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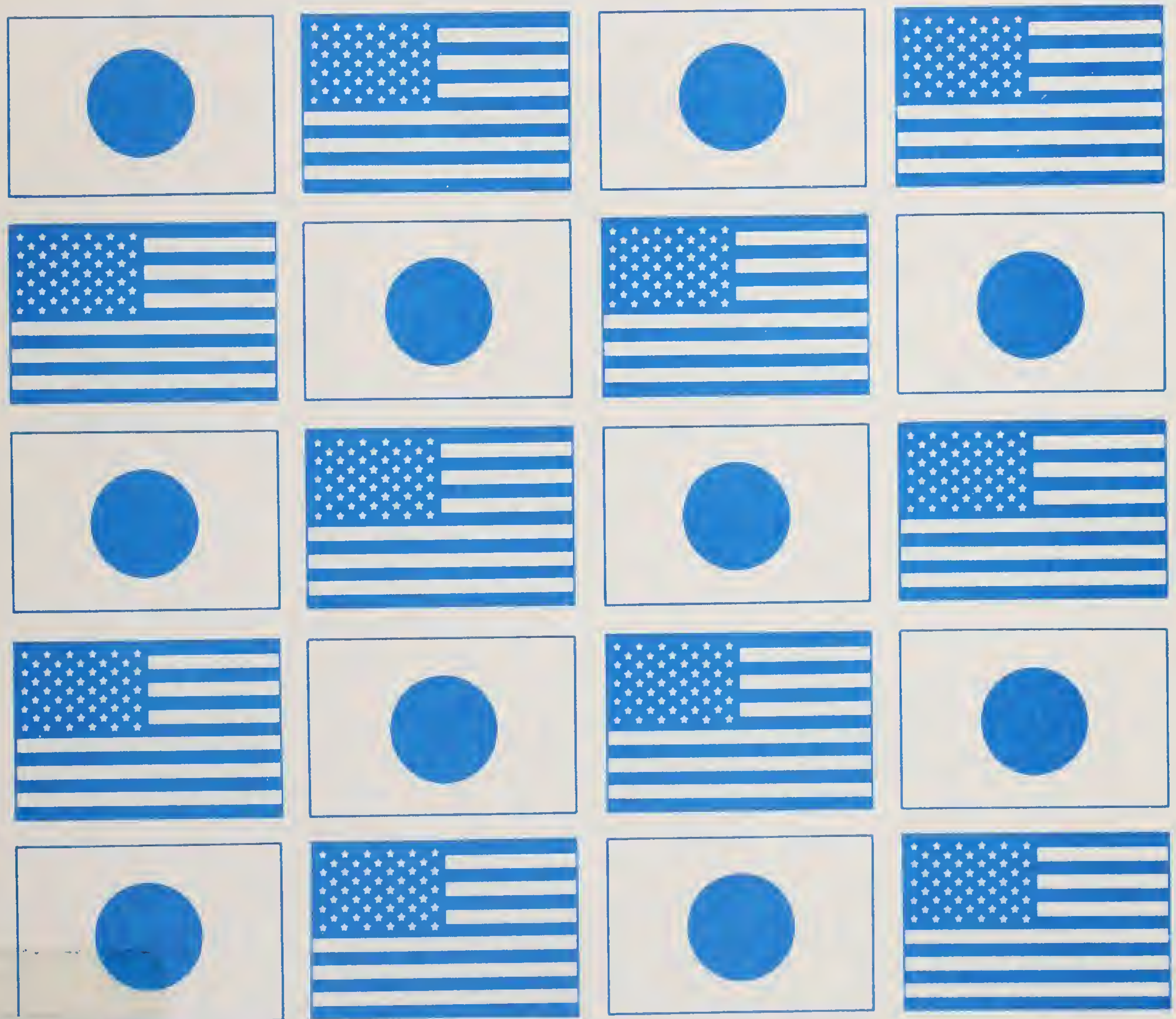

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PROCEEDINGS OF THE 21ST JOINT MEETING OF THE U.S.-JAPAN COOPERATIVE PROGRAM IN NATURAL RESOURCES PANEL ON WIND AND SEISMIC EFFECTS

Issued January 1990

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PREFACE

The UNITED STATES and JAPAN, in 1961, created the U.S.-JAPAN Cooperative Science Program. The U.S.-JAPAN Natural Resources Development Program (UJNR), one of three collateral programs making up the Cooperative Science Program, was created in January 1964. The objective of UJNR is to exchange information on research results and exchange scientists and engineers in the area of natural resources. The UJNR is composed of 17 Panels each focusing on specific technical subjects.

The Panel on Wind and Seismic Effects was established in 1968 to provide technologies for reducing damages from high winds, earthquakes, storm surge, and tsunami. The Panel provides for cooperative activities of 16 U.S. and six Japanese agencies with participating representatives of private sector organizations. Annual meetings alternate between JAPAN and the U.S. (odd numbered years in JAPAN; even numbered years in the U.S.). The Panel provides the vehicle to exchange technical data and information on design and construction of buildings, bridges, dams, and lifeline and waterfront structures, and to exchange high wind and seismic measurement records. Results from this Panel's efforts have impacted engineering practices and improved codes and standards in both countries.

The 21st Joint Meeting was held at the Public Works Research Institute, Tsukuba, JAPAN, during May 16-19, 1989. The Panel featured six themes: 26 of the 33 authored technical papers were presented plus two oral presentations each on the 1988 Armenia earthquake and two reports on task committee workshops. The Panel's nine task committees held their meetings during this period to review joint research projects and planned for future activities.

These proceedings include the program of the 21st Joint Meeting, Panel resolutions, technical papers, and task committee reports.

We are grateful for the partial support for the U.S. Panel from the National Science Foundation, Department of State, Department of Army, Federal Highway Administration, Bureau of Reclamation, Nuclear Regulatory Commission, Federal Emergency Management Agency, and the National Institute of Standards and Technology.

Noel J. Raufaste, Secretary
US-Side, Panel on Wind and Seismic Effects

The 21st Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects was held at the Public Works Research Institute, Tsukuba, JAPAN, from May 16-19, 1989. This publication, the proceedings of the Joint Meeting, includes the program, list of members, panel resolutions, task committee reports, and 33 technical papers.

The papers were presented under six themes: (I) - Wind Engineering, (II) - Earthquake Engineering, (III) - Storm Surge and Tsunami, (IV) - U.S.-Japan Cooperative Research Program, (V) - Code Development Process and Code Enforcement Responsibility, and (VI) - Armenia Earthquake (oral presentation).

KEYWORDS: accelerograph; Armenia; bridges; codes; concrete; design criteria; disaster; earthquakes; geotechnical engineering; ground failures; inelastic; lifelines; liquefaction; masonry; repair and retrofit; risk assessment; seismicity; soils; standards; storm surge; structural engineering; tsunami; and wind loads.

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Tuesday May 16

Opening Session	10:30-12:00	
Lunch	12:00-13:00	
Technical Session	13:00-16:50	

* Speaker

Session 1 (Theme I WIND ENGINEERING)

Chairman:N. Narita

13:00-13:20	A Summary of Federal Highway Administration's Wind Engineering Research Program Harold R. Bosch*
13:20-13:40	Wind Resistant Design Manual for Highway Bridges in Japan N. Narita, K.Yokoyama*, H.Sato
13:40-14:00	Field Measurements of Wind Pressures K. C. Mehta*
14:00-14:20	Discussion

Session 2 (Theme II EARTHQUAKE ENGINEERING)

Chairman:N. Narita

14:20-14:40	Behavior of the Nagara Dam during the Earthquake of December 17, 1987 Y. Sasaki*, T. Kuwabara, M. Nishimura, I. Seko, J. Kashiwagi, Y. Aoki
14:40-15:00	Analysis of Structural Response Records during the Whittier Earthquake, 1 October 1987 A. G. Brady*
15:00-15:20	Behaviors of Dams during Chiba Off Earthquake N. Matsumoto*, N. Yasuda
15:20-15:40	Discussion
15:40-16:00	Break

Guest Lecture (Theme II EARTHQUAKE ENGINEERING) 16:00-16:50

16:00-16:40	Survey for Estimation of Seismic Damage in South Kanto Area H. Oda*
16:40-17:00	Discussion

Adjourn

Wednesday May 17

Technical Session 9:00-12:00

Session 3 (Theme III STORM SURGE AND TSUNAMI)

Chairman:R. N. Wright

9:00- 9:20	Observations of Tsunamis on the West Coast of the United States J. F. Lander*
9:20- 9:40	Present State of Tsunami Numerical Simulation C. Goto*, K. Tanimoto
9:40-10:00	Discussion
10:00-10:20	Break

Session 4 (Theme II EARTHQUAKE ENGINEERING)

Chairman:R. N. Wright

10:20-10:40	Seismic Design Considerations for Offshore Oil and Gas Structures C. E. Smith*
10:40-11:00	Centrifuge Dynamic Model Tests in PHRI T. Inatomi, M. Kazama, S. Noda*, H. Tsuchida
11:00-11:20	Modeling of Earthquake Induced Pore Pressure Behavior R. H. Ledbetter, (A. G. Franklin*)
11:20-11:40	Seismic Design Method of Earth Embankment with Seismic Coefficient Y. Koga*, O. Matsuo
11:40-12:00	Discussion

Lunch 12:00-13:00

Technical Session 13:00-14:00

Session 5 (Theme II EARTHQUAKE ENGINEERING)

Chairman:R. N. Wright

13:00-13:20	Seismic Monitoring of Buildings : Analysis of Seismic Data M. Celebi, (A. G. Brady*)
13:20-13:40	Development of New Reinforced Concrete Buildings with High Strength and High Quality Concrete and Steel T. Murota*, M. Hirosawa
13:40-14:00	Guidelines for Identification and Mitigation of Seismically Hazardous Existing Federal Buildings H. S. Lew*
14:00-14:20	Discussion
14:20-14:30	Break

Task Committee Meetings 14:30-17:00

B:Large-Scale Testing Programs

E:Natural Hazard Assessment and Mitigation through Land Use Programs

F:Disaster Prevention Methods for Lifeline Systems

H:Soil Behavior and Stability During Earthquakes

I:Storm Surge and Tsunamis

Thursday May 18

Technical Session 9:00-12:00

Session 6 (Theme II EARTHQUAKE ENGINEERING)

Chairman:N. Narita

9:00- 9:20	U.S. Army Corps of Engineers Seismic Research Program W. E. Roper*
9:20- 9:40	The History and Status of Strong-Motion Earthquake Observation in Japan K. Ohtani*, H. Takahashi
9:40-10:00	Summary of Lifeline Earthquake Engineering J. D. Cooper, (H. R. Bosch*)
10:00-10:20	Discussion
10:20-10:40	Break

Session 7 (Theme V Code Development Process and Code Enforcement Responsibility)

Chairman:R. N. Wright

10:40-11:00	Minister-of-Construction's Agreement System in the Building Standard Law of Japan T. Murota*
11:00-11:20	History of Structural Regulations in Japanese Building Code and Outline of Wind-Resistant and Seismic Regulations Y.Ishiyama, Y. Ohashi*
11:20-11:40	Development and Enforcement of US Building Regulations J. G. Gross, R. N. Wright*
11:40-12:00	Discussion

Lunch 12:00-13:00

Technical Session 13:00-14:20

Session 8 (Reports of Workshops)

Chairman:R. N. Wright

13:00-13:30	Task Committee (F) Y. Sasaki*
13:30-14:00	Task Committee (J) H. R. Bosch*
14:00-14:20	Discussion
14:20-14:30	Break

Task Committee Meetings 14:30-17:00

A:Strong-Motion Instrumentation Arrays and Data

C:Repair and Retrofit of Existing Structures

D:Evaluation of Performance of Structures

J:Wind and Earthquake Engineering for Transportation Systems

Friday May 19

Technical Session 9:00-12:00

Session 9 (Theme IV U.S.-Japan Cooperative Research Program)

Chairman:R. N. Wright

9:00- 9:20	U.S.-Japan Coordinated Earthquake Research Program on Masonry Buildings -Seismic Test of Five Story Full Scale Reinforced Masonry Building, Part 2- S. Okamoto, Y. Yamazaki*, T. Kaminosono, M. Teshigawara
9:20- 9:40	Shear Strength of Masonry Wall Structural Systems H. S. Lew*
9:40-10:00	Inelastic Behavior of Reinforced Concrete Bridge Pier -Effect of Bilateral Loading and Circular Hollow Section- T. Iwasaki, O. Ueda, K. Kawashima*, K. Hasegawa, T. Yoshida
10:00-10:20	U.S. Coordinated Program for Masonry Building Research -A Fourth Year Status- J. L. Noland, (G. R. Fuller*)
10:20-10:40	Discussion
10:40-11:00	Break

Session 10 (Theme VI Reports of the Armenia Earthquake)

Chairman:R. N. Wright

11:00-11:30	Reports of the Armenia Earthquake -Discussion on Damages to Buildings- (Oral Presentation) M. Hirosawa*
11:30-12:00	US Experiences from Post-earthquake Fact Finding Team Visit to Armenia during 19-28 December (Oral Presentation) H. S. Lew*
12:00-12:10	Discussion

Lunch 12:10-13:00

Reports of Task Committees 13:00-15:00

Chairman:N. Narita

A:Strong-motion Instrumentation Arrays and Data
B:Large-Scale Testing Programs
C:Repair and Retrofit of Existing Structures
D:Evaluation of Performance of Structures
E:Natural Hazard Assessment and Mitigation through Land Use Programs
F:Disaster Prevention Methods for Lifeline Systems
H:Soil Behavior and Stability during Earthquakes
I:Storm Surge and Tsunamis
J:Wind and Earthquake Engineering for Transportation Systems

15:00-15:20 Break		
Adoption of Final Resolution	15:20-16:10	Chairman:N. Narita
16:10-16:20 Break		
Closing Session	16:20-16:40	
Press Conference	16:50-17:20	

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A. Strong-Motion Instrumentation Arrays and Data	*A.G. Brady M.K. Celebi A.F. Franklin W.B. Joyner F.G. McLean H. Meyers	*H. Tsuchida S. Hattori M. Ichikawa Y. Ishiyama K. Ohtani Y. Sasaki H. Yamanouchi
B. Large-Scale Testing Programs	*H.S. Lew A.J. Eggenberger C.E. Smith R.J. Smith	*K. Ohtani Y. Koga S. Noda S. Okamoto K. Yokoyama H. Yamanouchi
C. Repair and Retrofit of Existing Structures	*A.J. Eggenberger P.E. Gurvin H.S. Lew R.D. McConnell R.J. Morony J.B. Scalzi	*M. Hirosawa T. Iwasaki S. Okamoto H. Shinohara
D. Evaluation of Structural Performance	*G.R. Fuller M.K. Celebi R.D. Marshall R.D. McConnell J.B. Scalzi R.J. Smith	*S. Okamoto T. Hanafusa K. Masamura T. Murota N. Narita M. Okahara H. Shinohara H. Yamanouchi K. Yokoyama
E. Natural Hazard Assessment and Mitigation Through Land Use Programs	*S.T. Algermissen C.S. Barrientos A.G. Brady G.R. Fuller R.D. Marshall	*K. Kawashima S. Hattori T. Iwasaki M. Nagaoka Y. Sugimura

Task Committee-continued	US Side	Japanese Side
F. Disaster Prevention Methods for Lifeline Systems	*S.C. Liu M.K. Celebi J.D. Cooper T. Liu J.B. Scalzi J.S. Spencer S. Wu	*O. Ueda K. Kawashima T. Murota S. Noda K. Ohtani Y. Sasaki
H. Soil Behavior and Stability During Earthquakes	*A.G. Franklin F.G. McLean R.J. Smith	*Y. Sasaki Y. Koga N. Matsumoto M. Okahara Y. Sugimura H. Tsuchida
I. Storm Surge and Tsunamis	*H. Meyers C.S. Barrientos E.N. Bernard L. Butler	*T. Uda M. Ichikawa K. Masamura H. Takahashi K. Tanimoto
J. Wind and Earthquake Engineering for Transportation Systems	*J.D. Cooper A.G. Franklin J.H. Gates J.B. Scalzi	*S. Saeki M. Fujiwara T. Iwasaki K. Kawashima N. Narita M. Okahara Y. Sasaki K. Yokoyama

*Chairman

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

Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

May 16-19, 1989

The following resolutions are hereby adopted:

1. The Twenty-First Joint Panel Meeting provided an opportunity to exchange valuable technical information which was beneficial to both countries. In view of the importance of cooperative programs on the subject of wind and seismic effects, the continuation of Joint Panel Meetings is considered essential.

2. The following activities have been conducted since the Twentieth Joint Meeting:

   a. Guest researchers from both countries performed joint research that advanced the state of wind and earthquake engineering.

   b. Technical documents, research reports and proceedings of workshops were exchanged.

   c. The Japanese MOC document, "Manual for Repair Methods of Civil Engineering Structures Damaged by Earthquakes" was translated into English, published, and soon will be disseminated to appropriate US engineering community members.

   d. The Japanese MOC reports on Base Isolation Systems for Buildings were discussed with NIST in March 1989, for translation into English. They will be published and disseminated to appropriate U.S. engineering community members.

   e. Workshops and planning meetings were held:

      a) Fourth Joint Technical Coordinating Committee on Masonry, Task Committee (B), San Diego, CA, October, 1988.

      b) International Workshop on Sensor Technology Applied to Large Engineering Systems, Task Committee (D), NIST, Gaithersburg MD in September, 1988.

      c) Fifth Bridge Workshop, Task Committee (J), Tsukuba, May 9 - 13, 1989.

      d) Workshop on Disaster Prevention for Lifeline Systems, Task Committee (F), Tsukuba, May 11 - 13, 1989.

      e) Planning Meeting on Cooperative Research for Remedial Treatment of Liquefiable Soils, Task Committee (H), Tsukuba, May 13 - 14, 1989.

   f. Held a special Panel Theme on "Japanese and U.S. Code Development Process and Code Enforcement Regulations" to provide a better understanding of the history and evolution of both countries' building regulations.

3. Experiences from post-earthquake fact finding team visits to Armenia in December 1988 were reported by both sides. The Panel recognizes the importance of using these findings and experiences and encourages similar exchanges of information from future post-disaster investigations. Information transfer will include reports, seminars, and special Panel sessions as used at the Twenty-First Joint Meeting.

4. The Panel will continue seeking methods to contribute to the International Decade of Natural Disaster Reduction (IDNDR) such as exchanging Proceedings of Task Committees and Panel Meetings with their respective National Committees' of IDNDR.

5. The Panel approves each Task Committee's Mission Statement reports developed during the Twenty-First Joint Meeting. These reports present Task Committee objectives, current and proposed scope of work, and past and expected accomplishments.

6. The Panel recognizes the importance of the following items:

   a. Both sides are performing work contributing to the Panel's Program on Large-scale Testing of Precast Seismic Structural Systems (PRESSS). Information about the program will continue to be exchanged during the coming year.

   b. Continue to exchange data and information on application of active and passive response control systems for buildings and other structures and encourage the appropriate Task Committees, by joint work, to advance these technologies.

   c. Collect strong motion data on the performance of buried pipeline systems for 3-D downhole information.

   d. Obtain experimental verification of the effectiveness of retrofitting and strengthening methods for structures and soils.

   e. Disseminate information to all Panel members about opportunities for exchanging researchers to each country such as using the Japanese STA Fellowship Program and the related NSF Fellowship Programs.

7. Joint planning will begin on cooperative work in a new research area on verifying soil-pile interaction models for marine structures subjected to earthquake loads. This may be planned through correspondence by both chairmen of T/C (H) who will inform both Panel Side Secretaries about progress.

8. The Panel endorses the following proposed Task Committee workshops and planning meetings:

a. The Fifth Meeting of JTCCMAR, T/C (B) will be scheduled in Tsukuba, Japan October 1989, and a US - Japan Precast Seismic Structural Systems (PRESSS) Program Coordinating Meeting, T/C (B) will be planned for Japan in October, 1989.

b. Task Committees (C) and (D) are planning a joint workshop on Evaluation, Retrofit, and Strengthening Structures Damaged by High Winds and Earthquakes in May 1990.

c. Workshop on Earthquake Hazard and Risk Assessment, T/C (E) will be held in Japan in 1990.

d. Workshop on Disaster Prevention for Lifeline Systems, T/C (F) will be held in the U.S. in 1990.

e. Workshop on Design and Treatment Method for Mitigating Liquefaction Induced Damage, Task Committee (H) will be held in Japan in October 1990.

f. Workshop on Storm Surge and Tsunamis, T/C (I) will be held in Honolulu in October or November 1990.

g. Sixth Bridge Workshop, T/C (J) will be held in the U.S. just prior to the 22nd UJNR Joint Meeting.

Scheduling for the workshops and planning meeting shall be performed by the US and Japan chairmen of the respective Task Committee with concurrence of the Joint Panel chairmen. Results of each activity shall be presented at the next Joint Panel Meeting.

9. The Panel recognized the importance of continued exchange of personnel, technical information, research results, and recorded data that lead to mitigating losses from earthquakes and strong winds. The Panel also recognizes the importance of using available large-scale testing facilities in both countries. Thus, these activities should continue to be strengthened and expanded and, as appropriate, share Task Committee activities work at other meetings that have technical interest in the Task Committee activities. To facilitate these exchanges, the Panel will provide official endorsement.

10. The Twenty-Second Joint Meeting of the UJNR Panel on Wind and Seismic Effects will be held at NIST, Gaithersburg, Maryland, U.S., May 1990. Specific dates, program, and itinerary will be proposed by the U.S. Panel with concurrence of the Japan Panel.

Theme I

Wind Engineering

A Summary of the Federal Highway Administration's Wind Engineering Research Program

by

Harold R. Bosch[1]

ABSTRACT

Since the dramatic collapse of the Tacoma Narrows Bridge (Washington) in 1940, the Federal Highway Administration has actively pursued a program of research aimed at advancing our understanding of the effects of wind forces on structures and ensuring the aerodynamic stability of long-span bridges. This research has involved scale model testing in wind tunnels, analytical investigations, numerical modeling, instrumentation development, full scale measurements at bridge sites, software development, coordination of national research, and technology exchange. This paper will attempt to highlight some of the research activities which have been undertaken during the past 50 years with the aim of providing an overview of the Administration's wind engineering research program.

KEYWORDS: Bridge; cable-stayed; dynamics; stability; suspension; wind.

1. INTRODUCTION

Almost a half century ago, the dramatic collapse of the Tacoma Narrows Bridge sparked a major investigation into the effects of wind on suspension bridges. To coordinate the many activities which were to be undertaken, the Advisory Board on the Investigation of Suspension Bridges was formed. The Board was broadly representative of engineers responsible for specific suspension bridges, research engineers having competence in aerodynamics and suspension bridge theory, and representatives of industry with demontrated ability and leadership in the fabrication and erection of suspension bridges [1].

The first action of this Board was to recommend that the Bureau of Public Roads (BPR), now the Federal Highway Administration (FHWA), cooperate actively in a broad attack on the problems of the aerodynamic behavior of suspension bridges. The Bureau quickly responded by initiating agreements with: the Washington Toll Bridge Authority to conduct model studies in a special wind tunnel at the University of Washington [2]; the Golden Gate Bridge and Highway District for development of suitable bridge instruments, installation, and operation on the Golden Gate Bridge [3]; the American Institute of Steel Construction for analytical studies by the late Dr. Friedrich Bleich [4]; and the Oregon State Highway Department for theoretical analysis and static load testing on a suspension bridge model.

This cooperative research served to identify the mechanism of the wind action which caused the failure of the original Tacoma Narrows Bridge and firmly established the stability of the proposed design for the new bridge. In the process, it demonstrated that properly scaled model studies conducted in suitable wind tunnels could be used effectively to evaluate structural designs and would reproduce quite faithfully the observed bridge behavior.

The research also provided a number of significant observations concerning the response of suspension bridges to wind action [5]:

- Girder-stiffened suspension bridges are prone to restricted vertical or torsional oscillations. The strength and amplitude of these oscillations can be predicted from wind tunnel study.
- Girder-stiffened suspension bridges are also prone to catastrophic vertical or torsional oscillations depending on their depth to width ratio. Generally, narrow and deep structures have a tendency for vertical motion while wide and shallow structures have a tendency for torsional motion.
- Truss-stiffened suspension bridges are also subject to minor restricted oscillations; however, their main danger is from flutter resulting from a coupling of modes. The motion is usually dominated by the torsional component.
- In addressing the question of coupling, Dr. Bleich applied the classical theory of flutter to the behavior of suspension bridges. His analysis indicated that an effective means for combating the catastrophic behavior of truss-stiffened bridges would be to increase the torsional frequency through use of top and bottom lateral systems.
- Although the use of deck vents or slots proved to be effective on the new Tacoma Narrows Bridge, they are not generally effective on girder-stiffened suspension bridges.

Although this early research provided considerable insight into the problem of bridge aerodynamic stability, it revealed some inconsistencies and left a number of questions unanswered. Those involved in the work recognized the need for continued research leading to definite and consistent design criteria which would ensure the aerodynamic stability of suspension bridges and for establishment of a more permanent wind tunnel facility which would enable investigation of

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specific designs until suitable criteria are developed. Many felt that the Bureau of Public Roads should take on this obligation and coordinate the research efforts.

2. WIND RESEARCH PROGRAM

BPR, and later FHWA, gladly accepted this responsibility and embarked upon a broad research program which has involved coordinating national and international wind investigations, sponsoring contract research, conducting laboratory and field studies in-house, and providing technical guidance through committees, panels, or research councils. For more than 40 years, the Bureau has continued to play an important role in ensuring the aerodynamic stability of long-span bridges. Some of the major activities (Table 1) may be grouped into the following broad categories:

- Building a wind tunnel facility. Few wind tunnels in the U.S. are suitable for investigating the effects of wind forces on bridges and none are totally dedicated to that purpose.
- Developing laboratory equipment. Wind tunnel laboratories require specialized test equipment and custom support hardware which often is not available commercially.
- Conducting scale model studies. Designs were proposed for many new suspension and cable-stayed bridges which required testing. Alterations were proposed for a number of existing bridges which also required testing.
- Developing instrumentation systems. Durable, automated instrumentation systems were required to perform long term measurements at remote bridge sites. Few, if any, suitable systems were available commercially.
- Performing full scale measurements. Detailed wind and bridge response measurements are needed at bridge sites to establish the exact nature of local winds and to characterize bridge dynamic behavior.

A brief overview of specific projects within each of these categories will be presented in the sections which follow.

3. AERODYNAMICS LABORATORY

Following the reconstruction of the Tacoma Narrows Bridge, the temporary bridge testing facilities at the University of Washington were dismantled to make way for other new buildings. In response, the Bureau made a careful survey of existing wind tunnel facilities in the United States to determine if one might be adapted for the purpose of studying the effects of wind forces on suspension bridges. BPR determined that it would be more cost effective and expeditious to build its own specialized facility. During the 1950's, the aerodynamics laboratory and wind tunnel (Figure 1) were designed, constructed, and placed into service at BPR's Fairbank Highway Research Station in McLean, Virginia. The wind tunnel is a low velocity, open-circuit, laminar flow tunnel designed primarily for study of wind loads on long-span bridges [6,7]. During normal operation, air enters through the double inlet section of the tunnel, passes through a diffusion section containing a series of screens, and exits through a 6 by 6-ft (1.8 by 1.8-m) nozzle into the test area. Wind speeds up to 50 fps (15.2 mps) can be generated and the vertical angle of the wind can be changed by rotating the nozzle up and down.

4. LABORATORY INSTRUMENTATION

4.1 Turbulence Generator

During the period 1977-1983, FHWA sponsored research at Colorado State University to develop a new technique for laboratory simulation of large scale turbulence suitable for use in bridge section model studies [8]. This research led to design and implementation of a unique turbulence simulation system in the FHWA Aerodynamics Laboratory. The concept of this system is shown in Figure 2. The main components of the turbulence generator are two arrays of symmetrical airfoils and an electrohydraulic driving system. The airfoil assemblies are located just downstream of the wind tunnel nozzle. The airfoils at location A oscillate 180 degrees out of phase to produce horizontal velocity fluctuations while those at location B oscillate in phase to produce vertical fluctuations. Both sets of airfoils are driven by hydraulilc actuators and each can be operated independently through a master control system (Figure 3).

4.2 Force-Balance

During the period 1985-1986, FHWA designed and implemented a high frequency force-balance for measuring lift, drag, and pitching moment forces on bridge section models (Figure 4). The force-balance system actually consists of two separate balance mechanisms, one located at each end of the model. This rather unique feature enables detection of unbalanced loads on the bridge. The balance system is mobile and can be rolled into and out of the test section as necessary. Section models mounted in the force-balance can be rotated through a range of 180 degrees to vary the angle of attack.

5. SCALE MODEL STUDIES

5.1 Suspension Bridge Studies

During the period 1960-1968, wind tunnel studies (Table 2) were conducted to evaluate the aerodynamic stability of seven suspension bridges [9-16]: San Pedro-Terminal Island (California), Golden Gate (California), Tagus River (Portugal), Lion's Gate (Canada), Narragansett Bay (Rhode Island), New Quebec (Canada), and Bosporus (Turkey). Most studies were aimed at assessing proposed designs for new bridges, while some, like Golden Gate, were to investigate proposed alterations. All of these experiments were conducted under laminar flow conditions with the exception of Lion's Gate, where a parallel bridge was placed upwind of the model for a portion of the testing. Rigid section models with geometric length scales ranging from 1:40 to 1:75 were designed and fabricated for each test. The models were mounted on a spring suspension system which was carefully adjusted to provide the correct vertical and torsional frequencies. Wind speeds were varied from 0 to 50 fps (15.2 mps) while angle of attack was varied from -5 to +10 degrees. Model response was observed, recorded on strip chart, and later evaluated to obtain data on logarithmic increment, logarithmic decrement, amplitude, frequency, and critical velocity.

5.2 Generic Flutter Studies

During the period 1969-1971, wind tunnel studies were performed on a dozen generic sections in conjunction with analytical work being conducted by Sabzevari, Scanlan, and Tomko [17]. The models were chosen to be representative of truss, box, and girder type bridge sections. The research centered on developing an analytical model for bridge flutter, in which the aerodynamic information was to be obtained from experiment. Behavior of the rigid section models was observed in the wind tunnel and aerodynamic force coefficients were inferred from the response of the models. Experimental procedures were developed for efficient and reliable extraction of the necessary aerodynamic information.

5.3 Cable-Stayed Bridge Studies

During the period 1972-1980, FHWA conducted a series of model studies to investigate the aerodynamic behavior of cable-stayed bridge designs. Wind tunnel investigations were performed on the design for Sitka Harbor Bridge (Alaska), 14 design alternates for Luling-Destrehan Bridge (Louisiana), and the steel box design alternate for East Huntington Bridge (West Virginia). Like earlier experiments, these tests employed rigid section models, were conducted in laminar flow, and vertical wind angle was varied. In contrast with the earlier work, flutter coefficients were extracted, mean forces (lift, drag, and moment) were measured, and vortex-induced bridge response was carefully documented [18,19].

5.4 Bridge Rehabilitation Studies

During the period 1981-1983, Phase I of a comprehensive investigation into the aerodynamic behavior of the Deer Isle-Sedgwick Bridge (Maine) was accomplished. Section model tests were performed on a 1:25 scale section model of the existing bridge as well as three proposed modifications [20]. Laminar flow conditions were maintained throughout this phase of testing. A series of separate tests were conducted to identify general response characteristics, to extract flutter coefficients, to measure mean forces, and to study vortex-induced response.

5.5 Turbulence Simulation Studies

During the period 1984-1986, experimental procedures were developed to conduct section model studies in sumulated, large-scale turbulence [21]. Wind tunnel experiments incorporating simulated turbulence were performed on Deer Isle (Phase II) and Golden Gate section models using FHWA's unique turbulence generator [22]. The bridge models were subjected to various levels of turbulence and the resulting response compared to results from previous studies in laminar flow.

5.6 Generic Section Model Studies

During the period 1987-1988, special model studies were conducted on generic section models representing "Box" and "H" structural shapes, seven of each, with depth to width ratios ranging from 0.10 to 1.00. Initially, all the shapes were tested in both laminar and simulated turbulent flow conditions without varying the wind angle [23]. Flutter coefficients were obtained for each test condition. Similar tests are being conducted at the Chinese Aerodynamic Research and Development Center and the results from both laboratories are being compared. Following these experiments, another series of tests were performed in conjunction with research on analytical modeling of vortex-induced response [24]. For this study, three shapes exhibiting strong response to vortex shedding were selected from the original set of fourteen. Testing was performed only in laminar flow and attention was focused on regions of resonant response. The variation of response amplitudes within the resonant region(s) was carefully recorded as was the nature of vortex shedding in the wake and the coherence of shedding along the span of the model.

6. FIELD INSTRUMENTATION

6.1 Melpar System

In 1969, a data acquisition system was designed to monitor and record wind velocities and direction at remote bridge sites and to document their effects on suspension bridges [25]. The system was designed to operate unattended, being triggered into operation by wind velocities or bridge displacements which exceed predetermined threshold levels. To characterize the turbulent nature of the wind, 3-component propeller type anemometers were selected. These were to be spaced at intervals along the bridge span to sense wind components normal, parallel, and vertical relative to the roadbed. To characterize the bridge response, servo type accelerometers were selected. The accelerometers were to be installed in pairs, one near each edge of the roadbed, at key points along the bridge span. Accelerometers were also to be placed in a bridge tower to identify bending, twist, and sway of the tower. Provisions were also made for use of an aerovane type wind sensor which provides basic wind speed and direction data similar to that obtained by the National Weather Service. The data acquisition equipment included circuits or modules for signal conditioning, amplifying, filtering, multiplexing, and digitizing the incoming analog signals. A dual buffer was provided for intermediate storage of the digitized data. As one buffer fills, the system automatically switches to the other buffer and the data is dumped to one of two 9-track, 800 BPI digital tape drives. The system was designed to automatically sample up to 70 input channels at 8-bit resolution using a system clock rate of 500 samples per second and have the capability to store 10 hours of continuous wind and bridge data.

6.2 Kinemetrics System

By 1978, FHWA was expanding its field measurement program and developed a need for a second data acquisition system. A new system, similar to the prototype, was designed [26]. This new instrumentation system incorporated at number of technological advances and enhancements:

- 12-bit analog-to-digital conversion.
- Variable number of channels (64 max).
- Variable sampling rate (90,000 max).
- Variable filter frequencies and gains.
- Dual 1600 BPI 9-track tape drives.
- Special offset amplifiers.
- Uninterruptable power supply.

7. FULL SCALE MEASUREMENTS

Full scale measurements at bridge sites (Table 3) are an essential part of the wind research program. The wind data obtained provides detailed information regarding prevailing conditions (speeds, directions, angle of attack) and turbulence properties (intensity, spectrum, scale) which is often not available from other sources. The structural response data provides information on the structure's dynamic properties (frequencies, mode shapes, damping) which can be compared with computed values. Wind and structural measurements are often used in the design of wind tunnel experiments and to evaluate the effectiveness of aerodynamic retrofits. They can also provide a means for determining the adequacy of experimental and analytical techniques.

7.1 Ambient Vibration Surveys

In 1969, BPR awarded a contract to Teledyne Geotronics to investigate the feasibility of using the small amplitude vibrations normally present in suspension bridges to measure natural frequencies, mode shapes, and damping factors. Teledyne had previously been quite successful in using such an approach to determine the dynamic characteristics of large buildings. The survey technique was demonstrated on two moderate sized suspension bridges, the Newport Bridge (Rhode Island) and the Chesapeake Bay Bridge (Maryland). The test results were quite encouraging. On the Newport Bridge, the contractor identified 20 mode shapes, including five symmetric vertical, four anti-symmetric vertical, the first symmetric and first anti-symmetric torsional, six lateral, and three tower modes [27]. Mode shapes were in good agreement with calculated values although measured frequencies were about 20% higher. Damping estimates ranged from 3% for the first lateral mode to 0.4% for the fourth anti-symmetric vertical mode. Similar information was obtained for the Chesapeake Bay Bridge [28].

7.2 Wind and Bridge Response Studies

7.2.1 Newport Bridge

In 1971, FHWA installed its new wind and bridge response instrumentation system on the Newport Bridge (Rhode Island). Completed in 1969, the Newport Bridge is a truss-stiffened suspension bridge with a 1600-ft (488-m) main span, 688-ft (210-m) side spans, and 48-ft (14.6-m) roadway. Although this instrumentation was not fully deployed due to conflicts with painting crews on the bridge, it was used to record wind data from a portion of Hurricane Doria in August 1971. A total of five 3-component anemometers spaced at 350-ft (107-m) intervals along the span were in place at the time. Mean wind speeds of 60 mph (97 km/h) with peak gusts to 75 mph (121 km/h) were observed at the site. Wind data was recorded continuously for 10 hours. Wind angle ranged from 20 degrees for mild winds to 3 degrees for strong winds [29,30].

7.2.2 Sitka Harbor Bridge

In 1972, the instrumentation system was installed on the Sitka Harbor Bridge (Alaska). The Sitka Bridge is the first cable-stayed bridge in the United States to carry vehicular traffic. It is a five-span structure with a total length of 1053 ft (321 m) and a main span of 450 ft (137 m). The deck is 38 ft (11.6 m) wide and is supported by two steel box girders 30 by 72 in (76 by 183 cm). The two towers consist of free-standing pylon legs with no portal framing. Five 3-component anemometers and 10 accelerometers were mounted along the bridge span, 3 accelerometers were mounted at the top of one tower, and a strong motion accelerograph was installed at the base of one tower (Figure 5). Much of the wind observed at this site was parallel to the structure and did little to excite the span. Vortex-induced motion of the towers perpendicular to the plane of the stay cables was frequently observed, however. The tower legs were typically excited in the fundamental mode at wind speeds of 24 to 26 mph (39 to 42 km/h) and a frequency of 1.16 cps [31,32].

7.2.3 Perrine Bridge

In 1977, FHWA installed the instrumentation on the Perrine Bridge (Idaho). The Perrine Bridge is a 1500 ft (457 m) long structure over the 500 ft (152 m) deep Snake River canyon. The main span of the bridge is a steel deck arch with a span of 993 ft (303 m) and a rise of 210 ft (64 m). The deck, stringers, and floor beams rest atop box-shaped columns which span to the top chord of the arch truss. These columns are not braced and range in length from 7 to 160 ft (2.1 to 48.8 m). Pairs of accelerometers were mounted at mid height of three of the westerly columns (one short, one long, and one medium length) to monitor vibrations normal and parallel to the structure. To measure stresses in the columns, strain gages were mounted in the middle of each face near the base of the column. Three 3-component anemometers were installed on booms extended from the top chord facing west (one near the crown and one near each quarter point). Wind, traffic, and column vibration measurements were recorded on digital tape and strip chart for later analysis [33].

7.2.4 Pasco-Kennewick Bridge

In 1979, the instrumentation system was installed on the Pasco-Kennewick Bridge (Washington). The Pasco-Kennewick Bridge is the first segmental concrete cable-stayed bridge constructed in the U.S. The bridge is 2503 ft (763 m) long with a main span of 981 ft (299 m). Six 3-component anemometers were mounted along the deck with spacings ranging from 33 to 525 ft (10 to 160 m). Accelerometers were attached to each edge of the deck section at six stations along the main and side spans and three accelerometers were installed in the top of one tower. During the 24 month monitoring period, 49 hours of wind and bridge response data, representing approximately 100 events, were recorded [34]. Unfortunately, the highest wind speeds encountered were only 27 mph (43 mps). As expected, the bridge exhibited very little response to the given conditions. Measured frequencies and mode shapes compared quite well with computed values. Measured wind spectra was also quite similar to selected empirical models.

7.2.5 Deer Isle-Sedgwick Bridge

In 1981, FHWA refurbished the instrumentation system and installed it on the Deer Isle-Sedgwick Bridge (Maine). The Deer Isle Bridge is a girder-stiffened suspension bridge built in 1939 and similar in cross section to the original Tacoma Bridge. Immediately following the collapse of the Tacoma Bridge, this bridge was stiffened through addition of longitudinal and diagonal stays as well as cable clamps. Despite these modifications, the bridge has continued to oscillate in certain storm conditions. The Deer Isle Bridge has a main span of 1080 ft (329 m) with side spans of 484 ft (148 m) and approach spans of 130 ft (40 m). The deck is 23.5 ft (7.2 m) wide and the girders are 6.5 ft (2.0 m) deep. Six 3-component anemometers were mounted on booms facing seaward. Two aerovane type instruments were installed on the north bridge tower, one at the top and one near the bottom about 33 ft (10 m) above the water. Temperature sensors were located at the top and bottom of the same tower and outside as well as inside the instrument house located just north of the bridge. Twelve accelerometers were installed on the bottom flange of the stiffening girders at six stations along the main span and north side span. Three accelerometers were installed at the top of the north tower to measure tower bending and sway. More than 600 hours of wind and bridge response data have been recorded and data collection is continuing. Although the prevailing wind direction is from the northwest, wind events from most directions have been sampled. Wind speeds up to 80 mph (129 km/h), including a portion of Hurricane Gloria, have been captured. Bridge motion has been primarily in the vertical degree-of-freedom and frequencies as well as mode shapes compare reasonably well with calculations [35,36].

7.2.6 Luling-Destrehan Bridge

In 1984, a new instrumentation system was installed on the Luling-Destrehan Bridge (Louisiana). The Luling Bridge is the first major U.S. cable-stayed bridge to be built in the hurricane region of the Gulf of Mexico. The bridge consists of a five-span superstructure made entirely of weathering steel. The main span is 1235 ft (376 m) with side spans of 500 and 260 ft (152 and 79 m). The roadbed is 82 ft (25 m) wide and supported by twin trapezoidal box girders 14 ft (4.3 m) deep. Twenty-four groups of stay cables radiate from the two A-frame towers which are 400 ft (122 m) tall. The stay groups consist of either two or four cables each. Five anemometers have been installed atop special luminaire poles on the median barrier, three in the main span and one in each back span [37]. Thirty accelerometers were mounted on the bridge, 18 on the cables, 10 in the box girders, 2 at the top of the south tower. Storm activity at the site in 1984-85 was infrequent and maximum wind speeds of only 30 mph (48 km/h) were observed. As expected, the bridge remained quite stable [38]. Since that time, portions of Hurricanes Juan and Florence have been recorded, but wind speeds failed to exceed 35 mph (56 km/h). Data has been analyzed to determine bridge deck and cable frequencies as well as motion amplitudes. The results of measurements compare well with calculated values.

8. CONCLUSIONS

In this paper, the author has attempted to provide a brief summary of the Federal Highway Administration's wind engineering research program which has spanned half a century. The detailed tasks and results of each project highlighted could not be presented here and not all of the investigations performed could be mentioned. Some of the activities not covered involve application of computational fluid dynamics, development of computer software, implementation of design aids, cataloging of research results, dynamic calibration of wind sensors, and automation of wind tunnel testing. This research program, and others, have contributed significantly toward advancing our understanding of the effects of wind forces on structures and have ensured the aerodynamic stability of modern long-span bridges.

9. REFERENCES

1. Vincent, G.S., "Research as Related to Highway Bridges," Proceedings, American Institute of Steel Construction, National Engineering Conference, April 1962.
2. Farquharson, F.B., Smith, F.C. and Vincent, G.S., "Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge," University of Washington, Bulletin 116, Parts I-V, June 1949-June 1954.
3. Vincent, G.S., "Golden Gate Bridge Vibration Studies," ASCE, Journal of the Structural Division, Volume 84, ST6, October 1958.
4. Bleich, F., et.al., "The Mathematical Theory of Vibration in Suspension Bridges," BPR, Government Printing Office, 1950.
5. Vincent, G.S., "A Summary of Laboratory and Field Studies in the United States on Wind Effects on Suspension Bridges," Proceedings, Symposium on Wind Effects on Buildings and Structures, Teddington, England, June 1963.
6. Vincent, G.S., "Wind Tunnel for Aerodynamic Testing of Section Models of Suspension Bridges," Public Roads Magazine, Volume 30, No. 2, June 1958.
7. Bosch, H.R., "Aerodynamics Laboratory," FHWA Laboratory Flyer, Report No. FHWA/RD-88/070, August 1988.
8. Cermak, J.E., Bienkiewicz, B., and Peterka, J.A., "Active Modeling of Turbulence for Wind-Tunnel Studies of Bridge Models," FHWA, Report No. FHWA/RD-82/148, February 1983.
9. Wiles, E.G., "Aerodynamic Studies on Proposed San Pedro-Terminal Island Suspension Bridge," BPR Report, 1960.
10. Vincent, G.S., and Wiles, E.G., "Investigation of Aerodynamic Effects of Proposed Alterations of the Golden Gate Bridge," BPR Report, November 1960.
11. Vincent, G.S., "Aerodynamic Studies on Proposed Tagus River Suspension Bridge," BPR Report, 1961.
12. Vincent, G.S., "Aerodynamic Studies on Proposed Tagus River Suspension Bridge," BPR Supplemental Report, 1961.
13. Cayes, L.R., "Aerodynamic Studies on Lion's Gate Suspension Bridge," BPR Report, February 1964.
14. Cayes, L.R., and Michael, D.L., "Aerodynamic Stability of Proposed Narragansett Bay Suspension Bridge," BPR Report, December 1965.
15. Cayes, L.R., "Aerodynamic Stability of Proposed New Quebec Suspension Bridge," BPR Report, October 1966.
16. Cayes, L.R., "Aerodynamic Stability of Proposed Bosporus Suspension Bridge," BPR Report, June 1968.
17. Scanlan, R.H., and Tomko, J.J., "Airfoil and Bridge Deck Flutter Derivatives," ASCE, Journal of the Engineering Mechanics Division, December 1971.
18. Bosch, H.R., "Aerodynamic Investigations of the Luling Louisiana Cable-Stayed Bridge," FHWA, Report No. FHWA-RD-77-161, July 1978.
19. Cayes, L.R., "Aerodynamic Stability of Proposed Ohio River Cable-Stayed Bridge," FHWA, Report No. FHWA-RD-77-157, December 1977.
20. Bosch, H.R., "A Wind Tunnel Investigation of the Deer Isle-Sedgwick Bridge (Phase I)," FHWA, Report No. FHWA/RD-87/027, August 1987.
21. Huston, D.R., and Bosch, H.R., "Active Turbulence Generation for Section Model Studies," Proceedings, Sixteenth U.S.A.-Japan Conference on Wind and Seismic Effects, May 1984.
22. Huston, D.R., Bosch, H.R., and Tanaka, H., "The Effects of Large-Scale Upstream Gusting on the Aeroelastic Behavior of Section Models," Proceedings, Eighteenth U.S.A.-Japan Conference on Wind and Seismic Effects, May 1986.
23. Huston, D.R., "Flutter Derivatives From 14 Generic Deck Sections," Proceedings, ASCE Structures Congress, Orlando, 1987.
24. Ehsan, F., and Bosch, H.R., "Modeling the Effect of Spanwise Coherence of Aerodynamic Forces in Full-Bridge Response," Proceedings, Twentieth U.S.A.-Japan Conference on Wind and Seismic Effects, May 1988.
25. American Standard, Inc., "An Instrumentation System to Monitor and Record Wind Velocities, Directions, and Effects at Suspension Bridge Sites," FHWA, March 1971.
26. Bourgeois, B.S., et.al., "Wind Instrumentation of the Luling Bridge, Luling, Louisiana (Volume I)," Louisiana Transportation Research, Report No. FHWA/LA-79-1ST, March 1982.
27. McLamore, V.R., "Ambient Vibration Survey of the Newport Bridge," Teledyne Geotronics, Report No. 0370-2150, March 1970.
28. McLamore, V.R., "Ambient Vibration Survey of the Wm. Preston Lane Memorial Bridge," Teledyne Geotronics, Report No. 0370-2152, March 1970.
29. Gade, R.H., and Scanlan, R.H., "Aspects of Hurricane Winds as Recorded at an Instrumented Bridge," Proceedings, Eighth U.S.A.-Japan Conference on Wind and Seismic Effects, May 1976.
30. Gerhardt, C.L., "Monitoring and Recording Wind Velocities, Directions, and Effects at Long-Span Bridge Sites," FHWA, Report No. FHWA-RD-76-10, August 1975.
31. Gade, R.H., "Status of Investigation of Aerodynamic Behavior of the Sitka Harbor Cable-Stayed Bridge," Proceedings, Symposium on Full Scale Measurements of Wind Effects on Tall Buildings and Other Structures, London, Ontario, June 1974.
32. Nelson, D.D., Greene, D.M., and Keay, N.S., "Wind and Bridge Motion Data Recorded at the Cable-Stayed Bridge, Sitka, Alaska," FHWA, Report No. FHWA-RD-78-57, May 1977.
33. Cheng, D.H., "Aerodynamic Stability of Long Columns with Elastic End Constraints," FHWA, Report No. FHWA/RD-81/133, February 1982.

34. Bampton, M.C.C., et.al., "Pasco-Kennewick Cable-Stayed Bridge Wind and Motion Data," FHWA, Report No. FHWA/RD-82/067, February 1983.
35. Bampton, M.C.C., et.al., "Deer Isle-Sedgwick Suspension Bridge Wind and Motion Analysis," FHWA, Report No. FHWA/RD-86/183, March 1986.
36. Bosch, H.R., "Full Scale Studies on the Deer Isle-Sedgwick Suspension Bridge," Proceedings, Fifth U.S. National Conference on Wind Engineering, November 1985.
37. Thompson, H.A., et.al., "Proposed Plans for Measurement of the Aerodynamic Behavior of the Luling, Louisiana Cable-Stayed Bridge," Public Roads Magazine, Volume 45, No. 3, December 1981.
38. Bruce, R.N., et.al., "Dynamic Response to Wind and Wind Climate of the Luling-Destrehan Bridge at Luling, Louisiana," Louisiana Transportation Research, Report No. FHWA/LA-82-1ST, July 1986.

Table 1. Research Time-Phase Chart

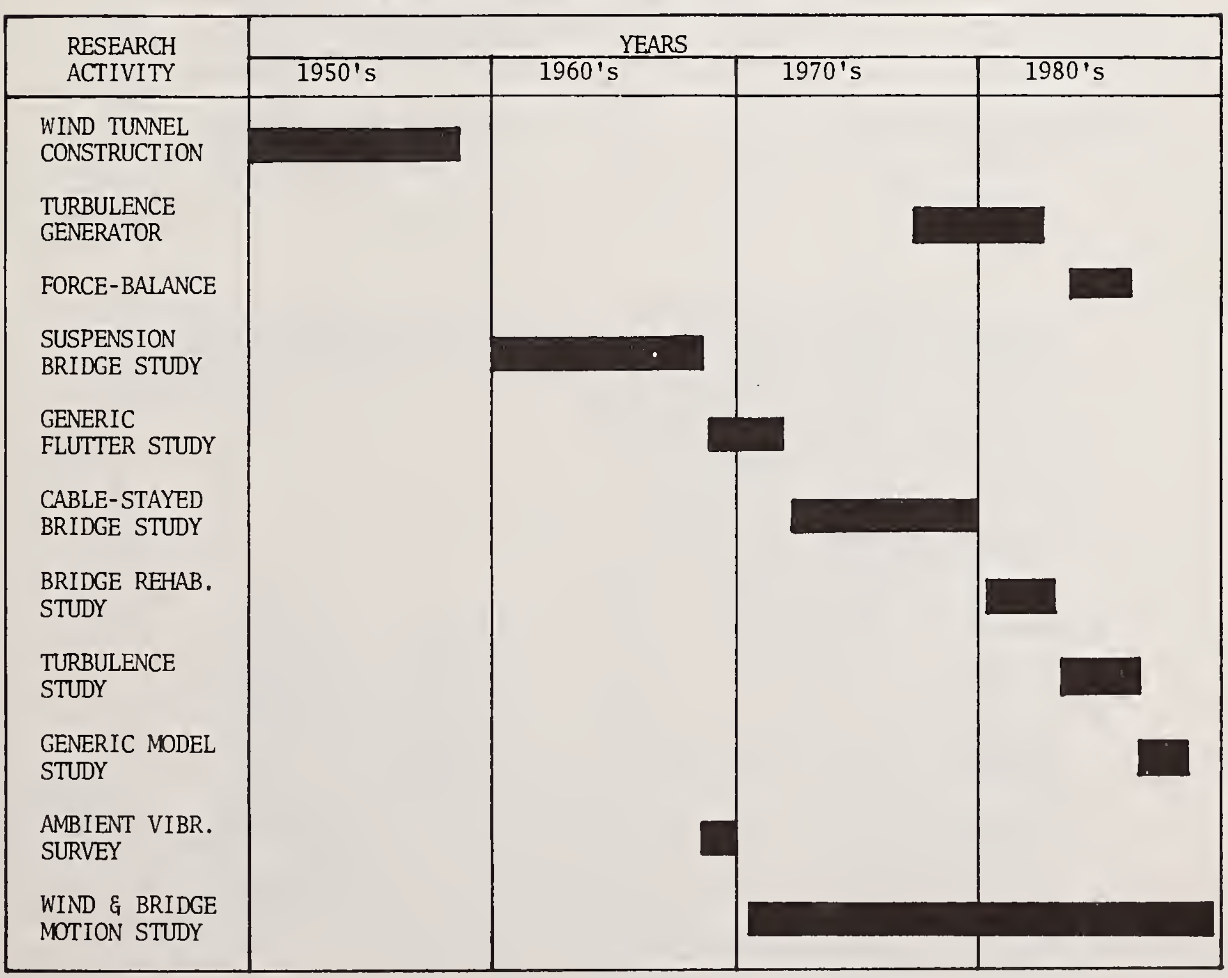


Table 2. Wind Tunnel Model Studies.

BRIDGE OR MODEL	LOCATION	DATE OF TEST	TYPE OF FLOW
SAN PEDRO-TERMINAL ISLAND	CALIFORNIA U.S.A.	1960	LAMINAR
GOLDEN GATE (Alterations)	CALIFORNIA U.S.A.	1960	LAMINAR
TAGUS RIVER	PORTUGAL	1961	LAMINAR
LION'S GATE	CANADA	1964	LAMINAR TURBULENT
NARRAGANSETT BAY	RHODE ISLAND U.S.A.	1965	LAMINAR
NEW QUEBEC	CANADA	1966	LAMINAR
BOSPORUS	TURKEY	1968	LAMINAR
GENERIC DECK SECTIONS	------	1969-1971	LAMINAR
SITKA HARBOR	ALASKA U.S.A.	1970	LAMINAR
LULING-DESTREHAN	LOUISIANA U.S.A.	1972-1976	LAMINAR
EAST HUNTINGTON (Steel Box Alt)	WEST VIRGINIA U.S.A.	1977	LAMINAR
DEER ISLE-SEDGWICK	MAINE U.S.A.	1981-1983	LAMINAR
GOLDEN GATE	CALIFORNIA U.S.A.	1984-1985	LAMINAR TURBULENT
DEER ISLE-SEDGWICK	MAINE U.S.A.	1985-1986	LAMINAR TURBULENT
GENERIC "BOX" & "H" SECTIONS	------	1987-1988	LAMINAR TURBULENT

Table 3. Full Scale Field Measurements

BRIDGE	LOCATION	DATE OF MEASUREMENT
NEWPORT	RHODE ISLAND	1969
CHESAPEAKE BAY	MARYLAND	1969
NEWPORT	RHODE ISLAND	1971
SITKA HARBOR	ALASKA	1972-1976
PERRINE	IDAHO	1977-1978
PASCO-KENNEWICK	WASHINGTON	1979-1980
DEER ISLE-SEDGWICK	MAINE	1981-Present
LULING-DESTREHAN	LOUISIANA	1984-Present

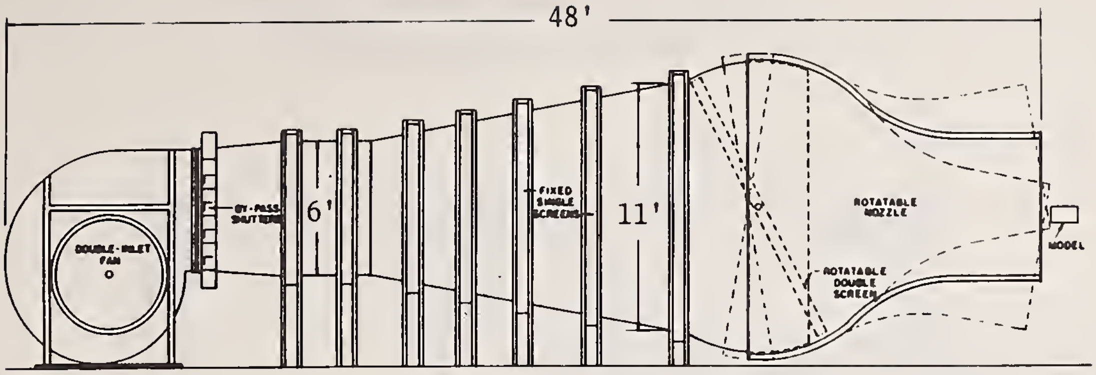
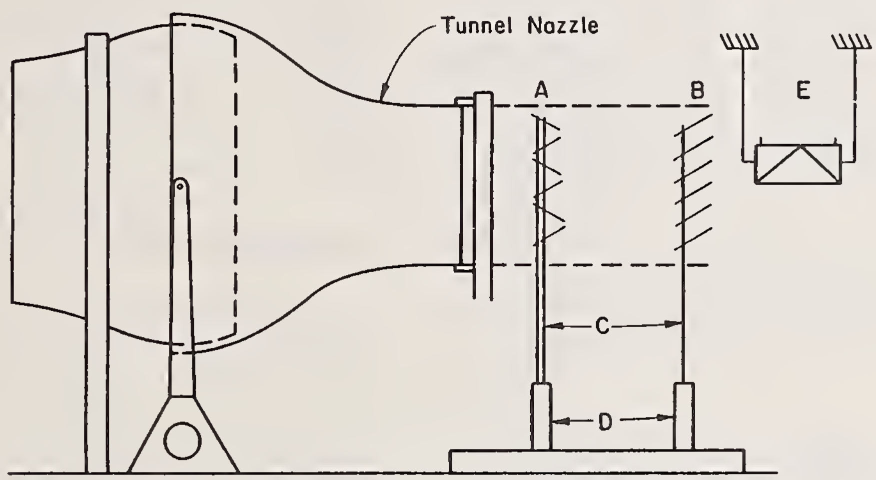


Figure 1. FHWA Wind Tunnel



A = Horizontal Gust Control
B = Vertical Gust Control
C = Linkage
D = Hydraulic Actuators
E = Bridge Model

Figure 2. Concept of Active Turbulence Generator.

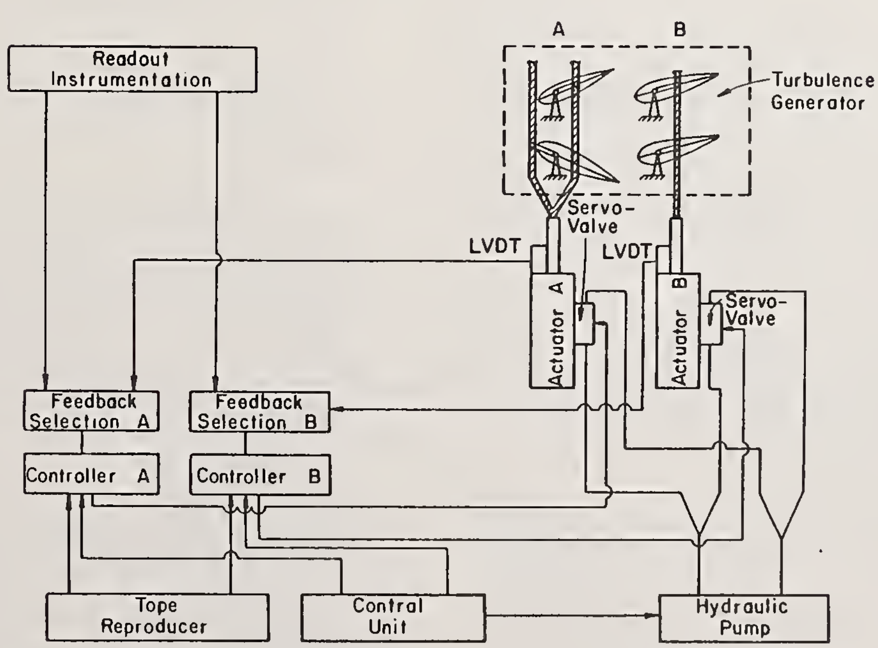


Figure 3. Control-Driving System

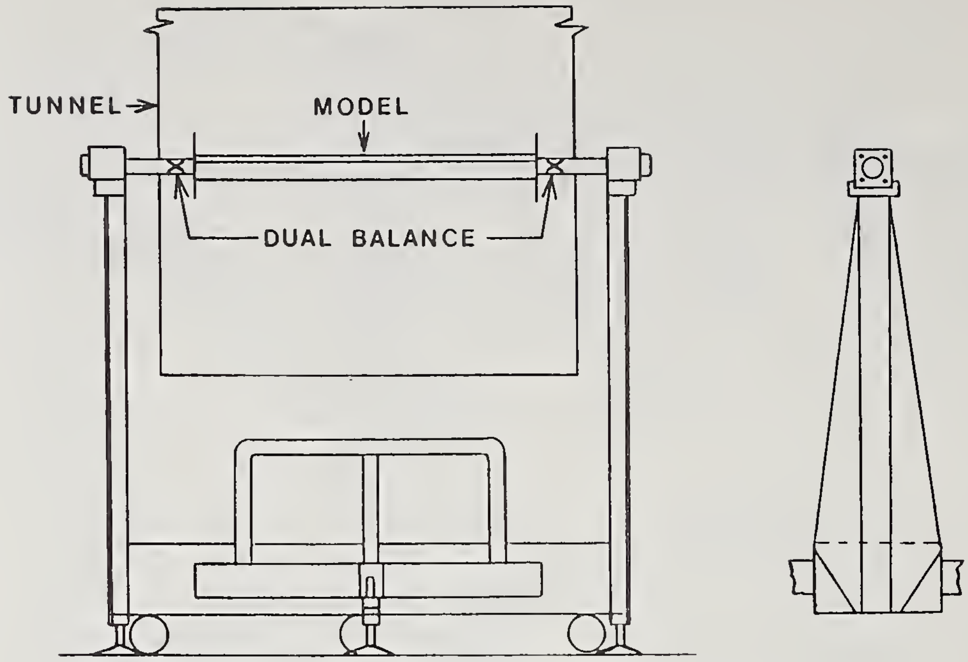


Figure 4. Dual Balance on Mobile Support.

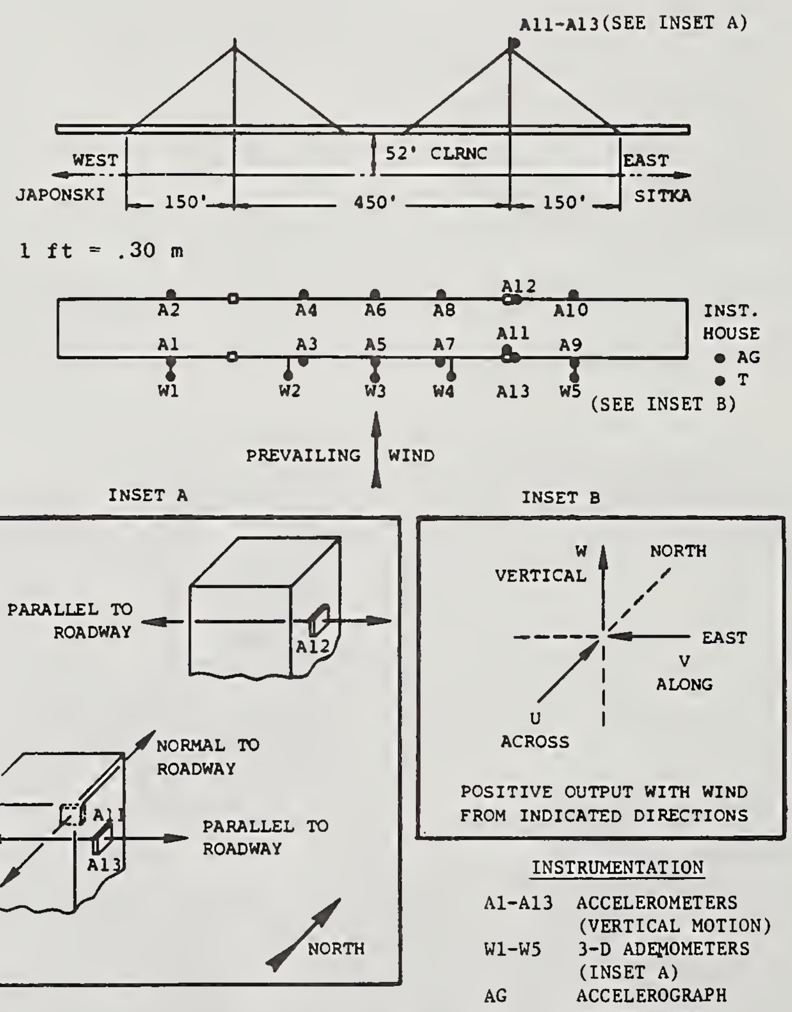


Figure 5. Instrumentation Configuration - Sitka.

Wind Resistant Design Manual for Highway Bridges in Japan

by

Nobuyuki Narita[1], Koichi Yokoyama[2] and Hiroshi Sato[3]

ABSTRACT

The new wind resistant design manual for highway bridges has been required recently in Japan. The working group was organized in the Japan Road Association in 1984 to prepare the manual. The working group was reformed into the wind resistant design committee in 1986, and the first draft of The Proposed Wind Resistant Design Manual For Highway Bridges was completed. The contents of the Manual are as follows.

1. GENERAL REMARKS
2. GUIDELINE OF WIND RESISTANT DESIGN
3. WIND PROPERTIES USED IN DESIGN (Basic Wind Speed, Design Wind Speed, Intensity of Turbulence, etc.)
4. WIND-INDUCED VIBRATIONS OF BRIDGE DECK (Prediction, Evaluation, Method of Alleviation, Flutter, Galloping, Vortex-Induced Vibrations, Gust Responses)
5. WIND-INDUCED VIBRATIONS OF TOWERS AND BRIDGE MEMBERS
6. WIND RESISTANT DESIGN AT ERECTION STAGES
7. WIND TUNNEL TESTING METHODS

KEY WORDS: Wind resistant design, Highway bridge, Design manual, Wind-induced vibration.

1. INTRODUCTION

The design of short-and medium-span highway bridges in Japan has been made according to The Specification For Highway Bridges (hereinafter referred to as 'the Specification'). With regard to wind resistant design, the Specification prescribes design wind load of bridges and dimensions of pipe structure, however, there is not a sufficient provision for wind-induced vibrations in the Specification.

The wind resistant design of long-span bridges in Japan has been mostly based on The Wind Resistant Design Criteria For Honsyu-Shikoku Bridges (hereinafter referred to as 'the Criteria'). The Criteria was originally formulated in 1967 and revised in 1976 by the ad hoc committee of the Japan Society of Civil Engineers. The Criteria prescribes the design wind load including the effect of gust and the evaluation method of wind-induced vibrations based on wind tunnel testing.

Since the revision of the Criteria, wind engineering has made a remarkable progress and a considerable amount of aerodynamic data of bridges have been acquired. The wind resistant design for limited kinds of bridges can be made without wind tunnel testing if reliable formulae for the estimation are provided. The prediction of wind-induced vibration will become more reasonable and reliable if the effect of turbulence is incorporated appropriately in the design.

From these reasons, the new wind resistant design manual for highway bridges has been required recently in Japan. The working group was organized in the Japan Road Association in 1984 to prepare the manual. The working group was reformed into the wind resistant design committee in 1986, and the first draft of The Proposed Wind Resistant Design Manual For Highway Bridges (hereinafter referred to as 'the Manual') was completed.

In this paper described are some of the features of the Manual. It should be noted that the final version of the Manual, which will be published in 1989, will not be necessarily the same as the first draft described in the paper.

2. PROCEDURE OF WIND RESISTANT DESIGN USING THE PROPOSED WIND RESISTANT DESIGN MANUAL

The procedure of wind resistant design using the Manual is shown in Fig. 1. The wind properties (basic wind speed, design wind speed, turbulence intensity etc.) are decided first. Then the design wind load is calculated according to the provisions of the Specification, and the static design of the bridge is made. This procedure is the same as the existing wind resistant design method.

At this stage, where major dimensions of the bridge are determined, wind-induced vibrations to be studied further are chosen considering the design wind speed, the deck width and the span length of the bridge (Table 1). For bridges with short span length, no study on wind-induced vibrations is required.

For other bridges, wind-induced vibrations specified above should be predicted using the formulae provided in the Manual. Then the wind-induced vibrations are evaluated. The formulae were established considering wind tunnel test results. Since only a few parameters are included in the formulae, the prediction error of the formulae is not negligibly small. Therefore, the formulae are established so that their prediction may become

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safer. It means that evaluation 'good' based on the formulae is almost always 'good', however, evaluation 'no good' based on the formulae is not always 'no good' because of the safety margin of the formulae. In case that the evaluation based on the formulae turns out to be 'no good', the bridge engineer can modify the desing or he can predict wind-induced vibrations by means of wind tunnel testing. The evaluation based on wind tunnel testing has priority over that based on the formulae.

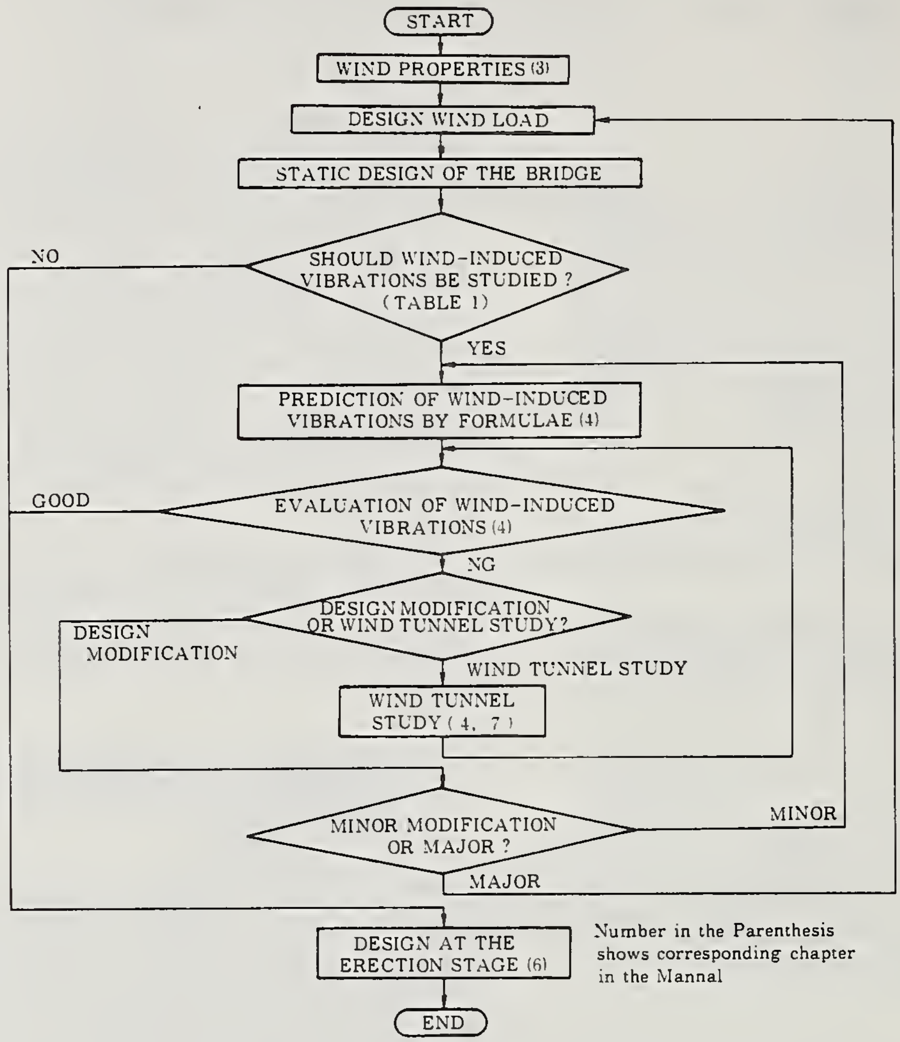

Fig. 1 Procedure of wind resistant design using the Manual

3. WIND PROPERTIES USED IN DESIGN

Wind properties are modeled fundamentally according to Davenport [1978, 1982], namely, lower mean wind speed but higher turbulence in rough terrain and low altitude, and higher mean wind speed but lower turbulence in smooth terrain and high altitude. The terrains are classified into four, namely, rough sea (Terrain I), open farmland (Terrain II), suburbs (Terrain III) and city centres (Terrain IV).

3.1 Basic wind speed U10

The basic wind speed U10 is defined as the mean wind speed over farmland (Terrain II) at an elevation of 10m, averaged over a period of 10 minutes. The probability that the annual maximum mean wind speed exceeds the basic wind speed is 10% in a 50-year period.

Using the meteorological data at weather stations in Japan, extreme wind speeds were estimated. The basic wind speeds were classified into 4 categories, namely 35m/s, 40m/s, 45m/s, 50m/s. It is shown in Fig. 2.

3.2 Design wind speed Ud

The design wind speed can be obtained from the following formula.

$$Ud = U10\ Eul \tag{1}$$

where, Eul: correction factor for altitude and terrains

The design wind speeds and the correction factors are shown in Fig. 3.

Table 1   Type of wind-induced vibrations to be studied further

Bridge Deck	Criteria	Wind-induced Vibration to be studied
Truss Deck for Cable-Stayed or Suspension Bridge	if $L > 385B/U_d$	then Flutter
Open Solid Deck for Cable-Stayed or Suspension Bridge	$L > 200B/U_d$	Vortex-induced Vibration
	$L > 346B/U_d$ and $B < 5D$	Galloping
	$L > 385B/U_d$	Flutter
Closed Solid Deck for Cable-Stayed or Suspension Bridge	$L > 200B/U_d$	Vortex-induced Vibration
	$L > 346B/U_d$ and $B < 5D$	Galloping
	$L > 577B/U_d$	Flutter
Girder Bridge	$L > 200B/U_d$	Vortex-induced Vibration
	$L > 346B/U_d$ and $B < 5D$	Galloping

where, L : main span length
B : bride deck width
U_d : design wind speed
D : bridge deck depth

Fig. 2 Basic wind speed

3.3 Turbulence properties

Typical values for turbulence properties such as turbulence intensities and power spectral density functions are provided in the Manual. Turbulence intensity Iu is shown in Fig. 4.

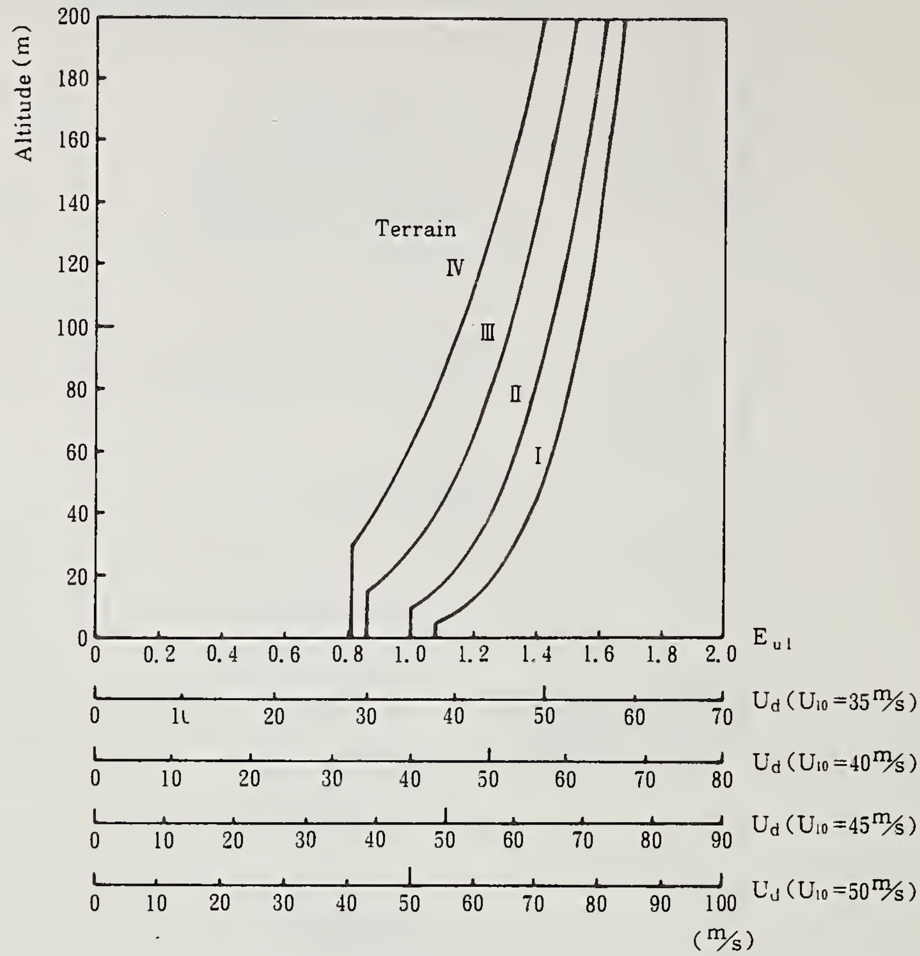

Fig. 3 Correction factor Eul and design wind speed

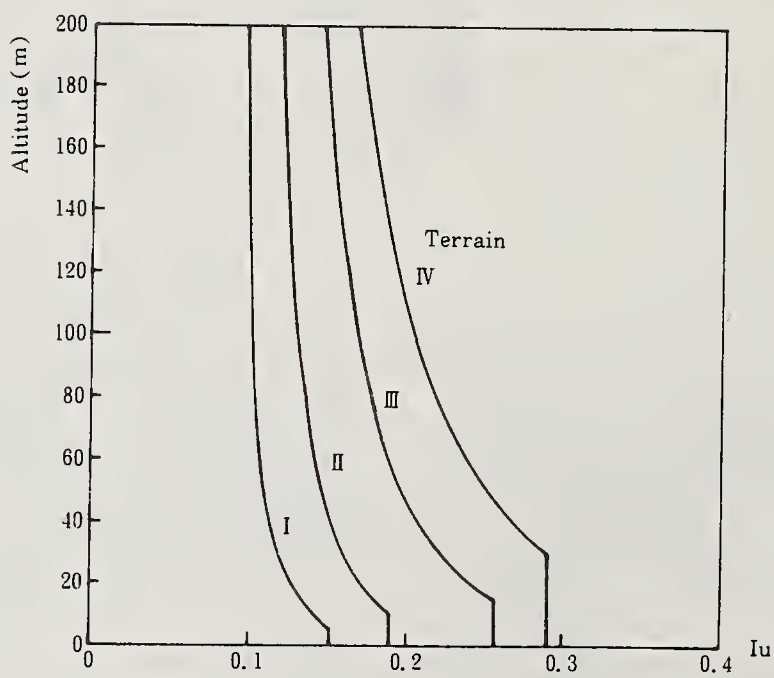

Fig. 4 Trubulence intentity

4. PREDICTION OF WIND-INDUCED VIBRATIONS BY THE FORMULAE

4.1 Flutter

Reduced critical wind speeds for flutter of solid decks are shown in Fig. 5. It seems that the reduced critical wind speed increases with an increase in deck width to depth ratio (B/D). The lowest reduced critical wind speed is about 2.5. As for a truss deck, the critical wind speed varies much with minor difference of configuration of truss and floor deck. But the reduced critical wind speed is higher than 2.5.

The formulae for the prediction of the critical wind speed of flutter is as follows:

$$Ucf = Ucfo\ Ef \qquad (2)$$
$$Ucfo = 2.5\ f\theta B \qquad (3)$$

where,

- Ucf: corrected critical wind speed for flutter.
- Ef: correction factor for the effect of turbulence.
- Ucfo: critical wind speed for flutter in smooth flow.
- $f\theta$: natural frequency of the 1st torsional mode.

Sinusoidal wave form observed in smooth flow changes into a random form in turbulent flow. The effect of turbulence on flutter, which is not so clear at the moment, will be incorporated into Ef.

4.2 Galloping

Galloping may take place for a solid deck whose deck width to depth ratio (B/D) is relatively small. Fig. 6 suggests that the critical wind speed for galloping is very high when B/D is over 5. For a solid deck whose B/D is less than 5, the lowest reduced critical wind speed is about 4.5.

The formula for the prediction of the critical wind speed of galloping is as follows:

$$Ucg = Ucgo\ Eg \qquad (4)$$
$$Ucgo = 4.5\ f\eta B \qquad (5)$$

where,

- Ucg: corrected critical wind speed for galloping
- Eg: correction factor for the effect of turbulence
- Ucgo: critical wind speed for galloping in smooth flow
- $f\eta$: natual frequency of the 1st bending mode.

Sinusoidal wave form observed in smooth flow changes into a random form in turbulent flow. It was found that onset velocity for negative damping, which causes galloping, increases remarkably in turbulent flow (Narita et al. [1987]). The effect of turbulence will be incorporated, for example, into Eg.

4.3 Vortex-induced vibrations

Vortex-induced vibrations may take place for a solid deck. The amplitude decreases with an increase in mass or mass moment of inertia and structural damping. The amplitude is very sensitive to the geometrical configuration, however, the amplitude decreases with an increase in deck width to depth ratio (B/D).

On the other hand, the variation of reduced wind speed for the occurrence of vortex-induced vibrations is smaller than that of the amplitude.

In the turbulent flow, the amplitude becomes smaller than in smooth flow. This effect becomes more significant with higher turbulence intensity and cross sections of large B/D. The ratios of the amplitude of vortex-

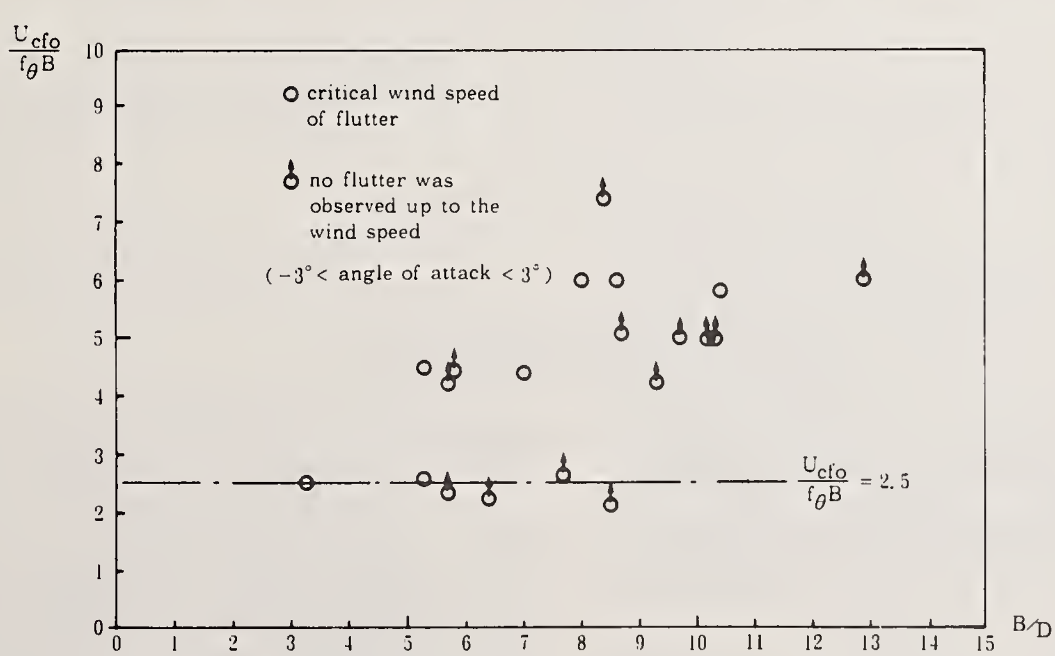

Fig. 5 Reduced critical wind speed for flutter of solid deck

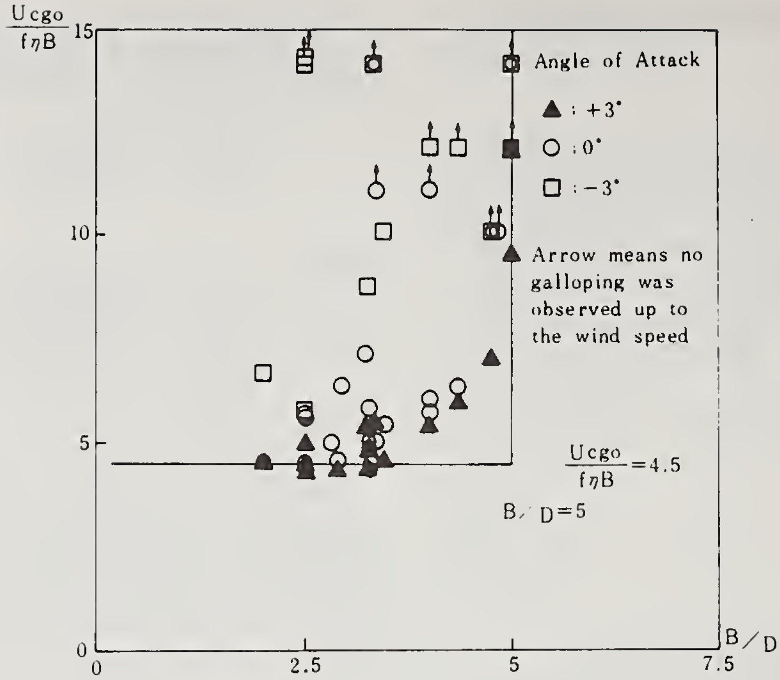

Fig. 6 Reduced critical wind speed for galloping of solid deck

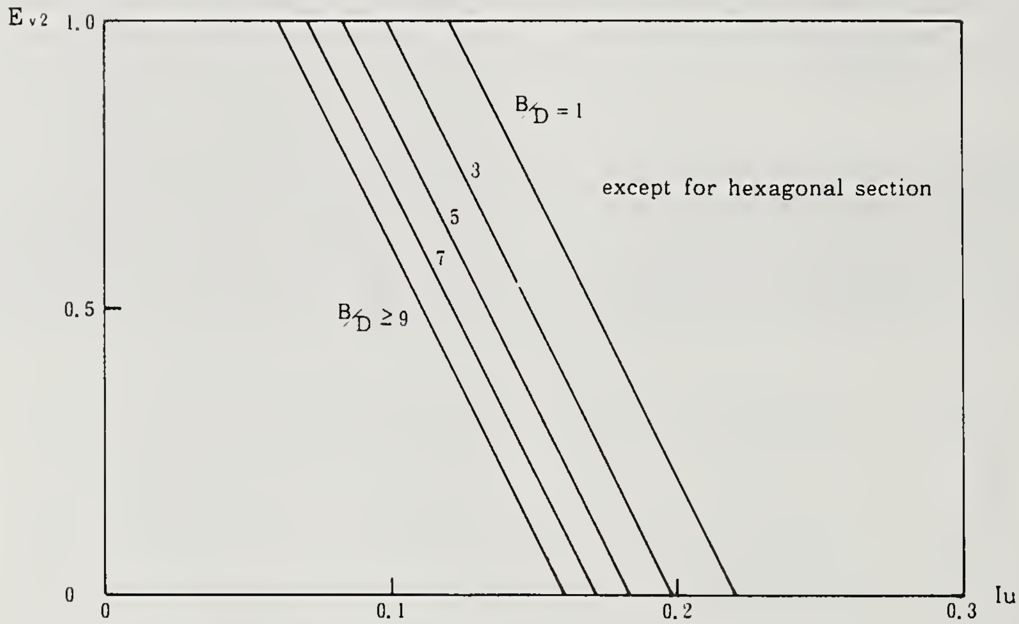

Fig. 7 Correction factor for effect of turbulence on amplitude of vortex-induced vibrations

induced vibration in turbulent flow to that in smooth flow suggested by several wind tunnel tests are shown in Fig. 7.

The formula for the prediction of the critical wind speed and amplitude of vortex-induced vibrations is as follows:

$$UcV = \begin{cases} \beta\eta \cdot f\eta \ B & \text{..... bending} \\ \beta\theta \cdot f\theta \ B & \text{..... torsion} \end{cases} \quad (6)$$

$$A = Ao \ Evl \ Ev2 \quad (7)$$

$$Ao = \begin{cases} \alpha\eta(D/B)^2/(mr\ \delta\eta) & \text{..... bending } (\eta/B) \\ \alpha\theta(D/B)^3/(Ir\ \delta\theta) & \text{..... torsion (deg.)} \end{cases} \quad (8)$$

where,

Ucv: wind speed for the maximum amplitude of vortex-induced vibration

$\beta\eta$, $\beta\theta$: reduced wind speed for vortex-induced vibration (Provision of Hanshin Expressway Public Corporation [1984] seems to be appropriate.)

A: corrected maximum amplitude of vortex-induced vibration

Ao: maximum amplitude of vortex-induced vibration for rigid model in smooth flow

Evl: correction factor for vibrational mode

(about $4/\pi$)

Ev2: correction factor for the effect of turbulence (Fig. 7)

$\alpha\eta, \alpha\theta$: effect of sectional shape. (Provision of Bridge Aerodynamics [1981] seems to be appropriate.)

mr, Ir: reduced mass or reduced mass moment of inertia

$\delta\eta, \delta\theta$: logarithmic decrement

5. PREDICTION OF WIND INDUCED VIBRATIONS BY WIND TUNNEL TESTING

Wind tunnel testing increases the reliability of wind resistant design of bridges. Since the prediction of wind-induced vibrations based on the formulae may often provide safer value, there is a fair chance that a wind tunnel study provides an economical design.

As is mentioned above, turbulence has significant effects on wind-induced vibrations. Therefore, standard wind tunnel testing methods in turbulent flow (full aeroelastic model test, taut-strip model test) are described in the Manual as well as conventional testing methods (spring-mounted rigid model test, measurement of steady aerodynamic forces).

6. EVALUATION OF WIND-INDUCED VIBRATIONS

6.1 Flutter and galloping

The following inequalities should be satisfied.

$$Ucf > Ur \quad \text{..... Flutter} \tag{9}$$

$$Ucg > Ur \quad \text{..... Galloping} \tag{10}$$

$$Ur = Ud \; Er1 \; Er2 \tag{11}$$

where,

Ur: reference wind speed for flutter and galloping

Er1: correction factor for the effect of gust and rapid growth of flutter or galloping

Er2: safety factor

6.2 Vortex-induced vibrations

The following inequality should be satisfied.

$$Ucv > Ud \tag{12}$$

In case that Ucv $\leqq$ Ud, the amplitude A should be examined in view point of safety of the bridge, fatigue and serviceability.

7. OTHERS

Besides the wind resistant design method of bridge deck mentioned above, the Manual describes the design methods for structural members such as towers and cables, and the design methods for bridges at their erection stages.

8. CONCLUSION

The new wind resistant design manual is now being prepared in Japan. The main features of the proposed wind resistant design manual for highway bridges are as follows.

a. The basic wind speeds of the whole country are provided.
b. The design wind speed and the properties of wind turbulence can be estimated considering the altitude of structure and the terrain around the structure.
c. The critical wind speed for flutter and galloping can be obtained by simple formulae as well as by wind tunnel testing.
d. The amplitude of vortex-induced vibrations can be estimated by simple formulae as well as by wind tunnel testing. The effect of turbulence on vortex-induced vibrations are incorporated.
e. The method of verification of flutter, galloping and vortex-induced vibrations are provided.
f.. The guideline of wind resistant design for bridge members and for bridges at their erection stages are described.
g. The standard wind tunnel testing methods are described.

ACKNOWLEDGEMENT

The authors would like to express their gratitude to all the members of wind resistant design committee in Japan Road Association. The discussion at the committee was essential for preparing the paper as well as the Manual.

REFERENCES

Davenport, A.G. 1978, "Wind structure and wind climate", Safety of Structures under Dynamic Loading, Trondheim, TAPIR, V.1, pp.209-256.

Davenport, A.G. 1982, "The interaction of wind and structures', Engineering Meteorology, Amsterdam, Elsevier, pp.527-572.

Hanshin Expressway Public Corporation 1984, Proposed Design Rule on Dynamic Effects of Wind on Bridges, (in Japanese).

ICE 1981, Bridge Aerodynamics, Proposed British Design Rules, TTL.

Narita, N., Yokoyama, K., Sato, H. and Nakagami, Y. 1987, "Aerodynamic characteristics of continuous box girder bridges relevant to their vibrations in wind", Seventh International Conference on Wind Engineering, Aachen, V.4, pp.283-292.

Field Measurements of Wind Pressures

by

Kishor C. Mehta[1]

ABSTRACT

This paper describes a field project sponsored by the National Science Foundation to obtain a reliable data base of wind induced pressures on building surfaces. A test building with the dimensions of 30 x 45 x 13 ft (9.1 x 13.7 x 4.0 m) and a 160 ft (48.8 m) tall meteorological tower are constructed in an open field. Wind speed, wind direction, and building surface pressures are measured simultaneously for a duration of 15 minutes when wind speeds exceed 20 mph (8.9 mps). The computerized data acquisition system records data at the rate of 10Hz for each of 40 channels. The meteorological data and pressure data instruments are installed and calibrated. Preliminary terrain characteristics and pressure results are obtained. Specific goal-oriented data collections are planned for the future.

KEY WORDS: Wind; Pressures; Field Data; Building

1. INTRODUCTION

As evidenced by wind-induced building damages around the world, many low-rise structures are susceptible to severe damage in windstorms. A greater understanding of what pressures the wind exerts on a building will assist in mitigating this damage. Toward that end, a unique facility has been constructed at Texas Tech University which permits research in wind effects on a low-rise building in the field.

The facility has the capability to measure pressures on the building surface at as many as 40 locations simultaneously. The 30 x 45 x 13 ft (9.1 x 13.7 x 4.0 m) test building can be rotated, allowing positive control over wind angle-of-attack. Meteorological conditions are monitored by instruments mounted on a 160 ft (48.8 m) tower located at the site. The test building and the tower are shown in Figure 1.

The research, sponsored by the National Science Foundation (NSF), concentrates on acquiring a reliable database of external and internal pressures on the test building. Other preliminary objectives include characterization of the terrain and verification of the pressure measuring system. This paper describes the facility, instrumentation, data acquisition system and preliminary results.

2. FIELD FACILITY

The field facility has two main components: the building and the meteorological tower. As shown in Figure 2, the outer test building is mounted on a track for rotation and the inner data acquisition building is fixed on a concrete slab. Pressure taps are mounted on surfaces of the test building while the computerized data acquisition system is housed in the fixed building.

The facility is located on a flat, open site, about two miles from the university campus. The city of Lubbock is situated on the high plains of Texas, elevation 3300 ft (1000 m) above sea level. The terrain is very flat, typically changing elevation at a rate of 5 in 1000. There are no significant hills or valleys within 20 miles (32 km) of the city. Most of the land is cultivated, the chief crops being cotton and sorghum. The flat and open nature of the countryside minimizes possible anomalies in wind characteristics caused by terrain. Additional descriptions of the surrounding terrain can be found in papers by Levitan, et al. [1] and Lakas, et al. [2].

The test building is a prefabricated metal building. It has smooth skin with no architectural features, making the building shape as simple as possible. The building has one door and one window. The entire test building is constructed on a rigid frame undercarriage. This carriage has a wheel in each corner which rests on a circular steel track embedded in the concrete slab. The undercarriage rests on the slab except during the rotation operation of the building. Hydraulic jacks at each corner lift the building (total weight of approximately 16,000 lb). Motors at two opposite corners rotate the building at the rate of 5 fps (1.5 mps). The building can be rotated a full 360°, providing positive control of the wind angle-of-attack. Anchor bolts are placed at 15° intervals around the circle.

The data acquisition building is a fixed structure of reinforced CMU construction, 12 x 12 x 8 ft (3.7 x 3.7 x 2.4 m). The meteorological instruments are mounted on a 160 ft (48.8 m) high three-legged guyed tower. The tower is guyed at two levels, 70 ft (21.3 m) and 130 ft (39.6 m). The base of the tower is 150 ft (45.7 m) west

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of the test building. Retractable booms extending 6 ft (1.8 m) from the tower are provided for wind speed anemometers at heights of 13, 33, and 70 ft (4, 10 and 21.3 m). A reference pressure box is located 50 ft (15.2 m) west of the test building. The box, of dimensions 5 x 4 x 3 ft (1.5 x 1.2 x 0.9 m), is placed below the ground surface. An 8 in. (20 cm) diameter PVC pipe extends underground from the reference pressure box to the data acquisition building.

3. INSTRUMENTATION

The meteorological instrumentation consists of four wind speed anemometers, two wind direction vanes, two temperature sensors, one relative humidity sensor, and one barometric pressure sensor. The wind speed anemometers are the 3-cup type and are located on booms at heights of 13, 33, and 70 ft (4, 10, and 21.3 m) and at the top of the tower, at the height of 160 ft (48.8 m). Wind direction vanes are placed at heights of 33 and 160 ft (10 and 48.8 m). Temperature, relative humidity, and barometric pressure sensors are mounted at the 13 ft (4 m) level on the tower.

Differential pressure transducers are used to measure external and internal pressures on the building. Two types of differential pressure transducers are used, thirteen Validyne model DP103-22N-7-S-4-H and twenty-eight Omega model PX 163-005 BD 5v. The transducers have a range of ±0.2 psi (±1.38 kPa). Validyne transducers have removable stainless steel diaphragms, and are of the variable reluctance type. Omega transducers have a fixed silicon diaphragm. Validyne transducers are more versatile and rugged.

The data acquisition system consists of an IBM PC XT microcomputer with an internal 20-megabyte hard drive. Analog voltages from the instruments are converted to digital form by a MetraByte DAS-8 high-speed A/D converter. The DAS-8 has a 12-bit resolution and an input range of ±5 volts. The system can record data at rates of up to 1200 samples per second (total for all channels). The data is streamed directly to a 20-megabyte Bernoulli removable cartridge drive, thus providing an easy way to transport the data from the field to the campus.

The pressures on building surfaces are measured through 3/8 in. (10 mm) diameter pressure taps mounted flush with the surface of the building. A 4 in. (102 mm) long 3/8 in. (10 mm) inside diameter flexible plastic tubing connects the pressure tap to a three-way solenoid valve. Ten inches (254 mm) of 3/16 in. (5 mm) inside diameter flexible plastic tubing connects the solenoid valve to the differential pressure transducer. The tubes are long enough to assemble the system in the field and are short enough to provide good response without distortion. A general setup of the pressure transmission system is shown in Figure 3. Approximately 35 ft (10.7 m) of 3/16 in. (5 mm) inside diameter flexible plastic tubing is used to connect the transducer to the reference pressure pipe in the data acquisition room. Dynamic response of the system is shown in Figure 4. The mean response values are good for frequencies up to 100 Hz; however the standard deviation values are not unity beyond 20 Hz. Recognizing that distorted responses will give ratios other than unity, undistorted measurements can be obtained with this system up to the frequency of 20 Hz. Details for calibrating the pressure measuring system are given in a paper by Ng and Mehta [3].

4. CURRENT PROGRESS

A wind tunnel study to determine the influence of the surrounding buildings on the external pressures has been carried out. Results of this study are outlined in the paper by Lakas, et al. [2].

Meteorological data have been collected during strong winds since early 1988. Of the first 63 records collected, 31 are of stationary winds. Analysis of the 31 records, each of 15-minute duration with mean wind speeds between 21 and 33 mph (9.4 and 14.8 mps), at the height of 33 ft (10 m), provides values for the power law coefficient, roughness length, and turbulence intensity. These values are shown in Table 1. Detailed analysis of these data are given in a master's thesis completed at Texas Tech University [4]. The azimuth zones for terrain surrounding the site are indicated in Figure 5.

Typical recordings of wind speed, wind direction, and pressure fluctuations, recorded simultaneously, are shown in Figure 6. The pressure values are positive, as expected, since this pressure tap is in the windward wall. The recordings indicate large fluctuations in wind speed and direction encountered in the field. Analysis of these recordings are in progress.

5. CONCLUSIONS

The field facility is constructed and operational. It is capable of collecting data that will assist in understanding wind loads on low-rise buildings. The facility has the potential for many other types of research. Topics such as measurement of the total wind force exerted on the building, load sharing and load path research, and effects of varied building geometry on pressures are possibilities. In addition, research in building ventilation is under consideration.

6. ACKNOWLEDGEMENTS

The research presented here is supported by NSF Grant No. CES8611601. Eleanora Sabadell and Michael P. Gaus, both of NSF, provide a continued encouragement to pursue field research. John D. Holmes, Senior Research Scientist from CSIRO, Australia, is a Fulbright scholar at Texas Tech during March-June, 1989, to

conduct research on this project. His significant contributions in calibrating reference pressure and analyzing recorded data are acknowledged.

7. REFERENCES

[1] Levitan, M.L., Mehta, K.C., Chok, C.V. and Millsaps, D.L., "An Overview of Texas Tech's Wind Engineering Field Research Laboratory," Proc., Sixth U.S. Conference on Wind Engineering, University of Houston, Houston, TX, March 1989.

[2] Lakas, B.D., Mehta, K.C., Kiesling, E.W., and Dunn, J.R., "Wind Tunnel Simulation of the Texas Tech Research Building: Effects of Terrain Features and Building Orientation," Proc., Sixth U.S. Conference on Wind Engineering, University of Houston, Houston, TX, March 1989.

[3] Ng, H.H.T. and Mehta, K.C., "Pressure Measuring System for Wind-Induced Pressures on Building Surfaces," Proc., Sixth U.S. Conference on Wind Engineering, University of Houston, Houston, TX, March 1989.

[4] Chok, C.V., "Wind Parameters of the Texas Tech University Field Site," a master's thesis submitted to the Graduate Faculty of Texas Tech University, Lubbock, TX 1988, 90 pp.

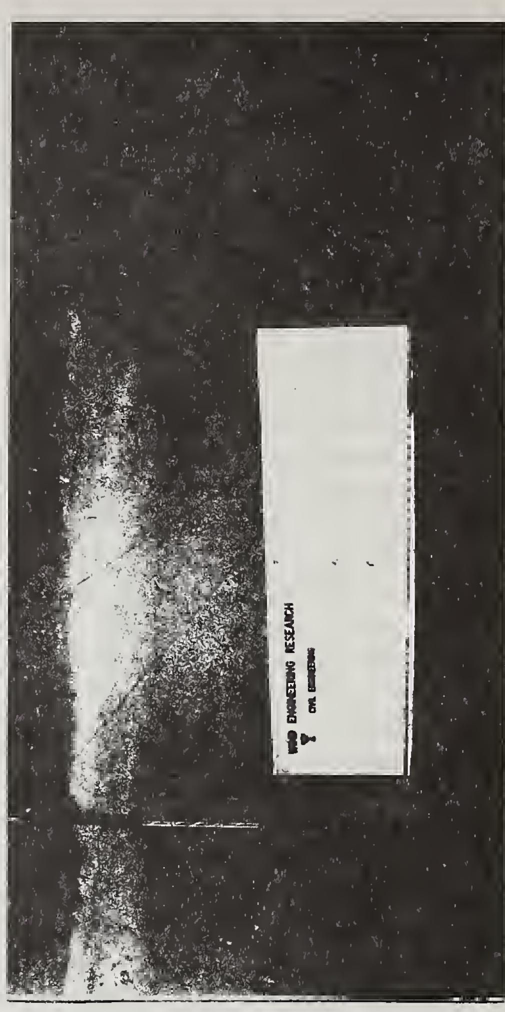

Fig. 1. Test building and meteorological tower

Table 1. Preliminary Terrain Parameters at the Test Site

Azimuth Zone (see Fig. 5)	No. of Records	Power-law Exponent α	Surface Roughness Length Z_o, ft (cm)	Turbulence Intensity at 33 ft (10 m)
270° - 70°	24	0.14 (0.14-0.17)*	0.045 (1.14)	0.17
70° - 160°	-	-	-	-
160° - 210°	3	0.15 (0.14-0.16)	0.059 (1.50)	0.18
210° - 270°	4	0.11 (0.10-0.13)	0.007 (0.18)	0.18

* Range of values

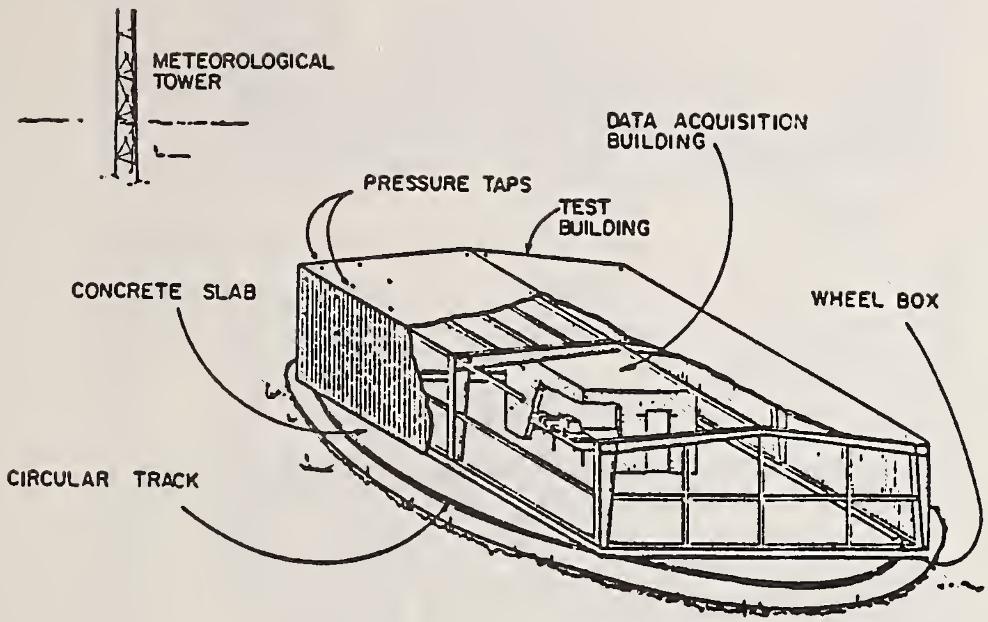

Fig. 2. Cutaway view of the test building

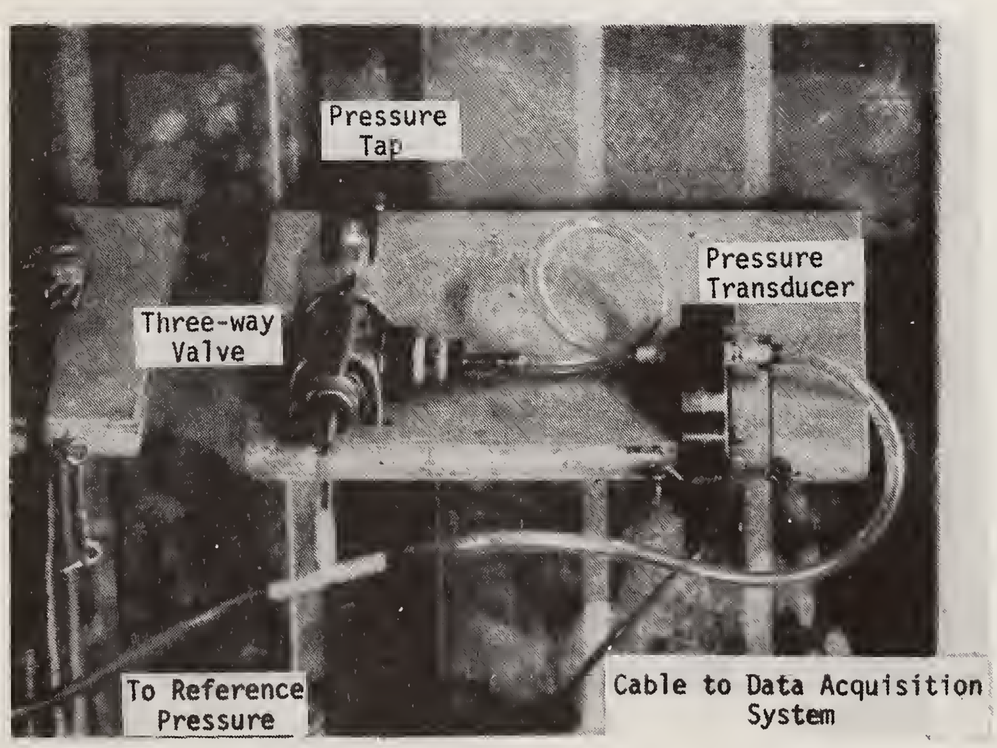

Fig. 3. Pressure transmission system

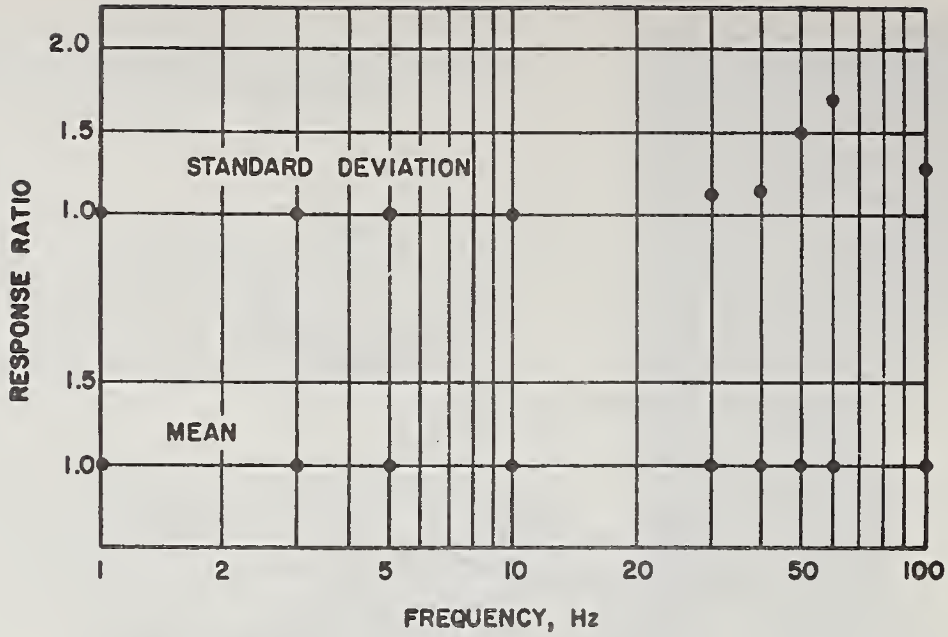

Fig. 4. Frequency response of the pressure transmission system

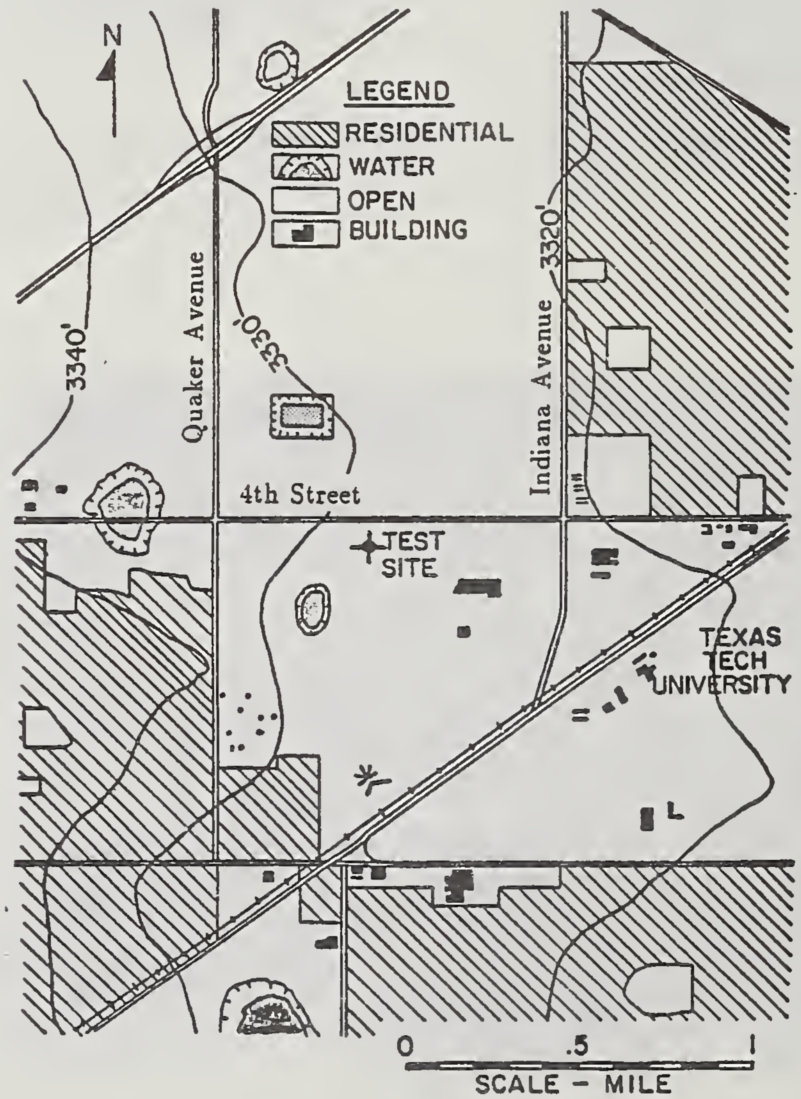

Fig. 5. Map of the site

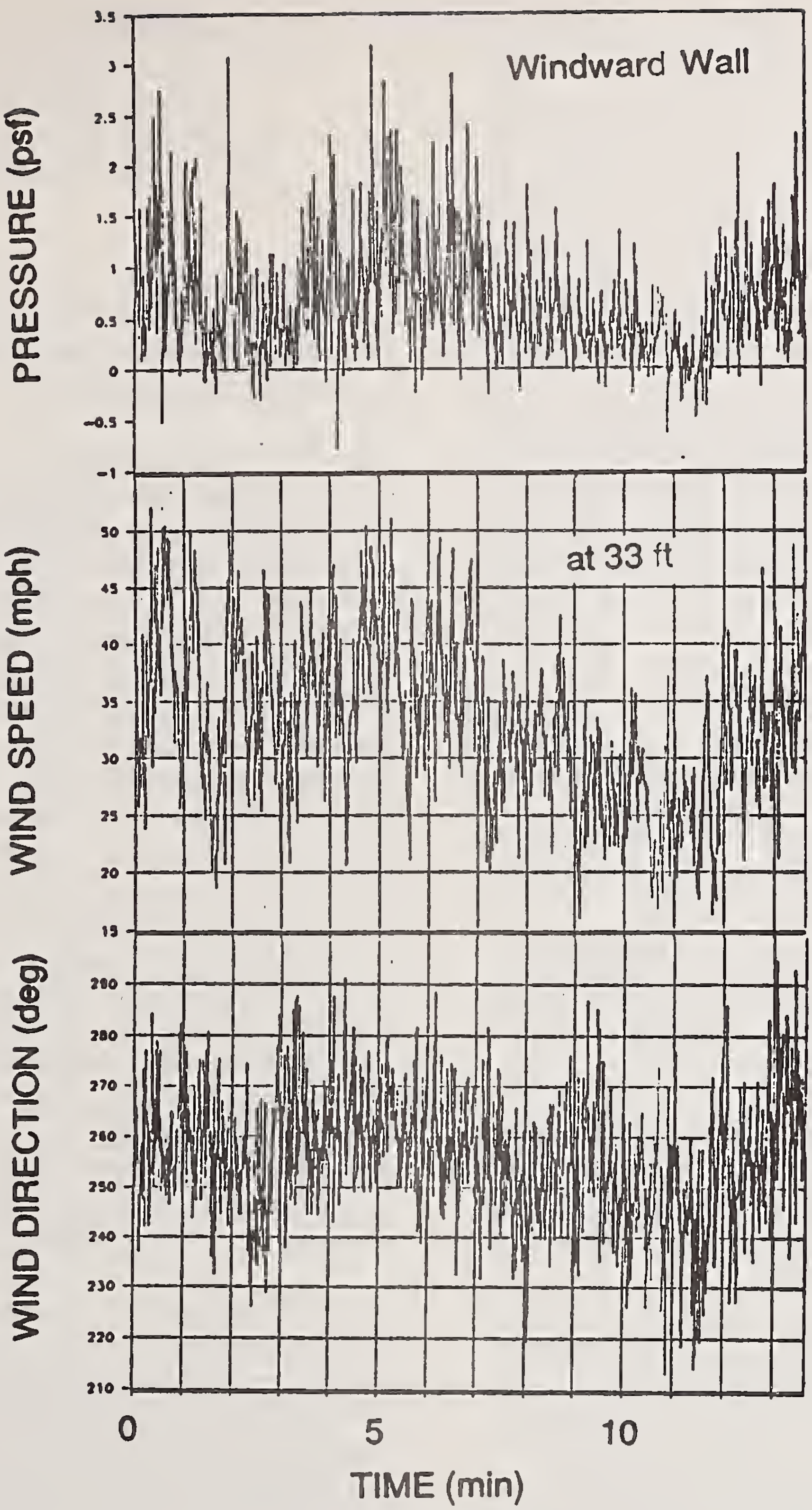

Fig. 6. Wall pressure, wind speed, and wind direction records

Transfer of Research Results in Wind Engineering

by

J. Eleonora Sabadell[1]

ABSTRACT

Brief comments on three types of transfer of research results will be presented here first, on the transfer of knowledge in general; second, on the transfer in the area of wind engineering; and third, when these exchanges occur between countries at different levels of development and resources.

1. TRANSFER OF KNOWLEDGE

Scientific and technical knowledge, in order to be put to use, has to follow certain processes: it must be generated (research); it has to be communicated and analyzed (information and intelligence); then assessed for adoption (evaluation, scenarios, alternatives); and lastly it has to be implemented into action (utilization, diffusion) when the innovation is proven appropriate (Vlachos, 1978).

The generation of new knowledge and the issues involved in identifying, organizing, managing and supporting needed research in general, and in wind engineering in particular, will not be discussed in this presentation.

Many of the problems that can arise during the process of communicating knowledge can be captured into two questions: where does the information come from, and where does it go. The different characteristics of the source of information as well as the material itself, e.g., disciplinary or multidisciplinary, private or public, basic or applied, to name only a few, are great in number and complex in nature. The same can be said about the receptor of this knowledge. What has been found is that sender and recipient often work at quite different "wavelengths".

The capabilities for analyzing and determining the quality and appropriateness of any exchanged piece of information vary widely among cultures, disciplines, localities, sectors, institutions, and levels of government. This can be a substantive problem, especially for the recipient, when its capacity is lower than the source. Then transfer may become dumping.

Next, and when information has been transformed into intelligence, the adoption process starts. For this purpose further evaluation is necessary, e.g., by building scenarios for different alternative use of the new idea, by ascertaining that it will be useful and timely in solving the specific problem at hand, by evaluating the costs and benefits of the different choices. During this part of the process the capabilities of the organizations and professionals involved in these evaluations are of critical importance. Unfortunately, this kind of expertise is not always available.

The last stage is the adoption of the new idea and its incorporation into a technique, measure, regulation or policy addressing a particular problem. The institutional arrangements supporting, carrying out and disseminating the innovation in its various forms, is the key factor for a successful transfer. The existing variety of national, regional, and local organizations is large enough as to make coordinated efforts in the adoption of innovations not an easy task.

An extra step that is often neglected, is the formal evaluation of the gains/losses made by adopting and implementing the innovation. Feedback on successes and failures is not easy to obtain and this is probably due in great measure to the amount of the free flow of information allowed or possible within and between countries.

2. TRANSFER OF NEW KNOWLEDGE IN WIND ENGINEERING

Wind engineering is a relatively young and interdisciplinary area which addresses a variety of issues, e.g., buildings aerodynamics, structural systems and materials and wind-load criteria, wind energy, dispersion of pollution, transport of

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particulates, airflow over complex terrain, to name only few. Other emerging concerns are in the area of social and economic damages assessment and mitigation, legal and insurance considerations, risk-benefit analysis (Meroney, 1987).

Wind engineering is also a very good example of the complexity of a given research field, and of the difficulties and opportunities investigators face when transferring the generated new knowledge to the practitioner.

To begin with, many disciplines are involved in wind engineering: fluid mechanics, aerodynamics, meteorology, structural mechanics, architecture, mathematics, computer science, forestry, agriculture, economics, psychology, and other social sciences. The difficulty of communicating between disciplines is a recognized problem, as it is to relate researchers within a single discipline but in different topical areas, e.g., wind characteristics and wind transport of pollutants.

In the United States the acknowledgment by the construction and insurance industries, and by regulatory agencies that results of wind engineering research should be integrated, or at least tested, in the design and construction of, and standards and codes for buildings and structures, has proven to be laborious, lengthy and often ignored especially for single-family homes. This fact is distressing because it has been estimated that losses from windstorms exceeds three billion dollars annually (Petak and Atkisson, 1982) and this figure continues to grow.

On the other hand, death and injuries caused by wind-related natural hazards are low because warning systems have improved markedly in the last two decades especially for hurricanes. In the case of tornadoes, warning is far from perfect, but engineered buildings and structures designed and built following adopted codes and better techniques can successfully protect their occupants and suffer limited structural damage. This is not the case for low-rise and mobile houses and commercial buildings where most loss of life and injuries still take place. These are some examples of how new knowledge has been transferred in certain areas better than in others.

The wind engineering community is composed of scientists and engineers whose work is much better integrated than in most other areas of inquiry. This fact is probably due to the youth of the field and to the early recognition of the complex nature of this very real problem. Even so, two main groups of researchers have been operating almost from the start: the wind tunnel/physical simulation group, and the full-scale structural behavior investigators. To these two groups all the other subgroups are attached. Communication between these researchers has been quite good but they continue to operate mainly on their own areas of interest. It is evident that there is room for improvement in cooperation and communication, as it is between the producers and the users of innovations. It has been found, time and time again, that the results of research where the user participates in a project from the start, have a much higher probability to be adopted into practice. Many wind engineering specialists have arrived to another important conclusion, and this is: that knowledge transferred is constrained by economic, legal and political factors far more than by the availability of technical and scientific knowledge.

3. TRANSFER OF RESEARCH RESULTS BETWEEN COUNTRIES

During the past four decades there have been many activities in the transfer of knowledge from countries with advanced state of experience to countries with less expertise, capabilities and resources. These transfers have been supported by governments, international organizations in all areas of aid, as well as financial institutions. The results have been mixed, with successes where cultural, economic, educational and other differences were well understood by the donor or conveyer country, and with failures where the transfer of knowledge or technology was done "as is" without any adjustment to the new environment.

Similarly, any wind engineering technique, measure, policy or any other innovation generated by a developed country, located mainly in a temperate zone, with stable population, rich in resources, with strong human capabilities and an excellent communication system, can be successfully transferred to another country that has limited resources and is often located in a completely different climatic zone of the world, only if those differences are well recognized from the start of any exchange, and if the consequences of any change are carefully assessed.

Another important point to be made is that transfer of knowledge does not occurred in only one direction. To the contrary, countries at any stage of their development have much to learn from each other.

4. REFERENCES

1. Vlachos, Evan, A Conceptual Model of the Knowledge Transfer Process, "Water Knowledge Transfer", by Neil Grigg, Ed., Water Resources Publications, Fort Collins, Colorado, 1978.

2. Meroney, Robert N., Proceedings of Seminar/Workshop on Wind Engineering, "The Past to the Future", Colorado State University, 1987.

3. Petak, W. J. and Atkisson, A. A., "Natural Hazard Risk Assessment and Public Policy: Anticipating the Unexpected", Springer-Verlag, New York, 1982.

Wind Resistance in Manufactured Housing in Hurricane Zones

by

G. Robert Fuller, P.E. [1]

ABSTRACT

Each year in the United States, significant damage to manufactured (mobile) housing is caused by high winds, hurricanes and tornadoes. In 1987, 115 deaths occurred in manufactured housing from wind-related causes. This paper is a review of recent research and investigations into the anchorage and installations of manufactured housing in hurricane zones.

KEYWORDS: Anchorages; foundations; hurricanes, manufactured housing; tie-downs; wind resistance.

1. WIND DAMAGE IN THE U.S.

According to a study funded by the National Science Foundation (Wiggins, 1977) wind damage to buildings in the United States had exceeded $2 billion per year by 1976. Estimated annual wind damage in 1988 was $4 to $5 billion. This doubling of cost from 1976 to 1988 was not only due to inflation, but also due to increased housing construction, particularly in coastal areas affected by hurricanes. The cost due to wind damage is higher than that due to any other natural hazard, including earthquakes, volcanoes, expansive soils and floods. Storms and weather kill more people in the U.S. than any other natural hazard averaging approximately 350 people per year (Gaus, 1985). Wind is one of the worst natural hazards in the United States [1].

Wind caused damage to buildings is most widespread for non- engineered and marginally engineered woodframe structures, such as housing, motels, stores, shopping centers, schools and churches. It was because of extensive damage to these types of structures, that the American Society of Civil Engineers (ASCE) in 1985 established a "Task Committee on Mitigation of Damages Due to High Winds." Included in its purpose was that the committee should "apply recent advancements in building aerodynamics to study this problem." [2]

The Tasks of the Committee were as follows:

- To apply new knowledge in building aerodynamics and wind engineering to make buildings, especially non-engineered buildings, more resistant to high winds.
- To review wind load related provisions and national and local building codes to determine their adequacy.
- To study the enforcement of provisions by building code officials, and compliance by contractors and builders.
- To recommend improvements in codes, standards, and design.
- To prepare reports for publication in ASCE Journals.

Initial results of this Task Committee were reported at an ASCE Structures Congress in New Orleans, September 1986, at a session on "Mitigation of Hurricane Wind Damage". Another conference was then held in Kansas City in November 1987, sponsored by the U.S. Wind Engineering Research Council and major Building Code organizations.

Because of the activities of the ASCE Task Committee and reports submitted to the two conferences, the National Science Foundation in 1988 awarded a grant to the University of Missouri - Columbia to study the response of wood framed houses to high winds and to develop a realistic and practical analytical model that could be used to predict the response. [3]

As reported in one of the original papers under this NSF Grant; "Great progress has been made in recent years in understanding the nature of wind damage to buildings and other structures, and how to mitigate such damage. It is now known that wind damage caused by hurricanes, tornadoes, and straightline winds can be greatly reduced by improving building construction practices, and by having better tie-down systems for mobile homes and parked aircraft. The costs associated with such

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improvements are rather modest the potential benefits are high." [1]

Recently, an issue of "Disaster Preparedness Report" by the National Weather Service, contained a summary of weather-related hazard fatalities in the United States. This report reveals that 341 lives were lost in 1987 and 349 in 1986; caused by weather (lightning, floods, tornadoes, high winds, heat, winter storms, hurricanes and tropical storms). [4]

Wind-related deaths totalled 115 in 1987: high winds - 33, thunderstorm winds - 23, tornadoes - 59. Of these, 51 deaths were in Texas; although 22 other states reported wind-related deaths. May 22 was the single deadliest day in 1987 when a tornado killed 30 people in Saragosa, Texas.

Analysis of locations where wind-related deaths occurred revealed that 30 deaths were in manufactured (mobile) homes: 24 from tornadoes, 1 from high wind, and 5 from thunderstorm strongwinds. Manufactured housing, which accounts for 6% of all U.S. housing, had 41% of all tornado deaths reported in 1987 [4].

Each year there are significant losses in terms of injuries, deaths, and property damage associated with manufactured homes exposed to high winds. This is critical when almost a third of all new single-family homes in America are manufactured housing.

2. WIND EFFECTS ON HOMES [5]

Manufactured home construction is by nature susceptible to damage by extreme winds. The units are lightweight and have large surface areas exposed to wind pressures. Single-wide units, 12'0" to 16'0" (3.7m to 4.9m) wide by 50'0" to 74'0" (15.3m to 22.6m) long, offer little resistance to overturning and sliding, except that provided by diagonal steel straps attached to screw-in ground anchors (See Figure 1).

Tornadoes, hurricanes, and "straight" winds can each create unique problems for manufactured housing. The sudden high intensity winds from tornadoes account for many deaths and injuries each year, and manufactured homes are frequently destroyed by tornado winds as low as 110 mph (176 km/hr). Long duration hurricane winds produce cyclic loads that cause fatigue in roofing and wall siding, and can weaken nailed connections and ground anchors. Straight winds can reach speeds of 65-80 mph (104-128 km/hr) but usually cause only moderate damage.

Damage to manufactured homes results from aerodynamic wind pressures that develop as air flows over and around the unit. External pressures pull walls, roof, and floor apart, except for windward walls. Drag forces tend to roll or slide the unit if it is not adequately anchored. Local pressures also cause material and connection failures at corners and roof eaves. Complete roof failures may occur because of wind uplift forces.

However, for the purpose of this paper, concentration will be on tiedown, anchorage and foundation failures of manufactured homes. Most manufactured housing is constructed on longitudinal steel chassis beams supported by piers of unreinforced concrete masonry units (cmu), spaced from 8'0" to 10'0" (2.4m to 3.1m) on center. Diagonal steel straps run from the steel chassis beams to steel rod anchors screwed into the ground. Some units, mostly double- wide homes, have perimeter unreinforced cmu walls with anchor bolts to the exterior wood frame.

Tiedown/anchorage failures range from the homes shifting off the foundation piers to complete pullout of ground anchors, allowing the unit to roll or vault in the wind. A shift off the piers can damage plumbing under the floor, or the piers punch through the floor. A more destructive failure occurs when the frame ties or over-the-roof straps break. This failure usually results in total destruction of the unit, because it is free to roll or tumble. A similar situation occurs if the ties stay intact, but if the ground anchors pull out of the soil.

Getting the proper size and type of anchorage is not always a simple operation. Relatively stiff soils are capable of resisting large pullout loads, but are highly resistant to penetration by screw-in ground anchors to the required depth. Loose soils may permit easy installation of ground anchors, but offer little resistance to pullout. [5]

3. INSTALLATION INVESTIGATION [6]

As a result of recent information collected on the number of deaths and injuries and amount of property damage associated with manufactured housing exposed to high wind forces, the U.S. Department of Housing and Urban Development (HUD) launched an investigation in 1988 into the installation of manufactured housing in hurricane (coastal) zones. This study was conducted by HUD's Monitoring Contractor, The National Conference of States on Building Codes and Standards (NCSBCS), with coordination by HUD engineers in the Manufactured Housing Compliance Branch. A major part of the study was the on-site inspection of 66 homes in ten coastal zone states (Alabama, Delaware, Florida, Georgia, Louisiana, Maryland, North Carolina, New Jersey, Rhode Island and Texas), from April to June in 1988.

State and local inspectors were asked to take NCSBCS and HUD engineers to "better installed" homes to evaluate installation practices considered to be satisfactory by the installers and local government inspectors. The purpose of the inspections was to determine not only the typical methods of installation used, but to understand the interaction between homeowners, dealers, installers, home manufacturers, local inspectors, and state regulators. The study included inspections conducted at manufactured housing parks, planned subdivision communities, and individual homeowner sites.

NCSBCS and HUD engineers identified specific deficiencies observed during on-site inspections, obtained information from installers, enforcement and building code officials, and reviewed installation manuals, ground anchor test data, and manufacturer's foundation and anchor designs. All this technical data was then analyzed to evaluate the on-site conditions observed to determine acceptable practices. A list of major deficiencies was then developed to be used in future training sessions for building code and State enforcement personnel. Finally, a "Manufactured Home Installation Inspection Guidebook" was drafted to be used as a training document, and also to be used by manufacturers and designers to prevent deficiencies from recurring.

4. INCORRECT INSTALLATIONS

Several deficiencies in the installation process were discovered, as shown in Figures 2 and 3, and in Photographs #1 through #18. The primary ones are summarized in the following seven categories:

4.1 Site Preparation:

Improper water drainage, topsoil not removed, incorrect placement of footings, lack of vapor barrier, poor soil bearing conditions.

4.2 Foundations:

Footings not below frost level; omitted under large opening column supports, perimeter walls, or ridgebeam posts; improper construction of block piers; caps or shims under steel beams; unreinforced block piers over 30" (0.8m) high.

4.3 Tiedowns:

Ground anchors at wrong angle, not in soil to correct depth, or extending out of ground an excessive distance, improper details or anchor test data; straps improperly installed or not tightened properly.

4.4 Marriage Wall Connections (Double-wide Units):

Inadequate connections at roof, floor or end walls; improper insulation or weatherproofing to prevent air or moisture infiltration; differential settlement or unlevel units.

4.5 Mechanical:

Defects or improperly installed plumbing, drain lines, heat ducts, or fixtures.

4.6 Electrical Work:

Improper field installation of fixtures and wiring connections; transportation damage to fixtures, appliances, or electrical system.

4.7 Special Site Conditions:

Inadequate design of foundation for flood plain, expansive clay soils, or earthquake forces; inadequate ventilation under homes.

5. CONCLUSIONS:

It becomes obvious from the data collected and investigations reported herein that there are several courses of action required to improve the resistance of manufactured housing to high wind forces. All corrective actions will involve specific organizations and levels of government. These are, in the opinion of the author, as follows:

. Research organizations (Universities and consultants);

. Designers (Manufacturers, anchor manufacturers, and consulting engineers);

. Local government and building officials (Building code and standards and inspectors);

. Installers and dealers;

. Federal and State Governments;

5.1 Research Organizations:

Funding by the Federal Government and the manufactured housing industry needs to continue, to develop analytical techniques for design of manufactured housing subjected to tornado, hurricane, or other high wind forces. Risk and hazard mapping development needs to continue, for incorporation into model building codes and standards.

5.2 Designers:

Manufacturers need to provide complete designs of each model for resistance to high wind forces, and provide specific details to show how wind forces are transmitted through the superstructure (shear walls and roof and floor diaphragms), to the frameand to the foundation and anchors. Test data for design needs to be supplied by anchor manufacturers and designers need to incorporate all information in adequate installation instructions and requirements.

5.3 Local Government and Building Officials:

Uniform inspection and enforcement programs need to be established by all states and municipalities for the installation and foundations of manufactured housing. The American National Standards Institute (ANSI), American Society for Testing and Materials (ASTM), and National Conference of States on Building Codes and Standards (NCSBCS) need to continue to promulgat standards and guidelines for adoption by local governments.

5.4 Installers and Dealers:

These industry representatives need to participate actively in trade organizations to obtain guidelines, standards, training, and local or state regulatory requirements. They also need to provide follow-up inspection to assure that homes continue to be level and are tied down properly. They also are required by the Federal Regulations to notify manufacturers of non-compliances with the Federal Standards and defects in construction.

5.5 Federal and State Governments:

State Governments need to develop uniform regulations, standards and enforcement programs, including onsite inspections of foundations and installations. The Federal Government, under the Manufactured Housing Program administered by HUD, should continue to foster research, develop guidelines for local governments, participate in and coordinate work of ANSI, ASTM, and NCSBCS, enforce the regulatory program for the design and construction of manufactured housing, and adopt updated wind design standards, as appropriate.

6. REFERENCES

[1] Liu, H. and Turner, E.J.; 1989: "Wind Damage Mitigation Strategies for the United States". University of Missouri, Columbia, MO 65211, Proceedings of Sixth U.S. National Conference on Wind Engineering, March 1989, Houston, TX.

[2] Fuller, G.R.; 1988: "Anchorages and Foundations for Wind Resistance of Manufactured Homes," U.S. Department of Housing and Urban Development, Proceedings of 20th Joint Meeting of US-Japan Cooperative Program in Natural Resources (UJNR), National Institute of Standards and Technology, S P 760, January 1989, Washington, DC

[3] Liu, H.; Gopalaratnam, V. and Nateghi, F; 1989: "Improving Wind Resistance of Wood-Frame Houses," University of Missouri, Columbia, MO Proceedings of Sixth National Conference on Wind Engineering, March 1989, Houston, TX.

[4] Peters, B.; 1988: "Disaster Preparedness Report, Natural Hazard Deaths in 1987 in the United States," National Weather Service, Southern Region, Ft. Worth, TX 76102.

[5] McDonald, J.R. and Mehnert, J.F.; 1989: "A Review of Standards of Practice for Wind Resistant Manufactured Housing," Texas Tech University, Lubbock, TX 79409, Proceedings of Sixth U.S. National Conference on Wind Engineering, March 1989, Houston, TX.

[6] U.S. Department of Housing an Urban Development; 1989: "Preliminary Study of the Installation of Manufactured Homes and Installation Monitoring by States and Local Governments," National Conference of States on Building Codes and Standards, Inc. January 1989, Herndon, VA.

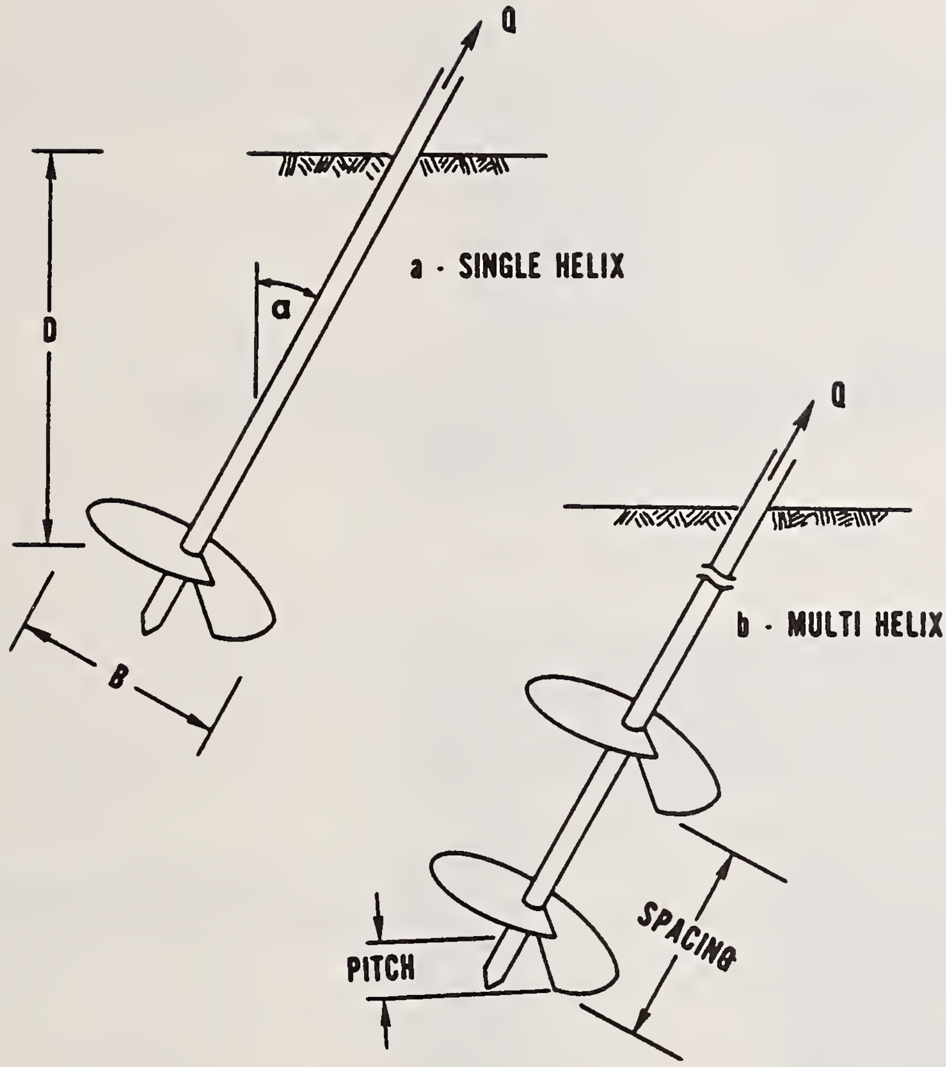


Figure 1 Helix and Multi-helix Anchors

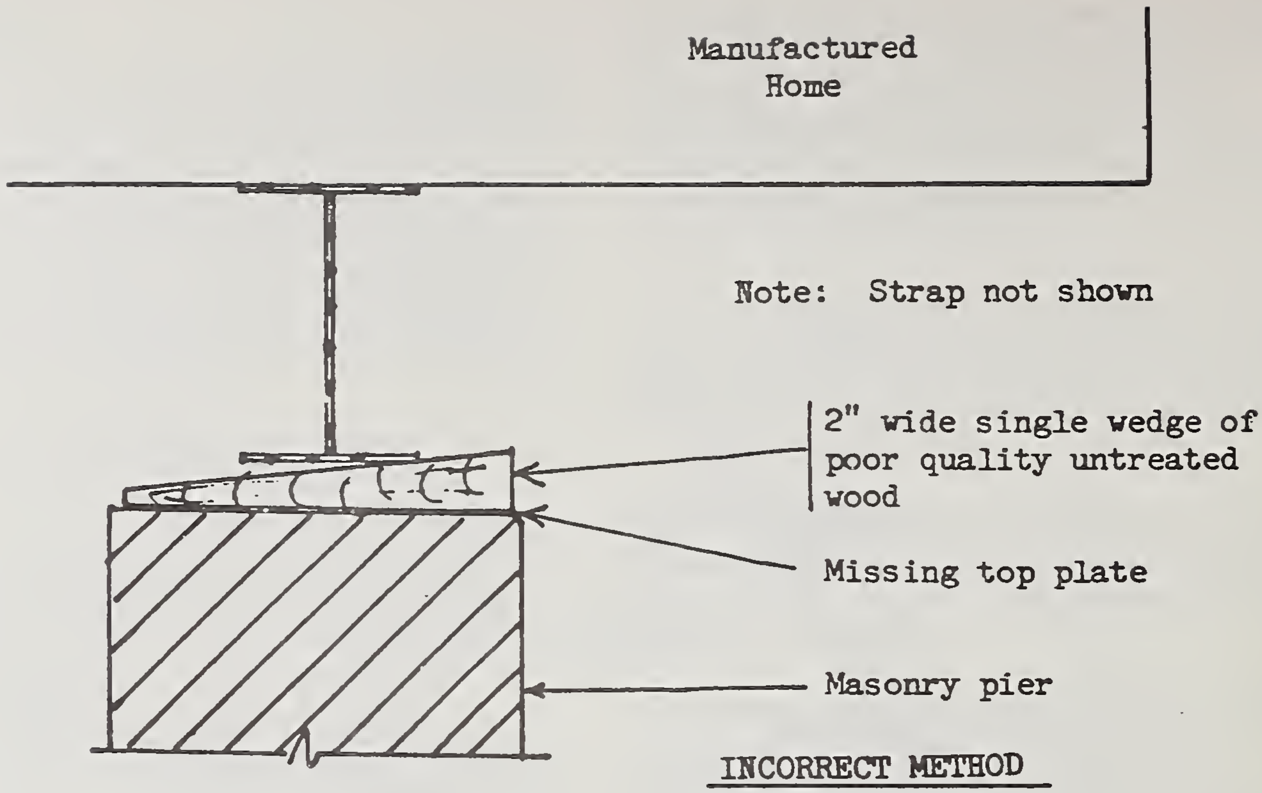


Figure 2 Improper Blocking and Shimming

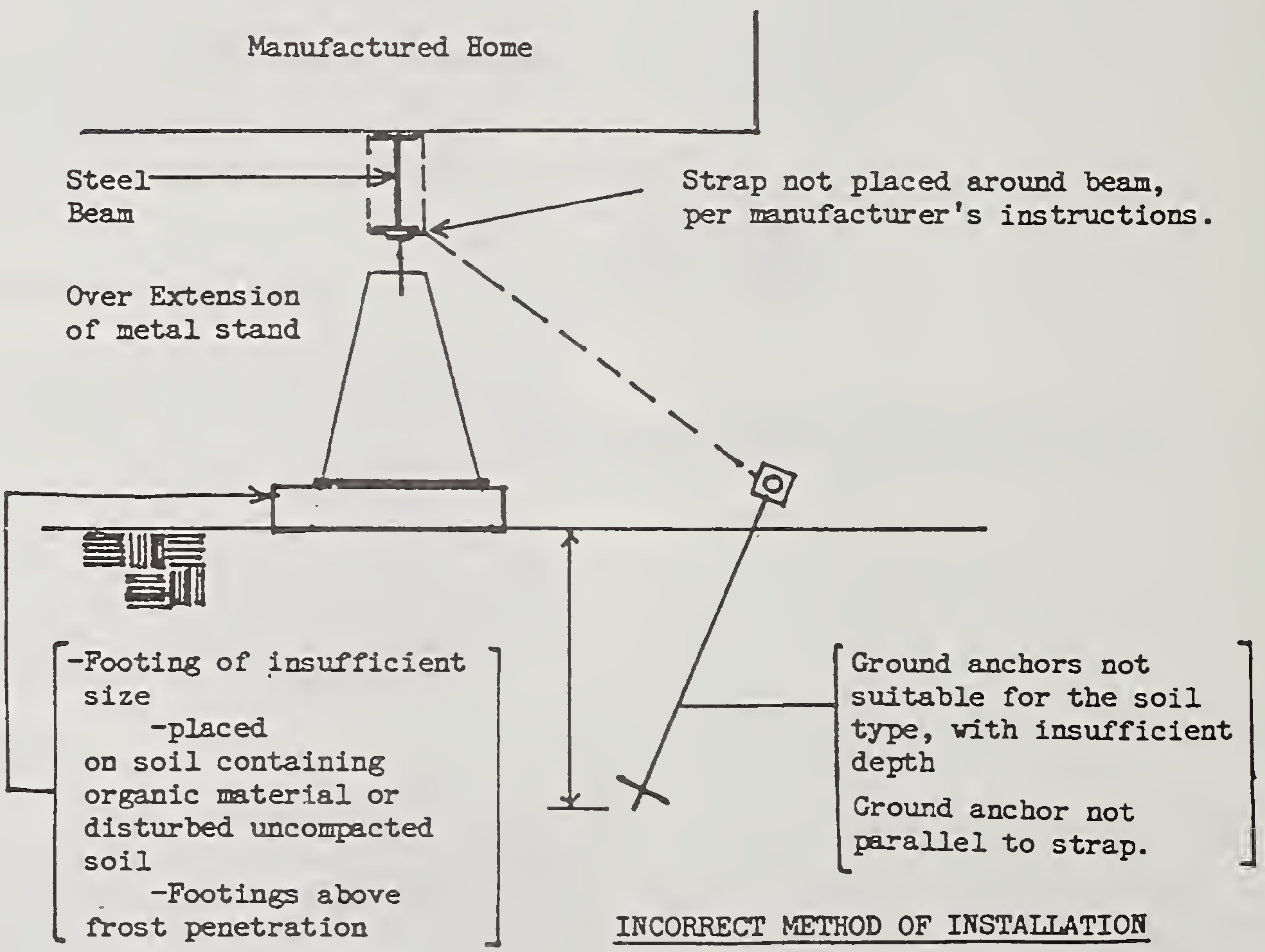
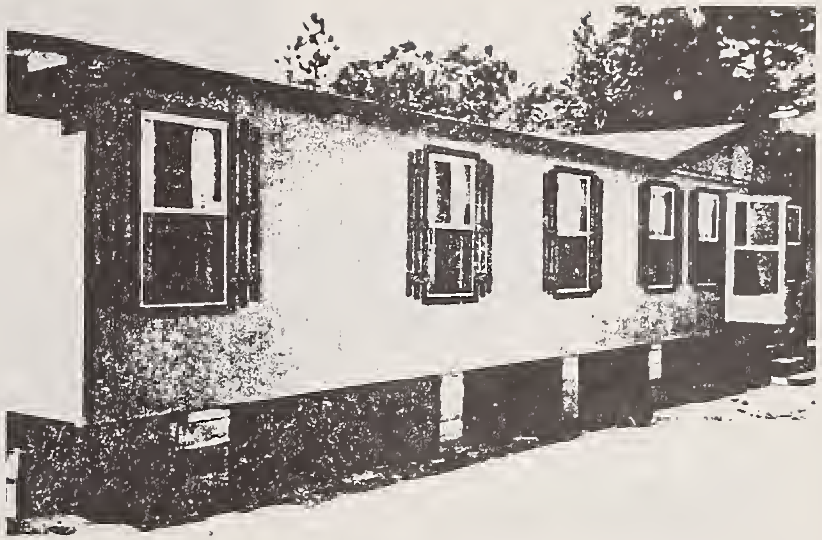


Figure 3 Incorrect Installation and Foundation

* Perimeter Footings
  Not Below
  Frost Line

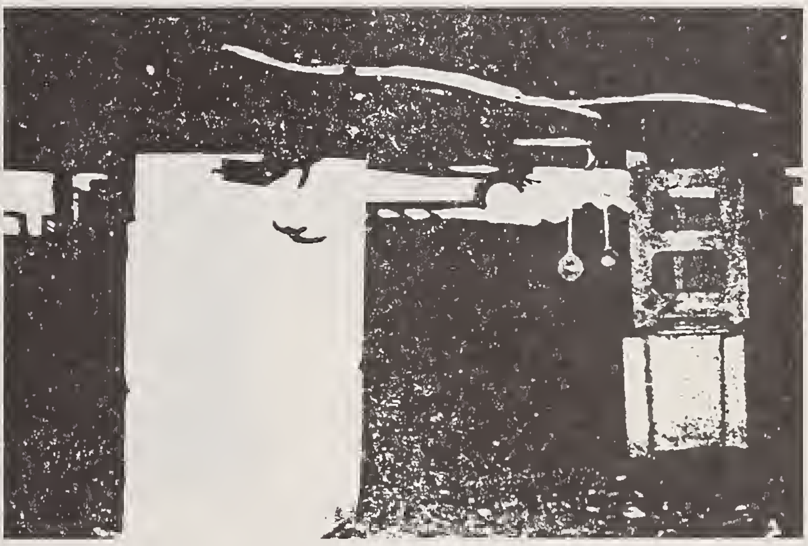
* Improper Location
  Of Piers



Photograph # 1

* Improper Cell
  Alignment

* No Footing Under
  Pier



Photograph # 2

* Improper Cell
  Alignment

* Absence Of Pier
  Cap

Photograph # 3

* Improper Drainage

* No Vapor Barrier

Photograph # 4

* Noncompacted Soil
  Underfooting

* Inadequate Protection
  Of Bearing Soil

Photograph # 5

* Organic Material
  Under Footing

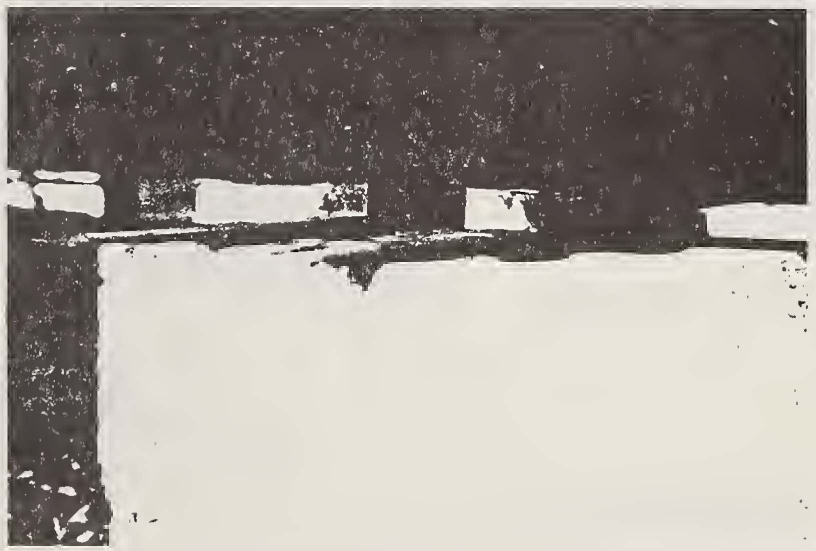
* Deteriorating Metal
  Stand

Photograph # 6

* Missing Solid
  Masonry Cap

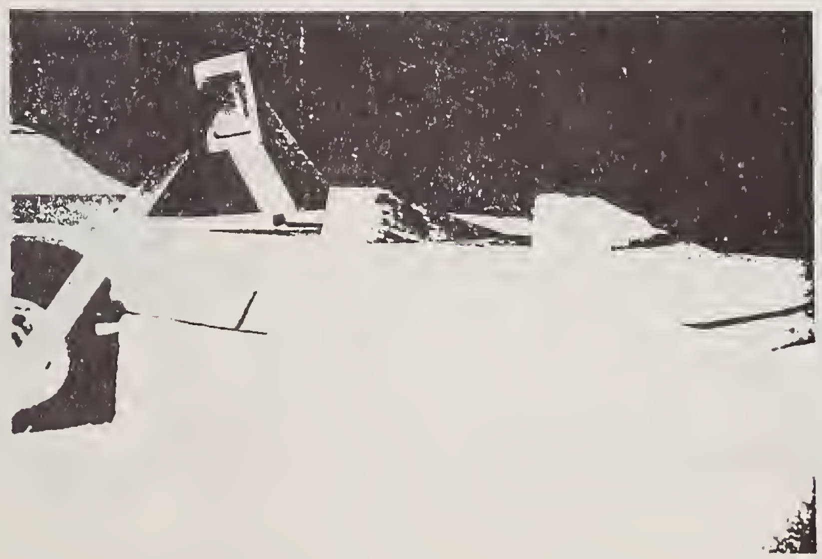
* Mis-Oriented
  Footing Blocks

Photograph # 7



* Missing Pier
  Cap

Photograph # 8



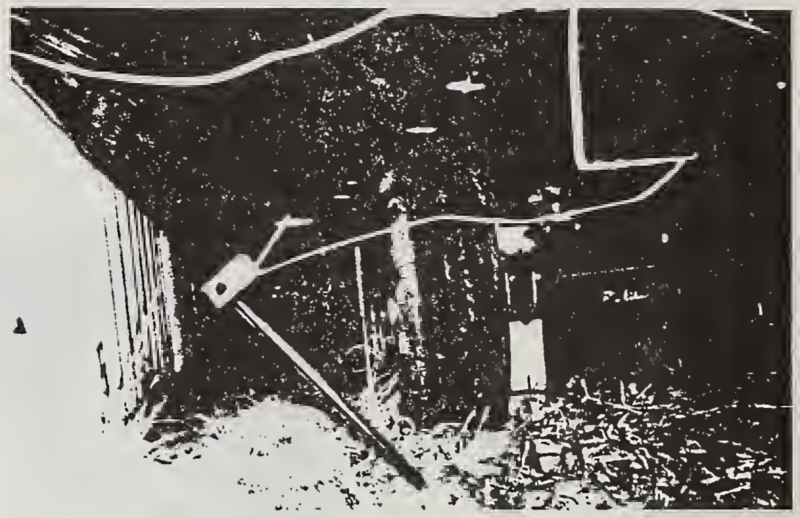
* Improper Shimming
  (without a pair)

* Improper Frame
  Tie Buckle

Photograph # 9

* Excessive Anchor Projection

* No Vapor Barrier

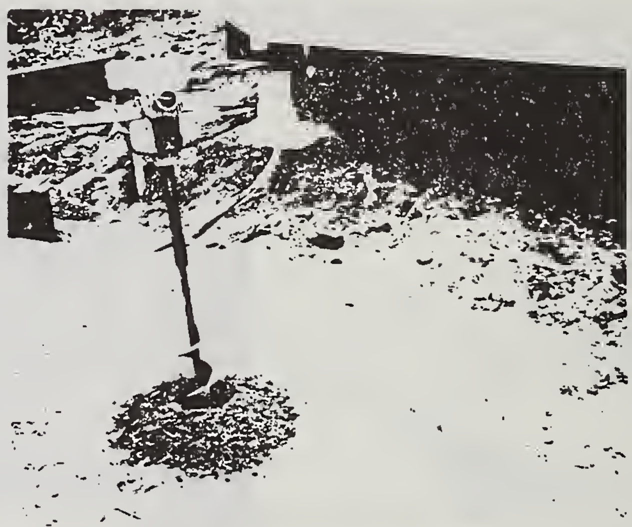


Photograph # 10

* Predrilled Hole With Poor Soil Preparation

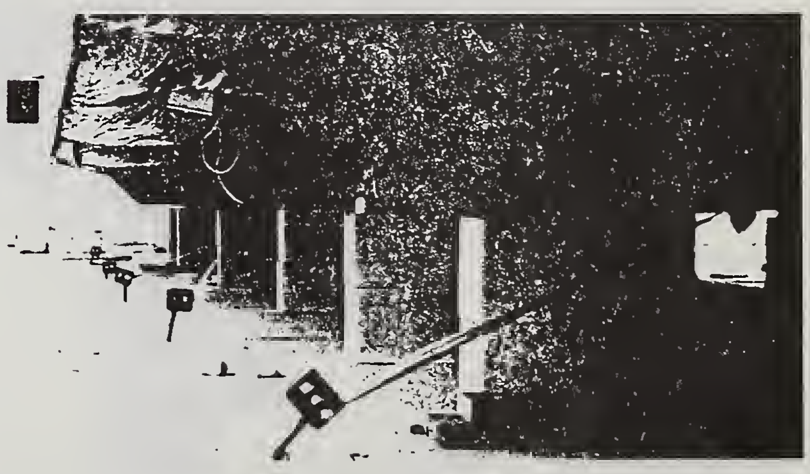
* Predrilled Hole Without Poured Concrete Collar

* No Vapor Barrier



Photograph # 11

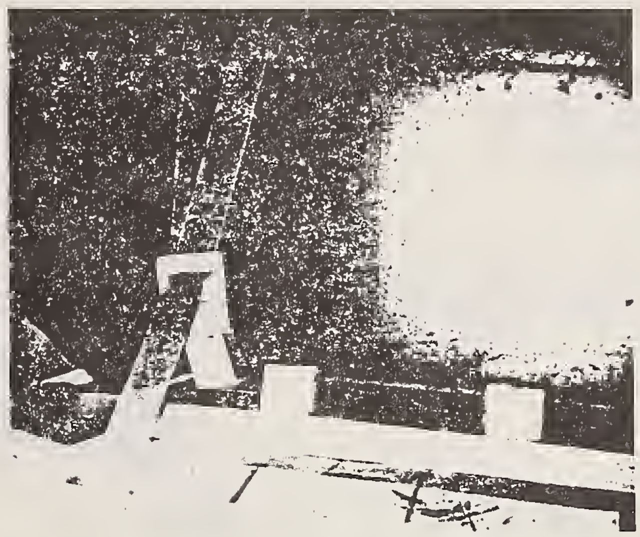
* Wrong Angle Of Anchor For Tie



Photograph # 12

* Improper Buckle Fastening

* Shimming Without A Pair



Photograph # 13

* Improper Buckle Fastening

* I-Beam Not Bearing On Pier

Photograph # 14

* Unacceptable Wrapping Of Frame Tie

* Poor Site Preparation

Photograph # 15

* Excessive Strap
  Angle

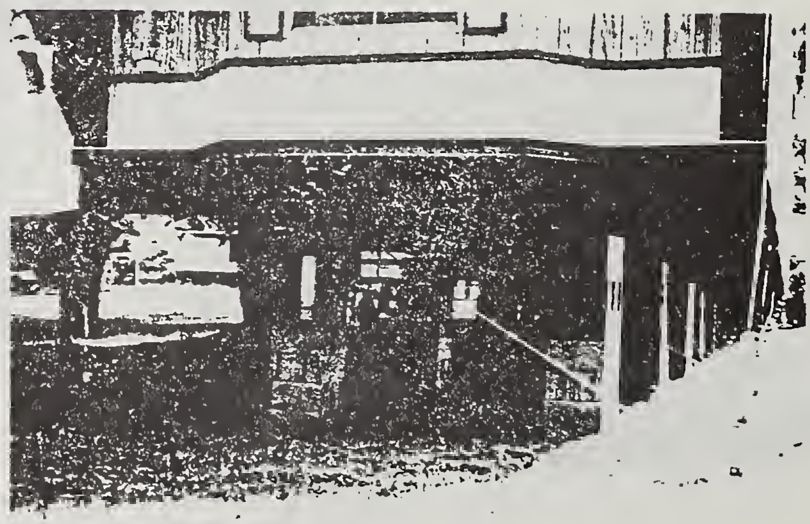
Photograph # 16

* Improper Pier
  Alignment To
  Marriage Wall

* Dryer Vented
  Beneath Home

Photograph # 17

* Tie Downs Incorrectly
  Located Between
  Piers



Photograph # 18

Theme II

Earthquake Engineering

Behavior of the Nagara Dam During the Earthquake of December 17, 1987

by

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SYNOPSIS

During the Chiba-ken Toho Oki Earthquake of December 17, 1987, 27 components of strong motion records were obtained at the Nagara Dam of the Water Resources Development Public Corporation located at a distant of about 29 km from the epicenter. The behavior of the dam during this earthquake was analyzed from these records and the dynamic properties of the dam were examined. Moreover, observed behavior of the dam was compared to the result of the dynamic finite element analysis.

KEY WORDS

Strong motion observation, Record of strong-motion acceleration, Dynamic behavior of earth dam, and Dynamic finite element analysis.

1. OUTLINE OF THE EARTHQUAKE AND THE STRONG-MOTION OBSERVATION FACILITIES AT THE NAGARA DAM

1.1 Outline of the Earthquake

An earthquake of JMA Magnitude 6.7 occurred near the offshore of eastern Chiba Prefecture at 11:08 AM on December 17, 1987. According to the Japan Meteorological Agency (JMA), the epicenter of this earthquake was located at 140°29' east longitude and 35°21' north latitude with the hypocenter depth of 58 km.[1] The seismic intensity of JMA scale was V (very strong) at Katsuura, Chiba and Choushi, and was IV (strong) at Tateyama, Tokyo, Yokohama, Mito, Ajiro, Kumagaya and Kawaguchiko. The range where the seismic intensity of V felt was almost within 50 km of the epicentral distance within Chiba Prefecture.

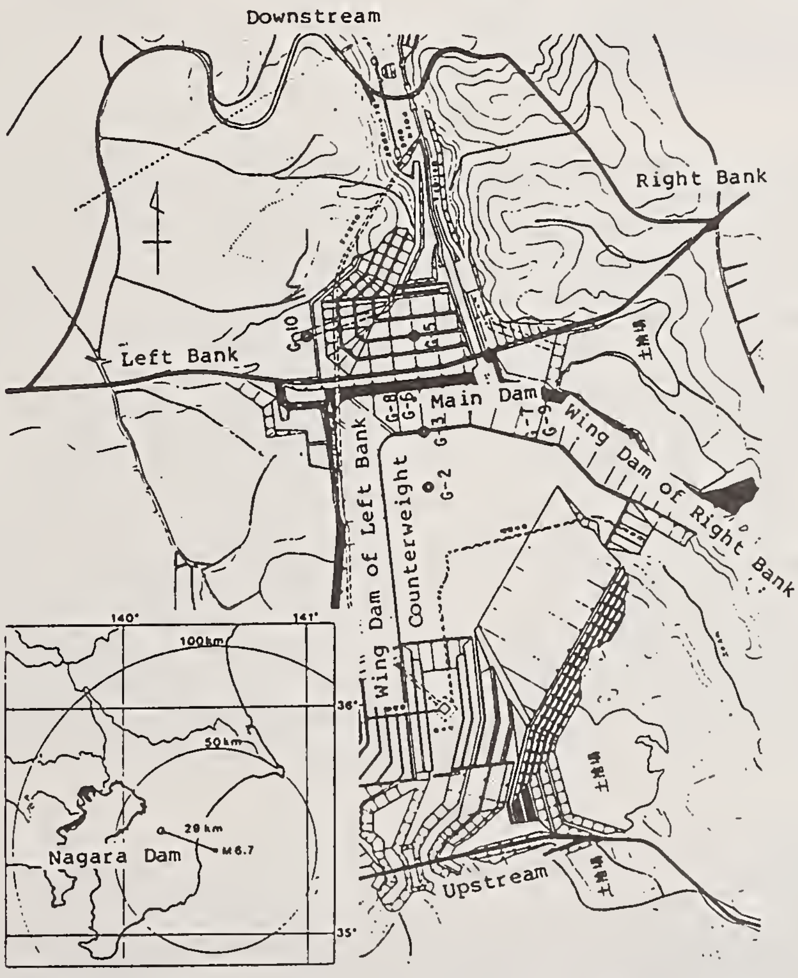

Fig. 1 Plan of the Nagara Dam

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2) Research Engineer, Ground Vibration Division, ditto
3) Visiting Research Engineer, Ground Vibration Division, ditto
4) Chief, the First Research Department, Research Institute, WRDPC
5) Head, Fill Dam Division, ditto
6) Fill Dam Division, ditto

Damages caused by this earthquake were reported; 2 persons dead, 161 persons wounded, 72,698 houses damaged, and the total amount of damages to the public facilities was about 6.88 billion yen. The damage features of this earthquake are reported in more detail in a separate paper.[2]

1.2 Outline of the Nagara Dam

The Nagara Dam is a zoned earthfil dam constructed at the border of Nagara Town and Ichihara City, which creates the reservoir of the capacity of 10 million cubic meters. This reservoir is one of the water conveyance facilities for supplying water to the Boso coastal industrial zone and its vicinities in Chiba Prefecture as well as residents in Chiba City and Kujukuri coastal region. Plan of the dam and its maximum cross section are respectively shown in Fig. 1 and Fig. 2. The 52 m high dam is constructed on the diluvium sandy layers. The cross section of the dam consists of three zones and counterweight fills. Construction materials for the dam are Kanto loam (Zone I, impervious zone), sandy soils (Zone II) and silty mudstone (Zone III). Wing dams for impervious purposes are located at both the right and left bank sides, and drains are provided inside the main Dam.

The dam site is located at a distance of 29 km from the epicenter in the west-northwest direction. An acceleration of about 260 gal was observed at the base of the dam. As a result of this earthquake motion, maximum settlement of 30 mm and a horizontal displacement of about 20 mm were observed in addition to hair cracks (maximum depth of 2 m) at several places in the pavement on the dam crest. However, this anomaly was not so severe and would not affect the safety of the dam.[3] Settlement distribution and location of cracks are shown in a separate paper.[3] Since it was the stage of test filling to the reservoir during the earthquake, the water level during the earthquake was lower (EL.: 57 m) than the surface of counterweight fill (EL.: 60 m).

1.3 Strong-Motion Observation Facilities at the Nagara Dam

Three-component strong motion accelerographs were installed at 10 places as shown in Figs. 1 and 2 within the dam.[4] G-1 and G-7 are electromagnetic type accelerometors. And G-8 to G-10 are popularizing type accelerometers which were installed on the concrete base on the dam crest and inside the water-collecting tunnel. A ten-second delay circuit was employed for records of G-1, G-2, G-4 and G-6. All records of each strong-motion accelerograph are stored on a magnetic tape in analogue form. Specifications of accelerograph are shown in Table 1.

Table 1 Specifications for Accelerograph Transducer

Place Code	G-1 ~ G-7	G-8 ~ G-10
Transducer Type	Electromagnetic Movable Coil Type	Piezoelectric Shear Type
Natural Frequency	3.0 Hz	Approx. 1 kHz
Damping Constant	More than 11	————
Max. Measuring Acceleration	1000 gal	1000 gal
Measuring Frequency Range	0.3 ~ 30 Hz	0.1 ~ 30 Hz

2. BEHAVIOR OF THE DAM DURING THE EARTHQUAKE

Strong-motion records were obtained during the event at 9 places out of ten other than G-9 accelerograph. Since the record is of analog type, it was converted into the digital quantity at the interval of 1/100 sec. and was processed by a band-pass filter shown in Fig. 3. Maximum accelerations are shown in Table 2. Since the records obtained by G-8 and G-10 are not synchronized with others, time was adjusted for G-8 and G-10 in such a manner that the occurrence time of maximum value of vertical motion becomes equal to that observed by other accelerographs. The maximum value at the base (G-1) was 256 gal in the upstream-downstream direction, 135 gal in the dam axis direction, and 86 gal in the vertical direction and at the crest (G-6), it was 371 gal in the upstream-downstream

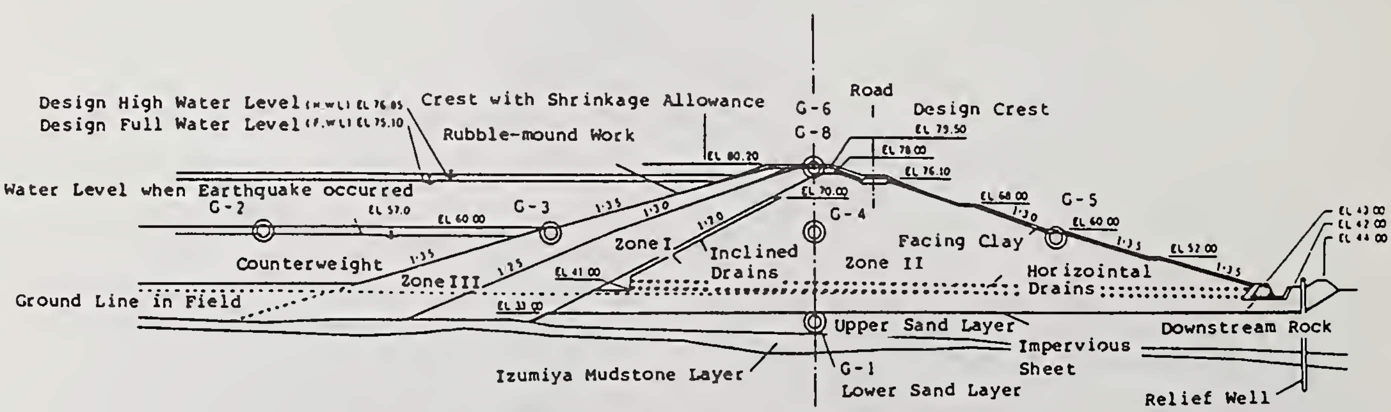


Fig. 2 Cross Section of the Nagara Dam

direction, 353 gal in the dam axis direction, and 326 gal in the vertical direction. The ratio between maximum acceleration of G-1 and G-6 was respectively 1.4 times (upstream-downstream direction), 2.6 times (dam axis direction) and 3.8 times (vertical direction). Time history of acceleration in the upstream-downstream direction and the displacement waveforms determined by numerical integration in frequency domain at G-1, G-4 and G-6 are shown in Fig. 4. Maximum acceleration at G-4, inside the dam, is smaller than that at the base (G-1). At the dam crest, relatively large acceleration values continue after the main shock compared to the base. By examining the waveform of the main shock portion in Fig. 4, it can be known that a phase lag occurs in the record of each point and waves propagate from base to the crest of dam. Distribution of maximum acceleration in the upstream-downstream direction at 6 points is shown in Fig. 5(a). Acceleration with the level same as the maximum acceleration at the crest occurred at the upstream and downstream slopes, and the values became almost half at the midheight of the enbankment. Also, the distribution of the maximum displacement derived from integration of acceleration is shown in Fig. 5(b), and displacement orbit of each

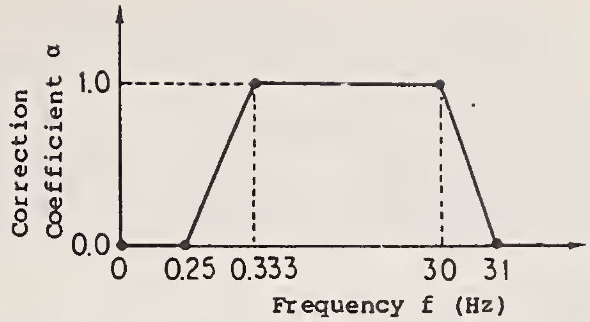


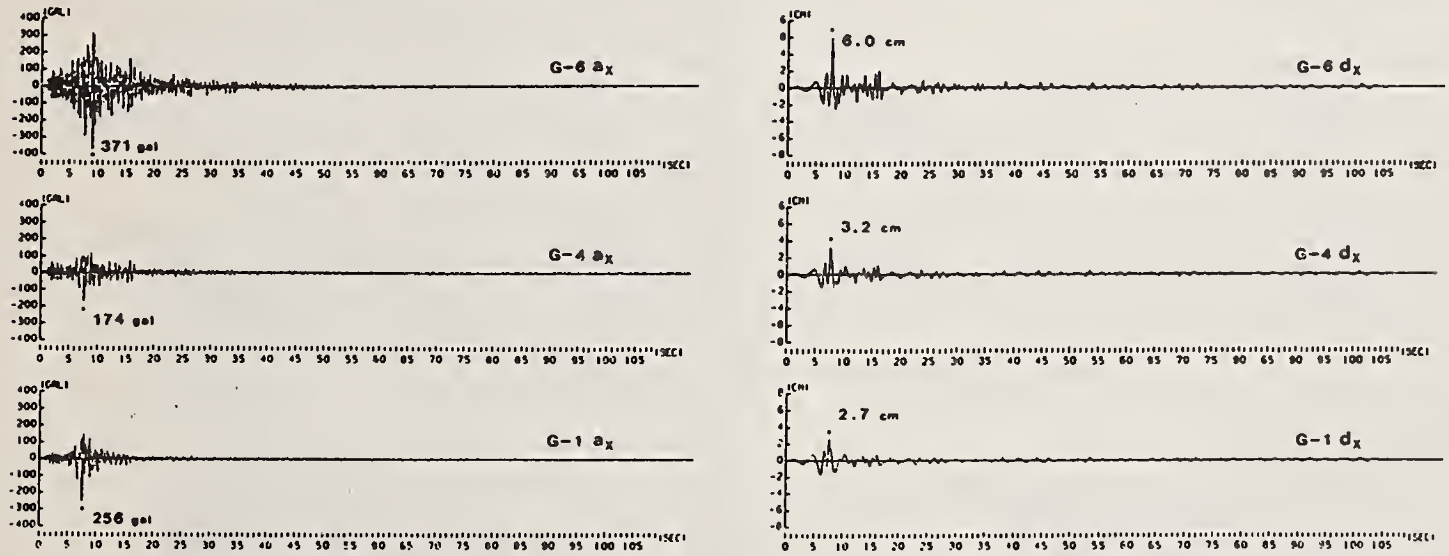
Fig. 3 Correcting Function of Band-Pass Filter

Table 2 Maximum Acceleration Observed at the Nagara Dam

Place Code	Measuring Direction Code	Maximum Acceleration (gal)	Place Code	Measuring Direction Code	Maximum Acceleration (gal)
G-1	X	256	G-6	X	371
G-1	Y	135	G-6	Y	353
G-1	Z	86	G-6	Z	326
G-2	X	263	G-7	X	257
G-2	Y	204	G-7	Y	288
G-2	Z	123	G-7	Z	179
G-3	X	347	G-8	X	497
G-3	Y	288	G-8	Y	385
G-3	Z	122	G-8	Z	329
G-4	X	174	G-10	X	274
G-4	Y	150	G-10	Y	147
G-4	Z	137	G-10	Z	109
G-5	X	380			
G-5	Y	303			
G-5	Z	182			

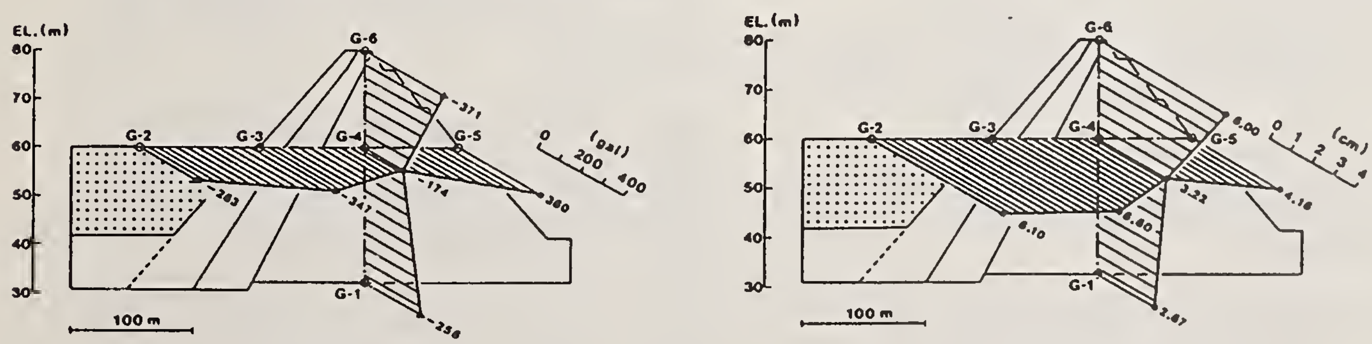
* Measuring Direction Codes
X: Upstream-Downstream Direction
Y: Dam Axis Direction
Z: Vertical Direction

Maximum acceleration is a corrected value after band-pass filter.



(a) Acceleration (b) Displacement

Fig. 4 Time History of Observed Acceleration and Calculated Displacement (Upstream-Downstream Direction)



(a) Acceleration (b) Displacement

Fig. 5 Distribution of Maximum Acceleration and Displacement (Upstream-Downstream Direction)

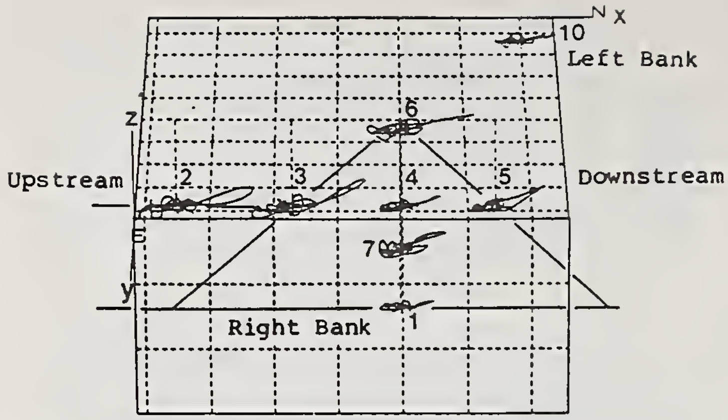


Fig. 6 Displacement Orbits at Observation Points

record is shown in Fig. 6. By looking at the displacement orbit, a large displacement directed from the downstream (north) to the direction 30° to 45° biased on the left bank northwest) can be recognized. This displacement occurred at about 8 sec. as shown in the displacement time history in Fig. 4(b). This portion of the waveform gives the maximum horizontal displacement for all observation points.

3. WAVE PROPAGATION IN THE DAM AND THE ESTIMATION OF DYNAMIC PROPERTIES OF EMBANKMENT MATERIALS[5]

3.1 Natural Frequency of the Dam

Since the record of G-6 is the result of an earthquake motion at G-1 which was amplified within the embankment, the natural frequency of the embankment can be derived from the transfer function between both points. The transfer function is calculated as the square root of the ratio of power spectrums of acceleration records between two observation points. As an example, the transfer function at the central section of the dam is shown in Fig. 7 to Fig. 9. In the horizontal components of the upstream-downstream direction and the dam axis direction, no dominant frequency showing significant amplification is recognized, and the response magnification is low as a whole. On the other hand, it can be known that the vertical component has a high amplification at around 2.7 to 3.0 Hz. Frequencies having large transfer magnifications recognized between base (G-1) and crest (G-6) are shown below in the order starting from the largest magnification.

Upstream-downstream direction (refer to Fig. 7): 5.4, 1.4 to 1.7, 4.2, 9.0 to 9.6, 10.7 to 10.9 Hz ... Amplified in a wide range of frequency.

Dam axis direction (refer to Fig. 8): 1.6, 2.9 to 3.2, 4.6 to 4.7, 6.3 to 6.6, 8, 8.7 to 9.3 Hz ... Amplified in a wide range of frequency.

Vertical direction (refer to Fig. 9): 2.7 to 3.0, 5.4, 7.7 to 7.9, 13.7 to 14.0 ... There are significant dominant frequencies.

Primary natural frequency of the maximum cross section was 1.6 Hz, which is derived from Matsumura's shear vibration solution[6] of a wedge-shaped embankment. From this solution and the above mentioned dominant frequency, the natural frequency of the dam is considered to be 1.6 Hz for horizontal shear vibration in both upstream-downstream and dam axis directions and 2.7 to 3.0 Hz for vertical vibration.

3.2 Wave Propagation Velocity within the Embankment

The upward propagation of an earthquake motion within the dam is seen in Fig. 4. Then, the propagating time of an earthquake motion from point (G-1) to the other upper observing points was determined by the two methods explained below. In the first method, cross correlation between time history of recorded waveforms at two observation points was taken as shown in Fig. 10, and then phase lag time which gives the peak correlation was used as a propagating time (Method 1), which means the mean propagating time for the overall waveforms. In the second method, attention is paid to the pulse showing the maximum amplitude which is considered to be the same phase at different observation points as shown in Fig. 11, and the time difference between two observation points of the rising zero cross time of the pulse is used as the propagating time (Method 2). The time derived by the Method 2 is the wave propagating time for a particular phase of the maximum amplitude. Values of wave propagating time within the embankment derived from acceleration components a_x and a_y in the upstream-downstream and the dam axis directions in these two methods are shown in Fig. 12(a). It can be known that the values of propagating time derived from both methods almost coincide with each other. Also, from the phase spectrum of transfer function between two observation points, the time difference determined from the gradient of the phase angle ϕ/frequency f also coincides with the propagation time as shown below (refer to Fig. 7 to Fig. 9).

$$\tau = \phi / (2\pi f) \quad \text{.......................} \quad (1)$$

Where, propagating time between two observation points is , their elevation difference is H, and the unit mass of the embankment between two points is , then the shear wave velocity is given by

$$V_s = H / \tau \quad \text{..........................} \quad (2)$$

Shear modulus is obtained by

$$G = \rho V_s^2 \quad \text{..........................} \quad (3)$$

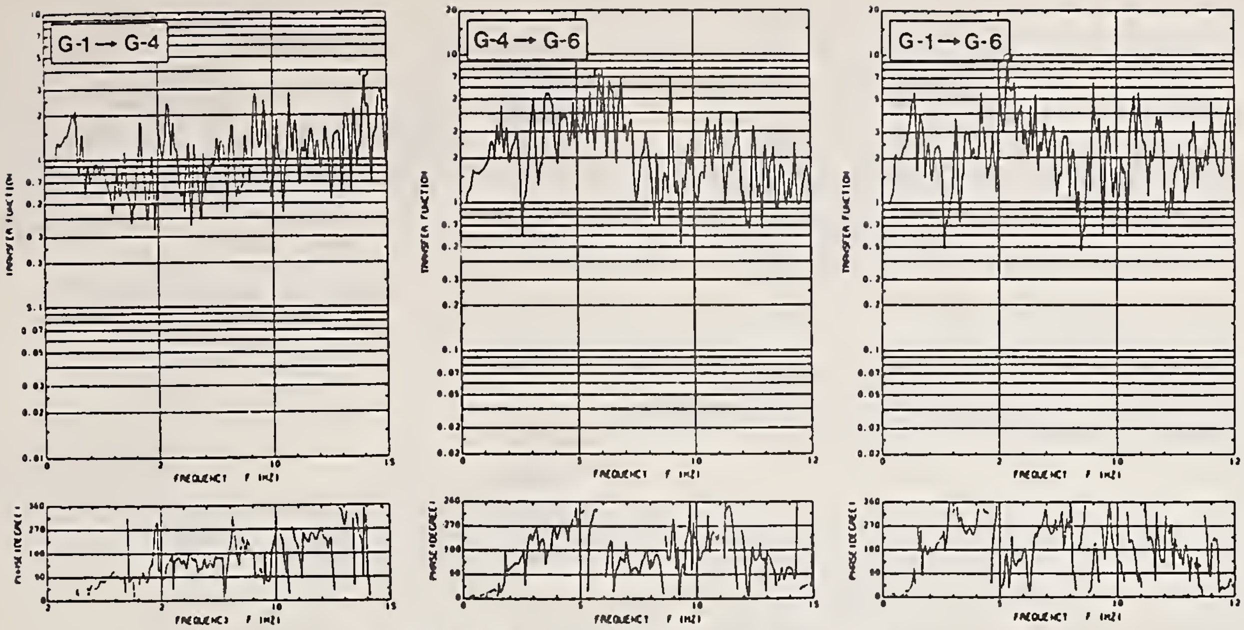


Fig. 7 Transfer Function of Acceleration Records
(Upstream-Downstream Direction)

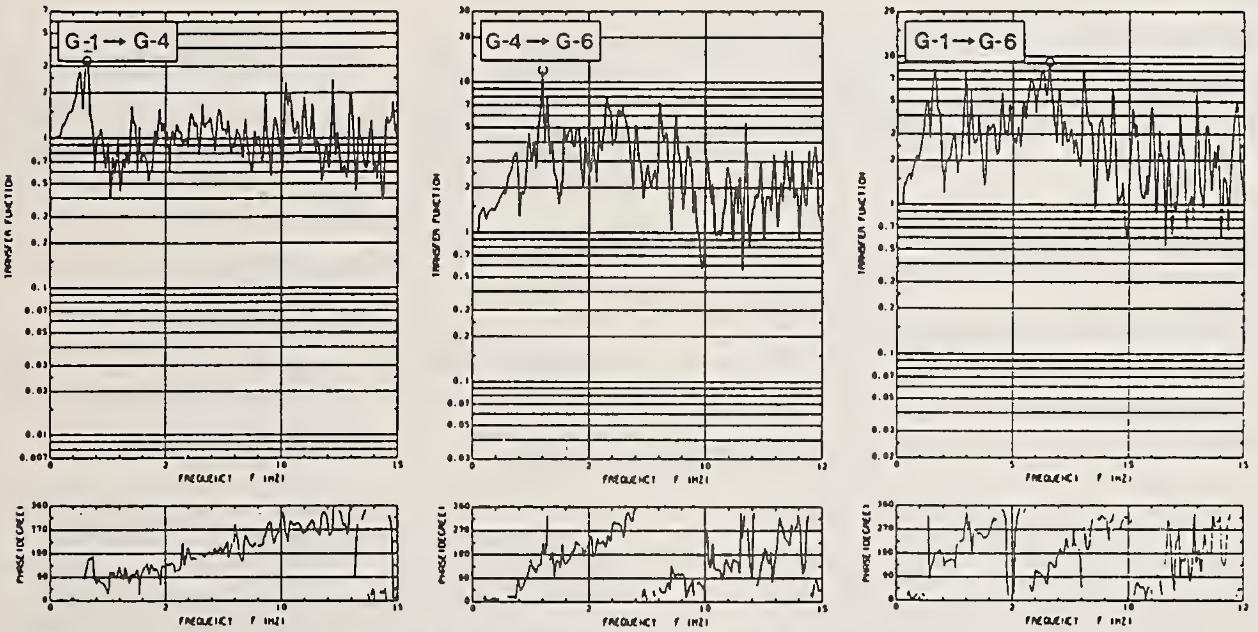


Fig. 8 Transfer Function of Acceleration Records
(Dam Axis Direction)

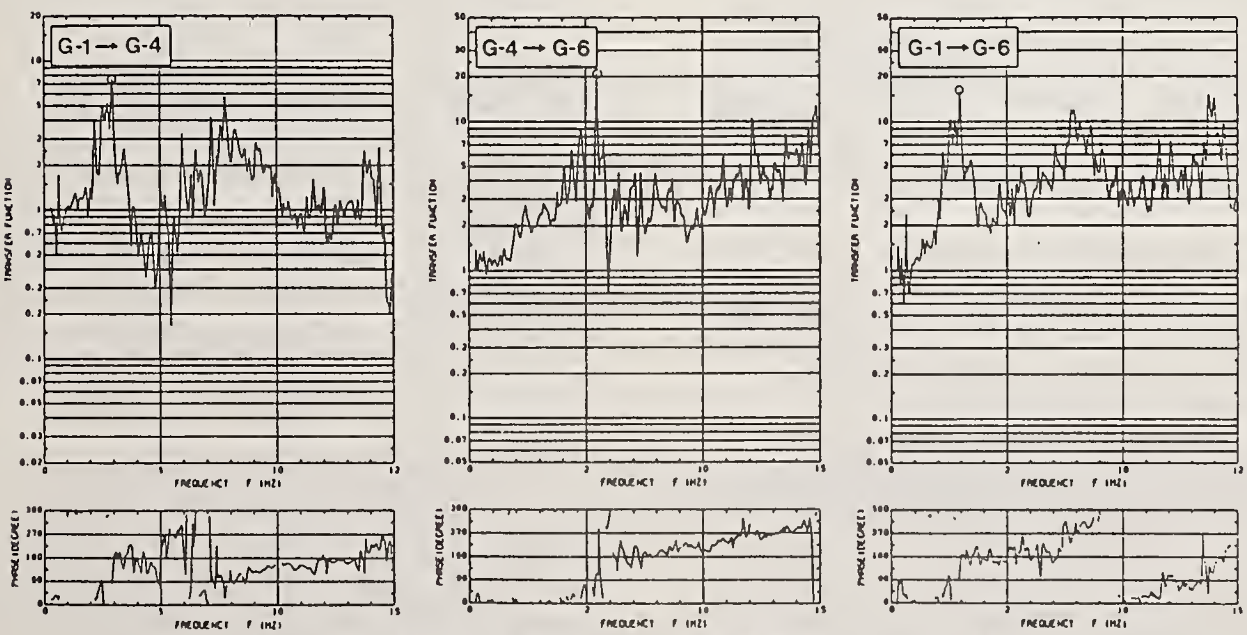


Fig. 9 Transfer Function of Acceleration Records
(Vertical Direction)

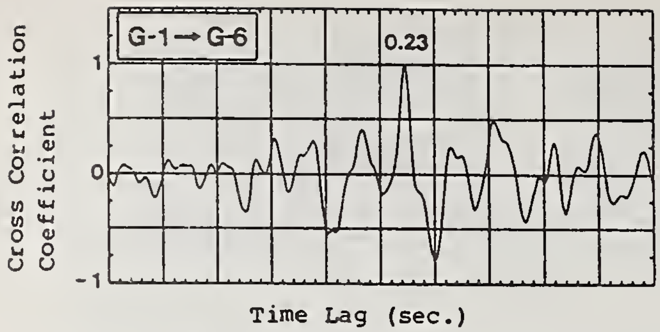


Fig. 10 Cross Correlation Coefficient of Acceleration Records (Upstream-Downstream Direction, Method 1)

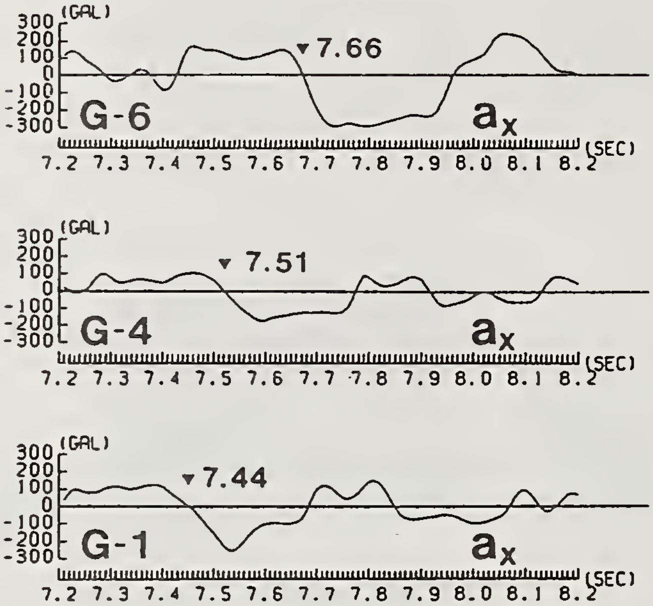
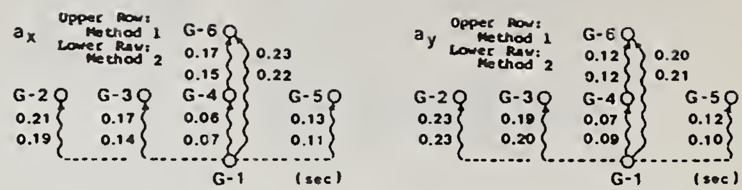
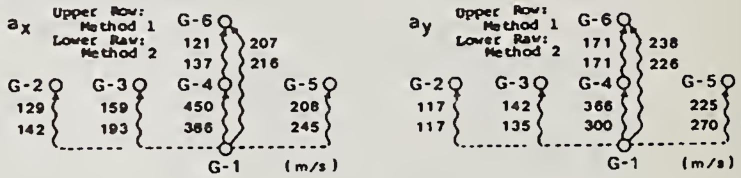


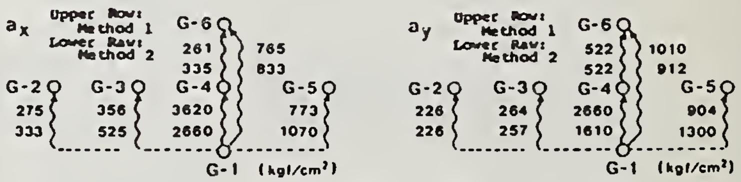
Fig. 11 Wave Propagation
(Upstream-Downstream Direction, Method 2)



(a) Wave Propagation Time τ



(b) Shear Wave Velocity V_s



(c) Shear Modulus G

Fig. 12 Results of Acceleration Records Analysis between Observation Points

V_s and G which are derived from the above are shown in Fig. 12(b) and (c) respectively. As shown by this Fig., the average shear wave velocity for the whole embankment determined from the records in the upstream-downstream direction at G-1 and G-6 is about 210 m/sec. On the other hand, the average shear wave velocioty for the upper portion of the dam determined from the records in the upstream-downstream direction at G-4 and G-6 is about 120 to 140 m/sec. This difference is thought to arise from the difference of the strain level in the embankment and confining stress level as stated later.

3.3 Dynamic Properties of the Dam Materials

Seismic prospecting which was made after the earthquake gives a formula which enables to estimate the shear wave velocity and the shear modulus as a function of the overburden pressure.[7] These are shown as vertical distribution curves in Figs. 13 and 14. These figures also indicate the results obtained by the methods 1 and 2 from the records of acceleration in the upstream-downstream direction. The results determined from the records of earthquake motion have the values 20 to 30% lower for V_s and 40 to 50% lower for G compared to V_{s0} and G_0 at a very small strain level which was obtained from seismic prospecting. Especially at the upper half portion (G-4 to G-6), V_s decreased to 1/3 to 1/4 and G decreased to 1/10 approximately compared to the lower half portion (G-1 to G-4). Both V_s and G are close to the results of seismic prospecting near the upstream slope (G-1 to G-3) and the downstream slope (G-1 to G-5).

Shear strain that occurred within the embankment and the counterweight during this earthquake will be determined as explained below from the multiple reflection theory assuming that the dam is as a homogeneous semi-infinite layer (uniform shear wave propagation velocity V_s). If the velocity of the earthquake motion observed on the ground surface is $v_0(t)$, then shear strain $\gamma(t, z)$ at a given depth z can be obtained by

$$\gamma(t,z) = \frac{1}{2V_s}\left\{v_0\left(t+\frac{z}{V_s}\right)-v_0\left(t-\frac{z}{V_s}\right)\right\} \quad \ldots\ldots \quad (4)$$

Time history waveforms of shear strains generated within the dam and counterweight were respectively calculated by using the velocity waveforms of G-6 and G-2, and the maximum value at the depth z was determined. The results are shown in

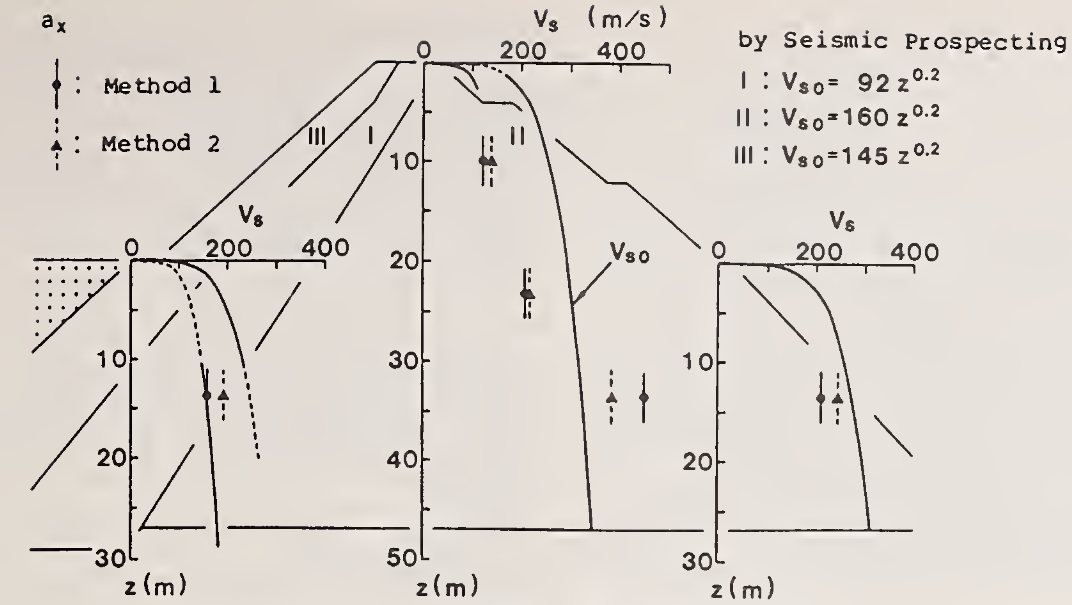


Fig. 13 Distribution of Shear Wave Velocity by Acceleration Records Analysis (Upstream-Downstream Direction)

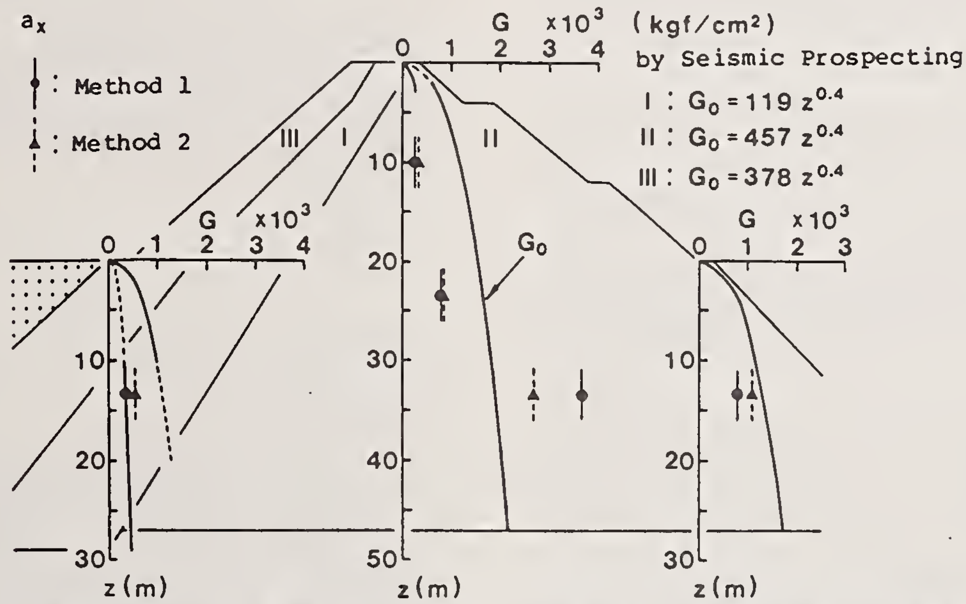


Fig. 14 Distribution of Shear Modulus by Acceleration Records Analysis (Upstream-Downstream Direction)

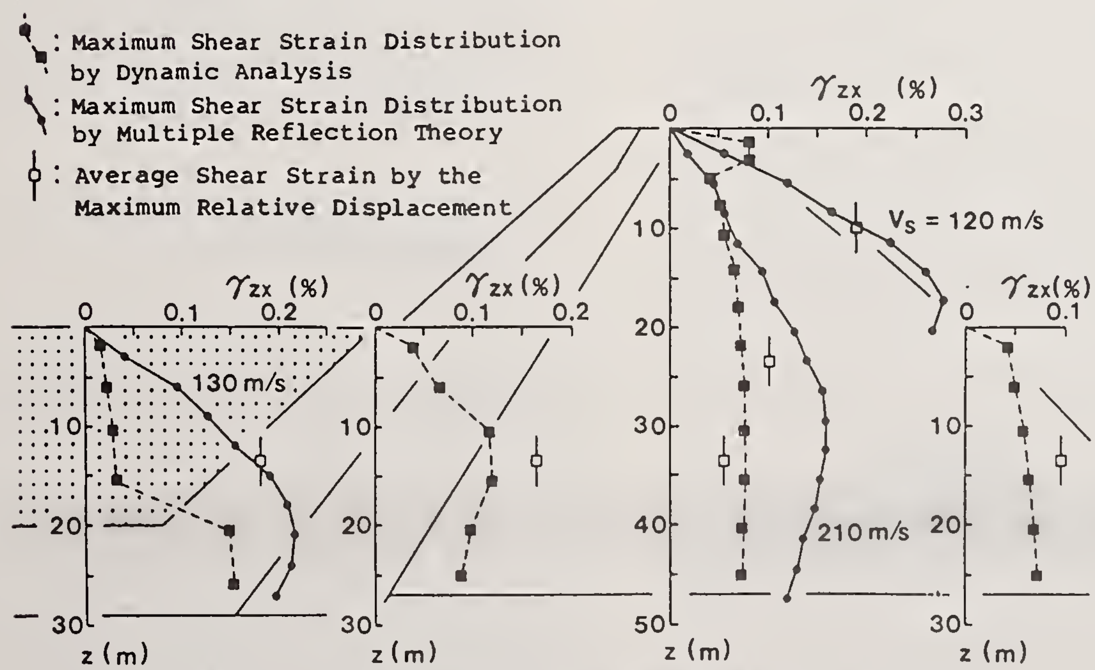


Fig. 15 Distribution of Maximum Shear Strain (Upstream-Downstream Direction)

Fig. 15. Also in the same figure, the maximum value of average shear strain is shown as reference, which was obtained from the division of the relative displacements at G-4 and G-6 to G-1 by the relative height of these two points to the base. From the time history waveform of strain shown in Fig. 16, it can be known that the strain of higher than 10^{-4} occurred repeatedly in the dam and counterweight fill and the maximum strain was in the order of 10^{-3}. By the occurrence of shear strain of this level, the shear modulus of the dam is considered to be lowered to about one-half in average of the initial shear modulus G_0 estimated from the results of seismic prospecting. Especially, a decrease of one order occurred in the upper half portion of the dam compared to the lower half portion.

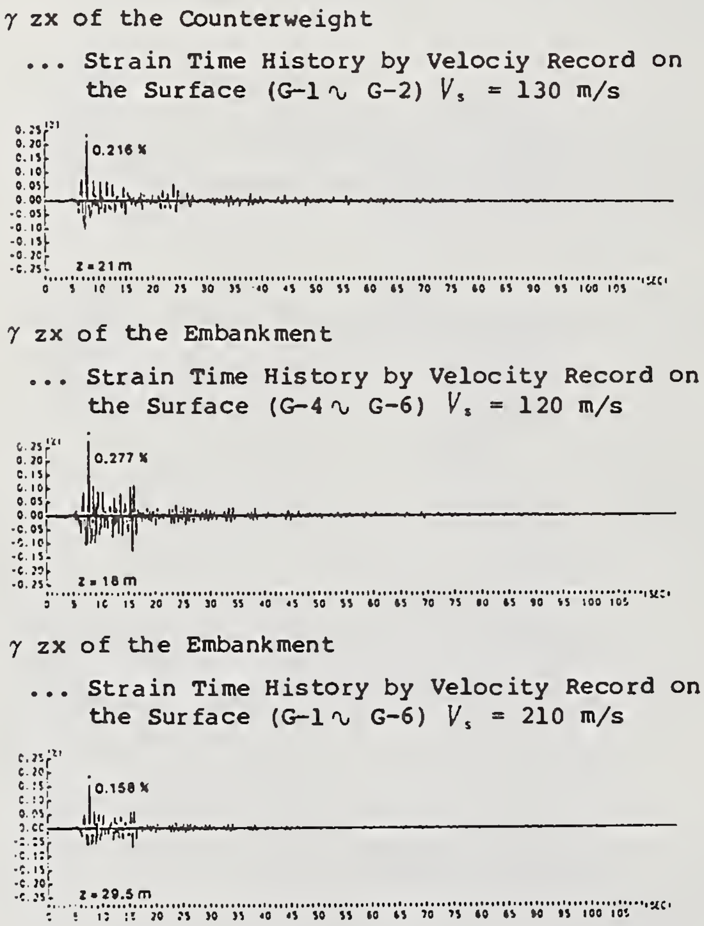


Fig. 16 Time History of Shear Strain by Velocity Records on the Surface (Upstream-Downstream Direction)

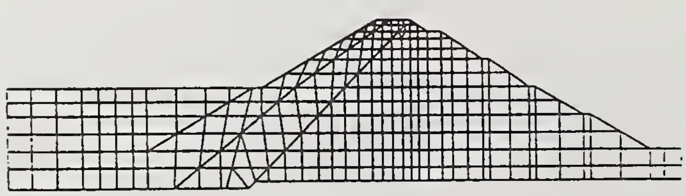


Fig. 17 Finite Element Model

The displacement amplification between the crest and the base for resonance can be determined if the embankment material is assumed to be viscoelastic. By utilizing this theoretical solution of shear vibration in a stationary state[6], the coefficient of viscosity can be calculated and, from the viscosity constant of the material can be calculated.

Displacement amplification factor becomes 2.25 (=6.00/2.67) from the maximum displacements at crest G-6 and base G-1 (refer to Fig. 5(b)), and the coefficient of viscosity is determined from the following equation:

$$\eta = \frac{2H^*}{2.4048}\sqrt{\rho G \gamma} \qquad (5)$$

where, height H* = 52 m by considering a standard triangular section for the dam, unit mass of the material is given by ρ = 1.75×10^3 kg/m^3, its shear modulus G = 80MPa (800 kgf/cm^2), coefficient γ = 0.12 which is determined from the displacement amplification factor of 2.25, and then η = 5.6×10^6Pa·s. The coefficient of viscosity and damping constant h has the following relation:

$$h = \frac{\eta\omega}{2G} = \frac{\pi\eta}{GT} \qquad (6)$$

As the period of the pulse, which can give the maximum amplitude in the displacement waveform of base G-1, is given by T = (7.56-7.37)x4 = 0.76 sec., and if this value is used as T then h becomes h = 0.29. On the other hand, from the pulse at crest G-6, T = (7.80-7.53)x4 = 1.08 sec. so that h = 0.20 is obtained. In the damping constant calculated as stated above, it should be noted that the recorded displacement waveform is not the one at resonance. From this theoretical relation between the deformation of the embankment during resonance and its viscosity coefficient, the coefficient of viscosity of η = 5.6 MPa·s and the damping constant of h = 0.2 to 0.3 are obtained for the embankment material.

4. BEHAVIOR OF THE DAM ESTIMATED BY FINITE ELEMENT ANALYSIS[7]

4.1 Input Data

In order to estimate the behavior, stress and deformation of the dam during the event, two-dimensional finite element analysis using QUAD-4[8] was performed for the maximum cross section shown in Fig. 2.

The model for analysis is shown in Fig. 17. Counterweight portion was limited to 55 m from G-2 point in the upstream side, the number of nodes is 359 and the number of elements is 351. The observed record for 30 seconds at G-1 point; (horizontal and vertical directions, refer to Fig. 4 for

Table 3 Dynamic Properties used in Dynamic Analysis

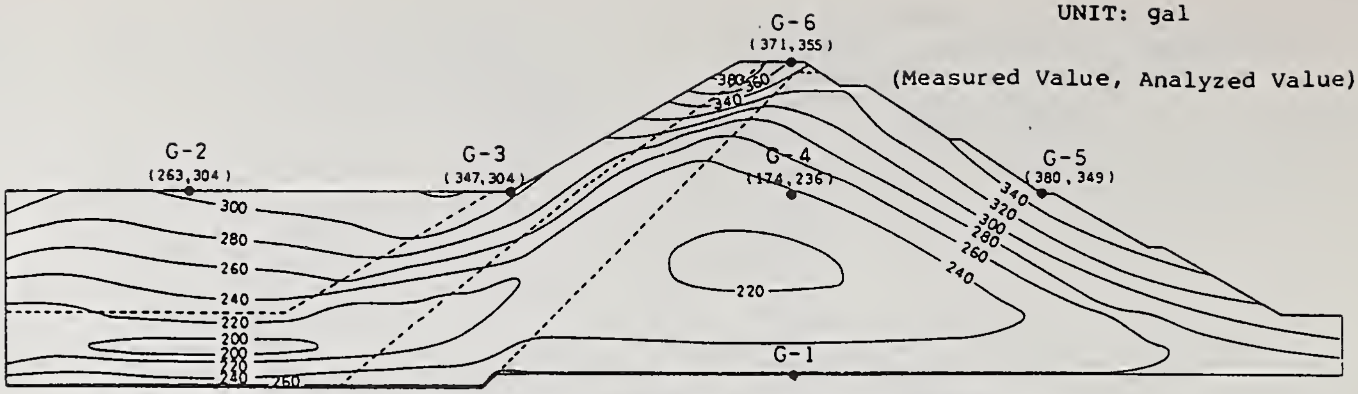
Properties / Zone	Shear Modulus: Shear Modulus in Small Strain G_0	Shear Modulus: Strain-Dependent of Shear Modulus	Poisson's Ratio υ	Damping Constant h
I	$G_0 = 100\frac{(7.32-e)^2}{1+e}\sigma m^{0.6}$	$\frac{G}{G_0} = \frac{4.0x10^{-3}}{4.0x10^{-3}+\gamma}$	0.40	$h = 0.15\frac{\gamma}{4.0x10^{-3}+\gamma} + 0.25$
II	$G_0 = 1100\frac{(2.17-e)^2}{1+e}\sigma m^{0.4}$	$\frac{G}{G_0} = \frac{1.3x10^{-3}}{1.3x10^{-3}+\gamma}$	0.35	$h = 0.25\frac{\gamma}{1.3x10^{-3}+\gamma} + 0.25$
III	$G_0 = 330\frac{(2.973-e)^2}{1+e}\sigma m^{0.5}$	$\frac{G}{G_0} = \frac{1.0x10^{-3}}{1.0x10^{-3}+\gamma}$	0.35	$h = 0.18\frac{\gamma}{1.0x10^{-3}+\gamma} + 0.25$
Counterweight	$G_0 = 100\frac{(7.32-e)^2}{1+e}\sigma m^{0.6}$	$\frac{G}{G_0} = \frac{4.0x10^{-3}}{4.0x10^{-3}+\gamma}$	0.40	$h = 0.15\frac{\gamma}{4.0x10^{-3}+\gamma} + 0.25$

horizontal motion) was input uniformly to the bottom boundary of the analysis model. Since there were no test values for the dynamic properties of the embankment materials, they were determined based on the published results of studies performed on a similar or the same kind of material. However, the seismic prospecting was conducted near the surface of the dam after the earthquake as stated before and the velocity of an elastic wave was measured down to the depth of several meters in each zone, so that the results of this measurement were also taken into account. Dynamic properties used in the analysis are shown in Table 3.

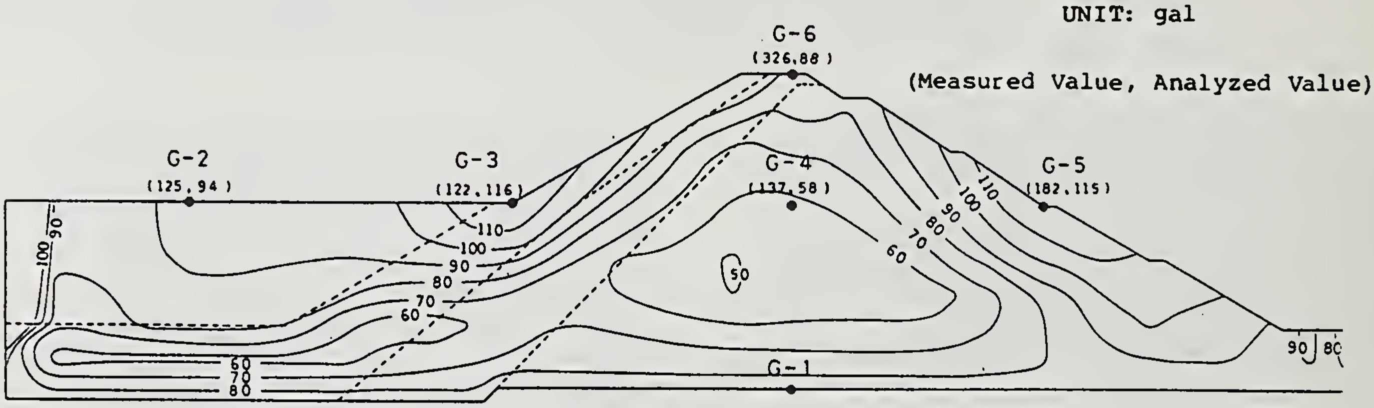
As shown in Table 3, dynamic shear modulus G_0 is given by function of confining stress (mean principal stress σ_m) and density (void ratio e). Therefore, incremental embankment analysis was made before the dynamic analysis for determining static stress and deformation. And G_0 calculated from the results of the above analysis was given to each element. The properties of the material used in the incremental embankment analysis were determined from the quality control test results during embankment work and the strength test results on undisturbed samples of materials after completion of embankment. The dissipation damping between the dam and foundation was assumed to be 0.25, which is somewhat large because the foundation was a sand layer, the difference in physical properties between the sand layer and the dam is considered to be relatively small, and frequency characteristics of earthquake motion are relatively similar between them.

4.2 Results of the Analysis

Distribution of the maximum response acceleration are shown in Fig. 18. The maximum response values at each portion correspond to the maximum values of input earthquake motion. The response acceleration decreases inside the dam and increases at the crest and the slopes of the dam, which agrees fairly well with the distribution tendency of measured values. However, the vertical maximum acceleration at the crest of the dam is considerably small compared to the observed value, so that the large amplification from the middle elevation to the crest of the dam is not well simulated. Fig. 19 shows the response acceleration time history and spectrum at the crest of the dam (G-6 point). The waveforms significantly indicate a response to long period components of input values, and indicated large responses with a spike-shaped short period which are seen in observed records do not appear. Similar results are obtained also at other observation points. Transfer function of horizontal acceleration at the crest of the dam is shown in Fig. 20. In this case, response is hardly made to the frequency components higher than 3 Hz, which indicates the predominance of low-order vibration mode with a simple deformation shape. The response frequency of the primary mode is 1.6 Hz and almost coincides with the primary natural frequency derived from the records of the earthquake motion observed and Matsumura's solution explained in the previous chapter. Fig. 21 shows the displacement time history for the crest of the dam (G-6) and the inside of the dam (G-4) observed and analyzed, where both observed values and analyzed values are the relative displacement to the base (G-1). The analyzed values are smaller than observed values at the crest of the dam and it is to the contrary inside the dam. This seems to occur from some difference between the properties of the materials of the dam used in the analysis and those of actual materials and also from the effect of errors in the calculating method. However, the waveforms are similar to each other, and the analyzed values agreed relatively well with the observed records inside the dam. From this, it is considered that analyzed values which are very close to actual values may be obtained even for strains or stresses which occur inside the dam. Fig. 22 shows the

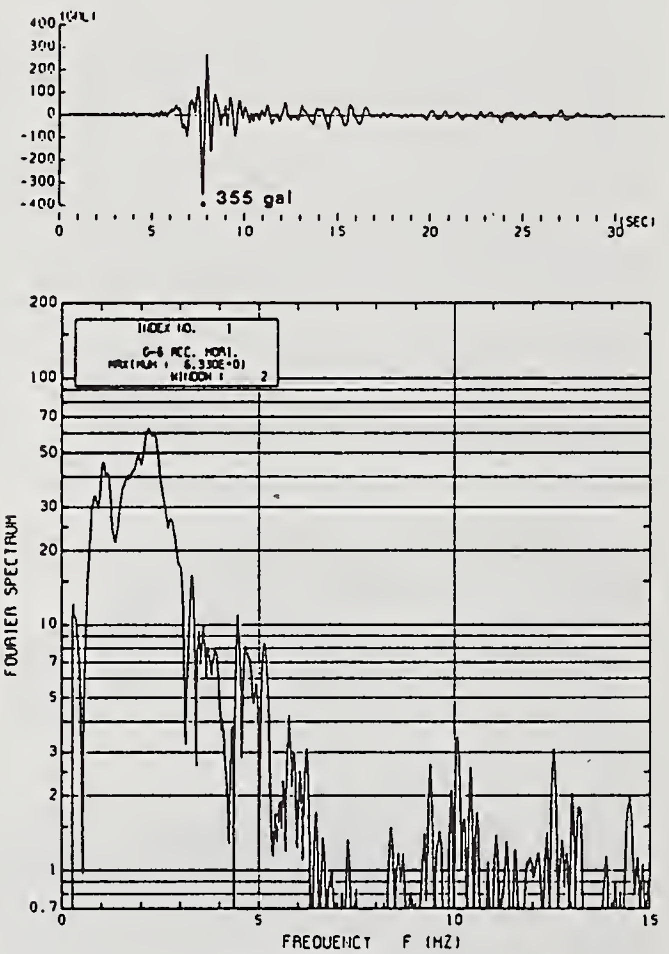


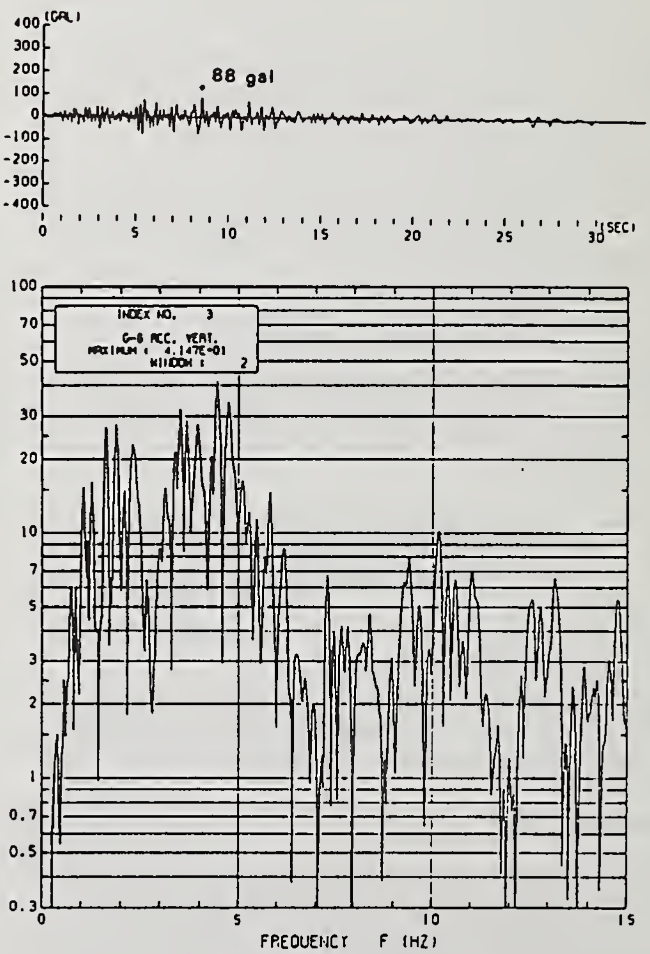
(a) Distribution of Maximum Horizontal Acceleration



(b) Distribution of Maximum Vertical Acceleration

Fig. 18 Distribution of Maximum Response Acceleration





(a) Horizontal Direction a_x (b) Vertical Direction a_z

Fig. 19 Analyzed Response Acceleration of the Dam Crest (G-6)

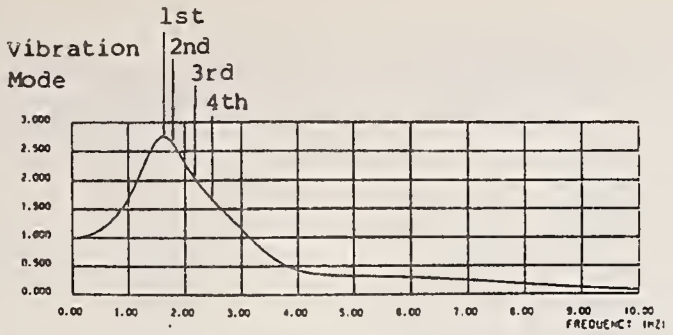
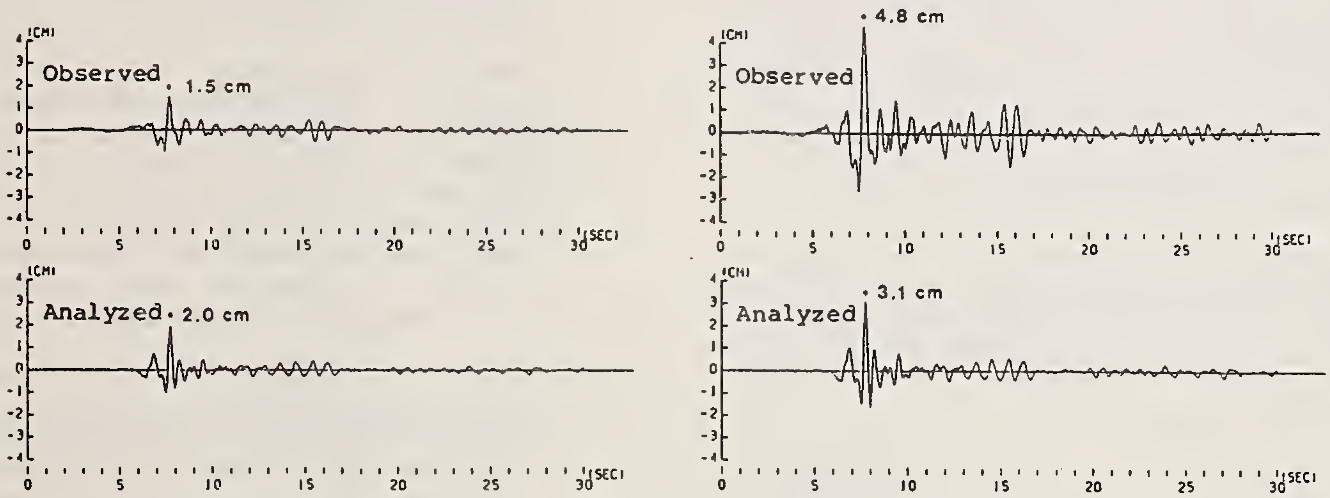


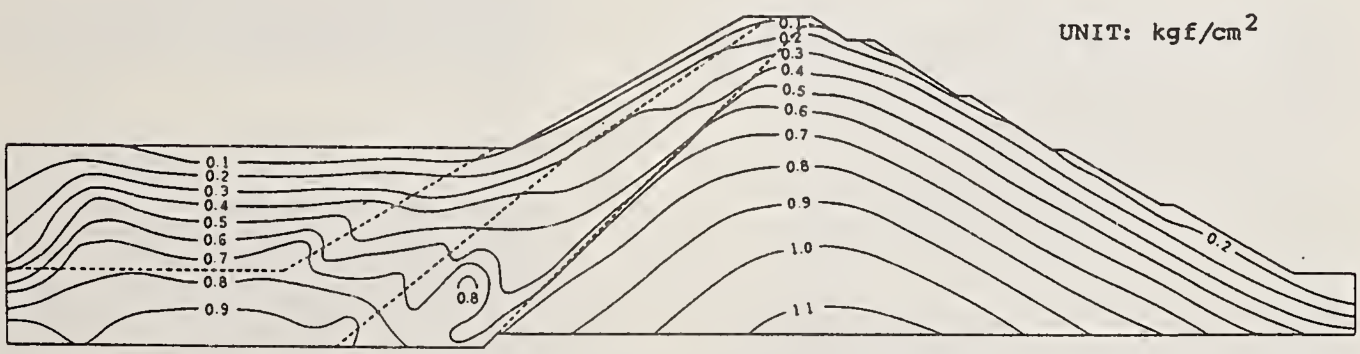
Fig. 20 Transfer Function of the Dam Crest (g-6)



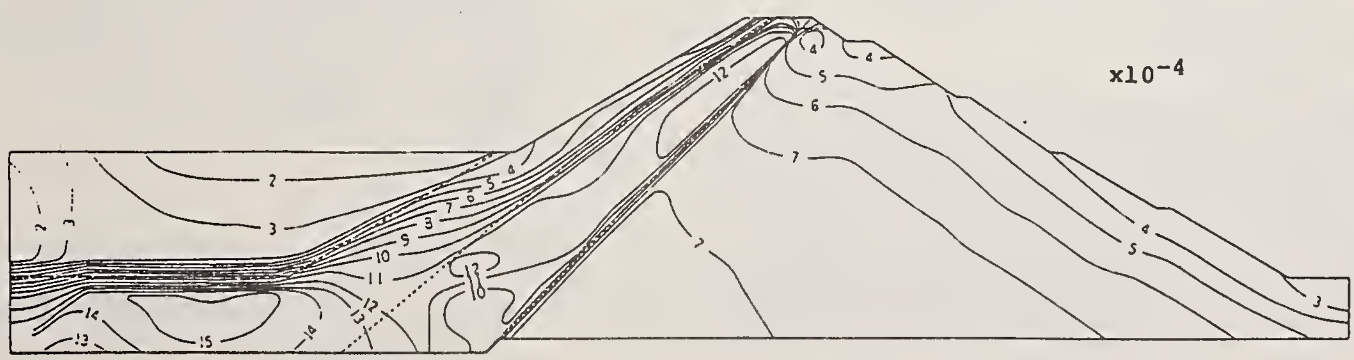
(a) Inside of the Dam (G-4)

(b) Crest of the Dam (G-6)

Fig. 21 Comparison of Relative Displacement in Horizontal Direction



(a) Maximum Shear Stress



(b) Maximum Shear Strain

Fig. 22 Distribution of Maximum Shear Stress and Strain distribution of maximum shear stress and maximum shear strain obtained by the analysis. Both values are dynamic values only due to earthquake motion without containing static stress and strain. Maximum shear strain occurs almost in the horizontal (vertical) direction. From the results shown in Fig. 22, the vertical distribution of maximum strain inside the dam and counterweight is determined and indicated in Fig. 15. According to this, the maximum strain is almost of the same order of the value easily determined by equation (4) from the records. According to Figs. 15 and 16, maximum value of shear strain inside the dam is large and exceeds 10^{-3}. Shear modulus decrease made by this large strain, reduces the generated shear stress. Also, in the distribution of shear stresses, they simply increase from the surface of the dam to the depth direction and are slightly concentrated in low elevation portion of Zone I.

The behavior of the dam estimated by the finite element analysis stated above has almost agreed with the behavior clarified from the records of actual observation.

5. CONCLUSIONS

1) Earthquake motion occurred during this event can be characterized by a large pulse of displacement toward the downstream direction.
2) Primary natural period of the Nagara Dam is about 1.6 Hz and is almost equal to the theoretical solution.
3) Shear wave velocity and shear modulus determined from the observation records are 20 to 30% lower for shear wave velocity and 40 to 50% lower for shear modulus than those obtained from experimental formulas based on seismic prospecting and laboratory tests.
4) Maximum shear strain generated at the main dam and counterweight was about 10^{-3}.
5) Damping constant was about 0.2 to 0.3.
6) The behavior of the dam clarified by the records of strong earthquake motion has been almost substantiated by the behavior of the dam estimated from the dynamic finite element analysis.

ACKNOWLEDGEMENT

The authors would like to express the deepest gratitude to Mr. Kimoto, Chief of the Boso Channel Construction Office, Water Resources Development Public Corporation, and other persons who offered the recorded data to the authors and are making efforts for maintaining strong-motion accelerographs for observing the precious records of the strong-motion earthquake.

REFERENCES

1) Japan Meteorological Agency: Outline of the Earthquake Occurred in December 1987 (in Japanese) (Jan. 1988).

2) Sasaki, Y.: Damage by the Chiba-ken Toho-Oki Earthquake, 21th Joint Meeting, Wind and Seismic Effect, UJNR (May, 1989).

3) Kimoto, E., Yamazaki, A., Fukushima, R., Seko, I., and Tanaka, H.: Situation of the Nagara Dam for Boso Channel in the Chiba-ken Toho-Oki Earthquake (in Japanese), Tsuchi-To-Mizu, No. 73, pp. 89-95 (Jan. 1988).

4) Seko, I., Kashiwagi, J., Saito, A., Aoki, Y., and Kogo, K.: Observation Records at the Nagara Dam for the Chiba-ken Toho-Oki Earthquake of December 17, 1987 (in Japanese), Report of Fill Dam Division, No. 8704, the First Research Department, Research Institute of Water Resources Development Public Corporation (Jan. 1988).

5) Sasaki, Y., Nishimura, M., and Kuwabara, T.: Dynamic Properties of the Embankment Materials of the Nagara Dam estimated from the Strong Motion Records during the Chiba-ken Toho-Oki Earthquake of December 17, 1987 (in Japanese), Technical Memorandum of Public Works Research Institute, No. 2702 (Jan. 1989).

6) Matsumura, M.: Deformation of Earth Embankment during Earthquake (in Japanese), Journal of Research No. 28, Public Works Research Institute (Oct. 1934).

7) Kashiwagi, J., Aoki, Y., Tsukamoto, K., and Kogo, K.: Dynamic Behavior of the Nagara Dam during the Chiba-ken Toho-Oki Earthquake (in Japanese), Symposium on the "Behavior of the Ground and Earth Structures during the Chiba-ken Toho-Oki Earthquake", pp. 205-212, JSSMFE (Jan. 1989).

8) Idriss, I.M., Lysmer, J., Hwang, R., Seed, H. Boiton: QUAD-4:A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Element Procedures, Report No. EERC73-16, University of California, Berkeley (1973).

Analysis of Structural Response Records During the Whittier Narrows, California, Earthquake, October 1, 1987

A. Gerald Brady[1]

ABSTRACT

The extensively instrumented building at 12440 Imperial Highway in Norwalk, California, was excited by horizontal ground accelerations peaking at 0.2 to 0.3 g during the M5.9 Whittier Narrows, California, earthquake, October 1, 1987. Specific frequencies corresponding to lower frequency modes are chosen from the accelerograms and their spectra. Refiltering to remove all frequencies lower than each chosen frequency, integrating for displacements, and separating out translations, torsions, and in-plane bending, permit graphic evidence of the amounts of each of these motions that are present at each modal frequency. The instrumentation was intended to provide just this type of engineering information.

KEYWORDS: Analysis; Building instrumentation; Combined modes; Structural response.

1. INTRODUCTION

The Whittier Narrows earthquake, October 1, 1987, magnitude 5.9, triggered 52 stations in the National Strong Motion Instrumentation Network operated by the U.S. Geological Survey for both the USGS and other federal and local agencies (Figure 1). These 52 stations represent 362 data channels (Etheredge and Porcella, 1987).

2. NORWALK STATION

The Norwalk building, directly south of the epicenter and 15 km distant, was extensively instrumented prior to the current USGS project for instrumenting special structures. It is a 7-story moment-resisting steel-frame building, instrumented jointly with Bechtel Power Corporation, the owner. Dimensions are 141m x 41m x 35m high. There are 27 channels from accelerometers within the building, and 6 channels at two ground level sites 30m and 131m from the building.

Horizontal peak ground accelerations were 0.2-0.3g at these free-field sites, 0.16-0.2g in the basement, and .41g at roof level.

3. PURPOSE OF THIS EXTENSIVE INSTRUMENTATION

When extensive instrumentation is installed, it is usually designed to investigate a few important modes at lower frequencies, and occasionally, special dynamic behavior. Accelerograms from significant events provide this data. Resonances in fundamental modes are often clear, after the strong input ground motion is over (Figure 2). During the strong motion, transients in higher modes are also present, sometimes concealing the contribution from lower modes.

Both lower mode resonances and higher mode transients may be the important, damaging, or destructive motions. The instrumentation here should show both, since it includes the basement, 1st and 2nd floors for damaging transients, as well as the 5th and 8th levels. The accelerations during this event were not sufficient to cause structural damage.

This paper shows how the use of various options in the USGS processing programs (Converse, 1984) can give a visual representation of the structural displacements at frequencies close to natural resonant frequencies. A few of the more significant results are presented.

4. PROCEDURE

We investigate the total structural response displacements, as represented by the motion at the accelerometer locations, at specific frequencies, which have been previously obtained from Fourier spectra.

4.1 Filter out all content at lower frequencies than the frequency of interest, and integrate twice for displacement, which puts a frequency-squared attenuation onto the higher frequencies. This leaves displacements at the chosen frequency as the only prominent motion.

4.2 Separate the translations from the torsion and horizontal in-plane bending of the floor slabs so that each can be looked at alone, even though the low-frequency modes might contain combinations of these motions (Figures 3 and 4).

5. AMPLITUDES OF SEPARATED COMPONENTS

Figure 5 shows displacements at four levels, in a graphic display of the north-south translation component, of a mode whose frequency was 0.76 Hz (Table 1). The corner frequency for the high-pass filter was 0.66 Hz. Figure 6 shows the only motion recorded in the east-west direction, a translation, in which a modal frequency of 2.4 Hz is sought. Both plots use consistent scales for each time history.

Table 1 lists the modal frequencies for N-S and

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E-W translations, and torsion, measured during the resonating part of these responses. The frequencies of these listed modes are closely proportional to the expected sequence 1, 3, 5. Transient peaks at the 8th level indicate the response during strong motion input. Transient peaks at the basement, downhole caisson and freefield levels indicate the extent to which the building amplifies or attenuates the freefield motion.

Table 2 lists frequencies and transient peak values for rocking and in-plane bending. Freefield rocking was not measured. In-plane bending is not associated with any specific input patterns. No measure of amplification or attenuation by the building for rocking or in-plane bending is therefore available.

The frequencies listed in Tables 1 and 2 are identified in Figure 7. From the transient peaks in Table 1 the ratio of basement peak to ground level peak is calculated and plotted against the corresponding frequency. As can be seen from this figure, there is a trend that low-frequency modal components be amplified, and high-frequency components attenuated.

6. CONCLUSIONS

Options available in the AGRAM programs permit plots to be drawn that show clearly the amount of each modal component (translations, torsion, in-plane bending) at each modal frequency.

Among the modes studied, there is a tendency for low-frequency modal components (less than 2 Hz) to be amplified (by 1 to 12 times), while higher frequency modal components (greater than 2 Hz) to be attenuated (by 0.6 to 1 times).

7. REFERENCES

Etheredge, E., and R. L. Porcella (1987). Strong Motion Data from the October 1, 1987 Whittier Narrows earthquake. U.S.G.S. Open-File Report 87-616.

Converse, April (1984). AGRAM, a series of computer programs for processing digitized strong-motion accelerograms, version 2.0. U.S.G.S. Open-File Report 84-525.

Table 1

Resonant Frequencies

and Transient Modal Peaks

		Mode: 1	2	3
N-S Translation				
Resonant Freq.	(Hz)	0.76	2.3	3.6
Transient peaks	8th	6	0.4	0.03
(cm)	B/C	2	0.25	0.04
	GL	2	0.25	0.07
E-W Translation				
Resonant Freq.	(Hz)	0.84	2.4	4.0
Transient peaks	8th	3.2	0.35	0.06
(cm)	B/C	1.0	0.12	0.05
	GL	0.85	0.18	0.07
Torsion				
Resonant Freq.	(Hz)	1.08	1.4	3.9
Transient peaks	8th	57	65	4
(μ rad)	B/C	18	35	4
	GL	-	26	5

Note: 8th: 8th level or roof.
B: Basement level
C: Caisson bottom level
GL: Ground level, 30 m and 131 m from the building

Table 2

Rocking and In-Plane Bending

		Mode: 1	2	3
Rocking				
Resonant Frequency (Hz)		1.1	~4	5.5
Transient peak (μ rad)		10	4	-
In-Plane Bending				
Resonant frequency (Hz)		0.76	2.4	
Transient peak offset				
at center (cm)	8th	1.8	0.8	
	B	0.3	0.15	

Note: 8th: 8th level or roof
B: Basement level

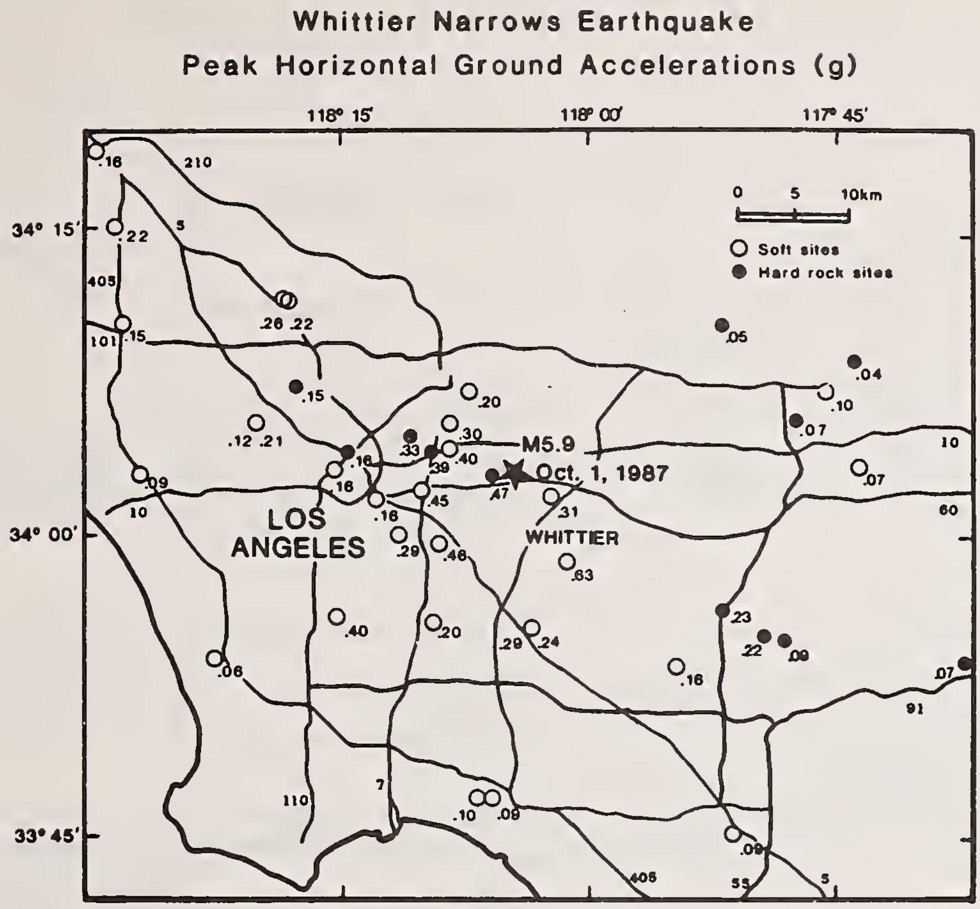

Figure 1. Epicenter of October 1, 1987, M5.9 Whittier Narrows earthquake. Peak ground accelerations at Norwalk, 15 km south of epicenter, are 0.24 and 0.29g.

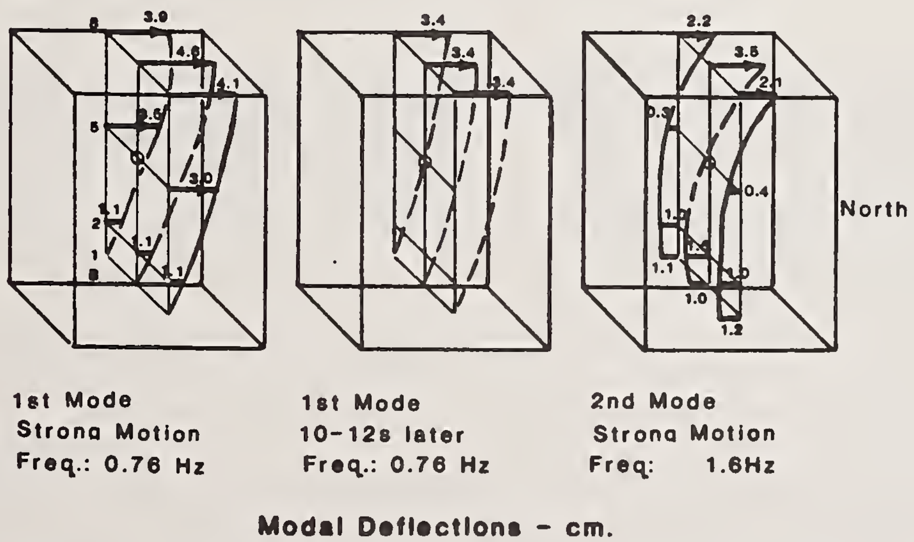

Figure 2. Estimates of 1st and 2nd mode N-S translation mode with combined in-plane bending of the floors, scaled from the original accelerograms prior to digitizing.

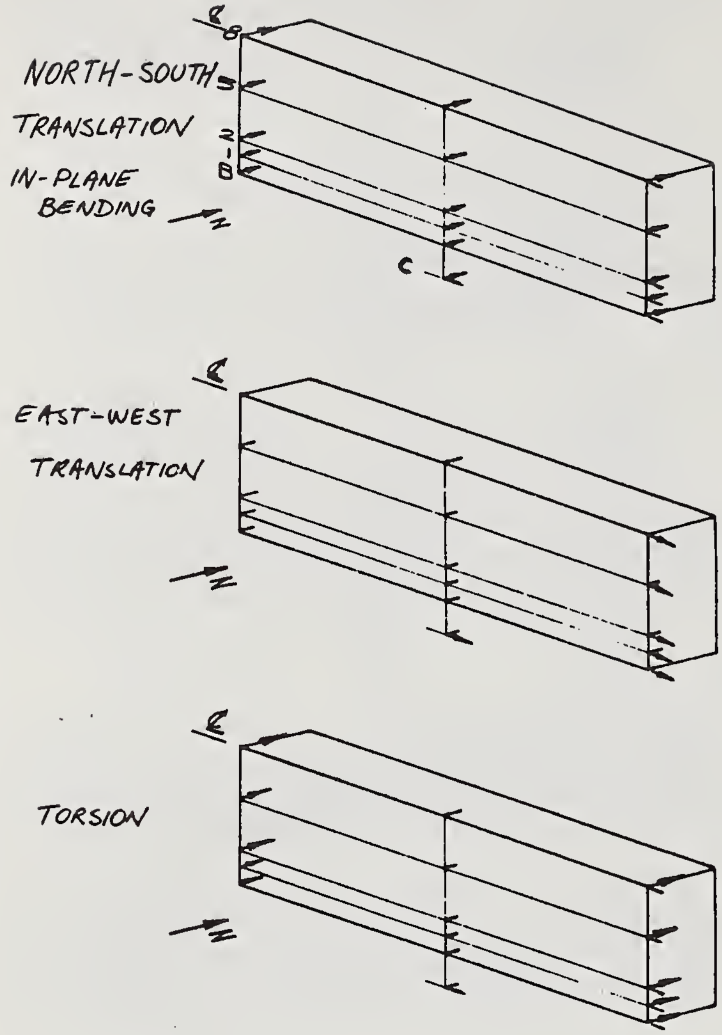

Figure 3. Grouping of sets of recorded channels for investigations of 4 different motions: N-S translation, in-plane bending, E-W translation, and torsion. Transducers are placed at the basement level, the 1st, 2nd, 5th, and 8th (or roof) floor levels, and in a foundation caisson, 9.1 m below basement level.

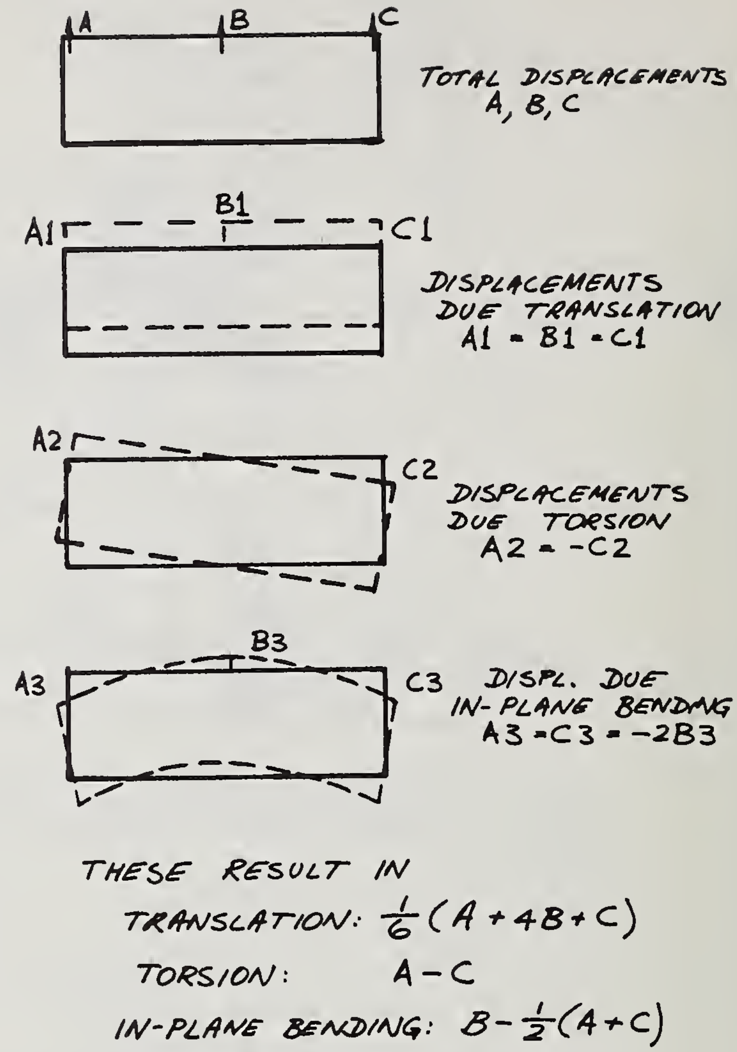

Figure 4. Plan view of an instrumented floor with the N-S motions (A, B, C), split into separate translation, torsion, and in-plane bending.

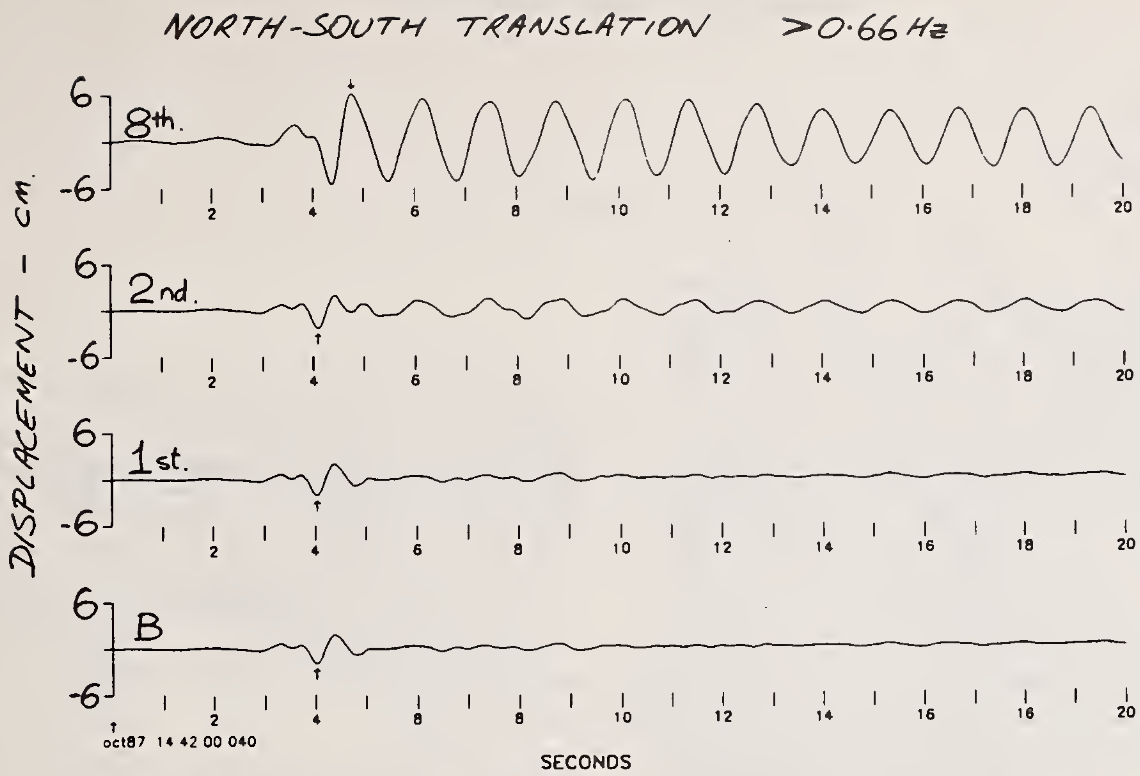

Figure 5. N-S translation displacements after filtering the relevant expressions and integrating twice. Displacements at 8, 2, 1, and Basement levels are shown.

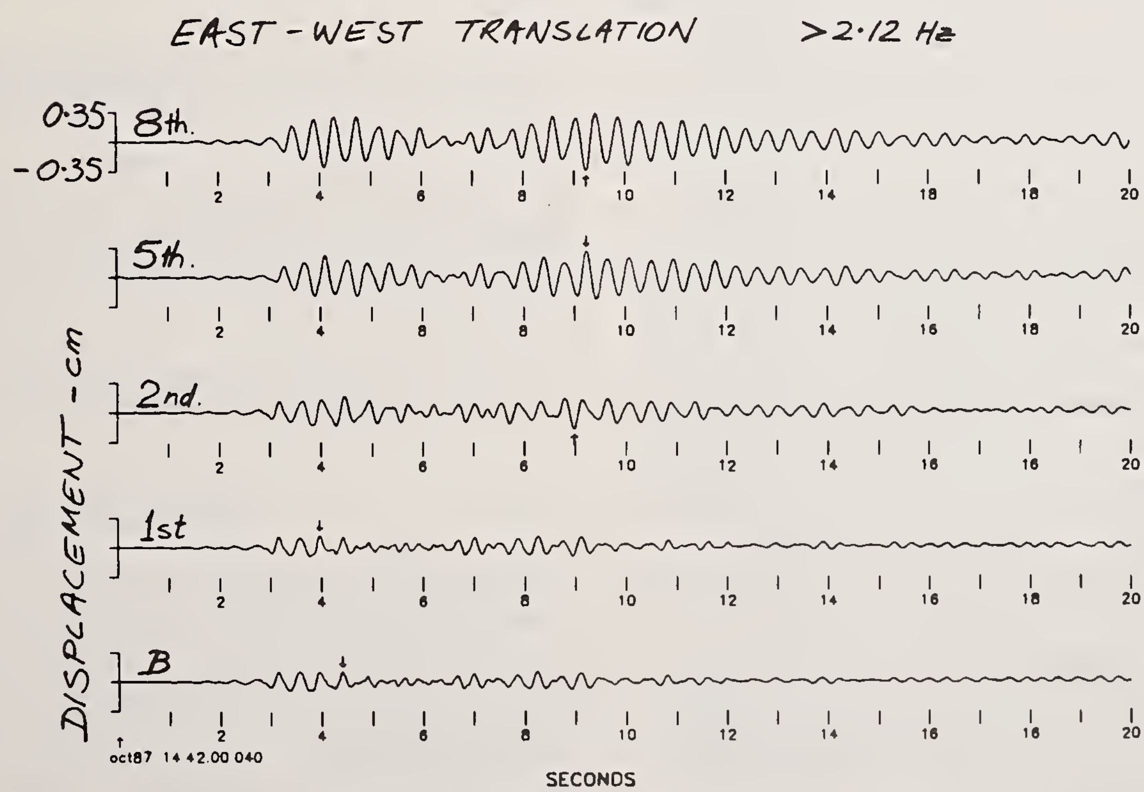

Figure 6. E-W translation displacements, filtered at a frequency designed to show 2nd mode, at 8, 5, 2, 1, and Basement levels.

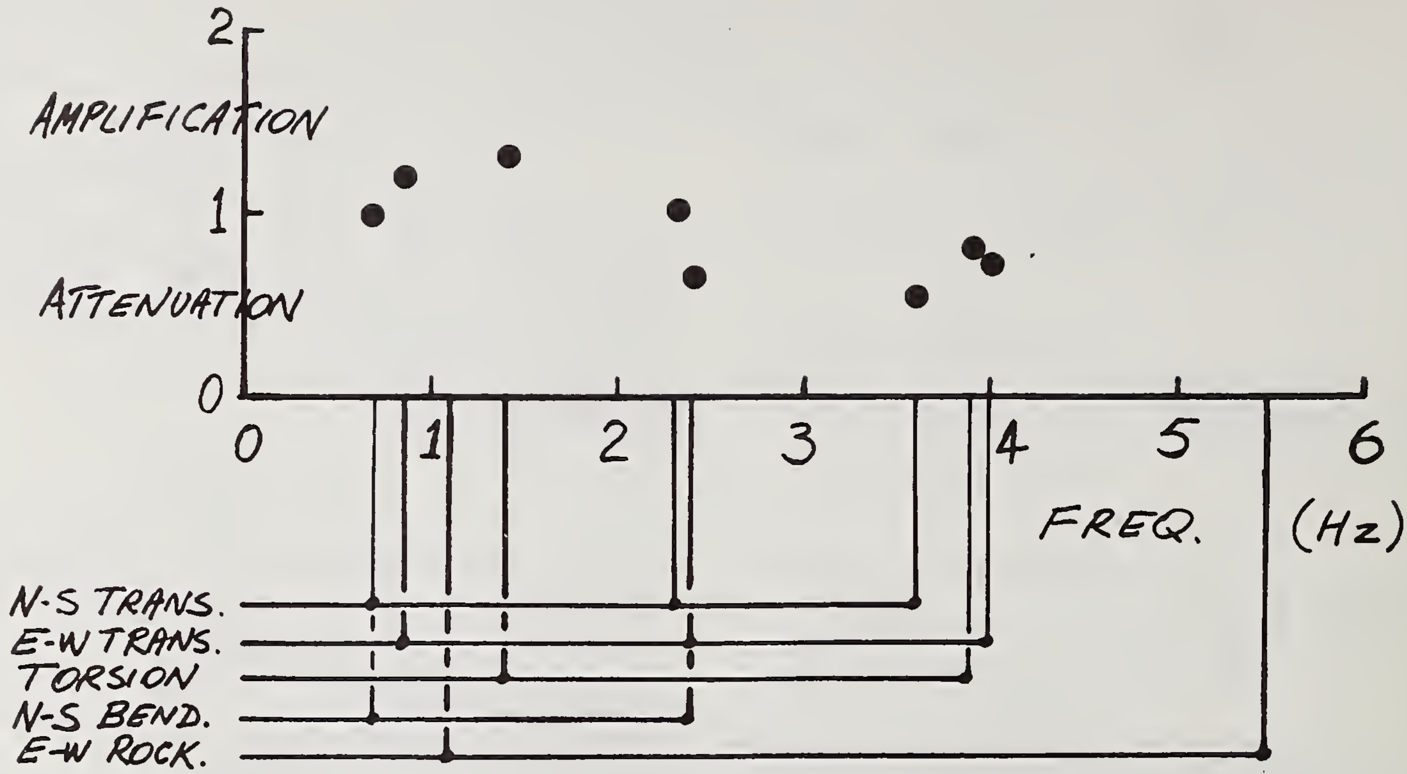

Figure 7. Building vibrational modes whose basement and downhole motion is amplified or attenuated with respect to the freefield motion. Modes are identified by their frequencies. There is a tendency for the low frequency modes to be amplified, and the high frequency modes to be attenuated.

Behaviours of Dams During Chiba off Earthquake

by

Norihisa MATSUMOTO*1, Nario YASUDA*2

[SUMMARY]

The paper describes the behaviours of dams during December 17, 1987 Chiba off earthquake. The magnitude of the event was 6.7 on the Richter scale. Houses and public works in Chiba Prefecture suffered damages due to this earthquake. We made investigations on the eleven dams which were located within about 30 kilometers from the epicenter. Surface cracking on the dam crest was found in two earthfill dams. However, cracks were limited only in the shallow depth. There were relations between the foundation geology and behaviours of dams.

[KEY WORDS] Dam, Earthquake damage, Foundation geology, Chiba off earthquake

1. INTRODUCTION

The earthquake of a magnitude of 6.7 on the Richter scale occurred on December 17, 1987, with the hypocentral depth of 58 km, at 35° 21' North latitude and 140° 29' East longitude, according to the Japanese Meteorological Agency. The paper describes the behaviours of dams during this earthquake.

2. INSPECTION OF DAMS IMMEDIATELY AFTER THE EARTHQUAKE

Inspection of dams were made at forty four dams immediately after the earthquake. These dams were all higher than fifteen meters and located in the areas where the Japanese Seismic Intensity Scale was equal to or greater than 4, and/or recorded peak acceleration was larger than 25 gals.

As a result of inspections, unusual behaviours were reported at five dams as shown in Table-1.

3. FIELD INVESTIGATIONS

We conducted field investigations on eleven dams as listed in Table-2. These dams include not only dams in Table-1, but also those dams located in the areas within thirty kilometers from the epicenter where the damage of earth structures were mostly observed. Yamakura Dam listed in Table-1, was excluded from our field investigations, since the unusual behaviour of Yamakura Dam was cracking of the abutment of the bridge in the reservoir not but the damage of the dam. The dams within the thirty kilometer radius from the epicenter were all fill dams. Therefore, Kameyama Dam, a concrete gravity type dam was added to the investigation, although its epicentral distance was greater than thirty kilometers. Kori Dam being farther than thirty kilometers from the epicenter were also added to the investigation, since the acceleration records were obtained at this dam. Locations of the dams were illustrated in Fig. 1, which also shows the outline of the area geology. Of eleven dams investigated, three–Kori, Nagara, and Konakagawa–were built on foundations of the Quaternary, and the other eight dams were on the late Tertiary.

We also made investigations on damaged river embankments of Tone river which are also earth structures similar to fill dams.

3.1 Investigation of dams

3.1.1 Kori dam

Maximum cross section of the dam is shown in Fig. 2. The foundation consists of Kasamori formation and Jizodo formation of the Pleistocene of the Quaternary period, and the impervious core of the dam was founded on the mudstone of Kasamori formation. The core was built with weathered mudstone, and the sands were used for random materials. The downstream slope were protected with grass, and furnace slugs were used for upstream riprap.

Caretakers of the dam were cleaning the boat in the reservoir at the time of earthquake, and they told us that they had not felt much shaking during the earthquake. Accelerometers were installed at three locations, the dam crest, middle elevation and the base. Although the accelerations in the dam axis direction at the crest and the base were not recorded, the up-down stream direction and vertical component were recorded. The peak value in the horizontal and vertical direction at the base was 21 and 13 gals, respectively, and the one at the crest was 62, and 177 gals, respectively. Since the instrument at the foundation malfunctioned

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during the main mothion, the peak values at the foundation were less than the real ones.

Leakage were measured by using the weir at the downstream toe of the dam. Leakage was the same value of 152 litre per minute before and after the earthquake and no change was observed at the dam.

3.1.2 Kameyama dam

The section and elevation of Kameyama Dam are shown in Fig-3. The geology of damsite consists of a series of mudstone and sandstone of Tertiary age. The sandstone outcrops on the higher elevation of the left alutment. The staff was in the dam management office when the earthquake occurred and they felt big shaking. But, nothing on the desk overturned in their office. Uplifts and leakage are measured two times a month at Kameyam Dam. Fig. 4 shows the leakage and uplift before and after the earthquake. Uplift No.8 and No.14 increased shortly after the earthquake, deceasing afterwards. No damage was observed at Kameyama Dam.

3.1.3 Katsuura Dam

Fig. 5 shows the section of Katsuura Dam. Sndstone known as Kiyozumi formation of Miocene, Tertiary age comprises the foundation rock. Kiyozumi sandstone which underlies the Kurotaki uncontormity is a hardened sedimentary rock. Weathered mudstone and loam were used for core materials and mudstone for random materials as shown in Fig. 5. Although four sccelerometers were installed at the downstream slope, no record was obtained due to malfunctioning of the instruments. Wet portions were found at the downstream berm. Since these wet areas gathered moss, they seemed to have no relations with the earthquake. No damage was observed at the dam.

3.1.4 Onjuku dam

Typical cross section of the dam is shown in Fig. 6. Bedrock of the site is comprised of mudstone of the Pliocene, Tertiary age. Weathered mudstone and loam are used for zone I, i.e. impervious material, and mudstone is used for zone II, i.e. semi-pervious material. The water level of the reservoir was 30 to 40 cm below the full reservoir level, when the event occurred. The leakage measured at the weir of the downstream increased just after the event, but returned to the previous value in a few days. The normal leakage amounts 40 litre per minute. No damage was observed at dam.

3.1.5 Nakuma dam

Fig. 7 shows the elevation and section of Nakuma Dam. The foundation consists of alternation of thin mudstone and sandstone of the Pliocene of Tertiary age. The core material is built with weathered mudstone and loam, and randum zone with mudstone. The upstream slope was protected with flexibly connected concrete blocks. Leakage is not measured at the dam. No damage was found at the dam.

3.1.6 Azuma No.1 dam

The section of the dam is shown in Fig. 8. The foundation rock consists of mudstone of the Pliocene of Tertiary age. Mudstone is used for random zone and weathered mudstone and loam for core materials. Neither change of leakage nor damage was found due to the earthquake.

3.1.7 Azuma No.2 dam

The plan is shown in Fig. 9. Azuma No.2 Dam is very close to No.1 Dam and the geology of the foundation is very similar to No.1 dam. Neither damage nor change of leakage was observed.

3.1.8 Arakine dam

The section of the dam is shown in Fig. 10. The geology of damsite is almost the same with that of Azuma No.1 Dam or No.2 Dam. Impervious core was built with loam and westhered mudstone, and random zone with mudstone. Although accelerometers were installed, no record was obtained due to disorder of the instruments. The water level was 20 to 25 cm below the normal pool level which is equal to the crest elevation of non-gated overflow spillway. According to the talks of the operationnal personnel, the wave of 1 meter height occurred during the earthquake, and some amount of water overtopped the spillway crest. The leakage measured at the downstream drain varied from 62.4 liter/min before the earthquake to 114.0 liter/min just after the event and to 108.0 liter/min on February 10th, 1988, as shown in Fig. 11. No damage was observed at the dam.

3.1.9 Uryuko dam

Section and elevation of the dam are shown in Fig. 12. No damage was found at the dam.

3.1.10 Nagara dam

Fig. 13 shows the typical cross section of Nagara Dam. The dam was just under the test filling of the reservoir after completion when the dam is subjected to the earthquake. The settlement was observed at the dam. Cracking of the asphaltic concrete of the dam crest pavement and opening of the concrete block of the spillway were observed at Nagara Dam.

Fig. 14 illustrates the geologic formation of damsite. There is Kazusa group below El. ±0m, and is Shimousa group above it. Both groups belong to middle Pleistocene age. Kazusa group is comprised of mudstone (El.–45.0m below) and of sand (El.–45.0m above). Shimousa group which exists higher than El.±0m consists of medium to fine grained sand (El ±0m to 20m) and of 2 to 3m thick mudstone at about El. 23m. Above it, there are sands of Jizodo and Narita formation. Number of blow of SPT of these sands exceeds 50, if they are not weathered. The minimum number of blow of SPT was 30 in the foundation sands of Naga Dam.

Loam is used for impervious Zone I, and silty mudstone is used for outer shell Zone III, and sands are used for Zone II.

The water level of the reservoir was very low when the event occurred and it was about El. 57.0m which is approximately equal to the elevation of the surface of upstream counterweight. The leakage from the drain increased from 44 liter/min before the earthquake to 106 liter/min 30 minutes after the earthquake, and reduced to 45 liter/min 24 hours after the earthquake. The increase of pore water pressures was 0.1 kgf/cm^2 at the maximum.

Cracking of asphaltic concrete of the dam crest was observed at four locations, all of which coincide with the location of piezometer hole. This implies that cracking is related with the stress concentration around the hole of asphaltic concrete pavement. The Water Resources Development Public Corporation (referred to WRDPC hereafter), the owner of Nagara Dam, excavated the exploratory pit to investigate the nature of cracking. Opening of the crack was so small that nobody could recognize it without very carefull observation. The direction of cracking was almost perpendicular to the dam axis. According to this investigation, cracking proved to reach two meter depth at maximum. Later, cracked portion was removed and backfilled with the same impervious material.

The maximum settlement in the dam body was 30 mm at the locations of middle elevation of upstream slope and left abutment as shown in Fig. 15. 700 mm camber had been provided for Nagara dam. The amount of settlement due to the earthquake is much less than the amount of total embankment consolidation expected for the dam.

The embankment settled several centimeters due to the earthquake at the contact with the spillway concrete block, while the settlement of 1 to 2 cm had already occurred before the earthquake. The opening of construction joint of the spillway block amounted 95 mm at the top just after the earthquake, although the joint had already been open to some amount before the earthquake. Ten accelerometers with three component sensors was placed at Nagara Dam. Peak acceleration at the dam base was 262 gals, and at the dam crest was 369 gals. Accelerograms recorded at Nagara Dam contain long period component of 1.0 Hz as well as short period component. Assuming that the natural period T(sec) of shear wave vibration is given by $T=4H/V_s$, where H is thickness of the layer in meters and V_s is shear wave velocity in meter per sec, $T=4H/V_s=1.0$ sec, if H=125m (dam height plus the depth of the sand layer) and $V_s=$ 500m/sec.

3.1.11 Konakagawa dam

Section of the dam is shown in Fig. 16. Several cracks were found at the dam crest paved with asphaltic concrete. As shown in Fig. 17, there were three cracks in the up-down stream direction near the left abutment, and three cracks in the dam axis direction, which are 2 to 3 mm wide. The concrete block for the upstream protection was displaced 2 to 3 cm at the contact of concrete block and the crest pavement. The slope of the concrete block protection work changes abruptly at El. 36.94m, and concrete blocks are separated 15 to 16 cm at this elevation in the central part of the dam. Furtheremore swelling of the protection work was observed in the lower part of this area.

3.2 Investigation of river embankments

River embankments were suffered damages such as cracking, slide and settlements due to Chiba off earthquake in and around Sahara City. Most of damaged embankments were located where the foundation is comprised of the old river bed deposit, and SPT value of the foundation is 10 from surface to 5 m, and ranges from 10 to 20 to the further depth of 10m. There were liquefaction craters in the nearly paddy field. Hitachi Tone river embankment was subjected to 50 cm subsidence. This embankment was also located in the old river bed. STP value was 1 to 2 up to 5 meter depth, and 30 at maximum from 5 to 9.5 meter in depth. The portions of the embankment also consisted of a extremely loose sand.

4. CONCLUSIONS

1) There were no major damages of dams due to 1987 Chiba off Earthquake.
2) Unusual behaviours were observed at Nagara and Konakagawa Dam.
   At Nagara Dam, the foundation motion seemed to be larger than other sites, because firstly there are thick sand deposits at the site foundation, and secondly the site is closer to the main earthquake area. Hair craking occurred at the creast in the direction perpendicular to the dam axis. The depth of cracks ranged 1 to 2 meters and the crack did not pass through from upstream to downstream surface.
   Cracking and swelling occurred at Konakagawa Dam presumably because the upstream slope had an abrumt change. The same type of damage had also occurred in the similar dams due to 1983 Mid Japan Sea Earthquake.
3) Damage of river embankment in Tone River were caused by liquefaction of loose sand deposit. The difference of damage between river embankments and embankment dams is mainly due to the difference of foundation condition.
4) Acknowledgements

We wish to thank River Bureau of Ministry of Construction, River Division of Chiba Prefectures and Water Resorces Development Public Corporation for their help to our study.

REFERENCES

1) N. Matsumoto, N. Yasuda, "Behaviours of Dams during 1987 Chiba Off Earthquake", Dam Engineering, No.25, 1988.
2) M. Ohmori, Y. Mizuyama, M. Horiguchi, "Kanto Area", Kyoritsu Press, 1986 (Geologic map was recompiled by Wakisaka based on this book).
3) Water Resources Development Public Corporation, "Investigations on Nagara Dam Related to Chiba Off Earthquake", 1988.
4) J. Kashiwagi, M. Aoyama, K. Tsukamoto, K. Kogo, "Analysis of Nagara Dam during Chiba Off Earthquake", Symposium on Behaviours of Foundation and Earth Structures during Earthquake", Japanese Geotechnical Sosiety, 1989.
5) Tone River Project Office, MOC, "Damage of River Structures", 1988

Table-1 List of dams reported unusual behaviours

Name of Dams	Type of Dam	Height (m)	Unusual Behaviours
Arakine	Earth Fill	33.5	Increase of leakage
Azuma No. 1	Earth Fill	24.5	Small failure of road slope near dam
Azuma No. 2	Earth Fill	21.0	Small failure of road slope near dam
Nagara	Earth Fill	52.0	Settlement, and cracking of asphaltic concrete pavement of dam crest
Yamakura	Earth Fill	22.5	Cracking and displacement of abutments of bridge near dam

Table-2 Investigated Dams

Name of Dams	Type of Dam	Height (m)	Year of Compretion	Owner	Epicentral Distance
Kori	Earth Fill	38.2	1972	Chiba Prefecture	53
Kameyama	Concrete Gravity	34.5	1981	Chiba Prefecture	38
Katsuura	Earth Fill	29.0	1971	Chiba Prefecture	32
Onjuku	Earth Fill	23.5	1978	Onjuku Town	23
Nakuma	Earth Fill	18.0	1976	Irrigation District	20
Azuma No.1	Earth Fill	24.5	1977	Ohara Twon	21
Azuma No.2	Earth Fill	21.0	1985	Ohara Twon	20
Arakine	Earth Fill	33.5	1978	Chiba Prefecture	21
Uryuko	Earth Fill	19.0	1951	Irrigation District	10
Nagara	Earth Fill	52.0	1985	WRDPC	28
Konakagawa	Earth Fill	19.4	1946	Irrigation District	24

Table-3 Peak acceleration at Nagara Dam

Location		G-1	G-2	G-3	G-4	G-5	G-6	G-8
Direction*	X	262	208	353	180	382	369	497
	Y	138	270	288	151	298	354	386
	Z	86	124	124	139	180	324	328
Accelerometer		PTK130	PTK130	PTK130	PTK130	PTK130	PTK130	SM10A

*Direction X: Up-down stream direction
Y: Dam axis direction
Z: Vertical direction

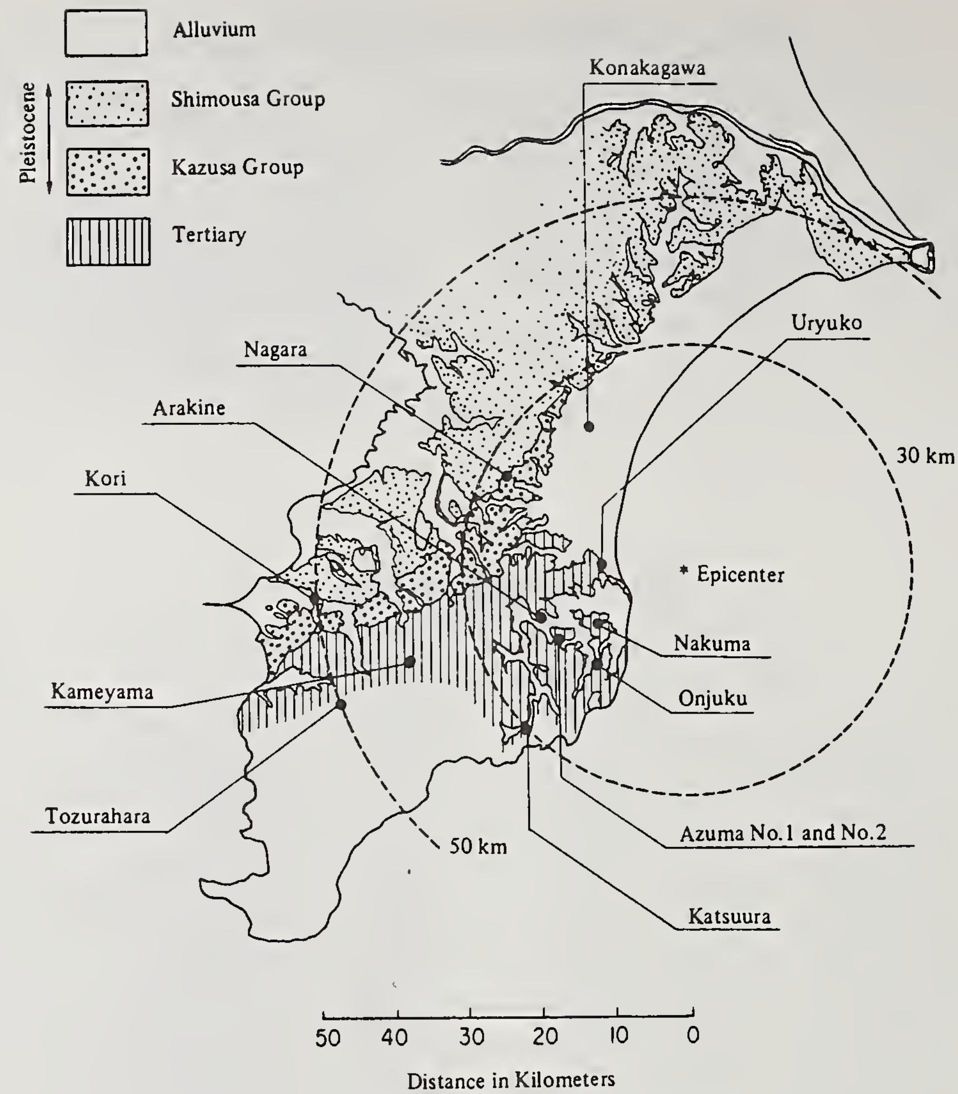


Fig. 1 Location of Dams and Local Geology

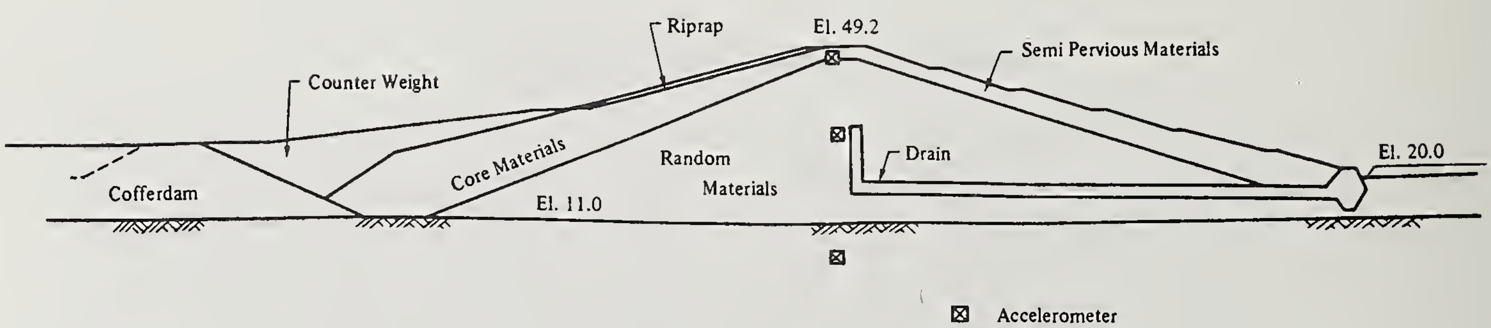


Fig. 2 Section of Kori Dam

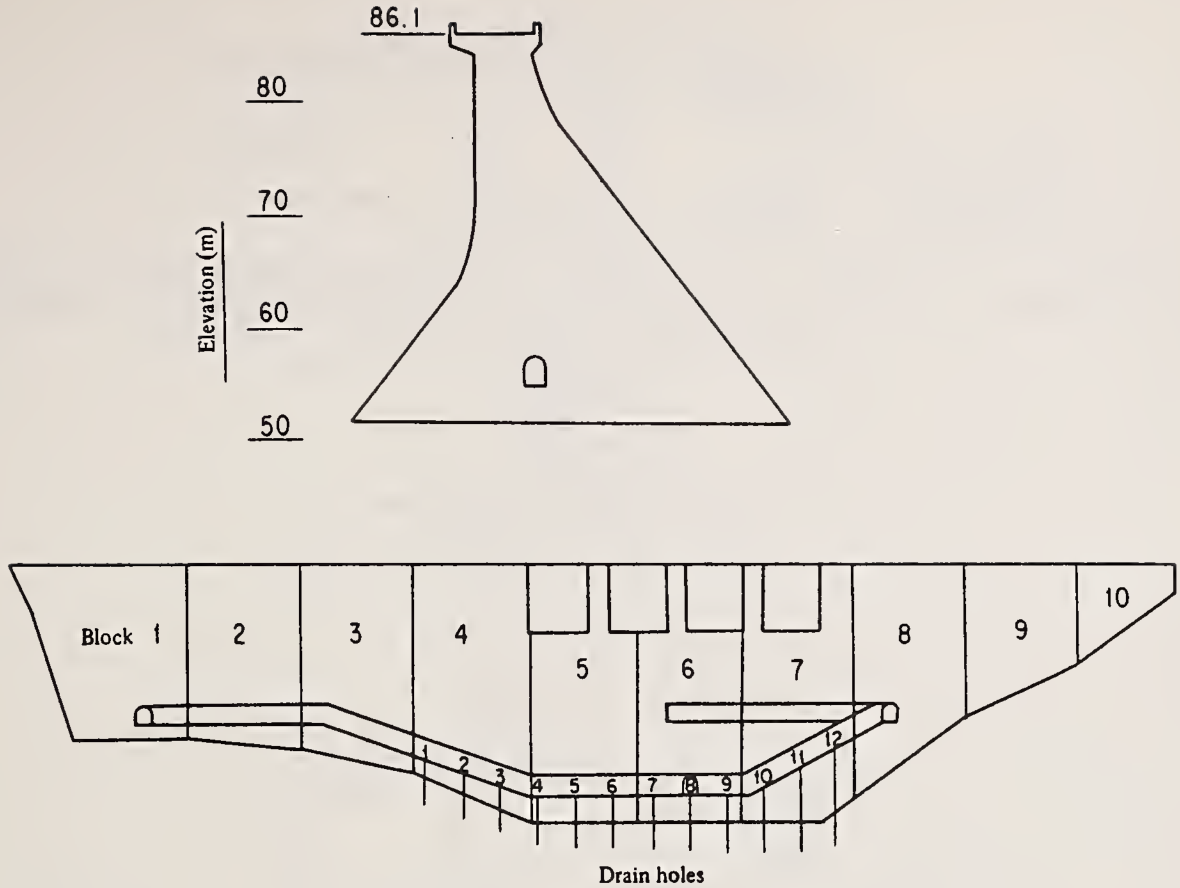


Fig. 3 Section and Elevation of Kameyama Dam

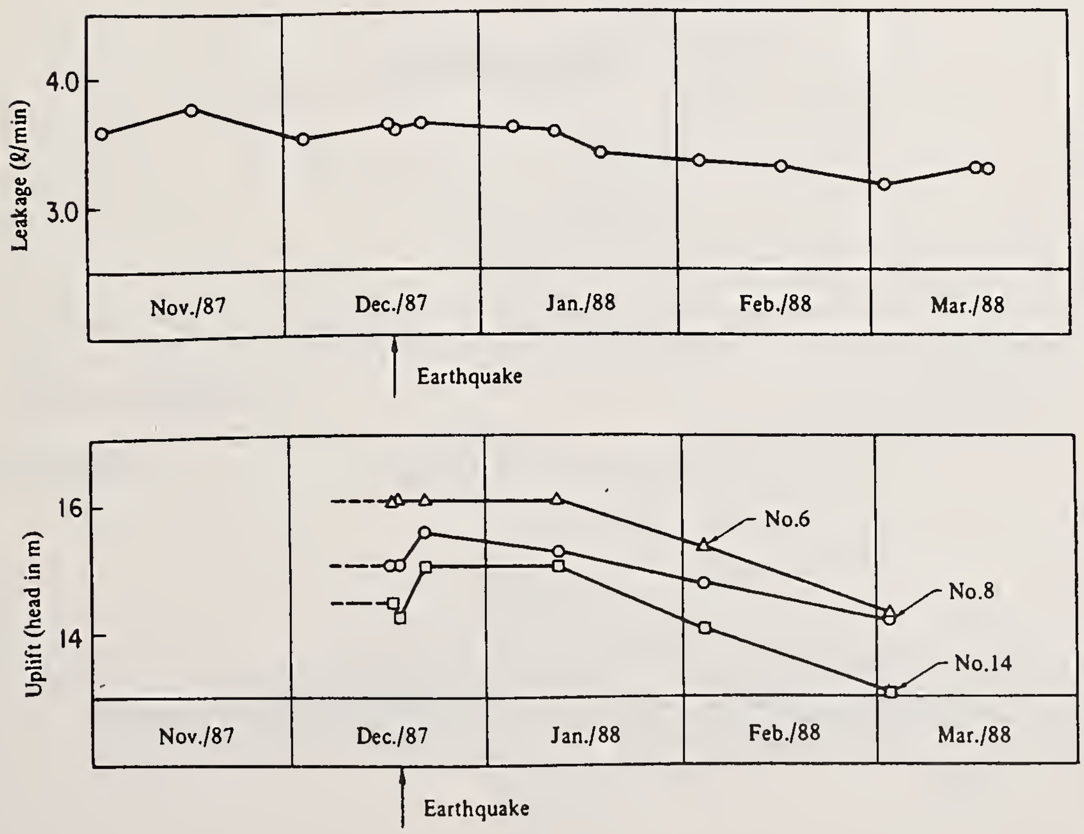


Fig. 4 Leakage and Uplift in Kameyama Dam

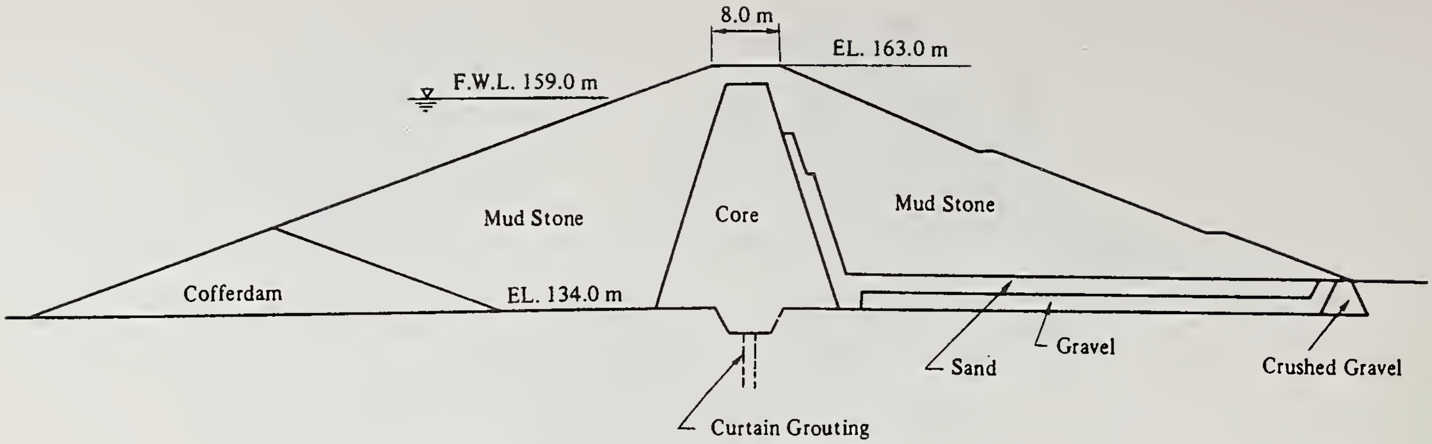


Fig. 5 Section of Katsuura Dam

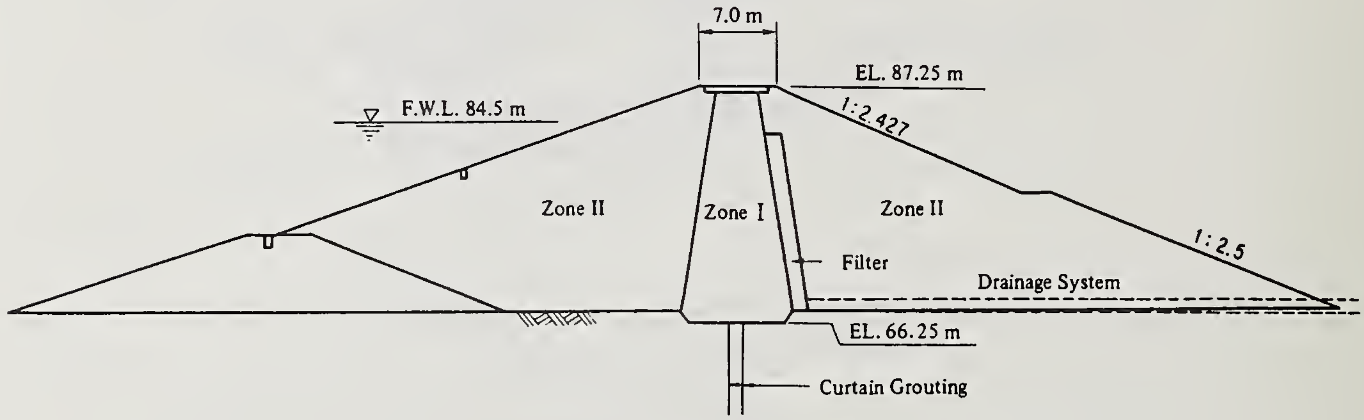


Fig. 6 Section of Onjuku Dam

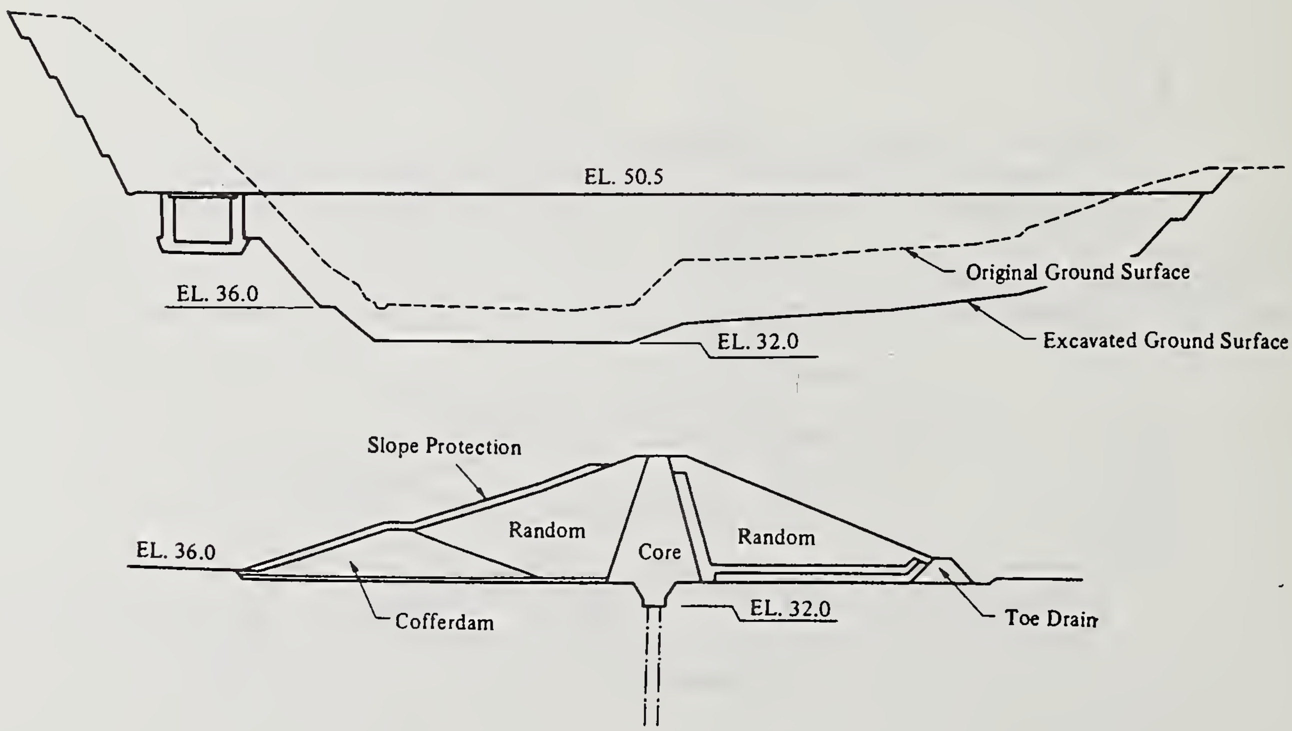


Fig. 7 Profile and Section of Nakuma Dam

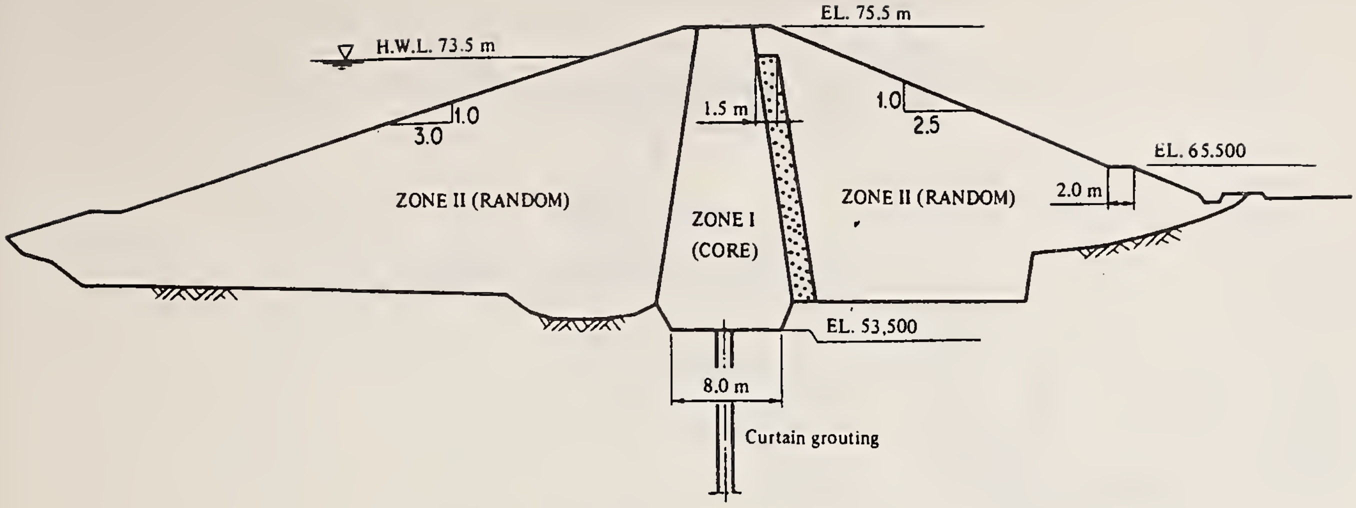


Fig. 8 Section of Azuma No.1 Dam

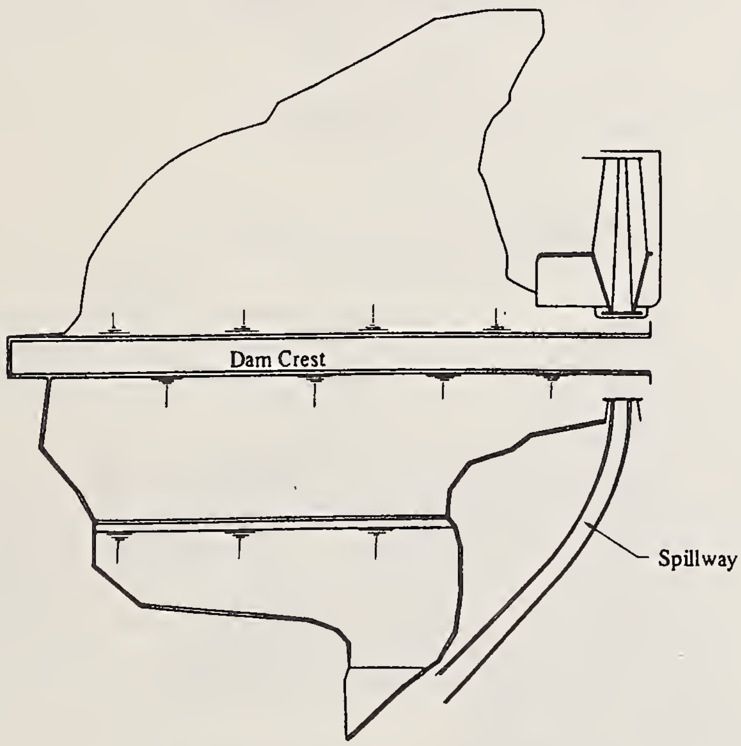


Fig. 9 Plan of Azuma No.2 Dam

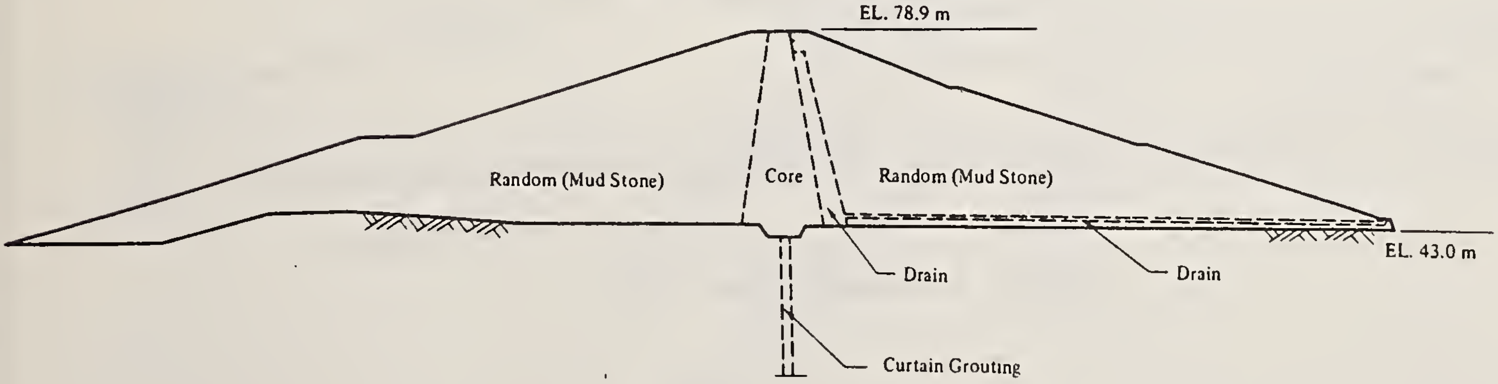


Fig. 10 Section of Arakine Dam

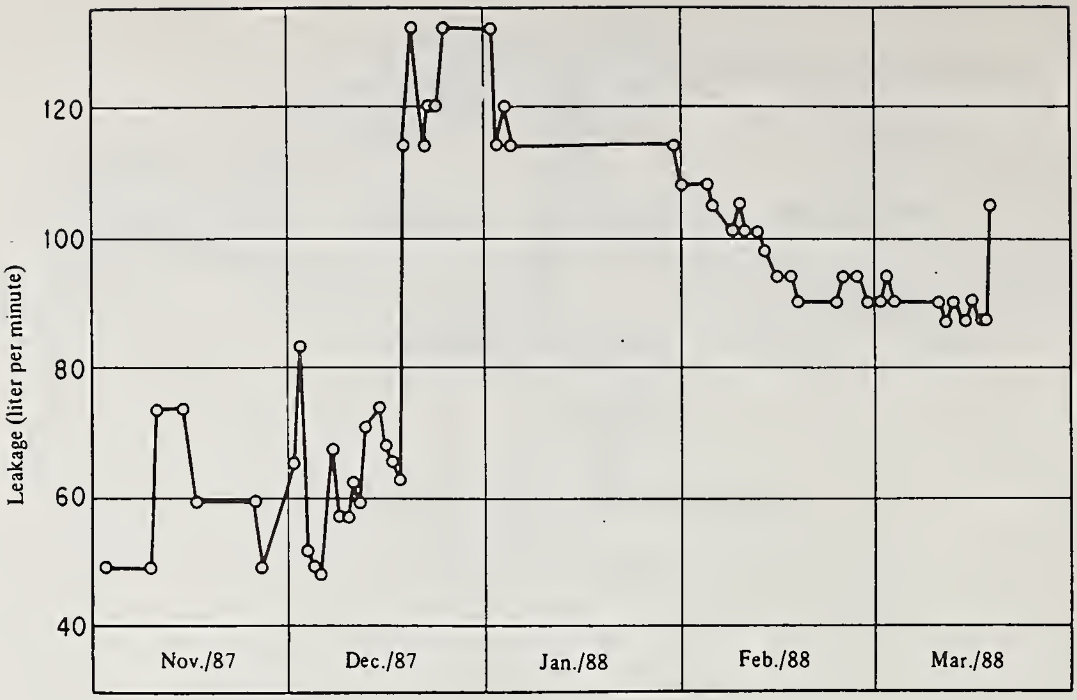


Fig. 11 Leakage in Arakine Dam

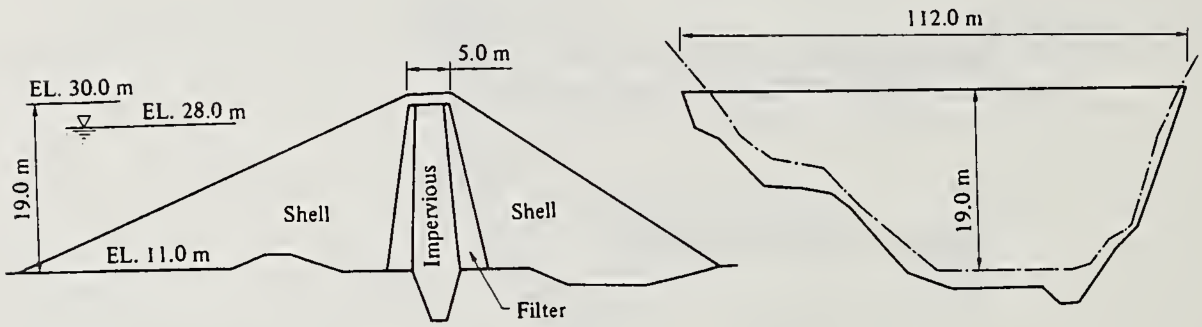


Fig. 12 Section and Profile of Uryuko Dam

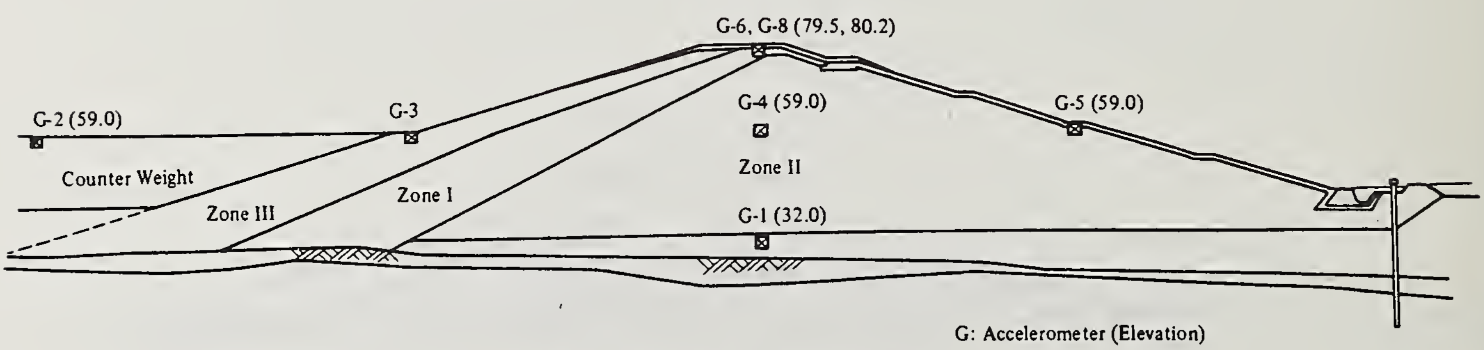
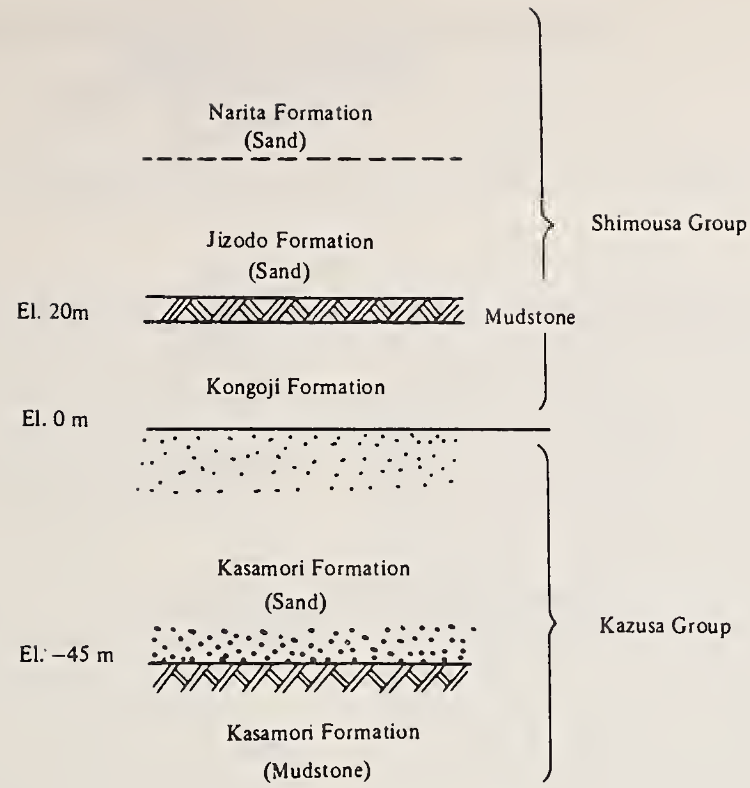
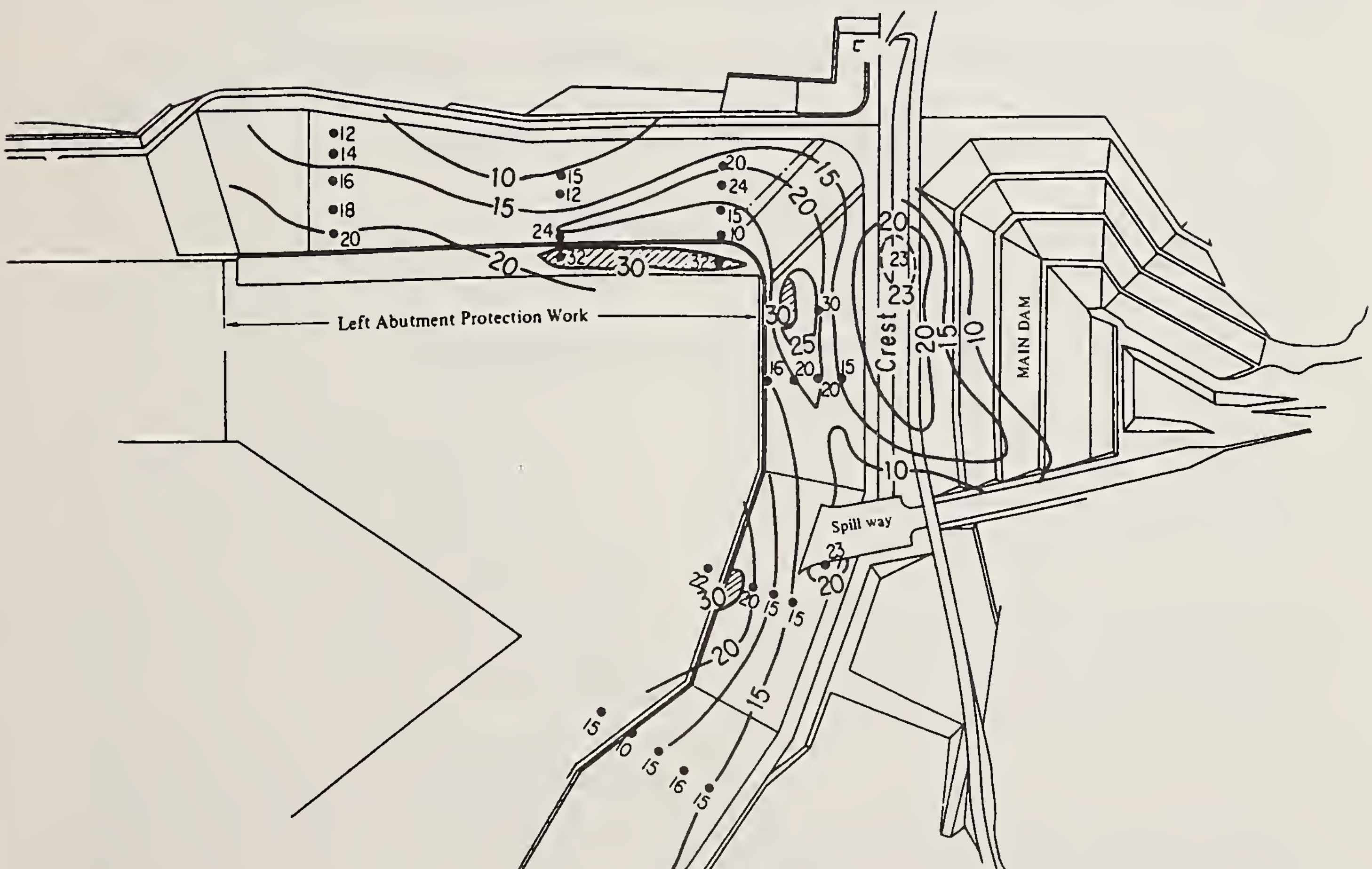


Fig. 13 Section of Nagara Dam



図−14 長柄ダム基礎の地質構成



図−15 長柄ダムにおける沈下

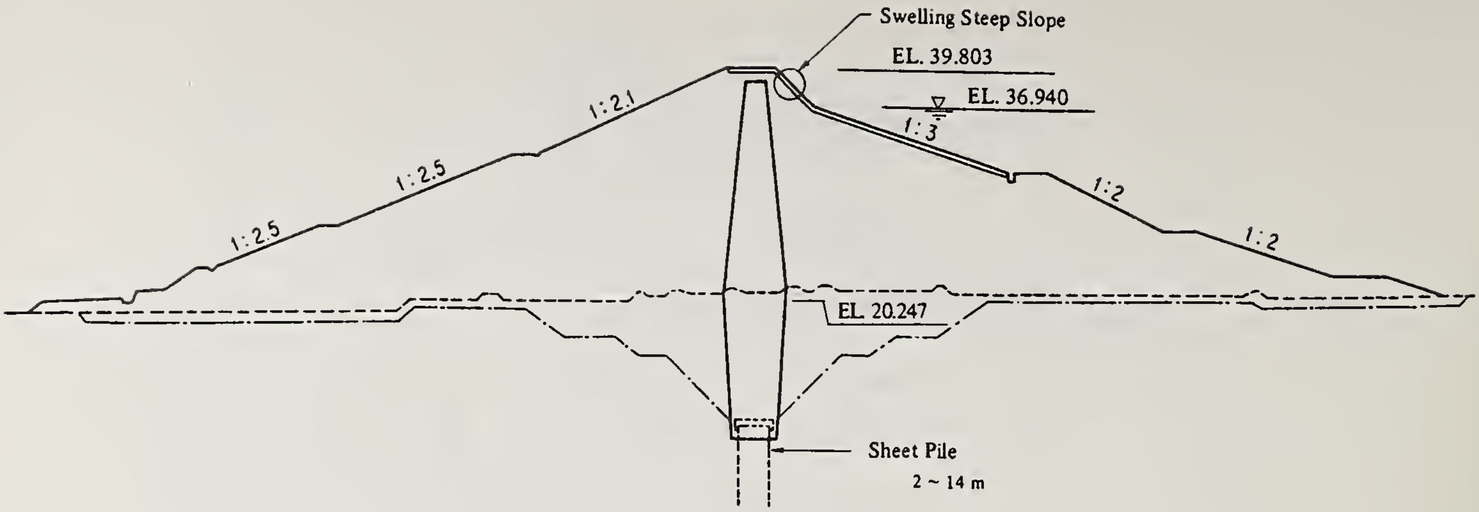


Fig. 16 Section of Konakagawa Dam

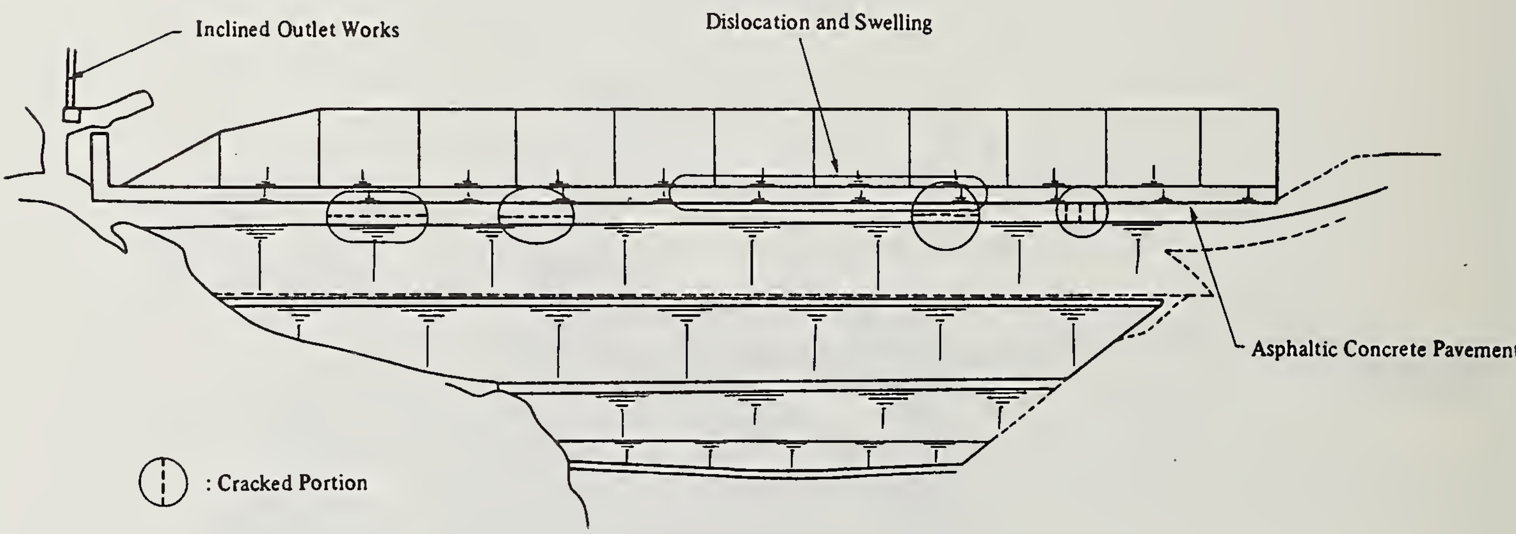


Fig. 17 Location of Cracks in Konakagawa Dam

Survey for Estimation of Seismic Damage in South Kanto Area

BY

Hideaki Oda*1 and Shigeru Tamura*2

SUMMARY

"Outline of Emergency Countermeasure Activities against Earthquake Disasters in South Kanto Area " was agreed and made in public by the Central Disaster Prevention Council on December 6, 1988, along with "the Survey for Estimation of Seismic Damage in South Kanto Area" which was conducted for the use of reference in making up the Outline.
This report aims to present the result of the Survey such as the surface acceleration, fire damages, damages to buildings and personal injury with the degree of the imminency of a great earthquake occurrence in South Kanto Area and the present earthquake disaster countermeasures in Japan for background information.

KEY WORD: Survey for Estimation of Seismic Damage, Earthquake, Earthquake Disaster Countermeasures, Outline of Emergency Countermeasure Activities against Earthquake Disasters, South Kanto Area, Fire Damage

1. BACKGROUND OF THE SURVEY FOR ESTIMATION OF SEISMIC DAMAGE

1.1 Earthquake Occurrence in Japan

Japan is located in the circum-Pacific seismic zone where the crustal movement is the most active and said to be one of the most famous seismic countries in the world.
While the Japan archipelago and the surrounding continental shelves amount to only 0.1% of the total area of the world, the energy of the earthquakes emitted from there holds about as much as 10% of what the earth generates in total In the vicinity of the Japan Archipelago the four plates of the Pacific, the Philippine Sea, the Eurasian, and the North American exist bordering one another, and on these interplates a great earthquake is likely to break out as a result of the particular concentration of their strain energy.
There are many types of earthquakes such as the Great Kanto Earthquake and the Tokai Earthquake along the Pacific coast which are though to break out periodically.
Along the Pacific coast, some types of great earthquakes has occurred periodically such as the type of the Great Kanto earthquake and that of the Tokai Earthquake.
The former broke out in 1605, 1707 and 1854, and the latter in 1703 and 1923 (Fig. 1).
In the South Kanto Area (consisted of Tokyo metropolitan, Kanagawa, Chiba and Saitama prefecture ;hereinafter the same) three types of great earthquakes which would cause serious damages are expected to break out, the type of the Great Kanto Earthquake, referred to above, that of the Earthquake occurring right beneath the South Kanto Area and that of the Earthquake occurring off the Boso Peninsula with the mutual contact among the Continental Plate, Philippine Sea Plate and the Pacific Plate (Fig. 2).
Since this region is the political and economic center of Japan and population as well as various functions have been greatly and densely accumlated, the resultant damages are likely to be tremendous and in a wide range taking secondary disasters such as fires into account.

1.2 The Present Earthquake Disaster Countermeasures in Japan

Earthquake disaster countermeasures in Japan have been promoted on the basis of the Disaster Countermeasures Basic Act (1962) and the Basic Plan for Disaster, a guiding principle adopted by the Central Disaster Prevention Council.
In addition, lessons learned from the San Fernando of Earthquake of February 1971, as to required countermeasures against earthquake disaster on a larger city of modern building structures, the Central Disaster Prevention Council has adopted the "Essentials of Earthquake Countermeasures for Larger Cities" in May 1971.
The same council has also decided in August 1975, and in May 1983, on "Contemporary Promotion of Urgent Disaster Prevention Countermeasures".
The present disaster countermeasures are being promoted in accordance with those laws and plans With respect to the emergency countermeasures against earthquake disaster, upon the breaking

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out of a disaster, or when such is anticipated the national administrative organs, local governments and public cooperations will proceed to take various emergency countermeasures to prevent or control such a disaster.
Should an eartquake occur, the muncipality will at first establish a Municipal Headquarters for Disaster Countermeasures to execute emergency operations.
If necessary, the prefectural government would establish its Headquarters for Disaster Countermeasures.
In the meantime, National Land Agency will make the assessment of its gravity, and if deemed necessary will call up the Meeting of Ministries and Agencies related to disaster countermeasures for exchanging pertinent information.
Should the need be recognized, the national government would establish the Headquarters for Major/Extraordinary Disaster Countermeasures, in accordance with the Disaster Countermeasures Basic Act, and proceed to excute a comprehensive disaster emergency countermeasures.
Concerning the promotion of earthquake prediction, since the proposal, in 1964, of an earthquake prediction plan by the Geodesy Council (an advisory organ for the Minister of Education), a systematic research on the earthquake prediction has started.
In 1969 the Coordination Committee for Earthquake Prediction was established in the Geographical Survey Institute, the Ministry of Construction, for facilitating exchange of information among various research organizations and for the composite assessment and judgement on such information.
This committee has designated 2 areas of intensified observation, and 8 areas of specified observation where the earthquake prediction is being explored.
A special emphasis is being placed on the improvement or the enchancement of earthquake prediction at the Tokai Area.
In 1976 the Headquarters for Earthquake Prediction Promotion(of which secretarist is established in the Science & Technology Agency) was established within the Cabinet for further promoting a comprehensive and systematic execution of related policies.
These systems for promoting earthquake prediction are as shown in Fig. 5.

1.3 Outline of Emergency Countermeasure Activities against Earthquake Disasters in South Kanto Area

For the earthquake countermeasures in the South Kanto Area, improvement and development of the system have been made according to the Disaster Countermeasures Basic Act, Basic Plan for Disaster Prevention prepared by the Central Disaster Prevention Council, operational plans for disaster prevention prepared by the designated administrative organs, local plans for disaster prevention prepared by the prefectures and Essentials of Earthquake Countermeasures for Larger Cities determined by the Central Disaster Prevention Council.
However, in the event of occurrence of an earthquake causing a disaster of extreme severity extensively in the South Kanto Area, close cooperation is required among the ministries and agencies responsible for the individual activity of emergency measures in order to perform them more efficiently and smoothly.
The Outline of Emergency Countermeasure Activities against Earthquake Disasters in South Kanto Area thus was formulated according to the purport of the "Contemporary Promotion of Urgent Disaster Prevention Countermeasures" decided by the Central Disaster Prevention Council to provide the outline for four activities of information, transport, medical service and first-aid which are specified as to be examined urgently among other activities for extensive and comprehensive disaster emergency countermeasures to be followed, by the Headquarters for Extraordinary Disaster Countermeasures and other related organs over the prefectural boundaries.
This outline of activities is based on the survey for estimation of seismic damage.
Besides, the contents of outline of activities is as shown in Table 1, and flow chart of government activities in Fig. 6.

2. SURVEY FOR ESTIMATION OF SEISMIC DAMAGE IN SOUTH KANTO AREA

2.1 The Beginning

The Survey for Estimation of Seismic Damage in South Kanto Area was conducted in order to know, should there occur an earthquake of an intensity similar to that of the Great Kanto Earthquake which caused the largest damage in the past, although the possibility of occurrence is not imminent, what phenomena would likely to occur and what extent of damage be caused in South Kanto Area which it is considered would be greatly affected in such event and thus to serve for furtherance of earthquake disaster countermeasures including preparation of the outline of emergency countermeasure activities and improvement of the accuracy of plans for earthquake disaster prevention.
Such estimation of damage is of course very difficult as there are many aspects involved which are not yet academically clarified such as behaviors of ground in an earthquake, effects on structures and mode of extension of fire.

But, upon the presently available knowledge, the damage estimation was made by a unified method over the South Kanto Area.

This damage estimation is in no way to predict what seismic damage is to occur and, as a matter of fact, its result has but to have some allowance according to the difference in the preconditions as well as the estimation method.. But, it is expected that it will serve for raising disaster prevention awareness of the inhabitants concerned, and examination and promotion of the earthquake disaster countermeasures by the administrarive organizations concerned.

Damage estimation was made with 7 panels established through cooperation of the government, ministries and agencies and other related organizations and discussion at the "South Kanto Area Seismic Damage Estimation Survey Committee" in National Land Agency, and the survey period was fiscal 1981 to fiscal 1987.

2.2 The Premise

The hypocenter of the assumed earthquake was Suruga Bay (Dislocation model by Kanamori and Ando, 1973), and scale was magnitude, 7.9, and conditions at the time of occurrence were 3 cases in Table 2.

2.3 The Outline of the Result

2.3.1 Surface acceleration

Based on the conditions of damage in the Great Kanto Earthquake, the values of bedrock acceleration were set up by the distance from the hypocenter, and the value of surface acceleration was caluculated for each mesh (1km by 1km) by multiplying the amplification factors of 18 ground types.

The values of surface acceleration thus assumed are as shown in Fig. 7, the area giving a surface acceleration equivalent to a seismic intensity of "6" or higher accounts for about 36 percent of the whole area.

2.3.2 Liquefaction

Obtaining the PL value of each of the meshes of 68 ground types having sandy soil distributed within a depth of 20m, possibility of liquefaction was estimated for each mesh according to the result of the study by Iwasaki et al.

The result of estimation of the possibility of liquefaction is as shown in Fig. 8, with the land along the rivers, reclaimed land along the coast and dune area giving higher possibility of liquefaction.

2.3.3 Damage to buildings

(a) Wooden buildings

Taking the cases of earthquake disaster in the past for reference and classifying the buildings into 4 types according to the time of construction and the number of stories, damage due to the shock was esimated from the deformation obtained for each mesh by input of the surface acceleration and that due to liquefaction was estimated with reference to the damage in the Niigata Earthquake.

Fig. 9 shows the result of estimation of the damage to wooden buildings.

"Heavy damage" accounts for 4.7 percent, and "half damage" for 5.1 percent, but those buildings of "heavy damage" which go to collaspe will not be much.

(b) Non-wooden buildings

Damage due to earthquake shock was estimated by comparing the prevailing earthquake-resisting performance of the existing buildings with the required earthquake-resisting performance obtainable from the seismic movement and building property.

Also, in reference to the damage in the Niigata Earthquake, damage due to liquefaction was estimated.

The result of estimation of the damage to non-wooden buildings is shown in Fig. 10.

"Heavy damage" accounts for 3.5 percent, and "half damage" for 3.7 percent, but those buildings of "heavy damage" which go to collaspe are very few.

2.3.4 Fire damage

With the cases of earthquake disaster in the past taken for reference and corrections made for the time and season, the number of fire cases was set up for each mesh, and in consideration of the fire fighting capacity, the number of spreading fires leading to fire of urban area was estimated.

Then, the spread areas were assumed, with the spreadability (incombustible zone rate) and stop of fire from spreading by road, etc. taken into consideration for each mesh.

The result of estimation of the fire damage is as shown in Fig. 11 through 13.

The burnt area is 2-6 percent, the number of the households in the burnt area is 1,220 to 3,770 thousands, and that of the buildings destroyed is 980 to 2,600 thousands.

2.3.5 Lifeline facilities damage

Based on the cases of earthquake disaster in the past and in consideration of earthquake ground motion, liquefaction and fire spreading, damage was estimated.

The result of damage estimation is as shown in Table 3, as a whole, electricity has the function obstructed for 21-43 percent, water supply for 32 percent, sewerage for 15 percent and damage to the subscribers' telephone accounts for 14-37 percent.

2.3.6 Personal injury

Based upon the data of the earthquake occurring after the Great Kanto Earthquake with involving death of more than 300 persons, the relation between the number of buildings destroyed and burnt down and the personal injury (killed and wound) was set up for estimation of the personal injury (Table 4).

As a whole, 83 to 152 thousand persons are estimated to be killed, and 120 to 205 thousand persons to be injured.

3. CONCLUSIONS

According to the result of this survey, if a large scale earthquake such as the Great Kanto Earthquake should occur in the South Kanto Area, the resultant damages are likely to be tremendous.

Since the prediction of a earthquake in the South kanto Area is quite difficult for the time being, it is especially helpful and useful to promote and sophisticate emergency countermeasures activities systems after the occurrence of disasters in order to minimize damages, and so the manual about emergency countermeasures activities was formulated this time.

Of course, it is very important not only to take emergency countermeasures after the occurrence of disaster but also to take every possible earthquake disaster countermeasure beforehand. Then it is and will be required to implement such countermeasures effectively with emphasis place on three points of (1) promotion of disaster prevention programs in urban areas, (2) strengthening disaster prevention systems and raising disaster prevention awareness and (3) promotion of earthquake prediction .

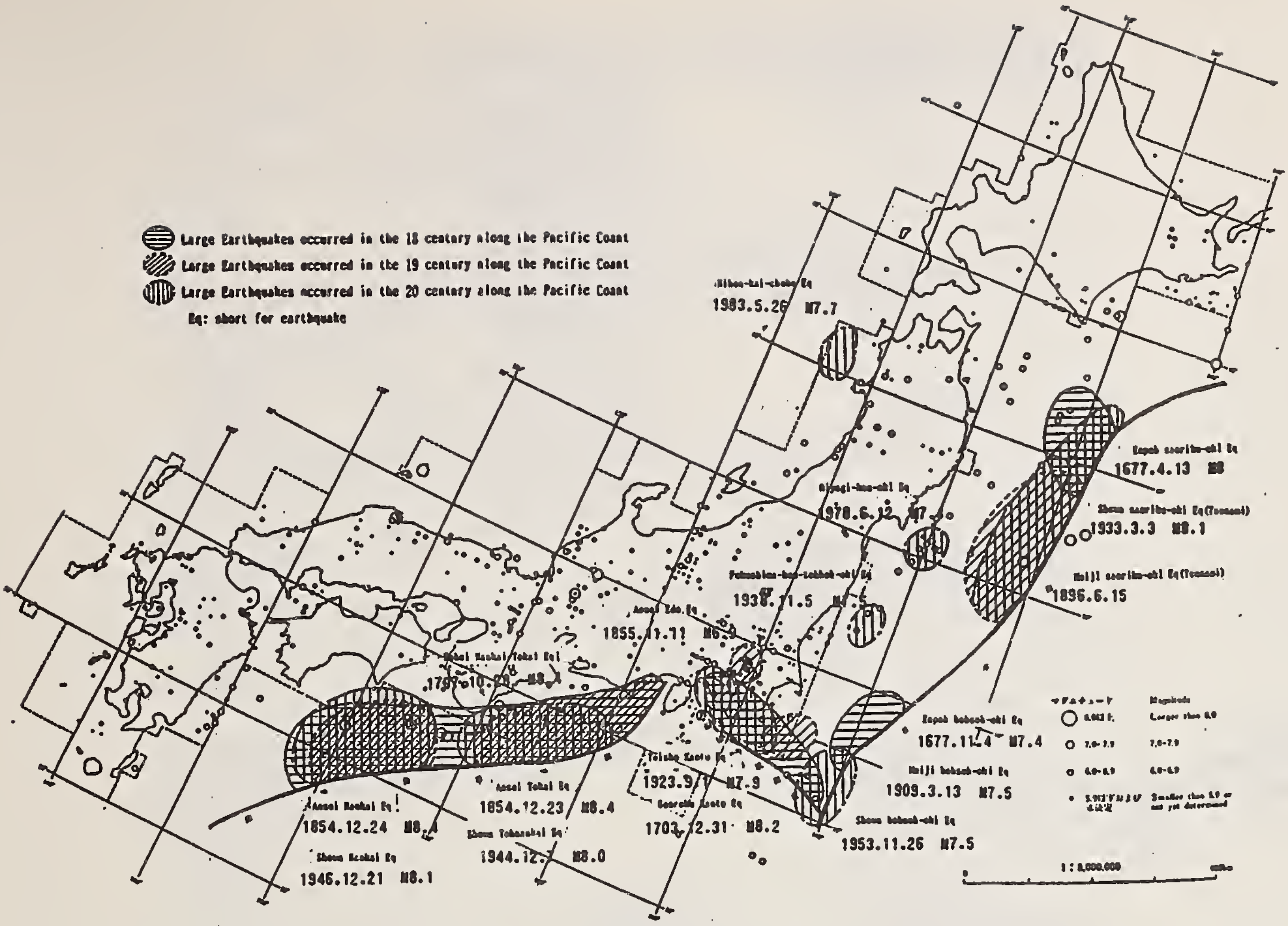

Fig. 1 Hypocenters of Large Scale Earthquakes Distribution Map in Recent Years

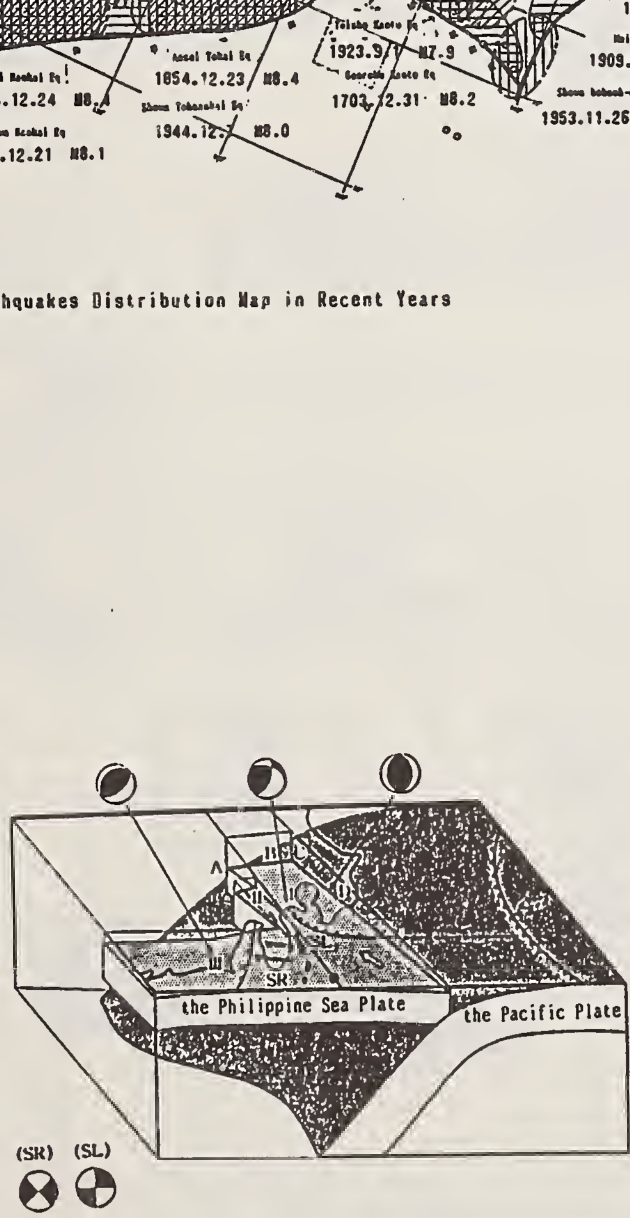

Fig. 2 Plate Model in the Kanto ·Tokai Area

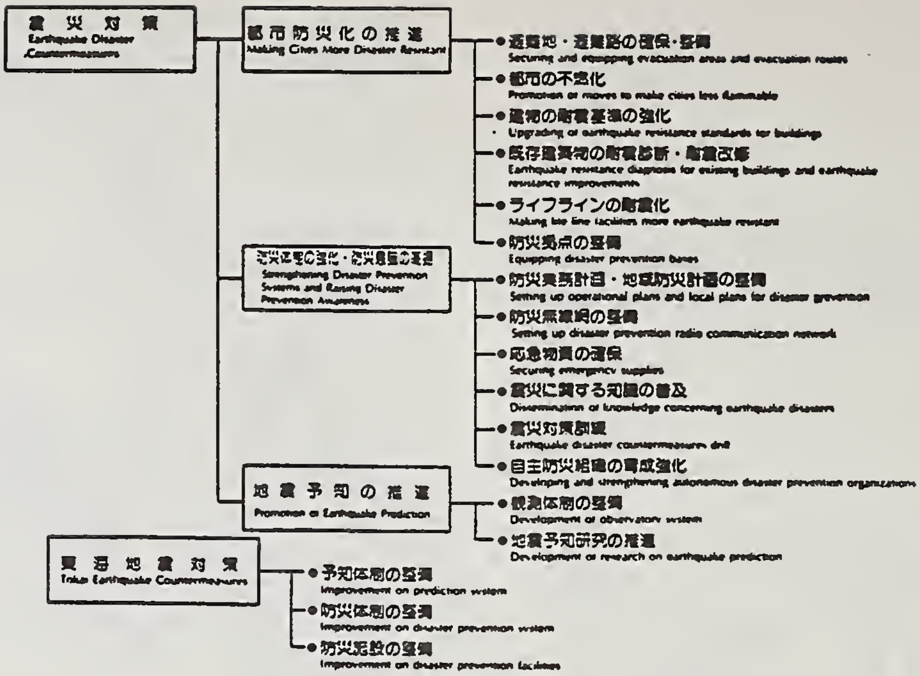

Fig. 3
震災対策の枠組
Framework of Earthquake Disaster Countermeasures

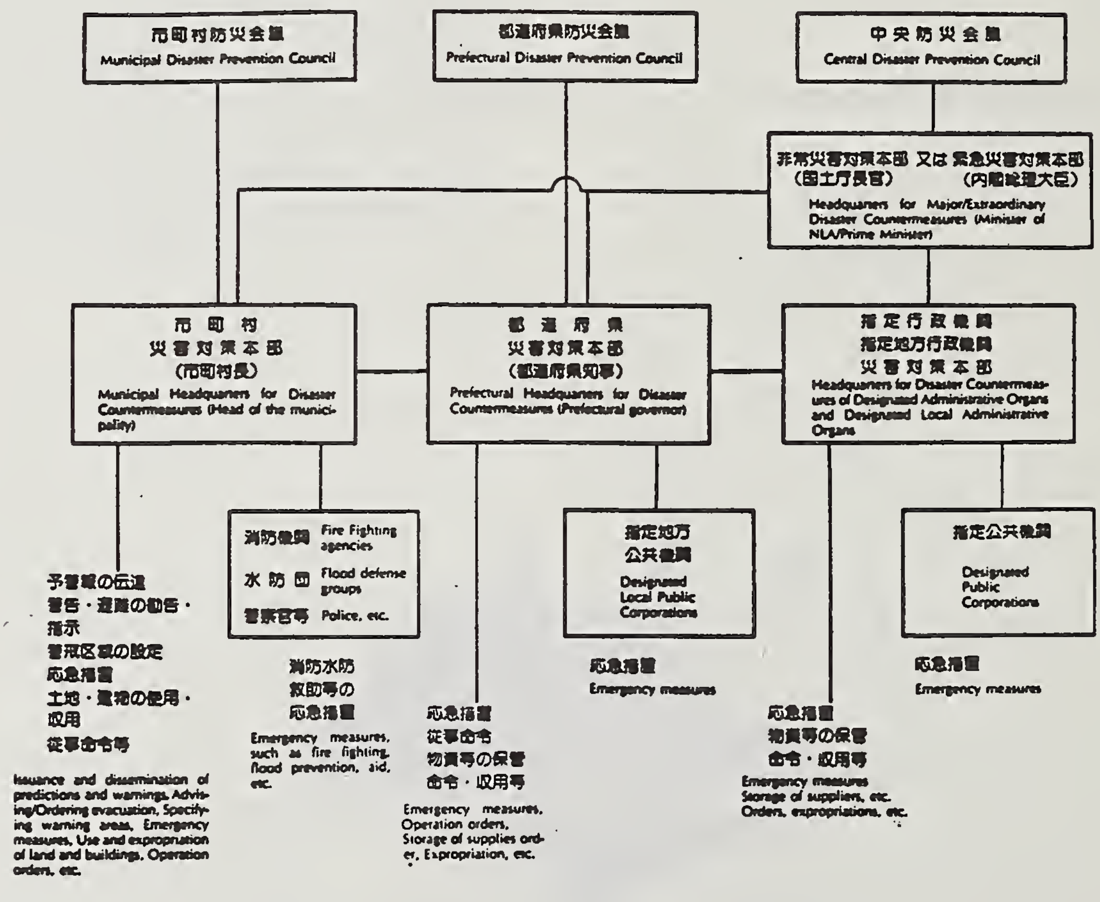

Fig. 4
非常災害発生時の主な応急処置
Main Emergency Measures to be Taken upon a Major Disaster

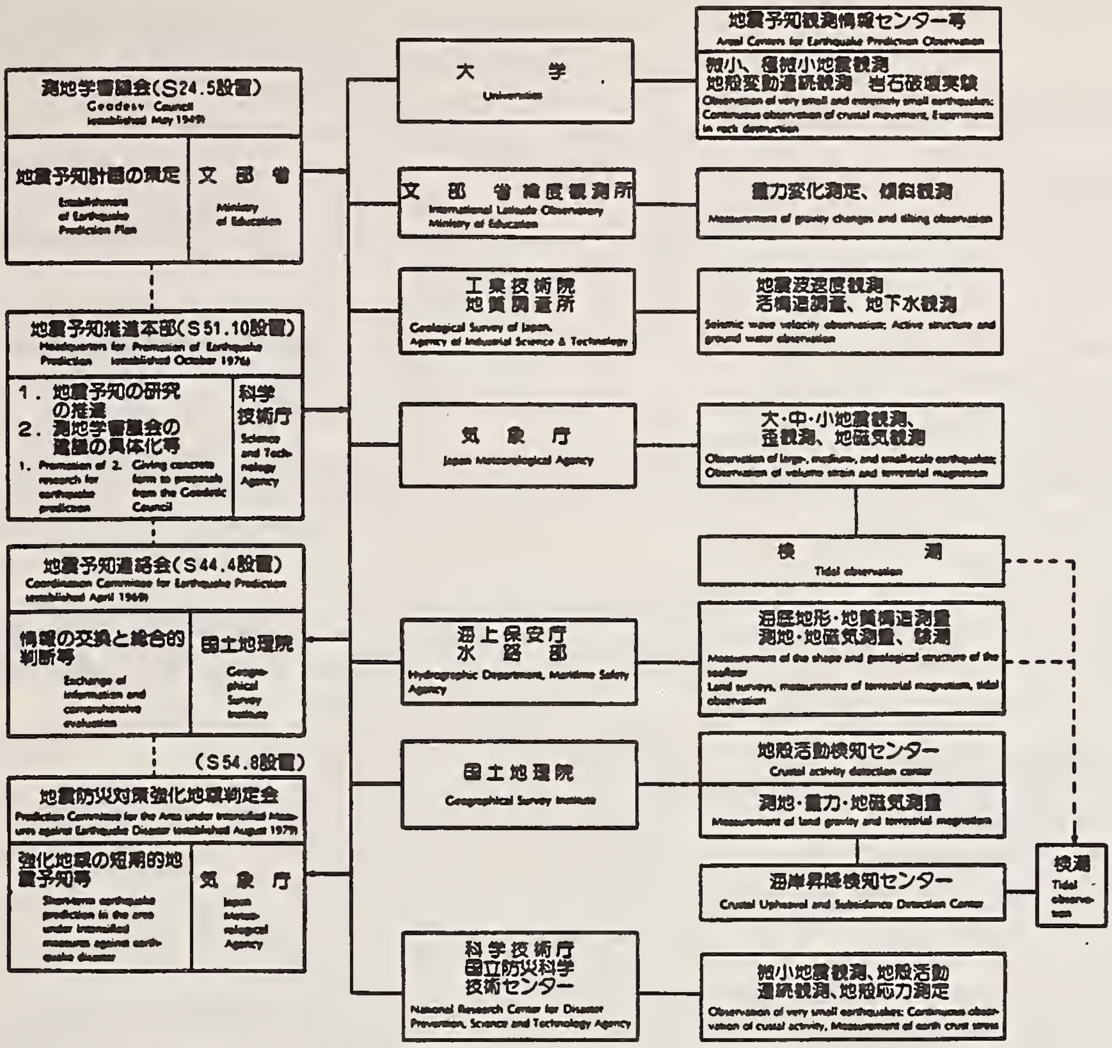

Fig. 5

地震予知の推進体制
Systems for Promoting Earthquake Prediction

Table 1

Outline of Emergency Countermeasure Activities against Earthquake Disasters in South Kanto District

CONTENTS

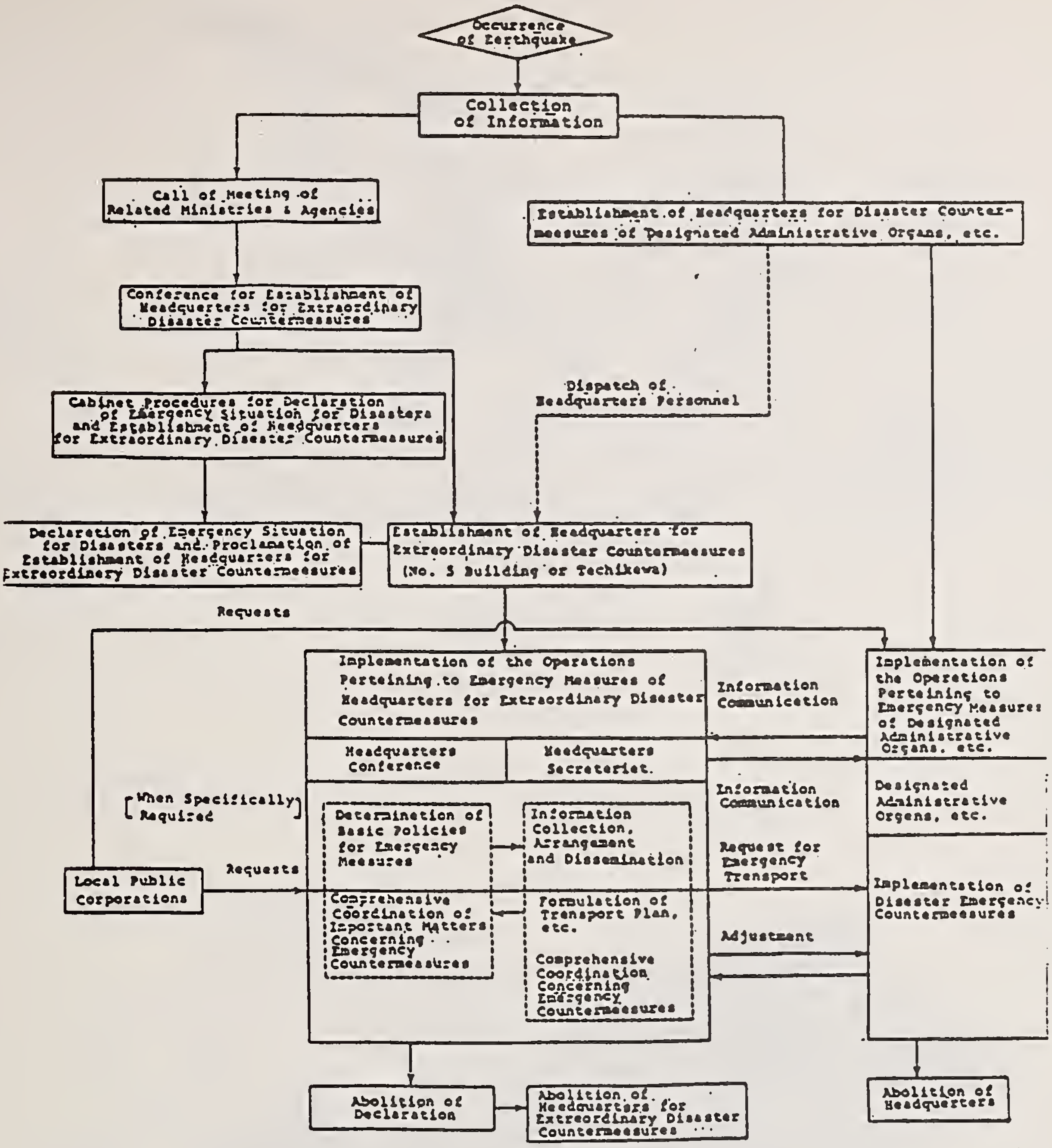

Fig. 6 Flow Chart of Government Activities

Table 2 Conditions at time of occurrence of earthquake

	Case 1	Case 2	Case 3
1. Season	Winter	Same as left	Autumn
2. Day of week	Weekday	Same as left	Saturday
3. Time	At about 5 p.m.	At about 2 a.m.	At about noon
4. Wind velocity	4m/sec	Same as left	10m/sec
5. Wind direction	Saitama and Chiba: NW Tokyo: NNW Kanagawa: N	Same as left	S
6. Weather	Fine	Same as left	Same as left
7. Humidity	50%	Same as left	80%
8. Tide level	AP 1.4m	Same as left	AP 1.0m

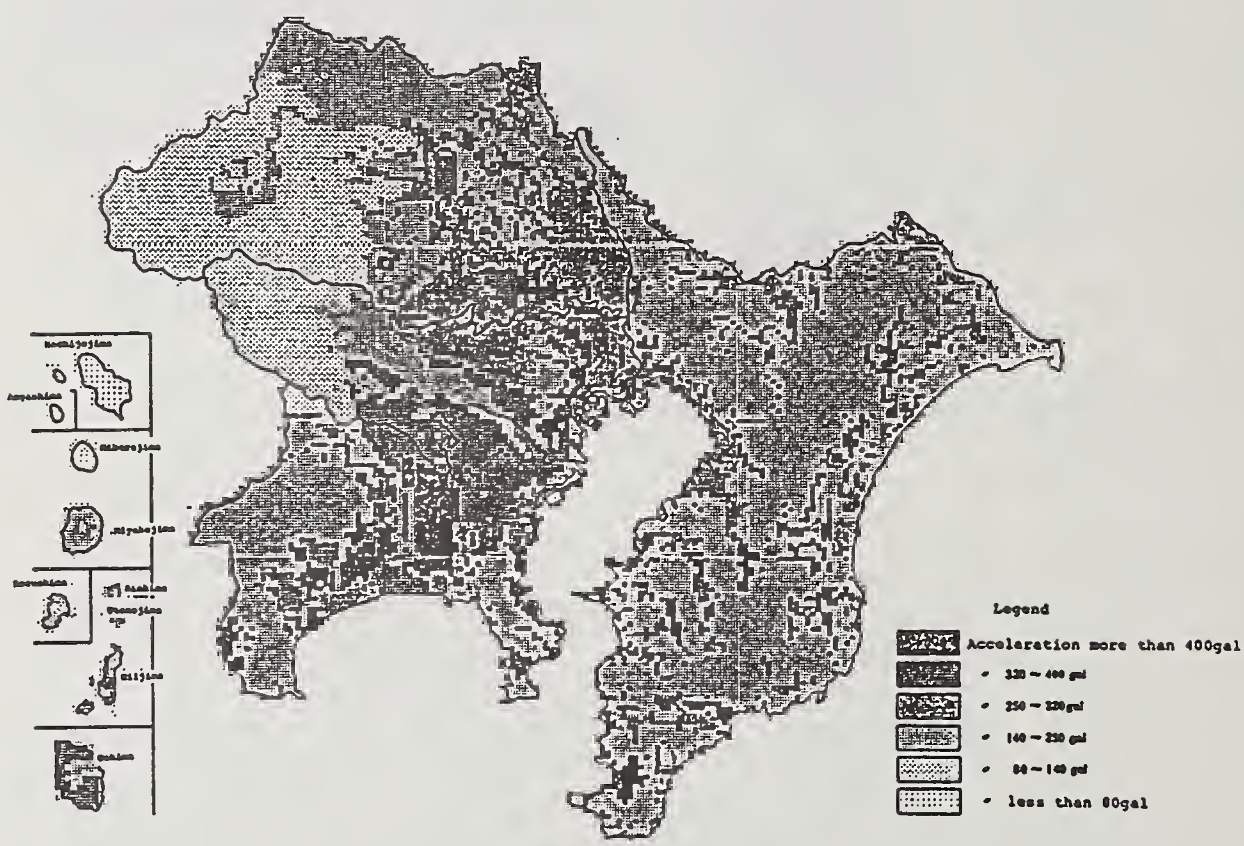

Fig. 7 Surface acceleration distribution map

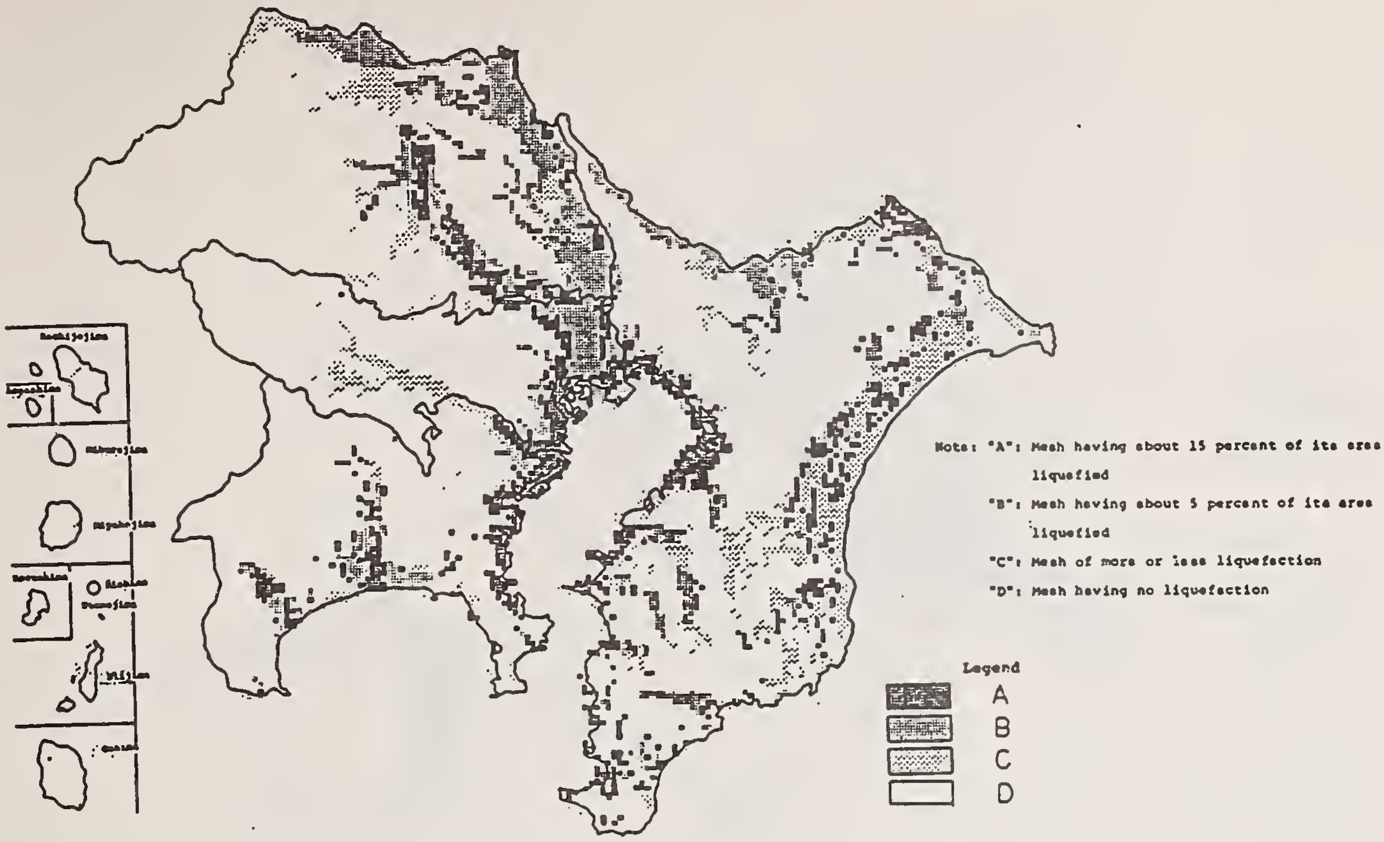

Fig. 8 Liquefaction possibility distribution map

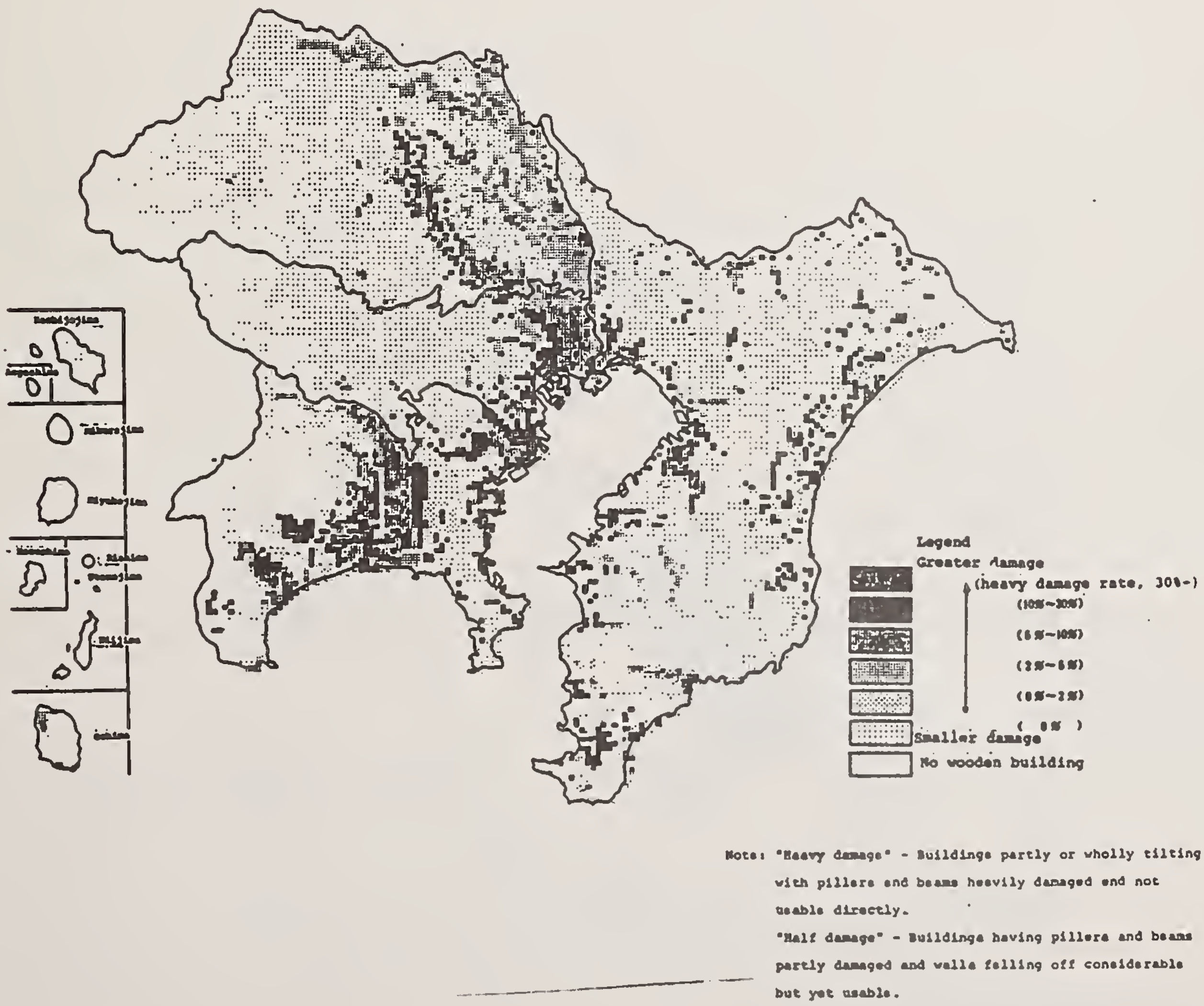

Fig. 9 Wooden building damage distribution map

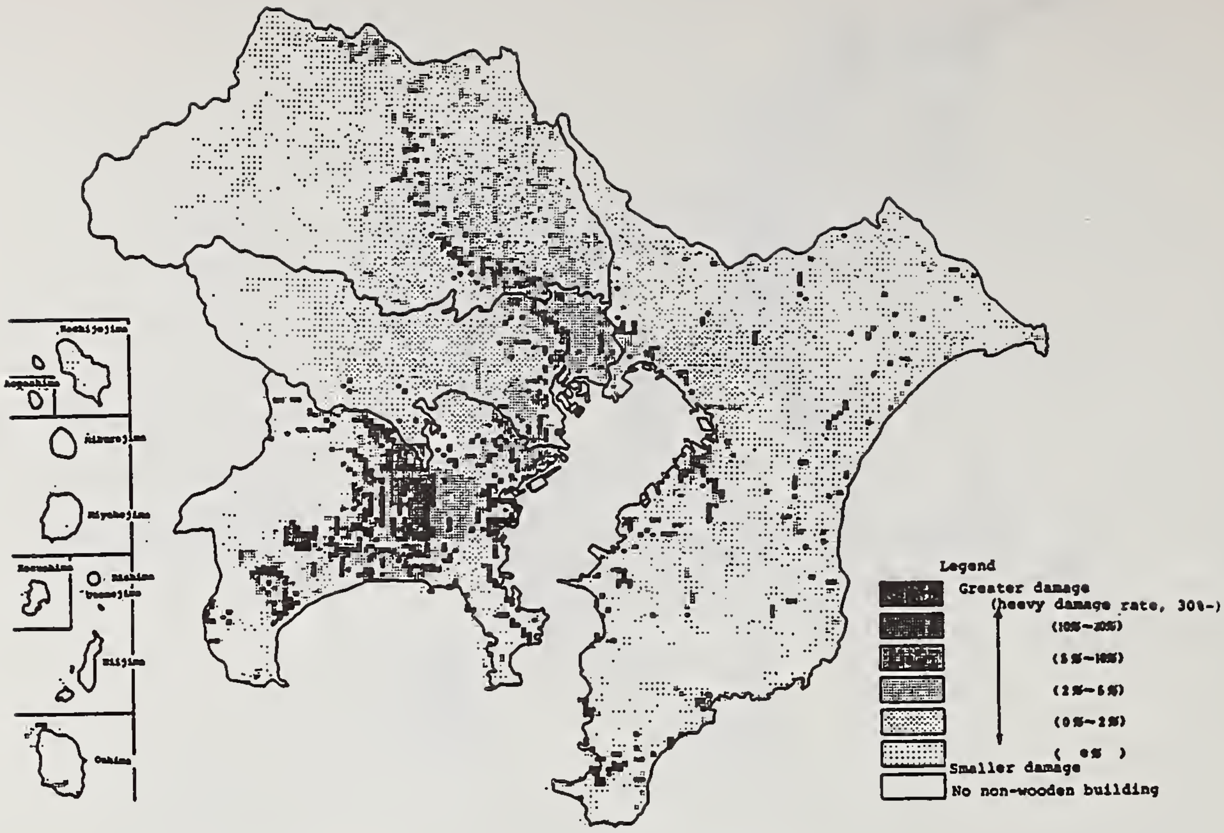

Fig. 10 Non-wooden building damage distribution map

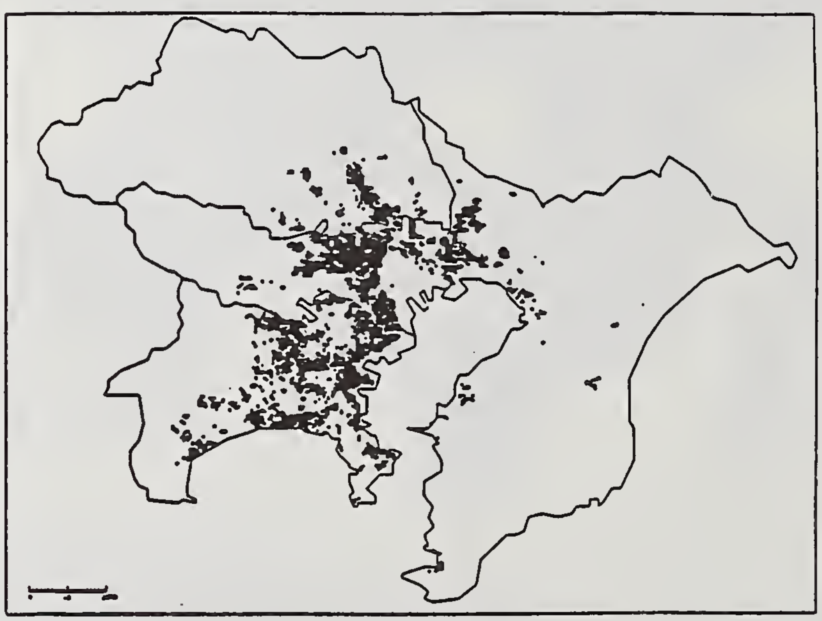

Fig. 11 Burnt area map (Case 1)

Fig. 12 Burnt area map (Case 2)

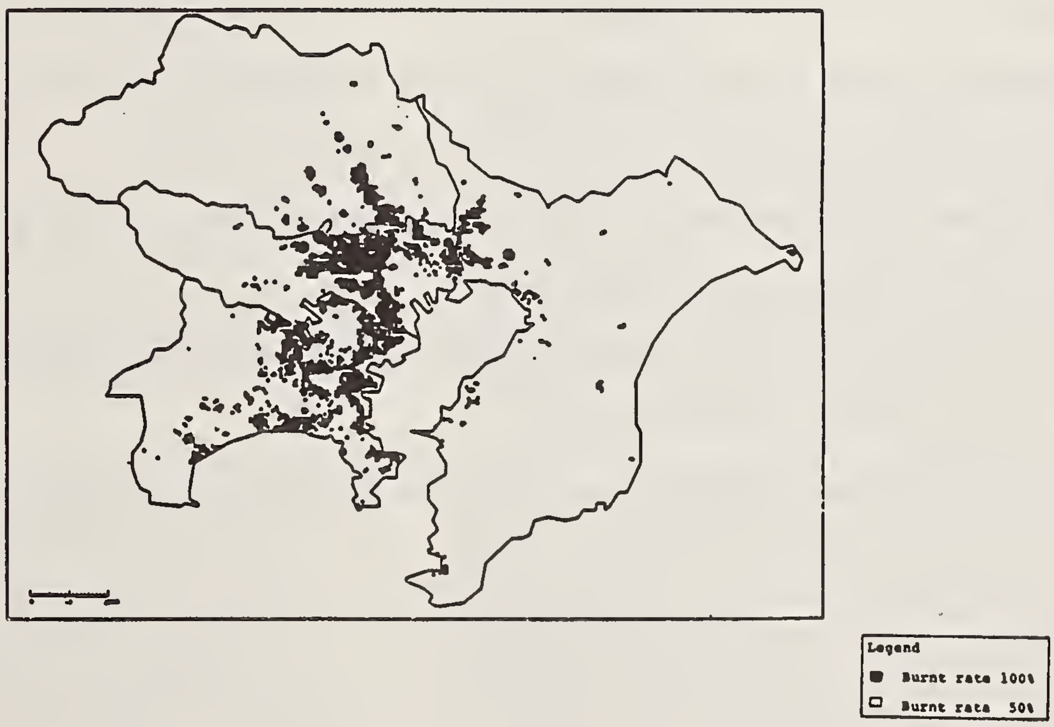

Fig. 13 Burnt area map (Case 3)

Table 3 Damage to lifeline facilities

Item	Case 1 (evening in winter)	Case 2 (midnight in winter)	Case 3 (noon in autumn)
Electricity	43%	21%	42%
Water supply	32%		
Sewerage	15%		
Telephone	37%	14%	37%

Table 4 Personal injury

	killed(1,000 persons)			injured(1,000 persons)		
	Case 1	Case 2	Case 3	Case 1	Case 2	Case 3
South Kanto Area	150	83	152	203	120	205

References

(1) the Research Group for Faults (1980) :Active Faults Sheets in Japan, the Publisher of Tokyo University, P13

(2) Kazuo Hamada (1986) :Earthquake Prevention, the Publisher of Morikita, P47

(3) Earthquake Disaster Countermeasures Division, Disaster Prevention Bureau, National Land Agency (1988) :Earthquake Disaster Countermeasures in Japan

(4) the Central Disaster Prevention Council (1988) :Outline of Emergency Countermeasure Activities against Earthquake Disasters in South Kanto Area

(5) National Land Agency (1988) :Result of Survey for Estimation of seismic Damage in South Kanto Area

Seismic Design Considerations for Offshore Oil and Gas Structures

by

Charles E. Smith*

ABSTRACT

The ability to determine the vibrational response of offshore platforms subjected to earthquake ground motions and to design for the resulting loads has received considerable attention since the quest for oil and gas resources extended to seismically active zones worldwide. It is the intent of this paper to review the major design considerations for offshore structures that are to be located in areas of potential seismic risk. Subjects of major importance relate to area seismicity, soil response, energy transfer between soil and platform foundation, platform response, and the consequence to operations resulting from that response. It is concluded that for the successful development of oil and gas resources and other mineral deposits in offshore environments, careful consideration must be given to the potential earthquake hazard, and appropriate design criteria must be evaluated and used to ensure safe and pollution-free operations.

KEYWORDS: Earthquake hazard, offshore platforms, offshore structures, oil and gas operations, seismic design criteria.

1. INTRODUCTION

During the past decade there has been a marked increase in the exploration and production of hydrocarbons in seismically active regions of the world. In the United States much of the effort has focused on the areas offshore southern California, and to a lesser extent, offshore Alaska. Both steel jacket and gravity-based platforms have been proposed to meet the operational requirements for developing resources in these areas[1]. The components of a typical steel-jacket platform are shown in Figure 1, and a general schematic of a typical arctic gravity platform is shown in Figure 2. In addition to the traditional environmental forces of wind, wave, current, and ice (for arctic areas), a significant consideration in the design of offshore facilities for seismically active areas is their structural response to earthquake ground motions.

However, the comprehensive inclusion of this consideration in a design process is a formidable task as shown in Figure 3. Major elements of this task include the characterization of area seismicity[2,3], site-specific response of the soil[4,5], transfer of seismic energy from soil to platform[6,7,8], platform response to absorbed energy[9,10], and the consequences associated with platform response to the earthquake induced motions[11,12]. An offshore facility should have sufficient strength and ductility to satisfy its intended purpose (drilling and production) during any anticipated loading condition without undue expense or risk. Design criteria should provide an engineer with the necessary procedures and parameters to enable him to develop a platform system (superstructure, structure, and foundation) that will have acceptable performance characteristics during a seismic event.

This paper reviews the major features which should be considered during the process of developing site-specific seismic design criteria for an offshore installation. In addition, ongoing research programs are presented to illustrate possible future technological developments. Some of the requirements are rigidly stated in design standards, such as the American Petroleum Institute (API) Recommended Practice 2A(RP2A)[13], whereas others are based on the application of experience and probabilistic methods. It is noted that offshore structures have several key structural and environmental differences as compared with typical onshore buildings, for which most of the experience on earthquake response has been obtained. Major differences include the following: (1) geology and response of subsea soils (lack of offshore seismic data and remoteness of most offshore sites), (2) a water column that exerts pressure on the soil deposit supporting the facility and provides a source of loading and dampening to the structure (effects of soil-fluid-structure interactions), (3) the special characteristics of the structure (deep pile foundation, large mass, large number of redundant members, large overturning moments, and the fact that the mass is concentrated at the top as opposed to being distributed over its height as in most building structures), and (4) the unusual properties of many offshore soils. Thus, the dynamic behavior of an offshore platform during an earthquake may be significantly different from its onshore counterpart, requiring a different set of design guidelines.

2. AREA SEISMICITY

For offshore structures located in areas subjected to earthquakes, the first step required in developing appropriate design criteria is a characterization of the seismicity of the proposed site. The engineer needs a qualitative description of the seismic threat which gives both the severity and likelihood of future occurrences of ground motions. As with land-based structures the time of occurrence, location of faults, magnitude of future earthquakes, likelihood of ground failure, (such as liquefaction and sliding) and the attenuation of ground motions with distance from the source represent major uncertainties in an assessment of the offshore seismic hazard[14].

The difficulties that occur in evaluating the seismicity of an offshore site differ from those normally encountered ashore. These difficulties stem from the fact that the seismic history of offshore areas is generally less complete than for neighboring onshore regions. The ability to delineate active faults or to accurately locate offshore earthquakes is generally less exact than for onshore events because of the lack of offshore regional seismograph networks[15]. Thus, the engineer must rely heavily on the predictions and extrapolations of seismologists and geologists in formulating an earthquake source model for a particular offshore site. The procedure taken in the development of the source model may range from the extremes of a fully deterministic approach to a fully probablistic method.

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Approaches such as these have inherent risks and are possible sources of unwarranted conservatism. However, until further knowledge is obtained on the regional tectonic environment, regional and local geology, and the compilation of historical records of earthquake occurrences, these approaches are the only prudent actions that can be taken.

3. SOIL RESPONSE

Ground motion characteristics can be modified by the soil properties existing at a particular site as shown in Figure 4, and it is easy to perceive that offshore ground motions might differ from onshore motions in several aspects. In fact, the soil response considerations can be one of the most important factors in the seismic design of an offshore structure and its foundation.

The differences between onshore responses and offshore responses can be attributed to several factors[16]. The attenuation factors may be different in saturated seafloor soils, especially if they are soft and contain entrapped gas. In addition, there are other variables which could cause differences in the responses: the reflection of seismic waves at the water column-soil interface could interact with site-specific wave forms to alter their characteristics, the weight of the water column, acting as a confining pressure on soil mass, could change the vertical component of motion, or local soil profiles could change the response if the sediments are wedge-shaped as they depart the shore (the entrapment or expansion of the seismic waves may lead to focusing or defocusing effects at a given site). All or part of these factors could contribute to the different response characteristics of offshore soils.

There are two general approaches for formulating the response of a particular site to a given earthquake. The soil response could be based on data obtained from an accumulation of strong motion records or it could be derived from an analytical method based on a soil response model, the latter incorporating the soil properties as determined by field and laboratory tests.

In principle, the accumulation of ground response records from a number of earthquakes could be used to characterize the response at a particular site. These records could be scaled to provide response data for earthquakes of varying intensities. However, because few records are available on the response of seafloor sediments, the current accepted procedure is to extrapolate onshore data for use at offshore sites. Research is being performed, as will be explained in a later section of this paper, to collect and analyze seismic data taken from actual seafloor sites.

Analytical methods are available to produce response data at a particular site based on the assessment of the base (bed rock) excitation. A "shear beam" or "soil column" can be modelled by using a number of layers which take into account soil properties and actual soil layer thicknesses.

Selection of appropriate inputs for the base excitation would come from a seismicity source model as previously discussed. The results of such an analysis are shown in Figure 5. It should be noted, however, that this method of analysis can only provide meaningful results if the appropriate distribution and nonlinear stress-strain properties of the soils are used in the calculations.

When selecting ground motion records for use in an analysis or design, the particular type of information specified must be guided by engineering judgment. Only those ground motion parameters which directly influence the structural response of the structure need to be considered. Peak acceleration, for example, has little to do with the response of a structure that has a three- or five-second fundamental period. Spectural velocity or peak ground velocity, however, appears to be particularly useful in the analysis of structures with fundamental periods in the range of typical offshore platforms[17].

4. SOIL-STRUCTURE INTERACTION

With the soil response defined for a particular site, the question remains: how is the soil energy transmitted to the structure. The central focus of this energy transfer is the coupled behavior between the foundation elements of the structure and the supporting soil components. Specifically, the reference here is to the local interactive forces and displacements between the soil mass and the structural elements.

For offshore platforms, there are two distinct aspects of the soil-structural interaction process which are inherent to the structure's physical size and mass. One aspect is the relative significant deformation between the so-called free-field soil system and the structural elements of the foundation system. This is especially true of pile-supported structures where the piles are subjected to large cyclic lateral loads. The other aspect is the possibility that the dynamic response of a structure-foundation system might result in the transmission of a significant amount of energy back into the soil mass. Thus, the existence of the structure could modify the motion of the soil system. For instance, in the case of an offshore gravity structure the calculated dynamic response resulting from earthquake-induced ground motions can be drastically reduced by including soil-structure interaction effects[18].

The main objective for developing improved soil-structure interaction models is to improve the analytical capabilities of the engineer to determine the vibrational response of structures. The features of such a model are shown in Figures 6 and 7. Having this model, the engineer can evaluate the potential for structural damage or failure and modify the design to mitigate earthquake effects. The soil-structure model should be capable of reflecting the three dimensional, inelastic, nonlinear, hysteretic, damping, and cyclic degrading characteristics of the various components of the soil-structure system. Several models have been developed for this purpose, but due to the complex nature of the problem, additional research is required and is being pursued to further define the technique[19,20].

5. STRUCTURAL RESPONSE:

The problems associated with the analysis and design of offshore structures subjected to earthquake forces are numerous and complex due to their many interactive components. Consideration must be given to the fact that these structures are either partially or totally immersed in water and will, therefore, exhibit different dynamic characteristics than if they were in air. Also, due to their large mass, there is a strong interaction during ground shaking with the soil below and adjacent to their foundation elements. Figure 8 shows some of the considerations that must be given to the components in the analysis of a soil-pile-structure system. Much information has been published on fluid-structure interaction and soil foundation interaction inherent in steel-jacket and gravity-based offshore structures. Figure 9 illustrates the frequency response of four different height platforms with the fluid-structure interaction not included and Figure 10 shows the response of the same platforms considering the interaction.

Numerical procedures for carrying out dynamic analyses of a complete offshore structure foundation system are similar to those for determining the dynamic response of other types of structures. However, the performance characteristics of an offshore structure, since it incorporates a number of unique features not found in onshore building construction and because of the magnitude of the environmental load which it must resist, rely greatly on the inelastic behavior of its components. Information on the inelastic, nonlinear structural behavior is essential for a realistic assessment of strengths and risks associated with an offshore system [21,22].

The specific algorithm used to describe performance characteristics of platform elements should be carefully considered. The inelastic behavior of braces, legs, and joint subassemblages is very complex and can deteriorate under the severe cyclic load reversals typically experienced during earthquake ground shaking[10,25]. Many analytical procedures have been proposed for use in finite element computer programs which mimic the postbuckling loadings and deformations of structural members[23].

The fluid-structure interaction or hydrodynamic effect manifests itself in two primary ways during the earthquake excitation of an offshore structure. This effect acts as an apparent additional mass due to the acceleration of the structure and also serve as a source of energy dissipation. It should be noted that if the structural members are flooded, the entrapped water mass must be included when calculating the effective dynamic mass of the structure. The so-called "added mass," is taken in the analysis as being equivalent to the mass of water displaced by the member projected in the direction of its motion. Changes in the mass and its assumed distribution can produce important effects on the frequency and mode shape responses of the structure[6].

The deck weight of a steel-jacket platform may comprise only a small portion of the total system weight; however, due to its position at the top of the structure, it can have a very important effect on the vibration properties of the structure. Therefore, the engineer must consider a reasonable range of deck weights when investigating the response of a platform. This is also true for other analytic assumptions, such as foundation stiffness and effects of marine growth. Research has shown that the interaction between soil and foundation elements is a significant factor in absorbing energy and limiting loads transmitted to the jacket[28]. Therefore, because many uncertainties exist in a seismic analysis, parameter studies should be undertaken to determine their influence on the dynamic response of a particular platform to a series of ground motions. This may be the only way in which the engineer can estimate with some assurance, that the structure will perform adequately during a seismic event[6,24].

Because of the large number of interactive systems, for purpose of analysis the structure is reduced to a complex mathematical model which can best be solved on a computer. However, analytical methodologies have been and are currently being verified in the field by comparing the measured earthquake ground motion responses of actual platforms to the responses derived from such analytical models[26,27]. One should, however, not totally accept analytical results without understanding the significance of the approximation incorporated in them. Early studies on the use of the time history method of analysis illustrate this point. If a structural analytical model is subjected to several earthquakes of nominally similar intensity, there is a wide variation in the computed responses. These results suggest that not only the peak intensity, but also the duration and phasing of ground motion pulses are significant to the platform's response. For the designer, an implication such as this means that he should review his analytical results with caution. His prediction of the loads and stresses in the structure based on past earthquake responses, might not match the reality of responses to future earthquakes[28].

6. DECK RESPONSE

A seismic analysis of a typical offshore structure has two major objectives. First, it must provide information that can be used to ensure that the structure as a unit will remain intact during an event, and second, it must provide information that can be used to ensure that the equipment and facilities on the deck will also retain their integrity during the event. Basically, the structure acts as a filter, altering the frequency content of the energy being absorbed at the base of the platform. Figure 11 illustrates the magnification in the vertical frequency response at deck level of four different height platforms subjected to the same vertical base excitation.

Ground motion components having frequencies close to the natural frequency of the platform are amplified while those away from the natural periods of the structure are phased out. Thus, the local vertical and horizontal accelerations to which the facilities and equipment on the deck are subjected may be significantly higher than the peak ground accelerations. It is important that this fact be recognized so that appropriate design criteria can be formulated to prevent or minimize damage to facilities and equipment on the deck due to an earthquake that the structure might survive with little or no damage. The usual procedure is to develop representative "deck response spectra" based on the response of the structure to several different earthquake records. With this spectra, and depending upon the natural period of the equipment or the equipment/deck support, either a static, a pseudo-static, or full-modal analysis can be used to determine required design forces. In essence, the natural period of the equipment or the equipment/deck supports determine its response to the seismic event.

If the equipment natural period is in the region of high energy input from the deck response, there could be a substantial amplification in the vibrational response. In a like manner, if the natural period of the system is such that no amplification is caused by the deck response, the system only undergoes a rigid body translation, and its motion is assumed to be equal to the deck's motions. Although most equipment has a high natural frequency and would respond statically to earthquake loading, the designer should be aware of situations in which the flexibility of deck support members might increase the systems dynamic response. For example, a very heavy container supported on a series of flexible beams which is subjected to vertical excitation or a very tall vessel on flexible supports which is subjected to rocking motion from the lateral excitation.

Because it is desirable to prevent deck equipment failures from happening before failures occur in the supporting structure, deck equipment is usually designed for allowable material stresses proportionally lower than those used within the structures. It is usually assured that equipment tie-downs tend to be the weakest link in the system and, as such, should be designed for even lower allowable stresses. In general practice, a one-third increase in the allowable material stress is permitted when incorporating load conditions, including those for earthquake-induced motion, in the design of deck members and equipment with no increase allowed for equipment tie-downs or brackets.

There are, however, two subsystems relating to deck-supported facilities, that should require a more elaborate design methodology. Both cranes and drilling rigs (due to their long, slender structure and correspondingly long natural periods) can exhibit independent natural responses to a seismic event with corresponding increases in forces and moments within the structures. Two recent references, one relating to cranes[11] and the other to drilling rigs[12], show the advantages of using a more refined dynamic analysis of the system to ensure that members are designed for realistic load conditions.

A few comments are made concerning safety-ralated equipment such as fire water lines, well control equipment, and personnel escape systems. The general consensus is that these items should be configured by assuming that, even with prudent attention to design variables such as prescribed loadings and material strengths, a failure will occur. That is, a failure-mode type of analysis should be conducted, with increased controls and redundancies designed into the systems to safely handle certain failure situations and mitigate the resulting hazard to personnel, the structure, or the environment.

7. GUIDELINES AND STANDARDS

The American Petroleum Institute (API), through several advisory committees, has established requirements for the seismic design of offshore platforms and these guidelines are stated in its Recommended Practice (RP)2A[13]. The API guidelines employ a dual-objective design philosophy in that a structure must provide adequate elastic strength and stiffness to withstand an earthquake of an intensity which has a reasonable "likelihood" of not being exceeded during the life of the structure. The guidelines also state that the platform should have adequate inelastic ductility so that it is able to withstand rare-intense earthquakes without collapsing, although major structural damage might occur.

Until recently, the API guidelines gave explicit recommendations for meeting the strength and the ductility objectives. The ductility calculation requirements, involving the calculation of inelastic deformations and strain energy, have been replaced by statements recommending good seismic design practices such as brace configuration and joint details. Several recent experimental and analytical studies have shown that approximately the same ductility goals can be accomplished by following these recommendations.

The API is currently developing a load and resistance factor design (LRFD) format for fixed offshore platforms. As part of this effort, LRFD factors for fixed offshore platform sites in seismically active areas will be specified. The LRFD format should provide a more consistent quantification of the bias and uncertainities contained in the seismic design process[29,30].

The Minerals Management Service (MMS) requirements are aligned with the API guidelines and are published in 30 CFR Part 250 of the U.S. Code of Federal Regulations[31]. The MMS, in verifying the design of platforms that may experience seismically-induced loadings, requires that dynamic analyses of the structures be conducted using earthquake-ground motions of the intensity appropriate to the site in question. The motions can be decided by applicable ground motion records or by response spectra consistent with the recurrence period appropriate to the design life of the platform. For design purposes, joint and member stresses from a "strength level" analyses must remain within allowable limits. As a second level of verification, some platform designs are required to demonstrate sufficient reserve capacity to prevent collapse, though not local failures, of the platform under a rare intense earthquake.

In lieu of a lengthy site-specific determination of the intensity and the characteristics of seismic ground motion, the MMS permits the use of "defensible" standardized spectra applicable to the region of the installation site. The spectra must reflect those site-specific conditions affecting frequency content and energy distribution.

When using the "time history" method for structural analysis, the MMS requires that at least three sets of ground motion time histories be used. These may consist of recorded or constructed (synthetic) earthquake time histories. The manner in which they are used must account for the potential sensitivity of the structure's response to variations in the phasing of the ground motion records. Ground motion descriptions are required to consist of three components corresponding to the two orthogonal horizontal directions and to the vertical direction.

8. SEAQUAKES

Another area of concern, especially for floating production systems where seismic effect are normally not considered or for other floating facilities subjected to long-term exposure periods, is a phenomenon called seaquake. During a seismic event, earthquake energy propagates within the earth's crust in the form of shear or compression waves. However, due to the inability of water to transmit shear, this energy is propagated through the water column in the form of compression waves. The literature contains relatively little information on the propagation of such waves or their effects on ships or offshore structures. Several instances have been reported of ships being severely shaken by seaquakes, with resulting damage indicating large deck accelerations[32].

Although there is little data available on this phenomoneon, the possibility exists that earthquakes may impose just as severe loadings on floating structures as for structures located on land. Since the mechanisms behind the characteristics of seaquakes are not well established, further investigations are necessary to better explain their probability of occurrence and the effects that they could have on permanent floating installations.

9. RESEARCH

The MMS has initiated two research projects to better understand the effects of earthquake loadings on the dynamic response of offshore platforms. The Seafloor Seismic Data Study project is being conducted by the Sandia National Laboratories to obtain and analyze seafloor earthquake motion data for seismically active areas off southern California. The program is focusing its efforts on the use of the Seafloor Earthquake Measurement System (SEMS) to collect and store seafloor seismic events. Figure 12 llustrates how seismic information obtained in this manner can be used to evaluate soil and structural response models where actual field measurements are being compared to calculated data.

The SEMS concept consists of two subsystems; a seafloor package and a shipboard unit as illustrated in Figure 13. The seafloor package captures and records seismic activity, and transfers the data on-command to the shipboard subsystem via an underwater telemetry system. The shipboard unit records the telemetered digital data on magnetic media. Situated on the seafloor is the seismic instrumentation package. The seafloor electronics package, referred to as the Data Gathering System (DAGS), which is connected to a seismic probe embedded in the seafloor sediments. The probe contains three accelerometers, configured triaxially to record the seismic motions, and a two-axis magnetometer to determine the absolute orientation of the accelerometers. The DAGS consists of a microcomputer controller, a data acquisition interface, volatile Random Access Memory (RAM), permanent nonvolatile bubble memory, an acoustic telemetry system, and a battery power pack. The major attributes of the DAGS include low power consumption for extended system life; broad seismic dynamic range-sensing capability; efficient use of limited digital data storage capacity; and a remote interrogation of the system from the surface. Low power consumption is achieved by use of CMOS technology and other low power components wherever possible. A nonvolatile magnetic bubble memory is used for long-term data storage (5 years) and is only activated when significant earthquake events are detected. This bubble memory device has the advantage of being de-energized for long periods of time without loss or degradation of stored data.

The SEMS uses low power accelerometers which are capable of measuring acceleration levels from 0.05 milli-G's to 10 G's over frequencies ranging from 0.2 Hz to 20 Hz. Each accelerometer signal is digitized at a 100 Hz rate by a high-precision data acquisition subsystem. The digital accelerometer data is placed into volatile RAM and analyzed by the microprocessor. The DAGS incorporates earthquake event detection through the use of pretrigger RAM buffering. The RAM is used to hold data while a determination is made whether these data should be saved or discarded. Short-term and long-term running averages are calculated and used to define the strength of seismic activity. If the system declares that the current data correspond to an earthquake event, the data are stored in the bubble memory. Allocation of the bubble memory is based on a hierarchal scheme which retains only the most significant seismic events.

Communication with the seafloor package is accomplished with the use of an acoustic telemetry data link. The DAGS can be controlled from a surface computer, thus allowing data retrieval, diagnostic analysis, and variability in operating parameters. The surface instrumentation consists of acoustic transmitting and receiving circuitry, referred to as the Buoy Repeater System (BRES), and a controlling IBM PC compatible computer. The PC serves as a user-friendly interface and allows direct communication with the seafloor instrumentation package. Additionally, data retrieved from DAGS is stored on floppy disks, and the earthquake data can be analyzed and graphically viewed onboard the ship during the interrogation[33].

By the end of the year (1989) it is envisioned that two SEMS units will be deployed offshore southern California. The two instruments, approximately 150 miles apart will form a seismic array and will provide an invaluable source for seafloor measurements of earthquake-induced seismic motions.

As an adjunct study, a project entitled "Seismic Response Analysis of Offshore Pile Supported Structures" is being conducted by the University of California, San Diego, to assess the reliability of current state-of-the-art computational pile-soil interaction models used to predict the of energy transmitted to a platform. Seismic accelerograms will be obtained from the SEMS units and from onboard adjacent instrumented platforms. Data that Tokai University recently obtained from an instrumented structure offshore Japan will also be used in the study[19]. It is anticipated that these studies will attest to using present day standards to accurately predict platform responses and, if necessary, to propose changes to design standards so that they are more reliable.

10. CONCLUSION

This paper was written to provide the reader with an understanding of the key considerations that need to be reviewed in determining the response and performance of offshore facilities to earthquake induced ground motion. The simplistic manner in which these considerations are presented should not be inferred as indicative of the procedure. Determining the response of these systems to intense seismic events represents a very complex, and at this time, not fully understood problem. The references cited should be consulted to acquire a more complete understanding and appreciation of these aspects and factors.

What is known is that offshore platforms have exhibited an inherent capability to perform well during earthquakes. In large measure, this capability is a consequence of the attention to details put forth in designing structures for the large wind, wave, and current forces that the platforms must resist. Because of the extensive research being conducted by both Government and industry, it is felt that the required technologies have been and will continue to be developed to provide safe and reliable operations in seismically active areas.

11. ACKNOWLEDGEMENTS

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12. REFERENCES

1. Brian Watt Associates, Inc., "A Study on the Feasibility of Production, Storage and Loading Systems in the North Aleutian Basin (OCS Lease Sale 92)," 284/BWA, Houston, Texas, September 1985.

2. Shimazaki, K., and Nakata, T., "Time Predictable Recurrence Model for Large Earthquakes," Geophysics Research Letters, 7, 1980, p. 279-282.

3. Wiggins, R.A., Sweet, J. and Frazier, G.A., "Seismic Risk for Offshore Structures," OTC 3113, Proceedings, Offshore Technology Conference, 1978, Vol. I, p. 529-533.

4. Idriss, I.M., Dobry, R. and Power, M.S., "Soil Response Considerations in Seismic Design of Offshore Platforms," OTC 2355 Proceedings, Offshore Technology Conference, 1975, Vol. III, p. 191-205.

5. Bea, R.G., "Moderator's Report," Session on Offshore Earthquake Geotechnology, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26-May 2, 1981, St. Louis, Missouri.

6. Bea, R.G., Audibert, J.M.E. and Akky, M.R., "Earthquake Response of Offshore Platforms" Journal of the Structural Division, Proceedings, American Society of Civil Engineers, Vol. 105, No. ST2, February, 1979, p. 377-400.

7. Bea, R.G., Litton, R.W., Nour-Omio, S. and Chang, J.Y., "A Specialized Design and Research Tool for the Modeling of Near-Field Pile-Soil Interactions," OTC 4806, Proceedings, Offshore Technology Conference, 1984, p. 249-254.

8. Liou, D.N. and Penzien, J., "Seismic Analysis of an Offshore Structure Supported on Pile Foundations," Report No. UCB/EERC-77/25, Earthquake Engineering Research Center, University of California, November 1977.

9. Marshall, P.W., "Design Considerations for Offshore Structures Having Non-Linear Response to Earthquakes," Preprint 3302, American Society of Civil Engineers Convention, October 1978, Chicago, Illinois.

10. Popov, E.P., Mahin, S.A. and Clough, R.W., "Inelastic Response of Tubular Steel Offshore Towers," Journal of Structural Engineering, Vol. 111, No. 10, American Society of Civil Engineers, October 1985.

11. Cardenas, J., "Drilling Rigs Designed for Earthquakes," Petroleum Engineer International, May 1988, p. 68-71.

12. Long, R.J., Watmough, D. and Effenberger, M., "Seismic Evaluation of Platform Cranes," OTC 6171, Offshore Technology Conference, May, 1989.

13. "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," API RP2A, 17th Edition, American Petroleum Institute, Washington, D.C., April 1987.

14. Arnold P., "Seismic Risk Analysis with Combined Randon and NonRandon Earthquake Occurrence," OTC 3111, Offshore Technology Conference, May, 1978.

15. Page, R.A., "Evaluation of Seismicity and Earthquake Shaking at Offshore Sites," OTC 2354, Offshore Technology Conference, May, 1975.

16. Reece, E.W., Ryerson, D.E. and McNeill, R.L., "Long-Term Measurements of Ground Motions Offshore," PROCEEDINGS, VOL. 1, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Rolla, Missouri, April 26-May 3, 1981.

17. Bea, R.G., Arnold, P., Idriss, I.M., Beebe, K.E. and Marshall, P.W., "A Study of Soil-Pile-Structure Systems in Severe Earthquakes," OTC 2749, Offshore Technology Conference, May 1977.

18. Pawsey, S.F., Knudson, W.C., Vahdani, Si and Bach, G.F., "Seismic Response of Gravity Structures with Soil Interaction," PROCEEDINGS OF BOSS '85, Delft, Netherlands, Jul 1985.

19. Nogami, T., "Soil/Pile Interaction Due to Seismic Response of Subsea Soils," Technology Assessment and Research Program for Offshore Minerals Operations--1988 Report, OCS Study MMS 88-0057, Minerals Management Service, Reston, Virginia.

20. Finn, W.D.L., Martin, G.R. and Lew, M.K.W. "Application of Effective Stress Methods for Offshore Seismic Design in Cohesionless Seafloor Soils," OTC 3112, Offshore Technology Conference, May 1978.

21. Marshall, P.W., "An Overview of Recent Work on Cyclic Inelastic Behavior and System Reliability," PROCEEDINGS, Structural Stability Research Council, Lehigh University, Bethlehem, Pennsylvania, 1982.

22. Mahin, S.A., Povov, E.P. and Zayas, V.A., "Seismic Behavior of Tubular Steel Offshore Platforms," OTC 3821, Offshore Technology Conference, May 1980.

23. Marshall, P.W., Gates, W.E., and Anagnostodoulos, S., "Inelastic Dynamic Analysis of Tubular Offshore Structure," OTC 2908, Offshore Technology Conference, May 1977.

24. Bea, R.G., "Earthquake and Wave Design Criteria for Offshore Platforms," Journal of the Structural Division, Vol. 105, NoST2, American Society of Civil Engineers, February 1977, pp. 401-419.

25. Mariow, R.S., "Effects of External Hydrostatic Pressure on Tubular Beam-Columns," Final Report, SWRI Project Number 06-1184, Southwest Research Institute, San Antonio, Texas, February 1988.

26. Chen, J., Ullmann, R.R. and Mason, A.B., "Measurement of Earthquake Ground Acceleration and Structural Response of a Fixed Offshore Platform," OTC 6172, Offshore Technology Conference, May 1989.

27. Mason, A.B., Chen, J., Beck, J.L. and Ullmann, P.R., "Model Parameter Identification of an Offshore Platform from Earthquake Response Reloads," ASCE Structural Congress, American Society of Civil Engineers, San Francisco, California, May 1989.

28. Wilson, B., "Earthquake Analysis Studies for Offshore Platforms," Annual Meeting, American Petroleum Institute, San Antonio, Texas, April 1982.

29. PMB Systems Engineering, Inc., "Seismic Load Effects Study," Prepared for the American Petroleum Institute, API PRAC Project 22 Technical Advisory Committee, 1985.

30. Moses, F., "Development of Preliminary Load and Resistance Design Document for Fixed Offshore Platforms," API PRAC Project 85-22, Case Institute of Technology, Cleveland, Ohio, January 1986.

31. U.S. Code of Formal Regulations, 30 CFR, Part 250--Oil and Gas and Sulphur Opertions in the Outer Continental Shelf, Subpart I - Platforms and Structures.

32. Selnes, P.B., Hove, K. and Bungum, H., "Seaquakes: A Potential Threat to Offshore Structures," Third International Conference on the Behavior of Offshore Structures, Cambridge, Massachusetts, August 1982.

33. Sleefe, G.E., "Seafloor Seismic Data Study," Technology Assessment and Research Program for Offshore Minerals Operations--1988 Report, OCS Study MMS 88-0057, Minerals Management Service, Reston, Virginia, 1988.

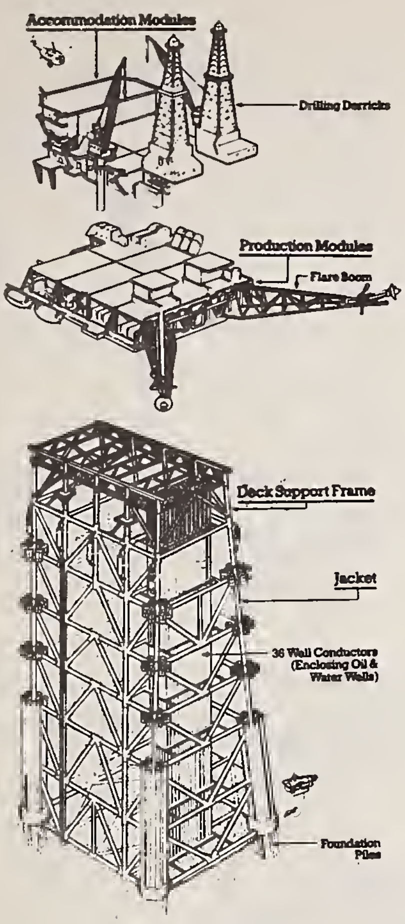

Figure 1. Typical components of a steel-jacket offshore platform.

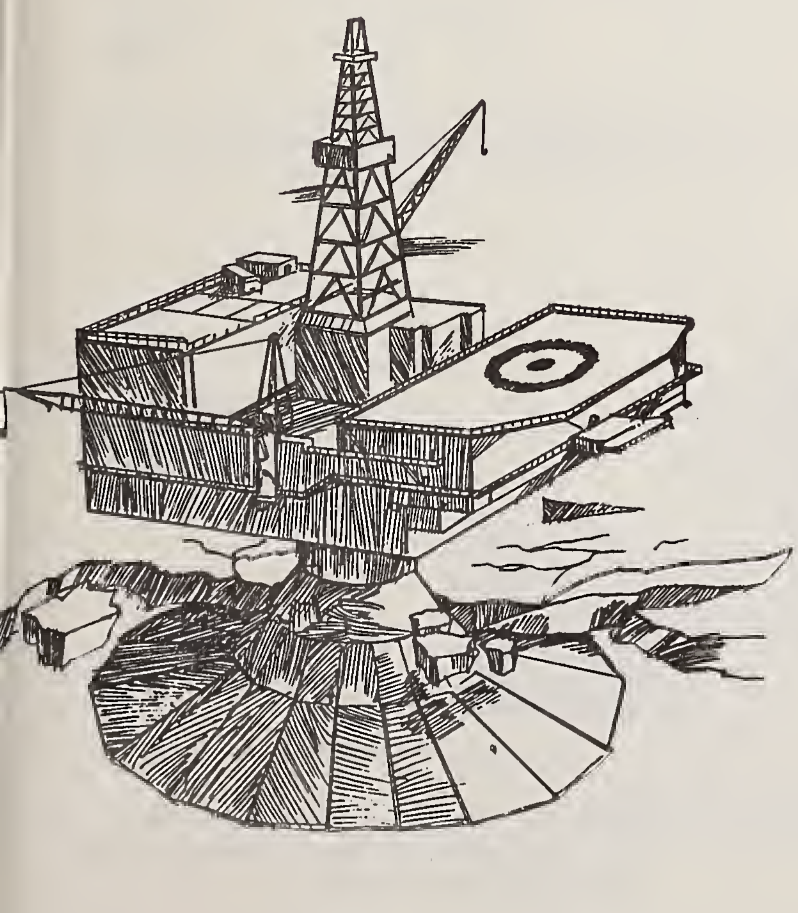

Figure 2. Typical arctic gravity platform.

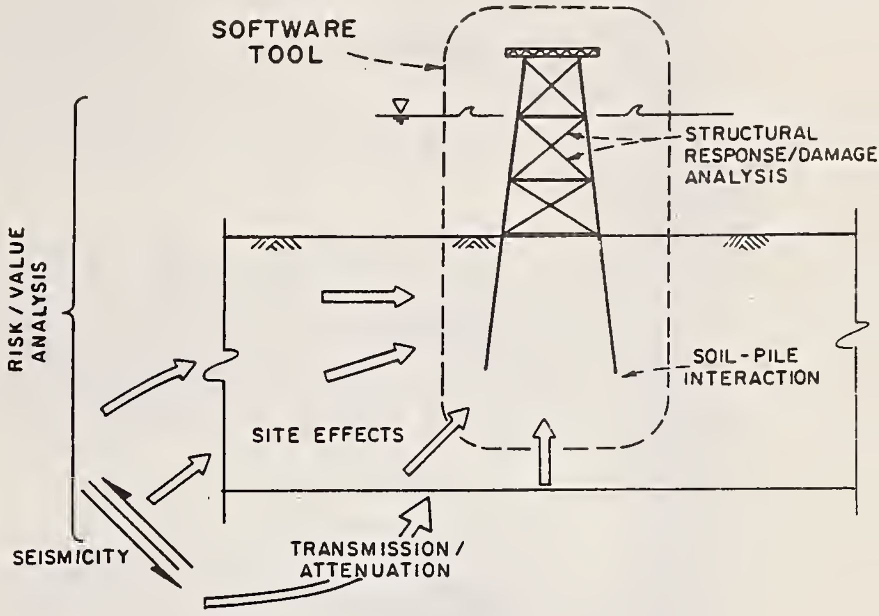

Figure 3. Areas of consideration for offshore earthquake engineering technology. (Taken from Reference 17)

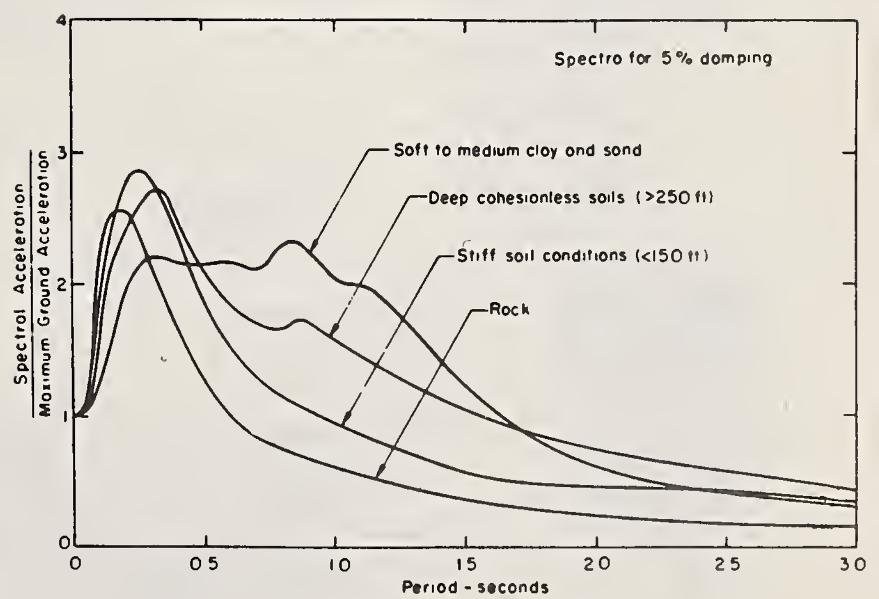

Figure 4. Average acceleration spectra for different site conditions. (Taken from Reference 4)

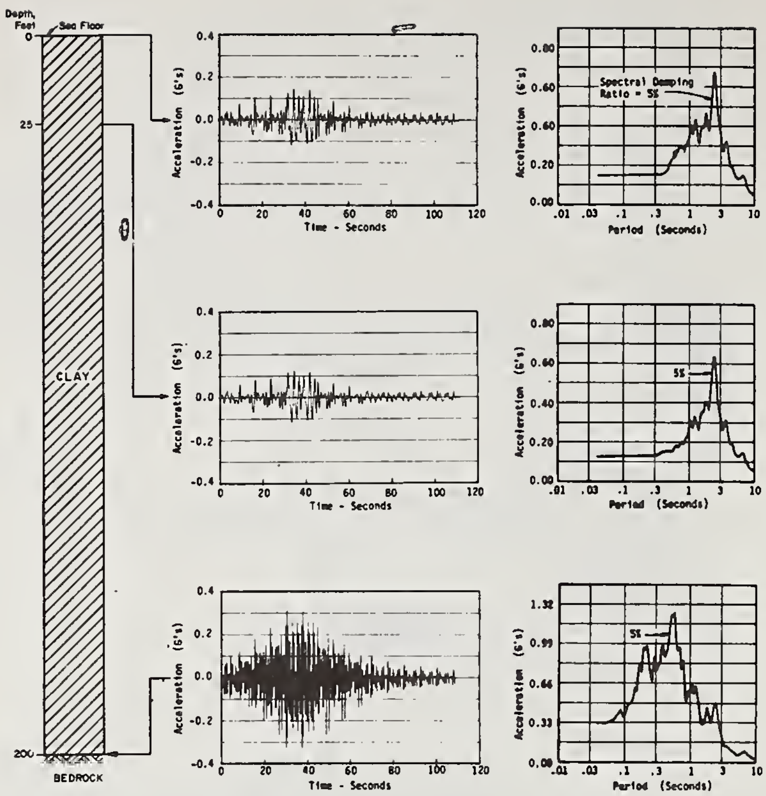

Figure 5. Example results of ground response analysis for an offshore platform site. (Taken from Reference 4)

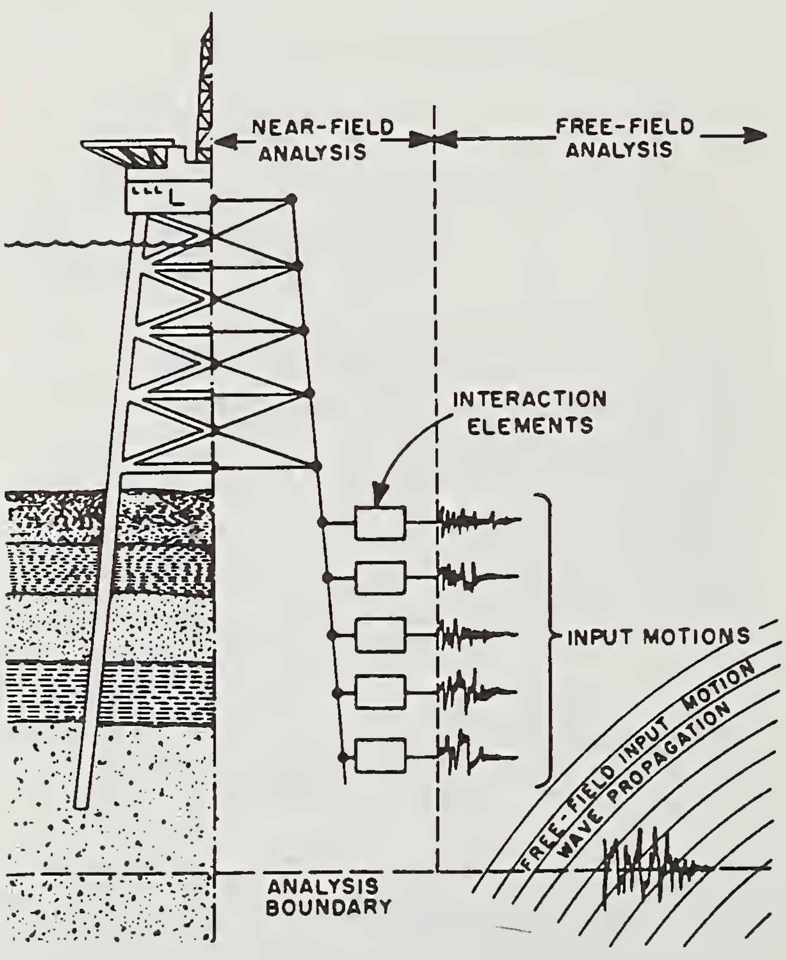

Figure 6. Offshore soil-pile-structure interaction model for earthquake response. (Taken from Reference 20)

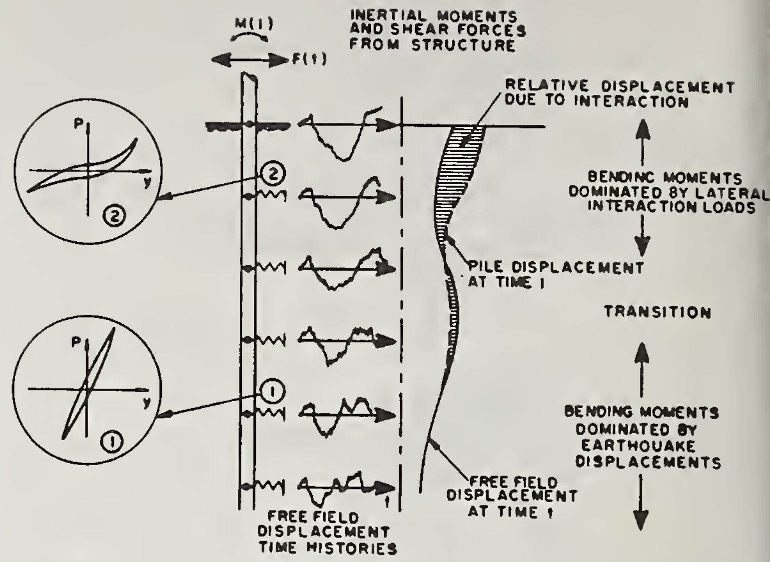

Figure 7. Qualitative description of soil-pile interaction under seismic loads. (Taken from Reference 20)

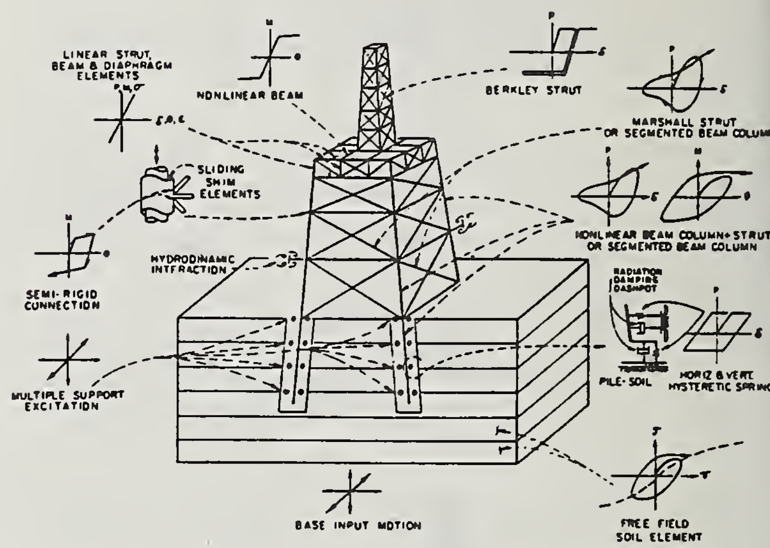

Figure 8. Typical finite element model required to evaluate the soil-pile-structure system. (Taken from Reference 17)

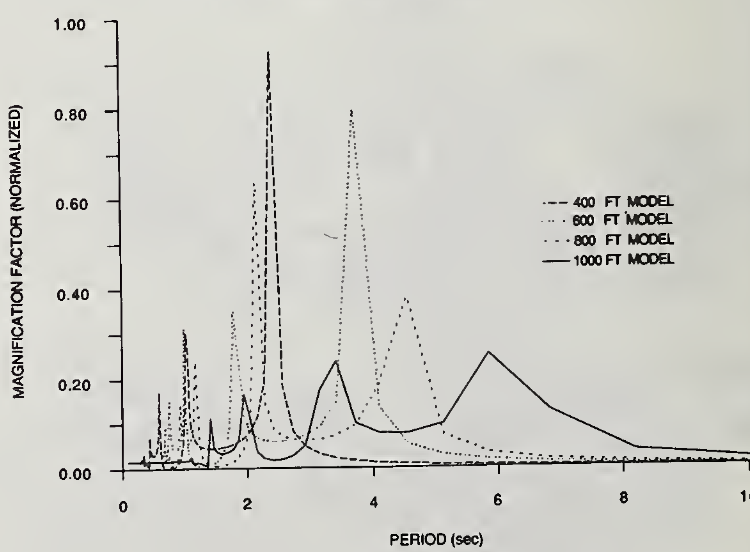

Figure 9. Frequency response at deck level due to horizontal base excitation fluid-structure interaction not included.

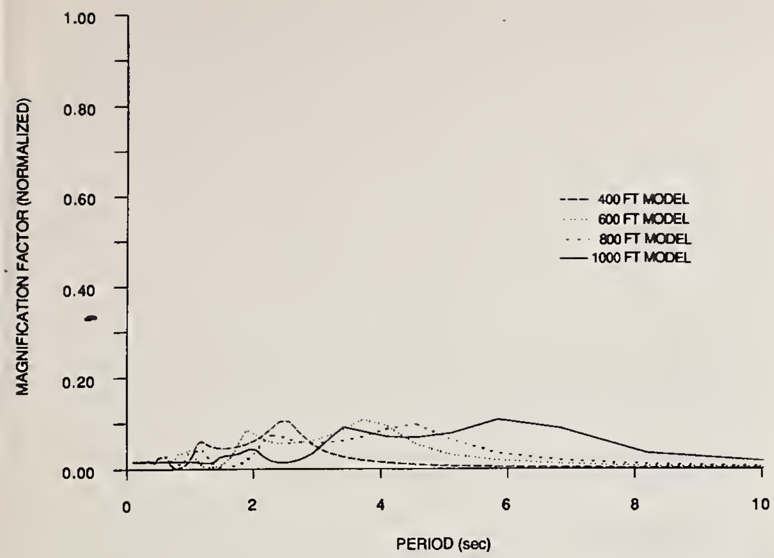

Figure 10. Frequency response at deck level due to horizontal base excitation; fluid-structure interaction included.

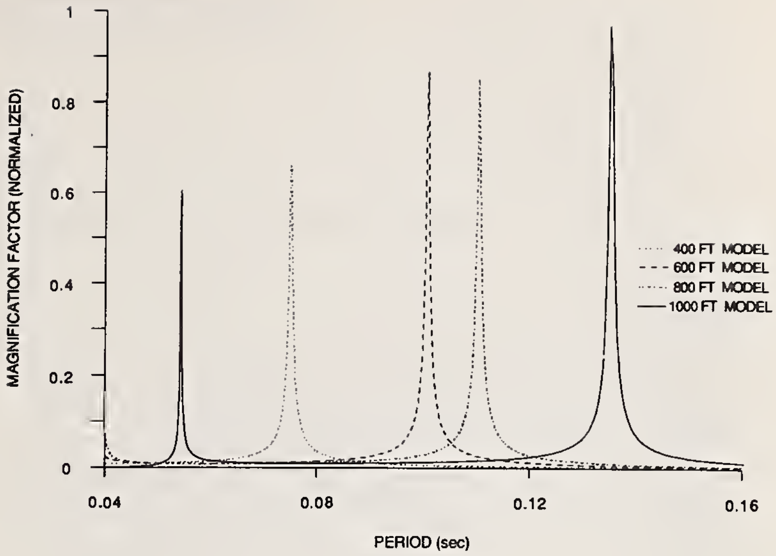

Figure 11. Frequency response magnification at deck level due to vertical base excitation.

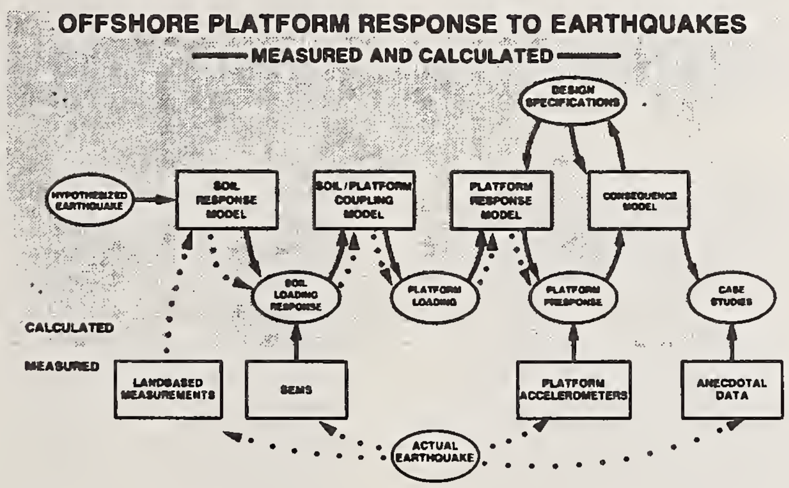

Figure 12. Evaluation methodology for offshore platform analysis and design.

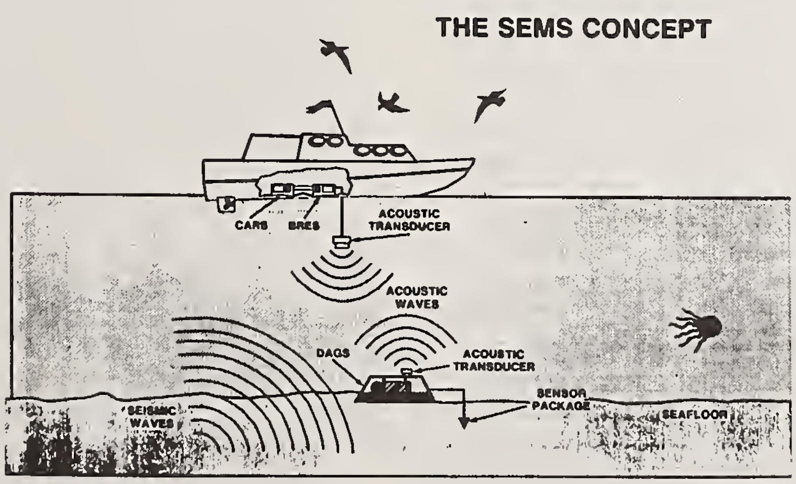

Figure 13. Illustration of the Seafloor Earthquake Measurement System (SEMS) for incorporating long-term data recordings,

Centrifuge Dynamic Model Tests in Phri

By

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SUMMARY

A centrifuge model test is one of the testing method on geotechnical engineering. This method is known to satisfy the similitude for model tests of geotechnical materials. The autors have developed earthquake simulators which can be accomodated in the centrifuge of the Port and Harbour Research Institute (PHRI), and carried out dynamic model tests in centrifugal field. The earthquake simulator is of electrohydraulic type and can generate scaled large earthquake motions. The paper introduces and illustrates the outline of the earthquake simulators and some experimental results using this apparatus.

KEY WORDS: Centrifuge, Dynamic Model Test, Earthquake, Test Equipment

1. INTRODUCTION

Soil structures and foundations in Japan have often suffered from great earthquakes. Studies in the dynamic geotechnical field are very important in Japan.

A centrifuge model test is one of the testing method on geotechnical engineering. Using this method we can make the same stress condition in a model ground as that in a real structure. Because the physical properties of geotechnical material, no matter which are static one or dynamic one, depend on its confining pressure, the use of a centrifuge is effective for both static and dynamic model tests. So far, only static model tests have been carried out in the centrifuge of PHRI since the centrifuge was constructed in 1981 (Ref.1). The use of the PHRI centrifuge for dynamic tests had to wait until recently when the authors decided to start developing the earthquake simulator which can be accommodated in the existing PHRI centrifuge (Ref.2). Obviously the centrifuge model test satisfies the similitude for models made of geotechnical materials and is, at present, the most promising method for testing model grounds and structures. However, many difficulties had to be overcome for developing the earthquake simulator and for applying to a dynamic model test in the centrifuge. The purpose of this paper is to introduce the outline of the earthquake simulator for PHRI and to show some results of application to the dynamic problem.

2. DESIGN OF THE EARTHQUAKE SIMULATOR

2.1 Scaling Law

The general scaling relationship for a dynamic model test is shown in Table 1 (Ref. 3). The scaling law for a centrifuge dynamic model test is, also given in Table.1, obtained by substituting n_ℓ, n_ε, n_ρ, n_g for 1/n, 1, 1, n in the general scaling law, respectively. A small-scale model and a full-scale structure are related through this relationship. According to this scaling law, if a model is constructed at a 1/50 scale, it is subjected to a 50G centrifugal acceleration to simulate the prototype structure. In the dynamic test, the frequency of the input motion applied to the model is 50 times higher, and the acceleration is 50 times larger. For example, in order to simulate a real earthquake motion, which has a peak acceleration of 0.2G, a duration of 60s and a frequency content up to 6Hz in the scaled model, the model is subjected to an excitation which has a peak acceleration of 10G, a duration of 1.2s and a frequency content up to 300Hz. If the mass of the model container and specimen is about 100kg, the capacity of the shaking force must be about 10kN.

In this scaling law it is found that the stress condition at homogeneous points is the same in the model and the full scale structure using the same material. That is a major merit of the centrifuge model test compared with the model test in ordinary 1G field. Using the centrifuge we can take account for the fact that the ground itself change its properties under the action of its own weight.

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2.2 Shaking System Selection

In order to design a centrifuge earthquake simulator, we had to consider advantages and disadvantages of various shaking systems. The authors examined various approaches such as the 'Bumpy Road' of Cambridge University, electromagnetic devices, electro hydraulic equipments, explosive firing and piezoelectric actuator(Ref.4-8). With the Bumpy Road, we will have to make a great deal of effort to change the input motion and to modify the PHRI centrifuge. The electromagnetic devices are too heavy and large to mount on the swing platform, and explosive firing seemed difficult to control the input motion. Moreover the authors could not find a large piezoelectric actuator in Japan. Consequently, it was decided to attempt to design an electrohydraulic type of shaker. Though this system is a little complex, we need no improvement on the PHRI centrifuge. This system satisfies the required capacities for shaking and the required reliability. Fortunately, the authors could find an accumulator, a servo valve and shut off valves which can be mounted on the swing platform. The reliability of the servo valve and of the shut off valves and the safety of the accumulator in a 60G centrifugal field were confirmed by the authors before making up the whole system.

3. OUTLINE OF THE EARTHQUAKE SIMULATORS

We made two earthquake simulators till today, one is a trial production system for feasibility study and the other is a version up system for practical experiment. But the fundamental mechanism of both system are the same. The earthquake simulator consists of an oil pressure supply unit, an actuator unit, a control unit and an oil tank. Fig.1 illustrates the arrangement with all the units of the version up system on the swing platform. Fig.2 shows the layout of the oil flow circuit of the simulator. The oil supply unit consists of an accumulator to supply oil pressure to the servo valve and three shut off valves. The A-shut off valve reduces oil leakage from the servo valve. The B-shut off valve is used in case of emergency. The C-shut off valve with flow control valve is used to avoid an impulse liquid flow to the servo valve. Once we accumulate oil pressure, we can experiment ten and several times during one flight.

3.1 Trial Production system

The model container box of the trial production system is fixed on the cylinder of the actuator. The dimensions of the shaking box are 400mm(L) X 180mm(D) X 270mm(H). It is supported by eight arms from four beams. The container is made of titanium to save the total shaking mass. The major specifications of the system are shown in Fig.3.

The system controller is composed of two control boxes. One is mounted on the swing platform and the other is located in the control room. They are connected by eight lines through slip rings. The controller has a sequence circuit to control the experiment from start to end. That is to say, if an operator presets the experimental conditions such as input wave, duration of shaking, amplitude and frequency, the only thing the operator needs to do during the flight is to press one button. The servo valve receives electrical signals from a function generator or Read Only Memory (ROM) and the actuator generates the input motion. Typical earthquake motions are memorized in the four ROMs. Each ROM has a length of 8192 words and we can select the time interval of the input motion from 0.001ms to 10ms.

3.2 Version up System

We improved on the trial production system to overcome the following drawbacks.

(1) We can not see the phenomena in visual because the control box hinders the soil container.

(2) We can select only one input motion during one flight because we have to set the input motion conditions before the centrifuge operation.

(3) It is inconvenient for making a model and for conducting several model tests continuously because the model container is fixed on the cylinder.

(4) The shaking force is small for exciting a satisfactory large model.

The major actuator specification of the version up system is also shown in Fig.3. The shaking force of the version up system is four times larger than that of the former one. And various model containers including a stacked ring apparatus are available because of using a shaking table. The dimensions of the shaking table are 700mm X 350mm. We can control this system and another instruments such as high speed camera from control room by a computer through RS232C interface and can set various test conditions. Input digital data are written in ROM whose capacity is 32K words, we can select input motion used in the test by setting the ROM address.

3.3 Data Acquisition System

There are two current way to collect data as follows. One is ordinary way with the transducer installed in the model, the other is optical way using cameras. In the former method we use a dynamic amplifier on the swing platform to save the slip ring channels and to improve the SN ratio. In the case of centrifuge dynamic model test, as an object is a very fast phenomena, we operate the analogue data recorder from a few seconds before the experiment starts to avoid failure in collecting the data. The data collected by analogue data recorder are converted into digital data after experiment. We can use both a high speed camera and an ordinary camera, which are effective for the phenomena with large deformation. We can trace movement of the target set up in the model by analyzing photographs.

4. APPLICATION OF THE SYSTEM TO THE DYNAMIC GEOTECHNICAL PROBLEM

4.1 Residual deformation of the gravity type caisson during earthquake

The authors discussed about sliding deformation of the gravity type caisson during earthquake using the model as shown in Fig.4. The model ground was constructed by pouring the dry Toyoura sand naturally from 20cm above the water surface into water. The model ground had wet density of 1.89 g/cm^3 and relative density of 38% We used water as a pore fluid material to avoid liquefaction, because the sliding behavior of the caisson on the ground without liquefaction is different from that with liquefaction. The input motion was the integrated displacement of the acceleration observed at Hachinohe Port in the 1968 Tokachi-oki earthquake, M=7.9. Four stage tests ware conducted as shown in Table.2. Fig.5 shows the time histories measured in the stage-4 test. It is found that the caisson slides in a moment when large acceleration takes place. The residual deformation at the top of the caisson and the maximum response acceleration of the caisson are shown in Fig.6 and Fig.7 with the maximum input acceleration in prototype scale. The following trends about the sliding behavior are found in these figure.

(1) The residual deformation increases with increasing the input acceleration. The caisson subjected to input acceleration of 200Gals slides 10cm in prototype scale.

(2) The pore water pressure did not rise remarkably, because the model fluid material was water. This result is predicted by the scaling law about the permeability of the fluid material.

(3) The pore water pressure decreases when the caisson is sliding.

(4) The maximum response acceleration of the caisson did not so much increase with increasing the input acceleration because of caisson sliding.

4.2 Confining pressure effects on the shear wave velocity of the sandy layers

It is well known that a shear modulus of the sand is in proportion to square root of effective confining pressure σ_v' . From elastic wave theory, it is also known that the shear modulus is equal to ρVs^2 . Thus, the shear wave velocity is in proportion to $(\sigma_v')^{0.25}$. This fact is important for evaluating the dynamic properties of the centrifuge model ground constructed. The authors studied on this point with the saturated sandy layers. Fig.8 shows the cross section of the model and the ground depth in prototype scale based on the scaling law. The sand and the pore fluid material used in the model and the model construction method are the same as those of the caisson sliding experiment described above. We conduct experiment with two different relative density model ground. Since it was difficult to detect the shear wave velocity directly in time domain, we estimate it by the response characteristics of the model ground in frequency domain. The input motion is a white noise, that frequency content of the acceleration is constant with frequency, which is generated by the authors in considering the response characteristics of the servo valve. Table.3 lists the test condition in prototype scale. In all cases we calculated the transfer function between the input acceleration and the acceleration response of the sand layer surface and search for the first natural frequency of the ground(f1). Typical transfer function is shown in Fig.9.

4.2.1 Average shear wave velocity of sandy layer

If it can be assumed that the model ground has an average shear wave velocity Vs, Vs can be identified by multiplying f1 by 4H, in which H represents layer thickness. Fig.10 shows the relation between the average shear wave velocity and the effective confining pressure at the middle depth of the ground. In this case the average shear wave velocity did not strongly depend on strain level because the model ground had high stiffness. The solid line described in this figure are regression line which is revolved using equation $Vs = a(\sigma_v')^b$, in which a and b represents constant number.

4.2.2 Distribution of shear wave velocity with depth

We estimated distribution of the shear wave velocity with depth by the following procedure.

(1) First we divide the ground into five layers in case-2 shown in Fig.8. In case-1 the ground is divided into three layers. It is assumed that the divided each layer has a constant shear wave velocity.

(2) We regard the shear wave velocity of the ground from surface to -2.4m depth as the average shear wave velocity estimated by the method described above at 10G centrifugal field.

(3) Next we assume the shear wave velocity of the ground from -2.4m to -4.8m depth as Vs2. Using Vs1 determined on the former stage and Vs2 assumed here, we can calculate the

response of the ground based on multiple reflection theory. If the dominant frequency calculated is agree with that obtained by the experiment at 20G centrifugal field, we adopt Vs2 as the shear wave velocity of the ground from -2.4m to -4.8m. If not so, we assume new Vs2 and calculate again. We calculate several times till obtaining good agreement.
(4) We can estimate the shear wave velocity of the ground under -4.8m depth similarly.

Fig.11 shows the distribution of the shear wave velocity obtained by this procedure. The line described in this figure inclines 0.25 power of the depth through the center point of the middle layer. These results indicate that the confining pressure affects on the shear wave velocity of the sandy ground even if its depth is less than 12m. And close agreement between the experiment and the element test conducted in the 1G field was obtained (Ref.9). The method described here is one of effective way for evaluating dynamic properties of soils in centrifugal field.

4.3 Modelling liquefaction

The research about liquefaction is important object using centrifuge because liquefaction is directly related to the effective confining pressure generated by its own weight. During recent several years some experiment about liquefaction using centrifuge have been conducted (Ref.10-13). It is necessary for modelling liquefaction in nG centrifugal field to reduce permeability of the fluid material n times smaller than that of prototype. Thus we studied relation between the coefficient of permeability and the viscosity of silicon oil as a fluid material. The permeability tests were conducted under 20 degrees centigrade using Toyoura sand of relative density 50% and 70%. Fig.12 shows the test results with viscosity. The results at the point of 1 centistokes represents that using a water as a fluid material. Fig.13 shows permeability reduction ratio of the silicon oil compared with the water. In this figure the coefficient of permeability of a 50 centistokes silicon oil is about 30 times smaller than that of the water. It is also found that the variation of the permeability is almost independent of the relative density.
Fig.14 shows the cross section of the model constructed in a stacked rings. The experiment was conducted in a 30G centrifugal field using 50 centistokes silicon oil. The model ground was constructed by removing the saturated sand into silicon oil without mixing the sand with the air, after getting the air out from the sand steeped in the silicon oil under vacuum. The ground constructed has relative density of 86%. The input motion used here is the same as that of the caisson sliding test described above. Table 4 lists the maximum response acceleration of the ground. The maximum response acceleration increases from bottom to surface. Fig.15 shows the time histories converted into prototype scale. The rise of excess pore pressure was remarkable at the point of the large acceleration took place. Fig.16 shows the distribution of the maximum excess pore pressure with depth. The brake line described in this figure represents initial effective stress. The excess pore pressure generates with increasing acceleration level and cause perfect liquefaction at the upper part of the layer in stage-3 test. According to further investigation, there was few settlement of the ground during vibration.

5.CONCLUSIONS

We introduced the earthquake simulator in PHRI centrifuge and explained some results of application to dynamic geotechnical problem. The earthquake simulator developed by the authors shows satisfactory performances for testing ground and structure models in the centrifuge. Obviously, the earthquake simulator in the centrifuge has a wide applicability to geotechnical problems such as soil-structure interaction. But there are many problems had to be overcome for applying to the dynamic geotechnical testing as follows.
(1) Technical problem
 * Development of more convenient equipment
 * Model construction method
 * Effects of side friction and rigid end wall of model container
 * Choice of the fluid material
 * Evaluation method of constructed model properties in the centrifugal field
(2) Internal problem of centrifuge model test
 * Coriolis effect
 * Influence of difference centrifugal acceleration with model depth
 * Relation between the dynamic properties and frequency, that is to say, an effect of strain rate with cyclic loading.
 * Impossibility of modelling structures made of large material such as gravel.

The technical progress of the dynamic centrifuge testing and the so-called 'modeling of models' analysis will solve these problems, and fruitful results are expected from the model tests in the coming few years.

REFERENCES

1) Terashi,M., "Development of PHRI Geotechnical Centrifuge and Its Application, " Report of the PHRI Vol.24, No.3, 73-122, (1985).
2) Kazama, M., Inatomi, T. and Ohtsuka,K., "Development of an Earthquake Simulator

for the PHRI Centrifuge," Technical note of the PHRI, No.607, 35p. (in Japanese 1988).

3) Inatomi,T, Kazama,M. and Ohtsuka, K., "Development of Earthquake Simulator in PHRI Centrifuge and Its Application", Proc.of 9WCEE, Vol.VIII, 831-836, (1988).

4) Schofield, A. N., "Dynamic and Earthquake Geotechnical Centrifuge Modelling," Proc. of Int. Conf. on Recent Advance in Geotech., 1081-1100, (1981).

5) Scott, R. F., "Centrifuge Model Testing at Caltech," Proc. of Soil Dynamics and Earthquake Engineering, Vol.2, No.4, 188-198, (1983).

6) Zelikson, A., Devaure, B., and Badel, D., "Scale Modelling of Soil Structure Interaction during Earthquake Using a Programmed Series of Explosions during Centrifugation," Proc. of Int. Conf. on Recent Advances in Geotech., 361-366, (1981).

7) Arulanandan, K., Canclini, J. and Anandarajah, A., "Simulation of Earthquake Motions in the Centrifuge," Proc. of ASCE, GT5, 730-742, (1982).

8) Whitman,R.V.,"Experiments with Earthquake Ground Motion Simulation," Proc. of Symp. on the application of centrifuge modelling to geotechnical design, 281-299, (1984).

9) Iwasaki, T., Tatsuoka,F. and Takagi,Y.: "Shear Moduli of Sands under Cyclic Torsional Shear Loading", Soils and Foundations, Vol.18, No.1, 39-56, (1978).

10) Schofield, A.N. and Venter,K.: " Earthquake Induced Pore Pressure in the Foundation of Sea Dyke", CUSD/D-SOILS/TR150, Dept. of Engineering, Cambridge University, 17p. (1984).

11) Whitman, R.V., Lambe,P.C. and Kutter, B.L.:"Initial Results from a Stacked Ring Apparatus for Simulation of a Soil Profile", Proc. of Int. conf. on Recent Advance in Geotech., 1105-1110, (1981).

12) Lee, F.H. and Schofield, A.N.:" Centrifuge Modelling of Sand Embankment and Islands in Earthquakes", Geotechnique 38, No.1, 45-58, (1988).

13) Hushmand,B., Scott, R.F. and Crouse, C.B. " Centrifuge Liquefaction Tests in a Laminar Box", Geotechnique 38, No.2, 253-262, (1988).

14) Koga, Y., Taniguchi, E., Koseki, J. and Morishita, T.:" Sand Liquefaction Tests Using a Geotechnical Dynamic Centrifuge", Proc. of the 20th Joint Meeting of US./JAPAN Panel on Wind and Seismic Effects, US/JAPAN Natural Resources Conference, 8p, (1988).

Table.1 Scaling relationship for a dynamic model test.

Quantity (Model/Prototype)	General scaling law	Scaling law in the centrifugal field
Length	n_ℓ	$1/n$
Mass density	n_ρ	1
Strain	n_ε	1
Acceleration	n_g	n
Time	$(n_\varepsilon n_\ell / n_g)^{1/2}$	$1/n$
Frequency	$(n_\varepsilon n_\ell / n_g)^{-1/2}$	n
Displacement	$n_\varepsilon n_\ell$	$1/n$
Stress	$n_\rho n_g n_\ell$	1
Stiffness matrix	$n_\rho n_g n_\ell / n_\varepsilon$	1
Pore water pressure	$n_\rho n_g n_\ell$	1
Coefficient of permeability matrix	$(n_\varepsilon n_\ell / n_g)^{1/2} / n_\rho$	$1/n$
Bending stiffness of pile	$n_\rho n_g n_\ell{}^5 / n_\varepsilon$	$1/n^4$

Table 2 Test condition

Stage No.	Centrifugal Acc. at 3.62m	Model Amax(G)	Prototype Amax(Gals)
1	50.3	2.4	47
2	50.2	5.6	109
3	50.2	10.1	197
4	49.9	12.8	252

Amax: The max. acceleration of the input motion

Table 3 The model test condition

Target centrifugal acceleration		Maximum acceleration of the input motion in prototype scale, unit:Gal			
Flight No.		1	2	3	4
	50G	---	---	60,61	---
Case-1	30G	---	85,84	---	---
	10G	111,110	---	---	---
	50G	29	53	79	82
	40G	34	61	80	89
Case-2	30G	43	82	126	173
	20G	66	124	209	276
	10G	107	---	---	---

Table.4 The maximum acceleration response of the ground

Measure point	The maximum response acceleration in prototype scale : Gal		
	Stage-1	Stage-2	Stage-3
A 5	145	165	231
A 4	115	150	213
A 3	101	130	195
A 2	76	110	189
A 1	75	135	315

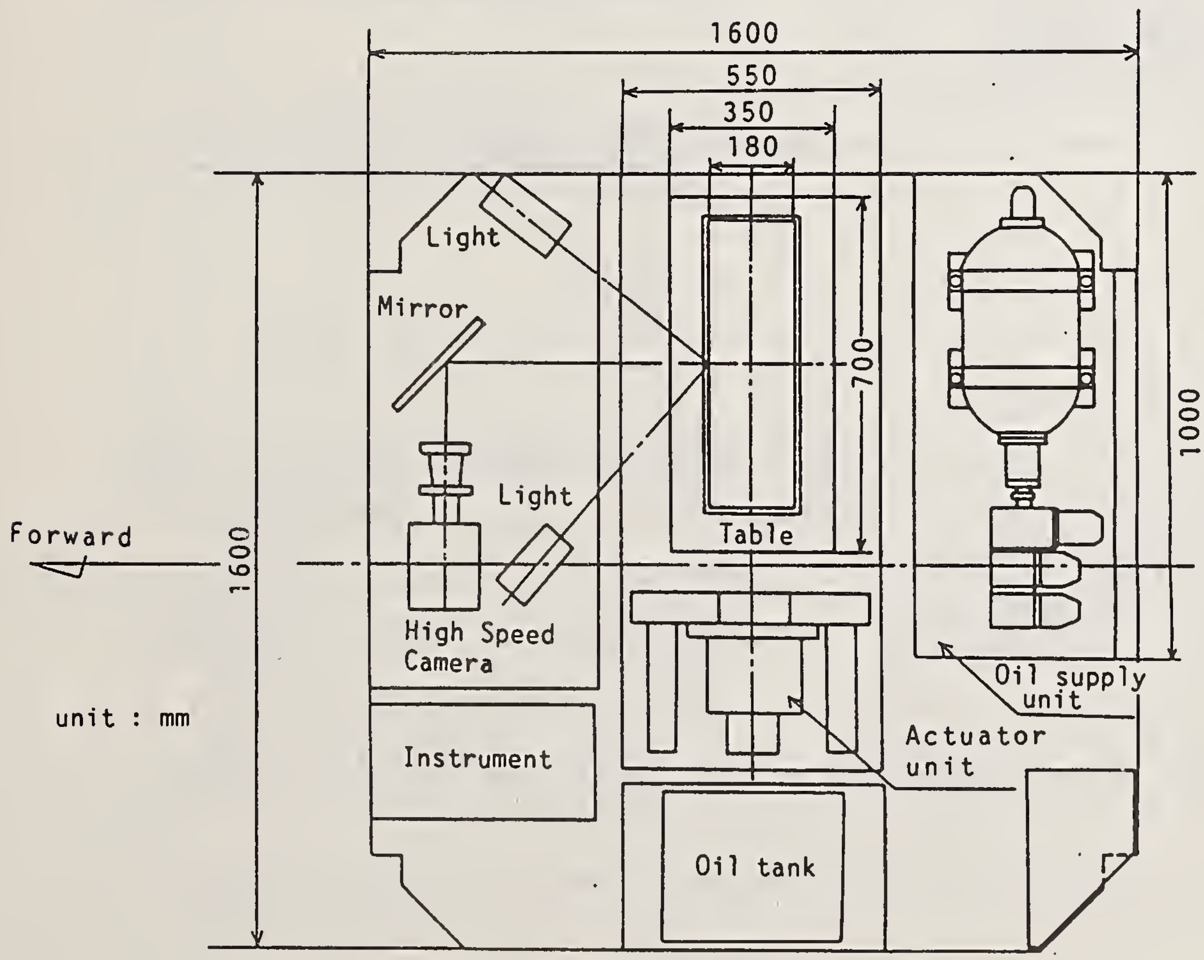


Fig.1 Arrangement of all the units on the swing platform.

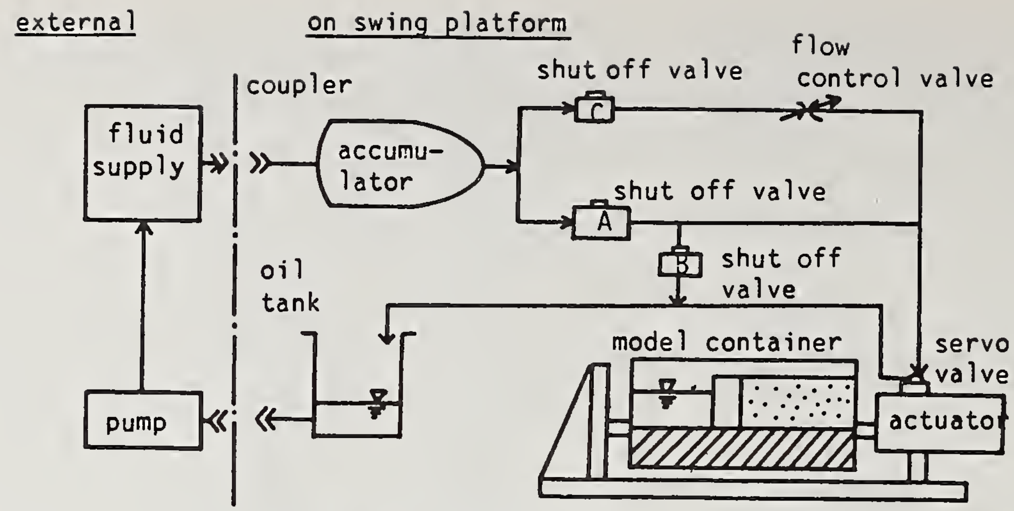
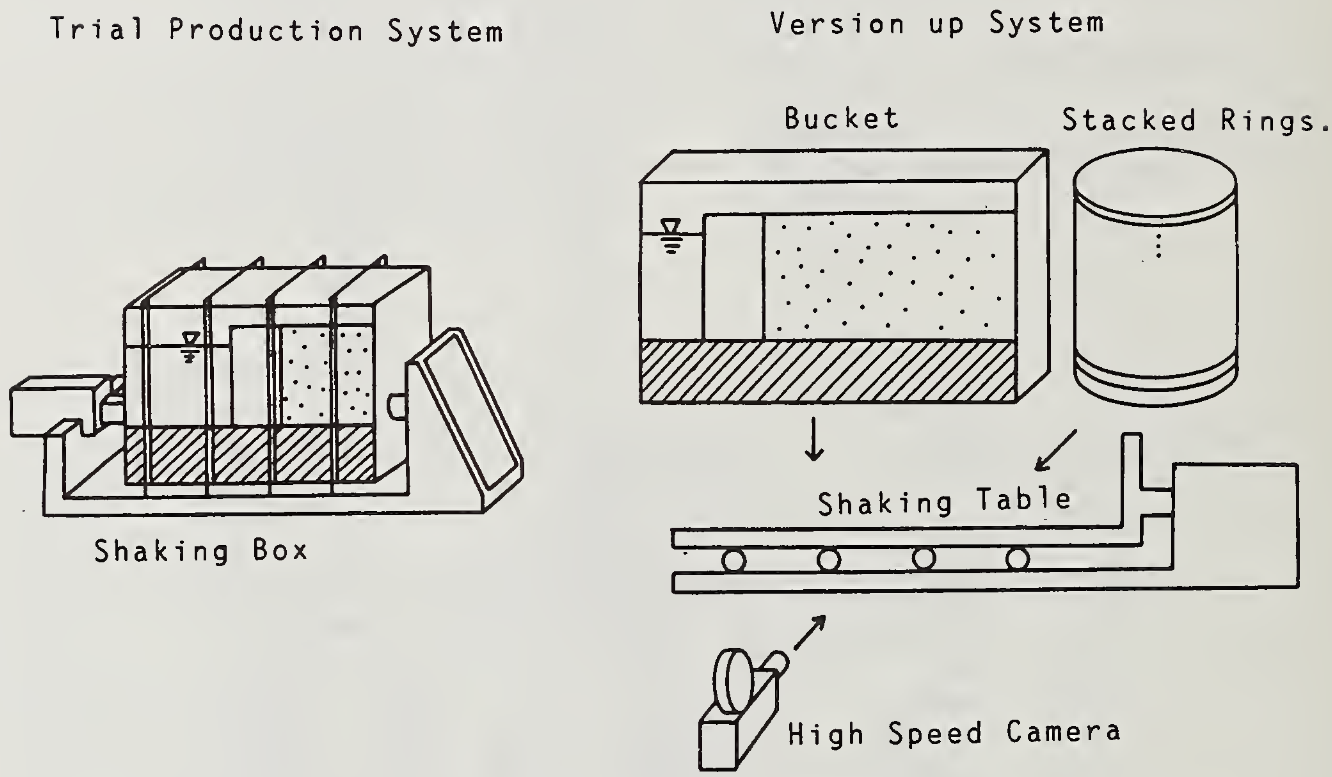


Fig.2 Oil flow circuit



Quantity	Trial production system	Version up system
Shaking force	about + 12kN	about + 50kN
Maximum displacement	+ 3mm	+ 6mm
Maximum velocity	+ 30cm/s	+ 45cm/s
Volume of the accumulator	4 liter	10 liter
Frequency content(nominal)	up to 300Hz	up to 250Hz
Payload	92kg	200kg

Maximum usable oil pressure 30MPa (ordinary use 21MPa)
The servo valve is controlled by LVDT displacement feed back system.

Fig.3 Specification of the earthquake simulator for PHRI centrifuge.

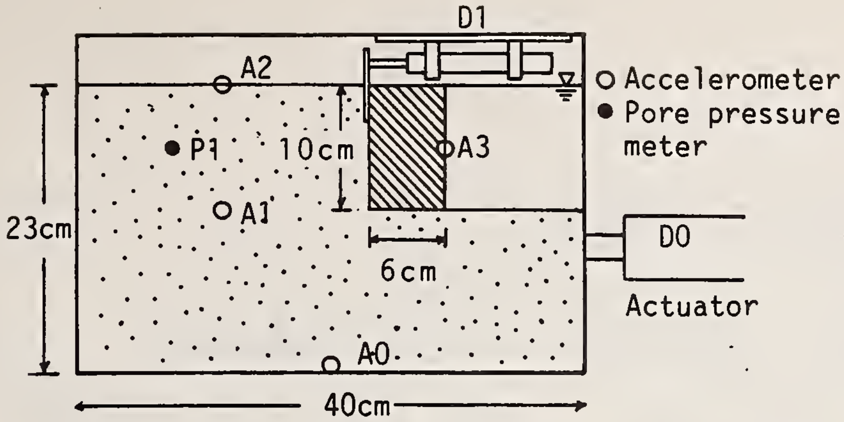


Fig.4 Cross section of the gravity type caisson model.

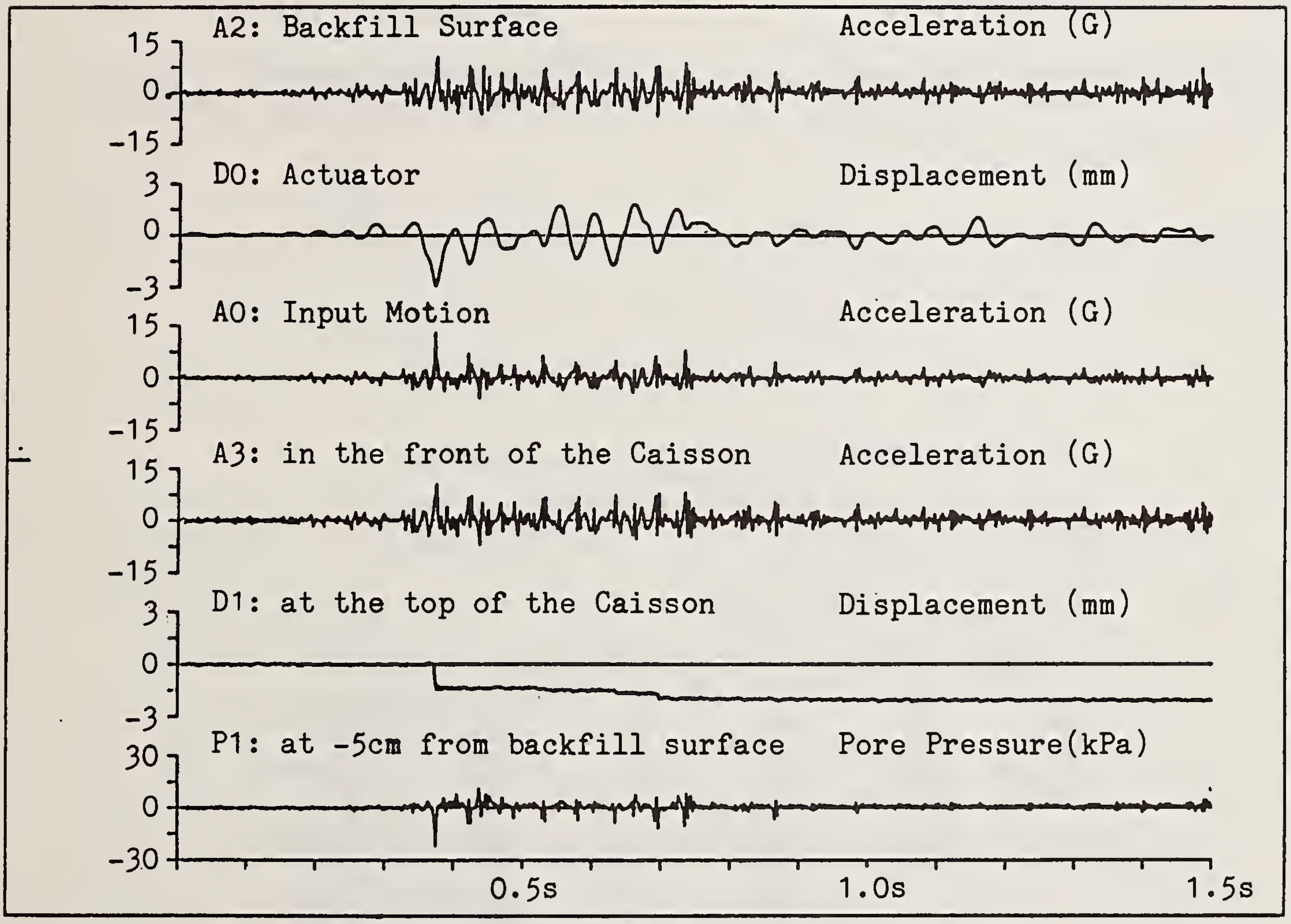


Fig.5 The time histories measured in the No.4 experiment.

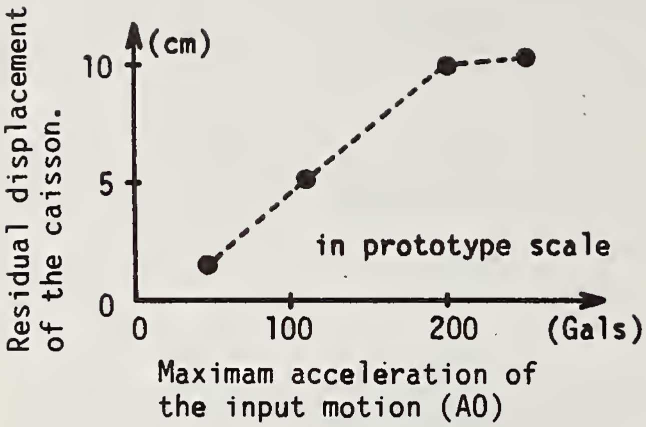


Fig.6 The residual displacement at the top of the caisson.

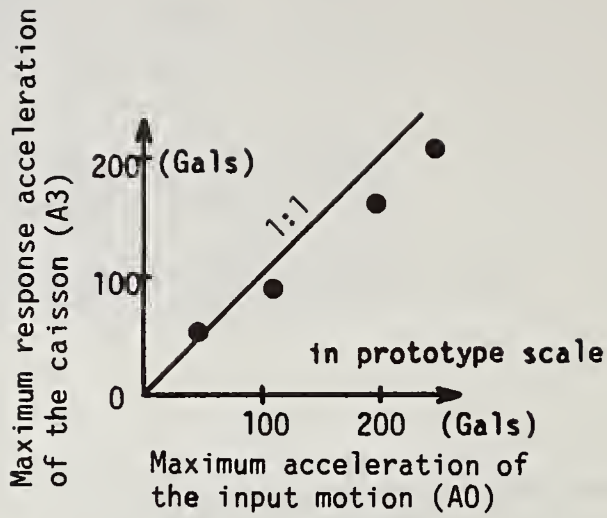


Fig.7 The maximum response acceleration of the caisson.

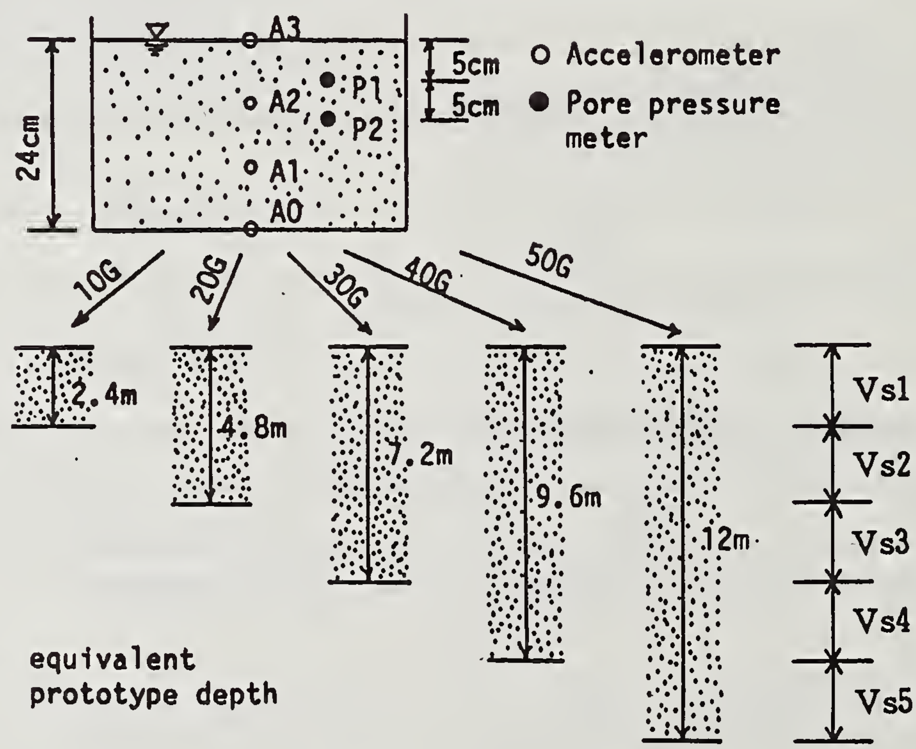


Fig.8 Cross section of the model and the equivalent prototype depth.

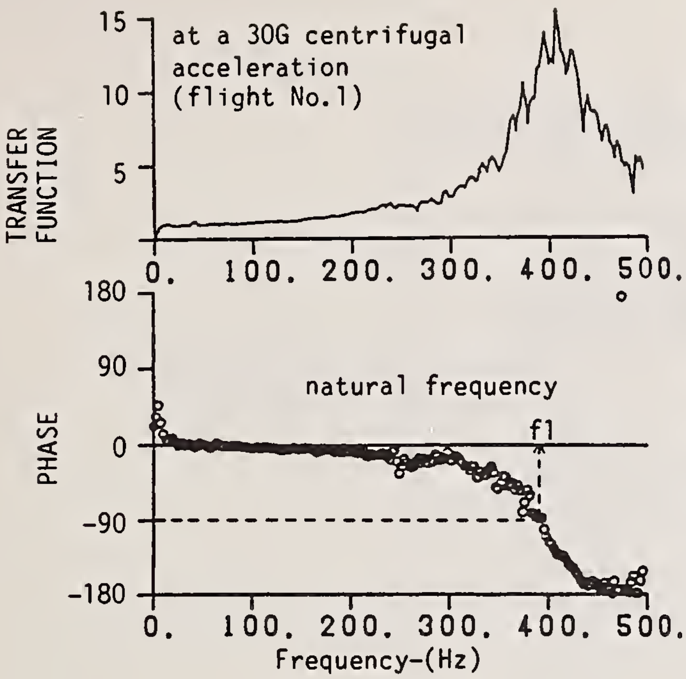


Fig.9 Typical transfer function at a 30G centrifugal field.

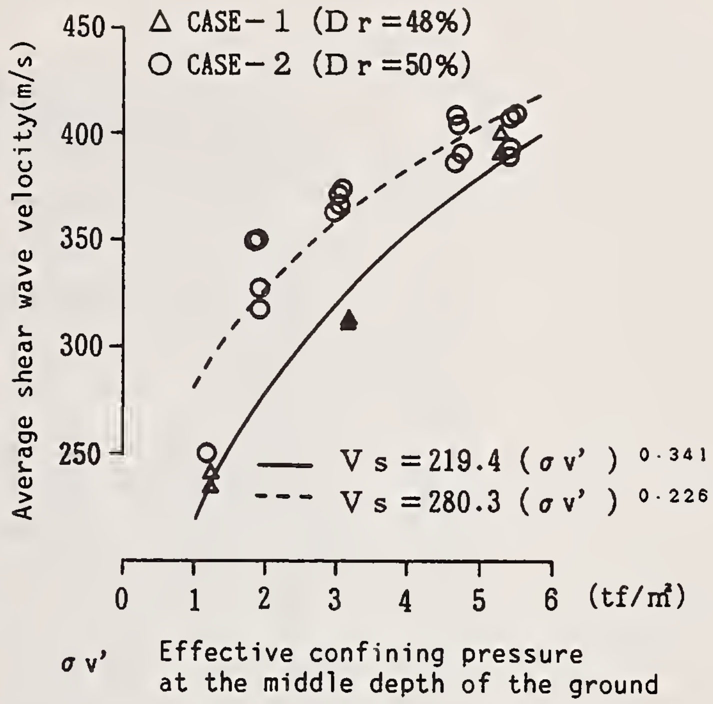


Fig.10 Relation between the average shear wave velocity and effective confining pressure at the middle depth of the ground.

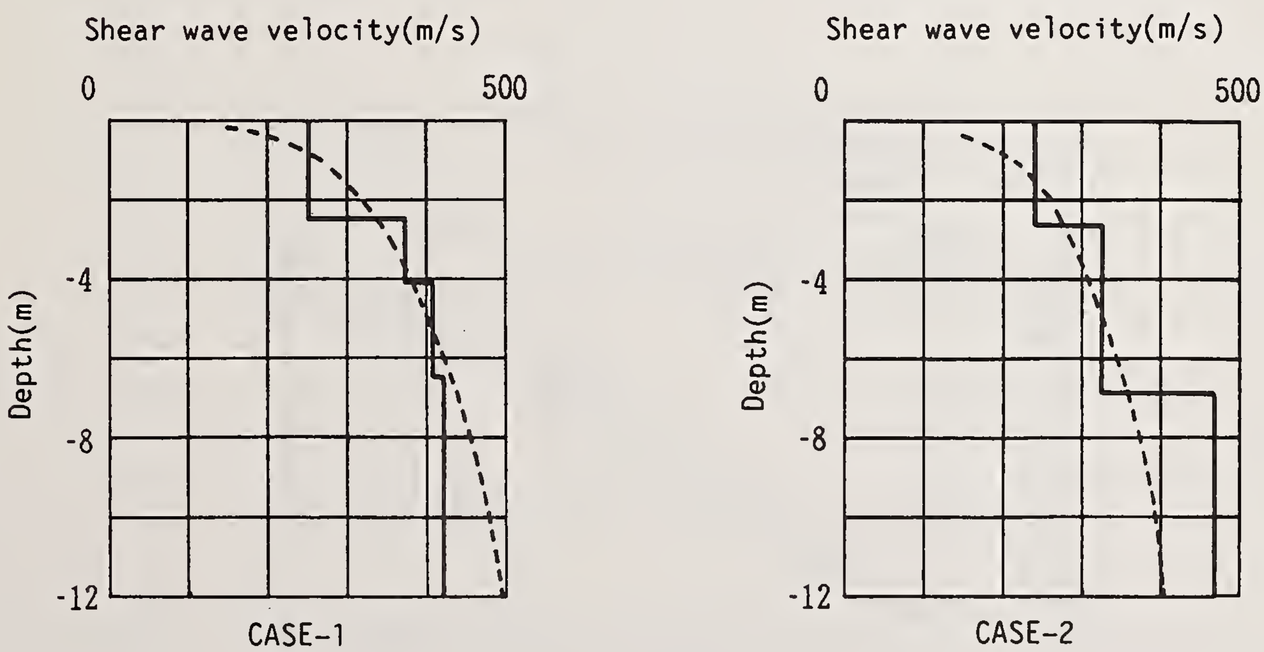


Fig.11 Distribution of the shear wave velocity with depth.

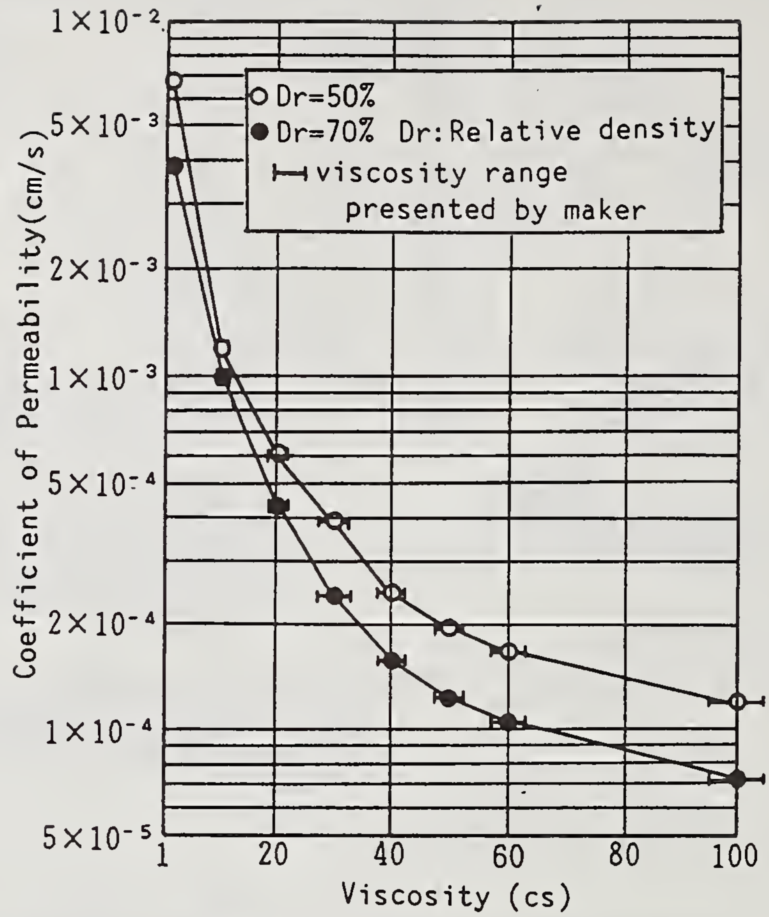


Fig.12 Relation between the viscosity and the coefficient of permeability.

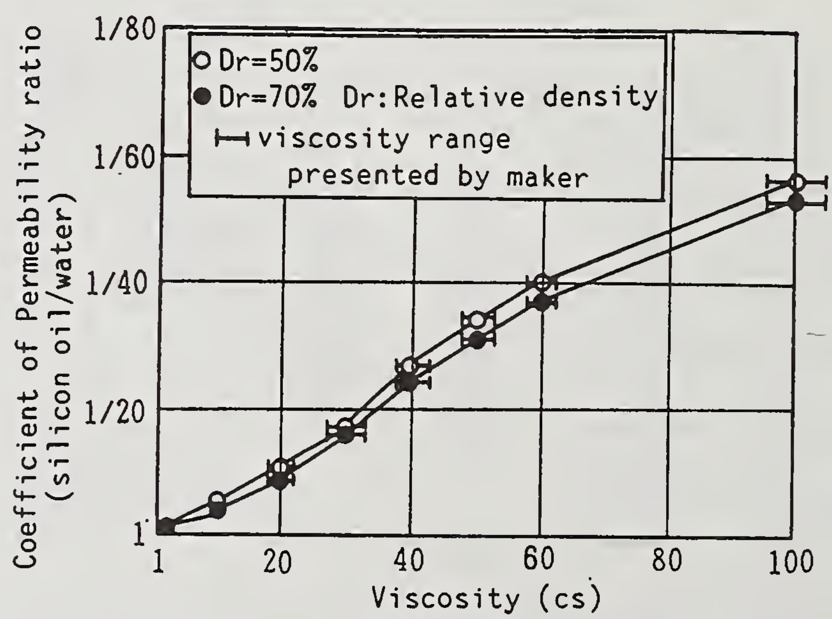


Fig.13 Permeability reduction ratio of the silicon oil compared with the water.

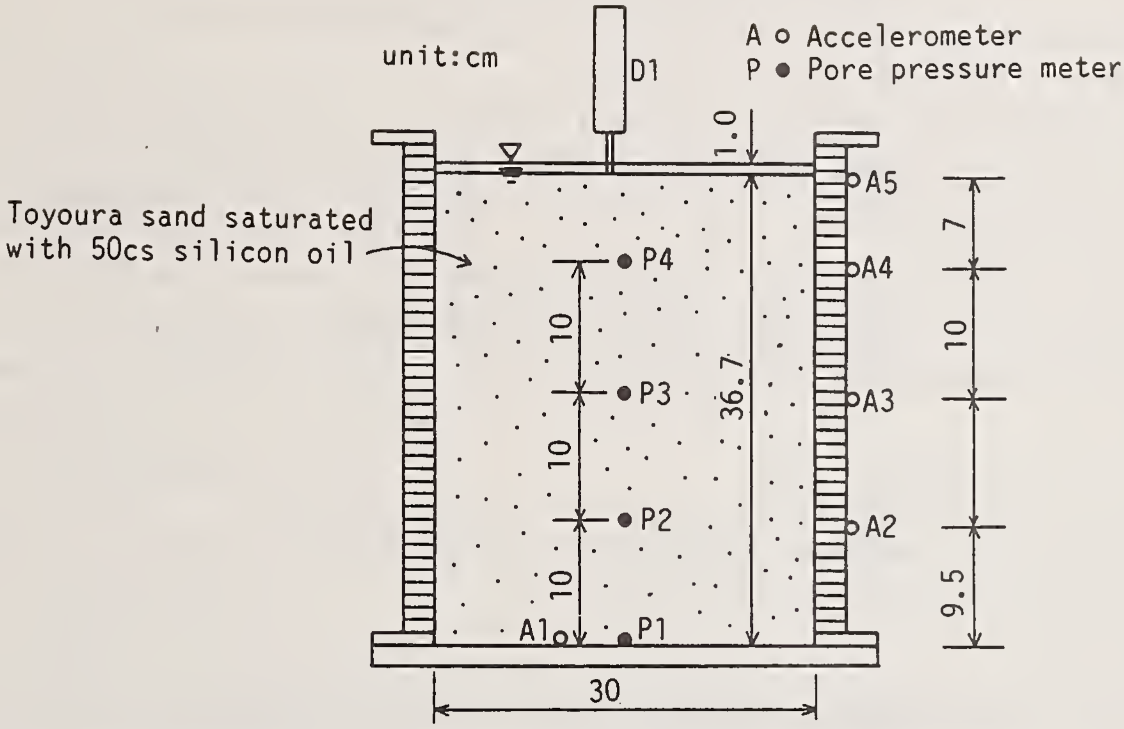


Fig.14 Cross section of the liquefaction model using stacked rings.

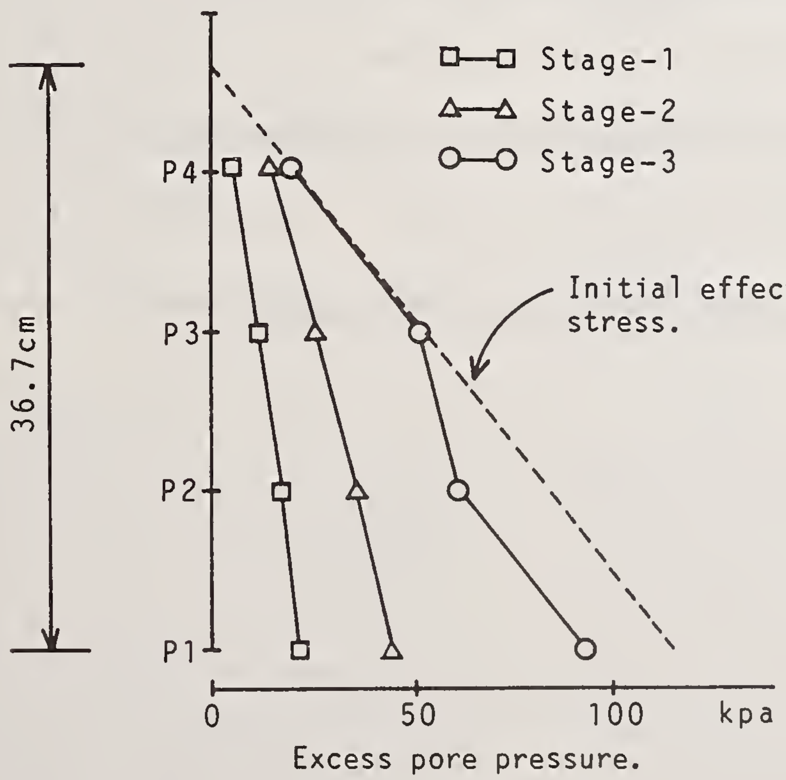


Fig.16 Distribution of the maximum excess pore pressure

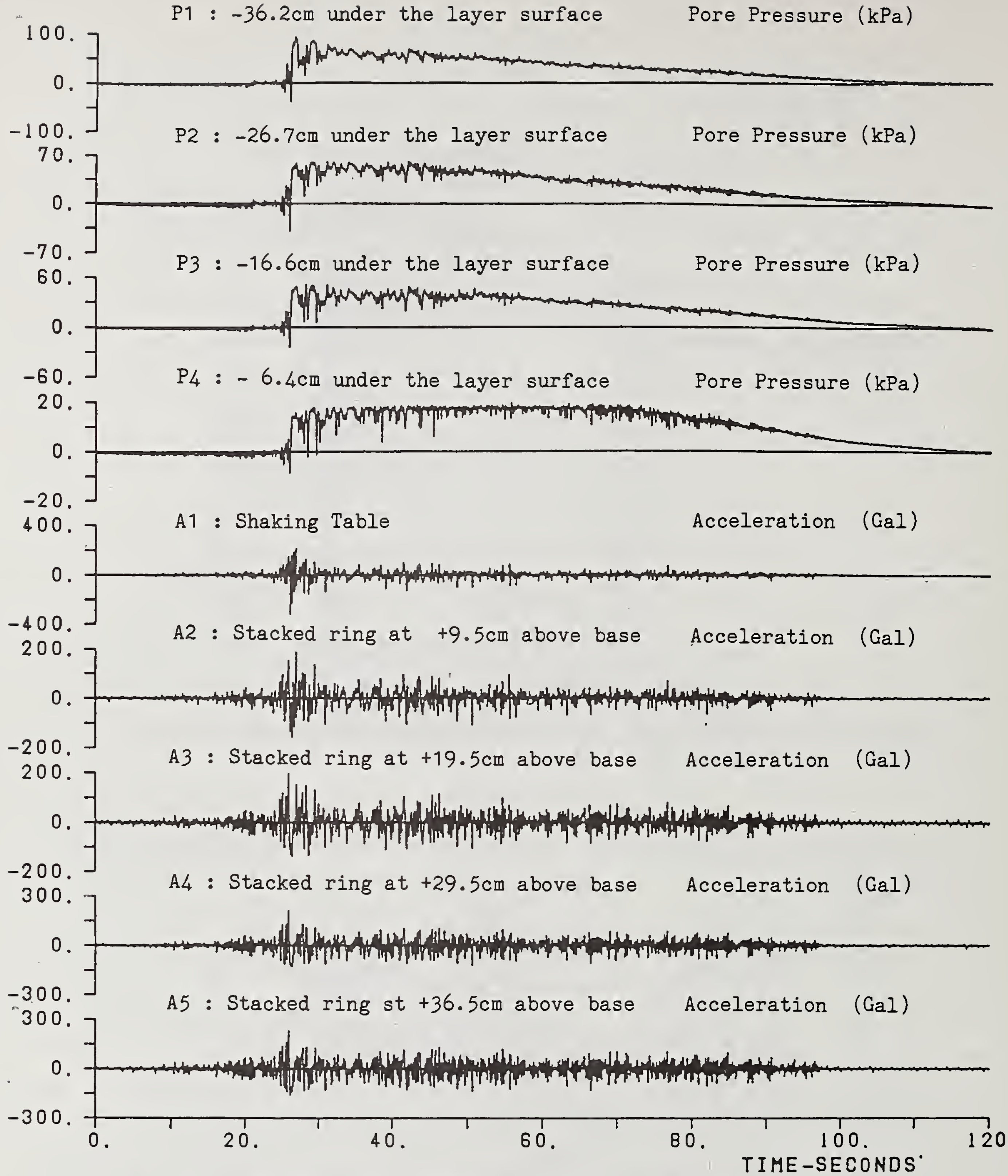


Fig.15 Time histories of stage-3 in prototype scale.

Modeling of Earthquake Induced Pore Pressure Behavior

by

Richard H. Ledbetter [1]

ABSTRACT

Centrifuge model testing with simulated earthquake excitation can provide an extensive data base for the detailed study of earthquake-induced prototype behavior, including pore fluid pressure response. In centrifuged models, prototype stresses can be produced at points in the model corresponding to those in the full scale structure by creating an artificial gravity field in the model. Each soil element in a centrifuged model may thus be expected to undergo the same response history as corresponding elements in a prototype for a given earthquake excitation. Centrifuge model tests for the study of earthquake-induced behavior have been conducted on the large geotechnical centrifuge at Cambridge University, England. The study of actual field-measured pore pressure behavior and response under earthquake loading compared to the centrifuge model tests shows that the pore pressure behavior in the centrifuge tests is consistent with the field measured behavior.

KEYWORDS: Centrifuge; earthquake; pore pressure.

1. INTRODUCTION

Study is indicating the same consistent behavior of actual pore fluid pressure under earthquake loading and physical tests yielding equivalent prototype response to earthquake excitation. Direct instrument measurements of excess pore water pressure induced by earthquakes have been made; (1) Ikehara (1970) reported measurements below a railroad embankment, (2) Ikuta et al. (1979) reported measurements near the basement walls of a building, (3) Ishihara (1981) measured pore water pressure rise at two depths within the artificial island of Ohgishima which felt three earthquakes, (4) Ishihara et al. (1981) measured excess pore water pressure at two depths within the artificial island of Owi, (5) Harp et al. (1984) reported measurements in lake shore sediments, and (6) Holzer et al. (1988) reported measurements from the Wildlife liquefaction array.

Centrifuge model testing with simulated earthquake excitation provides an extensive data base for the detailed study of earthquake-induced prototype behavior, including pore fluid pressure response. In centrifuged models, equivalent prototype stresses can be produced at points in the model corresponding to those in the full scale structure by creating an artificial gravity field in the model (Schofield, 1981). The ability to induce prototype stresses in a model is important because soil properties and responses are dependent on effective stresses. Each soil element in a centrifuged model may be expected to undergo the same response history as corresponding elements in a prototype for a given earthquake excitation.

In a centrifuged model, N is the model scale factor and the model is subjected to inertial accelerations of Ng, where g is the acceleration due to earth's gravity. For a saturated soil model subjected to an earthquake excitation, the behavior will depend on both the interaction between inertial effects and the local

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generation and dissipation of pore fluid pressures. The time scales for inertial effects, N, and for diffusion, N^2, can be brought together by slowing the diffusion process by using a more viscous pore fluid (Schofield, 1988). The time scales will be identical if a fluid of viscosity N times greater than water is used for modelling at Ng inertial acceleration.

The United States Nuclear Regulatory Commission and United States Army Corps of Engineers sponsored a series of centrifuge model tests to provide data for the study of earthquake induced settlements/deformations and liquefaction. The tests were conducted on the large geotechnical centrifuge at Cambridge University, England. Details of the Cambridge centrifuge and associated procedures for simulated earthquake testing have been described by Schofield (1981).

This paper presents typical centrifuge model pore fluid pressure response to earthquake excitation which is consistent with field measurements and indicates that pore fluid pressures respond to the peak motions of excitation. The test data and field measurements indicate that pore pressures dissipate rapidly and low permeability confining layers and/or low permeability liquefied soil may be necessary to sustain pore pressures at an elevated state in a liquefied soil mass.

2. FIELD PORE PRESSURE BEHAVIOR

Figure 1 (National Research Council, 1985) shows the soil profile at Owi Island, Japan, where the pore pressures in Figure 2 (National Research Council, 1985) were measured by Ishihara (1981) at the 6 and 14 meter depths during the Mid-Chiba earthquake in 1980. The measurements were in silty fine sand. An accelerograph measured surface motion. Note that: (1) the pore pressures responded immediately to the peak motion, they did not accumulate from the start of the earthquake, and they started dissipating immediately after the peak motion (they did not hold or continue to increase with the earthquake continued motion), (2) the pore pressure at the 14 meter depth did not dissipate as rapidly as that at the 6 meter depth, and (3) the soil profile shows clay layers above and below the 14 meter depth. The less rapid dissipation at the 14 meter depth illustrates the effect of the clay layers bounding the zone and that low permeability confining layers may be necessary to sustain pore pressures.

Figure 3 (Harp et al. 1984) shows pore pressure measured at a 1.3 meter depth in a saturated gravelly sand at the shore of Convict Lake, California, during the Mammoth Lakes earthquakes of 1980. The recorded surface acceleration is superimposed on the pore pressure record illustrating that the pore pressure followed the earthquake motions; peaking with the peak motion and not accumulating with continued motion. Low permeability layers would have been necessary to confine the gravelly sand in order for pore pressures to have been sustained. The site liquefied during the main shock and underwent lateral spreading; the pore pressure transducer was installed after the main shock and recorded the aftershock sequence.

3. LABORATORY PORE PRESSURE BEHAVIOR

3.1 Simple Shear Test

Figures 4 thru 6 (Ishihara et al. 1988) show pore pressure behavior of loose, medium, and dense Fuji river sand in undrained cyclic simple shear laboratory tests using irregular mutually perpendicular motions from three earthquakes recorded in Japan. Ishihara and Nagase (1988) were studying the effects of irregular and multi-directional loading on soil during earthquakes. Shown in each figure are the surface recorded earthquake components of motion that were used as test input and the shear strains and pore pressures for two stress ratios. Of note in these figures is that the pore pressures respond to the peak motions and do not significantly accumulate. This pore pressure behavior is consistent with field measured responses. The pore pressures in these tests are held at the levels induced by the peak motions because the tests are undrained, thereby locking in the pore pressures and not allowing them to dissipate during or after the earthquake loading, as might occur under field conditions unless there were confining low permeability layers.

3.2 Centrifuge Test

Figure 7 shows the schematics of a typical centrifuge physical model test illustrating an embedded structure, displacement transducers (LVDT), accelerometers (ACC), and pore pressure transducers (PPT). Earthquake simulation motion is input at the model base and transmitted through the model, primarily by shear stresses but also by normal stresses due to bending modes. De-aired blended silicone oil with a viscosity of N times the prototype viscosity was used as the pore fluid to model the drainage conditions in the prototype during the earthquake. Tests by Eyton (1982) have shown that the stress-strain behavior of fine sand is not changed when silicone oil is substituted for water as a pore fluid.

Figures 8 and 9 show the typical pore pressure response character in centrifuge tests with homogeneous models of Leighton Buzzard and Banding sand, ranging in relative density from 45 to 70 percent. The pore pressure records have been filtered and smoothed so that the main changes of the mean response can be compared to the earthquake acceleration input records. Major changes in the pore pressures were in response to the peak motion stages of the records; they stay at about the same level or start dissipating unless a larger motion peak occurs, do not accumulate with cyclic loading, and dissipate after the earthquake. Figure 10 shows the dissipation behavior of pore pressures. The pore pressure character in the centrifuge is consistent with field behavior. These sands were very fine,with nominal grain sizes of 0.1 and 0.19 mm for the Leighton Buzzard and Banding sands respectively, which is why they tended to hold the pore pressures during earthquake loading.

4. CONCLUSIONS

The study of actual field measured pore pressure behavior and response under earthquake loading compared to the laboratory test results of special multi-directional loading simple shear and centrifuge model tests is indicating that pore pressure behavior in centrifuge tests is consistent with field measured behavior. Centrifuge earthquake simulation testing can provide prototype truth data concerning earthquake-induced response in soils and can provide an extensive data base for detailed study, verification of analysis techniques, and to fill the voids in the knowledge of pore pressure behavior under earthquake excitation.

5. ACKNOWLEDGMENTS

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6. REFERENCES

1. Eyton, D.G.P. 1982. "Triaxial Tests on Sand With Viscous Pore Fluid," Part 2, Project Report, Engineering Department, Cambridge University, Cambridge, England.
2. Harp, E.L., Sarmiento, J., and Cranswick, E. 1984. "Seismic-Induced Pore-Water Pressure Records From The Mammoth Lakes, California, Earthquake Sequence Of 25 To 27 May 1980," Bulletin of the Seismological Society of America, Vol 74, No. 4.
3. Holzer, T.L., Youd, T.L., and Bennett, M.J. 1988. "In Situ Measurement of Pore Pressure Build-Up During Liquefaction," Wind and Seismic Effects, Proceedings Of The 20th Joint Meeting Of The U.S.-Japan Cooperative Program In Natural Resources Panel On Wind And Seismic Effects, U. S. Department of Commerce, National Institute of Standards and Technology.
4. Ikehara, T. 1970. "Damage to Railway Embankments Due to the Tokachioki Earthquake," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 10.
5. Ikuta, Y., Maruoka, M., Mitoma, T., and Nagano, M. 1979. "Record of Lateral Pressure Taken During Earthquakes," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol.
6. Ishihara, K. 1981. "Measurements of Insitu Pore Water Pressures During Earthquakes," Proceedings of The Int. Conf. on Recent Advances of Geotechnical Engineering and Soil Dynamics, St. Louis Missouri, Vol. I.
7. Ishihara, K., Shimizu, K., Yamada, Y. 1981. "Pore Water Pressures Measured In Sand Deposits During An Earthquake," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 21.
8. Ishihara, K., and Nagase, H. 1988. "Multi-Directional Irregular Loading Tests On Sand," Soil Dynamics and Earthquake Engineering, Vol. 7.
9. National Research Council. 1985. "Liquefaction of Soils During Earthquakes."
10. Schofield, A. N. 1981. "Dynamic and Earthquake Geotechnical Centrifuge Modelling," Proceedings of the Int. Conf. on Recent Advances in Geotechnical Engineering and Soil Dynamics, St. Louis, Missouri, Vol. III.
11. Schofield, A.N. 1988. "Geotechnical Centrifuge Modelling In 1988," CUED/D-SOILS/TR.213, Cambridge University Engineering Department, Cambridge, England.

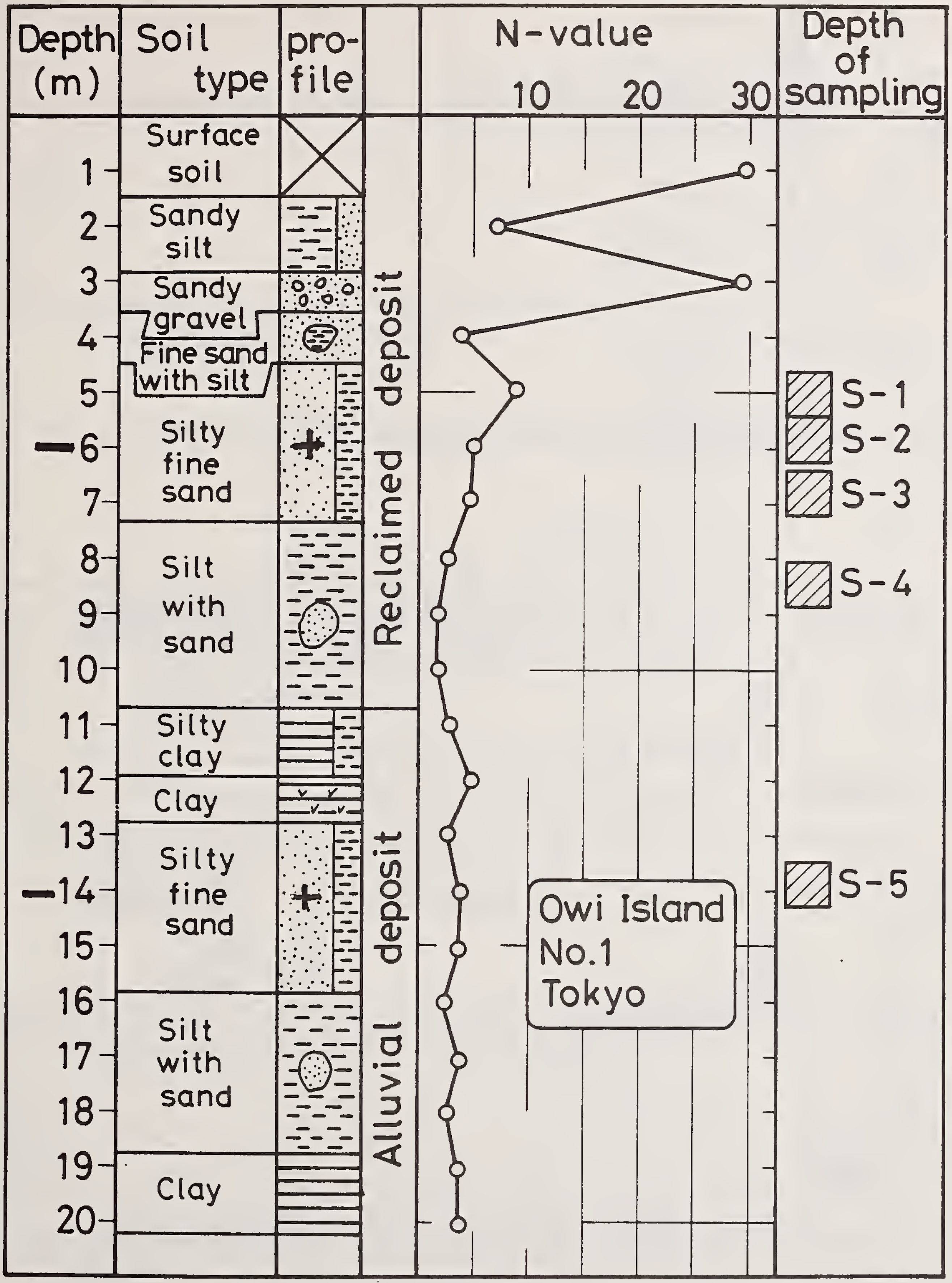


Figure 1. Owi Island soil profile for the Mid-Chiba earthquake, September 25, 1980 (from National Research Council, NRC,1985)

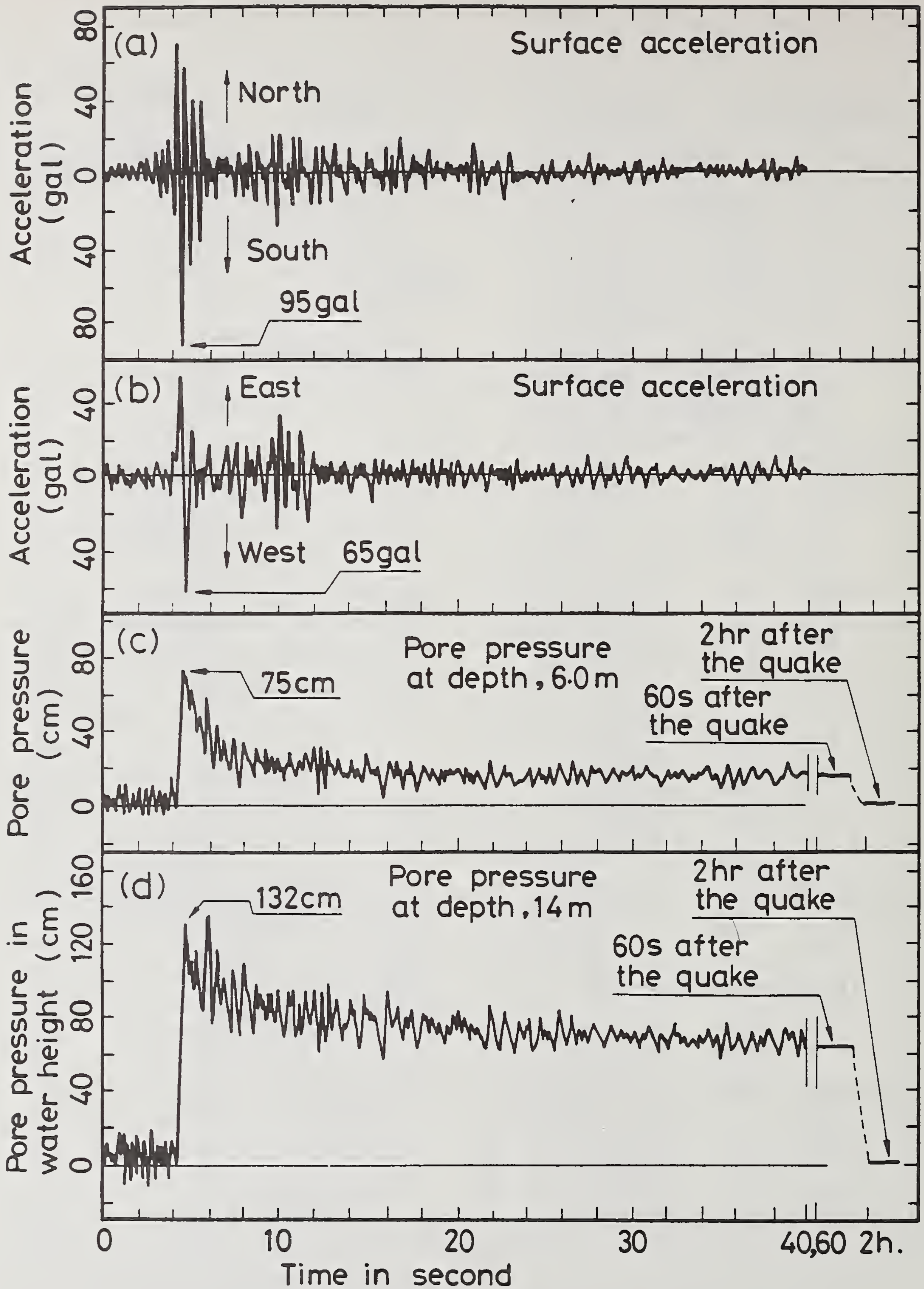


Figure 2. Surface acceleration components and pore water pressure responses at Owi Island, Mid-Chiba earthquake (from NRC, 1985)

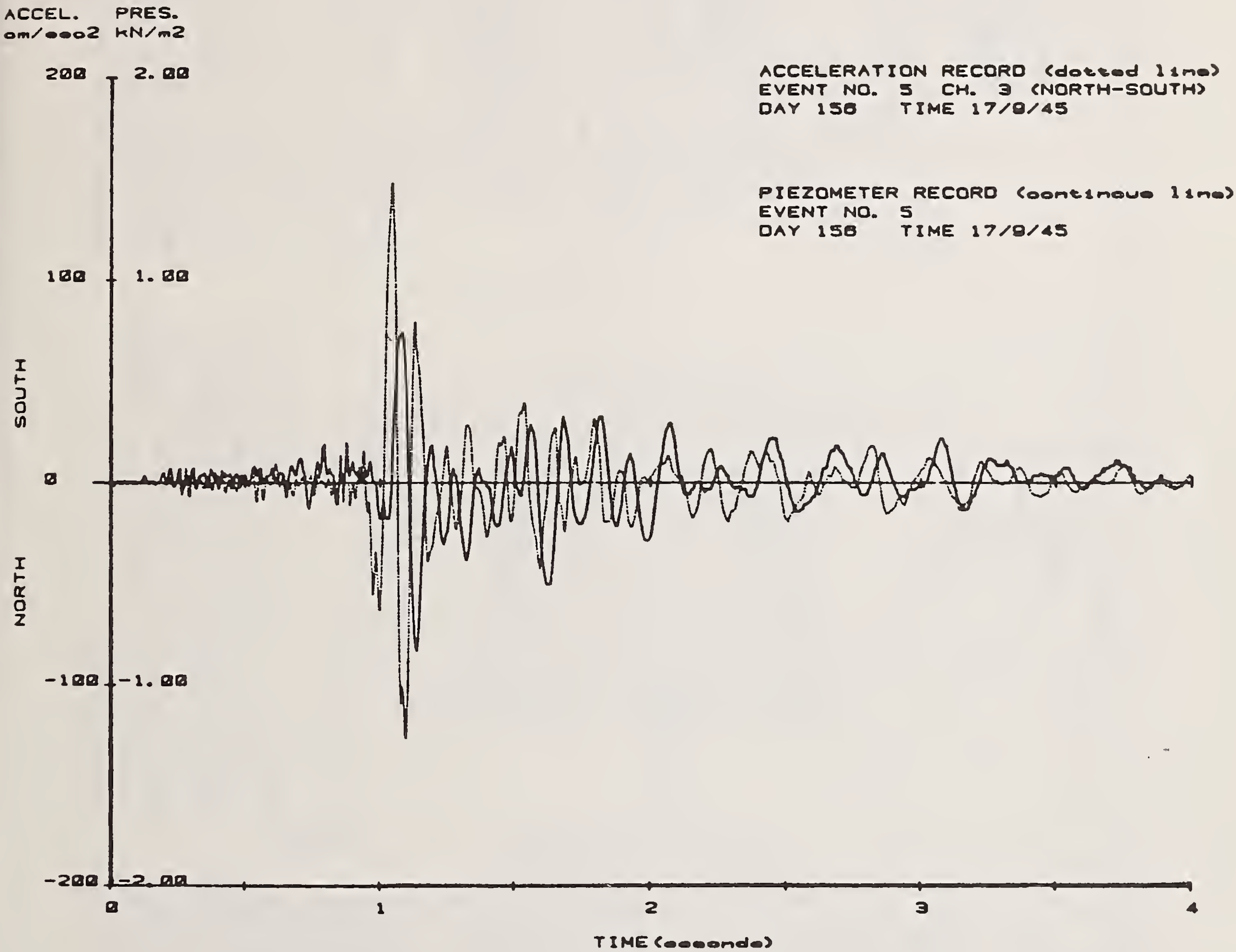


Figure 3. Typical comparison of pore water pressure response and horizontal acceleration, May 1980 Mammoth Lakes earthquake sequence (from Harp et al. 1984)

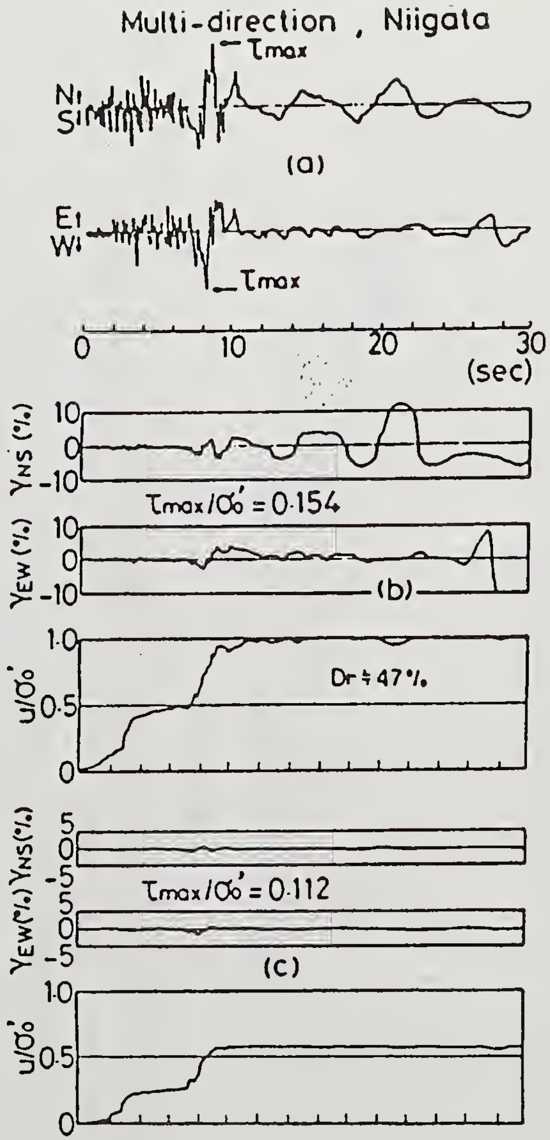


Figure 4. Pore water pressure response of loose Fuji river sand in undrained cyclic simple shear tests with irregular mutually perpendicular motions (form Ishihara et al. 1988)

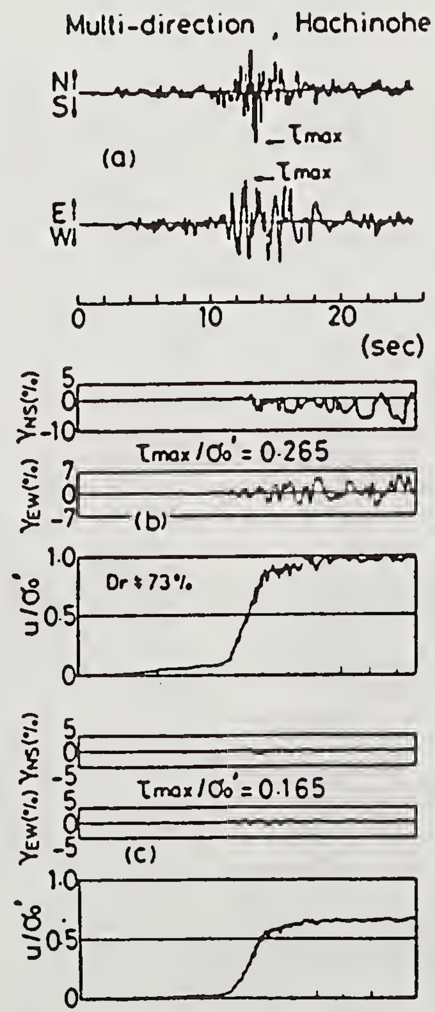


Figure 5. Pore water pressure response of medium dense Fuji river sand in undrained cyclic simple shear tests with irregular mutually perpendicular motions (from Ishihara et al. 1988)

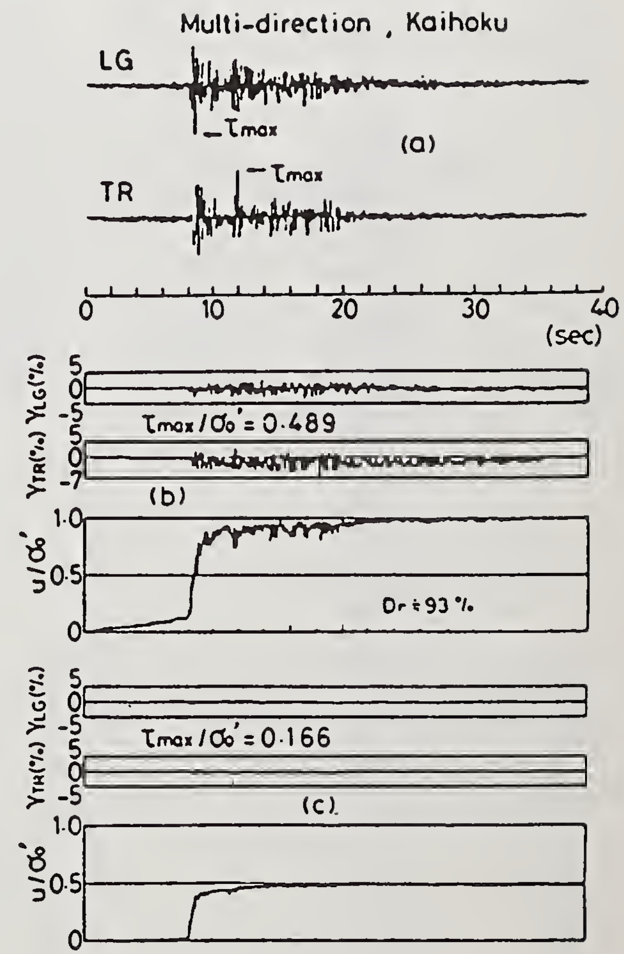
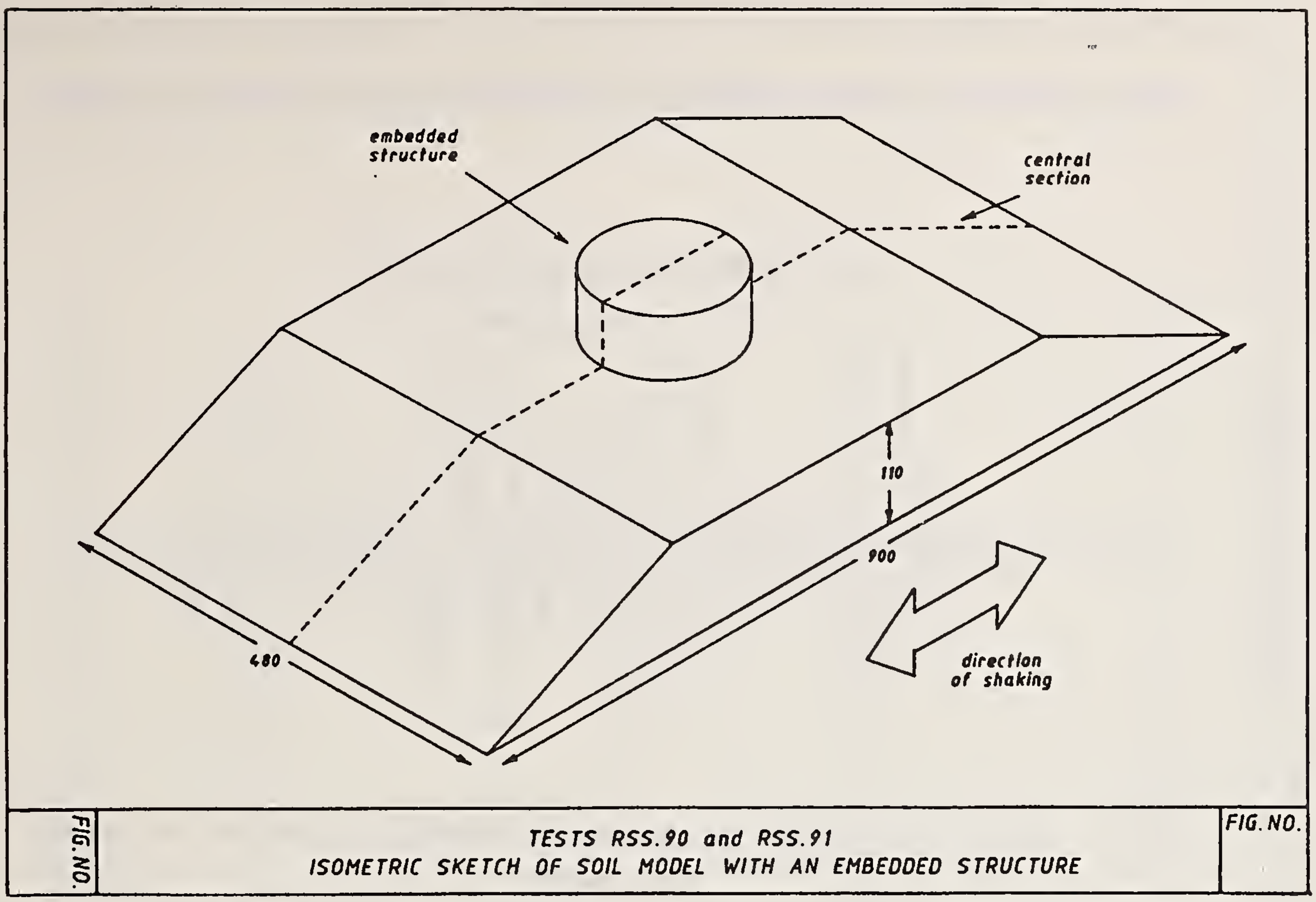
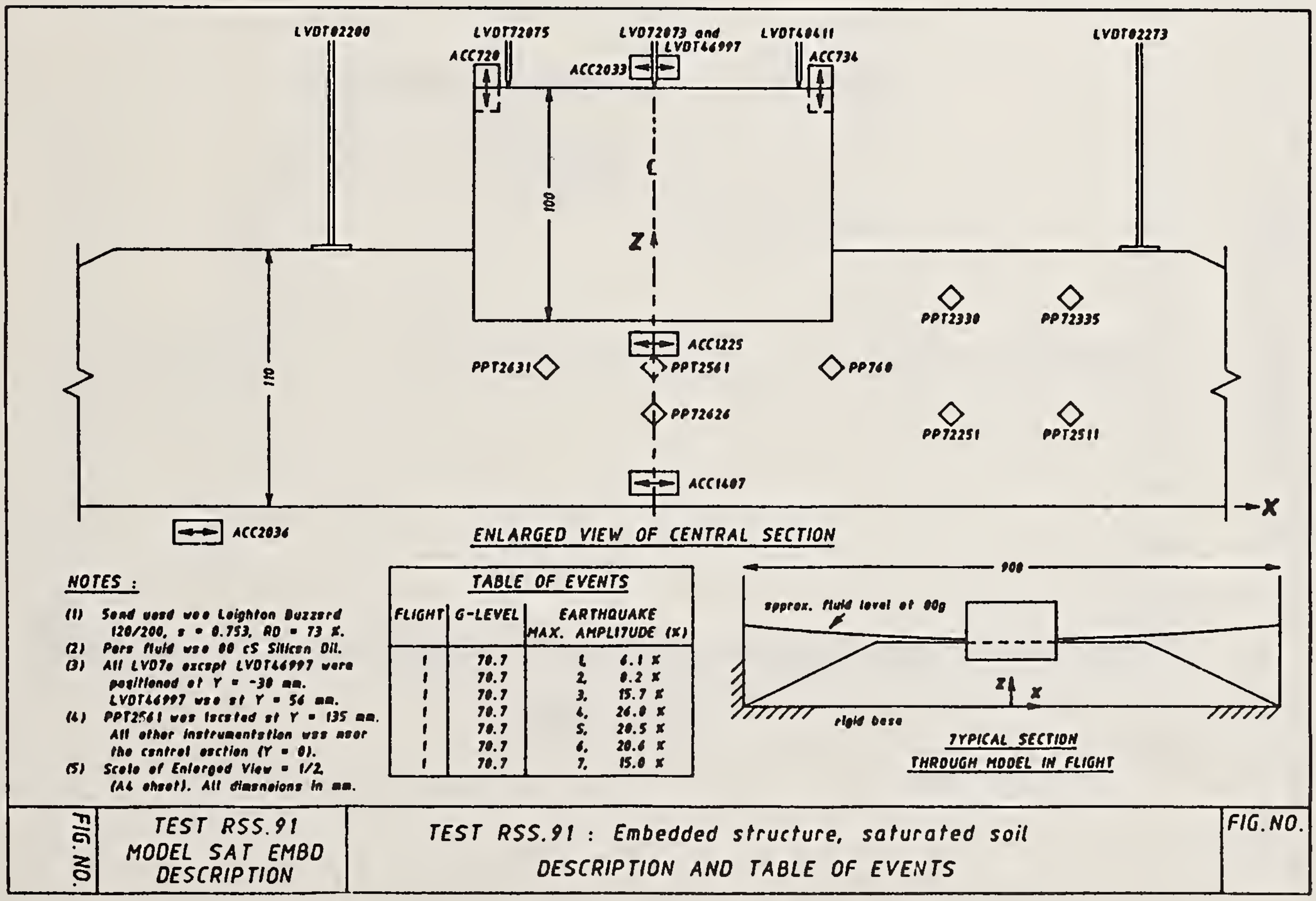


Figure 6. Pore water pressure response of dense Fuji river sand in undrained cyclic simple shear tests with irregular mutually perpendicular motions (from Ishihara et al. 1988)





FLIGHT	G-LEVEL	EARTHQUAKE MAX. AMPLITUDE (%)
1	70.7	1. 6.1 %
1	70.7	2. 8.2 %
1	70.7	3. 15.7 %
1	70.7	4. 26.0 %
1	70.7	5. 20.5 %
1	70.7	6. 20.6 %
1	70.7	7. 15.0 %

Figure 7. Schematics of a typical centrifuge physical model test

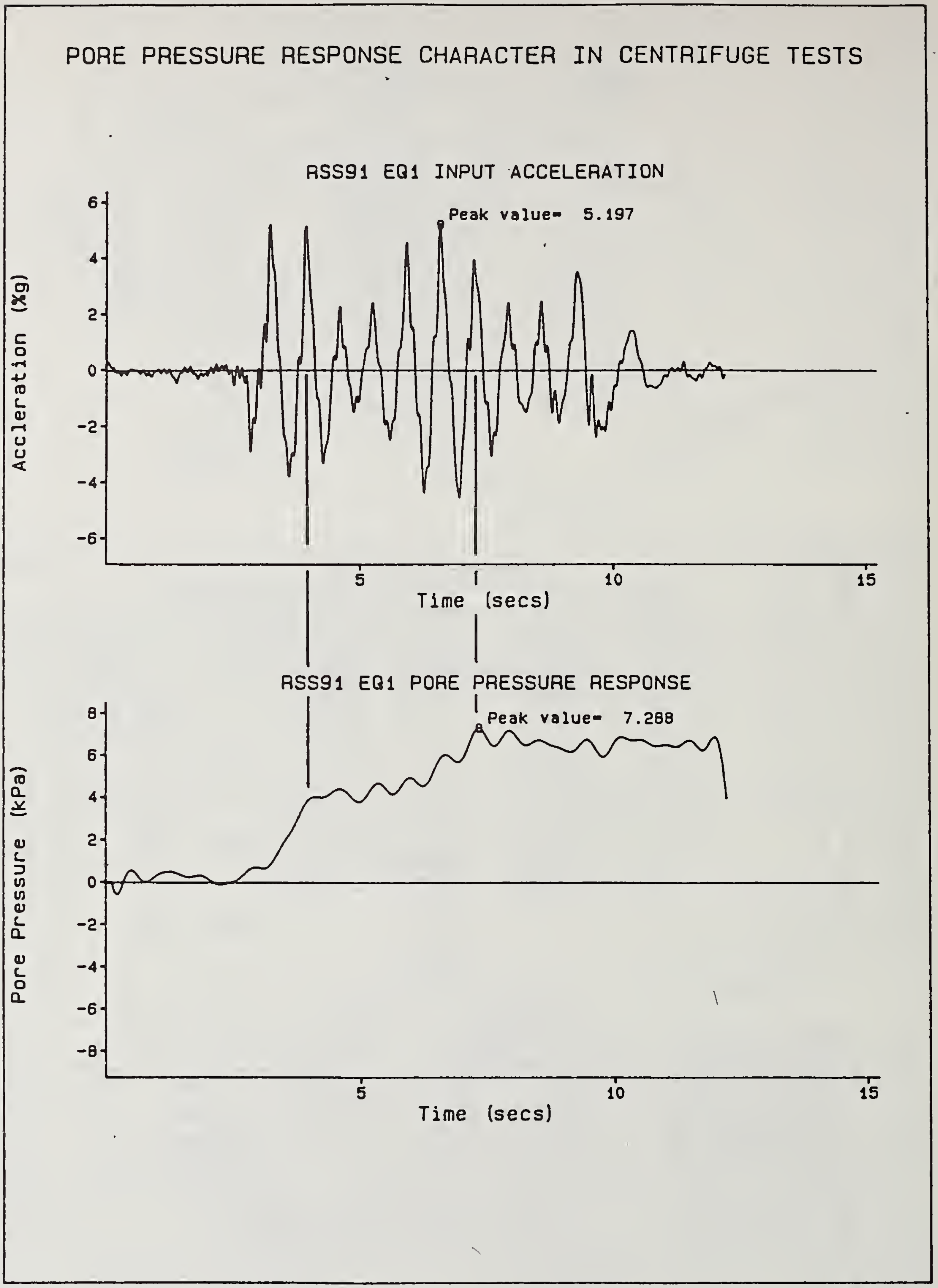


Figure 8. Typical pore fluid pressure response character in centrifuge tests with homogenous sand models

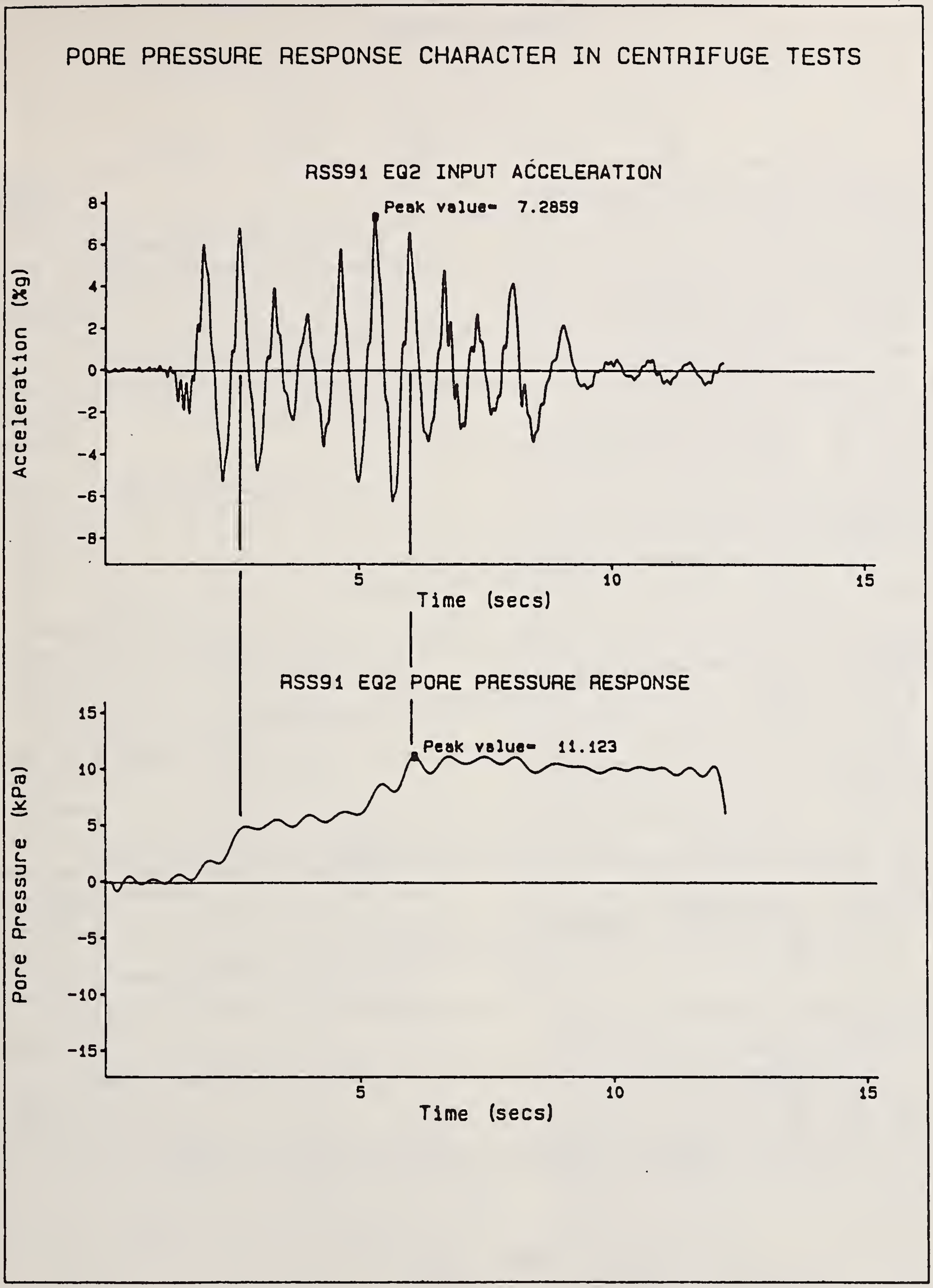


Figure 9. Typical pore fluid pressure response character in centrifuge tests with homogenous sand models

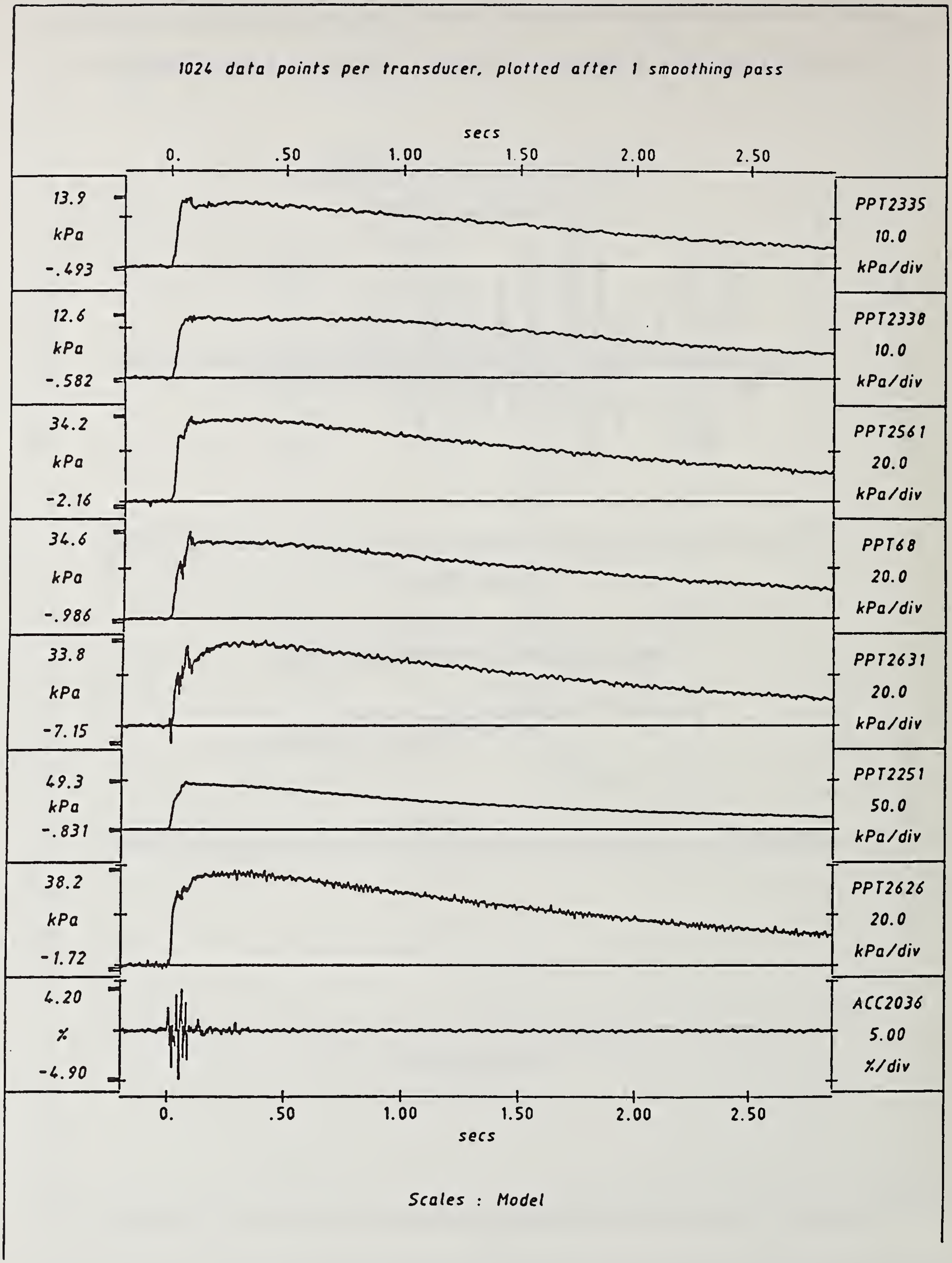


Figure 10. Typical pore fluid pressure dissipation behavior in centrifuge tests with homogenous sand models

Seismic Design Method of Earth Embankment With Seismic Coefficient

BY

Yasuyuki KOGA[1] and Osamu MATSUO[2]

ABSTRACT

The current practices in the seismic design method of earth embankments both in Japan and the U.S. are briefly described, and future research needs are identified. Next, some results of experimental and analytical study which has been conducted to improve the seismic coefficient method as a seismic design method for earth embankments are presented.

KEYWORDS: Seismic stability, Embankment, Seismic coefficient, Liquefaction

1. INTRODUCTION

During the past few decades, there have been major advances in our knowledge of the seismic behavior of earthen embankments, such as embankment dams, river levees, and road embankments, through the continuous efforts of strong earthquake motion observation, experimental and analytical studies. As a result of such efforts, the knowledge of liquefaction of subsuface ground during earthquakes and its associated damages to earth structures has remarkably progressed. Some of the results have already been introduced in design codes in Japan. However, the seismic design method for embankment structures does not seem to have been established but much yet remains to be improved.

This paper first outlines the current seismic design methods for embankment structures in Japan, compares with those in the U.S., and identifies future research needs concerning the seismic design methods. In the latter part of this paper, the study on the seismic coefficient method as a seismic design method for embankment structures is presented.

2. OUTLINE OF SEISMIC DESIGN FOR EMBANKMENTS

2.1 Practices in the Seismic Design Methods in Japan

In this paper, "embankment structures" generally indicate embankment dams, road embankments, and river levees. In Japan, "Dam Design Standard" published in 1957 was the first design code which specifies seismic design for embankments.[1] This design code adopted the pseudostatic analysis method combining the sliding surface method and the seismic coefficient method. Although the code has since been revised several times, there is no substantial change in the seismic design method. The current edition revised in 1978, however, recommends to conduct, in addition to the pseudostatic analysis, dynamic analysis and/or model vibration tests to ensure the seismic integrity of dams.[2]

For road embankments and river levees, the seismic design has not generally been applied. But some codes require to maintain seismic stability exceptionally for high embankments which are founded on soft ground. Accordingly, there recently appear some cases that seismic design or investigation is individually applied to embankments in the metropolitan area.

Table 1. summarizes current design codes in Japan which specify the seismic design method for earth structures. This table includes codes for various types of earth structures other than those for embankment structures. It is shown in the table that:

(1) All of the codes adopt pseudostatic analysis method as seismic design procedure.
(2) The analysis is based upon the sliding circle method and the Modified Fellenius method is adopted for the slice calculation.
(3) Design seismic coefficient kh is basically given by Eq.(1), which is commonly used for other engineering structures such as bridges and retaining walls.

$$k_H = \Delta_1 \cdot \Delta_2 \cdot \Delta_3 \cdot k_0 \quad (1)$$

1) Head, Soil Dynamics Division, Construction Equipment and Method Department, Public Works Research Institute, Ministry of Construction, Japan,
2) Senior Researcher, ditto.

where

k_0: standard horizontal seismic coefficient,
Δ_1: seismic zone factor,
Δ_2: ground condition factor,
Δ_3: correction factor depending on type, use, material property of the structure.

The value of the standard horizontal seismic coefficient k_o is empirically given as 0.2 in most codes and 0.15 in some codes. The seismic zone factor takes one of the values of 1.0, 0.85, or 0.7, depending on the seismic activity of the area. The ground condition factor Δ_1 is given as 0.9. 1.0, 1.1, or 1.2, depending on the stiffness or fundamental period of the ground. The factor Δ_3 takes 1.0 as a general principle, but code-1 in Table 1 gives Δ_3= 0.5-0.7 for embankment structures. This reduced value is considered to be due to the material characteristics of earthen embankment that soil is more ductile than steel or concrete. In other words, soil does not necessarily fail at the time of peak stress application but usually fails due to the cyclic loading during earthquakes. The value of Δ_3=0.5-0.7, however, is at most the empirical one.

Since the 1964 Niigata Earthquake, it has been widely recognized that liquefaction of the ground significantly affects the seismic stability of embankments. Hence, most of the codes shown in Table 1 require to assess liquefaction potential of the ground at sites where susceptibility of liquefaction is suspected.

Codes-6, 7, 9 and 11 adopt safety factor calculation formula in which effect of liquefaction is directly considered. In the formula, the shearing strength of saturated soil during earthquakes is given by Eq.(2). According to the equation, shearing strength is reduced due to the build up of porewater pressure, which is usually estimated from the liquefaction resistance factor Fl.

$$\tau_l = c' + (\sigma_0' - \Delta u)\tan\phi' \quad \text{......} \quad (2)$$

where

τ_l: shearing strength of saturated soil,
c', ϕ': shearing strength parameter in terms of effective stress,
σ_0': initial effective confining stress,
Δu: excess porewater pressure due to earthquake loading.

In seismic stability analyses of embankments by most Japanese design codes which consider liquefaction potential of saturated sandy deposits, liquefaction strength of the deposit is given by Eq.(2) using excess porewater pressure. The only exception is code-11, which allows the usage of dynamic strength based on generated strain without using excess porewater pressure in addition to the above mentioned soil strength.[3)]

As described above, it is rather rare at present to perform a seismic design or investigation for road embankments and river levees on the basis of design codes. Meanwhile, there are some cases that a detailed investigation which applies design code method and seismic response analysis method is performed for the particular newly planned and/or existing embankments. For embankment dams, static shear strength parameters are generally used.

Finally, with regard to embankment dams, which are defined as those with a height of 15m or more, the design procedure is based on the seismic coefficient method, not necessarily considering liquefaction-induced failure. This is guaranteed by the fact that the liquefaction susceptibility is precluded by removal of loose sandy strata and well-compaction of the embankment during construction. Further, it is verified that the above design procedure with careful construction would be a conservative one because the dams constructed that way have never been damaged during past earthquakes.

2.2 Practices in the Seismic Design Methods in the United States

In this section, current practices in the seismic design of embankment dams in U.S. are briefly summarized based on materials available to the authors.

The report,"Safety of Dams---Flood and Earthquake Criteria[4)]," which was made up by the Committee on Safety Criteria for Dams established in the National Research Council, provides valuable information on the current practices in U.S. According to the report, it seems that many of the federal agencies responsible for dam construction have been adopting the dynamic analyses, liquefaction potential analyses and permanent deformation analyses instead of pseudostatic analyses based on the seismic coefficient method. In particular, it is noteworthy that the U.S. Army Corps of Engineers rules out the seismic coefficient method. As for the state governmental agencies, not all the states furnish the earthquake design provision. Especially, many of the states located in low seismic activity zones do not prepare the provision or use the conventional seismic coefficient method. In contrast, some states located in high seismic activity zones require dynamic analyses and liquefaction potential assessment in addition to the pseudostatic analyses. In California, the state where the seismic activity is the most active in U.S., there are cases that the post-earthquake stability assessment of an existing dam against flow failure was performed.

2.3 Summary of Seismic Design Practices in Japan and U.S

The current practices in the seismic design of embankment structures both in Japan and U.S. may be summarized as follows:

In Japan

(1) Seismic design provisions are now under preparation even for relatively small-size embankment structures such as river levees, road embankments and the like.
(2) Many codes require liquefaction potential analyses.
(3) Pseudostatic method with circular sliding method and seismic coefficient is mainly used.
(4) In addition to the conventional pseudostatic method, the partly improved method, which takes loss of shear strength due to the build up of porewater pressure during earthquakes into account, is adopted in many design codes.

In the United States

(1) Seismic design procedure used is shifting from the conventional pseudostatic method to dynamic analysis, liquefaction potential analysis, and permanent deformation analysis.
(2) Some agencies are adopting such up-to-date seismic design procedures as the post-earthquake stability analysis against flow failure.

2.4 During- and Post-Earthquake Stability Analysis

It has been recognized that there are two types of eartquake-induced failure in terms of time when the failure occurs; i.e., during- and post-earthquake failure. Because these two types of failure have different mechanisms, their failure potential should be evaluated with different procedures. During-earthquake stability analysis includes the pseudo-static analysis and permanent deformation analysis, and is concerned with finite damages. On the other hand, post-earthquake stability analysis includes the pseudo-static analysis using steady-state strength[5)] and is concerned with flow failure. The latter may be required for such embankment structure that the embankment itself or the sandy strata in the underlying ground is of a fairly low density.

In Japan, however, there have been very few cases of flow failure in spite of frequent earthquake occurrence. This probably is caused by the following reasons: that the sandy layer existing in a high seismicity area may have been densified by frequently experiencing earthquakes since its formation; and that most of the embankment dams which have been constructed are fairly well-compacted. On the other hand, in Japan, earthquake damages to road embankments or river levees are considered to have much social impact to the economic activities or potential loss of life, even if the damages are within the moderate range. Thus, the procedure which can predict the extent of damage is needed.

2.5 Future Research Needs

Based on the discussions above, the future research subjects which are needed to establish the reliable seismic design procedure for embankment structures may be as follows:

(1) On the during-earthquake stability evaluation
- (1-1) Improvement of pseudostatic analysis method using seismic coefficient
  - -- Methodology on how to determine the dynamic shear strength of soil and the magnitude of seismic coefficient is particularly important.
- (1-2) Improvement of permanent deformation analysis techniques
  - -- Representation of soil stress-strain relation should be essential both in the conventional technique and in the rigorous technique based on the elasto-plastic approach.

(2) On the post-eathquake stability evaluation

-- Development of stability analysis technique using steady-state-strength of soil
-- Development of assessment technique to identify if steady-state is reached is particularly required.

(3) Development of reliable measurement techniques of soil density and dynamic shear strength

3. STUDY ON THE PSEUDOSTATIC ANALYSIS METHOD WITH SEISMIC COEFFICIENT

In this chapter is presented a part of the study that has been carried out in the PWRI to improve the conventional pseudostatic analysis method. The purposes of this study are:

(1) To clarify how to determine the dynamic shear strength of soil and seismic coefficient to be used in the analysis, and
(2) To correlate the safety factor with the damage extent of the embankment.

Case studies of river levees damaged by an earthquake and model shaking table tests as objective case histories are described in the following sections.

3.1 Shear Strength of Soils for Seismic Stability Analysis

3.1.1 Types of Shear Strength of Soils

Table 2 is a summary of shear strengths for seismic stability analysis of embankments. Among them, following three strengths frequently used are investigated.

$$S_1: \tau_f = \sigma_n{}'_0 \tan\phi' \quad \text{......} \quad (3)$$

$$S_2: \tau_f = (\sigma_n{}'_0 - u_e)\cdot\tan\phi' = \sigma_n{}'_0 (1 - u_e/\sigma_n{}'_0)\cdot\tan\phi \quad \text{....} \quad (4)$$

$$S_3: \tau_f = \sigma_n{}'_0 \tan\phi_D = \sigma_n{}'_0 R(N, \tau_f) \quad \text{......} \quad (5)$$

where
- $\sigma_n{}'_0$: initial effective confining stress,
- ϕ': shearing resistance angle,
- ϕ_D: dynamic resistance angle,
- R: dynamic strength ratio,
- u_e: excess porewater pressure due to earthquake loading.

Strength S_3 is called as a dynamic strength, which is determined as a magnitude of dynamic stress to cause a reference strain γ_f under a certain number of loading cycles. Strength S_2 is based on the concept that shearing strength is reduced through buildup of excess porewater pressure due to earthquake loading, whose detail has been described in Section 2.1.

3.1.2 Damage Examples

Analyzed are three sections of river levee which was damaged during the 1983 Nihonkai-chubu Earthquake, liquefaction of the foundation being the cause. These three sections were 100m apart with each other. While the configurations of the embankment sections were the same, the thickness and the liquefaction potential of the sandy layers were different, as shown in Fig.1.

3.1.3 Calculation Method

Stability analyses were conducted using circular sliding method of the modified Fellenius method (Eq.(6)).

$$F_s = \frac{R\Sigma\tau_f \ell}{\Sigma(WR\sin\alpha + kWy)}$$

$$= \frac{R\Sigma\{c'\ell + (W - u\cdot b)\cos\alpha \tan\phi'\}}{\Sigma(WR\sin\alpha + kWy)} \quad \text{......} \quad (6)$$

where
- τ_f : shearing strength of soil,
- c', ϕ': shearing strength parameters in terms of effective stress,
- W: weight of slice,
- k: seismic coefficient,
- l: length of sliding arc,
- u: porewater pressure,
- b: width of slice,
- R: radius of sliding circular arc,
- α: gradient of sliding arc to horizontal,
- y: vertical distance between gravity center of slice and center of sliding circle.

(refer to Fig.2)

3.1.4 Stability Analysis Results

In Fig.3 are the stability analysis results which show the relationship between seismic coefficient k and safety factor Fs for three sections. In the figure, S_2 indicates results of calculation without seismic coefficient, and S_2' vice versa. Calculation results indicate that the safety factor Fs depends much on the strength used. In both cases of S_2 and S_2', Fs decreases remarkably when k exceeds a certain value, which reflects that the excess porewater pressure is very sensitive to the seismic coefficient k.

Moreover, estimated maximum response accelerations from seismic response analyses are shown as a_{max} in the figure. Following formula was used to evaluate the seismic coefficient k.

$$k = C_r(N_c) \times \frac{a_{max}}{g} \quad \text{.............} (7)$$

where

$Cr(Nc)$: equivalence coefficient, i.e, coefficient to convert response acceleration history into seismic coefficient.

g: acceleration of gravity.

Equivalence coefficient Cr was evaluated as in the figure by means of the cumulative damage concept. Seismic coefficients calculated in this manner are plotted in the same figure. The validity of this method will be discussed in the next section.

As a result, the safety factor calculated for strength S_1 was fairly large, which cannot explain the fact that respective embankments were damaged during the earthquake. Fig.4 shows a relationship of safety factor Fs for strengths S_2, S_2', S_3 and observed settlements of the embankments. All the relations appear reasonable in that the settlement is large when Fs is small. However, Fs is too small in the cases where S_2 and S_2' were used, considering the fact that some embankments in the vicinity of these three sections were hardly damaged. Therefore, it is considered that Fs=1.0 cannot be a threshold of stability when using S_2 and S_2'.

On the other hand, it seems reasonable to use S_3, as the settlement of embankment of 6 m high was approximately 50 cm corresponding to Fs=1.0.

3.2 Investigation of Seismic Coefficient

3.2.1 Equivalent Seismic Coefficient

Consider first an embankment in Fig.5. The response acceleration a of sliding block B and shear stress τ at reference point P on a sliding surface during a random seismic motion will show such time histories as (a-1) and (b-1) in Fig.6. If the embankment is shaken as a rigid body, both time histories a-t, τ-t are similar. Next, consider a damage to this embankment against this seismic motion. Another seismic motion with a cyclic time history can cause a damage of the same amount to the embankment. The time histories of acceleration and shear stress are shown in (a-2) and (b-2). As the damage to an embankment is essentially caused by the deformation of soil elements of the structure, the stress time history in (b-1) is equivalent to that with amplitude τ_{eq} and number of cycles N_{eq} in (b-2). The relation is governed by the fatigue characteristics of a soil. The amplitude ratio Cr between (b-1) and (b-2) are obtained as below.

$$C_r = \frac{\tau_{eq}}{\tau_{max}} \quad \text{.............} (8)$$

On the other hand, according to the assumption of rigid body motion, the ratio of acceleration to shearing stress remains constant as below.

$$\frac{a_{max}}{\tau_{max}} = \frac{a_{eq}}{\tau_{eq}} \quad \text{.............} (9)$$

Therefore, we get

$$\frac{a_{eq}}{a_{max}} = \frac{\tau_{eq}}{\tau_{max}} = C_r \quad \text{.............} (10)$$

That is, random response acceleration time history (a-1) has been converted to cyclic response acceleration (a-2) by use of equivalence coefficient Cr which is determined from fatigue characteristics of soils and equivalent number of cycles.

Moreover cyclic response can be easily converted to static problem then shearing stress τ_{eq} is generated at point P on sliding surface by applying seismic coefficient $k_{eq}=a_{eq}/g$ to the sliding block B in Fig.5.

Consequently equivalent seismic coefficient is given by the following.

$$k_{eq} = \frac{a_{eq}}{g} = C_r \cdot \frac{a_{max}}{g} \quad \text{.............} (11)$$

3.2.2 Model Shaking Table Test

A series of shaking table tests were conducted for embankment model shown in Fig.7 by use of 6 types of acceleration wave form as given in Table 3. Cases 1, 2, 3 were excited with sinusoidal waves of constant loading cycles, 20, and different frequency. Case 4, 5, 6 were excited with random waves, which are shown in Fig.8.

As a consequence of shaking tests, the relationship between the maximum input acceleration and cumulative crest settlement were obtained as in Fig.9. This figure shows that the magnitude of the input acceleration to cause a certain settlement was larger for random waves than cyclic waves.

Meanwhile stability analyses for the model in Fig.7 using dynamic strength specified with respect to number of loading cycles, Nc=20, and failure strain, γ_f=5%, resulted in Fig.10. This shows that the critical seismic coefficient k_{eq} to give safety factor Fs=1.0 equals to 0.19.

Combining the coefficient and input acceleration shown in Fig.8, equivalence coefficient Cr is obtained by use of Eq.(10). Fig.11 shows the relationship between the crest settlement and Cr thus obtained. It is not decisive how large the critical settlement corresponding to Fs=1.0 should be adopted. If we take that is 5-10 mm, then the equivalence coefficient Cr is approximately 1.0 for cyclic loading and 0.3-0.6 for random loading. In the figure are plotted the equivalence coefficients Cr, which were obtained for Nc=20 by applying the cumulative damage concept to the initial liquefaction as a failure criterion. Both coefficients from the concept and shaking table tests coincide when critical settlement δ_{crit} =1-2mm is adopted. As the shear strain of the soil is small at the time of initial liquefaction, the crest settlement for this stage is supposed also to be small. It can be said that equivalence coefficients agreed fairly well in that sense.

3.3 Conclusions

Applicability of sliding surface method with seismic coefficient as aseismic calculation method of embankment was reviewed on the basis of damage examples and model shaking table tests of embankments.

Following results were obtained:

(1) In a sliding surface method with seismic coefficient, it is adequate to use dynamic strength in terms of total stress based on reference strain as the shear strength of soils.

(2) Seismic coefficient should be obtained by multiplying equivalence coefficient to the maximum acceleration as shown in Eq.(11). Equivalence coefficient can be roughly estimated by use of cumulative damage theory.

(3) The safety factor obtained using above mentioned shear strength of soil and seismic coefficient can be related to the settlement of an embankment.

4. SUMMARY

The current practices in the seismic design method of earth embankments in Japan and the U.S. were reviewed on the basis of the design codes available to the authors. Then the future research needs were identified.

Among various seismic stability analysis methods, the seismic coefficient method, i.e., pseudostatic analysis method, was investigated on the basis of case histories. The basis on how to determine the values of dynamic soil strength and seismic coefficient in the pseudostatic method was obtained from the study.

REFERENCES

1) Japanese Committee on Large Dam : Dam Design Standard, 1957.
2) --- : Dam Design Standard, Second revised edition, 1978.
3) Koga,Y. and Shimazu,A. and Taniguchi,E. : Consideration of Earthquakes in Highway Earthworks, Proc. 18th UJNR Joint Meeting, 1986.
4) Committee on Safety Criteria for Dams, National Research Council : Safety of Dams--Flood and Earthquake Criteria, National Academy Press,1985.
5) Poulos,S.J., Castro,G. and France, J.W. : Liquefaction Evaluation Procedure, Journal of Geotechnical Engineering, Vol.111, No.6,pp.772-792 June,1985.
6) Koga,Y. and Matsuo,O. : Stability Analysis of Seismically Damaged Embankments, Proc. 3rd International Conference on Case Histories in Geotechnical Engineering, 1988.
7) Koga,Y. and Matsuo,O. : Seismic Coefficient in Seismic Stability Analysis of Embankment, Proc. of 9th WCEE, Vol.3, pp.103-108, 1988.

Table 1 Codes for earthquake-resistant design of earth structures in Japan

No.	Agency	Title of code or standard	Objective earth structures	Shape of sliding surface	Slice calculation method	Magnitude of seismic coefficient	Excess pore water pressure	Minimum safety factor	Remarks
1	Public Works Research Institute, Ministry of Construction	New earthquake resistant design method (tentative) (1977)	Embankment dam, River dyke, Road embankment	Circular	Fellenius method (composite circle, modified Fellenius method, Wedge method)	$\Delta_1\ \Delta_2$ ko × (0.5 ~ 0.7) (ko = 0.2)	–	1.2 and over for dam	Dynamic analysis is desirable for a large scale dam Δ_1: seismic zone factor Δ_2: ground condition factor
2	Japanese National Committee on Large Dams	Dam design standard (2nd revised edition, 1978)	Embankment dam of 15 meters high and over	Circular (curve and straight line)	Fellenius method (composite circle, wedge method and modified Fellenius method)	High seismicity zone: 0.15 ~ 0.25 0.12 ~ 0.20 Low seismicity zone: 0.12 ~ 0.20 0.10 ~ 0.15	–	1.2 and over	Ditto
3	Japan Housing Corporation, Industrial Relocation Areas Promotion Corporation, Japan River Association	Revised disaster prevention flood control reservoir technical standards (tentative), (1980)	Embankment dam below 15 meters high	Circular	Fellenius method	High seismicity zone: 0.15 Low seismicity zone: 0.12	–	–	–
4	The Japan Port & Harbor Association (under the auspices of Ports and Harbors Bureau, Ministry of Transport)	Technical standards for port and harbor facilities (1979)	Slope of cohesive soil	Circular	Modified Fellenius method	(Seismic coefficient by area) x (Subsoil coefficient) x (Importance coefficient)	–	(Normally 1.3)	Application of design seismic coefficient gives excessive safety. Liquefaction to be studied seperately.
			Slope of sand and gravel	Straight line	Straight line sliding surface method	Ditto	–	1.0 and over	
5	Agricultural Structure Improvement Bureau, Ministry of Agriculture, Forestry and Fisheries	Design standards for land improvement project plan (dam design) (1981)	Embankment dam of 15 meters and over	Circular	Fellenius method (Wedge method, seismic coefficient circle method)	High seismicity zone: 0.15, 0.12 Low seismicity zone: 0.12, 0.10	–	1.2 and over	Liquefaction to be studied separately.
6	Industrial Location and Environmental Protection Bureau, Ministry of International Trade and Industry, Japan Mining Industry Association	Construction standards for tailings dam (1982)	Tailings dam	Circular	Modified Fellenius method	High seismicity zone: 0.15 Low seismicity zone: 0.12	considered	(1.2 and over)	Checking method is subject to handling of excess pore water pressure. Safety factor will also be subject to change accordingly.
7	Minamitama New Town Development Office, Tokyo Metropolitan Government	Design standards for housing site construction works (1984)	Embankment with slope height 7 meters and over, Cutting with slope height 10 meters and over	Circular	Modified Fellenius method	0.15	considered	Embankment: 1.1 Cutting: 1.2	–
8	The Japan Gas Association	Guideline for buried LNG Tank (1981)	Embankment surrounding buried tank	Circular (straight line, composite sliding surface)	Modified Fellenius method	$k_h = \alpha_1 \alpha_2 \alpha_3 k_o$ (k_o=0.15)	–	1.1	α_3 in design seismic coefficient is a correction factor in underground portion. Liquefaction to be studied seperately.
9	Public Corporation for Housing and Urban Development	Guideline for Seismic design of housing site (tentative) (1984)	Land for housing (embankment, cut slope and natural slope)	Circular Composite	Modified Fellenius method	$k_h = \Delta_1 \cdot \Delta_2 \cdot \Delta_3 \cdot k_o$ (k_o=0.2, Δ_3=0.5)	considered	1.0	
10	Japanese National Railway	Guideline for seismic design of railway embankment (tentative) (1983)	Railway embankment	Circular	(Not specified)	0.2	–	1.2	
11	Japanese Road Association	Manual for Road Earthworks (1986)	Road embankment on soft ground or of high importance	Circular, composite	Modified Fellenius method	$k_h = \nu_1 \cdot \nu_2 \cdot k_{ho}$ (k_{ho}=0.15)	considered	1.0	$\nu_1 = \Delta_1$ $\nu_2 = \Delta_2$

Table 2 Classification of soil strengths

Type	Strength criteria	Drainage condition	Loading pattern **)	Stress path & strength ***)	Time of strength mobilized	Corresponding loading condition	Reference
Static strength (Monotonous loading)	Mobilized maximum stress under a monotonous loading	Drained	a	A	during earthquake (high permeability)	Dead weight + seismic force	JNCOLD (1978)
		Undrained	a	B, B'	during earthquake (impulsive earthquake)	do	do
Dynamic strength (Cyclic loading)	Maximum stress of stress time history to cause a reference strain during a cyclic loading	Undrained	b	C	during earthquake	do	Seed (1966) Ishihara (1980)
	Maximum static strength after a cyclic loading	Incompletely drained*) (Pore pressure during a cyclic loading remains constant)	c	D	during earthquake & soon after earthquake	do Dead weight	JNR (1972) PWRI (1975) HUDC (1984) JMA (1980)
		Undrained*)	c	E	during earthquake & soon after earthquake	Dead weight + Seismic force Dead weight	Castro (1976) Seed (1979) Tokyo Metro. Govnt. (1983) Seed (1979)
		Drained*) (Pore pressure returns to initial pressure)	c	F	long after earthquake	do	

N.B. *) This drainage condition refers to that for a static loading after a cyclic loading. An undrained condition is assumed during a cyclic loading.
**) Refer to Fig.A1
***) Refer to Fig.A2

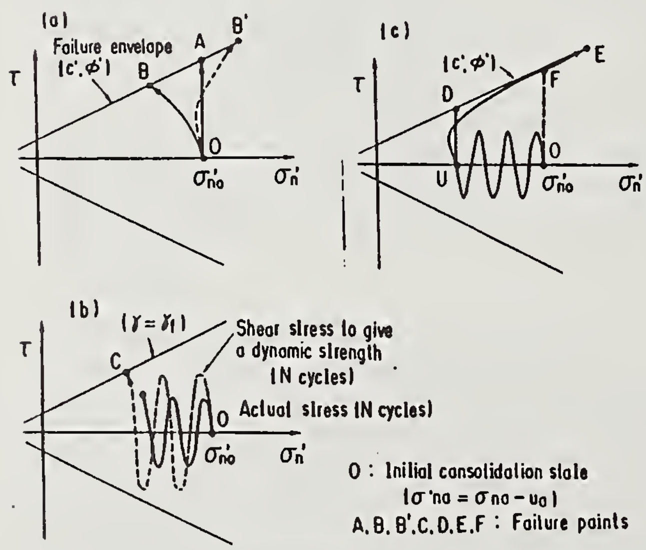


Fig.A2 Stress paths and shear strengths

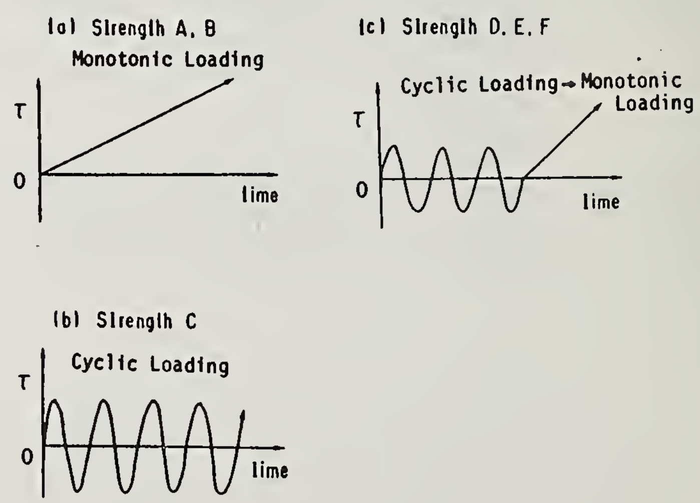
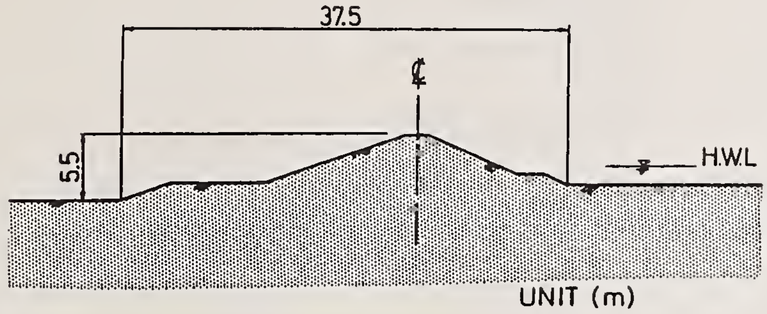


Fig.A1 Loading patterns to obtain shear strength

(a) Shape of the embankment



(b) Soil profiles at three locations

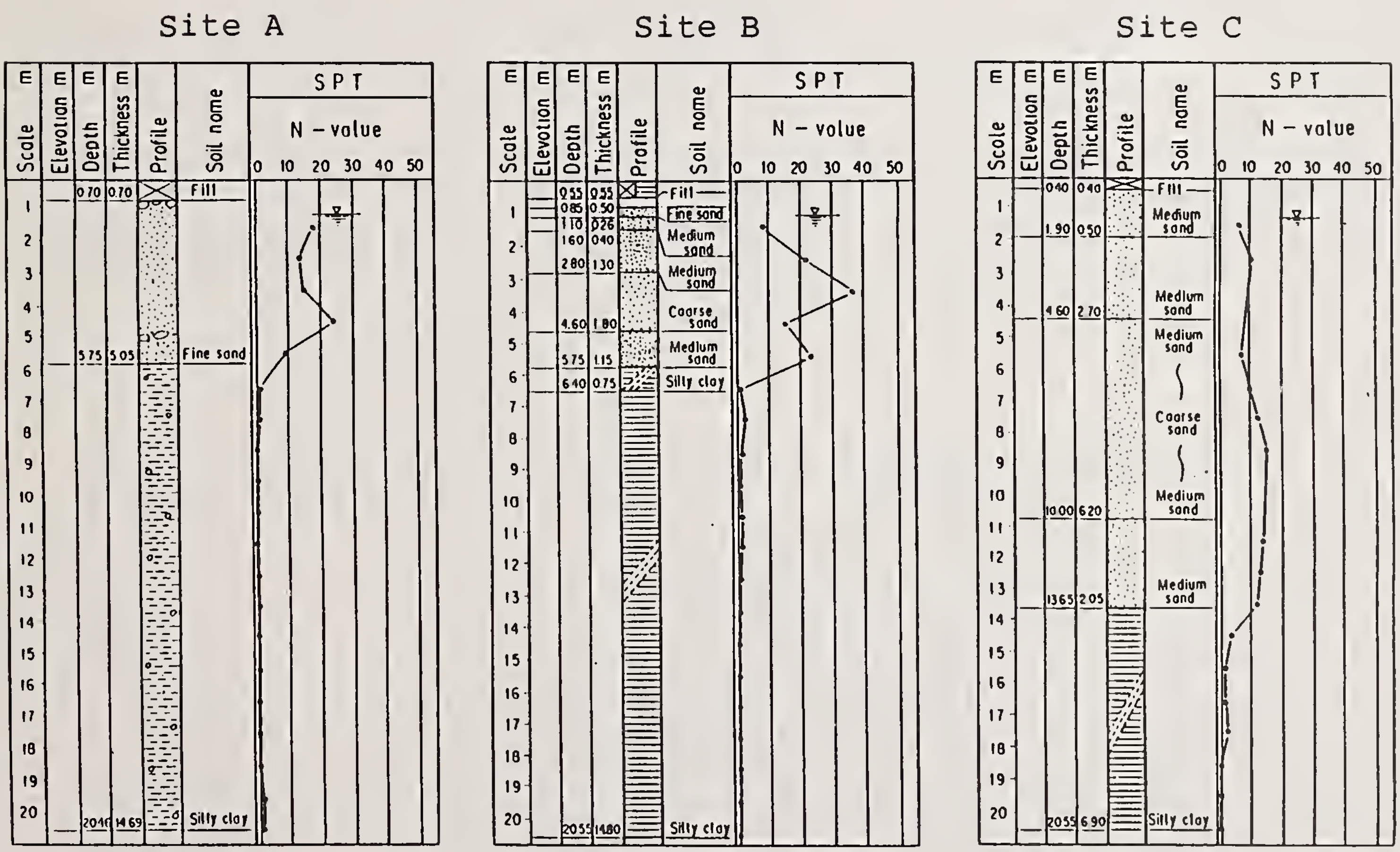


Fig.1 Shape of the embankment and soil profiles of the ground at three locations

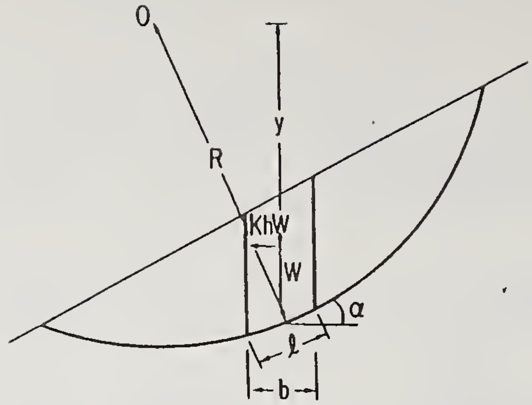


Fig.2 Key sketch for sliding surface method

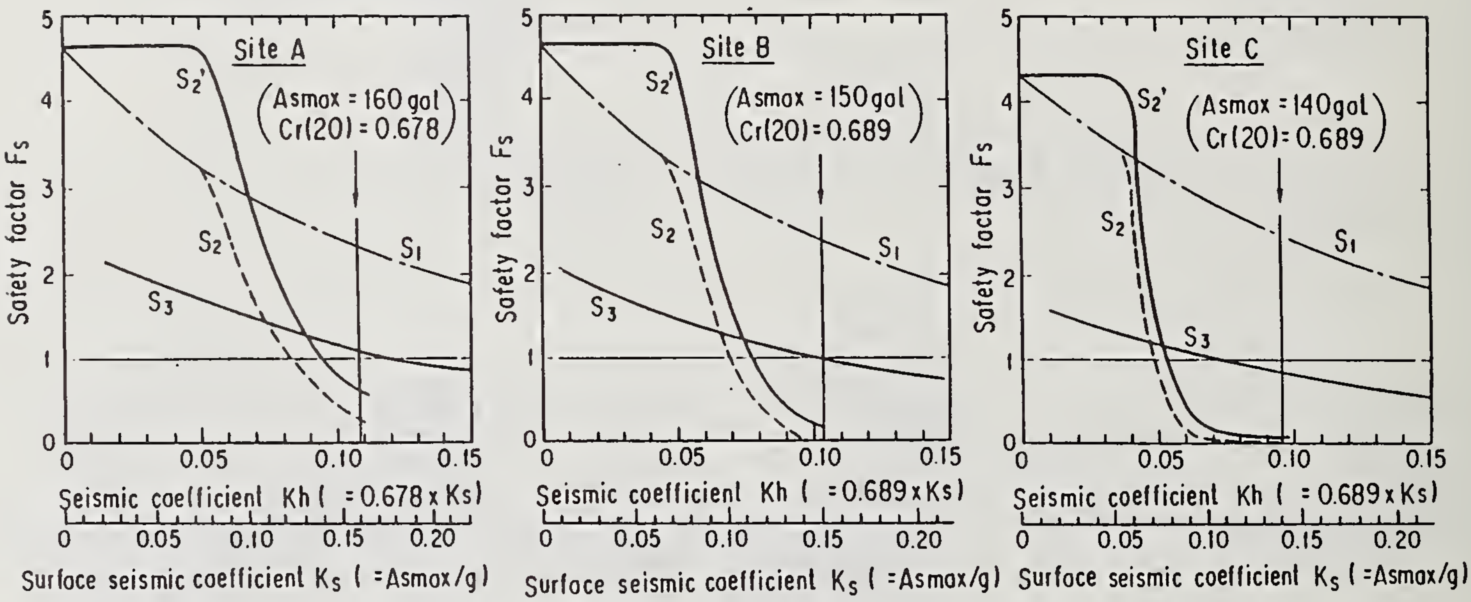


Fig.3 Seismic coefficient vs. safety factor

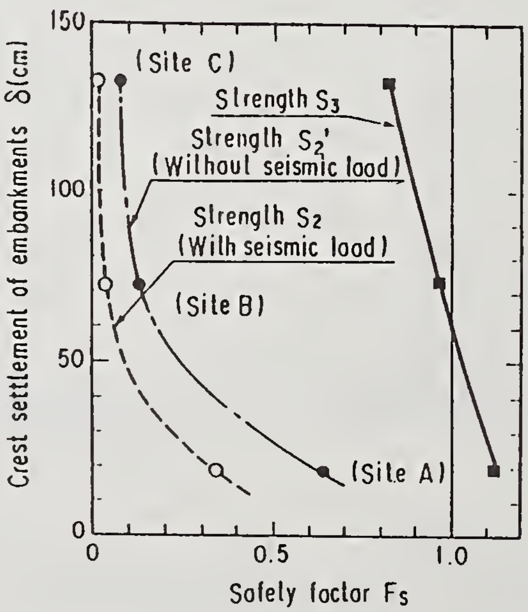


Fig.4 Safety factor vs. crest settlement

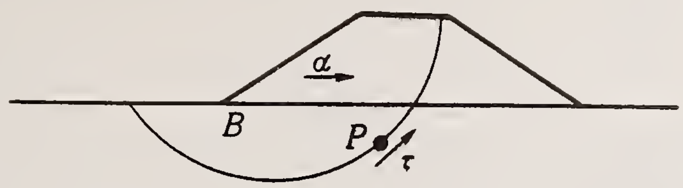


Fig.5 Key sketch for indicating the sliding block B and the point P on arbitrary sliding surface

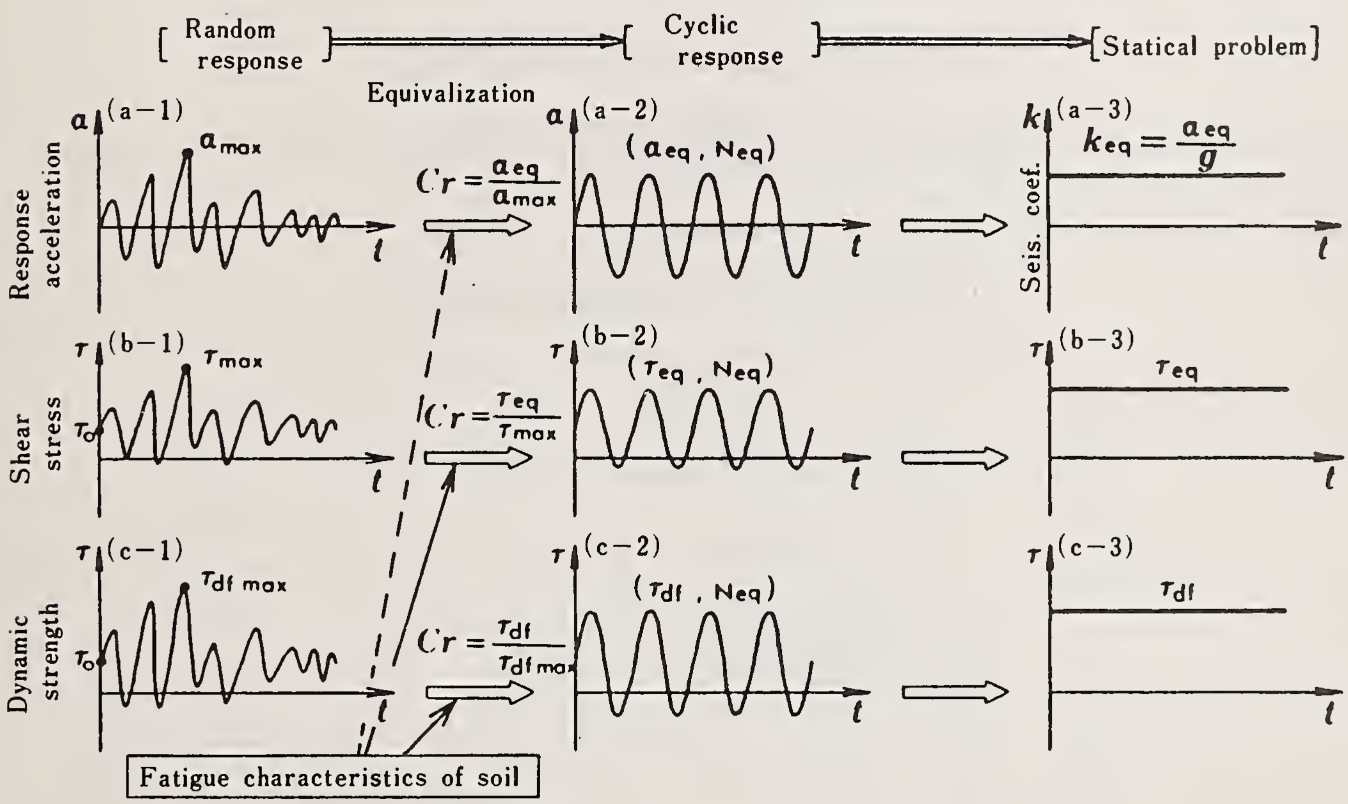


Fig.6 Transformation of irregular response problem to pseudostatic problem

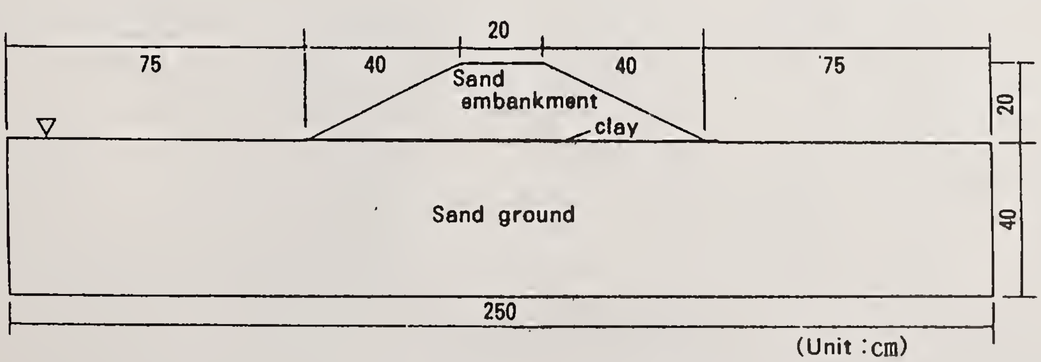


Fig.7 Model configuration

Table 3 Input acceleration wave forms

Case	Wave Form	Frequency f *)	Duration td	f×td	No. of steps	Max. Acc. Range
1	Regular (N=20)	f= 2.5 Hz	8 sec	20	4	117 − 409
2	do (do)	f=10Hz	2 sec	20	10	172 − 542
3	do (do)	f= 5 Hz	4 sec	20	6	90 − 358
4	Irregular (Shock Type)	f_0= 5 Hz	5 sec	25	14	94 − 991
5	do (Vibration Type)	f_0= 5 Hz	5 sec	25	8	112 − 832
6	do (Vibration Type)	f_0= 2.5 Hz	10 sec	25	7	95 − 774

*) f_0= Predominant frequency for irregular wave

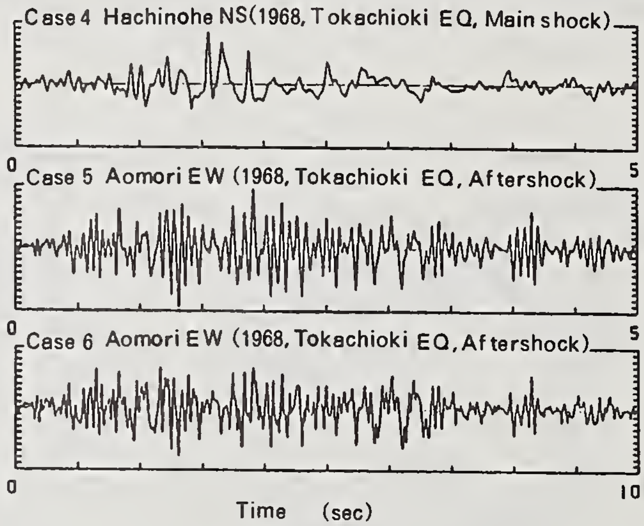


Fig.8 Input acceleration wave form

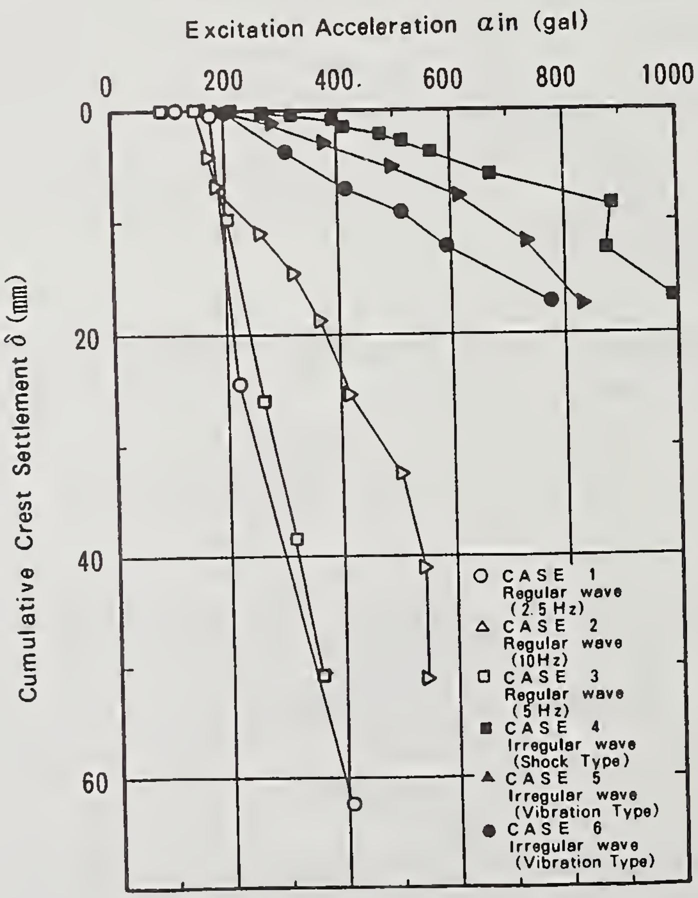


Fig.9 Input acc. vs. cumulative crest settlement

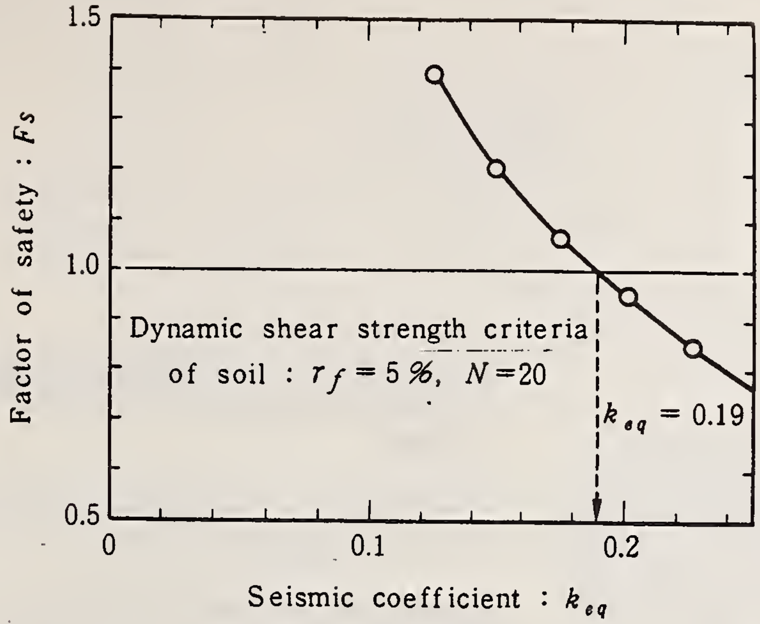


Fig.10 Safety factor vs. equivalent seismic coefficient

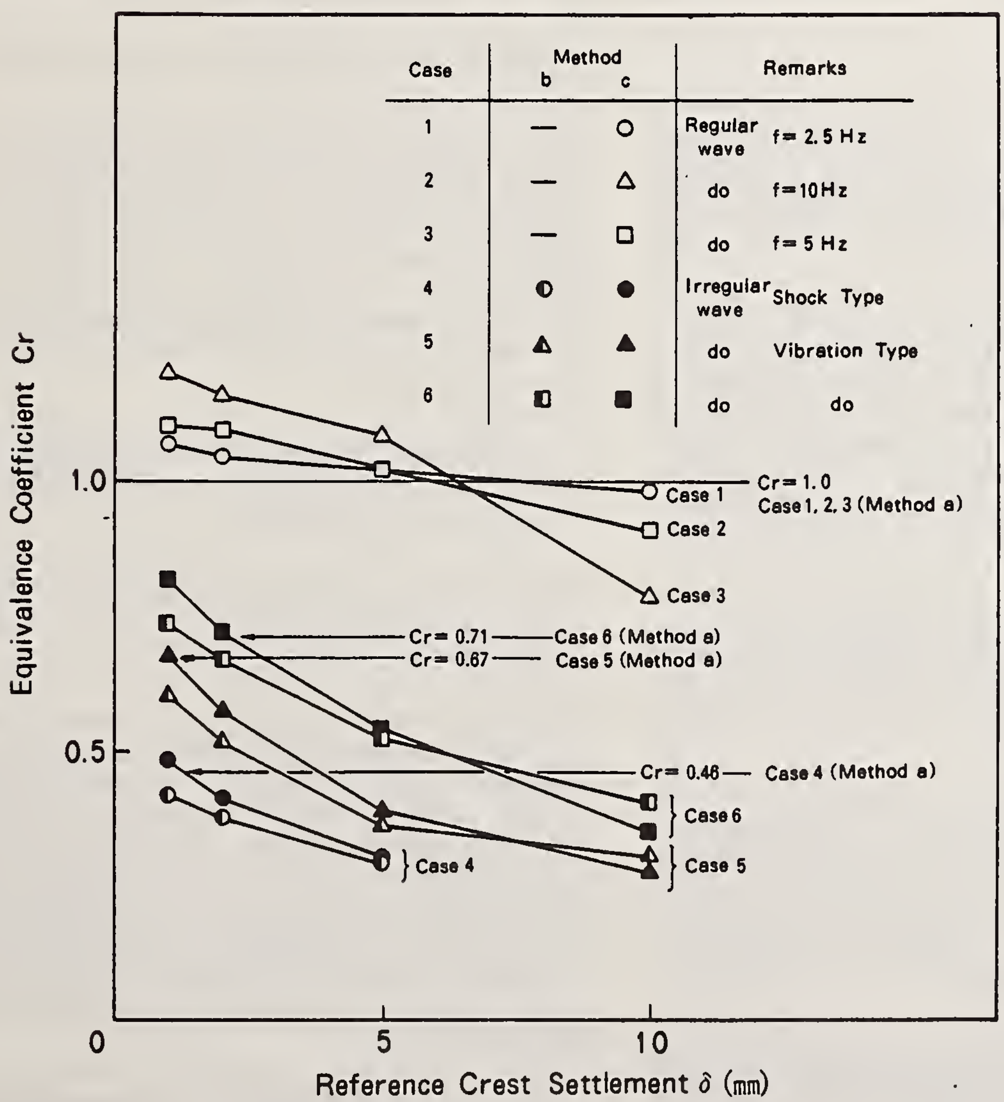


Fig.11 Failure criteria vs. equivalence coefficient

Seismic Monitoring of Buildings: Analyses of Seismic Data

by

M. Çelebi*

ABSTRACT

The main objective in seismic instrumentation of structures is to facilitate response studies that lead to improved understanding of the dynamic behavior and the potential for damage to structures under seismic loading. The purpose of this paper is: (1) to review the status of the programs for strong-motion instrumentation of structures in the United States and discuss various procedures and instrumentation schemes designed to best acquire response data from buildings and (2) to discuss preliminary results derived from recorded response data obtained from well-instrumented structures during the recent earthquakes.

KEYWORDS: earthquakes, records, spectra, instrumentation, diaphragm, interaction

1. INTRODUCTION

The purpose of this paper is to provide some insight into the objectives of programs for seismic monitoring of structures and review the status-quo with examples of records obtained from structures.

The main objective of any seismic instrumentation program for structural systems is to improve the understanding of the behavior, and potential for damage, of structures under seismic loading. The acquisition of structural response data during earthquakes is essential to develop and confirm methodologies used for analysis and design of earthquake-resistant structural systems. This objective can best be realized by selectively instrumenting structural systems to acquire data on strong ground-motion, and the response of structural systems (buildings, components, lifeline structures, etc.). As a long-term result one may expect design and construction practices to be modified to minimize future earthquake damage.

As part of its earthquake hazard reduction program, the United States Geological Survey (USGS) has pursued a structural instrumentation program to further structural response studies in selected targeted seismic regions shown in the map in Figure 1. A general review of the USGS program objectives and procedures is provided by Çelebi and others [1].

It is expected that a well-instrumented structure for which a complete set of recordings has been obtained, would provide useful information to:

- check the appropriateness of the design dynamic model (both lumped mass and finite element) in the elastic range;
- determine the importance of non-linear behavior on the overall and local response of the structure;
- follow spreading of the non-linear behavior throughout the structure as the response increases, and investigate the effect of the non-linear behavior on frequency and damping;
- correlate the damage with inelastic behavior;
- determine ground motion parameters that correlate well with building response and damage; and
- make recommendations to improve seismic codes.

The primary factors affecting selection of structures for instrumentation within the general USGS program are related to structural systems and construction materials that are likely to be repeated and those that have sufficient long-term engineering interest as well as site-related factors. These factors and parameters are weighted in the decision-making process of selection of structures for strong-motion instrumentation and can be categorized as follows:

- (a) structural parameters: materials of construction, structural systems, geometry, discontinuities, importance, age, and interest.
- (b) site parameters: probability of occurrence of a large earthquake, proximity to faults, site conditions, and expected damage and loss.

In the State of California, the most extensive program for instrumentation of structures is being conducted by the California Division of Mines and Geology (CDMG). To date, approximately 100 structures in California have been instrumented by the CDMG program [2]. The CDMG program aims to monitor typical buildings and structural systems according to a pre-established matrix reflecting structural types and heights. Therefore, in California, the USGS program concentrates on instrumentation of non-typical structures of special engineering interest.

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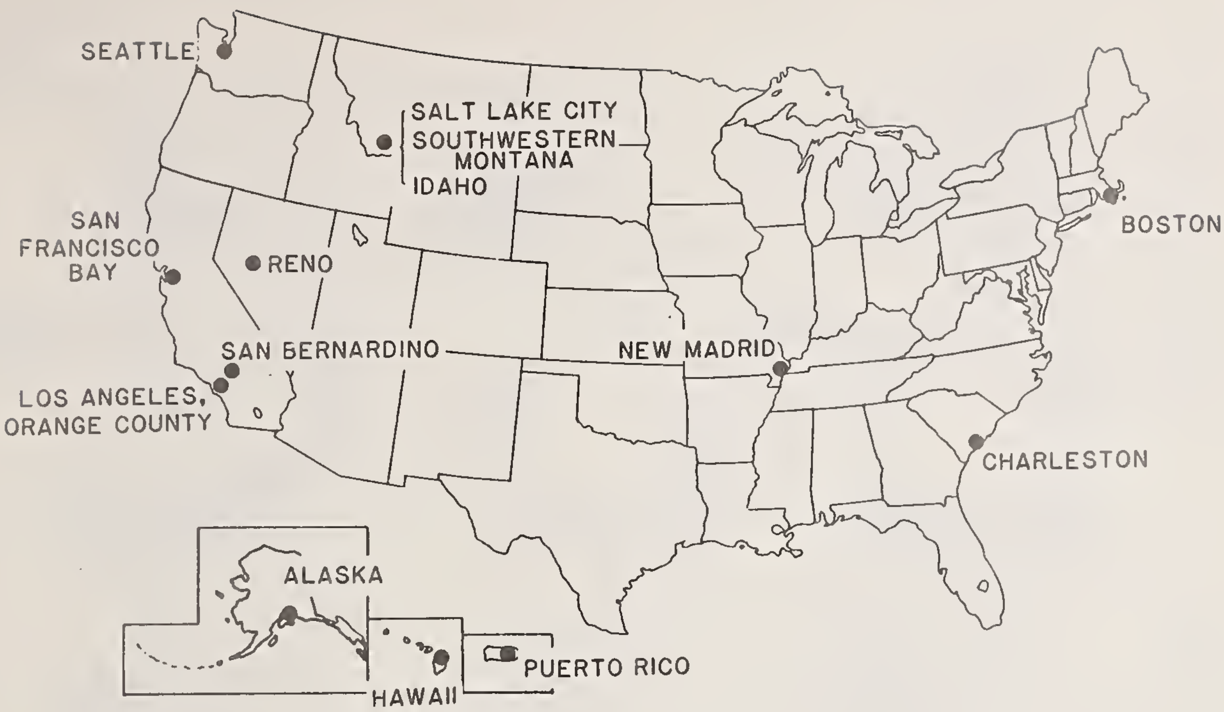


Figure 1. Targeted seismic regions for the USGS instrumentation of structures program.

Outside the United States, a significant number of buildings and other structures have been instrumented in Japan by various governmental and private organizations [3]. Italy has embarked upon an impressive structural monitoring program that also encompasses historical monuments [4]. It is known that there are instrumented structures in New Zealand and the Soviet Union.

2. CODE-TYPE INSTRUMENTATION

In the United States, independent of the USGS and CDMG programs described above, various codes in effect, whether nationwide or local, recommend different quantities and schemes of instrumentation. The Uniform Building Code (UBC) [5] recommends that for the two top seismic zones (Seismic Zones 3 and 4) a minimum of three tri-axial accelerographs be placed in every building over six stories in height, with an aggregate floor area of 60,000 square feet or more, and in every building over 10 stories in height, regardless of floor area. According to this recommendation, the tri-axial accelerographs are typically located in a building as shown in Figure 2a. The City of Los Angeles adopted the UBC recommendation in 1966—thus enabling numerous sensors in many buildings to record the motions during the 1971 San Fernando earthquake. However, in 1983, the City of Los Angeles changed the requirement of three accelerographs to only one—to be placed at the top of those buildings, meeting the UBC criteria described above.

Experience from past earthquakes, as well as the 1971 San Fernando earthquake, shows that the instrumentation guidelines given by the UBC code, for example, although providing necessary data for the limited analyses projected at the time, do not provide sufficient data to perform the model verifications and structural analyses now demanded by the profession. For example, torsional or rocking motions of a structure cannot be identified unless the structure is properly instrumented in key locations to obtain the necessary data. Typically, an extensively-instrumented building should contain sensors in key locations to record translational, torsional as well as rocking, motions of the structural system as seen in Figure 2b. Whenever physically possible, an extensive instrumentation scheme should include a free-field station. Extensive instrumentation for special cases such as buildings with flexible diaphragms and base-isolators requires variations in the schemes of instrumentation and are schematically shown in Figures 2c and 2d.

A recent example of records obtained during the 1 October 1987 Whittier–Narrows earthquake from UBC-type recommended instrumentation at the 900 South Fremont Building in Alhambra, California is shown in Figure 3. From these records, we can clearly identify the fundamental periods of the twelve-story building in its translational mode in the two horizontal axes at apoximately 2 seconds. This is demonstrated in Figure 4 showing the same recorded tri-axial accelerations (vertical, north-south and east-west directions) at the basement, 6th floor and 12th floor and the Fourier spectrum corresponding to each component. However, it is impossible to identify non-translational contributions (*e.g.*, torsional) and/or interaction effects (*e.g.*, rocking), if any, to the behaviour of the building—particularly when, as in this case, the tri-axial accelerographs were located near the center of rigidity of the building.

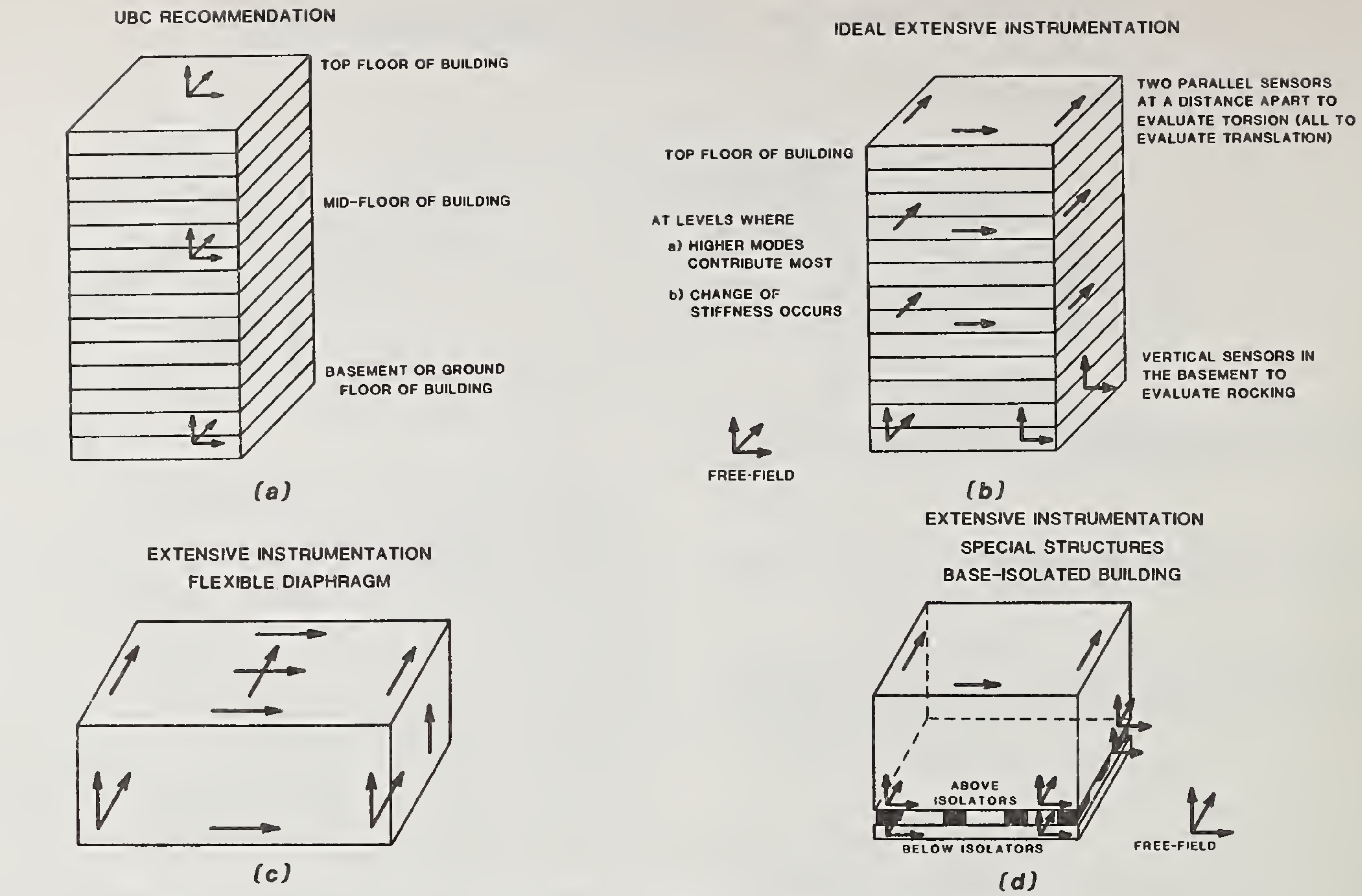


Figure 2. (a) Instrumentation according to Uniform Building Code recommendations. (b) An ideal extensive instrumentation for a regular building. (c) Instrumentation to record response of a flexible diaphragm. (d) Instrumentation to record response of a base-isolated building.

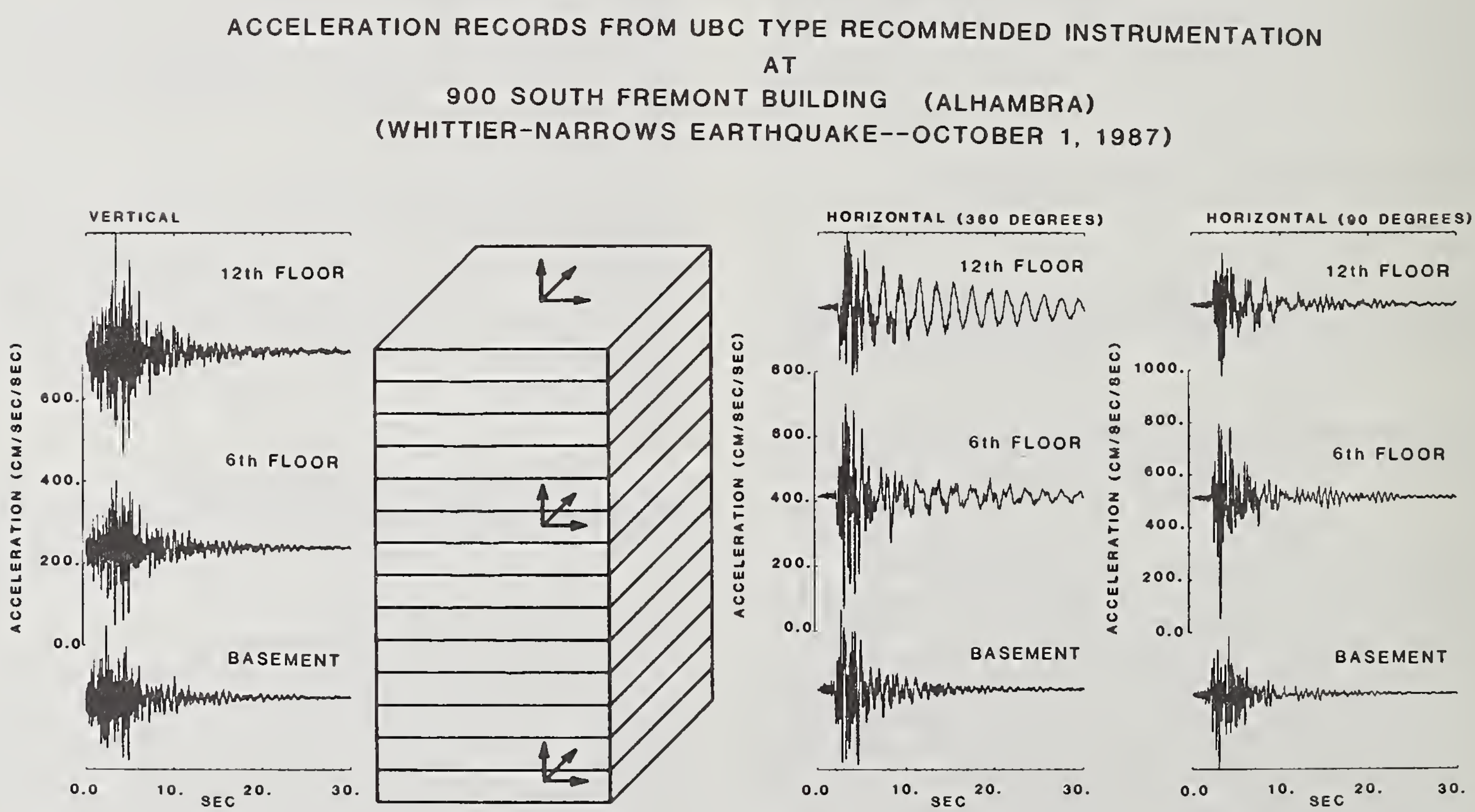


Figure 3. Acceleration records obtained at the 900 South Fremont Building (Alhambra) during the Whittier-Narrows earthquake of October 1, 1987 (Code-Type Instrumentation).

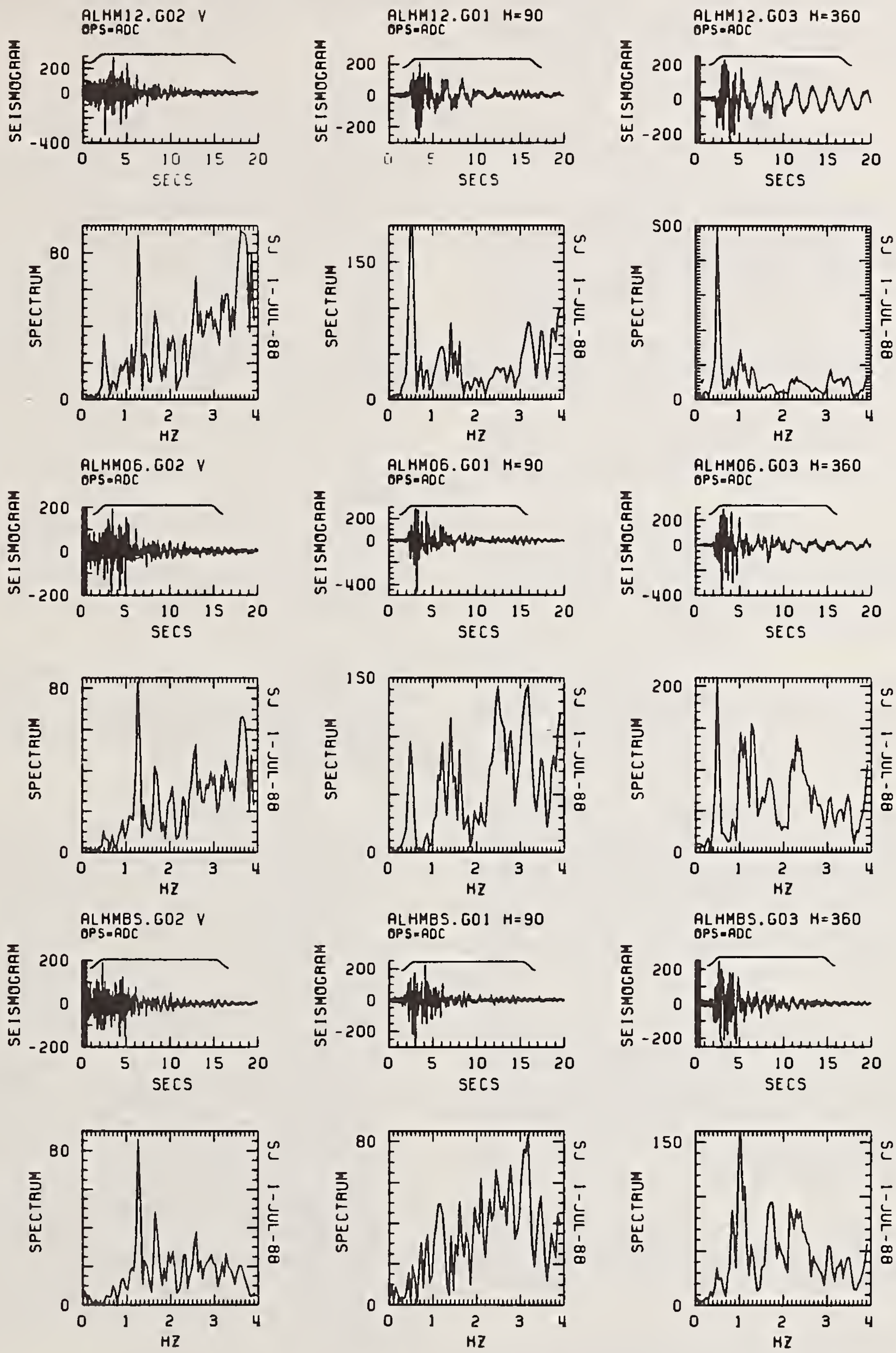


Figure 4. Acceleration records obtained at the 900 South Fremont Building (Alhambra) during the Whittier-Narrows earthquake of October 1, 1987 and the corresponding Fourier spectra.

While UBC recommendations are deemed not to be sufficient, nonetheless, records obtained during the 1971 San Fernando earthquake from buildings in downtown Los Angeles instrumented in accordance with UBC recommendations were used to derive relationships for determining fundamental periods of structures incorporating non-linear behavior [6]. These relationships have been incorporated into ATC-03-06 code provisions [7] and the 1988 edition of UBC [5].

3. EXTENSIVELY INSTRUMENTED STRUCTURES

Imperial County Services Building is perhaps the source of the most widely studied building-response data. During the 1979 Imperial Valley earthquake ($M_s = 6.5$), the building sustained substantial damage, primarily to its soft first story, and as a result was later demolished. The building was extensively instrumented with thirteen channels of accelerometers, a central recorder, and a three-component free-field accelerograph. The locations of sensors throughout the building are seen in Figure 5. The following conclusions were derived from the study of the complete set of records (Figures 6 and 7) [8]:

- the motion of the ground floor was affected by soil-structure interaction, as shown by the agreement of the data obtained with mathematical models that incorporate soil-structure interaction,
- the peak acceleration was approximately 50% higher and the nature of motions considerably different at the ground-floor level in comparison to the free-field motions,
- during the earthquake, the fundamental period of the building increased significantly (in the order of 150%), compared with the pre-earthquake periods determined from ambient vibration tests. Such drastic changes in the fundamental dynamic characteristics can only be attributed to the inelastic behavior of the structure during the earthquake,
- the initiation of the inelastic behavior and structural damage as apparent on the records and subsequent analyses reconfirmed that soft first stories exemplified in the Imperial County Services Building design are vulnerable during earthquakes unless specific design considerations are implemented.

Significant results have also been deduced from the study of the records obtained, during the 1979 Imperial Valley earthquake, at the Meloland Road–Interstate Highway 8 crossing [9]:

- peak accelerations recorded on embankment sites adjacent to each abutment were up to 30% higher than those recorded at the central column, indicating that the embankment fill material influenced the motion of the abutment,
- relative motion between the abutments and the surrounding fill material was evident,
- the recorded deck motions showed rotational as well as translational components.

During the $M_s = 6.1$ Morgan Hill earthquake of 24 April 1984, a complete set of acceleration records depicting the behavior of a flexible roof diaphragm were obtained from the West Valley College gymnasium in Saratoga, California. The gymnasium was instrumented by the California Division of Mines and Geology (CDMG) [10, 11].

The roof diaphragm of the 34.14 m × 43.90 m rectangular, symmetric gymnasium consists of 0.95 cm plywood over tongue-and-groove sheathing attached to steel trusses supported by reinforced concrete columns and walls. A three-dimensional schematic of the gymnasium and the locations of the eleven channels of force-balance accelerometers (connected to a central recording unit) and associated directions are provided in Figure 8. Only the main lateral load-resisting elements (shear walls) are depicted in the figure; however, the finite element model (also in the figure) developed for dynamic analyses took into account the columns as well. The recorded acceleration responses are shown in Figure 9. There were no free-field instruments at the site. Three sensors placed in the direction of each of the axes of the diaphragm facilitate the evaluation of in-plane deformation of the diaphragm. Other sensors placed at ground level measure vertical and horizontal motion of the building floor, and consequently allow the calculation of the relative motion of the diaphragm with respect to the ground level.

The following conclusions were derived based on studying the peak accelerations, relative displacements and Fourier spectra of the records obtained at the roof diaphragm level [12, 13]:

- the recorded peak accelerations in the N–S and E–W directions of the ground floor as well as the diaphragm differ considerably. The recorded peak ground-floor acceleration in the N–S direction (perpendicular to the narrower 34.14 m dimension) is 0.10 g while in the E–W direction (perpendicular to the 43.90 m dimension), it is 0.04 g. Consequently, the displacements in the stronger (N–S) direction are larger than those in the weaker (E–W) direction. Such differences in ground level accelerations are not generally accounted for in design considerations. The wall motion at the roof level is 1.5 times the amplitude of the floor level, while the horizontal motion of the center of the roof is 3 times the motion at the edges during the resonances in the two horizontal axes of the roof diaphragm evaluated 3.8 Hz and 3.9 Hz, respectively. The fundamental frequencies of the diaphragm are determined from the recorded accelerations to be 3.8 and 3.9 Hz in the N–S and E–W directions, respectively.
- at a frequency of about 4 Hz, the center of the roof diaphragm attains a peak acceleration of 0.42 g and 0.20 g in the N–S and E–W directions, respectively. The corresponding roof-edge peak accelerations are 0.14 g (average) and 0.065 g (average) in the N–S and E–W directions, respectively. Therefore, there is an amplification of motions at the center of the roof as compared with the edges by a factor of approximately 3 for both directions. From ground floor level to the edges, the same factor is approximately 1.5 in both directions.

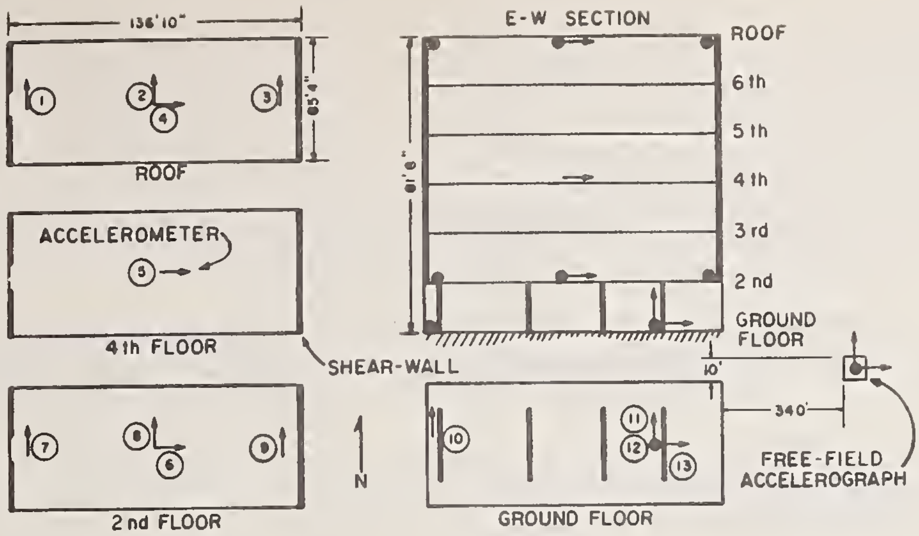


Figure 5. Floor plans and typical vertical section of the Imperial County Services Building. Locations of the sensors are identified.

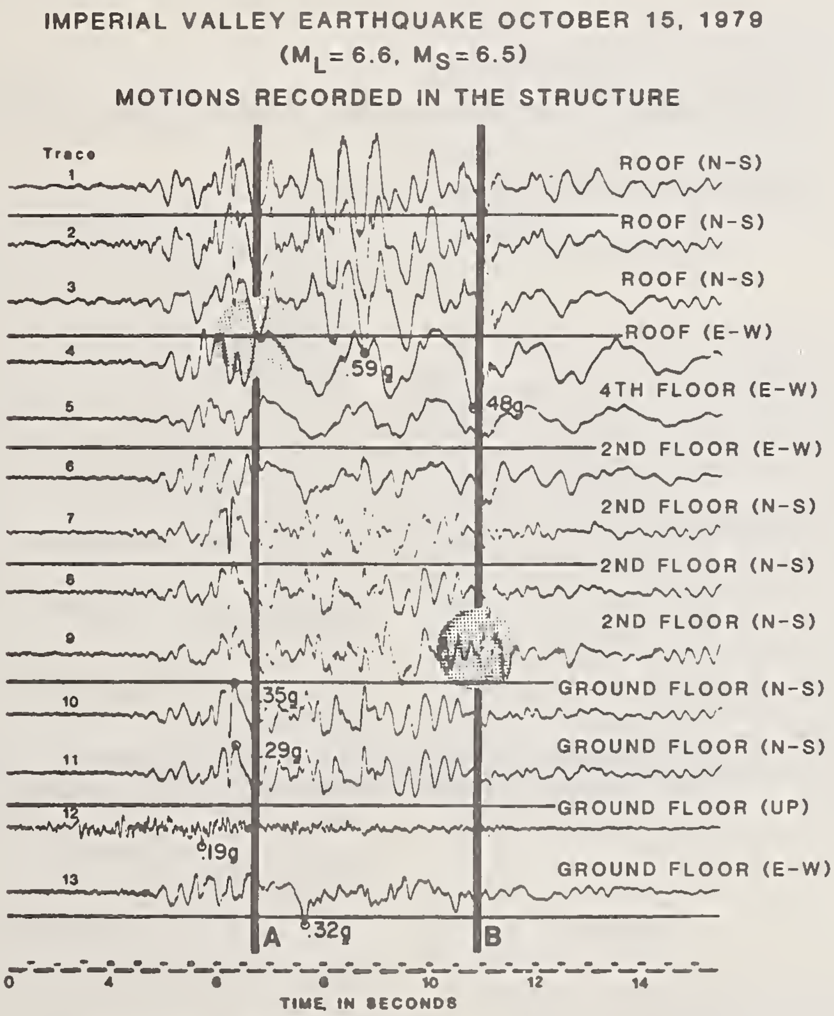


Figure 6. Acceleration records obtained at the Imperial County Services Building during the Imperial Valley earthquake of October 15, 1979.

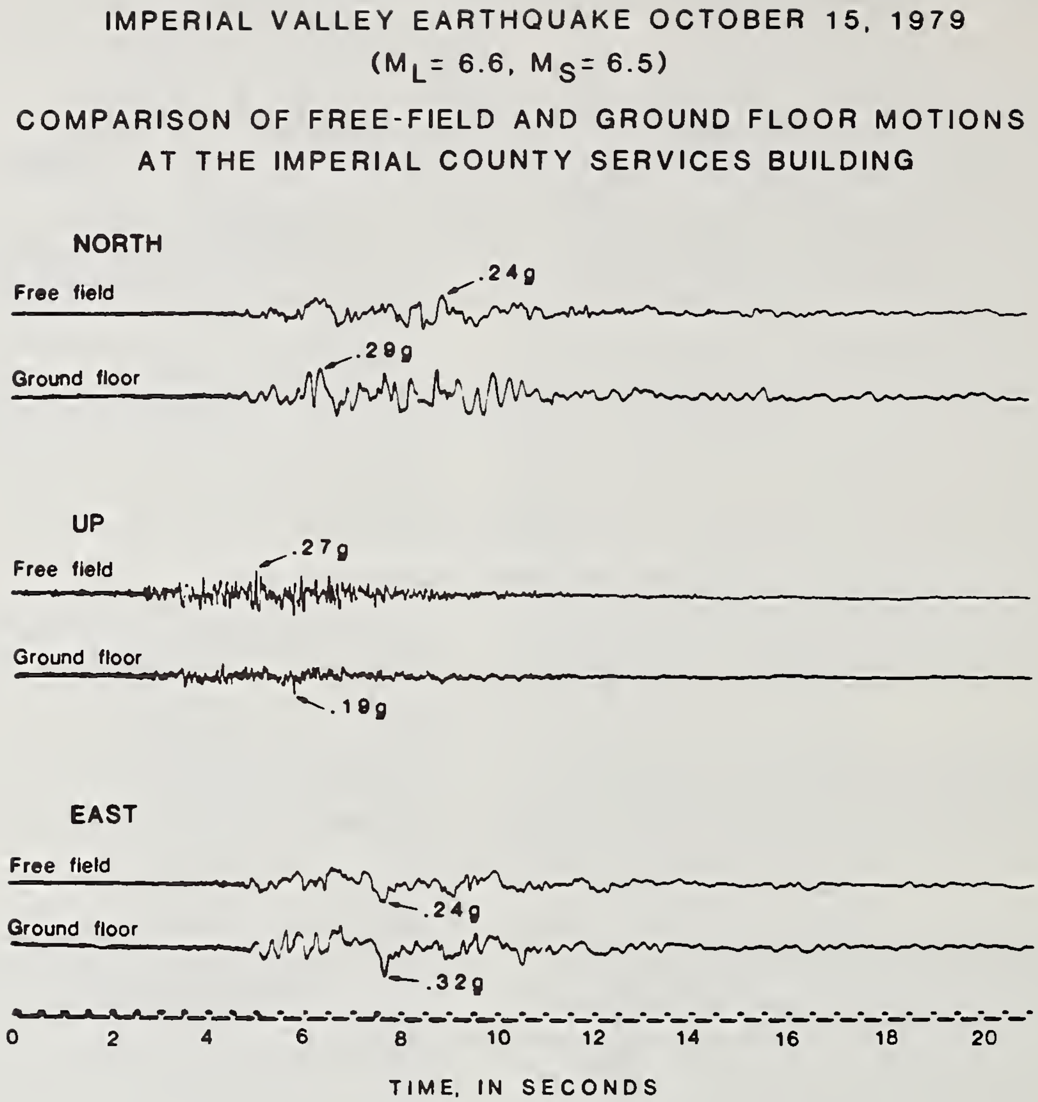


Figure 7. Comparison of free-field and ground floor motions recorded at the Imperial County Services Building during the Imperial Valley earthquake of October 15, 1979.

- furthermore, a very simple finite-element model consisting of beam elements (columns and shear walls) and plane-stress elements (diaphragm) [Figure 8], is used to match the fundamental frequency of the roof diaphragm. Within acceptable tolerances, the calculated displacement of the center of the diaphragm with those from recorded motions are matched for 5.0% damping as seen in Figure 10 which also shows the Fourier spectra derived from calculated responses.

- the calculated or the recorded displacements do not compare well with those from either the ATC–7 recommended formulas [14] or by calculations based on UBC seismic forces [5]. In addition, the recorded or calculated first mode of the structure is that of the diaphragm only and the frequencies, recorded or calculated, do not match with the frequencies determined from standard period formulas for structures. Therefore, in the design-analysis process, the standard code formulas for building periods should not be used to predict the periods of buildings with large-span diaphragms.

4. TORSIONAL RESPONSE OF A UNIQUE BUILDING

The 483 ft. (147 m) tall, 32-story 1100 Wilshire Finance Building in Los Angeles, California was instrumented in 1986 during its construction. The unique building plan abrubtly changes at the 12th level from a nominally rectangular shape to a triangle. General dimensions and the 21-channel instrumentation scheme of the building are provided in Figure 11. The building sits on spread footing founded on shale. Scaled horizontal acceleration time-histories at different levels of the building obtained during the October 1, 1987 Whittier–Narrows earthquake are provided in Figure 12. The records show that the motions at each level in the 208 direction at the NW corner of the building differ considerably from those in the SE corner. These differences indicate that torsional motions contribute significantly to the translational motions (*e.g.,* the peak accelerations of parallel motions are 0.19 g and 0.10 g on the 32nd floor and 0.26 g and 0.12 g on the 13th floor). The difference between the NW208 and SE208 accelerations at each instrumented level represents the rotational acceleration at that level (after division by the perpendicular distance between the two sensors). The recorded accelerations in the NW208 direction, the representative rotational acceleration (the difference between NW208 and SE208 accelerations), and corresponding Fourier spectra (all plotted to the same scale) for each of the top three levels are shown in Figure 13 and in Figure 14 for the ground level and the basement. The Fourier spectra of the total translational and the representative rotational motions at these top three levels show that the superstructure responded with a closely coupled torsional mode at a predominant frequency of approximately 0.8 Hz and another at 0.7 Hz. Furthermore the Fourier spectra amplitudes and the comparison of the amplitudes of the translational and torsional contributory accelerations show the significant level of torsional energy at the top three instrumented levels; however, the level of energy reduces considerably at the ground level and is not significant at the basement.

While further study of the records from this building are currently underway, it is important to note that the frequencies derived from the records presented herein do not in any way compare with the frequencies used in the design-analysis process. For example, the fundamental frequency of the building has been calculated (using fixed-base assumption) as 4.9 s (or approximately 0.2 Hz). We draw the following conclusions from the study of the motions recorded:

- higher modes dominate the response of the building to this earthquake.

- the dominant behavior of the building is a coupled torsional-translational mode (the torsional contribution to the translational motions recorded is very significant).

5. CONCLUSIONS

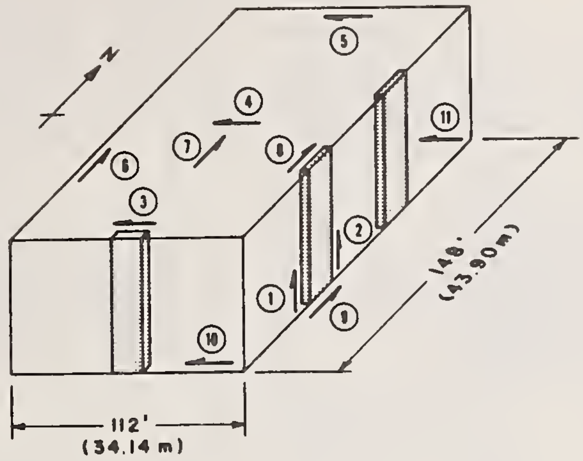
The objectives of seismic instrumentation of structures and various programs are reviewed in light of lessons learned from studying response data obtained from structures during past earthquakes. UBC-recommended instrumentation is discussed and compared with extensive instrumentation that permits complete response study of structures. It is shown that code-type instrumentation does not allow detection of torsional or rocking motions of a structure.

It is important to invest in seismic monitoring of structures. We have learned from study of the records obtained from instrumented structures. Well-instrumented structures can provide information on the real-life laboratory behavior of a structure that might not be retrevi-able using only the code-type recommended instrumentation, nor could it always be predicted during the design/analyses processes which requires certain idealizations. Examples presented show (a) that soft first stories require careful consideration, (b) that closely coupled higher torsional modes dominate the response of a vertically irregular structure, and (c) that the behavior of a flexible diaphragm can be identified by appropriately recording its response. These results may be significant in future analyses and modeling of such structures.

6. REFERENCES

1. Çelebi, M., Şafak, E., Brady, G., Maley, R., and Sotoudeh, V., 1985, Integrated instrumentation plan for assessing the seismic response of structures—A review of the current USGS program, USGS Circular 947.
2. Huang, M., Ventura, C., and Shakal, A., 1989, Strong-motion records from buildings, advance copy of paper submitted to ASCE Structures Congress, San Fransisco, CA, June 1989.
3. Watabe, M., and Nagata, M., 1989, Seismic instrumentation and its implications for high-rise buildings in Japan, advance copy of paper submitted to ASCE Structures Congress, San Fransisco, CA, June 1989.

4. Basili, M., and Bongiovanni, G., 1989, Seismic instrumentation of buildings in Italy, advance copy of paper submitted to ASCE Structures Congress, San Fransisco, CA, June 1989.
5. Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1970, 1973, 1976, 1979, 1982, 1985, 1988 editions.
6. Mulhern, M. R., and Maley, R., 1973, Building period measurements before, during and after the San Fernando earthquake, in San Fernando, California earthquake of February 9, 1971, U.S. Dept. of Commerce, NOAA, vol. I, Part B, pp. 725-733.
7. Applied Technology Council, 1978, Tentative provisions for the development of seismic regulations for buildings, Applied Technology Council—ATC 3–06.
8. Rojahn, C. and Mork, P. N., 1982, An analysis of strong motion data from a severely damaged structure—The Imperial County Services Building, El Centro, California, in The Imperial Valley, California, earthquake of October 15, 1979: U.S. Geol. Surv. Prof. Pap. 1254.
9. Rojahn, C., Ragsdale, J. T., Raggett, J. D., and Gates, J. H., 1982, Main-shock strong-motion records from the Meloland Road–Interstate Highway 8 overcrossing, in The Imperial Valley, California, earthquake of October 15, 1979: U.S. Geol. Surv. Prof. Pap. 1254.
10. Huang, M. J., Shakal, A. F., Parke, D. L., Sherburne, R. W., and Nutt, R., 1985, Processed data from the strong motion records of the Morgan Hill earthquake of 24 April 1984—Part II: Structural response records, Office of Strong Motion Studies, Report. No. OSMS 85-04, CDMG-SMIP, Sacramento, CA.
11. Shakal, A. F., Sherburne, R. W., and Parke, D. L., 1984, CDMG strong motion records from the Morgan Hill, California, earthquake of 24 April 1984, Office of Strong Motion Studies Report No. OSMS 84–7, CDMG, Sacramento, CA, 101 p.
12. Çelebi, M., Brady, G., Şafak, E., and Converse, A., 1986, Performance of an earthquake-excited roof diaphragm, Proc. ASCE Conference on Dynamic Response of Structures, U.C., Los Angeles, March 31–April 2, 1986, pp. 863–870.
13. Çelebi, M., Bongiovanni, G., Şafak, E., and Brady, G., 1989, Seismic Response of A Large-Span Roof Diaphragm, in print Earthquake Spectra (May 1989 issue).
14. ATC–7, 1981, Guidelines for the design of horizontal wood diaphragms, Applied Technology Council, Palo Alto, CA.



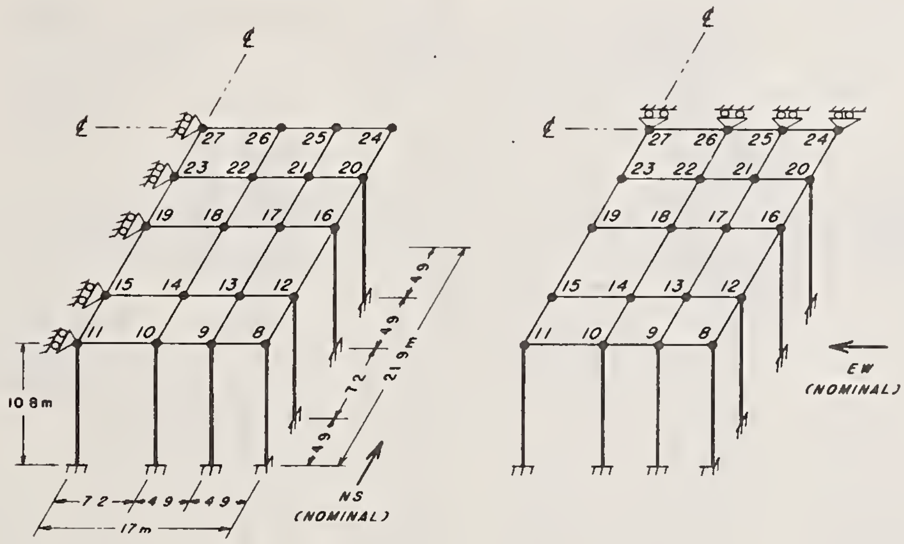


Figure 8. Three-dimensional schematic of the West Valley College Gymnasium (Saratoga, California) and the locations and directions of eleven channals of force-balance accelerameters. Only the main lateral load resisting shear walls are shown (top figure). The finite element model (quarter of the gymnasium–bottom figures) shows the columns as well as the shear walls all of which are modeled as beam elements.

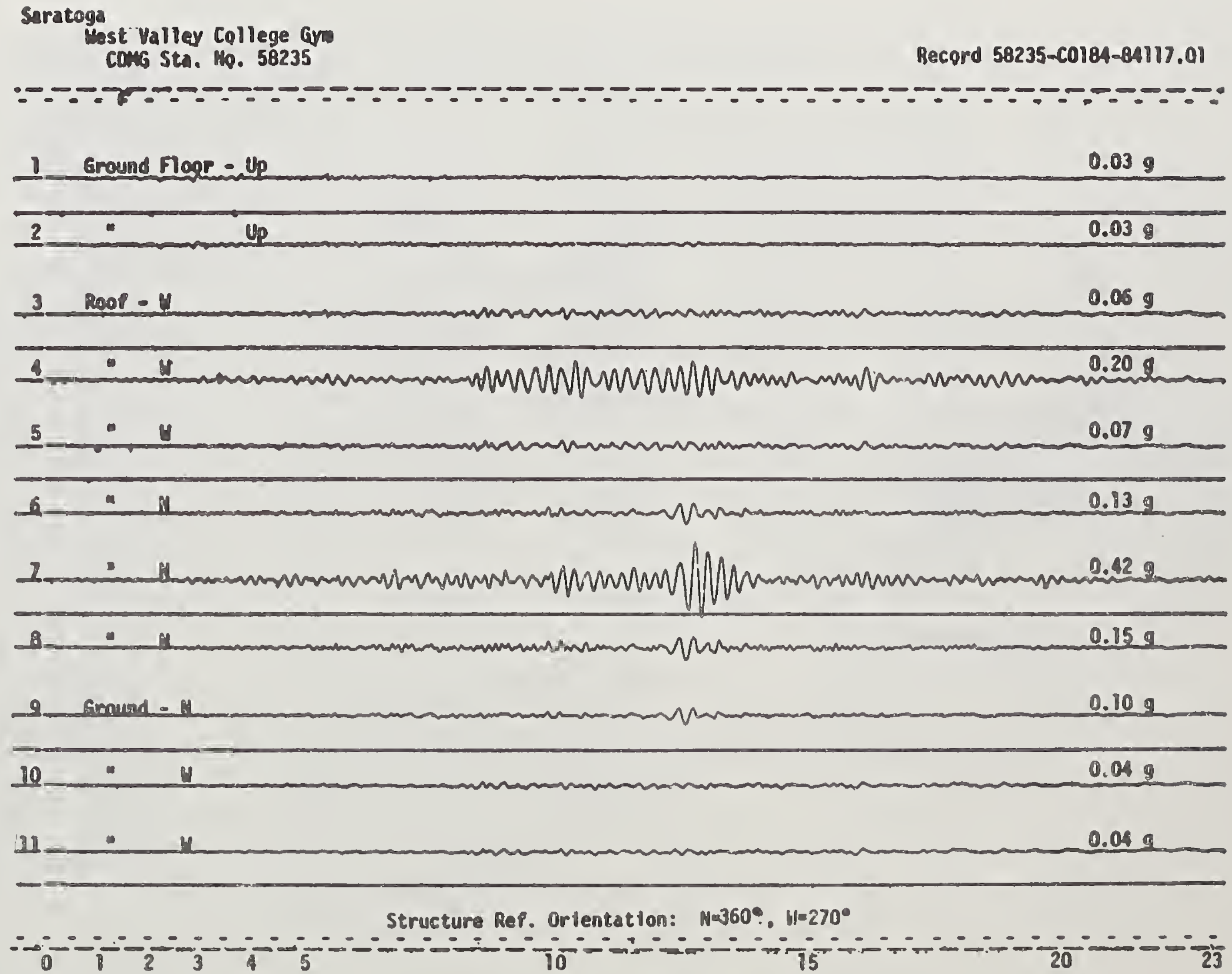


Figure 9. Acceleration responses recorded at the West Valley College Gymnasium (Saratoga, California) during the 24 April 1984 Morgan Hill earthquake [$M_s = 6.1$].

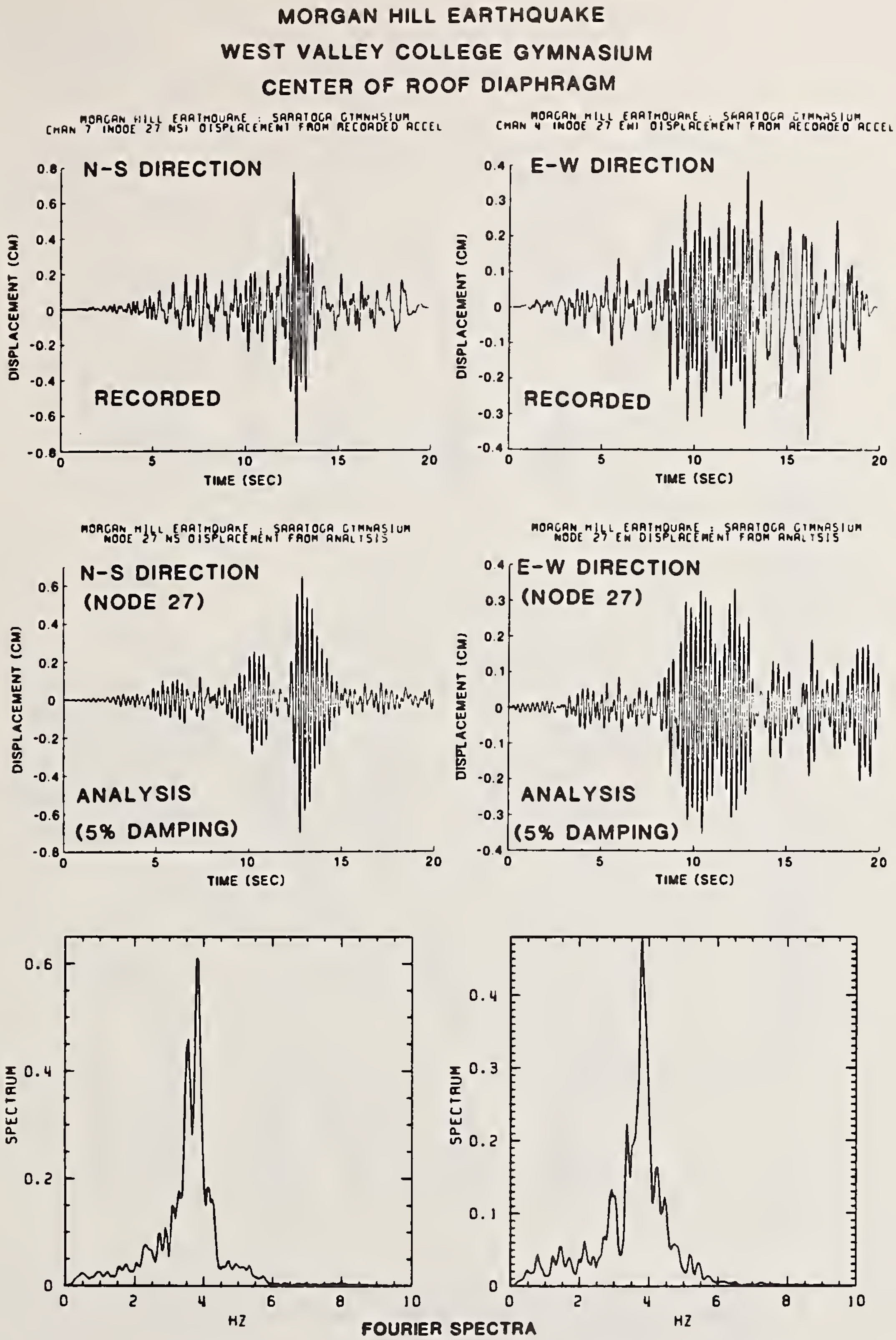


Figure 10. Recorded and calculated displacements at the center of the roof diaphragm in the N-S and E-W directions respectively. The calculated displacements are from dynamic analyses using the finite element model and ground level motions as input. The Fourier spectra corresponds to calculated displacements.

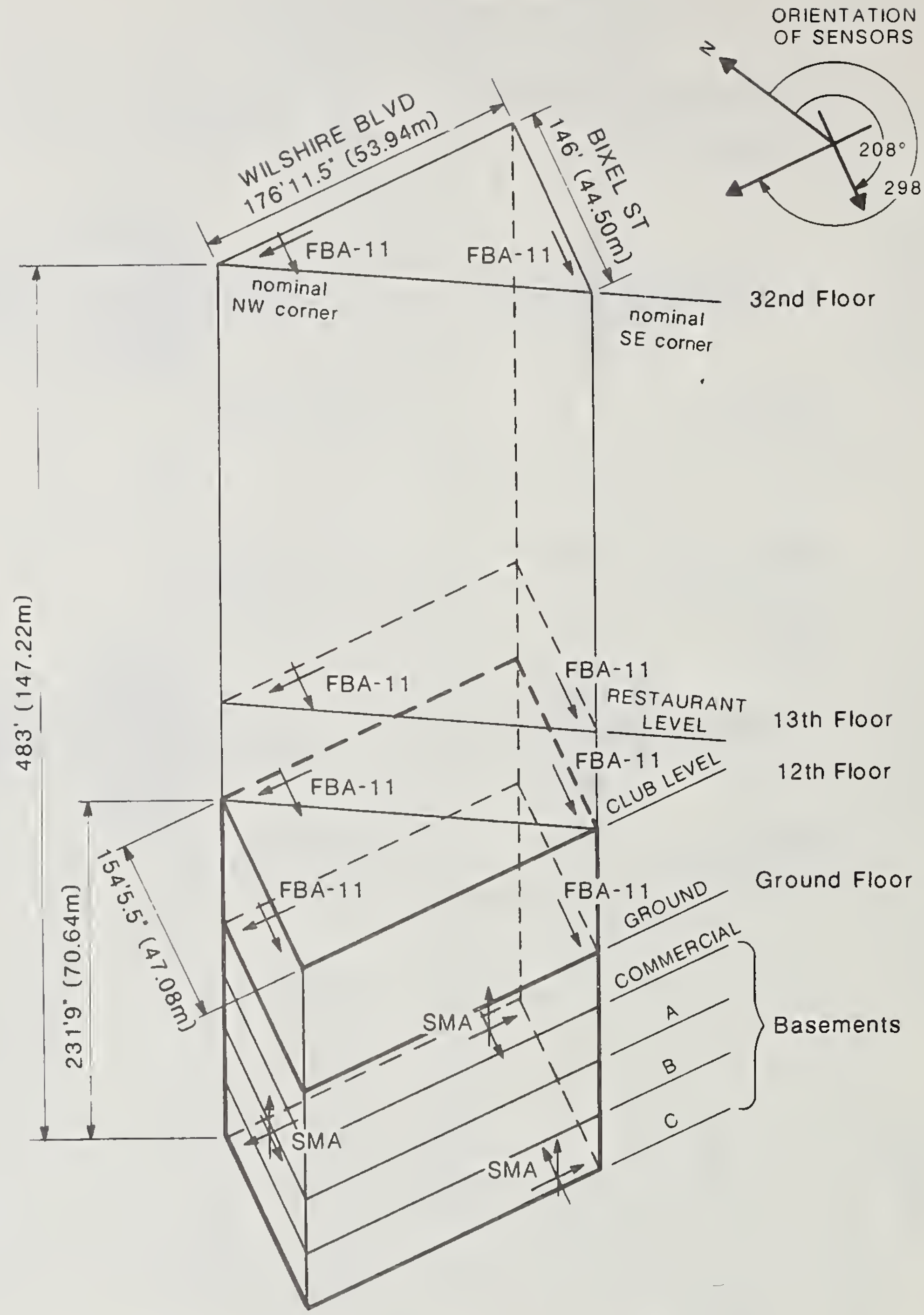


Figure 11. General dimensions and locations as well as orientations of the strong-motion instrumentation scheme of 1100 Wilshire Finance Building (Los Angeles, California).

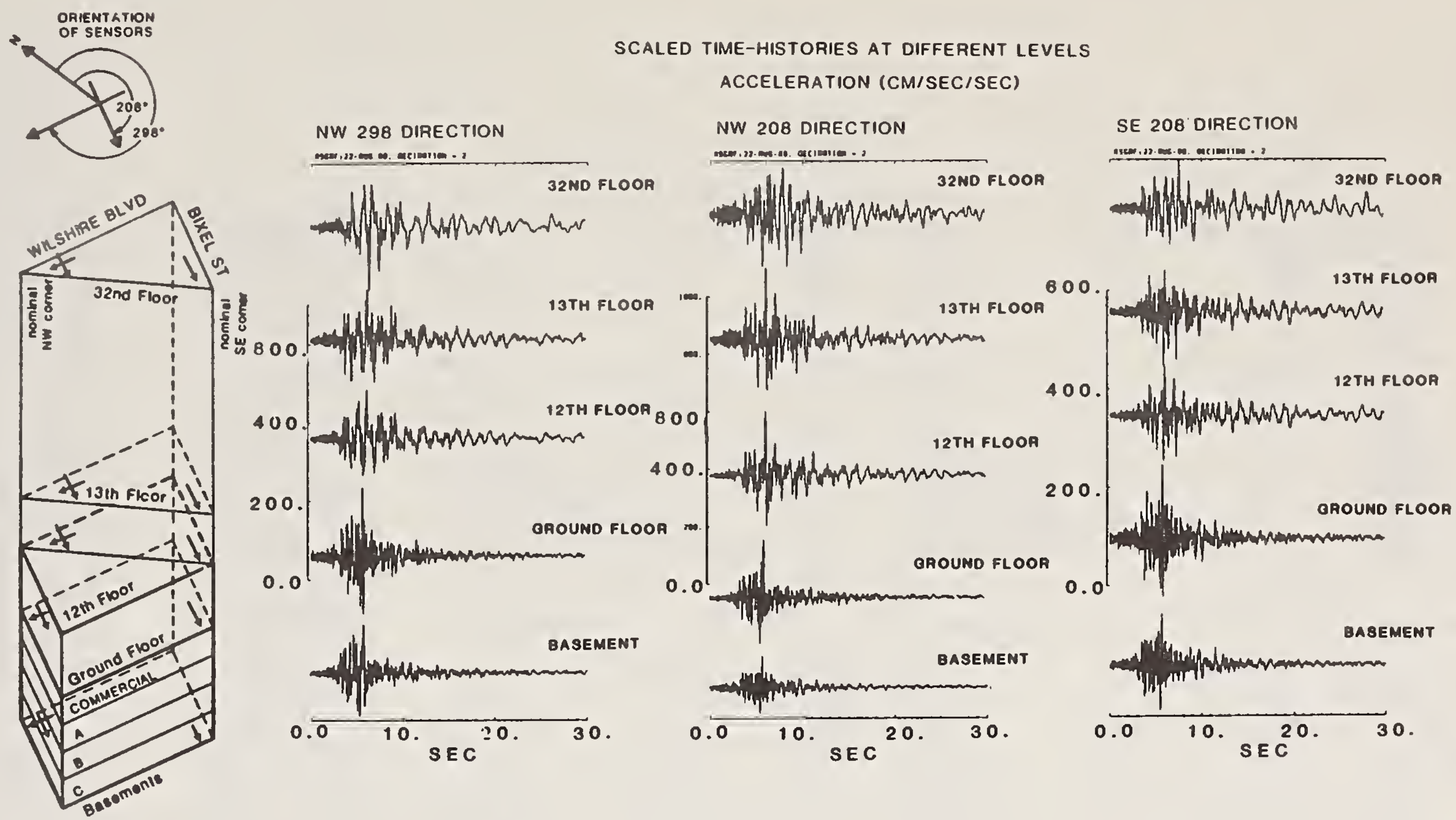


Figure 12. Scaled horizontal accelerations recorded during the Whittier-Narrows earthquake of October 1, 1987 at 1100 Wilshire Finance Building.

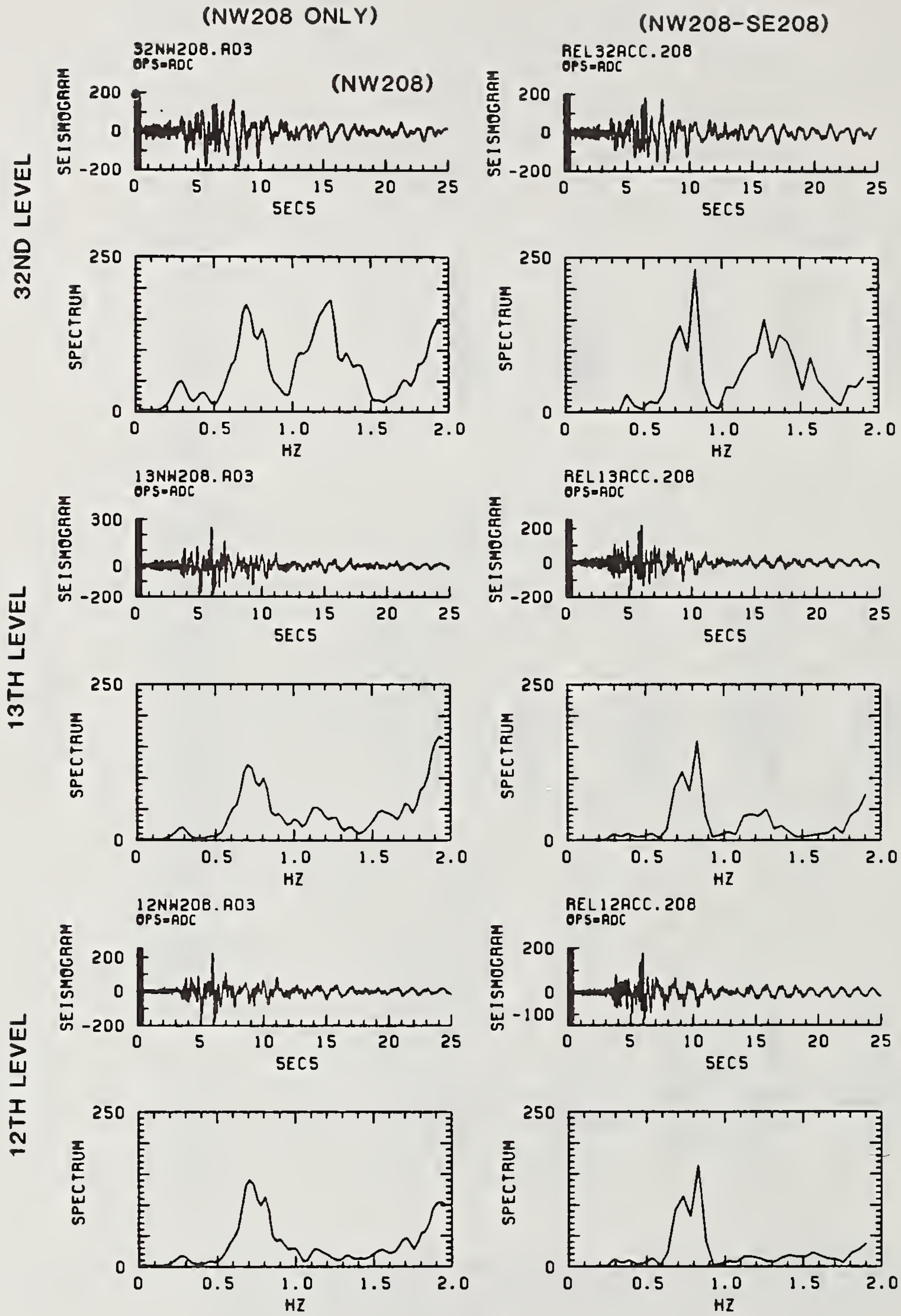


Figure 13. On the left column are the NW208 component accelerations and their Fourier spectra (all plotted to the same scale) for 32nd, 13th and 12th levels respectively. On the right are the representative rotational accelerations (differences between NW208 and SE208 recordings) and their Fourier spectra corresponding to the same instrumented levels.

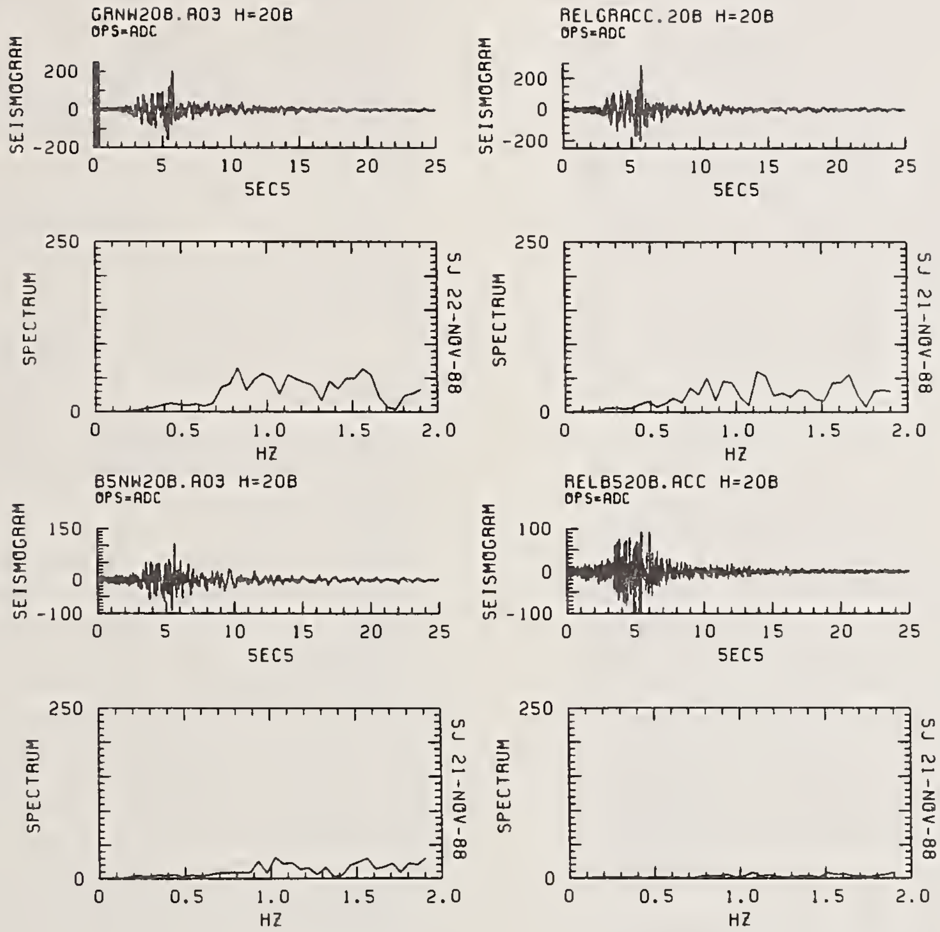


Figure 14. On the left column are the NW208 component accelerations and their Fourier spectra (all plotted to the same scale) for the gound level and the basement respectively. On the right are the representative rotational accelerations (differences between NW208 and SE208 recordings) and their Fourier spectra corresponding to the same instrumented levels.

Development of Advanced Reinforced Concrete Buildings Using High-Strength Concrete and Reinforcement

by

Tatsuo Murota[1] and Masaya Hirosawa[2]

ABSTRACT

This paper describes the outline of a MOC's new research project on reinforced concrete buildings using high-strength and high-quality concrete and reinforcement. The project is carried out by BRI from 1988 to 1992. Concrete of 300 to 1200 kgf/cm^2 strength and reinforcement of 4000 to 12000 kgf/cm^2 yield strength are treated in the project. Guidelines for structural design and construction of reinforced concrete buildings using concrete of less than 600 kgf/cm^2 strength and reinforcement of less than 6000 kgf/cm^2 yield strength are expected as fruit of the project.

KEYWORDS : high-strength concrete, high-strength reinforcement,high-rise reinforced concrete building.

1. INTRODUCTION

Recently, research and development on reinforced concrete buildings using high-strength concrete has been actively promoted in Japan. Based on these activities, high-rise RC buildings which have images remarkably different from those of common RC buildings have come to appear in Japan.

Technology development on such a new field of RC building construction seems to be more actively promoted in the future and it is sure that the Ministry of Construction will be requested to have technical standards for building administration for such buildings, sooner or later.

Based on the background, the Ministry of Construction (MOC) decided to manage a five-year research project titled as "Development of Advanced Reinforced Concrete Buildings Using High-Strength Concrete and Reinforcement" (hereinafter "New RC"). New RC Project started in 1988 fiscal year with major objectives as follows:

(1) To survey and analyze results of research and development conducted by private companies using a proper common measure to establish a technology system for advanced reinforced concrete buildings (New RC buildings). If necessary, to execute basic researches for the systematization.

(2) To understand the mechanical behavior of buildings using high-strength concrete and steel bars and to develop design systems suitable for New RC buildings.

Results of the project will be utilized as technical standards for building administration and will encourage further research and development on advanced reinforced concrete buildings.

2. RANGE OF MATERIAL STRENGTH

Strength of concrete and reinforcement treated in the project ranges from 300 to 1200 kgf/cm^2 (4300 to 17400 psi) for concrete and from 4000 to 12000 kgf/cm^2 (57 ksi to 174 ksi) for reinforcement. In Fig.1 major research fields in the project are shown on a plane where vertical axis is yield strength of reinforcement and horizontal axis is concrete strength.

In the figure, small zones A and B surrounded by dotted lines correspond to areas for current common RC buildings and for high-rise RC buildings of about twenty through thirty stories constructed in the Minister-of-Construction's agreements specified in Article 38 of the Building Standard Law, respectively.

Zones I, II-1, II-2 and III which are zones treated in the project, cover very large area compared to two zones stated above. Therefore it is obviously unreasonable to think that we could understand mechanical behavior of New RC buildings simply by extrapolating the knowledge on current reinforced concrete structures.

So, in the project, theoretical examination of experimental data on current knowledge on high-strength concrete structures will be emphasized to construct new technology systems. This means that current technical knowledge on reinforced concrete buildings will also be re-examined in the project.

Because of the wide range of material strength, the character of researches in the project will be basic and the project may not yield much practical results in some zones far apart from the current reinforced concrete technology. Practical results are expected in Zones I and II-1, because these zones are comparatively close to the boundary of current technology.

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3. OBJECTIVES OF RESEARCH AND DEVELOPMENT AND FINAL RESULTS EXPECTED

Table 1 shows five objectives of research and development and the corresponding final results expected in the project. Some of the results will be applicable to refine current reinforced concrete technology. Guidelines for structural design or construction in the table will not give full details of technology but will give basic principles that are not enough for practical use.
It is because the guidelines which details technical specifications for structural design or construction often have a tendency to impede the development of relevant technology, and also future technical development will possibly change the high-strength concrete technology so much as we can not predict.

4. RESEARCH ORGANIZATION

Building Research Institute is in charge of conducting the project. Research committees are set up in Japan Institute for Construction Engineering (JICE) as shown in Fig. 2 to organize universities, private companies, etc. Three committees are assigned to determine research schemes, to coordinate research works, to promote the project and to integrate research results. The chairman of these three committees is Prof. H. Aoyama, University of Tokyo.
Technical Committees are in charge of making research programs in detail, implementing research works and integrating research results in five particular fields. The chairmen of the technical committees are Prof. F. Tomosawa, University of Tokyo for Concrete Committee, Prof. S. Morita, Kyoto University for reinforcement Committee, Prof. S. Otani, University of Tokyo for Structural Element Committee and Prof. T. Okada, University of Tokyo for Structural Design Committee. Construction and Manufacturing Committee will start in 1991. These committees have cooperative relations with various groups as shown in Fig.2.

5. ITEMS FOR RESEARCH AND DEVELOPMENT

Research items under consideration in each technical committee are as follows:

Concrete Committee

(1) Development of materials necessary for making high- and ultra high-strength concrete.
Quality standards of the materials.
(2) Physical properties of high- and ultra high-strength concrete.
(3) Mix proportion design, casting and curing works and quality control.

Reinforcement Committee

(1) Development of high- and ultra high-strength reinforcement. Mechanical properties of reinforcements.
(2) Mechanical properties of confined concrete.
(3) Constitutive equation of RC elements and application of the finite element method.
(4) Bond between concrete and reinforcement. Anchorage and arrangement of reinforcement.

Structural Element Committee

(1) Mechanical properties of beams and columns.
(2) Mechanical properties of shear walls.
(3) Effect of shear force on beams, columns and shear walls.
(4) Mechanical properties of beam-column connections and frames.
(5) Mechanical properties of foundations.

Structural Design Committee

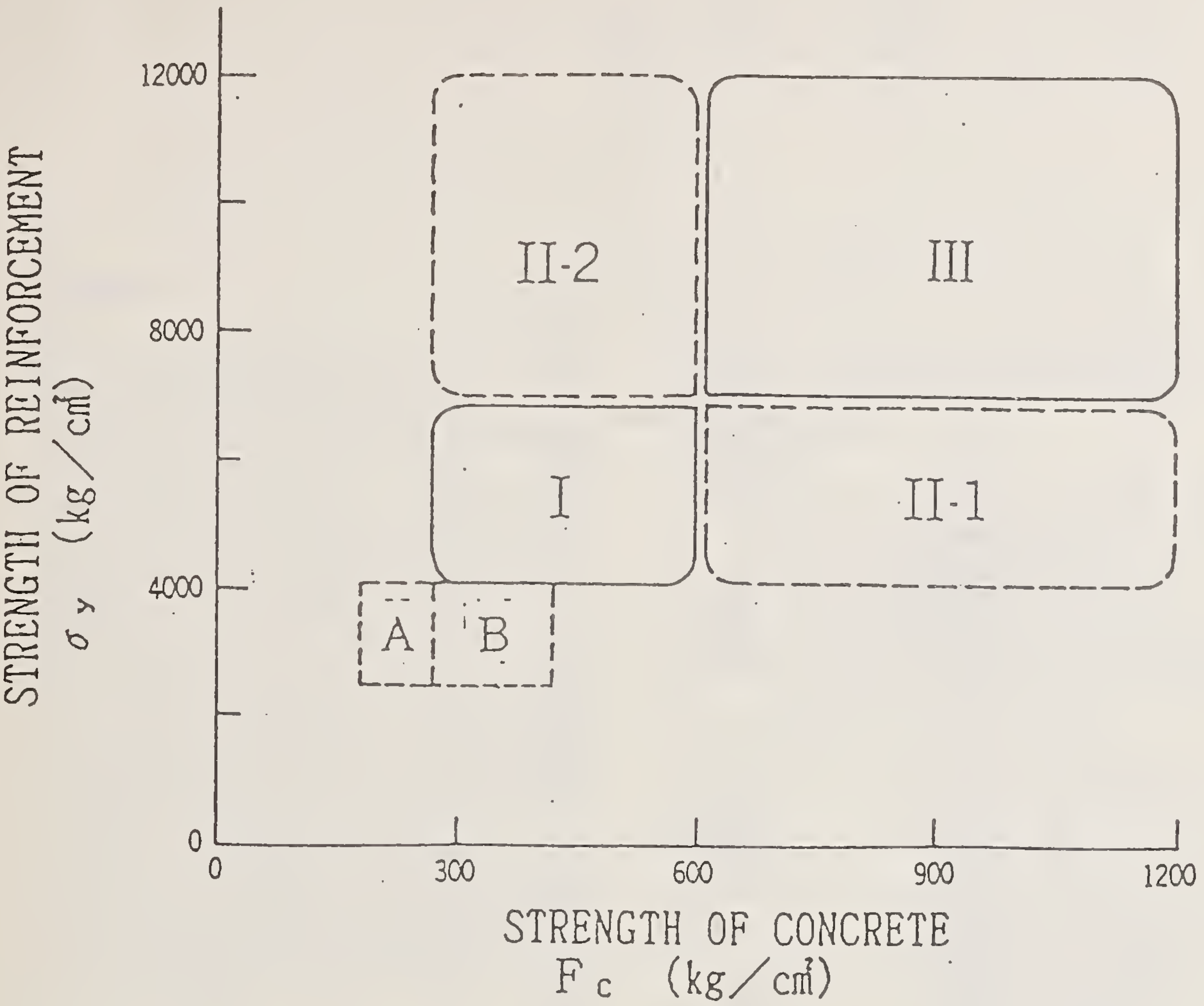
(1) Methods for modeling and analysis of behavior of structural frames in each zone.
(2) Practicable types of structural frames.
(3) Design loads and requirements for structural performance.
(4) Design methodology.

6. CONCLUDING REMARKS

The scope of the research and development in New RC Project includes research fields far beyond frontiers of current knowledge on reinforced concrete structures, where it is sure for us to encounter much difficulty.
In spite of that, the project dares to challenge the difficulty in order to know the future scope of reinforced concrete technology. This project has aroused great interest in private companies, universities, etc. and the cooperation and assistance proposed by those organizations will produce much fruit in this project.
One of the most important technology elements in high-strength reinforced concrete building engineering is related to quality control of concrete. In Japanese practice in the structural design of reinforced concrete buildings, the accuracy of quality control of concrete has never been reflected positively on structural design. This project will pay attention to the effects of the accuracy of quality control of concrete on structural design of high-strength reinforced concrete buildings.

Table 1 Objectives of research and development and final results expected

Objectives	Final Results Expected	
	For New RC	For Current RC
1) Development of high-strength and high-quality materials	· Methods for mix proportion method and quality control of concrete (Zone I) · Methods for production and arrangement of reinforcements (Zone I) · Guidelines for developing materials for Zone II and III	· Revision of design method for mix proportion and quality control method for concrete · Revision of reinforcement quality standards · Revision of upper limits of reinforement strength
2) Evaluation of properties of structural members and frames	· Methods of analysis	· Revision of current methods of analysis
3) Development of design and construction guidelines	· Structural design guideline (Zone I) · Construction guideline (Zone I) · Draft guidelines for structural design and construction (Zone II, III)	· Revision of structral design methods · Revision of construction standards
4) Feasibility study on RC buildings in Zone II-1	· New type high-rise buildings	
5) Feasibility study on RC buildings in Zone III	· New images of RC buildings	



ZONE : TYPE OF RC BUILDINGS & MATERIALS

A : LOW-RISE BUILDINGS IN COMMON
B : HIGH-RISE BUILDINGS CONSTRUCTED IN LAST DECADE
I : HIGH-STRENGTH CONCRETE & REINFORCEMENT
II-1: ULTRA HIGH-STRENGTH CONCRETE & HIGH-STRENGTH REINFORCEMENT
II-2: HIGH-STRENGTH CONCRETE & ULTRA HIGH-STRENGTH REINFORCEMENT
III : ULTRA HIGH-STRENGTH CONCRETE & REINFORCEMENT

Fig.1 Strength of materials and fields of research and development

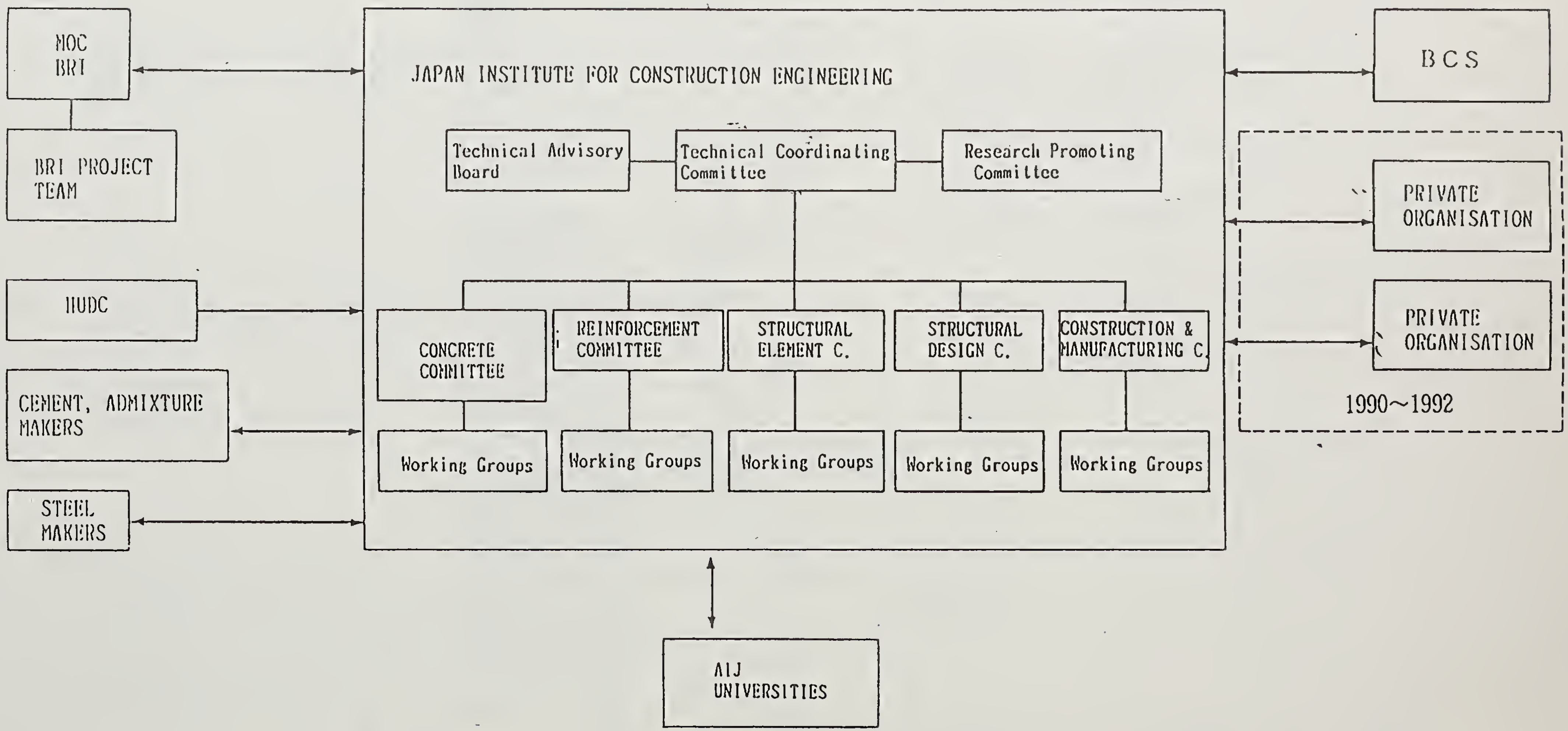


Fig.2 Organization for research and development

Guidelines for Identification and Mitigation of Seismically Hazardous Existing Federal Buildings

by

H. S. Lew[1]

ABSTRACT

This report, **Guidelines for Identification and Mitigation of Seismically Hazardous Existing Federal Buildings**, was prepared by the Interagency Committee on Seismic Safety in Construction in support of the National Earthquake Hazards Reduction Program, the President's plan to implement the Earthquake Hazards Reduction Act of 1977 (Public Law 95-124). The **Guidelines** are intended for consideration and use, as appropriate, by Federal agencies in their plans for mitigation of seismic hazards in existing buildings. Some Federal agencies have their mitigation plan in operation. It is not the intent of these **Guidelines** to supercede the existing plans.

KEYWORDS: buildings; earthquakes; earthquake hazard; evaluation; existing buildings; federal agencies; guidelines; mitigation; seismic safety; strengthening

1. INTRODUCTION

1.1 Background

Existing buildings that were not designed to be earthquake-resistant constitute one of the major potential hazards to life and property. A building may be vulnerable due to ground shaking caused by the earthquake, soil liquefaction, landslides, surface rupture, and also by tsunami. The huge inventory of existing buildings makes the reduction of the earthquake hazard a complicated and costly issue that must be worked out carefully with full consideration of the economic impact.

There are several factors which make it technically and economically difficult to identify and upgrade many buildings which could be hazardous in terms of life safety or post-earthquake operational capability:

(a) while most areas of the United States are susceptible to some level of seismic hazard, many areas have not required buildings to be designed to resist seismic forces;

(b) seismic design criteria have been upgraded over the years and it may not be feasible to bring all existing buildings, including those which were designed according to old codes but may not meet current codes, into conformance with the provisions of current codes; and

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(c) historically, some buildings not designed to current codes have performed adequately during earthquakes, while other buildings designed to specific codes have not performed satisfactorily.

This report provides guidelines to Federal agencies for use in their programs to identify, evaluate, and strengthen existing buildings to reduce the threat to life safety and to reduce major damage to critical and essential buildings.

1.2 Scope

The basic goal for strengthening of existing buildings is to provide life safety and, where necessary, post-earthquake operational capability. Buildings vulnerable to structural damage pose the most serious hazard to life safety. First priority must be given to identifying structures that might be susceptible to structural damage and/or collapse. Nonstructural components that could be a threat to life safety should be identified next.

Buildings exempted from investigation under these guidelines (Sect. 3.2.1) should be investigated for nonstructural hazards, but these investigations could be incorporated into regular maintenance inspections or even be a special investigation for buildings housing toxic and explosive substances.

This report presents a systematic methodology for identifying hazardous conditions and supplying decision makers with information on the extent of the hazard and the feasibility of mitigation. A viable strategy for mitigation of hazards can be developed and targets for implementation established.

Section 2 provides recommendations pertinent to implementing a mitigation program for hazardous buildings. Guidelines for prioritizing buildings and scheduling the seismic evaluation are presented. The final decision is left to the responsible agency based on budget constraints and overall agency programs.

Section 3 describes a procedure for identification of hazardous buildings and Section 4 describes qualitative evaluation procedures. Section 5 makes recommendations for requirements for levels of acceptable performance for existing buildings. Section 6 suggests mitigation techniques.

It is recognized that Federal agencies must have flexibility in dealing with hazardous buildings which can have significantly different characteristics. Additional constraints are imposed when a building is leased or built under a grant program instead of being owned by the agency.

1.3 Definitions

Critical Building - a building which, in case of "failure" may cause secondary effects such as release of toxic substances, fire or explosion.

Essential Building - a building which must be safe and useable for essential or emergency functions during and after a major earthquake.

Federal Building - a building owned, leased, assisted, or regulated by Federal agencies.

2. PROGRAM FOR SEISMIC EVALUATION OF EXISTING BUILDINGS

Implementing a program for seismic evaluation of existing buildings will depend upon availability of funds and the priority of mitigation action relative to other requirements. It would be generally infeasible, in terms of time and funding, for an agency to carry out detailed engineering analyses on every building in its inventory. Therefore, simplified methods will be required for identifying vulnerable buildings and needed retrofit measures. Priorities can be set only when there is knowledge of the degree of hazard, potential effect on current occupants or agency's mission, and the feasibility of mitigation.

Following is a recommended schedule which is reasonable and feasible:

a. All buildings located in NEHRP* seismic map areas 6 and 7 or UBC* seismic zones 3 and 4 and with occupancy of 100 or more people should be evaluated within five years.

b. All critical and essential buildings located in NEHRP seismic map areas 6 and 7 or in UBC seismic zones 3 and 4 should be evaluated within five years.

c. All other buildings in NEHRP seismic map areas 6 and 7 or in UBC seismic zones 3 and 4 should be evaluated within eight years.

d. All critical and essential buildings located in NEHRP seismic map areas 3, 4 and 5 or in UBC seismic zones 1, 2A and 2B should be evaluated within eight years.

e. Buildings in NEHRP seismic map areas 3, 4 and 5, or in UBC seismic zones 1, 2A and 2B should be evaluated, on a case-by-case basis, within twelve years.

f. All nonstructural components, which may be a threat to life safety, of buildings in NEHRP seismic map areas 3 through 7 or UBC seismic zones 1 through 4 should be evaluated within five years.

3. IDENTIFICATION OF HAZARDOUS BUILDINGS

3.1 General

The potentially large cost of an engineering investigation of all existing buildings in areas of seismic risk makes it necessary to approach the identification of hazardous buildings in a carefully planned manner. No single approach would meet the need of all agencies, because:

a. The number of buildings in agency inventories varies greatly.

b. The diversity of buildings; i.e., use, occupancy, size, location, type and age, complicates establishing a single approach.

c. Availability, completeness, and accuracy of information on each building vary widely.

d. The best strategy for prioritizing the investigation and mitigation efforts of each agency would not be uniform.

In general, buildings located in high seismic hazard areas should be evaluated first. The following screening factors may be considered in establishing an effective evaluation program.

* See references

3.2 Screening Factors

The first step in dealing with a large inventory of existing buildings is to apply a screening process that eliminates unnecessary evaluation and identifies buildings requiring further evaluation.

3.2.1 Primary Screening Factors

Buildings, except critical and essential buildings, which fall in any of the following categories need not be evaluated for earthquake vulnerability:

a. Those designs that meet or exceed the provisions of: the NEHRP Recommended Provisions, or the 1976 edition (or later) of the Uniform Building Code.

b. Those located in NEHRP map areas 1 and 2 or in UBC seismic zone 0.

c. One story wood-frame and one story pre-engineered metal buildings.

d. Buildings, except essential buildings, occupied by fewer than 6 people.

e. One- and two-family houses which are two stories or less.

3.2.2 Secondary Screening Factors

After the primary screening, agencies should set priorities for qualitative evaluation and retrofitting. As a minimum, the priorities should consider:

Seismicity
Site conditions
Structural types
Occupancy
Building use

Some methods for prioritizing structures are suggested in Appendix A.

3.2.3 Critical Buildings

a. All critical buildings located in NEHRP map areas 3 through 7 or in UBC seismic zones 1 through 4 should be evaluated. The type of evaluation will depend upon the nature of the hazardous substance, such as toxic chemicals and explosives, contained in the buildings.

b. Critical buildings located in NEHRP map areas 1 and 2 and in UBC seismic zone 0 need not be evaluated.

3.2.4 Essential Buildings

a. All essential buildings located in NEHRP map areas 6 and 7 or in UBC seismic zones 3 and 4 should be evaluated in the initial phase of program.

b. Essential buildings located in NEHRP map areas 3, 4, and 5 and in UBC seismic zones 1, 2A and 2B may be deferred from the initial phase of program.

In some instances, it may be more economical and reasonable to evaluate, at the same time, a complete complex consisting of a number of buildings in lieu of individual priorities.

4. EVALUATION OF EXISTING BUILDINGS

Qualitative Evaluation based on examination of available design documentation and field inspection shall be carried out for the structural system and for each exterior or interior nonstructural system or component which may pose a seismic hazard. It should be a two step approach: preliminary evaluation and detailed evaluation.

4.1 Preliminary Evaluation

The preliminary evaluation is intended primarily to reduce further the large inventories and avoid unnecessary investigation costs. It should determine, at the least practical cost, whether the structure provides an acceptable degree of safety or a more extensive evaluation is required.

For preliminary evaluation, as well as determining the acceptable degree of safety, ATC-21 (FEMA-154) Handbook on "Rapid Visual Screening of Buildings for Potential Seismic Hazard" (ATC-1988)* may be used. The Handbook provides a procedure for determining if a building needs further analysis without performing a detailed structural analysis.

4.2 Detailed Evaluation

A detailed evaluation of buildings not eliminated in the preliminary evaluation will be required to determine the earthquake resistant capacities of critical elements of the building and the extent of any deficiencies. ATC-14 "Evaluating the Seismic Resistance of Existing Buildings" (ATC-1987)* or ATC-22 "Seismic Evaluation of Existing Buildings" (ATC-1989)* may be used.

The ATC-14 methodology includes a state-of-the-art review of existing documents and has incorporated information from earlier methodologies. It is based on assumptions that one or more of the following events pose danger to human lives:

- the entire building collapses
- portions of the building collapse
- components of the building fail and fall
- exit and entry routes are blocked, preventing the evacuation and rescue of the occupants

The fundamental approach in ATC-14 is to ascertain whether there is a complete lateral resisting system with a coherent load path and whether appendages and veneer are properly attached. The adequacy of seismic performance of the structural system and components and exterior and interior nonstructural systems is expressed in terms of the Earthquake Capacity Ratio. The methodology is applicable to all parts of the U.S.

ATC-22, which is based on ATC-14, provides a step-by-step procedure for evaluating existing buildings. The methodology is to identify structural weaknesses that have been observed in past earthquakes to lead to failure and falling of components or to partial or total collapse, with an attendant loss of life.

4.3 Earthquake Capacity Ratio

The earthquake capacity ratio can be expressed in terms of the ratio of seismic capacity to seismic demand of critical structural members. The capacity to demand ratios can be computed using the procedure described in ATC-14, ATC-22 or other appropriate procedures.

The earthquake capacity ratios for the critical elements of the lateral force resisting system are indices of the structural

* See references

resistance of the existing building. The lower the values the higher the potential risks. If the lowest value of the earthquake capacity ratio is less than unity, the building should be considered a life safety hazard and so reported to the building owner.

5. REQUIREMENTS FOR MITIGATION OF HAZARDOUS BUILDINGS

Determine the level of acceptable performance for an existing building exposed to earthquake forces based on the predominant performance requirement imposed on the building, i.e., basic life safety of the occupants or post-earthquake operational capability.

5.1 Basic Life Safety Requirements

The basic life safety requirements for an existing building are that during a major earthquake it: 1) does not collapse, partially or totally and 2) performs adequately to provide unobstructed ingress and egress. This level is the minimum performance requirement for an existing building normally occupied by personnel and located in NEHRP map areas 3 through 7 or in UBC seismic zones 2A, 2B, 3, and 4. This level should provide life safety for the occupants, containment of hazardous or lethal contents, and a safe means of egress after a major earthquake. However, the damage incurred by the building may not be repairable. Recommended actions for the abatement of the structural deficiencies in buildings with basic life safety performance requirements are:

a. Where the building's earthquake capacity ratio as defined in Section 4.3, is 0.80 or greater, no action is required.

b. Where the building's earthquake capacity ratio is less than 0.80 but greater than 0.50, mitigation is recommended within 10 years.

c. Where the building's earthquake capacity ratio is 0.50 or less, mitigation is recommended within 5 years.

5.2. Post-Earthquake Operational Capability Requirements

Post-earthquake operational capability requirements are intended to provide continued operation or function of the building during and immediately after a major earthquake. This level is the maximum performance requirement for an existing building. Recommended actions for the abatement of the structural deficiencies in buildings with post-earthquake operational requirements are:

a. Where the building's earthquake capacity ratio is 0.90 or greater, no action is required.

b. Where the building's earthquake capacity ratio is less than 0.90 but greater than 0.50, mitigation is recommended within 10 years.

c. Where the building's earthquake capacity ratio is 0.50 or less, mitigation is recommended within 5 years.

5.3 Cost Impact Study

Perform a cost impact study to determine a reasonably accurate estimate of the total costs to strengthen the building. If the total cost exceeds projected budgetary constraints, the continued use of the building should be limited to storage; the building should be phased out; or other strategies, such as lowering seismic hazard exposure by changing occupancy requirement should be assessed carefully.

6. MITIGATION TECHNIQUES

6.1 Strengthening Methods

Strengthening the structural and nonstructural systems of an existing building may be a viable strategy for mitigating earthquake hazards. Various strengthening methods may be applied. The basic principles of earthquake-resistant design should be followed. It is recommended that the guidelines given in the Technical Manual of the Army, Navy, and Air Force "Seismic Design Guidelines for Upgrading Existing Building" or "Techniques for Seismically Rehabilitating Existing Buildings" by URS/John A. Blume & Associates* be used. These documents provide guidelines for upgrading both structural and nonstructural members. They provide the conceptual development for seismic upgrading and detailed techniques for strengthening structural and nonstructural elements with illustrated examples. They further provide guidelines for the cost effectiveness of upgrading existing buildings based on data obtained from the preliminary evaluation and the detailed structural analysis.

6.2 Quality Assurance Requirements

Whenever repair and strengthening procedures are implemented, quality assurance should be as rigorous as that required for new construction.

The requirements for inspection and material testing for new work also apply to modification of existing structural components or systems. However, special procedures are necessary to assure the quality of alterations involving those techniques which are no longer used in new construction. Therefore, the overall adequacy of a repair or strengthening program cannot be guaranteed by conformance of work to code and testing requirements for new construction alone.

APPENDIX A

METHODS FOR PRIORITIZING INVESTIGATIONS OF EXISTING STRUCTURES

A number of methods for establishing priorities have been developed. Each agency should determine the factors suitable to its purposes. The following brief descriptions of some methods are offered for information only.

a. The Naval Facilities Engineering Command has developed a computer program to search automated data files on structures and prioritize them by applying factors selected according to size, age, replacement cost and usage. This program can be easily expanded to include other factors if desired.

b. "Handbook on Establishing Priorities for Seismic Retrofitting of Buildings" by Federal Emergency Management Agency.

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* See references

7. REFERENCES

ACI 318, "Building Code Requirements for Reinforced Concrete, ACI 318-83", American Concrete Institute, Detroit, Michigan 1983.

Army, Navy, Air Force, 1988, Seismic Design Guidelines for Upgrading Existing Buildings, Departments of the Army (TM-809-10-2), Navy, (NAVFAC P355.2), and the Air Force (AFM 88-3, Chap. 13, Sect. B), Washington,D.C.

ATC, 1987, Evaluating The Seismic Strength of Existing Buildings, Applied Technology Council Report ATC-14, Redwood City, CA.

ATC, 1988, Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, Applied Technology Council Report ATC-21 (FEMA-154), Applied Technology Council, 3 Twin Dolphin Drive, Redwood City, CA 94065.

ATC, 1989, Draft Seismic Evaluation of Existing Buildings, Applied Technology Council Report ATC-22, Redwood City, CA 94065.

NEHRP, 1988, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Building Seismic Safety Council, Washington, D.C.

UBC, 1988, Uniform Building Code (Revised Every Three Years), International Conference of Building Officials, Whittier, CA.

URS/John A. Blume & Associates, Draft Techniques for Seismically Rehabilitating Existing Buildings, Federal Emergency Management Agency, Washington, D.C. 1989.

U.S. Army Corps of Engineers Seismic Research Program

by

William E. Roper[1]

ABSTRACT

The Corps Research Program is focused on addressing the seismic problems of the Civil Works Water Resource Program. It includes seismic challenges related to new construction as well as operating existing structures and repair, maintenance and rehabilitation of existing structures. The research program is developed cooperatively with field users, the laboratories, and headquarters program managers. The research program has four primary areas: (1) liquefaction potential of fine-graded soils, (2) seismic response of concrete dams and abutment structures, (3) earthquake hazard evaluations for engineering sites, and (4) improvement of foundation soils susceptibility to liquefaction.

KEYWORDS: Liquefaction; Seismic; Earthquake.

1. INTRODUCTION

The Corps Seismic Research Program is very applied in character and carefully structured to meet the highest priority needs of the Civil Works program. To ensure proper prioritization of the research program, close coordination between the major user and developer organization is required. In the Corps of Engineers, these organizations are the headquarters Research and these organizations are the headquarters Research and Development Directorate, the field District offices, the laboratories, and the headquarters Technical Monitors (representing the functional headquarters directorates other than R&D). Changes this past year to effect better coordination among the four principal organizational elements have resulted in the establishment of field review groups for all research program areas. The field review groups are composed of project engineers and managers who are specialists in the seismic and earthquake engineering areas from representative Corps districts across the United States. They are individually recommended by the headquarters R&D Directorate and appointed by the field Division Engineers. They participate in an annual program review of the Seismic Research Program with the laboratory specialists and headquarters Technical Monitors. This annual meeting reviews the content of the proposed program and establishes the priority of individual work units within the overall program. An illustration of the principal elements involved in the program development and the interrelationships is shown in Figure 1. This approach is proving very effective in ensuring that field input is incorporated into the research program. The field review group members also act as technology transfer agents to assure that the new technology tools are effectively utilized in the field. This has resulted in a much closer working relationship between the field, laboratory and headquarters Technical Monitors.

The current Seismic Research Program has four major components that will be summarized separately in the remainder of the paper.

2. EARTHQUAKE HAZARD EVALUATION

This effort is a part of the Rock Research Program that is conducted through the Geotechnical Laboratory at the U.S. Army Waterways Experiment Station.[2] The objective of this program is to provide earthquake, geologic and seismic logic evaluation procedures for estimating site specific earthquake ground motions for engineering evaluations at major Corps projects. It is primarily focused on evaluating the durability of dams where failures would be catastrophic. The Corps need reliable methods to assess the sources and the maximum potential for earthquakes. We must also develop methods in order to judge the susceptibility of sites to the effect of earthquakes, assign site-specific earthquake ground motions, and be able to generate accelerograms and response spectra for the engineering analyses.

The current work expands the Waterways Experiment Station collection of strong-motion earthquake records and develops techniques for field evaluations for estimating susceptibility of sites. This work will also generate parameters for earthquakes, ground motions and develop characterizations of ground motions for engineering analysis. These characterizations will be used to improve computer program models for generating synthetic seismograms.

Some specific accomplishments from this research include seismic zoning diagrams for the eastern United States as shown in Figures 2 and 3. The zone diagrams provide guidance to our field organizations for seismic design considerations. Also, charts for earthquake ground motion related to damage and for ground motions related to earthquake magnitude have been developed, using available data. A methodology for assessing susceptibility of a foundation soil to liquefaction by earthquake shaking has also been developed.

Milestones during FY 1989 include publication of a catalog of recommended accelerograms and development of accelerograms for hard rock formations. This effort is scheduled to be completed in 1991.

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3. SEISMIC RESPONSE OF CONCRETE DAMS AND ABUTMENT STRUCTURES

This effort is part of the Structural Engineering Research Program at the Waterways Experiment Station.[3] The research is addressing the coupled response of the structure-reservoir-foundation system to earthquake ground shaking. The results will be used to revise the Corps criteria for seismic analysis of concrete dams and abutment structures. The current Corps criteria assumes linear elastic behavior of the structure and loads as determined by the seismic coefficient method. Recent research indicates that these criteria do not adequately consider the locally expected earthquake motion, the dynamic behavior of the structure, the hydrodynamic response to the reservoir, the dynamic effect of the foundation, and monolithic linear behavior of the structures concrete.

The analysis procedure that has been developed is applicable to seismic analysis of nonoverflow monoliths for concrete gravity dams. The analysis procedure includes the dynamic behavior of the structure, hydrodynamic response of the reservoir, dynamic behavior of the foundation, and absorption of the reservoir bottom. A rigorous time history finite element analysis program was developed to include behavior of the foundation and reservoir interactions. This analysis procedure has demonstrated that tensile stresses significantly change form slight variations of the input variables. Therefore, it is important to understand the importance of each part of the problem, such as foundation interaction, and appropriate values to correctly model them nominally.

A simplified procedure taking into account the multiple parameters has been developed into a computer program titled "SDAM." SDAM (Simplified Dam Analysis) is an interactive computer program which performs a seismic analysis of nonoverflow monolith of a concrete gravity dam. The hydrodynamic loads are developed by using a simplified procedure developed by Dr. A. K. Chopra, University of California at Berkeley. The hydrodynamic and gravity loads are calculated based on an input response spectra or a given spectra acceleration.

SDAM generates a finite element grid internally from the six dimensions shown in Figure 4. This grid is never seen by the user so the user does not have to be concerned with node and element number. SDAM uses the simplified loads and applies them to the finite element grid and then calculates stress values at the centroid of each element. The centroidal stresses are then extrapolated to produce stresses on the upstream and downstream faces of the dam as shown in Figures 5 and 6. Since the maximum seismic stresses occur on the surface of gravity dams, these plots are valid for determining levels of stress occurring in the structure.

Specific milestones for this year include documentation of a 3-D analysis for spillway monoliths, and publication of a technical report on Chopra's simplified procedures for spillway analysis.

4. LIQUEFACTION POTENTIAL OF FINE GRAIN SOILS

The liquefaction effect is part of the soils programs conducted at the Waterways Experiment Station.[4] The objective of this research is to develop criteria and methods for evaluating the liquefaction susceptibility of fine grain soils. This will address a problem relevant to a number of Corps dams which are located in seismicly active areas and are underlaid by fine grain or plastic alluvial soils. Presently, the cyclic strengths have been judged against standards developed for clean sands. Recently, available data on liquefaction occurrences show that fine-grained soils may also be susceptible to liquefaction. A better understanding of the limits of applicability of present analysis methods to such soils and what modifications of these methods is needed.

The research approach will include: (1) evaluation of field occurrences of fine-grain soil liquefaction, (2) development of a laboratory test program to assess dynamic strength and liquefaction susceptibility of fine-grain soils on the basis of plasticity, grain size distribution, and density, and (3) analysis of test results and developed procedures for determining in-situ liquefaction potential in soils containing various amounts and types of fines. The effort will include evaluation of field occurrences of clayey soil liquefaction worldwide as to ground motion characteristics, in-situ test data, and engineering properties. This will include a compilation and analysis of data on dynamic laboratory test for materials close to the "A" line on the Casagrande plasticity chart. The study will also include the examination of data bases at various geological earthquake engineering research institutes in the People's Republic of China.

During 1989, 576 isotropically-consolidated cyclic triaxial tests, on controlled-mixture specimens will be conducted to evaluate functional relationships between liquefaction resistances and fines type and content. Hollow

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cylinder cyclic torsional stress tests will also be conducted to assess an appropriate factor to convert "conventional" cyclic triaxial strengths to cyclic simple sheer strengths. This work is being accomplished under contract with the University of Colorado at Denver.

5. IMPROVEMENT OF FOUNDATION SOILS SUSCEPTIBILITY TO LIQUEFACTION

This research is being conducted as part of the Repair, Evaluation, Maintenance and Rehabilitation (REMR) research program at the Waterways Experiment Station.[5] This study is to determine the feasibility and effectiveness of various techniques for improving liquefiable foundation conditions under existing structures and to ensure safety against failure due to earthquake excitation. The research results will be of direct benefit to the Corps dams where current evaluations reveal inadequate margins of safety against earthquake-induced liquefaction. The research may offer cost effective alternatives for remedial action to improve earthquake-safe dam foundations.

Guidelines for remedial action have been developed as a part this research effect and published as a technical note GT-SR-1.2 in the REMR notebook.

6. CONCLUSION

The requirements for practical, applied engineering tools for seismic design will continue to grow, particularly in the area of remedial action. In developing these engineering tools, a partnership between the field user and laboratory researcher is important for providing review and feedback to the research program. This partnership also assists in transferring the new technology into field application.

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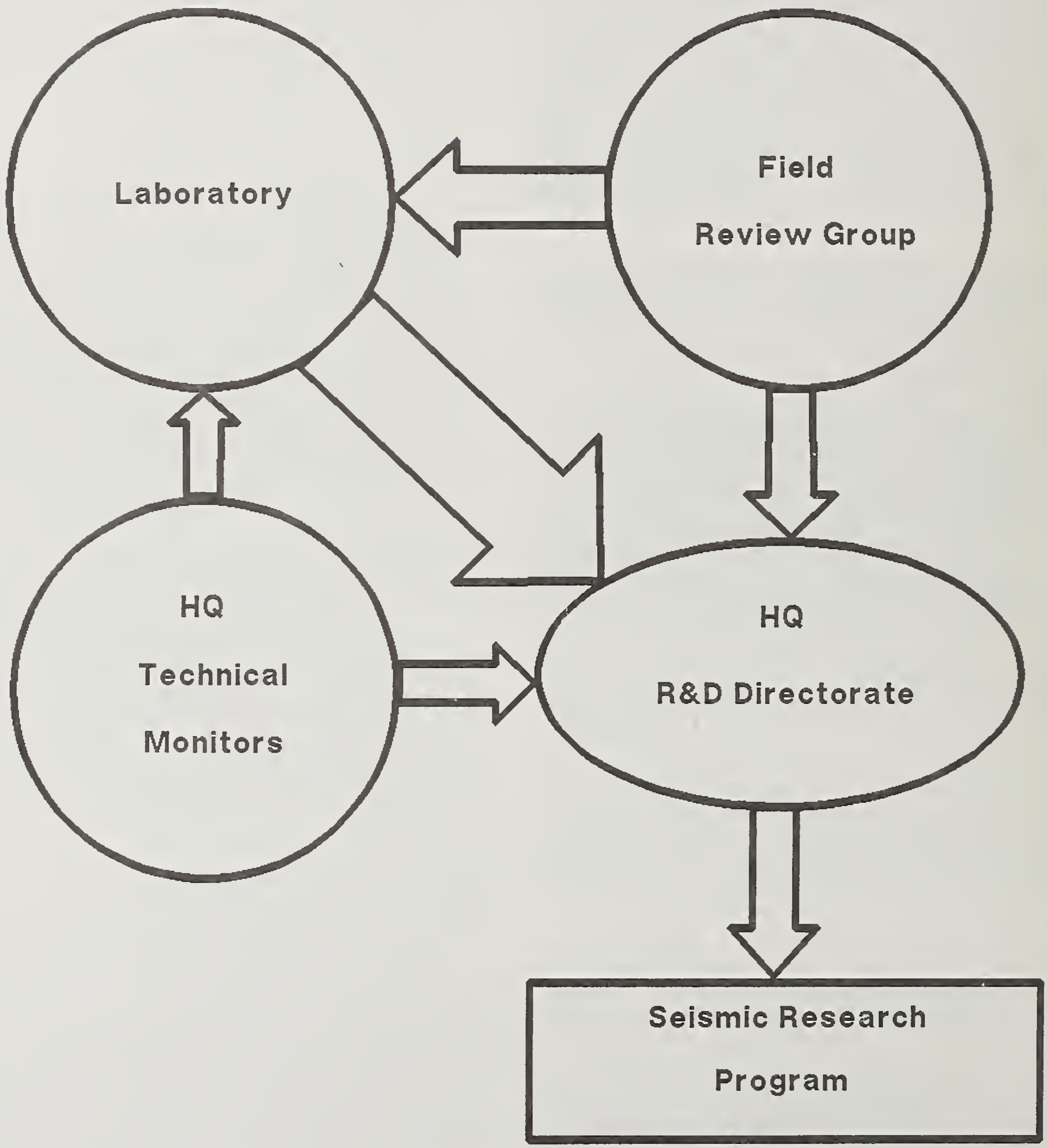
Figure 1. Research Program Development Process



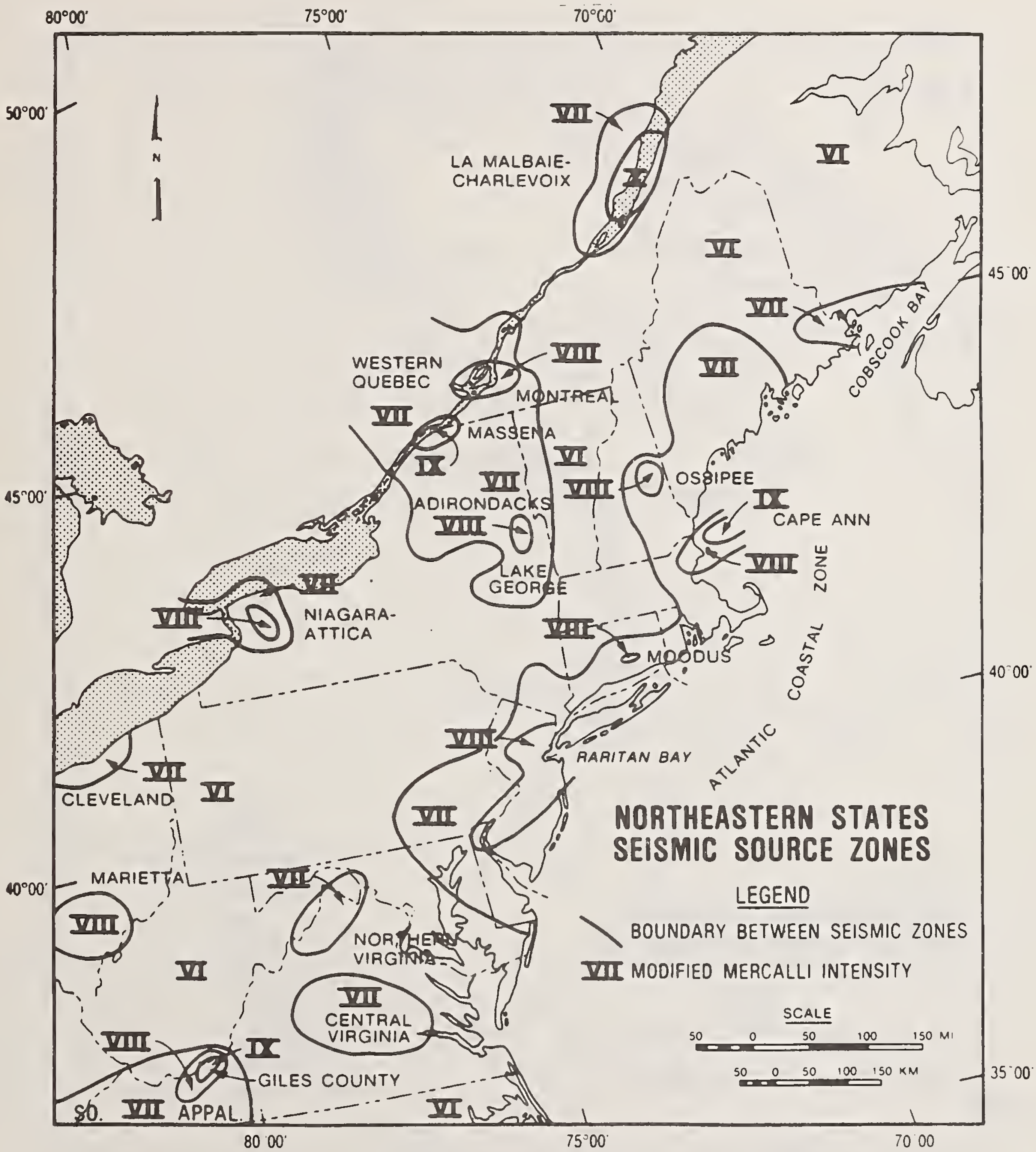


Figure 2: Seismic Source Zones for Northeastern United States

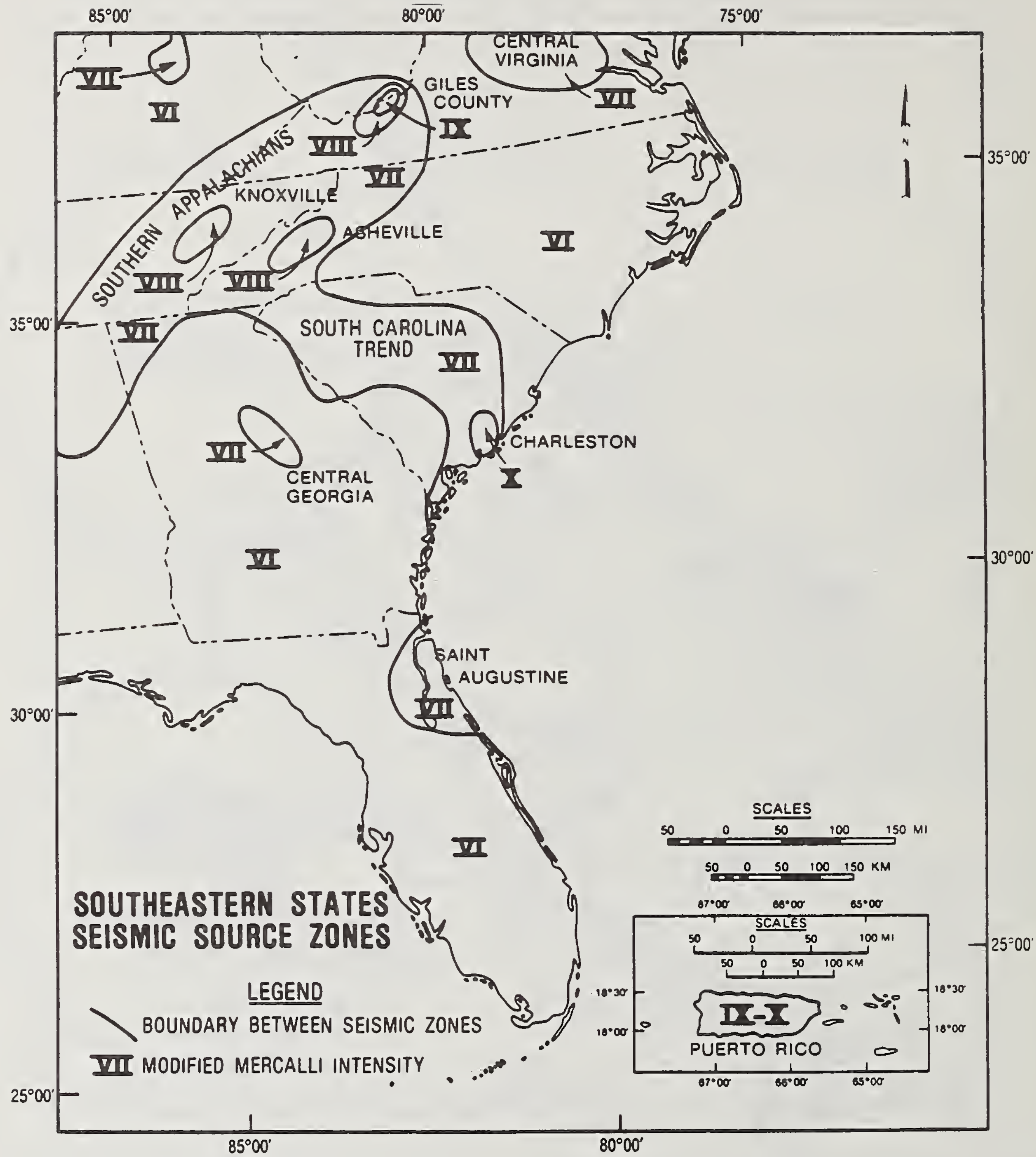


Figure 3: Seismic Source Zones for Southeastern United States

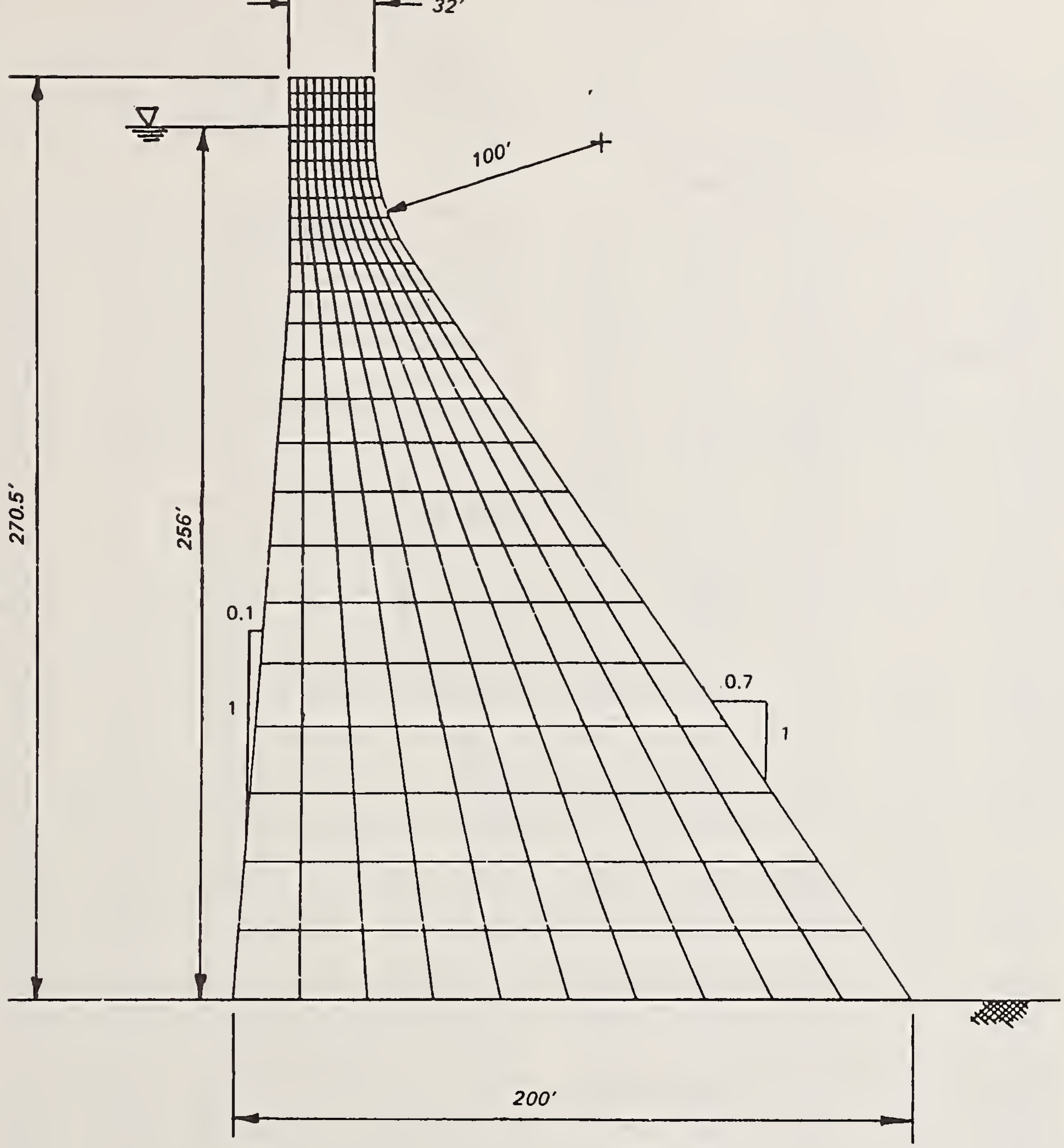


Figure 4: SDAM Finite Element Grid System

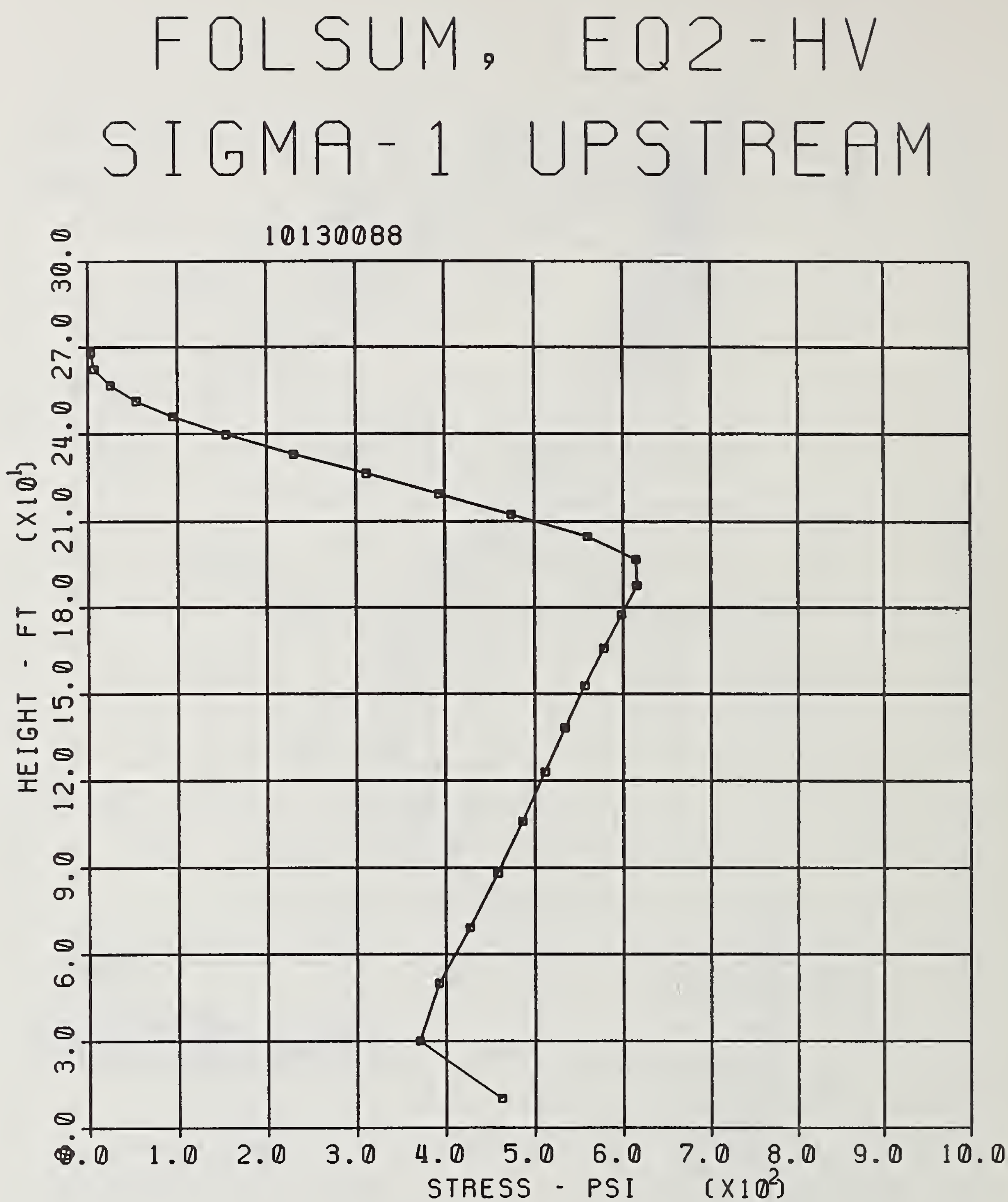


Figure 5: Stresses on Upstream Face of Folsum Dam Using SDAM

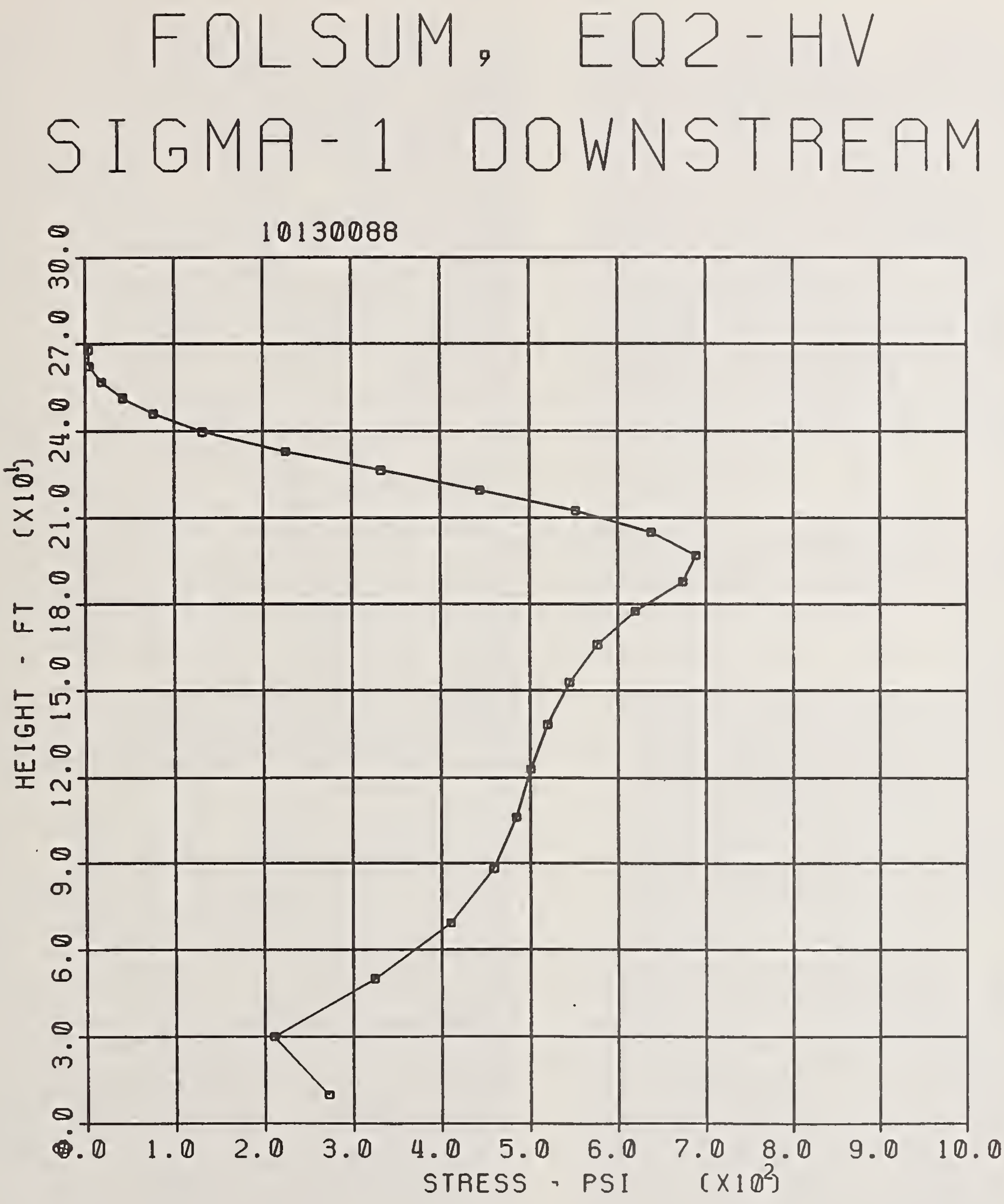


Figure 6: Stresses on Downstream Face of Folsum Dam using SDAM

The History and Status of Strong-Motion Earthquake in Japan

By

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SUMMARY

The strong-motion earthquake observation in Japan has the progress history over 35 years. Japan has about 1,000 observation sites and more than 2,000 strong-motion seismographs. However, that whole situation is not always understood to the full. Strong-Motion Earthquake Observation Council was conducted the Distribution plan for the target to push on with installation after 10-15 years by the grasping of that whole situation as possible as they can. This report describes the general view of the history and present status on strong-motion earthquake observation and the outline of the distribution plan in the future.

Keyword

Strong-motion earthquake observation, Strong-motion Earthquake Observation Council, Distribution plan, Observation network.

1. INTRODUCTION

The strong-motion earthquake observation in Japan, which try to catch the seismic motion during strong earthquake without scale-over, has passed away over 35 years. At the beginning age, the concerned organizations of strong-motion earthquake observation were scarcely and the whole situation was grasped fully. By the increasing of observation sites, manifold of observation equipment, complexity of the concerned organizations nowadays, it was very difficult to understand the present status of the network of strong-motion earthquake observation at nowadays. This report describe the outline of present status of strong-motion earthquake observation in Japan and the distribution plan for the target to 10-15 years in the future placed under the making clear the state of observation through the reconsideration works of "distribution Plan of the Strong-Motion Earthquake Observation" by conducting the Strong-Motion Earthquake Observation Council (SMEOC). And also, we discuss that progress history by considering with the lacking of detailed discussion of that history and adjusted literatures.

2. THE PROGRESS HISTORY

The persons concerned in trial construction of a seismograph obtaining such strong-motion records held meetings, the "Committee for the Standard Strong Motion Accelerograph" was organized, and the actual work of this committee was started after the grant-in-aid for Developmental Scientific Research from the Ministry of Education in the fiscal year 1951 had been obtained. This instance was the starting point for the strong-motion earthquake observation in Japan. Thus, in March 1953 a prototype of a seismograph for measuring the acceleration of strong-motion earthquakes was completed, and it was named "SMAC type" after the initial letters in the name of the committee. This accelerograph No.1 was installed in an underground vault of the Earthquake Research Institute of the University of Tokyo, and the observation was started on June 29, 1953. On the other hand, by the Grant-in-aid for Architectural Technique Research from the Ministry of Construction in the fiscal year 1954, another type of strong-motion seismograph was designed by the same committee, according to specifications a little simpler than those of the SMAC type, and this new type was accomplished in May 1955 and named "DC type".

On October 9, 1953 the work of making the project for observation of strong-motion earthquakes was commenced by the Seismological Subcommitte, Disaster Prevention Committee, Resources Council, Prime Minister's Office, and was completed as "Recommendation concerning the Project of Strong-motion Earthquake Observation" on January 31, 1955, which was submitted to the Prime Minister. This recommendation demand the realization of the following points.

1) To complete the facilities for observation of strong-motion earthquake in three years.
2) To establish an organization for liaison between observers and to entrust this organization with agreements about observational operations, exchange of observational informations and data,

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and with carrying out of various sorts of study.
3) To take appropriate measures for obtaining the budget which is required for measurement of strong-motion earthquakes and for administration of the said liaison organization.

With this recommendation as a momentum, into the budget under the jurisdiction of the Ministry of Construction there were included the expenses necessary for examination the antiseismic resistance of buildings, and 25 strong-motion accelerographs were additionally installed in the fiscal year 1956 and 1957. Further, as a result of encouragements concerning establishment of strong-motion accelerographs, given by the Seismological Society of Japan, the Architectural Institute of Japan and the Japan Society of Civil Engineers, the number of accelerographs set up in buildings by civil expenses was gradually increased. In 1958, an accelerograph of SMAC type was set up in a dam for the first time.

With the increase of the number of accelerographs as is mentioned above, the records of measurements began to be accumulated, and the necessity of materialization of the liaison organization mentioned above in 2) and 3) was more and more increased. For some time after the said recommendation the Seismological Subcommittee of the Resources Council played the role of the liaison organization, but with the increases in the number of accelerographs and records, a stronger organization became indispensable. For this reason, apart from the Resources Council, the Strong-Motion Earthquake Observation Committee was organized, and commenced its activity from December 21, 1956. However, this committee had no definite estimate at all, so that the maintenance of accelerographs and measurements were tolerably continued by favourable cooperation from manufactures of accelerographs and civilians and by a part of Grant-in-aids for Developmental Scientific Research allocated to the Earthquake Institute of the University of Tokyo. And then, in November 1958 the Resources Council was presented "the Report on Strong-Motion Earthquake Observation" to the Minister of State for Science and Technology by the collection of state of the strong-motion earthquake observation under the considering the re-examine stage of the central mechanism for this observation, and demand the promotion of the work of strong-motion earthquake observation. By the efforts of the persons concerned in this committee, "Strong-Motion Earthquake Records in Japan, Volume 1" was published in March 1960, and thereafter the style of this publication has been regarded as the standard of record publication in Japan.

It happened that on June 16, 1964, the Niigata Earthquake occurred and a considerable number of buildings and bridges were damaged from destruction of the soft ground of their foundations and became unusable. From this fact, demands were made for the establishment of earthquake-proof construction technique on the soft ground. In addition, a strong-motion accelerograph, which had been installed by the chance in a building, accomplished its function and furnished valuable scientific data which are considered to clarify the dynamic properties of loose soil and the changing process of buildings in Niigata, and this fact reaffirmed the necessity for increasing of accelerographs. Motivated by these circumstances, the recommendation entitled "On Intensification and expansion of Antiseismic Engineering Study" was submitted to the Prime Minister from the Science Council of Japan on November 17, 1964. This recommendation demanded the following measures:
1) Increase in the number of strong-motion seismographs for engineering use and practical utilization of the data.
2) Special measures for establishment of antiseismic engineering technique on the soft ground.
3) Intensification and expansion of education and study of earthquake engineering and soil engineering at the universities and institutes.

After this recommendation was submitted, in the fiscal year 1965 the Strong Earthquake Observation Center was established in the Earthquake Research Institute, University of Tokyo, and a part of the expenses necessary for the administration of Strong-Motion Earthquake Observation Committee was the first time recognized to be defrayed at national expense. Motivated by the Niigata Earthquake, however, while on the one hand the establishment of accelerographs became again active, on the other hand an increase in the budget for the Strong Earthquake Observation Center was not to be expected, and therefore the necessity of intensification of the said liaison organization was afresh keenly felt.

Thus, on June 21, 1967, the

Strong-Motion Earthquake Observation Council was established, the expenses for its administration being recognized as as estimate allocated to the National Research Center for Disaster Prevention, Science and Technology Agency, and this council was now able to promote the project of strong-motion earthquake observation in cooperation with hitherto existing Strong-Motion Earthquake Observation Committee. This council as a liaison organization has been preparing a plan of distribution of strong-motion accelerographs all over the country and making an effort to secure the budget corresponding to the increase in the number of accelerographs and records.

The Strong-Motion Earthquake Observation Council, in compliance with the demands from every field of life, has collected the data concerning all of observation points where strong-motion accelerographs are installed all over the country and registered them in the "Resister Book". This book contains the data as of March 1969, and the additional ones have been supplemented. In 1972, the Strong-Motion Earthquake Observation Council presented a future plan of distribution of strong-motion accelerographs with the consideration of the importance in distributing them in the vacant area of the country so as to record the strong-motion earthquakes which will occur at any part of Japan, and also in measuring the ground vibration of various types of soil conditions during strong-motion earthquakes. The standards of distribution recommended in this plan are as follows:

1) Each of the prefecture should have 20 stations as an average expect Tokyo, Osaka and Hokkaido, and the stations in each prefecture should be distributed as uniformly as possible.
2) In a city having more than 200,000 population, the number of accelerographs should be increased according to the population, because of the importance of the prevention of earthquake disasters in such a congested area.
3) In principle, two or more accelerographs should be installed at each stations.

According to these standards the total number of accelerographs will be 2,100. The plan gives the priority of the installation to the ground having especially soft soil and to the region where a high probability of the occurrence of an earthquake disaster is expected.

At the almost same time for the study of distribution plan as above mentioned, the Science Council of Japan presented the Demand entitled "On Research Promotion for the Prevention of Seismic Hazards of the Constructions" to the Prime Minister on October 1971. On the occasion of presentation of this demand, the Society of Construction Workers, the Architectural Institute of Japan, the Japan Society of Mechanical Engineers, the Japan Society of Agricultural Civil Engineering, the Japan Society of Civil Engineers, the Seismological Society of Japan, UNESCO and the other societies were presented the demand of almost same contents to the authorities concerned. It was strongly requested the following aims in that demand which will be practised the appropriate measures without delay at the government office concerned:

1) Propulsion of installation of strong-motion accelerographs and intensity of management structures.
2) Special measures for the promotion of antiseismic engineering research for the general structures include the ground.
3) Promotion of the research for the establishment of the new design criteria of the buildings, such as a piped type structure, off-shore structure, plant structure, which were not experienced the strong earthquakes.

With the stimulation of the active cooperation between the organizations concerned with the strong-motion earthquake observations, the recommending with the installation of accelerographs for the high rise buildings based on the administrative advice of the application to the Article 38 of the "Building Standard Code",the number of the installations increased remarkably by the private organizations. The observation points were remarkably concentrated to the big cities ,such as Tokyo, Osaka. And, the new problems, which the maintenance and keeping of accelerographs could not followed up by the rapid increasing of the number of accelerographs,were arise. On August 1974, the Administrative Management Agency was done the administrative inspection to the aseismic measures at the big cities. Based upon this inspection, the following recommendation was presented to the Ministers of related organizations.

"The actual data for the propagation of seismic waves and the dynamic behaviour of structures are not so enough, because the dynamic anti-seismic design method is newly developed. To improve the

the following urgent measures,

1) The adjustment of the strong-motion earthquake observation network which can be caught the large earthquakes occurred at any regions.
2) The strengthening of the organization for the collection and analyses of the observed data.
3) The appropriate maintenance of accelerographs and the advice of accelerograph installation for the public or important structures.

By the considering with the progress of earthquake engineering and engineering seismology, the Resources Council was looked over again the implementing condition for the recommendation presented on 1955, and presented the new recommendation entitled "On the strong-motion earthquake observation for the necessity to estimate the seismic risk", which was putted stress on the promotion of observation at the ground, to the Minister of State for Science and Technology. This recommendation was requested the following measures;

1) To adjust and expand the network at ground for the understanding of characteristics of seismic motion during large earthquakes at the different soil conditions of different areas.
2) To attempt the improvement the recording accuracy for the grasping of totally properties of strong motion.
3) To attempt the adjustment of the collection and management systems for the observed records.
4) To attempt the adjustment of the regulation and propulsion systems intentionally and synthetic to the totally researches and developments for the ground motion during large earthquakes.

The International Workshop on Strong-Motion Earthquake Instrument Arrays was held under the sponsorship by the National Science Foundation and UNESCO, and the auspices of the International Association for Earthquake Engineering in Hawaii in May 1978. The goal of this workshop was to develop a a workable plan for the possible future deployment of dense strong-motion earthquake instrument arrays with primary emphasis on ground motion studies. The following resolutions, should be implemented, was approved;

1) The International Association for Earthquake Engineering in collaboration with the International Association of Seismology and Physics of the Earth's Interior form an International Strong Motion

The Ministry of Construction was recognized the expansiveness of the current accelerographs, the complication of maintenance and replaying of the records, was set about the development of the popular edition accelerographs of the having the definite accuracy and low cost, and also having the easy treatment by using the new technology focused to the electronics on June 1979. This development was conducted based on the regime of the technical estimation of construction, 5 manufactures were subscribed to this development. As the result of the estimation, these 5 accelerographs from 5 manufactures were approved to reach the developmental targets. These targets were instituted to the following;

1) To be able to record simultaneously 2 horizontal components and 1 vertical component.
2) To be able to record the acceleration in the range between 30 to 1,000 gals in the frequency range 0.3 - 15 Hz, and to prepare the replay or digitized systems with the interval less than 1/100 sec.
3) The accuracy of the replay or digitized acceleration is not exceeding with (recorded acceleration X 10 % + 10 gals).
4) To be able to record several times, and maintain the operating time of more than 2 minutes by the one start.
5) The starting is done by the vertical acceleration, and it is able to set up the starting acceleration with the range 5 - 20 gals.
5) The cost should be sett low-priced.

By the reason of the necessity to increase the dense-array strong-motion earthquake observations, the Strong-Motion Earthquake Observation Council examined the new type of seismograph. In October 1983, after two years discussion, the council published the report relating the new type of strong-motion seismograph. The main specifications of this seismograph has the following;

1) To be able to record the seismic motion of acceleration with the range of 0.1 -2,000gals.
2) To be able to record faithfully within the frequency domain of 0.05 - 25 Hz.
3) To be able to record the absolute time data with the accuracy of 1/1000 sec simultaneously with the seismic motion.
4) To be able to record from the initial movement part by using 5 sec delay equipment.

5) To be able to record the same timing sampling and same absolute time by the connection with plural seismographs.
6) To easy use the analysis of record from using the digital cassette tape.

Based on the present analysis of the some conditions of development of strong-motion earthquake observation, the Strong-Motion Earthquake Observation Council was established the working committee to study the new distribution plan for the index of the project of strong-motion earthquake observation for the target of 10 - 15 years in the future in 1983. In September 1989, this council was published the report of distribution plan of strong-motion seismographs". The following sections of this report is explained the present status and the future plan of installation of seismographs based on that report.

As is mentioned above, the SMAC type accelerograph was developed in 1953. Based on the experiences of the use of SMAC type in the subsequent period, improvements upon this were put one after another. The SMAC type has now 12 variations: A, A-2, B, B-2, C and C-2, which are waxed paper type, and D, E, E-2 and Q, which are scratch record film type, and M and T, which are analogue cassette tape type. To correspond with the digital measurement, the Ministry of Construction was developed new SMAC-MD in March 1989. The main properties of this digital instrument are as follows;

1) To be able to measure 9 channels for the maximum.
2) To be able to increase the capacity of the recording memory. to be able to max. 4 Mbyte by using 1 Mbyte IC memory card.
3) To be able to read the maximum acceleration value in a moment.
4) To be able to transmit the seismic information.
5) To have the compensation of 3 hours stoppage of electric current.

3. PRESENT STATUS

As is mentioned above, the history of strong-motion earthquake observation is nearly concerned with the project by using the SMAC type accelerographs. Nowadays, the whole condition of the strong-motion earthquake observation is limited to the condition of using SMAC type accelerographs. It is the present status that the whole situation off using the other type seismographs, such as moving coil type, servo magnetic type and so on, is not arranged. Therefore, the Strong-Motion Observation Council was investigated the almost 70 % of the whole condition in cooperation with the manufactures.

Fig. 1 shows the distribution of SMAC type accelerographs (include DC type, and so forth) registered to the Strong-Motion Earthquake Observation Council as of March 1989. Fig. 2 shows the increasing trend of installed accelerographs,which include the non-registerd accelerographs. the total number of accelerographs is reached to over 1,500 instruments. Fig. 3 shows the number of recorded earthquakes. More than 1,500 earthquakes were recorded in the over 35 years till the fiscal year 1989.

To investigate the distribution of seismographs all over the country,the map which was devised to the mesh from the origin located Nihonbashi, Tokyo, was made. We made the 2 kind of mesh map, 25 km X 25 km mesh (25 km mesh map) and 50 km X 50 km mesh (50 km mesh map). We did not considered the part of small island area. Fig. 4 and Fig. 5 show the 25 km and 50 km mesh lines superposed on the map of Japan, respectively. Figs. 6 and 7 show the outline of Japan by the mesh map, respectively. 25 km mesh map has 644 meshes, and 50 km map has 177 ones.

The installed area of SMAC type accelerographs illustrate to the 25 km mesh map in Fig. 8. The mesh ,which has more than one SMAC at least, is counted to 198, the ratio is 30.6 %. Fig. 9 shows the same figure to the 50 km mesh map. The installed meshes are 112, the ratio is 63.3 %.

Now, we use many kind of seismographs for the observation at the same to the use of SMACs. Fig. 10 shows the installed area of all kind of seismographs. The installed meshes of 25 km map are 292, the ratio is 40.5 %. The vacant meshes are 485. Fig. 11 shows the same manner map of 50 km mesh. The installed meshes 146, the vacant meshes are 31.

4. DISTRIBUTION PLAN OF SEISMOGRAPHS

The Strong-Motion Earthquake Observation Council was published the distribution plan on February 1972, as mentioned above. This plan is recommended the 2,100 accelecographs installation all over the country. During 25 years from the publishing this plan,the installation and collection of records were smoothly underwent. Nevertheless, based on the consideration of the more increase trend of concentration of the seismographs at the big city, the developments of the research for the seismic risk and the earthquake prediction, it was important thing to reconsider the distribution by the base of efficiently and economically observation. The Strong-Motion Earthquake Observation Council was established the "sub-committee for the examination of new distribution plan" in November 1983. As the result of examination, the council was adjusted the report on the distribution plan of seismographs all over the country on September 1988.

At the examination, it is aim to the efficiently collection of data by the correction of unbalanced installation and the consideration of local seismic risk. And also, that works were enforced by the esteem of the way of thinking of former plan and the changing of social and economical states in present. This new plan was made by the visual point to the observation at ground, which has very high capability of utilization of records. The observation at the structures will be made the appropriate plan by the each organization.

The large or huge earthquake will occur at the off-shore of Pacific ocean, the moderate or large earthquake will occur at the Japan sea are and the moderate and shallow earthquake will occur at the inland area. The seismic activities based on the historical data show the local properties, but generally speaking, any place in Japan has the possibility of attacking of the strong-motion earthquakes by the distribution of the active fault all over the country. For the catching of the record during earthquake at the one area at rare intervals, the fundamental distribution is necessary to install the equal density all over the country. And then, it is reasonable to change the distribution density at locally by the consideration of the research result of the seismic risk and the prediction of earthquake occurrence. The aim of distribution was set up to the two kind of areas, the one is the mesh of fundamental observation (fundamental mesh) for the covering to similar density to the whole Japan, the other is the mesh of intensified observation (intensified mesh).

The distribution distances were determined to the following objects by the considering with the target of observation.

1) To be able to record the vibration of Seismic Intensity 5 (JMA scale) at the earthquake of greater than M6.5.

2) To be able to record the vibration of Seismic Intensity 4 at the earthquake of greater than M6.0.

The empirical equations of the relation between the Magnitude and the area (km) of the greater than some intensity were presented by Prof. Muramatsu and Dr. Katsumata. Prof. Muramatsu was presented the equation between M and area (Sv),which is the area of greater then intensity 5, as shown Eq. 1. And, Dr. Katsumata was presented the equation between M and Siv (the area of greater than intensity 4) as shown Eq. 2.

$$\log Sv = M - 3.2 \qquad (1)$$

$$\log Siv = 0.82M - 1.0 \qquad (2)$$

It is assumed that the area of generation of greater than intensity 5 or 4 vibration is the square of L (km), L = 45 km for the M6.5 by the Eq. 1, and L = 91 km for M6.0 by the Eq. 2. By the reference of this value, the size of fundamental mesh is defined to the 50 km X 50 km for the equality distribution of seismographs based upon the above items 1) and 2).

The size of intensified mesh is defined to the condition to be able to record of Seismic Intensity 5 at the earthquake greater than M6.0. From Eq. 1, L = 25 km for M6.0. Then, the size of intensified mesh is determined to 25 km X 25 km. The area of intensified mesh was determined the following three items.

(1) The area of high seismic activity

It is a nearly impossible to predict the future earthquake occurrence except the greater than M8.0, like the Tokai Earthquake, because the prediction for the smaller earthquake is the face of research. And, some researchers were presented the method to estimate of the strength of seismic motion for future earthquake by statistic manner. But, they have the great difference by the difference of analytical method and the using data. Therefore, The relatively high seismic activity area was determined by the proposed method by the "Development of New Anti-seismic Design Method"by the Ministry of Construction (1972 - 1976). Fig. 12 shows the section of seismic activities. In this figure, A is the area of high seismic activity, B is the moderate activity and C is low activity. The intensified area was determined by this high seismic activity zone.

(2) Area of specific and intensified

observation, designated by the Coordinating Committee for Earthquake Prediction.

The designation of area of specific observation is given to areas of historical disastrous earthquakes, active faults, high seismicity, and to areas important from social and economic aspects, like densely populated area. When anomalous phenomena which could be considered precoursors to an earthquake, such as land uplift or subsidence, are observed by either the nationwide network of geodetic surveys, seismic observation and tide-gauge observation or by other observations in the area of specific observation, such an area is designated as an area of intensified observation. Fig. 13 shows the area of specific and intensified observation , which were revised on 1978. These areas are appropriated to designate the intensified mesh of the strong-motion earthquake observation. Some parts of these areas are duplicated to the high activity zone. Then, the areas of western Akita Pref. and northwestern Yamagata Pref., southwestern Niigata Pref. and Northern Nagano Pref.,Iyo-nada and Hyuga-nada region are determined to the intensified observation mesh of this distribution plan.

(3) The densely populated areas

In the densely populated areas, the number and sort of constructed structures are so many, it is predicted to the social effect is very severe at the earthquake hazard. And, the disaster of large and important structures has a possibility of secondary hazard unless the hazard of themselves. By consideration of this point, the densely populated areas and the construction sites of large and important structures are designate to the intensified area of this plan.

Based on the determination of fundamental mesh and intensified mesh, the mesh which should be installed seismographs in the future and that distribution plan were examined. The examined ideas are as follows;

1) Dissolution of vacant mesh of seismograph

All of vacant fundamental and intensified mesh will be set 3 observational sites. At one site, 2 seismographs are installed at the ground and structure. It is necessary that more seismographs on large and long structures will be installed. Table 1 shows the object meshes for examination by considering the duplicated meshes between the fundamental and intensified observation. The map of the object meshes for examination is shown in Fig. 14. This figure explain the final distribution of the fundamental and intensified meshes. There are 92 fundamental meshes, 151 intensified meshes foe the area of high seismic activity zone, 93 intensified meshes foe the area of specific observation and 50 intensified meshes foe the area of intensified observation, designated by the Coordinating Committee for Earthquake Prediction. And, the total number of meshes is 386. The distribution of installed seismographs is shown in Fig. 15, which was drawn on the Fig. 14. By the Table 1, the vacant meshes are 20 (22 %) in the fundamental observation area, and 140 (48 %) in the intensified observation area. To dissolute these vacant meshes according with the above mentioned target of installation, it is necessary to set up 480 (= 160 X 3) observation sites and 960 (= 480 X 2) seismographs.

2) Densely installation for the scarcely number of seismographs mesh

It is no little number of meshes which were set up the plural observation sites already. But, It is important to attempt the densely installations, according with the above target (1) The meshes which has not 3 observation site, are 55 % of total fundamental observation meshes, and 60 % of intensified observation meshes. The installed condition is not same with each mesh, which has one seismograph at least. If it is assumed that 2 observation sites, and 2 seismorraphs for each site will be need to attempt in the future, 80 (72 X 0.55 X 2) sites, 160 (80 X 2) seismographs for the fundamental meshes, and 190 (154 X 0.6 X 2) sites, 380 (190 X2) seismorraphs for the intensified meshes will be need to install in the future.

For the achievement to the targets of (1) and (2) together, it is necessary to install 750 sites, 1500 seismographs in total in the future. They are the minimum numbers for the achievement to the targets. Therefore, it is necessary that more seismographs will be installed on the structure. In addition with the increasing of installation within the fundamental and intensified observation meshes in this distribution plan, the following items are requested.

1) To propel the installation on the ground at the new observation site.
2) To request the plural installation on the ground at the different soil conditions, if possible. And, to strongly request the installation on the rock layer.
3) To attempt the dissolution of the vacant mesh at the densely populated area, with dependent of the above mentioned items.

According with the publication of this report of new distribution plan, the actualization of this plan will be desired by the specific cooperation of every organizations with reflection of this plan's point of view to the enforcement installation planning of the seismographs.

5. ACKNOWLEDGMENT

The Report of Distribution Plan of Strong-Motion Seismographs was completed by the specific efforts of the member of the Strong-Motion Earthquake Observation Council, the Secretary Meeting of this council and the Working Committee for studying the New distribution Plan. This report was arranged in order by the authors, quote from that Report. The authors would like to thank every concerned gentlemen. And, for the history of the strong-motion earthquake observation, we feel uneasy to escape some items our memory. We shall be happy if you will point out.

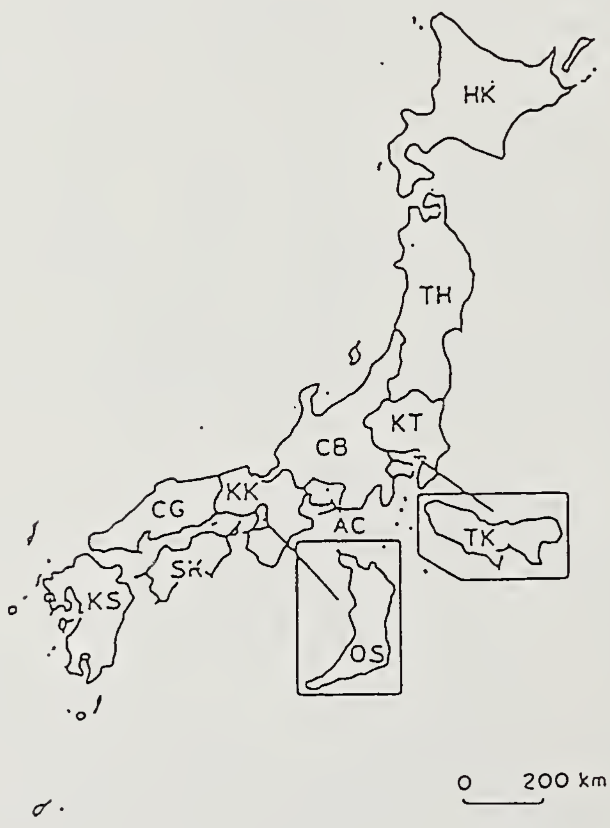
6. REFERENCES

1) "The Report of Distribution Plan of Strong-Motion Seismographs" edited by the Strong-Motion Earthquake Observation Council, published by National Research Center for Disaster Prevention, 1988
2) I. MURAMATSU "Relation between the Distribution of Seismic Intensity and the Magnitude of Earthquake" Education and Research Report of University of Gifu, 4(3), 1969, pp.168 - 176
3) M. KATSUMATA, K. TOKUNAGA "Corresponding with the Area of Seismic Intensity 4 and the Magnitude of Earthquake, and the Seismic Intensity and the Acceleration" Shinken-jiho 36 1971, pp.89 - 96
4) S. HATTORI, Y. KITAGAWA, M. OTUKA "Regional Coefficients of Earthquake for Aseismic Designs" BRI Research Paper, No. 75, 1977

Table 1. The Object Meshes for Examination

<table>
<tr><th colspan="2"></th><th colspan="2">Total No. of Meshes</th><th colspan="2">Mesh No. of Installed Area</th><th colspan="2">Mesh No. of Uninstalled Area</th></tr>
<tr><td colspan="2">Meshes for Fundamental Observation (50kmX50km) *</td><td colspan="2">9 2</td><td colspan="2">7 2</td><td colspan="2">2 0</td></tr>
<tr><td rowspan="3">Meshes for Intensified Observation (25kmX 25km)</td><td>I : High Seismic Zone</td><td>1 5 1</td><td rowspan="3">2 9 4</td><td>6 0</td><td rowspan="3">1 5 4</td><td>9 1</td><td rowspan="3">1 4 0</td></tr>
<tr><td>II : Area of Specific Obs.</td><td>9 3</td><td>5 1</td><td>4 2</td></tr>
<tr><td>III : Area of Intencified Obs.</td><td>5 0</td><td>4 3</td><td>7</td></tr>
<tr><td colspan="2">Total</td><td colspan="2">3 8 6</td><td colspan="2">2 2 6</td><td colspan="2">1 6 0</td></tr>
</table>

* Excepted the Duplicated Meshes for Intencified Observation



Districts		Number of Places	Number of SMACs*
Hokkaido	(HK)	48	82
Tohoku	(TH)	62	115
Tokyo	(TK)	182	488
Kanto	(KT)	94	163
Aichi	(AC)	29	52
Chubu	(CB)	105	183
Osaka	(OS)	38	93
Kinki	(KK)	47	72
Chugoku	(CG)	43	65
Shikoku	(SK)	30	53
Kyushu	(KS)	35	59
Total		712	1425

* SMAC type + DC type

Fig. 1. Distribution of Strong-Motion Accelerographs (as of March 1989)

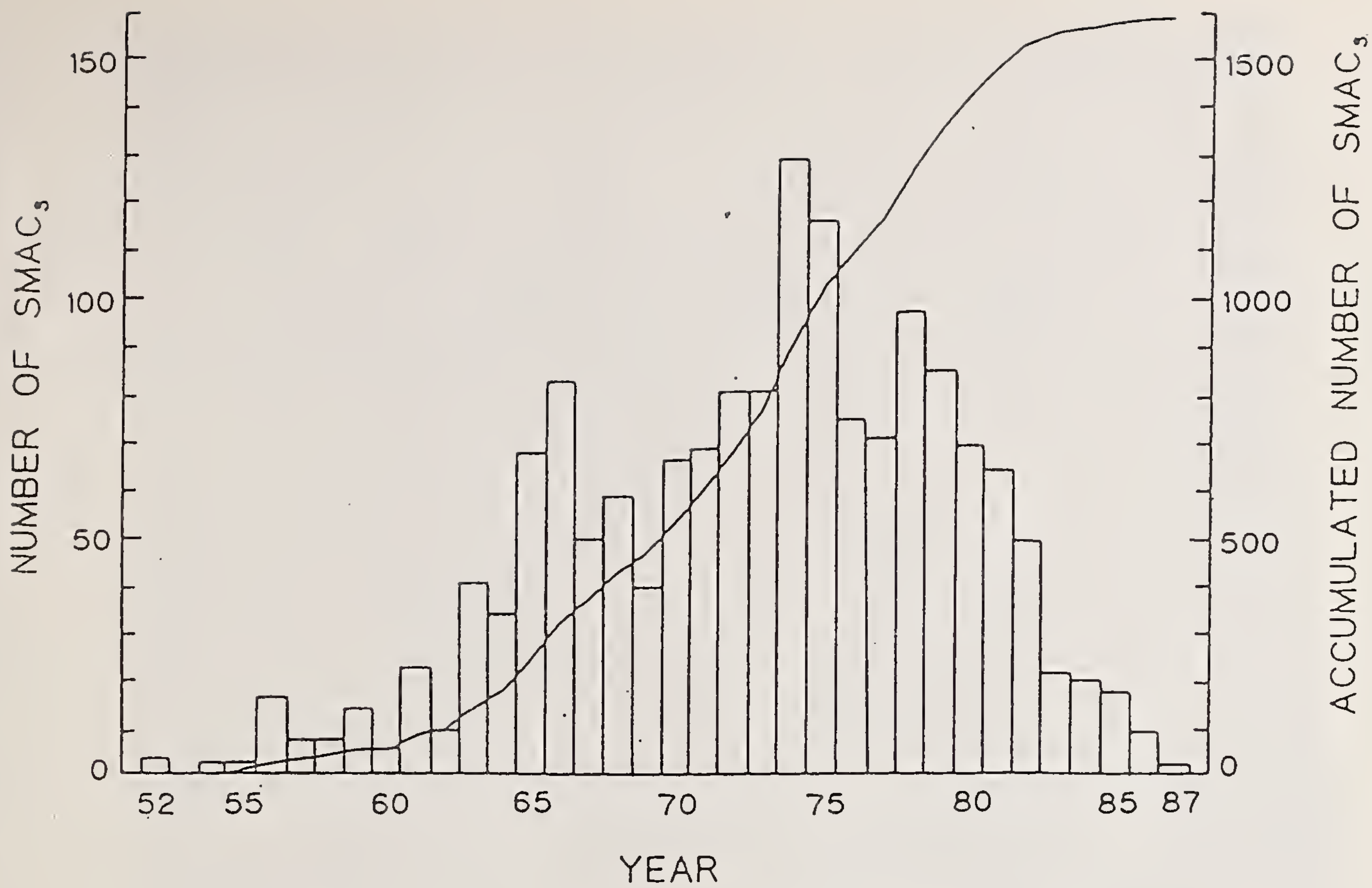


Fig. 2. The Increasing Trend of Installed Accelerographs

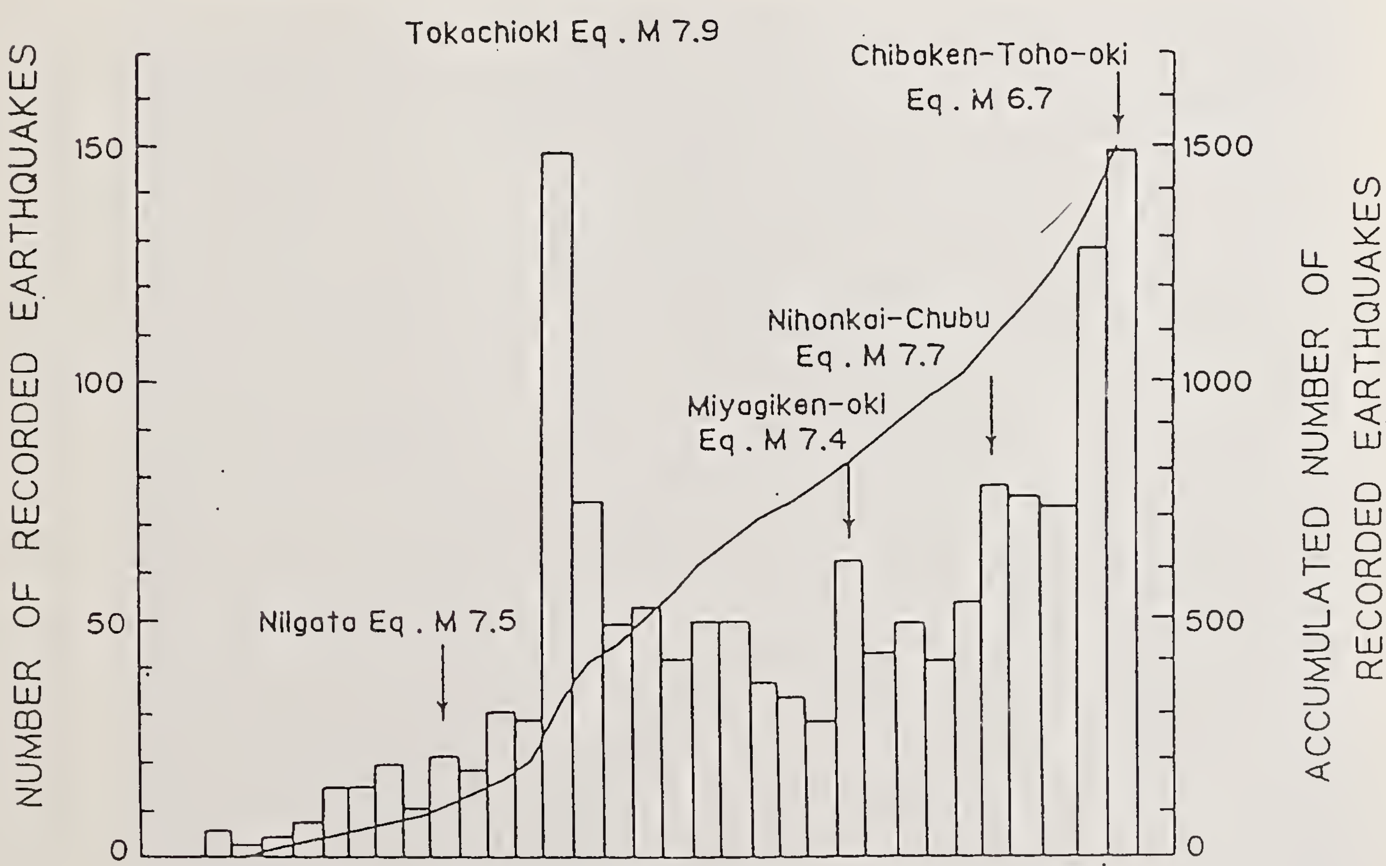


Fig. 3. The Number of Recorded Earthquakes

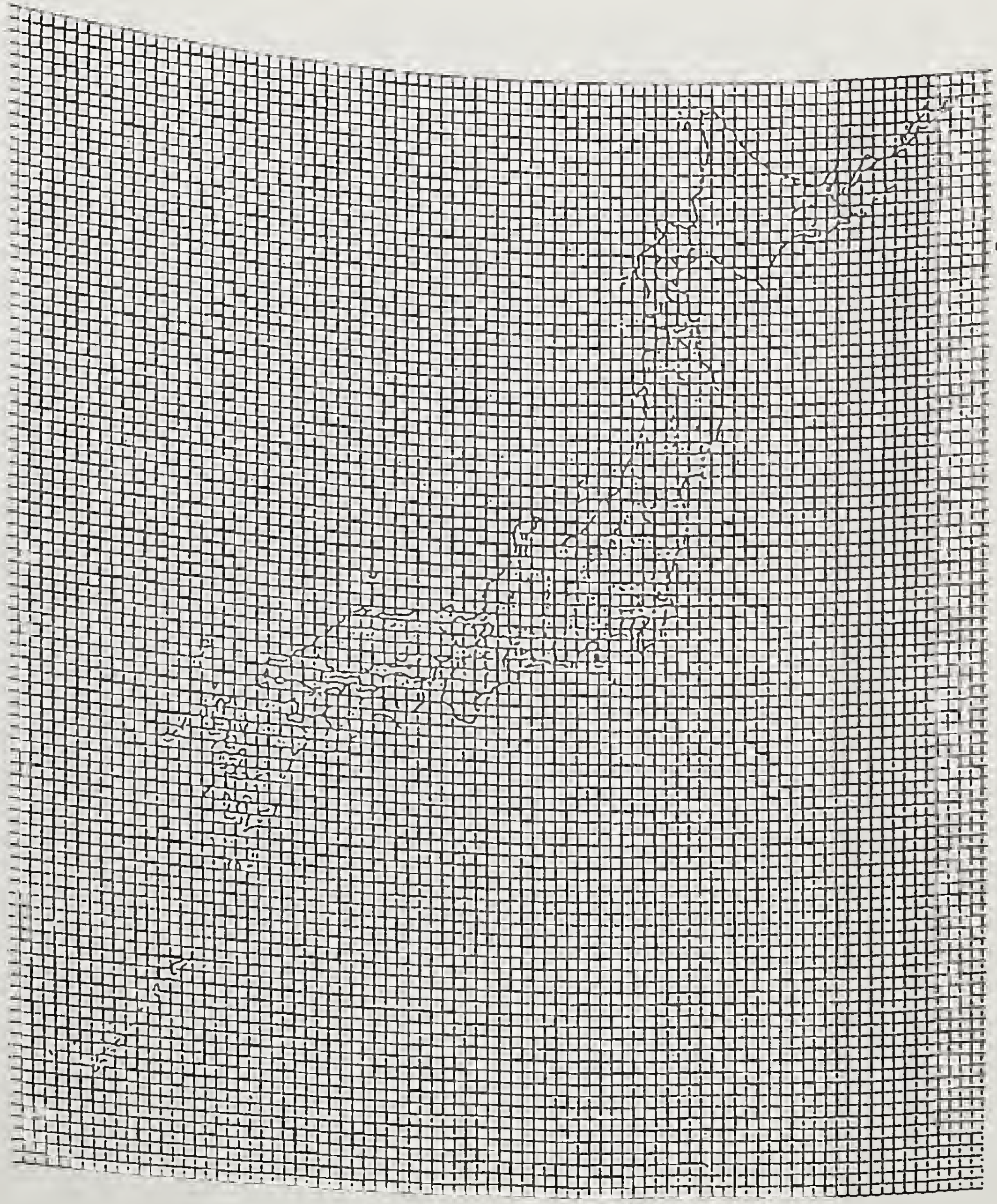


Fig. 4. Map of Japan (Duplicated with the 25 km Mesh Lines)

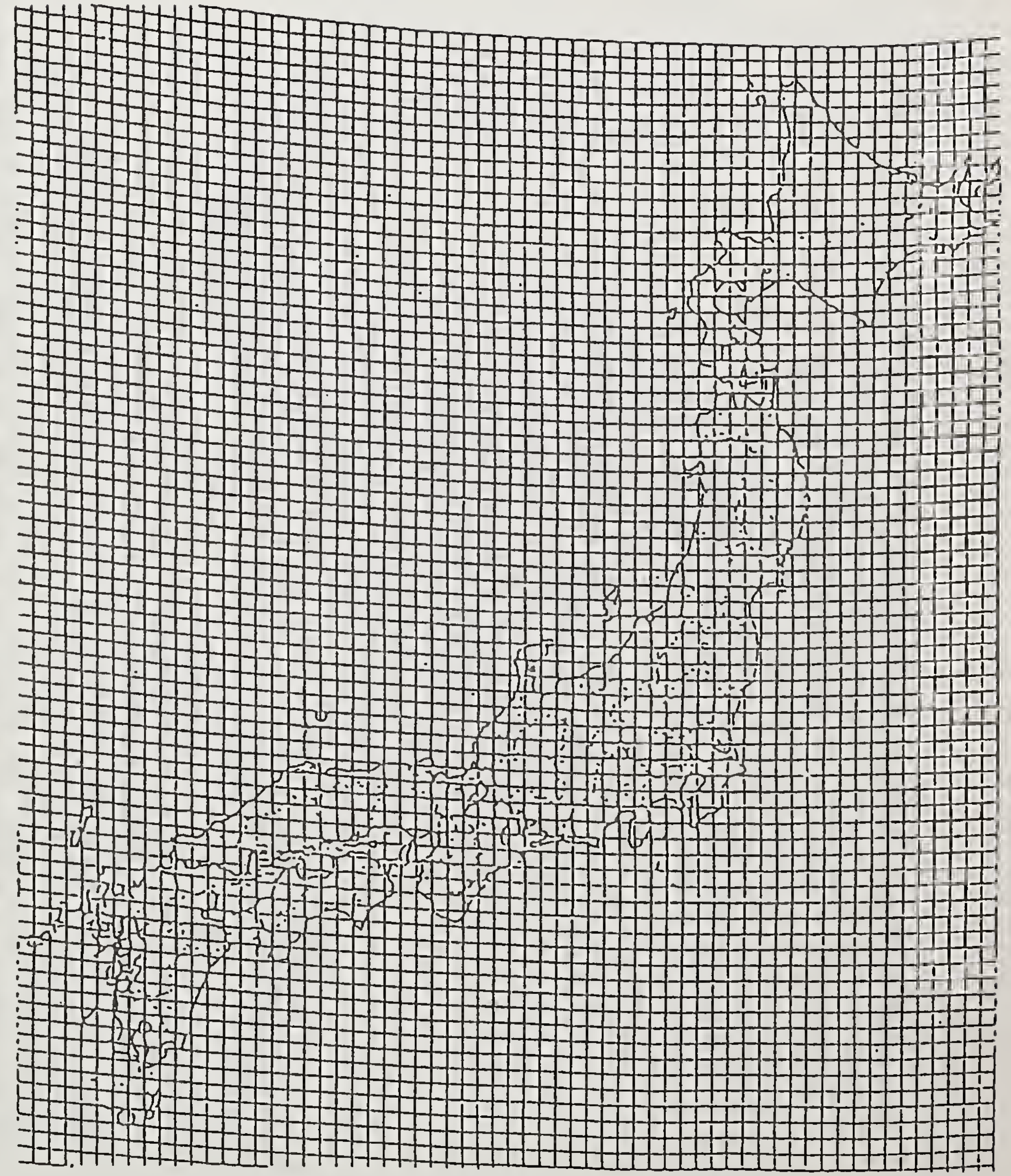


Fig. 5. Map of Japan (Duplicated with the 50 km Mesh Lines)

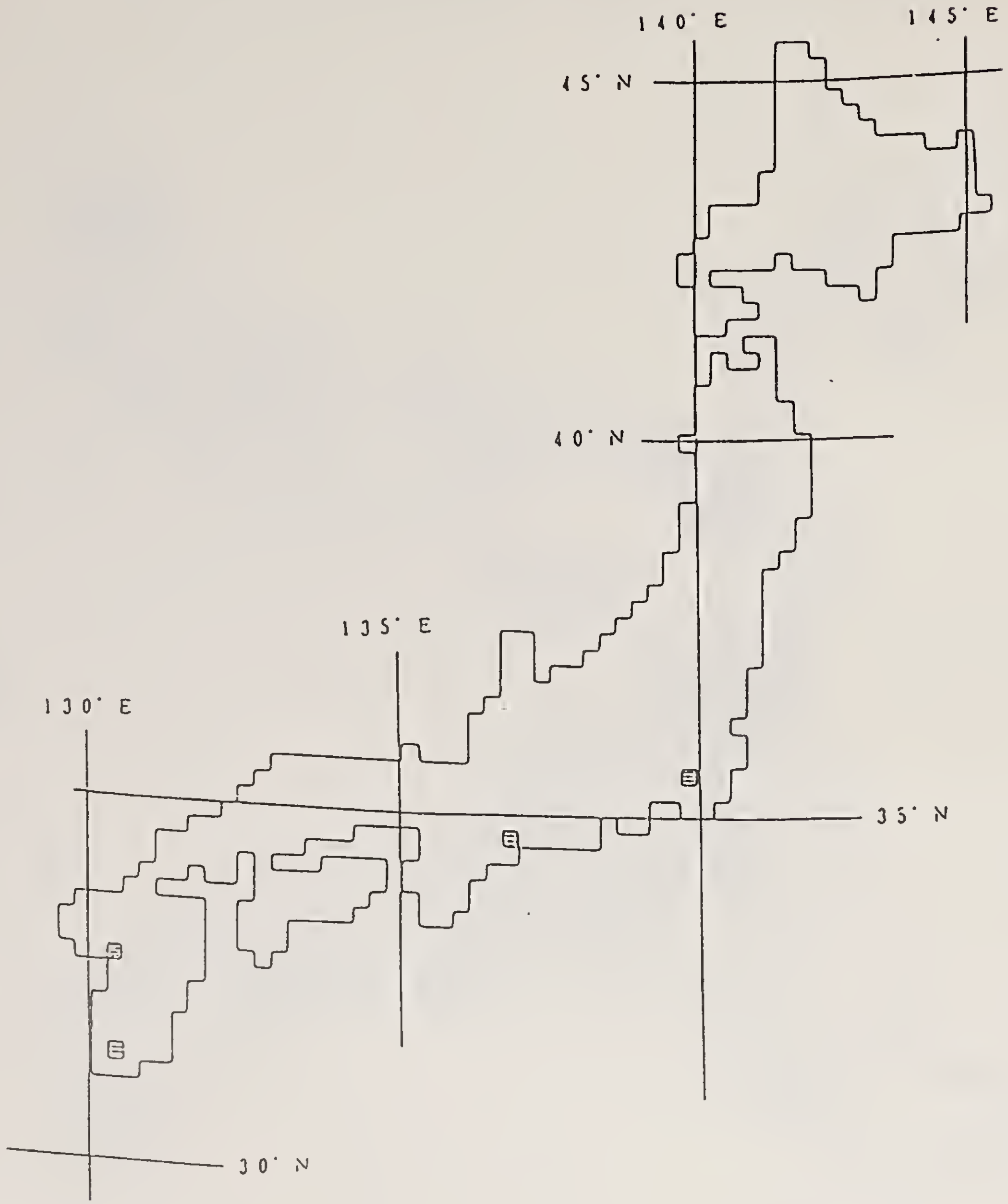


Fig. 6. Outline of Japan by the 25 km Mesh Lines

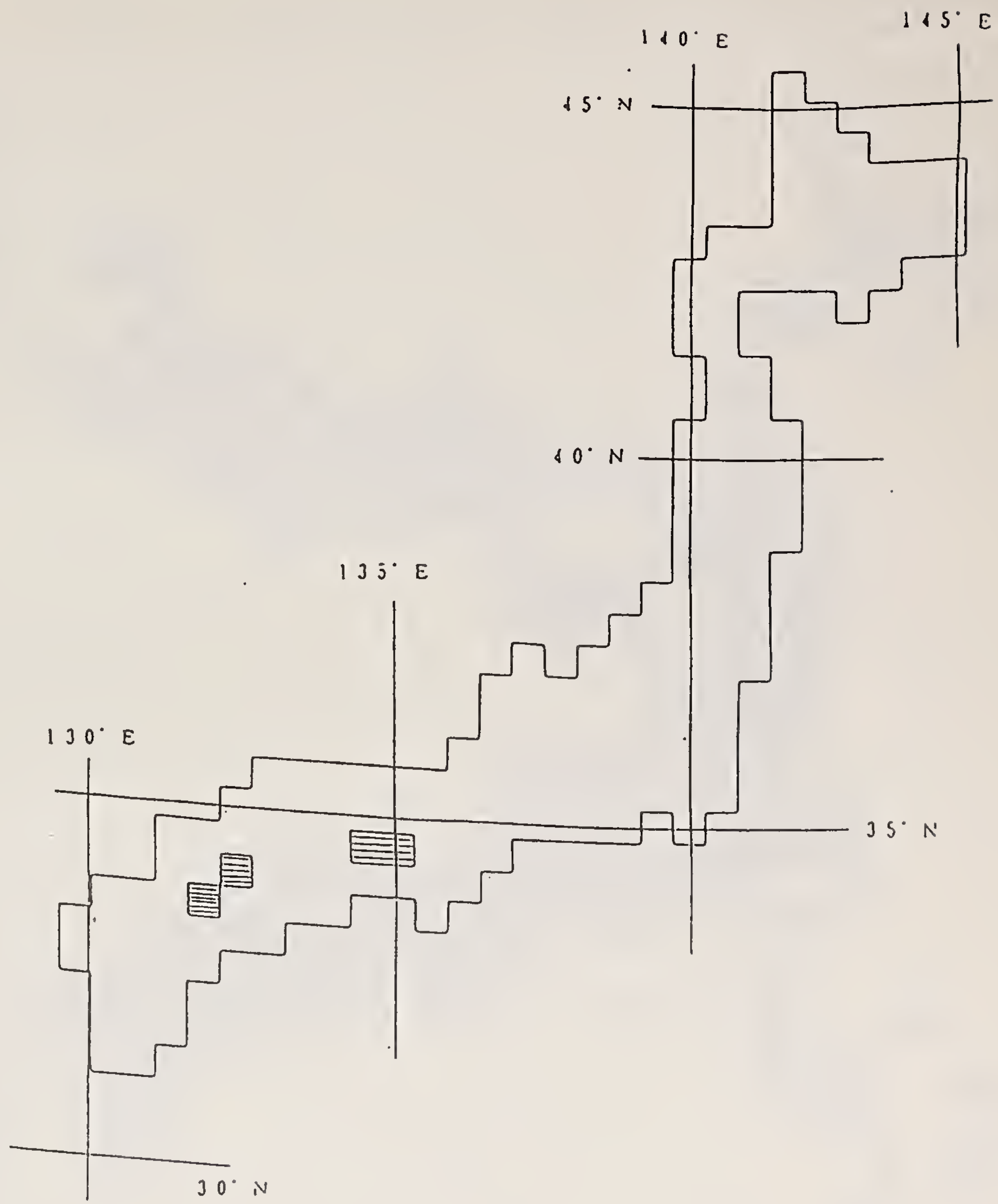


Fig. 7. Outline of Japan by the 50 km Mesh Lines

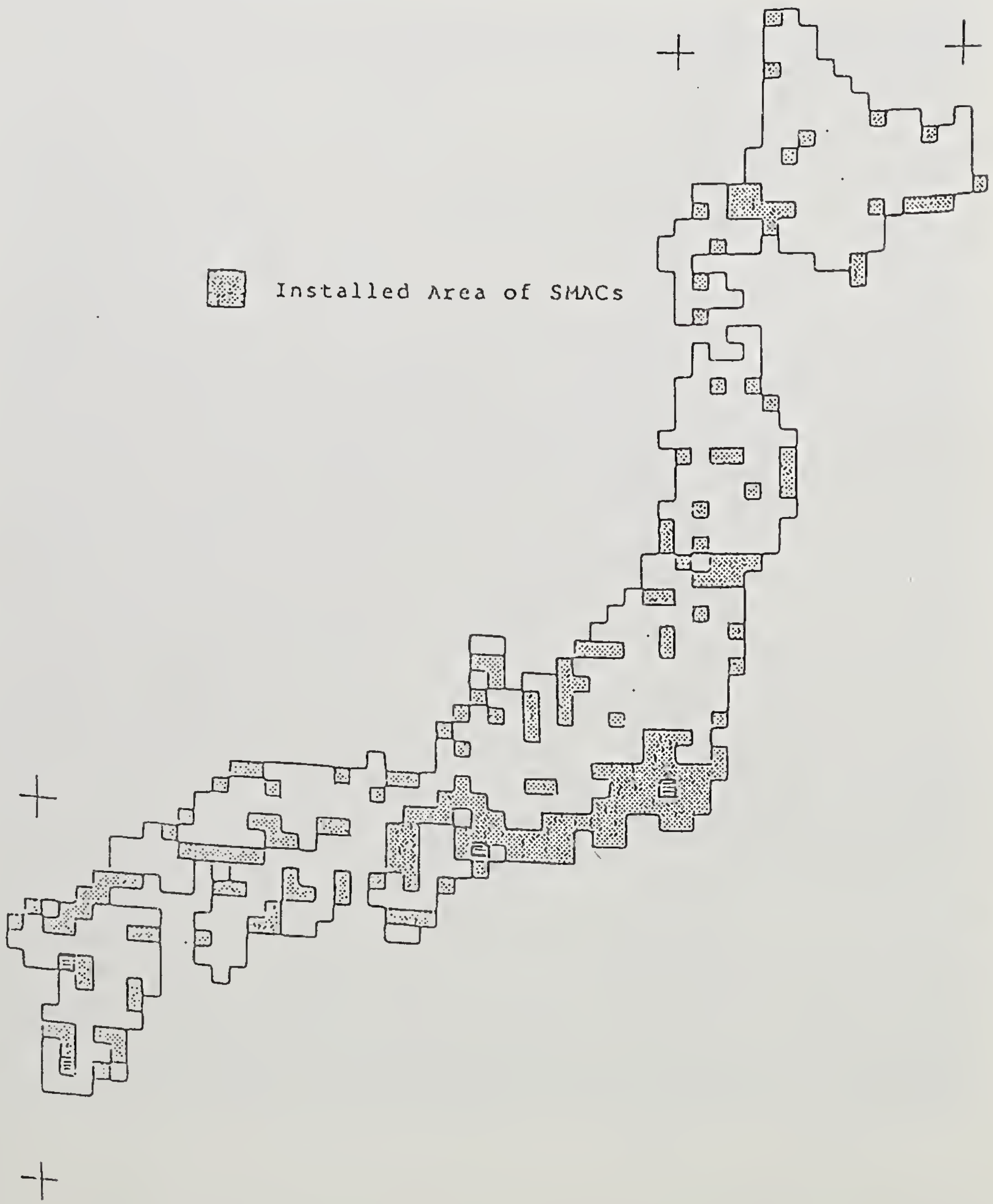


Fig. 8. Installed Area of SMACs (25 km Mesh Map)

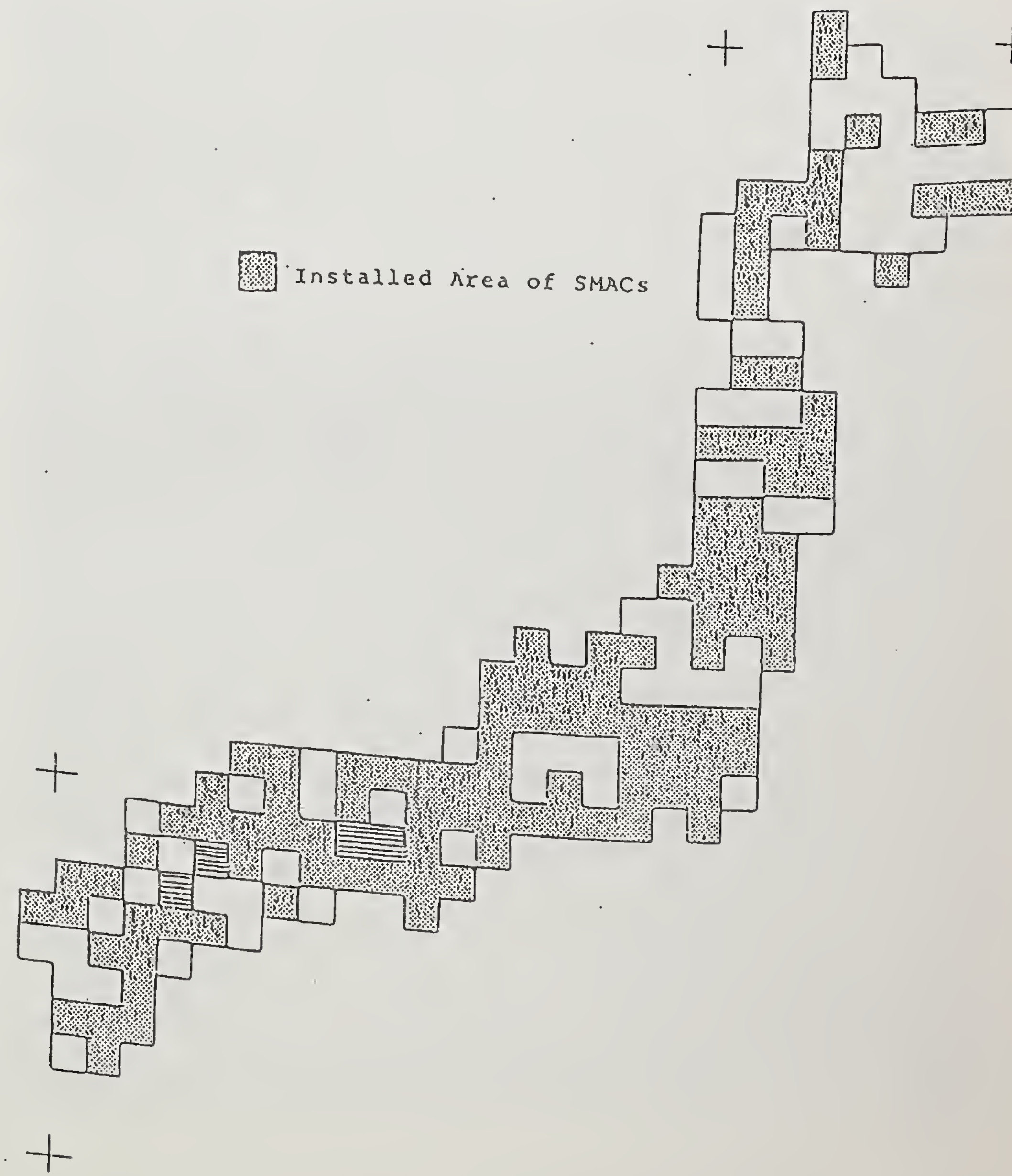


Fig. 9. Installed Area of SMACs (50 km Mesh Map)

Fig. 10. Installed Area of Seismographs (25 km Mesh Map)

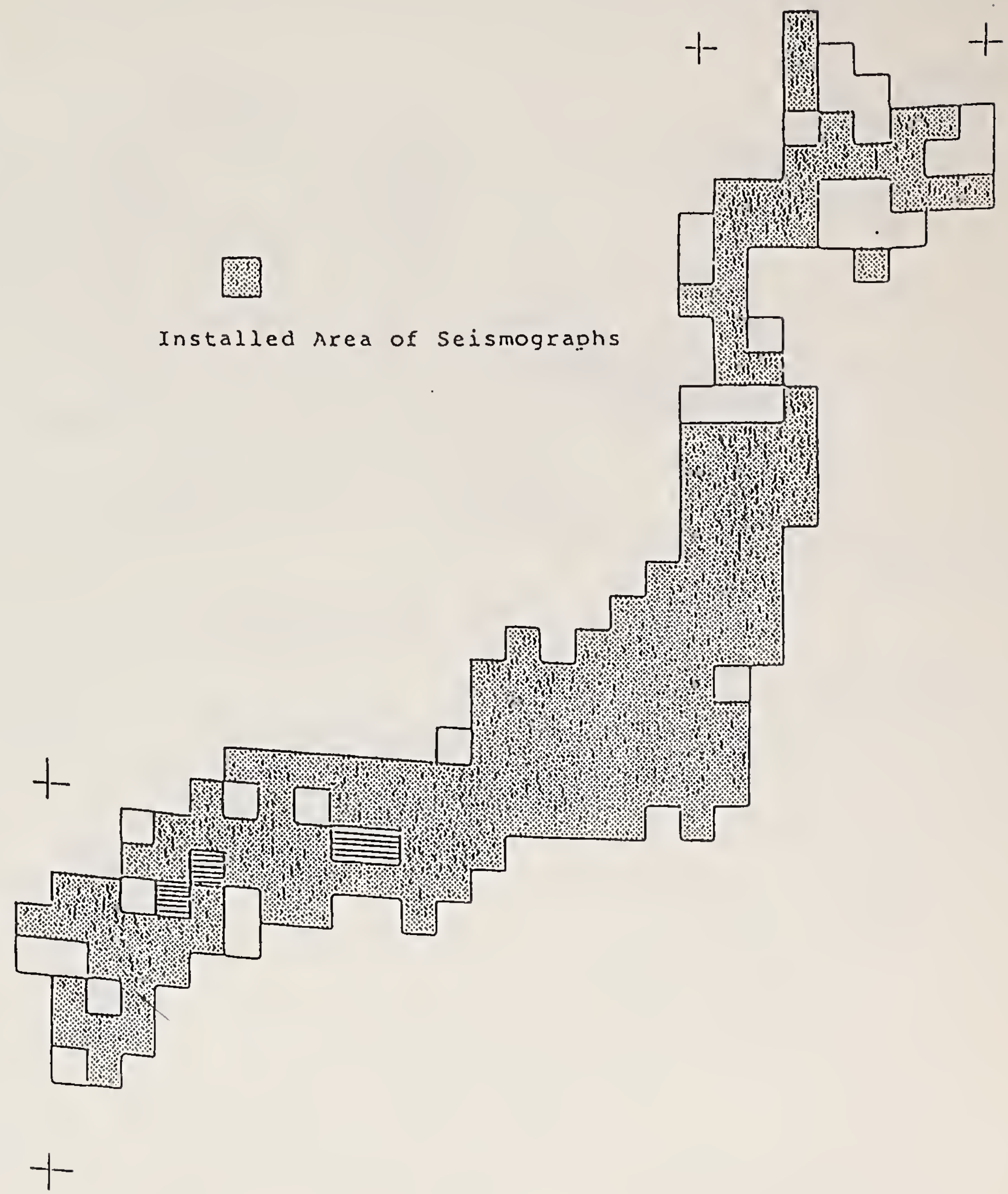


Fig. 11. Installed Area of Seismographs (50 km Mesh Map)

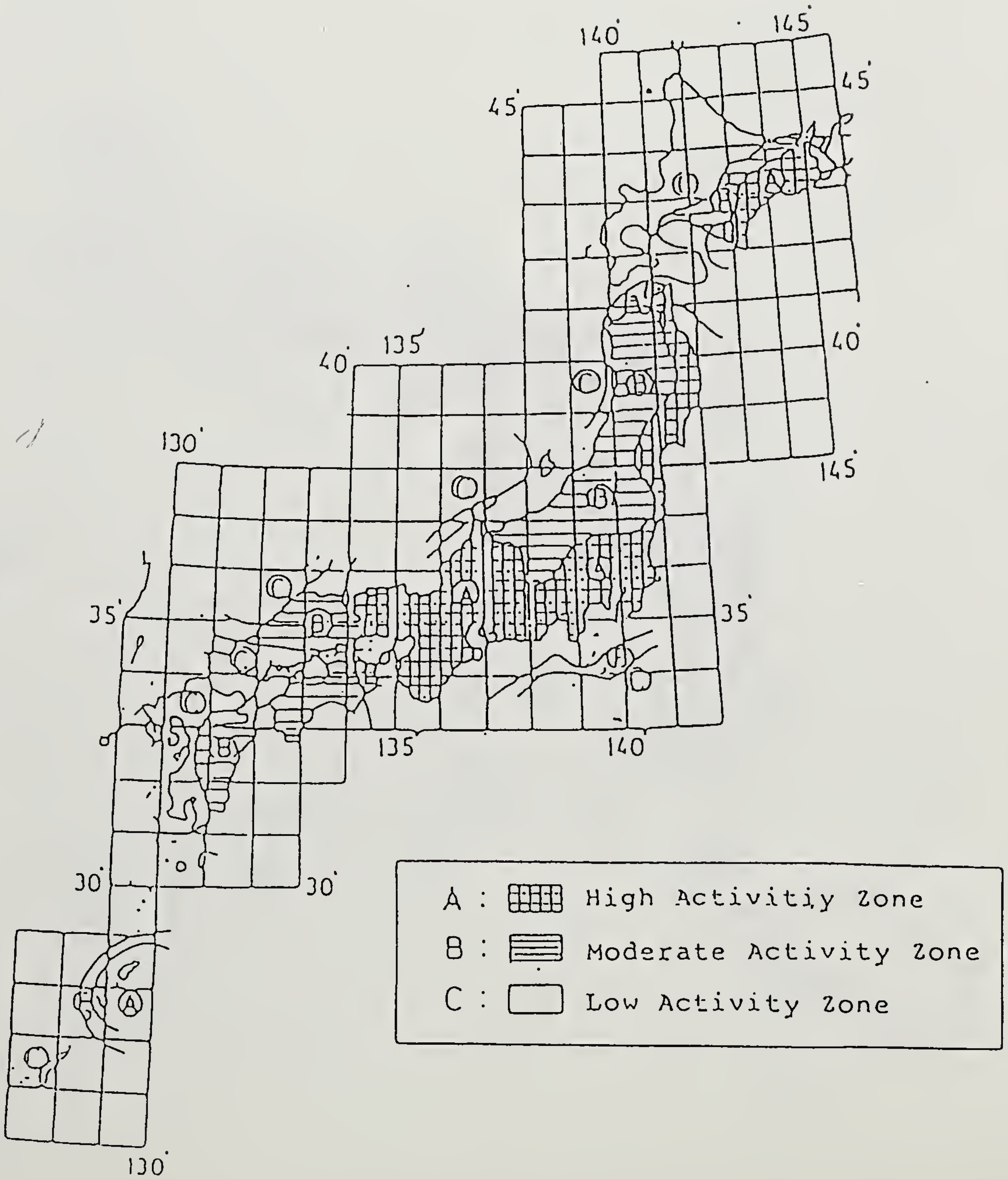


Fig. 12. The Section of Seismic Activities

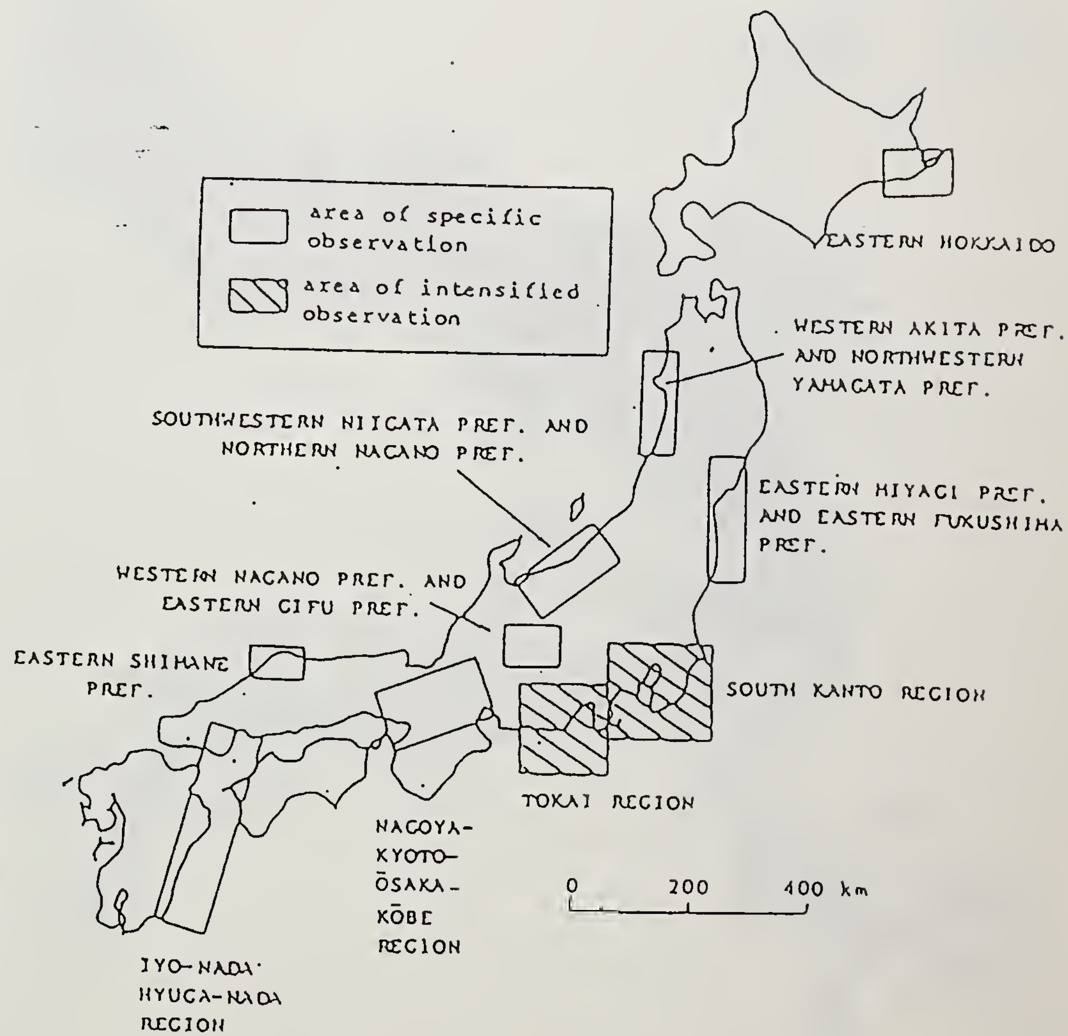


Fig. 13. Area of Specific and Intencified Observation (designated by The Coodinating Commiittee for Earthquake Prediction)

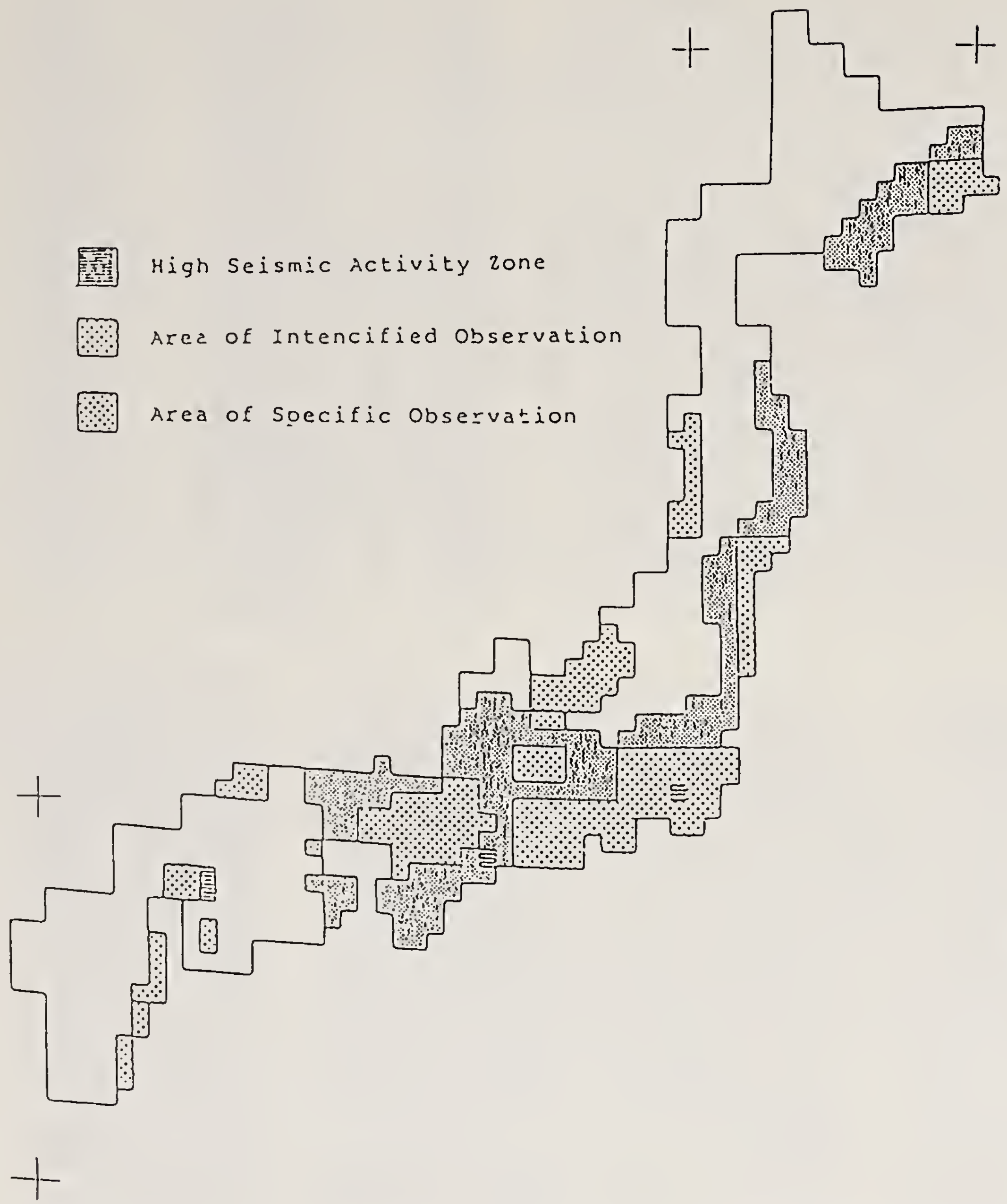


Fig. 14. Arcas for Fundamental and Intencified Observation
(Map of the Object Meshes for Examination)

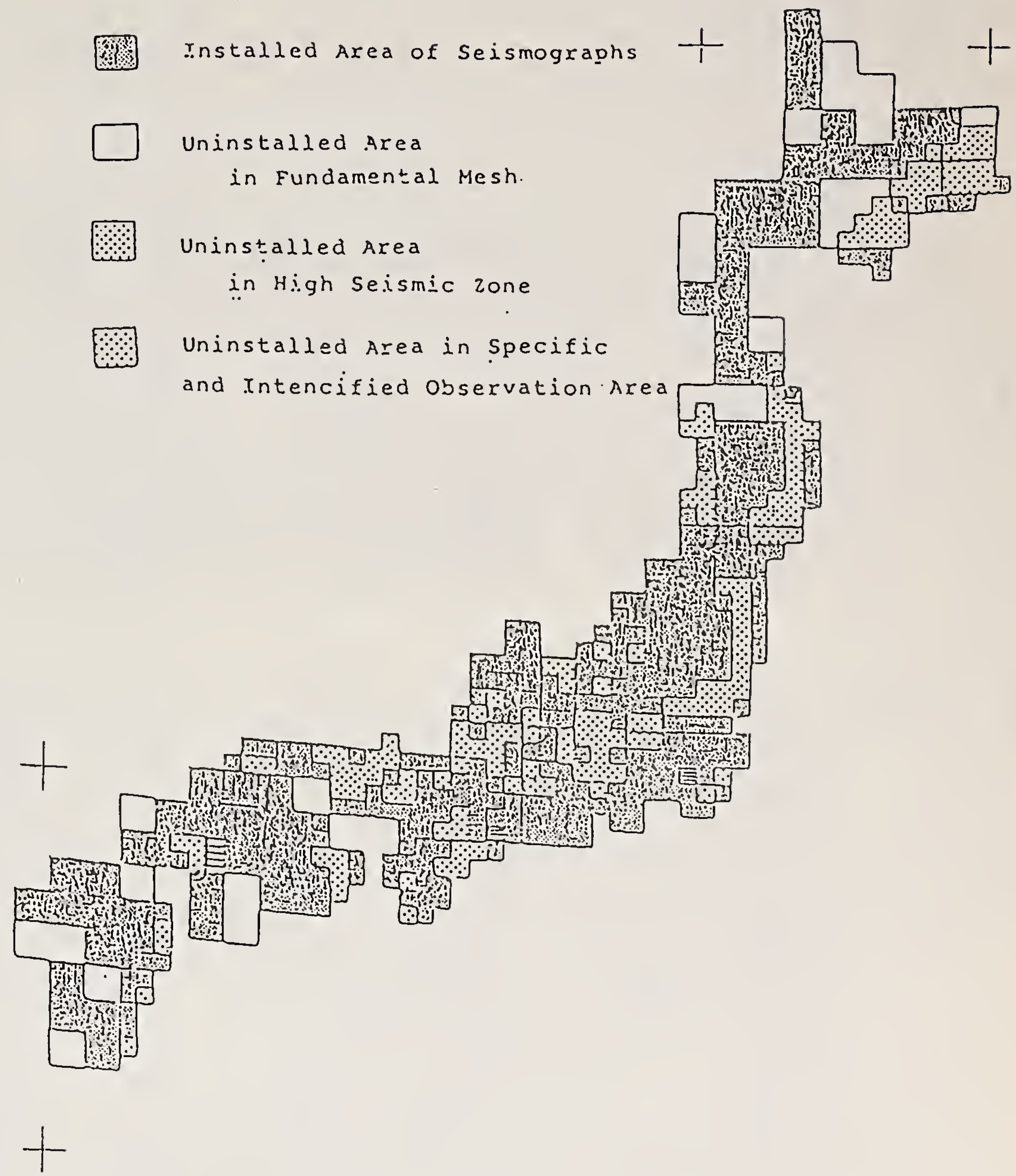


Fig. 15. Installed Area of Seismographs

Summary of Lifeline Earthquake Engineering

by

James D. Cooper[1]

ABSTRACT

This paper reviews the current status of lifeline earthquake engineering design practice and major research efforts underway which ultimately will lead to the establishment of more formal seismic design criteria. Earthquake engineering studies have been conducted for many years. However, it was not until the 1971 San Fernando earthquake that major focus was placed on the damage to structures and facilities other than buildings. At that time, C. Martin Duke defined a new subarea of earthquake engineering he termed "lifeline earthquake engineering." His general definition of lifelines encompassed those facilities which are required to support and maintain emergency operations and to facilitate longer term reconstruction following an earthquake. He noted that little attention had been given to the seismic design of transportation, gas and liquid fuel, water and sewer and electric power and communications facilities.

Since 1971, major research has been conducted which has lead to the identification of lifeline design guidelines and establishment of a major lifeline earthquake program in the United States, which is summarized herein.

KEYWORDS; Earthquake; lifelines; research; design.

1. INTRODUCTION

Lifelines are defined as those utilities, facilities, and structures that are required to function following an earthquake to facilitate search and rescue, provide emergency services, allow for the movement of goods and materials and form a network which is required for post event reconstruction. Electric power and communications, gas and liquid fuel, transportation and water sewer systems form todays modern lifeline network. Each lifeline is comprised of numerous components, some of which are critically vulnerable to earthquake induced damage and whose loss of function renders the lifeline useless. For example, a break in a major gas, oil or water transmission pipeline can cause system failure. However, a break in a gas or water service line may eliminate service to a very local area only. In either case, failure may induce secondary problems such as fire caused by escaping gas from a broken pipe or the inability to control fires becasue of a broken water pipe.

Attention was focused on the lifeline problem following the 1971 San Fernando earthquake. Since then, numerous post earthquake investigations have been made and damage assessment reports prepared documenting the performance of lifelines. Basic research is providing engineers an understanding of how and why components of lifelines perform the way they do. Yet little has been published in the open literature to provide detailed guidance for the design and construction of lifelines to resist strong ground shaking.

The design and construction of integrated lifelines involves the application of multidisciplinary topics and experience gained from previous earthquakes where weaknesses in design, construction, system architecture, and management have been highlighted. The performance and reliability of lifelines can affect a broad geopolitical area requiring the involvement of community leaders, public officials and the private sector to mitigate damaging effects. Pre-earthquake considerations include the identification of expected variations in earthquake intensity, engineering factors, and policies influencing risk and reliability. The planning process for each lifeline system will significantly influence the expected outcome of the effects of a seismic event. The process includes an understanding of the geologic factors that produce the seismic intensity levels that cause structural damage and ground failure. This knowledge supports systematic evaluation of hazards and risks required for planning reliability needed to mitigate earthquake damage.

The easiest, most cost effective way in which to mitigate seismically induced damage is to upgrade seismic design and construction procedures for new construction. This approach will take decades to enhance the seismic resistance of lifelines, but will ultimately reduce the potential for catastrophic impact in the event of a great earthquake. Limited seismic design procedures and details for lifelines have been developed over the past 20 years. However, some newer structures incorporating these procedures have been exposed to relatively strong earthquakes and have performed quite satisfactorily. Design and construction costs associated with enhanced seismic resistance is still being evaluated. A range of from one to 20 percent increased project costs for upgraded seismic resistance has been suggested and is highly dependent on the level of seismic detailing incorporated into the design. Nonetheless, these costs distributed over time become an inexpensive investment to significantly reduce the sudden loss potential from a great earthquake.

A second method to mitigate seismically induced damage is to retrofit existing lifelines. Retrofit is typically <u>much more costly</u> than planning in advance for the seismic design of new construction. However, retrofit can be cost effective even in regions outside of the traditionally thought of seismically active zones. For example, if potentially vulnerable transportation routes are identified, a large number of bridges along these critical routes could be "restrained" at low cost. Since this retrofit technique typically costs between 10 and 20 thousand dollars per bridge, 150 bridges could be retrofit for the approximate cost of one new structure! For other types of lifelines, retrofit may not be cost effective except in unusual circumstances. Retrofit details and designs are becoming more common in traditional seismically active areas, although retrofit is not yet a generally accepted policy in these areas.

Following is a summary of recent and ongoing design and research activity that will have important impact on the seismic design and construction of lifelines.

2. LIFELINE DESIGN METHODS

The American Society of Civil Engineers established the Technical Council on Lifeline Earthquake Engineering (TCLEE) in 1974 to establish the means by which the civil engineering profession could undertake a comprehensive role in elevating the state of the art of lifeline earthquake engineering. In 1974, no major

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organization or agency was committed to the problem area. The Society's goals and structure provided a unique opportunity to focus attention on this area. Technical committees corresponding to each of the lifelines were established to study the problems of planning, design, construction and operationof each lifeline to mitigate the effects of earthquakes and develop procedures with which sound design could be achieved. Additionally, a committee on seismic risk was established to develop procedures for seismic risk analysis, including risk from geological hazards.

One of the major goals of TCLEE was to develop and assemble advisory notes for each respective lifeline. the ASCE publication, "Advisory Notes on Lifeline Earthquake Engineering" (1984) presents the most comprehensive overview of the current (1989) state of the art of seismic design practice for each topical area and provides some insight into how lifeline facilities have performed in past earthquakes. The publication provides information and approaches in developing improved designs for new lifeline facilities. Significant research has been conducted or funded by the National Science Foundation (NSF), the Federal Emergency Management Agency (FEMA) and the National Institute of Standards and Technology (NIST) which warrants an update of this publication.

Since 1984 some utilities, privately owned gas and oil companies and individual agencies have developed their own criteria for seismic design. A general approach for the design of lifelines has evolved and is outlined herein. Perform or conduct site specific studies which reflect the hazard exposure to the lifeline. Typically, geological and seismological investigations are conducted which identify local and regional geology and historical earthquake activity. In some cases, field exploration studies are conducted. Criteria are developed which are based on risk considerations and potential ground motions including accelerations, velocities and displacements at the site, distance from causative fault and duration of shaking. Structural design parameters are established which include the development of design spectra. Special structural considerations including estimates of damping and ductility are factored into the criteria. Allowable deformation and force criteria are then developed. Special detailing particularly for foundations, anchorages and connections are then identified. These general steps have been applied to the design of electric power and communications facilities, gas and oil transmission systems and transportation structures including bridges.

3. FUTURE FEDERAL LIFELINE DESIGN GUIDES

In 1977, the Congress of the United States enacted the Earthquake Hazard Reduction Act of 1977 to reduce risks of life and property from future earthquakes. This act forms the basis for the formation of the National Earthquake Hazards Reduction Program (NEHRP). Several specific objectives cited in the NEHRP include the charge to develop seismic design and construction standards for Federal use; develop guides or facilities that are Federally owned, constructed or financed to ensure serviceability following an earthquake and coordinate the development of guides for the consideration of seismic risk in the development of Federal lands.

In 1978, the Interagency Committee on Seismic Safety in Construction (ICSSC) was established as part of the NEHRP to assist Federal departments and agencies involved in construction to develop earthquake hazard reduction measures for incorporation in their ongoing programs. The measures will be based on existing standards when feasible.

In meeting its responsibilities, the ICSSC cooperates with State and local governments and private organizations in developing nationally applicable earthquake hazard reduction measures. Five subcommittees of the ICSSC are responsible for responding to the charge contained in the NEHRP:

1. Standards for New and Existing Buildings
2. Lifelines
3. Evaluation of Site Hazards
4. Federal Domestic Assistance, Leasing and Regulatory Programs
5. Post Earthquake Response Investigations

The mission of the Lifelines Subcommittee is to identify existing guidelines or standards for seismic design, construction and retrofit of energy, transportation, water, and telecommunication systems; to recommend Federal adoption of such standards when found adequate; and to encourage development of new standards where there are significant omissions. The Subcommittee will also study techniques for evaluating the seismic vulnerability of existing lifelines and for improving their resistance to seismic effects and ease of repair.

The Lifelines Subcommittee is also considering strategies which will permit identification of those lifeline facilities important in the emergency, immediate recovery, and long-term economic recovery periods, and provide guidance for appropriate levels of seismic protection for each type.

The subcommittee is currently in the process of reviewing technical literature pertaining to seismic resistant design methodology of lifelines prior to developing a formal recommendation to the ICSSC or specific guidelines for Federal agencies use.

4. LIFELINE RESEARCH

Several Federal agencies and organizations are actively pursuing lifeline earthquake engineering research. They include the National Science Foundation (NSF), The National Center for Earthquake Engineering Research (NCEER) at the State University of New York in Buffalo, and the Federal Emergency Management Agency (FEMA).

The NSF is funding several research studies relating to siting and geotechnical systems research which focuses on the fundamental engineering issues related to ground shaking and the effects it has on geological-structure interactions. Specific engineering investigations are being conducted to better understand the process of subsurface ground damage to lifeline systems including utility lines and energy transmission systems. Studies include a University of Colorado investigation on the effect of geological layered strata in modifying the strong ground motion experienced by tunnels and buried pipelines, including amplification of ground motion at the surface.

The 1987 Whittier Narrows, California earthquake caused extensive damage to water distribution pipelines. While theoretical modeling of earthquake induced damage has been actively pursued, theoretical models are based upon damage mechanisms that are not well understood, and there is very little data related to the direct observation of earthquake induced damage. Such data is needed to validate these models so that they can be used with confidence in deriving seismic design criteria. At present, there is no code for buried pipelines.

An Old Dominion University study involves a thorough field investigation and analysis of the damage to water lifeline systems around Whittier, California. In addition, the predictions of the existing models are being compared with the observed behavior, and based on this comparison the models are being modified to correlate with the observations.

A field experiment is being performed by Weidlinger Associates in which underground pipelines are placed within the Parkfield segment of the San Andreas Fault Zone. The experiment will help examine the validity of current assumptions regarding the behavior of underground continuously welded steel pipe and jointed ductile iron pipe subjected to near surface fault offset. Both active and passive instrumentation is included to permit evaluation of important assumptions made in current prediction and design methods.

The Whittier Narrows earthquake also offers an opportunity to assess the impact that research conducted since the 1971 San Fernando earthquake has had on the earthquake design and performance of lifelines. Dames and Moore, Inc., is conducting research including the collection, review, and documentation of lifeline system performance data from the October 1, 1987 Whittier Narrows earthquake; the development of preliminary response and recovery models for different lifeline systems using data from the Whittier Narrows earthquake and the 1971 San Fernando earthquake; and assessment of the impact that research in the lifelines area conducted after the 1971 San Fernando earthquake has had on the improvement of seismic design and emergency response and preparedness procedures for lifelines.

The NSF also sponsors the NCEER at Buffalo, New York in cooperation with the State of New York, and public and private corporations. NCEER is conducting research to improve basic knowledge about earthquakes, engineering practice and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. A major focused effort is underway to conduct research in the area of lifeline system failure. Here, effort is directed at two lifeline systems; namely crude oil transmission systems and water delivery systems.

Research on water delivery systems addresses issues relating to ground motion studies; system performance, vulnerability and serviceability; and risk assessment and societal impact. Research on crude oil transmission systems involves the development of seismic risk assessment for a crude oil pipeline system which traverses large spatial areas. Factors will include studies of peak ground accelerations and permanent ground displacements. In addition to ground motion studies which include space-time correlation and geological, topographical and ground motion data analyses, numerous detailed studies are being conducted to better understand the hazards associated with liquefaction and large ground deformation, soil-structure interaction, wave propagation and fault crossings. Detailed studies on system vulnerability, response and serviceability include evaluating damage and repair of piping systems, establishing system reliability and enhancing analysis capability.

Completion of these studies will result in a greater fundamental understanding of how these two types of systems respond and interact when subjected to earthquake activity. These results will provide important information needed for the development of detailed seismic design guides for piping systems.

In recent years, FEMA has become a major sponsor of lifeline earthquake engineering studies. Recently, the Building Seismic Safety Council completed the development of a seismic hazard abatement action plan for lifelines for FEMA to provide a basis for the planning of a long-range lifelines research program. The plan was developed through a workshop consensus process utilizing the expertise of numerous professionals and organizations that deal with the diverse areas of design, construction, operation, and maintenance of lifeline systems. The Action Plan recommended 67 priority projects related to lifeline systems. Details are contained in the FEMA publication entitled "Abatement of Seismic Hazards to Lifelines: An Action Plan" and the six volume FEMA report entitled "Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of an Action Plan."

FEMA sponsored a follow-up study to develop expert recommendations and consensus for prioritorizing and implementing the action. An ad hoc committee on lifelines was established for performing this task. The committee observed that earthquakes pose a profound threat to the reliability and continued survivability of all lifeline systems. The committee made recommendations to implement a nationally coordinated and comprehensive program for mitigating the risk to lifelines from seismic and other natural hazards and focus project activities in the following four high-priority project areas: Improve the awareness and education on lifeline hazards mitigation to lifeline service providers, users, and regulators; Develop information on vulnerability of lifeline systems to natural hazards and identify procedures for minimizing lifelines vulnerability; and Develop and recommend regulatory actions for adopting hazard mitigation standards and criteria for lifeline systems.

Finally, FEMA initiated a study with EQE, Inc., to assess the potential impact of damaging earthquakes on lifeline systems in general and determine the vulnerability of a yet to be identified single lifeline in one region as a case model. This initial effort is expected to contribute a considerable amount of needed information to promote awareness on the importance of mitigating hazards to lifeline systems.

This summarizes the major research efforts being conducted which will add to the basic understanding of the performance of lifeline systems. Additional research is being conducted by private organizations and institutes but is generally not available to the public.

5. CONCLUSION

Although significant attention has been focused on the effects earthquakes have on lifelines, relatively little guidance is available for the seismic resistant design of these facilities. The most definitive seismic design criteria available in the United States for a specific set of structures is the American Association of State Highway and Transportation Officials Bridge Specifications. More general seismic design guidance for lifelines is documented in the ASCE publication "Advisory Notes on Lifeline Earthquake Engineering."

Important research has and is being conducted that will provide information on which to update seismic design guidance for lifelines. Minimal fiscal resources could be invested now to update design guidance for selected lifeline structures. For example, much has been learned about the dynamic effects of fluids in tanks, piping behavior, welding and detailing and anchorage systems to update design guidance for new tanks and piping systems. Of course, many unanswered questions remain which are identified in detail in the FEMA publication "Abatement of Seismic Hazards to Lifelines: An Action Plan." However, many new lifeline structures will be constructed before research studies which are identified in the Action Plan can be completed. As engineers, we must apply today's available knowledge from lessons learned in past earthquakes to the design of tomorrows lifeline structures.

6. REFERENCES

1. Abatement of Seismic Hazards to Lifelines: An Action Plan, Federal Emergency Management Agency Earthquake Hazards Reduction Series 32, FEMA, Washington, DC, August 1987.
2. Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of an Action Plan, Volumes 1-6, Federal Emergency Management Agency Earthquake Hazards Reduction Series 26-31, FEMA, Washington, DC, July 1987.
3. Advisory Notes on Lifeline Earthquake Engineering, American Society of Civil Engineers, New York, New York, 1983.
4. Evaluation of Site Hazards, U.S. Geological Survey Open File Report 88-362 prepared for the Interagency Committee on Seismic Safety in Construction (ICSSC TR-10), Reston, Virginia, 1987.
5. National Earthquake Hazards Reduction Program: Fiscal Year 1988 Activities, Federal Emergency Management Agency, Washington, DC, 1988.
6. Reducing Earthquake Hazards: Lessons Learned from Earthquakes, Publication 86-02, Earthquake Engineering Research Institute, Berkeley, California, 1986.
7. Second-Year Program in Research, Education and Technology Transfer, Technical Memorndum NCEER-87-0028, National Center for Earthquake Engineering Research, Suny, Buffalo, New York, March 1988.

A Strong Motion Data Catalog for Personal Computers

by

Lee Wesley Row, III[1] and Herbert Meyers[2]

ABSTRACT

Technical advancements in the field of earthquake engineering can be impeded by lack of essential strong motion data. Catalogs which document these records are valuable tools for engineers and researchers needing to locate data applicable to specific studies. The personal computer-based strong motion catalog, called SMCAT, combines the efficiency of automated data management systems with the convenience of personal computers to provide users with an alternative information source that can be accessed interactively. The 12,700 record database which documents the largest public domain holding of strong motion data includes information on triggering events, recording station characteristics and peak ground motion values. This paper describes the development and features of this catalog.

1. INTRODUCTION

Many studies regarding the effects and potential impacts of strong ground motions rely on the analysis of historic accelerograms. The need for convenient access to existing strong motion data has been widely documented. The U.S. National Workshop on Strong Motion Instrumentation notes that research advancements in the field of earthquake engineering can, and may often, be impeded by insufficient data (ref. 4).

With the thousands of processed accelerograms already in existence, it can be a major undertaking for engineers and researchers to locate all of the data applicable to a particular study. Strong motion catalogs which document this vital data resource have proven to be extremely useful for investigators needing to locate data pertinent to their application. Although these catalogs have appeared in a variety of formats, such as printed listings, on-line information banks, and graphic work stations (see refs. 1, 2 and 3), they all tend to have their drawbacks as far as either being inconvenient, limited in scope or not up-to-date. An alternative solution to documenting this resource is a personal computer-based catalog system known as SMCAT. SMCAT, an abbreviation for Strong Motion Catalog, utilizes the many advantages of personal computers to provide a convenient database which contains ample information.

The catalog documents the National Geophysical Data Center (NGDC) and World Data Center-A (WDC-A) Strong-Motion Data Archive -- the largest public domain holding of digitized strong motion data. The catalog is accessed via a menu-driven, front-end program which allows for interactive searches and retrievals from the database. Special characteristics of the catalog provide for a multitude of features such as simple updates and linkages with existing databases. A combination of the efficiency of automated database management systems with the convenience of personal computers attributes to the practicality and effectiveness of this catalog.

The SMCAT catalog is a dBase III+ formatted database which documents the contents of the NGDC/WDC-A Strong Motion Archive. The initial release of the catalog contains over 12,700 entries or records. Each record in the database is comprised of 29 fields of information which describe the triggering event parameters, recording station characteristics, and peak recorded ground motion values for every processed accelerogram in the WDC-A archive (table 1). Most of the information fields used in the catalog are those deemed most useful for engineers and researchers by the U.S. National Workshop on Strong Motion Instrumentation (ref. 4). Additional useful data such as recording

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site geology, fault plane parameters and structural characteristics were left out of the present catalog because of the difficulty involved in obtaining much of this information.

The scope of the catalog encompasses all of the public domain strong motion data dating from 1933 (the first U.S. accelerograph recording) to the present that were contributed to WDC-A from numerous government agencies, educational institutions, and industries located around the world. Although almost half of the records originated from accelerographs installed in the United States, there are also a significant amount that originated from fifteen other countries (fig. 1). Records in the archive are representative of a broad range of recording environments from structural to free field (fig. 2). Structural records include those that were obtained from a diverse selection of dams,bridges, and utilities, as well as various types of buildings. Recording site geologies cover almost the entire spectrum from hard rock to alluvium and fill.

2. DATA COLLECTION

Information used in the catalog was collected from three main sources. Most of the data pertaining to the triggering event and station characteristics are extracted from reports and technical papers published by agencies responsible for the collection and processing of accelerograms. Unfortunately, most of these reports are published rather promptly and, hence, do not contain comprehensive information. Seismological catalogs were used as a supplement to fill in any information regarding the triggering event. Finally, any information that still remained unknown, such as peak ground motion values, was obtained from the archived accelerogram data files.

Most of the collected information was entered into the catalog database manually. However, automated procedures can be used whenever large sets of data, such as the CALTECH series or the Taiwan SMART-1 array data, are available in redundant file formats. In this case, a short FORTRAN or C program was quickly produced to extract as much information as possible from the file headings and data points.

In theory, the process of incorporating information into the SMCAT database seems rather straightforward; in reality, however, the process is typically complex. The difficulty in developing this type of catalog can be attributed to the hodgepodge of data and documentation formats. Ideally, all of the information should be contained in the heading of the accelerogram data file; this would not only aid in the development of catalogs, but would undoubtedly benefit the users of these data. Compilation of this catalog would become a trivial task if all of the necessary information were written into the files in an internationally standardized format.

3. INFORMATION RETRIEVAL

The catalog is disseminated to users in a dBase III+ format on 5 1/4 inch floppy diskettes. A front-end program is included for interactive searches and retrievals. This menu-driven program acts as an interface between the user and the catalog enabling the user to access the database without dBase III+ software. A search is initiated by defining the search parameters and specifying the range of each parameter. For instance, a typical search may include locating all of the records obtained from free field sites located at a maximum distance of 20 kilometers from the epicenter and that recorded a peak acceleration greater than 20% g. Up to five fields can be selected to specify a search condition (table 2). The data output can be directed to a printer, monitor, or stored in an output file. Stored output can be re-searched with a new set of parameters if necessary. Three types of output file formats are available: dBase III+, ASCII, and Lotus 1-2-3. This option greatly increases the flexibility of the product by not restricting the user to a dBase III+ format.

4. ADDITIONAL FEATURES

The use of dBase III+ as a database management system technology greatly increases the capabilities of this catalog. Several features inherent in dBase and other features that have been designed into the catalog program provide an adequate foundation for users to expand upon. New fields can be added to the catalog without significantly affecting the performance of the search program. Suitable candidates for additional fields could pertain to site geology, structural characteristics, or fault plane parameters. Any additional fields that are not part of the original database structure cannot be accessed by the program. For users that prefer management software other than dBase, the SMCAT program will export the entire database, or a selected portion of the database, to either an ASCII or Lotus 1-2-3 formatted file.

If modification of the existing database is not desirable, then the catalog can be linked to an external database through a related field such as event date. Use of the program can be avoided altogether by exporting the database to the desired file format and using the appropriate software to access the records. Editing or altering of the catalog can easily be accomplished by using one or more of the editing utilities available in dBase.

5. CONCLUSIONS

The SMCAT strong motion catalog is designed to assist engineers and researchers in locating accelerograms suitable for particular applications. With the diverse and extensive selection of historic recordings that are documented in this catalog, a user can find records applicable to most studies. Special features contribute to the flexibility of the catalog which enable users to tailor or apply the system to suit specific needs.

6. REFERENCES

1. Coffman, J.L. and Godeaux, S. 1985. "Catalog of Strong-Motion Accelerograph Records," World Data Center-A for Solid Earth Geophysics Report SE-38, 79 p.

2. Polhemus, N.W. and Jones E. 1984. "Interactive Database for Graphic Retrieval of Strong-Motion Accelerograms," in Proceedings of the Eighth World Conference on Earthquake Engineering, July 1984, San Francisco, California, v. 2, p. 87-91.

3. Converse, A., 1978. "Strong-Motion Information Retrieval System User's Manual," U.S. Geological Survey Open File Report 79-289, 51 p.

4. Iwan, W.D. 1981. "U.S. Strong-Motion Earthquake Instrumentation, Proceedings of the U.S. National Workshop on Strong-Motion Earthquake Instrumentation," April 1981, Santa Barbara, California, 55 p.

Fields that Comprise SMCAT

Field Name	Description	Type of Field	Character Width
EQDATE	Event Date	Numeric	6
EQTIME	Event Time	Numeric	8
EPILAT	Epicentral Latitude	Numeric	6
E_NS	Latitude North/South Flag	Character	1
EPILONG	Epicentral Longitude	Numeric	7
E_EW	Longitude East/West Flag	Character	1
EQNAME	Event Name	Character	30
DEPTH	Focal Depth	Numeric	3
INT	Maximum Intensity (MM)	Numeric	2
MAG	Magnitude (M_L)	Numeric	4
CODE	Station Code	Character	7
COMPONENT	Instrument Component	Character	7
STATION	Station Location	Character	30
STATE	State (recording site)	Character	2
COUNTRY	Country (recording site)	Numeric	5
STRUCTURE	Structure Type	Character	3
EPIDIST	Epicentral Distance	Numeric	3
MAXACCEL	Maximum Acceleration	Numeric	10
MAXVEL	Maximum Velocity	Numeric	8
MAXDISP	Maximum Displacement	Numeric	7
INSTRUMENT	Instrument Type	Character	7
FLOOR	Floor Number (recording Site)	Numeric	2
STALAT	Station Latitude	Numeric	6
S_NS	Latitude North/South Flag	Character	1
STALONG	Station Longitude	Numeric	7
S_EW	Longitude East/West Flag	Character	1
TAPE	NGDC Data Set Name	Character	7
TYPE	Data or Record Type	Character	1
FILE	Data Set File Number	Numeric	4
		Total Characters:	190

Table 1. Table lists the 29 fields that make up the SMCAT database.

Searchable Fields in the SMCAT Database

Field	Units	Range
Event Date	UTC	1933 - present
Epicentral Coordinates	degrees	global
Event Name		
Focal Depth	kilometers	0 to 9999
Maximum Intensity	MM	1 to 12
Magnitude	M_L	1 to 10
Station Name		
Country (recording site)		
State (recording site)		
Structure Type (recording site)		
Building Floor Number (rec. site)	floor #	-9 to 99
Epicentral Distance	kilometers	0 to 9999
Maximum Acceleration	gals	0 to 99999
Maximum Velocity	cm/sec	0 to 9999
Maximum Displacement	centimeters	0 to 999
Station Coordinates	degrees	global
NGDC Data Set Name		

Table 2. Table lists the fields that can be searched by the SMCAT program. Up to five fields can be searched at one time with the program.

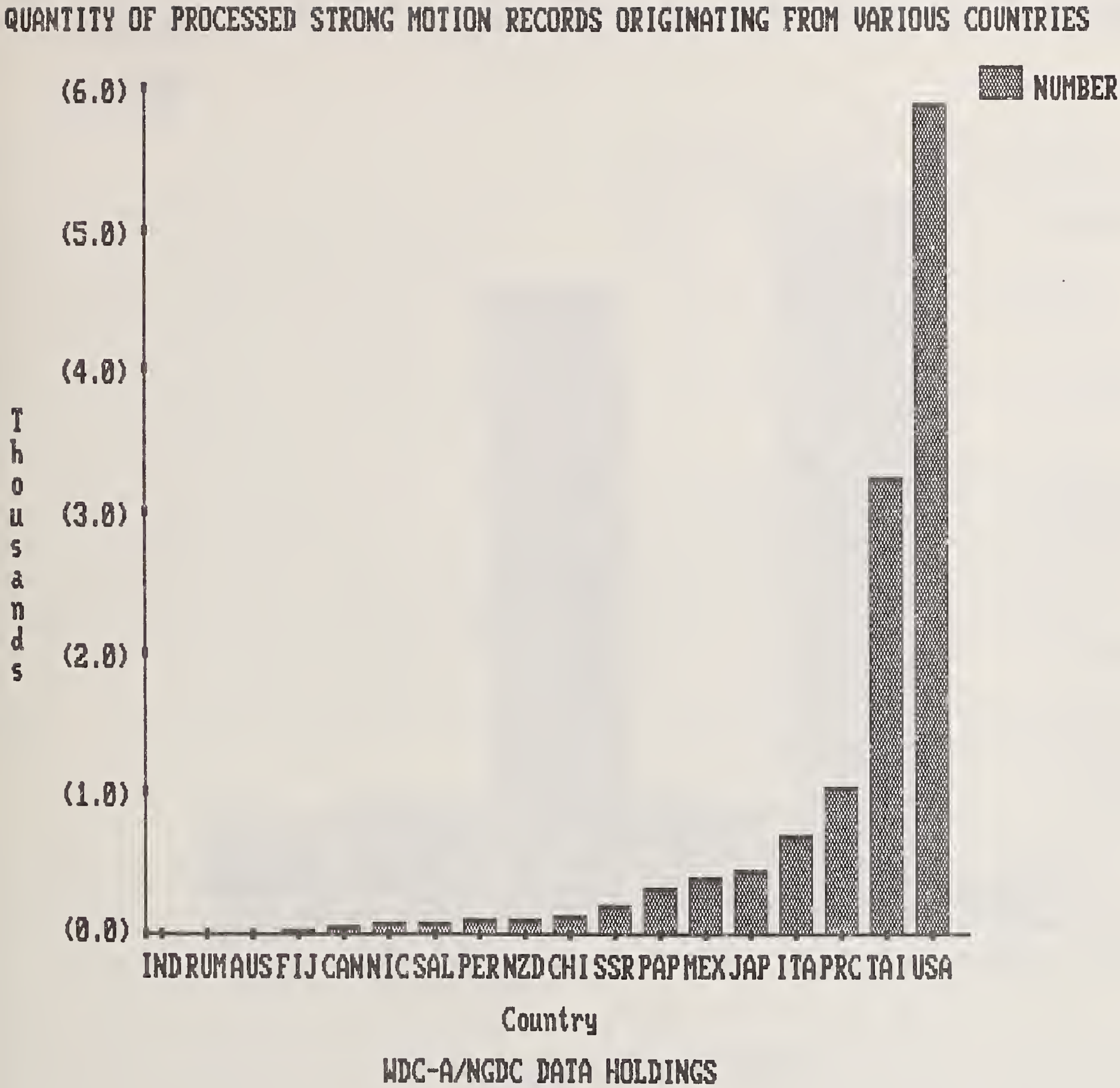


Figure 1. Chart shows the quantity of processed strong motion records originating from various countries represented in the SMCAT catalog. One record is defined here as a single component record with only one phase of processing. A record with all three phases of processing (raw, filtered, and response spectra) is considered three separate records. Country codes are as follows: IND: India, RUM: Rumania, AUS: Australia, FIJ: Fiji, CAN: Canada, NIC: Nicaragua, SAL: El Salvador, PER: Peru, NZD: New Zealand, CHI: Chile, SSR: Union of Soviet Socialist Republics, PAP: Papua New Guinea, MEX: Mexico, JAP: Japan, ITA: Italy, PRC: Peoples Republic of China, TAI: Taiwan, USA: United States. Less than ten records originated from each of the countries of India, Rumania, and Australia.

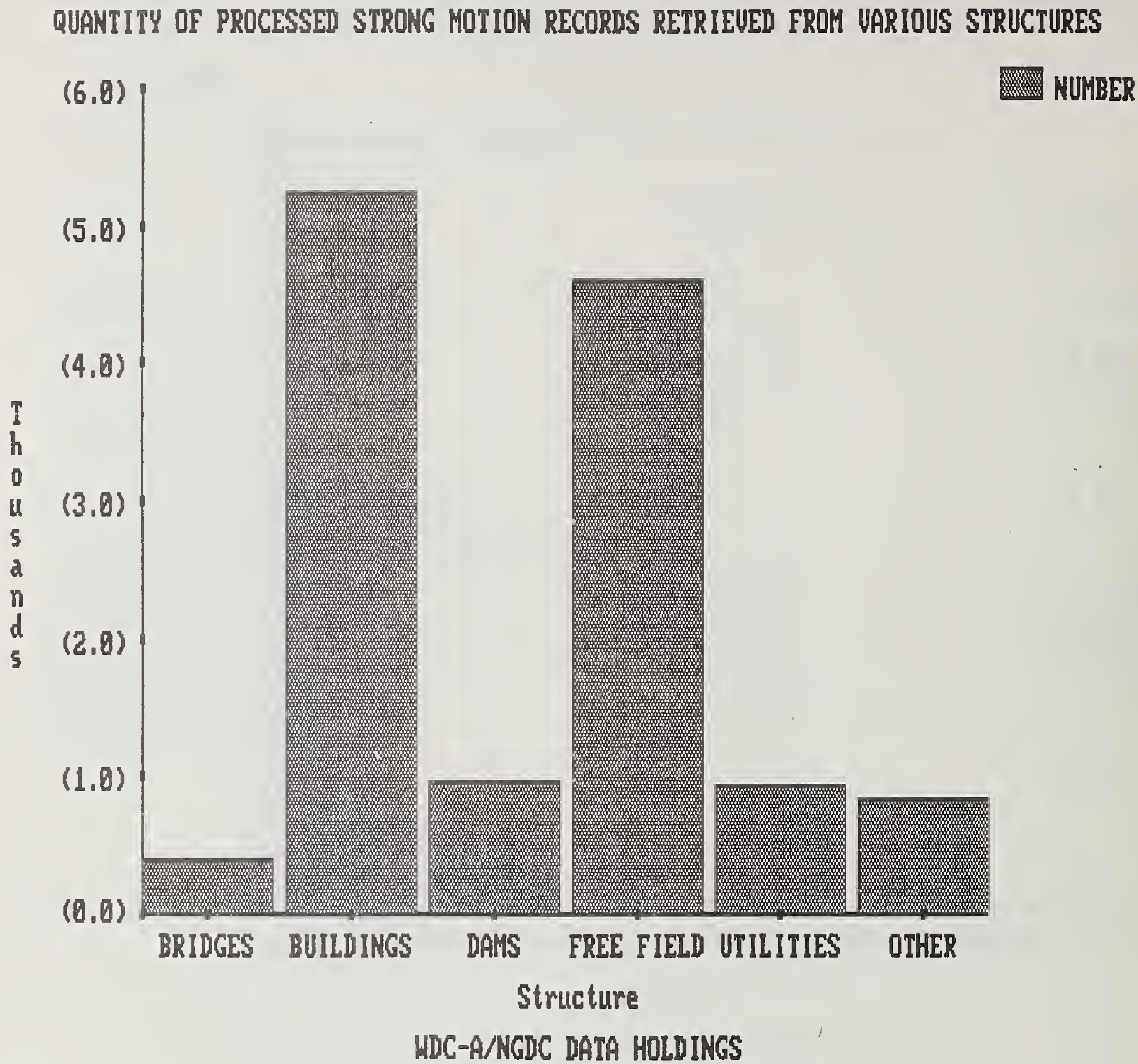


Figure 2. Chart shows the quantity of processed strong motion records retrieved from the various structures represented in the SMCAT catalog. One record is defined here as a single component record with only one phase of processing. A record with all three phases of processing (raw, filtered, and response spectra) is considered as three séparate records.

Nonlinear Seismic Analysis of Concrete Gravity Dams

by

Lucian G. Guthrie[1]

ABSTRACT

This paper describes an investigation of the behavior of concrete gravity dams subjected to earthquake motions of sufficient strength to induce cracking. Nonlinear, dynamic finite element analyses of three dams subjected to the Parkfield earthquake motion are conducted with the ADINA 84 code. In the analyses the essential characteristics of static preloading, bidirectional seismic motion, dynamic concrete cracking, and hydrodynamic interaction are modeled. The results show that cracked zones can propagate through the cross sections at various elevations. The U.S. Army Corps of Engineers' guidance for seismic evaluation of gravity dams compares conservatively to these results but incorrectly locates the elevation of cracking through the section in some cases. A procedure to estimate the relative permanent displacement across such cracked sections is developed using the sliding block analysis.

KEYWORDS: Concrete gravity dams; earthquake analysis; finite element analysis.

1. INTRODUCTION

U.S. Army Corps of Engineers publication ETL 1110-2-303 (U.S. Army CE, 1985) provides design guidance for determining the earthquake-induced loading, performing the dynamic stress analysis, and evaluating the results for concrete gravity dams. That guidance requires an analysis to estimate the extent of cracking, and a sliding stability analysis on planes where the greatest cracking occurs. Cracking is indicated when a linear elastic analysis calculates combined maximum tensile stresses greater than 0.1 times the unconfined compressive strength of the concrete using damping ratios of 7 or 10 percent. The sequence of analysis is shown in Figure 1.

Where cracking is indicated an estimate of the location and extent of cracking and an evaluation of the severity of this cracking are required. To accomplish this a dynamic nonlinear analysis of the dam is required to determine the location and extent of cracking. This is followed by a sliding stability analysis of any section above a plane determined to be cracked.

To confirm or appropriately modify the guidance given in ETL 1110-2-303, as described in Figure 1, an analytical investigation was conducted on the behavior of concrete gravity dams subjected to earthquake motion of sufficient strength to induce cracking (Mlakar, 1987). This paper describes that investigation and reports on the results.

2. DYNAMIC NONLINEAR ANALYSIS INVESTIGATION

2.1 Tasks of Investigation

The investigation encompassed the following tasks:

1) Performed nonlinear analyses on three given nonoverflow gravity dam cross sections of 185, 300, and 638 feet (56, 91, and 194 meters) in height using earthquake-induced ground motion of sufficient strength to induce cracking. The results of the analyses were used to estimate the extent of cracking possible and any permanent displacements likely to occur along planes of maximum cracking.

2) Evaluated the interim procedure contained in ETL 1110-2-303 for analyzing the response of a gravity dam to a maximum credible earthquake.

3) Developed a simplified method of analysis to estimate upper bounds for permanent displacements due to vibratory ground motion without resorting to multiple acceleration time-history analyses.

2.2 Particulars of the Nonlinear Analyses

2.2.1 Computer Code and Discretization

The nonlinear analyses were performed using the general purpose finite element code ADINA (ADINA Engineering, 1984). The cross section of each structure was modeled using 9-node, isoparametric, quadrilateral, plane stress elements of unit thickness. The discretization of the finite element meshes is shown in Figure 2. This degree of discretization was found to adequately reproduce the stress distributions within gravity dams (Cole and Cheek, 1986).

[1] U.S. Army Corps of Engineers, HQUSACE (CEEC-ED), Washington, DC 20314-1000

2.2.2 Constitutive Modeling

The constitutive behavior of the concrete of the dams was described with the ADINA concrete material model (Bathe and Ramaswamy, 1979). For the static and the seismic loadings imposed, this description provided an essentially linear behavior in compression and a linear behavior in tension up to the stress level at which cracking occurred. When this level of stress was reached, spatial zones of cracked material were defined. In these zones the tensile stiffness across the cracked surface was reduced to zero and the shear stiffness was reduced to half the uncracked value. The effects of strain rate on material behavior were approximated as described in Mlakar, 1987.

2.2.3 Loadings

In the nonlinear analysis, both static and seismic loadings were considered. The self-weight of the cross section was applied as a mass proportional body force. The hydrostatic loading of the reservoir was accomplished through a set of pressure loadings along the upstream surface of the structure. Both of these loadings were linearly increased from zero initially to their full static value in a rise time of 2 seconds. This rise time was selected to achieve equilibrium under the static loading prior to the arrival of the strong earthquake motion. The horizontal and the vertical earthquake loading were applied as uniformly prescribed displacements along the base of the dams. Finally, the hydrodynamic loading of the reservoir was approximated by adding concentrated nodal masses on the upstream face corresponding to the distribution used in the linear elastic analysis as described in Cole and Cheek, 1986.

The earthquake record (Figure 3) used in the analyses was the N65W horizontal and vertical components of the 1966 Parkfield, California earthquake recorded at Temblor No. 2 Station, CIT File Nos. B037-1 and B037-3. This loading is representative of strong earthquake motions likely to be encountered at rock sites, upon which all Corps of Engineers concrete dams are founded.

2.2.4 Discussion of Results

When excited by an earthquake motion strong enough to initiate cracking, the dams sections analyzed cracked completely through their cross sections. This is consistent with limited scale model tests (Niwa and Clough, 1980). In some cases the cracked zones grew through stable stages. In other cases, the cracking was virtually instantaneous once it began. The practical implication of this may be that dams which crack during an earthquake do crack completely through their cross section. However, further analytical and experimental study is required to support this conclusion in general.

The three dams analyzed cracked completely at various elevations. The shortest structure cracked through its base while the taller ones cracked through the upper elevations. The elevation of complete cracking may be sensitive to the flexibility of the foundation which was not considered in this nonlinear analysis. However, a state-of-the-art linear analysis which included this feature (Fenves and Chopra, 1984) indicates that the maximum total stress occurs at the upstream edge of the base. This is the same location at which cracking initiated in each of the nonlinear analyses conducted in this investigation.

The Procedure of ETL 1110-2-303 conservatively evaluated the safety of each of the four dam-earthquake combinations considered in this work. But in two cases, the simple linear procedure was misleading in that the elevation of cracking through the cross section differed from that observed in the nonlinear analysis. It is important that this elevation be correctly predicted so that a subsequent estimate of permanent displacement along the cracked plane is meaningful. This prediction requires that the inelastic behavior following initial cracking be incorporated in the evaluation procedure. This might be simply achieved with a static nonlinear resistance function and an inelastic response spectrum.

3. <u>STABILITY OF CRACKED SECTIONS</u>

It has been shown that extremely strong ground shaking can cause cracking through the cross section of a concrete gravity dam. In fact, this may have occurred as shown in Figure 4, at the Koyna Dam during the December 1967 earthquake (Chopra and Chakrabarti, 1971). The Corps' analytical procedure for the maximum credible earthquake (Figure 1) would require an estimate of permanent displacement in such a case. A simple procedure for this estimation is developed and illustrated in Mlakar, 1987.

3.1 <u>Description of Sliding Analysis</u>

In the proposed procedure the cracked surface is assumed to be planar and either horizontal or inclined (Figure 5). The portion of the monolith above this crack is assumed to be a rigid body subject to sliding along the cracked plane as the portion of the monolith below the crack moves horizontally due to the ground motion. This idealization resembles the sliding block analysis widely employed for earth embankments (Newmark, 1965), (Franklin and Chang, 1977), (Hynes-Griffin and Franklin, 1984) and should be no less applicable to gravity dams.

The limiting acceleration is calculated from equilibrium of the monolith above the crack and the amplification of ground motion is computed from an approximation consistent with the simplified stress analysis of gravity dams as described in Cole and Cheek, 1986.

3.2 Discussion of Results

The procedure leads to a conservative result in each of the two examples illustrated in Mlakar, 1987. The method may be improved through an experimental examination of the frictional coefficient employed to compute the limiting acceleration. An analytical study of the amplitude and frequency modulation of ground motion within the cracked structure may also lead to a better procedure.

3.3 Overturning Stability

Corps of Engineers criteria (U.S. Army CE, 1985) states that an overturnig mode of failure of a gravity dam due to an earthquake-induced excitation is not likely because of the large base width to height ratios of gravity dam cross sections and the nature of the excitation--peak oscillatory motion of very short duration.

4. CONCLUSIONS

The funding limitation for the Mlakar, 1987 study permitted the investigation of a rather limited number of cases. In addition, foundation interaction effects were not taken into account, only one earthquake record was used, and the hydrodynamic loading of the reservoir was approximated by adding concentrated nodal masses on the upstream face corresponding to the distribution used for the linear elastic analysis. So, broadly applicable conclusions are not supported by this study alone. However, the following observations can be made from the results of the study:

1) When excited by an earthquake motion of sufficient strength to initiate cracking, the three dam cross sections analyzed cracked completely through. In some cases, this cracking progressed through stable stages and in others, it was virtually instantaneous.

2) The cracking through the dam cross sections occurred at the base, at the elevation of downstream slope change, and at a lower elevation. The shortest structure cracked through its base while the taller ones cracked through the upper elevations.

3) The procedure in ETL 1110-2-303 conservatively evaluated the safety of the four cases considered. However, in two cases the simplified dynamic analysis procedure incorrectly located the elevation at which cracking occurs through the cross section.

4) The sliding block analysis is a rational bases to estimate the permanent relative displacements along cracked planes through a dam cross section.

5. REFERENCES

ADINA Engineering, 1984, "Automatic Dynamic Incremental Nonlinear Analysis Users Manual," Report AE 84-1, ADINA Engineering, Watertown, MA.

Bathe, K. J., and Ramaswamy, S., 1979, "On Three-Dimensional Nonlinear Analysis of Concrete Structures," Journal of Nuclear Engineering and Design, Vol 52, pp 385-409.

Chopra, Anil K., and Chakrabarti, P., 1971, "The Koyna Earthquake of December 11, 1967 and the Performance of Koyna Dam," Report UCB/EERC 71-1, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Cole, R. A., and Cheek, J. B., 1986, "Seismic Analysis of Gravity Dams," Technical Report SL-86-44, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Franklin A. G., and Chang, F. K., 1977, "Earthquake Resistance of Earth and Rock-Fill Dams: Permanent Displacement of Earth Embankments by Newmark Sliding Block Analysis," Miscellaneous Paper S-77-17, Report 5, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Hynes-Griffin, M. E., and Franklin, A. G., 1984, "Rationalizing the Seismic Coefficient Method," Miscellaneous Paper GL-84-13, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Mlakar, Paul F., 1987, "Nonlinear Response of Concrete Gravity Dams to Strong Earthquake-Induced Ground Motion," Technical Report SL 87-7, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Newmark, N. M., 1965, "Effect of Earthquakes on Dams and Embankments," Geotechnique, Vol 15, No. 2, pp 139-160.

Niwa, A., and Clough, R. W., 1980, "Shaking Table Research on Concrete Dam Models," Report UCB/EERC 80-05, Earthquake Engineering Research Center, University of California, Berkeley, CA.

U.S. Army Corps of Engineers, 1985, "Earthquake analysis and Design of Concrete Gravity Dams," Engineer Technical Letter 1110-2-303, Washington, DC.

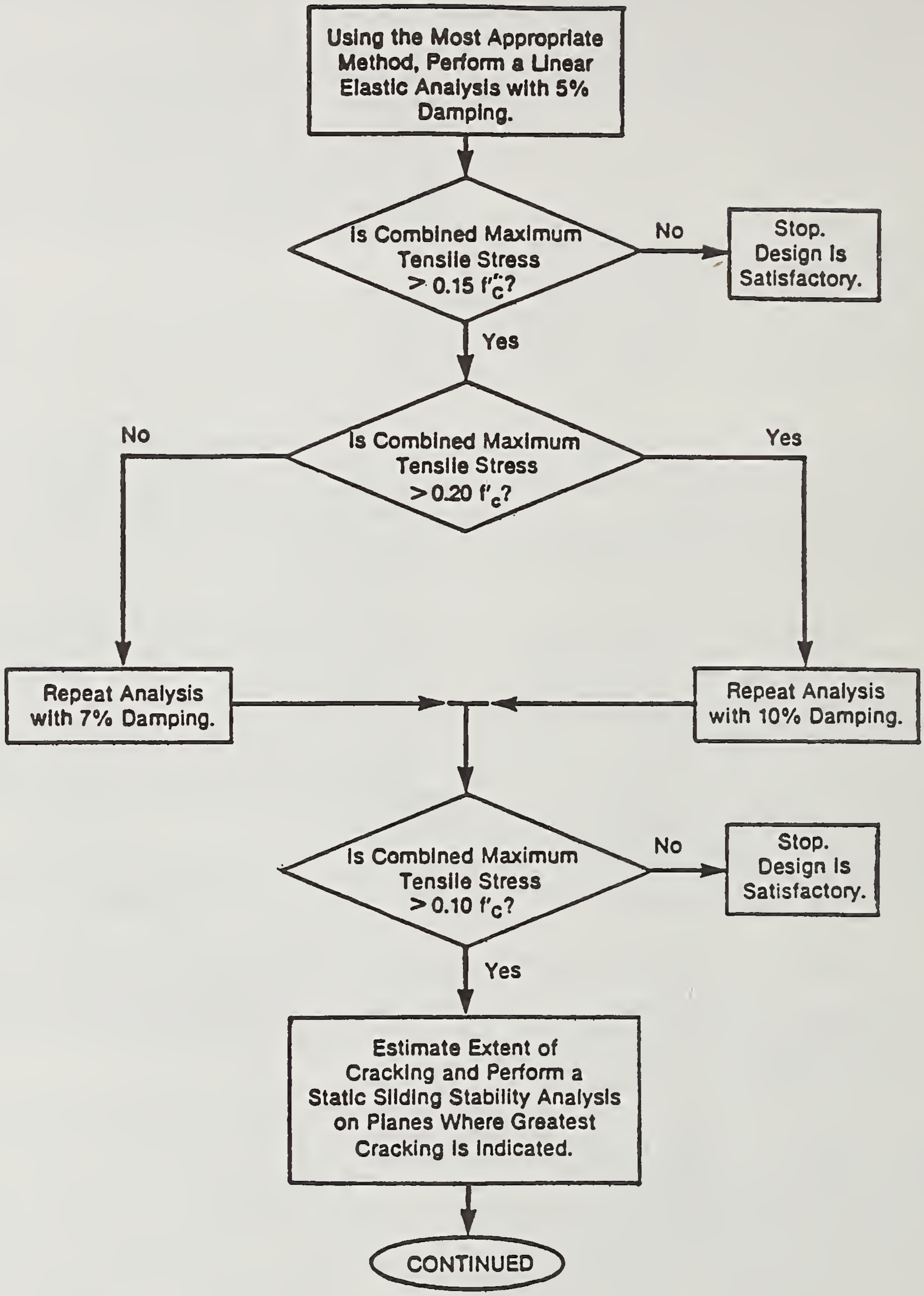


Figure 1a

Figure 1. Sequence of analysis for the maximum credible earthquake (DA 1985) (Sheet 1 of 2)

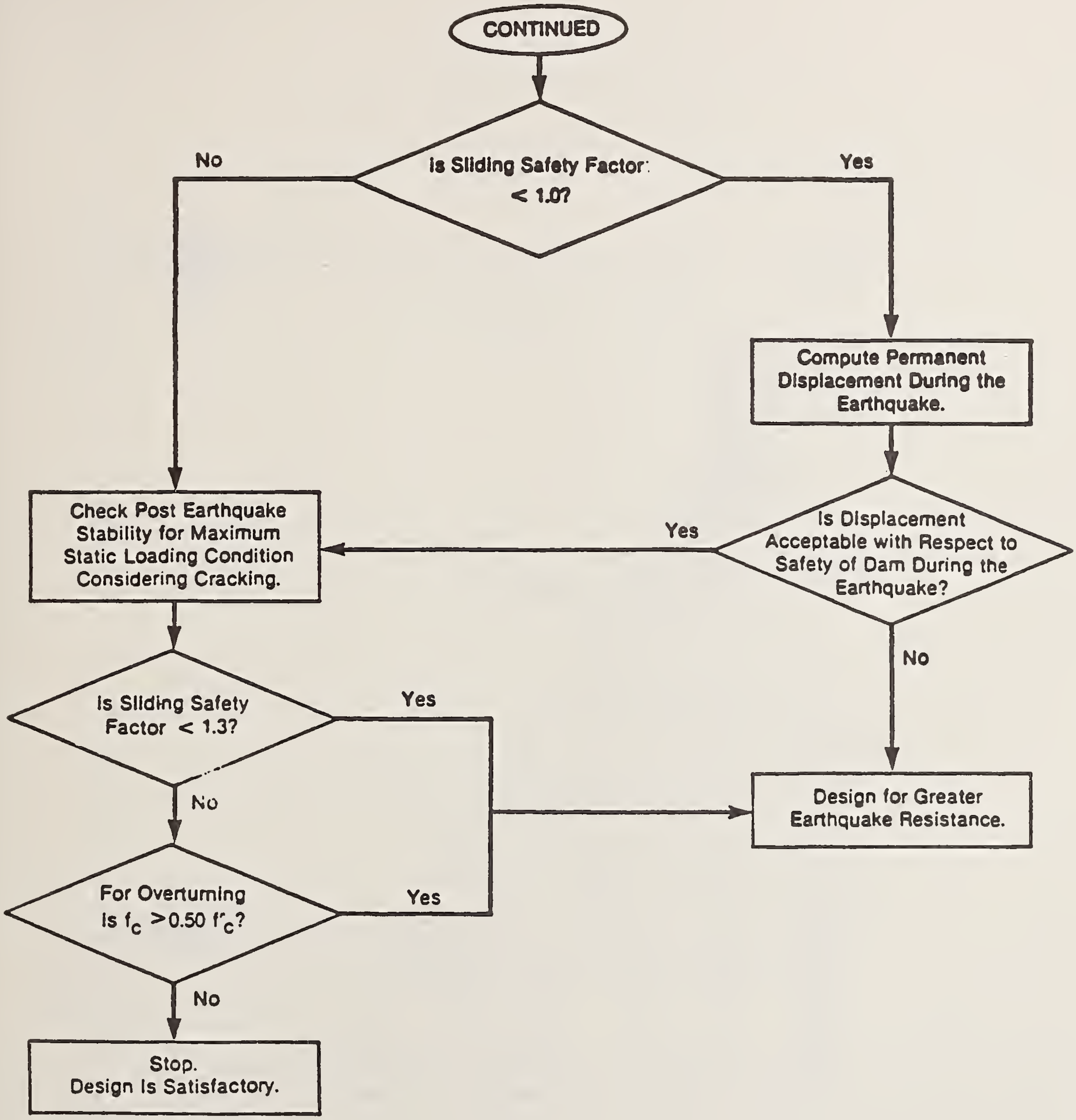


Figure 1b

Figure 1. (Sheet 2 of 2)

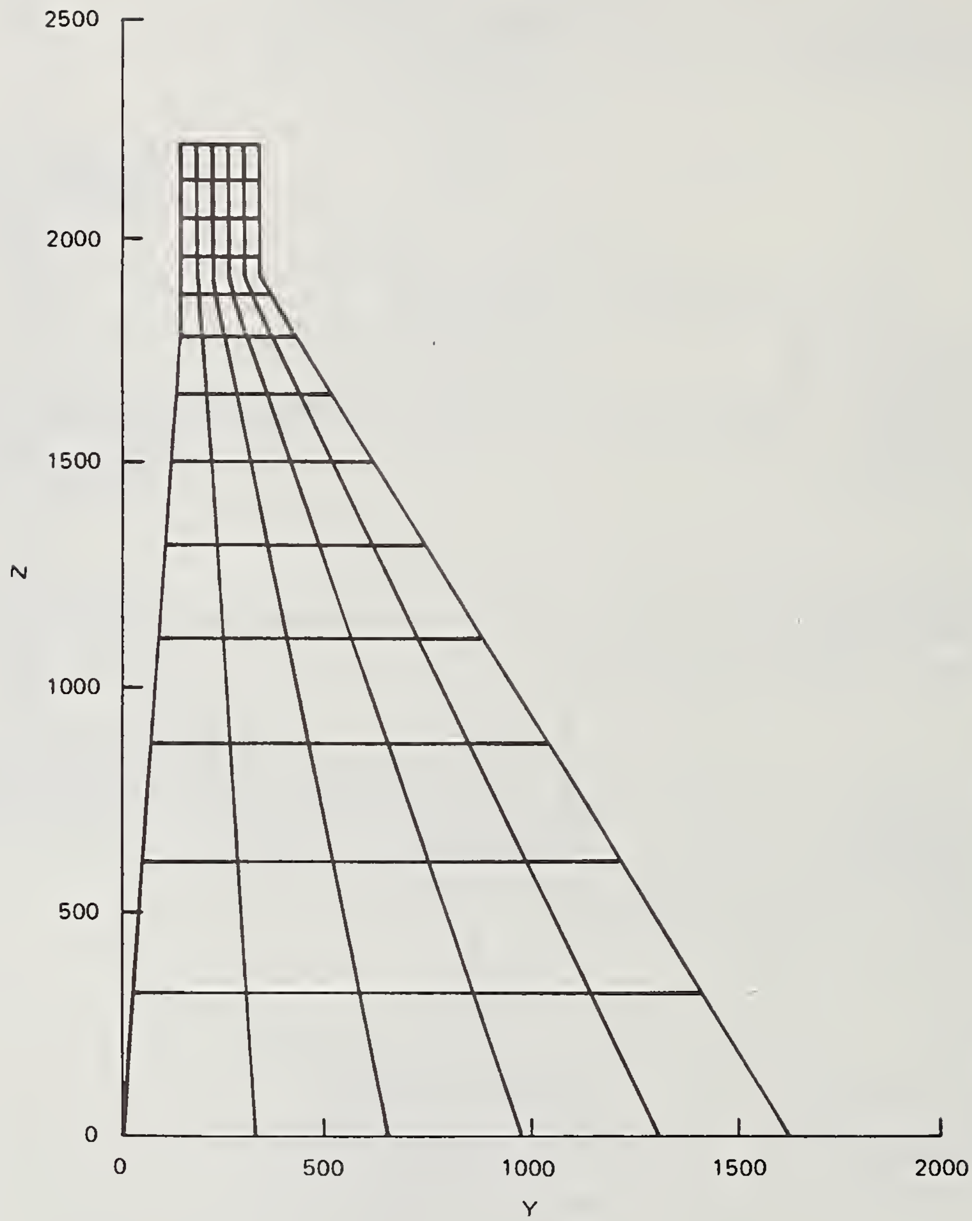


Figure 2. Finite element grid of Russell Dam.

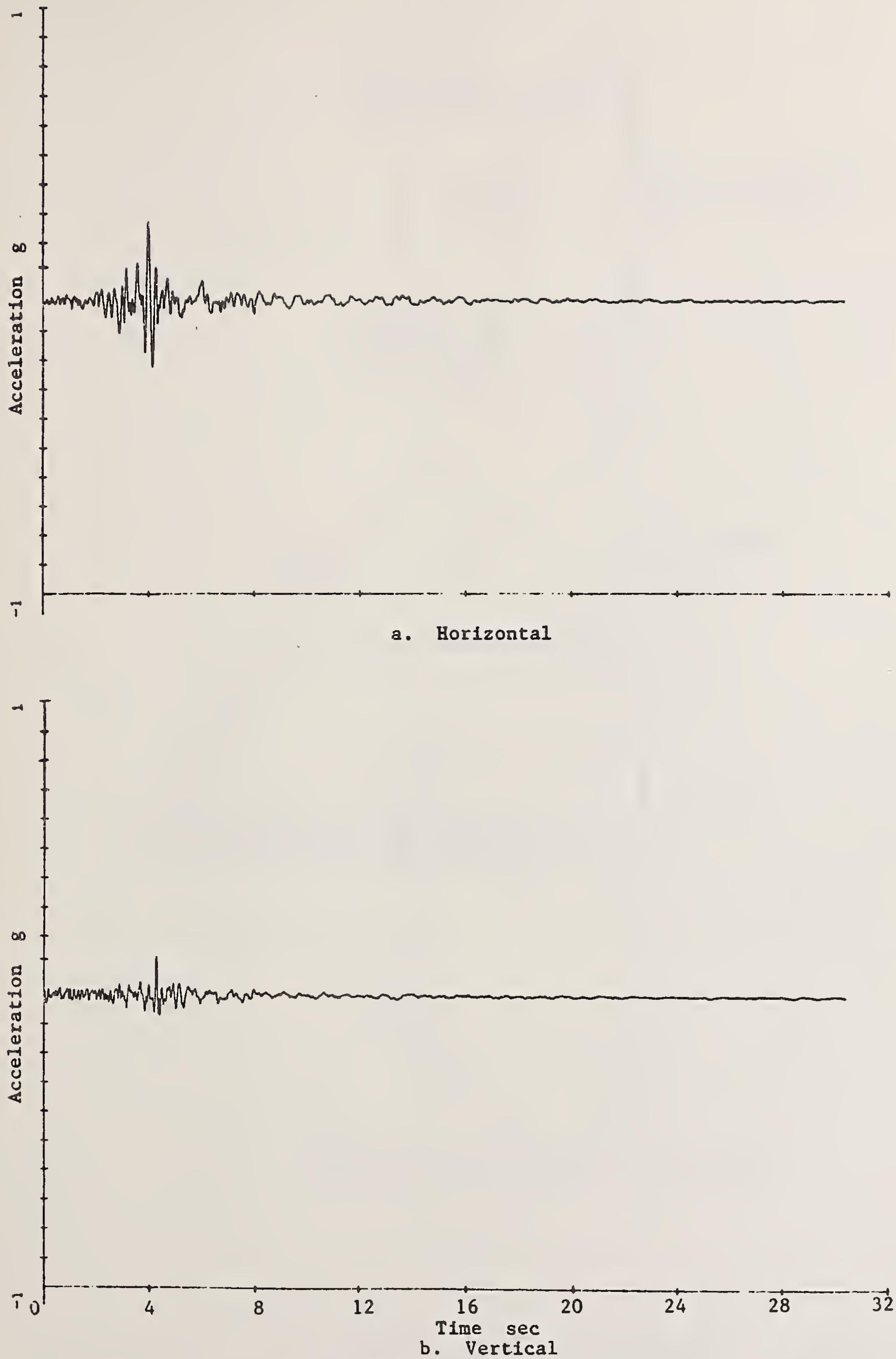


Figure 3. Parkfield earthquake motions used in analyses

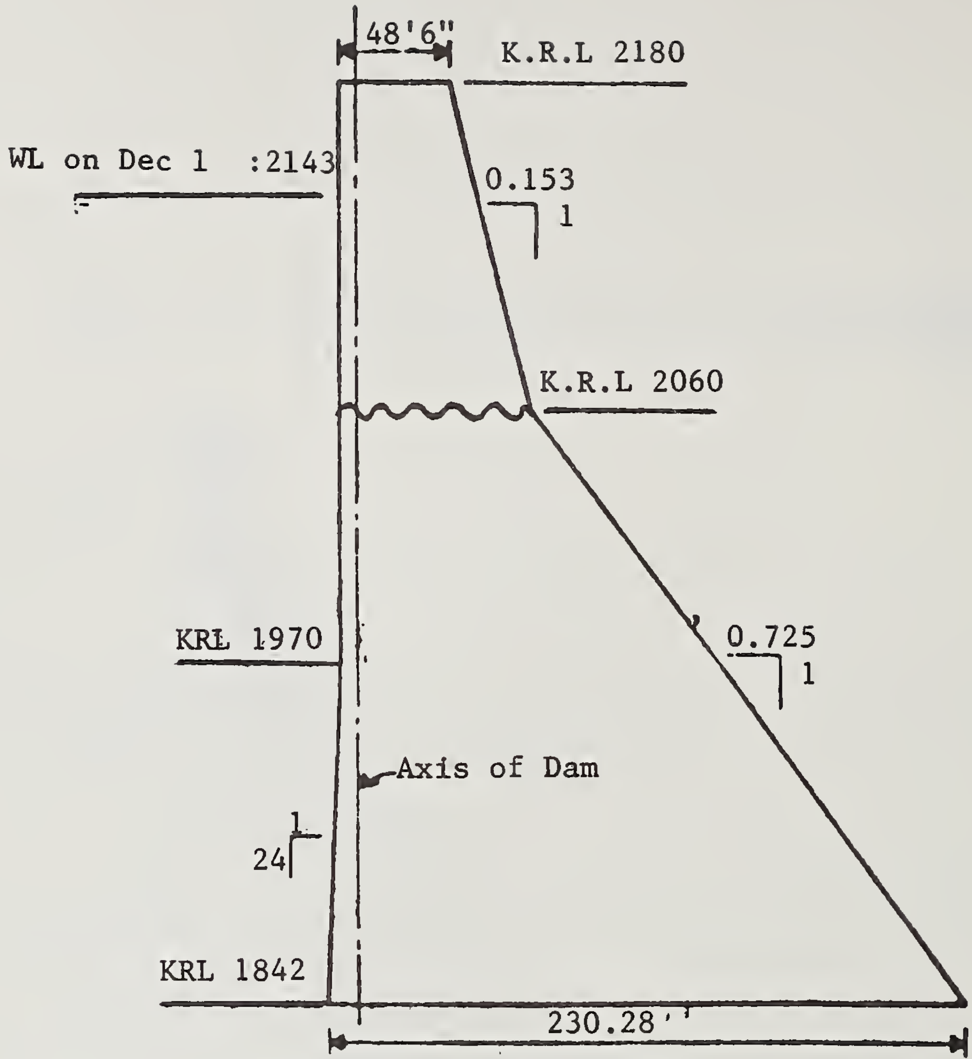
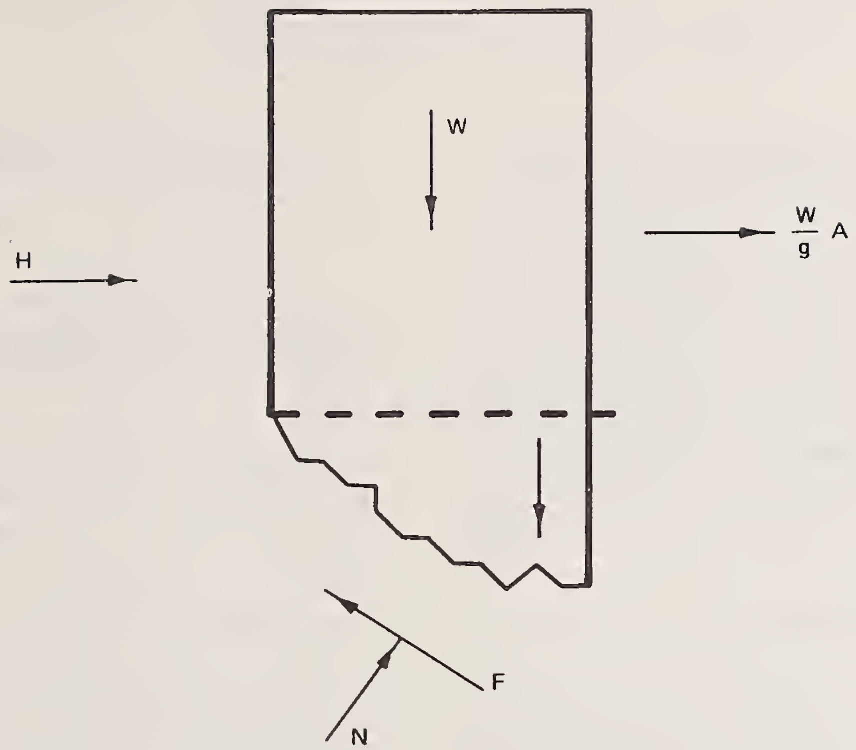
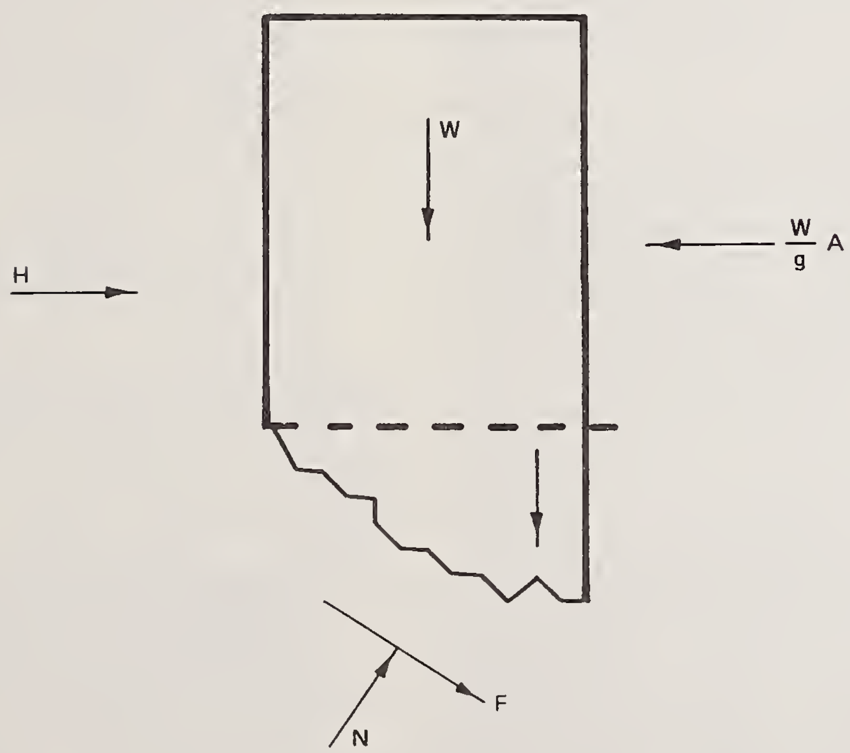


Figure 4. Cracking of Koyna Dam



a. IMPENDING SLIDING DOWNSTREAM



b. IMPENDING SLIDING UPSTREAM

Figure 5. Free body diagram of cracked dam.

Seismic Microzonation

By

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ABSTRACT

This paper reviews the basic principles of seismic microzonation--the multidisciplinary process that leads to the division of a region into smaller areas expected to experience the same relative severity of an earthquakes physical effect such ground shaking. Since the original pioneering research by Japanese scientists and engineers in the early 1900's, many significant advances in understanding have been made. The time seems to be right for renewed efforts in seismic microzonation to be made worldwide.

1. WHAT IS TO BE DONE?

1.1 Design Goal

The goal is to devise a set of standard procedures that can be used throughout the world to produce seismic microzonation products. Products produced in a seismic microzoning study are applied in land-use, building codes, design and construction practices, repair and strengthening of existing buildings, and response and recovery planning. These applications can save lives and economic resources.

Although the pre-earthquake environment is more optimal, experience shows that seismic microzonation studies are accepted more readily as a strategy for reducing losses from earthquake hazards in the post-earthquake environment. Political considerations are usually less of an impediment in the post-earthquake environment.

1.2 Background

Seismic microzonation, the division of a region into smaller areas expected to experience the same relative severity of an earthquake hazard (for example, ground shaking, surface fault rupture, earthquake-induced ground failure, tectonic deformation, or tsunami runup) is an important part of the process of evaluating earthquake hazards and assessing the risk in an urban area. Seismic microzonation is the part of the process of evaluating earthquake hazards that provides the prospective user of an area with the design criteria that will permit him to select the most suitable part of the area for the proposed use.

2. HOW WILL IT BE DONE?

2.1 Compilation of Seismic Microzonation Experience

Seismic microzonation has been performed in many countries throughout the world. However, there is no standard procedure for seismic microzonation, and the results have varied widely from country to country (see proceedings of the three international conferences on seismic microzonation held in the United States and the proceedings of the microzonation conference held in Algeria).

2.2 Technical Procedure

The key to the creation of a standard procedure in seismic microzonation is to obtain explicit answers to the following questions:

-- Where are the earthquakes occurring now?
-- Where did they occur in the past?
-- Why are they occurring?
-- How often do earthquakes of a certain size (magnitude) occur?
-- How big (severe) have the physical effects been in the past?
-- How big can they be in the future (e.g., next 50 years)?
-- How do the physical effects vary spatially and temporally?
-- How have these physical effects impacted various types of buildings and lifeline systems?

Although these questions appear to be simple, the answers typically require detailed research and technical studies that integrate geologic, seismological, and engineering data on two scales:

1) Evaluation of seismic hazards on a regional scale: (a map scale of about 1:100,000 to 1:1,250,000). This part of a microzoning study establishes the physical parameters of the region needed to evaluate the earthquake hazards of ground shaking, surface fault rupture, tectonic deformation, and tsunami runup. Technical tasks such as the following are required:

   a. Compilation of a catalog and map of the prehistorical, historical, and current seismicity.

b. Performance of neotectonic studies (mapping, age dating, and trenching) to acquire information on recurrence times in the past several thousand years not provided by historical seismicity.

c. Preparation of a seismotectonic map showing the location of active faults and their correlation with seismicity.

d. Preparation of a map showing seismogenic zones and giving the magnitude of a maximum earthquake and the frequency of occurrence for each zone.

e. Specification of regional seismic wave attenuation laws and their uncertainty.

f. Preparation of probabilistic ground-shaking hazard maps in terms of peak bedrock acceleration, peak bedrock velocity, exposure times, and probabilities of nonexceedance.

2) Evaluation of seismic hazards on an urban scale: (a map scale of about 1:5,000 to 1:25,000). This part of a microzonation study integrates the seismotectonic and other physical data acquired in the region of the study (Part 1 above) with site-specific data acquired in the urban area to produce seismic microzonation maps. Technical tasks such as the following are required:

a. Acquisition, synthesis, and integration of existing and new geologic, geophysical, and geotechnical data to characterize the soil/rock columns in terms of their physical properties and their response to various levels of ground shaking.

b. Preparation of ground-shaking hazard maps showing the dynamic amplification factors for soil/rock columns in terms of amplitude and frequency composition of ground shaking and the level of dynamic shear strain for a range of seismic loads.

c. Preparation of a map showing the potential for surface fault rupture and tectonic deformation.

d. Preparation of a map showing the potential for liquefaction.

e. Preparation of a map showing the potential for landslides.

f. Analysis of the vulnerability of various types of buildings and lifeline systems under a range of seismic loads.

3. WHAT LEADS US TO BELIEVE IT CAN BE DONE?

3.1 Basic Data

The basic data required for seismic microzonation are available in many countries throughout the world; what is missing is a standard procedure. Microzonation on the regional and urban scales requires the best available information on: 1) seismicity, 2) the nature of the earthquake source zone, 3) seismic wave attenuation, 4) local ground response, and 5) building and lifeline response.

3.2 Scientific and Engineering Problems

A number of technical issues (i.e., questions for which expert judgment is divided between "yes" and "no") have been identified for the problem of microzoning the ground-shaking hazard. They are summarized below to provide examples of their range and complexity. Considerations of structural response and potential vulnerability will not be discussed here. Similar technical problems can be stated for the ground-failure hazard.

Seismicity - The record of historical seismicity in both the United States and other countries varies considerably in length and completeness. Lack of completeness can introduce biases in statistical analyses unless careful judgments are made. Incorporating geologic evidence of recent faulting as well as geodetic data improves the likelihood of establishing the best possible recurrence rates for earthquakes. If geologic and geophysical data are not available, it may be extremely difficult to estimate the maximum magnitude in an area, and indeed, it is possible that a number of geographic areas may not have experienced their maximum magnitude earthquake. Use of the record of historical seismicity alone may cause underestimation of the maximum magnitude.

The issues include the following:

a. Will the uncertainty involved in using catalogs of instrumentally recorded and felt earthquakes representing a short time interval and a broad regional area permit a precise specification of the frequency of recurrence of major earthquakes on a local scale?

b. Can the seismic cycle of individual fault systems be determined accurately and, if so, can the point in the cycle be specified?

c. Can the location and magnitude of the largest earthquake that is physically possible on an individual fault system or in a seismotectonic province be specified accurately? Can the frequency of this event be specified?

d. Can seismic gaps be identified and their earthquake potential evaluated accurately?

e. Can discrepancies between the geologic evidence for the occurrence of major tectonic movements in the geologic past and the evidence provided by current and historical patterns of seismicity in a geographic region be reconciled?

Seismogenic Zones - No standard method has been adopted for delineating seismogenic zones. Usually, each cluster of earthquake foci on active faults is considered as a source zone; however, scientific judgment is involved in drawing the boundaries of source zones. For example, one danger is that two or more regions having different seismotectonic characteristics will be incorrectly combined and the resultant analysis will suggest some average but nonexistent physical condition. In defining seismogenic zones, all available information is used to establish the physical correlations between earthquake occurrences and geologic processes and tectonic structures, including: 1) location of the boundaries of crustal blocks which are undergoing contrasting displacements, 2) history of vertical and horizontal regional tectonic movements, 3) the seismic cycle and history of active faults, and 4) tectonic stress. Each seismic source zone is chosen so that it encloses an area of seismic activity and, to the extent possible, an area of related tectonic elements. Although time-dependent models are now available, earthquakes are commonly assumed to have equal probability of occurrence anywhere in a source zone, to have an average rate of occurrence that is constant in time, and to follow a Poisson distribution of recurrences.

The technical issues include the following:

a. Can seismic source zones be defined accurately on the basis of the record of historical seismicity? On the basis of geology and tectonics? On the basis of the record of historical seismicity generalized by geologic and tectonic data? Which approach is most accurate?

b. In assessing the earthquake ground-shaking hazard for a region, can a magnitude be assigned accurately to the largest earthquake expected to occur in a given period of time on a particular fault system or in a particular seismic source zone?

c. Can the physical effects of earthquake source parameters such as stress drop and seismic moment be quantified and incorporated in zoning maps?

Seismic Wave Attenuation - Characterization of the ground motion close to an active fault is one of the most important yet most difficult parts of the problem of constructing a ground-shaking hazard map. The empirical strong ground motion data are currently too limited to resolve all of the technical issues concerning the attenuation characteristics of both near- and far-field ground motion, even though unique ground-motion data have been acquired in the near field in the 1979 Imperial Valley, California, earthquake and in other locations worldwide. These data have reinforced current thinking in some areas and revised it in others, but have not resolved all of the controversial issues concerning seismic wave attenuation. Frequency-dependent effects of the transmission path on earthquake ground motion have not been quantified fully because of limited data. Observational and instrumental data indicate that the regional seismic attenuation rates depend on the physical properties (i.e., Q structure) of the Earth's crust and upper mantle in a region, that the attenuation rates can vary considerably from region to region, and that Q is frequency dependent.

Attenuation curves are required to specify how values of peak ground motion (or spectral velocity ordinants) decrease as distance from the causative fault increases. Such curves are essential when constructing a zoning map of the peak-acceleration ground-shaking hazards. The problem is that many peak-amplitude attenuation curves having substantial differences exist in the literature. The question of magnitude dependence of attenuation is important in probabilistic ground-shaking hazard estimation because it sharply influences the estimated level of maximum ground motion in two cases: 1) areas having a high rate of seismicity, and 2) when long periods of time are considered.

The technical issues include:

a. Can the complex details of the earthquake fault rupture (e.g., rupture dimensions, fault type, fault offset, fault slip velocity) be modeled accurately enough to give precise estimates of the amplitude and frequency characteristics of ground motion close to the fault? Far from the fault?

b. Do values of peak ground-motion parameters or spectral velocity ordinants saturate at large magnitude?

Local Ground Response - Since the early 1900's, literature of earthquake engineering and engineering seismology has recognized and documented that structures founded upon unconsolidated material (soil) are damaged more frequently and usually more severely in earthquakes than structures founded on rock. The damage distribution on many occasions (for example, in the 1967 Caracas, Venezuela, and the 1985 Mexico earthquakes) has been recognized as being related to the specific properties of the site geology. Many past studies have used empirical ground-motion data and analytical models to define the frequency-dependent effects that have been and still are controversial; only acquisition of ground-motion data recorded at sites underlain by rock and a variety of soil columns close to the fault from large- to great-magnitude earthquakes will resolve these arguments.

The technical issues include the following:

a. For various soil types, is there a discrete range of peak ground-motion values and levels of dynamic shear strain where the ground response is repeatable and essentially linear? Is there a range where nonlinear effects dominate?

b. Can the physical effects of selected physical properties of the soil and rock column (e.g., thickness, lithology, geometry, water content, shear-wave velocity, and density) be modeled accurately? Which of these physical properties control the spatial variation, duration, and amplitude and response characteristics of ground motions in a geographic region for the fault-site geometries?

c. Can the variation of ground motion with depth below the surface be modeled accurately in order to estimate the ground-shaking effects on underground lifeline systems?

3.3 Seismotectonic Analogs

The optimal approach is to select countries that have analogous seismotectonic settings. Most of the zones of seismic activity in the United States have counterparts and analogs in other areas of the world. Much more can be learned about the tectonic setting, earthquake mechanics, earthquake hazards, and risk for parts of the United States by studying tectonic analogs in other countries. Certain aspects of source zones in the U.S. that are not clearly understood (i.e., pronounced overburden masking basement feature, lack of long historic record of seismicity, etc.) become impediments or road-blocks to understanding, particularly if efforts are concentrated solely on specific features in the U.S. Critical keys to the understanding of major strike-slip faults like the San Andreas fault may come from research and field mapping of similar features in say Turkey, Guatemala, New Zealand, Venezuela, the Philippines, Japan, Alaska, Iran, Pakistan, western China, and the U.S.S.R. The similar statement can be made for normal and thrust faults. A comparison of some of these fault systems indicates that parts of them are in various stages of their earthquake cycle. By studying analogous critical earthquake-generating features in other countries, it may be possible to "catch" forerunning or precursory features of large shocks and to apply that experience prior to the occurrences of the next large earthquake on, say, the San Andreas fault or other fault systems in the U.S.

Intraplate earthquakes and the state of stress in Australia, Canada, northern Europe, the U.K., parts of Africa, and peninsular India show many similarities to those associated with the Central and Eastern U.S. Many shocks in those areas seem to occur along old fault systems that have moved many times throughout geologic history in response to various plate-tectonics events. The configuration of major tectonic elements and the reactivation of fault systems in the Southeastern U.S., the site of the Charleston earthquake in 1886, are similar to those of west Africa near Accura, Ghana, and the Benue trough of Nigeria. The tectonic setting of the New Madrid seismic zone in the Central U.S. is similar to that of the seismic zone that extends into the interior of Australia near Adelaide. Several zones of intraplate shocks are similar to those of the eastern and Central U.S. in that they are characterized by very large areas for a given level of energy release.

A great deal can be learned from studies in other countries about the repeat time of large earthquakes along given segments of strike-slip and convergent plate boundaries. The average repeat time is a function of the long-term rate of plate movement and the geometry of the rupture zone. Variations in repeat time at a given place appear to be associated with the length of the rupture zone and the amount of seismic slip associated with the last large shock in that zone.

Since the historic record of earthquakes along the Alaska-Aleutian arc is so short, information from convergent zones near Japan, New Zealand, India, Pakistan, the U.S.S.R., the Lesser Antilles, Mexico, Central and South America, and other similar areas can be applied to earthquake-related problems for Alaska and the Aleutians. Similarly, the unusual style of plate motion to the north of Puerto Rico and the Virgin Islands--thrust faulting on nearly horizontal planes with the slip vector nearly parallel to the plate boundary--is similar to that in the western most Aleutians and in the Andaman Islands.

4. WHAT IS THE BENEFIT?

4.1 Potential Applications of Seismic Microzonation Products

Applications of seismic microzonation products (maps, data, analyses) can be made in terms of land-use, building codes, construction practices, repair and strengthening of existing buildings, and response and recovery planning. The benefits in any given country where seismic microzonation has been performed include:

a. Improving the current building code, identifying options for modifications that incorporate the scientific and engineering lessons learned from past destructive earthquakes.

b. Improving of regional and urban land-use practices, identifying options for alternatives to current practices that might reduce potential losses.

c. Improving design and construction practices for new buildings, specifying options for alternatives to current practices that might be more effective in ensuring high quality.

d. Improving the current practices to repair and strengthen existing buildings, suggesting options for alternatives to current practices that might be more effective.

e. Evaluation of plans for emergency response and disaster recovery.

4.2 Implementation

Three groups of professionals will implement the new knowledge provided by seismic microzonation: the design professional, the urban and regional land use planner, and the emergency manager.

5. REFERENCES

1. University of Washington, Proceedings of the First International Conference on Microzonation for Safer Construction-Research and Application, Seattle, Washington, October 30 - November 3, 1972, 2 volumes.

2. University of Washington, Proceedings of the Second International Conference on Microzonation for Safer Construction-Research and Application, San Francisco, California, November 26 - December 1, 1978, 3 volumes.

3. University of Washington, Proceedings of the Third International Earthquake Microzonation Conference, Seattle, Washington, June 28 - July 1, 1982, 3 volumes.

4. Ministry of Urban Development and Construction, Proceedings of the International Conference on Seismic Microzonation, Ech Cheliff, Algeria, October 11-12, 1984, 2 volumes.

5. Thiel, Charles C., 1988, Seismic Microzonation: An Approach to Seismic Land-Use Planning in Hays, W. W., (Editor), Proceedings of Conference 41 on "A Review of Earthquake Research Applications in the National Earthquake Hazards Reduction Program: 1977-1987, U.S. Geological Survey Open-File Report 88-13-A, pp. 320-342.

6. Woodward Clyde Consultants, 1984, Seismic Microzonation of Ech Cheliff Region, Algeria, Report prepared for Controle Technique de la Construction, Algiers, Algeria; 6 volumes.

7. Hays, W. W., 1985, The Realization of a Seismic Microzonation Study in the Chlef Region of Algeria Following the 1980 El Asnam Earthquake, in Kusler, Jon A. (Editor), Proceedings of International Symposium on Housing and Urban Development After Natural Disasters, pp. 51-57.

8. Panel on Seismic Macro- and Microzonation, 1981, in State-of-the-art in Earthquake Engineering: Proceedings of the Seventh World Conference on Earthquake Engineering, Special Volume, pp. 47-68.

Full Scale Test on a Three-Storied Wood-Framed Building

by

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Abstract

A full-scale wood-framed building of three stories was subjected to the lateral loads, and the structural performance against the seismic and wind loads were analyzed. The experiments were carried out on the specimens whose floor and walls perpendicular to the loading direction were not sheathed with sheet materials as well as on the complete structure and the influence of the shear stiffness of a diaphragm and that of a bearing wall perpendicular to the loading direction on the structural performance was investigated. The experimental results were compared with the calculation from the finite element model, frame model and also Tuomi's model based on the load-slip relations of a nail joint, and the applicability of these calculation methods to the design procedure was investigated.

KEY WORDS:Static loading, Finite element analysis, Horizontal deformation, Ultimate load, Shear wall, Diaphragms.

Introduction

In relation to the revision of Japanese Building Standard Law [1] in 1987, the full scale tests on a Wood-framed House of three stories were carried out to investigate the structural safety against the seismic and wind loads.

Building Standard Law, Enforcement Order requires that the structural safety of wooden houses having more than two stories and/or exceeding 500m² in total floor area should be confirmed by means of structural calculation.

A purpose of this study is to analyze the performance of the shear walls and diaphragms in an actual structure, and to present the useful data for the structural design of the three storied Wood-framed Houses.

Description of specimens

The specimen was a full-scale Wood-framed Construction of three stories whose plans and elevations are shown in Figs.1 and 2. Each story had 66 square meters floor area of 9.1 meters long in the longitudinal direction (loading direction) and 7.28 meters wide in the transversal direction. Each story had three continuous walls in the longitudinal direction at the both sides and in the center of the plan. Exterior walls were sheathed with 9.5 millimeters thick Douglus-fir plywood outside and 12 millimeters thick gypsumboard inside. Interior walls were sheathed with 12 millimeters thick gypsumboard on both sides. Nail spacings were 10 centimeters around the perimeters of a sheathing sheet and 20 centimeters along the intermediate support. Subfloor was 12 millimeters thick lauan plywood and roof sheathing was 9.5 millimeters thick Douglus-fir plywood. No elastmeric adhesives were applied to the joints between subfloor and floor joists except for the joint just under the wall.

Specimens were divided into following three types(See Fig.3).

Specimen A: Specimen whose floor framings were not sheathed with subfloor and walls in the transversal direction

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were sheathed with no sheathing materials.

Specimen B: Specimen whose floor framings were sheathed with subfloor but whose wall framings in the transversal direction were not sheathed with sheathing materials.

Specimen C: Specimen whose floor framings were sheathed with subfloor and wall framings were sheathed with sheathing materials.

Test methods

The lateral static load was applied at the top of the specimen with three hydraulic jacks as shown in Fig.5. Horizontal and vertical displacements of each story were measured with the electric transducers and tension force of a hold-down bolt connecting each story was measured as shown in Fig.6.

In the Specimen A, the monotonously increasing loads were applied horizontally at the top of three continuous walls so that each displacement of the loading point was equal. In the Specimens B and C, the reversed cyclic loads as shown in Fig.7 were applied so that the horizontal load applied to the central wall was one and a half times as much as those applied to the south and north walls.

Forced vibration tests were also carried out before and after the static loading test of the Specimens B and C by employing a rotating-mass generator situated on the ceiling of the third story, and the response of each story was measured with the electromagnetic pickups.

Experimental results

Horizontal resistance

The relation between the horizontal loads and horizontal displacements at the top of the south, central and north walls of the Specimen A is shown in Fig.8, and the relations between the total applied loads and average displacement of the south, central and north walls of the Specimens B and C are presented in Figs.9 and 10 respectively. Table 1 summarizes the lateral load for the prescribed displacement ratio and the maximum load. The prescribed displacement ratio is defined by the ratio of horizontal displacement to the height of loading point in the Specimen A and the ratio of average displacement of three walls to the height of the loading point in the Specimens B and C.

The lateral load for the prescribed displacement ratio and the maximum load of the south and central walls in the Specimen A was respectively 36 to 48% and 55 to 69% smaller than those of the north wall. Horizontal displacement ratio for the maximum load of the south and north walls of the Specimen A was both 0.032 and that of the central wall was approximately 20% smaller than those of the south and north walls. The total lateral loads of three walls in the Specimens B and C were respectively 20 to 23% and 10 to 15% smaller than those of the Specimen A. This may be caused by the difference of the shear stiffness and strength between the south and north walls. The lateral load of the Specimen C was approximately 10% greater than that of the Specimen B. It is considered that the load applied to the south wall was not transferred to the north wall in the Specimen B because of the torsional deformation of the specimen, while the torsional moment was supported by the walls in the transversal direction in the Specimen C. Horizontal displacement for the maximum load in the Specimen B (average of three walls) was approximately two-thirds of that of the Specimen C. This summarizes that the Specimen B had less deformability than the Specimen C because of the lack of the shear stiffness of the walls in the transversal direction.

Horizontal displacements of each story are presented in Figs.11 to 13. They showed that the horizontal deformations were the most remarkable on the second story in all the specimens.

Description of failure

In all the specimens the failure concentrated on the second story especially on the south and central walls. Fig.14 shows the failure of the south wall in the Specimen C. Exterior plywood buckled at the compression side and were torn at the tension side around an opening, and the nail joints connecting plywood and studs failed along the vertical fasteners of plywood. Similar failures were observed on the interior walls sheathed with the gypsumboard.

Effects of floor stiffness and perpendicular walls

The horizontal deformations of the floor in the Specimens B and C are presented in Figs.15 and 16 respectively. These figures show that the floor of each story in the Specimen B turned remarkably according to the increase of the horizontal deformation, while the torsional deformation of the floor in the Specimen C was very small in comparison with the Specimen B. This proves that the torsional moment caused by the difference of the shear stiffness of the walls in the longitudinal direction were supported by the walls in the transversal direction and it reduced the torsional deformation of the floor. Figs.15 and 16 show also the shear deformations of the floor of the Specimens B and C were very small.

Distribution of shear forces

Figs.17 and 18 show the ratio of the lateral forces supported by the continuous walls of each story in the Specimens B and C. The distribution of forces to the walls in the Specimens B and C was obtained from the relation between the shear deformation and the lateral load in each wall of the Specimen A. The parenthesized values in the figures are the ratio of the amount of the bearing walls. The amount of the bearing walls is defined by the products of the wall coefficient and the sum of the length of the bearing walls without openings. The ratio of shear forces of each wall was almost constant regardless of the horizontal deformation, and close to the ratio of the amount of the bearing walls. This proves that the lateral load can be distributed according to the amount of the bearing walls in case of designing the lateral stiffness and strength of the structure.

Uplift of bearing walls

Fig.19 shows vertical displacement of the bearing walls on the first story of the south wall, and Figs.20 to 22 show drawing forces of the bearing walls at the end of each story in the Specimen C. Drawing force of the bearing wall on the third story was very small, and it became larger on the lower story than the higher story. Drawing force of the north wall was the largest on the first story, and that of the center was the smallest and approximately a half of that of the north wall.

Dynamic characteristics

Forced vibration tests using a rotating-mass vibration generator were carried out in order to evaluate the dynamic characteristics of the Specimens B and C.

Figs.23 and 24 show typical frequency response curves for EW-direction (the longitudinal direction) in forced vibration tests, and Table 2 summarizes the natural frequencies evaluated from the response curves.

Microtremor exited vibrations of the completed house with the live load were also measured after finishing the exterior and interior. The natural frequencies of the completed house evaluated from the fourier spectra of observed records are summarized in Table 2.

The natural frequencies of torsional mode of the Specimens B and C were compared to evaluate the effects of the walls in the transversal direction on the torsional stiffness. The results were summarized that the natural frequency was risen from 4.8 to 8.8Hz by sheathing the walls in the transversal direction. This suggests that the floor of the specimen was sufficiently rigid so that the perpendicular walls increased the torsional stiffness.

In order to investigate the effects of damages of the bearing walls on dynamic characteristics, the natural frequencies of translational mode before and after the static loading tests of the Specimen C were compared. The results were summarized that the damage decreased the natural frequency from 5.8 to 3.1Hz. The natural frequency of the completed house decreased from 5.8 to 4.8Hz comparing with the unfinished structure. It seems the natural frequency decreased because of the increase of weight of finish and live load.

Comparison of the calculated values with the experimental results

Horizontal displacements of each story in the Specimen C were compared with the calculated values computed from the finite element model, frame model and Tuomi's model.

In FEM model, walls were modeled with the triangular plates as shown in Fig.25. The modulus of elasticity and the shear modulus of elasticity of wall elements were assumed $36,400 kgf/cm^2$ and $1,285 kgf/cm^2$ for exterior walls and $30,000 kgf/cm^2$ and 750 kgf/cm^2 for interior walls respectively from the racking test of wall panels.

In Frame model, shear walls, lintel walls and end joists were modeled with the beam element as shown in Fig.26. The modulus of elasticity of a column element of 12-by 10cm in section corresponding to the shear wall of ninety-one centimeters in length was assumed $1,254,300 kgf/cm^2$ for exterior walls and $812,800 kgf/cm^2$ for interior walls from the racking test of wall panels. The equivalent section of lintel walls was assumed in consideration of the composite action between lumber and plywood. Rigid zone was also assumed at the joint of beams and columns.

Tuomi's model[2] based on the slip of nail joint between lumber and plywood sheet in shear wall(See Fig.27) was applied to predict the shear deformation and ultimate load[3][4]. The slip modulus and strength of a nail joint was assumed 691kgf/cm and 100kgf for plywood-lumber joints and 238kgf/cm and 38kgf for gypsumboard-lumber joints from the racking test of wall panels.

Figs.28 to 30 show the comparison of the experimental results in the Specimen C with the computed results from each model. For the computation of the shear resistance of each story in the Specimen C, the shear resistance of each wall was simply summed up neglecting the torsional deformation. The calculated results from each model had a tendency to underestimate the horizontal stiffness when the horizontal displacement was very small (when γ is approximately less than 0.03) but it seems that the calculation from these models are applicable to the practical use.

Table 3 shows the ultimate load of each story computed from Tuomi's model. The computed value of the second story was the smallest which coincides with the fact that the second story was most damaged in the experiment, and the computed value of the second story showed relatively good agreement with the maximum load in the experience. This proves that Tuomi's model is applicable to predict the ultimate properties of Wood-framed building.

Conclusions

To summarize the results of this study, the following conclusions were lead.

1. The shear resistance of the north wall of the specimen was approximately one and a half times as large as that of the south wall. However the difference of shear resistance between the south and north walls affected very little on the torsional deformation of the structure in case it had sufficient lateral stiffness in the transversal direction.
2. The shear deformation of the diaphragm was small enough to assume that the diaphragm was rigid in case of calculating the lateral resistance of the construction.
3. The lateral forces distributed on the walls were approximately proportional to the amount of the bearing walls.
4. Drawing forces at the end of the wall were the largest on the first story and the smallest on the third story. This suggests that the drawing forces of the walls on the upper story should be considered to design those of the lower story.
5. FEM model, Frame model and Tuomi's model can be applied to the practical use to predict the lateral resistance of Wood-framed building, and Tuomi's model is proper to predict the ultimate properties of Wood-framed building.

References

1. Building Standard Law, Notification No.56, Ministry of Construction, 1982.
2. TUOMI,R.L. and McCUTCHON,W.J. "Predicting Racking Strength of Light-frame Walls", Preprint for ASCE Meeting, October 1977.
3. YASUMURA,M. "Racking Resistance of Wooden Frame Walls with Various Openings", CIB W18/19-15-3, September 1986.
4. YASUMURA,M.,MUROTA,T.et al. "Experiments on A Three-Storied Wooden Frame Building", Proceedings of the 1988 International Conference on Timber Engineering, Vol.2, September 1988.

Table 1. HORIZONTAL RESISTANCE, MAXIMUM LOAD AND DISPLACEMENT RATIO FOR MAXIMUM LOAD

(Unit:kgf)

DISPLACEMENT*		1/600	1/300	1/200	1/150	1/120	1/60	Pmax	Rmax**
SPEC-IMEN A	SOUTH:	2,050	3,190	3,890	4,500	5,090	6,450	8,700	0.032
	CENTER:	1,750	2,350	3,240	3,450	3,550	3,680	4,160	0.025
	NORTH:	3,870	5,610	6,800	7,660	8,550	11,310	13,590	0.032
SPECIMEN B		6,240	9,080	10,650	12,690	13,980	16,660	16,700	0.016
SPECIMEN C		6,970	9,900	11,520	13,570	14,890	17,970	18,670	0.026

* Ratio of horizontal displacement to the height at the loading point
** Displacement ratio for the maximum load

Table 2. NATURAL FREQUENCY OF SPECIMENS

SPECIMEN		FIRST MODE	SECOND MODE
		Hz	Hz
SPECIMEN B	BEFORE LOADING	4.8 (TORSION)	6.5
SPECIMEN C	BEFORE LOADING	5.8	8.8 (TORSION)
	AFTER LOADING	3.1	7.0 (TORSION)
FINISHED HOUSE		4.8	7.2 (TORSION)

Table 3. Ultimate Strength of each wall computed from the Tuomi's model(Unit:kgf)

Wall	South	Center	North	Total
1F	6,110	5,104	10,837	22,051
2F	6,902	4,246	9,807	20,955 (18,670)*
3F	8,170	4,248	10,577	22,995

* The parenthesized values represent the ultimate load of the experiment in the Specimen C.

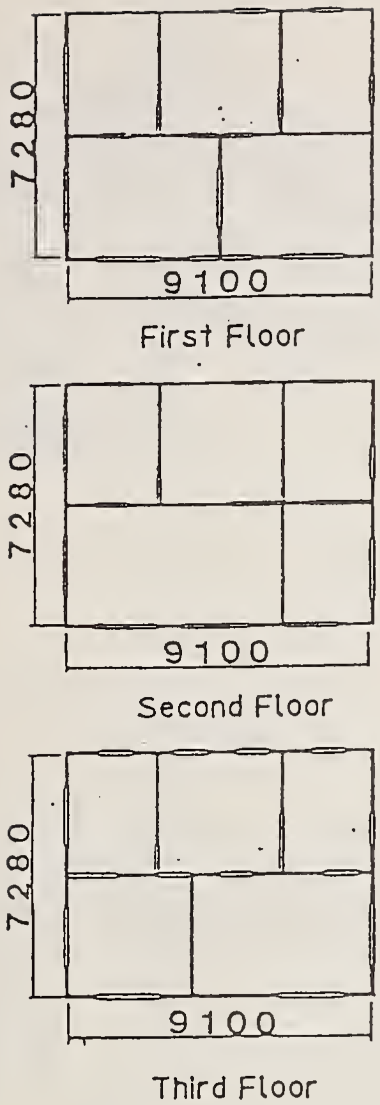


Fig.1 Plans of a specimen.

Fig.2 Elevations of a specimen.

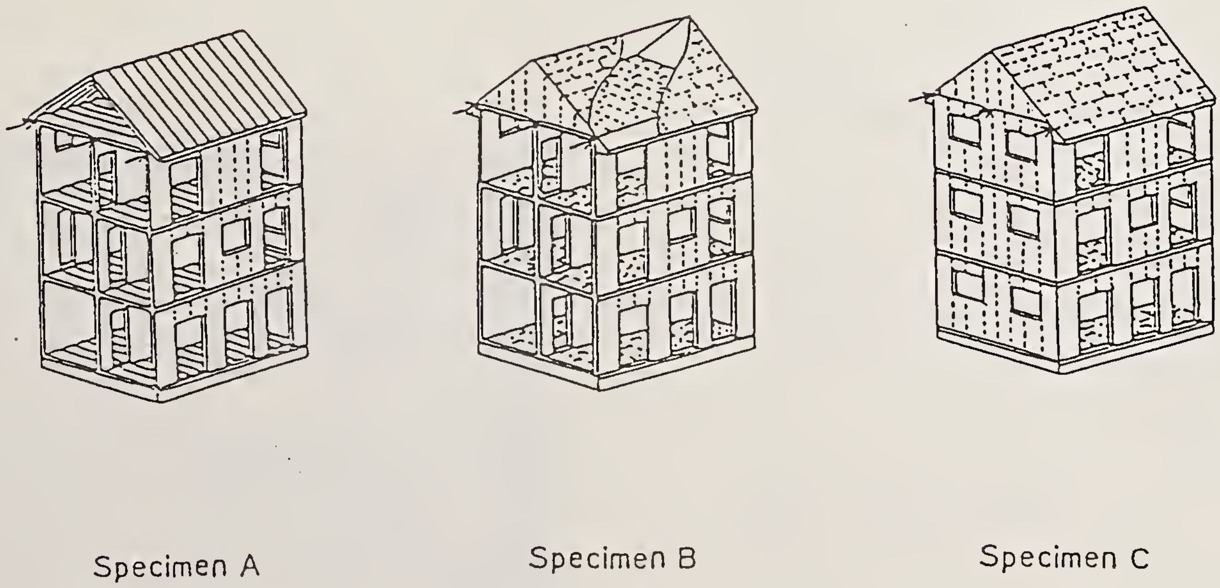


Fig.3 Kinds of specimens.

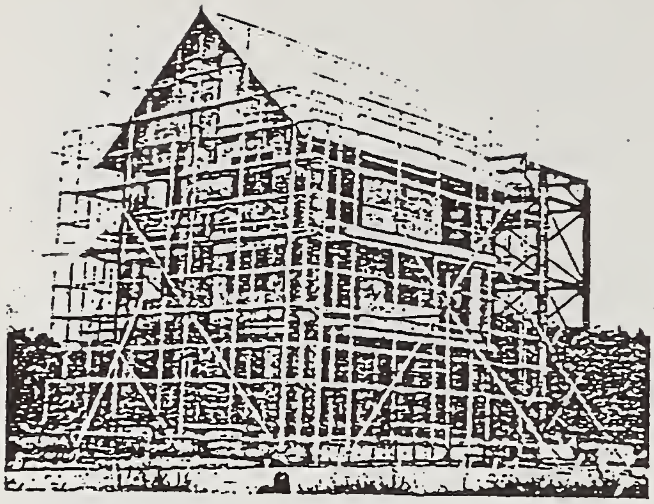


Fig.4 Exterior view of a specimen.

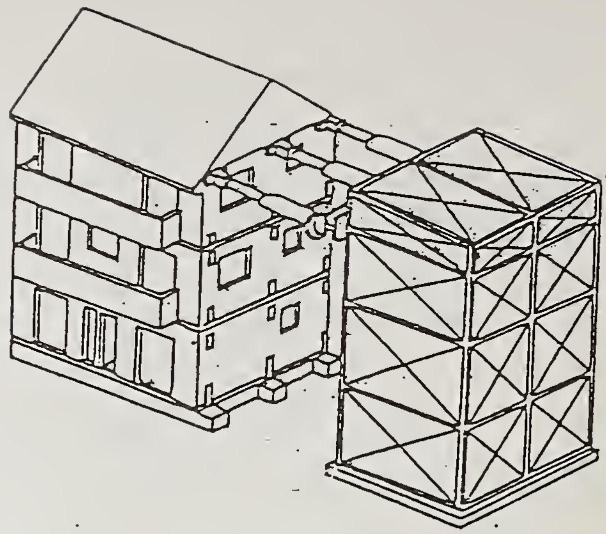


Fig.5 Test set-up.

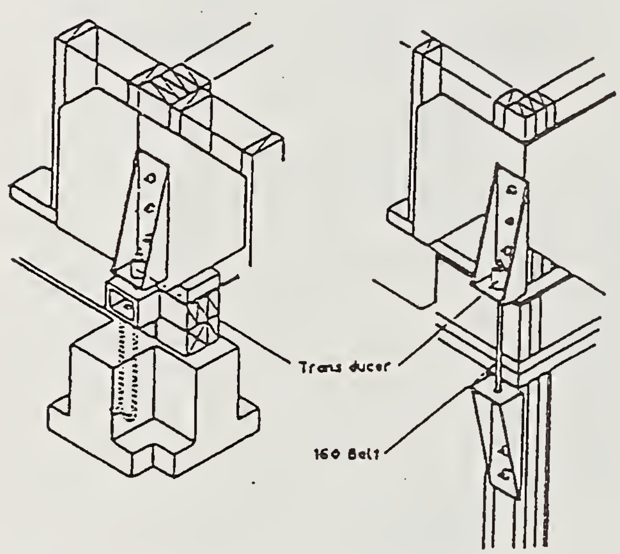


Fig.6 Measurement of a tension force.

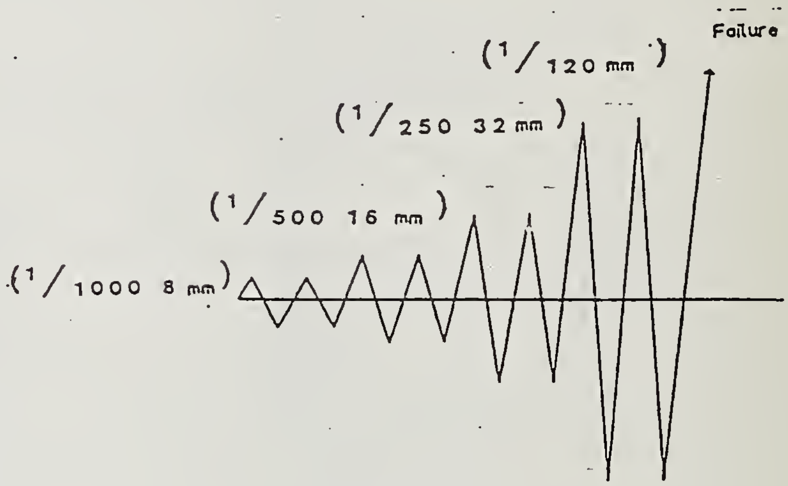


Fig.7 History of loading.

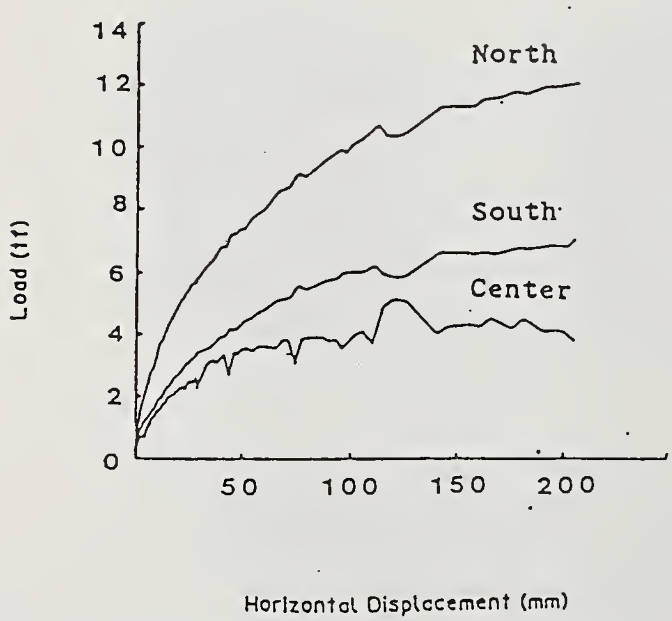


Fig.8 Load-displacement curves in the Specimen A.

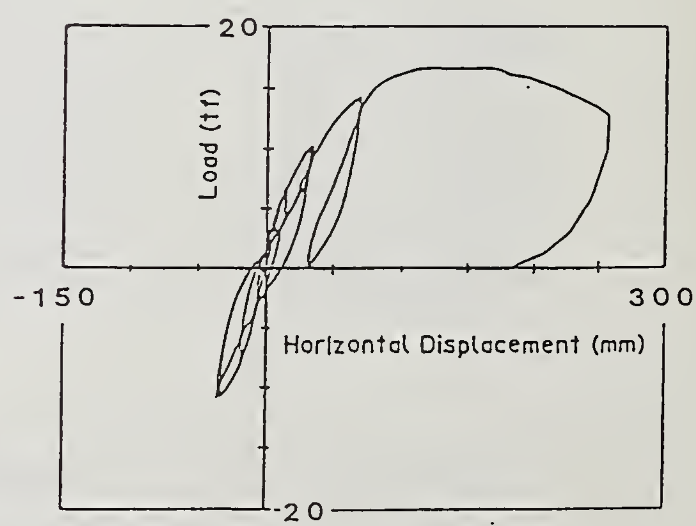


Fig.9 Load-displacement curves in the Specimen B.

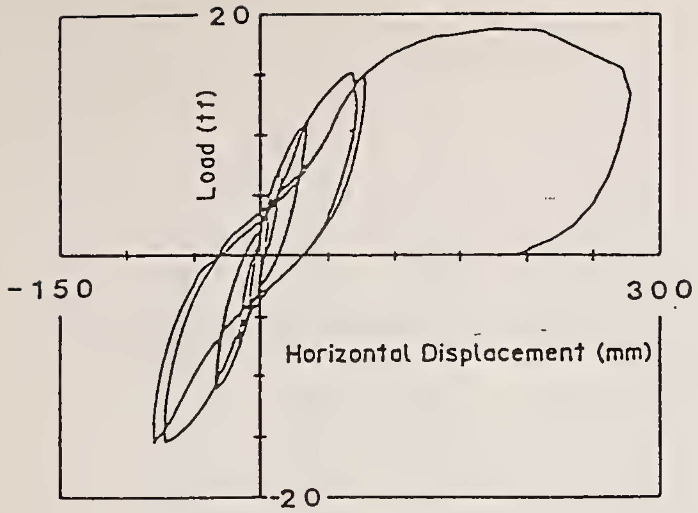


Fig.10 Load-displacement curves in the Specimen C.

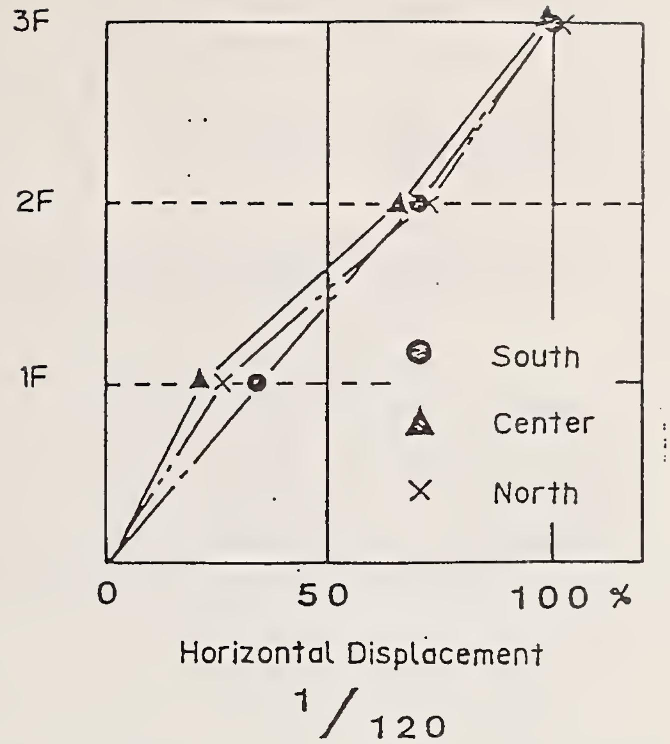


Fig.11 Horizontal deformation of each story in the Specimen A.

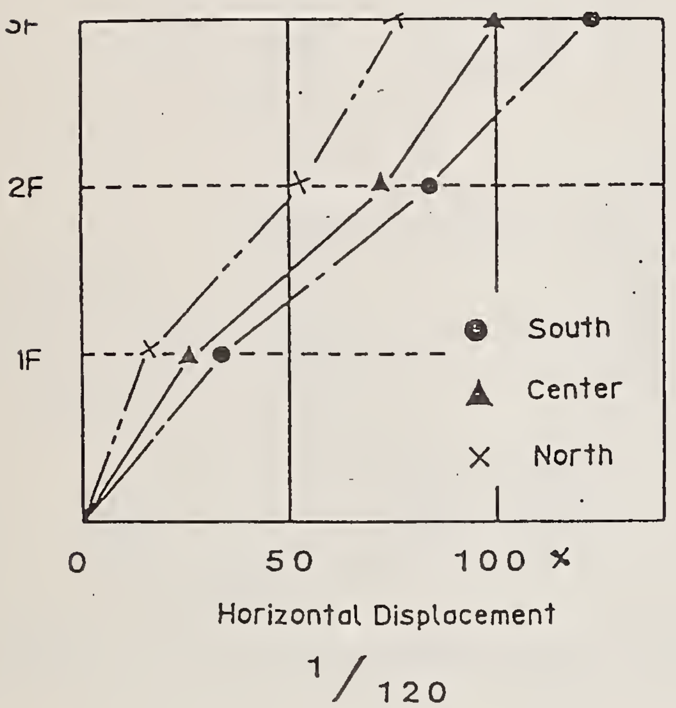


Fig.12 Horizontal deformation of each story in the Specimen B.

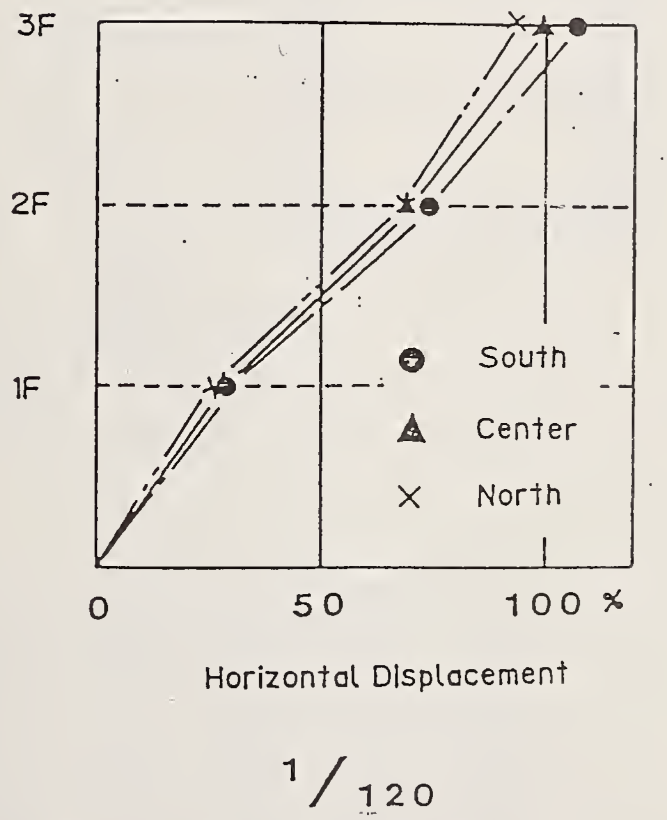


Fig.13 Horizontal deformation of each story in the Specimen C.

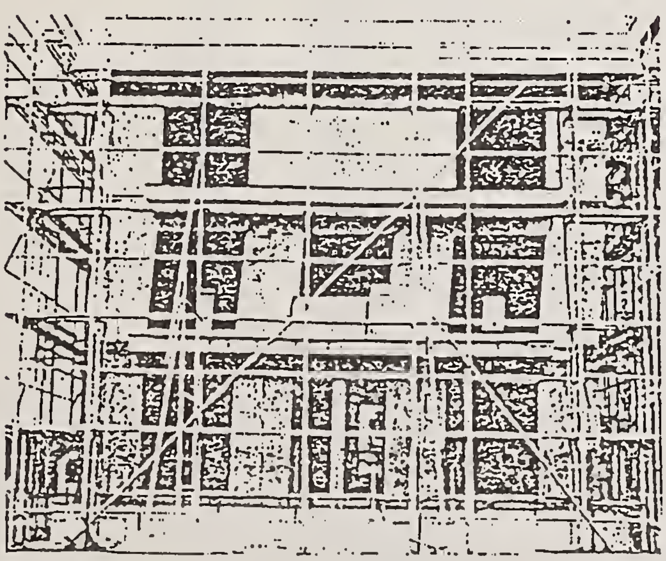


Fig.14 Failure of the south wall in the Specimen C.

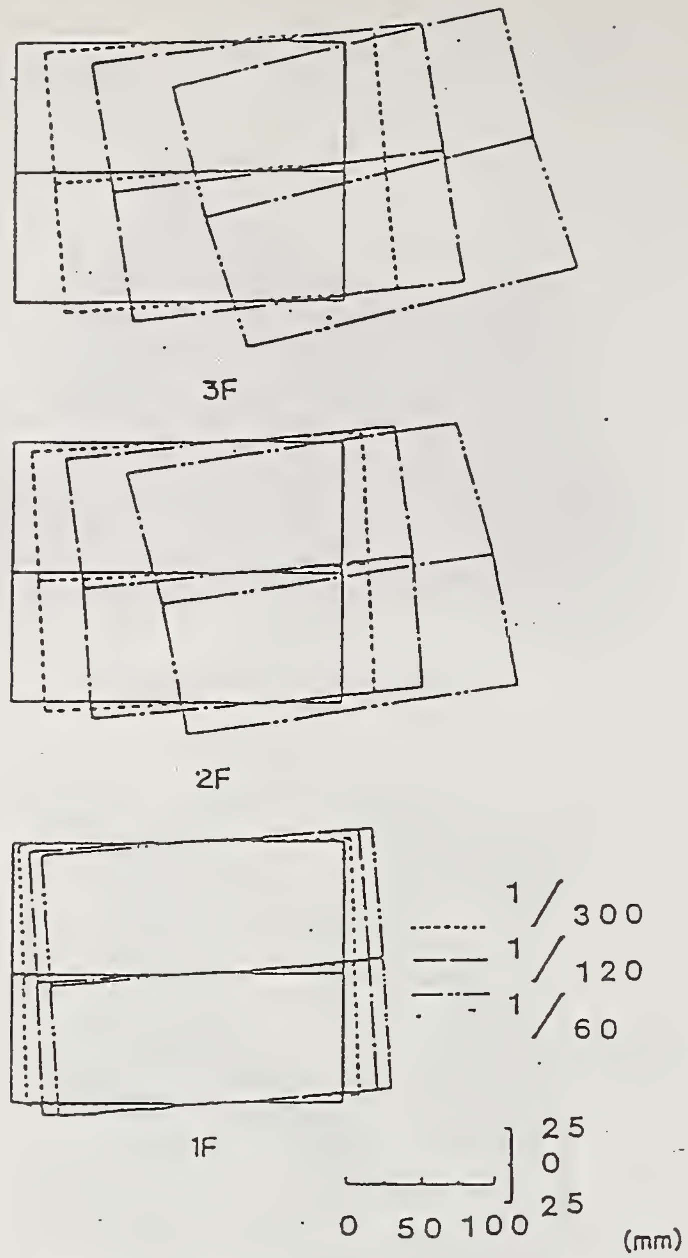


Fig.15 Horizontal deformation of the floor in the Specimen B.

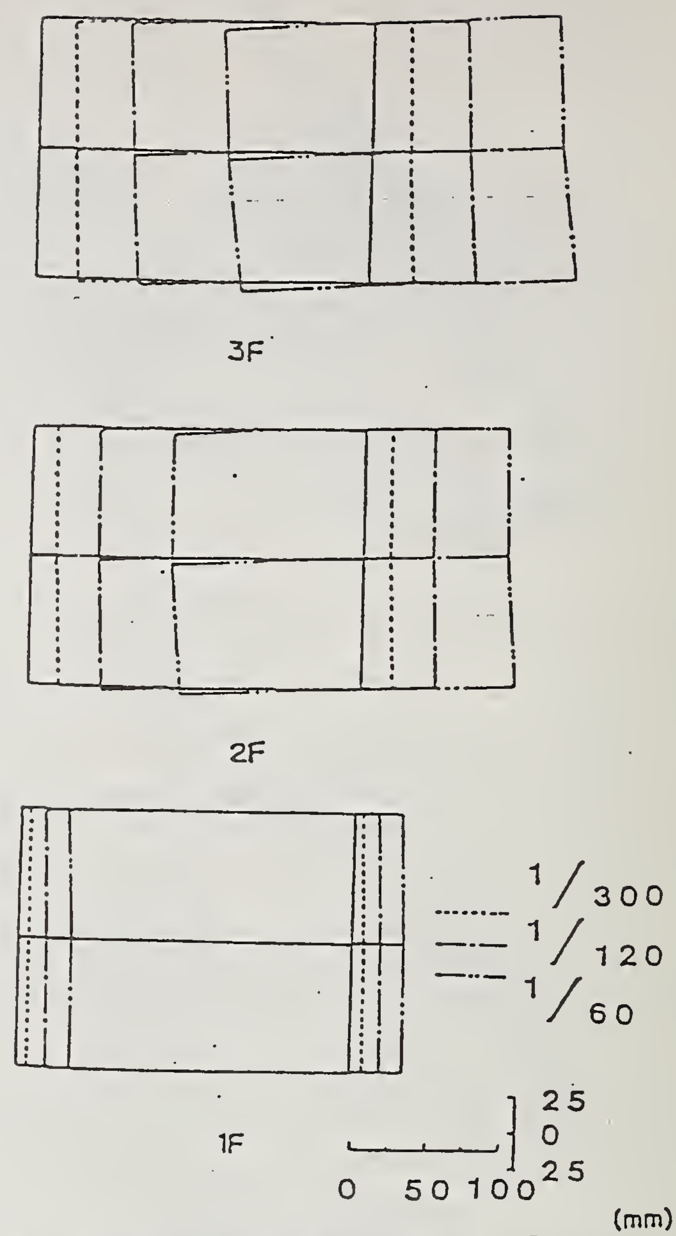


Fig.16 Horizontal deformation of the floor in the Specimen C.

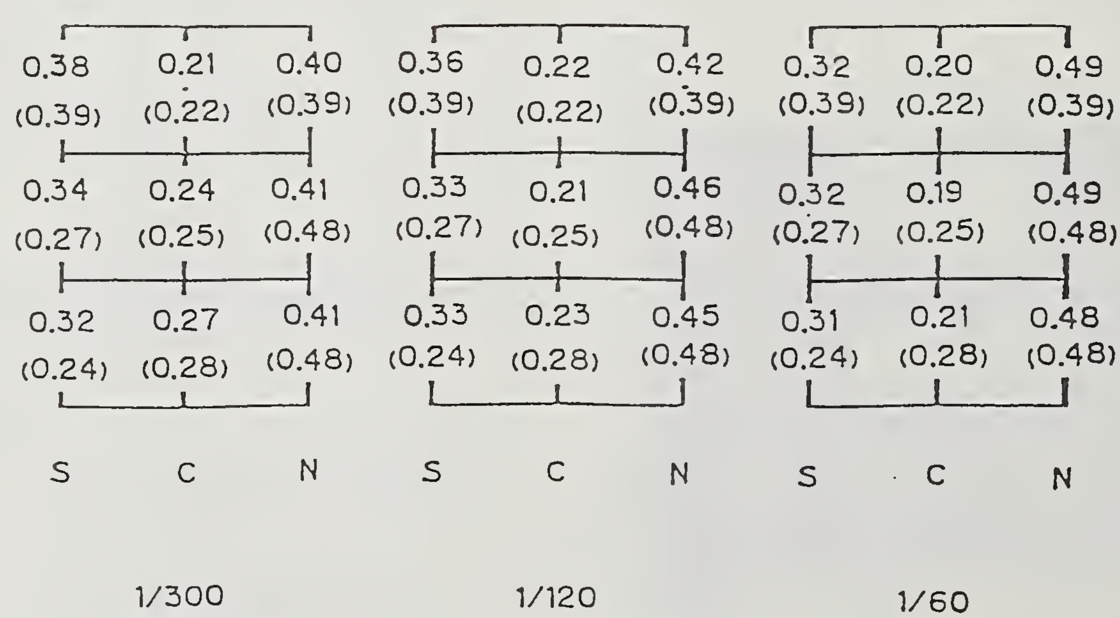


Fig.17 Distribution of the horizontal load in the Specimen B.

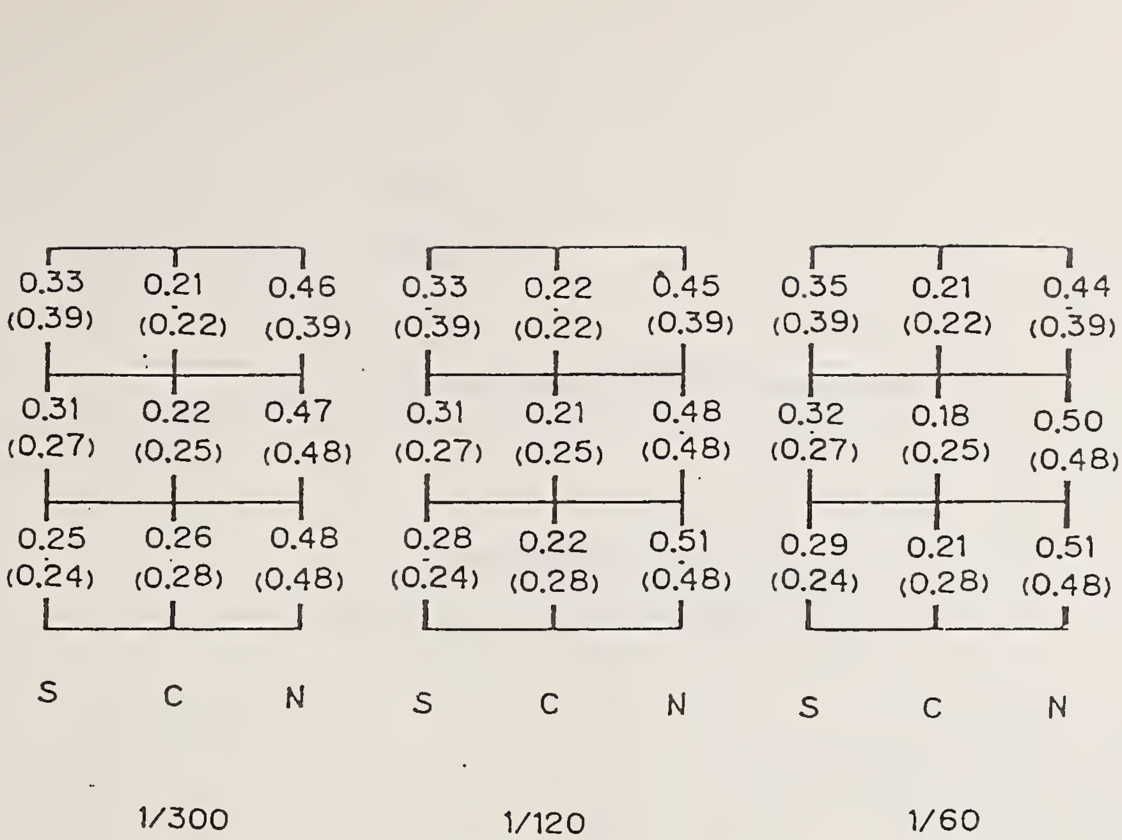


Fig.18 Distribution of the horizontal load in the Specimen C.

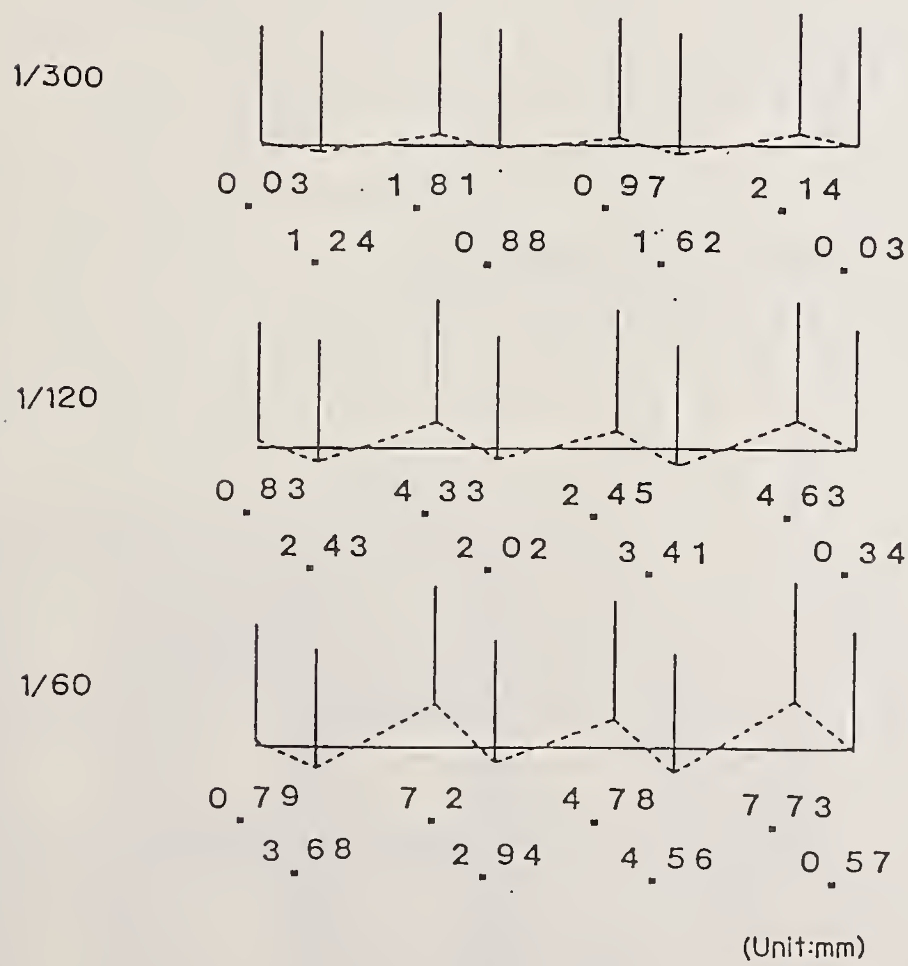


Fig.19 Vertical displacements of the south wall on the first story.

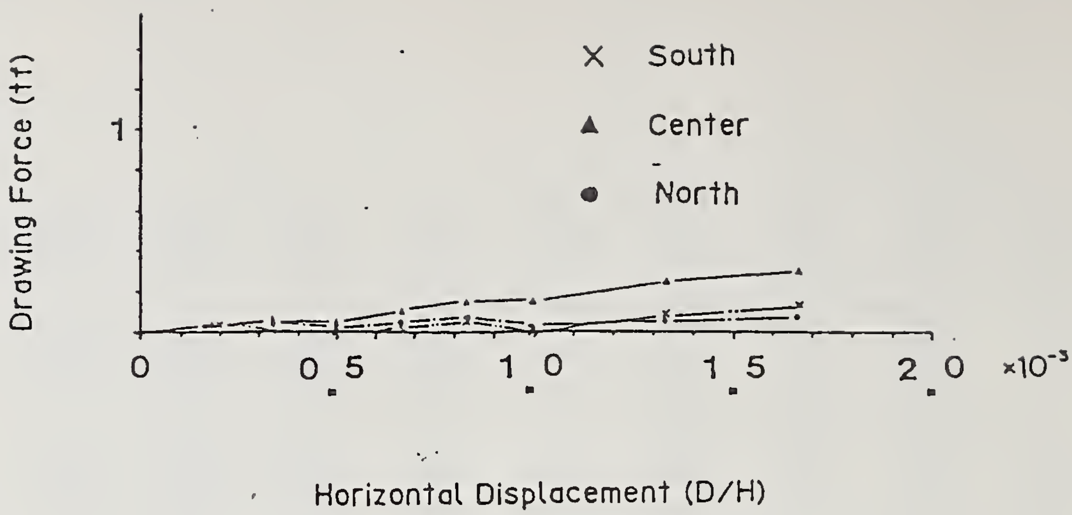


Fig.20 Drawing force of the walls on the third story.

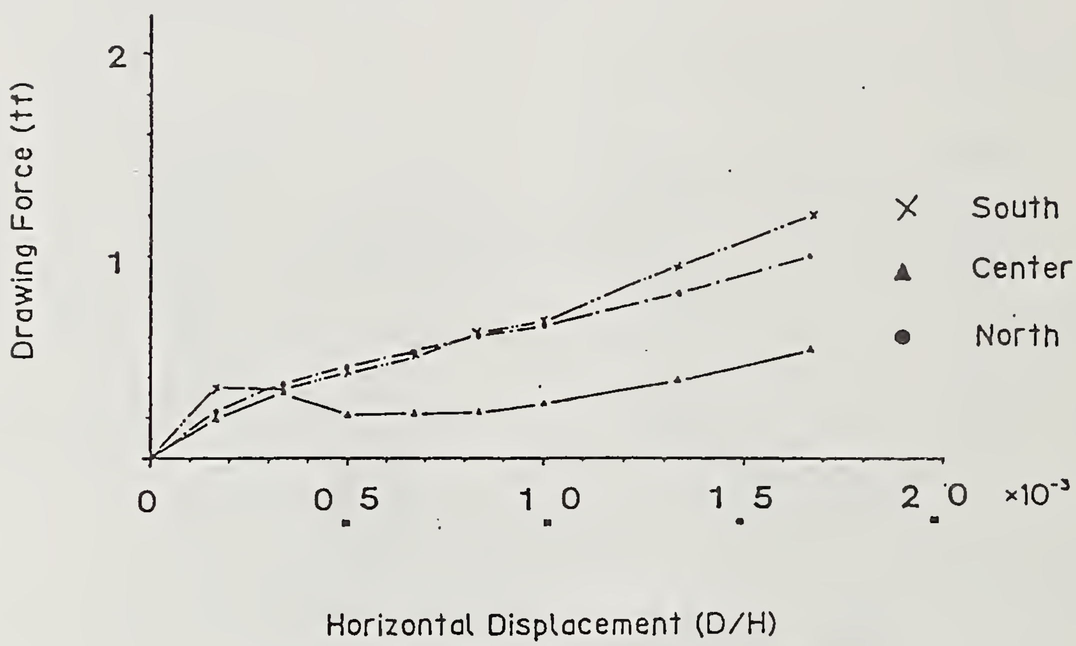


Fig.21 Drawing force of the walls on the second story.

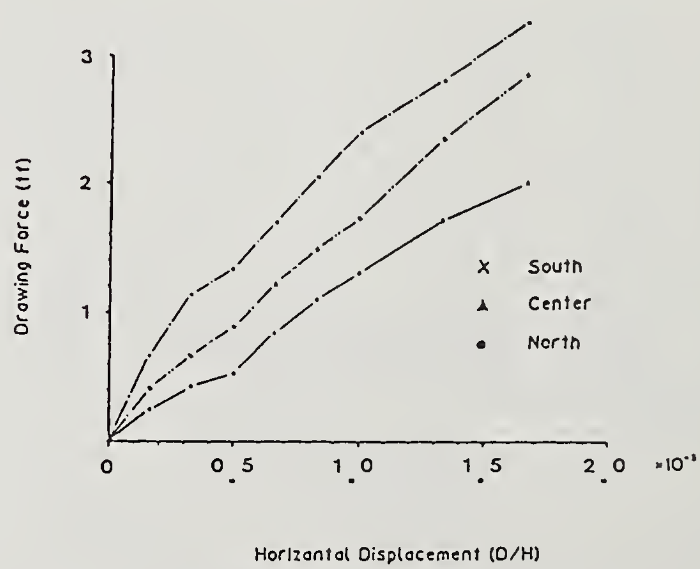


Fig.22 Drawing force of the walls on the first story.

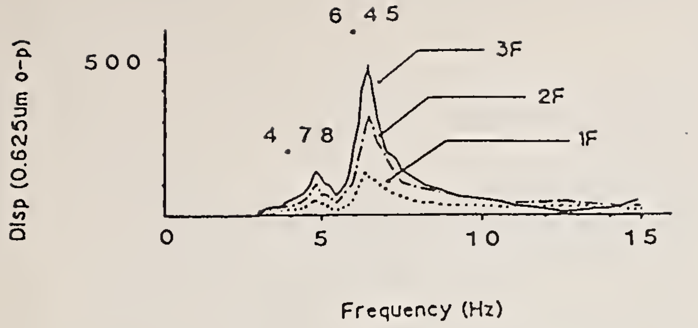


Fig.23 Frequency response curves for EW-direction in the Specimen B.

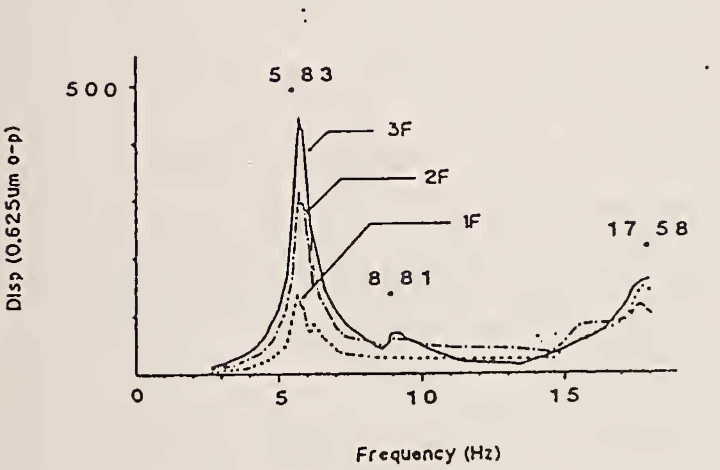


Fig.24 Frequency response curves for EW-direction in the specimen C.

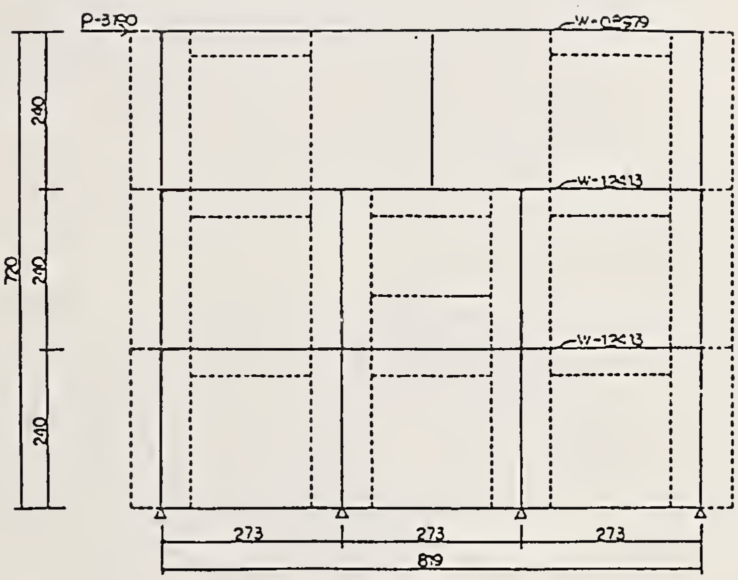


Fig.26 Frame model.

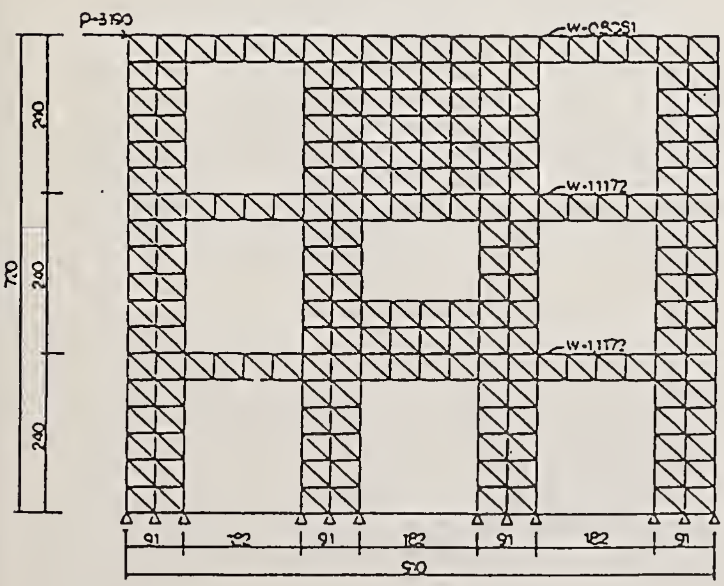


Fig.25 Finite element model.

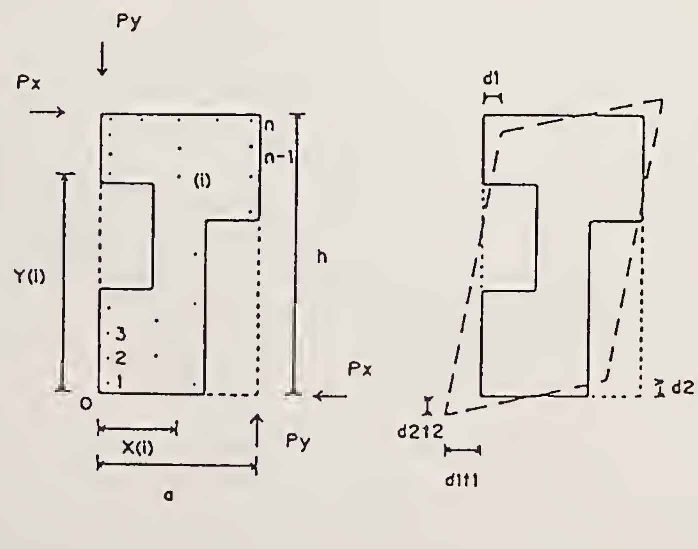


Fig.27 Racking model of a shear element in Tuomi's model.

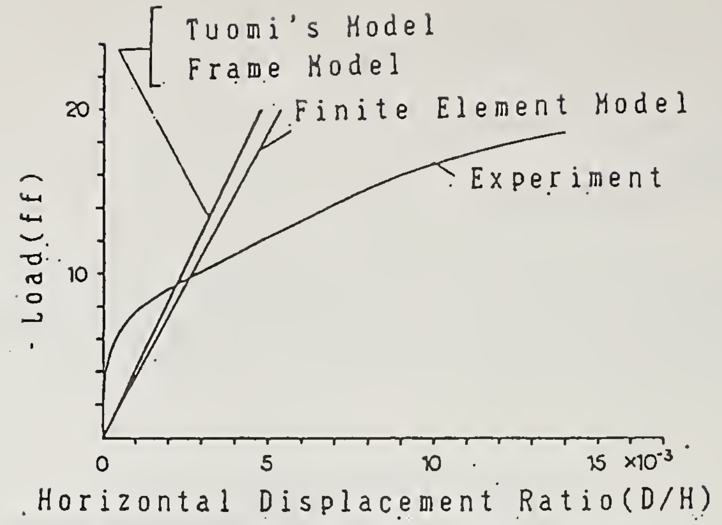


Fig.28 Comparison of the experimental results with the calculated results on the first story of the Specimen C.

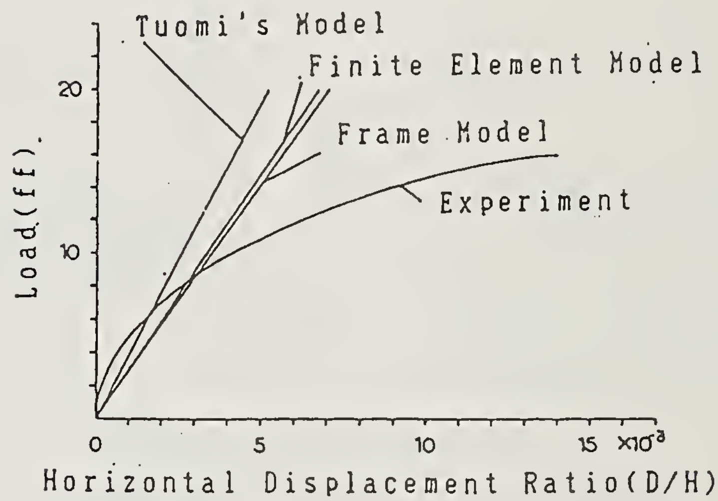


Fig.29 Comparison of the experimental results with the calculated results on the second story of the Specimen C.

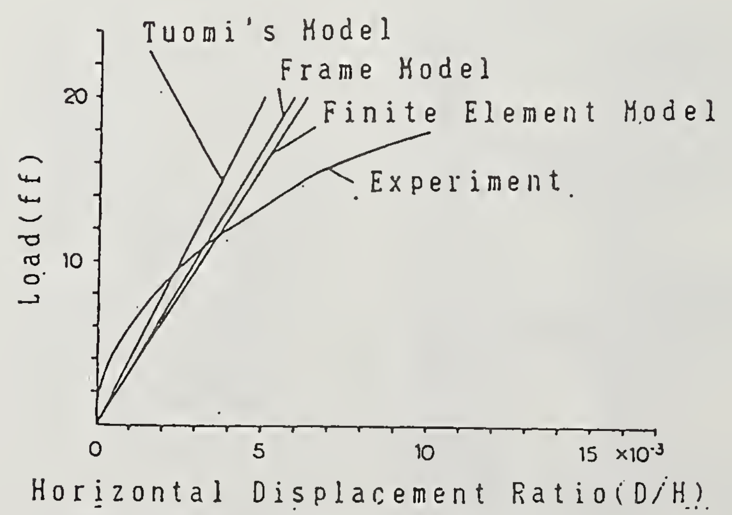


Fig.30 Comparison of the experimental results with the calculated results on the third story of the Specimen C.

Damage by the Chiba-ken Toho-Oki Earthquake

by
Yasushi SASAKI*

ABSTRACT

An earthquake of magnitude 6.7 occurred off the eastern shore of Chiba Prefecture at approximately 11:8 AM on December 17, 1987.

This earthquake took two human lives, when they were crushed to death under fallen concrete block walls and a stone lantern, and caused damage to various civil engineering structures and buildings.

Reports were already published on this earthquake, and studies are still being continued. In this report, the author summarizes the damage of this earthquake based on the two papers[1),2)] published previously.

KEY WORDS: Earthquake, Disaster, Chiba-ken Toho-Oki Earthquake.

1. OUTLINE OF EARTHQUAKE

According to the Meteorological Agency[3)], the epicenter of the earthquake has located at 35°21' north latitude and 140°29' east longitude with the hypocenter depth of 58 km, and the magnitude of the earthquake was 6.7.

Seismic intensity of V on the JMA scale was recorded at Katsuura, Chiba, and Choshi, IV at Tateyama, Tokyo, Yokohama, Mito, Ajiro, Kumagaya and Kawaguchiko, and III at Ohshima, Mishima, Utsunomiya, Chichibu, Niijima, Maebashi, Nikko, Koufu, Onahama, Shizuoka, Shirakawa, Karuizawa, Iida and Hachijoujima.

Fig. 1 shows the seismic intensity at various places. It also shows approximate boundaries between the seismic intensities V and IV, between IV and III and between III and II. The range where the seismic intensity of V was observed was the area at the distance of less than 50 km from the epicenter within Chiba Prefecture. It was reported that the intensity of earthquake motion varied even within the range of seismic intensity of V in Chiba Prefecture shown in Fig. 1 according to the results of detailed survey.

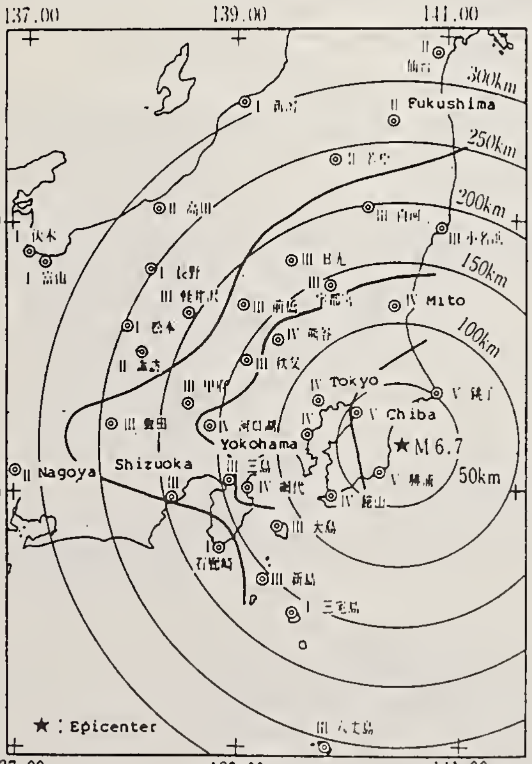


Fig. 1 Seismic intensity (JMA scale) at various places

JMA seismic intensity scale / Acceleration of earthquake (gal)	0 No feeling slight	I No feeling slight 0.8	II Weak 2.5	III Rather strong 8
Kawasumi's seismic intensity	1	2	3	4, 5

IV Strong 25	V Very strong 80	VI Disastrous 250	VII Very disastrous 400
6, 7	8	9, 10	11, 12

Fig. 2 Distribution of Kawasumi's Intensity near epicenter (after Harukawa[3)])

* Head, Ground Vibration Division, Public Works Research Institute, Ministry of Construction

Harukawa[3] conducted a questionnaire survey by asking questions of the residents in 47 cities, towns and villages in Chiba Prefecture and determined a seismic intensity distribution in Chiba Prefecture based on about 2,900 answers obtained. The results of Harukawa's survey are shown in Fig. 2. As "Kawasumi's seismic intensity" is used as indexes of seismic intensity in his survey, Kawasumi's intensity is compared to the JMA scale in Fig. 2.

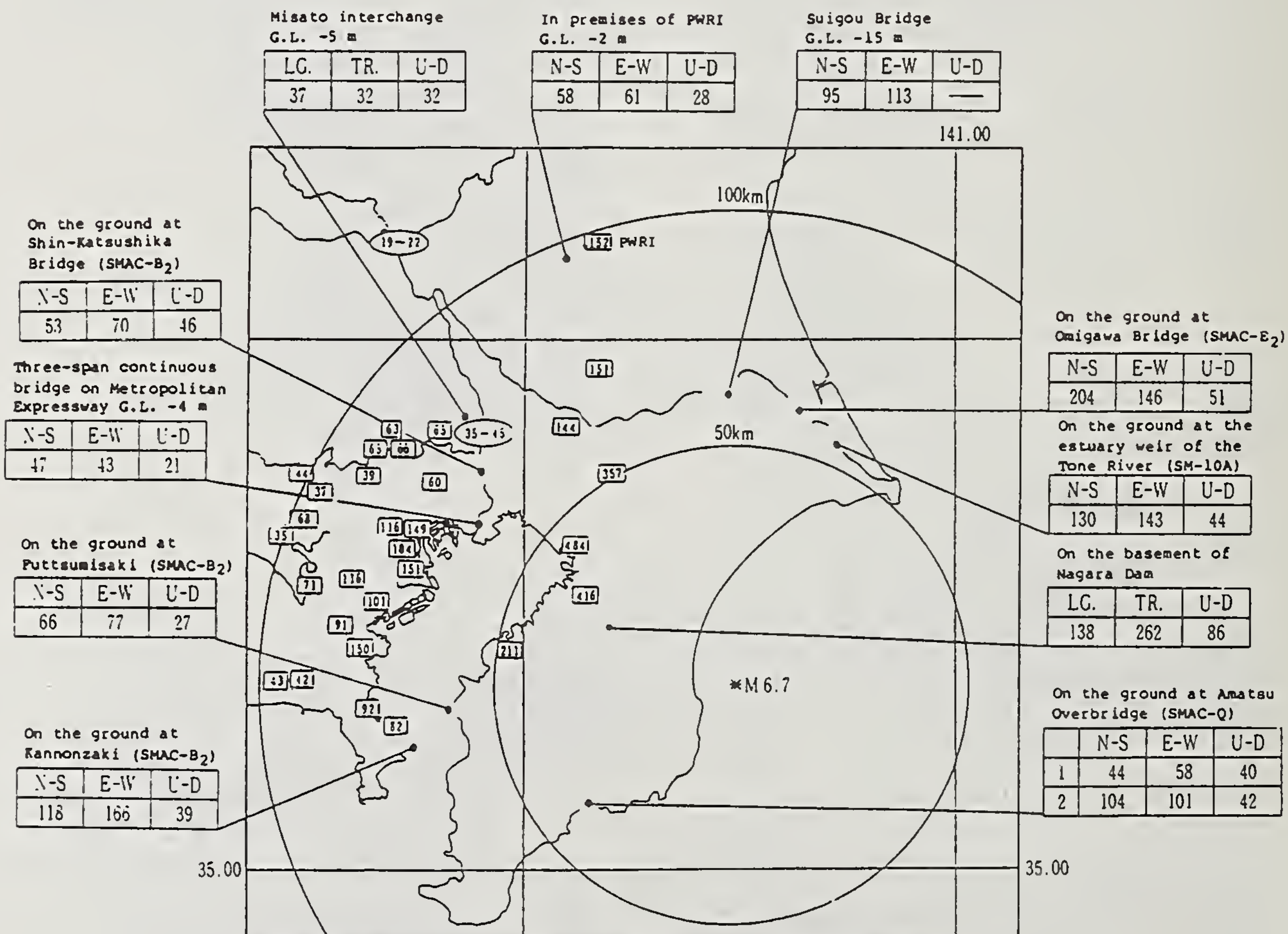
According to the results of the survey, it can be known that there were areas near Tougane City and Chounan Town where the earthquake motion was locally stronger.

From the study of earthquake mechanism, Mizone[5] reported that this earthquake was caused by the strike slip fault in the northeast-southwest direction on P-axis (principal axis of maximum stress) which was created inside the plate in the Philippines Sea. One of nodal planes (potential shear plane) is almost directed toward north-south which is the direction of distribution of aftershocks.

By this earthquake, many strong-motion seismographs for engineering purposes installed in Tokyo and Chiba Prefecture were trigered. Recorded maximum values were summarized in the prompt report of the earthquake.[6] Among these records, numerically processed and corrected for instruments by PWRI are shown in Fig. 3 which were obtained on ground and underground near ground surface. However, part of the records were not digitized since the maximum acceleration values were not large. Fig. 3 also shows the maximum acceleration[7] (maximum value of the resultant in two horizontal directions) observed by the Tokyo Gas Co.

Fig. 4 shows the relation between maximum acceleration and epicentral distance. Fig. 4 also shows the values for Class 1 ground of the following regression equation[8] which indicates the distance attenuation of maximum horizontal acceleration obtained from previous records of strong earthquakes:

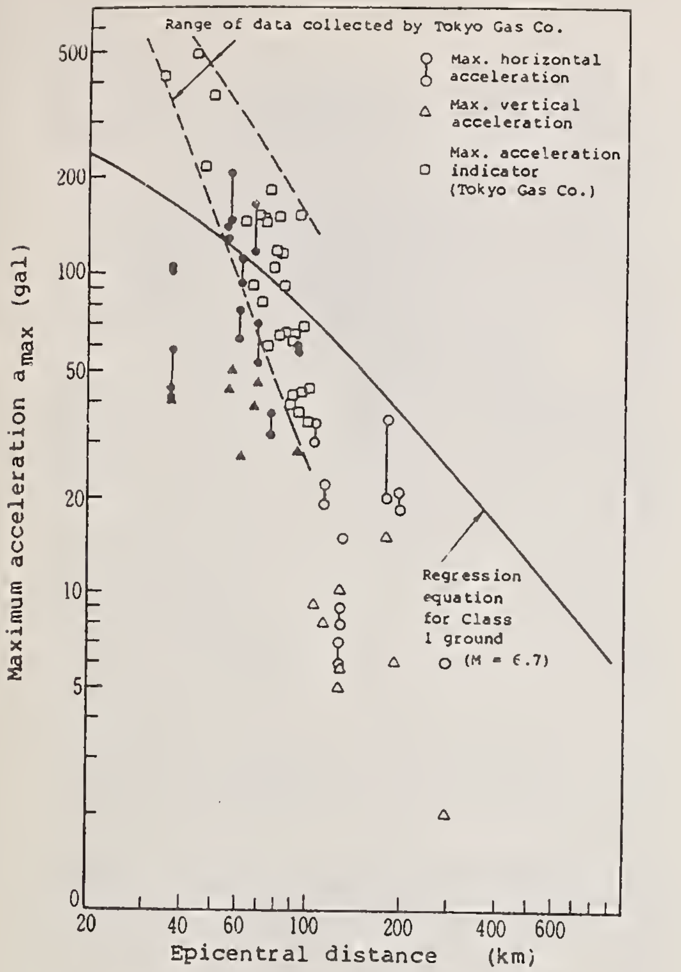
$$A\ max = \left\{ \begin{array}{ll} 987.4 \times 10^{0.216\ M} & \text{(Class 1 ground)} \\ 232.5 \times 10^{0.313\ M} & \text{(Class 2 ground)} \\ 403.8 \times 10^{0.265\ M} & \text{(Class 3 ground)} \end{array} \right. \times (\Delta + 30)^{-1.218} \quad \ldots\ldots\ldots\ldots\quad (1)$$



□ : Observed value (resultant in 2 directions) by Tokyo Gas Co.

○ : Observed values on the ground (respective values in two directions before numerical processing instrument correction) by strong-motion seismographs installed by the Ministry of Construction and others; unit in gal; value shown outside Fig. 3 is the maximum acceleration after numerical processing instrument corrections.

Fig. 3 Maximum acceleration at various places



* Values of max. horizontal acceleration and max. vertical acceleration after numerical processing instrument corrections are shown black. Readinga from analog records are shown in white.
* Data recorded by Tokyo Gas Co. are resultant max. acceleration for two horizontal directions.

Fig. 4 Epicentral distance and maximum acceleration

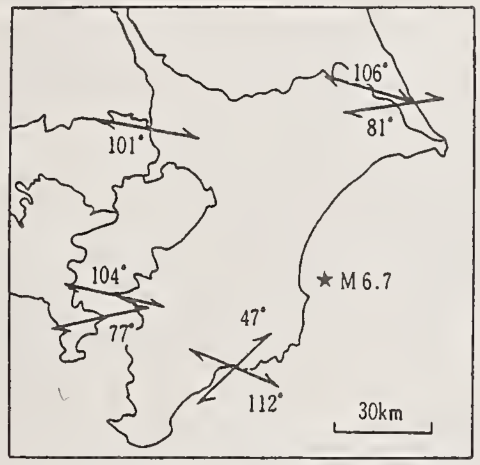
where, M: Magnitude, Δ : epicentral distance.

Tendency of the records obtained this time was almost similar to those of previous earthquakes, the size of acceleration was also similar to those of previous earthquakes, and the attenuation due to distance was greater compared to the regression equation based on many previous earthquake records.

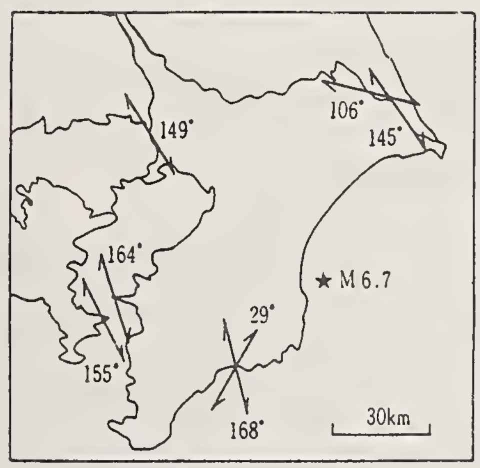
As understandable from Figs. 3 and 4, large maximum acceleration values were observed in part of the region. This occurred probably because of local variation in ground conditions.

If the acceleration in two horizontal orthogonal directions and the acceleration in a vertical direction are observed at a certain observation point and are vector-synthesized, then it is possible to obtain the locus of acceleration vector at that point. This kind of locus is called the orbit of acceleration record. The orbit has a complicated shape but is normally distorted in a certain direction and the acceleration in that direction is large. This direction is called the principal axis of the acceleration of earthquake motion or the dominant direction of vibration.[9]

The principal axis can be determined also relative to velocity or displacement. Principal axis of acceleration and the principal axis of displacement of the records obtained by the present earthquake are shown in Fig. 5. Earthquake motion is dominant in the direction of the principal axis, and some dominant directions of earthquake motion at observation points shown in Fig. 5 are orthogonal to the epicentral direction but others are not, thereby indicating the irregularity. This seems to occur because the earthquake motion is strongly affected by local topography or ground conditions near the observation points.



(a) Acceleration records



(b) Displacement records

Fig. 5 Principal axis of earthquake motion (projection of dominant principal axis to horizontal plane)

2. OUTLINE OF THE DAMAGE

The outline of the damage caused by the earthquake is shown in Tables 1 and 2.

This earthquake took two human lives, when they were crushed to death under fallen concrete wall blocks (Ichihara City, Chiba Prefecture) and a stone lantern. (Mobara

Table 1 General Aspect of damage (As of March 22, 1988, surveyed by the National Land Agency)

Class		Unit	Amount of damage
Persons	Dead persons	Persons	2
	Wounded persons	Persons	161
Houses	Totally collapsed	Houses	16
	Half collapsed	Houses	102
	Partially damaged	Houses	72,580

Table 2 Damage to facilities (as of March 22, 1988, partially as of February 29, surveyed by Chiba Prefecture)

Facilities		Number of places	Amount of damage (million yen)
Civil engineering facilities	Facilities related to Ministry of Construction	153	2,781
	Facilities related to Ministry of Agriculture, Forestry & Fishery (fishing ports)	13	197
	Facilities related to Ministry of Transport (ports & harbors)	10	5
	Subtotal	176	2,983
Other facilities, etc	Farmland, agricultural facilities	390	2,688
	Agricultural common use facilities	6	59
	Agricultural crops	-	4
	Non-common use facilities	-	3
	Devastation of forest land	100	864
	National school facilities	1	4
	Public school facilities	55	235
	Cultural properties	2	3
	Children's welfare facilities	3	5
	The disabled rehabilitation & welfare facilities	4	26
	Welfare facilities for the aged	2	10
	Subtotal	563	3,901
	Total	739	6,884

City, Chiba Prefecture), and 161 persons were wounded. Among the damage to houses, broken ridge roofing tiles were remarkable. Ridge roofing tiles of many houses were broken at many places mainly in the eastern part of Chiba Prefecture close to the hypocenter. Among approximately 72,700 damaged houses, most houses were reported to be partially damaged, and the contents of damage were mostly broken ridge roofing tiles.

Distribution of the damaged wooden houses is shown in Fig. 6. Statistics of the total number of houses were not available, so that the total number of households in cities, towns and villages at the end of February 1988 was used instead. And ratio of the

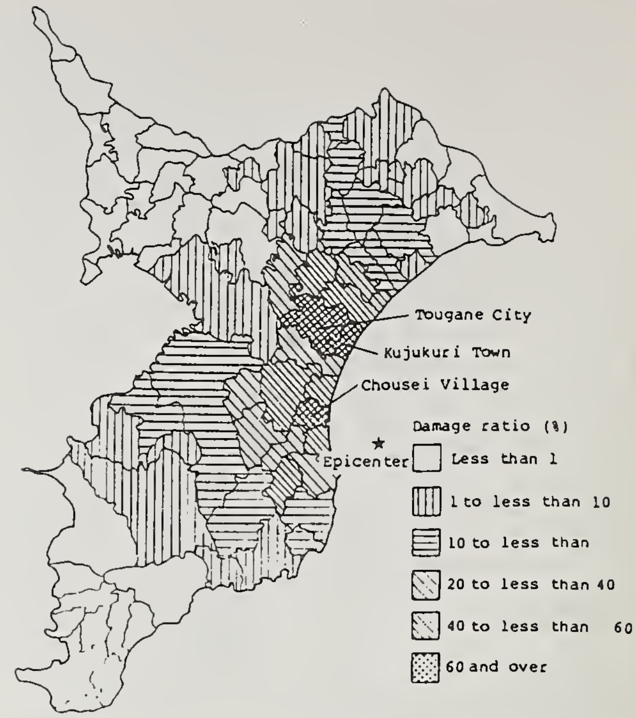


Fig. 6 Distribution of damage ratios for houses by city, town and village

number of damaged houses to the total number of households was determined as damage ratio for each city, town and village and is shown in Fig. 6. Damage ratios of houses in Tougane City and Chousei Village exceed 60%, and districts having large damage ratios are distributed mainly around both these districts. This means that slightly larger earthquake motions acted mainly to both the districts during the earthquake.

Cracks which might cause slope failure were noticed in slopes at 10 places (2 places in Narutou Town, 3 places in Matsuo Town, 1 place in Isumi Town, 1 place in Chounan Town, 2 places in Tougane City, and 1 place in Ichihara City in Chiba Prefecture), so 167 persons from 47 households in two cities and 4 towns evacuated to safe places.

Damage to public and other facilities occurred at 739 places in both Ibaraki and Chiba Prefectures, and the total amount of damage was reported to be 6.88 billion yen.

Among the damage to civil engineering facilities, the damaged facilities under the jurisdiction of the Ministry of Construction are shown by kind of facility in Table 3.

Facilities which were relatively damaged, will be explained later mainly with respect to the facilities under the jurisdiction of the Ministry of Construction. Damage given to river facilities were the settlement and cracking of levees and revetments. Damage to road facilities under the jurisdiction of the Ministry of Construction were the settlement of fills and depression of road surfaces which occurred on National Highway No. 126 at Daikata in Tougane City.

Table 3

Damage to the facilities under jurisdiction of the Ministry of Construction (as of April 4, 1988, surveyed by the Disaster Prevention Section, Ministry of Construction)

Class	Projects executed by central government		Projects executed by Local (Prefectural) Government		Total	
	Number of places	Amount	Number of places	Amount	Number of places	Amount
		(million yen)		(million yen)		(million yen)
Rivers	11	962	45	1,082	56	2,044
Steep slopes			1	2	1	2
Roads	1	54	90	629	91	683
Bridges			5	52	5	52
Sewerage			5	248	5	248
Total	12	1,016	145	2,013	158	3,029

Facilities under the jurisdiction of the local government were damaged at 45 places in rivers, 5 bridges, 90 places on roads and 5 places of sewerage. Damage to sewer facilities occurred in both Tougane and Mobara Cities; manholes and pipe ducts were damaged (at 883 places in Tougane City & 61 places in Mobara City), and the Choho pump station as well as water supply pipes and a sludge disposal yard of the Kawanakajima terminal treatment plant were damaged in Mobara City. Places of damage to rivers, bridges and road facilities related to subsidized projects in Chiba Prefecture are shown in Fig. 7. It can be known in this Figure that more damage occurred also in the eastern part of Chiba Prefecture which is close to the epicenter.

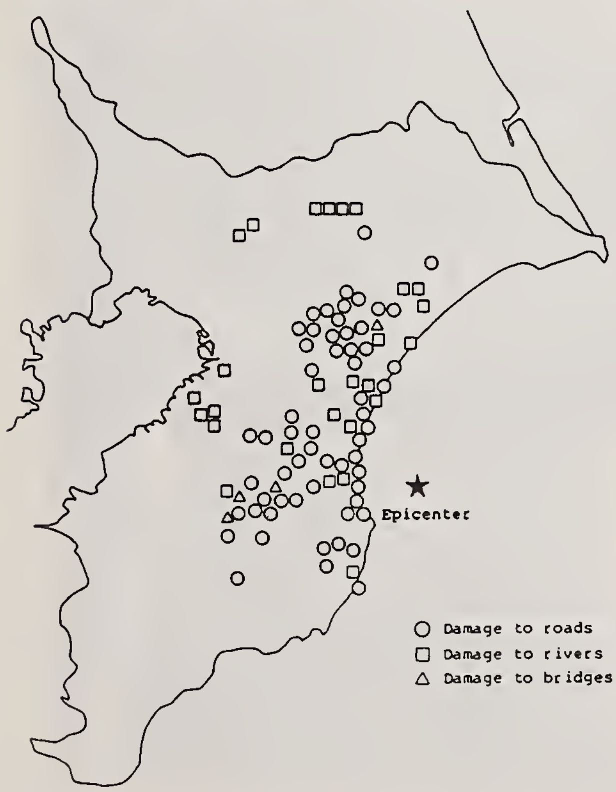


Fig. 7
Places of damage to rivers, bridges and roads (related to subsidized projects in Chiba Prefecture)

In addition to the damage to facilities, the phenomena of sand boils were observed at various places. The places where sand boils were surveyed in detail by Chiba Prefecture[11] and Yasuda et al.[11] One of the characteristics seen at the places of sand boils was that they occurred not only in reclaimed land, at beaches and lakes, but also in reclaimed land in mountainous regions. By comparing the epicentral distance of the places where liquefaction occurred to the limit epicentral distance explained by Kuribayashi and Tatsuoka,[12] it can be known that the liquefaction occurred during the earthquake at the places having longer epicentral distances compared to previous experience. An example of the occurrence of liquefaction at a place farther than Kuribayashi-Tatsuoka's limit epicentral distance was also reported with respect to the Lake Hachirougata[13], and the liquefactio seems to occur at the places exceeding the limit epicentral distance depending upon ground conditions or earthquake motion characteristics.

As seen in the cases of Chounan Junior High School and the levee of the Tone River, some structures are considered to be damaged directly by the liquefaction. But structures located near the places of sand boils were not always damaged during this earthquake. Probably, the reason of this feature may be related to the depth and thickness of liquefied layer. Liquefaction in Urayasu City located along the coast of the Tokyo Bay will be explained later.

3. DAMAGE TO RIVER FACILITIES

In the main stream of the Tone River, the phenomena of sand boils were observed not only in protected low land's and flood channels near the Tone River Lower Reach Construction Office, which manages the lower reach, but also at other places.

The places, where the phenomena of sand boils was observed, were the areas which were considered to be old river courses or the crossing portions of old rivers in all cases; the ground seems to be easily liquefied in these areas. In a wide place such as flood channel, the sand boiling was observed along continuous crack-shaped holes in the probable direction of the bank of the old river. In an area along the levee, the sand boiling continued in the direction parallel to the alignment of the levee. Photo 1 shows the sand boil in the flood channel near 39 km of the right bank.

Near the right bank of 47 km of the Tone River shown in Photo 2, the sand boil was observed in a paddy field at the protected low land side, at the toe of the slope at the protected land side, and at the berm on the water side. In the berm at the protected low land side, several stripes of cracks 20 cm wide and approximately 160 cm deep were created, and the levee crown was

Photo 1 Sand boil in flood channel near 39 km (Sawara City) at the right bank of the Tone River

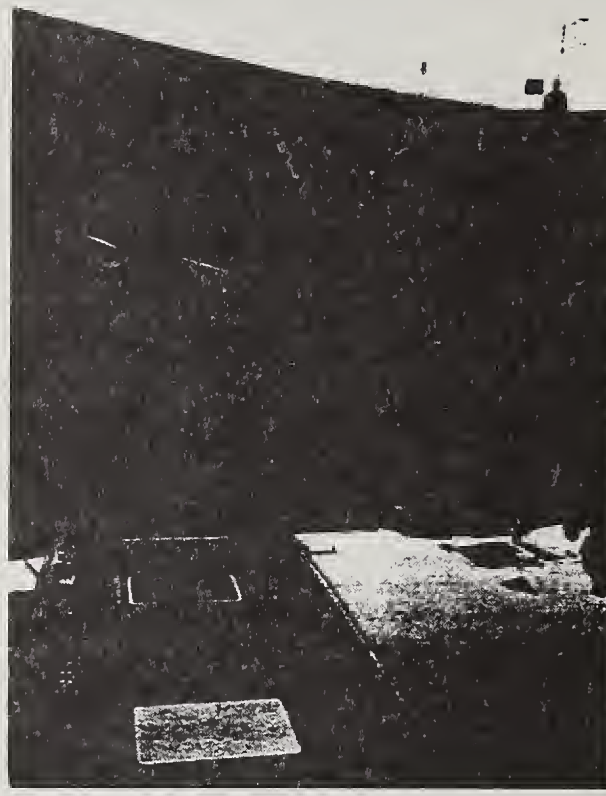


Photo 3 Depressed levee at the right bank upstream of Yokotone Gate

Photo 2 Cracks in inner berm near 47 km at the right bank of the Tone River

Photo 4 Depressed levee at the right bank downstream of the Yokotone Gate

settled. Though the amount of settlement was not accurately found, it looked about 30 cm maximum during observation in field surveys.

Transverse cracks were observed near the places where the maximum settlement occurred during field surveys, but these cracks were not found immediately after the earthquake. Therefore, in some cases, cracks in levees seem to appear on the surface with a time delay after the dispersion of pore water pressure inside the liquefied ground. This should be noted during the survey after an earthquake.

Photos 3 and 4 show the situation of damage near the Yokotone River Gate located at the confluence of the Yokotone River near 40 km at the left bank.

Photos 5 and 6 and Fig. 8 show the situation of damage near 1.25 km at the left bank of the Hitachi Tone River.

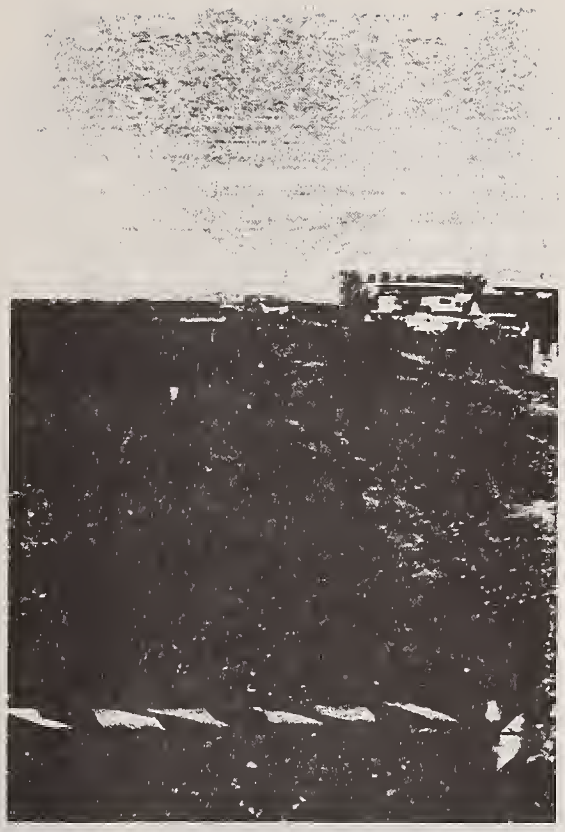


Photo 5 Overall view of the damaged levee near 1.25 km at the left bank of the Hitachi Tone River

Photo 6 Cracks in the levee near 1.25 km at the left bank of the Hitachi Tone River

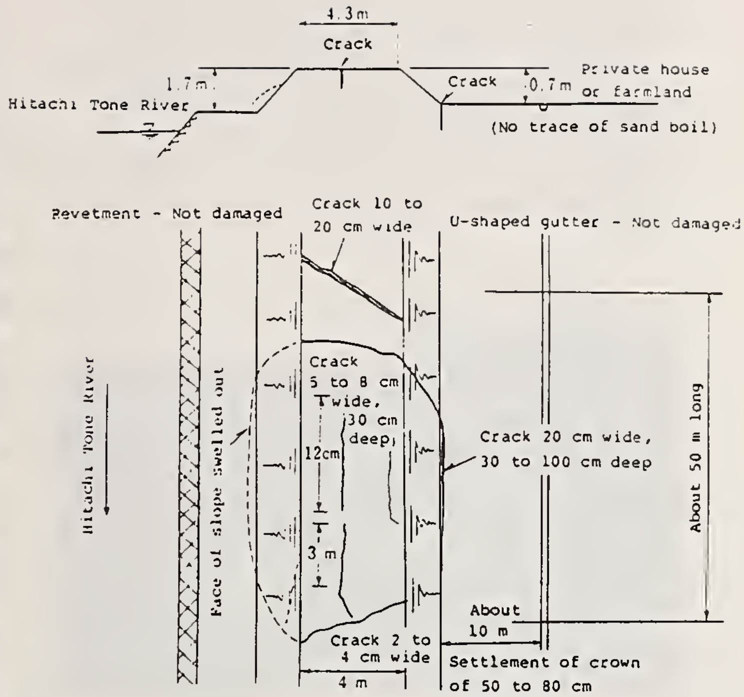


Fig. 8 Damage to the levee near 1.25 km at the left bank of the Hitachi Tone River

The levee at this place had been constructed in a tidal marsh previously, and the phenomenon of sand boiling was not observed near the damaged place. The levee was relatively low with a height of about 2 m, and the material of the levee was sand of almost uniform grain size. Automatic Ram sounding and Dutch Cone sounding were performed here immediately after the earthquake, and qc value of the levee was 7 to 10 kg/cm so that the material was loose. Also, part of the levee was so loose that it settled by itself.

Immediately below the levee, loose sand of almost uniform grain size, probably having a similar grain size distribution to that of the levee, was located to a thickness of 1.5 to 2.0 m and, below this sand, there was a sand layer which was relatively dense.

At the protected low land side of the damaged place, private land filled to 1.0 to 1.5 m above the original ground was located adjacent to the damaged place. There were U-shaped concrete gutters at the boundary between the private land and the river reservation. But no damage to the private land and the concrete gutters was noticed. One of the causes is considered to be the liquefaction in the levee below the groundwater level and in a very thin portion of the loose sand layer immediately below the levee. And one of the remote causes will be the raised groundwater level inside the levee by the fill on the private land at the protected low land side as explained above.

4. DAMAGE TO ROAD FACILITIES

On the National Highway No. 126 at Daikata in Tougane City in Chiba Prefecture, a difference in level of about 7 cm was created on the fill at the connection with an overbridge above JR Tougane Railway Line by the earthquake, also a depression of about 20 cm deep, 4 m wide and 100 m long was created on the road surface of the National Highway No. 126 at Daikata.

Photo 7 Difference in level created at the connection with an overbridge on the National Highway No. 126 at Daikata

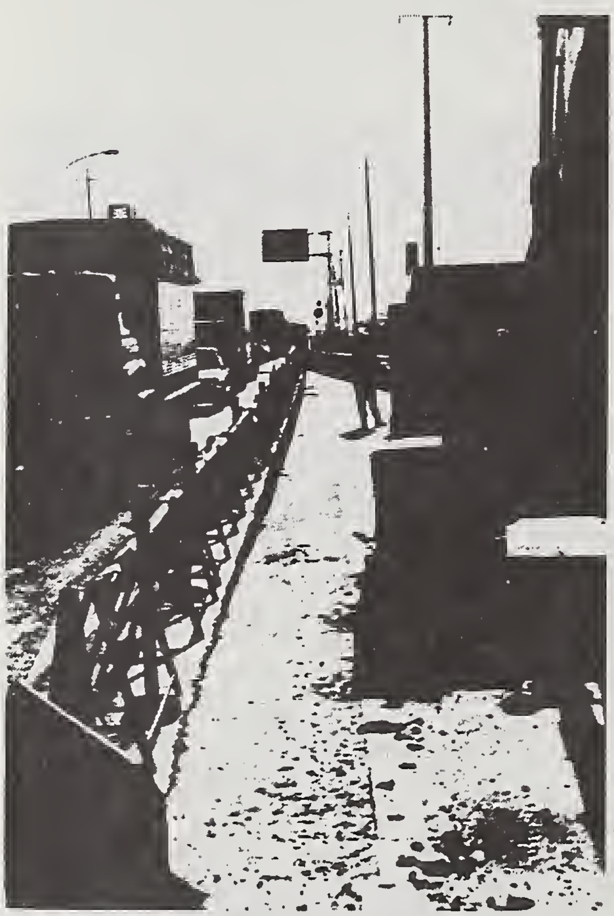


Photo 8 A depression created on the road surface on the National Highway No. 126 at Daikata in Tougane City

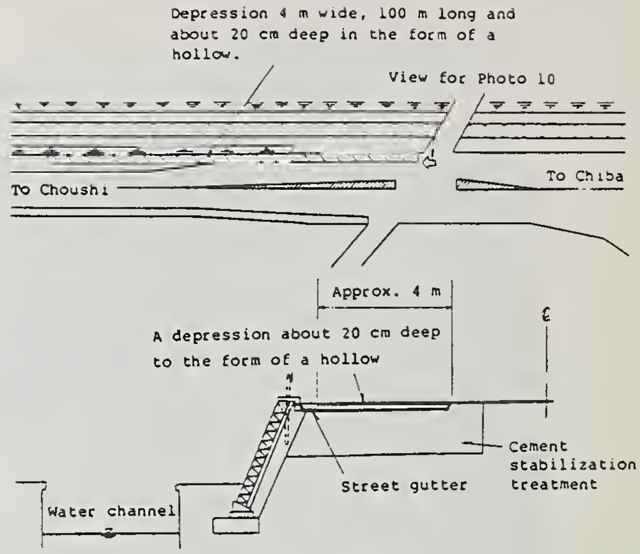


Fig. 9 Depression of road surface on the National Highway No. 126 at Daikata, Tougane City (No. 10 point shown in Table 3 and Fig. 6)

Photo 9 Damage to road surface on the Iioka Ichinomiya Line of Prefectural Road at Ichinomiya

Photo 7 shows the situation of emergency repair of the fill at the connection to the Daikata Overbridge and the difference in level at the curb, and Photo 8 and Fig. 9 shows a partially depressed road surface.

The portion where a depression was created on the road surface was on a low earth fill which probably had been widened before the earthquake, and only the newly filled portion was settled resulting in a depression with the maximum level difference of 20 cm on the road surface.

Photo 9 shows the situation of damage to roads on Iioka-Ichinomiya Line of the prefectural road at Ichinomiya. In this section, the road runs through pine forests on flat land, and the road surface meandered over a length of 400 m. On December 18, 1987 when the field survey was conducted, a level difference of 4 to 5 cm created on part of the road surface had been already repaired by asphalt paving by emergency measures.

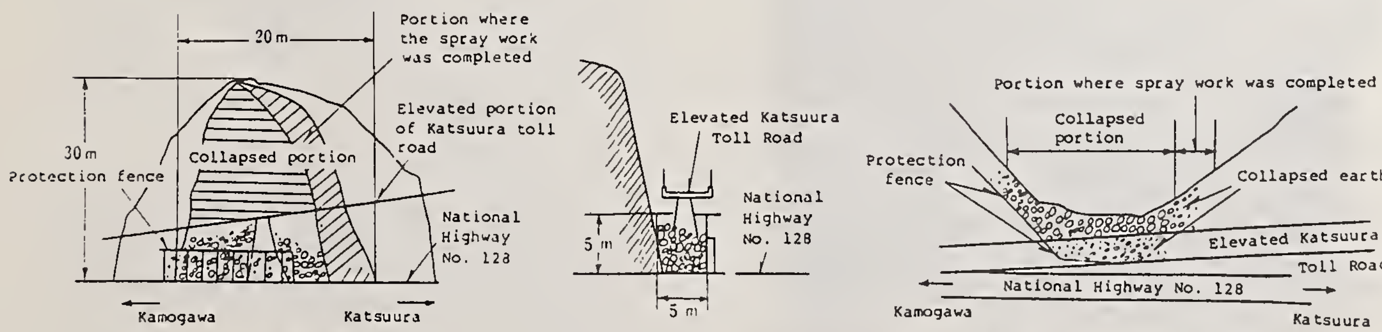
The meandering in this section was created by the earthquake in such a manner that the road surface was waved upward, downward and laterally so that the paved surface was separated from the raised or lowered from the roadside gutters. At the furthest north end portion of the damaged section, upward and downward wave on the road surface had the wavelength of 15 to 16 m, the wave height of the swell was about 8 cm, and the swell with 5 to 6 waves continued. The difference in level between the road surface and the top of roadside gutters was the largest at the central part of the damaged section, and the maximum difference in level was approximately 18 cm. Gaps of about 5 cm were observed between both the ends of gutters and the road and private land, thereby indicating the trace of horizontal movement of paved road relative to the adjacent natural ground.

Photo 10 shows the situation of the collapse of cut-slope at a road side on the National Highway No. 128 at Katsuura.

At the time of the occurrence of the earthquake, the site for the highway was in the stage of spraying works with the top soil and plants excavated from the face of slope as shown in Fig. 10. And the surface layer of weathered sandstone slope collapsed from the earthquake. According to the workers working on the site at the time of the earthquake, the whole of the face of the slope collapsed in a moment leaving a cloud of dust. No spring water from the slope was recognized. The first report of the damage indicated the collapsed volume of about 1,500 m^3. Part of collapsed earth was scattered to the National Highway No. 128 and the Katsuura toll road, but most of the collapsed earth was stopped by the protection fences installed for the slope works, causing almost no disturbance to road traffic.

In addition to these damage, relatively minor damage, such as cracks in road surfaces and differences in level, were caused also on the roads in Ibaraki Prefecture. Fig. 11 shows the situation of damage which occurred on the National Highway No. 125 near the boundary between Sakuragawa Village and Higashi Village. This section is a fill section 2 to 3 m high crossing exit roads, and transverse cracks near the boundary between fill and cut and roadside cracks in fill portion were caused.

Photo 10 Damage to the National Highway No. 128 in Katsuura City



(a) Front view of the damaged portion (b) Side view of the damaged portion (c) Plan view of the damaged portion

Fig. 10 Damage to the National Highway No. 128 (Katsuura City)
(No. 11 point in Table 3 and Fig. 6)

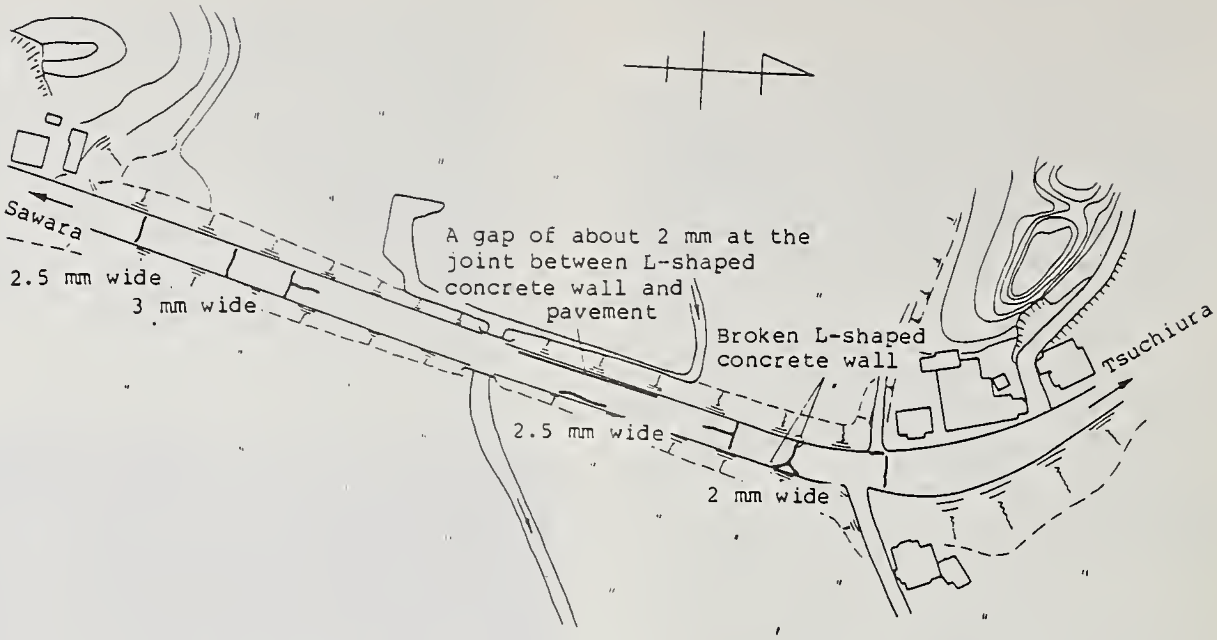


Fig. 11 Damage to National Highway No. 125 (at Sakuragawa)

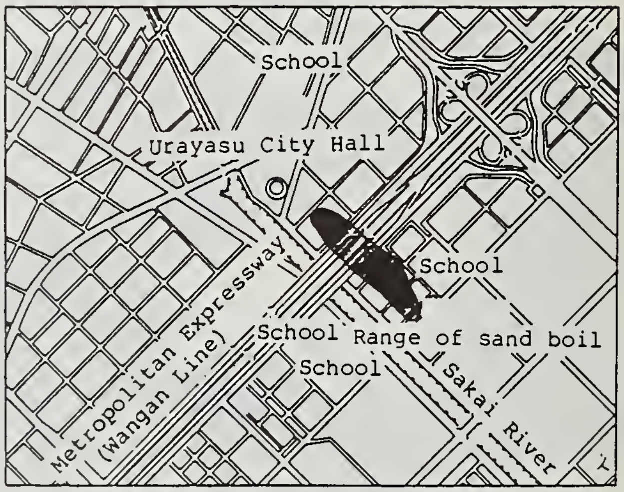
5. SAND BOIL IN URAYASU DISTRICT

Phenomena of sand boil were recognized after the earthquake also in Urayasu City and Takasu in Chiba City located along the Tokyo Bay in Chiba Prefecture.

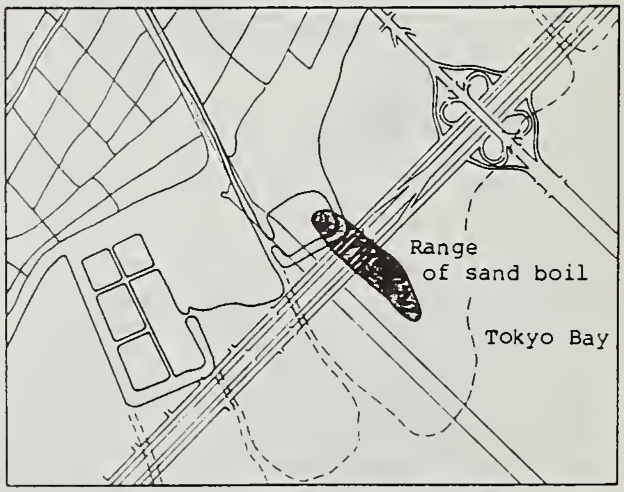
Among them, an example of sand boil occurred near Mihama in Urayasu City as shown in Photo 11. The sand boil occurred in a strip-like area at the south side of the Urayasu City Hall. Fig. 12(a) shows the range of sand boil summarized by the city office. Fig. 12(b) shows the former coastal line read from an old topographic map (surveyed in 1945) near the relevant area. Ground in this area was formed by filling-up recently, and the whole neighboring area including the strip-like area where the sand boil occurred this time has newly filled ground.

Nevertheless, the phenomenon of sand boil occurred only in a part of the area along the Sakai River, and this seems to be

Photo 11 Sand boiling in Urayasu City



(a) Range where sand boiled



(b) Range of sand boil and the location of former coastal line based on partially corrected map in 1945

Fig. 12 Places of sand boiling in Urayasu City (No. 25 point in Table 3 and Fig. 6) strange. The reason may not be found unless a detailed survey is made. However, the former coastal line coincides with the lagoon-shaped portion projecting out in the form of a sand bar, so that there may be a thin earth layer containing relatively fine grained soil below and near the surface layer. Thus, it is possible that the groundwater level in the thin earth layer was locally higher than that in the surrounding portion.

Fortunately, damage to structures in the sand boiling area as not reported except for cracks in road surfaces, houses were not deformed even one week later. Although occurrence of liquefaction in the ground was apparent during this earthquake, it did not cause damage to structures at some placed. The reason for this should be reviewed in detail based on the information concerning the magnitude of ground motion, ground conditions, groundwater levels, etc.

6. DAMAGE TO BURIED STRUCTURES

The earthquake caused damage not only to houses and fill structures such as embankments but also to sewer pipelines, manholes, water supply pipes and gas pipes which were buried underground as stated previously.[14)]
According to the case studies of damage to buried structures by the earthquakes which occurred in the past, the direct causes of the damage are the liquefaction of the ground and the accompanied flow failure of the ground in many cases. Liquefaction of the ground and the flow of the ground often accompany the sand boil or cracks in the ground surfaces, so that these abnormal states on the ground are greatly helpful for detecting the location of damage to underground structures which otherwise cannot be found directly by visual observation. However, damage to underground structures will not cause any abnormal states on the nearby ground surface in some cases. This occurs because the mechanism of causing damage to the buried structures is very complicated.

Therefore, it will be checked here whether the location of damage to buried structures by the present earthquake can be correlated to the changes in the nearby ground surface. This checking will be made with respect to the damage to water supply pipes. According to the data reported by the Chiba Prefectural Government, 652 cases of damage to the waterworks in the whole of Chiba Prefecture were reported.[15)] Breakdown of these cases is: 122 cases in Keiyou district, 11 cases in Kimitsu district, 16 cases in Katori district, 3 cases in Youkaichiba district, 222 cases in Sanbu district, 257 cases in Chousei district, and 21 cases in Isumi district. The number of cases of damage occurred in both Chousei and Sanbu amounts to 73% of the whole.

Table 4 Damage to water supply pipes and kinds of pipes
(Unit: Number of places)

Kind of pipe	Form of damage			Total
	Broken joint	Detached joint	Broken pipe	
Asbestos pipe	14	6	59	79
Vinyl pipe	43	10	28	81
Steel pipe	14	6	10	30
Ductile cast iron pipe	1	9	0	10
Total	72	31	97	200

Table 5 Damage to water supply pipes and pipe diameter
(Unit: Number of places)

Pipe diameter	Form of damage			Total
	Broken joint	Detached joint	Broken pipe	
Less than 100 mm	52	16	71	139
100 to 200 mm	16	10	20	46
200 to 300 mm	2	4	6	12
More than 300 mm	2	1	0	3
Total	72	31	97	200

△: Damage to water works (Chousei 87, Sanbu 113)
•: Damage to roads (Chousei 37, Sanbu 24)
·: Hatched portion shows mountainous area (hilly land).
o: Areas where the damage is concentrated.

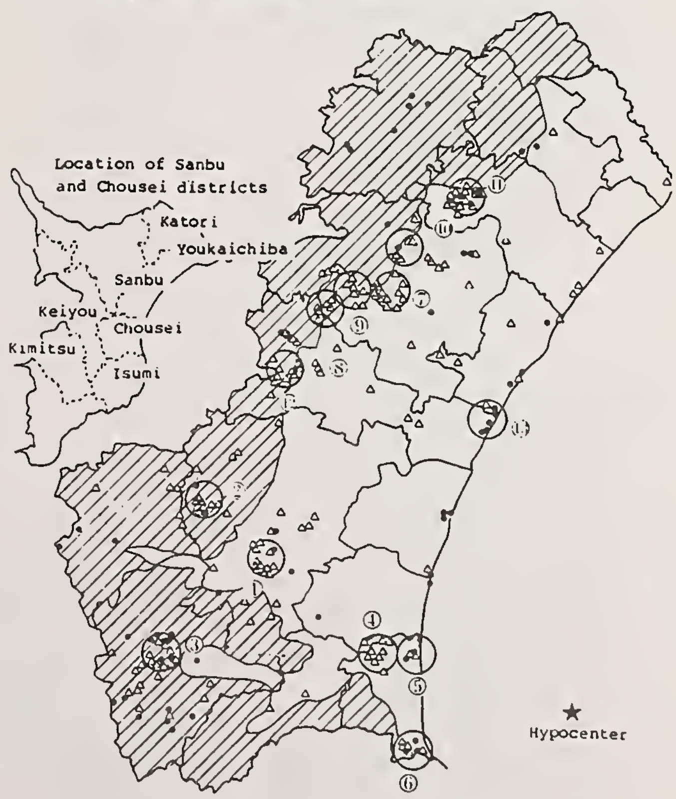


Fig. 13 Distribution of the damaged portions of water supply pipelines in Sanbu and Chousei districts

Among the places of damage to waterworks in both the districts, the restoration works were performed at 87 places in Chousei district and at 113 places in Sanbu district. Relations between the damage and the kind and diameter of pipes in 200 places in total are shown in Tables 4 and 5. And the places of damage are shown in Fig. 13. Fig. 13 also shows the places of damage in both Chousei and Sanbu districts, among the damage to roads indicated in Fig. 7.

The kinds of pipes which were more damaged were asbestos pipes and vinyl pipes; contents of damage were broken pipe body in the case of asbestos pipes and broken joints in the case of vinyl pipes. Pipes with smaller diameters were more damaged, and most damage as broken joints and broken pipe bodies of small pipes with diameters smaller than 100 mm.

From Fig. 13 showing the distribution of the places of damage to water supply pipes, it can be known that the damage to water supply pipes is distributed in the portions close to mountainous regions and mountainous portions along valleys at the western end of Kujukuri low land. This occurs because villages are concentrated in these areas and thus the density of water supply pipes there is relatively high. On the other hand, the damage to roads occurred more in the districts along valleys in mountainous areas and in the districts near the coast. Most damage to these roads as made on earth fills.

From the situation of this distribution, it is difficult to find certain characteristics of the relation between the damage to roads and damage to water supply pipes. However, according to the damage distribution in a small range, damage at more than 6 places was concentrated in 13 circular ranges with the radius of 1 km. Topographic features of these 13 ranges (districts) are as shown in Table 6.

Topographic features of the districts, where the damage as concentrated, were the change in the ground conditions.

As shown in Table 6, some districts have damage to roads while other districts have damage to water supply pipes but no damage to roads among the 13 districts. Thus, the location of damage to underground water supply pipes cannot be detected based on the

Table 6 Characteristics of the districts where the damage to water supply pipes as concentrated

Region	District name	Density of damage to waterworks: Number of cases shown in ( )	Density of damage to roads: Number of cases shown in ( )	Characteristics of topographic outline at damaged place
Chousei	(1) Downtown area in Mobara City	4.1 (13)	0.3 (1)	Fill, and boundary between fill and other topography (such as dune)
	(2) Kokufuzeki area in Mobara City	2.9 (9)	0.6 (2)	Flattened land, plain at the bottom of valley, or boundary between low position of dune and slope (steep slope)
	(3) Central part in Chounan Town	3.5 (11)	2.2 (7)	[Same as (2) above] + high fill
	(4) North area in Ichinomiya Town	2.9 (9)	0 (0)	Natural levee, sand bar, and boundary between these and other topography (reclaimed land, plain on coast)
	(5) Near Shin-Ichinomiya Bridge in Ichinomiya Town	0.6 (2)	0.6 (2)	Natural levee
	(6) Toromi area in Ichinomiya Town	1.6 (5)	1.0 (3)	[Same as (2) above] + plain on coast
Sanbu	(7) Built-up area in Tougane City	4.1 (13)	0 (0)	Fill (Most Downtown area was developed after 1975)
	(8) Yamaguchi area in Tougane City	2.5 (8)	0 (0)	Boundary to low position of terrace in low hinterland [Locally the same as (2) above]
	(9) Daizudani area in Tougane City	2.2 (7)	0 (0)	[Same as (2) above] + fill, low hinterland
	(10) Matsunokyo area in Tougane City	1.9 (6)	0.3 (1)	[Same as (2) above] + fill
	(11) Central part of Narutou Town	6.0 (19)	0.6 (2)	Fill, flattened land (reclaimed land), boundary between natural levee and plain on coast
	(12) West area of Ohami Shirasato Town	3.2 (10)	0 (0)	[Same as (2) above] + fill
	(13) East coast area of Ohami Shirasato Town	0.3 (1)	1.6 (5)	Sand bar

Note: Density of damage = Number of damage cases/km^2

abnormal states on ground surfaces alone, such as by damaged roads. However, if there are damaged roads and water supply pipes are located nearby, then the water supply pipes are also damaged in many cases.

Damage to structures buried relatively close to the ground surface are caused by the deformation of the ground such as settlement or cracks near the buried structures or by large strains in the ground created during the earthquake. Therefore, for detecting the location of damaged underground structures, it will be helpful to know the abnormal states (residual deformation) occurring on the ground and also to find the traces of dynamic strains created during the earthquake. However, at present, it is difficult to clarify the proper traces capable of suggesting the places where large strains actually occurred in the ground during earthquakes.

7. BEHAVIOR OF THE NAGARA DAM

The Nagara Dam was under construction in Nagara Town, Chousei County, Chiba Prefecture by the Water Resources Development Public Corporation. And hair cracks were observed at several places on the crown of this dam body and a slight settlement was measured after the earthquake. Accurate records of anomaly and valuable acceleration records by strong-motion seismographs were obtained during the earthquake since the dam reservoir was in the stage of test filling.[16]

The Nagara Dam is a zone type earth fill dam (embankment dam) 52 m high. Construction work for the dam was started in January 1973, the embankment was built from October 1980 to May 1985, and test filling of water to the reservoir was started in April 1986. Water level during the earthquake was near El = 57.0 m.

Fig. 14 shows the plan of the Nagara Dam, and Fig. 15 shows its section. According to the visual observation immediately after the earthquake, no anomaly was recognized on the riprap work for the slope of the embankment. But it was reported that hair cracks were recognized at several places on the top asphalt and on the pavement surface of a shifted road near the crown. Fig. 16 shows the location of the hair cracks noticed.

Also, traces of several cm of settlement of embankment were found at the transitional part between the spillway and dam body. Also, slightly increased gaps in concrete block joints of spillway were recognized at the top.

On January 1988, a follow-up survey was made of the cracks previously recognized at the crown of the embankment. The crown asphalt was peeled off and lime water was poured into the hair cracks. The maximum depth where the traces of lime water were recognized was about 1 m. When the discontinuous surface was scraped with a planting trowel, the surface was peeled off to the depth of about 2 m maximum. The hair cracks were not continued from upstream side to downstream side, and it was verified that the permeability coefficient of the embankment directly below the hair cracks satisfied the standard value of construction management, thereby assuring impermeability.

Measuring points for surveying were installed on the crown of the dam and on the upstream and downstream slopes, and the measurements were made on December 14 before the earthquake. The results were compared to the results of surveying performed on

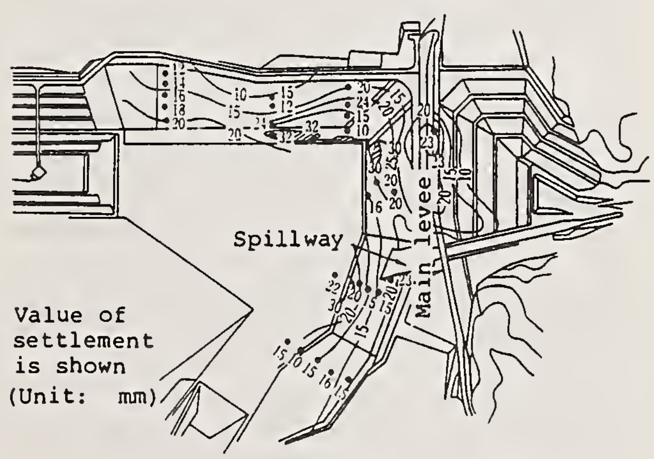
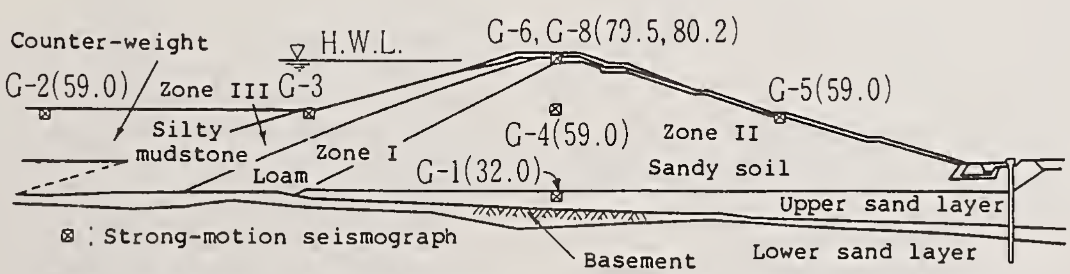


Fig. 14 Plan of the Nagara Dam (including settlement)



Note: Elevation (m) of strong-motion seismograph installed is shown in ( ).

Fig. 15 Cross section of the Nagara Dam (including the location of strong-motion seismographs)

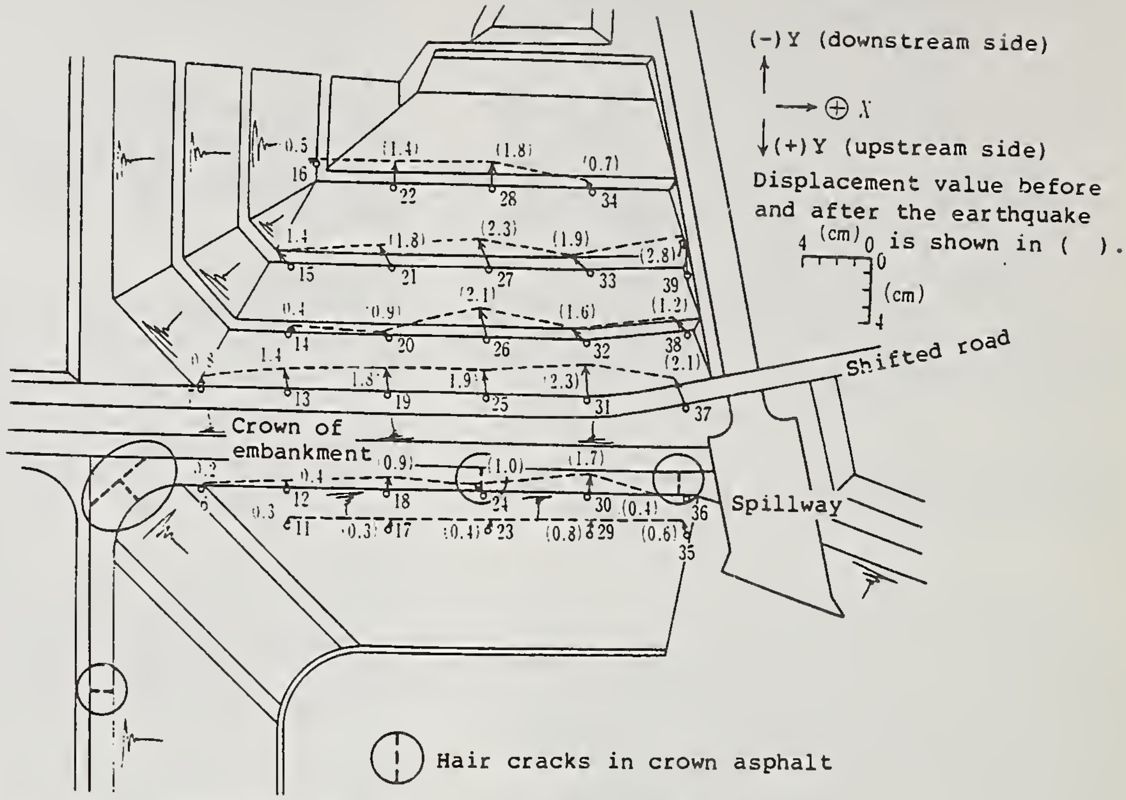


Fig. 16 Location of verified hair cracks and the distribution of horizontal displacements of dam body

December 21 after the earthquake, and the settlement and horizontal displacement of the embankment were arranged as shown in Figs. 14 and 16.

Settlement of the crown of the embankment was about 20 mm, and also a maximum settlement of 30 mm was recognized at the toe of the upstream slope. The horizontal displacement occurred toward the downstream side at both the upstream and downstream slopes, and the maximum horizontal displacement at the downstream slope was about 23 mm.

The strong-motion seismographs were installed at 10 places within the dam and on natural ground on the Nagara Dam site. Location of the strong-motion seismographs installed is shown in Fig. 15. Among the 10 strong-motion seismographs, 7 of them were of electromagnetic type, and the remainder were popularizing type. Nine out of ten seismographs were trigered during the earthquake and the records of 27 components were obtained.

Maximum acceleration values recorded are shown in Table 7. At the foundation surface (G-1) directly below the dam body, 262 gal was recorded in upstream-downstream direction and 138 gal in dam axis direction. At the crown (G-6), 369 gal was recorded in upstream-downstream direction, and 354 gal in dam axis direction. Horizontal acceleration in upstream-downstream direction was amplified by 1.4 times approximately.

In addition, pore water pressure gauges were installed at the Nagara Dam; bore hole type at 15 places and placing during construction stage type at 41 places. Earth pressure cells were also installed at 6 places.

According to the pore water pressure gauge installed near EL-46 m, a maximum rise of about 0.05 kgf/cm^2 in pore water pressure was recognized in Zone III.

At the upstream side of the embankment adjacent to Zone III, a counter-weight comprising loam and Narita sand was constructed up to EL = 59 m. This counter-weight was not fully compacted, and thus a possibility of a rise in pore water pressure by the action of earthquake motion acted to the inside of counter-weight was considered. This possibility will be roughly examined hereinafter.

The counter-weight about 20 m thick can be presumed to have almost homogeneous soils. Therefore, the shearing strain γ (z, t) due to the propagation of SH wave at the point with the depth Z in the soil layer can be expressed as shown below by using the velocity υ (t) at the ground surface (Z = 0).

$$\gamma(z, t) = \upsilon(t + z/Vs) - \upsilon(t - z/Vs) / (2 \cdot Vs) \quad .. (2)$$

where, Vs is the shear wave velocity in the counter-weight.

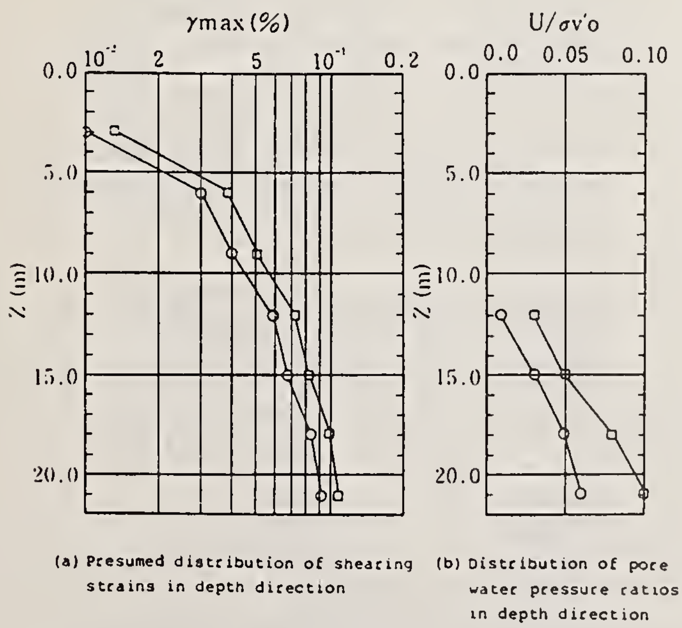
Since an accurate value of Vs is unknown, it

Table 7 Maximum acceleration recorded on the strong-motion seismographs installed at the Nagara Dam

Instrument Code	Measuring direction	Maximum acceleration (gal)	Instrument Code	Measuring direction	Maximum acceleration (gal)
G-1	X	262	G-6	X	369
G-1	Y	138	G-6	Y	354
G-1	Z	86	G-6	Z	324
G-2	X	270	G-7	X	262
G-2	Y	208	G-7	Y	294
G-2	Z	124	G-7	Z	177
G-3	X	353	G-8	X	497
G-3	Y	288	G-8	Y	386
G-3	Z	124	G-8	Z	328
G-4	X	180	G-10	X	281
G-4	Y	151	G-10	Y	148
G-4	Z	139	G-10	Z	111
G-5	X	382			
G-5	Y	298			
G-5	Z	180			

* Measuring direction codes:
X: Upstream-downstream direction
Y: Dam axis direction
Z: Vertical direction

is assumed to be Vs = 200 m/s based on the wet bulk density (γt = 1.62 tf/m^3). As the value of v(t), a time history of velocity derived by integrating the horizontal acceleration at G-2 was used for calculating the shearing strain inside the counter-weight from equation (2), and the distribution of its maximum values in the depth direction is indicated in Fig. 17(a).



(a) Presumed distribution of shearing strains in depth direction

(b) Distribution of pore water pressure ratios in depth direction

□: Where the records in upstream-downstream direction were used.
o: Where the records in embankment axis direction were used.

Fig. 17 Estimated maximum shearing strain and estimated pore water pressure inside the counter-weight fill

Here, the pore water pressure rising characteristics in the saturated soil layer due to the repeated action of shearing strain is needed, however the soil testing data is not available. Then, the following relation[17] which was derived by the author et al for Sengenyama sand is used.

$$\frac{u}{\sigma_{vo}} = \frac{\Sigma(\gamma_a - 0.05)}{1.076\ \Sigma(\gamma_a - 0.05) + 0.633} \quad \text{.. (3)}$$

where, γ_a is the amplitude (%) of shearing strain.

An example of the time history of the shearing strain which would have occurred inside the counter-weight fill is shown in Fig. 18. Pore water pressure ratio occurred inside the counter-weight was determined by using equations (2) and (3) and is shown in Fig. 17(b).

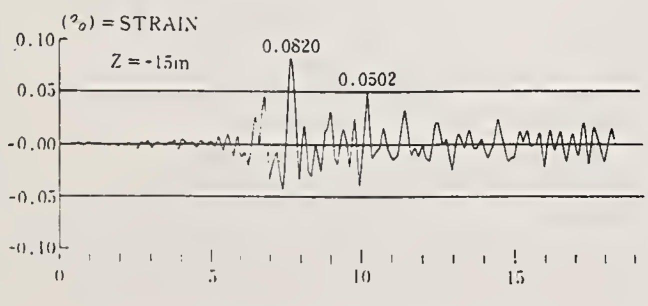


Fig. 18 An example of time history of estimated shearing strains (Z = -15 m)

From these results, the pore water pressure ratio at the depth of about 15 m inside the counter-weight becomes almost equal to 0.03 to 0.05. From this, it can be known that the pore water pressure of about 0.05 kgf/cm^2 can be generated.

8. CONCLUSION

The earthquake reported above gave a feeling of relatively large earthquake motions to people in the metropolitan region. But the magnitude of the earthquake was 6.7 and the depth of the hypocenter was deep, at 58 km. Therefore, the damage to structures occurred only in local, limited areas where the ground conditions were poor, and the life of citizens was not greatly disturbed. Road traffic was restricted in some sections immediately after the earthquake but, in general, this did not cause traffic disturbance. However, there was a danger of the occurrence of slope failures in certain areas, and the residents had to evacuate to safe places. Also, roofing tiles of many houses damaged at the same time, and a long period of time was necessary for repairing the roofs due to the shortage of labor and materials.

Places of boiling of sand and water were distributed in a wider range compared to the previous experience. This disclosed the possibly of dangerous places very susceptible to the earthquake on the new ground located near the metropolitan zone. This should be taken as a lesson for the future.

Acknowledgement

In the stage of the present survey, the author received data and materials from many organizations and persons including the Flood Control Section, Disaster Prevention Section, and Disaster Measures and Surveys Office, River Bureau, Ministry of Construction; Road Disaster Prevention and Measures Office, Road Bureau, Ministry of Construction; Tone River Lower Reach Construction Office, Kasumigaura Construction Office, and Chiba National Highway Construction Office, Kanto Regional Construction Bureau; Ryugasaki Civil Works Office, Ibaraki Prefecture; Public Waterworks Section, Chousei and Sambu Civil Works Office, Waterworks Bureau, Chiba Prefecture; City of Mobara and City of Tougane; and waterworks construction firms in Chousei and Sanbu. Tokyo Gas Co. gave the author the values of maximum acceleration immediately after the earthquake. And the Water Resources Development Public Corporation gave the author the records taken at the Nagara Dam. The author would like to express the greatest gratitude here to all the persons concerned.

REFERENCES

1) Sasaki, Y.: Prompt Report on the Chiba-ken Toho-Oki Earthquake of December 17, 1987, Civil Engineering Journal 30-1, pp. 47-55, 1988.
2) Sasaki, Y., Matsumoto, H., Kuwabara, T. and Seko, I.: Damage by the Chiba-ken Toho-Oki Earthquake, Tsuchi-to-kiso, Vol. 136, No. 12, pp. 5-11, 1988.
3) Earthquake and Volcano Division, Meteorological Agency: Summary of Earthquakes and Volcanoes in December 1987.
4) Harukawa, M.: Seismic Intensity Distribution in Kujukuri Plain and the Vicinities during the Chiba-ken Toho-Oki Earthquake and the Problems Suggested (2nd report), Proceedings of Symposium by Kanto Branch of the Japan Geological Society, pp. 18-24, 1988.
5) Mizoue, M.: Outline of the Chiba-ken Toho-Oki Earthquake, Proceeding of Symposium by Kanto Branch of Japan Geological Society, pp. 1-8, 1988.
6) Conference for Promoting the Strong Earthquake Observation Project: Prompt Report on Strong-Motion Accelerograms No. 37, published by the National Research Center for Disaster Prevention, Science and Technology Agency, 1988.
7) Osawa, R.: Special Issue, Earthquake Measures by Enterprises, Part 1, Earthquake Measures by Tokyo Gas Co., News from Earthquake Engineering Promotion Society, No. 93, 1987.
8) Kawashima, K., Aizawa, K. and Takahashi, K.: Method of Estimating Maximum Earthquake Motion and Earthquake Response Sp]ectrum (Part 4), Re-examination of the distance attenuation formulas for maximum acceleration, velocity, displacement and response spectrum, PWRI Technical Memorandum No. 1993, 1983.
9) Yoshimi, S., Sasaki, Y. and Kuwabara, T.: Riview of the Principal Axis of Three-dimensional Earthquake Motions in Strong Earthquake Records, Proceedings, 43rd Annual Meeting of the JSCE, Part 1, pp. 936-937, 1988.
10) Kusuda, T. et al.: Liquefaction Phenomena Observed in the Chiba-ken Toho-Oki Earthquake, Proceedings, Symposium by Kanto Branch, Japan Geological Society, pp. 31-34, 1988.
11) Yasuda, S., Touno, I. and Morimoto, I.: Survey Report of the Chiba-ken Toho-Oki Earthquake, Chishitsu & Chousa, 1988.
12) Kuribayashi, E., Tatsuoka, F. and Yoshida, S.: Ground Liquefaction History after the Meiji Era in Japan, PWRI Technical Note No. 30, 1974.
13) Sasaki, Y.: Lessons from the Damages Caused by the Nihonkai-chubu Earthquake (1983) - Lessons from the damage to the

levees of the Hachirougata Lagoon - Kisokou, pp. 77-84, May 1987.
14) Kanto Central Committee, Japan Gas Association: the Chiba-ken Toho-Oki Earthquake and City Gas, 1988.
15) Geographical Survey Institute, Public Works Research Institute and Building Research Institute, Ministry of Construction, National Land Development Technology Center: Report on the Development of Disaster Information System, pp. 176-190, 1988.
16) Kimoto, E., Yamazaki, A., Fukushima, R., Seko, I. and Tanaka, H.: Situation of the Nagara Dam of Bosou Water Conveyance Channel during the Chibaken Toho-Oki Earthquake, Tsuchi-to-Mizu, pp. 89-95, 1988.
17) Sasaki, Y. and Masui, H.: Calculations of the Generated Amount of Pore Water Pressure based on Cumulative Shearing Strains, Proc. 43rd Annual Meeting, JSCE, Part 3, pp. 448-449, 1988

Theme III

Storm Surge and Tsunami

Observations of Tsunamis on the West Coast of the United States

by

James F. Lander[1]

ABSTRACT

Although the recorded history extends back 150 to 200 years for the United States West Coast the effect of tsunamis there has been studied little. Major harbors of Seattle and San Franciso are well protected by narrow entrances and the southern coast of California is protected by a broad continental slope and offshore islands. Fourteen tsunamis with amplitudes of 1 meter or more or which were destructive have been reported. However, until 1960 none caused more than a few thousand dollars in damage. The effects of the 1946 Aleutian tsunami were stronger near Santa Cruz, California while the Chilean tsunami of 1960 caused about $1,000,000 in damage to southern California. The 1964 Gulf of Alaska tsunami was by far the most severe causing $13,000,000 in damage and 16 fatalities, mostly in Crescent City, California. These different areas of principal effects probably relate to the different orientations of the source of the wave. The 1812 Santa Barbara, California tsunami, 1960 Chilean tsunami and 1964 Alaskan tsunami are considered as the design tsunamis. Future tsunamis from the Shumagin gap, in Alaska, and Peru-Chile gap are potentially damaging. Population growth, coastal and harbor development, increased recreational use of beaches, and increased number of small fishing and recreational craft are increasing the risk of damage from future tsunamis.

Keywords: Tsunami; U.S. West Coast; Tsunami Hazard

1. INTRODUCTION

Permanent settlement of the West Coast by Europeans began in 1769 with the establishment of the Spanish mission at San Diego, and by American and British traders in Washington and Oregon territories in the early 19th century. Thus, a period of 150 to 200 years or less in some areas is available for the historical study of tsunamis. Recording tide gages were put into permanent operation in San Francisco, and San Diego, California and Astoria, Oregon, in 1854. These gages were probably the earliest in operation in the Pacific. In 1854 these gages recorded unusual oscillations which were identified as sea waves from the Tokaido and Nankaido, Japan, earthquakes. These may have been the first recorded tsunamis identified with earthquakes, and allowed the scientists of the time to calculate the average depth of the ocean between Japan and California. Despite this relatively long history, the study of West Coast tsunamis has received little attention.

Tsunami warnings for the West Coast are provided by the regional facility at Palmer, Alaska. Due to the response time they cannot provide warnings for locally generated tsunamis on the West Coast. Warnings for remote sourced tsunamis are sent to each state government and military commands and relayed to county officials. Response to the 1964 warnings was mixed. Some officials succeeded in making evacuations and others did little. In fact, in response to the warning, people crowded the wharfs in San Francisco and Los Angeles to see the 1964 tsunami arrive.

2. TECTONIC SETTING

The West Coast owes most of its good fortune with respect to the tsunami hazard to geography and tectonics. The major population centers of Seattle and the San Francisco Bay are well protected by large bays with restricted entrances. The coasts of northern California, Washington and Oregon are rugged and exposed to the sea. The communities there are small, seldom more than a few thousand people. Southern California is protected by a broader continental slope and a chain of offshore islands.

Unlike most of the Pacific coasts, the West Coast of the United States does not have typical subduction zone tectonics. The principal earthquake faults including the famous San Andreas are strike-slip without notable vertical offset and occur mainly on land. The 1906 San Francisco earthquake did not produce a tsunami even though portions of the break were under water. Offshore areas in southern California do have normal and reverse faulting

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and have caused small local tsunamis four or five times in historical times. Figure 1 shows the main tectonic features relative to tsunamis on the West Coast.

3. LARGE AND DAMAGING TSUNAMIS OBSERVED ON WEST COAST

Fourteen tsunamis have been observed on the West Coast with amplitudes of one meter or more. Nine have caused some damage including one tsunami with an amplitude of less than one meter (see Table 1). Remote sourced tsunamis from Japan, Kamchatka, the Aleutian Islands, Gulf of Alaska, Hawaii, and Chile have all caused at least minor damage. Local tsunamis have been reported observed four or five times but some of the earlier reports are questionable. The 1812 tsunami was once believed to have had a 15 m run up but now is believed to have had only a 3.5 m wave. The 1927 California tsunami was the best documented local tsunami. Prior to 1960 no tsunami caused more than a few thousand dollars of damage. The most destructive tsunamis were the 1960 tsunami from Chile and, the 1964 tsunami from the Prince William Sound, Alaska which was the most important. Even these two events which caused about $1,000,000 and $13,000,000 in damage respectively on the West Coast were overshadowed by the much more significant effects elsewhere which received most of the scientific attention. Due to the orientation of the generating zones these are thought to be the most hazardous sources possible for tsunamis and are the design tsunamis for remote sourced tsunamis for the West Coast.

4. COMPARISON OF THE 1946, 1960 AND 1964 TSUNAMIS

Figure 2 shows the amplitudes recorded or observed on the West Coast of the United States for the 1946, 1960 and 1964 tsunamis. The 1946 Aleutian tsunami was devastating to Hawaii and resulted in the establishment of the Pacific Tsunami Warning Center. On the West Coast it was observed widely but damage was limited to the area north and south of the San Francisco Bay. A maximum amplitude of 2.7 meters was observed at Noyo, California, where 100 fishing vessels were washed ashore. At Santa Cruz, California, one man was drowned and houses and sheds flooded.

The 1960 Chile tsunami's greatest effects were in the Long Beach/Los Angeles, California, area. There was one fatality, and $500,000 to $1,000,000 damage there, mostly to small craft and floating boat slips. It caused minor damage elsewhere including $30,000 to two boats at Crescent City, California.

The 1964 tsunami most strongly affected the Washington, Oregon, and northern California coasts with most of the damage (about $8,000,000) at Crescent City. Another $1,000,000 damage was done to small craft at San Rafael and Sausalito, California, marinas well inside the San Francisco Bay. Abundant loose logs from the lumbering industry contributed to the damage on the coasts of Washington, and Oregon. The waves tended to follow stream and rivers concentrating damage there. Figure 3 shows the effect at Seaside, Oregon. The effects were notably less in southern California. A total of 16 fatalities and about $13,000,000 in damage occurred. One fatality occurred the following day at Klamath River, California, due to continued high wave activity in resonance with the continental shelf.

5. RISK FACTORS

The local tsunamis are small and limited in effect. The length of expected fault rupture is believed to limit the maximum wave heights to 4 to 6 m, and the range of coastline affected to about 100 km. Also, there are no large rivers to create submarine deltas and deposits or steep sided trenches to give rise to a tsunami due to a submarine landslide. The history of local tsunamis may be too short to be fully confident that the worst has been observed. This is particularly true for the area off the coasts of Washington and Oregon where there is evidence of subduction of the Juan de Fuca Plate but no history of earthquakes or tsunamis from the area of subduction.

Seismic gaps in the Shumagin, Alaska, area and in the Peru-Chile area are potential source areas for remote sourced tsunamis damaging to the West Coast in the future and may have effects similar to the 1960 and 1964 tsunamis.

A particular hazard to the northern part of coast is damage caused by logs which litter the beaches. These act as battering rams and also end up as substantial debris on land. In the southern area much of the damage is caused to small craft insecurely berthed in floating slips. Currents set up by the tsunamis can dislodge the boats which became additional floating projectiles.

The tsunami hazard is growing due to changing demographics including: population growth, the increased use of beaches for recreation, increased numbers of small fishing and pleasure craft, and increased development of harbor areas.

6. CONCLUSIONS

1. The West Coast probably has experienced its design tsunamis in the 1812, 1960, and 1964 events with the 1964 event being the most destructive.

2. The West Coast is relatively protected from tsunamis due to protected coasts and harbors. There is apparently only a minor local tsunami risk.

3. A significant current hazard probably exists from remote-sourced tsunamis from seismic gap areas in Shumagin, Alaska, and South America.

4. The tsunami hazard is growing due to changing demographics with increased coastal development, recreation use of beaches, population growth, and numbers of small craft.

5. Logs on the beaches of the Northwest and small craft in floating slips add to the hazard.

6. Several localities such as Crescent City, Half Moon Bay, Santa Cruz, and Avila, California report larger amplitude waves than other communities which may relate to the shape of their harbors or to their exposure to the open ocean. Tsunamis follow rivers inland affecting communities which also cluster near the river mouths.

7. Variations in the tsunami height reported at locations along the coast and for tsunamis are probably related to the orientations of the tsunami source.

7. REFERENCES

1. Agnew, Duncan Carr, "Tsunami History of San Diego," Earthquakes and Other Perils, San Diego Region, San Diego Association of Geologists, 1979, p. 117-138.
2. Bache, A.D., Report of the Superintendent of the United States Coast Survey for 1855, Washington, D.C., 1856, p. 96, and appendix, 50 p. 342-346.
3. Bancroft, Hubert H., History of California, Vol. 4, 1886, p. 78.
4. Berkman, S.C. and J.M. Symons, The Tsunami of May 22, 1960 as Recorded at Tide Stations, United States Department of Commerce, Coast and Geodetic Survey, Washington, D.C., 1964, 79 p.
5. Byerly, Perry, "The California Earthquake of November 4, 1927," Bulletin of the Seismological Society of America, Vol. 20, No. 2, June 1930, p. 53-66.
6. Green, C.K. "Seismic Sea Wave of April 1, 1946, as Recorded on Tide Gages," Transactions of the American Geophysical Union, Vol. 27, No. 4, August 1946, p. 490-500.
7. Grouginis, V.J., J.W. Joy, and R.R. Putz, The Reported California Tsunami of December, 1812, unpublished manuscript, 120 p.
8. Heaton, Thomas H. and Stephen H. Hartzell, "Earthquake Hazards on the Cascadia Subduction Zone," Science, American Association for the Advancement of Science, Washington, D.C., 1987, Vol. 236, p. 162-168.
9. Heaton, Thomas H. and Parke D. Snavely, Jr., "Possible Tsunami Along the Northwestern Coast of the United States Inferred from Indian Traditions," Bulletin of the Seismological Society of America, Vol. 75, No. 5, 1985, p. 1455-1460.
10. Hogan, O.W., W. W. Whippe, and C. Lundy, Tsunamis of 27 and 28 March, 1964, State of Washington Coastline, United States Army Corps of Engineers (Seattle, Washington, unpublished file report, 1964 29 p.)
11. Joy, J.W., "Tsunamis and their Occurrence along the San Diego County Coast", Westinghouse Ocean Research Laboratory Report, No. 68-567- OCEAN-RL, July 15, 1968, Sàn Diego, California.
12. Larsen, L.H., "Tsunamis Hazard Along Washington's Coast," Natural Hazards in Washington's Coastal Zone, Washington State Department of Emergency Services, Olympia, Washington, 1979, p. 35-38.
13. Magoon, O.T., Structural Damage by Tsunamis, American Society of Civil Engineers Specialty Conference on Coastal Engineering, 1965 Proceedings, Santa Barbara, California, 1965, p. 35-38.
14. Marine Advisers, Inc., Examination of Tsunami Potential at the San Onofre Nuclear Generating Station, Report A-163, Los Angeles, California, September 1965, 59+ p.
15. McCulloch, D.S., "Evaluating Tsunami Potential," in Evaluating Earthquake Hazards in the Los Angeles Region, United States Geological Survey Professional Paper 1360, United States Geological Survey, Reston, Virginia, 1985, p. 375-413.
16. National Academy of Sciences Committee on the Alaska Earthquake, The Great Alaska Earthquake of 1964, Oceanography and Coastal Engineering, National Academy of Sciences, Washington, D.C., 1972, 556 p.
17. O'Brien, M. P., Preliminary Report on Seismic Sea Waves from Aleutian Earthquake of April 1, 1946, Technical Report Wave Project Fluid Mechanics Laboratory, University of California at Berkeley: April 25, 1946.
18. Roberts, James A. and Chen-Wu Chien, "The Effects of Bottom Topography on the Refraction of the Tsunami of 27-28 March 1964: The Crescent City Case," Ocean Science and Ocean Engineering 1965--Joint Conference Marine Technology Society and American Society Limnology and Oceanography, Washington, D.C.,Trans., Marine Technology Society, Washington, D.C., Vol. 2, 1964, p. 707-716.
19. Salsman, Garrett S., The Tsunami of March 9, 1957 as Recorded at Tide Stations, Technical Bulletin No. 6, United States Coast and Geodetic Survey, July 1959, 18 p.
20. Soloviev, S.L. and Ch.N. Go, A Catalogue of Tsunamis of the Eastern Shore of the Pacific Ocean, Academy of Sciences of the USSR, "Nauka" Publishing House, Moscow, 1975, 204 p. (translation by Canada Institute for Scientific and Technical Information, National Research Council, Ottawa, Ontario, Canada K1A 0S2, 285 p.)

21. Spaeth, M.G. and S.C. Berkman, Tsunami of March 28, 1964, as Recorded at Tide Stations and the Seismic Sea Wave Warning System, The Great Alaska Earthquake of 1964, Oceanography and Coastal Engineering, National Academy of Sciences, 1972, p. 38-110.

22. Symons, J.M. and B.D. Zetler, The Tsunami of May 22, 1960 as Recorded at Tide Stations, Preliminary Report, United States Department of Commerce, Coast and Geodetic Survey, Washington D.C., 1960, 29 p.

23. Wiegel, R.L., "Tsunamis," Earthquake Engineering, Prentice-Hall, Englewood, Cliffs, New Jersey, 1970, p. 253-306.

24. Wilson, Basil W. and Alf Trum, "Runup Heights of the Major Tsunami on North American Coasts" in The Great Alaskan Earthquake of 1964; Oceanography and Coastal Engineering, National Academy of Sciences, Washington, D.C., 1972, p. 138-180.

25. Zerbe, W.B., Tsunami of November 4, 1952, as Recorded at Tide Stations, United States Department of Commerce, Washington, D.C., 1953, 62 p.

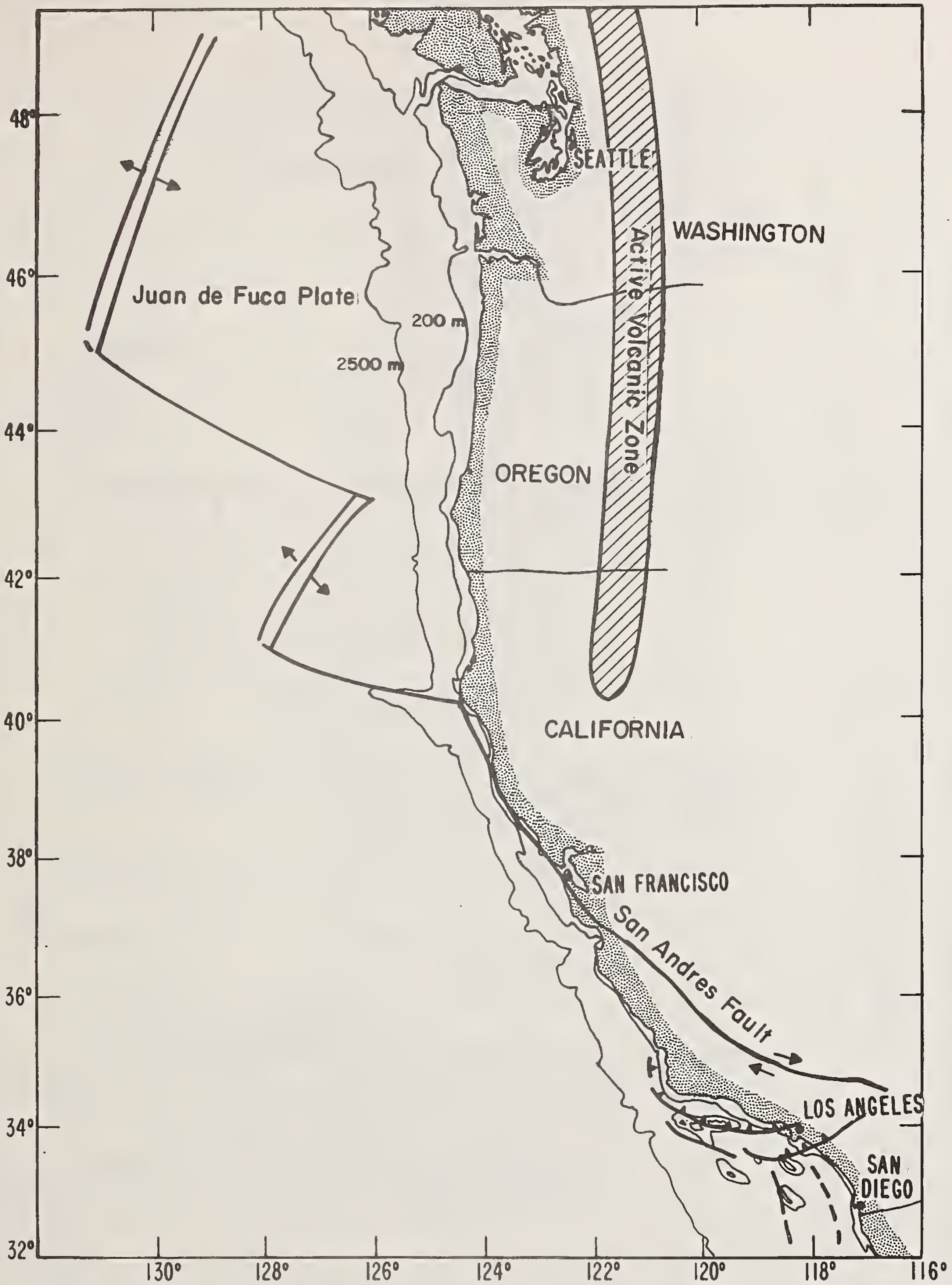

Fig. 1. Schematic of major tectonic features relating to tsunamis on the west coast

Table 1. Tsunamis Affecting the West Coast of the U. S.

Damaging or at Least 1 Meter Amplitude

Date	Amplitude (M)	Locality Most Affected	Source
1812	3.5	Gaviota (S. Calif.) Negligible Damage	S. Calif.
1859	4.6	Half Moon Bay (C. Calif.) Schooner Damaged	Unknown
1862	1.2	San Diego (S. Calif.)	S. Calif.
1868 (Aug.)	1.8	San Pedro (S. Calif.)	Chile
1868 (Oct.)	4-5	San Francisco (C. Calif.)	N. Calif.
1877	3.6	Gaviota (S. Calif.)	Peru-Chile
1878	2.0	Wilmington (S. Calif.) Unspecified Damage	Chile?
1896	1.5	Santa Cruz (C. Calif.) Dike Destroyed, Ship Damaged	Japan
1927	1.8	Surf and Avila (S. Calif.)	S. Calif.
1946	2.7	Santa Cruz (S. Calif.) Some Damage, 1 Killed	Aleutian Is.
1952	1.4	Crescent City & Avila (N. & S. Calif.)	Kamchatka
1957	1.0	San Diego (S. Calif.) Minor Damage	Aleutians
1960	1.7	Crescent City & Los Angeles (N. & S. Calif.) $1,000,000 in Damage, 1 Killed	Chile
1964	4.3	Crescent City (N. Calif.) $12,000,000 in Damage, 16 Killed	Gulf of Alaska
1975	0.4	Catalina Island (S. Calif.) $2,000 in Damage	Hawaii

TSUNAMI AMPLITUDES ON UNITED STATES WEST COAST

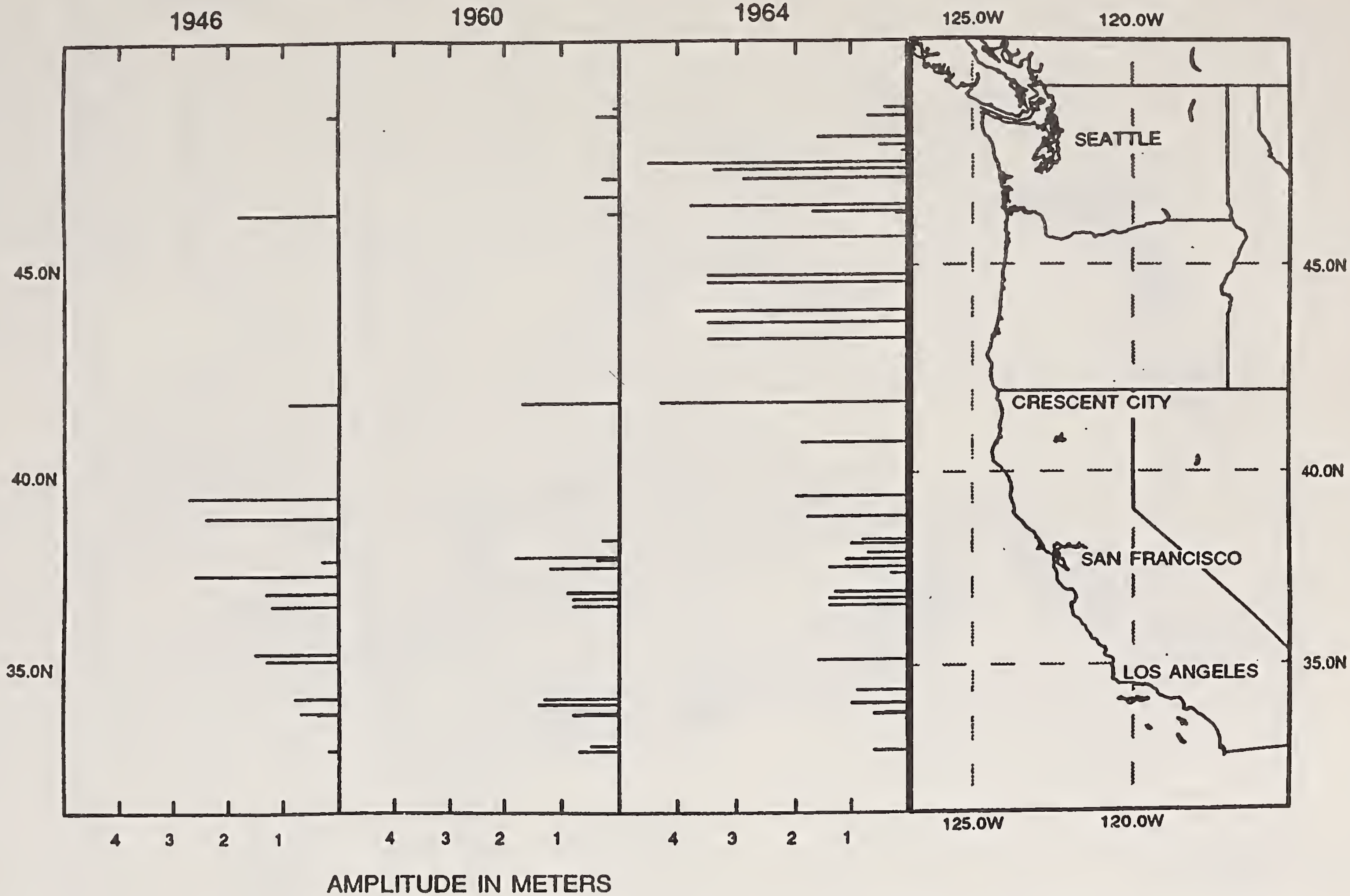

Figure 2. Tsunami amplitudes at selected localities along the west coast of the United States for the tsunamis of April 1, 1946, in the Aleutian Islands, May 22, 1960, in Chile and March 28, 1964, in the Gulf of Alaska.

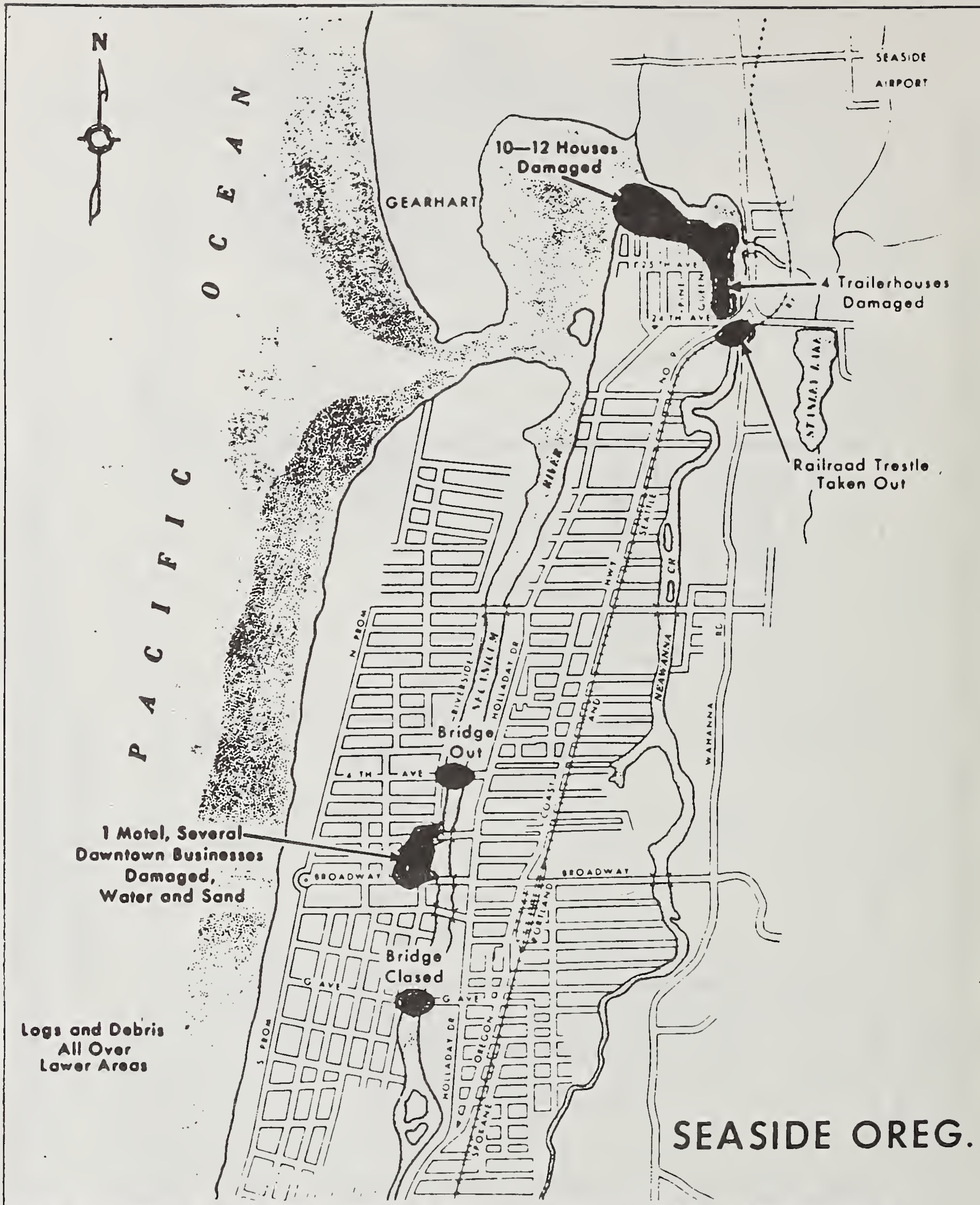

Figure 3. Location of major tsunami damage in Seaside, Oregon (from Spaeth and Berkman, 1972).

Present State of Tsunami Numerical Simulation

by

Chiaki Goto*[1] and Katsutoshi Tanimoto*[2]

SUMMARY

Present state of tsunami numerical simulation which is generally carried out to predict tsunami hazard in Japan is briefly reviewed. As recent knowledge, simulations for transoceanic propagation and soliton fission of tsunamis and behavior of spread materials due to tsunami are also introduced. Finally, through a knowledge of numerical error, remarks for computation are explaned.

KEY WORDS : Tsunami, Distant Tsunami, Soliton Fission, Behavior of Spread Materials, Numerical Computation, Numerical Error.

1. INTRODUCTION

Located in northwestern part of the Pacific Ocean seismic belt, Japan has been facing to tsunami hazard and is widely known as the place where tsnamis often attack. Especially, the Sanriku Coast in northeastern coast of main island of Japan experienced crushing calamities caused by the Great Sanriku Tsunamis occured in 1896 and in 1933. Due to the two tsunamis, more than 30,000 people were killed and about 10,000 houses were lost. Thus, the Japanese term "tsunami" had a chance to become the international word.

For the past twenty years, many tsunami walls and breakwaters have been constructed along Japanese coast line as the tsunami protection works. However most of them Were constructed for the purpose tof the pretection against middle class of tsunamis and a few structures were designed against huge tsunamis. For countermeasures against huge tsunamis, the total protection plan with the construction of defence structures, the management of hazardous area and tsunami warning system should be considered. From the reason and the limited data avilable on tsunamis, many research attempts for the prediction of tsunami hazard have been made by numerical simulation.

2. NUMERICAL COMPUTATION OF TSUNAMI

2.1 Computation in deep sea region

The finite difference and the finite element methods have been developed for reproducing the tsunami propagation based on the long wave theories. Here, a numerical simulation of tsunamis by using the finite difference method which is usually carried out to predict tsunami hazard is introduced. The computation of tsunamis is generally carried out in two separate parts: deep sea and shallow sea area, because of a limit of computer memory capacity and computation time. In deep sea, the computation area is covered with coarse grid and a coast line is approximated as a vertical wall. However the grid length might be varied dependent on the length of tsunami source, general grid length is set so as to be about 5 km long and become gradually smaller approaching a shallow sea. Figure 1 shows a computation area on Sanriku Coast located in northern Japan[1]. Computation areas are divided four regions, region A to region D, the grid lengths are 5.4 km, 1.8 km, 0.6 km and 0.2 km respectively. In this figure, tsunami sources reported by Hatori[2] are also presented. The numerals inidicate the occarence year of tsunamis.

Generally, the Governing equations in deep

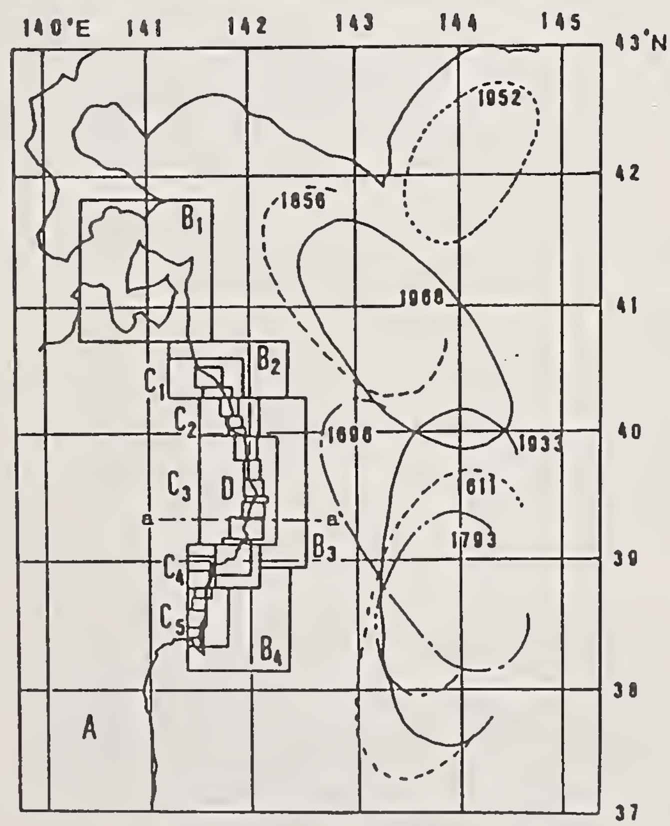

Fig.1 Computation region

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sea are the linear long wave theory as follows.

$$\frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \qquad (1)$$

$$\frac{\partial M}{\partial t} + gh\frac{\partial \eta}{\partial x} = 0 \qquad (2)$$

$$\frac{\partial N}{\partial t} + gh\frac{\partial \eta}{\partial y} = 0 \qquad (3)$$

where

- (x, y) : spatial coordinates.
- t : time.
- g : gravity acceleration.
- h : still water depth.
- η : tsunami height.
- (M, N) : tsunami discharge flux in the (x, y) direction.

There are some cases which takes terms of the Coriolis and the bottom friction into consideration. The effects of the terms, however, are usually small because tsunamis attacking Japan islands travel only short distance in deep sea.

The initial tsunami profile is assumed to be the same as crustal movement calculated on a fault model. The seabed deformation can be approximated as instantaneous change and the initial water surface can be also asuumed to have the same profile as the deformated sea bottom by earthquake.

The computation for the deep sea, mentioned above, are primarily performed with the purpose of calculating the input condition for the computation of shallow sea areas. In addition, the reliability of the computation results should be considered only for the sea of deeper than 200 m.

Figure 2 shows a result of computation in deep sea for Great Meiji Sanriku Tsunami occured in 1896.[1] The vertical scales of the figures shown are exaggerated compared to the horizontal ones. The maximum tsunami height at t = 0 s is 4m.

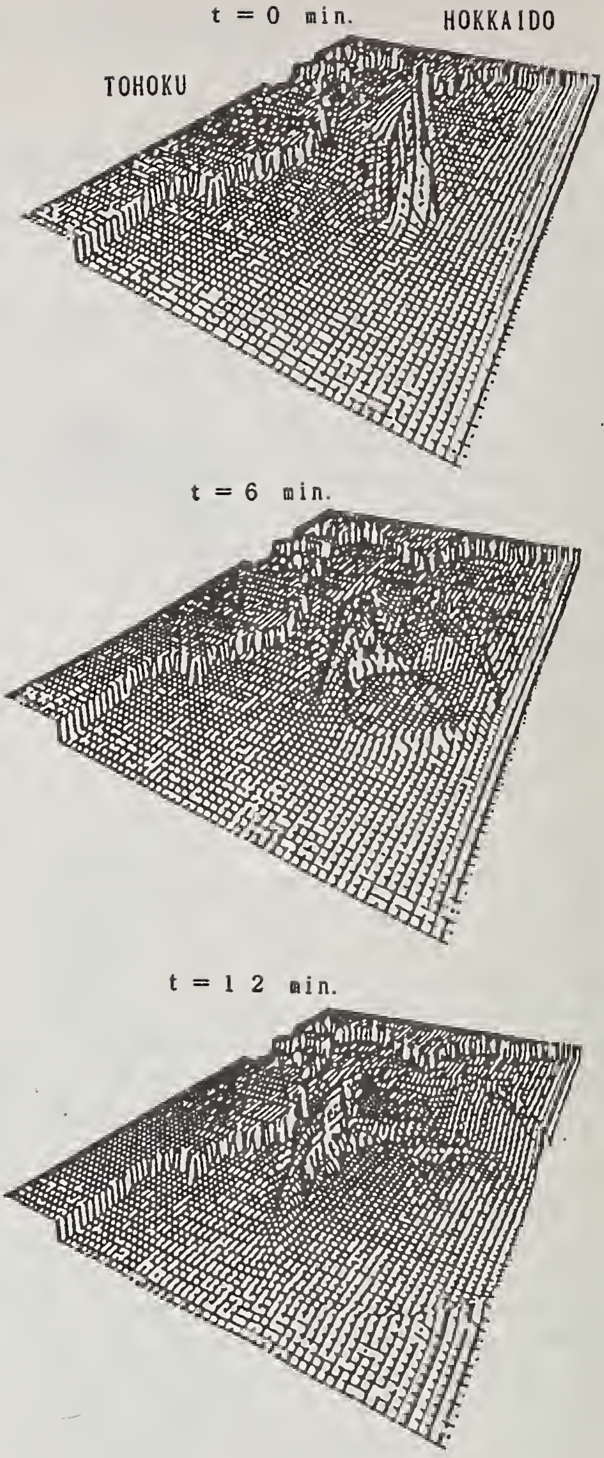


Fig. 2 Computation result for Great Meiji Sanriku Tsunami (in deep sea).

2.2 Computation in shallow sea region

The computation for a shallow sea is carried out for the purpose of predicting tsunami behavior in the vicinity of the coast and on the land. Generally the grid size is less than 50 m. The governing equations used for the computation is the shallow water theory, which takes sea bottom friction into consideration, as follows.

$$\frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \qquad (4)$$

$$\frac{\partial M}{\partial t} + \frac{\partial}{\partial x}\left(\frac{M^2}{d}\right) + \frac{\partial}{\partial y}\left(\frac{MN}{d}\right) + gh\frac{\partial \eta}{\partial x} + \frac{gn^2}{d^{7/3}} M\sqrt{M^2 + N^2} = 0 \qquad (5)$$

$$\frac{\partial N}{\partial t} + \frac{\partial}{\partial y}\left(\frac{N^2}{d}\right) + \frac{\partial}{\partial x}\left(\frac{MN}{d}\right) + gh\frac{\partial \eta}{\partial y} + \frac{gn^2}{d^{7/3}} N\sqrt{M^2 + N^2} = 0 \qquad (6)$$

where

- d : total water depth, $d = h + \eta$.
- n : Manning's roughness coefficient[3].

As the offshore boundary condition in shallow sea region, the results of the computation in deep regions is given. Homma's weir formula[4],[5] is usually applied to the estimation of overtopping rate or over flow

(a) Computation result with both effects of run-ups on land and coastal structures.

(b) With effct of run-up on land and without effect of coastal structures.

(c) Without both effects.

Fig. 3 Computation results for the 1896 Great Meiji Sanriku Tsunami (in shallow sea area).

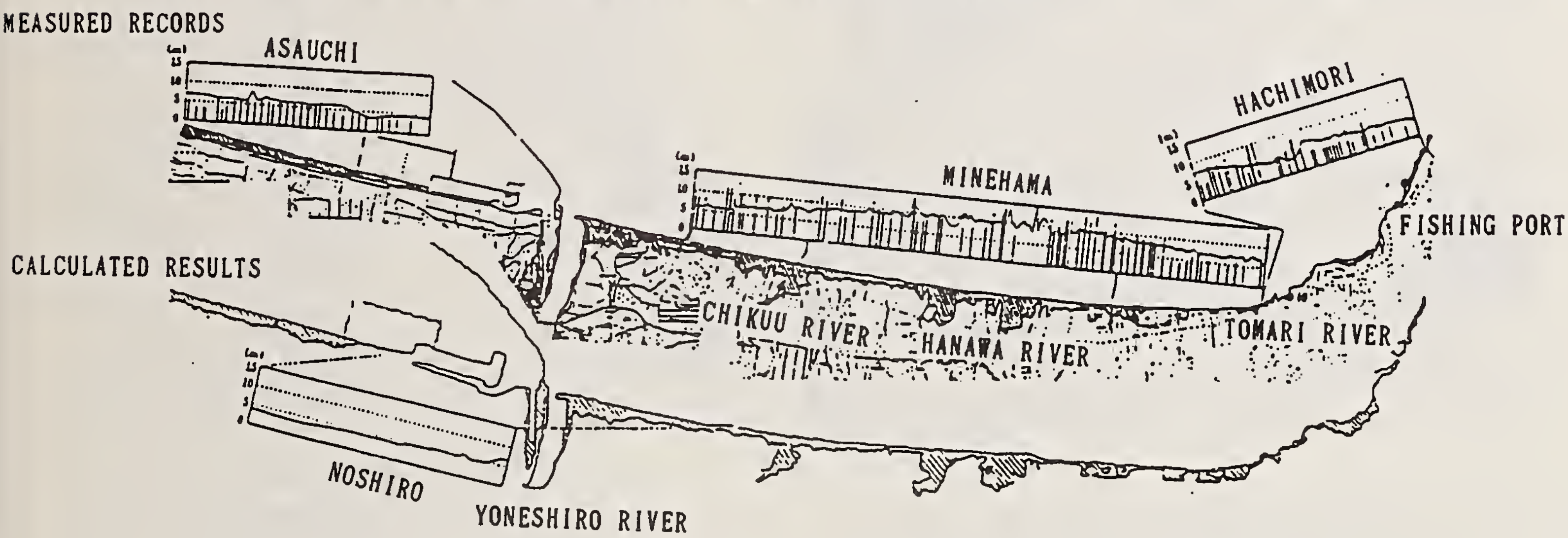


Fig. 4 Computation results for the 1983 Nihonkai-Chubu Tsunami.

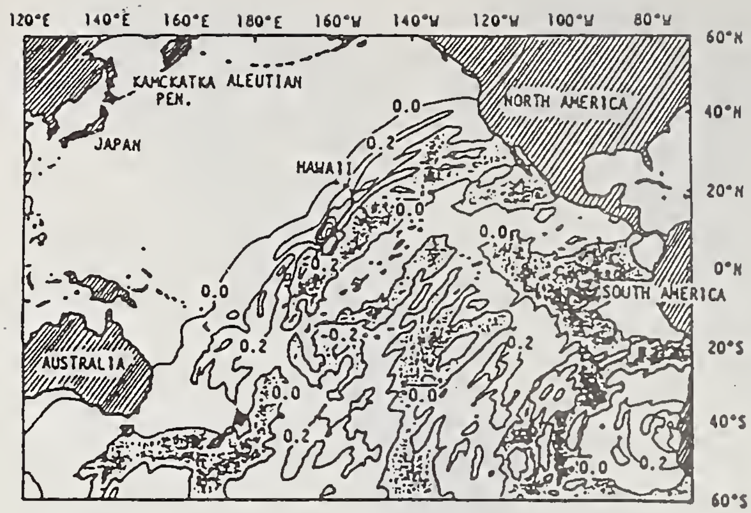
rate on sea walls and breakwaters. As the boundary condition at the tsunami front on land, there are two methods. One is a method that assumed a weir formula[6] and the other is a method that a continuous topography is approximated by series of discontinuous horizontal steps[7]. Because the reliability of both methods are not clear from a physical point view, we have to use the fine grid near shoreline.

An example of computational results is shown in Fig. 3.[1] The computation is carried out for the Great Meiji Sanriku Tsunami at Miyako bay. The computatioal results at 40 minutes after tsunami occurence are presented for the different conditions: i.e. with and without tsunami run-up and effects of structures. The result which is taken both run-up on land and structures into account is apparently different with tsunami velocity distribution from those without consideration.

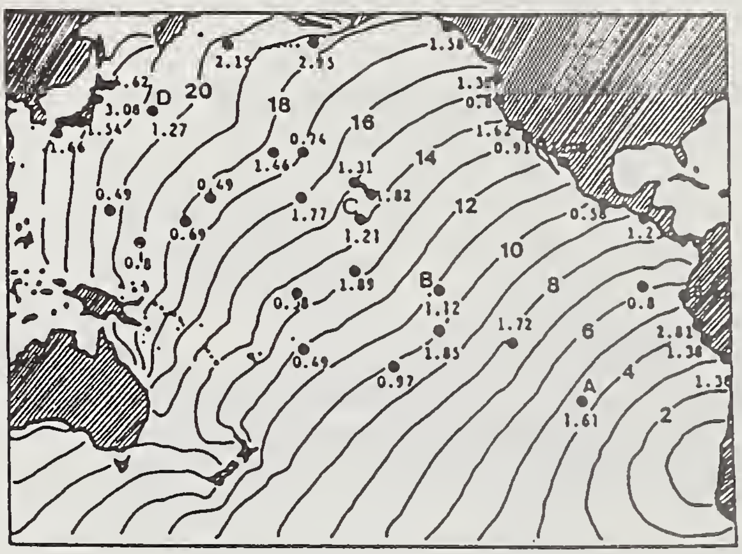
Fugure 4[8] shows a result of computation of the 1983 tsunami occured at Japan Sea. In this figure, tsunami run-up heights and inundation area are compared with records. Since the grid size of the computation is 30 m long and is considered to be fine enough, it seems that the results is quite reliable. Aida[9),10] proposed parameters denoted K and κ to evaluate the accuracy of numerical result as follows;

$$\log K = (1/N) \log K_i \qquad (7)$$

and



(a) Water height distribution at 15 hours after tsunami occurence



(b) Tsunami travel-time curve

Fig. 5 Computation results of the 1960 Chilean Tsunami.

$$\kappa = \sqrt{(\log K_i)^2/N - (\log K)^2}, \qquad (8)$$

where

N : numbers of data,

K_i : ratio of record to numerical result.

The values of K and κ is 0.99 and 1.28 respectively in this computation.

3. RECENT DEVELOPMENTS IN TSUNAMI COMPUTATION

3.1 Distant tsunami

Numerical computation of the transoceanic propagation of tsunamis generated on the Chilean coast to Japan can be carried out by use of a super computer.

Figures 5 and 6[11),12)] show numerical results of the 1960 Chilean Tsunami. The governing

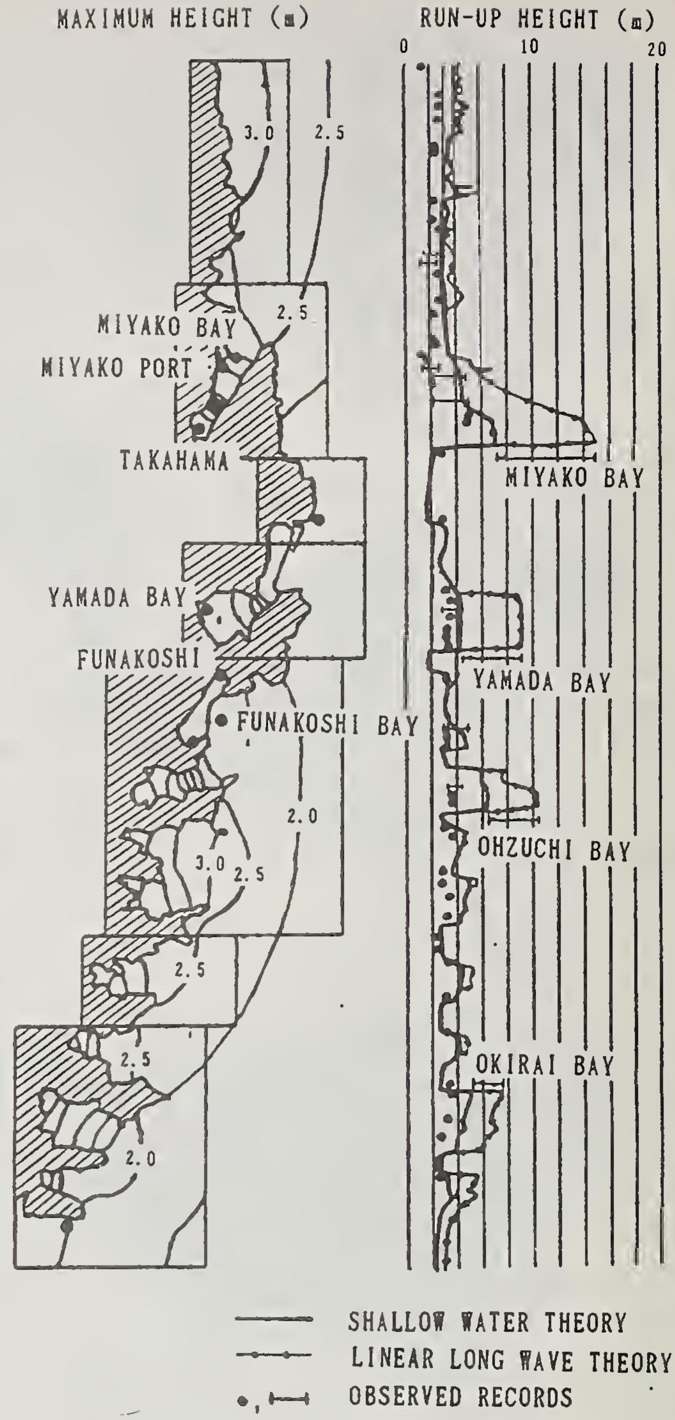


Fig. 6 Computation results of the 1960 Chilean Tsunami (tsunami height along Sanriku Coast).

equations of the computation are the linear Boussinesq equations with Coriolis terms described in the latitude-longitude coordinate system as follows :

$$\frac{\partial \eta}{\partial t} + \frac{1}{R\cos\lambda}\left[\frac{\partial}{\partial \lambda}(M\cos\lambda) + \frac{\partial N}{\partial \psi}\right] = 0 \qquad (9)$$

$$\frac{\partial M}{\partial t} + \frac{gh}{R}\frac{\partial \eta}{\partial \lambda} = -fN + \frac{1}{R}\frac{\partial}{\partial \lambda}\left[\frac{h^3}{3}F\right] \qquad (10)$$

$$\frac{\partial N}{\partial t} + \frac{gh}{R\cos\lambda}\frac{\partial \eta}{\partial \psi} = fM + \frac{1}{R\cos\lambda}\frac{\partial}{\partial \psi}\left[\frac{h^3}{3}F\right] \qquad (11)$$

$$F = \frac{1}{R\cos\lambda}\left[\frac{\partial^2}{\partial t\partial x}(u\cos\lambda) + \frac{\partial^2 v}{\partial t\partial y}\right]$$

where

(λ, ψ) : latitude and longitude coordinates,

f : Colioris factor,

(M, N) : tsunami discharge flux in the (λ, ψ) direction.

In the computation, grid length is set for 1/6 degree, which corresponds to 20 km along the equator.

Figure 5 shows distribution of tsunami height at 15 hours after tsunami occurence and travel-time curves drawn for every 1 hour. The difference between tsunami heights simulated with the linear long wave theory and those simulated with shallow water theory are exhibited along the Sanriku coast in Fig.6. Since the linear long wave theory does not included sea bottom friction terms, the resulting tsunami heights are large compared with observed data. On the other hand, the result with the shallow water theory including wave nonlinearity and bottom friction shows good agreement with observed data. The CPU time to reproduce 30 real hours is 30 minutes with the aid of super computer NEC SX-1.

3.2 Soliton Fission of Tsunami

The Nihonkai-Chubu earthquake which occured in the middle part of Japan Sea in 1983 generated a great tsunami. Many accurate data of run-up heights, inundation areas and tide gauge records were obtained and collected. In addition, soliton fissions as a result of the interaction between the nonlinear and the dispersion effects during the tsunami propagation was observed and recorded at the front of tsunami.

A tsunami formed in a solitary wave train is quite dangerous to protection works because of its rapid growth in height. Detail of the soliton fission of tsunamis should be analized by numerical computation.

Figure 7[13] shows an example of numerical computation for evolution of solitons. The computation is carried out by the nonlinear dispersive long wave theory with bottom friction terms[14]. The result has small numerical oscillations becase coarse grid size is adopted for the limitation of computer memory. Therefore, the simulation does not seem to be carried out relevantly. However we can recognize in the figure that run-up front of the first and the second wave have growing to solitary wave train. As described above, the numerical computation for soliton fission of tsunamis has been developed recently, but there are so many problems related to soliton behavior left unsolved.

3.3 Spread Materials Due to Tsunamis

As for the behavior of materials caused by a tsunami, only a few studies have been

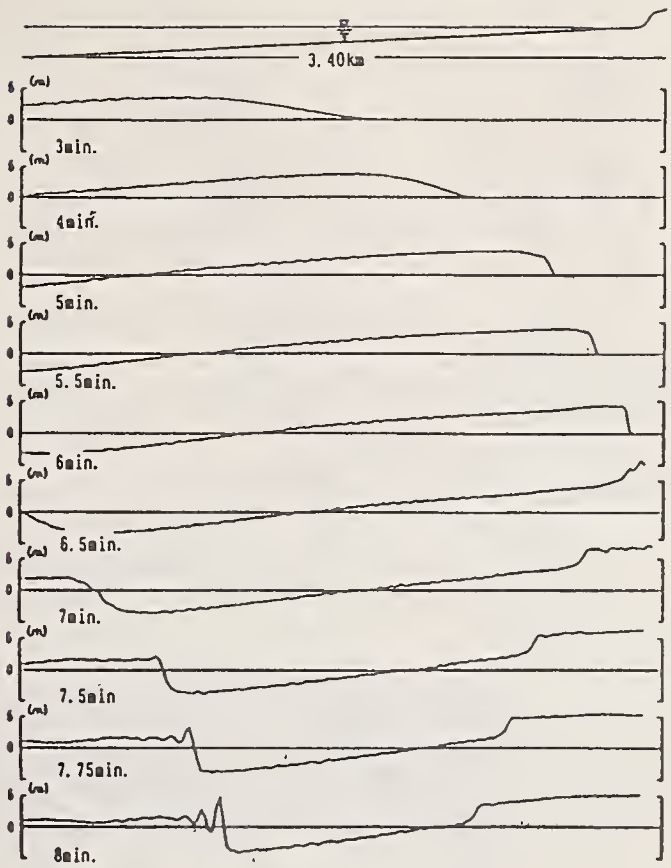


Fig.7 Computation result for tsunamis with soliton fissions.

made. They are limited to the numerical simulation for the spread of timber and oil.

As for the timber spread, it is reported that about 4,000 timbers were floated away from Miyako port on North Sanriku coast when the Tokachi-oki tsunami hit in 1968. Once floated and washed away by tsunamis, timbers often become destructive forces to houses and port facilities.

The simulation model of the timber spread is developed by Goto[15]. The motion of timber consists of two parts, a avereged motion and diffusion. The averaged motion is calculated for the mean current induced by tsunami, on combining the drag and added mass effects of timbers. The diffusion is estimated by a assumption that spread of timbers around their center of gravity follows a random process.

Figure 8 shows a result of practical application for numerical simulation of timber spread. It is assumed that 6,000 timbers are being stored and are washed away by tsunami. In the figure, black points indicate location of timbers at each time.

For oil spread, it is reported that oil tank in Niigata city faced on the Japan Sea was damaged by the 1964 earthquake and oil was discharged and spread over the water.

Fig. 8 Computation results for timber spreading.

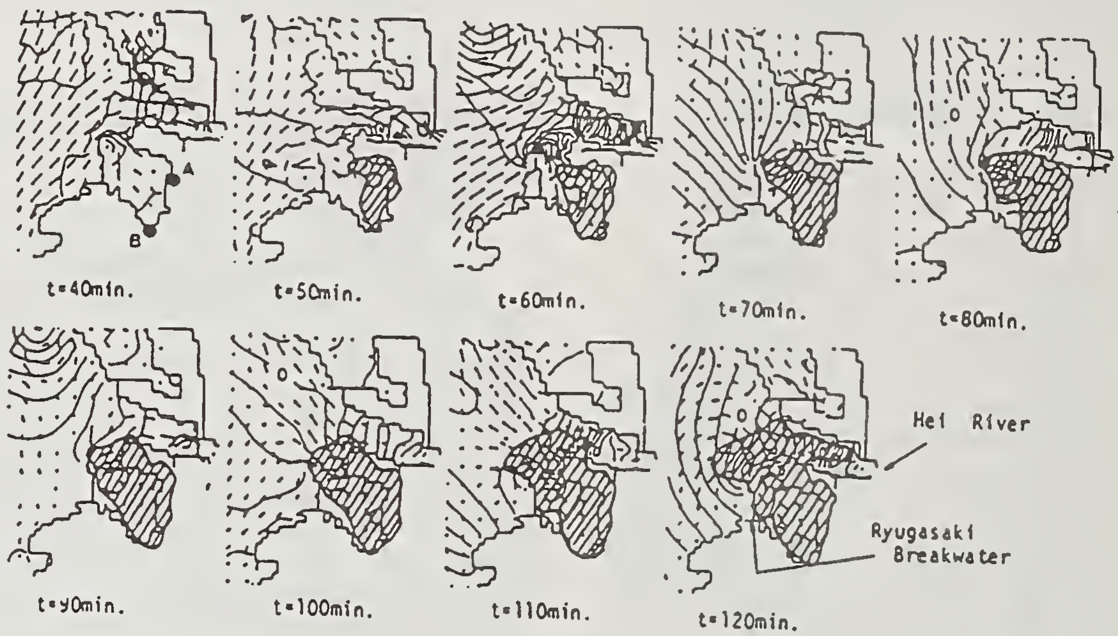


fig. 9 Computation results for oil spreading.

Unfortunately, five houres after the earthquake the oil caught on fire and fire continued 15 days.

The spread of oil by tsunami was first computed by Goto[16]. The method without the capillarity can be applied to the gravity-inertia and gravity-viscous regimes. In a large oil spill, the surface tension becomes important after several days, while a tsunami may continue for several hours. Therefore, the method is sufficient to predict the oil transported by tsunami.

Figure 9 shows an example of the results. There are two tanks at points A and B. Each of tanks contains 1,000 m^3 of oil and is surrounded by dikes of 40 cm high. It is assumed that the tanks are ruptured by an earthquake and leaked oil is stored inside the dikes. When the tsunami water level exceeds the surrounded dikes, oil begins to be floated and transported.

4. NUMERICAL ERROR AND REMARKS FOR COMPUTATION

4.1 Classification of Numerical Error

Once the initial tsunami profile is determined exactly, the accuracy of tsunami numerical simulation is closely related with the governing equations and the grid length used. Especially, aliasing and truncation errors related with grid length is very important because the errors appear as the artificial dispersion or dissipation effects and causes the phase shift or the damping of tsunami height[17),18)].

4.2 Governing Equations

Usually, the long wave theory is used as a model representing the propagation of a tsunami. The theory is derived by the assumption which vertical acceleration of water particle is small as compared with the acceleration of gravity and the relative depth is small. The equation is described the motion of long waves as follows :

(local term) + (pressure term) +(nonlinear term) + (dispersion term) = 0.

If the dominant terms of the equation consists in the first two terms, it is called the linear long wave theory, if three terms, the shallow water theory, and if all terms, the nonlinear dispersive long wave theory.

For a tsunami occurred in the sea near Japan, the dominant terms of the equation can be considered as only a local and a pressure terms. And the nonlinear term is kept small while the tsunami is traveling in deep seas and the dispersion term is known as small in both areas[19)]. For on the reason, the linear long wave theory is used In deep sea, whereas the shallow water theory is used in shallow seas.

In conventional simulations, there are some cases used by the linear long wave theory as governing equations even if in shallow seas. However the accurate wave celerity is faster than ones by the linear long wave theory and is described by the equation as follows :

$$C = \sqrt{gh}[\ 1+ (3/2)(H/h) - (2\pi h/L)^2/6 + \cdots] \qquad (12)$$

Where

C : tsunami celerity,

H : tsunami height,

L : tsunami length.

Therefore, we have to pay attention to the difference of celerities which causes the phase sift and gives serious influences to maximum tsunami height and velocity distribution.

4.3 Grid Length

Aliasing and truncater errors of the numerical computation are very important, which causes calculated tsunami height to be smaller. In order to keep error small, grid length Δx should be proposed to satisfy the following equation,

$$C_0T/\Delta x \geq 30 \qquad (13)$$

where

C_0 : wave celerity of the liner long wave theory,

T : period of tsunami.

Wave celerity is proportional to the square root of sea depth, therefore, the computed grid length should be made gradually smaller as the tsunami approachs shallower.

To obtain more accurate results, we can use of the knoeledge of the artificial dispersion effects introduced by numerical scheme because the effects have the similar to the physical correct dispersion. The condition, that long wave theory without dispersion terms can give the same wave profile computed with the theory with dispersion effects, requires the relation as following :

$$(\Delta x/h)\sqrt{1-(c\Delta t/\Delta x)^2} = 0.5 \qquad (14)$$

Therefore, the grid length of numerical simulation should be determied so that the relation Eqs.(13) and (14) are satisfied.

Even if there are no breaking waves, the boundary condition at the wave front on land has an important influence on the accuracy of the numerical results. Goto and Shuto[19)] proposed a condition related with grid size which are successfully applied to the problem, as following

$$\Delta x/sgT \leq 10^{-4} \qquad (15)$$

where

s : bottom slope from the shore line to the point of 100m deep.

5. CONCLUSION

In the paper, numerical simulation method for tsunami propagation and run-ups is briefly reviewed. Recently, tsunami numerical simulation for run-up on land and trans-oceanic propagation been developed and appried to practical problems. But, even now, the accurate velocity and wave force distributions necessary to calculate satabilities of coastal structures are quite difficult and there are so many problems which remain unsolved.

REFFERENCES

1) Shuto, N., C. GOTO and F. Imamura : Use of numerical simulation as a means of tsunami warning, Proc. of JSCE, Vol.393, pp.181-189, 1985(in Japanese).

2) Hatori, T. : Tsunami sources off northeastern coast of the Pacific Ocean in Japan, Earthquake, Vol.27, 1974(in

Japanese).
3) Houston, J.R. and H.L. Butler : A numerical model for tsunami inundation, U.S.Army Eng. W.E.S., Tech. Rept. HL-79-2, 54p, 1979.
4) Goto, C. and N. Shuto : Effects of large obctacles on tsunami inundation, Terra Scientific Publishing Company, pp.551-525, 1983.
5) Aida, I., H. Tsubokawa and M. kawaguchi : Numerical experiment on behavior of tsunamis exceeding the design height of a sea wall - Case studies for Matsuzaki and Taro -, Earthquake, Vol.41, pp.343-350, 1988(in Japanese).
6) Aida, I : Numerical experiments of inundation of tsunamis, Bull. Earthq. Res. Inst., Vol.53, pp.441-446, 1977(in Japanese).
7) Iwasaki, T and A. Mano : Two-dimensional numerical computation of tsunami run-ups in the Eulerian description, Proc. of 26th Conf. on Coastal Eng., JSCE, pp.70-74, 1979(in Japanese).
8) Hasegawa, K. and K. Inagaki : Numerical simulation of the Nihonkai-Chubu Tsunami, Unic Cop., 1984(in Japanese).
9) Aida, I. : Tsunami source models of the 1983 Nihonkai-Chubu Tsunami, Bull. Earthq. Res. Inst., Vol.59, pp.93-104, 1984(in Japanese).
10) Aida, I. : Numerical simulation of old tsunam is originated off the Sanriku coast, Bull. Earthq. Res. Inst., Vol.56, pp.71-101, 1977(in Japanese).
11) Imamura, F., O. Nagano, C. Goto and N. Shuto : Numerical simulation of the transoceanic propagation of the 1960 Chilean Tsunami, Proc. of 34th Conf. on Coastal Eng., JSCE, pp.172-176, 1987(in Japanese).
12) Goto, C., F. Imamura and N. Shuto : Study on numerical simulation of the trans-oceanic propagation of tsunami Part 1: Governing equation and mesh length, Earthquake, Vol.41, pp.515-526, 1988(in Japanese).
13) Hasegawa, K. and K. Inagaki : Numerical simulation of the Nihonkai-Chubu Tsunami (Part 2), Unic Cop., 1987(in Japanese).
14) Goto, C. : Equations of nonlinear dispersive long wave for a large Ursell number, Proc. of JSCE, Vol.351, pp.193-202, 1984(in Japanese).
15) Goto, C. : A simulation model of timber spread due to tsunamis, Proc. of 30th Conf. on Coastal Eng., JSCE, pp.594-597, 1984(in Japanese).
16) Goto, C. : A simulation model of oil spread due to tsunamis, Proc. of JSCE, Vol.357, pp.217-224, 1985(in Japanese).
17) Imamura, F. and C. Goto : Truncation error in numerical tsunami simulation by the finite difference method, Coastal Eng. in Japan, Vol.31, No.2, pp.245-263, 1988.
18) Sayama, J., F. Imamura and C. Goto and N. Shuto : Study on an accurate numerical simulation method for tsunamis in deep sea, Proc. of 34th Conf. on Coastal Eng., JSCE, pp.177-181, 1987(in Japanese).
19) Goto, C. and N. Shuto : Numerical simulation of tsunami propagations and run-ups, Terra Scientific Publishing Company, pp.439-451, 1983.

Theme IV

U.S.-Japan Cooperative Research Program

U.S.-Japan Coordinated Earthquake Research Program on Masonry Buildings—Seismic Test of Five Story Full Scale Reinforced Masonry Building, Part 2

By

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SUMMARY

In order to investigate the validity of the design method and to know the total behavior of a RM building, static cyclic loading and pseudo dynamic tests of a five story full scale RM building was performed from mid-November, 1987 to the end of January, 1988. The test building in the extended service load range was very stiff with only minor structural distress. It was found that the characteristics of the test building in elastic range are very sensitive to the condition of foundation. The maximum lateral force capacity reached 968 ton, at an overall building drift of 4/800 (0.5%) in the yield load phase test.

Response of the pseudo dynamic test using the Taft EW 1952 (300gal) was within the hysteresis loop obtained from the yield load phase test. The ultimate deformation limit state was marked by rapid strength degradation at an overall building drift of 7/800 (0.875%).

The maximum shear carrying capacity and load-deformation relationship of the test structure was simulated well by a non-linear frame analysis.

KEY WORDS: Reinforced Masonry, Full Scale Test, Static Cyclic Load, Pseudo Dynamic Test, Non-Linear Frame Analysis

1. INTRODUCTION

Under the auspices of the UJNR on Wind and Seismic Effects, both the U.S. and Japan are working since 1984 on a coordinated earthquake research program on masonry structures. The target of the Japanese program is the development of comprehensive design guidelines for medium rise RM structures, in particular, the five story apartment building, to meet the countrie's need of high density residential construction. Based on a detailed understanding of the structural behavior of RM building derived from analytical and experimental research programs on materials, components, sub-assemblages and planar frame structures, the design guidelines (draft) was developed.

The five story full scale prototype test on a reinforced concrete masonry building was planned and carried out in order to experimentally verify analytical models which are assumed and dealt with in the draft of the design guidelines. The main portion of the full scale test consisted of three static cyclic load phases, those were service load phase, yield load phase, and ultimate load phase (see Table 1). The lateral load distribution was derived from the Japanese Building Code. Overall test plan, design and construction, and brief test results were presented at the last (20th) UJNR meeting held in Washington D.C. May 1988 [Ref.1].

In this paper, adding to the review of the test structure and loading method, results of each static cyclic load phase test and pseudo dynamic test carried out just before the ultimate load phase test are discussed. Pseudo dynamic test was performed in order to simulate the response of the damaged building due to the secondary shock after the main shock of an earthquake. Finally, large amplitude of deformation was applied to the test structure in order to obtain the deformation capacity of the test structure and the deformation characteristics of the structural elements in the ultimate state.

This paper also discusses on the reason why there is much difference between ultimate capacity calculated by the design guidelines and the one obtained from the test, as well as on the elastic behavior by mean of frame model analysis. A frame model analysis was applied because it is very simple and very convenient for current design method.

2.GEOMETRY AND LOADING OF TEST STRUCTURE

Even though detailed reference on the geometry and loading of the five story test building should be made to [Ref.1], a brief iteration of the most important geometrical data, load application and reference notation is presented in Fig.1 to allow quick cross references.

The five story full scale test specimen represented a module of a prototype apartment building with four parallel load bearing frames with openings (Frames Y1 through Y4, Fig. 1) in the critical loading direction under investigation. The floor area is 13.79m (loading direction) x 15.19m (transverse direction, including 2.4m wide balcony) and the total building height is 14m from the top of foundation to the roof slab (each story is 2.8m in height). The standard and corner concrete units used in the test building are shown in Fig.2. Running bond was used in the test building. Both vertical and horizontal rebars were arranged in 20cm spacing module basically. Wall length ratios are $15.20cm/m^2$ in the loading direction, and $23.30cm/m^2$ in the transverse direction,

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respectively. If the test building consists of 8 dwelling units per floor plan (4 of the test building modules side by side), those values are 14.89cm/m^2 and 18.15cm/m^2 in each direction, respectively.

Loads were applied to the center line (X2) of the test structure by means of 11 servo-controlled hydraulic actuators depicted schematically in Fig. 1(a). For the cyclic load testing, floor level loads were applied in forced control as a fraction of the roof level force as stipulated by the Japanese Building Code. Roof level loading was under displacement control to eliminate the torsional mode. For the pseudo dynamic seismic simulation test the five-story displacements were applied to one of the actuators at each floor level while the other servo-controlled actuator was force slaved to the first. Only at the roof level, where three actuators applied the lateral loading, see Fig.1(a), the two outside actuators were driven in displacement control again to eliminate the fundamental torsional mode.

Wall and beam/girder components were denominated by W or G, respectively, followed by a number and letter sequence indicating the plan location. Preceding digits 1 to 5 denote the story level under consideration. Thus, e.g., $_1W_{1A}$ denotes the first story wall labeled 1A at the lower left hand corner in Fig.1(b) at the intersection of reference lines Y1 and X1, etc.

3. SERVICE LOAD PHASE RESPONSE [Ref.2]

Crack pattern development in selected frames in loading direction is shown in Figs. 3 and 4. Base shear versus displacement relationship obtained through static cyclic loading tests is shown in Fig. 5 and corresponding roof level displacement envelope is shown in Fig. 6 together with indications of the major events detected during loading.

The service load phase covers the loading up to a nominal base shear stress level of 8.0kg/cm^2 which is approximately twice the nominal value corresponding to the 0.2G service load level required by the Japanese Building Code.

Up to the loading of <u>4.0kg/cm^2</u> nominal base shear stress level, the test structure showed very little distress and a very stiff overall response (The first story stiffness was 3,237ton/cm). Very small flexural cracks in the lintel beams and a slight opening of horizontal bed joint cracks in first and second story wall elements were observed at this loading stage. Also, cracking at slab corner started from this load level. The response of the test building up to this load level can be classified as virtually linear elastic.

During the loading between 4.0kg/cm^2 and <u>8.0kg/cm^2</u> nominal base shear stress level, flexural cracks at the horizontal bed joints of walls and the lintel beams in all stories developed, as shown in Fig. 3. Very small diagonal cracks starting from a corner of small rectangular openings located at the bottom of first and second story wall elements $_1W_{4A}$ and $_2W_{4A}$ started to develop. Cracks at floor slabs, perpendicular to the loading direction, started to open due to flexural deformation of beams concerned. Almost no cracks were observed in the orthogonal walls X_1 to X_3.

The first yield in any instrumented reinforcement was recorded in the flexural reinforcements of long wall elements $_1W_5$ and $_1W_8$, and in the building corner reinforcement in $_1W_{9B}$, all at 8.0kg/cm^2 nominal base shear stress level, as shown in Fig. 7.

The first story secant stiffness at this stress level decreased in 2,417ton/cm from initial elastic stiffness, 3,237ton/cm.

4. YIELD LOAD PHASE RESPONSE [Ref.2]

<u>Crack Development and Reinforcement Yield Development</u>

During the loading up to an overall building drift angle of <u>1/1,200</u> (0.083%), wall element $_1W_{4B}$ and a joint element located above the first story wall $_1W_{2B}$ exhibited diagonal cracking. At the loading of 9.0kg/cm^2 nominal base shear stress level, horizontal bed joint cracks of transverse wall elements were detected in the first story. Figure 7 shows flexural reinforcement yield development up to a building drift angle of 2/800 (0.25%). According to this figure, the reinforcements in all first story wall elements along the base of the building yielded up to a building drift angle of 2/800 (0.25%). The flexural reinforcement arranged at lower side of most beam elements also yielded until this deformation level. The flexural reinforcements arranged at upper side of all beam elements except $_2G_{7B}$ and $_2G_{6B}$ did not yield due to existence of RC floor slab.

At <u>2/800</u> (0.25%) drift angle load level, the first diagonal cracks occurred in most of all wall elements in the first story and also in the second story long wall element $_2W_8$. The stiffness of the building significantly dropped at this deformation level due to this diagonal cracking as seen in Fig. 6. The first diagonal cracking in a beam element also was detected in the second story short beam $_2G_{5B}$. Remarkable diagonal cracking from a corner of a small rectangular opening located at the bottom of the first story wall element $_1W_{4A}$ occurred, and radiated cracking in floor slabs adjacent to wall edges (including straight cracking along with a beam face) occurred as well at this deformation level.

At a deformation level of <u>3/800</u> (0.375%), diagonal cracks in first story wall elements increased in number. Face shell spalling in the second story short beam $_2G_{5B}$ as well as toe crushing of the long wall elements $_1W_5$ and $_1W_8$, which indicate the onset of major structural deterioration, started at this deformation level. In the first story orthogonal walls, vertical cracking was detected at the place distant 1.0-1.2 meters from the walls arranged in the loading direction, which is considered to be produced by excessive deflection of the walls in the loading direction. Lateral load still increased up to a deformation level of 4/800 (0.5%) drift angle at which the maximum lateral load (base shear) 968tons was recorded. This almost corresponds to the total building weight, 996tons.

Change in Stiffness and Deflection Mode

The two cycles service load level loading was held after the service load phase and the yield load phase responses in addition to the first service load level loading as shown in Table 1. Table 2 shows story stiffnesses of the test building under various loading level. It is clearly understood from this table that all the story stiffnesses changed at almost same rate of stiffness degradation. This also indicates that the building was considerably damaged under the yield load phase response up to a deformation level of 4/800 (0.5%) drift angle.

A plot of story drift percentage for each story, shown in Fig. 8, indicates that the deflection mode did not significantly change through cyclic loading up to the maximum lateral load level. One can just notice that the first story drift percentage is slightly increasing with increasing building drift.

Story rotation of the long wall element W_5 is depicted in Fig. 9. It is clearly noticed that flexural deflection is definitely caused by the rotation at the foot of the first story wall element ${}_1W_5$ and that, among three stories above the 3rd story, the sign of the story rotation is just opposite to the corresponding one in the first story, even the magnitude of story rotation at upper three stories is small. The fact mentioned in the latter suggests existence of considerable flexural reaction by beam elements in these stories. The shear and flexural deflection percentages in the total deflection of the first story wall element ${}_1W_5$ are 40% and 60% respectively through the building deformation level of the drift angles of 1/800 (0.125%)- 6/800 (0.75%).

The shear deflection versus story deflection relationship in the first story wall elements ${}_1W_{1B}$, ${}_1W_{2B}$ and ${}_1W_3$, depicted in Fig. 10, also clarifies that the shear deflection is, roughly speaking, 50% or a little less in the total story deflection up to the building deformation level of 4/800 (0.5%) drift angle.

Yield Hinge Development

The consecutive yield hinge development is depicted in Fig. 11. This almost coincides with reinforcement yield development as shown in Fig. 7. The difference between them is due to two kinds of difinition of yielding, that is, reinforcement yielding and structural element flexural yielding, as schematically shown in Fig. 12. The point 1 in Fig. 12 just corresponds to a reinforcement steel yielding point (2,000 micro strain in this case), whereas the point 2 was determined as a point where sudden change in slope on stress-strain relationship was observed, at which the structural element concerned is considered to have yielded.

Based upon Fig. 11, hinges were made at feet of most wall elements (65% of total number of hinges which were made finally) in the first story, and at second and third story short beams, ${}_2G_{5B}$ and ${}_3G_{5B}$, until a building drift angle reached 1/800 (0.125%). Among those hinges, the ones at feet of two first story long wall elements, ${}_1W_5$ and ${}_1W_8$, were first formed at 1/1,200 (0.083%) building drift angle. Up to 2/800 (0.25%) building drift angle, all the first story wall elements and some second story ones yielded at their feet. Beam elements, almost 60% of all, also yielded at their lower side throughout the entire building.

Up to 3/800 (0.375%) building drift angle, 64 - 66% of total number of hinges were made in wall and beam elements arranged in frames Y1 and Y4, whereas 80% in those arranged in frames Y2 and Y3 in which long wall elements were arranged.

During the loading up to 4/800 (0.5%) building drift angle, another yield hinges were newly made mostly at those elements arranged in frames Y1 and Y4 so that hinges more than 80% were formed in these frames until this drift angle. Most of all beam elements (83%) yielded at their lower side by then. Upper side of beam elements did not yield throughout the testing except second and fourth story beam elements in frame Y1 and two second story beam elements in frame Y4. This is considered to be caused by existence of RC floor slab reinforcements.

Strain Distribution in Reinforcing Bars

Many strain gages were attached to surface of reinforcing bars vertically and horizontally arranged in walls, beams and RC floor slabs to measure effectiveness of those bars especially arranged in the orthogonal elements. All the vertical reinforcements in the frame X_1 yielded at upper surface of building foundation(line H_1), when a building deformation level reached 1/1,200 (0.083%) drift angle, and also yielded at line H_2, 90 centimeters above the line H_1, under 1/800 (0.125%) drift angle loading. The first story vertical reinforcements in the interior frame X_2 yielded under relatively large deformation as compared with those in the exterior frame X_1.

All those reinforcements in the frames X_1 and X_2 yielded at floor slab surface seimaltaneously when flexural reinforcements arranged in concerned wall elements in the loading direction yielded. The yielding of vertical reinforcements arranged in the walls which are located perpendicular to the loading direction, thus clearly showed effectiveness of all those reinforcements against lateral loading.

Similar tendency was observed on floor slab reinforcement associated with beam flexural reinforcement. However, strain level through all the measurements of slab reinforcements were very low as compared with the one in vertical reinforcements arranged in orthogonal walls, such as the frame X_1.

5. PSEUDO DYNAMIC TEST [Ref.3]

The objective of the subsequent five-degree-of-freedom pseudo dynamic test was twofold, namely to investigate the response characteristics, i.e., natural frequency, mode of response and lateral force distribution, of the test structure after damage against 1.0G level lateral load accumulated, and to obtain the dynamic response of the damaged building to an after-shock or a subsequent seismic event with a 0.3G maximum acceleration.

The earthquake record of the Taft EW 1952 with a time window from 2.68 to 8.98 seconds was selected as the driving seismic motion for the pseudo dynamic test, since the first mode response was dominant with virtually equal amplitudes in both the positive and negative directions under preliminary computer analysis using the record.

The pseudo dynamic floor level response of the damaged test structure is depicted in Fig. 13 for the roof floor level displacement and base shear force time histories. After 6.3 seconds passed in pseudo dynamic response, the free vibration response produced a natural period of vibration of 0.54 second, slightly longer than the measured forced vibration response (0.41 sec.) after the yield load phase test.

Visual inspection of the test structure, i.e., trace of crack propagation and damage area assessment, did not indicate any additional structural damage or stiffness deterioration during the 300gals secondary seismic event. The obtained hysteretic response (Fig. 14) to the secondary seismic event stayed well within the hysteresis envelope obtained under the yield load phase test. Figure 14 again indicates that the test structure can still perform in a stable manner without increased loss of structural integrity.

Finally, Fig. 15 depicts the variation of peak response of the test structure along the height. The dominance of the first mode response is clearly visible as well as the limiting response envelope obtained at the ultimate lateral load limit state during the cyclic testing of the yield load phase. Of particular interest is the shape of the lateral load distribution over the building height, Fig. 15a, which is almost linear (inverse triangular) as opposed to the shape of the imposed lateral force distribution for the cyclic load testing which was based on the Japanese Building Code. The straight line inverse triangular response is indicative of the formation of flexural mechanisms at the base of the first story wall elements. This tendency is emphasized by almost straight line lateral displacement distributions along the building height, see Fig. 15b, compared to the displacement envelopes obtained during the cyclic yield load phase test.

6. ULTIMATE LOAD PHASE RESPONSE [Ref.3]

Subsequent to the maximum lateral load limit test and the pseudo dynamic seismic test, the test structure was subjected to increased cyclic lateral deformation levels, beyond the 4/800 (0.5%) building drift at maximum load, to investigate the strength degradation characteristics and to evaluate the ultimate failure mechanism. Overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and 7/800 (0.875%) were imposed in single load cycle with full reversal. At a total building drift of 7/800 (0.875%) the ultimate deformation load test was terminated since a lateral load carrying capacity of less than 50% of the maximum lateral load level was obtained on the reverse deformation cycle.

The base shear versus drift relationship for the overall building (roof level) and the first story level are depicted in Fig. 5. In the initial deformation cycle load levels of 98%, 90% and 69% of the maximum lateral load were achieved at overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and the 7/800 (0.875%), respectively. A comparison between the overall building drift and the first story drift shows first story drift levels of 1/102 (0.98%), 1/70 (1.43%) and 1/42 (2.38%) for the corresponding overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and 7/800 (0.875%). The cyclic ultimate deformation load test showed that 80% of the maximum lateral load capacity was still maintained at overall building drift of 1/120 (0.83%) and first story drift of 1/60 (1.67%).

The lateral deformation characteristics of the test building over the building height is depicted in Figs. 16 and 17. Individual story displacements and story drifts are plotted over the height of the test structure for various overall building drift levels in Figs. 16a and 16b, respectively. A significant increase in first story deformation and drift for overall building drift levels of 6/800 (0.75%) and 7/800 (0.875%) can be seen, with very little additional deformations in the upper stories two to five. The rapid increase in story drift without an increased rotational component is a clear sign for shear deformations and the development of a shear mechanism in the first story walls. This tendency is also depicted by the overall deformation modes for lateral load bearing frame Y2 in Fig. 17, where deformation states at 3/800 (0.375%) and 7/800 (0.875%) radians are compared. The additional upper story deformations are negligible compared with the first story displacements.

The above findings are confirmed by large diagonal cracks in all first story walls as depicted by the ultimate crack pattern of all four load bearing frames in Fig. 18. The opening of these diagonal cracks in the first story wall components occurred at deformation levels of 5/800 (0.625%) and 6/800 (0.75%) building drift, which corresponds to the increased lateral deformations in the first story at these load levels. Few of the cracks in the upper stories as well as the cracks in the floor slabs and in the transverse walls extended or widened during the ultimate deformation load phase test. Thus, the overall deformation mechanism encountered in the test structure clearly shifted from an overall flexural behavior up to and at the ultimate load limit state (5/800 (0.625%) building drift) to a shear dominated mechanism in the first story during the ultimate deformation load phase test.

7. FRAME ANALYSIS [Refs. 4 and 5]

Model

The test building is idealized to be frames composed of beam members representing the wall and beam components. In order to represent the nonlinearlity of members, walls and beams, one-component model consisting of flexural spring at the both edges of a member and a shear spring at the center of a member is considered. Beam model has rigid zones at both ends. Flexural spring characteristics representing flexural behavior and shear spring characteristics representing shear behavior of a member are modeled to be a tri-linear one with cracking and yielding points.

For flexural spring, yield rotation angle(τ_y) is assumed to be 1/800 for 1m long wall and beam, 1/1,500 for 2m long wall and 1/3,000 for 4m long wall. Maximum flexural strength is calculated by the equation in Ref. 7. Those values are listed in Table 3. For shear spring, nominal shear cracking stress is assumed to be $15kg/cm^2$. Therefore shear cracking deformation angle(γ_c) becomes almost 1/4,000.

The value of Young's Modulus (E) is defined from the prism compressive test results. In this analysis mean values of E and F_m are $196t/cm^2$ and $196kg/cm^2$, respectively. Shear Modulus (G) is gotten assuming that poisson's ratio(ν) is 0.2 ($G=86t/cm^2$).

The joint part of wall and beam is treated as rigid zone in this analysis. In the elastic analysis this rigid zone is assumed as illustrated in Fig. 19 and in the inelastic analysis full part of beam-wall joint is assumed to be rigid zone. The effective width of transverse members for elastic stiffness are determined by Ref. 7.

Typical component and subassemblage test can be traced with this beam model and the validity of an inelastic beam (member) model used in this paper is verified [Refs. 4 and 5].

Foundation

Test building is fixed to testing floor (depth = 2.00m). Test building is too rigid to consider the foundation be fixed. In this analysis foundation beam ($I=121.67x10^5cm^4$, $A=10,575cm^2$) and testing floor ($I=600x10^5cm^4$, $A=18,000cm^2$) and the rotational stiffness (see Table 3 c) due to prestressing bars are considered.

Results of Elastic Stage

Four cases listed in Table 4 are analized. Among Model 1 through Model 3, different effective width of transverse members to inertia moment is considered. In Model 1, rectangular section without taking account of transverse members is considered. In Model 2, the full width of transverse members are taken into each member's inertia moment. The inertia moment of Model 3 is defined according to the design guidelines [Ref.7]. The design guidelines recommend that incremental factor of inertia moment due to transverse members is up to 2.0, and inertia moment of almost all members with transverse members are limited to be 2.0 times as that of rectangular section (I_o). These three cases, Model 1 through Model 3, are assumed to be under fixed end condition. In Model 4 the effect of the foundation is considered, other conditions besides the foundation are the same as corresponding ones in the Model 3.

The vibration periods are tabulated in Table 5. Experimental results are also listed in this table. Experimental results are obtained from the flexural matrix gotten by the each floor unit load test. Model 4 gives the closest results to the test ones among these Models. The limitation of effective inertia moment up to $2*I_o$, is reasonable. The recommendation for effective width of transverse members in the design guidelines is also reasonable. Vibration modes between Model 3 and Model 4, are different each other. It clearly shows the necessity to consider the effect of foundation condition in elastic stage.

Overall Behavior in Inelastic Stage

Monotonic loading analysis is carried out. Inertia moment and foundation condition are the same as the Model 4. In this case, however, full part of the panel zone surrounded with walls and beams is taken into account as a rigid zone. This is based on the fact that the purpose of this analysis is to predict the lateral load capacity of the test building from the component strength at their critical section, and also that the stiffness degradation of each member is so dominant to the total stiffness of the test building that rigid zone of wall-beam panel is assumed not to affect on the total stiffness of the test structure. Maximum moment capacity of each member is calculated with considering the effect of all re-bars in transverse members.

Figure 20 shows the each story shear force vs. story drift angle relationship. Good agreements between test results and this analysis are observed. This means that the effect of re-bars in transverse member on flexural strength and characteristics of flexural and shear springs are evaluated properly in this analysis. These spring characteristics are defined based upon the component test results conducted in past years.

Development of Yield Hinges

Test result shows that the yielding of wall W_5 occurred at first in the nominal shear stress of $8kg/cm^2$ loading step. But it means that the extreme edge re-bar reached its yield strain at this loading level. The re-bars' strains in middle part of wall W_5 increased rapidly from building drift angles of 1/800-2/800 (0.125%-0.25%). This fact matches well to the analytical results.

8. CONCLUSIONS

Conclusions obtained from the seismic test on the five story full scale reinforced masonry building are listed as follows:

Service and Yield Load Phase Tests

1. The test building showed very little distress and a very stiff overall response against the loading of 4.0kg/cm^2 nominal base shear stress level which corresponds to the 0.2G design load level. The first story drift angle of the test building against the 0.2G design load level was 1/4,530. Almost no crack was observed under this design load level loading. The test building certainly exceeded design requirements of carrying lateral load capacity corresponding to 0.5G base shear which is specified in a draft of a seismic design guidelines for medium rise RM buildings.

2. The actual lateral load capacity obtained during the test at an overall building drift angle of 4/800 (0.5%) was 968tons which is almost equivalent to the weight of the test building (dead load + live load) or a base shear coefficient of 1.0G.

3. The overall development of failure mechanisms was governed by formation of flexural yield hinges at the base of the first story walls, especially walls W_3, W_5 and W_8, and also at the beam ends throughout the entire building. All first story walls clearly yielded in flexure long before the development of major diagonal cracks which ultimately dominated the first story wall limit behavior.

4. Significant portions of the transverse walls contributed to the overall load transfer in the form of wide flanges to the four load bearing frames. Activation of all the transverse wall reinforcement certainly contributed to the high maximum lateral load capacity. Large contribution of floor slab to beam strength can be considered from the observed transverse crack patterns in the reinforced concrete floors and also from strain distribution in the floor slab reinforcements associated with flexural reinforcements in the beams.

Pseudo Dynamic Test

5. In spite of the comparatively large magnitude of the input acceleration, 300 gals, response of the specimen was within the hysteresis loop obtained from the yield load phase test, and there was no progress in damage of the specimen. The lateral force distribution was more similar to inverted triangular distribution than that imposed in the static loading test, and also the deflection mode was nearly linear compared with the one in the yield load phase test. The natural period of 0.54sec. was obtained from the free vibration by means of the pseudo dynamic test method.

Ultimate Load Phase Test

6. Over 5/800 (0.625%) building drift angle, lateral load capacity decreased gradually and deflection started to concentrate to the first story. Flexural deflection mode was changed to shear failure mode which was dominated by only the first story displacement. After 6/800 (0.75%) building drift angle, shear cracks at the first story walls expanded and lateral load capacity of the test building decreased rapidly with concentration of deflection to the first story. The overall building drift angle was approximately 1/120 (0.83%) when the strength decreased to 80% of the maximum strength.

Frame Analysis

7. Total behavior of the test building was simulated by a non-linear frame analysis. In this analysis, wall and beam characteristics are assumed based on a component test and simple beam theory. The elastic analysis showed that rigid zone recommended in the RM design guidelines is reasonable. And the ultimate analysis also showed that the lateral load capacity of the test building can be estimated by the flexural strength at critical section of each component. It is also important for discussing the elastic behavior of such a rigid structure to consider the condition of foundation. And in inelastic stage, all the reinforcing bars in the transverse members are effective on the ultimate shear carrying capacity.

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The writers would like to record the generous cooperation of Dr. H. Mizuno, Messrs H. Kato, H. Isoishi, T. Nishi, and M. Matsuno for performing the full-scale RM building test.

REFERENCES

[1] S. Okamoto, Y. Yamazaki, T. Kaminosono, and M. Teshigawara, "Seismic Test of the Five Story Full-Scale Reinforced Masonry Building", 20th Panel Meeting on Wind and Seismic Effects, UJNR Cooperative Research Program, Washington D.C., May 17-20, 1988.

[2] Y. Yamazaki, et al., "Seismic Test of Five

story Full Scale Reinforced Masonry Building-Outline of Test and, Service and Yield Phase Response", Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October, 1988.

[3] T. Kaminosono, et al., "Seismic Test of the Five-Story Full-Scale Reinforced Masonry Building-Pseudo Dynamic Test and Ultimate Phase Response-, "Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October 1988.

[4] M. Teshigawara, "Introduction of Dynamic Inelastic Frame Analysis on Reinforced Masonry Structures," Proceedings of the Second U.S.-Japan Joint Technical Coordinating Committee Meeting on Masonry Research, Keystone, USA, 1986.

[5] M. Teshigawara, T. Kaminosono, Y. Yamazaki and T. Nishi," Seismic Test of Five Story Full Scale Reinforced Masonry Building- Analyses on Service and Yield Phase Response-", Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October, 1988.

[6] "Design Standard for Reinforced Concrete Building," Architectural Institute of Japan.

[7] T. Kaminosono, et al., "Draft of Design Guidelines for Reinforced Masonry Buildings," Proceedings of the Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, Tomamu, Hokkaido, Japan, October 1987.

Table 1 Test and Loading Sequence

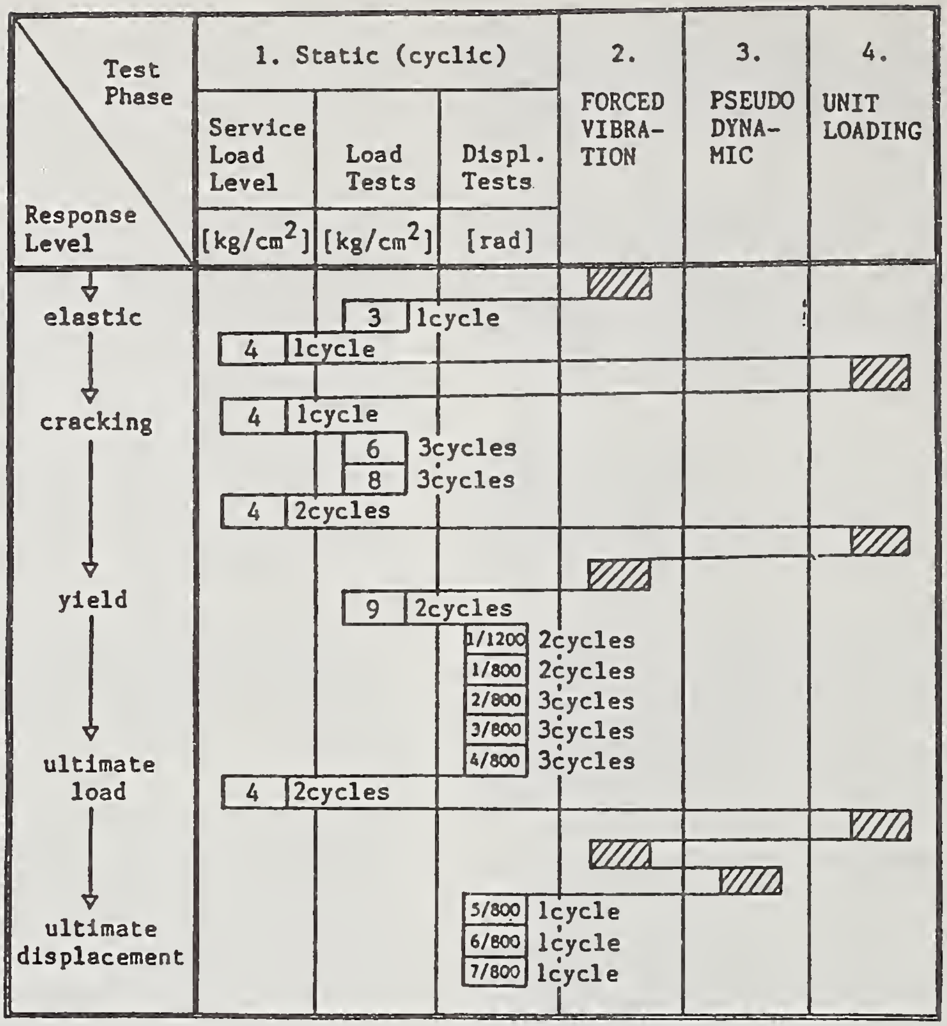


Table 2 Story Stiffness of 5 Story Full Scale Test Building

[ton/cm]

story	$4kg/cm^2$ Stress Level			$8kg/cm^2$ Stress Level	1/800 building Drift Angle Level
	Virginal Loading	After Service Load Phase	After Yield Load Phase		
5	1975 (100)	1660 (84.2)	230 (11.6)	1641 (83.1)	844 (42.7)
4	2520 (100)	1906 (75.6)	340 (13.5)	1978 (78.5)	1240 (49.2)
3	2763 (100)	2158 (78.1)	363 (13.1)	2180 (78.9)	1329 (48.1)
2	3237 (100)	2401 (74.2)	394 (12.2)	2464 (76.1)	1525 (47.1)
1	3237 (100)	2392 (73.9)	291 (9.0)	2417 (74.7)	1554 (48.0)

Table 3 a) Characteristics of Member-Beam-

Member	My_1	My_2
${}_RG_3$	889	9,150
${}_5G_3$	1,228	9,589
${}_4G_3$	1,355	9,940
${}_{3,2}G_3$	1,566	9,940

My_1: Ultimate moment when lower side is in tention [t.cm]
My_2: Ultimate moment when upper side is in tention [t.cm], including the re-bars in slab (28D10 and 8D13)
Mc : Flexural cracking moment = My/3
$I = 15.6*10^5 cm^2$
$I < 2*I_o$, $I_o = bD^3/12$ $(7.8*10^5 cm^4)$
$\tau y = 1/800$ rad.
A : Area for shear stiffness = $1{,}501 cm^2*1.2$, 1.2:effect of RC slab
Qc= Strength of shear cracking = $15 kg/cm^2*A = 27.0$ ton
Foundation Beam. $I=721.67*10^5 cm$, $A=28{,}575 cm^2$

Table 3 b) Characteristics of Member-Wall-

Member		My	I,Qc, y,A
	5	10,544	
	4	8,178	$I=249.55*10^5$ cm^4
W_{4A}	3	8,781	Qc=68.0 ton
	2	6,784	$\tau y=1/1{,}500$
	1	4,192	$A=3{,}781$ cm^2
	5	31,301	
	4	34,997	$I=1164.0*10^5$
W_5	3	43,395	Qc=143.3
	2	47,091	$\tau y=1/3{,}000$
	1	50,787	A=7961
	5	7,559	
	4	11,555	$I=249.55*10^5$
W_{4B}	3	16,200	Qc=68.0
	2	20,510	$\tau y=1/1{,}500$
	1	21,058	A=3,781

My : Ultimate moment [t.cm]
$Ts = 15 kg/cm^2$
Qc = Ts*A*1.2. 1.2 : Effect of transverse wall
A : Area for shear stiffness and strength

Table 3 c) Characteristics of Rocking

Location	K_T(t.cm)
2m length wall	$420*10^4$
4m length wall	$1,700*10^4$

K_T=2* 32*Es*D^2/1=105D^2
Es=2,100t/cm_2 32=8cm_2 1:length of PC-bar(320cm)
D:wall length

Table 4. Analytical Condition of Model 1-4

	Model 1	Model 2	Model 3	Model 4
Transverse member	Not considered	Full width considered	According to the RM D.G[7]	Same as the Model 3
Foundation	Not considered (Fix)	Not considered (Fix)	Not considered (Fix)	Considered
Shear area	Rectangular only			

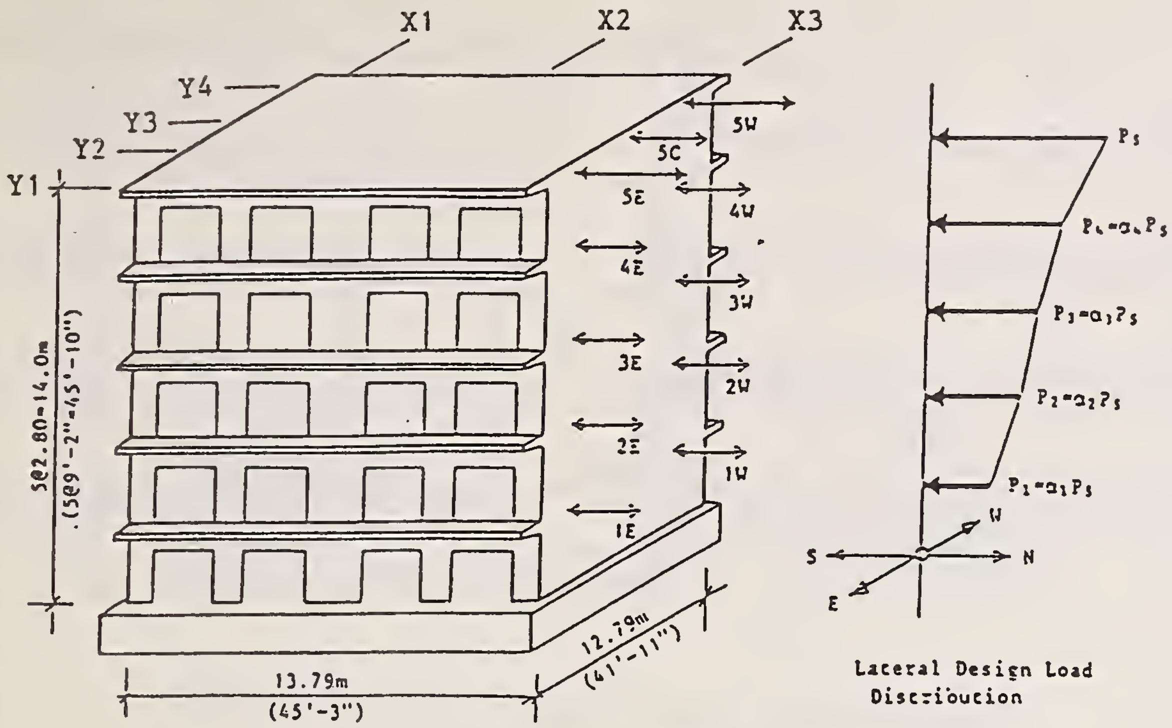
E = 196 ton/cm^2 G = 86 ton/cm^2

In the elastic analysis for Model-1 to Model-4, rigid zone of wall-beam joint is assumed as shown in Fig. 19 (according to A.I.J. standard for RC structure [Ref.6]).

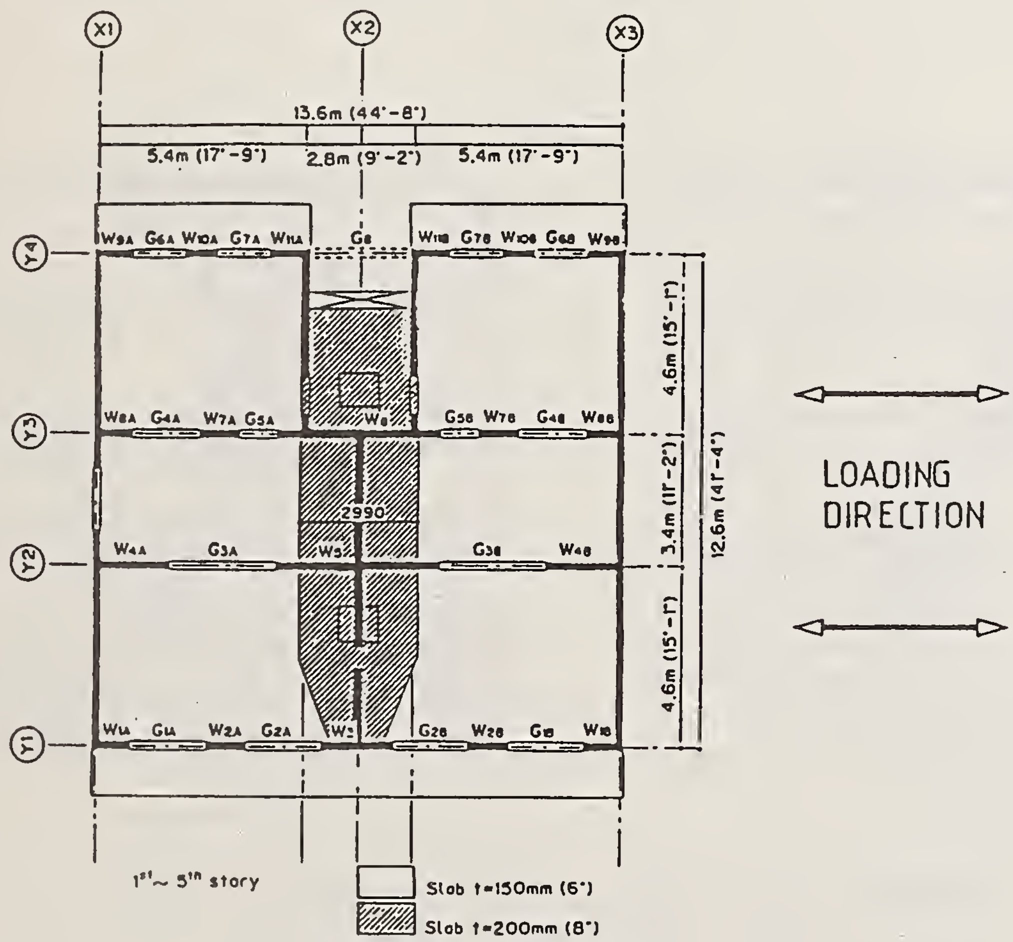
Table 5 Natural Period (sec.)

MODE	Model					TEST
	M 1	M 2	M 3	M 4	M 5	
1	0.184	0.12	0.159	0.169	0.166	0.157
2	0.055	0.039	0.048	0.052	0.051	0.049
3	0.029	0.022	0.026	0.028	0.027	0.026
4	0.020	0.016	0.018	0.019	0.019	0.019
5	0.016	0.013	0.015	0.015	0.016	0.013
	FIX END CONDITION					

* note: It is very difficult to evaluate the proper rotational stiffness of foundation because of its pre-stressed PC bars. Model 4 ignores the effect of this pre-stressed. In supplementary analysis, Model 5, rotational spring due to pre-stressed PC bars was changed to be 1,000 times rigid as compared with Model 4. Natural period does not changed so much(T1=0.169 sec in Model 4, T1=0.166 sec. in Model 5). And the first vibration mode of Model 5 closes to the test results than that of Model 4. It is supposed that rotational stiffness assumed in Model 4 would be enough rigid.



a) Test Setup and Load Distribution



b) Plan Geometry and Reference Notation

Fig.1 Geometry and Loading of Test Structure

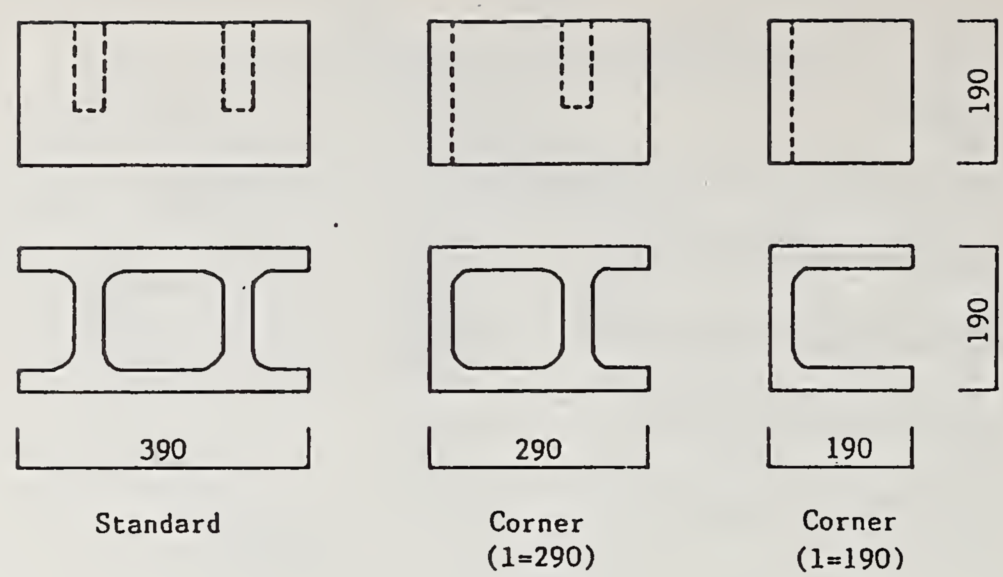


Fig.2 Concrete Block Units [scale in mm]

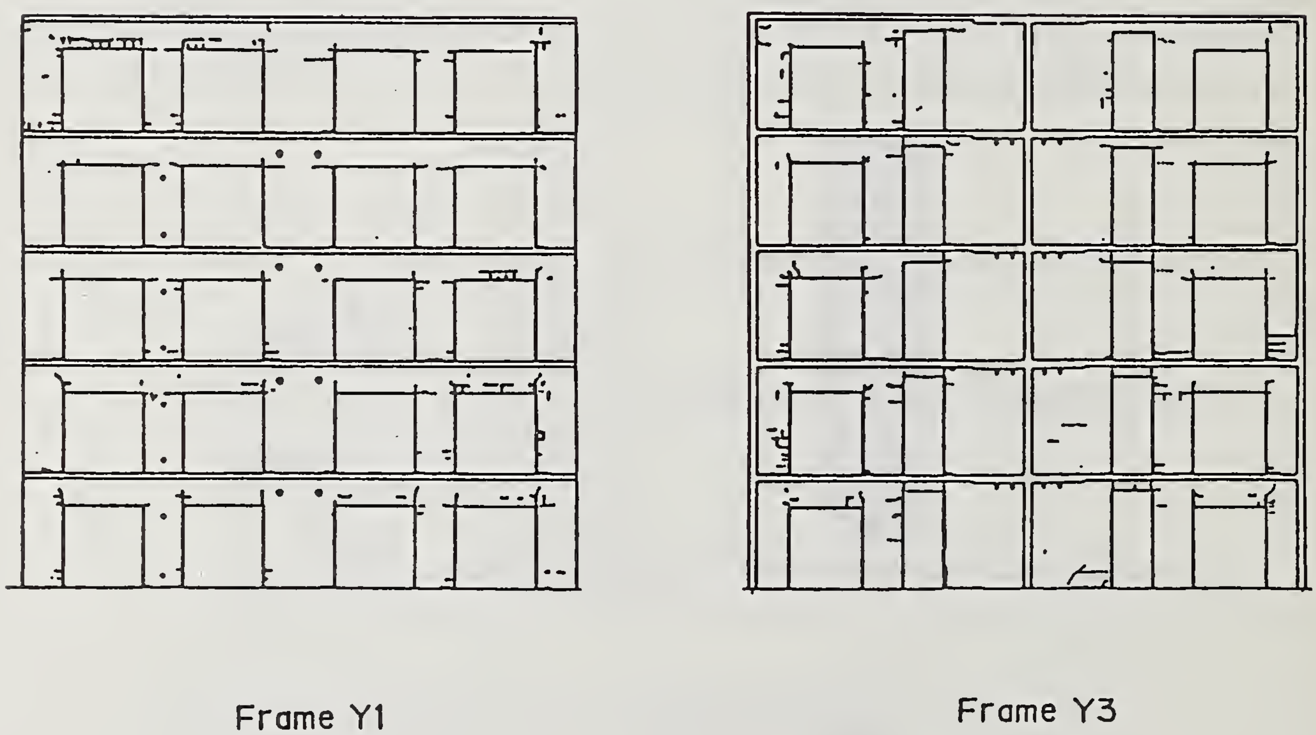


Fig.3 Crack Pattern of Frames -Y1 and -Y3 at τ=8kg/cm^2

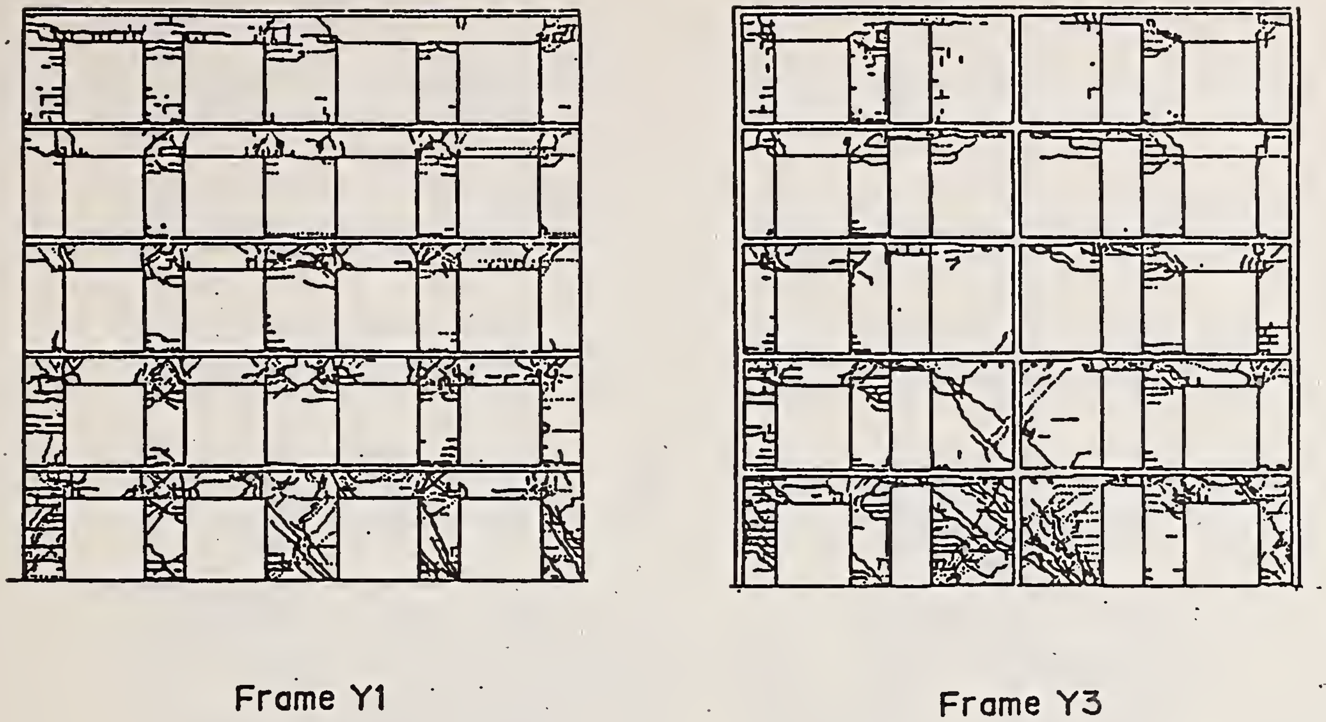


Fig.4 Crack Pattern of Frames -Y1 and -Y3 at R_t=4/800 rad.

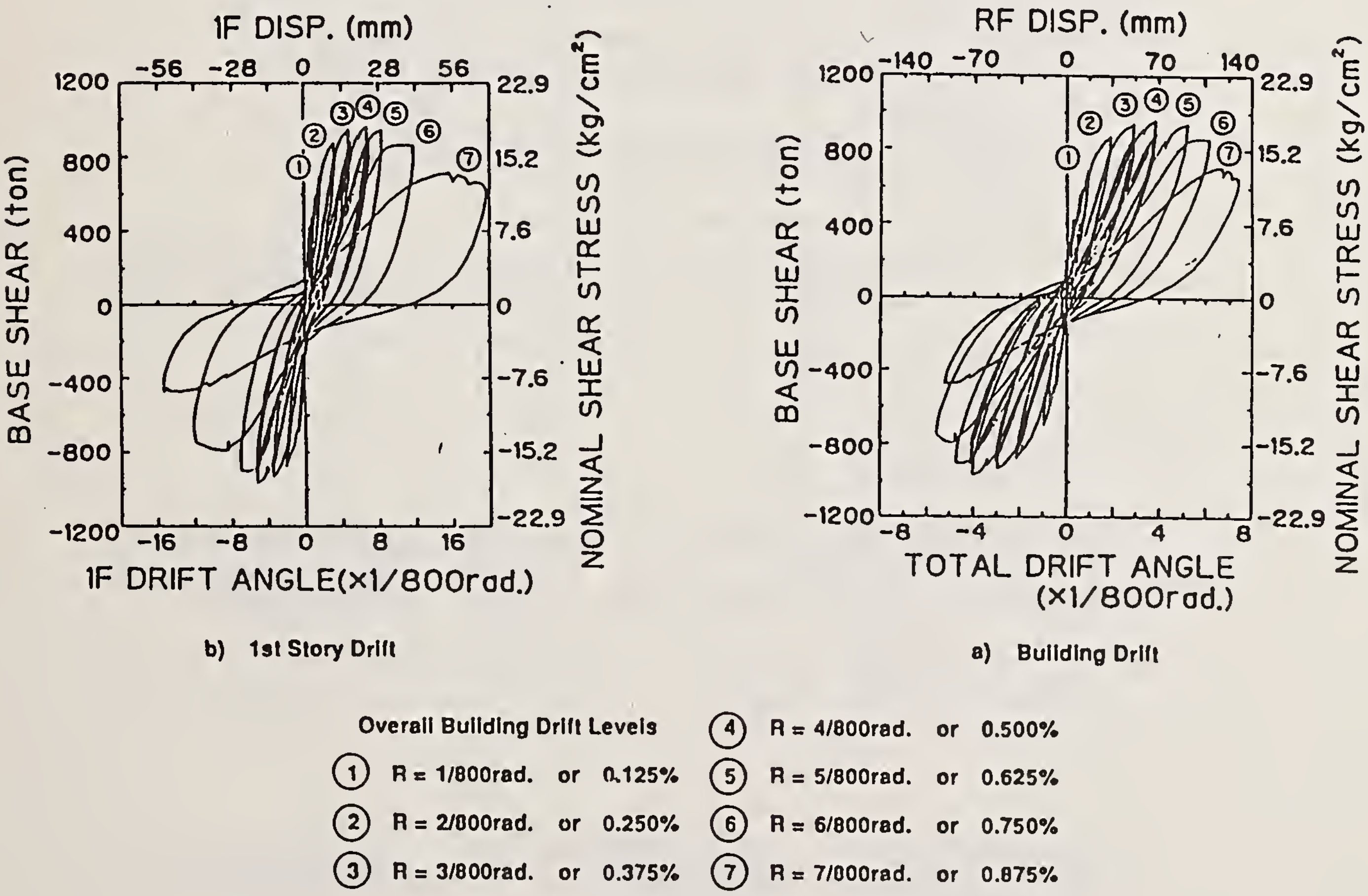


Fig.5 Base Shear vs. Displacement Relationship
Obtained through Static Cyclic Loading Tests

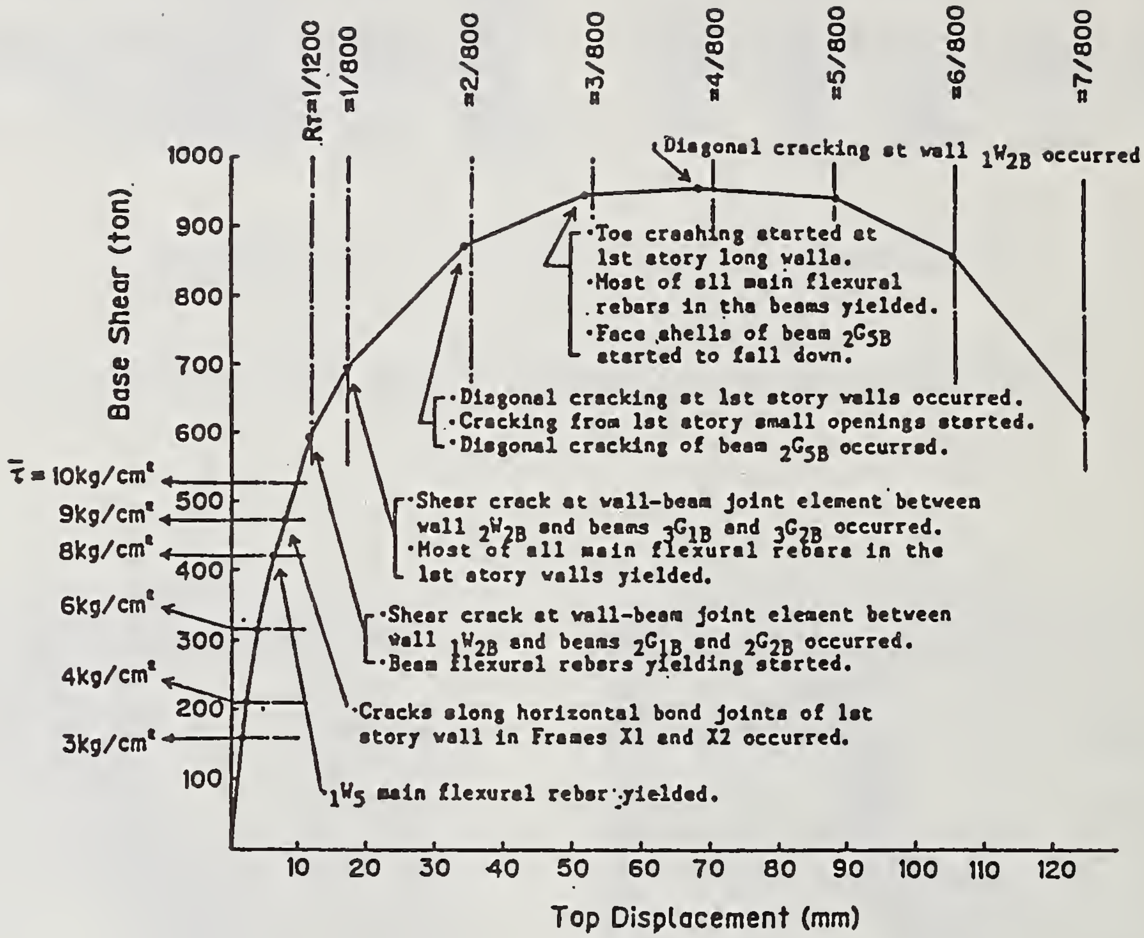
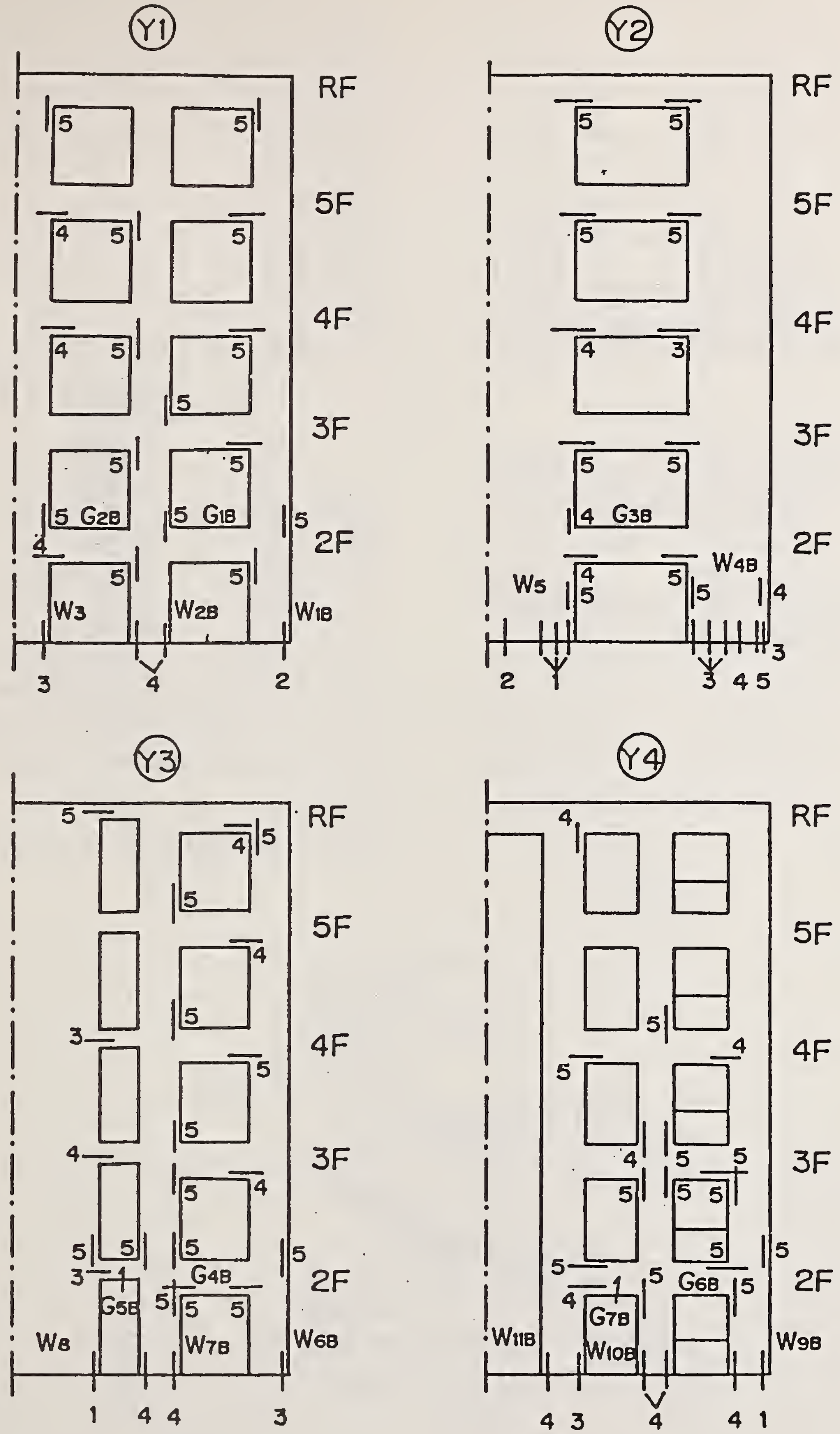


Fig.6 Base Shear vs. Top Displacement Envelop Curve for Static Cyclic Loading Tests



Building Drift
1 -1/2000(0.050%)
2 -1/1670(0.060%)
3 -1/1200(0.083%)
4 -1/800(0.125%)
5 -2/800(0.250%)

Fig.7 Consecutive Reinforcement Yield Development

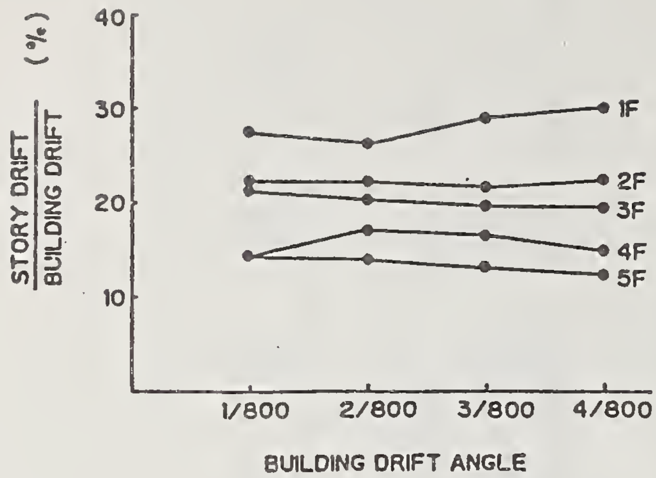


Fig.8 Story Drift Percentage during Yield Load Phase Pesponse

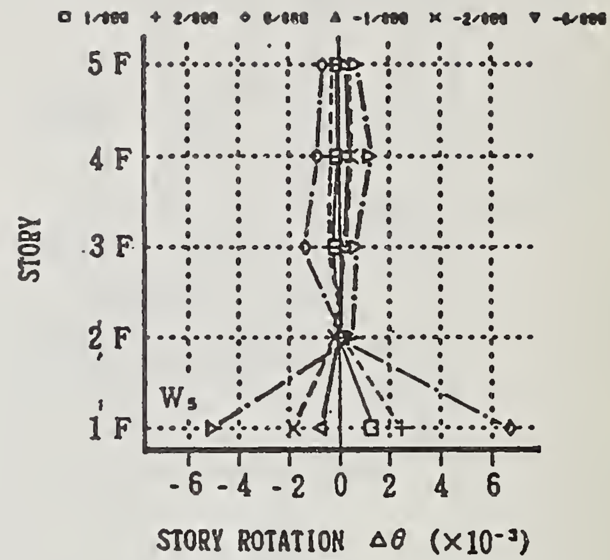


Fig.9 Story Rotation of Wall W5

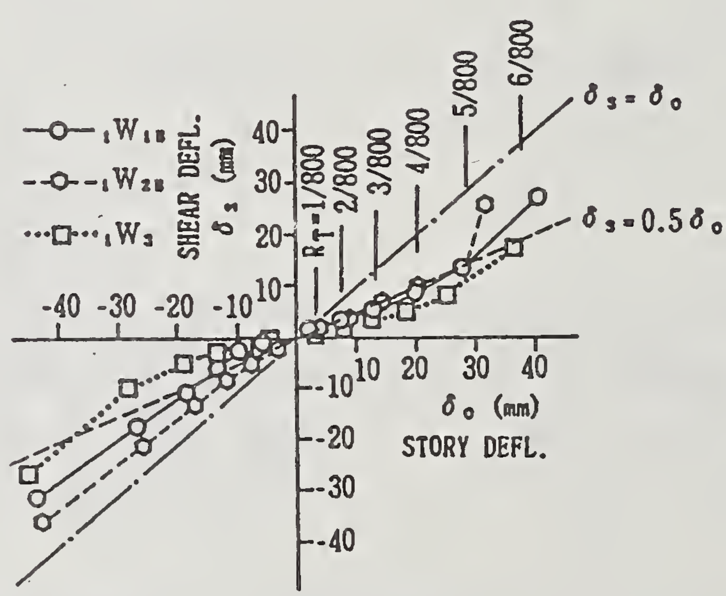
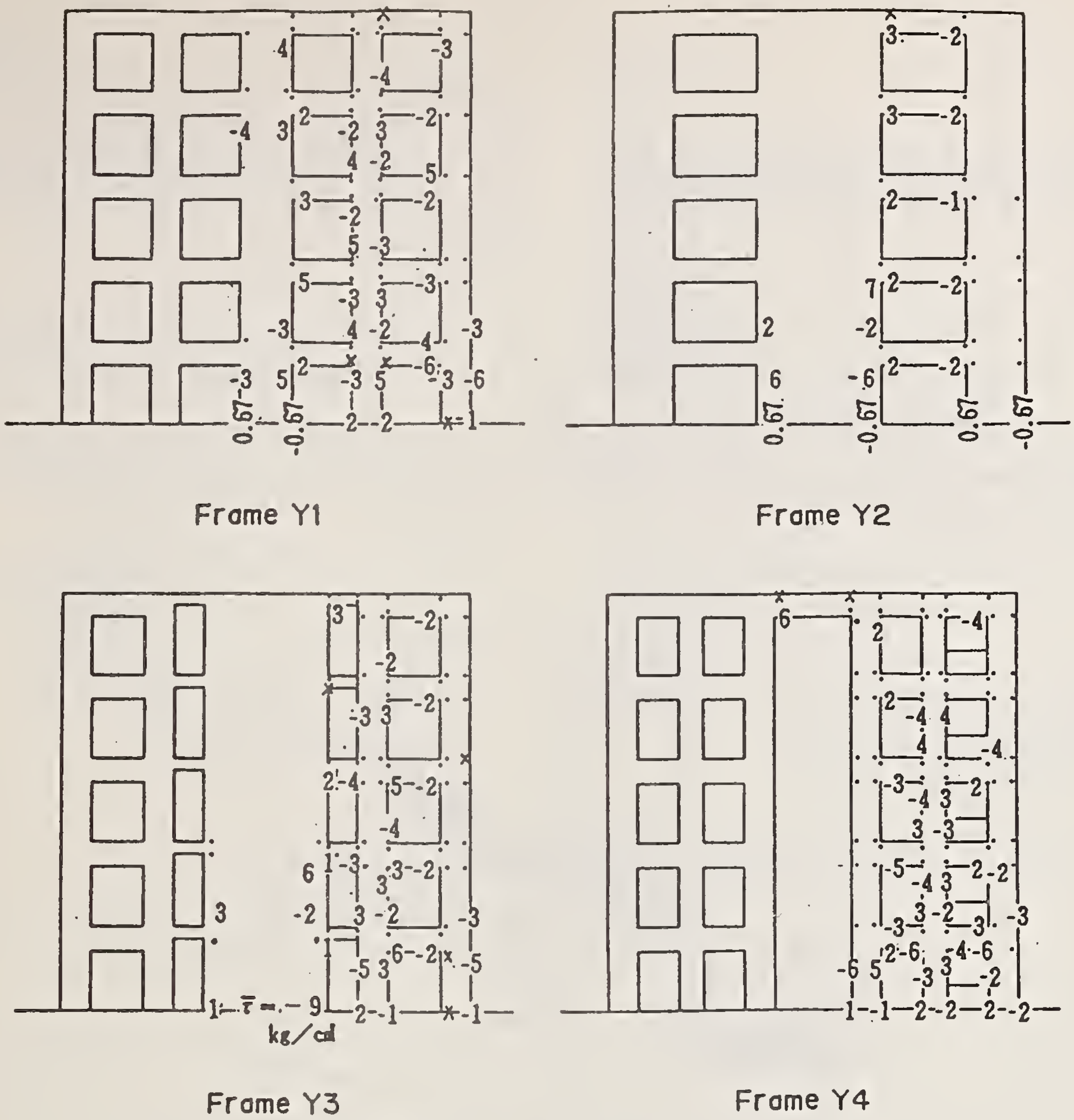


Fig.10 Shear Deflection vs. Story Deflection on First Story Walls



Numerals : indicate a building drift angle scaled in × 1/800
• : strain gage locations
× : dead strain gage locations

Fig.11 Yield Hinge Development

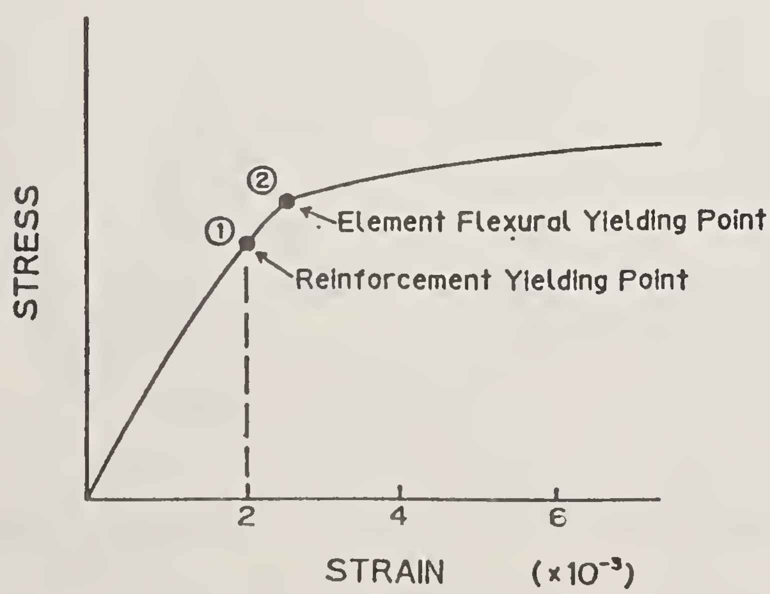
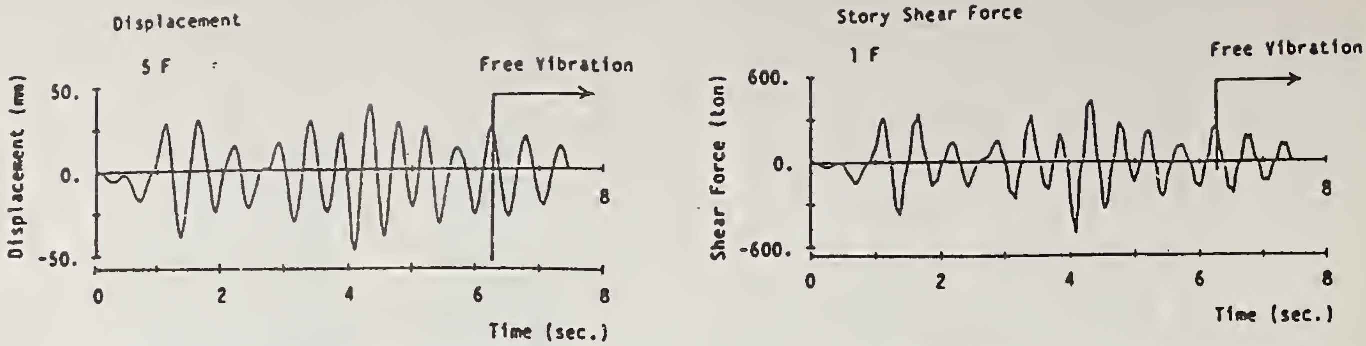


Fig.12 Definition of Yielding



a) Time History of Horizontal Displacement

b) Time History of Lateral Force

Fig.13 Pseudo Dynamic Test Floor Level Response to Taft EW 1952 (0.3g)

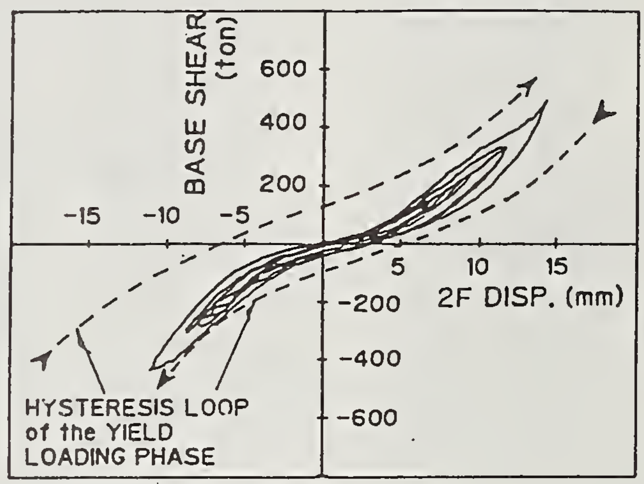
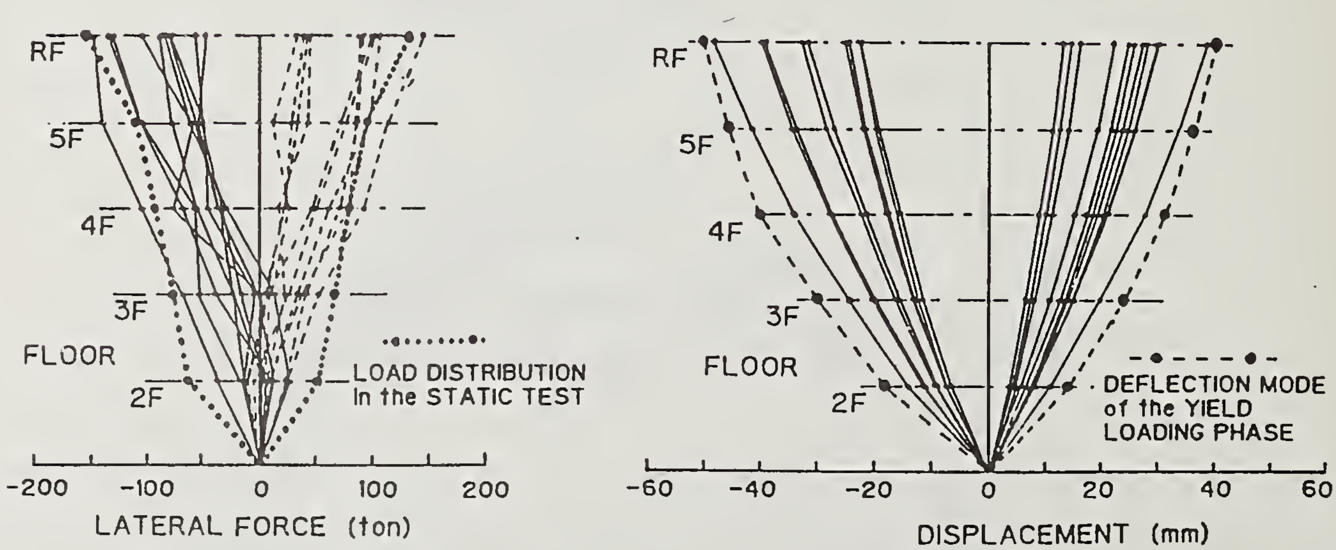


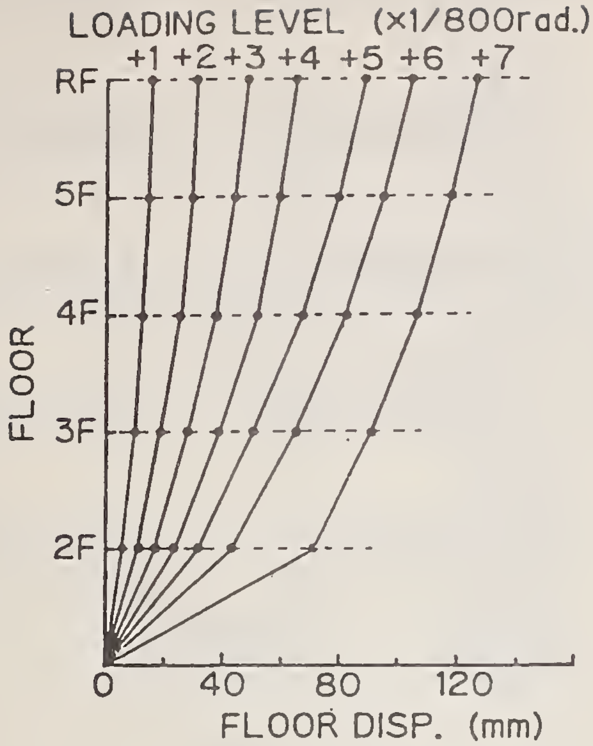
Fig.14 Relationship Between Base Shear and the First Story Displacement During Pseudo Dynamic Test



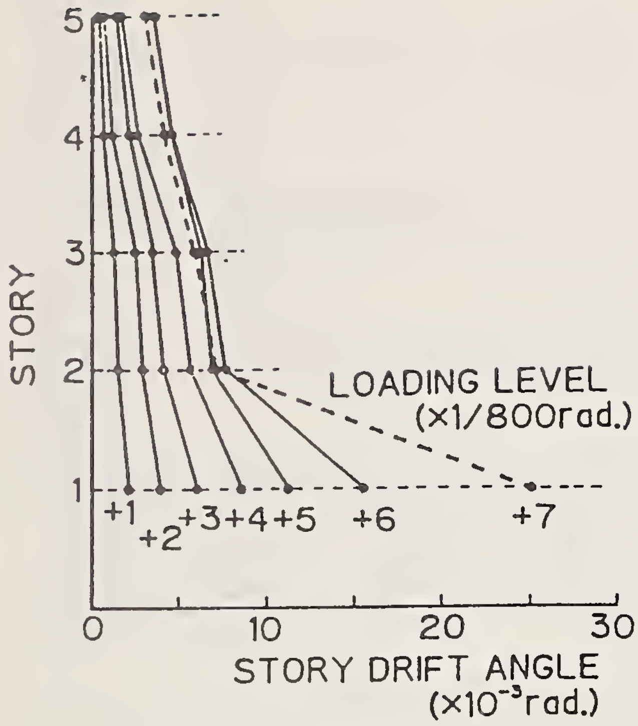
a) Lateral Force Distribution at Peaks of the Response

b) Deflection Mode at Peaks of the Response

Fig.15 Peak Response to Taft EW 1952 (0.3g)

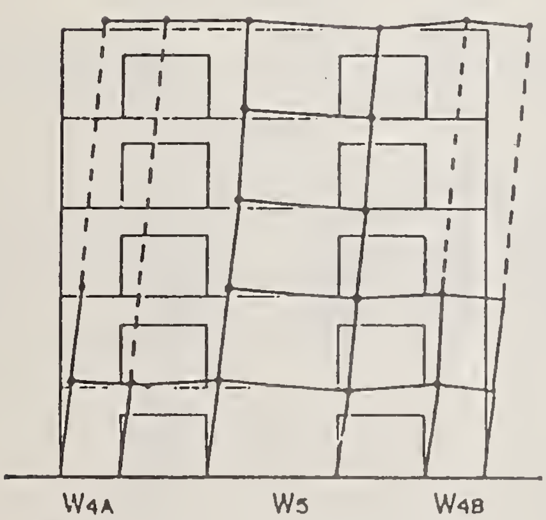


a) Building Deflection Mode

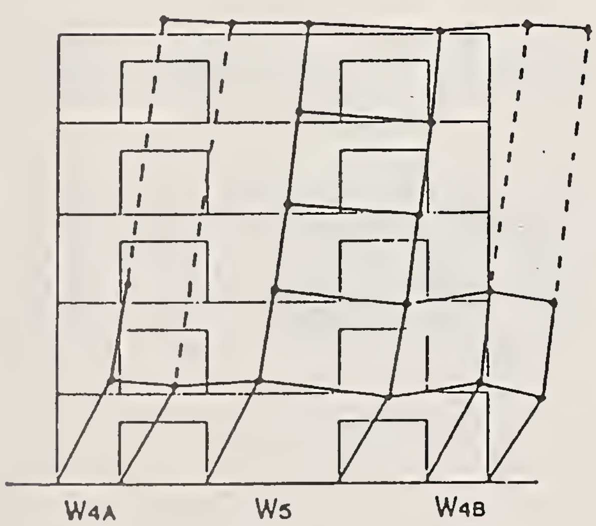


b) Loading Level vs. Story Drift Angle

Fig.16 Progressive Deformation History

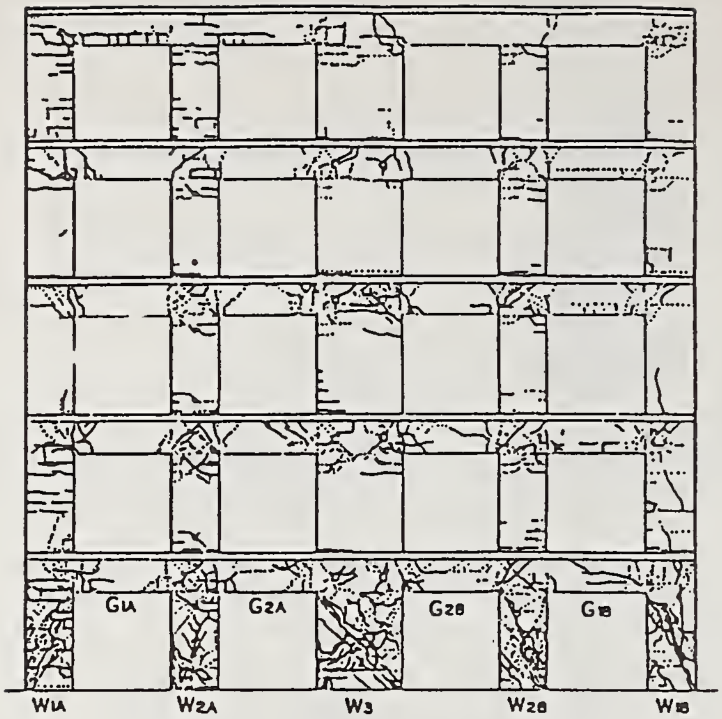


FRAME-Y2 (+3/800rad.)

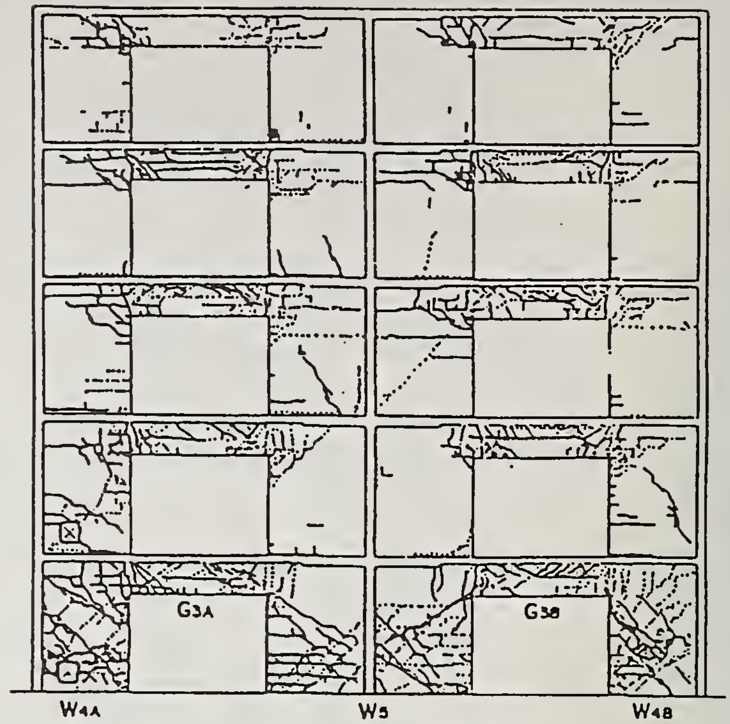


FRAME-Y2 (+7/800rad.)

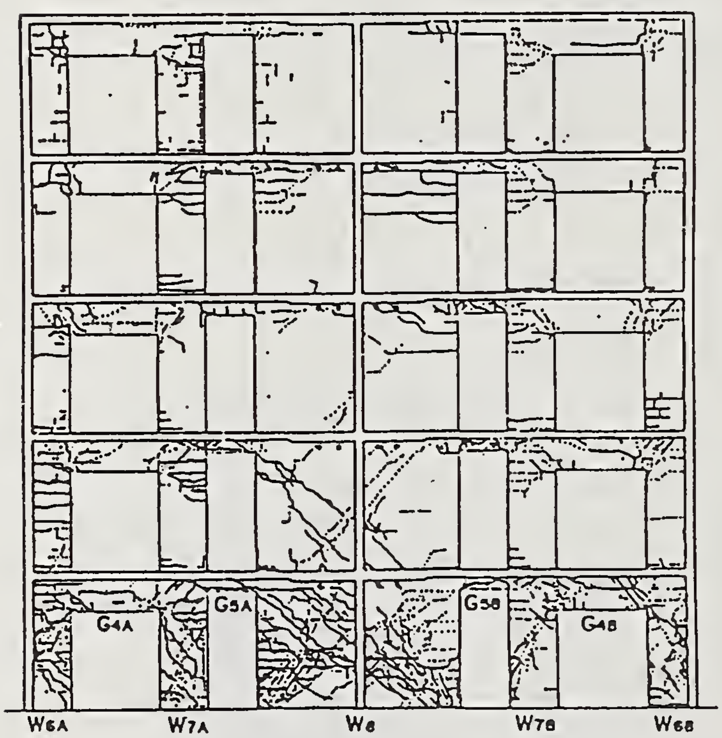
Fig.17 Overall Deflection Patterns of Frame Y2



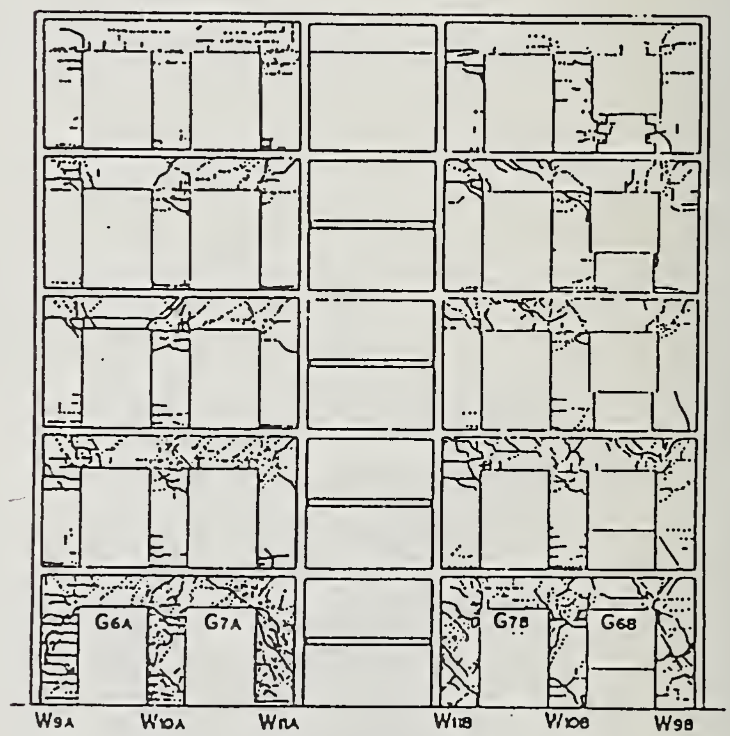
a) Frame Y1



b) Frame Y2



c) Frame Y3



d) Frame Y4

Fig.18 Ultimate Crack Patterns in Lateral Load Bearing Frames

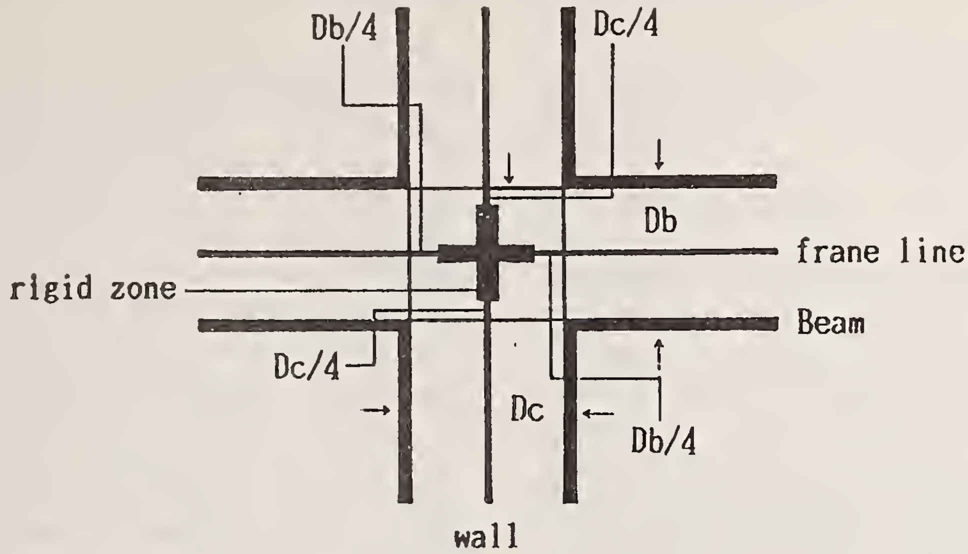


Fig.19 Assumption of rigid zone

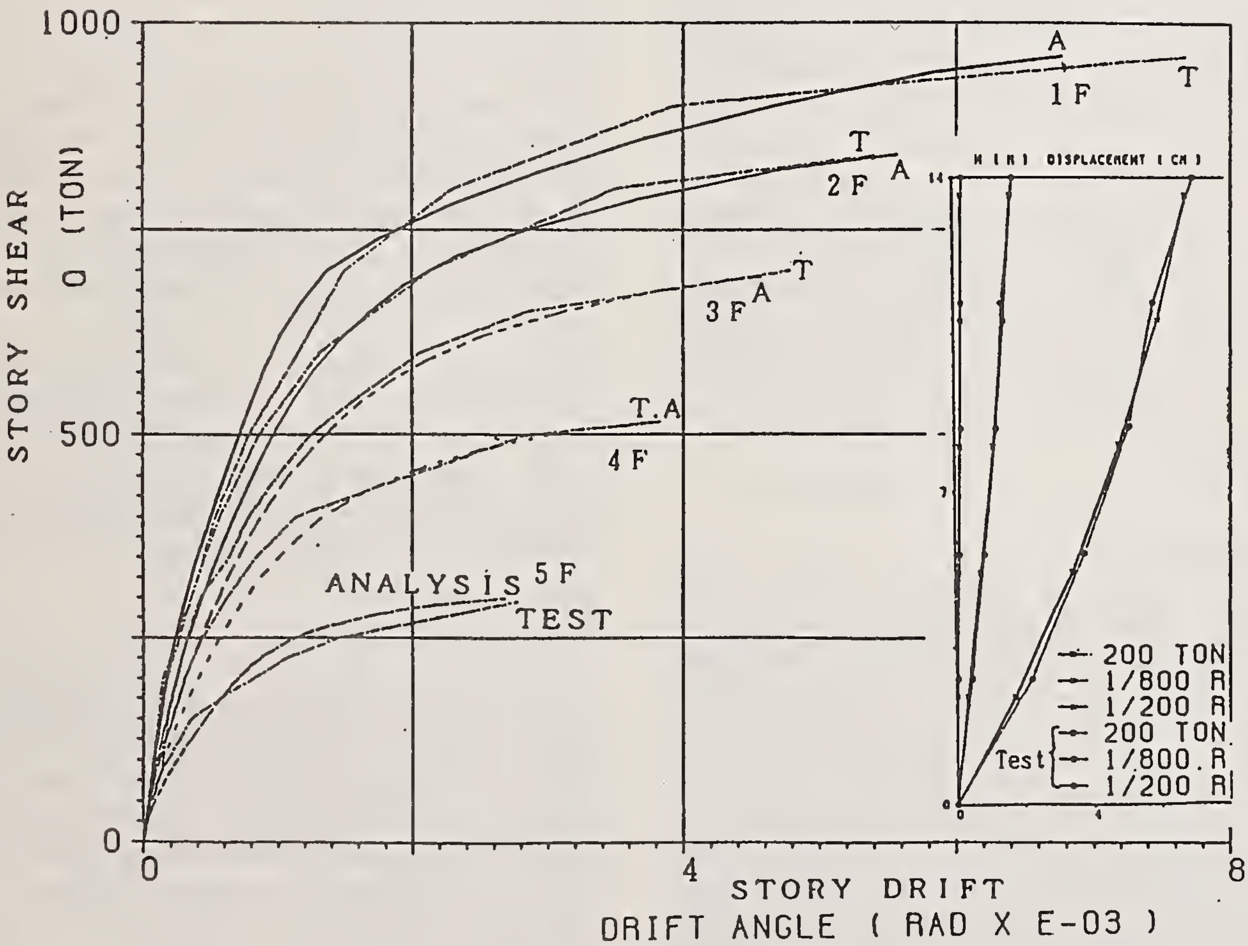


Fig.20 Story Shear vs. Story Drift Relationship

Influence of Horizontal Reinforcement on the Shear Resistance of Masonry Walls

by

Charles W. C. Yancey[1]

ABSTRACT

Ten concrete masonry wall specimens were tested to determine the effect of varying the amount and distribution of horizontal reinforcement on in-plane shear resistance. The walls were constructed with hollow concrete block and varying amounts of bed joint reinforcement and reinforcing bars located in grouted bond beams. They were subjected to reversed cyclic, in-plane lateral loads and essentially a constant axial compressive load. Results of the tests indicate that relatively small amounts of horizontal reinforcing are effective in increassing post-cracking strength. In-plane shear strength did not increase proportionally with increasing amounts of reinforcing.

KEYWORDS: axial load; bond beams; concrete masonry; cracking load; cyclic loading; energy absorption; horizontal reinforcement; shear failure; shear walls; ultimate strength

1. INTRODUCTION

1.1 Background

This series of tests initiated the second phase of a masonry research program being conducted at the National Institute of Standards and Technology (formerly the National Bureau of Standards). The primary objective of the research program is to gather data necessary in defining in-plane shear capacity and deformation behavior of shear-dominated masonry walls. During the first phase of the program, unreinforced masonry walls were tested to obtain a benchmark for comparison of results from tests on reinforced walls. Variables investigated during the Phase I testing include: magnitude of axial compressive stress, aspect ratio, masonry type, mortar type, and grout strength. Phase II testing is divided into three series:
horizontally-reinforced walls, vertically-reinforced walls, and walls with combined vertical and horizontal reinforcement. This paper reports the results from tests conducted on ten horizontally-reinforced, masonry walls.

2. TEST SPECIMENS

2.1 Materials

The masonry units were 8x8x16, hollow, two-core, concrete blocks with a compressive strength of approximately 1800 psi, based on gross cross-sectional area. The mortar was proportioned in accordance with ASTM designation Type S. The water/cement ratio used for the grout was based on trial batches to produce a compressive strength approximately equal to that of the block units. Bed joint reinforcement consisted of 9-ga, ladder-type, galvanized wire. Bond beam reinforcement consisted of combinations of #3, #4 and #5 deformed bars.

2.2 Wall Details

The walls were fabricated in the laboratory, using running bond construction. They were 48 in. long and 7 5/8 in. thick, and had a net area of the horizontal cross section of 196.5 sq. in. The height of the walls was 56 in., resulting in an aspect ratio

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(height/length) of 1.17. An initial axial force of 39,300 lbf was applied to all walls. This force caused an axial stress of 200 psi on the net area. Based on results a series of test run during Phase I, this axial stress level was sufficient to promote a shear mode of failure. As shown in figure 1, there were two different bond beam configurations: 1)a single bond beam at mid-height, and 2) two bond beams at the second and sixth courses. Table 1 summarizes the details of the ten specimens.

3. TEST PROTOCOL

3.1 Test Setup

The walls were tested in a structural test rig consisting of servo-controlled hydraulic actuators that are capable of applying any combination of forces or displacements in the six-degrees-of freedom at the top of a wall specimen. The six degrees of freedom are three translations along and three rotations about the three orthogonal axes. Vertical loads were applied through the use of four actuators which were attached to the ends of the upper and lower crossheads. In-plane lateral load was applied by an actuator which was aligned with the central plane of the wall. Upper and lower loading shoes were attached to the ends of the walls to simulate a fixed-fixed boundary condition.

3.2 Instrumentation

The horizontal and vertical actuators were equipped with load and displacement transducers to measure the overall forces and moments applied to the upper crosshead and displacements of the actuator pistons as they moved the crosshead. The in-plane horizontal displacement of the walls was measured by linear variable differential transformers (LVDTs) which were vertically spaced along the loaded edge as show in figure 2. Vertical and diagonal displacements were measured using four LVDTs which were mounted on one face of the specimens. A number of specially designed strain gage devices, called leaf spring transducers (LSTs), were attached to one face to measure localized deformations in the block units. The LSTs were positioned such that they did not cross mortar joints. Figure 3 gives the locations of the LSTs deployed on wall specimen R1.

3.3, Test Procedure

The test procedure was standardized for all specimens except wall R8. The reversed, cyclic loading history illustrated schematically in figure 4 was used as the guide. First, the specified vertical compressive load, 39,300 lbf, was applied. While maintaining this load at in-plane lateral displacements were applied at the top edge of the walls. The first displacement excursion was 0.02 in., or about 25% of the estimated displacement at first cracking. A first cracking displacement of 0.08 in. was predicted based on test results from Phase I. Three reversed cycles of lateral displacement were applied at each increment of 0.02 in. up to the maximum excursion of 0.06 in. Thereafter, the displacement was increased until a major crack formed (i.e. First Major Event or FME). The direction of the crosshead was then reversed to a displacement equal to that recorded at FME. The degradation and stabilization cycles indicated in figure 4 were followed as the walls were subsequently loaded to failure. As the lateral displacement magnitude increased, the axial load gradually increased. Wall R8 was tested under load control in the vertical direction.

4. TEST RESULTS

All ten of the wall specimens exhibited shear modes of failure as characterized by cracks propagating along each major diagonal. Table 2 presents a summary of the results for two limit states: 1) first cracking and 2) ultimate load. The steel content percentages given in column 4 were computed by dividing the area of the reinforcement (i.e. bars or galvanized wire) by the product of the wall's thickness and height. The total horizontal load at first cracking and the corresponding lateral displacement at the top of wall, the magnitude of the ultimate load and the lateral displacement at ultimate load are listed in columns 5 through 8.

The displacement values, referred to as "global in-place displacements," are those measured by the displacement transducer located in the horizontal load actuator.

Figure 5 presents load-displacement curves for four walls with varying amounts and distributions of reinforcement: 1) wall R1 was unreinforced and provides a benchmark for comparing response characteristics; 2) wall R4 had no bond beam but contained joint reinforcing at every horizontal mortar joint; 3) wall R5 contained a mid-height bond beam,with 2 - #4 bars, but had no joint reinforcing; and 4) wall R10 had a mid-height bond beam, with 2 - #4 and 1 - #5 bars, and joint reinforcing at every course. The load and deflection values at FME and ultimate are noted on the curves. The bracketed numbers indicate the number of loading cycles applied to the specimens at the time of occurrence of the limit state.

Figure 6 provides a qualitative comparison of behavior for the same four walls in the form of schematic drawings of the crack pattern at the end of testing.

The energy absorbed during each cycle of loading was computed using the load-displacement data. Bar charts quantifying the energy absorption magnitude per cycle are presented in figure 7 for walls R1, R4, R5, and R10. The vertical scale is the same for all four bar charts to more clearly illustrate the relative magnitudes of energy absorption.

5. DISCUSSION OF RESULTS

The qualitative and quantitative results from wall R1 provide a baseline of comparison for the other wall specimens. The majority of the cracks that existed in wall R1 at the end of testing were formed in the first few cycles of inelastic deformation and progressed as testing continued. Strength and stiffness degradation occurred as the result of grinding of mortar joints and crushing of block units along cracks in the bed and head joints. The FME and ultimate load, occurring in the direction of initial deflection, were of the same magnitude. The number of cycles of loading before reaching ultimate was 17. Referring to figure 7, it is observed that the maximum energy level in any cycle was about 3 kip-in. Comparing the results from walls R2 and R4 with those of wall R1, it was observed that cracks formed in the two specimens with bed joint reinforcement at about a 20% higher load than in the unreinforced wall. The cracks tended to remain closed, and there was less grinding and crushing along cracks in walls R2 and R4 than in wall R1. As borne out by figure 6, cracking was more uniformly distributed over the faces of R2 and R4 than for R1. Ultimate load was increased by the presence of bed joint reinforcement placed either at every course or every other course. The displacement and number of cycles at ultimate load for walls R2 and R4 were approximately double the corresponding value for wall R1. Comparison of results from R2 with those from R4 indicate no significant difference in wall response as a function of the spacing of the bed joint reinforcing.

Referring to table 2, the comparative entries for wall R1 and walls R5 or R6 indicate a significant increase in the ultimate load levels as a result of using a reinforced bond beam. The presence of the mid-height bond beam confined crack propagation to the lower half of the specimens for the first 20 cycles of loading. The final crack pattern for wall R5 is presented in figure 6. There was considerable grinding of mortar joints and crushing and grinding of block units as the blocks moved relative to each other. When the test results from walls R5 and R6 are taken as a bond beam set and compared to the joint reinforcing set formed by walls R2 and R4, it is observed that the individual contribution of each form of reinforcement to an increase in ultimate load is about the same. Moreover, comparable values were obtained for deflection at ultimate load, number of cycles at ultimate load and first cracking load from each set.

The displacement at ultimate load was between 3/16 and 1/4 in. for all reinforced walls except R10 (mid-height bond beam and joint reinforcing every course). The displacement value for wall R10 was about 1/2in. Its first cracking load level was significantly higher than any other in this series. Moreover, wall R10 had the second highest ultimate shear stress in the series. When comparing the ultimate shear resistance of this wall with that of walls R6 and R8, which had about the same steel content, the significantly higher values obtained by R10 may be attributable to the presence of the bed joint reinforcing.

The following observations summarize the preliminary results:

1. Small amounts of horizontal reinforcing are effective in increasing post-cracking strength.

2. In-plane shear strength does not increase proportionally with increasing amounts of reinforcing.

3. Bed joint reinforcing placed in alternating courses is as effective in increasing in-plane shear strength as when placed in every course.

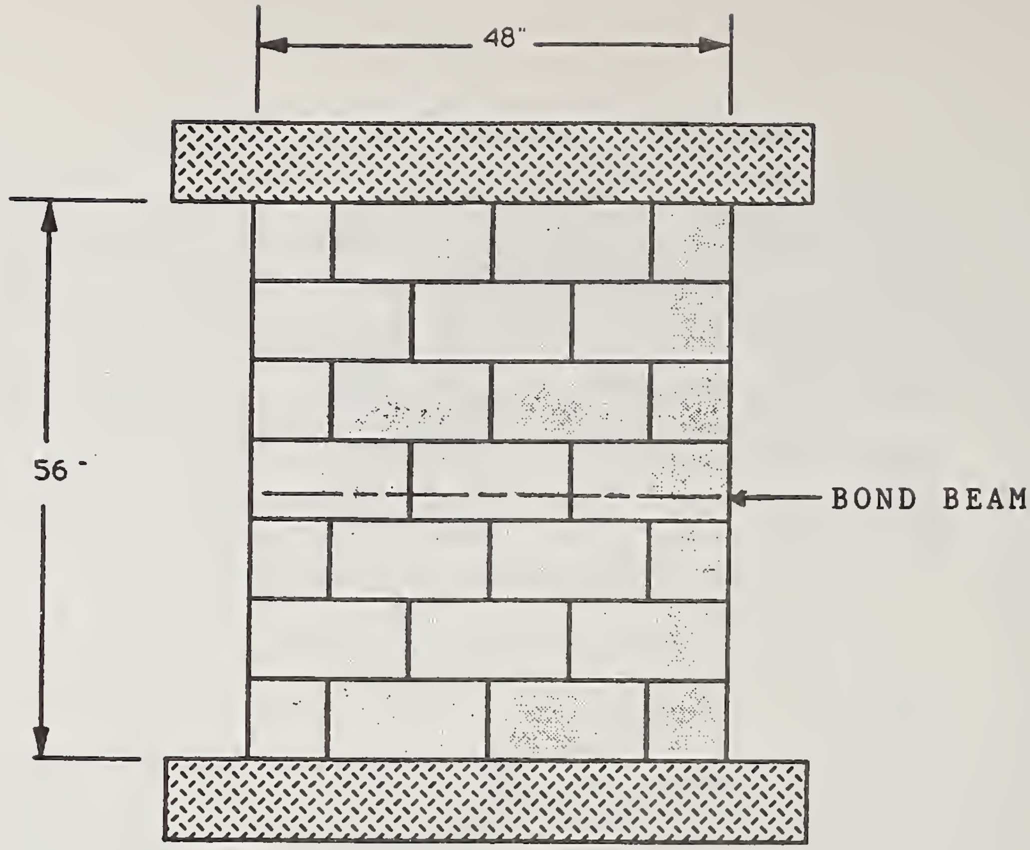
TABLE 1 - TEST SPECIMEN DESCRIPTION

SPECIMEN NAME	ASPECT RATIO	NO. OF BOND BEAMS	BOND BEAM LOCATION	BOND BEAM REINF.	JOINT REINF. PATTERN	JOINT REINF. AMOUNT
R1	1.17	0	N/A	N/A	UNREINF.	N/A
R2	1.17	0	N/A	N/A	EVERY OTHER COURSE	9 ga @ 16" on center
R3*						
R4	1.17	-0-	N/A	N/A	EVERY COURSE	9 ga @ 8" on center
R5	1.17	1	MID-HEIGHT	2 - #4	UNREINF.	N/A
R6	1.17	1	MID-HEIGHT	3 -#5	UNREINF.	N/A
R7	1.17	2	2nd & 6th courses	1 - #5	UNREINF.	N/A
R8	1.17	1	MID-HEIGHT	3 - #5	UNREINF.	N/A
R9	1.17	1	MID-HEIGHT	1 - #3	EVERY OTHER COURSE	9 ga @ 16" on center
R10	1.17	1	MID-HEIGHT	2 - #4 1 - #5	EVERY COURSE	9 ga @ 8" on center
R11	1.17	2	2nd & 6th courses	1 - #5	UNREINF.	N/A

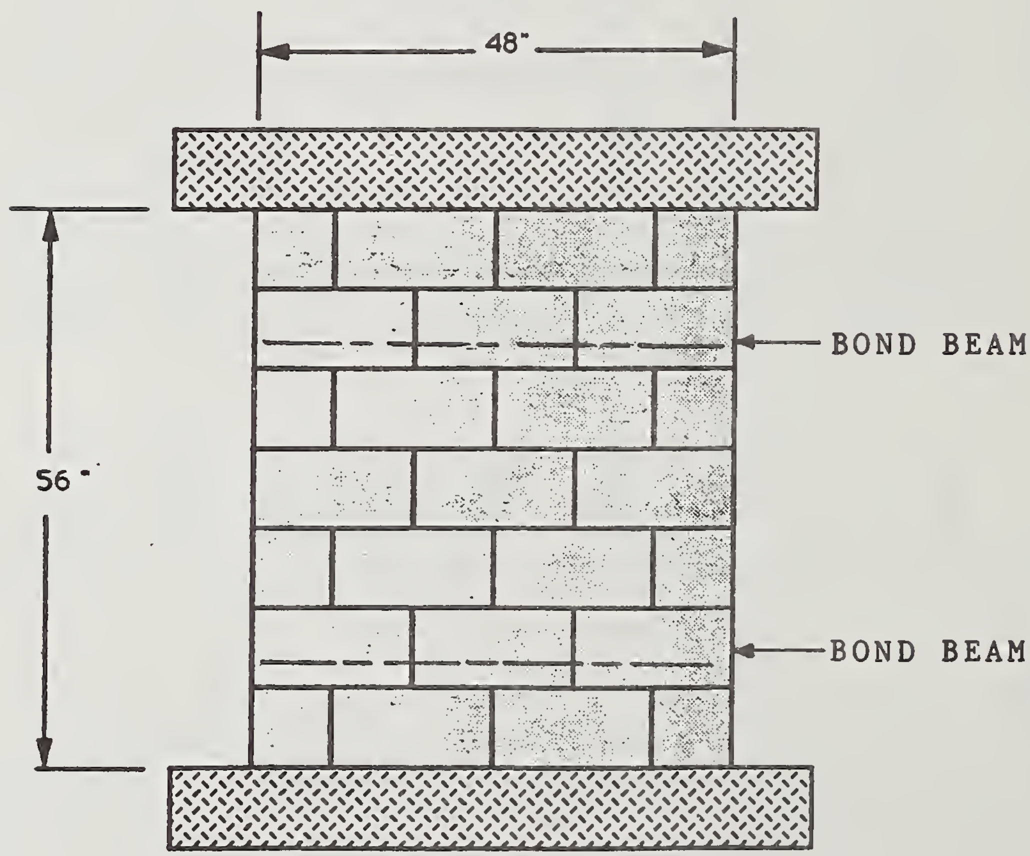
* Specimen broken before testing

Table 2 - LOAD AND DEFLECTION AT FIRST CRACKING & ULTIMATE

WALL SPECIMEN	NO. OF BOND BEAMS	JOINT REINF.	STEEL CONTENT %	FIRST CRACKING LOAD (KIPS)	DEFL. AT CRACKING LOAD (IN.)	ULT. LOAD (KIPS-PSI)	DEFL. AT ULT. LOAD (IN.)
R1	0	NONE	0	23.8	0.0.09	27.5(140) -23.8(121)	0.12 -0.09
R2	0	16 IN. O.C.	0.0242	26.7	0.10	28.9(147) -35.1(179)	0.20 -0.28
R4	0	8 IN. O.C.	0.0566	26.2	0.12	31.9(162) -33.4(170)	0.20 -0.24
R5	1	NONE	0.0936	30.2	0.12	42.5(216) -45.4(231)	0.24 -0.30
R6	1	NONE	0.218	27.8	0.12	32.7(166) -35.1(179)	0.18 -0.18
R7	2	NONE	0.145	30.6	0.18	35.2(179) -36.8(188)	0.26 -0.26
R8	1	NONE	0.218	23.3	0.12	26.8(136) -26.6(136)	0.24 -0.24
R9	1	16 IN. O.C.	0.0757	21.9	0.15	36.3(185) -39.7(202)	0.23 -0.38
R10	1	8 IN. O.C.	0.215	36.6	0.24	43.1(220) -45.2(230)	0.48 -0.48
R11	2	NONE	0.145	29.5	0.20	33.2(169) -38.6(196)	0.25 -0.25



MID-HEIGHT BOND BEAM



BOND BEAMS AT 2ND AND 6TH COURSES

Figure 1 - Location of Bond Beams

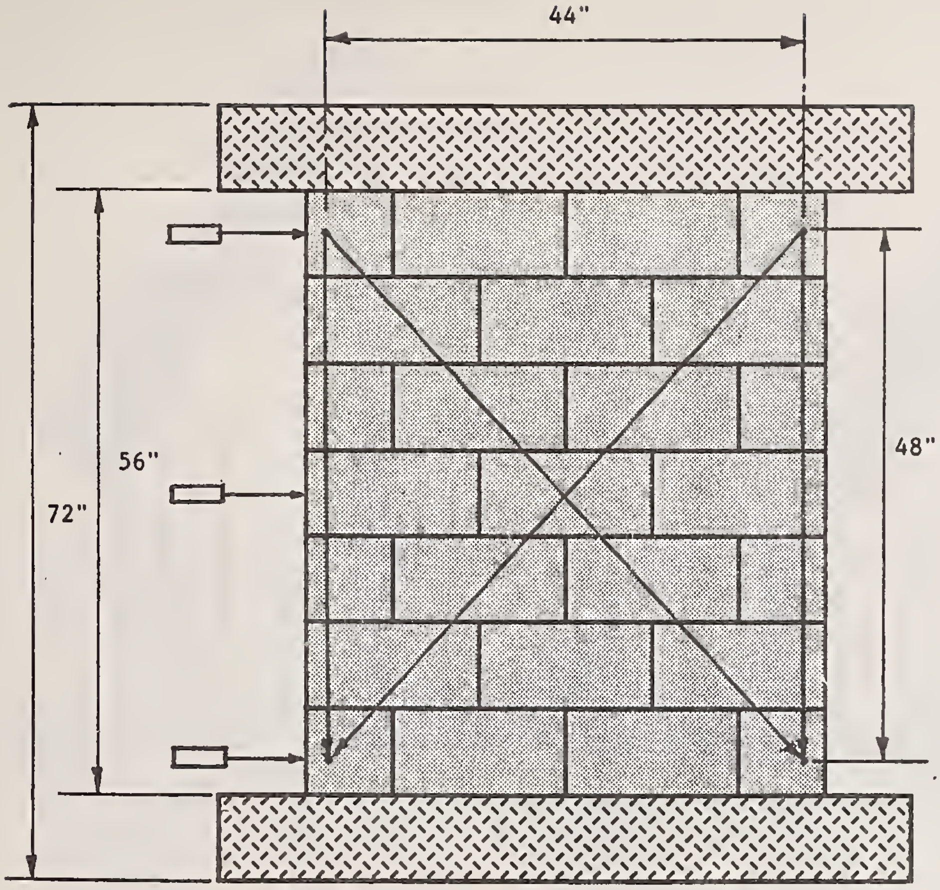


Figure 2 - LVDT Locations

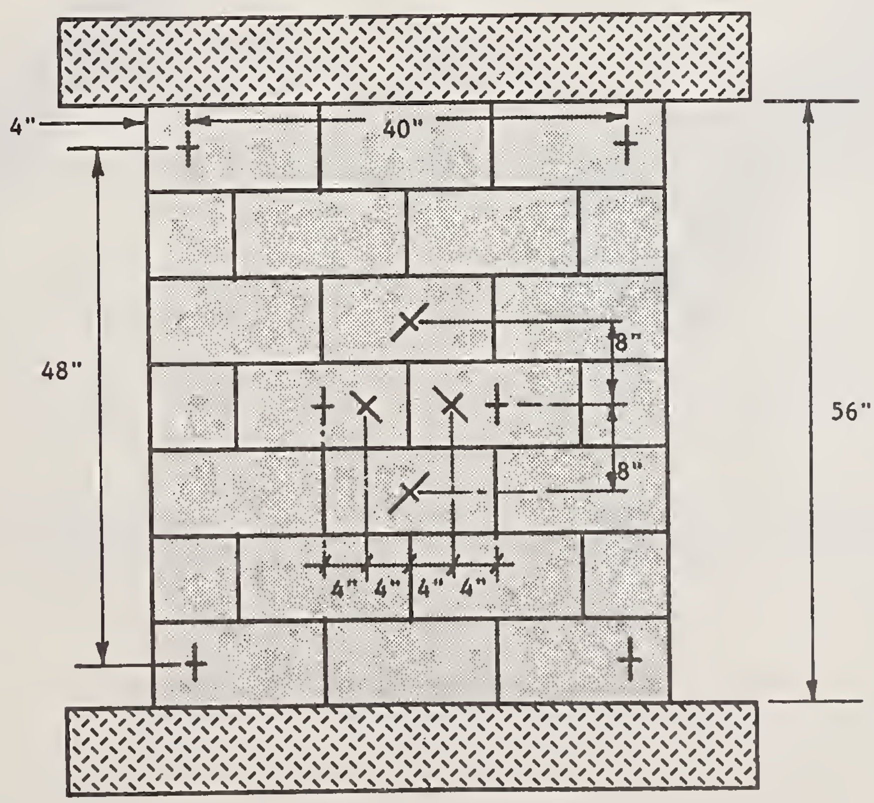


Figure 3 - LST Locations

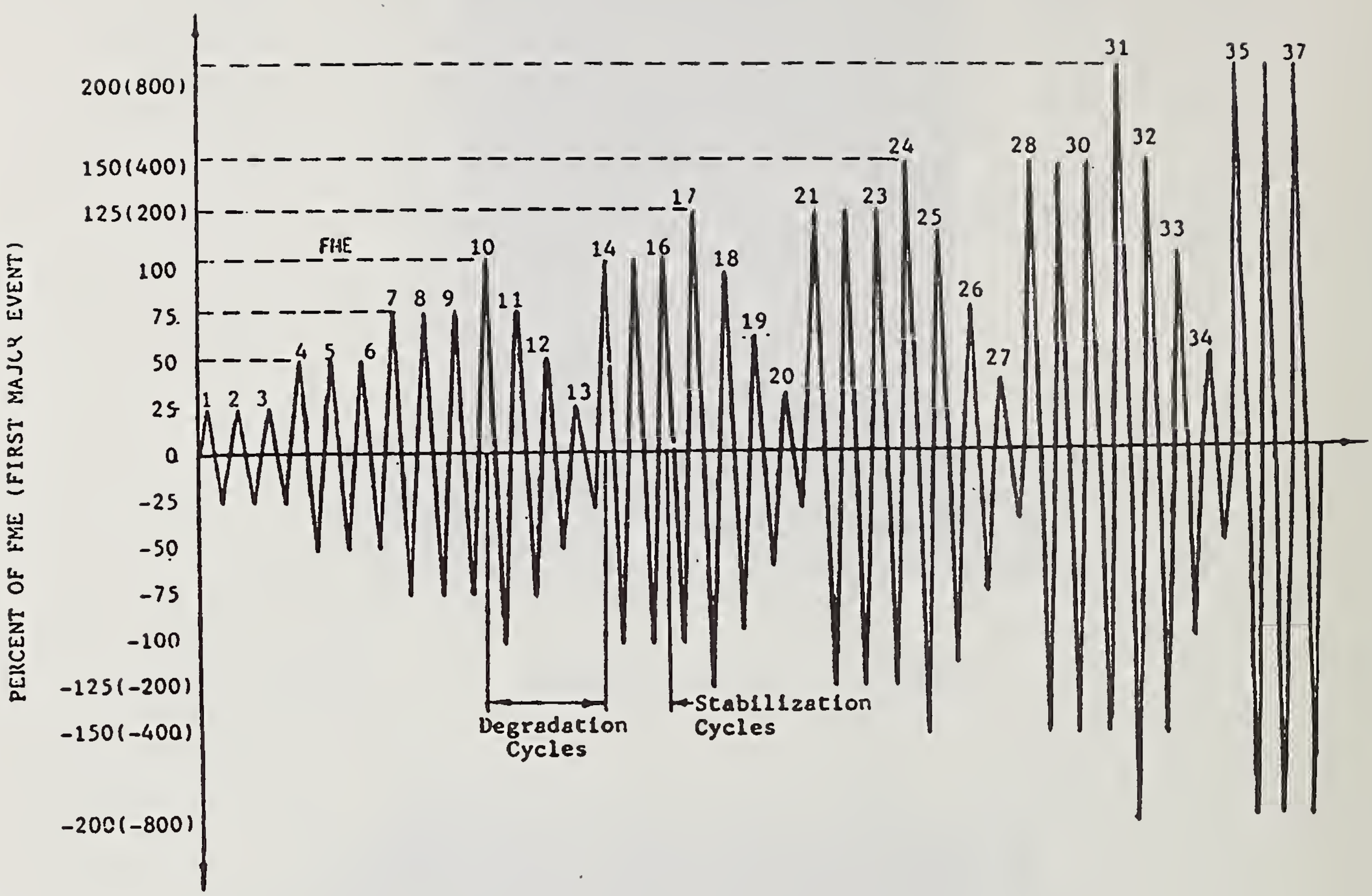
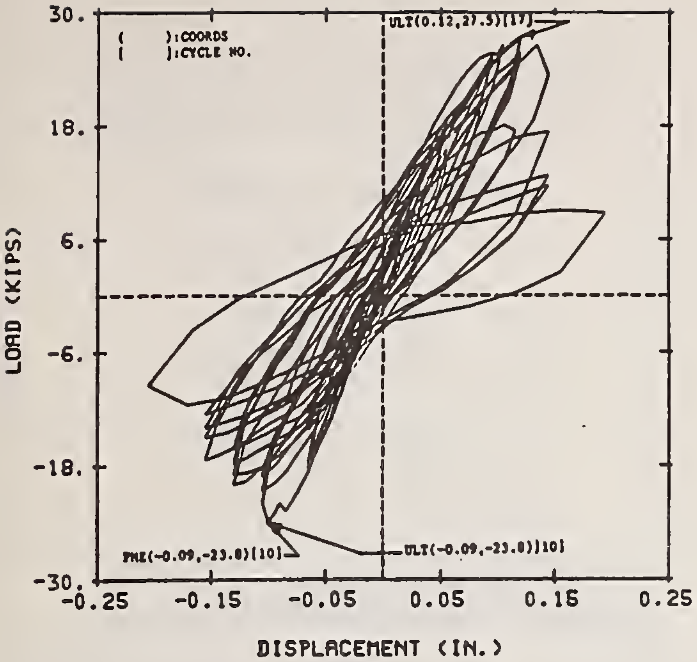
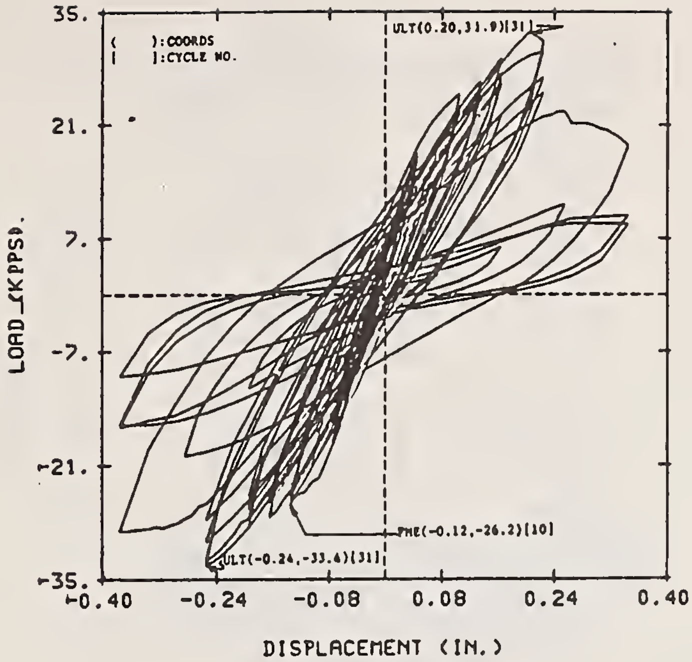


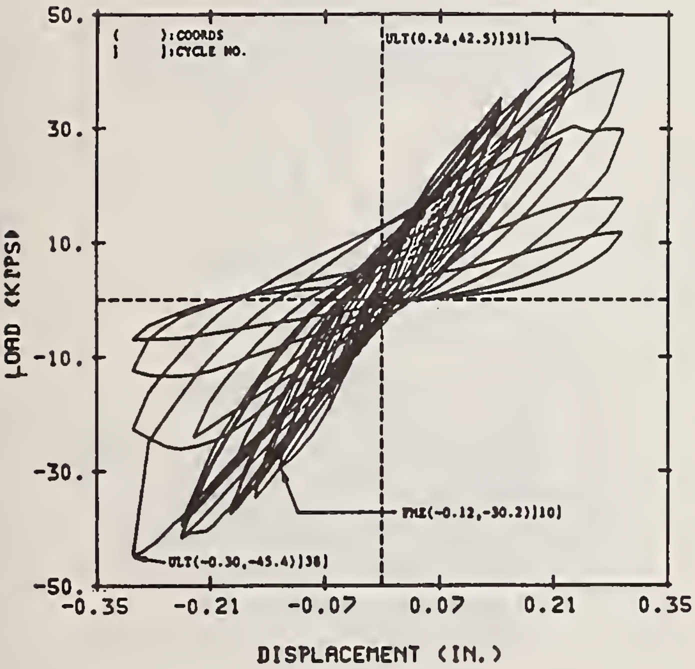
Figure 4 - Cyclic Loading History Used For Masonry Walls



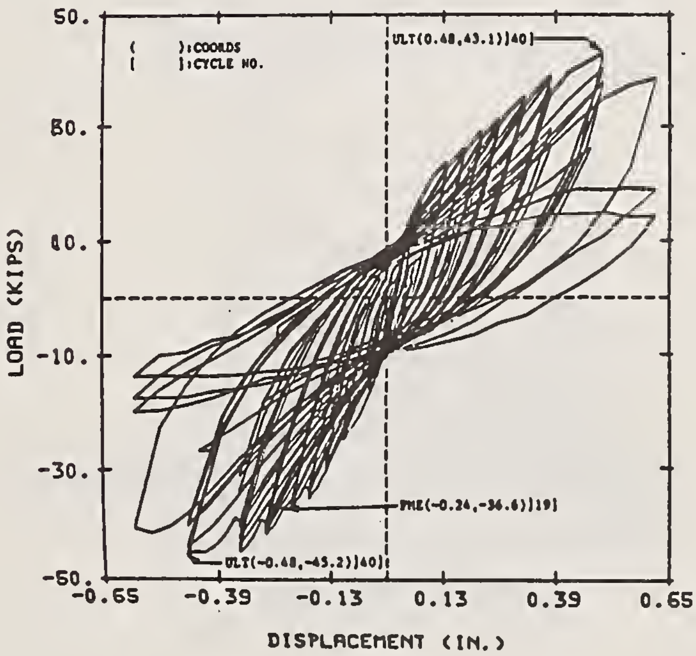
UNREINFORCED WALL (R1)



NO BOND BEAM, JOINT REINF. EVERY COURSE (R4)



MID-HEIGHT BOND BEAM
NO JOINT REINFORCING (R5)



MID-HEIGHT BOND BEAM
JOINT REINF. EVERY COURSE (R10)

Figure 5 - Load Displacement Curves

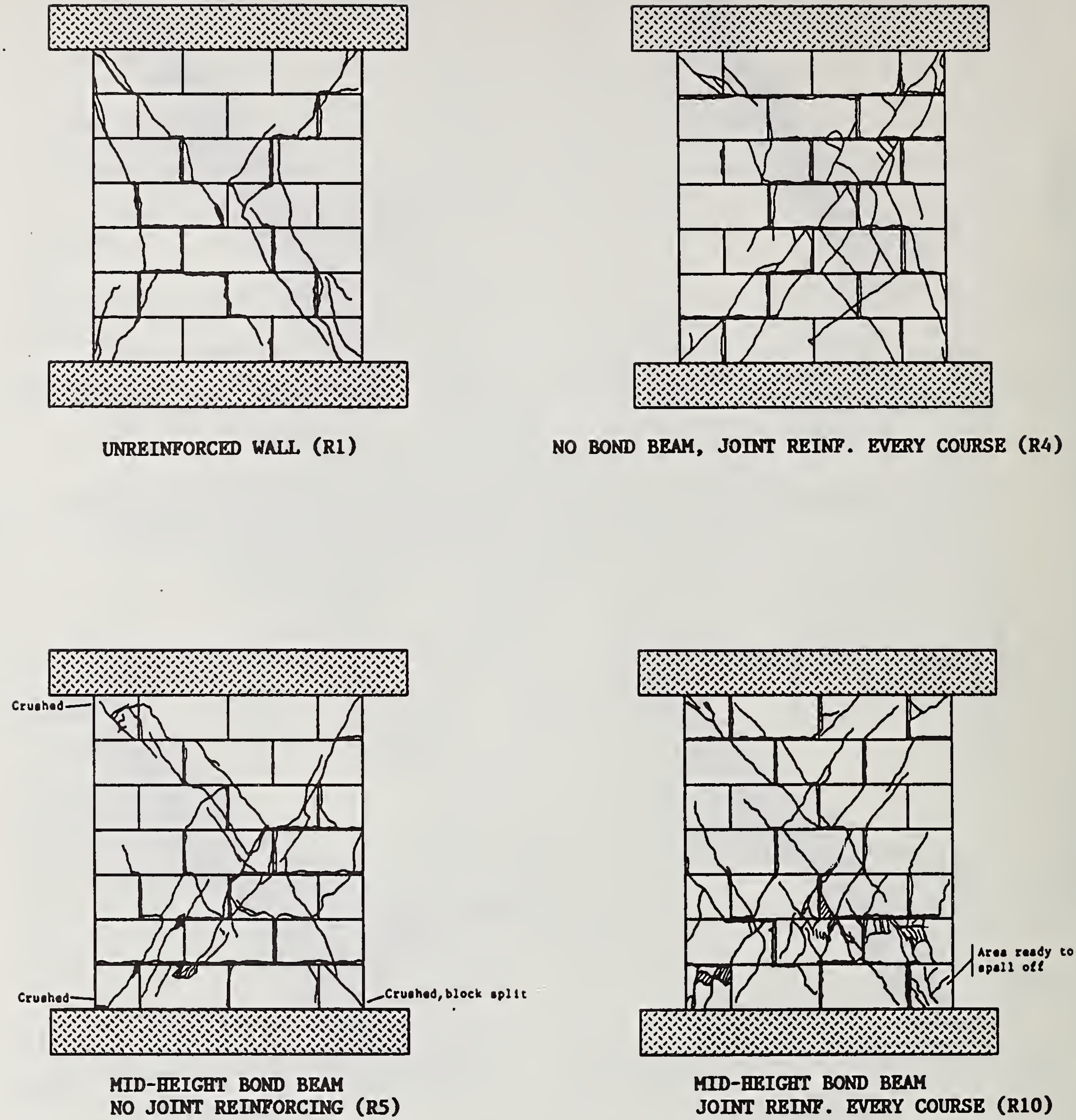


Figure 6 - Crack Patterns at End of Testing

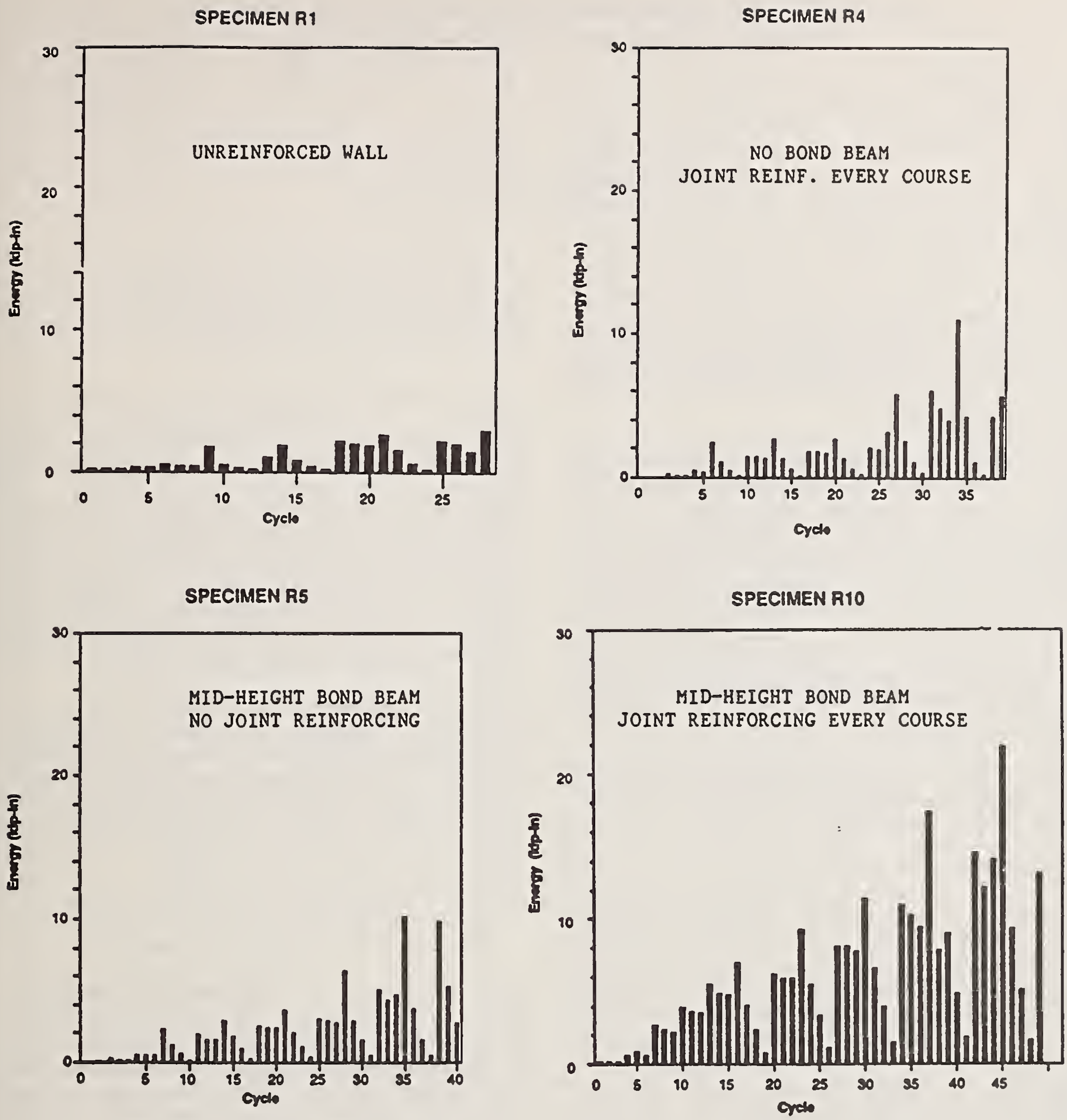


Figure 7 - Energy Absorption Per Cycle

Inelastic Behavior of Reinforced Concrete Bridge Pier—Effect of Bilateral Loading and Circular Hollow Section

by

Toshio Iwaskai[1], Osamu Ueda[2],
Kazuhiko Kawashima[3], Kinji Hasegawa[4]
and Takeshi Yoshida[5]

ABSTRACT

Dynamic loading tests using large specimens of reinforced concrete bridge piers have been conducted to study an effect of bilateral loading and circular hollow section on the nonlinear hysteresis behavior of reinforced concrete bridge piers.

It was found from the tests that deterioration of load bearing capability is larger in the specimens subjected to the bilateral loading as compared with the specimen subjected to unilateral loading. It was also found that the specimen with hollow section subjected to a series of step-wise load reversals shows much significant failure than the specimen with solid section due to spalling-off of the concrete into the hollow space as well as the spalling-off of concrete outside the specimen.

KEYWORDS: Seismic design, reinforced concrete (RC) bridge piers, highway bridges, dynamic loading tests, ductility

1. INTRODUCTION

A number of highway bridges has been damaged in the past earthquakes. Particularly as represented by the damages caused by the Miyagi-kèn-oki Earthquake in 1978, significant damages of the RC bridge piers were likely to be developed by strong ground shaking compared to the superstructure or foundation. Seismic design for highway bridges has been made in accordance with the seismic coefficient method in which structural members are designed so that they would stay within elastic range against lateral force equivalent to the horizontal acceleration of 0.2 G to 0.3 G. It is important, however, to properly evaluate the dynamic characteristics of RC piers including energy dissipating capability.

Many studies have been made on nonlinear hysteretic behavior of RC piers under various loading conditions such as static, dynamic and earthquake-like loadings. However, in such loading tests conducted in the past, the loading was mostly made only in one direction. The seismic force acting to RC piers during an earthquake comprises two-directional components. It is required to evaluate the effect of simultaneous loading in two horizontal directions.

On the other hand, for high RC piers, hollow sections are often adopted for aiming to reduce the weight of pier.

In the RC piers with hollow section, core concrete is not fully confined by the tie bars surrounding the outside of concrete since there is a hollow space inside the pier, so that it is anticipated that the hollow section has less amount of ductility as compared with the solid section. Because few experiments have been made for the hollow sections, study for the nonlinear behavior of RC piers with hollow sections seems to be quite important for adopting the RC hollow cross sections for high bridge piers.

From the viewpoint described above, the bilateral loading in two horizontal directions was conducted by using two loading actuators. Loading tests were also conducted for RC pier models with circular hollow section. Effect of the bilateral loading and hollow section on the nonlinear hysteresis behavior of RC piers was studied.

2. TEST SPECIMENS

2.1 Specimens for Bilateral Loading

Three specimens were used for the bilateral loading. As shown in Fig. 1, the specimens have the rectangular section of 80 cm x 40 cm, and the height of 2.4 m from the base to the loading point. Thus, the shear span ratio defined as a ratio of the pier height to the effective width is 6.8 and 3.2 for weak axis and strong axis, respectively. The main reinforcement is of eleven SD30 with a diameter of 13 mm (longitudinal

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reinforcement ratio of 1.0 %). It was placed only along the long side wall with covering thickness of 5 cm. All the axial reinforcements are arranged continuously from the footing to the top of the pier. Lateral ties are of SR24 with a diameter of 9 mm and placed at 20 cm interval, which is equal to the half of the thickness of 40 cm in short-side direction.

High-early-strength portland cement with the maximum grain size of aggregate of 10 mm is adopted. Unconfined compressive strength on the day of loading test was 311 to 402 kgf/cm^2.

2.2 Specimens for Circular Hollow Section

Two specimens as shown in Fig. 2 were used to study the effect of circular hollow section. The specimens have the circular hollow section with the outside diameter of 800 mm and inside diameter of 518 mm, and the height h is 2.475 m from the base of the pier to the loading point. For the main reinforcement, fifty SD30 bars were placed for both specimens P-46 and P-47. Diameter of the main reinforcement was varied as a parameter to be investigated, i.e., ϕ 10 mm for P-46 and ϕ 13 mm for P-47. Thus, the main reinforcement ratio is 1.2 % and 2.1 % for P-46 and P-47 respectively. Main reinforcement was placed continuously from the footing to the top of pier. The lateral ties of SR24 with ϕ 9 mm were placed at the intervals of 20 cm.

High-early-strength portland cement was used for concrete with the maximum grain size of aggregates of 10 mm.

3. LOADING PROCEDURE

In the experiments for studying the bilateral loading effect, the specimen was anchored to the reaction floor with bolts as shown in Fig. 4. Two dynamic actuators, which were installed to the reaction wall in such a manner that two loading directions became orthogonal to each other, were fixed to the head of the specimen. Although axial force of about 5 to 20 kgf/cm^2 acts to bridge pier due to dead load of superstructure, it was disregarded in this test because of the restrictions of the experimental equipment and its limited effect.

Two loading directions were set equal to the two principal axes of the section as shown in Fig. 5, in which the first and second loading directions are defined here as the direction normal and parallel, respectively, to the long side of the cross section. For comparison with the bilateral loading condition, unilateral loading was also made, in which loading was made in the first loading direction.

The yield displacement corresponding to one ductility factor was defined ; δ_1 for the first loading direction and δ_2 for the second loading direction. It was defined as the displacement at loading point at which reinforcing bars at the extreme tension fiber firstly reached yield strain. When subjected to the loading in the relevant direction, the yield displacements δ_1 and δ_2 are 12 mm and 5 mm, respectively.

The specimens were subjected to a series of step-wise increasing symmetric displacement cycles as shown in Fig. 5.

In the first loading direction, the loading amplitude was increased with the step-size of one ductility factor such that $1\delta_1$, $2\delta_1$, $3\delta_1$, . In the second loading direction, the step-size for the loading amplitude was assumed one ductility factor (δ_2, $2\delta_2$, $3\delta_2$, $\cdots$) for P-49 and two times of ductility factor δ_2 for P-50 ($2\delta_2$, $4\delta_2$, $6\delta_2$, $\cdots$).

Phase between the first and second loading amplitudes was shifted by 90 degrees as shown in Fig. 6, so that hysteresis between the two loading amplitudes becomes elliptic. Thus, the number of loading is 10 cycles in the first loading direction, but it is 10.5 cycles in the second loading direction. No loading was made in the first and last 0.25 cycle in second loading directions. The loading rate was determined in such a manner that the velocity amplitude in the first loading direction became constant with 10 cm/sec.

In the experiments for studying the effect of hollow section, the lateral force was applied with an actuator as shown in Fig. 7.

The specimens were subjected to a series of step-wise symmetric displacement cycles. At each step, 10 cycles of loading with the same displacement amplitude was carried out. The step-size was determined by increasing the displacement in each step by a displacement ductility factor of one. It should be noted here that due to malfunction of measurement system the yield displacement δy used in this test was determined from the strain of main reinforcement at 10 cm above the base. Therefore, the yield displacement δy used in this test is about 4 % larger than the correct value.

The loading velocity was assumed as 25 cm/sec. The axial force equivalent to the dead load of the superstructure was also disregarded in the test.

4. EFFECT OF BILATERAL LOADING

4.1 Failure Mode

Failure mode after step-wise loading is shown in Figs. 8, 9 and 10, in which cracks, spalling-off of concrete and buckling of main reinforcement normal to the first loading direction are presented. When flexural failure occurs at the base of the pier subjected to unilateral loading, the failure normally progresses in the order of

a)horizontal cracks in concrete at the base,
b)spalling-off of concrete near the base,
c)outward buckling of main reinforcement,and
d)rupture of main reinforcement.

Even in the bilateral loading the same progress of the failure basically occurs. In the unilateral loading, the spalling-off of concrete occurs at a time on the whole plane orthogonal to the first loading direction. In the bilateral loading the spalling-off begins from a corner and then spreads to the whole plane orthogonal to the first loading direction as the loading progresses.

Table 1 shows the progress of damages in terms of loading displacement at which spalling-off of concrete occurs and the number of main reinforcement ruptured. The spalling-off of concrete begins from $4\delta_1$ loading in the unilateral loading. In the bilateral loading, the spalling-off begins from $4\delta_1$ and $3\delta_1$ loading in specimen P-49 and P-50, respectively.

As to the rupture of main reinforcement, the rupture begins from $6\delta_1$ loading in the unilateral loading. Four reinforcements are ruptured at this loading step. On the other hand in P-49, the rupture begins at $6\delta_1$ loading, in which 6 reinforcements are ruptured. In specimen P-50, the rupture begins from the $4\delta_1$ loading in which one bar is ruptured, and 7 bars are ruptured at the next loading step.

From such evidence, it can be said that the damages generally progress more promptly in bilateral loading compared to unilateral loading. Number of main reinforcement ruptured significantly depends on loading pattern. This is certainly developed because the bars, specially at the corner, are subjected to alternation of larger tension and compression in the bilateral loading than in the unilateral loading.

4.2 Strength and Ductility

Fig. 11 shows hysteresis loops between loading force and lateral displacement in the first loading direction. It is seen in Fig. 11 that the hysteresis curves of P-49 and P-50 have considerably round shapes compared to the unilateral loading.

Fig. 12 shows the comparison of the envelope of the hysteresis loops in the first loading direction.

It can be seen in Fig. 12 that envelope of the hysteresis loop of P-49 is almost identical with that of P-37, while the envelope of the hysteresis loop of P-50 is much smaller than that of P-37. It seems that as long as the ratio between the loading displacement in the first and second directions is δ_2/δ_1, the effect of bilateral loading is less significant. On the other hand, when the ratio is doubled to $2\delta_2/\delta_1$, the bilateral loading effect becomes considerable.

Fig. 13 compares the hysteresis loops in the second loading direction. In both specimens, a projected loop can be observed at the beginning. This corresponds to the loading at the first 0.25 cycle in which loading was made only in the second loading direction.

Table 2 summarizes the yield load Py, the maximum load Pu, yield displacement δy and ultimate displacement δu. Only averaged values developed between in positive and in negative loading directions are presented in Table 2.

Although the ductility factor is 17 % smaller in P-49 than that in P-37, the yield load Py and the maximum load Pu are almost the same between P-49 and P-37. However, the ductility factor and the maximum load Pu of P-50 are 25 % and 12 % smaller than those of P-37. Thus, the bilateral loading effect is significant for P-50.

4.3 Energy Dissipating Capability

The hysteresis loop generally has a curve presented in Fig. 14, and the area surrounded by the hysteresis loop represents the energy absorbed by the pier during one loading reversal. Fig. 15 compares the amounts of energy absorbed during load reversals due to deformation in the first loading direction. Energy absorption increases up to $3\delta_1$ loading during which difference between the three specimens is less significant. Over $3\delta_1$ loading, the energy absorption of the specimen P-50 significantly decreases due to rupture of the main reinforcements, while the energy absorption of the specimen P-49 takes the maximum at $4\delta_1$ loading and then decreases thereafter.

Energy absorption capability was also studied in terms of an equivalent hysteretic damping ratio he which is defined as

$$h_e = \frac{1}{2\pi} \cdot \frac{\Delta W}{W} \quad \text{-------------------} \quad (1)$$

in which, W and ΔW represents an elastic energy and an energy dissipated during one load reversal, respectively, as defined in Fig. 14.

It is interesting to note that the hysteretic damping ratio is larger in P-50 than P-49, even for the loading displacement up to $3\delta_1$. This is due to the hysteresis loop with round corner.

5. EFFECT OF HOLLOW SECTION

5.1 Failure Mode

The failure mode on the plane parallel to the loading direction after each load reversal is shown in Figs. 18 and 19. Flexural failure occurred in the three specimens at their bases. Because the main reinforcement ratio is 0.9 % higher in P-47 than in P-46 by the tie reinforcement ratio being the same, the failure at the base is much significant in P-47 than in P-46, i.e., after load reversal with $5\delta y$, concrete at the base is completely crushed and distortion similar with the shear failure occurred. Three and six main reinforcements were ruptured during $6\delta y$ and $7\delta y$ load reversals, respectively, in P-46, while no rupture of the main reinforcement was developed in P-47 up to $6\delta y$ loading. This clearly shows that the deformation developed at the base of P-47 is close to the shear failure rather than the flexural failure.

Spalling-off of concrete into the hollow space occurred from $7\delta y$ loading in P-47 and from $5\delta y$ loading in P-48. This is greatly different from the failure mode developed in P-29 with solid cross section.

5.2 Strength and Ductility

Fig. 20 shows the hysteresis loops between loading force and displacement of pier. Fig. 21 compares the envelopes of the hysteresis loops presented in Fig. 20.

In accordance with the difference of the failure mode described in the preceding section, there is a significant difference in the envelope of the hysteresis loop. In the specimen P-29, the capability for supporting the lateral force decreases gradually as the number of main reinforcement ruptured increases. Even in the specimen P-46, deterioration of the load in accordance with the increase of loading displacement is not necessarily significant. However, in the specimen P-47, the load significantly decreases after reaching the maximum strength. In the hollow circular section, the concrete is confined at the outside by the main reinforcement as well as lateral ties but not confined at the inside. This is considered to be related to the sudden decrease in the strength due to the spalling-off of concrete into the hollow space.

5.3 Energy Dissipating Capability

Fig. 22 shows the equivalent hysteretic damping ratio defined by Eq. (1). It should be noted here that the yield displacement is not identical between the hollow and solid sections. It is seen in Fig. 22 that decreasing rate of the damping ratio after reaching the maximum value is much larger in hollow section than in solid section.

6. CONCLUSIONS

In order to study the effect of the bilateral loading and of hollow section on the nonlinear hysteretic behavior of RC piers, a series of dynamic loading tests was conducted. From the results presented herein, the following conclusions may be deduced:

(1) Effect of bilateral loading

i) When the specimens are subjected to a series of step-wise increasing symmetrical bilateral displacement cycles with the step size of one ductility factor for the relevant directions, progress of the failure and deterioration of load bearing capability is approximately identical with the specimen subjected to the unilateral load reversals. However, the step size in the second direction is doubled ($2\delta_2/\delta_1$), the progress of failure and deterioration of load bearing capability is substantially decreased as compared with the specimen subjected to the unilateral load reversals.

ii) Amount of the energy dissipation during load reversals is smaller in the specimen subjected to the bilateral loading, in particular, ratio between the loading displacement in the first and second directions becomes $2\delta_2/\delta_1$.

(2) Effect of hollow section

i) In the circular hollow section, the concrete is confined at the outer side by main reinforcement and lateral ties. However, because there is an empty space inside, spalling-off of concrete into the inner space develops.

ii) In consequence, especially in a specimen having a high main reinforcement ratio, the load bearing capability suddenly decreases after the spalling-off of the inner concrete. The final failure occurs without the rupture of the main reinforcement.

iii) Decreasing rate of the equivalent damping ratio after reaching the maximum value is much larger in hollow section than in solid section.

REFERENCES

1) Iwasaki, T., Kawashima, K., Hagiwara, R., Hasegawa, K., Koyama, T. and Yoshida, T. : Experimental Investigation on Hysteretic Behavior of Reinforced Concrete Bridge Pier Columns, 17th Joint Meeting, Wind and Seismic Effects, UJNR, 1985.
2) Iwasaki, T., Kawashima, K., Hasegawa, K., Koyama, T. and Yoshida, T. : Effect of Number of Loading Cycles and Loading Velocity of Reinforced Concrete Bridge Piers, 19th Joint Meeting, Wind and Seismic Effects, UJNR, 1987.
3) Iwasaki, T., Ueda, O., Kawashima, K., Hasegawa, K. and Yoshida, T. : Dynamic Strength and Ductility of Reinforced Concrete Bridge Piers, 20th Joint Meeting, Wind and Seismic Effects, UJNR, 1988.
4) Kawashima, K. and Koyama, T.: Effect of Number of Loading Cycles on Dynamic Characteristics of Reinforced Concrete Bridge Pier Columns, Structural Eng./Earthquake Engr., Proc. JSCE, Vol. 5, No. 1, pp. 183 - 191, 1988, 4.
5) Kawashima, K. and Koyama, T.: Effect of Cyclic Loading Hysteresis on Dynamic Behavior of Reinforced Concrete Bridge Pier, Structural Eng./Earthquake Eng., Proc. JSCE, Vol. 5, No. 2, pp. 343 - 350, 1988, 10.

Table 1 Comparison of Failure Modes

Loading Type		Unilateral Loading	Bilateral Loading	
Specimen		P − 37	P − 49	P − 50
Initiation of Spalling of Cover Concrete		Step 4	Step 4	Step 3
Number of Main Reinforcement Cut during Loading	Step1～3	0	0	0
	Step 4	0	0	1
	Step 5	0	0	7
	Step 6	4	6	−

Table 2 Comparison of Load and Displacement Performance

Loading Type		Unilateral Loading	Bilateral Loading	
Specimen		P − 37	P − 49	P − 50
Load	Yield Load P_y (tf)	7.1	7.2	5.9
	Maximum Load P_u (tf)	8.4	8.1	7.4
	P_u/P_y	1.18	1.13	1.25
Displacement	Yield Displacement δ_y (cm)	0.91	1.04	0.88
	Ultimate Displacement δ_u (cm)	5.7	5.4	4.1
	δ_y/δ_u	6.3	5.2	4.7

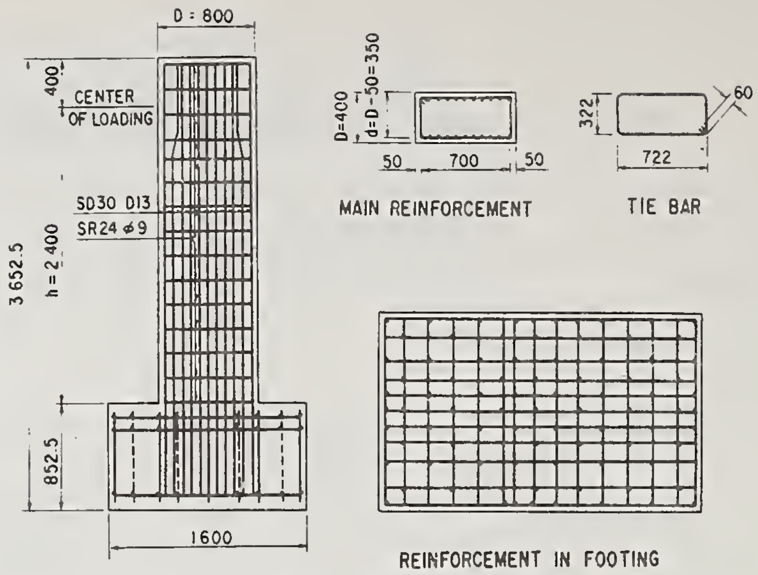


Fig.1 Test Specimens for Bilateral Loading

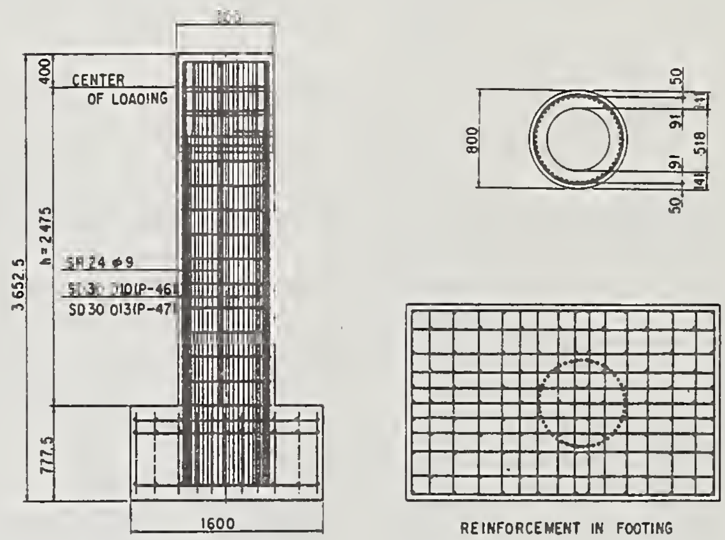


Fig.2 Test Specimens with Circular Hollow Section

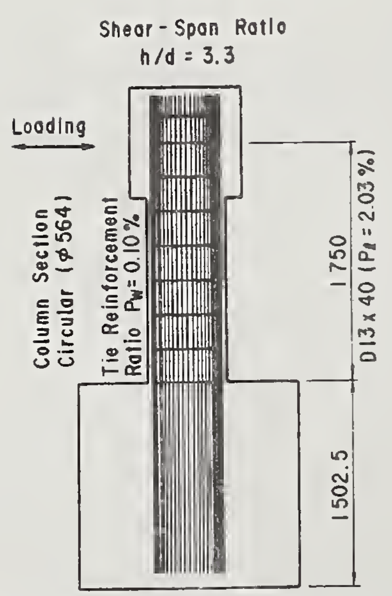


Fig.3 Test Specimens with Circular Section

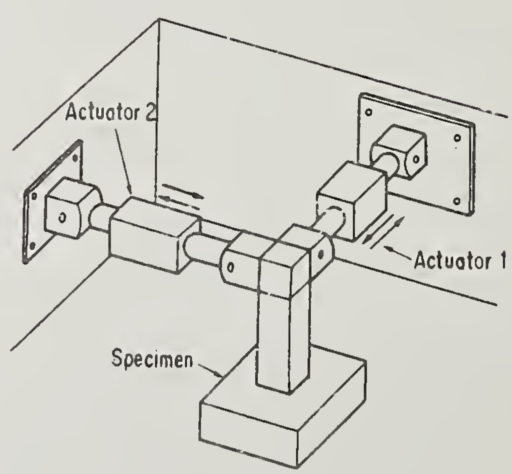
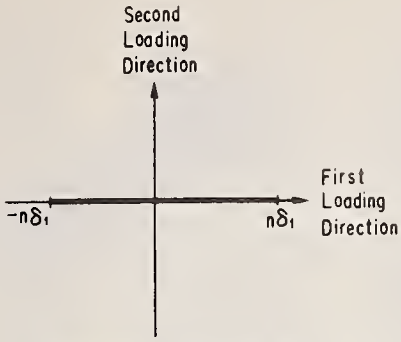
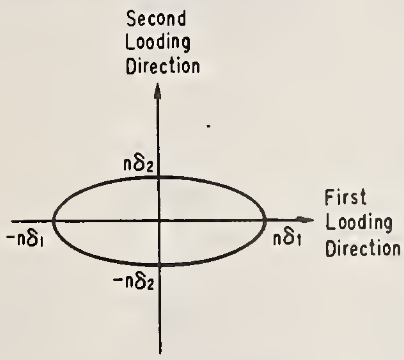


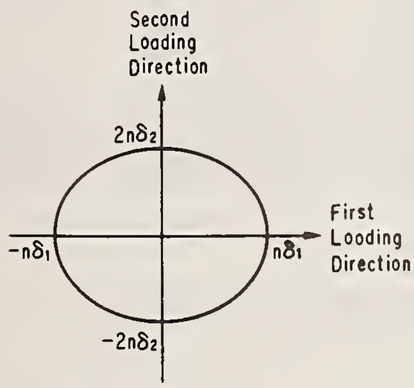
Fig.4 Set-up for Bilateral Loading



(a) Unilateral Loading in P-37

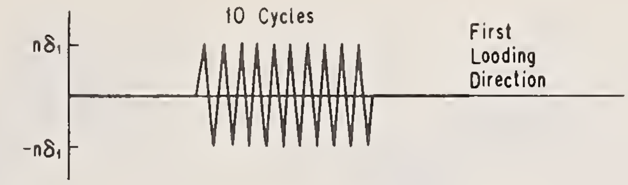


(b) Bilateral Loading in P-49

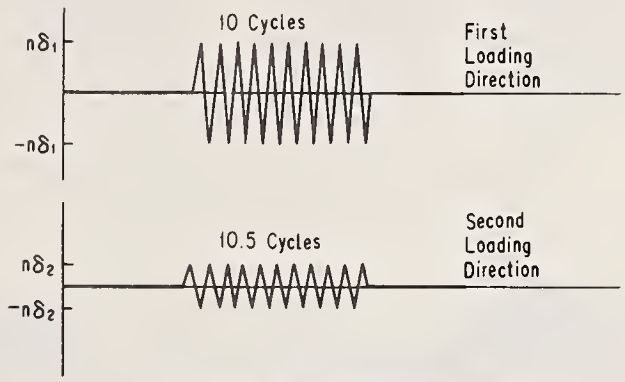


(c) Bilateral Loading in P-50

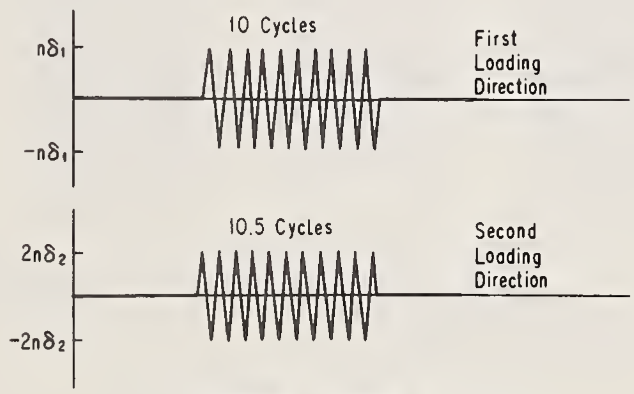
Fig.5 Loading Locus in Unilateral and Bilateral Loading



(a) Unilateral Loading in P-37



(b) Bilateral Loading in P-49



(c) Bilateral Loading in P-50

Fig.6 Loading Hysteresis in First and Second Direction

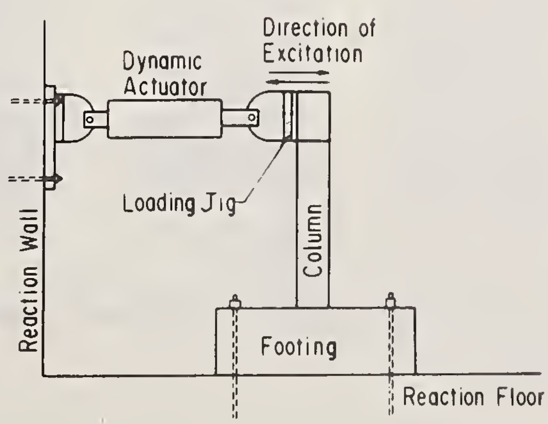
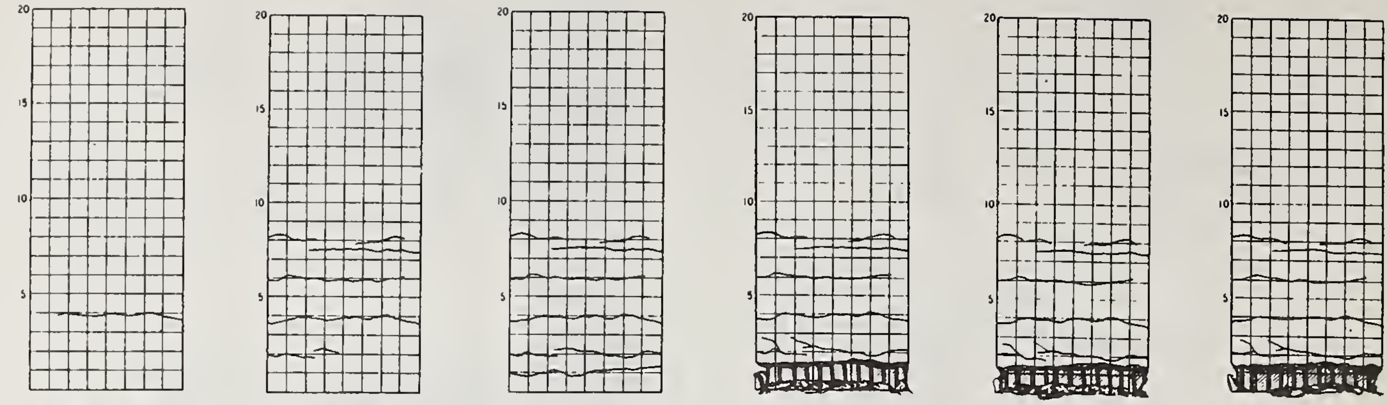
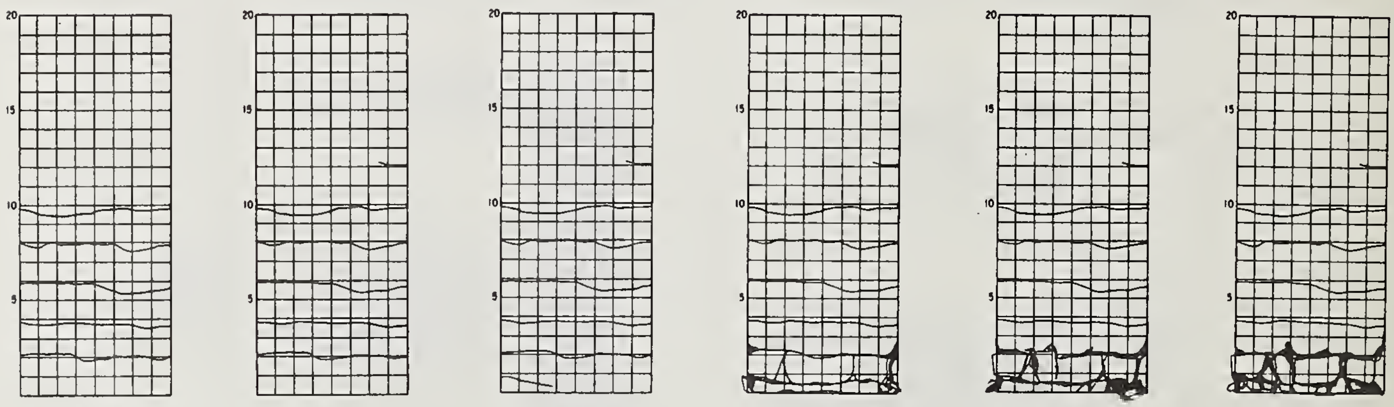


Fig.7 Experimental Set-up in Circular Hollow Section



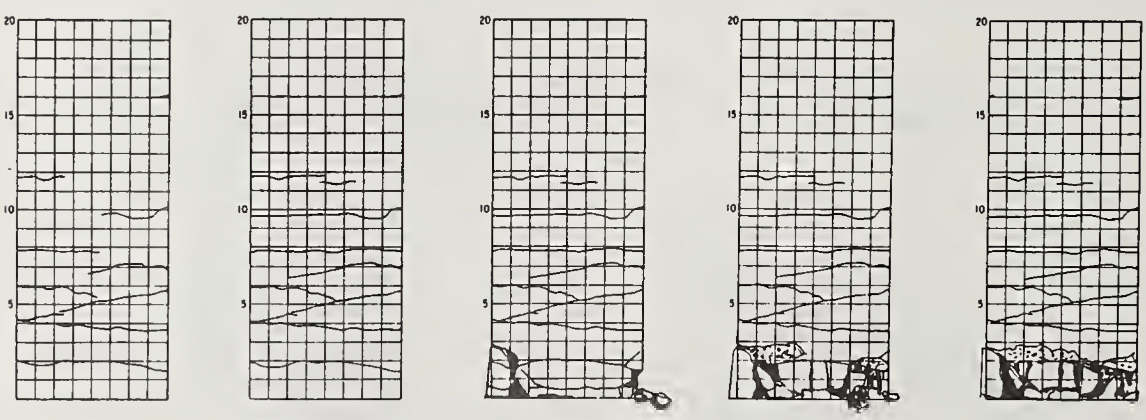
(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5 (f) step 6

Fig.8 Failure Mode for Specimen with Unilateral Loading (P-37)



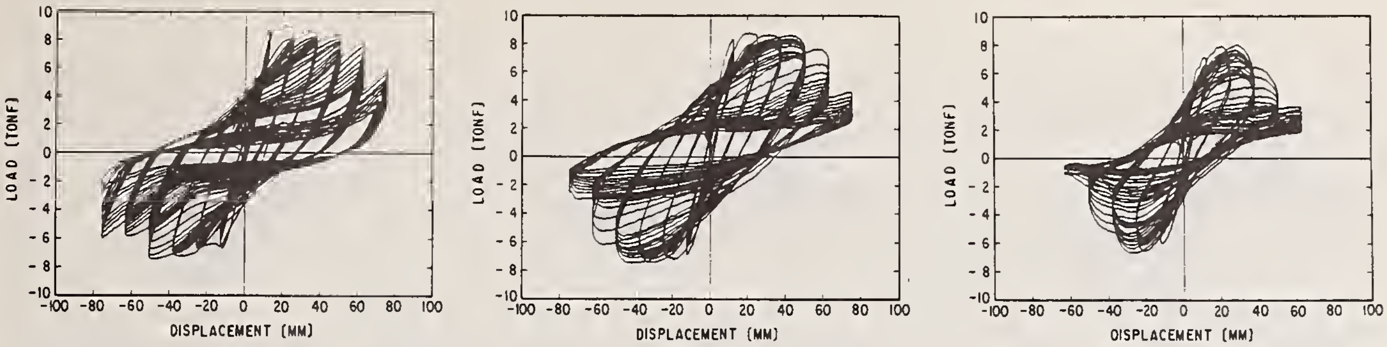
(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5 (f) step 6

Fig.9 Failure Mode for Specimen with Bilateral Loading (P-49)



(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5

Fig.10 failure Mode for Specimen with Bilateral Loading (P-50)



(a) Unilateral Loading (P-37) (b) Bilateral Loading (P-49) (c) Bilateral Loading (P-50)

Fig.11 Comparison of Hysteresis Loops in First Direction

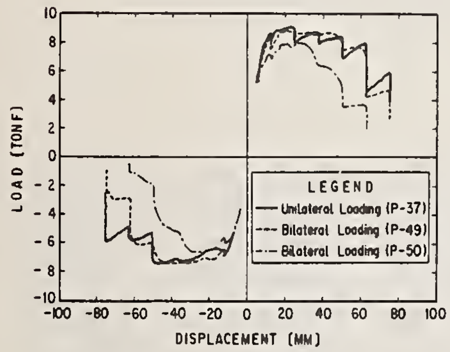
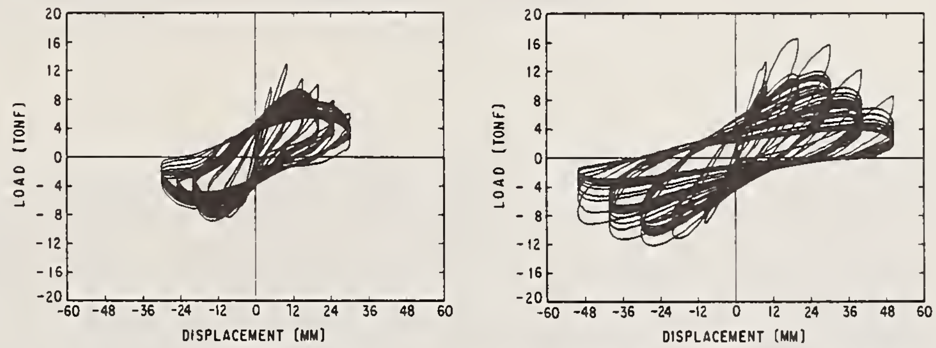


Fig.12 Comparison of Envelope of Hysteresis Loops



(a) Bilateral Loading (P-49) (b) Bilateral Loading (P-50)

Fig.13 Comparison of Hysteresis Loops in Second Direction

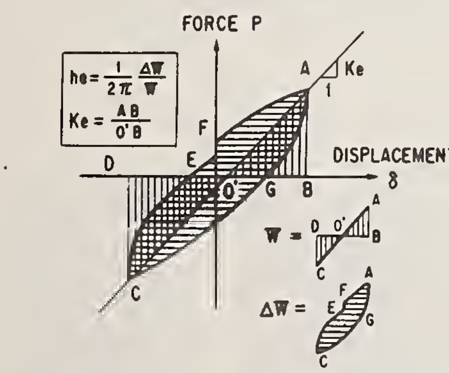


Fig.14 Definition of Equivalent Hysteretic Damping Ratio

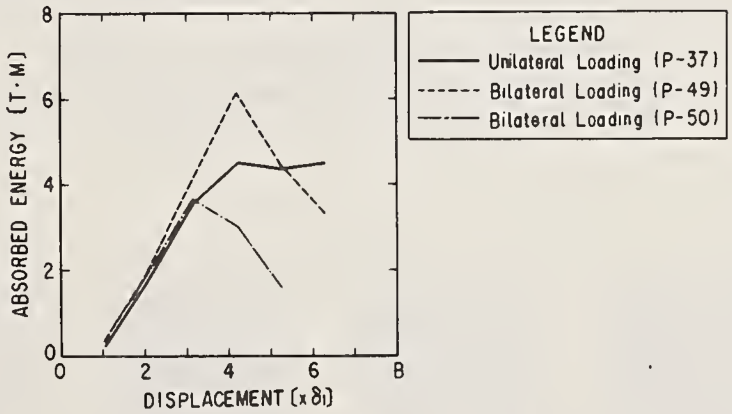


Fig.15 Comparison of Energy Dissipation

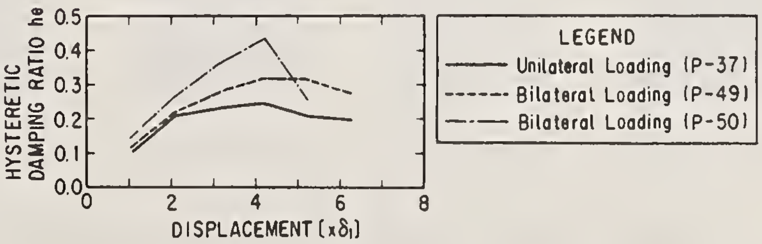
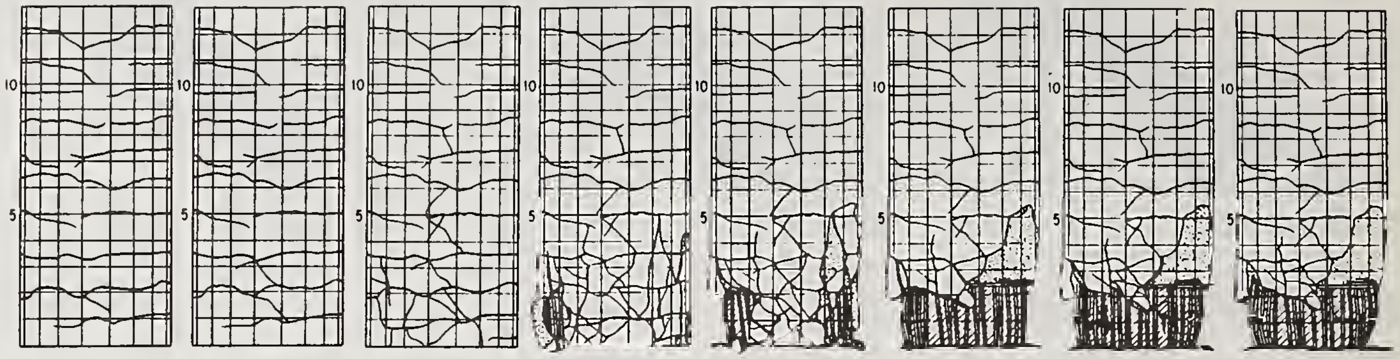
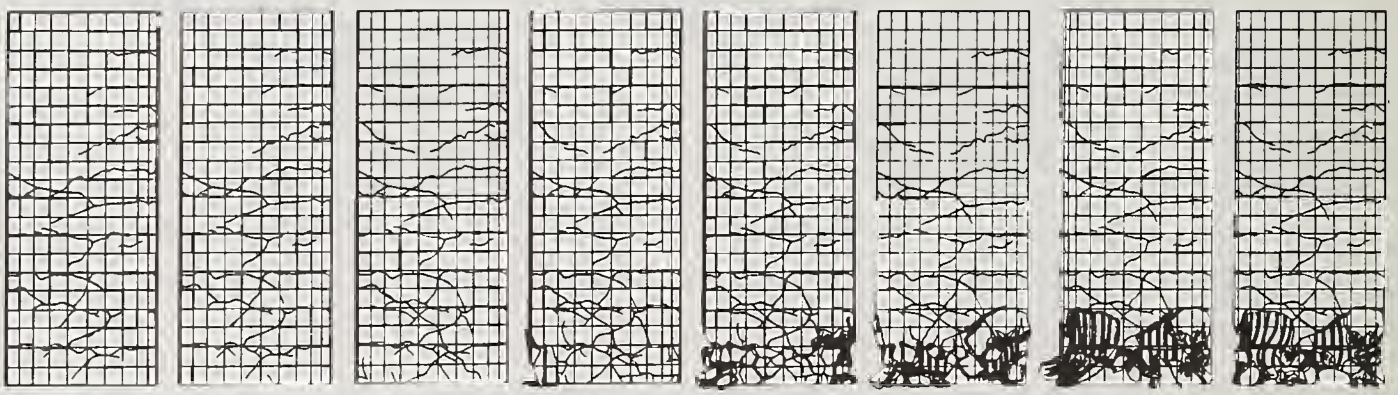


Fig.16 Comparison of Equivalent Hysteretic Damping Ratio



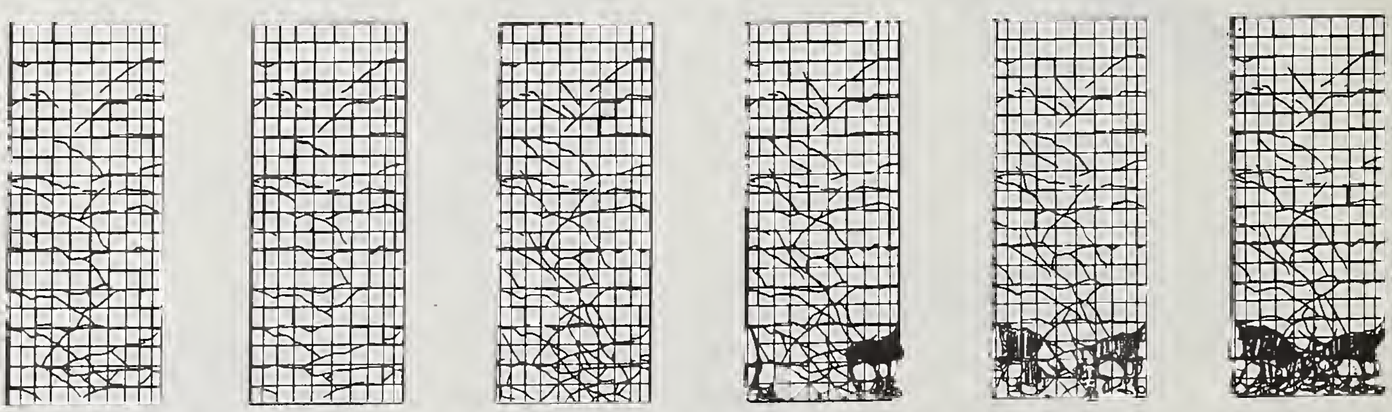
(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5 (f) step 6 (g) step 7 (h) step 8

Fig.17 Failure Mode for Specimen with Circular Section (P-29)



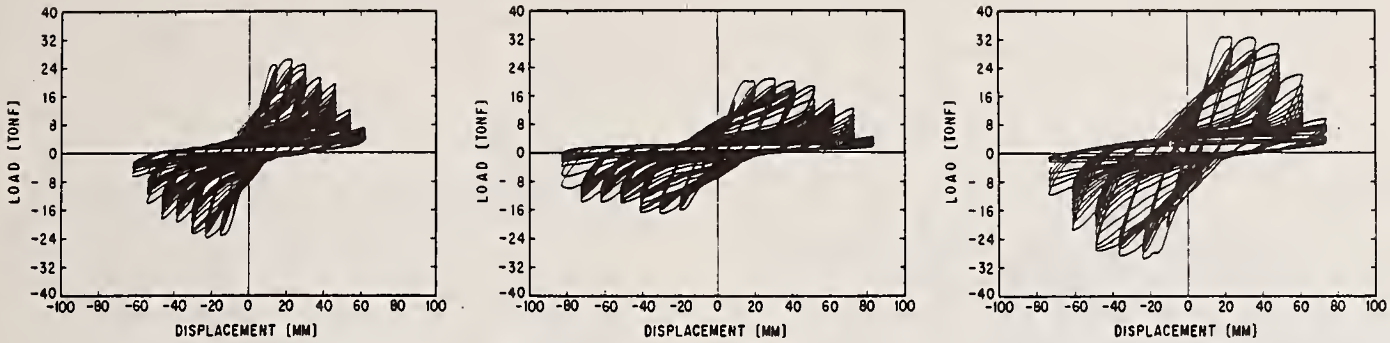
(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5 (f) step 6 (g) step 7 (h) step 8

Fig.18 Failure Mode for Specimen with Circular Hollow Section (P-46)



(a) step 1 (b) step 2 (c) step 3 (d) step 4 (e) step 5 (f) step 6

Fig.19 Failure Mode for Specimen with Circular Hollow Section (P-47)



(a) Circular Section (P-29)

(b) Circular Hollow Section (P-46)

(c) Circular Hollow Section (P-47)

Fig.20 Comparison of Hysteresis Loops

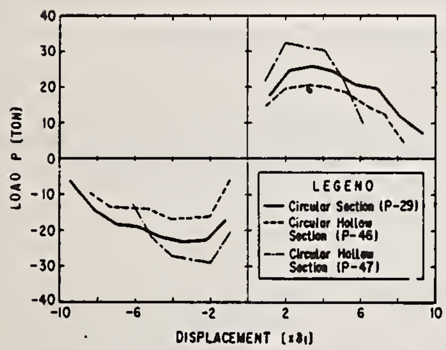


Fig.21 Comparison of Envelope of Hysteresis Loops

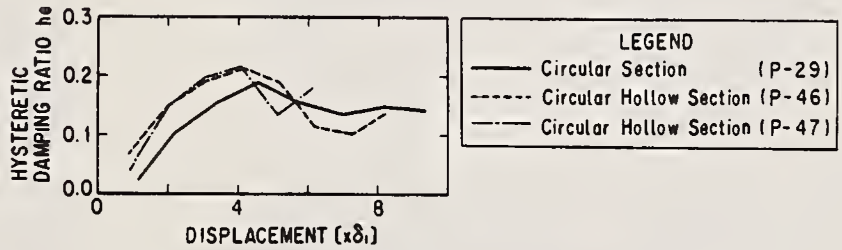


Fig.22 Comparison of Equivalent Hysteretic Damping Ratio

U.S. Coordinated Program for Masonry Building Research—A Fourth Year Status

by

James L. Noland[1]

ABSTRACT

The U.S. Coordinated Program for Masonry Building Research is comprised of 28 specific research tasks. The primary purpose is to support development of a limit state design standard for masonry buildings. The. U.S. program has, in turn, been coordinated with a parallel program in Japan. Overall, the U.S. program is approximately 50% complete. Several individual U.S. research Tasks are complete and others well underway. Many technical reports have been published and significant technical accomplishments made.

KEYWORDS: Reinforced masonry; TCCMAR; Limit state design; status.

1. INTRODUCTION

1.1 Overview

The U.S. Coordinated Program for Masonry Building Research is an integrated set of specific research tasks being executed by the U.S. Technical Coordinating Committee for Masonry Research (TCCMAR/U.S.). The U.S. Program, in turn, is coordinated with a parallel program conducted in Japan to exchange information and concepts for the mutual benefit of both countries.

The U.S. program addresses structural issues and is intended to provide design & criteria recommendations necessary to support limit state design techniques for reinforced masonry. The program consists of both analytical and experimental research tasks and is organized on a project basis as shown in Figure 1. The individual research tasks are coordinated and time-phased, as shown in Figures 2 and 3, to avoid duplication of effort and to transfer information between the task researchers on a timely basis. Research tasks which comprise the U.S. Coordinated Program are listed in Table 1 with a brief statement of purpose for each. This paper is an update of the previous status report (1) and is based upon a detailed status report of November 1988 (2).

1.2 Current Status

By May 1989 the U.S. program will have been under way for approximately four years. This paper briefly reviews the activities, progress, and accomplishments of the program during that period.

2. ADMINISTRATIVE CHANGES

Dr. H.S. Lew of the National Institute of Standards and Technology (USA) has replaced Dr. Scribner as TCCMAR/U.S.-UJNR Co-Liaison. Refer to Figure 1.

Gregg Borchelt, Director of Engineering, Brick Institute of America has replaced G. Dalrymple on the Industry Participation Panel and Ro Whitlock on the Industry Observers Panel. Refer to Figure 1.

3. PROGRESS

3.1 Individual Task Status

The status of individual research Tasks in terms of percent completion is listed in Table 1. A more complete description of the progress and status of each task may be found in reference (2). The completion percentages are based upon the total time period of each task as opposed to the time period of Fall 1985 to December 31, 1987 used in the previous report (1).

3.2 Reports

The number of technical reports published by TCCMAR/U.S.researchers continues to increase. Arrangements have been made so that the reports are available to anyone from the Earthquake Engineering Library in

[1] Atkinson-Noland & Associates
Boulder, CO 80302, U.S.A.

Berkeley. The reports available are listed in Table 2.

A large number of technical papers have been presented and/or published in various meetings and conferences including JTCCMAR meetings. These are listed in reference (2).

3.3 Design Standards Development

A critical issue in the U.S. is, and has been the transfer of technology developed by research into practice. In the U.S. an important factor in implementing research into practice is the incorporation or provision for new technology by the design standards.

In the U.S., design standards are developed by non-governmental professional groups using various forms of consensus processes. Hence, the NSF-supported U.S. Coordinated Program for Masonry Building Research cannot develop standards, but will develop recommendations (Task 10).

Discussions and negotiations over the last few months have served to establish mechanisms and processes for transfer of technology (i.e. research data and recommendations) to a consensus design standards development committee sponsored by U.S. professional societies. A one-year pre-standardization effort to collect and document background information pertinent to limit state structural design and to develop a draft limit state design standard for masonry has been organized. The pre-standardization work along with the technology developed by the coordinated masonry research will be utilized by the national masonry standards committee to produce a limit state design standard for masonry. The basics of the process are illustrated by Figure 4.

4. TECHNICAL ACCOMPLISHMENTS

The U.S. program is proceeding generally according to expectations. At this point in time, which is about the half-way point, technical accomplishments are beginning to become apparent. It must be recognized, however, that the major technical results are yet to come.

Because the primary objective of the U.S. program is to provide a basis for limit state design and an associated generic modeling capability, the following seem significant among the results achieved to date:

a. A finite element model (FEM) has been formulated and implemented capable of post-peak strength response of reinforced shear walls. The material models have the capability to represent the reduction in peak compressive strength due to tensile cracking and allow reorientation of initial cracks to a final crack state. The FEM is formulated such that Gauss points are essentially independent thereby allowing use of a coarser mesh in modeling. Correlation with experimental results has been very good. Work will continue in this area to verify correlation with experimental results.

b. Structural component models (SCM) can now predict yield and maximum force levels for in-plane flexural reinforced masonry walls. Correlations with experimental results are very good. Studies have led to the development of capacity reduction factors for in-plane flexural masonry walls. Work will continue to develop a family of SCM's for system analyses.

c. A non-linear lumped parameter model (LPM) has been developed to do time-history analyses of masonry structures. A new non-linear "spring" element has been introduced to represent masonry shear walls in building system models. Studies have been done to understand the dynamic behavior of masonry structures under ground motions with different v/a ratios. The LPM has been extended and used to provide understanding of the dynamic behavior of masonry structures allowed to uplift.

For future design applications and studies, U.S. ground motions have been categorized with respect to v/a rates.

d. A database has been developed for the cyclic behavior of single-story reinforced masonry shear walls with different in-plane compressive loads and reinforcement rates. Limit states have been identified for application in the design recommendations to be made.

e. Tests of two-story coupled masonry shear walls have revealed the need for high accuracy in the modeling of the coupling beams. Limit states for this type of system were identified.

f. Force-deflection characteristics of prestressed precast concrete plank floor diaphragms have been established experimentally both parallel and perpendicular to plank direction. Limit states have been identified. Data has been accumulated for other floor systems as well. This information will be the basis of spring elements for use in the lumped parameter models.

g. Reinforced concrete and clay masonry walls responded elastically when subjected to out-of-plane dynamic loads typical of design ground motion in soil type 1 in the highest U.S. seismic zones. The walls had reinforcement ratios of 0.07 balanced to 0.5 balanced, vertical loads of 0 to 80 psi (36.3kg) and slenderness ratios of 40 to 50. Under design motions, maximum lateral displacements were about 6 inches (2.4 cm). For higher levels of ground motion, the walls sustained vertical loads, but some permanent lateral deflection was noted.

h. Tests of grouted hollow clay and concrete unit prisms in compression indicated that masonry is a generic material, i.e., that no unusual behavioral traits are present to separate grouted clay and concrete masonry. The compressive tests established stress-strain curves including the post-peak descending branch and limit state values. Tests of prisms loaded in eccentric compression established strain gradient effects and K1, K2, K3 stress block values for strength design.

5. RESOLUTIONS OF THE FOURTH JTCCMAR MEETING

1. The fourth meeting of JTCCMAR was held in San Diego, California, U.S.A. on October 17, 18, and 19, 1988. The meeting was attended by 18 researchers from the United States, 16 researchers from Japan, 7 observers from the United States and 10 observers from Japan.

2. There was a successful exchange of information and ideas between the researchers from both countries on the subjects of masonry material behavior, behavior of assemblies, behavior of systems, testing procedures, analytical/modeling methods, results of the Japanese tests on a 5-story building, seismic design procedures and future work.

3. It was recognized that, while progress has been made in developing an understanding of the behavior of masonry, there is still an urgent need for additional information. The participants therefore recommend that sponsoring agencies take appropriate measures to continue support for masonry research.

4. The participants recognize that both countries have many common research needs regarding masonry construction as well as mechanical properties of masonry units, components, and systems.

5. It was recognized that continuous and in-depth effort is required to establish the

seismic performance characteristics of masonry structures and to develop improved construction and design procedures. However, the participants realize that the research programs of both countries primarily address central issues. They recognize that, while the knowledge developed will provide a basic foundation for improved masonry technology, many other important issues exist which should be addressed.

6. Significant progress has been made by both sides in testing and modeling masonry materials, components, subassemblages, and prototypes which will lead to a better understanding of masonry structural behavior and to the development of adequate seismic design procedures for masonry buildings.

7. The participants recognized that the full-scale test carried out in japan has and will make a great contribution to the understanding of the overall behavior and performance of masonry structural systems. They recommend that another full-scale test on a masonry structural system be carried out in the United States. It is also recommended that information obtained from both full-scale tests be exchanged.

8. The technical communication which has been carried out by sub-groups and individuals of TCCMAR/U.S. and TCCMAR/Japan has been valuable and should be continued. Researchers should be exchanged as required to enable a meaningful exchange of concepts and information between individuals and groups with similar research interest.

9. The participants from both countries agree that proper seismic performance of masonry structures requires consideration of damage control and minimization of life safety threats.

10. It was recognized that the joint project will result in an improved technological base for the design of RM buildings.

11. It was recognized that the results of this coordinated research program could also be beneficial to other countries which have a seismic hazard, especially developing countries.

12. Considering that the U.S. program is planned to continue until 1993 and considering the benefits of the mutual exchange between the U.S. and Japanese programs, it is hoped that the joint relationship be continued so the mutual benefits may be maximized.

13. The participants realized that a reinforced masonry building association should be established in Japan as soon as possible to promote, by continuous effort, the development and promulgation of a more rational design procedure for RM buildings, including the low-rise category, so that the joint U.S.-Japan relationship may be continued.

14. It was recognized, by both sides, that new concepts in masonry technology should be investigated to enhance economy and reliability.

15. The fifth meeting of JTCCMAR will be held in Japan in the fall of 1989.

6. ACKNOWLEDGEMENTS

Primary financial support for the U.S. side of the Joint program has been provided by the National Science Foundation. Dr. A.J. Eggenberger is the cognizant program manager. The support of the U.S. masonry industry is gratefully acknowledged.

The benefits of the U.S.-Japan cooperation are recognized. The

generous sharing of data and experience by TECCMAR/Japan has been most welcome.

7. REFERENCES

1. Proceedings: 20th Joint Meeting of the U.S.-Japan Cooperative Program in Natural Resources - Panel on Wind and Seismic Effects, N.J. Raufaste, ed., NIST, January 1989.

2. Status Report: U.S. Coordinated Program for Masonry Building Research, TCCMAR/U.S., November 1988.

U.S - JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

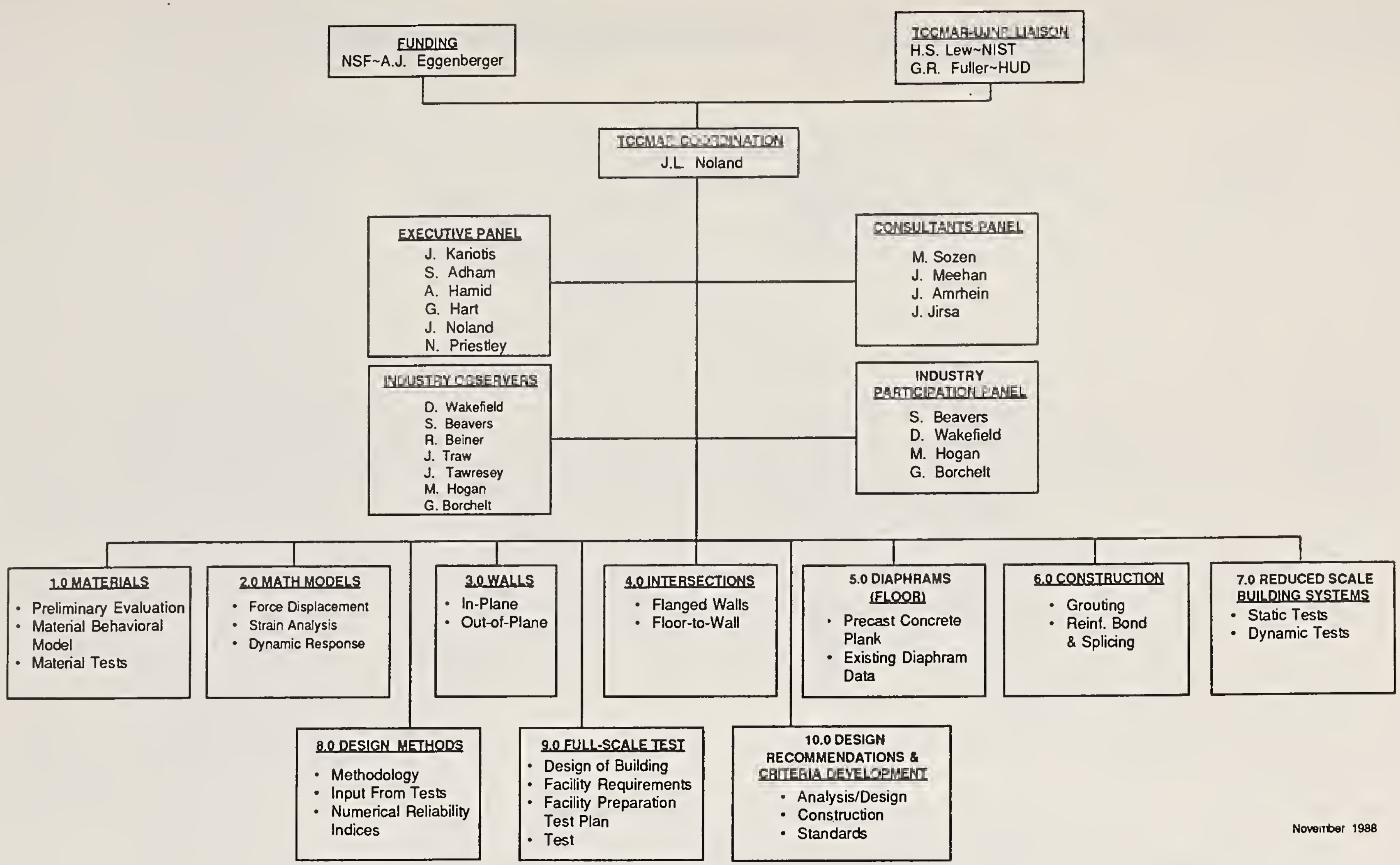


FIGURE 1 - TCCMAR ORGANIZATION (U.S.)

U.S.-JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

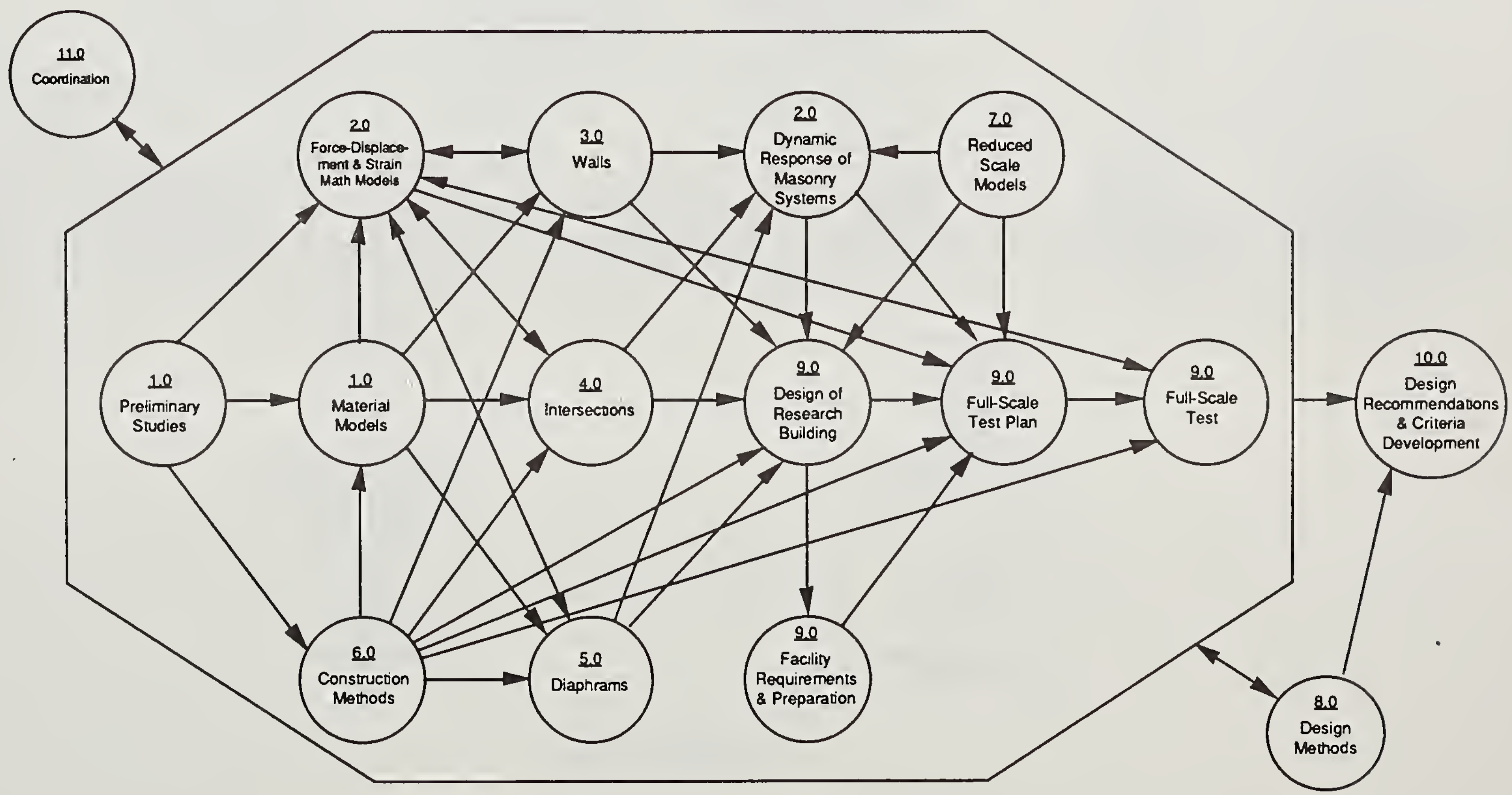


FIGURE 2 — TASK INGREDIENTS-DEPENDENCE CHART (U.S. PROGRAM)

November 1988

U.S.-JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

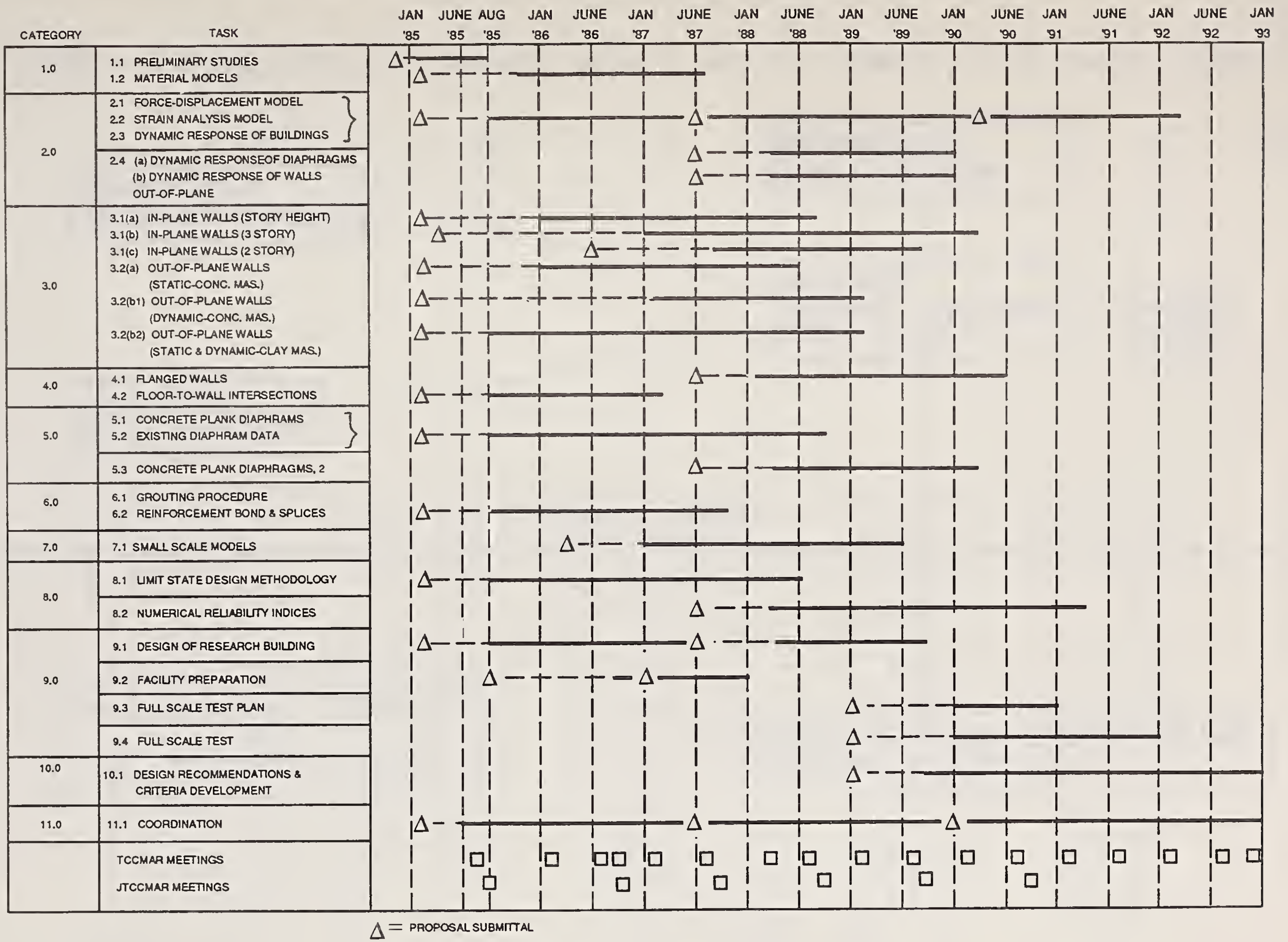


FIGURE 3 — TASK SCHEDULE (U.S. PROGRAM)

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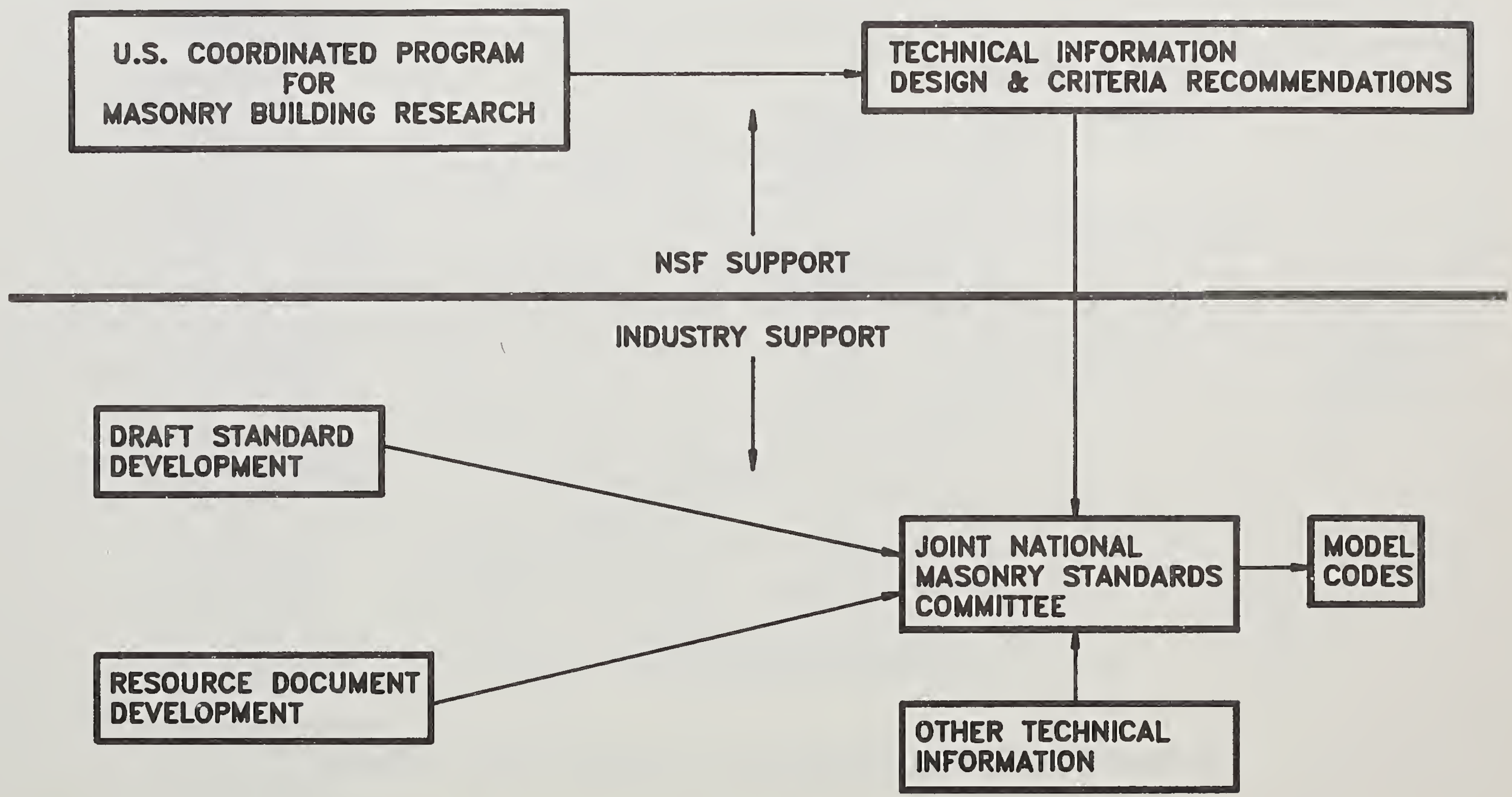


FIGURE 4 – INCORPORATION OF RESEARCH RESULTS INTO THE STANDARDS DEVELOPMENT PROCESS

TABLE 1 - RESEARCH TASKS COMPRISING THE U.S. COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

Category	Task (Researcher)	% Complete	Title - Purpose
1.0	1.1 (Atkinson)	100	Preliminary Material Studies -- To compare the behavior of clay and concrete unit masonry. To provide a basis for selection of the type or types of masonry to be used in subsequent tasks. To establish standardized materials test procedures for all the experimental tasks.
1.0	1.2 (Hamid) (Brown)	95	Material Models -- To measure the parameters required for development of the flexural compression stress-block. To determine uniaxial and biaxial material properties for analytical models (Tasks 2.1 and 2.2) including post-peak behavior. To evaluate non-isotropic behavior.
1.0	1.3 (Atkinson)	-0-	Material Tests -- To critically review and assess existing tests of masonry materials and assemblages to determine the usefulness of data produced with respect to the needs of analytical models and design methodology developed in the program. To revise existing tests as required and/or suggest new tests. The work will be done in coordination with Category 2 and 10 Tasks to establish accuracy requirements.
2.0	2.1 (Englekirk)	60	Force-Displacement Models for Masonry Components -- To develop force-displacement mathematical models which accurately characterize reinforced masonry components under cyclic loading to permit pretest predictions of experimental results. To develop models suitable for parameter studies and models suitable for design engineering.
2.0	2.2 (Ewing)	65	Strain Analysis Model for Masonry Components -- To develop a strain model for reinforced masonry components in conjunction with Task 2.1. To identify regions of large strain thus assisting in experimental instrumentation planning. To develop a simplified model to be used to provide data for strength design rules and in-plane shear design procedures.

TABLE 1

Category	Task (Researcher)	% Complete	Title-Purpose
2.0	2.3 (Kariotis)	65	Dynamic Response of Masonry Buildings -- To develop a generalized dynamic response model to predict interstory displacements using specified time histories. To correlate force-displacement models and to investigate force-displacement characteristics of structural components in the near-elastic and inelastic displacement range. To provide data for building test planning.
2.0	2.4(a) (Porter)	-0-	Dynamic Response of Diaphragms -- To develop an analytical non-linear model of load displacement history of horizontal diaphragms. To provide associated displacements and stiffnesses for an integrated dynamic spring model. This Task will provide a computer model extension using a lumped-parameter mass parameter spring of the experimental data collected from Tasks 5.1 and 5.2. This work will provide input for Task 2.3.
2.0	2.4(b) (Mayes) (Sveinsson)	15	Dynamic Out-of-Plane Response of Reinforced Masonry Walls -- To develop analytical models based upon the results of Task 3.2(b) which can be used to predict out-of-plane response of masonry walls of various shapes, sizes, and internal construction. To conduct response studies based on independent variation of parameters. The models will interface with the models of Tasks 2.3 and 2.4(a).
3.0	3.1(a) (Shing)	90	Response of Reinforced Masonry Story-Height Walls to Fully Reversed In-Plane Lateral Loads -- To experimentally establish the behavior of story-height walls subjected to small and large amplitude axial force, and bending moments considering various reinforcement ratios and patterns.
3.0	3.1(b) (Hegemier) (Seible)	10	Development of a Sequential Displacement Analytical and Experimental Methodology for the Response of Multi-Story Walls to In-Plane Loads -- To develop a reliable test methodology for investigating structural response, through integrated analytical and experimental studies of three-story reinforced hollow unit masonry walls. The methodology will be the basis of studying the response of a full-scale masonry research building in Task 9.4. To develop analytic models in conjunction with Tasks 2.1, 2.2, and 2.3.

TABLE 1

Category	Task (Researcher)	% Complete	Title-Purpose
3.0	3.1(c) (Klingner)	40	Response of Reinforced Masonry Two-Story Walls to Fully Reversed In-Plane Lateral Loads -- To establish the behavior of two-story walls subjected to small and large amplitude reversals of in-plane lateral deflections, axial force and bending moments considering the effect of openings, floor-wall joint details, reinforcement ratios and coupling between shear walls. To develop analytical models in conjunction with Tasks 2.1, 2.2, and 2.3.
3.0	3.2 (Hamid) (Mayes) (Harris)	95	Response of Reinforced Masonry Walls to Out-of-Plane Static Loads -- To verify the behavior of flexural models developed using material models, to evaluate the influence of unit properties, bond type and reinforcement ratios upon wall behavior. To provide stiffness data for correlation with dynamic wall test results [(Task 3.2(b)].
3.0	3.2 (Adham) (Mayes)	100	Response of Reinforced Masonry Walls to Out-of-Plane Dynamic Excitation -- To experimentally determine effects of slenderness, reinforcement amounts and ratios, vertical load and grouting on dynamic response as needed for mathematical response models and the development design coefficients for equivalent static load methods.
4.0	4.1 (Priestley)	20	Response of Flanged Masonry Shear Walls to Dynamic Excitation -- To experimentally investigate the dynamic behavior of flanged shear walls, in particular, the behavior of T-section walls and the significance of dynamic, as opposed to static or quasi-static testing, for in-plane loading. To develop analytical models to investigate the flange-web shear lag phenomena, and to identify the interaction between flange width, height, reinforcement content, and ductility level. (In conjunction with Tasks 2.1, 2.2, and 2.3).
4.0	4.2 (Hegemier)	80	Floor-to-Wall Intersections of Masonry Buildings -- To determine the effectiveness of intersection details to connect masonry wall components. To construct a nonphenomenological analytical model of intersection behavior for use in building system models.

TABLE 1

Category	Task (Researcher)	% Complete	Title-Purpose
5.0	5.1 (Porter)	85	Concrete Plank Diaphragm Characteristics -- To investigate experimentally concrete plank diaphragm floor diaphragms with stiff supports to determine modes of failure and stiffness characteristics including yielding capacity in terms of distortion as needed for masonry building models.
5.0	5.2 (Johnson) (Porter)	85	Assembly of Existing Diaphragm Data -- To assemble extensive existing experimental data on various types of floor diaphragms, to reduce to a form required for static and dynamic analysis models.
5.0	5.3 (Porter)	-0-	Concrete Plank Diaphragm Characteristics Continuation -- To investigate experimentally the behavior of concrete plank floor diaphragms with flexible supports to determine modes of failure and stiffness characteristics including yielding capacity in terms of distortion as needed for masonry building models.
6.0	6.1 (TBD)	-0-	Grouting procedures for Hollow Unit Masonry -- To identify methods of grouting hollow unit masonry such that the cavity is solidly filled and reinforcement is completely bonded.
6.0	6.2 (Tulin)	100	Reinforcement Bond and Splices in Grouted Hollow Unit Masonry -- To develop data and behavioral models on the bond strength and slip characteristics of deformed bars and lap splices in grouted hollow unit masonry, as needed for building modeling.
7.0	7.1 (Abrams)		Small Scale Models -- To provide experimental test data on the dynamic behavior of three-story reinforced concrete masonry buildings built with 1/4 scale hollow concrete units. To demonstrate the viability of constructing and dynamically testing reduced scale building system models for basic behavior studies.
8.0	8.1 (Hart)	95	Limit State Design Methodology for Reinforced Masonry -- To select an appropriate limit state design methodology for masonry. To select and document a procedure to compute numerical values for strength reduction factors. To review program experimental research tasks to assure that statistical benefits are maximized and proper limit states are investigated.

TABLE 1

Category	Task (Researcher)	% Complete	Title-Purpose
8.0	8.2 (Hart)	15	Numerical Reliability Indices -- To develop numerical values of statistically-based strength reduction factors using program experimentally developed data, other applicable data, and judgment. To complete development of the methodology.
9.0	9.1 (Kariotis)	100	Design of Reinforced Masonry Research Building - Phase 1 -- To develop the preliminary designs of the potential research buildings which reflect a significant portion of modern U.S. masonry construction. to select a single configuration in consultation with TCCMAR which will be used as a basis for defining equipment and other laboratory facilities using methods developed in Category 2 tasks and the associated load magnitudes and distributions. Phase 2 - To prepare final drawings and specifications for construction of the five-story test specimen.
9.0	9.2 (Hegemier) (Seible)	75	Facility Preparation -- Define, acquire, install and check-out equipment required for experiments on a full-scale five-story reinforced masonry research building.
9.0	9.3 (Hegemier) (Seible) (Priestley)	-0-	Full Scale Masonry Research Building Test Plan -- To develop a detailed and comprehensive plan for conducting static load-reversal tests on a full-scale five-story reinforced masonry research building.
9.0	9.4 (Hegemier) (Seible) (Priestley)	-0-	Full Scale Test -- To conduct experiments on a full-scale five-story reinforced masonry research building in accordance with the test plan and acquiring data indicated. To observe building response and adjust test procedures and data measurements as required to establish building behavior.
10.0	10.1 (Noland)	-0-	Design Recommendations and Criteria Development -- To develop and document recommendations for the design of reinforced masonry building subject to seismic excitation in a manner conducive to design office utilization. To develop and document corresponding recommendations for masonry structural code provisions.

TABLE 1

Category	Task (Researcher)	% Complete	Title-Purpose
11.0	11.1 (Noland)	60	Coordination -- To fully coordinate the U.S. research tasks. To enhance data transfer among researchers and timely completion of tasks. To schedule and organize TCCMAR and Executive Panel meetings. To establish additional program policies as the need arises. To stimulate release of progress reports and dissemination of results. To coordinate with industry for the purposes of informing industry and arranging industry support. To interface with NSF and UJNR on overall funding and policy matters.

TABLE 2 - RESEARCH TASK REPORTS

Task No.	Author/s - Title
1.1-1:	Atkinson and Kingsley, Comparison of the Behavior of Clay & Concrete Masonry in Compression, September 1985.
1.2(b)-1:	Young, J.M., Brown, R.H., Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient, May 1988.
2.1-1:	Hart, G and Basharkhah, M., Slender Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01), September 1987.
2.1-2:	Hart, G. and Basharkhah, M., Shear Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01), September 1987.
2.1-3:	Nakaki, D. & Hart, G., Uplifting Response of Structures Subjected to Earthquake Motions, August 1987.
2.1-4:	Hart, G., Sajjad, N., and Basharkhah, M., Inelastic Column Analysis Computer Program (INCAP, Version 1.01), March 1988.
2.3-1:	Ewing, R., Kariotis, J. El-Mustapha, A., LPM/I, A Computer Program for Nonlinear, Dynamic Analysis of Lumped Parameter Models, August 1987.
2.3-2:	Ewing, R., El-Mustapha, A., Kariotis, J., Influence of Foundation Model on the Uplifting of Structures, July 1988.
3.1a-1:	Scrivener, J., Summary of Findings of Cyclic Tests on Masonry Piers, June 1986.
3.1b-1:	Seible, F. and LaRovere, H., Summary of Pseudo Dynamic Testing, February 1987.
4.1-1:	Limin, H., Priestley, N., Seismic Behavior of Flanged Masonry Shear Walls, May 1988.
4.2-1:	Hegemier, G., Murakami, H., On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part I: Experimental
4.2-2:	Hegemier, G., Murakami, H., On the Behavior of Floor-to-Wall Intersections in Concrete Masonry construction: Part II: Theoretical
6.2-1:	Scribener, J., Bond of Reinforcement in Grouted Hollow-Unit Masonry: A State-of-the-Art, June 1986.
6.2-2:	Soric, Z. and Tulin, L., Bond Splices in Reinforced Masonry, August 1987.
8.1-1:	Hart, G., A Limit State Design Method for Reinforced Masonry
9.1-1:	Kariotis, J.C., Johnson, A.W., Design of Reinforced Masonry Research Building, September 1987.
9.2-1:	Seible, F., Report on Large Structures Testing Facilities in Japan, September 1985.
9.2-2:	Seible, F., Design and Construction of the Charles Lee Powell Structural Systems Laboratory, November 1986.
9.2-3:	Seible, F., The Japanese Five-Story Full Scale Reinforced Masonry Building Test, January 1988.
11.1-1:	TCCMAR, Summary Report: U.S. Coordinated Program for Masonry Building Research, September 1985 to August 1986.

Theme V

Code Development Process and Code Enforcement Responsibility

Minister-of-Construction's Agreement System in the Building Standard Law of Japan

by

Tatsuo Murota

ABSTRACT

The Minister-of-Construction's Agreement System in the Building Standard Law of Japan is divided by five categories. They are concerned with the followings:

(1) Special building materials or methods of construction unanticipated under the law.
(2) Materials, building elements or equipments having property equal or superior to that specified in the law or cabinet orders.
(3) Structural design of tall buildings exceeding 60m in height.
(4) Computer programs used for the structural calculation of buildings.
(5) The bearing capacity of wooden shear walls.

This paper describes the outline of those categories of the Agreement System.

KEYWORDS

Building Standard Law, Ministry-of-Construction's Agreement System

1. INTRODUCTION

The purpose of the Building Standard Law (hereinafter Law) is to safeguard the life, health and properties of the people by providing minimum standards concerning the site, structure, equipment and use of buildings, and thereby to contribute to the furtherance of the public welfare.
The minimum standards are provided in Chapter II of the Law and they are supplemented by the Building Standard Law Enforcement Order (hereafter Cabinet Order) or notifications issued by the Ministry of Construction.
Major purposes of the Minister-of-Construction's Agreement System (hereinafter Agreement System) are to make the minimum standards responsible to technology development and to supplement the imperfection of the minimum standards.
The Agreement System are divided into five categories according to the purposes:

(1) Recognition of special building materials or methods of construction unanticipated under the Law.

In Chapter II of the Law, minimum standards are provided concerning the site, structural and equipment of buildings. Article 38 in the Chapter refers to special materials or methods of construction as:

Article 38. The provisions of this Chapter or those of orders or ordinances based thereon shall not apply to buildings using special building materials or methods of construction deems that the said building materials or methods of construction are equal or superior to those specified in the said provisions.
The purpose of this Article is to correspond to technology development by recognizing new materials, new types of structures or construction methods, etc. that are not anticipated under the Law. The first category of Agreement System is based on this Article, and has played an important role in promoting technology development.

(2) Recognition of materials, building elements or equipments having property equal or superior to that specified in the Law or Cabinet Order

Minimum standards concerning materials, building elements or equipments in the Law or Cabinet Order are provided in two ways: (1) by showing examples or (2) by describing minimum requirements. Either of the ways is not perfect to provide minimum standards correctly, and it is necessary to supplement the imperfection by using the expression "equal or superior to".
A typical example of the former is seen in Cabinet Order Article 108:

(Fire Preventive Construction)
Article 108. Fire preventive construction in Article 2 item (8) of the Law shall be as described in each of the following items:

(1) Regarding walls whose studs and beds are made of non-combustible materials, the said fire preventive construction shall be as described in any of the following (a) through (c):
   (a) Metal lath with mortar finish of 1.5cm or more in thickness:
   (b) Cemented excelsior board or gypsum board with mortar or plaster finish of 1cm or more in thickness:
   (c) Cemented excelsior board with metal board over mortar or plaster coating:
(2) ----------
(3) ----------
(4) In addition to those mentioned in the preceding items, the said fire preventive construction shall be that designated by the Minister of Construction upon hearing the opinion of the Director General of the

Fire Defense Agency as having fire preventive property equal or superior to that of the above-mentioned.

Article 108-2 is an example of the latter:

(Non-combustible Materials)
Article 108-2. Building materials having non-combustibility as specified by Cabinet Order under Article 2 item (9) of the Law shall be those which are designated by the Minister of Construction as having the property described in each of the following items (excluding item (2) in the case of those used for external finish of buildings). against the heat of normal fires.
(1) Materials which neither burn nor develop deformation, melting, cracking or other hampering fire protection;
(2) Materials which do not generate smoke or gas hampering fire protection.

The second category of Agreement System is concerned with the recognition of materials, buildings elements or equipments having property equal or superior to that specified in the Law or Cabinet Order.

(3) Recognition of structural design of buildings exceeding 60m in height

Article 20 in the Law provides that the safety of structure shall be confirmed through structural calculation for buildings exceeding a certain limit of scale.
In Cabinet Order Article 81-2, it is provided that "Structural calculation for buildings exceeding 60m in height shall be made by those methods which are recognized by the Minister of Construction as being valid in confirming the structural safety of the building concerned".
The third category of Agreement System is concerned with the provision.

(4) Recognition of computer programs used for the structural calculation of buildings

In cases where a building owner intends to construct any building as specified in Law, he shall submit an application to the building official for confirmation that the plan conforms to the provisions of laws.
Article 1 of the Building Standard Law Enforcement Regulations provided the form of application for confirmations, and also provides that the building administrative agency may establish, by regulations, provisions that, if the building is designed by a qualified architect, all or some of the drawings and documents necessary for confirmation may be omitted.
The fourth category of Agreement System is conceded with the evaluation of computer programs for structural calculation. If a computer programs are recognized to be valid by the Minister of Construction, some parts of structural calculation sheets prepared by a qualified architect using the computer program can be omitted in case of application for confirmation.

(5) Recognition of the bearing capacity of wooden shear walls

Cabinet Order Article 46 requests wooden buildings to be provided with shear walls of a certain extent of total shear strength. A method for calculation of the shear strength is also provided in the article.
According to the method, value of "Multiplier" is necessary for calculation. Multiplier is a physical quantity which is determined by experiments as function of shear strength and rigidity of bearing walls. The fifth category of Agreement System is concerned with recognition of the value of Multiplier for bearing walls.

2. PROCEDURE OF APPLICATION FOR MINISTER-OF-CONSTRUCTION'S AGREEMENT

Procedures of application for the Minister-of-construction's Agreement differ with the categories. Details of the procedures are described in Reference 1), and so the procedures only for category 1 and 2 will be touched upon here.

(1) Procedure of Application for Category 1 Agreement System

The flow of application is as follows (see Fig. 1). The period from the beginning of procedure 2 to the end of procedure 3 is about four months, in general.

① Prior consultations
The applicant is suggested to have prior consultations with the Building Guidance Division of the Ministry of Construction, regarding the application.

② Application for technical examination
The applicant is then required to apply to the Building Center of Japan, for technical examination on the proposed technology.

③ Application for recognition
Subsequently the applicant is required to apply to the Minister of Construction for recognition. In this case, the applicant shall attach a document issued by the Building Center of Japan certifying the result of the technical examination in procedure 2 .

(2) Procedure of Application for Category 2

The flow of application for Category 2 Agreement System is as follows (see Fig. 2). The period from the beginning of procedure 2 to the end of procedure 3 is about three months, in general.

① Implementation of tests
Prior to the application, tests required for the application shall be conducted at the testing institutions designated by the Minister of Construction.

② Application for technical examination
Those materials which have passed the above tests shall be subjected to technical examination at the Building Center of Japan.

③ Application for recognition and designation
Following the above technical examination, the applicant is required to submit the document certifying the result of the examination above to the Minister of Construction to apply for recognition and designation. Those materials which have been recognized to satisfy the specified requirements, will be designated by the Minister of Construction as materials having specific property.

3. NUMBER OF AGREEMENT ISSUED BY THE MINISTER OF CONSTRUCTION

Table 1 shows the number of Agreement's issued by the Minister of Construction for last five years (1984 to 1988 FY).

4. REFERENCES

1) OUTLINE OF THE APPROVAL & CERTIFICATION SYSTEM UNDER THE BUILDING STANDARD LAW, The Building Center of Japan, 1986.

Table 1 Number of Agrement's issued by the Minister of Comstruction in last five years

FIELD	1984	1985	1986	1987	1988
1. High-rise Construction	24	31	42	71	87
2. RC Construction	35	52	58	68	60
3. Steel Construction	29	40	62	85	85
4. Wooden Construction	0	3	9	5	6
5. Foundation	34	38	38	48	61
6. Computer Program	32	15	25	20	21
7. Base Isolation System	0	4	6	8	14
8. Membrane Structure	5	12	12	19	13
9. Elevatory and Playing Equipment	60	66	50	122	173
10. Plumbing	0	0	17	3	3
11. Fire Preventive Construction	384	392	370	336	451
12. Fire Compartment	81	98	76	81	108
13. Separation Wall	11	6	11	18	17
14. Prefabricated Housing	154	132	85	138	158
TOTAL	849	889	861	1022	1257

Fig.1 Application procedure for Category 1 Agrement System

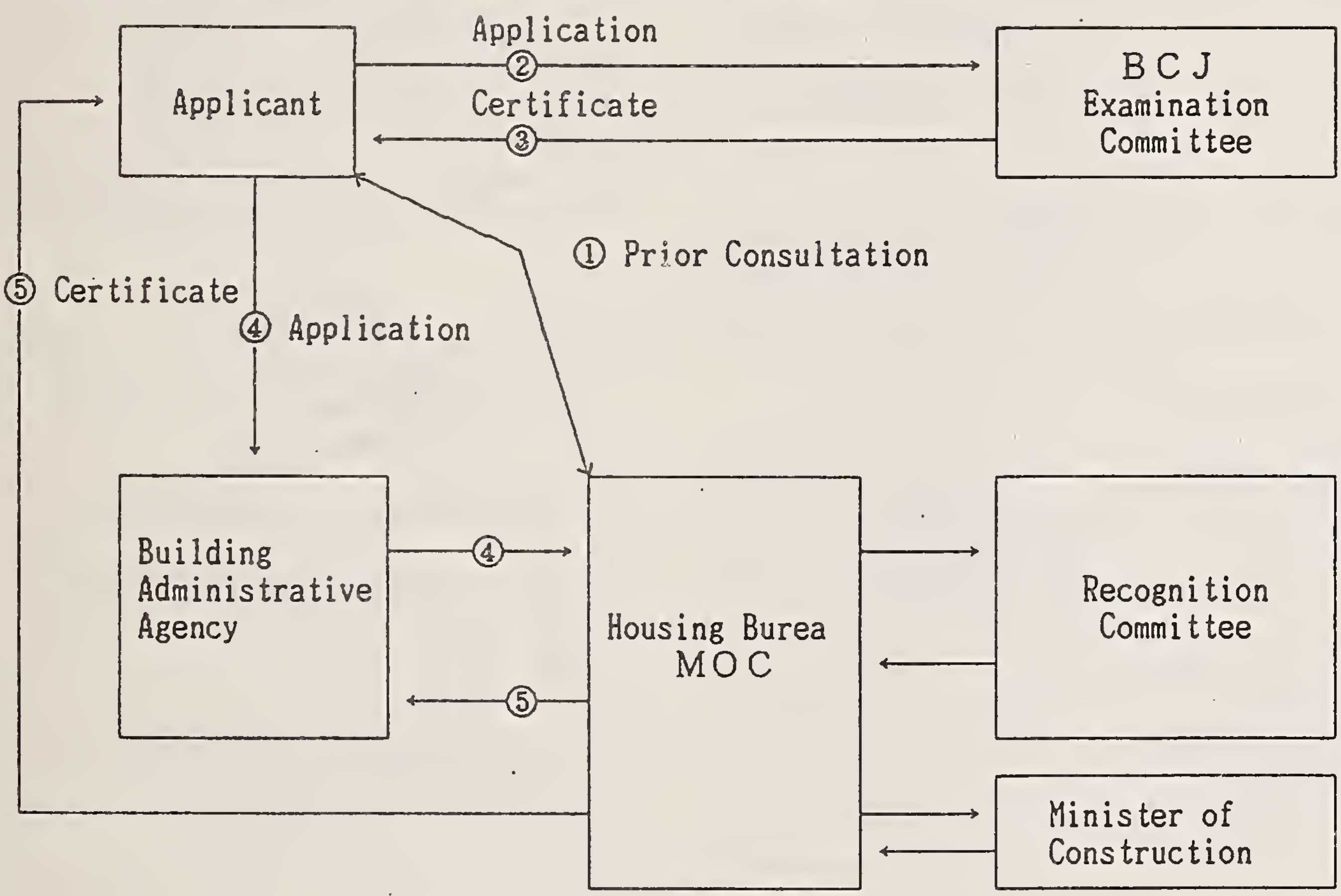
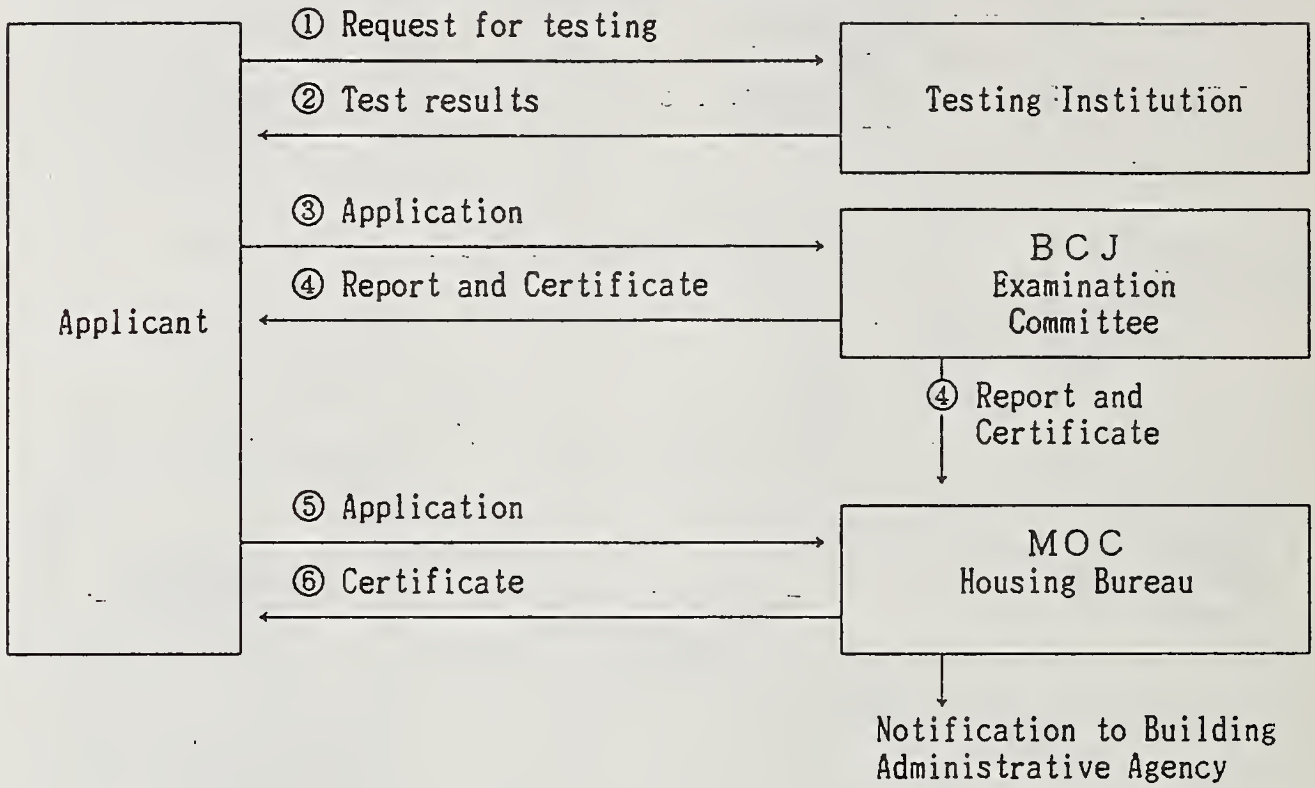


Fig.2 Application procedure for Category 2 Agrement System



History of Structural Regulations in Japanese Building Codes and Outline of Wind-Resistant and Seismic Regulations

By

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SUMMARY

This paper presents a outline of Building Standard Law which prescribes the basic building restriction of Japan and brief histroy of Japanese building restriction after Meiji-Restoration when Japan set course for modern nation. And this paper presents histories and current codes of seismic design method and wind-resistant design method. Opinions on the current codes are also summarized.

KEY WORDS: Building Restriction, Building Standard Law, Seismic Design Method, Seismic Force, Urban Building Law, Wind Force, Wind-Resistant Design Method

1. OUTLINE OF JAPANESE BUILDING RESTRICTION

1.1 Feature of the land and cities of Japan

Japan is an earthquake-prone country lying to the west of the circum-Pacific earthquake belt. Approximately, 70% of the land is either hilly or mountainous and the remaining 30% is alluvial land that contains most of the urban areas, and is subject to the influence of earthquakes. Japan, therefore, has sustained much damage due to earthquakes, throughout her long history.

Japan is also situated in the path of typhoons and many buildings are damaged by typhoons every year.

Most of the urban areas in Japan can be regarded as a continuation of old towns where wooden buildings are densely packed. In such areas, dry air in winter, seasonal strong winds and foehn phenomena that arise in certain areas because of the mountainous land, produces a very large degree of danger of the occurence of fires and conflagrations.

The urban areas in Japan have been subject to the attack of conflagrations in the past.

The cause of conflagrations is not only accidental fires but also the simultaneous outbreak of fires in many places due to a large-scale earthquake. The Great Kanto Earthquake (1923) was typical of the latter, with approximately 450,000 buildings being destroyed by fire, and some 140,000 persons dead or missing.

Thus, it is highly necessary that buildings in Japan be able to safely withstand attacks of earthquakes, typhoons and fires.

1.2 Purpose and constitution of Building Standard Law

Building Standard Law was enacted in 1950 considering the abovementioned conditions, and since then has been subjected to many revisions in accordance with the advances in technology and the conditions of building disasters that have occurred.

The building standard statute consists of Building Standard Law, Building Standard Law Enforcement Order as cabinet order, Building Standard Law Enforcement Regulations as Ministry of Construction Order, and Notifications of the Minister of Construction based on these regulations.

The objective of the statute is to "safeguard the life, health, and properties of the people by providing minimum standards concerning the site, structure, equipment, and use of buildings, and thereby to contribute to the furtherance of the public welfare."

The statute comprises the "general provisions" on procedures for building construction, the "building codes" on standards for safety and hygiene of all buildings in the whole country, and the "zoning codes" on standards for ensuring the safety and appropriate environments in urban areas to be applied to city planning areas based on City Planning Law, as well as penal provisions.

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1.3 Outline of the regulations of Building Standard Law

1.3.1 General provisions (Procedures for building construction)

① Building confirmation

Generally in the case where a building is to be constructed, the building client must receive the cofirmation of a building official of the municipality, as to whether the plan for the building is in conformity with the laws relating to the site and structure of the building, and the building equipment.

② Design and supervision by architect

The design and the supervision of construction works of buildings exceeding a certain scale must be performed by qualified architects.

③ Inspection after the completion of construction

Once the construction work have been completed, the building client must submit a notification of this effect, to the building official. The said building must receive an inspection by a building official, as to whether the building is in conformity with the law relating to the building site, structure and the building equipment. A certificate of inspection is then granted when it is confirmed.

④ Periodic inspection and periodic report system

The owners of certain buildings such.as hospitals, hotels, department stores, theaters, collective dwellings and offices exceeding certain scales, and the owners of escalators and elevators and other building equipments must have a qualified person inspect the conditions of such buildings and building equipments, every 1 to 3 years, and must report the results of investigations to the municipality.

1.3.2 Building codes

Building codes stipulate standards for structural safety, fire prevention and sanitation.

Regarding structural safety, the codes lay down the standards for detailed structure on wooden construction, masonry construction, reinforced masonry construction, steel-frame construction, reinforced concrete construction, steel encased reinforced concrete construction and no reinforcing bar construction and structural calculation.

In Building Standard Law, the basic idea concerning structural strength is that structures must be safe against loads and external forces such as fixed load, imposed load, snow load, wind force, soil pressure, water pressure, seismic force, etc.

In order to ensure the safety of a building, Building Standard Law Enforcement Order determines minimum standards according to the structural type. It also provides that structural calculations must be performed to confirm the safety of large buildings. The range of building for which structural calculations must be performed is as follows.

type of construction	scale
wooden structure	$n \geqq 3$ or $A \geqq 500\,m^2$
others	$n \geqq 2$ or $A \geqq 200\,m^2$

(where n:number of stories of the building
A:total floor area of the building)

The method of the structural calculation was revised in 1980, and has been in force since June 1, 1981. Adding to the conventional allowable stress design method for elastic range, this new method of structural calculation determines design method considering ultimate stress including the stress within a plastic range. In addition, story drift, rigidity factor and eccentricity factor should be checked to ensure a balanced structural plan.

The Enforcement Order prescribes the loads, the external forces, the allowable stress of building materials necessary for structural calculation, and also the methods of calculation that are necessary for structural safety in accordance with the height and structure of buildings.

Regarding the structural calculation, to cofirm structural safety, at least five kinds of loads and external forces must be checked, namely, fixed load, imposed load, snow load, wind force and seismic force.

Values of allowable stress are provided for the common materials such as timber, steel, concrete, etc. Dual allowable stress system is adopted, namely, allowable stress for permanent loads and allowable stress for temporary loads are provided.

Values of allowable stress for permanent loads are half to one third of ultimate stress of materials, values of allowable stress for temporary loads are equivalent to yield level or elastic limit of materials.

The stress generated in each of the elements by the combinations of loads and external forces, does not exceed the allowable stress.

The combinations of loads and external forces are shown in table 1.

Regarding seismic design, adding to the allowable design method, limitations on story drift, rigidity factor, eccentricity factor and ultimate lateral shear strength of each story are provided.

As for fire prevention, the codes stipulate that roofs and external walls of buildings in a specified area be made fire preventive construction, that department stores and other buildings for special use be made fireproof construction, and that fire compartments be created in large scale buildings according to a specified floor space. Standards for emergency facilities, including hallways, staircases, smoke-exhaust equipment, etc. are also stipulated.

To secure sanitation, the codes lay down standards for natural lighting and ventilation of rooms, structure of lavatories, structure of elevators and escalators, etc.

1.3.3 Zoning codes

The zoning codes prescribe relationship between sites and roads, land use designated districts, ratio of total floor area to site area, height of a building, restrictions on buildings in fire protection districts, etc.

2. HISTORY OF BUILDING RESTRICTION OF JAPAN

2.1 Before Building Standard Law

With the end of the Edo Period and the Meiji Restoration, Japan set the course for a modern nation. Abolishing the insulation policy of the Edo Era, the newly established Meiji government carried out various measures to open the country and catch up with western advanced nations.

Among them was restructuring the capital city of Tokyo. At that time, most houses in Tokyo were of wooden construction and, therefore vulnerable to disasters, especially fires. The government took various steps to make housing in Tokyo fireproof and renovate the old urban structure in Tokyo.

One of the initial actions was the "Ginza Brick Construction Program" in the wake of the 1872 large fire in Ginza. The program was designed to make housing in Ginza fire-resistant through the brick construction introduced from Western Europe and to reconsruct streets.

In the further effort for full-fledged restruction of Tokyo, Tokyo City and Ward Revision Ordinance, the first city planning legislation in Japan, was enforced in 1888.

Meanwhile, in 1886, prefectural governments which have ports established regulations on hygiene of Nagaya apartment houses against the background of the spread of infectious diseases, such as cholera.

Around 1900, building restrictions on hygiene and fireproofness were set up in many prefectures.

In these restrictions, there were few structural regulations.

With the outbreak of World War I, the Japanese economy made outstanding progress, causing concentration of the population and industries in urban areas and making it necessary to establish urban planning and building regulations in big cities other than Tokyo.

In 1919, City Planning Law and Urban Building Law was enacted for six major cities. The two laws served as the starting point of modern building regulations in Japan.

The two laws provided for such systems as building lines regulations and zoning systems. They also stipulated standards for buildings in terms of fireproofness, safety in structural strength, and guarantee of hygiene.

Regarding structural safety, the codes laid down the standards for detailed structure on wooden construction, masonry construction, steel-frame construction, reinforced concrete construction and no reinforcing bar construction and structural calculation., Fixed load and imposed load were provided in regulations of structural calculation.

In accordance with progress in construction technology, and economic and industrial growth, Urban Building Law underwent several revisions. In 1923, the Great Kanto Earthquake rocked the Kanto Area, destroyinga huge number of buildings. Fires caused by the earthquake burned down wooden houses in the region, killing more than 100,000 people.

In reflection on the massive damage, Urban Building Law set up seismic force and made it obligatory to calculate the seismic strength of building. Regulations on safety of buildings in Japanese legislation have since centered on fireproofness and earthquake-proofness.

In the 1930s, the Japanese economy was subject to the military policy. As a result, building restrictions through Urban Building Law were suspended in 1943, excepting some on fire-proofness, and the war-time standards on building structures were established to conserve goods and materials.

After the World War II, Urban Building Law was restored. But in accordance with the enactment of the Constitution of Japan, Building Standard Law was enforced, abolishing Urban Building Law. (Technically speaking, Building Standard Law was basically the extension of Urban Building Law. Regarding structural calculations, revised standard of the war-time standard was adopted.)

The application area of Building Standard Law is the whole nation, not limited districts as in Urban Building Law. City Planning Law, a sister law of Urban Building Law, remains unchanged after the war.

2.2 Changes in building restrictions by Building Standard Law

Nearly 40 years have passed since Building Standard Law was enacted in 1950. During the period, Japan has made remarkable progress in economy and building technology. And Building Standard Law has been revised several times. Following are major revisions in the structural codes.

The codes underwent major revisions, while taking into account progress in building technology and damages to buildings caused by earthquakes.

The Tokachi Earthquake in 1968 inflicted heavy damage on reinforced concrete buildings, considered highly earthquake-resistant, leading to review of earthquake-proofness of reinforced concrete buildings.

As a result, the structural codes of Building Standard Law stipulated ductility of buildings with the revision of reinforcement for shearing of pillars of reinforced concrete buildings in 1971.

At that time, the development of computer technology also promoted structural analysis knowhow. Through computerized seismic analysis, construction technology of highrise buildings was developed around 1970.

Against the background of these factors, the Ministry of Construction led the development project for new seismic design method from 1972 to 1977 with a view to thoroughgoing revisions of seismic design method in Building Standard Law.

In 1978, the Miyagikenoki Earthquake caused extensive damage to buildings, making it urgent to revise seismic design method in the law.

And in 1981, the drastic revision took effect with dynamic considerations on seismic calculation. For instance, the seismic force, which had been set at more than 0.2 of seismic coefficient, was fixed in accordance with natural periods of buildings. The revision also called for two-stage designing -designing against the seismic force equivalent to self weight of a building, in addition to the conventional allowable stress designing against seismic force of 20% of the self weight of the building. Moreover, retained ultimate lateral shear strength was to be calculated with consideration for ductility of a building, not allowable stress.

Revisions and addition have been made to standards for detailed structure in accordance with progress in structural materials, such as steel products, and development of new structuralforms, including wood frame construction.

3. OUTLINE OF SEISMIC DESIGN AND ITS HISTORY

3.1 History of seismic design method

After the Meiji Restoration in 1868, the government abandoned the 230 year isolation from foreign countries and resumed overseas trade..The government invited many researchers, scientists and engineers to Japan in order to develop Japanese Industries as well as to absorb Western culture as soon as possible.

The Imperial University of Technology was founded in 1877. Education of building engineering started then.

The Yokohama Earthquake (M=5.4) occurred in 1880 and caused moderate damage. The earthquake was not very severe, but was strong enough to frighten resident researchers from overseas countries. In the same year the Seismological Society of Japan was founded by the effort of Dr. John Milne who was a mineralogist from England.

The Architectural Institute of Japan (AIJ) was founded in 1886.

In 1891, the Nobi Earthquake (M=7.9) occurred and caused very severe damage. In the next year(1892) the Earthquake Investigation Committee was founded and research on seismology was accelerated. The San Francisco Earthquake (M=8.3) occurred in 1906. In 1914 Riki Sano proposed the concept of seismic coefficient.

The 1923 Great Kanto Earthquake occurred near Tokyo and killed more than one 100,000 people. Most of the deaths occurred because of the fires which spread after the earthquake. Next year (1924), one article was supplemented into Urban Building Law which had been already enforced at that time. The article says, "Horizontal seismic coefficient shall be more than 0.1." That means each building shall be designed and constructed so that it can resist a lateral force of at least 10 % of its weight. In 1925 the Earthquake Investigation Committee was rechartered as the Earthquake Research Institute.

After the adoption into law of the horizontal seismic coefficient, many researchers tried to develop a practical method for calculating stresses in the structure caused by seismic forces. There were only a few engineers who could analyze the stresses of a structure subjected to the horizontal forces. In 1933, the year when Sanrikuoki (M=8.4) and Long Beach (M=6.25) earthquakes occurred, Kiyoshi Muto proposed the well-known Muto's D value method to solve this problem.

There had been only one allowable stress for each material which corresponded to allowable stress for permanent load or about one-third of the yield level of the material. During the World War II, because of the shortage of construction materials, a dual allowable stress system was introduced into Japan. That is, the stress caused by permanent loads was to be less than the allowable stress for permanent loads, and the stress from temporary loads was to be less than the allowable stresses for temporary loads.

In 1950, after the World War II, Building Standard Law and its enforcement order replaced the old regulations. At that time the horizontal seismic coefficient became 0.2 and the allowable stress for temporary loads became twice the allowable stress which had been used.

The height limitation of buildings had been 31 meters or about 100 feet since adoption of Urban Building Law. The objective of the height limitation was to restrict the volume of the buildings in urban area. In 1963 the limitation was abolished. In 1964, the Niigata Earthquake(M=7.5) occurred which caused the tilting of buildings because of liquefaction of saturated sand. In 1968, Kasumigaseki Building, the first highrise building in Japan, was completed.

After the San Fernando Earthquake (M=6.4) in 1971, a five year national research project for establishing a new seismic design method was carried out from 1972 to 1977 by the Ministry of Construction, with the cooperation of the Building Research Institute, Public Works Research Institute, universities, private companies and many other organizations. The new seismic design method was proposed in 1977.

In 1978, the Miyagikenoki Earthquake (M=7.5) hit the Sendai area and its occurrence accelerated adoption of the new seismic designmethod. The new seismic design method, which had already been proposed, was reviewed and evaluated for use as a practical design method. The new seismic design method has been included in Building Standard Law Enforcement Order, Notifications by the Minister of Construction, etc. which constitute the current seismic design code since 1981. Though the code introduced most of up-to-date knowledge of earthquake engineering, there still remain many items to be improvedas the damage by the Mexico Earthquake in 1985 (M=8.1) suggested.

3.2 Outline of the current seismic design code

The purpose of the current seismic design code is that buildings should withstand moderate earthquake motions with almost no damage, and should not collapse nor harm human lives during severe earthquake motions. In order to satisfy the purpose, buildings are to be designed using one or more of thefollowing design procedures.

1) Structural requirements : Buildings shall meet the relevant structural requirements specified by Building Standard Law Enforcement Order, Notifications of the Minister of Construction, Specifications of the Architectural Institute of Japan, etc.

2) Stresses : The stresses caused by the lateral seismic shear for moderate earthquake motions shall not exceed the allowable stresses for temporary loads.

3) Story drift : The drift of each story of the building caused by the lateral seismic shear for moderate earthquake motions shall not exceed 1/200 of the story height. This can be increased to 1/120, if nonstructural members will have no severe damage at the increased story drift.

4) Eccentricity, stiffness, etc.: The stiffness eccentricity Re(see note of Table 3) of each story shall be less than 0.15. The lateral stiffness ratio Rs(see note of Table 4) of each story shall be greater than 0.6.

5) Ultimate lateral shear strength :
The ultimate lateral shear strength of each story shall not be less than the specified ultimate lateral shear.

The above design procedures are applied to each building according to its structural type, floor area, height, etc.

The lateral seismic shear Qi of i-th story is determined by

$$Qi = Ci\ Wi \qquad (1)$$

where Ci is the lateral seismic shear coefficient of the i-th story and Wi is the weight of the building above the i-th story.

Lateral seismic shear coefficient of the i-th story is given by

$$Ci = Z\ Rt\ Ai\ Co \qquad (2)$$

where Z is the seismic hazard zoning coefficient(Fig. 1), Rt is the design spectral coefficient (Fig. 2), and Ai is the lateral shear distribution factor(Fig 3). The standard shear coefficient Co is 0.2 for moderate earthquake motions and 1.0 for severe earthquake motions.

For buildings higher than 31 m, or with irregularities exceeding the specified limits, the calculated ultimate lateral shear strength of each story must not be less than the specified ultimate lateral shear Qun :

$$Qun = Ds\ Fes\ Q \qquad (3)$$

where Ds is the structural coefficient, Fes is the shape factor, and Q is the shear for severe earthquake motions which is given by Eqs.(1) and (2) letting $Co = 1.0$.

The structural coefficient Ds is determined by the energy absorbing capacity of the structure (Table 2). The shape factor Fes is the product of the factor Fe(Table 3) and the factor Fs (Table 4).

3.3 Opinions on current seismic design code

3.3.1 Favorable opinions

Generally speaking, the current seismic design code has been favorably accepted by most engineers, mainly because of the following reasons:

① Architects as well as engineers have come to recognize the importance of structural planning and many buildings have been designed so they are in good structural balance.
② Many buildings have been designed, considering not only strength but also ductility, because a two step design, i.e., allowable stress design and ultimate strength design, has been introduced.

3.3.2 Unfavorable opinions

In spite of the above favorable opinions on the current design code, there are several demerits or items to be improved as follows:

A. Shape factor Fes:

① In order to satisfy the requirements related to the shape factor, engineers are apt to intentionally eliminate shear walls and braces which have been proven to be very effective against earthquake motions.

② If the requirement for stiffness eccentricity Re is not satisfied, the ultimate lateral shear strength should not merely be increased but should be increased so that the torsional vibration will be reduced.

③ Current procedure for estimating Fes does not seem to reflect the real behavior of buildings. Fes should be determined not only by the elastic stiffness but also by the behavior after yielding.

④ Evaluating stiffness of partition walls and other nonstructural members is very difficult, so that Fes estimated using such stiffness may not indicate real characteristics of the buildings.

⑤ In order to only satisfy the requirements for stiffness eccentricity and for lateral stiffness ratio, engineers are apt to unnecessarily increase sizes of columns.

⑥ In order to control the stiffness, slits along columns and walls are often provided, which may not improve the total behavior of the building.

B. Structural coefficient Ds:

① Ultimate lateral shear strength is only evaluated as the sum of lateral strength of columns, walls, braces, etc. The deformation at which the lateral strength is attained, however, should be considered.

② Structural behavior of the building can not be expressed only by the unique value of Ds.

③ Ds is evaluated at each story, but it should be evaluated as a unique value for the building, considering the total behavior of the building.

④ The way to determine the structural coefficient Ds depends mainly on analyses for single degree of freedom systems. Therefore Ds evaluated by the current method does not necessarily express the real behavior of buildings which should be regarded as multi degree of freedom systems.

C. Other items;

① The seismic hazard zoning coefficient Z ranges between 1.0 and 0.7 . It does not sufficiently reflect seismicity in Japan which seems to be more diversified.

② Not included is the importance factor which is considered in most seismic regulations in the world. The factor should be included.

③ Regulations for substructure are not sufficiently provided, compared to super structure.

④ Damage by earthquakes shows that non-structural members, building equipment, furniture, fences, etc. may be more hazardous than main structure which is already designed against earthquakes. Therefore more provisions for nonstructural items should be provided.

⑤ Consideration of dynamic behavior of buildings is well covered in the code, but nonlinear behavior of structures after yielding is not sufficiently covered.

⑥ Dynamic analysis/design method, e.g., modal analysis and time history analysis, should be included.

⑦ In the case of extension for old buildings, there is difficulty in satisfying the current requirements.

D. Overall concept;

In addition to the above comments, there seems to be a more crucial point concerning the overall concept for the current design code. That is "The current design code has stipulated too much in detail." This happened in spite of the fact that at the beginning stage of drafting the current code it was confirmed that design philosophy should be clarified, and design detail should not be specified in the code so that it could be determined by each engineer. Some related opinions are as follows:

① Regulations in detail make most engineers just follow the stipulated design procedures without considering the overall structural design. Many engineers are apt to rely too much on computer programs which can handle any design routes from the beginning to the end of the structural design without man-machine interaction.

② Factors such as Ai, Ds, and Fes should not be, specified in the code but be appropriately determined by engineers.

③ An ideal case may be that the code should only clarify the criteria of probability of damage by earthquakes and experts on structural designing would then just report to building officials that the criteria is satisfied, in order to get building permit.

④ Current design procedures require much more time for designing buildings and also for checking documents for building permit than the previous code.

3.3.3 Closure

The current seismic design code, which is often called the new seismic design method, has been enforced since 1981 and has been accepted favorably by many engineers and building officials. The above mentioned opinions, however, indicate many useful suggestions not only to improve the current code in Japan but also to consider in developing new codes in other countries.

4. <u>OUTLINE OF WIND-RESISTANT DESIGN AND ITS HISTORY</u>

4.1 <u>History of wind-resistant regulation</u>

In 1886, the Architectural Institute of Japan (AIJ) was founded. In 1890, Tatsuzo Sone introduced translated paper on wind-resistant design of Eiffel-Tower. In 1906, Riki Sanô presented paper on wind force. Those were the first papers on wind-resistant design of buildings in Japan.

In Taisyo-Era, several typhoons struck Japan, and did considerable damages to wooden buildings. Consequently Syozo Uchida and others proposed draft regulation for wooden buildings.

Urban Building Law, which had been already enforced at that time, had structural calculation regulation. But it didn't have wind force regulation.

Because of damages by typhoon, wind force regulation was provided as follows in Enforcement Regulation of Urban Building Law of Tokyo.

wind force

75kg/m² ($h \leq 6m$) (where h : height from ground)
100kg/m² ($h > 6m$)

In 1934, the Muroto Typhoon, which had the largest magnitude, struck Kansai district, western part of Japan. A lot of buildings suffered very severe damages, and there were heavy casualties. These damages promoted reseach on wind-resistant design of buildings.

Yosikatsu Tsuboi, Kiyosi Muto and others introduced foreign research papers on wind-resistant design. Hantaro Nagaoka who was a famous physicist criticized wind-resistant design method of buildings.

Japan-China War occurred in 1937, and Japan became under war codition.

AIJ proposed draft standard of structural calculation for steel structures. In this draft standard, wind force regulation was provided as follows.

$W = C \times q \times A$

W ; wind force (kg)
C ; coefficient of wind force
q ; velocity-presser (kg/mm^2)

height	q for building	for tower
$h \leqq 15m$	100	200
$15m < h \leqq 30m$	130	260
$30m < h \leqq 50m$	160	320
$50m < h$	200	400

where A ; action area (m^2)
h ; height from ground (m)

During the World War II, Yoshiro Taniguchi researched into wind resistant design of hangers for airplanes.

Because of the shortage of construction materials, war-time standards on building structure was provided. In this standards, wind force was as follows.

$q = 40\sqrt{h}$ q ; velocity-presser (kg/mm^2)
h ; height from ground (m)

After the World War II, Japan Standard 3001, which was the regulation of structural calculation and was revised standard of war-time standard, was provided. In this standard, velocity presser q became $60\sqrt{h}$.

Building Standard Law was provided instead of Urban Building Law, in 1950. Japan Standard 3001 was adopted as the regulation of structural calculation in Building Standard Law Enforcement Order.

Notification by the Minister of Construction on reduction coefficient for wind force was provided in 1952.

AIJ proposed "Technical Recommendation for Highrise Building" in 1964. In this recommendation, velocity-presser q was provided nearly equivalent to $120\sqrt[4]{h}$. This recommendation was not compulsory.

Research on dynamic effect of wind had been done. "Draft Standard of Loads for Buildings", which was proposed by AIJ in 1975, presented data for dynamic effect of wind.

The regulations of structural calculation of Building Standard Law Enforcement Order was revised in 1981. At that time, velocity-presser q was revised as follows.

$q = 60\sqrt{h}$ ($h \leqq 16m$)
$120\sqrt[4]{h}$ ($h > 16m$)

In 1981, AIJ proposed "Recommendation of Loads for Buildings", in which dynamic effect of wind and statistics were introduced. This recommendation also was not compulsory.

4.2 <u>Present regulation of wind force</u>

4.2.1 Regulation of Building Standard Law Enforcement Order

The regulation of structural calculation of Building Standard Law Enforcement Order adopts allowable stress method. Concerning the wind resistant design, the regulation says that the stress caused by the combination of permanent stresses and temporary stress by wind force shall be less than the allowable stresses for temporary load. The allowable stresses for temporary load are equivalent to yield level or elastic limit of materials.

Wind force is provided as follows.

$P = C \times q \times A$

P ; wind force (kg)
C ; coefficient of wind force
q ; velocity-presser (kg/mm^2)
$q = 60\sqrt{h}$ ($h \leqq 16m$)
$120\sqrt[4]{h}$ ($h > 16m$)
A ; action area (m^2)

4.2.2 "Recommendation of Wind Force for Buildings" by AIJ

Regarding the wind force, "Recommendation of Loads for Buildings" by AIJ says as follows.

$P = q \times C_f \times G_f \times A$

P ; wind force (kg)
q ; velocity-presser (kg/mm^2)
C_f ; coefficient of wind force
G_f ; gust influence coefficient
A ; action area (m^2)

where
$q = (1/2)^2 \cdot \rho \cdot V_z^2$
ρ ; density of air
V_z ; design wind velocity

$V_z = V_0 \cdot E \cdot R$

V_0 : normal wind velocity see fig.4 (mean return period of 50 years)

E : wind velocity distribution coefficient see table 5.

R : return period conversion coefficient of wind velocity

$R = 0.61 - 0.10 \log_e [\log_e (t/(t-1))]$

t : return period

C_f is given by wind tunnel test.

G_f is given from table 6 or by precise analysis.

4.3 Opinions on current wind resistant design code

4.3.1 Opinions

Opinions on current wind resistant design code are as follows.

① Current wind resistant design code is provided for the buildings, area of which exposed to wind are comparatively small, and rigidity of which are comparatively high. Consideration to dynamic effect of wind is necessary for buildings, area of which exposed to wind are large, and rigidity of which are low.

② Current wind resistant design code is provided uniformly for all over country, and isn't based upon statistic data, and the roughness of the ground of the construction site of building isn't considered.

③ Formula of velocity pressure changes at height of 16m. That is irrational.

④ Although damages of substructure by strong wind were larger than super-structure regulations for substructure are not sufficiently provided.

⑤ Current wind resistant design code is based on the allowable stress method, but it is necessary to include fatigue design, deformation design, rigidity design, etc.

4.3.2 Closure

Current wind resistant design code is provided basically forty years ago, and have not adopted results of later studies. It is necessary to introduce resent studies of wind force in code.

Table 1 Combination of stresses for structural calculation

Kind of Stress	Conditions estimable regarding loads and external forces	General case	Case of heavy snow district
Permanent stress	Normal time	G+P	G+P+S
Temporary stress	Snow season	G+P+S	G+P+S
	Storm	G+P+W	G+P+S+W
	Earthquake	G+P+K	G+P+S+K

where, G,P,S,W and K represent respectively the following stresses (stresses combined of axial stress,bending monent,shear and others):

G : stresses caused by fixed load
P : stresses caused by imposed load
S : stresses caused by snow load
W : stresses caused by wind force
K : stresses caused by seismic force

Table 2 Structural Coefficient Ds*

Behavior of Members	Type of Frame		
	(1) Ductile moment frame	(2) frame other than listed in (1) & (3)	(3) **
A. Members of excellent ductility	0.3	0.35	0.4
B. Members of good ductility	0.35	0.4	0.45
C. Members of fair ductility	0.4	0.45	0.5
D. Members of poor ductility	0.45	0.5	0.55

* Values can be decreased by 0.05 for steel encased reinforced concrete buildings and for steel buildings.

** Frame with shear walls or braces for reinforced concrete and steel encased reinforced concrete buildings, and frame with compressive braces for steel buildings.

Table 3 Shape Factor Fe and Stiffness Eccentricity Re

Re*	Fe
less than 0.15	1.0
$0.15 \leq Re \leq 0.3$	linear interpolation
more than 0.3	1.5

* $Re = e/re$
where e is the eccentricity of the center of stiffness from the center of mass, and re is the elastic radius, i.e. square root of the torsional stiffness divided by the lateral stiffness.

Table 4 Shape Factor Fs and Lateral Stiffness Ratio Rs

Rs*	Fs
more than 0.6	1.0
$0.3 \leq Rs \leq 0.6$	linear interpolation
less than 0.3	1.5

* $Rs = r/\bar{r}$
where r is the lateral stiffness, which is defined as the value of the story height divided by the story drift caused by the lateral seismic shear for moderate earthquake motions, and $\bar{r}$ is the mean value of r.

Table 5 (a) Class of roughness of ground surface

Class of roughness of ground surface	Condition of ground surface near the construction site of the building
I	flat ground with almost no obstacles
II	district with some obstacles like crops or with thin trees and low rise buildings
III	district with dense trees and low buildings or with thin middle or high rise buildings
IV	district with dense middle or high rise buildings

Table 5 (b) Distribution coeficient E

Height from ground \ Class of roughness of ground surface	I	II	III	IV
$Z \leq Z_b$	1.12	1	0.87	0.72
$Z_b < Z \leq Z_G$	$1.20(Z/10)^{0.1}$	$(Z/10)^{0.15}$	$0.80(Z/10)^{0.2}$	$0.52(Z/10)^{0.8}$
$Z_G < Z$	1.62			

Table 5 (c) Z_b , Z_G

Class of roughness of ground surface	I	II	III	IV
Z_b	5	10	15	30
Z_G	200	250	350	450

Table 6 Gust influence coefficient G_f

Class of roughness of ground surface	G_f
I	2.1
II	2.2
III	2.4
IV	2.8

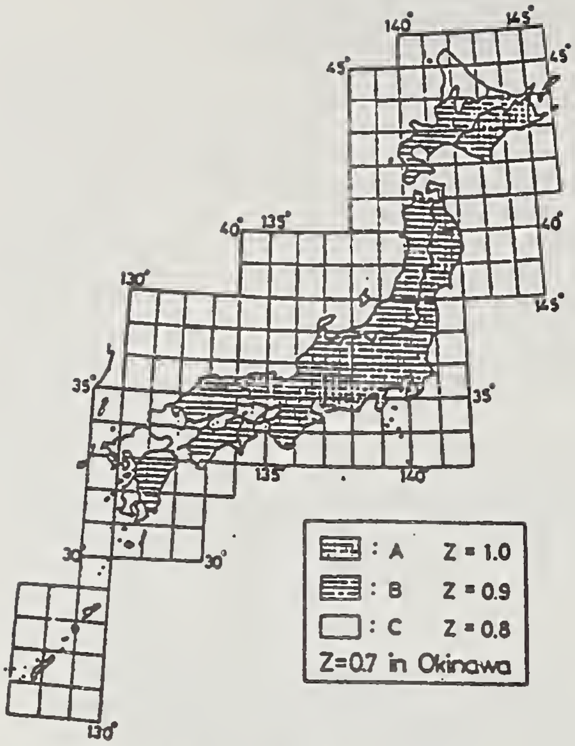

Fig. 1 Seismic Hazard Zoning Coefficient, Z

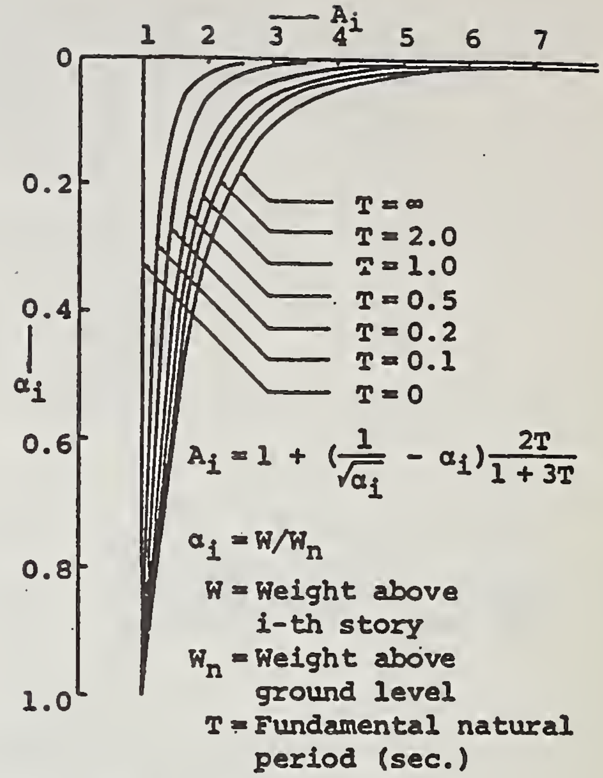

Fig. 3 Lateral Shear Distribution Factor, A_i

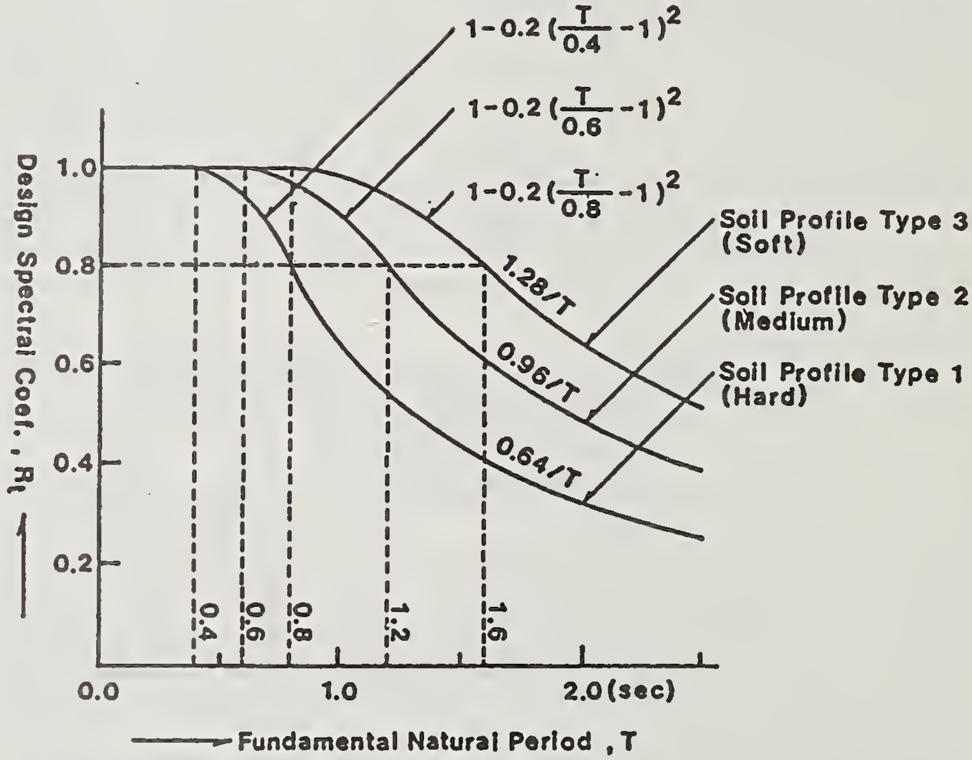

Fig. 2 Design Spectral Coefficient, R_t

Through the precise analysis of the structure, the foundation, the soil, etc., the value of R_t can be reduced to 0.75 of the value given by this figure. But the value shall not be less than 0.25.

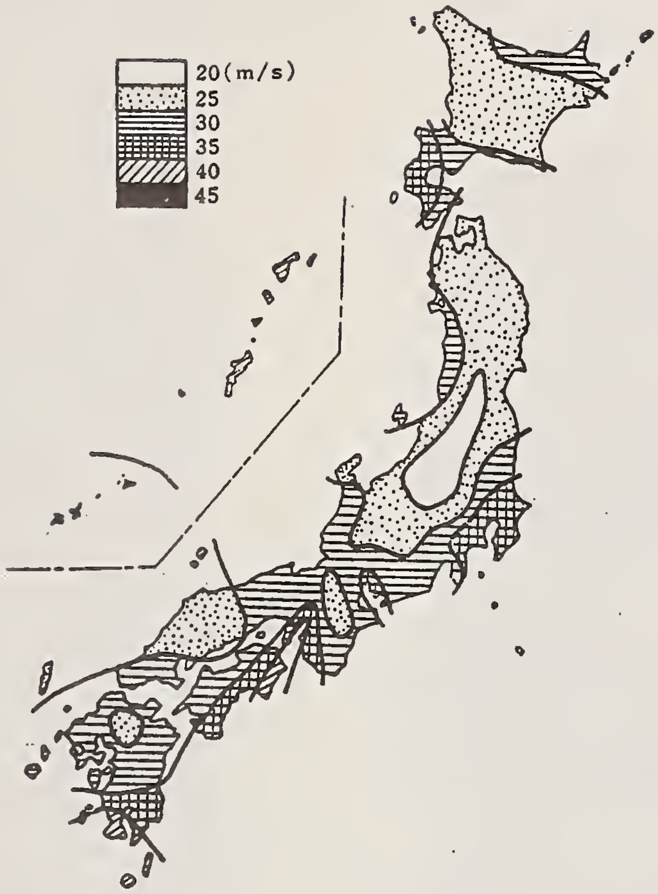

Fig.4 Normal wind velocity V_0

Development and Enforcement of U.S. Building Regulations

by

James G. Gross and Richard N. Wright[1]

ABSTRACT

U.S. practices for development and enforcement of building standards and codes are described for the guidance of those who seek to develop or apply improved building products or practices. Trends, such as mitigation of effects of natural hazards, conservation of energy, water and environment, and efforts to reduce barriers to beneficial innovations or trade, lead to demands for changes in building products and practices.

Understanding of the U.S. building regulatory system and the roles of participating organizations is essential to introduction of new products or practices and to improvement of the building regulatory system.

KEYWORDS: Accreditation; Building; Building Code; Certification; Product Approval; Regulation; Standard.

1. BUILDING AND BUILDING REGULATIONS

1.1 Introduction

Historically, building regulations were created and enforced to satisfy the objectives of protecting the "health" and "safety" of building occupants and the public in general [1]. In recent years, a third objective "welfare" has been added and has received increased emphasis. Furthermore, during recent years, building regulations have become more stringent. New provisions have been added, with very few deletions. Enforcement of more stringent regulations raises concerns for adverse effects on the cost of building construction [2].

The Constitution of the United States leaves the authority to regulate building design and construction to the states. In the past, this authority usually has been exercised by political jurisdictions at the local level, such as cities and counties. Recently, there has been a broad movement by the individual states, as described in section 2, to reclaim authority for the development and implementation of statewide building codes. International efforts to increase trade in building products and services seek elimination of use of standards and certification systems as barriers to international trade by establishing a framework of common rules [3]. These are expected to lead to international harmonization of building standards, regulations, product approval systems, and requirements for professional licensing. These efforts are likely to require changes in U.S. building regulations and regulatory practices.

This paper begins with a description of the building process, its principal actors, and the role of standards in the building regulatory system and in the building process. Interfaces between the building regulatory system and the building process are described. Technical and policy trends leading to changes in building regulations and regulatory processes are noted.

1.2 Building Process

A simplified, schematic diagram of the building process is shown in figure 1. The early phases, feasibility study and program analysis, decide where the building will be located, how much space will be provided and what characteristics

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are required for the space. The principal actors in this activity are the owner and the architect. It is important to obtain adequate professional inputs in programming. Specialists, such as the engineering geologist and the structural engineer, may guide in site selection and in selection of a structural scheme well adapted to the site conditions, respectively.

The design phases, schematic design and design development, are concerned with the identification and elaboration of a solution meeting the requirements developed in programming. Principal participants in design include the owner, architect, structural engineer, mechanical engineer, foundation engineer, etc.

The contracting phase, involving preparation of contract documents and bidding or negotiation for selecting the contractor, requires preparation of detailed working drawings and specifications by the design team or potential contractors, and the preparation and evaluation of contractors' offers. The construction phase involves the actual construction and its review for compliance with the plans and specifications by the designer, as the owner's agent, and the building regulatory authority.

The final part of the building process, occupancy, has as principal actors, the persons using the building, and those who maintain or remodel it.

1.3 Building Regulatory System

Major interactions of the building regulatory system with the building process are also shown in figure 1. The principal elements of the building regulatory system are: (1) the building regulations denoted as "codes," and (2) the regulatory agency or agencies which enforce and interpret the codes.

The regulations speak directly to certain aspects of the building and the building process:

- o Land use, defining the natural and man-made environments,
- o Design, controlling the performance of the building in the natural and human environments,
- o Construction, controlling how the design is implemented, and
- o Occupancy, controlling how the building is used, maintained, remodeled, or removed.

The usual interactions of the building regulatory system and the building process are shown in figure 1. The codes themselves are referred to by the planners and designers during the programming and design stages to assure that these regulations are complied with. The building regulatory system reviews the planning and design when appropriate documents describing the designer's conception are made available to the building regulatory system. This interaction is denoted "plan review" in figure 1. Following approval, after any necessary modifications, field enforcement by the building regulatory system promotes compliance with the design during construction. A view of interactions and roles in the building regulatory system is shown in figure 2.

1.4 Aspects of Building Regulation

Standards and codes may be either performance or prescriptive in nature [4]. A performance standard expresses the desired attributes of the product or process in question: any scheme which results in achievement of these attributes meets the standard. This approach promotes economy and innovation since it focuses on the users' requirements. A prescriptive standard is a specific definition of an acceptable process or product. The desired attributes often are unstated: the process or product may meet the standard but may not meet the users' needs. Prescriptive standards have the advantage that they are objective. It is relatively straightforward to evaluate and determine whether or not the standard is complied with. However, prescriptive standards do not assure satisfaction of the users' needs, let alone in an optimal fashion. Performance standards have the disadvantage that it is conceptually more difficult to evaluate whether they are complied with. As indicated in figure 3, a laboratory-type activity may be required to evaluate compliance. Efforts have been made to standarize the process of evaluation of performance standards [5].

Performance standards should not be seen as the antithesis of prescriptive standards. Both have their place. Performance standards are essential for description of desired attributes of the building. It is most meaningful to the user to say: "the building must have less than one percent probability of unserviceability from earthquake per year." Prescriptive standards are useful statements to the designer or to the builder of exactly what is wanted. Ideally, prescriptive standards will be related to performance standards so that the performance intended is explicit.

Land use regulations may be intended for protection from natural hazards or protection of property values. The major elements of land use regulations are:

- o The general plan, which is a prescriptive standard for a geographical area describing how land use is to develop.
- o Zoning regulations, which describe the conditions for land use for various purposes. One set of zoning regulations may be applied to a variety of general plans.
- o Subdivision regulations prescribe in detail how land is to be used when subdivided for a specific purpose. Applications of land use regulations for natural hazards mitigation are described in [6]: economic consequences of overly stringent land use regulations are discussed in [2].

Design and construction standards and codes deal with the various physical systems of the building, such as the structure, mechanical systems, plumbing and electrical, or with specific aspects of performance such as fire safety. Section 2 provides a detailed description of the development of building codes. This is representative of development of other standards and codes.

Housing codes control the conditions of occupancy of buildings. These codes generally are concerned with public health under normal conditions of use, rather than mitigation of natural hazards.

Mobile homes, housing units built on permanent chassis, are uniquely subject to Federal building regulations. Originally mobile homes were considered to be often-transported housing and regulated under transportation authorities of the various states. However, it became common for a mobile home to be transported only once to a building site where it remained throughout its useful life as lower-than-normal cost housing. In recognition of their public importance, Congress established a national system of regulation for mobile homes in 1974 [7].

Other manufactured homes are factory-built, in search of improved quality and economy, but are similar to conventional houses when erected on site. Efforts are continuing [8] to create a regulatory system for factory-built housing that will provide a national market to manufacturers while imposing consistent requirements on conventional and factory-built housing.

Rehabilitation of existing buildings and preservation of historic buildings pose special regulatory requirements. First, current building standards and codes may not apply to the materials and systems of older, existing buildings. Second, it may be unduly expensive to require the same performance level of existing buildings as is required of new ones. Technical studies of the National Bureau of Standards in cooperation with the National Conference of States on Building Codes and Standards [9] led to the development of improved technical bases for regulations, improved regulatory process, new technology and evaluation tools, and cost-effective decision methods. The National Institute of Building Sciences developed rehabilitation guidelines [10] incorporating this work: these guidelines, in turn, are referenced in state and local building codes.

Licensing for professional services in the building process also is a state responsibility in the United States. Such licensing includes design services of architects and engineers, and licensing of contractors. Professional licensing in the U.S. requires graduation from an accredited, professional academic program, experience under the supervision of a licensed professional, and successful completion of a written examination. The 1988 U.S.-Canada free trade agreement, and similar, anticipated agreements with the European Community [11] have caused the National Society of Professional Engineers [12], and similar organizations for other licensed professionals, to explore national systems for professional licensing.

2. BUILDING CODES

2.1 Introduction

There are many types of codes that fall into the broad category of building codes. The various types are reviewed in section 1. For purposes of illustration and discussion, this section will deal only with building codes. Much of the discussion relative to the development and administration of building codes also applies to the other building-related codes.

2.2 Status

There are now 39 states that have statewide building codes, in various forms, which are implemented in different ways [13]. Thirty-five are based upon one of three model codes, and four are state written. Some states allow entirely voluntary use at the local level, while twenty-two others require mandatory usage for the entire state (Alabama, Alaska, California, Connecticut, Florida, Indiana, Kentucky, Massachusetts, Michigan, Minnesota, Montana, New Jersey, New Mexico, New York, North Carolina, Ohio, Oregon, Rhode Island, Utah, Washington, West Virginia, Wisconsin). Eleven states (Arkansas, Idaho, Illinois, Iowa, Kansas, Maryland, Mississippi, Nevada, New Hampshire, Oklahoma, South Carolina) require mandatory application of the state code only for state buildings. Five states have legislation under consideration to adopt statewide codes.

Building regulation and enforcement have been widely cited in the literature [2] as "adding significantly to the cost of building construction." The two principal constraints which have been identified and cited for increasing the cost of construction are: (1) some building codes have requirements that are excessive, or are so prescriptive as not to permit alternate designs which would satisfy the regulatory objective; and (2) the time delay involved to carry out the building regulatory process. The first deals primarily with the technical content of the building codes, and the second deals primarily with the process of administering building codes.

Several national organizations have been formed to improve both the code content and the process for building code enforcement. Three of these organizations are associations of building officials; each promulgates a model building code (i.e., the Building

Officials and Code Administrators International (BOCA), the National Building Code; the Southern Building Code Congress International (SBCCI), the Standard Building Code; and the International Conference of Building Officials (ICBO), the Uniform Building Code). These "model" codes are not, in a strict sense, a code. A code is a standard or other set of conditions and requirements which is made mandatory by government bodies, either through direct legislation or through administrative regulation, based on authority granted by a legislative body. A model code is a standard or other set of conditions and requirements that is recommended for use as a code or regulation. It is developed and promulgated with the intent that it be adopted for regulatory application.

The Council of American Building Officials (CABO) is a consortium of the three model code organizations, with its offices in the Washington, DC metropolitan area. It promulgates the One and Two Family Dwelling Code and the Model Energy Code. The One and Two Family Dwelling Code is widely used for the regulation of single family attached and detached dwellings. It is also used by many states as the basis for regulating modular housing. Modular housing is housing which is made up of prefabricated volumetric boxes produced in a manufacturing plant and moved to the site where the boxes are arranged and attached to each other so as to form a complete structure. The Model Energy Code represents a codification of the American Society of Heating, Refrigerating and Air Conditioning Engineers (ASHRAE) standards addressing energy conservation for buildings. It is used as the basis for energy conservation in the model building codes as well as most of the states of the Nation.

The model building codes cited above are gaining in acceptance and use by state and local governments. Of the 39 states with statewide building codes, 35 are based on either the National, Standard, or Uniform model codes. The major cities of the United States also appear to be moving toward the adoption and use of the three model codes. The Association of Major City Building Officials (AMCBO), made up of the 31 cities of over 500,000 population, reports that 80 percent of these cities now use the model codes as the basis for their regulations, although many of these cities amend the models. The model building codes are not uniform in format, scope, or content; however, there is a great deal of similarity in scope and technical content.

A major advantage of model building codes is that each of the promulgating organizations, BOCA, ICBO, and SBCCI, has well organized code revision procedures for keeping the model code up to date and responsive to the latest advances in technology. A code change cycle takes a year, so it is necessary for proposed code changes to be submitted nearly a year ahead of the time that a final decision is made to change the code. The process includes two public hearings. At the first, an appointed code changes committee hears testimony and studies the documentation submitted with the proposed change. Following this hearing, the code changes committee makes recommendations for approval, approval as modified, or disapproval. These recommendations are published and distributed to the membership and other interested parties. Anyone may challenge the recommendations in writing with supporting documentation. Challenges are heard at the final hearing before the full membership and there finally acted upon. Unchallenged recommendations of the code changes committee are automatically ratified by the membership. The model codes are issued in their entirety on a three-year cycle with supplements issued during intervening years. The latest complete edition is 1988 for the Standard and Uniform Model Codes and 1987 for the National Model Code. The last complete One and Two Family Dwelling Model Building Code was issued in 1989. It, too, is printed on a three-year cycle.

2.3 Administration and Enforcement

The efficient application of building regulations requires technically sound, clear, and correct documents, and efficient processes for applying these regulations. It is the opinion of many experts that the inefficient enforcement of building codes contributes more unnecessary costs to buildings than do ineffectual technical constraints. The essential elements of an effective regulatory process include:

- o Clear governmental authority, responsibility, and accountability with little or no conflict or duplication,
- o Efficient process to eliminate conflicts, overlaps, duplication and delays,
- o Qualified administrative and technical personnel,
- o Certification and training programs for building code enforcement personnel, and
- o Sufficient funding to provide for the above.

The building code enforcement process generally includes three major steps:

- o Plan review and permit issuance,
- o Inspection during construction, and
- o Approval for occupancy and use.

In addition, activities which do not affect each building but are essential to the overall building code administration and enforcement include:

- o Pre-permit consultation,
- o Procedures for appeals, and
- o Adoption and change procedures.

These elements of the process, if cumbersomely and inefficiently administered, can cause excessive cost to both the applicant and enforcement official. Excessive time delays in conduct of the process affect the overall time for construction, and thus total construction cost. While pre-permit consultation is not a mandatory part of the enforcement process, it is receiving increased attention as an opportunity early in the design process to resolve differences in understanding the building code requirements and as a means to significantly smooth out and reduce the time required for plan review.

A recent development is one-stop permitting. This idea is being embraced by designers and building officials alike to simplify the permit process. The following benefits accrue to the participants.

- o An applicant goes to only one location for services,
- o Duplicate requirements are reduced, which avoids unnecessary confusion,
- o Time is saved by both code users and the building department staff, and
- o User uncertainty and misunderstandings are reduced or eliminated.

In 1982, the President's Commission on Housing Report [2] called for "development of models of efficient building department administration appropriate to various size States." The National Conference of States on Building Codes and Standards (NCSBCS), under contract to the Federal Trade Commission, has developed four model administrative procedures for effective state building code administration of residential buildings and a "model one-stop permit processing for one and two family dwellings." NCSBCS has developed model processes for nonresidential construction as well.

A recent study by the Business Roundtable, and reported in their report E-1, "Administration and Enforcement of Building Codes and Regulations," provides excellent background dealing with many of these issues [14].

3. BUILDING STANDARDS

3.1 Introduction

Standards, which are referenced or incorporated, provide the bases for most of the technical requirements in building codes. While most of these standards are materials specifications or test methods, there are numerous design standards, particularly structural design standards, which are referenced in the codes. Many of these same standards are widely used as a basis for design contracts and construction specifications as well as building regulations. Because of this broad regulatory and contractual use, standards provide the principal vehicle for the transfer of advancements from research in building technology.

3.2 Standards Used in Building Codes

There are nearly 2,000 standards referenced in U.S. building codes. A typical U.S. building code references 300 to 400 primary standards. In a National Bureau of Standards report [15] these standards are listed for major city and state codes. Although this report is more than ten years old, the situation is much the same today, with the exception that more states and major cities now rely on the model building codes. The codes studied included the three model building codes (National Building Code, Standard Building Code, and Uniform Building Code), 20 state building codes, and the building codes of the 30 largest cities. The standard referenced most often was the American Concrete Institute Standard 318, Building Code Requirements for Reinforced Concrete, which was referenced 123 times in 36 codes. The second most referenced standard was the American Institute of Steel Construction Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings, which was referenced 62 times in 34 codes. Both of these standards are structural design standards, which indicates the widespread reliance on such standards for regulatory purposes. Approximately 2,000 standards were referenced by one or more of the codes studied. Further, these primary standards in turn reference many secondary and tertiary standards. Compliance with a referenced standard often is mandated by the building code.

There is a heterogeneous array of standards development organizations in the United States engaged in standards-making activities. The standards referenced in building codes referred to above were developed by 91 different organizations.

The American Society for Testing and Materials (ASTM), the largest U.S. standards organization, develops approximately 8,500 standards. Six types of standards are developed: "test methods," "specifications," "practices," "terminology," "guides," and "classifications." ASTM publishes a 3-volume compendium of 663 standards [16] specifically for building regulatory use.

3.3 Standards Development Processes

Most national standards in the United States are produced through a "voluntary system" made up of government and industry, producers and consumers, institutions and individuals. The system is called "voluntary," since participation of interested parties is on a voluntary basis; and the standards produced are available for voluntary use in building regulations and in building contracts. When so used, compliance with the standard is mandatory.

The voluntary standards are produced by two general processes, leading to "industry" standards and "consensus" standards. "Industry" standards are those promulgated by a given industry or industrial group. An industry standard reflects an agreement by the promulgating industry and is controlled by that industry. Such standards are often produced under limited participation processes. Frequently, however, the standard reflects the view of some affected parties as well.

"Consensus" standards are most often developed by standards organizations devoted to standards writing. Some professional societies and industry trade associations also develop consensus standards as part of their activities and, in doing, ensure the opportunity for broad representation and participation. Consensus standards reflect the view of all interested and concerned parties.

Due to the rigorous procedural requirements and broad participation in the process, consensus standards generally receive the highest level of recognition by regulators and others involved in the building process. They are more readily accepted and used than are industry standards developed under limited participation processes.

The American National Standards Institute (ANSI) serves as the coordinating body for consensus standards produced by other recognized, authoritative organizations. It encourages the development of standards through the consensus process, and it is concerned with the well-being of the U.S. standards generating system. It also coordinates U.S. activities related to international standards participation, particularly the International Organization for Standards (ISO) and International Electrotechnical Commission (IEC). The principal criterion for American National Standards is the concept of "consensus" which must be used during development. Consensus is defined by ANSI as:

> "Substantial agreement reached by concerned interests according to the judgment of a duly appointed authority, after a concerted attempt at resolving objections. Consensus implies much more than the concept of a simple majority but not necessarily unanimity."

A standards developer may be accredited by ANSI to use one or more recognized methods of developing evidence of consensus. These are (1) Accredited Organization Method, (2) Accredited Standards Committee Method, and (3) Accredited Sponsor Using Canvass Method. Each of these methods is further defined and has as a guiding principle the development of consensus for approval, revision, reaffirmation, and withdrawal of ANSI standards. American National Standards are produced under a due process, which means that any person, organization, company, government agency, or individual with a direct and material interest has a right to participate. Due process allows for equity and fair play. Minimum due process requirements include:

- o Openness

  Participation shall be open to all persons who are directly and materially affected by the activity in question. Timely and adequate notice shall be provided of the initiation and development of a new standard or a substantively revised standard.

- o Balance

  The standards development process shall have a balance of interest and shall not be dominated by any single interest category.

- o Interest Categories

  The interest categories appropriate to the development of a consensus in any given standard activity are a function of the nature of the standard being developed, but must include at least producer, user, and general interest.

Written Procedures

Written procedures shall govern the methods used for standards development and shall be available to any interested person.

Appeals

An identifiable, realistic, and readily available appeals mechanism for the impartial handling of substantive and procedural complaints must be in the written procedures.

Notification of Standards Development

Standards activities shall be announced in suitable media, as appropriate, to demonstrate provision of opportunity for participation by all directly and materially affected persons.

Consideration of Views and Objections

Prompt consideration shall be given to the written views and objections of all participants.

Consideration of Standards Proposals

Prompt consideration shall be given to proposals for developing new standards or revising or withdrawing existing standards.

Records

Records shall be prepared and maintained to provide evidence of compliance with the written procedures.

4. PRODUCT APPROVAL

4.1 Introduction

Product approval and product acceptance have different meanings to different users. Product acceptance, on the part of designers, contractors, and owners, suggests the need for assurance that the product will perform as expected under conditions of intended use. Regulators of building construction, on the other hand, look at product approval from the viewpoint of assurance that the regulatory needs of the jurisdiction will be complied with. Product approval, in response to regulatory needs, is the use for which the following discussion generally applies.

There is no coherent national product approval system in the United States. A recent study of the National Institute of Building Sciences [17] describes the current procedures. Product approval, by and large, is dependent upon the final judgment of the building official. Taking into account the seriousness of the consequence of a potential failure, he makes his judgment based upon the information available. Building officials in the United States rely on others to provide assistance in evaluating products for their compliance with the intent of the building code. The essential elements contributing to product approval are standards, testing, and certification that the product or service supplied meets the applicable standard or other criteria. In the case of conventional products meeting recognized standards, the problem is somewhat less difficult than approval of innovative products for which standards have not been developed. The regulator relies on laboratory reports, experience and certification, be it self-certification or certification by another third party, and evaluation systems of others. The model building code organizations and their umbrella organization, CABO, provide evaluation services which are widely relied upon by the building officials. Although there is a lack of uniformity in approval of conventional products which are produced to a standard, it is in the area of innovative products where standards do not exist that the most difficulty lies. Lack of standards and performance criteria discourages innovation and places the regulatory official in a difficult position of accepting or rejecting innovation. If there is doubt, he must primarily be concerned for the regulatory requirements and health and safety of the occupants.

4.2 Evaluation Services

Evaluation systems of each of the model codes and the National Evaluation System (NES) of CABO are highly regarded and widely relied upon by building officials for making judgment relative to the compliance of conventional products with the model building codes and for evaluation of innovation. These organizations evaluate submissions of proponents describing products with supporting documentation. Based upon information submitted and additional data which may be called for, determination is made as to the compliance with specific requirements of the model codes. Reports when issued are distributed to the membership of the respective model code organizations and are available to other interested parties upon request. Each of the model code organizations issues hundreds of reports including those developed under the NES of CABO. A broad range of conventional and unusual products are covered. In recent years, evaluation services have been formalized and are gaining in credibility and acceptance. The model code organizations are developing acceptance criteria for use by proponents of building products and systems to provide guidelines on required performance to meet the code intent. As of January 1989, ICBO has issued 17 sets of acceptance criteria addressing such broad subjects as criteria for expansion anchors in concrete, criteria for quality control manuals, and criteria for residential chimney liners. These criteria are developed and adopted in conjunction with public hearings conducted by the evaluation committees of ICBO. They may be revised from time to time through the same public hearing process.

4.3 Laboratory Accreditation

It is recognized generally that national consensus standards provide for the essential qualities of the products falling under the scope of the standard. Testing, however, does not have the same credibility, particularly testing by laboratories lacking recognized qualification for the testing. Laboratory accreditation has come into being to meet the need of further validation and assurance of a laboratory's capability. There are many different programs available for the competence of testing laboratories in the United States. Laboratory accreditation serves to meet the needs of:

- o Laboratories seeking independent evaluation of technical proficiency and national recognition of their competence,
- o Industry requiring the services of competent testing laboratories,
- o Governmental regulatory agencies requiring evaluation of public safety,
- o Purchasing authorities requiring assurance of product conformance to specifications (standards), and
- o Consumers seeking assurance of product safety and performance.

Although there are many laboratory accreditation programs in the United States, only a few are national in scope that relate to construction. The three major national programs are briefly discussed below.

NVLAP

The National Voluntary Laboratory Accreditation Program (NVLAP) was established in 1976. It is administered by the National Institute of Standards and Technology and is available on a voluntary basis to all laboratories be they public or private. In the area of construction technology, NVLAP currently has programs that address thermal insulation, carpeting, construction testing services, solid-fuel room heaters, acoustical testing services, and seals and sealants.

A2LA

The American Association for Laboratory Accreditation (A2LA) Program was established by the American Council of Independent Laboratories, in 1978, by a group of concerned individuals to develop a management system to verify a wide variety of testing laboratories. A2LA grants accreditation in a field of testing and, in some cases, on specific standards. Fields of testing currently recognized which relate to construction regulation are construction materials, geotechnology, and thermal testing.

AASHTO

In 1988, the American Association of State Highway and Transportation Officials (AASHTO) initiated their laboratory accreditation program. This program addresses soils, concrete and asphalt materials. It is standards specific and implements ASTM Standards E994 [18] and E548 [19]. The criteria for the evaluation of laboratories in specific fields of construction materials testing are based on the following ASTM standards:

ASTM E3666-83	Standard Practice for Evaluation of Inspection and Testing Agencies for Bituminous Pavement Materials
ASTM D4561-86	Standard Practice for Quality Control Systems for an Inspection and Testing Agency for Bituminous Pavement Materials
ASTM D3740-80	Standard Practice for Evaluation of Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction
ASTM C1077-87	Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation

These ASTM standards include criteria on the quality assurance system of the laboratory, qualifications and training of laboratory personnel, evaluation of laboratory equipment, and evaluation of the ability of laboratory personnel to conduct tests according to the standard methods of test for which accreditation has been requested.

4.4 <u>Certification</u>

Certification programs play an important part in quality control/quality assurance programs of the United States as related to building products and related services. The extent of these services is not well known and the effectiveness of certification in assuring quality and compliance with standards has not been analyzed comprehensively. Some certification programs appear to be well run, well documented, and very reliable. Others appear to provide little that contributes to quality control and quality assurance. Certification is a process by which a given product is asserted to meet a specific standard or other specific criteria. Certification generally takes one of two forms, either certification by a third independent party or self-certification by the producer or proponent of the product.

NBS Special Publication 703 [20] identifies 109 private sector organizations in the United States which engage in product certification activity. The directory includes organizations which: administer programs and certify that products meet some criteria; administer programs using an independent third party certifier; or serve as an independent third party certifier. This report identifies only product-related programs. It does not address programs certifying services or professional skills, nor does it include programs which do not specify criteria or standards which a product must meet in order to be approved. An extensive effort was made to make this document complete when it was prepared in 1985. It is interesting to note that 73, out of a total of 109, of the organizations identified are agencies which certify construction products. From this information, it appears that the construction industry is the sector most dependent upon certification programs for regulatory compliance and quality assurance.

The American National Standards Institute has two relevant standards: Z34.1, covering third party certification programs, and Z34.2, covering self-certification. ANSI also has a program for accrediting certifiers. This program for accrediting certifying agencies had only two agencies in the program at the end of 1988, both construction product certifiers. As of April 1989, a third organization has been accredited, and two more certifying agencies have applied and are being processed for accreditation. All five of these agencies are primarily concerned with construction products.

5. PRESSURES FOR CHANGE

5.1 Introduction

Building regulations and regulatory practices are under steady pressure to incorporate advances in knowledge from research. In areas such as earthquake and wind engineering, energy conservation, and indoor air quality, provisions are updated to incorporate recent results and enforcement procedures are modified to test and assure compliance. However, two additional main trends are causing major changes in building regulations and regulatory practices. One is the effect of advanced computation and automation - changing what we build and how we build to take advantage of the growing information-processing capabilities based on microelectronics. The other is international harmonization of building regulations and regulatory practices, stimulated by the European Community effort to create a single European market by the end of 1992, and motivated by an international desire to reduce non-tariff barriers to beneficial trade in construction products and services.

5.2 Advanced Computation and Automation

Advanced computation and automation will provide great changes in what we build and how we build [21]. Building standards and codes, written on paper, can be replaced by computer programs that will incorporate the requirements of the code, provide automatic data processing capabilities in applying the code in specific instances, and guide the regulator in evaluating the building material, component or system under consideration [22]. Traditional, printed standards and codes can be replaced by "knowledge-based expert systems." These techniques facilitate updating standards and codes to introduce new technologies. Moreover, they facilitate checking revisions for consistency with other provisions.

Establishment of a computer-based information system for a project, and the development of information interface technologies allowing automatic exchange of data between the project information system and the information system of the regulator, will allow regulatory evaluation of a project without physical exchange of plans and specifications between the designers and the regulatory authority.

Advances in computers, making them smaller, faster and cheaper, allow much improved measurements of the properties of building materials, components and systems in the factory or in the field to assist in the enforcement of building regulations.

Most of the above efforts presently are at the research stage, but improved textual representations of standards and building codes are already available in computer-based information systems.

5.3 International Harmonization

The European Community (EC) has work underway to permit free flow of goods, services, capital, and people within western Europe by the end of 1992. Plans to implement this objective are underway in European construction. The EC is actively developing a program for standards, regulations, certification and testing which will make products and services acceptable in all of the EC countries.

This action was brought about by the Single European Act adopted in 1986. The basis for the Act is that any product legally manufactured in an EC member state must, in principle, be admitted to the markets of all the other members. This requires unification and acceptance of various systems of building standards, building regulations, product testing and certification. The existing European national standardization bodies have underway an immense effort to harmonize existing standards and develop new ones so the construction products will be acceptable to all the countries. The standards being developed will be European Committee for Standardization (CEN) standards, not International Organization for Standardization (ISO) standards [11].

In all of this effort there is little U.S. involvement or input. However, a profound effect on U.S. building regulations and regulatory practices can be anticipated. The General Agreement on Tariffs and Trade and bilateral agreements, such as the U.S.-Canada free trade agreement of 1988, call for the elimination of non-tariff barriers to trade. Therefore, U.S. building standards, building regulations, and product approval systems must be harmonized to allow trade between the United States and other countries in building products and services. This process will lead to products and services meeting U.S. requirements being eligible, without further testing or approval, for international trade, and comparable access of foreign products and services to the U.S. market.

6. SUMMARY AND TRENDS

Health, safety and welfare regulations for buildings are fragmented and vary greatly, both in technical requirements and their administration. There is widespread interest in improving the regulation of buildings, with much attention given to this subject by building officials, design professionals, government at all levels, and the public.

6.1 Building Code Development and Adoption

- o Adoption of model building codes

  Many national organizations and commissions have recommended the adoption of one of the three major model building codes -- National, Standard, or Uniform. These model codes are technically similar, are updated annually, and reissued every three years. The adoption of model building codes without amendment, to the extent feasible, is encouraged by many building industry organizations.

- o Develop uniform format for model codes

  The National Institute of Standards and Technology and the National Institute of Building Sciences, among others, have studied the development of a uniform format for use in the model building codes. The American Society of Civil Engineers and the American Institute of Architects have recommended that the model code organizations develop a uniform format to facilitate model code usage and updating so as to be technically and administratively harmonious and as uniform as is practicable.

- o Adoption of statewide building codes

  A task force, composed of the National Association of Home Builders, NCSBCS, and the model code organizations, has proposed that each state adopt consistent building regulation on a statewide basis with enforcement at the local level. This is envisioned as a means of improving construction productivity.

- o Recommend changes for updating building codes

  When potential improvements in the technical content of building codes and their usage are noted, architects, engineers, contractors, and owners are being encouraged to submit proposed changes to the model code promulgating body to update code content and to facilitate its use.

6.2. Code Administration and Enforcement

Architects, engineers, and contractors are concerned with the processes used for administration and enforcement of building regulations at the local level. Particular items of concern include time required to receive permits, recognition of pre-permit counseling, one-stop permitting, and procedure for appeals from the building official's decision.

- o Model administrative and enforcement processes

  Model processes from NCSBCS are available for implementation.

- o Appeals boards

  Design professionals are being encouraged by their professional societies to seek membership on code appeals boards.

- o Training of building officials

  Training and certification of building officials at all levels is being implemented by model code organizations and states with support by contractors and professionals.

- o Use of computers and other aids

  Automation of the building administration and enforcement process to improve the quality and reduce the time required for building regulation is rapidly growing.

6.3 Product Approval

State, local, and Federal governments and the private sector are studying the development of a national product approval system meeting the requirements given in section 4. A product approval, based on nationally or internationally-recognized standards, should be available locally for acceptance of products in national or international trade. This will require: (1) developing an active U.S. advocacy role in international standards activities, and (2) establishment of a coherent system for acceptance of innovative building products (those for which there are no national standards or products for which nonstandard use is proposed) and improvement of the acceptance and quality assurance of products for which there are applicable international standards.

6.4 Automated Exchange of Construction Information for Regulatory Review

Machine representation of standards and codes is under development to allow efficient, automated review of compliance of designs. Moreover, these techniques could be used to assure consistency when standards or codes are modified to improve provisions. The family of national and international standards required for automatic exchange of construction data (drawings, specifications, etc.) between the diverse computing systems of owners, designers, and regulators is being developed to improve the economy, accuracy and timeliness of plan review. These capabilities also will facilitate transmittal of plans and specifications to manufacturers, fabricators at the construction site, or the transmittal of changed conditions from the site to engineering and regulatory offices.

7. REFERENCES

1. Sanderson, R.L., Codes and Code Administration, Building Officials Conference of America, Chicago, 1969.

2. The Report of the President's Commission on Housing, Washington, 1982.

3. Cooke, P.W., "A Review of U.S. Participation in International Standards Activities," NBSIR 88-3698, National Bureau of Standards, January 1988.

4. Working with the Performance Approach in Building, Publication 64, International Council for Building Research, Studies and Documentation, Rotterdam, January 1982.

5. Dikkers, R.D., "Model Documentation for Building Regulation," TN 775, National Bureau of Standards, May 1973.

6. Wiggins, J., et.al., "Land Use Planning and Natural Disaster Mitigation," in Building Practices for Disaster Mitigation, BSS 46, National Bureau of Standards, February 1973.

7. National Manufactured Housing Construction and Safety Standards Act of 1974, P.L. 93-383, 88 Stat. 633, 42 U.S.C., Section 5401.

8. Field, C.G., "The Changing Building Regulatory System," Pacific Rim Conference of Building Officials, April 1989.

9. Gross, J.G., Pielert, J.H., Cooke, P.W., "Impact of Building Regulations on Rehabilitation - Status and Technical Needs," TN 998, National Bureau of Standards, May 1979.

10. National Institute of Building Sciences, Rehabilitation Guidelines 1980, Department of Housing and Urban Development, 1980.

11. Cooke, P.W., "A Summary of the New European Community Approach to Standards Development," NBSIR 88-3793; National Bureau of Standards, June 1988.

12. Stephens, K., "U.S., Europe Draft Accord on Acceptance of Engineering Credentials," Engineering Times, National Society of Professional Engineers, March 1989.

13. "Directory of State Building Codes and Regulations," National Conference of States on Building Codes and Standards," 1987.

14. "Administration and Enforcement of Building Codes and Regulations," Report E-1, Construction Industry Cost Effectiveness Project, The Business Roundtable, 1982.

15. Vogel, B.M., "Standards Referenced in Selected Building Codes," NBSIR 76-1140, National Bureau of Standards, 1976.

16. "ASTM Standards in Building Codes," American Society for Testing and Materials, 1988.

17. "Building Product Approval/Acceptance Processes," National Institute of Building Sciences, June 1987.

18. "Standard Guide for Laboratory Accreditation Systems," ASTM E994-84.

19. "Standard Practice for the Preparation of Criteria for Use in the Evaluation of Laboratories and Inspection Bodies," ASTM E548-84.

20. Breitenberg, M.A., "Private Sector Product Certification Programs in the United States," SP 703, National Bureau of Standards, 1985.

21. Wright, R.N., "Computer Integrated Construction," Proceedings, International Association of Bridge and Structural Engineers, February 1988.

22. Wright, R.N., Lyons, J.W., "Machine Representation of Standards," Standardization News, ASTM, August 1986.

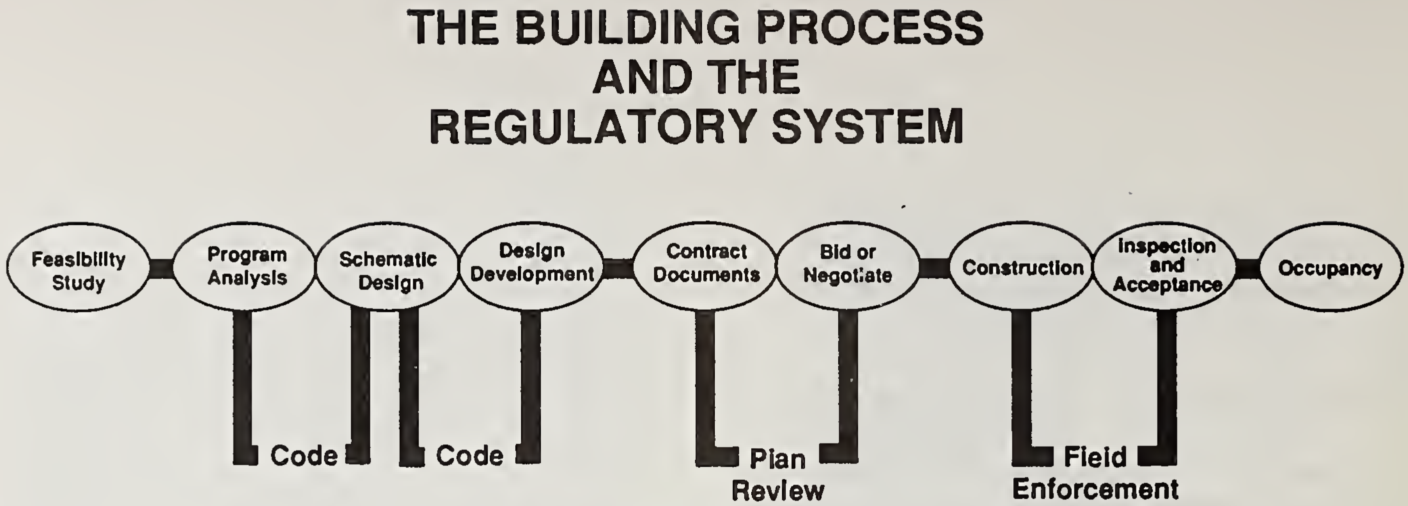

Figure 1. The Building Process and Regulatory System

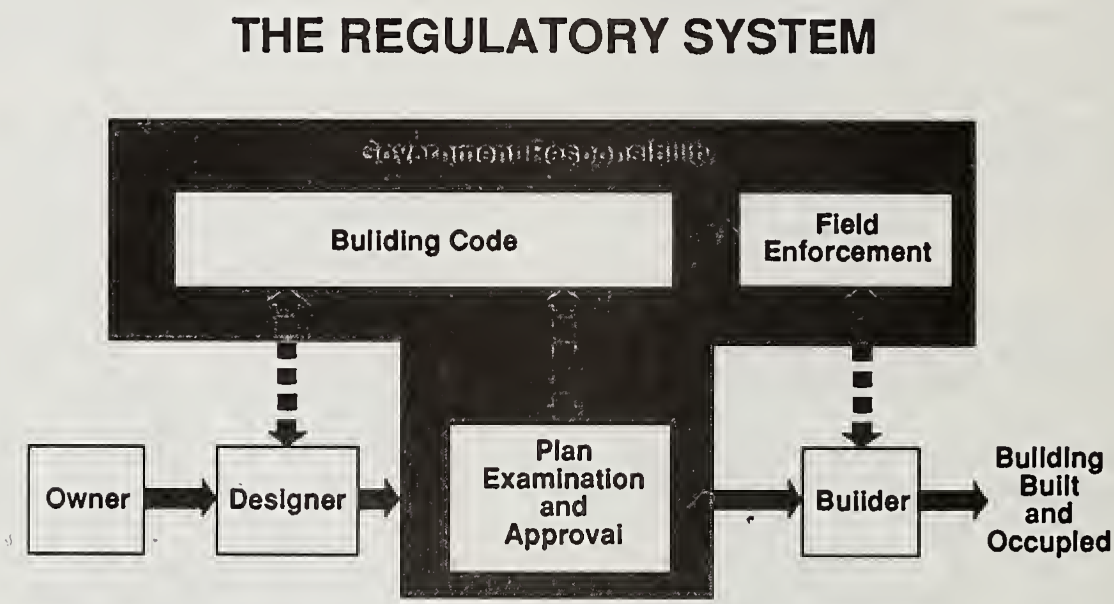

Figure 2. Roles in the Building Regulatory Process

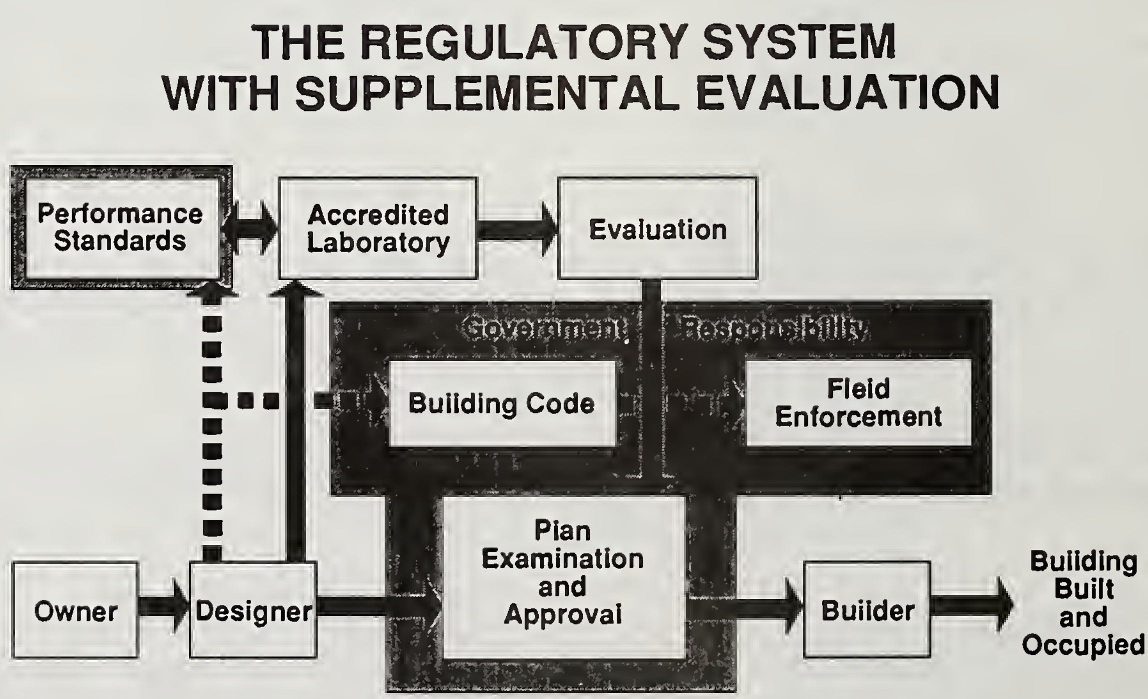

Figure 3. Supplementary Evaluation in the Building Regulatory System

Appendix:

Task Committees A–J Reports

Report of Task Committee on

(A) STRONG-MOTION INSTRUMENT ARRAYS AND DATA

Date: May 18, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees:			
	Japan Side -	H. Tsuchida (Chairman)	(PHRI)
		K. Ohtani	(NRCDP)
		M. Seino	(MRI)
		T. Kuwabara	(PWRI)
		K. Tamura	(PWRI)
	U.S. Side -	A. G. Brady (Chairman)	(USGS)
		J. F. Lander	(Univ. of Colorado)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

MISSION STATEMENT

1. Objective

The objective of the task committee is to coordinate the provision of strong-motion earthquake data to researchers and to the practicing engineering community. Task committee members are involved with instrumentation; recording, processing, and analyzing strong-motion data; research directed to ensuring high-quality data disseminated to users; and the analysis of ground motion and structural motion in dynamic response to earthquakes.

2. Scope of Work

The T/C coordinates strong-motion related research, processing strong-motion data, and disseminating information on the recorded behavior of structures. In addition to regular exchange of data and publications, the T/C's technical approach includes:

* Plan and conduct T/C workshops and meetings, held generally in conjunction with UJNR joint meetings; often with IASPEI/IUGG conferences and with World Conferences on Earthquake Engineering.

* Conduct related technical sessions at professional society meetings.

* Create a procedure for the dissemination of significant strong-motion digital data with regard for the rights and expectations of the owner, the users of data, and the earthquake engineering community.

* Create an up-to-date information exchange on computer processing procedures and related research so all users of processed data are aware of valid ranges and practical limitations.

* Develop a research program for appropriate instrumentation and dynamic analysis of structures with base isolation or other response-reducing systems, under earthquake excitation.

* Develop a research program using temporary arrays and data derived to study site response.

3. <u>Accomplishments</u>

* A workshop on strong-motion earthquake recording was held at USGS in Menlo Park, CA, August 1987.

* The US side presented the Japanese side with a manual and floppy disks containing a global strong motion data catalog and an access program prepared by NOAA's National Geophysical Data Center in May 1989.

* A Workshop was held in Tokyo on 6 August 1988 on the Processing of Strong-Motion Records, under the sponsorship of IASPEI, and International Association of Earthquake Engineering (IAEE). The workshop was scheduled to coincide with the 9th World Conference on Earthquake Engineering (9WCEE), 2-9 August 1988 in Tokyo and Kyoto. Task Committee chairmen and members were responsible for the organization and review of preparatory processing results by participants. A pre-proceedings containing these results was ready for the workshop; a final Proceedings of the workshop has been published. A total of 52 attended; 31 Japanese, 21 others.

* A Workshop was held on 1 August 1988 in Tokyo on the effects of surface geology; T/C members provided significant involvement.

* All Japanese T/C members and several US members attended the 9WCEE in Tokyo and Kyoto,1988. Many papers addressing the objectives of the T/C were presented.

* Dr. A.G. Brady lectured to three groups of Japanese earthquake engineers, including PWRI and BRI, during his stay in Japan for the 9WCEE.

* Data tapes and reports on the Whittier Narrows, CA earthquake of 1 October 1987 were delivered to the Japanese T/C at the 9WCEE.

4. Future Plans

* The T/C Chairmen will investigate funding sources for a 3rd Workshop on Processing Strong Motion Records, concentrating on array data.

* Coordinate the analysis of records from arrays: downhole 3-D arrays (geophysics and earthquake engineering) and structural arrays (structural engineering).

* The T/C recognizes the efforts of the International Working Group on the Effects of Surface Geology and encourages wider participation by Panel members.

5. Information Exchange

* will be performed within the domestic community through technical meetings, conferences, seminars, and workshops.

* will be performed through participation in post-earthquake field investigations with concentration in areas where strong-motion stations have recorded data.

* will be performed with the counterpart side through T/C workshops; visitor exchanges, seminars, lectures; and participation in annual Joint Panel meetings.

* The most crucial information exchange is rapid description of recovered strong motion (and subsequently digital strong-motion data on tape or disk) from significant earthquakes, for example Whittier Narrows, CA of 1 October 1987, M5.9; Supersition Hills, CA 24 November 1987, M6.6; Eastern Chiba Offshore, 17 December 1987, MB 6.0; South Coast of Honshu, JAPAN, 18 March 1988, MB 5.5.

6. Impact

* Most managers who process strong-motion recordings use techniques based on, or closely associated with, the work of this T/C and its members. Foreign networks fitting this category include:

AUSTRALIA - Australian Seismological Center, Bureau of Mineral Resources, PO Box 378, Canberra ACT 2601.

CANADA - Pacific Geoscience Center, PO Box 6000, Sidney, BC. V8L 4B2.

ITALY - The Italian National Commission for Research and Development on Alternative Energy Sources (ENEA), Rome.

TURKEY - University of Bogagici, 80815 Bebek, Istanbul.

YUGOSLAVIA - Institute of Earthquake and Engineering Seismology, Skopje.

* Use of seismological and strong-motion data, in convenient form, in countries other than where recorded.

* Dissemination of digital data from significant earthquakes increases the research data base; application of this data in design practice aids in reduction of earthquake hazards.

7. Barriers

* Difficulty of funding for T/C joint endeavors.

* Joint research efforts with close interaction between participants are difficult unless visiting research appointments can be made.

Report of Task Committee on

(B) LARGE-SCALE TESTING PROGRAM

Date: May 17, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki-ken 305

Attendees:	Japan Side -	K. Ohtani (Chairman)	(NRCDP)
		Y. Yamazaki	(BRI)
		S. Nakata	(BRI)
		H. Yamanouchi	(BRI)
		Y. Koga	(PWRI)
		K. Yokoyama	(PWRI)
	U.S. Side -	H. S. Lew (Chairman)	(NIST)
		G. R. Fuller	(HUD)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

"MISSION STATEMENT"

1. Objective

The objective of the Task Committee is to develop performance data of full-scale dynamic properties of structures to better resist wind and seismic loads through both laboratory testing of prototype structures and field testing of structures in situ. Improved full-scale test data will validate the results of small-scale model tests and substantiate the results of computer analyses of structural behaviour.

2. Scope of Work

The Task Committee develops its research agenda in coordination with other appropriate T/Cs, such as T/C C and D, for full-scale evaluation of buildings, and other structures except bridges.

* Plans and conducts workshops and joint meetings to identify research topics and develops joint test programs.

* Coordinates research projects carried out by various laboratories in the U.S. and Japan. Facilitates publication of research results and implementation of findings in codes and standards.

* Facilitates exchange of research personnel, technical information and available testing facilities.

3. Accomplishments

* The second phase of the U.S.-Japan Coordinated Program for Masonry Building Research has been funded by the National Science Foundation for U.S. researchers. Both analytical and experimental research, including component and subassemblage tests are being carried out at several U.S. universities.

* The Japan side published the technical document entitled "Design

Guidelines for Medium Rise Reinforced Masonry Buildings". Taking the opportunity of the publication of the document, the Reinforced Masonry (RM) Building Promotion Society of Japan was established in May 1989 to further the RM building systems and disseminate technical informations.

* The 4th Joint Technical Coordinating Committee on Masonry Research (JTCCMAR) was held in October 1988 at San Diego, CA. Progress on individual research projects were presented by principal investigators and future coordination of the Joint Program was discussed.

* The results of a full-scale 5-story masonry building test performed in Japan were published in the Journal of the Masonry Society in 1988.

* Precast seismic structural systems was selected as the fourth topic in the Joint U.S.-Japan Program on Large-Scale Testing during the 20th Joint Panel Meeting. A meeting of the U.S. Planning Group was held at the University of California at San Diego on July 1988. The scope of precast structure research program in the U.S. and the planning for a T/C workshop were discussed.

* The first workshop of the Precast Seismic Structural Systems (PRESSS) was held on November, 1988 at San Diego, CA. The purpose of the workshop was to develop a comprehensive U.S. research agenda for PRESSS. Over 35 research ideas were presented; they are being evaluated by the executive committee to develop a comprehensive multi-year research program. On the U.S. side, the proposal will be submitted to the National Science Foundation for funding in 1990.

* On the Japan side, two preliminary meetings were held to discuss the research program for precast seismic structural systems. The Government of Japan approved funding for the PRESSS research program to begin in 1989.

4. Future Plans

* The testing of a five-story full-scale masonry structure will be carried out at the University of California at San Diego beginning October 1989. The U.S. Technical Coordinating Committee on Masonry Research will be held during the first week of July 1989.

* The 5th meeting of the JTCCMAR is scheduled in October 1989 at Tsukuba, Japan. The joint masonry program will be reviewed including the testing requirements for a full-scale 5-story masonry structure in the U.S.

* A Joint U.S.-Japan PRESSS Program Coordinating Meeting is planned for October 1989 in Japan. The scope and schedule of the Joint Coordinated Program of Precast Seismic Structural Systems will be discussed in detail during this meeting.

* Continue exchange of information of the following topics between appropriate U.S. and Japan organizations.
  a. Seismic performance of composite and mixed construction; a U.S.-Japan Joint Workshop is being planned,
  b. Application of high strength construction materials, such as concrete and steel, to structures in high seismic zones,
  c. Application of base isolation and active controlled systems to

critical structures located in high seismic and wind zones, and

d. Testings of large-scale structures other than buildings and bridges.

* Exchange of information on large-scale testing facilities and large-scale testing programs should be encouraged.

5. Information Exchange

* Technical reports and research documentation were exchanged with participating organizations in both countries.

* The results of the 5-story full-scale test of masonry structure in Japan were published by the Journal of the Masonry Society thereby widely disseminating the T/C's research.

6. Impact

* The results of the past joint research projects on reinforced concrete and steel structures have been published in U.S. technical journals. A number of key findings have been incorporated into U.S. and Japanese building codes and design guidelines.

* As a results of joint research projects, the state of practice and understanding in earthquake engineering has been improved in both countries. The results from the joint program have further stimulated research in both countries.

* Facilitate joint research projects utilizing available testing facilities in both countries.

7. Barriers

* There are no major technical barriers for the attainment of the Task Committee mission. However, different building practices and needs require careful planning for effective joint research projects.

* Securing funding to support exchange of research personnel is difficult in both countries. Funding agencies of both countries should be informed about the technical benefits resulting from the joint panel activities and encouraged to participate researcher exchanges and joint research programs.

Report of Task Committee on

(C) REPAIR AND RETROFIT OF EXISTING STRUCTURES

Date: May 18, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees:	Japan Side -	M. Hirosawa (Chairman)	(BRI)
		M. Fujiwara	(PWRI)
		S. Nakata	(BRI)
	U.S. Side -	H. S. Lew (Acting Chairman)	(NIST)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

Mission Statement

1. Objective

The objective of the Task Committee is to encourage research and provide technology transfer activities in materials, components, and total building systems related to evaluating the performance of existing building subjected to environmental and earthquake forces. The research topics provide data and information for the repair and retrofit of existing buildings.

2. Scope-of-Work

The T/C work includes information exchanges and planning and hosting workshops and seminars. Workshops are generally held in conjunction with the annual Panel's meeting. They provide T/C members with an opportunity to review related research and to direct plans for future research, share and implement results. The T/C conducts related technical sessions at professional society's meetings. The T/C studies new materials and methods to accomplish repair and retrofit operations on buildings e.g., fiberglass reinforced plastics, and exploits use of automation and robotics for repair and retrofit. Work focuses on coordinating research projects in Japan and the U.S. to minimize duplication and to maximize benefits.

3. Accomplishments

Several accomplishments were achieved during the past two years:

* Held the 4th Workshop on Repair and Retrofit of Existing Structures at the 19th Joint Meeting, JAPAN, 1987.

* Held a workshop to develop a research agenda for repair and retrofit of different types of buildings, September 1987, in San Francisco. The workshop proceedings were published.

* FEMA published cost data for various methods of repair and retrofit of existing buildings.

4. Future Plans

* A joint workshop is being planned on the design approaches and construction methods for repair and retrofit of buildings, May 1990 in the U.S. The workshop will focus on the use of newly developed methods for repair and retrofiting of buildings.

* In the U.S., FEMA will publish methods to strengthen existing buildings.

* In Japan, the Guidelines for Evaluating and Strengthening Existing Buildings will be revised, and the Guidelines for Evaluation of Earthquake Damage and Repair Methods of Buildings will be published.

5. Information Exchange

* Publications of T/C workshops were widely distributed and made available to the design professionals through technical presentations at workshops and at other national meetings of professional societies.

* Exchange of information with the JAPAN-side has been achieved using the published workshop reports and individual's research papers.

6. Impact

* Contributed to development of US and Japan design criteria for repairing reinforced concrete, steel, and other buildings.

* T/C recommendations were incorporated into building codes and professional practices as illustrated by rehabilitation programs used by the City of Los Angeles to upgrade existing masonry buildings to resist earthquake forces.

* Influenced U.S. and Japan design practices to upgrade existing structures.

7. Barriers

* Greater progress need be made to implement available research results. One approach is to create a central coordinating group who would be in contact with researchers, professionals, and decisionmakers. This group would maintain a library of the T/C activities and of researchers participating in related meetings and workshops. A data bank of reports could be established and made available by electronic methods and accessed by personal

computers. This barrier would be eliminated when sufficient funds are identified to support this endeavor including the identification of appropriate person/organization to lead the effort.

* Securing funding to support the workshop participants is difficult in both countries.

Report of Task Committee on

(D) EVALUATION OF STRUCTURAL PERFORMANCE

Date: May 18, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees:	Japan Side -	T. Murota (Acting Chairman)	(BRI)
		H. Yamanouchi	(BRI)
		K. Kurashige	(MRI)
	U.S. Side -	G. R. Fuller (Chairman)	(HUD)
		K. C. Mehta (Temp. Member)	(Texas Tech. Univ.)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

MISSION STATEMENT

1. Objective

Thousands of structures in each country are considered inadequate to resist probable high winds or earthquakes. An important aspect of disaster mitigation is determining the capacity of existing structures to resist wind and seismic forces, so necessary repair, strengthening, and retrofit can be planned. To provide adequate performance evaluation, each country must coordinate development of condition assessment, screening, and structural analysis methodologies. Structures must then be analyzed and instrumented, and evaluated after disasters.

2. Scope of Work

* Develop a complete inventory of "benchmark" structures that have been analyzed and instrumented for measurement of wind and seismic response.

* Coordinate development of a uniform system of screening and complex structural analysis of wind and seismic resistance and capacity of structures in each country.

* Develop advanced sensor technology, instrumentation, and "expert" systems to provide condition assessment of existing structures.

* Evaluate masonry and precast concrete buildings, and seismic-isolation systems, and include these findings in each country's catalog of "benchmark" structures for post-disaster performance evaluation.

* Plan and conduct workshops at future UJNR Joint Panel Meetings,

cooperatively with Task Committees "C" and "B".

3. Accomplishments

* Exchanged several post-earthquake investigation reports prepared by each country, such as Japan Sea, Mexican, and Whittier Narrows Earthquakes.

* Presented Applied Technology Council ATC-14 Report on "Evaluating the Seismic Resistance of Existing Buildings," which enables engineers to analyze the seismic resistance of existing buildings and to identify structural weaknesses and areas needing strengthening (7/88).

* Exchanged documents on "Measurements of High Winds on High Towers at Kashima Coast (3/87), and "Report of Observation at Meteorological Observation Tower No. 2" by Meteorological Research Institute in Japan.

* Received Japanese MOC first and second year reports on state-of-art and catalog of buildings with seismic isolation or wind damping systems. The U.S. Side is exploring ways to have the documents translated into English (5/89).

* The U.S. Side sponsored an "International Workshop on Sensors and Measurement Techniques for Assessing Structural Performance." A research agenda for sensor technology and measurements related to structural performance was developed (9/88).

* A research workshop sponsored by NSF and coordinated by UJNR Panel members was held in Washington, DC to develop a U.S. research agenda for masonry. A working group was related to structural performance of masonry buildings. Evaluation techniques, nondestructive testing, and condition assessment were identified as priority research needs (8/88).

* As a part of the U.S.TCCMAR Program, completed a project on "Nondestructive Evaluation of an Existing Masonry Building" in Williamsburg, VA (6/88).

* Meetings of U.S. TCCMAR and The Masonry Society (TMS) was held in Salt Lake City, Utah in January 1989. Discussed nondestructive testing methods and analysis of masonry buildings for resistance to wind and earthquake forces.

4. Future Plans

* Develop a reporting system and data collection form to catalog existing structures in the U.S. and Japan that have been evaluated for wind and seismic resistance or have been instrumented. Compile a computer data file for exchange between countries and report its progress at each UJNR Joint Panel Meeting.

* Encourage and enlist the participation of private industry, consulting engineering firms, universities, and other governmental

agencies involved in the instrumentation, evaluation, and condition assessment of existing structures.

* Encourage each country to develop sensor technology and instrumentation of structures, for making condition assessments and for determining the long term environmental and aging effects on building components and materials.

* Translate and disseminate two Japanese MOC Reports on seismic isolation systems and wind damping systems into English.

* Plan with T/C "C" a workshop to be held in conjunction with 22nd UJNR Panel Meeting in US and ASCE Structures Congress in Baltimore, MD in May 1990.

5. Information Exchange

* Exchange methodologies for evaluation of structural performance and reports of actual post disaster evaluations of structures.

* Provide updated data of "benchmark" structures evaluated and instrumented.

* Continue the collection of related technical papers and literature, and encourage development of an annotated bibliography of information for periodic exchange.

* Prepare papers and reports for annual UJNR Joint Panel Meetings and associated workshops.

* Exchange reports on U.S. ASCE Structures Congress held in San Francisco, CA, in May 1989, Sixth U.S. Conference on Wind Engineering held in Houston, TX in March, 1989, and similar meetings in the U.S. and Japan dealing with evaluation of structural performance of existing structures.

* Obtain and evaluate the Texas Tech Report by Dr. Kishor Mehta on structures damaged by tornadoes and hurricanes.

* Obtain and exchange documents and tornado probability maps developed under a US-NSF project.

* Obtain complete information on California Division of Mines and Geology (CDMG) instrumentation of approximately 130 structures (90 to 100 buildings, plus bridges and other structures.).

6. Impact

* Enables the development of revised standards and building code provisions to prevent destruction from high winds and earthquakes.

* Provides engineers with the capability to conduct condition assessment of existing structures for the purpose of developing cost effective repair, retrofit, strengthening or demolition programs.

* Will increase the confidence level of the public, regulators, policy makers and engineers, to be assured that the majority of major structures will perform adequately to resist high wind and earthquake forces.

7. Barriers

* Difficulty in identifying priorities and inadequate resources to document load carrying capacity of existing buildings subjected to highly probable extreme winds and earthquakes.

* Difficulty in developing probability risk maps for tornadoes, typhoons, and hurricanes.

* Limited resources for instrumenting buildings, for conducting post-disaster evaluations, and for participating in foreign travel for coordination of activities.

Report of Task Committee on

(E) NATURAL HAZARD ASSESSMENT AND MITIGATION THROUGH LAND USE PROGRAMS

Date: May 17, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees:	Japan Side –	K. Kawashima (Chairman)	(PWRI)
		S. Noda	(PHRI)
		K. Yokoyama	(PWRI)
		K. Tokida	(PWRI)
		K. Hasegawa	(PWRI)
		H. Matsumoto	(PWRI)
	U.S. Side –	A.G. Franklin (Acting Chairman)	(WES)
		N. Raufaste	(NIST)
		A.G. Brady	(USGS)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

Mission Statement

1. Objective

Natural hazard assessment and mitigation activities represent the analysis and application of a broad range of geological, seismological, and engineering research to the problems of the earthquake resistant design of buildings, land use planning and zoning, disaster preparedness, and mitigation of damage and life loss. The Task Committee emphasizes the improvement of methods for natural hazard assessment and techniques for applying these assessments. Specific technical areas of concern are:

1. Probabilistic Earthquake Hazard Assessment
2. Attenuation of Seismic Waves
3. Prediction of Strong Motion
4. Seismic Zonation
5. Earthquake Losses
6. Delineation of Seismotectonic Elements
7. Application of Hazard Assessment to Seismic Provisions of Building Codes

Estimation of natural hazard losses are important to the Committee because these establish, in part, the quantitative bases for mitigation activities.

2. Scope of Work

* Plan and conduct T/C workshops and meetings. These meetings and workshops are generally held in conjunction with annual Panel on

Wind and Seismic Effects joint meetings.

* Conduct cooperative research programs.

* Exchange research personnel.

* Exchange information on a timely basis.

3. Accomplishments

* Papers have been presented at Panel on Wind and Seismic Effects technical meeting during the past several years.

* Exchanged publications throughout the year. About 20-30 papers are exchanged per year.

4. Future Plans

* Organize a workshop in Japan during 1990, for the purpose of discussing and exchanging ideas on the state-of-the-art knowledge and practice on earthquake hazard and risk (loss) assessment.

* Improve knowledge about researchers exchange programs by disseminating information about fellowship programs and about other financial assistance; and by exchanging of information on institutions, research programs, and related laboratory facilities where opportunities for visiting researchers exist.

5. Information Exchange

* Between United States and Japanese sides through annual joint meetings, T/C meetings and workshops.

* Visits by researchers and joint investigations of earthquakes.

* With the U.S. and Japanese communities through publications, technical meetings, conferences, seminars and workshops.

6. Impact

* Increased public awareness of the importance of hazard and risk assessment.

* Improved public safety through implementation of results reported at Panel on Wind and Seismic Effects meetings and workshops. As an example of implementation is the use of probabilistic ground motion maps in U.S. in Applied Technology Council and Building Seismic Safety Council seismic design studies.

7. Barriers

* Lack of available funds for T/C workshops and travel to annual meetings of Panel on Wind and Seismic Effects.

Report of Task Committee on

(F) DISASTER PREVENTION METHODS FOR LIFELINE SYSTEMS

Date: May 17, 1989

Place: Public Works Research Institute
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees:

Japan Side -	Y. Sasaki (Chairman)		(PWRI)
	T. Murota		(BRI)
	K. Tokida		(PWRI)
	K. Tamura	(Observer)	(PWRI)
	T. Kuwabara	(")	(PWRI)
	H. Sugita	(")	(PWRI)
U.S. Side -	A. G. Brady (Acting. Chairman)		(USGS)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

MISSION STATEMENT

1. Objective

Lifeline systems such as gas, oil, water, and sewage pipelines; and power, communication, and transportation systems are crucial to the survival and health of a city or community. Earthquakes affecting such systems cause severe social and economic disruptions to communities and cause human suffering to the residents. T/C "F" focusses on improving 1) the behavior of lifeline systems during earthquakes and 2) engineering and other seismic countermeasures such as damage estimation techniques and inspection procedures. These technologies will provide users with improvements to existing standards of practice for safety and serviceability of lifeline systems.

2. Scope of Work

* Plan and conduct T/C workshops and meetings; they generally are held in conjunction with the annual UJNR Joint Meeting. The purpose is to exchange the state-of-art knowledge and practice, and to identify cooperatively key opportunities for exchange and studies which effectively implement research results.

* Conduct related technical sessions at professional society meetings.

* Create a research program on a selected lifeline system (similar in concept to the Large-Scale Testing Program). For example, through a joint demonstration, produce criteria for emergency evaluation and on-line monitoring systems for lifeline operation.

* Conduct special activities such as translating, printing, and distributing MOC PWRI's publication, Manual for Repair Methods for Civil Engineering Structures Damaged by Earthquakes.

* Develop performance standards and a manual on repair, restoration, and retrofit of lifelines.

3. Accomplishments

* Researchers of both sides presented more than ten papers on lifeline systems at past Joint Panel meetings and discussed the state-of-art and identified research needs.

* A U.S. - Japan Joint Workshop on lifeline earthquake engineering was held in May of 1989 in Japan.

* Exchanged technical reports on disaster prevention for lifelines, such as buried pipeline systems, and relevant specifications.

* Translated the MOC document on repair methods.

* NIST (formerly NBS) and PWRI have carried out cooperative research on seismic loading effects on structural components of lifeline systems such as column-like structures and buried pipes using large-scale models.

* PWRI instrumented ductile pipes in soft ground at Sodegaura in Chiba and started observation as one of the U.S. - Japan cooperative research program.

4. Future Plans

* Instrument ductile pipes at different orientations to faults at Parkfield, CA (sponsorship by NSF & NCEER).................. 5/89

* Continue observation of instrumented buried pipes and exchange data

* Distribute to US engineering community the English version of PWRI Manual on Repair Methods for Civil Engineering Structures Damaged by Earthquakes 10/89

* Develop a knowledge base for seismic disaster mitigation 5/90

* Develop an engineering manpower database 5/90

* Conduct Joint Workshop on Disaster Prevention for Lifeline Systems in the U.S. .... /91

5. Information Exchange

* Domestic community: through technical meetings, conference sessions, seminars, and workshops.

* Between U.S. and Japan: through T/C workshops, and annual Joint

Panel meetings.

* Visits by researchers and participation in technical meetings such as NCEER-sponsored activities and post-earthquake field investigations by EERI.

* Short-term (3 to 12 months) researcher exchanges to participate in on-going lifeline research such as performed at the University of Michigan and PCA in the U.S. and at BRI in JAPAN.

6. Impact

* Increased public awareness of problems with lifeline earthquake engineering using results from workshops e.g., "Abatement of Earthquake Hazards to Lifelines", Denver CO, Nov 1986.

* Contributed to increase field engineers' technical knowledge on repair methods, through Seminars with lectures by T/C members using the MOC Manual, in nine major cities in Japan including Tokyo and Osaka; more than two thousand participants attended seminars.

* Contributed to the assessment of design and operating standards for lifeline systems through T/C member participation in the Building Seismic Safety Council.

* Contributed to the technical development program of the Technical Council on Lifeline Earthquake Engineering (TCLEE/ASCE) via T/C members participation in the committee work.

7. Barriers

* Insufficient funds to conduct annual Task Committee Workshops and to publish reports.

* Lack of participation by U.S. industrial organizations and utilities in research or in implementation of findings due to caution on taking risks and a reluctance to adopt new technologies and practices.

Report of Task Committee on

(H) SOIL BEHAVIOR AND STABILITY DURING EARTHQUAKES

Date : May 18, 1989

Place : Public Works Research Institute,
Ministry of Construction
1, Asahi, Tsukuba-shi, Ibaraki 305

Attendees :	Japan Side -	K. Tokida(Chairman)	(PWRI)
		T. Iwasaki	(PWRI)
		Y. Koga	(PWRI)
		T. Matsumoto	(PWRI)
		S. Noda	(PHRI)
		M. Okahara	(PWRI)
		Y. Yamazaki	(BRI)
		H. Matsumoto (Observer)	(PWRI)
		J. Koseki (Observer)	(PWRI)
	U.S. Side -	A. G. Franklin (Chairman)	(WES)
		W. E. Roper	(CORPS)
		C. E. Smith	(MMS)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in May, 1988, the work of this Task Committee was drafted for adoption by the panel as follows;

Mission Statement

1. Objective

To enhance technology for predicting the dynamic behavior of soils, for establishing procedures to analyze dynamic soil-structure interaction and for modifying the dynamic behavior of foundations and earth structures, to assure their safe performance during earthquakes. Government agencies with responsibility for public works have a duty to the public to provide for public safety and to make economical use of public funds. Consequently, they have a need for more accurate methods of predicting soil behavior, and for improved, economical methods of controlling soil behavior, to assure the seismic safety of public works and to minimize the costs of pconstruction by avoiding unnecessary over-design.

2. Scope of Work

* At the annual joint panel meetings, present technical papers describing technological developments and the state of the art and practice related to soil behavior and stability during earthquakes.

* Exchange relevant information and technical data relating to field performance, research and methods of practice.

* Plan and conduct T/C workshops in coordination with proposed or ongoing cooperative research programs.

* The task committee will plan and conduct programs of cooperative research, by means of coordination of research tasks, exchange of researchers between U.S. and Japanese government laboratories, exchange of results, and technology transfer through joint publication of research results and recommended practice. By bringing greater resources to bear on problems of mutual interest, and by coordinating research tasks to make the best use of the respective strengths of the two sides' research capabilities, the benefits realized from available research funds can be amplified.

* Exchange visiting researchers. This may be done in connection with cooperative research programs, and independently of them, while will further the aims of the cooperative research programs, technical information exchanges, familiarization with methods of practice on the respective sides, and mutual understanding and cooperative relationships between workers on the two sides.

3. Accomplishments

* U.S. researchers presented 4 papers, and Japanese researchers presented 10 papers, on soil behavior and stability during earthquakes, at the 20th and 21st joint panel meetings.....5/88,5/89

* Mr. Yoshitake Hachiya, of the Port and Harbor Research Institute, completed a one-year visit as a guest researcher in Pavement Systems at the U.S. Army Engineer Waterways Experiment Station..3/88

* A T/C (H) Workshop on Remedial Treatment of Potentially Liquefiable Soils was held at Jackson Lake, Wyoming...........5/88

* Mr. Osamu Matsuo, of the PWRI, completed a one-year guest researcher assignment, performing research in Dynamic Behavior of Offshore Structures, at Southern California University.................8/88

* Mr. Nario Yasuda of the PWRI is a guest researcher at the U.S. Bureau of Reclamation, working on Seismic Behavior of Soils..1/89

* Proceedings of the Workshop on the In-Situ Testing, held at San Francisco, California in 1985 was published and disseminated to the Panel members...4/89

* A planning meeting for cooperative research on preventive measures against soil liquefaction was held prior to the 21st joint meeting..5/89

4. Future Plans

* Identification of research tasks on preventive measures against soil liquefaction that are of common interest, in accordance with the results of the planning meeting..........................9/89

* Develop plans for a cooperative research program to verify soil-pile interaction models for marine structures subject to earthquakes,

measurement of sea floor earthquake motions, and the stability of gravity based structures during seismic events.................9/89

* Initiate research tasks on the development of in-situ assessment methods for soil liquefaction, under the cooperative research program..10/89

* Hold the 1990 U.S. - Japan Workshop on Design and Treatment Methods for Mitigating Liquefaction Induced Damage, in Japan...10/90

5. Information Exchange

* T/C workshops, exchange of visitors, and participation in annual joint panel meetings, including presentation of technical papers.

* Exchange of researchers between U.S. and Japanese government laboratories.

* Visits by researchers and participation in technical activities such as post-earthquake field investigations.

* With the professional community in the U.S. and Japan; by publication of papers in journals; publication of research reports by the respective agencies; participation in professional society meetings and conferences; seminars; and workshops.

6. Impact

Through the exchange of researchers and the diffusion of earthquake-related technical data, experience, and information on methods of practice, the state of technology in geotechnical earthquake engineering has been raised on both sides. Through the cooperative research program on in-situ testing of soils which was carried out after the 1983 Nihonkai-Chubu Earthquake, improved and more accurate methods of using in-situ tests for evaluating the liquefaction potential of soils were achieved; these have been in general use in the United States since 1985.

7. Barriers

* Insufficient funds allocated to earthquake engineering research.

* Administrative restrictions on foreign travel.

Report of Task Committee on

(I) STORM SURGE AND TSUNAMI

Date: May 17, 1989

Place: Public Works Research Institute
Ministry of Construction
Tsukuba, Ibaraki 305

Attendees:	Japan Side -	T. Uda (Chairman)	(PWRI)
		K. Tanimoto	(PHRI)
		K. Kurashige	(MRI)
		M. Okada	(MRI)
		N. Oyagi	(NRCDP)
	U.S. Side -	J. F. Lander (Acting Chairman)	(Univ. of Colorado)
		W. E. Roper	(COE)

Following the request in item 8 of the Final Resolutions adopted at the 20th Joint Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

MISSION STATEMENT

1. Objective

Both storm surge and tsunami are hazards capable of inflecting damage of disastrous proportions. Storm surges are associated with hurricanes and typhoons where high winds and the mounding of water under the low barometric pressure of the storm's eye can cause extensive flooding and severe wave action. Tsunamis are predominately caused by underwater earthquakes and, to a lesser extent volcanic activity. Depending on the distance from the source tsunamis arrive within minutes to up to a day after generation.

The main objective of this T/C is to mitigate these hazards through shared technologies, research, information, and cooperative work.

2. Scope of Work

* Exchange results of research on storm surge and tsunami occurrence, generation, propagation, and coastal effects. This includes observations on historical, current, and theoretical tsunamis. Of particular interest is the effort by US and JAPAN to acquire deep ocean tsunami measurements.

* Exchange results and status of anti-storm surge and tsunami; activities including analysis of the problem, planning, warning, and engineering approaches.

* Exchange information on important planned and ongoing projects relating to storm surge and tsunamis.

* Exchange information on development of technologies such as computer programs to predict travel times, run-up heights, and wave characteristics and analysis; improved instrumentation, and use of satellite communications for detection and warning.

* Facilitate dissemination through exchange of literature, technical reports at joint meetings, special workshops, joint projects, and direct interaction among participants.

3. Accomplishments

* Exchanged digital bathymetric data with the Japanese side; US side received a computer program to calculate tsunami travel time charts.

* Exchanged computer program forecasting storm surge run-up.

* Exchanged technical reports on related subjects.

4. Future Plans

* Develop exchanges of data and information on activities related to deep ocean tsunami measurements.

* A Second Tsunami Workshop (proposed location, Hawaii in October or November 1990) to address advances since the first Workshop and to bring together computer modelers and potential users of modeling results is planned.

* Explore and promote joint undertakings such as instrument testing, joint publications, scientist exchange, satellite communications, and assistance in contacts within the US and JAPAN.

* NOAA has adopted the travel time program developed by Mr. Okada (MRI/JMA) using NOAA gridded bathymetric data to run on NOAA computers. The first ever computed travel time charts have been prepared for the Caribbean Sea using this program developed by Mr. Okada. A copy of the published paper will be sent to the Japanese side.

* The activity area of the Task Committee will be expanded to include members with strong interest on the engineering aspects of storm surge and tsunamis in U.S. side.

* The U.S. side has compiled detailed information on all tsunamis affecting the U.S. and possessions. It is being printed and will be exchanged this summer.

5. Information Exchange

* Continue exchange of publications, bibliographic data, technical reports, and personal contacts.

6 Impact

* Increased cooperation among scientists working in these fields.

* Hasten implementation of techniques developed from the US to Japan and vice versa

* Facilitate wider dissemination of information and technology.

7. Barriers

* Insufficient funding for government and private sector organizations to participate in T/C workshops and the Panel's annual joint meetings.

* The tsunami problem is less serve in the US than in JAPAN which leads to a smaller community of researchers. There are fewer US researchers focusing on tsunamis at any or several locations; thus highlighting the importance of T/C periodic meetings and workshops.

* There are different approaches and degree of significance in dealing with problems in the US and JAPAN e.g., in US the hazard is from a remote source which leads to emphasizing developments of warning and evacuation schemes while in JAPAN the engineering approach is favored due to more locally generated tsunamis.

Report of Task Committee on

(J) WIND AND EARTHQUAKE ENGINEERING FOR TRANSPORTATION SYSTEMS

Date: May 18, 1989

Place: Public Works Research Institute, Ministry of Construction
1-banchi Asahi, Tsukuba-shi, Ibaraki-ken 305, Japan

Attendees:	Japan Side -	S. Saeki(Chairman)	(PWRI)
		T. Fujitani	(MRI)
		T. Iwasaki	(PWRI)
		Y. Sasaki	(PWRI)
		M. Fujiwara	(PWRI)
		K. Yokoyama	(PWRI)
		M. Okahara	(PWRI)
		K. Kawashima	(PWRI)
		K. Tokida	(PWRI)
		K. Minosaku(Observer)	(PWRI)
	U.S. Side -	H. R. Bosch(Acting Chairman)	(FHWA)
		A. G. Franklin	(WES)

Following the request in item 8 of the Final Resolutions adopted at the 20th UJNR Joint Panel Meeting held in Gaithersburg in 1988, the "Mission Statement" of this Task Committee was drafted for adoption by the Panel as follows;

[Mission Statement]

1. Objective

The objectives of this Task Committee are to plan, promote, and foster research on the behavior of highway bridges when subjected to wind and seismic forces and to disseminate research results and provide specifications and guidelines based on the task committee's findings. Surface transportation systems for movement of goods and people play a vital part of commerce and intercourse between people. Highway bridges are especially influenced by the forces of wind and earthquakes because of their open exposure to those forces.

2. Scope-of-Work

The scope of the work is applicable mostly to highway bridges without any limitation on their size and function: such as existing bridges and new bridge designs; whole system of bridges; and/or to single components of a bridge. The mission is performed through:

conducting workshops, exchanging researchers,
developing methods for design evaluations and test procedures,
inspection techniques, rehabilitation and maintenance specifications and policies,
and performing cooperative research programs and other relevant cooperative administrative activities.

3. Accomplishments

* Conducted coordinated research studies on the seismic performance of bridge piers and columns. Details of this work involved determing the performance of reinforced concrete piers and columns subjected to dynamic cyclic loading, performing model tests on the failure of reinforced concrete piers, testing full-scale concrete columns and for the behavior of concrete filled steel tubes.

* Held the fourth bridge workshop in San Diego, U.S. in 1988 and the fifth bridge workshop in Tsukuba, Japan in 1989, dealing with the determination and evaluation of the performance and strengthening of bridge structures, structural monitoring, repairs, safety and non-destructive evaluation. Special attention was given to the seismic and wind loadings of modern cable -stayed bridges, vibration suppression, base isolation and dynamic control techniques.

* Both sides presented technical papers at the UJNR Joint Meeting and participated in technical study tours such as factories, construction sites, and instrumented structures.

4. Future Plans

* Conduct the 6th bridge workshop just prior to the 22nd Joint Panel

meeting in the U.S. .

* Continue the coordinated experimental research study on the seismic performance of bridge piers and columns, and continue emphasis on base isolation.

* Develop a coordinated research study on seismic, aeroelastic, and aerodynamic response of cable-supported bridges with emphasis on cable inspection, vibration and corrosion protection.

* Develop a coordinated research study to compare the seismic design criteria for bridges in JAPAN and the U.S.

* Develop a coordinated research study on seismic response control, system identification techniques, and non-destructive evaluation of bridge structures.

5. Information Exchange

* Exchanged technical reports, research program documentation, construction logs, design plans, and assorted photographs and TV videos.

* Mr. S. Unjoh from PWRI worked in University of Southern California and Mr. Ray Zelinsky from Transportation Bureau of California State worked in PWRI to engage in research studies relating to this Task Committee.

6. Impact

* Greater uniformity was achieved in wind engineering test procedures and modeling, resulting in more efficient solutions to the aerodynamic bridge problem.

* The workshops and study tours in the two countries have facilitated technology transfer, as an example, in the area of cable protection and vibration suppression of cable-stayed bridges.

* With respect to the earthquake protection of bridges, the free exchange of literature, instrumentation technology, and earthquake response data has led to the improvement of design and retrofitting guidelines and to calibration and verification of specifications.

* A set of priorities for research needed from a more international viewpoint were established.

7. Barriers

There are no major technical barriers to achieve the Task Committee mission. However, financial barriers, especially funding needed to participate in workshops and panel meetings and for conducting actual bridge tests and large scale model tests, limit the effectiveness of the work in this Task Committee's work.

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5. AUTHOR(S)

Noel J. Raufaste, Editor

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NATIONAL INSTITUTE OF STANDARDS AND TECHNOLOGY
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GAITHERSBURG, MD 20899

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9. SPONSORING ORGANIZATION NAME AND COMPLETE ADDRESS *(Street, City, State, ZIP)*

SAME AS ITEM #6 ABOVE.

10. SUPPLEMENTARY NOTES

☐ Document describes a computer program; SF-185, FIPS Software Summary, is attached.

11. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here)*

The 21st Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects was held at the Public Works Research Institute, Tsukuba, JAPAN, from May 16-19, 1989. This publication, the proceedings of the Joint Meeting, includes the program, list of members panel resolutions, task committee reports, and 33 technical papers.

The papers were presented under six themes: (I) - Wind Engineering, (II) - Earthquake Engineering, (III) - Storm Surge and Tsunami, (IV) - U.S.-Japan Cooperative Research Program, (V) - Code Development Process and Code Enforcement Responsibility, and (VI) - Armenia Earthquake (oral presentation).

12. KEY WORDS *(Six to twelve entries; alphabetical order; capitalize only proper names; and separate key words by semicolons)*

accelerograph; Armenia; bridges; codes; concrete; design criteria; disaster; earthquakes; geotechnical engineering; ground failures; inelastic; lifelines; liquefaction; masonry; repair and retrofit; risk assessment; seismicity; soils; standards; storm surge; structural engineering; tsunami; and wind loads

13. AVAILABILITY

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