NSF/CEE-84004

PB84-209196

Center for Civil Engineering Earthquake Research

PROCEEDINGS
of the
FIRST USA - JAPAN BRIDGE ENGINEERING
WORKSHOP

Public Works Research Institute
Tsukuba, Japan Feb. 20-22 1984

Report No. CCEER-84-2
Editors Bruce Douglas
and Toshio Iwasaki

• Reno

Engineering Research and Development Center
College of Engineering
University of Nevada Reno

INFORMATION RESOURCES
NATIONAL SCIENCE FOUNDATION

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/CEE-84004	2.	3. Recipient's Accession No. PB84-209196

4. Title and Subtitle
USA-Japan Bridge Engineering Workshop, Proceedings of the First, Tsukuba, Japan (February 20-22, 1984)

5. Report Date 1984

6.

7. Author(s)
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8. Performing Organization Rept. No.

9. Performing Organization Name and Address
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College of Engineering
Reno, NV 89557

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C)
(G) CEE8318486

12. Sponsoring Organization Name and Address
Directorate for Engineering (ENG)
National Science Foundation
1800 G Street, N.W.
Washington, DC 20550

13. Type of Report & Period Covered
Proceedings

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

A schedule of the workshop and an itinerary of the study tour are presented, along with lists and biographical sketches of both Japanese and American participants. Papers delivered at the workshop are provided, and focus on such topics as: specification and seismic design forces for earthquake-resistant design of highway bridges and substructures of highway bridges in Japan, ductility demand for bridge piers, embankment of sandy layers, seismic-resistant bridge design criteria in California, static and dynamic response of cable stayed bridges, and dynamic field testing of full-scale highway bridges. The need for coordinated research between the U.S. and Japan in the following areas is cited: repair procedures for seismically damaged bridge structures, experimental field testing of bridges, and studies of the effects of soils on the behavior of bridges. Appendices explore the main bridges of the Kan-Etsu Expressway.

17. Document Analysis a. Descriptors

Meetings	California	Soils
Dynamic structural analysis	Earthquakes	Soil mechanics
Earthquake-resistant design	Bridge foundations	Japan
Bridges (structures)	Design standards	

b. Identifiers/Open-Ended Terms
Kan-Etsu Expressway (Japan)   B. Douglas, /PI

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		218
	20. Security Class (This Page)	22. Price

(See ANSI-Z39.18)   *See Instructions on Reverse*   OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

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PREFACE

During the Fifteenth Joint Meeting at the U.S. - Japan Panel on Wind and Seismic Effects of the UJNR held at Tsukuba, Japan May 17-20, 1983, several of the papers were devoted to bridge earthquake engineering. As a result of the mutual beneficial interaction which took place between the bridge engineering communities of both countries at the May, 1983, UJNR meeting, it was suggested that it would be very beneficial to hold a workshop in the near future where topics on bridge earthquake engineering could be more fully explored. The First USA - Japan Bridge Engineering workshop resulted from this suggestion. The financial sponsor for the US participants was the NSF through a grant CEE 8318486 awarded to the University of Nevada at Reno.

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SCHEDULE AND AGENDA

Schedule of U.S.-Japan Bridge Workshop
(at Public Works Research Institute)

February	20 (Mon)	Opening Session (S-1)
		Technical Sessions (S-2)
		Visit the Building Research Institute
		Reception (to be hosted by the Director General of PWRI)
	21 (Tue)	Technical Sessions (S-3,4,5)
		Observation the Facilities of PWRI
		Reception (to be sponsored by Japanese members)
	22 (Wed)	Technical Sessions (S-6,7)
		Discussions (S-8)
		Resolutions (S-9)
		Reception (to be sponsored by U.S. members)
	23 (Thu)	Study Tour
		Visit the Kan-etsu Expressway (bridge construction sites)
	24 (Fri)	Study Tour
		Visit Tokyo Metropolitan Expressway (viaduct construction sites)
	25 (Sat)	Study Tour
		Visit the Kajima Institute of Construction Technology

Itinerary of Study Tour

Thursday, February 23 - Saturday, February 25

Course: Tsukuba - Takasaki - Minakami - Tokyo

Feb. 23 (Thu)	8:00	Lv.	Tsukuba
	10:30 10:54	Ar. Lv.	Kumagaya Station (by Express "Yukemuri 3")
	11:26	Ar.	Takasaki
		Lunch	
		Visit the Kan-etsu Expressway (bridge construction sites)	
	17:30	Ar.	Minakami (Hotel Minakami-kan, Tel 02787-2-3221)
24 (Fri)	8:00	Lv.	Hotel Minakami-kan
	9:00 9:16	Ar. Lv.	Jyomo-Kogen Station (by Super Express)
	10:10 10:21	Ar. Lv.	Omiya Station
	10:47 11:00	Ar. Lv.	Ueno Station
	11:20	Ar.	First Construction Office, Metropolitan Expressway Public Corporation
		Lunch	
	13:00	Lv.	above Office
	14:00 15:30	Ar. Lv.	Metropolitan Expressway Construction Site
	17:00	Ar.	Akasaka Tokyu Hotel
25 (Sat)	9:00	Lv.	Akasaka Tokyu Hotel
	10:00	Ar.	Kajima Institute of Construction Technology
		Lunch	
	13:00	Lv.	above Institute
	14:00	Ar.	Akasaka Tokyu Hotel

Remark: Feb. 23, 24 : escorted by Sato, Arakawa, Ikeda, Hagiwara
Feb. 25 : escorted by Iwasaki, Ikeda

AGENDA

Mon. Feb. 20

1:00 Session-1 Opening Session (At 8F, PWRI)
(Chairmen: Narita, Douglas)
Address by Dr. R.Iida, Director-General, PWRI
Address by Mr. J.D.Cooper, U.S. Chairman, T/C(J)
Address by Dr. M.Murakami, Japan Chairman, T/C(J)
Introduction of All Participants

1:35 Adoption of Agenda

1:40 Group Photograph

1:45 Session-2 General Presentations (At 8F, PWRI)
(Chairmen: Murakami, Fleming)

1:45	U-1	"AASHTO Guide Specifications - Seismic Design for Highway Bridges," (Cooper)
2:05	J-1	"Outline of the Specifications for Earthquake-Resistant Design of Highway Bridges in Japan," (Iwasaki)
2:20	U-2	"Seismic Resistant Bridge Design Criteria in California," (Gates)
2:40	J-2	"Outline of the Specifications for Substructures of Highway Bridges in Japan," (Kaminaga)
2:55	U-3	"NBS Large Scale Seismic Testing Project," (Lew)
3:10	J-3	"Sesmic Design Forces of Highway Bridges in Japan," (Kawashima)
3:25	U-4	"Transportation Research Board Bridge Engineering Activities," (Spaine)

3:40 Break

3:55 Leave PWRI

4:00 Arrive BRI
Visit BRI Lab's

5:00 Leave BRI

5:10 Arrive Kenshu-Kaikan (Hotel)
Tel. (0298)64-2844

Tue. Feb. 21

8:40 Leave Kenshu-Kaikan

9:00 Session-3 General Presentations (At 8F, PWRI)
(Chairmen: Ohshima, Gates)

9:00	U-5	"Box Girder Bridge Hinge Restrainer Test Program," (Selna)
9:15	J-4	"Effects of Soil Liquefaction," (Arakawa)
9:30	U-6	"Implementation of the Analytical Capabilities Required for the Aseismic Design of Bridges," (Imbsen)
9:45	J-5	"Dynamic Response Analysis for Seismic Design of Highway Bridges in Japan," (Hagiwara)
10:00	U-7	"Seismic Response of Meloland Road Overpass," (Werner)
10:15	J-6	"Ductility Analysis of Reinforced Concrete Piers," (Kobayashi)

10:30 Break

11:00 Session-4 General Presentations (At 8F, PWRI)
(Chairmen: Arakawa, Goins)

- 11:00 U-8 "Bridge Foundation Problem," (Arden)
- 11:15 J-7 "Earthquake Resistant Design of Tokyo Bay Crossing Bridge," (Hirukawa)
- 11:30 U-9 "Full-Scale Dynamic Testing and Earthquake Response of Suspention Bridges," (Scanlan)
- 11:45 J-8 "Earthquake Resistance Design of the Meiko Nishi Bridge," (Takahashi)

12:00 Lunch

1:00 Session-5 General Presentations (At 8F, PWRI)
(Chairmen: Sato, Lew)

- 1:00 U-10 "AASHTO and Bridge Earthquake Engineering," (Freidenrich)
- 1:15 U-11 "Seismic Design Guidelines for Highway Bridges - A Brief Overview of Their Use," (Dodson)
- 1:30 J-9 "A Dynamic Analysis of a Multi-Span Continuous Bridge," (Hara)
- 1:45 U-12 "Highway Bridge Design Specifications for Seismic Loads in the United States," (Goins)
- 2:00 U-13 "Cable Stayed Bridges - Static and Dynamic Response," (Fleming)
- 2:15 J-10 "Seismic Design of Ajigawa Bridge," (Nakajima)
- 2:30 U-14 "Dynamic Field Testing of Full Scale Highway Bridges," (Douglas)
- 2:45 J-11 "Earthquake Resistant Design of Bannosu Viaduct," (Higuchi)

3:00 Break

3:20 Visit PWRI Lab's
5:00 Leave PWRI
5:10 Arrive Kenshu-Kaikan

Wed. Feb. 22

8:40 Leave Kenshu-Kaikan
9:00 Session-6 Future Programs (At 8F, PWRI)
(Chairmen: Yamamoto, Spaine)

- 9:00 J-12 "Concrete Division, PWRI," (Kobayashi)
- 9:10 J-13 "Research Plan on Earthquake Resistance," (Ohshima)
- 9:20 J-14 "Behavior of Concrete Filled Steel Tubes," (Part-3 Beam-Columns Members) (Miyata)
- 9:30 U-15 "National Science Foundation," (Douglas)
- 9:40 J-15 "Ground Vibration Division, PWRI," (Arakawa)
- 9:50 J-16 "Future Programs at Earthquake Engineering Division, PWRI," (Iwasaki)
- 10:00 U-16 "Natioral Bureau of Standards," (Lew)

10:15 Break

10:45 Session-7 Future Programs (At 8F, PWRI)
(Chairmen: Kobayashi, Selna)

10:45 J-17 "Earthquake Resistant Design and Tests of the Katashina-gawa Bridge," (Kadotani)
10:55 U-17 "Federal Highway Administration," (Cooper)
11:05 J-18 "Observation of the Behavior of Multi-Span Continuous Girder Bridges," (Miyauchi)
11:15 U-18 "State of California," (Gates)
11:25 J-19 "Hanshin Expressway Public Corporation," (Nakajima)
11:35 U-19 "National Cooperative Highway Research Programs," (Spaine)
11:45 J-20 "Earthquake Resistant Design of Akashi Kaikyo Bridge," (Yamagata)

12:00 Lunch

1:00 Session-8 Discussions on Future U.S.-Japan Coordinated Programs (At 8F, PWRI)
(Chairmen: Iwasaki, Douglas)

3:30 Break

4:00 Session-9 Resolutions (At 8F, PWRI)
(Chairmen: Murakami, Cooper)
5:00 Leave PWRI
5:10 Arrive Kenshu-Kaikan

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RICHARD W. ARDEN IS PRESIDENT OF SEA ENGINEERS/PLANNERS WITH CORPORATE HEADQUARTERS IN SPARKS, NEVADA AND BRANCH OFFICES IN LAS VEGAS, NEVADA AND SEATTLE, WASHINGTON. AS ADMINISTRATIVE HEAD OF THE FIRM, MR. ARDEN HAS DIRECTED IT TO THE FOREFRONT IN ENGINEERING, PLANNING, GEOTECHNICAL, AND SURVEYING SERVICES. HE HAS ACTED AS PRINCIPAL-IN-CHARGE, AND COORDINATES THE ACTIVITIES OF THE FIRM'S PROJECTS WITH GOVERNMENTAL AGENCIES AND PRIVATE CLIENTS AND OVERSEES THEIR COMPLETION DURING THE DESIGN AND CONSTRUCTION STAGE.

THE FIRM HAS DESIGNED BRIDGES IN THE NORTHERN AND SOUTHERN PART OF THE STATE OF NEVADA. SEA RECEIVED A SPECIAL AWARD FROM THE PACIFIC SOUTHWEST REGION PORTLAND CEMENT ASSOCIATION IN RECOGNITION OF CREATIVE DESIGN AND EXPRESSIVE USE OF CONCRETE ON THE GREG STREET BRIDGE IN SPARKS, NEVADA. MR. ARDEN HAS PERFORMED NUMEROUS GEOTECHNICAL INVESTIGATIONS FOR BRIDGES ON THE INTERSTATE AND SECONDARY SYSTEM IN THE STATE OF NEVADA. MR. ARDEN HAS WRITTEN NUMEROUS REPORTS AND STUDIES FOR THE BENEFIT OF INDIVIDUAL CLIENTS.

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Mr. Cooper serves in a supervisory capacity in the Structures Division. As such, he oversees all contract, staff, and Federal-aid research activity of the division, which encompasses two projects and 10 tasks in the Federally Coordinated Program (FCP) in R&D. These projects are dedicated to improving the structural adequacy, safety, strength, economy and serviceability of bridges, tunnels, and other highway structures. Prior to this position, he served a Presidential Internship in Engineering and Science, with the Office of Research, Federal Highway Administration and was an instructor at Syracuse University.

Mr. Cooper has authored or co-authored many research papers, and is a member of several scientific and professional societies, including the Earthquake Engineering Research Institute, Sigma Xi, and the American Society of Civil Engineers, where he serves as Chairman of the Program and Publications Committee of the Technical Council on Lifeline Earthquake Engineering. He currently serves as a consultant to the Trans-European Motorway Project through the United Nations. He also acts in an advisory capacity to other elements of the Department of Transportation. He served as a member of the Editorial Board of the Journal of Civil Engineering Design. He was a recipient of an Outstanding Young Men of America Award in 1974 in recognition of ability, accomplishments, and service to community, a Community Leaders and Noteworthy Americans Award in 1976 in recognition of achievements and outstanding service to community and State, and the Federal Highway Administrator's Superior Achievement Award in 1981. He is a registered Professional Engineer.

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Jim Dodson is a native Nevadan who was born in Reno in 1948. Educated in local schools, he received a Bachelor of Science degree in Civil Engineering from the University of Nevada at Reno in 1971.

His entire career has been spent with the Nevada Department of Transportation. After completing the Department's eighteen month Rotational Engineer Program, he was assigned as an Assistant Resident Engineer in Elko, Nevada. Projects he worked on during the three years spent in the Construction Division included a twin bore interstate highway tunnel and three interstate highway bridges.

In 1975, he transferred into the Bridge Division as a Senior Bridge Designer. With a construction background, he continued to work with field personnel on structurally related construction matters. He also developed and taught a course on bridge construction to Department field personnel.

In 1980, Dodson was temporarily assigned to the Program Engineer to develop a prioritization process for State funded construction projects. During this time, he also worked with Federal Highway Administration personnel in the development of their Preconstruction Engineering Management System.

In 1982, Dodson became Chief Bridge Engineer. His efforts since assuming this position have been concentrated in strengthening the maintenance inspection and bridge replacement program areas.

In charge of a relatively small staff which is located in a seismically active area, he is interested in providing ways for his staff as well as others similar in make up to stay abreast with current seismic design practice.

BRUCE M. DOUGLAS, Director
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Reno, Nevada 89557

Dr. Douglas obtained his B.S.C.E. degree from the University of Santa Clara in 1959, and his M.S. and Ph.D. degrees in Engineering Mechanics from the University of Arizona in 1965. He has been at the University of Nevada at Reno (UNR) for the last 20 years, where he has served as an assistant, associate and full professor of civil engineering. During the period 1974-1976, he served as the associate director of the Seismological Laboratory in the Mackay School of Mines, and as the chairman of the Civil Engineering Department at UNR between 1976-1984. He is a member of a number of professional societies including the International Association of Bridge and Structural Engineering, the Earthquake Engineering Research Institute and the American Arbitration Association which he serves as a panel member. He also is a member of the Transportation Research Board Committee for Dynamics and Field Testing of Highway Bridges, and has participated as one of the United States delegates to workshops on bridges and/or earthquake engineering in New Zealand, China and Japan.

His recent research interests have been primarily related to the lateral dynamic testing of full scale highway bridges at high amplitudes, and examining earthquake response data obtained from highway bridges. The focus of these studies has been to apply system identification methods to both types of bridge response data in order to identify the bridge foundation and structural parameters which significantly affect the distribution of seismic loads. He has published a number of papers on this subject.

John F. Fleming

Date of Birth: December 15, 1934

Birthplace: Indiana, Pennsylvania

Education:
- Carnegie Institute of Technology, B.S. Degree
  Civil Engineering 1957
- Carnegie Institute of Technology, M.S. Degree
  Civil Engineering 1958
- Carnegie Institute of Technology, Ph.D. Degree
  Civil Engineering 1960

Present Position:
Professor of Civil Engineering
University of Pittsburgh

Dr. Fleming has been a member of the faculty of the University of Pittsburgh, Department of Civil Engineering, since 1969. He is engaged in teaching and research in the area of structural engineering with emphasis on computer analysis of structural systems for static and dynamic loadings. One of his particular research interests, in recent years, has been the static and dynamic analysis of cable stayed bridges. He has authored a number of publications on this subject. He is presently completing a National Science Foundation sponsored project on Siesmic Analysis of Cable Stayed Bridges.

Immediately prior to joining the faculty of the University of Pittsburgh, Dr. Fleming was a project engineer, with the consulting engineering firm, General Analytics, Inc., for four years. He was responsible for supervising projects in structural analysis and design and in developing computer capabilities in the company. From 1960 through 1965, Dr. Fleming taught in the Civil Engineering Department of Northwestern University in Evanston, Illinois.

Dr. Fleming is a Registered Professional Engineer in the State of Pennsylvania, and is a member of the American Society of Civil Engineers. He has acted as an engineering consultant for a number of companies, such as, United States Steel Corporation, and Westinghouse Electric Corporation, in developing computer programs for structural analysis and design. He is presently very interested in the applications of computer graphics as an analysis and design tool.

Jack Freidenrich was educated in the Paterson, New Jersey school system and subsequently graduated with honors from Southern Methodist University with a Bachelors Degree in Civil Engineering. He was elected to National Honorary Mathematics and Engineering Fraternities, and did graduate work at New York University in structures and soils. He is a licensed professional engineer, a member of the American Society of Civil Engineers, and member and past president of the Mercer County Chapter of the New Jersey Society of Professional Engineers. In 1976, he was designated "Engineer of the Year" by the Professional Engineers Society of Mercer County, and subsequently by the Central Jersey Engineering Council.

Mr. Freidenrich is the past president of the Northeast Association of the State Highway and Transportation Officials, Chairman of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures, Chairman of the AASHTO Route Numbering Committee, Chairman of the Transportation Research Board (TRB) NCHRP Panel 20-7, AASHTO representative on the Joint AASHTO/Association of County Officials/Association of County Engineers Committee, member of the AASHTO Standing Committee on Highways, member of TRB Advisory Panel SP20-5, past member of the AASHTO Executive Committee, and a member of the Rutger's University Civil and Environmental Engineering Advisory Committee. For 15 years, he served as instructor in the evening engineering program at Trenton Junior College (Mercer County Community College) and the School of Industrial Arts.

Mr. Freidenrich joined the Department in 1943 as a Junior Engineer, and for the past 12 years has served as Director of Engineering and Operations. In December of 1983, he was appointed Assistant Commissioner for Engineering and Operations. At the recent Annual AASHTO Meeting in Denver, Colorado, Jack Freidenrich was presented with the 27th Thomas H. MacDonald Award, granted annually by AASHTO in recognition of outstanding achievement by an individual in the field of highway administration, engineering or research.

James H. Gates
Structural Mechanics Engineer
Office of Structures Design
California Department of Transportation

December, 1983

BIOGRAPHICAL SKETCH

Mr. Gates received a Bachelor of Science Degree in Civil Engineering from the University of Cincinnati in 1959. His total professional experience since graduation has been with the California Department of Transportation.

He is a registered Civil Engineer in California and is a member of the American Society of Civil Engineers, the Earthquake Engineering Research Institute, and is active on a number of technical committees.

Between 1960 and 1970 he was involved in the design of over 100 bridges including many prize winners.

Since the 1971 San Fernando earthquake he has been involved primarily in seismic related work, including the development of the current CALTRANS seismic criteria for bridge design, the instrumentation of bridges for strong-motion recording, the measurement of ambient vibration data on various bridges, the development of seismic training programs, and the writing of a number of technical papers.

He has also been involved in the development and implementation of improved computer programs for the design of bridges, including, the development of the real-time interactive system used at CALTRANS, the improved column and footing analysis programs, and the development of a simplifying pre-processor for modal analysis of bridges.

He also participated in the development of the Applied Technology Council project, ATC-6, which resulted in a national specification for the seismic design of bridges. He was a member of the 1981 NSF team from the US which visited New Zealand to develop a cooperative research program, and was a member of the 1983 United States-Japan Panel on Wind and Seismic Effects.

BIOGRAPHICAL SKETCH

Veldo M. Goins, P.E.
Bridge Engineer
Okla. Dept. of Transportation

Veldo M. Goins is forty-four years of age, having grown up and graduated from Highschool in Ada, Oklahoma, a small town 85 miles southeast of Oklahoma City. He attended the University of Oklahoma receiving a BS in Civil Engineering in 1963 and attended graduate school on a part-time basis while working full-time. He received professional registration in 1967.

Mr. Goins has worked for the past twenty years for the Oklahoma Department of Transportation, and for the last twelve years has held the position of Bridge Engineer in charge of the Bridge Division responsible for the supervision of seventy-five engineers and technicians in the design, preparation of plans and maintenance inspection of bridges on the State Highway System of Oklahoma.

In 1972, he was appointed as member of the AASHTO Bridge Committee and Chairman of the Technical Committee for Loads and Load Distribution, both positions still held. This committee and sub-technical committees write and update the "AASHTO Standard Specifications for Highway Bridges."

He has served on the following committee assignments:

Member 12 years, Vice-Chairman 2 years-AASHTO Subcommittee for Bridges and Structures
Chairman 12 years - AASHTO Technical Committee for Loads and Load Distribution
Member 4 years - ASCE Loads and Forces on Bridges
Member Project Panel - ATC-6-1 - Seismic Design Guidelines for Highway Bridges
Member Project Panel - ATC-6-2 - Seismic Retrofit Guidelines for Highway Bridges
Participant - ATC-12 - Seismic Engineering Workshop, New Zealand - 1981
Member Project Panel - NCHRP Project 20-5 Topic 14-22, Distribution of Wheel Loads on Highway Bridges

Roy A. Imbsen, P.E.
Engineering Computer Corporation

Mr. Imbsen is a co-founder and principal partner in ECC, which was established 1976. As Vice President, he manages the Structural Mechanics and Research Section and is currently conducting bridge-related research and development.

Mr. Imbsen began his professional career with the Bridge Department, California Department of Transportation, after receiving a BSCE degree from the University of Illinois in 1962. During his fourteen year tenure with the California Bridge Department, he was involved in various bridge design, construction and research activities. As a Senior Bridge Engineer in Research and Development, he helped develop and implement various new methodologies for bridge design, including the seismic design criteria for bridges developed after the 1971 San Fernando earthquake.

At ECC, Mr. Imbsen has continued to direct various bridge-related research projects for the Federal Highway Administration, Transportation Research Board - National Cooperation Highway Research Program, National Science Foundation and California Department of Transportation. These projects have included the development of workshops and computer programs for implementation on newly-developed seismic design methodologies, the development of new methods for evaluating the load-carrying capacity of existing reinforced concrete bridges and the development of new design criteria for considering thermal effects in concrete superstructures.

During this time, Mr. Imbsen has continued his education. He obtained his MS degree in 1972 and is completing the dissertation requirements for a PhD in Engineering.

Mr. Imbsen is registered as a professional engineer in the states of California, Illinois and Washington. The organizations to which he holds professional memberships include the ASCE, ACI, EERI, SEAOC and TRB.

H. S. LEW

Education: (Degrees)

Washington University, B.S. Degree
Architectural Engineering, 1960
Lehigh University, M.S. Degree
Civil Engineering, 1963
University of Texas, Ph.D. Degree
Civil Engineering, 1967

Position:

Leader, Construction Safety Group
Structures Division
Center for Building Technology
National Engineering Laboratory

Dr. Lew presently serves as Leader, Construction Safety Group in the Center for Building Technology, National Bureau of Standards. He plans and manages overall technical programs dealing with productivity and safety as it relates to construction of structures. He serves on various technical and administrative committees of national, professional and standards organizations. He serves on a number of advisory committees of engineering societies and industry which provide guidance to research activities in several universities. He also serves as Secretary of the U.S.-Japan Joint Panel on Wind and Seismic Effects, U.S.-Japan Cooperative Program on Natural Resources. He is currently serving on standing board committees of the American Concrete Institute (ACI) and was the President of the National Capitol Chapter of ACI.

Dr. Lew was Assistant Professor of Civil Engineering at the University of Texas at Austin prior to his service with the Bureau, and is Associate Professorial Lecturer at the George Washington University in Washington, D.C. Prior to his graduate studies, he worked at a large consulting engineering firm as a structural engineer.

Dr. Lew has published numerous papers and reports. He is recognized for his work in investigating and reporting on the cause and prevention of structural failures during construction. He is a member of the American Society of Civil Engineers, the American Concrete Institute, the American National Standards Institute, the National Safety Council, and the Structural Stability Research Council. He is chairman and member of several technical committees of these organizations including Chairman of ACI Committee 228 on Nondestructive Testing of Concrete, and Member of ACI Committee 318 on Standard Building Code.

Dr. Lew is a registered professional engineer. He is a member of the honor societies of Sigma Xi and Chi Epsilon. Dr. Lew is the recipient of several honors and awards including the ACI Wason Medal for the "Most Meritorious Paper in Materials Research," 1977; the National Safety Council's Cameron Award for "Outstanding Construction Safety Work," 1982; and the CBT Communications Award in 1980 for excellence in dissemination of research results of building technology. He is a 1980 recipient of the U.S. Department of Commerce Bronze Medal Award for "Outstanding technical contributions in construction failure investigations." He is also a 1982 recipient of the U.S. Department of Commerce Silver Medal Award for "Outstanding contributions in enhancing the safety of the work environment during building construction."

Biographical Sketch

Robert J. Reilly

Projects Engineer
Transportation Research Board
National Academy of Sciences
2101 Constitution Avenue, NW
Washington, DC 20418
202/334-3224

As Projects Engineer for the National Cooperative Highway Research Program, Robert Reilly is responsible for technical administration of AASHTO-sponsored, contract research in the area of structural engineering. This includes preparation of NCHRP publications and presentations of research findings at meetings of AASHTO and other interested organizations. He presently monitors more than 20 projects representing total funding of almost $4 million. In recent years, there has been a strong interest by NCHRP's sponsors in research on the structural evaluation, repair, and rehabilitation of existing bridges.

Robert Reilly is a graduate of Manhattan College (1960) and earned MS and PhD degrees in Civil Engineering from the University of Maryland (1962 and 1967). Before joining the TRB staff in 1972, he served on the Civil Engineering faculty at the University of Maryland and worked as an engineer for the U.S. Government, a ready-mix concrete company, and several consulting firms. While at the University of Maryland, he taught undergraduate and graduate courses in the areas of structural engineering, applied mathematics, engineering materials, computer methods, and solid and fluid mechanics. During this period, he also worked on three highway bridge research projects sponsored by the Maryland Department of Transportation.

While at the Veterans Administration in 1972, he worked on a program for structural rehabilitation of VA buildings following the loss of lives in the collapse of hospital buildings in the 1971 San Fernando earthquake. Robert Reilly has lectured in programs sponsored by the AISC, AASHTO, and AISI, and has published articles in the ASCE Structural Division Journal and International Journal of Computers and Structures. He is married, has five children and resides in Bowie, Maryland.

January 25, 1984

Robert H. Scanlan

Professor Scanlan, who has been at Princeton University in the Department of Civil Engineering since 1966, was born in Chicago and received his early training there in the public schools, Armour Institute of the Illinois Institute of Technology, and the University of Chicago. He later did graduate studies at M.I.T. and the Sorbonne, Paris, receiving a doctoral degree from each of these institutions.

He has had a range of technical and research experience including industry, government, and the university. His university experience immediately prior to accepting a professorship at Princeton was as director of the Structures and Mechanics Division at Case Institute of Technology, Cleveland. He is currently Director of the Princeton program in Structures and Mechanics.

His principal areas of interest include vibrations, stress, structural dynamics, and fluid-structure interaction, with emphasis on the effects upon civil engineering structures of earthquake and wind. He was recently chairman of the ASCE Committee on Dynamics for the Engineering Mechanics Division and Task Committee on Wind Forces of the ASCE. He is a founding member of the Wind Engineering Research Council, a national organization. He is author of over 100 technical papers in his areas of interest and has been consultant in these areas to agencies of the U.S. and foreign governments, and to private firms. He is co-author of three texts, most recent of which is one on Wind Engineering (with E. Simiu), Wiley, 1978.

He has carried out research related to wind and earthquake effects upon structures, notably on the stability and reliability under wind of long bridges. He has been consultant on the aerodynamics of a number of bridges, buildings, and other structures.

He has been a consultant and expert witness on a variety of structural and mechanical engineering concerns, notably as regards stress, fatigue, and design reliability. He is a Registered Mechanical Engineer in the State of Ohio.

Lawrence G. Selna
3173 Engineering I Building
University of California
Los Angeles, California 90024
USA

Phone (213) - 825-5502

Position: Professor, Department of Civil Engineering
University of California
Los Angeles, California

Education: Ph.D. in Structural Engineering
University of California, Berkeley
September 1967

Professional Experience: Registered Civil and Structural Engineer in California

Research Interest: Professor Selna is performing full scale seismic testing of bridge components. The components used for seismic retrofitting of reinforced concrete box girder bridges are subjected to cyclic displacement histories which eventually cause failure of the restraining device. The bridge structure surrounding the restrainer and the restrainer itself are constructed to full scale for use in the experimental work. The testing program is conducted at the University of California, Los Angeles and is sponsored jointly by the National Science Foundation and California State Department of Transportation. The experimental program has been underway since July 1982.

Teaching Interest: Professor Selna teaches structural engineering courses with emphasis on design and Earthquake Engineering. A course on Bridge Design is a featured course in the design sequence.

Biographical Sketch

Lawrence F. Spaine

Engineer of Design
Transportation Research Board
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, DC 20418
(202) 334-2950

For the past 18 years Mr. Spaine has been the Engineer of Design for the Transportation Research Board. He is a staff engineer in the Technical Activities Division with wide latitude for independent decision and action in planning and directing the Board's activities in design technology. He is charged with the responsibility of the Board's mission of stimulating and correlating research activities and disseminating research information on design aspects of transportation systems. Specific areas of responsibility include photogrammetry and aerial surveys, geometrics, hydrology, hydraulics, environmental design, utilities, pavement design and structures design. Mr. Spaine also serves on approximately 28 advisory panels in the TRB administered National Cooperative Research Program.

Mr. Spaine has a broad background in Civil Engineering having served six years with the U.S. Army Engineers, three years as a faculty member at North Carolina State University, ten years as assistant bridge engineer for the Seaboard Railroad Company and three years as structural engineer for consulting firms. At the railroad he was charged with the responsibility of design and construction of the railroad and highway bridges on the system's right-of-way.

Mr. Spaine was graduated from North Carolina State University with a BSCE degree. He is credited with graduate studies in structures, foundations and transportation economics. He is a registered professional engineer in Virginia and North Carolina and is a member of numerous professional societies including AREA, ASCE, ASTM, ACI and ARTBA.

BIODATA — S.D. WERNER

Mr. Stuart Werner is Associate and Technical Director of Agbabian Associates, a consulting engineering company located in the Los Angeles, California area. Mr. Werner's technical specialty is earthquake engineering, and he has extensive experience in earthquake-related projects dealing with seismic wave propagation, seismic risk and criteria development, soil/structure interaction, and structural response. He has been principal investigator of several NSF-sponsored studies of the seismic behavior of bridges; these have included evaluations of traveling wave effects on bridge response and, most recently, an assessment of seismic response characteristics of the Meloland Road Overpass using bridge motions measured during the 1979 Imperial Valley earthquake. Mr. Werner has earned an M.S. degree from Northwestern University and an Engineer's Degree from MIT, both in Civil Engineering. He is a registered Civil Engineer in California and Pennsylvania, and is a member of ASCE, EERI, the Seismological Society of America, and the Structural Engineer's Association of Southern California.

JAPAN BIOGRAPHICAL SKETCHES

PERSONAL CAREER

NAME : Ryuichi IIDA

POSITION : Director-General
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi 1-banchi, Toyosato-machi
Tsukuba-gun, Ibaraki-ken, 305
JAPAN
Tel. 0298-64-2211

DATE OF BIRTH : January 30, 1930

PERMANENT ADDRESS : Tokyo Prefecture, JAPAN

EDUCATION : Bachelor of Civil Engineering
Tokyo University
Doctor of Civil Engineering

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1953-1964	Research Engineer Dam Structure Division Public Works Research Institute
1964-1972	Head, Dam Structure Division
1972-1973	Head, Fill Dam Division
1973-1979	Director, Dam Department
1979-1980	Director Planning and Research Administration Department, PWRI
1980-1982	Assistant Director-General, PWRI
1982-1982	Deputy Director-General, PWRI
1982-	Director-General, PWRI

MAJOR SUBJECT AND ITS SUMMARY : Dam Engineering, Rock Mechanics

TRAVEL ABROAD : U.S.A., Italy, Spain, France, Portugal and Indonesia

MAJOR PUBLICATION :
"Rock Mechanics," May, 1987

PERSONAL CAREER

NAME : Nobuyuki NARITA

POSITION : Director
Planning and Research Administration Department
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi-1, Toyosato-machi, Tsukuba-gun
Ibaraki-ken, 305, JAPAN
Tel. 0298-64-2211

DATE OF BIRTH : July 10, 1934

PERMANENT ADDRESS : Chiba Prefecture, JAPAN

EDUCATION : In Structural Engineering, Bridge Engineering, River Engineering and Concrete Engineering at Department of Civil Engineering,Faculity of Engineering, Tokyo University
In Structural Engineering and Bridge Engineering at Graduate Course, Faculty of Engineering, Tokyo University
Doctor of Engineering

SPECIALITY : Wind resistant design of civil engineering structures

MAJOR AREAS OF EXPERIENCE :

1960-1964	Research on earthquake engineering
1960-	Research on wind resistant design of bridges
1970-1980	Head, Structure Division, Structure and Bridge Department, Public Works Research Institute Ministry of Construction
1980-1983	Director, Structure and Bridge Department Public Works Research Institute Ministry of Construction

MAJOR SUBJECT : Wind resistant design of bridges

TRAVEL ABROAD : United states, United Kingdom, Portugal, Burma and Tanzania

MAJOR PUBLICATION :
"Wind resistant design of a cable stayed bridge with solid bridge girder," March, 1978

PERSONAL CAREER

NAME:	Michimasa IKEDA
POSITION:	Bridge Instructor, Planning and Research Administration Department, Public Works Research Institute, Ministry of Construction
ADDRESS:	Asahi-1, Toyosato-machi, Tsukuba-gun, Ibaraki-ken, 305, Japan
DATE OF BIRTH:	August 3, 1952
PERMANENT ADDRESS:	Hiroshima, Japan
EDUCATION:	Bachlor of Engineering, University of Tokyo, 1975 Certificat D'Etudes Supérieures (Circulation et Transports), Ecole Nationale des Ponts et Chaussées, 1983
SPECIALITY:	Civil Engineering

MAJOR AREAS OF EXPERIENCE:

1977	Kanto Regional Construction Bureau, Ministry of Construction
1978-1979	General Affairs Division, Road Bureau, Ministry of Construction
1980-1981	National Expressway Division, Road Bureau, Ministry of Construction

MAJOR SUBJECT: Bridge Engineering, Economy of Transport

TRAVEL ABROAD: France, U.K., W.Germany, Italy etc.

MAJOR PUBLICATION:

1) Economic Effects of Expressway, 1981
2) La Comparaison des Systemes Fraçais et Japonais de Financement et de Péage Autoroutiers, 1983

PERSONAL CAREER

NAME: Shun-etsu ODAGIRI

POSITION: Senior Engineer
Planning Division
Planning and Administration Department
Public Works Research Institute
Ministry of Construction

ADDRESS: Asahi-1, Toyosato-machi, Tsukuba-gun, Ibaraki-Ken,
305 JAPAN

DATE OF BIRTH: August 6, 1950

PERMANENT ADDRESS: Aomori Prefecture

EDUCATION: Bachelor of Electrical Engineering
Nagoya Institute of Technology

SPECIALITY: Electrical Engineering

MAJOR AREAS OF EXPERIENCE:

1975-1983	Engineer, Kanto Regional Construction Bureau
1983-	Senior Engineer, Planning Division, Public Works Research Institute

MAJOR SUBJECTS:

Communication Engineering
- Micro-wave Radio Communication-

PERSONAL CAREER

NAME : Misato MURAKAMI

POSITION: Director
Structure and Bridge Department
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi 1-banchi, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305, JAPAN
Tel. 0298-64-2211, Ext. 209

DATE OF BIRTH : June 10, 1933

PERMANENT ADDRESS : Yamaguchi prefecture, JAPAN

EDUCATION : Bachelor of Civil Engineering, Yamaguchi University
Doctor of Engineering, Nagoya University

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1958-1961	Kinki Regional Construction Bureau, M.O.C.
1961-1963	Road Bureau, M.O.C.
1963-1967	Kinki Regional Construction Bureau, M.O.C.
1967-1973	Japan Highway Public Cooperation
1973-1977	Shikoku Regional Construction Bureau, M.O.C.
1977-1982	Chugoku Regional Construction Bureau, M.O.C.
1982-1983	Director Earthquake Disaster Prevention Department PWRI
1983-	Director Structure and Bridge Department PWRI

MAJOR SUBJECTS AND SUMMARY : Bridge Engineering

TRAVEL ABROAD : West Germany, Malaysia

MAJOR PUBLICATIONS :

"Design and Stress Analysis of the Suspension Bridge KANMON," 1960-1961

"A Study of Erection Method of Cable for Suspension Bridges," 1977

PERSONAL CAREER

NAME : Kazuya OHSHIMA

POSITION : Head
Foundation Engineering Division
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi-1, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305 JAPAN
Tel. 0298-64-2211

DATE OF BIRTH : August 1, 1945

PERMANENT ADDRESS : Kyoto Prefecture

EDUCATION : Bachelor of Engineering, Kyoto University

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1974-1977 Research Engineer, Foundation Engineering Division, PWRI
1977-1982 Osaka Construction Department
Hanshin Expressway Public Cooperation
1982-1983 Design Division
Hanshin Expressway Public Cooperation
1983- Head, Foundation Engineering Division, PWRI

MAJOR SUBJECTS AND ITS SUMMARY :

Foundation Engineering
Bridge Engineering (Substructure)

MAJOR PUBLICATION :

"A STUDY of Plugging Effect of Pile End" 1976
"Design and Construction of YAMATO GAWA Bridge" 1982

PERSONAL CAREER

NAME : Koji KAMINAGA

POSITION : Research Engineer
Foundation Engineering Division
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi-1, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305 JAPAN
Tel. 0298-64-2211

DATE OF BIRTH : April 30, 1954

PERMANENT ADDRESS : Miyagi Prefecture

EDUCATION : Bachelor of Engineering, Tohoku University

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1977-1979 Road Construction Division
Mie Prefecture Government
1979-1981 Hamamatsu Construction Office
Ministry of Construction
1981- Research Engineer, Foundation Engineering Division, PWRI

MAJOR SUBJECTS AND ITS SUMMARY :

Foundation Engineering
Bridge Engineering

MAJOR PUBLICATION :

"Foundation Works in the Vicinity of Existing Structures" 1983

PERSONAL CAREER

NAME : Kunio YAMAMOTO

POSITION : Head, Structure Division
Structure and Bridge Department
Public Works Research Institute
Ministry of Construction

ADDRESS : 1 Asahi, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305 JAPAN
Tel.ᵖ0298-64-2211

DATE OF BIRTH : October 15, 1942

PERMANENT ADDRESS: Aichi Prefecture

EDUCATION : Master of Civil Engineering
Nagoya University

SPECIALITY : Structural Engineering

MAJOR AREAS OF EXPERIENCE :

1967-1971	Kinki Regional Construction Bureau, M.O.C.
1971-1973	Toll Road Division, Road Bureau, M.O.C.
1973-1976	Chubu Regional Construction Bureau, M.O.C.
1976-1978	Chief, Road Construction Section Development and Construction Division Okinawa General Bureau, Prime Minister's Office
1978-1980	Chief, First Planning Section, Road Division Kinki Regional Construction Bureau, M.O.C.
1980-	Head, Structure Division Structure and Bridge Department, PWRI

MAJOR SUBJECTS AND ITS SUMMARY :

Wind resistant design of bridges

MAJOR FIELD FOR RESEARCH WORK :

On the planning of the ring road around the Ise Bay, 1975

PERSONAL CAREER

NAME: Hiroshi SATO

POSITION: Research Engineer, Structure Division,
Structure and Bridge Department,
Public Works Research Institute,
Ministry of Construction

ADDRESS: Asahi-1, Toyosato-machi, Tsukuba-gun,
Ibaraki-ken, 305, Japan

DATE OF BIRTH: December 6, 1953

PERMANENT ADDRESS: Yokohama City

EDUCATION: Bachlor of Engineering, University of Tokyo, 1976
Master of Engineering Science, University of Western Ontario, Canada, 1983

SPECIALITY: Civil Engineering

MAJOR AREAS OF EXPERIENCE:
1976- Research on Wind Resistant Design of Bridges

MAJOR SUBJECT: Wind Engineering

TRAVEL ABROAD: Canada

MAJOR PUBLICATION:
On the Aerodynamic Forces on a Rectangular Prism in Smooth and Turbulent Flow, Including Motion-Induced Effects, 1983

PERSONAL CAREER

NAME : Shoichi SAEKI

POSITION : Head, Bridge Division
Structure and Bridge Department
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi-1, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305 JAPAN
Tel. 0298-64-2211

DATE OF BIRTH : July 21, 1938

PERMANENT ADDRESS : Hyogo Prefecture

EDUCATION : Bachelor of Civil Engineering
Nagoya Institute of Technology

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1962-1966	Research Engineer, Birdge Division, PWRI
1967-1973	Office for Kanmon Bridge, Japan Highway Public Corporation
1974-1975	Senior Research Engineer, Bridge Division PWRI
1975-	Head, Bridge Division, PWRI

MAJOR SUBJECT AND ITS SUMMARY :

Research on superstructure of Bridge

TRAVEL ABROAD : Indonesia, Thailand, Philippines, U.S.A. and China

MAJOR PUBLICATIONS :

"Handbook for the Bridge Design and Erection,"(Joint work), Society for Constructin Works, 1978
"Erection of Long-Span Bridges,"(Joint work), Sankaido Publication, 1979
"Analysis of Cable-Stayed Bridge," Technical Memorandum of the PWRI, 1978
"Resistance of Concrete Beams subjected to Shear," Technical Memorandum of the PWRI, 1978
"Reliability Analysis of Steel Highway Bridges," Technical Memorandum of the PWRI, 1977
"Ultimate Resistance of Steel Piles Subjected to Bending," Annual Meeting of Japan Society for Civil Engineers, 1975
"Study on Hybrid Girders," 10-th Congress of IABSE, 1977
"Design of Non-Composite Steel Girders Based on Load Factor Design Method," Technical Memorandum of the PWRI, 1976
"Erection of Suspension Bridges," Annual Meeting of the Japan Society for Civil Engineers, 1968

PERSONAL CAREER

NAME: Toshitaka Miyata

POSITION: Bridge Division
Structure and Department
Public Works Research Institute
Ministry of Construction

ADDRESS: Asahi-1, Toyosato-machi
Tsukuba-gun, Ibaragi-ken
305 JAPAN
Tel. 0298-64-2211

DATE OF BIRTH: OCT.27,1949

PERMAMENT ADDRESS: Hiroshima-ken,Japan

EDUCATION: Bachelor of Engneering in Civil Engneering
M.S.in Civil Engineering Kyoto University

SPECIALITY: Civil Engineering

MAJOR AREA OF EXPERIENCE.

1975-1977	Kinki Regional Construction Bureau
1977-1979	Land Planning Agency
1979-1981	Road Bureau
1981-	P.W.R.I

MAJOR SUBJECTS AND SUMMARY:
Structure Engineering
Traffic Engineering

TRAVEL ABROAD: U.S.A,Thailand

PERSONAL CAREER

NAME : Kazuo SATO

POSITION : Director
Earthquake Disaster Prevention
Department
Public Works Research Institute
Ministry of Construction

ADDRESS : Asahi 1-banchi, Toyosato-machi
Tsukuba-gun, Ibaraki-ken
305, JAPAN
Tel. 0298-64-2211, Ex. 209

DATE OF BIRTH : April 5, 1932

PERMANENT ADDRESS : Gunma Prefecture

EDUCATION : Bachelor of Engineering in Civil Engineering
Tokyo University, Japan

SPECIALITY : Civil Engineering

MAJOR AREAS OF EXPERIENCE :

1957-1960	Researcher, Traffic Engineering Division, PWRI, Ministry of Construction
1960-1965	Chief Engineer, Tokyo and Ohmiya Highway Construction Offices, Kanto Regional Construction Bureau, M.O.C.
1965-1966	Senior Engineer, Express Highway Investigation Division, Tokyo Branch, Japan Highway Public Corporation
1966-1968	Deputy Head, Hitachi Construction Office, Kanto Regional Construction Bureau, M.O.C.
1968-1972	Deputy, Head, Second Highway Division, Road Bureau, M.O.C.
1972-1977	Head, Hamamatsu and Osaka Highway Construction Offices, M.O.C.
1977-1980	Deputy Director, Investigation Department, Technology Center for National Land Development
1980-1981	Senior Planning Officer for Environment, Kanto Regional Construction Bureau, M.O.C.
1981-1983	Inspector, Minister's Secretariat, M.O.C.
1983-	Director, Earthquake Disaster Prevention Department, PWRI, M.O.C.

MAJOR SUBJECTS : Road Planning
Poor Ground Treatment
Landslide Treatment
Earthquake Disaster Prevention

TRAVEL ABROAD : England, France, Portugal, Philippines

MAJOR PUBLICATION : "Investigation on Countermeasures of Earthquake Disaster at Tokai District," March, 1977

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1965-1968	Engineer, CHUO Express Highway Construction Bureau, Japan Highway Public Corporation (J.H.P.C.)
1969-1973	Engineer, Long Spanned Bridges Section of Headquarters, J.H.P.C.
1973-1976	Researcher, Foundation Engineering Division of Laboratory, J.H.P.C.
1976-1979	Chief of Bridges, Numata Express Highway Construction Office, J.H.P.C.
1979-1981	Deputy Head, Tokyo Bay Crossing Highway Research Office, J.H.P.C.
1981-	Head, Ground Vibration Division, Earthquake Disaster Prevention Department, PWRI, Ministry of Construction

MAJOR SUBJECTS AND SUMMARY :

Survey and Design of Bridges
Earthquake Engineering for Bridges

TRAVEL ABROAD : France, Germany, Austria

MAJOR PUBLICATIONS:

1) Vibrational Characteristics of A Huge Foundation, June 1974, Civil Engineering, June 1974 (in Japanese)
2) Field Vibration Test of Multi Column Foundation, 30th Annual Meeting of Japan Society of Civil Engineering, October 1975 (in Japanese)
3) Planning and Design of High Rise Bridges with Consideration of Seismic Effect, 11th IABSE, September 1980

PERSONAL CAREER

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MAJOR SUBJECTS : Earthquake Engineering

TRAVEL ABROAD : U.S.A., Peru, Mexico, Argentina, Venezuela, Burma

MAJOR PUBLICATIONS :

1) "Soil-Structure Interaction of A Highway Bridge with Use of Recorded Strong-Motion Accelerations", 7th WCEE, 1980
2) "Correlative Investigations on Theoretical and Experimental Behavior of A Curved Model Bridge Structure", Report No.EERC 76-26, Earthquake Engineering Research Center, Univ. of California, Berkeley, 1976
3) "Preliminary Analysis of Finite Ground Strains induced during Earthquakes and Effects of Spatial Ground Motions on Structural Response", 4th U.S. National Congress on P.V.P.T., ASME, 1983
4) "Attenuation of Peak Ground Motion and Absolute Acceleration Response Spectra", 8th WCEE, 1984
5) "Dense Instrument Array Program of The Public Works Research Institute and Preliminary Analysis of the Records, 8th WCEE, 1984
6) "Procedure of Instrument Correction and Displacement Calculation for SMAC-B2 Accelerograph Records with Considering Accuracy of Digitization", Proc. of Japan Society of Civil Engineers, 1982
7) "Effects of Damping Ratio on Earthquake Response Spectra", Proc. of JSCE, 1983

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Earthquake Engineering

TRAVEL ABROAD : None

MAJOR PUBLICATIONS :

1) Analysis of Finite Ground Strains Induced During Earthquakes, Proc. of 17th Earthquake Engineering Conference, Japan Society of Civil Engineers, July 1983 (in Japanese)

2) Experimental Analysis of Recording Accuracy of Digital Strong-Motion Accelerograph, Technical Memorandum No.2019, Public Works Research Institute, August 1983 (in Japanese)

PERSONAL CAREER

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Doctor of Engineering, Kyushu University (1981)

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MAJOR AREAS OF EXPERIENCE :

1961-1973	Public Works Research Institute, M.O.C.
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1975-1981	Head, Ground Vibration Division, Earthquake Disaster Prevention Department, PWRI, M.O.C.
1981-	Head, Earthquake Engineering Division, do.

MAJOR SUBJECTS AND SUMMARY :

Earthquake Engineering
Soil Dynamics

TRAVEL ABROAD : U.S.A., India, Peru

MAJOR PUBLICATIONS :

1) "A History of Soil Liquefaction in Japan," 6 WCEE, January 1977
2) "Seismic Analysis of A Highway Bridge Using Strong-Motion Acceleration Records," International Conference on Microzonation, November 1978
3) "A Summary of Experimental and Analytical Seismic Research Performed on Bridges in Japan," Bridge Workshop, January 1979
4) "Ground Failures and Damages to Soil Structures from the Miyagi-ken-oki, Japan Earthquake of June 12, 1978," 2nd U.S. National Conference on Earthquake Engineering August 1979
5) "Progress Report of Research Works in Bridge Earthquake Engineering at the Public Works Research Institute, Japan," 7 WCEE, September 1980
6) "Free Field and Design Motions during Earthquakes," International Conference on Recent Advances in Geotechnical Earthquake Engineering, St. Louis, Missouri, U.S.A., 1981
7) "Soil Liquefaction Potential Evaluation with Use of the Simplified Procedure," do.

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1977- Research Engineer, Public Works Research Institute
Ministry of Construction

MAJOR SUBJECT AND SUMMARY :

Earthquake Engineering

TRAVEL ABROAD : U.S.A.

MAJOR PUBLICATIONS :

1) "Dynamic Behavior of a Reinforced Concrete Bridge Pier Subjected to Cyclic Loadings", The Sixth Japan Earthquake Engineering Symposium, December, 1982
2) "Study on Earthquake Resistance of Bridges Supported by Caisson Foundations", The Sixth Japan Earthquake Engineering Symposium, December, 1982
3) "Reversed Cyclic Loading Test of Bridge Pier Models", Technical Memorandum of PWRI No.1801, October, 1981
4) "Vibration Tests of Bridge Models Using a Large Shaking Table", The 13th Joint Meeting of U.S.-Japan Panel on Wind and Seismic Effects, May, 1981
5) "Experimental Studies on Seismic Behavior of Structural Members Using a Dynamic Structural Testing Facility at PWRI", The 14th Joint Meeting of U.S.-Japan Panel on Wind and Seismic Effects, May, 1982
6) "Dynamic Response Analysis of Shizunai Bridge Damaged by the Urakawa-oki Earthquake of March 21, 1982", The 15th Joint Meeting of U.S.-Japan Panel on Wind and seismic Effects,May, 1983

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Master of Engineering in Civil Engineering, Tokyo University (1983)

SPECIALITY : Civil Engineering

MAJOR AREA OF EXPERIENCE :
1983- Research Engineer, Public Works Research Institute
Ministry of Construction

MAJOR SUBJECT AND SUMMARY :
Earthquake Engineering

PERSONAL CAREER

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1964-1977	Public Works Research Institute, MOC
1977-1981	Takizawa-Dam Construction Office Water Resource Development Cooperation
1981-	Head of Concrete Division Public Works Research Institute, MOC

MAJOR SUBJECT AND SUMARRY :
Concrete Structure
Concrete Material

TRAVEL ABROAD : U.S.A.

MAJOR PUBRICATIONS :

1) "The Shear Strength of Reinforced Concrete Beam-Column Joints", Thesis of U.T., December 1978
2) "Study on the Steel Mixed Reinforced Concrete Structure", Research Report of PWRI, No. 1677
3) "Introduction to an Earthquake Evaluation Test for Effects to Retrofit of Reinforced Concrete Bridge Pier Elements", 13th Meeting of UJNR
4) "Effect of Resin Injection for Repairing Different Types of Ruptures in RC Beams", Review of 37th General Meeting of CAJ.

PERSONAL HISTORY

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1960-1962	Wakato Bridge Project (JHPC)
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1974-1976	Dupty of Structual Engineering Section of Engineering Dept. (JHPC)
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1979-1981	Chief of Himeji Construction Office (JHPC)
1981-1983	Senior Staff Engineer, Management and Economic Research Office (JHPC)
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MAJOR SUBJECT AND SUMMARY : Structual Engineering

TRAVEL ABROAD : ----------

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TRAVEL ABROAD : ----------

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1968-1976	Osaka Construction Bureau, JHPC
1976-1979	Colombo Plan Expert, Public Works Department, Malaysia
1980-1983	Okinawa Construction Office, JHPC
1983-	Tokyo Bay Crossing Bridge & Tunnel Research office, Planning and Research Department, JHPC

TRAVEL ABROAD : Malaysia

PERSONAL HISTORY

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1974-1977	Nagasaki Construction Office, N.D.K.
1977-1980	Structural Engineering Section, Nagoya Construction Section, N.D.K.
1980-1982	Chief of Construction Section, Meiko-Nishi Bridge Construction Office, N.D.K.
1982-	Present post

MAJOR SUBJECTS AND SUMMARY:
Foundation Engineering
Earthquake Engineering

TRAVEL ABROAD: Philippine, Hongkong, Thailand, Korea

MAJOR PUBLICATIONS:

1) "Pore pressure build-up inalluvium due to pile driving"
Asian Conference of soil mechanics and foundation engineering,
India, 1974

2) "Design of cast-in-place concrete pile"
Kensetsutosho Co. Ltd., Japan, 1974

3) "Construction of concrete structure"
The foundation engineering, Japan
September 1978

4) "Viaducts of Kinki Expressway"
The bridge engineering, Japan
March 1979

PERSONAL CAREER

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1976-1979	Structure Section of Sendai Construction Bureau Japan Highway Public Corporation
1979-1982	Concrete Section of the Laboratory Japan Highway Public Corporation
1982-	Structure Section of Tokyo Second Construction Bureau Japan Highway Public Corporation

MAJOR SUBJECTS AND SUMMARY:
Optimum Design of Structures
Structural Dynamics

TRAVEL ABROAD: U.S.A.

MAJOR PUBLICATIONS:
1) "Optimum Design with Several Design Variables", JSCE, 1976
2) "Fatigue Test and Analysis of PC Connecting Girder", JHPC, 1980

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MAJOR SUBJECTS AND SUMMARY :
Bridge Engineering

MAJOR PUBLICATIONS :
1) "Design of 4-span Continuous Prestressed Concrete Bridge", MEPC Report, March 1981
2) "Measurement of Thermal Stress of Kahei-ramp", MEPC Report, March 1983

PERSONAL CAREER

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1978-80 Field Office, Third Construction Office, M.E.P.C.
1980-83 Engineering Administration Section, Engineering Department, M.E.P.C.
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MAJOR SUBJECTS AND SUMMARY :

Bridge Engineering
Structure Design

TRAVEL ABROAD : Europe, USA, Canada, Mexico

MAJOR PUBLICATIONS :

1) "An Application of Cylindrical Shell Theory to Civil Engineering Structures", JSCE Annual Meeting, 1970
2) "Modern Bridge Design Manual", Japan Society of Civil Engineers, 1982
3) "Automatic Road Design with a Computer", Japan Road Congress, 1971
4) "Aethetics of Road Bridges (1)-(3)", Japan Road Society, 1978,1979, 1980
5) "Some Long Term Obsrevation Results of Light-weight Concrete Structures in Japan", RILEM-ACI Symposium, Hungary, 1984

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1981 - 1983	Advisory Officer, Engineering Division
1983 -	Chief of Design Section, Engineering Division

TRAVEL ABROAD : Europe (West Germany etc.),
People's Republic of China

MAJOR PUBLICATIONS:

1) "On Earthquake Response and Earthquake Resistant Design of a Long span cantilever truss bridge" TRANSACTIONS OF JSCE, Vol. 5, 1973

2) "Structural Chracteristics and Aerdynamic Stability of Long span cable stayed bridges.
The Bridge and Foundation Engineering Vol. 9
No.7, 8, 10, 1975 (in Japanese)

3) "Design of Yamatogawa bridge (Cable stayed Bridge)".
The Bridge and Foundation Engineering Vol. 13
No.7, 1978 (in Japanese)

4) Noulinear Behavior of Pile foundations during strong Earthquake motion"
The Bridge and Foundation Engineering Vol. 17, No.5 1983
(in Japanese)

5) "Earthquake Resistant Design of Ajigawa Bridge (Cable stayed Bridge)
Hanshin Expressway Pub. Corp. Technical report No.2 1982
(in Japanese)

6) "Evaluation of Structural safety for design of Foundations".
"The Bridge and Foundation Engineering Vol. 17 No.5, 1983
(in Japanese)

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1959 - 1969	Ministry of Construction
1969 - 1970	Japan Highway Public Corporation
1970 -	Honshu-Shikoku Bridge Authority

Major Subjects and Summary :

Design and Construction of Bridge

Travel Abroad : U.K, France, W. Germany

Major Publications :

1) Undersea Blasting for Japan Mainland-Shikoku Bridge Route-By Electromagnetic Firing Method-

2) Practicial Study of Under water Drilling and Brasting (Ultra sonic firing system)

PERSONAL CAREER

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Major Areas of Experience :

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Major Subjects:	Design of Foundations
Travel Abroad :	U.S.A., Canada, Australia

Major Publications :

1)"Investigation of Bearing Capacity of Foundation Ground of Honshu-Shikoku Bridge" 9th Internationa Conference on Soil Mechanics and Foundation Engineering

2)"In-situ Dynamic Deformation Properties of Soft Rock Masses" 5th International Society for Rock Mechanics

PERSONAL CAREER

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Major Subjects and Summary : Earthquake Engineering

Travel Abroad :

Major Publications:

1) Three dimensional oscillation analysis of truss girders by the thin walled elastic beam theory, vol. 9 Transactions of TSCE, 1977.

2) Vibration analysis of bridges considering the characteristics of foundation-ground soil system, No. 25 symposium of structural engineer in Japan.

JAPAN PAPERS

J-1 TO J-20

J-1

Outline of the Specifications for Earthquake-Resistant Design of Highway Bridges in Japan

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ABSTRACT

This paper will briefly introduce the Specifications for Earthquake-Resistant Design of Highway Bridges issued by the Japan Road Association in March, 1980. The Specifications are one part of the Specifications of Highway Bridges, which consist of five part; Part I General Specifications (1980), Part II Steel Bridges (1980), Part III Concrete Bridges (1978), Part IV Substructures (1980), and Part V Earthquake Resistant Design (1980).

INTRODUCTION

The Specifications of Highway Bridges apply to the design of highway bridges with span lengths not longer than 200 meters. The Specifications for Earthquake-Resistant Design basically stipulate to employ the seismic coefficient methods and provide two methods in determining design seismic coefficients. One is the conventional seismic coefficient method that applies to the design of relatively rigid structures. The other is the modified seismic coefficient method considering structural response that applies to the design of relatively flexible structures.

CLASSIFICATION OF GROUND CONDITIONS

The ground conditions will be clarified into four groups according to Table 1, in which characteristic value of ground (T_G in s) can be calculated by the following equation.

$$T_G = 4\sum_i \frac{Hi}{Vsi} \quad (s) \qquad (1)$$

where Hi : thickness of i-th subsoil layer (m),
Vsi : shear wave velocity of i-th subsoil layer at low strain (m/s).

The baserock for calculation of Eq.(1) is stipulated to take on the soil layer that has a shear wave velocity at low strain equal to 300 m/s or higher.

Table 1 Classification of Ground Conditions and Value of ν_2

Group	Characteristic Value T_G (second)	Value of ν_2
1	$T_G < 0.2$	0.9
2	$0.2 \leqq T_G < 0.4$	1.0
3	$0.4 \leqq T_G < 0.6$	1.1
4	$0.6 \leqq T_G$	1.2

LIQUEFACTION OF SANDY SOIL LAYER

For saturated alluvial sandy layers which are judged to be vulnerable to liquefaction, liquefaction potential shall be checked with use of liquefaction resistance factor (F_L). For those soil layers which are judged to liquefy, bearing capacities, subgrade reaction constants and other soil constants shall be either neglected or reduced in the seismic design.

SEISMIC COEFFICIENT METHOD

In the seismic coefficient method, which applies to relatively rigid structures, the horizontal design seismic coefficient (k_h) shall be determined by

$$k_h = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_0 \qquad (2)$$

where k_0 : standard horizontal design seismic coefficient (=0.2),
ν_1 : seismic zone factor,
ν_2 : ground condition factor,
ν_3 : important factor.

The values of ν_1, ν_2 and ν_3 are shown in Tables 2, 1 and 3, respectively.

Table 2 Seismic Zone Factor ν_1 for Highway Bridges

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

Table 3 Importance Factor ν_3 for General Highway Bridges

Group	Definitions	Value of ν_3
1	Bridges on expressway (limited-access highways), general national highways and principal prefectural highways. Important Bridges on general prefectural highways and municipal highways.	1.0
2	Other than the above	0.8

Note: The value of ν_3 may be increased up to 1.10 for special cases in Group 1.

MODIFIED SEISMIC COEFFICIENT METHOD

The modified seismic coefficient method shall apply to bridges with piers higher than 15 meters above the ground surface. In the modified seismic coefficient method, the horizontal design seismic coefficient (k_{hm}) shall be determined by

$$k_{hm} = \beta \cdot k_h \qquad (3)$$

where β is the magnification factor shown in Fig.1, and k_h is given by Eq.(2). It is specified to estimate the natural period for the individual system consisting of each substructure and the part of superstructures supported by it by the following equation.

$$T = 2.01\sqrt{\delta} \qquad (4)$$

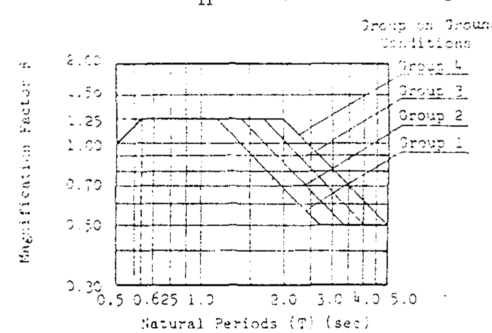

Fig. 1 Magnification Factor for the Modified Seismic Coefficient Method

where T : fundamental natural period (s) of the system,
δ : maximum horizontal displacement (meter) of the pier when subjected to the dead weight of the section of superstructure supported by the substructure and also to 80 percent of the dead weight of the substructure above the ground surface.

SEISMIC MOTIONS IN DYNAMIC ANALYSIS

An article is introduced concerning seismic motions to be utilized in dynamic response analyses. Dynamic response analyses are necessitated for those bridges which are significantly different from normal bridges, and those which are constructed on extremely soft soils, in order to precisely investigate the earthquake resistivity of bridges in terms of displacements, ductilities, etc.

SEISMIC COEFFICIENT IN DUCTILITY ANALYSIS

In order to avoid brittle failure during earthquakes, it is extremely important for reinforced concrete structures to have adequate ductility. It is stipulated that the seismic coefficient (k_{hd}) in ductility analysis shall be determined according to the following equation.

$$k_{hd} = \begin{cases} \nu_4 \cdot k_h & \text{(in the seismic coefficient method)} \\ \nu_4 \cdot k_{hm} & \text{(in the modified seismic coefficient method)} \end{cases} \quad (5)$$

where ν_4: structural characteristics factor (1.3 or greater).

DESIGN DETAILS

A special attention is paid to the design of structure details, in consideration of the damage previously experienced to bridge structures. To this aim provisions are specified for bearing supports and devices for preventing bridge girders from falling.

CLOSING REMARKS

It is considered necessary in the near future to improve and amend the current Specifications on the following subjects.

(1) Seismic design force
(2) Estimation of liquefaction potential of soil layers
(3) Design details of bearing supports and connections
(4) Ductilities of reinforced concrete piers
(5) Modified seismic coefficient method considering the effect of soil-structure interactions
(6) Seismic design of continuous girder bridges

J-2

Outline of the Specifications for Substructures of Highway Bridges in Japan

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Preface

Substructures of bridges normally mean the portions below the bearings and can be roughly divided into the substructural body and foundation. The Specifications for Highway Bridges Part IV Substructures prescribes the standards for the design and construction of these structures, and this report will outline these standards.

1. Design of Substructural Body

Design of structural members is basically performed by the allowable stress method based on the theory of elasticity. When considering the influence of earthquakes upon the acting loads, the allowable stresses are increased 1.5 times.

2. Foundation Design Approach

For the design and calculations of the foundation, both the theory of plasticity and the theory of elasticity will be employed. Design related to the ground such as calculations of bearing capacity is performed based on the theory of plasticity and the safety factor is established for this purpose. For the design of the foundation body, calculations are performed on the basis of the theory of elasticity as same as substructural body.

Foundation design system can be roughly divided into the three categories of spread foundation, caisson foundation and pile foundation, and each category is defined for the purpose of design method as shown in Tables 1 and 2. Bearing mechanism and mathematical model of each type of foundation are shown in Fig. 1.

Table 1 Classification of Spread Foundation and Caisson Foundation

Foundation type \ D_f/B	0	$\frac{1}{2}$	1
Spread foundation	←		
Caisson foundation		→	

Where, D_f: Effective depth of embedment (m)
B: Width of short side of foundation (m)

Table 2 Classification of Caisson Foundation and Pipe Foundation

Foundation type		\ $\beta \cdot \ell$	0	1	2	3	4
Caisson foundation			←				
Pile foundation	Short pile (with finite length)			↔	↔		
	Long pile (with semi-infinite length)					→	

Where, ℓ: Effective depth of embedment of caisson or pile (cm)
β: Characteristic value of caisson or pile (cm^{-1})

$$\beta = \sqrt[4]{\frac{kD}{4EI}}$$

EI: Flexural rigidity of caisson or pile (kg/cm^2)
D: Width or diameter of caisson or pile (cm)
k: Coefficient of subgrade reaction in horizontal direction of caisson or pile (kg/cm^3) (average value at the point of $\ell/2$ from ground surface for caisson foundation, and at the point of $1/\beta$ from ground surface for pile foundation)

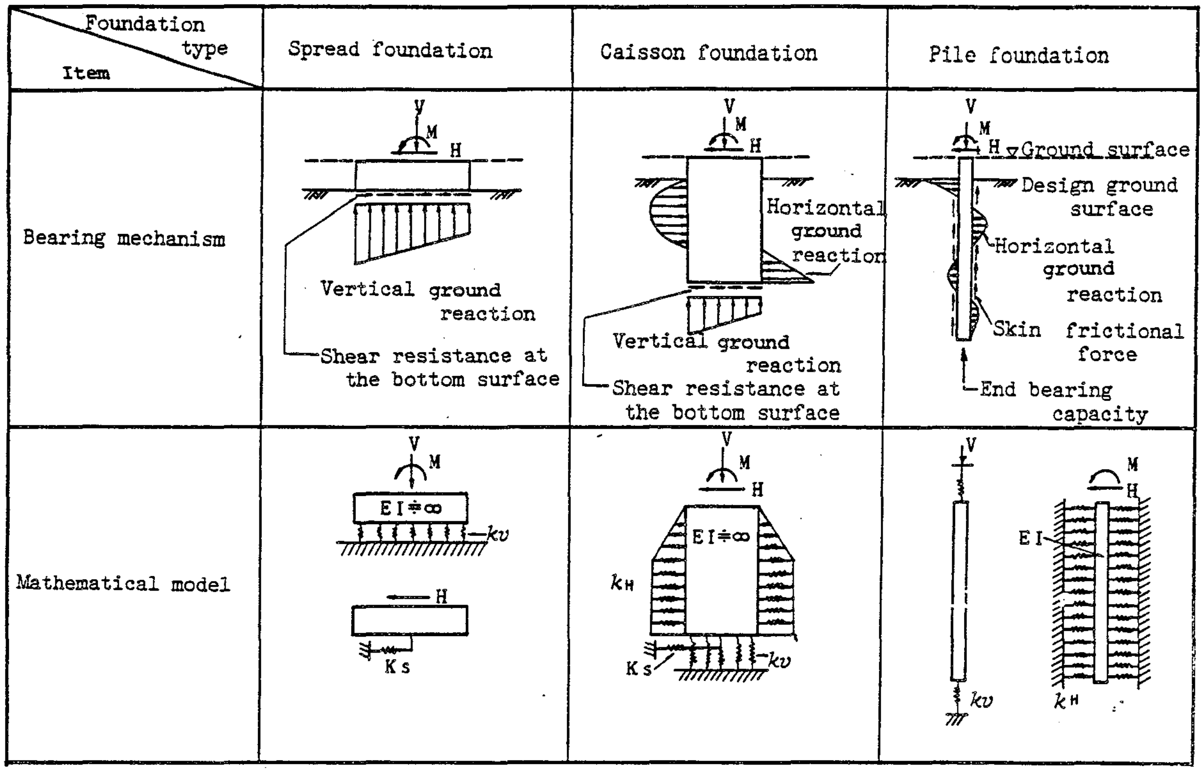

Fig. 1. Bearing Mechanism of Foundation and Its Modelization

3. Design of Spread Foundation

Spread foundation is required to be designed so as to meet the following provisions:

(a) Foundation is to be stable in its vertical and horizontal bearing (sliding) and against overturning. For this purpose, the following requirements must be met:

1) Vertical ground reaction at the bottom face of the spread foundation shall not exceed the allowable vertical bearing capacity of ground at the bottom.
2) Shear resisting force at the bottom face of spread foundation shall not exceed the allowable shearing capacity of ground at the bottom.
3) The position of resultant loads operation acting to the spread foundation shall be within 1/6 of width of bottom face from its center during normal time and within 1/3 during earthquake.

(b) The displacement of foundation shall not exceed the allowable displacement.

(c) Stress of each portion of foundation shall not exceed the allowable stress.

4. Design of Caisson Foundation

In the design of caisson foundation, it should be assumed as a rule that the vertical load is supported only by the vertical ground reaction at the bottom of caisson and that the horizontal load is supported by the horizontal ground reaction on the front surface and by the shear resisting force at the bottom. With respect to the horizontal resistance, the side frictional resistance can be considered by increasing the horizontal coefficient of ground reaction by 20%. Under the assumptions stated above, the caisson foundation will be designed so as to meet the following requirements:

(a) Caisson foundation shall be stable for vertical bearing and against rotating and sliding. For this purpose, the following requirements must be met:

1) Vertical unit reaction of ground at the bottom of caisson shall not exceed the allowable vertical unit bearing capacity of ground.
2) Maximum horizontal unit reaction of ground at the front surface of the caisson shall not exceed the allowable horizontal unit bearing capacity of ground at that position.
3) Shear resisting force at the bottom of caisson shall not exceed the allowable shearing capacity between the bottom face and ground.

(b) Displacement of the caisson foundation shall not exceed the allowable displacement.

(c) Stress of each portion of caisson shall not exceed the allowable stress.

5. Design of Pile Foundation

The following basic assumptions are made in the design of pile foundation:

1) Vertical and horizontal forces acting to the pile foundation are supported only by the piles, and resisting forces by the bottom and front of footing are neglected.
2) Footings are handled as a rigid body and, for this purpose, a thickness greater than a predetermined value should be used.
3) As a rule, piles should be so arranged that each pile will uniformly receive long-term continuous load. The effect of grouped piles will be neglected by maintaining the minimum center-to-center distance of piles greater than 2.5 times the pile diameter.

In the design of pile foundation, the structural specifications should be so determined as to meet the following requirements basing upon the concepts stated above:

1) Reactions (force in axial direction and force perpendicular to the axis) which occur at the pile head due to the load acting to the pile foundation shall not exceed the allowable bearing capacity of ground at pile head.
2) Stress in pile body shall not exceed the allowable stress of pile.
3) Displacement of pile foundation shall not exceed the allowable displacement.

Reaction of each pile and displacement of footing are determined by solving the basic equations with footing displacement as unknown quantity and each pile as spring support.

J-3

SEISMIC DESIGN FORCES OF HIGHWAY BRIDGES IN JAPAN

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INTRODUCTION

Horizontal design seismic coefficient k_H for highway bridges with span length less than 200 m are currently specified in the Part V Seismic Resistant Design Specifications of Highway bridges as

$$k_H = \begin{cases} \nu_{ZONE} \times \nu_{GC} \times \nu_{IMP} \times k_0 & (T < 0.5 \text{ sec}) \quad \text{(1-a)} \\ \nu_{ZONE} \times \nu_{GC} \times \nu_{IMP} \times \beta(T) \times k_0 & (T \geqq 0.5 \text{ sec}) \quad \text{(1-b)} \end{cases}$$

in which ν_{ZONE}, ν_{GC}, and ν_{IMP} represent modification coefficients in accordance with the zone of the site, ground condition and importance of the bridge, respectively, and k_0 and $\beta(T)$ represent a standard seismic coefficient (=0.2) and amplification factor in accordance with natural period of the bridge, respectively. Eq.(1a) represents the design seismic coefficient for seismic coefficient method, and Eq.(1b) represents the design seismic coefficient for modified seismic coefficient method considering structural response, which is to be applied for bridges with natural period longer than 0.5 sec.

Fig.1 shows the design seismic coefficient k_H specified by Eq.(1) assuming that ν_{ZONE} and ν_{IMP} are equal to 1.0 (normal condition). Although seismic coefficient k_H as shown in Fig.1 is widely used for seismic design, the validity of the design seismic coefficient is not very apparent.

PROPOSED SEISMIC COEFFICIENT

According to the original definition of the seismic coefficient method proposed by Sano in 1916, the seismic coefficient k_H is defined as the maximum acceleration developed in the structure divided by an acceleration of gravity G. Assuming that the maximum structural response acceleration with a natural period T and damping ratio h be approximated by an acceleration response spectrum $S_A(T, h)$, the seismic coefficient $\tilde{k}(T)$ may be represented as

$$\tilde{k}(T) = S_A(T, h) / G \quad (2)$$

It should be noted here that an actual damping ratio h of the bridge be used in Eq.(2). It is apparent that the damping ratio h is not generally independent of natural period T, i.e., damping value associated with viscous and structural damping for fundamental mode is most likely predominant in tall-pier bridges, whereas a radiational damping from substructure to surrounding subsoils is much more pronounced than viscous and structural damping in short-pier bridges, especially when the substructures are embedded deeply in ground. Fig.2 shows a relation between damping ratios and natural periods for highway bridges, which were obtained by many field forced vibration tests[1]. For general bridges designated as group A in Fig.2, the damping ratio h may be written as

$$h = 0.02 / T \quad (3)$$

On the other hand, according to the multiple regression analysis of free-field ground motions recorded in Japan, absolute acceleration response spectral amplitudes $S_A(T, h)$ of 5% damping of critical are given in terms of earthquake magnitude M and epicentral distance Δ for three subsoil conditions as

$$S_A(T, 0.05) = a(T, GC) \times 10^{b(T, GC)M} \times (\Delta + 30)^{-1.178} \qquad (4)$$

in which coefficients a(T, GC) and b(T, GC) are given for specific natural period T and subsoil conditions [2]. The response spectral amplitude $S_A(T, h)$ of arbitrary damping ratio h can be approximately estimated from $S_A(T, 0.05)$ as (refer to Fig.3)

$$S_A(T, h) = S_A(T, 0.05) \times \{ \frac{1.5}{40h + 1} + 0.5 \} \qquad (5)$$

Then, substituting Eqs.(3), (4) and (5) into Eq.(2), the seismic coefficient $\tilde{k}(t)$ can be obtained as

$$\tilde{k}(T) = 1/G \cdot \{a \times 10^{bM} \times (\Delta + 30)^{-1.178}\} \times \{\frac{1.5}{0.8/T + 1} + 0.5\} \quad (6)$$

Fig.4 shows the seismic coefficient $\tilde{k}(T)$ defined by Eq.(6) for three combinations of earthquake magnitude M and epicentral distance Δ. Comparing Fig.4 with Fig.1, it is observed that the seismic coefficient $\tilde{k}(T)$ defined by Eq.(6) takes the maximum value at shorter natural period as compared with the design seismic coefficient k(T) specified by Eq.(1). It is also seen that although the design seismic coefficient k(T) is close with the seismic coefficient $\tilde{k}(T)$ for the condition of M=7 and Δ=50 km, variations of seismic coefficient in accordance with natural period and subsoil conditions are appreciably different between k(T) and $\tilde{k}(T)$.

FUTURE INVESTIGATION

It is not apparent that which combinations of earthquake magnitude and epicentral distance can be assured by using the seismic coefficient of 0.2 to 0.3 in Eq.(1). Future investigations are considered necessary from the following two points;

1) The combination of earthquake magnitude M and epicentral distance Δ shall be determined considering the safety level required for design of highway bridges. It is possible to use probabilistic procedure to estimate expected spectral amplitude($\tilde{k}(T)$) for certain return period T_R instead of evaluating the combination of M and Δ. It is considered appropriate to provide two seismic coefficients, i.e., one corresponding to ordinary seismic loading and the other to extremely large seismic loading. The combination of M and Δ, or T_R, is necessarly to be properly selected for the two levels of loading.

2) Eq.(6) consideres only elastic structural response, and unelastic response is considered indispensable, especially when seismic loadings corresponding to extremely large earthquake are introduced in design.

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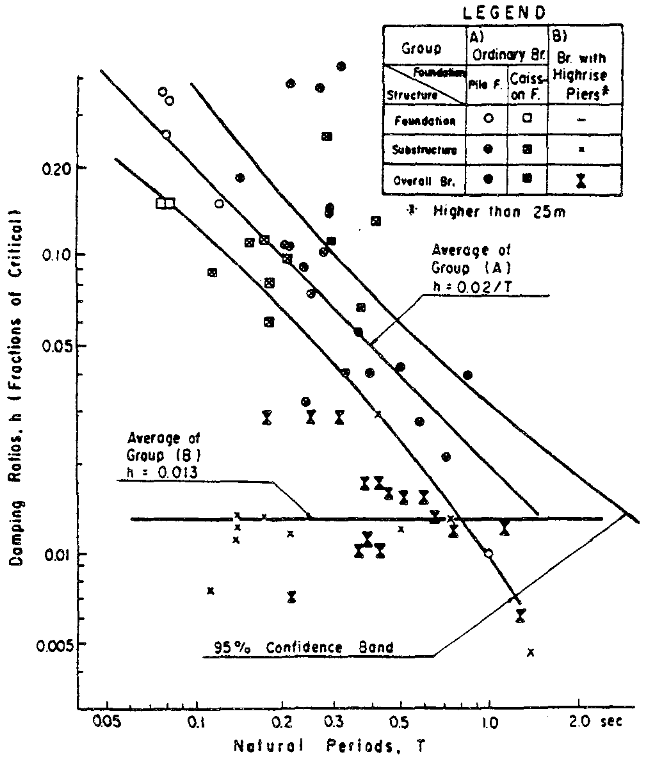


Fig.2 Natural Period vs. Damping Ratio of Highway Bridges by Field Forced Vibration Tests[1]

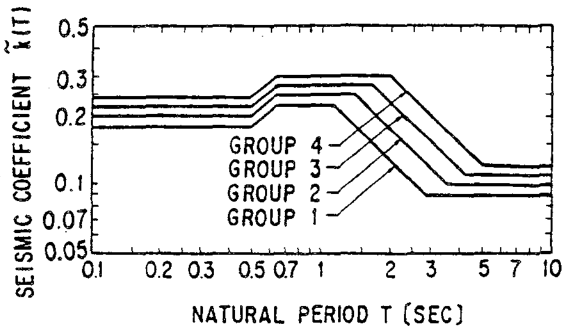


Fig.1 Design Seismic Coefficient of Highway Bridges

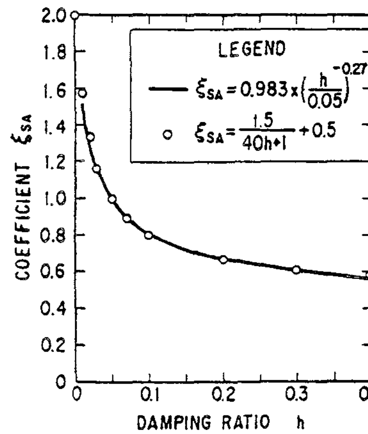
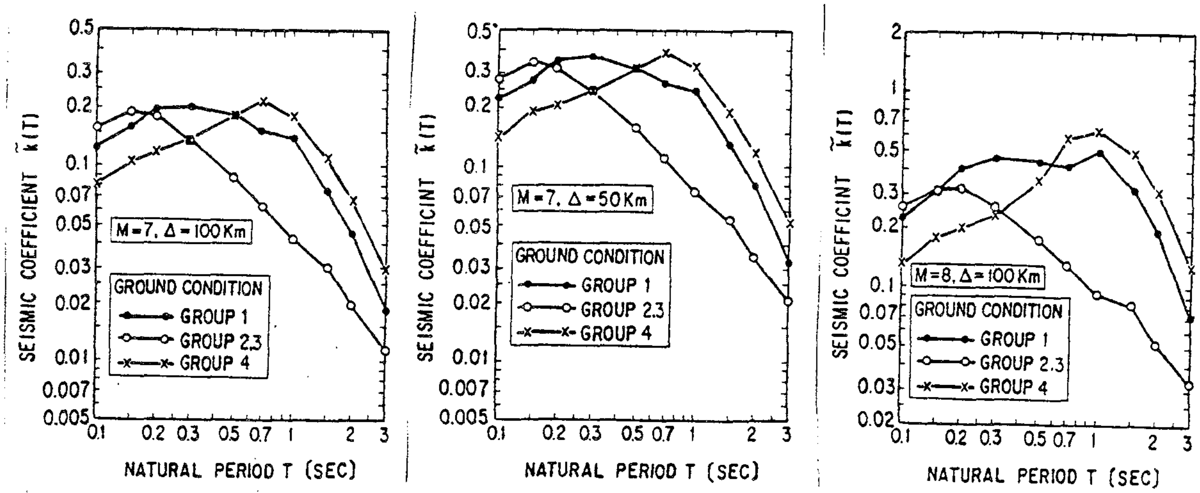


Fig.3 Modification Coefficient of $S_A(T, 0.05)$ with Respect to Damping Ratio h



(a) M=7, Δ=100km (b) M=7, Δ=50km (c) M=8, Δ=100km

Fig.4 Seismic Coefficient defined by Eq.(6) for Specific Combinations of Earthquake Magnitude and Epicentral Distance

J-4

EXPERIMENTAL STUDY OF EMBANKMENT ON SANDY LAYERS

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INTRODUCTION

During the past earthquakes, many earth structures such as road embankments and river dikes have been damaged. Typical examples of damaged embankments at plans were seen in Akita area during The Nihonkai-chubu Earthquake of 1983. Many embankments cracked and settled, in spite of the low height. Most of these damaged embankments were situated on the beds of the old rivers and lowlands between dunes where the soils are very liquefiable. These tendencies are the same as the cases of the Niigata Earthquake of 1964 and the Miyagiken oki Earthquake of 1978. For reducing the effects of earthquakes, it is important to make soil structures to be stable during earthquakes.

In consideration of the matters described above, embankment model experiments using a large shaking table were conducted to make clear the effects of liquefaction of the grounds on embankment failure.

METHODS OF EXPERIMENTS

Embankment models of sands were made on sandy grounds in a large container with length of 6 m, width of 3 m and height of 2 m, which were placed on the shaking table. The steel container has two sides made of transparent glasses and two rotational walls with bottom hinges. Experiments for four sandy grounds with different relative densities were carried out.

The dimensions of the models are shown in Fig. 1. The height of the grounds was 110 cm, the height of the embankments was 60 cm, the top width of the embankment was 100 cm and the slope was 1 : 1.5.

The wet sands were putted into the soil container and the ground and embankment of the above dimensions were made. Water was poured through the bottom of the container up to the level of the ground surface. Ground models were formulated with layers of thickness of 20 cm. To make ground models, three methods of compaction were employed, i.e., stepping methods (compaction with foot), mallet falling methods and rammer methods. Four cases of the experiments are tabulated in Table 1, where compaction methods are also shown. At the formation of embankment, the thickness of sand placement was 20 cm and stepping method was only used.

The physical properties of sands used for experiments are shown in Table 2. The ground of Case 1 was loose and the relative density was 36%. The ground of Case 2 was looser and the relative density was 19%. The ground of Case 3 was dense and the relative density was 62%. The ground of Case 4 was denser and the relative density was 88%. The relative densities of embankments were about 40% for Cases 1, 2 and 3, and 62% for Case 4.

At shaking table tests, sinusoidal motions with the frequency of 5 Hz were inputted. Three levels of table accelerations were set, i.e., 200, 400 and 600 gals. It took 5 $\sim$ 10 seconds from beginning of shaking to the set levels. The set levels of accelerations were kept constant for 50 seconds. When models completely failed at lower levels of accelerations, tests for higher levels are ignored. Accelerations measured during real tests at each case were shown in Table 3.

RESULTS OF EXPERIMENTS

The embankment at Case 1 - ground was loose - and Case 2 - ground was looser - failed at step 1 (objective acceleration : 200 gals). One at Case 3 - ground was dense - was not deformed remarkably at step 1, however it failed at step 2 (objective acceleration : 400 gals). One at Case 4 - ground was very dense - was not deformed remarkably at step 1 and 2, but the surface of the embankment failed at step 3 (objective acceleration : 600 gals). Photos show the outlines of failure mode at the last step of Case 1 and Case 4. Fig.2 shows the relation between the relative density and the maximum acceleration at the occurrence of the failure. It is clear that the acceleration for the failure rises according to the increase of relative density of the sandy layers.

At case 1 and 2, a tension crack occurred in the center of the embankment when the acceleration reached to 150 $\sim$ 200 gals. After that, the top of the embankment began to settle down and the tension crack grew larger. At the end of the tests the crack widhts were 14 cm at Case 1 and 25 cm at Case 2 and the average settlements of the top of the embankment were 20 cm at Case 1 and 23 cm at Case 2. The embankment settled down and the basement of the embankment flew to both sides of the slope. The ground surfaces were pushed up by the movement of the lower part of the embankment.

Before the occurrence of the crack and the settlement, pore water pressures began to rise in the ground surrounded the embankment and then in the ground under the embankment. Sand boils were seen at the ground surface near the side of the soil box. These matters show that the embankments lost their stability when the ground liquefied, settled down and flowed outsides.

At Case 3, several cracks appeared on the top of the embankment at about 15 seconds later at step 2 (real acceleration was 341 gal). However the cracks and settlement did not grow so largely as in Case 1 and 2. The average settlement of the top of the embankment was 6.5 cm. In this case, the basement of the embankment also flew outside due to the liquefaction of the ground as appeared in Case 1 and 2, but the degree was not so high as Case 1 and 2 and the large crack did not appear in the top center of the embankment. Pore water pressure rised but not so remarkably as in the Case 1 and 2.

At Case 4, the surfaces of the embankment slops began to fail at about 15 seconds later at step 3 (real acceleration was 682 gals) and many small cracks appeared at the top of the embankment. The maximum acceleration at the top was over 1,000 gals. However the ground and the inside of the embankment wasn't damaged apparently and pore water pressures didn't rises.

CONCLUSIONS

Failure patterns of the embankments on the sandy layers during earthquakes were ascertained through the experiments of the ground and embankment model by a shaking table. Ascertained points are follows:

1) It is clear that the failures of the embankments on the loose sandy layers are caused by the liquefaction of the ground.
2) Failure patterns and degrees of failure of the embankments on the sandy layers depend remakably on the realtive densities of the layers.
3) In the case of loose ground, the grounds arround the embankment liquefy and flow outsides at first and then the embankments settledown.

REFERENCES

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Table 1 Methods of Compaction

Case No.	Grounds	Embankments
1	5 Times of Compaction with Foot	5 Times of Compaction with Foot
2	1 Time of Compaction with Foot	the same as above
3	5 Times of Compaction with Mallet	the same as above
4	5 Times of Compaction with Rammer	the same as above

Table 2 Physical properties of Sands Used for Experiments

Items	Sign	Unit	Value
Specific Gravity of Soil Particle	Gs	–	2.704
Optimum Moisture Content	w_{opt}	%	15.5
Maximum Dry Density	$\gamma_{d,max}$	gr/cm³	1.668
Maximum Void Ratio	e_{max}	–	0.972
Minimum Void Ratio	e_{min}	–	0.613
60% Grain Size	D_{60}	mm	0.30
10% Grain Size	D_{10}	mm	0.15
Mean Grain Size	D_{50}	mm	0.26
Coefficient of Uniformity	U_c	–	2.0

Table 3 Accelerations of Shaking Table

Case No.	Step No.	Target Acceleration (gal)	Observed Acceleration (gal)	Remarks
1	1	200	201	Damaged
2	1	200	186	Damaged
3	1	200	223	Not Damaged
	2	400	341	Damaged
4	1	200	221	Not Damaged
	2	400	425	Not Damaged
	3	600	682	Damaged

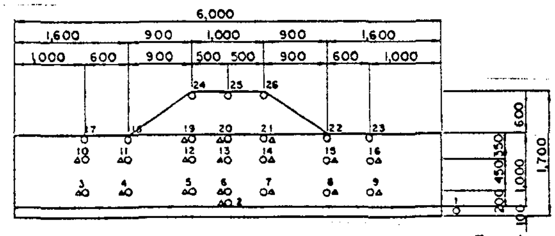

Fig. 1 Ground and Embankment Model

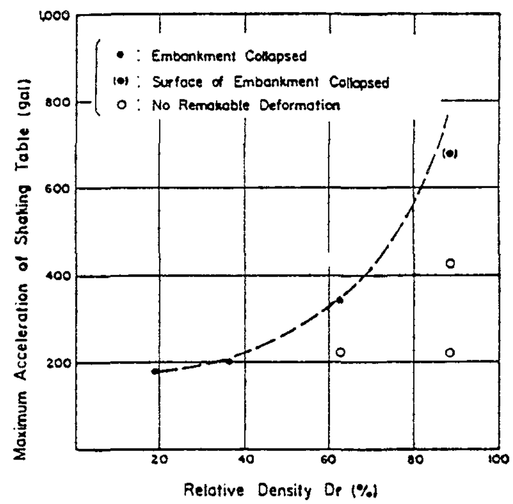

Fig. 2 Relation between Relative Density of Ground and Maximum Acceleration of Shaking Table

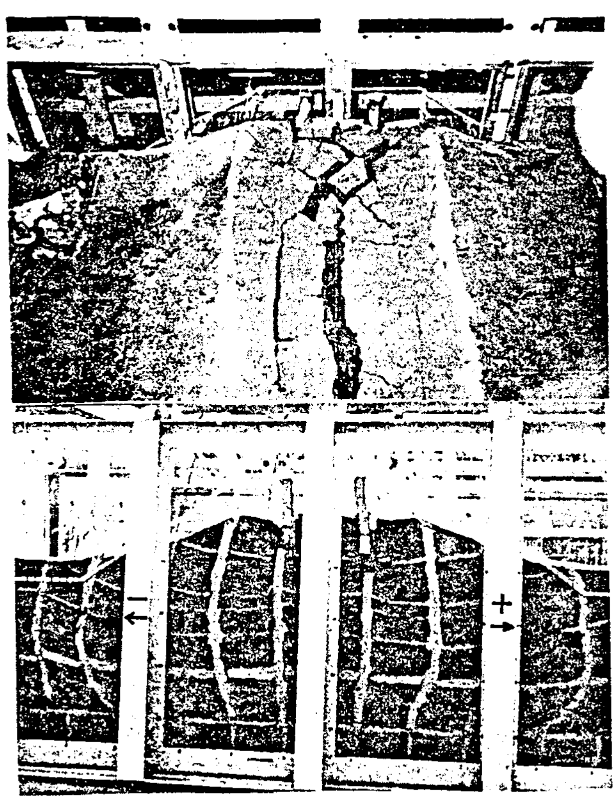

Photo. 1 Failure Mode of Case 1

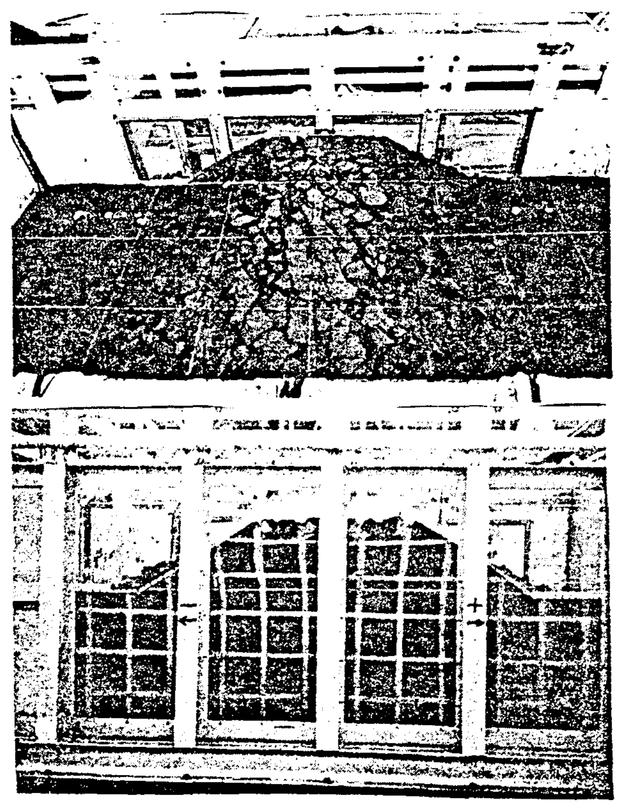

Photo. 2 Failure Mode of Case 4

Dynamic Response Analysis for Seismic Design of Highway Bridges

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GENERAL

Seismic design of highway bridges in Japan has been performed by static analysis using seismic coefficients since 1926. Seismic loads are considered as a static horizontal force expressed by eq.(1) in the design.

$$F_E = K_h \cdot W \qquad (1)$$

where, F_E : design seismic load
K_h : design horizontal seismic coefficient (standard value=0.2)
W : weight

In addition to the static analysis, the latest provision(1) enforced in 1980 requires dynamic response analysis when it is necessary. According to the principles of the provision, designers decide the necessity of dynamic response analysis and select the input seismic motion and analytic model.

PURPOSE OF DYNAMIC RESPONSE ANALYSIS

The purpose of dynamic response analysis is to check the seismic performance of the following bridges after designing by static analysis (seismic coefficient method or modified seismic coefficient method). (Fig.1)

(1) Bridges for which the static analysis is not adequate because of their complicated seismic responses (long-span bridges, high-pier bridges, and complicated bridges with various soil conditions and substructure types or dimensions).
(2) New-type bridges which have not experienced strong seismic excitations and whose seismic performance has not been verified by past earthquakes.

INPUT SEISMIC MOTIONS FOR DYNAMIC RESPONSE ANALYSIS

Input seismic motions for dynamic response analysis are selected from the followings:

(1) average response spectrum (Fig.2, Fig.3, etc.)
(2) recorded strong seismic motion obtained at the construction site.
(3) recorded strong seismic motion obtained at the site of similar ground conditions.
(4) seismic motion at the base rock estimated from the recorded strong seismic motion.

The maximum acceleration of the input seismic motions is usually set to be 100-200 cm/sec^2 at the base rock or foundation level according to the seismic activity at the construction site. Two or Three input seismic motions are used for the dynamic response analysis considering the perdominant frequencies of the bridge, ground, and seismic motion.

DYNAMIC RESPONSE ANALYSIS METHODS

Bridges are usually considered as a linear (elastic) system in the dynamic response analysis. If the seismic excitation is expected to be very strong at the construction site, it is desirable to perform non-linear response analysis.

There are two methods for dynamic response analysis as follows:

(1) response spectrum analysis

(2) time history response analysis

Response spectrum analysis is simpler than time history response analysis. Average response spectrum, in which informations on several seismic motions are included, is available in the response spectrum analysis. For these reasons, response spectrum analysis is preferable in the seismic design. When non-linear response analysis is necessary or the periods of the bridge and ground are not separated, response spectrum analysis is unavailable or unreliable, and it is necessary to perform time history response analysis.

Damping constants (h) for the dynamic response analysis are usually set to be as follows:

(1) superstructure - h = 0.02

(2) substructure - h = 0.02 - 0.10

(3) soil - h = 0.10 - 0.20

FUTURE RESEARCHES

The following items are prior research themes to improve the dynamic response analysis for the seismic design.

(1) Selection of the bridges for which dynamic response analysis is necessary

(2) Establishment of input seismic motions

(3) Estimation of spring constants of the soil-foundation system.

(4) Estimation of damping constants of the structure and soil.

(5) Effect of soil-structure interaction.

(6) Establishment of analytic models to represent the non-linear behavior of structural members and soil (Fig.4).

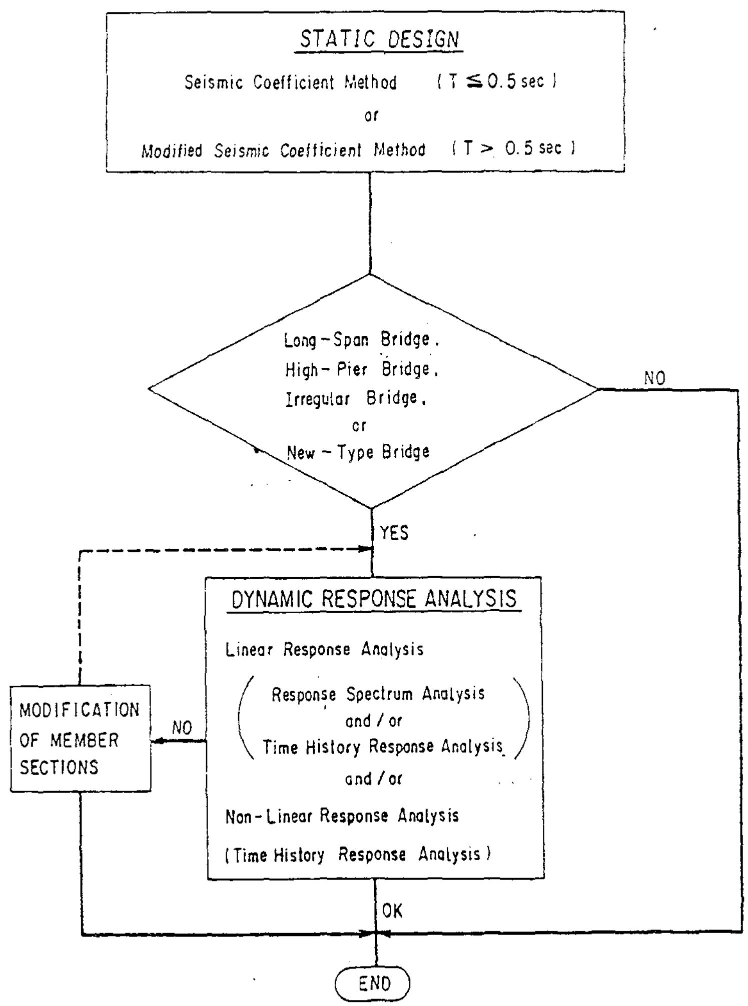

Fig. 1 Seismic Design of Highway Bridges

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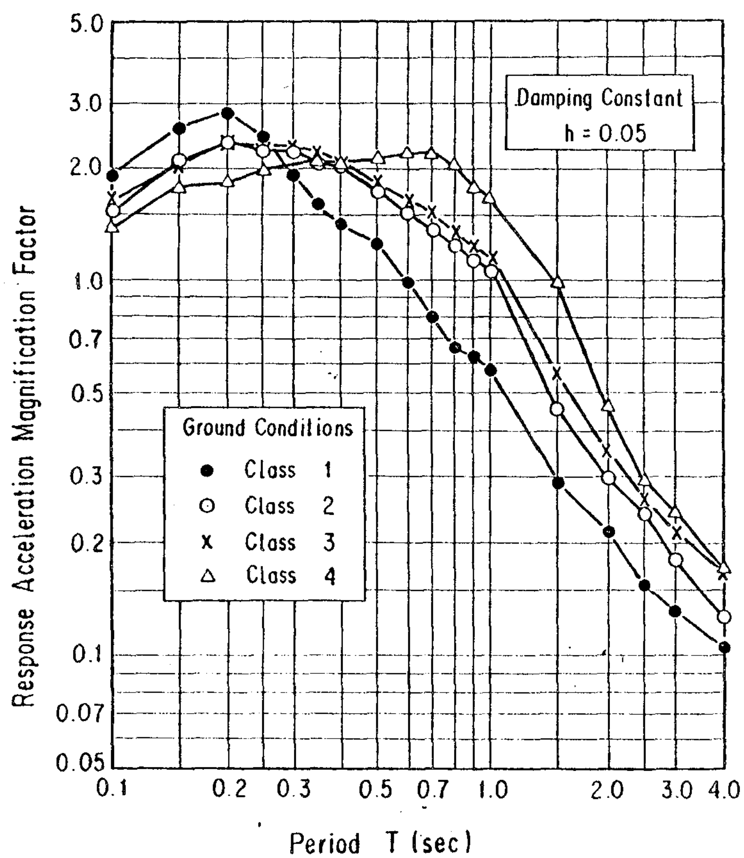

Fig. 2 Average Response Acceleration Magnification Factor Obtained from 277 Strong Seismic Motion Records (h = 0.05)

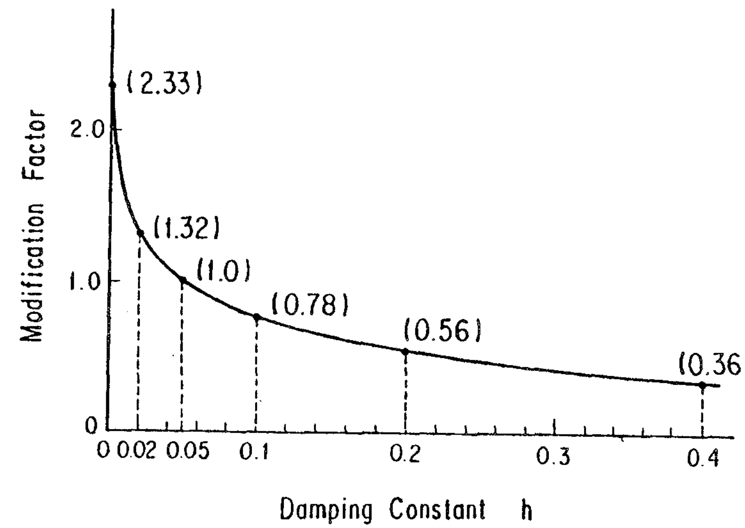

Fig. 3 Response Spectrum Modification Factor with Regard to Damping Constant

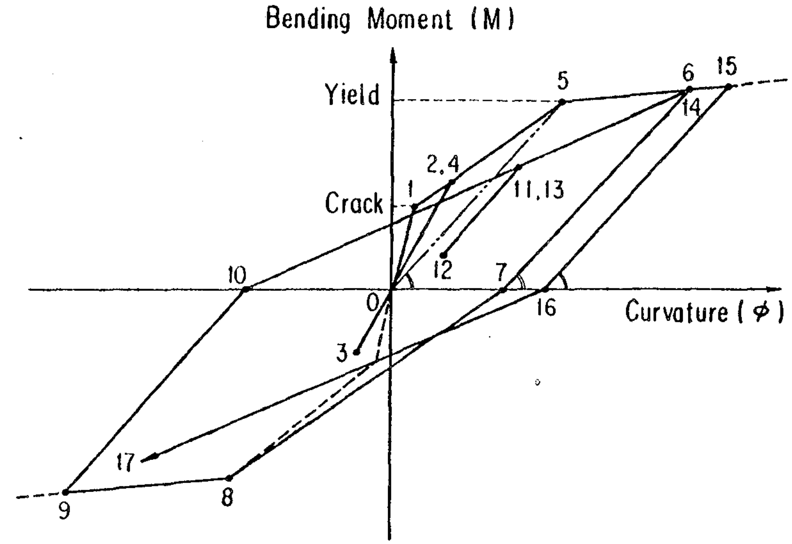

Fig. 4 Degrading Tri-Linear Model for the Bending Moment - Curvature

DUCTILITY DEMAND FOR BRIDGE PIERS

J-6

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1. GENERAL

JRA Specifications for Highway Bridges Part V, Earthquake Design (1980), which is applied to all of the highway bridges with the span length not longer than 200 m, recommends to check the deformability of bridge piers and bridge abutments for avoiding brittle failure caused by earthquakes. The checking method or the conception for deriving the requisite deformability is very unique although it has requirement of further researches.

This paper briefly introduces the checking method and the theoretical background.

2. OUTLINE OF THE CHECKING PROCEDURE

(1) Design with ordinary methods against earthquake force with seismic coefficient K_h

(2) Calculate of ultimate displacement (δ_u) and allowable displacement ($\delta_a = 1/3 \times \delta_u$) of the pier top

(3) Calculate of equivalent natural period (T_{eq}) and equivalent damping factor (h_{eq}) at $\delta = \delta_a$

(4) Search of the seismic coefficient in ductility analysis (K_{hd}) which makes the displacemrnt of the pier top equal to allowable displacement (δ_a) taking the influence of T_{eq} and h_{eq} into account

(5) Check whether $K_{hd} \geqq 1.3 \times K_h$

3. CALCULATION METHODS AND THEORETICAL BACKGROUNDS

The ultimate displacement of the pier top (δ_u) is calculated with eq. (1)

$$\delta = \int_0^H \phi y \cdot dy \qquad \text{----- (1)}$$

H : Height of the pier

ϕ : Curvature at the section $(= \varepsilon_0 / x_0)$

ε_0: Extreme compresive fiber strain at the section

x_0: Distance between extreme compresive fiber and neatral axis

y : Distance from the top of the pier

The relations between stresses and strains are as Fig.1 and Fig.2 . The "ultimate" is defined when it reaches at ε_0=0.0035 or ε_s=0.002. It is recommended that the allowable displacement of the pier top is taken 1/3 of the δ_u with consideration of the test results that the ductilities of concrete pier decrease under alternatingly repeated loading. The load at the allowable displacement (P_a) is derived with the load-displacement relation drown from the result of calculations. Equivalent stiffness (K_{eq}) is obtained by $K_{eq}=P_a/\delta_a$. Equivalent natural period when the displacement reachs to δ_a is calculated by eq.(2).

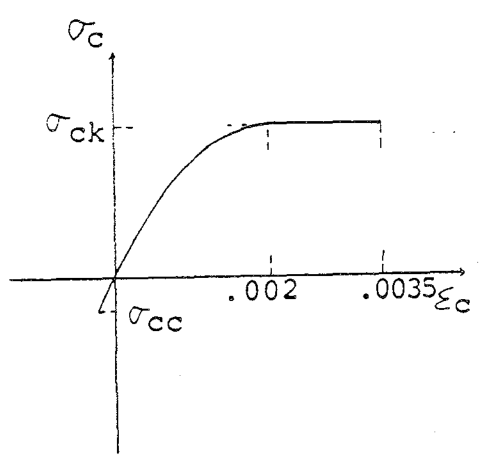

Fig. Stress-Strain Relation
Concrete

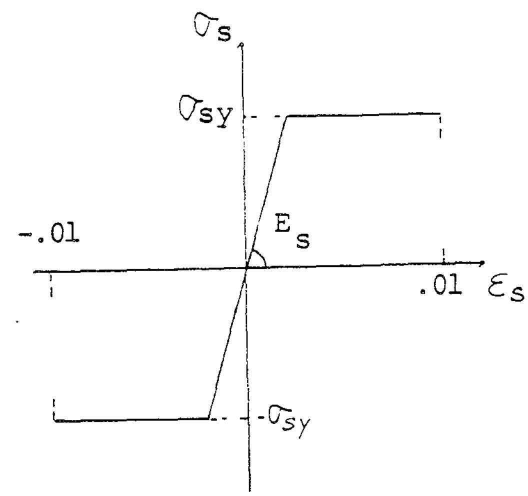

Fig. Stress-Strain Relation
Reinforcing bar

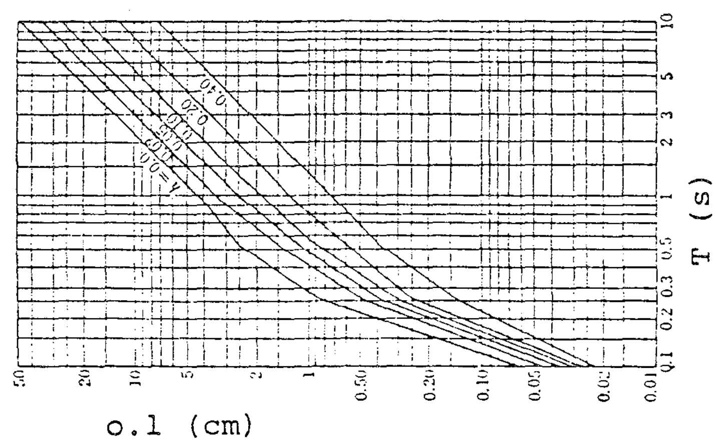

Fig.3 Example of Response Spectrum Curve

$$T=2\pi\sqrt{W/gk} \qquad \text{----- (2)}$$

where $W=W_u+0.3xW_p$

W_u: Weight of the superstructures supported by the pier

W_p: Weight of the pier

g : acceleration of the gravity force ($9.8\ m/s^2$)

Equivalent damping factor is calculated by eq.(3).

$$h=0.02+0.2x(1-\frac{1}{\sqrt{\mu_a}}) \qquad \text{----(3)}$$

where $\mu_a=\delta_a/\delta_y$

δ_y: Displacement of the pier top at the yielding of reinforcement

Eq.(3) is quoted from Gulkin and Sozen's proposal.

The seismic coefficient $K_{\mu a}$ when the displacement reaches at δ_a is calculated by eq.(4).

$$K_{\mu a}=\delta_a/\alpha(2\pi/T)^2 \qquad \text{----(4)}$$

where α : Maximum absolute accelation

As $\alpha\cdot(2\pi/T)^2$ is the displacement under the earthquake which acceleration is 1.0g, we can calculate it with Response Spectrum Curves and the result has been drown taking into account both damping factor and the ground conditions. One of them is shown in fig.3. As this fig. is drown in the condition that the acceleration is 0.1, $K_{\mu a}=\delta_a/(10x\delta_{0.1})$ where $\delta_{0.1}$ is the displacement when holizontal acceleration is 0.1g. If $K_{\mu a}$ is greater than $1.3xK_h$, it is estimated that the structure is safety on ductility.

4. FUTURE STUDY

Problems need further study is as follows.

(a) The rationality of the assumption that the allowable ductility is to be 1/3

(b) The method for deriving the damping factor

(c) The rational method for improving the ductility of the concrete structure and other means acceptable when enough ductility is not obtained.

References

1). Japan Road Association : Specification for Highway Bridges, Part V, Earthquake Resisting Design, 1980

2). Minoru Ohta: "A Study on Earthquake Design for Reinforced Concrete Bridge Piers of Single-Column Type", Report of PWRI No. 153.

3). P. Gulkan and M. A. Sozen: "Inelastic Responses of Reinforced Concrete Structure to Earthquake Mortions" Jour. of ACI, Proc. Vol.71, No.12, 1974

Earthquake Resistant Design of Tokyo Bay Crossing Bridge

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1. Outline of Tokyo Bay Crossing Bridge & Tunnel

The Tokyo Bay Crossing Bridge & Tunnel is the marine road of 15km in length to cross the center of Tokyo Bay from Kawasaki city of Kanagawa prefecture to Kawasaki city of Chiba prefecture. Nihon Doro Kodan is carring out investigation to construct an exclusive road for automobile traffic with the design speed of 80km/hr and six lanes.

From the viewpoint of structural type, it consists of three kinds of structures, i.e., an immersed tunnel to secure the passage of large vessels in the middle of bay, bridges with the clearance of small vessels under it at the both side of tunnel and man-made islands which conect the tunnel to the bridges.
(Fig-1)

With regard to type of bridges, continuous steel box girder for superstructure and three different types of multi-pile foundation, underwater footing foundation and open caisson foundation for substructure are being studied.

The seabed at the planned location of route is deposited with a soft layer of alluvial silty-clay, which is 20m-30m in thickness. The maximum depth of water is about 30m around the middle of bay. Moreover, this area was attacked by a number of severe earthquake in the past.

Therefore, in the designing of the Tokyo Bay Crossing Bridge & Tunnel, the most important problem to be solved is to ensure the stability of structures during the earthquake attacking.

2. Earthquake Resistant Design of Bridges

The earthquake resistant design of bridges shall be carried out in accordance with the seismic coefficient method or the modified seismic coefficient method, which complies with Earthquake Resistant Design of Specifications of Highway Bridges: Japan Road Association, 1980, in principle. In addition, the dynamic analysis shall be adopted to certify the safety for these large-scale bridges which are constructed on the extremely soft soil deposits and which are expected to deform considerably during earthquake.

Since the time history analysis is used as the dynamic analysis, it is necessary to determine an input earthquake motion, which is based on the acceleration response spectra of the anticipated earthquake at the Tokyo Bay area.

3. Determination of Input Earthquake Motions

It is recommended that the magnitude of input earthquake motions are determined on the basis of the levels of earthquake resistivity required to the planned bridges. Then, those two levels are specified as follows.

1) It is necessary that bridges shall maintain thier functious against the motion which is expected to occur once during thier life time.

2) It is also necessary to prevent bridges from collapsing to the motion which is expected to occur rarely at the Tokyo Bay.

The two earthquake motions are assumed, i.e., one has the return period of 150 years for the first level (L1), and another corresponds to Kanto Earthquake (M=7.9) for the second level (L2).

The expected response spectra of the bedrock to the above mentioned two earthquake motions were calculated as shown in Fig-2. Subsequently, the strong motion records, whose response spectra closely resemble those of L1 and L2 respectivity, are selected from past earthquakes, and are modified to fit to those response spectra. Here, the acceleration records at the Kaihoku Bridge during the Miyagiken-Oki Earthquake and at the Hachinohe port during the Tokachi-Oki Earthquake were adopted, and input earthquake motions were obtained by modifying the amplitude of the acceleration records so that their response spectra coincide with those of L1, L2 respectivity, as shown in Fig-3,4. Fig-5,6 shown the modified motions of L1 and L2 as input earthquake motions.

Their characteristics and application rang are as follows,

1) Modified motion L1 : Acceleration motion whose response sepctrum characteristic is similar to that of expected response spectrum of earthquake with a 150 year return period, is applied to the structures with natural period of 0.3sec. to 3 sec.

2) Modified motion L2 : Acceleration motion whose response spectrum characteristic is similar to that of expected response spectrum of earthquake as large as Kanto earthquake, is applied to the structures with natural period of 1sec. to 3sec.

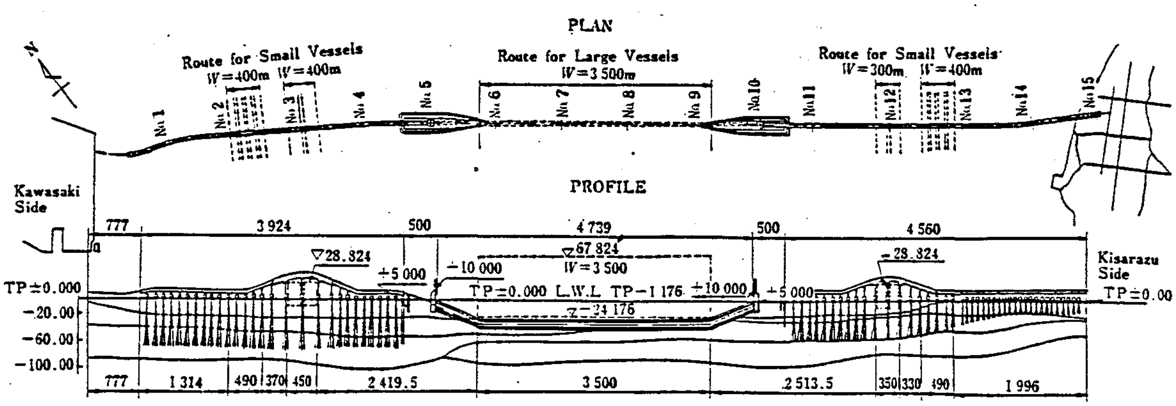


Fig-1. General View of Tokyo Bay Crossing Bridge & Tunnel

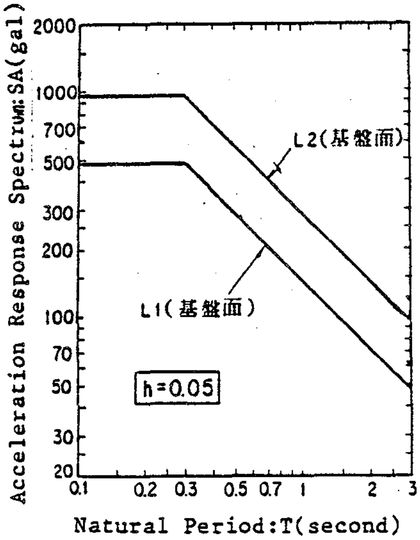


Fig-2. Acceleration Response Spectra

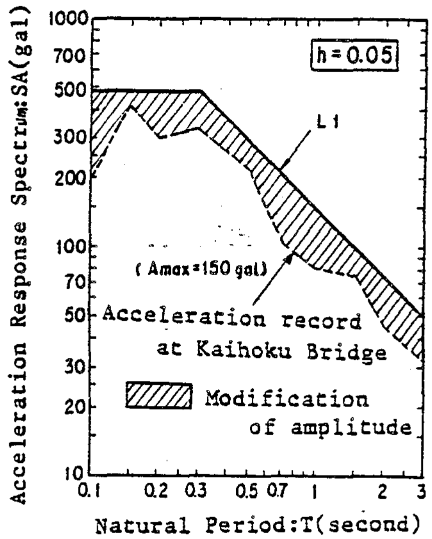


Fig-3. Modification of Amplitude for L1

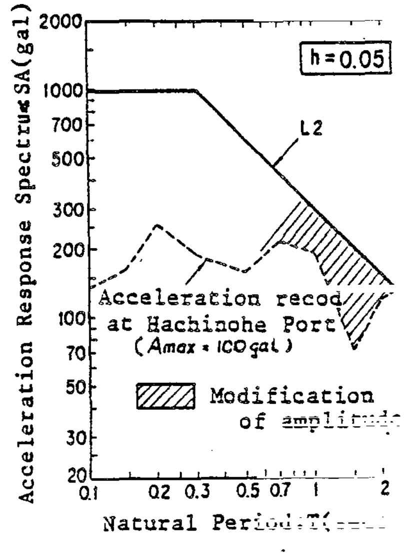
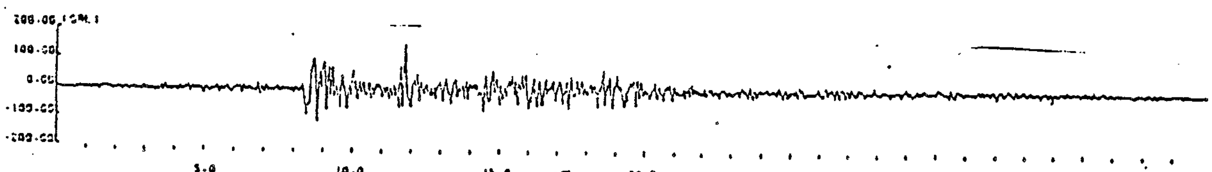
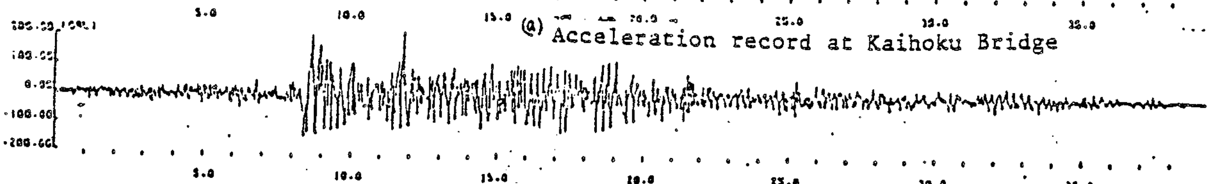


Fig-4. Modification of Amplitude for

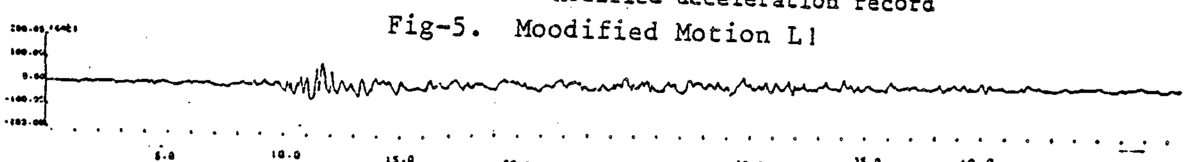


(a) Acceleration record at Kaihoku Bridge

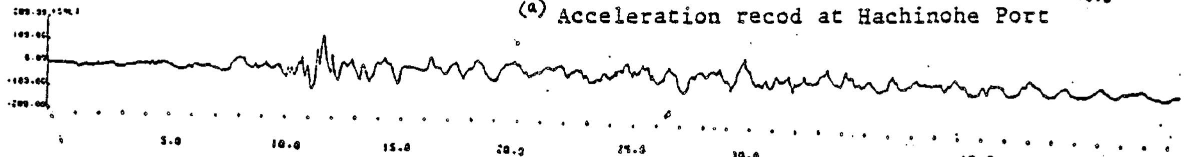


(b) Modified acceleration record

Fig-5. Moodified Motion L1



(a) Acceleration recod at Hachinohe Port



(b) Modified acceleration record

Fig-6. Modified Motion L2

J-8

EARTHQUAKE RESISTANCE DESIGN OF THE MEIKO NISHI BRIDGE

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TOKYO, JAPAN

GENERAL

The Meiko Nishi Bridge, or the Nagoya Harbor West Bridge is the first of three newly projected bridges which expected to form a connecting link over the Nagoya Port.

In order to integrate into the enviroment and also to provide a landmark for the Nagoya Port area, thoughtful consideration is given in the design of the bridge.

The Meiko Nishi Bridge is a cable-stayed bridge with a 3-span continuous steel box-girder. Its span arrangement is 175m+405m+175m and its cable arrangement is double plane multiple fan cable. The tower is A-shaped.

As the ground of this bridge site is very poor, the special attention is needed to secure safety against earthquake and to reduce its cost. An elastic stopper system using cables to connect the tower and the box-girder is adopted. The purpose of this system is to obtain the dampening effect of the longitudinal thermal force exerted to the tower base and the piers, while the damper cables function as stoppers for the box-girder.

THE ELASTIC STOPPER SYSTEM

Among various elastic stopper systems, this particular system that connects the tower and the box-girder by strand cables, named the Meiko Cable Damper, is adopted.

The rigidity of the cable is determined as follows,

$$K = \frac{E \cdot A}{L} \cdot \cos\theta$$

where, K: spring coefficient (rigidity of the cable)
E: modulus of elasticity of the cable
A: cross section area of the cable
L: length of the cable
θ: angle formed with the cable and horizontal level

Lateral reaction at the tower base of this system is designed to be 1/2 of that of firmly fixing system. This resulted in that the spring coefficient is K=11000 t/m.

The cable is pre-tensioned to prevent release of stress within the range of the designed temperature (±30 C.).

VIBRATORY PROPERTY OF THE SYSTEM

If the box-girder and the tower is connected with flexibility in longitudinal direction, the natural period of the structure becomes longer. This is very effective to reduce response to earthquake, but this increases deflection of live load and bring out structural disadvantage. In order to decrease static deflection and dynamic responses at the same time, elastic damper system was adopted.

Fig. 1 shows that at the designed spring coefficient natural period of the structure makes little difference, even though the spring coefficient is doubled or halved. This means that response of the structure to earthquake also makes little difference.

DYNAMIC ANALYSIS

One of the results of spectrum analysis is shown in Fig. 2. Supposing the spring coefficient is changed, calculated response of the structure makes little difference. Dynamic response of the structure was calculated on the base of seismic records, but the result was much the same.

EARTHQUAKE RESISTANCE DESIGN

The seismic inertia force is generally estimated by static analysis, replacing the inertia force with static load. As for the Meiko Nishi Bridge, earthquake resistance design was planned, adopting modified seismic method considering the response to earthquake. The calculated natural period and vibration mode of this bridge system were taken into account, when modifying. The maximum design seismic coefficient became 3.0 as shown in Fig. 3.

Dynamic response was calculated, using some seismic records with the maximum 150 gal. acceleration. The result of the dynamic analysis proved that the earthquake resistance design of this bridge is appropriate.

The erection of the tower of the Meiko Nishi Bridge was accomplished and now the box-girder is under construction. This bridge is scheduled to be completed at the end of 1984. On completion of the bridge, the vibration test is planned to examine its structure system.

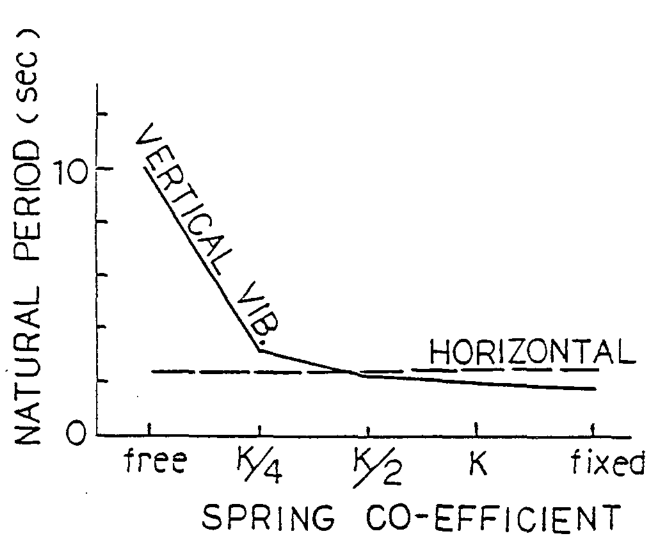

Fig. 1 NATURAL PERIOD OF THE SUPERSTRUCTURE

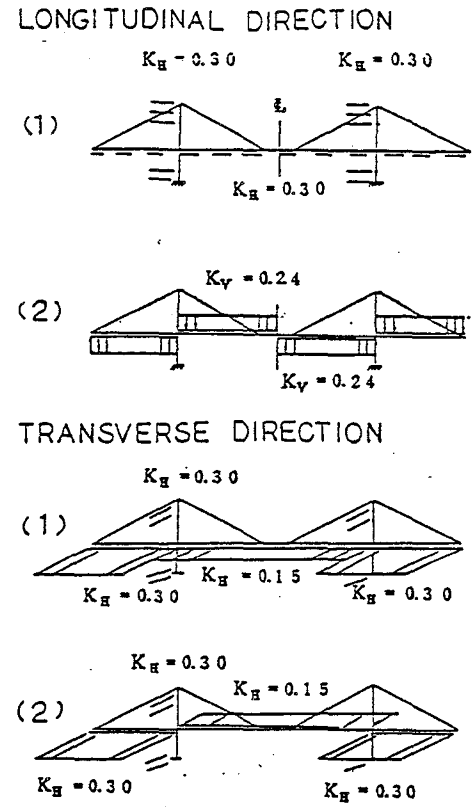

Fig. 3 DESIGN SEISMIC COEFFICIENT

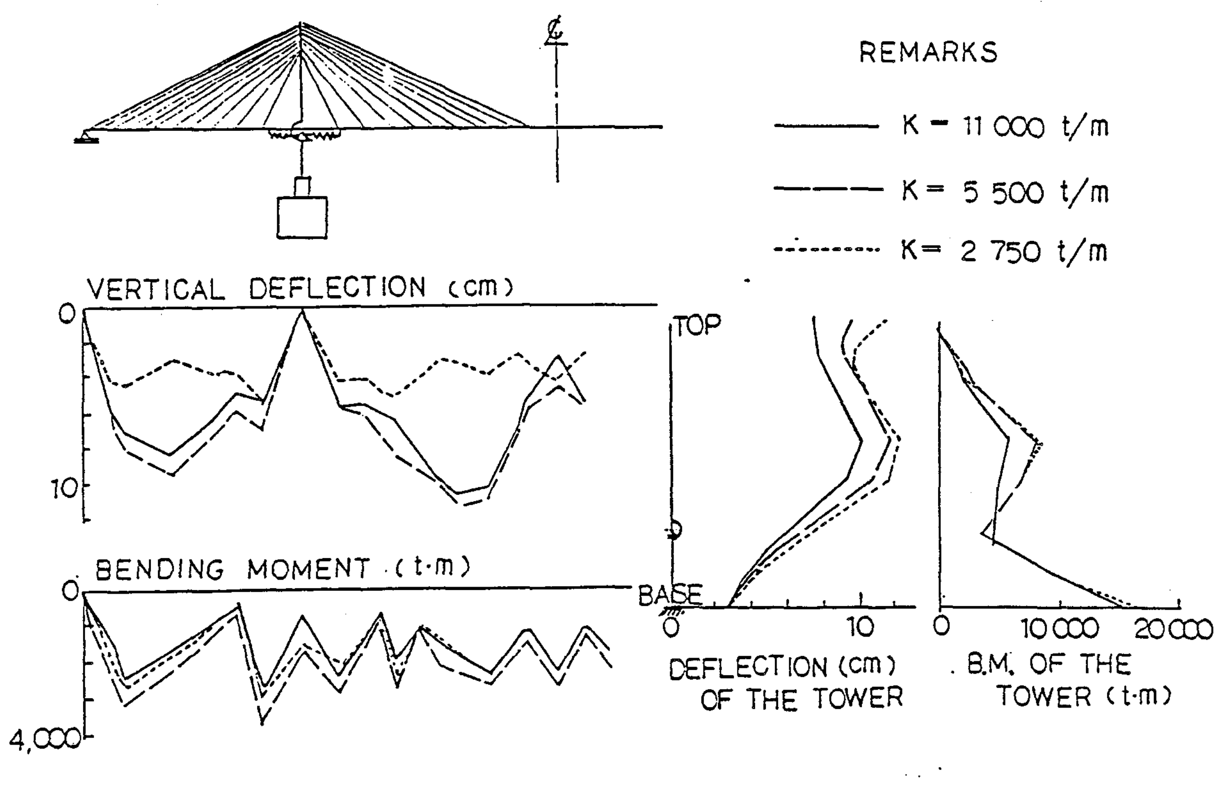

Fig. 2 DYNAMIC RESPONSE OF SPECTRUM ANALYSIS

A DYNAMIC ANALYSIS OF A MULTI-SPAN CONTINUOUS BRIDGE

J-9

Mitsuo Hara

Design Section
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GENERAL

Because of the ease of design and construction, simple span girder type bridges have been prevalent to date on urban expressways. Recently however, from the point of veiw of aesthetics, aseismic characteristics and the noise from the bridge expansion joints, the use of continuous girder types has been on the increase for intra-urban bridges. Accompanying this trend is the utilization of an increased number of spans per bridge, 8 to 12 spans being the norm.

This paper describes the dynamic analysis as carried out on one such 10 span continuous bridge constructed by the Metropolitan Expressway Public Corporation in Kawaguchi City.

OUTLINE OF BRIDGE

The important part of the design of multi-span continuous girder bridges lies in the effective distribution of the seismic loads and axial loads due to temperature from the superstructure to the piers and substructure. By considering a number of support types these loads are applied to what is in effect a flexible substructure. Here, steel piers are used and the horizontal loads from earthquake and temperature effects are distibuted to each pier with respect to its relative stiffness. The overall stability of the structure is then analyzed.

The bridge is shown in Figure 1. The individual spans are 50m with a total bridge length of 500m. It is composed of two steel box girder bridges, having R.C. decking, the central section having facility for on/off ramps. Over this stretch of bridge the overall cross section varies with the number of girders reducing from 3 to 2. The transverse beams included in the superstructure are constructed integrally into the box girders. The steel piers are of the independent twin column type, with moveable bearings at the two end piers and hinged at the 9 internal ones. The foundation structure is a footing set on 1.5m diameter reverse piles.

DYNAMIC ANALYSIS

The dynamic model testing was carried out on a 1:25 scaled model using a vibration table at the civil engineering research laboratories of the Ministry of Construction. The response modal analysis was carried out by considering the entire structure as a multi-mass plane frame. The input earthquake was in accordance with the Highways Association Standard V aseismic design provisions, using a response spectrum having an acceleration intensity of 255gal, a damping factor h = 0.02 and a design seismic coefficient Kh = 0.33.

Figure 2 shows the stiffnesses of each pier as a spring factor applied at the pier cap. In order to limit the displacement of the end piers along the lateral axis their stiffnesses have been made larger than those for the internal piers.

DYNAMIC ANALYSIS RESULTS

The results of the dynamic analysis for this bridge are as follows.

First, by considering the pier base moments and the modes of vibration for the response along the longitudinal axis it was found that the dominant mode is the fundamental mode. This is similar to the assumptions made in the Modified Seismic Coefficient Method. Namely for this case the superstructure acts as a solid body with respect to the substructure, the entire bridge exhibiting, so to speak, the simple response for a single mass. The seismic method for response in the longitudinal axis can thus be considered to approximate closely,that for static design.

Second, the results given in Figure 3 show that, for analysis along the lateral axis, the fundamental through to the 5th harmonic all have a significant influence upon the response values. From this it can be seen that the vibration along the lateral axis is made up of a large number of individual superposed modes.

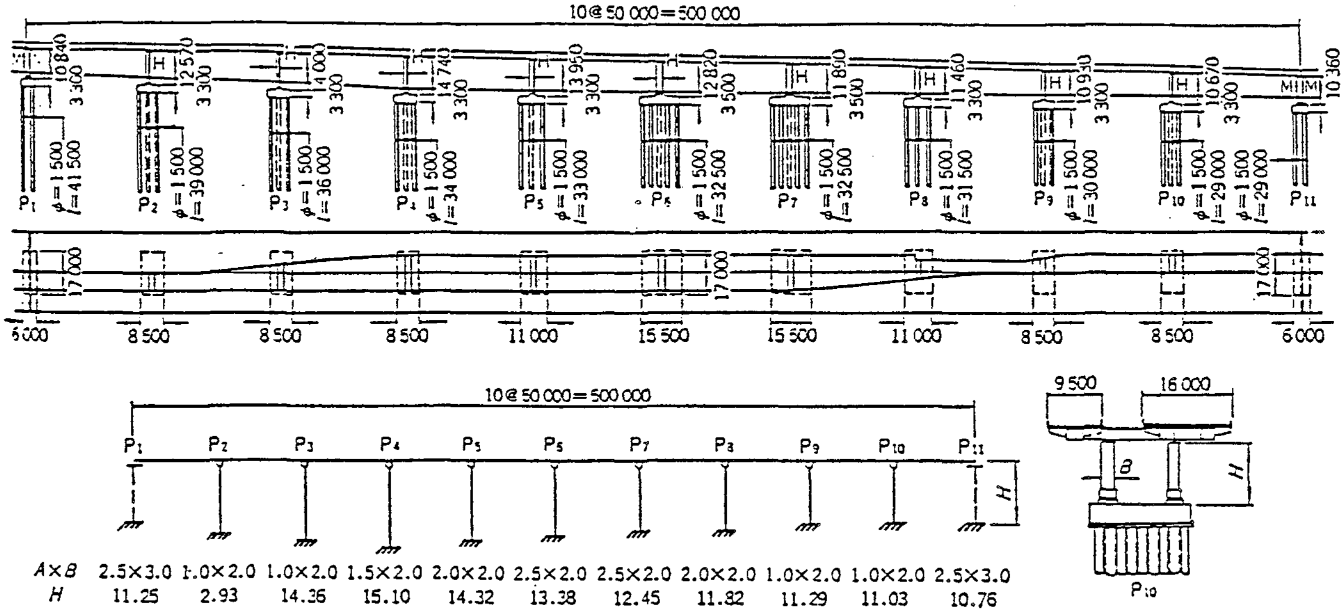
Figure 3(2) shows the result for an equivalent superstructure model having a uniform cross section. Since, in this instance, the actual cross section is not uniform, Figure 3(2) indicates the substantial difference in conditions represented by the model results. Also from the results in Figure 3(1) it can be seen from the small displacements with respect to the entire structure, that the two end piers, whose stiffnesses were increased, do in fact fulfill the intended function.

Table 1.2 gives the respective pier displacements and the pier base moments from the dynamic analysis. By considering the more important, design-wise, pier base moments and comparing the values with those from the static design the overall average response is seen to be 86% of the static design values. This result stems from the vibration mode being composed of a large number of dominant harmonic modes. However if each pier is considered separately it can be seen that for piers P6 and P7 the values are higher than the static values by 8% and 4% respectively, indicating the tendency for the moments to be attracted to the central portion of the bridge.

CONCLUSIONS

The design and construction of the actual structure was carried out taking into account both the model vibration tests and the results of analysis. The accuracy of the design method was however, checked by observing the behaviour of a 10 span continuous girder bridge for seismic and temperature effects etc.

The results of the vibration tests carried out on a 12 span continuous girder bridge were presented in 1981 at the 13th Joint Meeting of the UJNR at Tsukuba, Japan.



A: Longitudinal dimensions (m)

B: Lateral dimensions (m)

Figure 1 : Outline of 10 Span Continuous Bridge

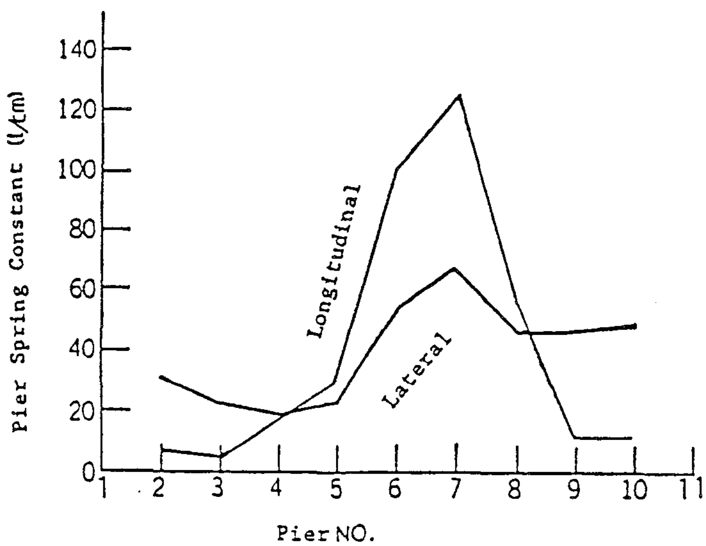
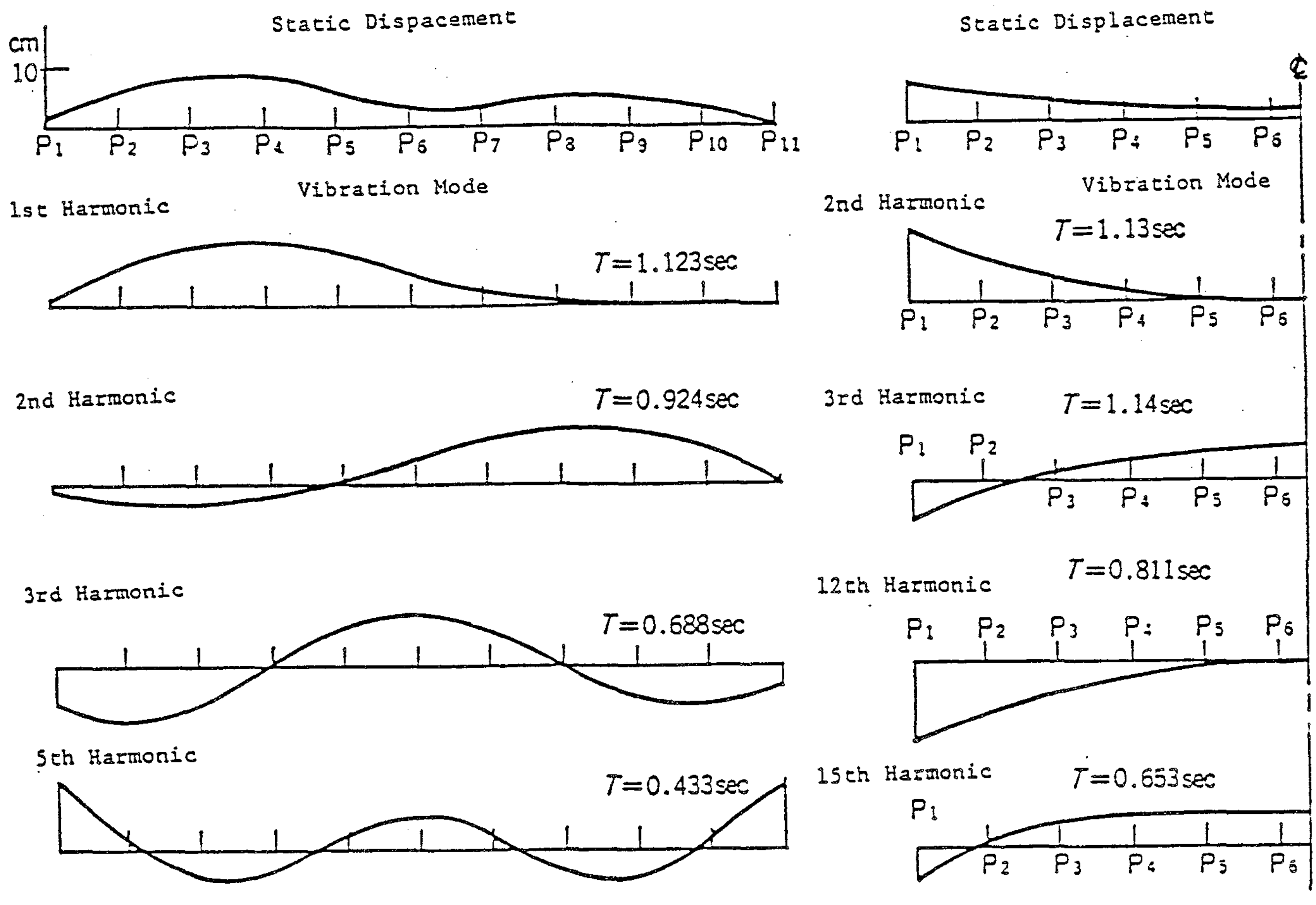


Figure 2 : Distribution of Pier Spring Constant for 10 Span Continuous Girder Bridge



(1) 10 span continuous bridge

(2) Model bridge

Figure 3 : Static Displacements and Vibration Modes for the Lateral Axis

Table 1 : Pier Cap Displacements for the Lateral Axis of a 10 span continuous girder bridge (mm)

		P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	Average
Displacements	Static Analysis δ_1	12.5	58.5	86.7	89.0	67.3	40.0	33.3	46.7	52.0	38.6	10.5	47.7
	Dynamic Analysis δ_2	14.3	64.6	101.4	109.4	88.7	59.2	47.3	53.4	52.8	36.2	10.7	58.0
	δ_2/δ_1	1.14	1.14	1.17	1.23	1.32	1.48	1.42	1.14	1.02	0.94	1.02	1.22

Table 2 : Pier Base Moments along the Lateral Axis (t·m)

		P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	Total
Bending Moment	Static M_1	7 374	5 271	6 283	5 316	5 442	6 468	6 155	5 801	6 009	4 743	6 740	$\Sigma M_1=65\ 602$
	Dynamic M_2	6 173	4 279	5 459	4 749	5 197	6 955	6 388	4 873	4 527	3 245	5 065	$\Sigma M_2=56\ 910$
	M_2/M_1	0.8	0.81	0.87	0.89	0.96	1.08	1.04	0.84	0.96	0.68	0.75	Average .86

Earthquake Resistant Design for Cable Stayed Bridge - Ajigawa Bridge, Hanshin Expressway

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H. Nakajima

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J-10

Ajigawa Bridge[1)] now under construction is one of the longest cable stayed bridges in Japan, which is an asymmetrical three span structure with a center span of 350 m, end spans of 120 m and 170 m, and has diamond shaped towers 111 m high from the top of its supporting footing. (Fig. 1)

The seismic design procedures of the bridge are outlined in the flowchart appearing in Fig. 2. As seen in the flowchart, the features of the deisgn[2)] are soil - structure interaction being taken into account, the acceleration response spectrum being established especially for the site of the bridge and the dynamic analysis of the structure as a whole being performed for the seismic design.

Model for Soil - Structure Interaction Analysis

The plane strain finite element model for the soil-structures interaction analysis is shown in Fig. 3 for the ground motion in the transverse direction of the bridge. The superstructure of the bridge in the model is simplified without losing its dynamic characteristics. In this analysis, the dilluvial stratum with an N-value of the standard penetration test greater than 50 and a shear wave velocity greater thatn 300 m/sec is assumed to be the bed rock from the engineering judgement.

Parameters β and G appearing in the figure are damping coefficient and shear modulus of the soil under the given ground motion, which are calculated by using the computer program "SHAKE" with the sounded data on the soil. The input bed rock motions for one dimmensional soil column model used for "SHAKE" are TAFT N21E record and the earthquake ground motion recorded near the site (call this "NANKO") with their amplitude modified to be 100 cm/sec^2, which corresponds to 200 cm/sec^2 on the ground surface. According to the results given by this computer work, the input motion for the soil-sturcture interaction model the accelaration amplitude of about 180 cm/sec^2 in case of the TAFT N21E and about 190 cm/sec^2 in case of the local record "NANKO" (the maximum accelaration amplitude recorded is 17 cm/sec^2)

Acceleration Response Spectrum for the Bridge

The time history analysis was done by using the model and input motions mentioned above to get the acceleration response spectrum on the top of the pile supported spread footing, where the tower of the bridge is embedded. Fig. 4 shows the lateral acceleration response spectra by the TAFT N21E record and by the local record together with the response spectrum for the soil type IV, which is given in the Japanese code and the proposed response spectrum for the bridge. It is observed that both response spectra by the TAFT record and the local record have the peak around the period of one second and this is considered due to the dynamic characteristics of the soil at the site.

In determining the design response spectrum for the bridge, this characteristics was taken into account and more importantly the upper bound of 400 cm/sec^2 on the response spectrum was set by referring to the existing code, design guidelines for the other bridges of this kind and by the engineering judgements that the damping ratio could be much greater than 2 % when such a large acceleration acts on the bridge and that the response acceleration by the local earthquake record whose maximum acceleration is rather small could be considered to be smaller when the maximum acceleration of the record gets larger.

Multi Mode Analysis for the Superstructure

The multi-mode dynamic analysis for the superstructure of the bridge was carried out by using the acceleration response spectrum. The lumped mass frame model of the bridge was used for the analysis. The period of the first mode is about 3.7 sec and the effective mass for that mode is 90 %. All members of the superstructure are checked by the force induced by the earthquake and a part of the tower member (one close to the supporting footing) is designed by the earthquake force.

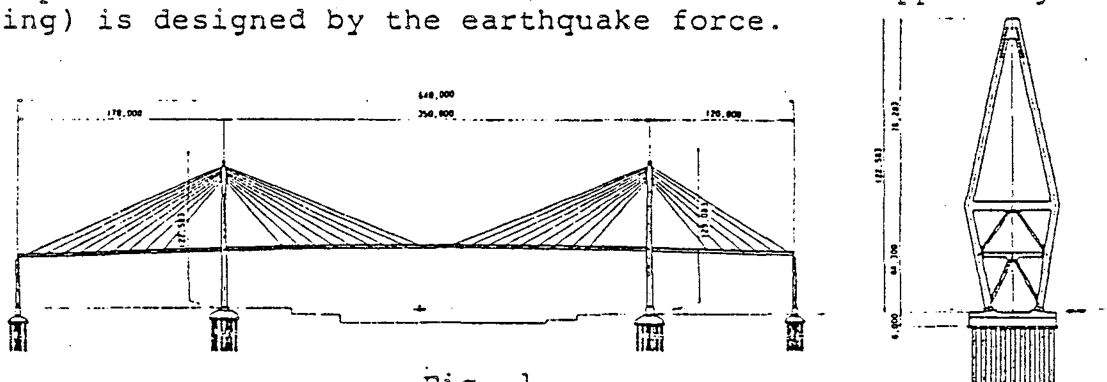


Fig. 1

selection of input earthquake

data on soil

$\beta-\gamma$ and $G-\gamma$ curve by indoor test

computer analysis by "SHAKE"

determine coefficients of soils during the earthquake

simplified pile foundation model → ← simplified superstructure model

soil-structure interaction analysis model (F.E.M.)

computation of eigenvalues for the model acceleration response spectrum on the top of the footing

↓ ← existing design response spectra

comparison and evaluation

↓

determine design acceleration response spectrum

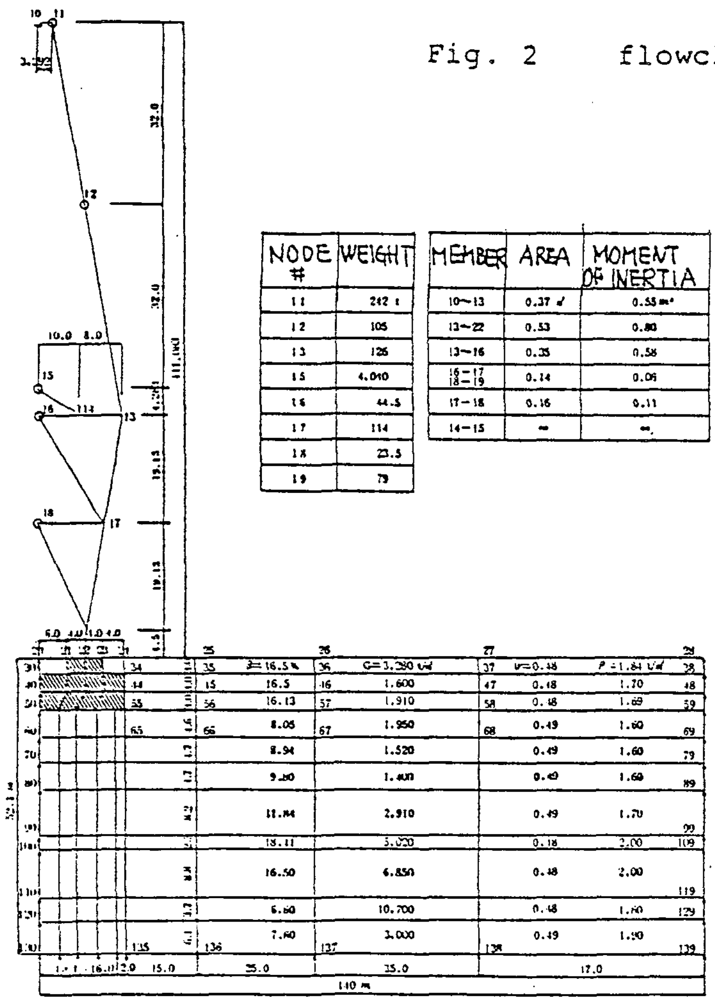
↓

multi mode analysis for superstructure

↓

determine design force

Fig. 2 flowchart



NODE #	WEIGHT
11	242 t
12	105
13	126
15	4,040
16	44.5
17	114
18	23.5
19	79

MEMBER	AREA	MOMENT OF INERTIA
10~13	0.37 m²	0.55 m⁴
13~22	0.53	0.80
13~16	0.35	0.58
16−17 18−19	0.14	0.09
17−18	0.16	0.11
14−15	∞	∞

Fig. 3

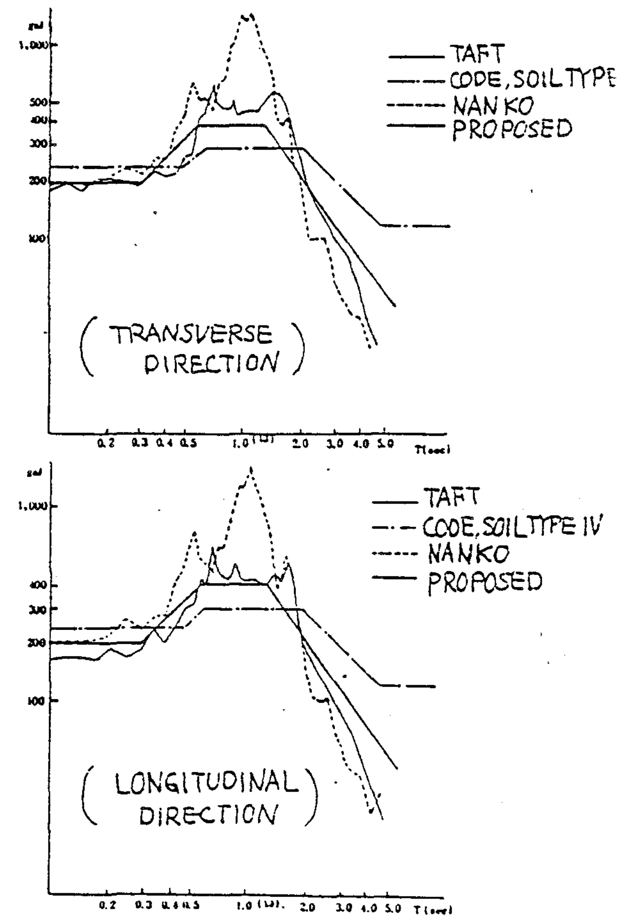


Fig. 4

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2) EMI, S, "Seismic Design of the Ajigawa Bridge", 1982, Hanshin Exgpressway Public Corp. Technical Report, in Japanese

Earthquake Resistant Design of Bannosu Viaduct

J-11

Honshu-Shikoku Bridge Authority
Kozo Higuchi

I. Introduction

Bannosu Viaduct in Kojima-Sakaide Route of Honshu-Shikoku Bridge Project is a combined highway and railway bridge.

These piers are 40 to 70 meters high with I section as shown in Fig.I. Upper part of column supports only highway load.

Thus, distribution of mass and rigidity of the upper part is very small comparing to the lower part. And dynamic analysis is required to check the safety in addition to the modified seismic coefficient method which is generally used.

2. Dynamic Analysis

There are several problems to apply dynamic analysis in design.

These are as follows:

I) Input motion
2) Analytical model
3) Assessment of result

Therefore, we have performed several type of dynamic analysis as shown in Table I.

Earthquake input motion used in Case I ~ Case 4 is the horizontal acceleration of the top of the footing. It is obtained by the method of wave propagation theory in multi-layered system considering non-linearity of shear modulus and damping coefficient. In that calculation the Kaihoku Tr earthquake motion record by the Miyagi-ken-oki Earthquake in I978 is inputted at the design ground surface.

Response spectrum curve used in Case 5 is obtained from the average spectrum of ground type of group 4 by cutting its maximum β as shown in Fig.2.

In elasto-plastic analysis used in Case I and Case 2, the moment-curvature ($M - \phi$) relation of the pier section is assumed by a degrading tri-linear model proposed by Prof.K. Muto as shown in Fig.3 and Fig.4. The relation between stress and strain of concrete and reinforcing bars are assumed in Fig.5. However the $M - \phi$ is assumed as linear in Case 3 ~ Case 5.

Calculated maximum responses of bending moment are presented in Fig.6. In these figures the design value (obtained by the modified seismic coefficient method) is also presented.

And the followings matters are pointed out.

(I) The bending moment at the bottom of pier (Sec.C in Fig.I) by elastic analysis (Case 3 and Cace 4) is nearly equal to the design value. But at the bottom of upper part (Sec.B in Fig.I) is approximately 4 times the design value and approximately 3 times in Case 5.

So it is understood that oscillation behavior of upper part is significantly different from those assumed in the modified seismic coefficient method.

(2) Case I or Case 2 are considered to show the real behavior. Thus, it is unreasonable to apply the result of Case 3 or Case 4 to the allowable stress method.
(3) The result of Case 5 is nearly equal to that of Case I. So we have judged that elastic dynamic analysis by Case 5 is effective to apply for the design.

3. Remarks

Througn above mentioned investigation we proposed the practical dynamic analysis method for the design.
Further, we have performed repeated loading test using I/4 or I/ 4√2 model of upper part to determine M-φ curve and hysterisis and I/IO model of lower part to determine the rigidity of I section pier.

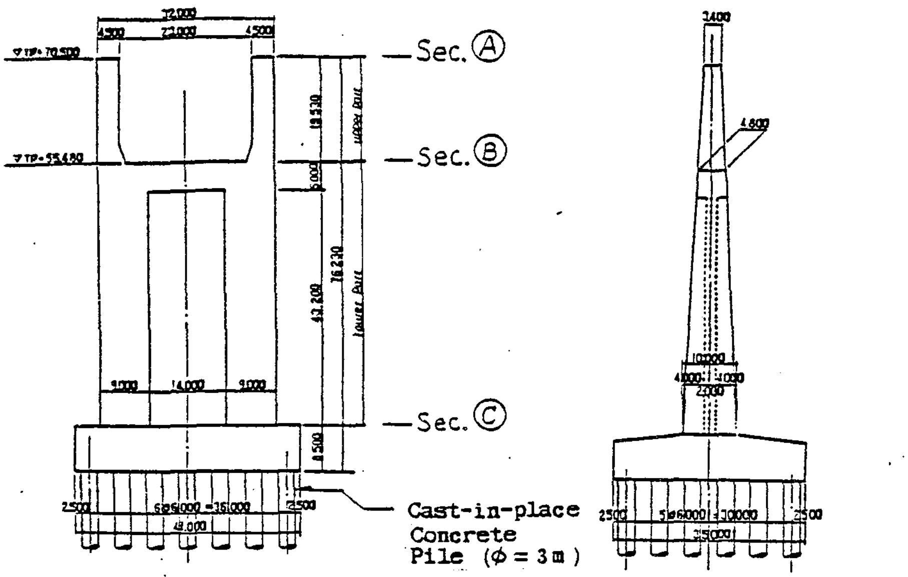


Fig. 1 General View of the Pier of Bannosu Viaduct (4P)

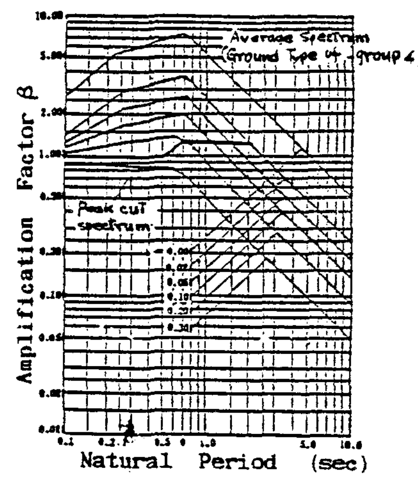


Fig. 2 Response Spectrum Curve

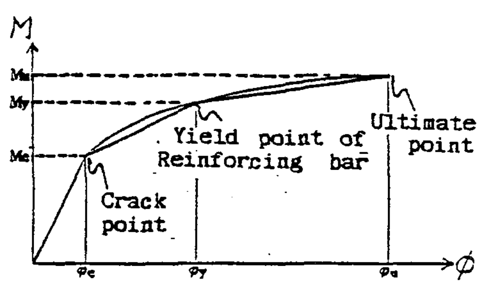


Fig. 3 M-φ Curve

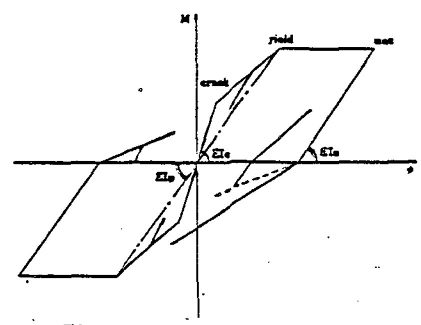


Fig. 4 Hysterisis Curve

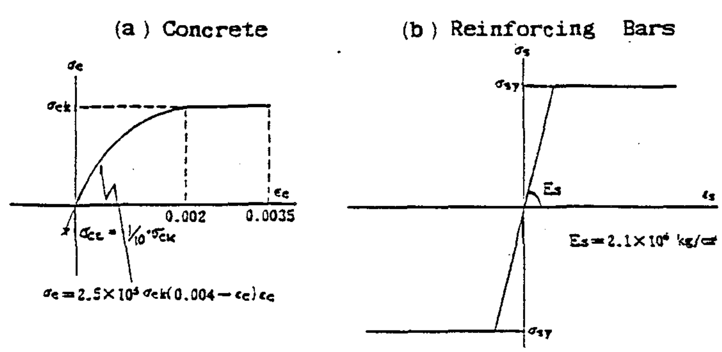
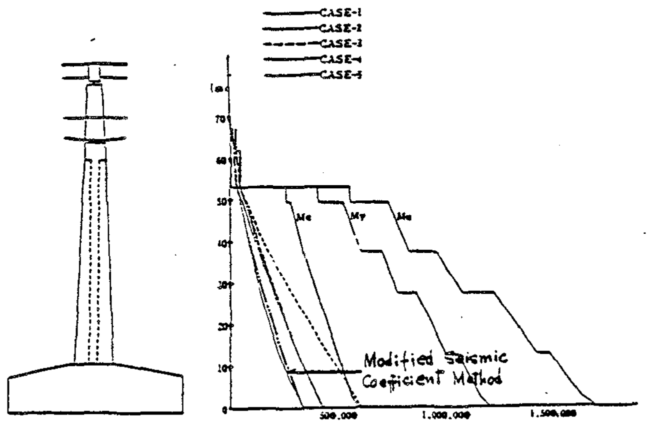


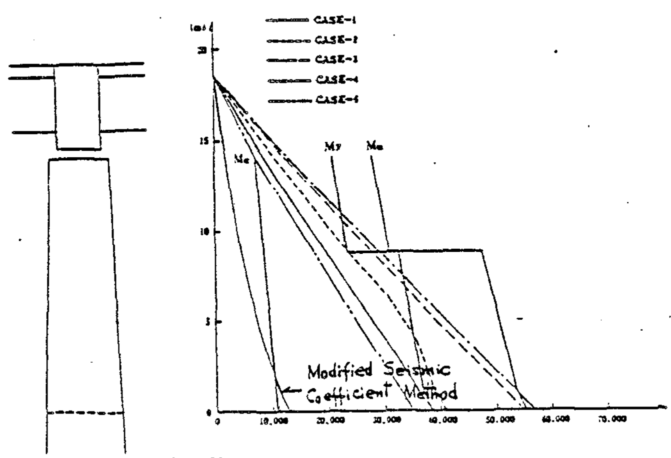
Fig. 5 stress - Strain Relation

Table 1 Case of Dynamic Analysis

	Rigidity of Pier	Analytical Method	Maximum Acceleration at the Bace of Footing	Input Motion
Case 1	Elasto-Plastic	Time History Analysis (Numerical Integration Method)	gal 200	Kaihoku
Case 2	〃	〃 (〃)	300	〃
Case 3	Elastic	〃 (〃)	200	〃
Case 4	〃	〃 (Model Analysis)	200	〃
Case 5	〃	Response Spectrum Tecnique (〃)	200	Doken (βmax =1.25)



(a) Lower Part



(b) Upper Part

Fig.6 Maximum Bending Moment in Transverse Direction

FUTURE PROGRAMS AT CONCRETE DIVISION, PWRI

by Shigetoshi KOBAYASHI

J-12

1. Introduction

Concerning earthquake resistance of concrete structure and earthquake resisting design concept, following three themes are to be carried out at Concrete Division, Geology and Chemistry Department PWRI, hereafter.

1) Study on earthquake resisting strength of concrete members
2) Study on ductility and earthquake resisting strength of joints of concrete structure
3) Study on repair of earthquake-damaged concrete structure

The following chapters show the outline of above themes.

2. Study on earthquake resisting strength of concrete members

Elastoplastic property of reinforced concrete members were analyzed theoretically in the former studies and the results were reflected in the present design method. In order to certificate the elastoplastic peroperty of RC members experimentaly, items shown in table 1. will be investigated.

3. Study on ductility and earthquake resisting strength of joints of concrete structure

Joint section of RC structure is apt to be most fragile part against earthquake attack. In this theme ductility and earthquake resisting strength of joint, especially joint of footing and pier of large bridge, will be clarified and design method of joint will be reconsidered to increase ductility and strength. (see table 1.)

4. Study on repair of earthquake-damaged concrete structure

This theme will be carried out in co-operation study with Earthquake Engineering Division. At Concrete Division, study on repair material and repairing method is being carried out.
(see "Future Programs at Earthquake Engineering Division")

Table 1. Items of Future Programs

Theme	1984	1985-
Earthquake Resisting Strength of Concrete Members	o Experimental analysis on effect of Cut-off of main reinforcement to shear strength o Effect of loading histry of earthquake resisting strength	o Experimental analysis on effect of Cut-off of reinforcement to property of RC members o Experimental analysis on property of rigid framed pier and box section pier
Ductility and Earthquake Resisting Strength of Joints of Concrete Structure	o Analysis on stress of joint of footing and pier of large bridge o consideration of anchorage length of reinforcement	o Reconsideration of the arrangement of reinforcement for increasing ductility of joints o Study on the method for increasing ductility of joints by concrete material

J-13

Research Plan on Earthquake Resistance by Foundation Engineering Division

Kazuya Ohshima

Head, Foundation Engineering Division,
Public Works Research Institute.

In the design of foundations for bridge structures in this country, the sizes and section performance of the foundations are being governed by seismic forces in many cases in recent years. Therefore, it can be said that most researches on the foundations performed in the Foundation Engineering Division are related to the earthquake resistance. However, only studies of the horizontal strength of foundation structures will be reported here. The future research plan on the horizontal strength of foundation structures by the Foundation Engineering Division will be explained below.

1. Study of Coefficient of Subgrade Reaction

In the design of foundations for structures, many different calculating formulas for the coefficient of subgrade reaction are currently being used depending upon the kind of structure. Because of this, in recent years the sizes of foundations sometimes varies depending upon the formula used and foundation types have consequently been greatly diversified (for example, the coefficient of subgrade reaction varies depending upon whether a pile with a large diameter is considered as pile or caisson in design).

Because of the reason stated above, the Division is planning to unify the calculating method for the coefficient of subgrade reaction and to establish the unified design method for the horizontal strength of foundation through the horizontal loading test and various calculations.

2. Study of Bearing Capacity of a Foundation on Slopes

Foundations of bridges are often built on slopes in hilly lands or mountainous areas, but there are still many uncertain items involved in the design of foundations built on such ground. Because of this, foundation models will be placed on slopes whose strength varies in several steps and the correlations between the kind of ground, width of the front of foundation, angle of slope of ground, length of slope, angle of inclination of load, and the conditions of development of plastic zone of ground at the front of foundation, failure mode, ultimate bearing capacity will be studied.

3. Study of Design Method for Bridge Seats

One of the remarkable damages to substructures during earthquake is the failures of concrete bridge seats. Bridge seats must be designed so as to assure the shear resisting force required against horizontal force during earthquake. In view of this, experiments will be performed in order to study the effective methods for anchoring supports and for improving the ductility of reinforced steel bars.

4. Study of Reinforced Concrete Bodies

In order to study the shear strength of bridge pier, model specimens with circular and rectangular sections will be produced and biaxial tests performed to find the influence of steel ratio, spacing of hoops and the presence of shear reinforcement in horizontal direction upon the shear strength. Also, the ratio of axial force to horizontal load will be changed and then the influence of the magnitude of axial force upon the shear strength and ductility of the pier stud will be studied.

J-14

Behavior of Concrete Filled Steel Tubes (Part-3 Beam-Column Members)

Bridge Division, Structure and Bridge Department

Shoichi Saeki
Toshitaka Miyata
Koichi Minosaku

1, Introduction

In 14-th and 15-th UJNR Joint Meetings, we introduced features of behavior of Concrete Filled Steel Tubes as short column members and bending members.

This paper continuously describes an outline of tests on beam-column members, and also introduce our future research programs.

2, Problems

Principal problems on Concrete Filled Steel Tubes as beam-column members are as follow.

(1) Evaluation of flexual rigidity

From the results of former two tests, it is clear that Concrete Filled Steel Tubes has high toughness and do not lose capacity even when large displacement produces. However, it must be examined that Concrete Filled Steel Tubes has high toughness in case of beam-column members.

In case of beam-column members, the secondly bending moment must be considered in the design.

In calculating the secondary bending, a horizontal displacement must be calculated. Then, calculating methods of flexual rigidity which takes into consideration a condition of inner-concrete and a composite degree between steel tubes and concrete must be established.

(2) Necessity and arrangement of shear connectors

From the result of tests on short column members, in an elastic region, it is clear that concrete separates from steel tubes due to the difference of possion's ratio between concrete and steel tubes.

Under axially loads, members do not usually resist as composite structure in an elastic region. Therefore, if horizontal loads act on members under axially loads, it is feared that Concrete Filled Steel Tubes cannot exhibit high capacity and toughness, because it takes behave like a built-up beam.

For this reason, rational arrangement of shear connectors is necessary. Further, the arrangement methods of shear connectors must be examined.

3, Outline of the tests

The purpose of our research is to get basic data for a design formula under axially loads and bending, and to clear up how concrete strength and thickness of steel tubes influence behavior of members.

In first series of tests, four specimens, as are shown in Figure-1 and Table-1, will be set, and tests will be carried out by using Large Structure Bi-axial Testing Machine which was installed last year.

In tests,horizontal loads will be increased under constant axial load (700 ton), and loads and displacement in two directions, and strains of steel tubes and concrete will be mesured.

4,Future research program

From now on, we will examine about two items as is indicated below.

(1) Beam-column members

We will examine necessity and an arrangement methods of shear connectors (stud,etc.), and also behavior of members under repeative loads.

(2) Connections

In order to apply Concrete Filled Steel Tubes to piers, we will examine forms and structual details of connections that can effectively transmit high toughness and rigidity to beam members.

Table-1. Dimension of specimens

TYPE	Diameter (mm)	Thickness (mm)	Height (mm)	Steel	Concrete Strength (kg/cm^2)
A	700	6	3500	SM50Y	180
B	700	6	3500	SM50Y	240
C	700	6	3500	SM50Y	300
D	700	12	3500	SM50Y	180

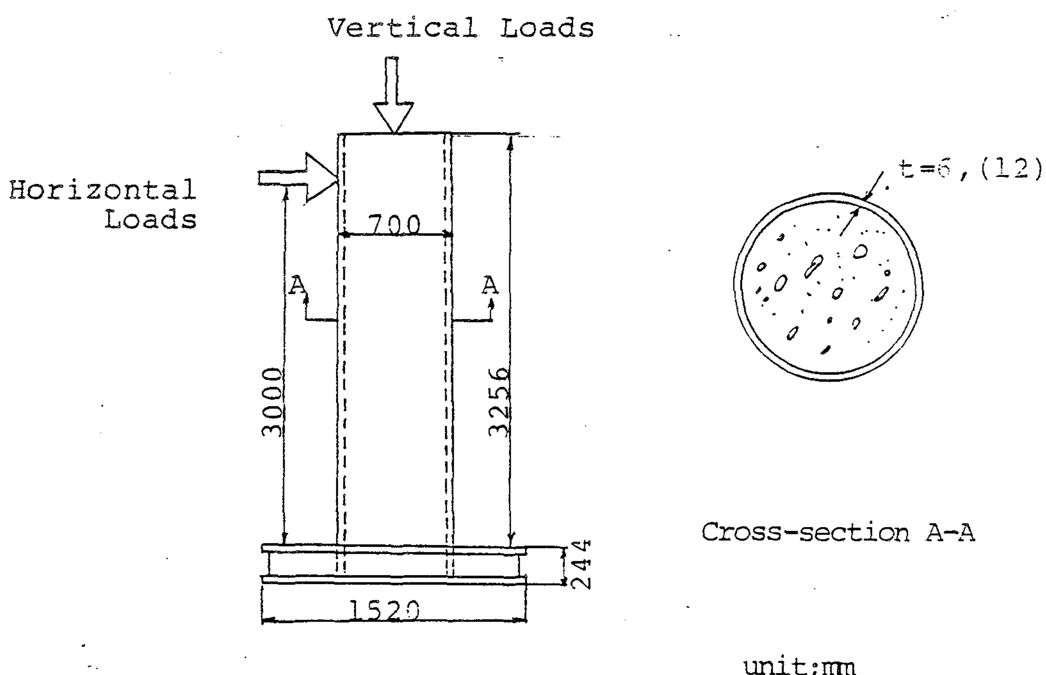


Figure-1. Configuration of specimens

J-15

FUTURE PROGRAM

Tadashi Arakawa

Ground Vibration Division
Earthquake Disaster Prevention Department
Public Works Research Institute
Ministry of Construction, Tsukuba, Japan

I. Observation and analysis of strong motions

1) Countrywide strong motion observation network

2) Dense instrument array program

II. Study of Ground motion and earthquake response of civil engineering Structures

1) Design forces related to seismic resistant design based on ground condition

2) Seismic resistant design of bridges for destructive earthquakes

3) Development of seismic resistant design procedure of large structures for long-period ground motions

4) Development of seismic resistant design procedure of underground structures considering ground strains induced during earthquake.

5) Seismic resistant design of road embankment on the liquefiable sandy layers

6) Seismic resistant design of underground structures affected by liquefaction of sandy layers

7) Seismic risk map for possible Liquefaction

FUTURE PROGRAMS AT EARTHQUAKE ENGINEERING DIVISION, PWRI

BY TOSHIO IWASAKI*

INTRODUCTION

Future programs in bridge earthquake engineering to be conducted at Earthquake Engineering Division, Earthquake Disaster Prevention Department, PWRI, in the fiscal year of 1984 starting April 1, 1984, will be summarized by the following seven topics. In the parenthesis are indicated sponsoring organizations.

1) Study on Effectiveness of Seismic Design of Highway Structures with Consideration of Total Cost Consisting of Construction Cost and Expected Damage Expenditure (Ministry of Construction)
2) Seismic Design Method of Reinforced Concrete Bridge Pier Columns with Use of Results of Dynamic Experiments and Dynamic Response Analyses (Ministry of Construction)
3) Procedure for Estimating Earthquake Resistance of Existing Highway Bridges (Ministry of Construction)
4) Procedures for Evaluating Damage Extent and Repairing Damaged Bridge Structures (Ministry of Construction)
5) Seismic Design Method of Continuous Girder Bridges (Ministry of Construction)
6) Dynamic Analysis of Pile-Caisson Foundations for Long-Span Suspension Bridges (Honshu-Shikoku Bridge Authority)
7) Study on Seismic Behavior of Rubber Bearing Supports for Elevated Highway Bridges (Hanshin Expressway Public Corporation)

The following chapters will briefly introduce some characteristics of the above studies.

Study on Effectiveness of Seismic Design (1981 ~ 1985)

At present, seismic design of general high bridges are done according to the JRA Specifications of Highway Bridges , in which the conventional seismic coefficient method is used, with use of the seismic coefficient varying from 0.13 to 0.24 for bridges with the pier height of 15 meters or less. In this study, a cost-benefit analysis will be done by evaluating relationships between the design seismic forces and the total cost consisting of construction cost plus expected damage expenditure. For estimating expected damage expenditure secondary effects of earthquake damage (such as economic losses) are considered, in addition to the direct structural damage losses. This study will provide a fundamental concept of seismic design of bridges, in consideration of optimization of investment for bridge construction, and will give appropriate levels of seismic forces according to the importance of bridges.

Study on Reinforced Concrete Bridge Pier Columns (1983 ~ 1987)

In order to depvelop ample design methodology of bridge piers, dynamic experiments on earthquake resistance of RC pier columns are undertaken using the Dynamic Structural Testing Facility at PWRI. The dynamic experiments will be performed with emphasis on the following:

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a) Effects of Numbers of Dynamic Cycles
b) Effects of Bi-Directional Horizontal Loading
c) Effects of Horizontal and Vertical Loading
d) Effects of Eccentric Loading

Those tests will be conducted by varying axial reinforcements, shear span ratios, hoops and ties, and cross-sectional shapes.

In addition to the above experiments, dynamic response analyses will be conducted with consideration of non-linear characteristics of supporting soils and inelastic behavior of RC columns obtained from the above experiments.

Study on Earthquake Resistance of Existing Bridges (1981 ~ 1985)

In order to approptiately strengthen, prior to the outbreak of an earthquake, existing bridge structures which seem to be vulnerable to earthquake disturbances, procedures for estimating earthquake resistance of existing highway bridges are currently investigated. A special attention is paid to the effects of interactions between surrounding soils and bridge substructures with pile foundations or caisson foundations. The results of this study will provide effective means of retrofitting existing bridges.

Study on Evalucation of Damage Extent and Repairing Methods for Bridges (1981 ~ 1985)

From the experiences of the past strong earthquakes it seems important to conduct investigations on evaluation methods of damage extent and repairing methods for bridges. This study will provide a guidline for of evaluating damage extent and levels of repairing, and structures for selecting appropriate repairing works for damaged bridge structures.

Seismic Design of Continuous Girder Bridges (1983 ~ 1987)

A number of continuous girder bridges sustained damages during recent strong earthquakes such as the Miyagi-ken-oki Earthquake of 1978, the Urakawa-oki Earthquake of 1982, and the Nihon-kai Chubu Earthquake of 1983. It seems that the current procedure for designing substructures of continuous bridge girders is not necessarily suitable. Especially, allotments of seismic forces acting from superstructures to substructures, should be carefully studied through experiments and analyses. According to the current design specifications[1], seismic forces acting on substructures are propotional to the reactions acting on the substructure crowns from superstructures. In this study dynamic tests with use of four separete shaking tables will be conducted, and the results will provide appropriate seismic design methods for substructures of continuous girder bridges.

Dynamic Analysis of Pile-Caisson Foundations (1982 ~ 1984)

With the commission from The Honshu-Shikoku Bridge Authority, a study on dynamic analysis methodology is now undertaken for pile-caisson foundation, namely a composite foundation having piles plus caisson. This particular type of foundations is under consideration as those for long-span suspention bridges of Honshu-Shikoku Bridges.

Seismic Behavior of Rubber Bearing Supports (1983 ~ 1984)

Rubber bearing supports may be popularly used in the near future as bearing supports of urban elevated highway bridges, in order to increase seismic resistance of bridges and also to reduce noise pollution problems. With the commisior from the Hanshin Expressway Public Corporation, seismic behavior of rubber bearing supports will be experimentally evaluated.

Summary

Future research programs to be conducted at Earthquake Engineering Division, PWRI are summazied in the above. Among various topics, study on seismic design method of RC bridge pier columns will be emphasized for the fiscal year of 1984 and a few years later.

References

1) Japan Road Association: "Specifications of Highway Bridges, Part IV Earthquake-Resistan Design," May, 1981

EARTHQUAKE RESISTANT DESIGN AND TESTS OF THE KATASHINA-GAWA BRIDGE

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J-17

GENERAL

The Kan-etsu Expressway shown in Fig.1 extending 300 kilometers in length links Tokyo, capital of Japan, with Niigata, the largest city on the Japan Sea, for speedy automobile transportation. Since some portions under construction pass through well-known mountainous regions and rivers in Japan, the expressway calls for big-scale bridges such as Katashina-gawa Bridge, three sets of three-span continuous-truss curved bridge, shown in Fig.2.

The bridge has to cross over a huge U-shaped valley with a wide and flat bottom 100 meters lower than the river terrace. That, therefore, needs very high piers whose maximum height is 70 meters, as well as long spans the maximum of which is 168 meters long.

Since Japan is a typical country in a seismic zone with the addition of severe topology, all consideration for the countermeasures against earthquake has been taken in planning and designing the bridge.

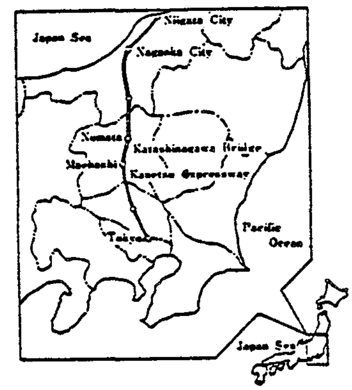

Fig.1 Location

Since a multi-fixed support system was adopted to the central three-span structure with nearly equal height piers, the natural period of the structural system became longer. Consequently, the reduced design force for earthquake has been able to be shared evenly by the multi-fixed piers.

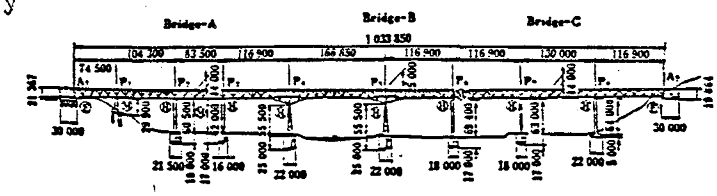

Fig.2 General View

A hollow type pier with two cells is adopted to make the structure more flexible. Since piers from P2 to P8 are 55 to 70 meters high, a steel reinforced concrete structure is applied to ensure the good ductility for ultimate loading conditions.(See Fig.3)

Since the structure is extremely complicated as well as huge, we have made an earthquake registant design in the following items and are going to make further investigation mentioned later.

MODIFIED SEISMIC COEFFICIENT METHOD AND DYNAMIC ANALYSIS

Since the bridge has a relatively longer dominant natural period than 0.5 sec, we basically applied the modified seismic coefficient method shown in Fig.4 which determined an equivalent seismic force in response to the dominant natural period of the structure. In order to obtain natural frequencies by solving an eigenvalue problem, we considered a lumped mass beam model shown in Fig.5 at first. However, more accurate truss model with lumped masses shown in Fig.6 had to be taken into account after the experimental model vibration test mentioned

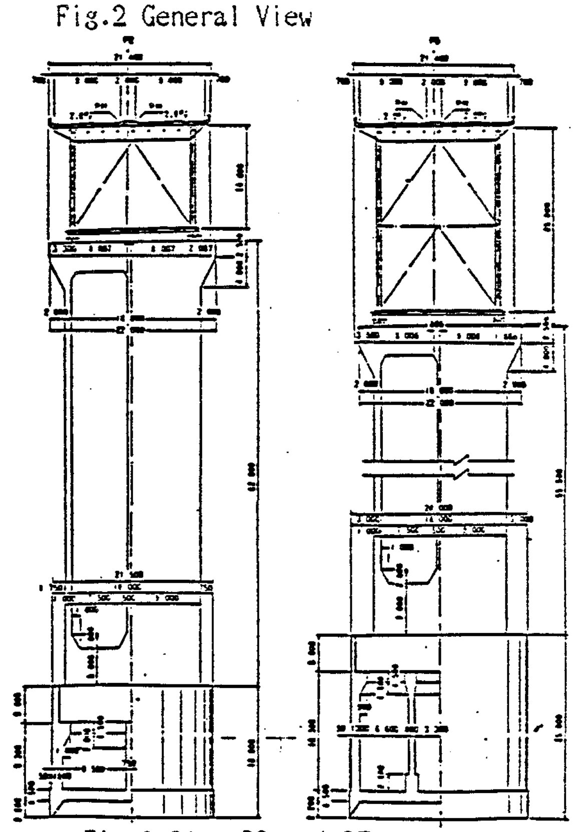

Fig.3 Pier P2 and P5

later. The natural periods of both models and corresponding seismic coefficients are as follows:

longitudinal

	beam model	truss model	
A bridge	0.764 sec	0.893 sec	0.25
B bridge	1.854 sec	1.848 sec	0.20
C bridge	1.096 sec	1.216 sec	0.25

transversal

entire bridge	1.55 sec	0.25

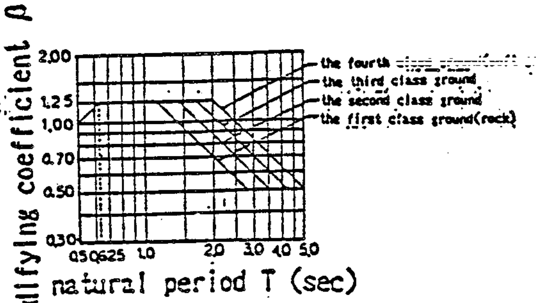

Fig.4 Seismic Coefficient

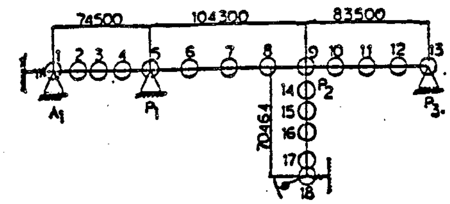

Fig.5 Lumped Mass Beam Model

The dynamic analysis was made in order to compute response using the lumped mass truss model. The averadge response spectrum in the seismic code for highway bridges was used for the modal analysis. According to the analysis, some members such as lower chords at abutment A1 and upper chords at pier P1 found to be over the static design stresses obtained by the modified seismic coefficient method.

EXPERIMENTAL MODEL VIBRATION TEST

In order to find a rational dynamic analysis, a 1/100 experimental model of the bridge was tested by vibrating it on the shaking table the maximum loading capacity of which was 100 tons. The results are as follows:

- Since the model vibrates not only horizontally but also vertically to a great extent, greater stresses were developed in the chord members especially in the center of the span.
- The design natural frequencies calculated by the similarity law for bridge A,B and C were smaller than those of the lumped mass truss model analysis like this:

longitudinal

A bridge	0.867 sec
B bridge	1.390 sec
C bridge	-

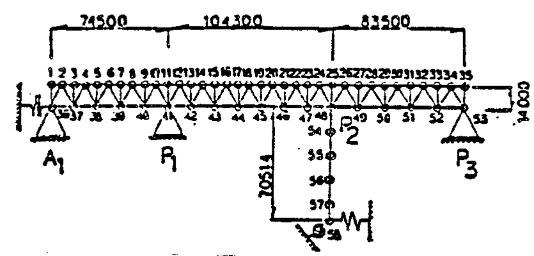

Fig.6 Lumped Mass Truss Model

The difference seems to be because the foundations of experimental model were fixed while the analitic model was supported by the springs equivalent to the ground. According to the Fig.4, it could change the design seismic coefficient for A bridge from 0.2 to 0.25. That is one of the reason the truss model mentioned above was analized to know more accurate value.

VIBRATION TEST OF THE SUBSTRUCTURE

The vibration test for the actual pier P5 and P6 was carried out installing the vibration generator, 50 ton maximum excitation force, on the pier top to check the theoretical results and design parameters. Then we made a simulation by use of two analytic models. One is a lumped mass model pier shaft with spring support equivalent to the ground condition shown in Fig.7, and the other is a lumped mass model supported by the ground simulated by the finite element model with various boundary conditions such as free, fixed, viscous and transmitting, shown in Fig.8. The results are as follows:

for P5, longitudinal

	experiment	spring ground	F.E.M. ground
1st natural frequency	1.53	1.56	1.54
2nd natural frequency	6.4	6.6	6.5
3rd natural frequency	10—11	10.2	11

The modal damping constants of the spring ground model were 1.3 % for the first mode , 6.5 % for the second mode and 25 % for the other modes. The spring constants was 13 times larger than that in design. As far as the finite element analysis was concerned, the damping constants for concrete and ground were 1.3 % and 5 % respectively. The shear rigidity of the ground for this model was 2 times larger than that in the design.

According to the test, the first mode of the resonance was governed by the vibration of the pier shaft itself just like fixed support. However, the foundation began to vibrate greatly at and after the second peak and the soil- structural interaction was getting remarkable.

Eventhough the displacements due to the test were small like a few centimeters at the pier top, the smaller the damping constants are , the larger the response displacements are. Moreover, softer springs make the structure more flexible, in other words those two facts could make the structure more earthquake-resistant than what that is actually are.

One of the results of the simulation by the finite element model is shown in Fig.9. As far as its boundary condition is concerned, the transmitting boundary gave the the best fitting and was superior to the equivalent spring model especially in soil-foundation interaction.

The natural periods were able to be obtained by measuring small disturbances caused due to wind or traffic and making frequency analysis on P2, P3, P5, P6, P7 and P8. Since P5 and P6 were also vibrated by the oscillator, the results from the small disturbances were reviewed by those of the vibration test and got good agreements.

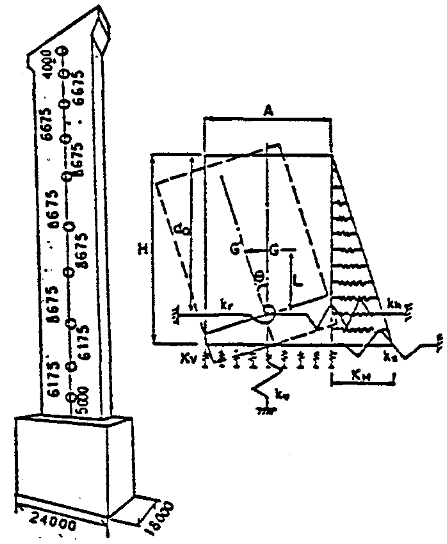

Fig.7 Spring Model

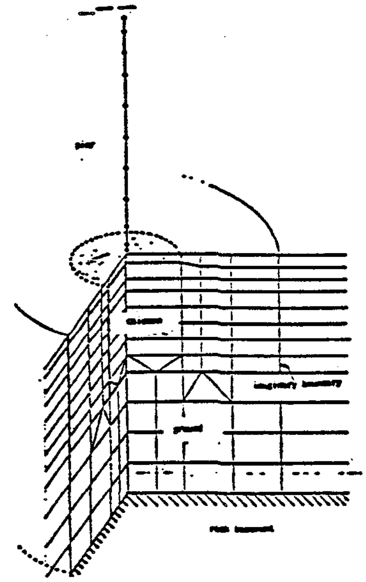
Fig.8 F.E.M. Model

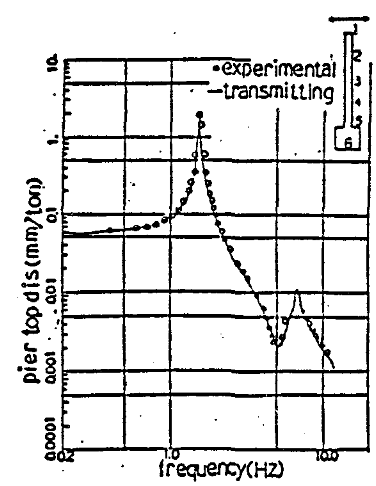

Fig.9 Resonance Curve

VIBRATION TEST OF THE SUPERSTRUCTURE AND FUTURE RESEARCH

In order to confirm the results of the dynamic analysis in the design, the vibration test for the entire structure is scheduled after the completion of the superstructure now under construction. The data obtained so far will be useful to know the dynamic behavior more precisely in the analysis after the vibration test of the entire structure.

Since the vibration tests, so far and upcoming, are limitted to the small displacements the maximum of which is a few centimeters, it is desired to make a consistent observation at the earthquake in order to evaluate the difference between test-loading and actual earthquake force. It is now under discussion to install strong motion accelerometers.

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2. The Vibration Test on the Substructure of the Katashina-gawa Bridge,1983, Japan Highway Public Corporation

OBSERVATION OF THE BEHAVIOUR OF MULTI-SPAN CONTINUOUS GIRDER BRIDGES

J-18

Hiroyoshi Miyauchi

Design and Research Division
Department of Engineering
Metropolitan Expressway Public Corporation

The research into multi-span continuous girder bridges was started in proper by the Metropolitan Expressway Public Corporation in 1978, with three such bridges at present having been completed. The general outline of the bridges is shown in Figure 1, with structural outlines given in Table 1.

The biggest problem with respect to the design of multi-span continuous girder bridges is what proportion of the horizontal load from the superstructure due to earthquake and temperature should be assumed imparted to the respective piers. This distribution varies for each of the three bridges mentioned, and the design is carried out based on static and dynamic analyses, together with model experiments. The accuracy of the design processes of these three bridges is scheduled to be verified by observation of the behaviour of the bridges beginning 1984.

The content of the observations will be as follows.

A) Structural Behaviour Due to Temperature, Shrinkage and Creep.

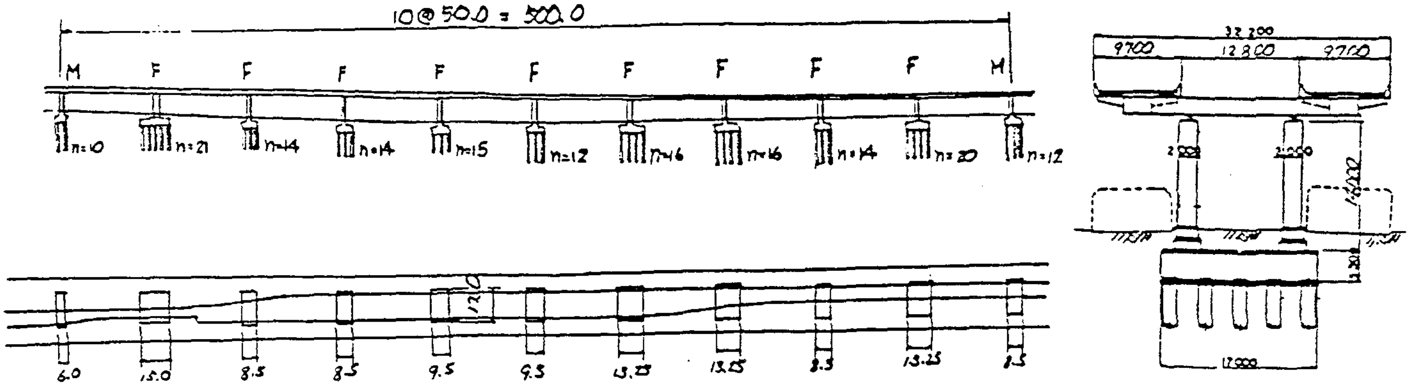
1. Measurement of structure and ambient temperatures.
2. Measurement of girder expansion and contraction.
3. Measurement of pier inclinations.
4. Measurement of variations in damper cable tensions.

B) Structural Behaviour During Earthquake

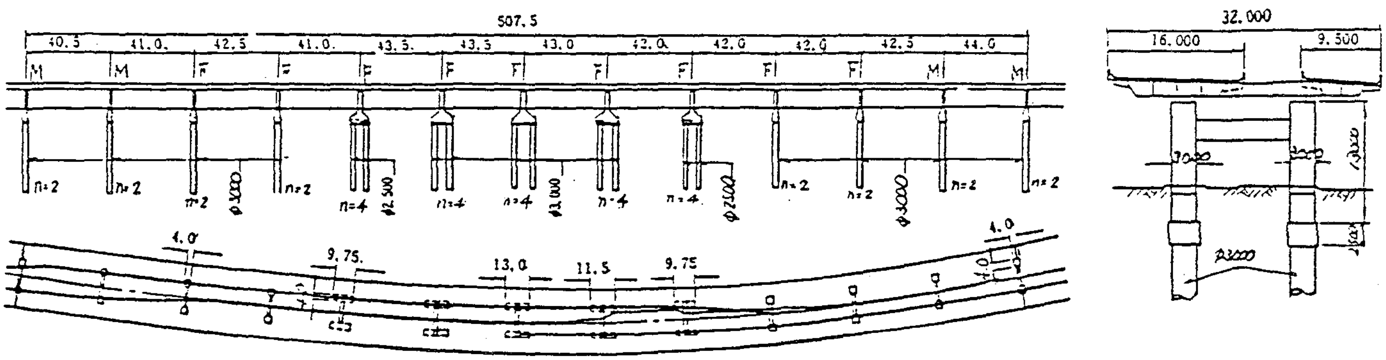
1. Measurement of vibration using seismometers and strong quake instrumentation.

C) Measurement of Non-Uniform Pier Settlement.

BRIDGE A



BRIDGE B



BRIDGE C

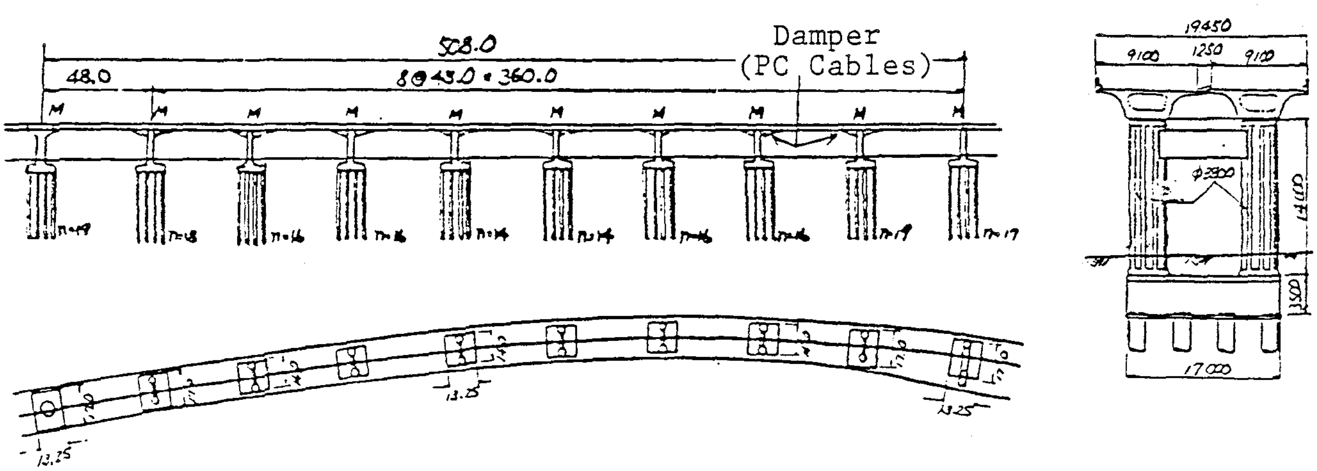


Figure 1: Multi-span Continuous Girder Bridges: General Layout

Table 1. Structural Details of Multi-Span Continuous Girder Bridges

		BRIDGE A	BRIDGE B	BRIDGE C
Superstructure	Main Girder	10 span continuous steel box girder	12 span continuous steel box girder	9 span continuous P.C. box girder
Superstructure	Deck	R.C. deck	R.C. deck	R.C. deck
Superstructure	Supports	Steel plate bearing	Rubber bearing	Steel plate bearings with dampers (P.C. cable)
Substructure	Pier	Independent twin steel column pier	R.C. pile bent pier (H-type)	R.C. pier (H-type)
Substructure	Foundation	Footing with reverse piles (ϕ1500)	Large diameter reverse piles (ϕ2500, ϕ3000)	Footing with reverse piles (ϕ1500)
Distribution method for the horizontal load due to combinations of earthquake and temperature.		Piers taken as flexible structures, the horizontal load being distributed with respect to the relative stiffnesses.	With respect to the pile bents the entire super/substructure is taken as flexible, with the distribution of the horizontal load calculated by varying pile numbers and pier stiffnesses etc.	Main girders are connected to the piers by P.C. cables with the horizontal load calculated by taking into account the stiffnesses of the pier-damper combinations.

J-19

Future Programs of Hanshin Expressway Public Corp.

1) Ground motion to be considered in Osaka-Kobe Area

2) Ultimate strength and ductility of RC and steel bridge-structure columns

3) Earthquake resistant design of PC cable stayed bridge

4) Dynamic Response of multi-span continuous bridge with rubber shoes

Earthquake Resistant Design of Akashi Kaikyo Bridge

J-20

Honshu-Shikoku Bridge Authority
Mamoru Yamagata

I. Introduction

The proposed Akashi Kaikyo Bridge is a long span suspension bridge having deep foundations embedded into soft sediments.

In designing the bridge with deep foundations, the interaction between structure and soil may be of major importance.

The purpose of this paper is to provide a brief information about the progress of studying method by the Lumped Mass-Spring-Dashpot Model with consideration of the interactions.

2. Subsurface soil condition

The subsurface soil condition at the tower pier of 2P is shown in Fig. I.

The basement consists of granites of late Mesozoic, over which such layers as Miocene Kobe Group, Pilo-Pleistocene Akashi Formation, upper Diluvium and Alluvium exist.

3. Dynamic analysis by the lumped mass-spring-dashpot model

The effect of interaction is one of the important problems to be solved.

Several methods to deal with this problem have been proposed by scholars, designers and so on.

In several methods, the finite element method and the mass-spring method seem to be very popular in Japan.

In designing Akashi Kaikyo Bridge, the lumped mass-spring-dashpot method based on the wave propagation theory has been proposed as a suitable method added to conventional methods.

As shown in Fig-2(a), the ground surrounding a foundation is represented by series of springs and dashpots in this design model.

The equation of motion of the rigid foundation is written as

$$\begin{bmatrix} m & 0 \\ 0 & J_G \end{bmatrix} \begin{Bmatrix} \ddot{x}_G \\ \ddot{\theta} \end{Bmatrix} + \begin{bmatrix} K_{xx}+i\omega C_{xx} & K_{x\phi}+i\omega C_{x\phi} \\ K_{x\phi}+i\omega C_{x\phi} & K_{\phi\phi}+i\omega C_{\phi\phi} \end{bmatrix} \begin{Bmatrix} x_G \\ \theta \end{Bmatrix} = -\begin{bmatrix} m & 0 \\ 0 & J_G \end{bmatrix} \begin{Bmatrix} \ddot{U}'_G \\ \ddot{\phi}' \end{Bmatrix} + \begin{Bmatrix} Q \\ M \end{Bmatrix}$$

where

- m, J_G : mass and its moment of inertia
- $K_{xx}, K_{\phi\phi}, K_{x\phi}$: equivalent spring coefficients
- $C_{xx}, C_{\phi\phi}, C_{x\phi}$; equivalent damping coefficients
- ω : excitation frequency
- x_G , θ : relative displacement of and rotation about the mass center of the founfation
- U'_G , ϕ' : effective seismic motion
- Q, M : external force

The equivalent spring and damping coefficients are expressed as follows;

$$K_{xx} = Ga\left(cu1 + \frac{G_s}{G}\delta su1 \right)$$

$$K_{\phi\phi} = Ga^3\left[c\phi 1 + \left(\frac{z_c}{a}\right)^2 cu1 + \frac{G_s}{G}\delta s\phi 1 + \frac{G_s}{G}\delta\left(\frac{\delta^2}{3} + \frac{z_c^2}{a^2} - \delta\frac{z_c}{a}\right) su1 \right]$$

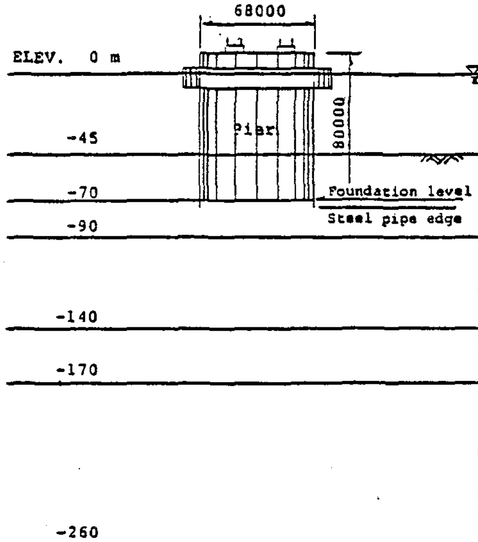
$$K_{x\phi} = -Ga\left[z_c cu1 + \frac{G_s}{G}\delta\left(z_c - \frac{h}{2}\right) su1 \right]$$

$$C_{xx} = \frac{Ga}{\omega}\left(cu2 + \frac{G_s}{G}\delta su2 \right)$$

$$C_{\phi\phi} = \frac{Ga^3}{\omega}\left[c\phi 2 + \left(\frac{z_c}{a}\right)^2 cu2 + \frac{G_s}{G}\delta s\phi 2 + \frac{G_s}{G}\delta\left(\frac{\delta^2}{3} + \frac{z_c^2}{a^2} - \delta\frac{z_c}{a}\right) su2 \right]$$

$$C_{x\phi} = -\frac{Ga}{\omega}\left[z_c cu2 + \frac{G_s}{G}\delta\left(z_c - \frac{h}{2}\right) su2 \right]$$

Comparison of the results from the FEM and the lumped mass-spring-dashpot model shows that the agreement in the general trend is good but the stiffness obtained from the FEM model is slightly larger.
The Honshu-Shikoku bridge Authority intends to continue the study of this model.



	Density γ_t (t/m^3)	S' wave Velocity Vs (m/s)	P' wave Velocity Vp (m/s)	Poisson's Ratio $*\nu$	Initial Shearing modulus $G_0 = \frac{\gamma_t}{g} \cdot V_s^2$ (t/m^2)
Water					
Akashi Formation Sand and Gravel	2.0	400	1,800		31,020
		700	2,000	0.43	100,000
		800	2,200	0.42	130,600
Kobe Group K_1 Sandy layer	2.2	1,000	2,500	0.40	224,500
K_2 Muddy layer	2.2	900	2,500	0.43	181,800
K_3 Muddy layer	2.2	1,100	3,000	0.42	271,600
Granite	2.5	2,000	4,500 6,000	0.48	1,020,400

$$*\nu = \frac{1-2(V_s/V_p)^2}{2\{1-(V_s/V_p)^2\}}$$

Fig.1 Subsurface soil condition at Tower pier 2P

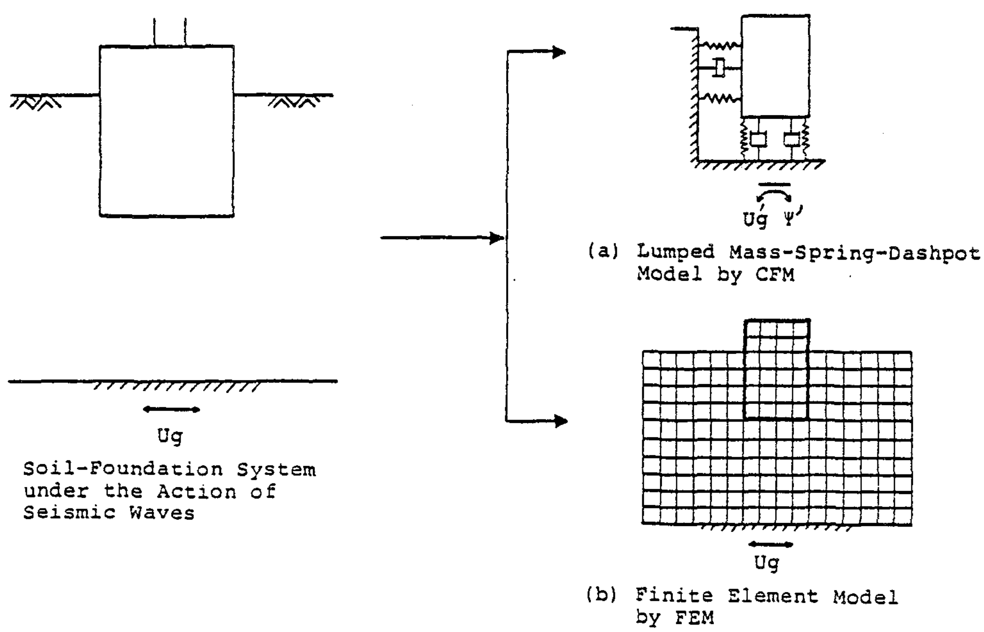


Fig.2 Soil-Foundation Interaction System and Its Models by CFM

USA PAPERS

U-1 TO U-19

AASHTO GUIDE SPECIFICATION
Seismic Design for Highway Bridges

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U-1

The 1971 San Fernando earthquake represented a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971, the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California. In 1973, the California Department of Transportation (CalTrans) introduced new seismic design criteria for bridges that included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. In 1975, AASHTO adopted Interim Specifications which were a slightly modified version of the 1973 CalTrans provisions and made them applicable to all regions of the United States. In addition to these code changes the 1971 San Fernando earthquake also stimulated research activity on the seismic problems related to bridges. By 1977, significant earthquake engineering research studies relating to highway bridges had been completed and the Federal Highway Administration (FHWA) funded a study with the Applied Technology Council (ATC) to develop recommended bridge seismic design guidelines. In 1983, AASHTO adopted the ATC work as a Guide Specification.

Since the seismic risk varies across the United States, specifications were developed for four seismic performance categories (SPC) to which bridges are assigned based on (1) the bridge importance classification and (2) the seismicity of the area in which the site is located. Bridges are classified first according to their relative importance--either as essential or all others. Essential bridges are determined based on their social/survival and security/defense classification. Essential bridges are those that must keep functioning during and after an earthquake. The second aspect of classification covers the differing degrees of complexity and sophistication of seismic analysis and design specified. SPC-D bridges include those designed for the highest level of seismic performance with particular attention to methods of analysis, design, and quality assurance. SPC-C bridges include those where a slightly lower level of seismic performance is required but the potential for damage is slightly greater than SPC-D. SPC-B bridges include those where a lesser level of seismic performance is required and a minimum level of analysis and specific attention to support design details are provided. SPC-A bridges include those where no seismic analysis is required but attention to certain design details for superstructure support is provided.

The primary basis for the development of the specification was to minimize the hazard to life and provide the capability for bridges to survive during and after an earthquake with essential bridges to remain functional. To meet this objective, the following philosophy was adopted.

Small-to-moderate earthquakes should be resisted within the elastic range of the structural components without damage. Realistic seismic ground motion intensities should be used in the design procedure. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridges. And where possible, damage that does occur should be readily detectable and accessible for evaluation and repair.

Two different approaches were combined in the specification to satisfy the above philosophy. They are mainly "force design" and "displacement control" criteria. Minimum requirements are specified for bearing support lengths of girders at abutments, columns, and hinge seats to account for some of the important relative displacement effects that cannot be calculated by current state-of-the-art methods. Member design forces are calculated to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. Design requirements and forces for foundations are intended to minimize damage since most damage that might occur will not be readily detectable.

The methodology varies in complexity as the SPC increases from A to D. For SPC-A bridges the only design requirement is one of providing minimal bearing support lengths for girders at abutments, columns, and expansion joints. Even though the level of seismic risk of these bridges is very small, prevention of superstructure collapse is deemed necessary and hence the requirements. Design for the level of seismic forces in these regions is not considered necessary.

Elastic member forces for SPC-B bridges are determined by a single mode spectral approach. Design forces for each component are obtained by dividing the elastic forces by a reduction factor (R). For connections at abutments, columns and expansion joints, the R-factor is either 0.8 or 1.0 and they are therefore designed for the expected or greater than expected elastic forces. Foundations are also designed for the elastic forces. For columns and piers the R-factor varies between 2 and 5 and they are therefore designed for froces lower than that expected from an elastic analysis and are assumed to yield when subjected to the forces of the design earthquake. Design requirements to ensure reasonable ductility capacity of columns in SPC-B are not specified whereas they are for higher performance category bridges.

For SPC-C and -D bridges the general approach is similar to SPC-B; however, several additional requirements are included. For columns, additional requirements are included to ensure that they are capable of developing reasonable ductility capacity. For connections and foundations alternate design forces to those determined by the procedures of SPC-B are also permitted. These are based on the maximum shears and moments that can be developed by column yielding when the bridge is subjected to the design earthquake forces. Horizontal linkage and tie down requirements at connections are also provided. For SPC-D bridges, settlement slabs are required to reduce the chance of abutment backfill settlement.

Ground motion intensities to be used with the specifications are adapted from seismic risk maps and associated design spectra developed for the

"Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06). The ATC-3-06 maps are based on (1) an appraisal of expected ground motion intensities, (2) the probability that the design ground shaking will be exceeded is approximately the same in all parts of the United States, and (3) frequency of occurrence of earthquakes in various regions of the country. It is possible that the design earthquake ground shaking might be exceeded, although the probability of this happening is quite small.

Ground motion is characterized by the two parameters - Effective Peak Acceleration (EPA), A_a and Effective Peak Velocity-Related Acceleration (EPV), A_v. Although these parameters do not at present have precise definitions in physical terms, they should be considered as normalizing factors for the construction of smoothed elastic response spectra for ground motions of normal duration. The EPA is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 seconds, while the EPV is proportional to spectral ordinates at a period of about 1 second. The constant of proportionality (for a 5 percent damped spectrum) is set at a standard value of 2.5. Thus when the ordinates of a smoothed spectrum between the periods mentioned above are divided by 2.5, the EPA and EPV are obtained. The EPA and EPV thus obtained are related to peak ground acceleration and peak ground velocity but are not necessarily the same or even proportional to peak acceleration and velocity.

The specifications provide for two methods of analysis which vary according to the refinement in the mathematical idealization. They are the single mode spectral analysis method and the multi-mode spectral analysis method. All methods assume simultaneous support excitation. The single mode spectral method requires a calculation of a period. The method is based on the premise that the mode shape of the vibrating structure can be assumed and represented by a shape function which can be expressed mathematically in terms of a single generalized coordinate taken as the amplitude at the point of maximum displacement. It is designed to approximate the dyanamic character of the bridge. The method consists basically of the following steps: 1. Determine the period for the assumed mode; 2. Determine the corresponding seismic coefficient as a function of A_v, structure period, and soil type; 3. Determine the maximum displacement due to the seismic loading; and 4. Determine the component forces corresponding to the maximum displacement.

In summary, there has been a tendency in most parts of the United States to view earthquakes as primarily a California problem. However, more than 70 million people are exposed to moderate or major earthquake risk. The use of AASHTO Guide Specification will lead to the improved seismic resistance of highway bridges in all regions of the United States.

U-2

January, 1984

SEISMIC RESISTANT BRIDGE DESIGN CRITERIA IN CALIFORNIA

by
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Office of Structures Design

INTRODUCTION

The design of seismic resistant bridges in California made a dramatic turn in February, 1971. The heavy damage in San Fernando was unprecedented in the history of bridge design in California (3). The development of a modern seismic bridge design criteria in California was started as a direct result of this earthquake.

This paper will describe some of the important aspects of the current California Department of Transportation seismic bridge criteria, (1,2).

FACTORS CONSIDERED IN THE SEISMIC DESIGN CRITERIA

The force level portion of the criteria was designed to be easily modified as new developments were made in earthquake engineering. Each component represents the independent influence of a different discipline (4,5). The following factors, which affect the seismic forces which go to a structure, are included in the criteria:

A The peak rock acceleration, determined from seismological studies of fault activity and attenuation data gathered from historic events and theoretical studies.

R The acceleration spectra for rock based on actual recorded data and theoretical studies.

S The soil amplification factor, based on both computer studies and actual recorded data.

Z The ductility and risk reduction factor, is based on observed damage plus computed and experimentally determined data. A variable factor to consider risk based on structure period is included in this factor.

The product of the first three factors (ARS) results in an elastic response spectra curve for the site that would approximate the response from a maximum credible event on the closest fault.

Division by the factor Z, after the distribution of the ARS forces, gives a design force for the portion of the structure under consideration. The factor Z is component dependant, thus the design force depends not only on the seismicity and site conditions, but on the structural component being designed.

RECENT CRITERIA DEVELOPMENTS

Many concepts developed for the ATC-6 criteria (8), were significant departures from previous design provisions. Several of these new concepts were incorporated into the current California Department of Transportation seismic bridge criteria (1,2), however California retained the maximum credible seismic force level philosophy developed for previous versions of their criteria (4,5). The primary new components in the California criteria which are based on the ATC-6 criteria are:

- Uncertainty in the direction of loading is accounted for by combining loads in two orthogonal directions by adding 30% of one direction loading to 100% of the loads in the other direction.

- Column spiral reinforcement in plastic hinge zones is specified based on work in New Zealand by the University of Canterbury (6,7). Both the shear and confinement reinforcement design has been revised based on these and recent ACI requirements.

- Bridge bent analysis is performed by the distribution of overstrength plastic moments in the bent.

- The foundations and column connections are required to be designed to carry the plastic moment in the column when it can be shown that the column undergoes plastic action.

FUTURE NEEDS AND DEVELOPMENTS

There is an increasing need for a simple design criteria for small bridges. The small two and three span bridge makes up over 80 percent of the total worldwide inventory of bridges. The response of these small bridges in high seismic areas is greatly influenced by the abutments and simplified design techniques and reliable details could significantly reduce damage.

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NBS Large Scale Seismic Testing Project

U-3

Introduction

A large portion of modern bridge structures constructed in zones of high seismic activity are supported by bents consisting of one or more columns. Nearly all present design codes for bridge column seismic details have had their basis in the extensive research done on building columns. Inelastic response of these bridge structures under seismic loading normally involves plastic hinging of the columns. Behavior is consequently different in concept from that required of building frames, where a design approach is adopted to insure that beams hinge before columns.

Damage to bridge columns in the San Fernando earthquake highlighted the need for reassessment of existing seismic design practice for bridges. Since 1971 column design requirements have been changed and now require additional confinement steel to avoid compression buckling of longitudinal reinforcement and continuous steel at the footings and bent cap to avoid the pullout problem. New colums have not been tested to date, and controversy still exists as to the amount of confining reinforcement required to insure adequate ductility without significant degradation in strength.

In January of 1979 the National Science Foundation funded the Applied Technology Council to conduct a workshop on Earthquake Resistance of Highway Bridges. This group specifically identified the effect of scale on the energy absorbing characteristics of bridge columns as being of critical interest. Besides the issue of hinging ocurring in the column, rather than in a connecting beam as in a building, bridge columns were also identified as being significantly larger and lower stressed. The effect of large scale reinforcement -- # 14 and # 18 bars -- was also of interest. Is large scale transverse reinforcement as efficient as that typically used in building columns, or do differing bond and yield characteristics make large scale structures more vulnerable during a seismic event ? To answer these questions the National Bureau of Standards has undertaken a program to investigate the performance of large scale bridge columns subjected to simulated seismic events.

Research Objectives

The NBS test program will address the effects of scale factor on bridge column design in response to the need to determine whether the behavior of small sections can be extrapolated to large cross sections. Inherent in this program of study will be the assessment of ductility capacity, effect of high axial load, and the effect of moment shear ratio. The program plan will consider columns designed in accordance with current California Department of Transportation specifications, which meet or exceed those outlined in _Seismic Design Guidelines for Highway Bridges_. Specimens at full, 1/3, and 1/6 scale were selected and detailed. The tests have been designed to take advantage of the NBS Large Scale Seismic Testing Facility to obtain "benchmark" data. This facility has been developed to provide full computer control of axial column loads up to 12,000 kips (53.4 MN), lateral loads up to 1500 kips (6.8 MN), and column yield moments of 40,000 kip-ft (54 MN-m). Columns can be as large as eight feet (2.4m) in diameter and 58 feet (17m) in height.

Test Specimens

Single column bents constitute the critical case for seismic loading, since multiple columns can act as moment resisting frames when properly detailed. Since single columns are widely used in seismic regions, the experimental investigations will include only single column bents. Results of the tests will lead to details applicable to both single and multiple column bents since boundary conditions in the critical regions are similar. During transverse seismic loading single column bents are typically restrained against rotation at the tops by only the torsional resistance of the superstructure. However, it is prudent to assume that no such restraint will exist in a strong motion event, particularly if non-monolithic joints are utilized. The optimum general experimental specimen should be of a single cantilevered column design. Two types of cantilevered bridge columns have been designed, based on full-scale 60-inch (1.5m) diameter bridge columns currently is use in the state of California. The basic specimen geometries are as follows:

Type A: Thirty foot (9m) column (high moment/shear ratio) with continuous longitudinal reinforcement through the footing for evaluation of transverse reinforcement performance in the plastic hinge region under varying axial loads.

Type B: Fifteen foot (4.5m) column (low moment/shear ratio) to investigate shear performance of transverse reinforcement.

Each column is cast monolithically with a rectangular base structure measuring 26 feet (8m) long, eight feet (2.5m) wide, nine feet (2.8m) tall, and weighing 380 kips (173,000 kg). The base is supported by eight 600 kip (2.7 MN) roller systems bearing on four hardened rail tracks. A special 4000 kip (18MN) capacity hinged roller mechanism sits atop the column to provide an interface to the vertical load system that minimizes induced lateral loads to the machine. The majority of the lateral load is carried by a torsion resistant tie-back frame which connects the specimen to the upper portion of a heavily post-tensioned 45 foot (14m) high reaction wall. Lateral loads are applied to the base structure using a double acting servo controlled hydraulic actuator, which is also affixed to the reaction wall. The exceptional size of the base structure is required to produce yield moment at the column base before uplift of the base structure.

For both type A and B specimens, axial load will be varied from 0.06 to $0.3f'_cA_g$, and will be held constant via commputer control while other variables are being investigated. Energy absorbtion characteristics will be determined via load deflection hysterisis plots. Lateral loads will first be applied to achieve ten complete reversed cycles at yield deflection, after which the load cycles will be repeated for successively higher lateral deflections until destruction of the column. Plans currently include provisions to retro-fit at least one column after failure to ascertain if retrofit repair of damaged columns, now widely used in some areas, is effective for resisting further seismic loads. Three 1/6 scale model tests are scheduled to be conducted during the spring of 1984. Pending completion of the computer controlled Large Scale Seismic Testing Facility in June of 1984, the first of three initial full-scale specimens will be tested.

U-4

TRANSPORTATION RESEARCH BOARD
BRIDGE ENGINEERING ACTIVITIES
L. F. Spaine, Engineer of Design

The Transportation Research Board (TRB) is a unit of the National Research Council (NRC), the operating agency of the National Academy of Sciences (NAS) and its companion academy, the National Academy of Engineering (NAE). The two academies are private, not for profit, honorary organizations, both operating under a charter enacted by the United States Congress in 1863. The purpose of the academies is to advance science and technology and to act as an official advisor to the federal government and other agencies upon request. The NRC encompasses all of the operating units under both academies.

The TRB is a nonprofit organization supported by contributions, primarily from the state highway and transportation departments, the U.S. Department of Transportation (USDOT) and various other participating organizations and individuals. The specific role of TRB is to identify transportation issues and problems, encourage research and studies to address the problems, provide national forums through conferences and workshops to report research results, undertake research projects when appropriate and record and disseminate useful findings through publication programs. TRB deals strictly with transportation related problems.

There are two separate divisions in TRB in which bridge engineering activities are addressed. These are the Cooperative Research Programs, which encompasses the National Cooperative Highway Research Program (NCHRP), and Regular Technical Activities involving standing committees and task forces. The following paragraphs describe, in a brief way, activities under these two division.

The National Cooperative Highway Research Program is supported on a continuing basis by funds from participating departments of the American Association of State Highway and Transportation Officials (AASHTO), with the full cooperation and support of the Federal Highway Administration (FHWA) of USDOT. The program is administered by TRB. The program does not operate on a grant basis. It deals in applied, contract research totally committed to providing timely solutions for operational problems facing highway and transportation administrators and engineers.

Since its inception in 1962 the NCHRP has included numerous studies of interest to bridge engineers. In response to a growing national awareness of bridge problems the sponsors have referred an increasing number of bridge research projects to the program. In the past four years more than one third of NCHRP funds have been allocated for studies of problems in the area of bridge engineering. The present annual funding for NCHRP is approximately $6.3 million. There are currently over twenty active projects in the bridge engineering area representing total funding of approximately $4.0 million.

Many of the studies referred to the program have been directed to development of improved methods of design and construction, with the ultimate goal of modifying the AASHTO Standard Specifications for Highway Bridges. With an estimated 45 percent, or over 250,000, of the nation's bridges classified as deficient an increasing amount of research has been aimed at structural evaluation, repair and rehabilitation.

Several research emphasis areas are worthy of note. For example, welded connections, used extensively in construction of steel bridges over the past twenty-five years, have developed fatigue problems in some instances. Studies under NCHRP have resulted not only in the development of repair and retrofitting techniques for fatigue susceptible welded connections but also in the complete revision of AASHTO's provisions for consideration of fatigue in design of steel bridges.

Another area of active research over the life of the program has been directed at the problem of bridge deck durability. Deterioration caused by environmental contamination has reached serious proportions, and numerous studies have addressed design, construction, maintenance, materials and protective systems perceived to be associated with or solutions to this problem.

A third area receiving emphasis over the past several years is that of evaluation of repair techniques for damaged steel and concrete members. This problem results primarily from damage due to impact by over-height vehicles. Guidance has been developed to assist the engineer in assessing the severity of the damage and selection of the most appropriate repair techniques.

The bridge research to be initiated this current year will include three new projects. The major thrust will be the development of procedures and equipment for assessment of the load capacity and remaining life of existing bridges. A second project will deal with the distribution of wheel loads on highway bridges with the ultimate aim of providing information for revision of appropriate provisions in the AASHTO Bridge Specifications. The third project will validate analytical procedures to design simple span precast prestressed girders for continuity over piers.

The NCHRP represents a small portion of the bridge research conducted in the United States. It has been a productive program, however, because it has provided operating agencies with an opportunity to pool their funds toward economic and timely solutions for common bridge problems.

The Regular Technical Activities division of TRB performs an entirely different function from that of NCHRP. Here is housed over 170 standing committees and task forces that involve approximately 3,000 individual members from the various transportation technical disciplines. This vast pool of talent provides an opportunity for a mutual exchange of information, identification of problems and solutions and an evaluation of research findings through review of papers and reports. The committees are responsible for developing the TRB Annual Meeting program and for the conduct of seminars, workshops and specialty conferences. An important function of the technical committees is to identify research needs and develop research needs statements.

The TRB Annual Meeting is the single most important activity of TRB. It provides a national and international forum for the presentation and discussion of nearly 1,000 papers and reports. This is accomplished in over 200 technical program sessions and an equal number of committee and task force meetings. Attendance at the five-day meeting exceeds 4,000. There are currently eleven TRB standing committees involved exclusively with design, construction, evaluation and maintenance of bridges and other transportation structures. Fifteen sessions at the 1984 Annual Meeting were devoted to topics under these general categories.

Another important activity of the Regular Technical Activities division is the conduct of workshops and specialty conferences. The most notable contribution in the bridge area was the Bridge Engineering Conference held in St. Louis, Missouri in 1978. This conference produced 68 technical papers, and these were published in Transportation Research Records 664 and 665. Attendance at this conference exceeded 600. A Second Bridge Engineering Conference will be held in Minneapolis, Minnesota, September 24-26, 1984. The program for this conference has been developed and will include 14 technical sessions covering a broad range of papers in the area of bridge design, construction, maintenance, rehabilitation, testing, foundations, hydraulics and computer applications. Proceedings from this conference will be published in the Transportation Research Record series and distributed in September 1984.

In conclusion, another of the TRB's important activities is its publications program. Papers and reports to be published are subjected to a peer review process. Once accepted for publication they are included in one of several report series. Publications released by TRB total about 10,000 pages annually. Due to the diversity in topic matter the reports are classified according to one or more of 29 subject areas and distributed to sponsors, affiliated members and subscribers through a selective distribution system. They are also available for purchase on an individual basis.

BOX GIRDER BRIDGE HINGE RESTRAINER TEST PROGRAM

Lawrence G. Selna (1) U-5

INTRODUCTION

Background

Post-earthquake investigations of box girder highway bridges after the 1971 San Fernando Earthquake showed that the decks pulled apart at the hinge and bearing seat locations. In 1980, two box girder spans of the Fields Landing Overhead near Eureka, California dropped when the hinge seats opened during the Trinidad-Offshore Earthquake.

By the end of 1982 a significant amount of bridge strengthening or retrofitting had been performed by the State of California, Department of Transportation (CALTRANS). The unrestrained joint seats represent the prime focus of the CALTRANS retrofit program. Six hundred ninety bridges out of 1240 identified as deficient had joint restrainers installed by that time.

Objective

The objective of the present paper is to discuss the box girder bridge restrainer test program. The development of the testing facilities, equipment, and specimens at UCLA is emphasized.

FACILITIES AND EQUIPMENT

Reaction Frame

A reinforced concrete reaction or loading frame composed of a test bed and two reaction blocks is capable of resisting 1500 kips (6666 kN) and is depicted in Figs. 1 and 2. A bridge specimen's anchor bars are shown protruding through the west reaction block of the loading frame in Fig. 3. Alternating tension-compression forces up to 1500 kips (6666 kN) can be applied to full scale bridge components. The 4 ft (1.22 m) thick reaction blocks have numerous 3 in. (7.6 cm) diameter pipe passages used for tension anchors which restrain the actuators and components to be tested. The 32 ft (9.75 m) clear distance between the reaction blocks permit static and dynamic structural testing of components up to 22 ft (6.71 m) in length.

Actuators and Controllers

One 18 in (45.7 cm) diameter by 12 in (30.5 cm) stroke actuator and two 12 in (30.5 cm) diameter by 24 in (61.0 cm) stroke actuators are used in the testing. They are controlled by Moog servovalves and controllers. A 5 HP pump drives the system with a maximum speed of 1 in/min (2.54 cm/min).

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SPECIMENS

Bridge Test Components

The dimensions used in old construction are the same for most bridges. Only the cross section depth is changed for different span lengths. The thicknesses used in the test components are as follows: 1) webs or stems - 12 in (30.5 cm); 2) hinge diaphragm - 10 in and 25 in (25.4 cm and 63.5 cm); 3) bolster - 9 in (22.9 cm); 4) soffit slab - 5.5 in (14.0 cm); and 5) deck slab - 7.5 in (19.1 cm). The bridge component length dimension on each side of the hinge is 10 ft (3.05 m) making a total length of bridge that is represented equal to 20 ft (6.10 m). The bridge component width used in the test is 10 ft (3.05 m) which is equivalent to the distance between webs or stems of an actual bridge. The height of the test component is 4 ft (1.22 m). All except the height dimension are standardized box girder bridge dimensions.

Grade 60 reinforcement is used in the tested components. The reinforcement configuration is shown in Fig. 4. For the cable restrainer test 20 - #14 bars are used to anchor the west bridge component to the reaction block (Fig. 3). The anchor bars are necessary because of the projected 750 kip (3,333 kN) cable strength. A similar number is used to join the east bridge test component to the actuators (Figs. 1 and 2). The large anchor bars are spread outward to the stems in the hinge region so that only the standard reinforcement of the hinge diaphragm and bolster remain at that location (Fig. 4). Therefore an accurate experimental representation of the hinge diaphragm is obtained.

Type C1 Hinge Restrainer

The Type C1 Hinge Restrainer consists of 7 - 3/4 in (1.91 cm) ϕ cables which lash the hinge diaphragms together (Figs. 1 and 2). The seven cables are anchored on the face of the west bolster, pass through both hinge diaphragms, wrap around a drum located on the face of the east bolster, pass back through the second hole, and are anchored again on the west bolster. The cables are swaged to 1 in (2.54 cm) studs which pass through a plate for the anchorage.

CONCLUSION

The testing program for reinforced concrete box girder hinge restrainers is underway in the Department of Civil Engineering at the University of California, Los Angeles. Full scale box girder bridge test components and prototype restrainers used in the CALTRANS earthquake retrofit program are subjected to cyclic loadings which represent seismic inputs.

ACKNOWLEDGMENTS

Funding was provided by National Science Foundation Grant CEE-8207543 and State of California, Department of Transportation Research Technical Agreement Number RTA 13945.

Technical guidance for the work was given freely by CALTRANS bridge engineers Oris Degenkolb, Ray Zelinski, and James Gates of the Office of Structures Design.

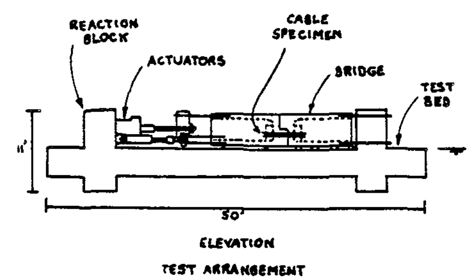

Fig. 1 - Elevation View of Bridge and Reaction Frame

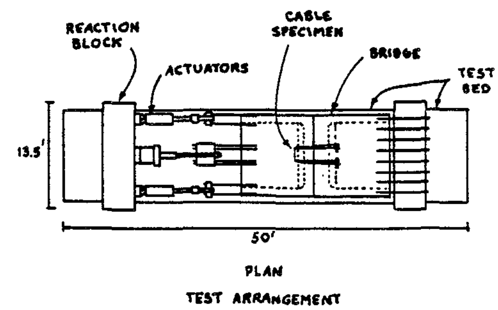

Fig. 2 - Plan View of Bridge and Reaction Frame

Fig. 3 - Bridge Component Installed in Reaction Frame

Fig. 4 - Reinforcement for Box Girder Hinge Component

IMPLEMENTATION OF THE ANALYTICAL CAPABILITIES REQUIRED FOR THE ASEISMIC DESIGN OF BRIDGES

by

R. A. Imbsen

U-6

Both the current AASHTO (American Association of State Highway and Transportation Officials) "Standard Specifications for Highway Bridges" (1), which was upgraded following the 1971 San Fernando Earthquake, and the more recently adopted AASHTO guide specification, "Seismic Design Guidelines for Highway Bridges" (2), require that a single-mode or multi-mode response spectrum analysis be conducted in the seismic design of bridges to be located in the higher seismic zones. Because the analytical procedures involved in seismic analyses are new to many bridge designers, it has been difficult to implement these new methodologies within the United States. Recognizing this problem, the National Science Foundation elected to fund a project to develop the computer program, SEISAB (SEIsmic Analysis of Bridges), and conduct pilot workshops to aid in this implementation effort.

In addition to being used as a design tool to facilitate the implementation of the new design codes, SEISAB is also being extended to bring to the profession the nonlinear capabilities that were developed at the University of California, Berkeley, as part of an investigation entitled "An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances" (3). These nonlinear capabilities of SEISAB are being designed for use by the researcher or bridge designer involved in the following design-related activities:

- Conducting parametric studies to establish procedures and design coefficients for new or improved aseismic design specifications
- Conducting detailed dynamic analyses studies on complex bridges
- Investigating newly developed aseismic design strategies that include energy dissipation
- Developing design procedures that include the complex effects of soil-structure interaction.

Extending SEISAB to include both newly-developed elements unique to bridges and nonlinear analysis capabilities provides a vehicle for implementing the state-of-the-art methodologies emerging from the universities into the bridge engineering profession.

In line with the primary objective of developing a usable design tool, SEISAB-I was developed with an effective means of user communication by incorporating a problem-oriented language written specifically for the bridge engineer. The free-format SEISAB language consists of simple, easy-to-remember commands, natural to the bridge engineer in describing a bridge. Using a minimum amount of user input data, the program completely generates a mathematical model. SEISAB-I, which contains linear dynamic analysis capabilities, was well received in its initial pilot workshop, presented to a selected group of highly qualified bridge engineers from the California Department of Transportation. Three subsequent workshops that included the use of SEISAB-I for both the design and the retrofitting of bridges were equally successful.

In regions of high seismicity it has generally been economically unfeasible to design and build bridges that resist earthquake loads elastically. Thus, in order to achieve acceptable performance, designers have relied on the post-elastic behavior of certain components. This has generally meant that columns or piers could be expected to yield during a major earthquake. This design strategy also requires that other nonductile components, such as bearings, be designed to resist seismic forces elastically.

Recently, though, there has been a growing interest in using different design and retrofitting strategies in regions of high seismicity. These strategies, which utilize concepts such as isolation, energy dissipation and/or restraint, often employ special bearing devices designed to behave nonlinearly during a major earthquake. However, many of these design strategies are relatively new and lack performance histories during an earthquake. In addition, the effect of these strategies cannot be adequately evaluated by experimental research which investigates only the performance of isolated components. Because full-scale testing of prototype designs is very expensive, such testing is usually economically unfeasible. Therefore, analytical techniques must be relied on if these new aseismic design strategies are to be properly evaluated. In many cases, the analytical methods currently used to evaluate these strategies are based on simple, single-degree-of-freedom idealizations of a given bridge. However, the geometry and articulation of most bridges makes the validity of such simple idealizations questionable, especially in view of the fact that there are other nonlinear components in the bridge. To properly evaluate these new strategies and identify those situations where they will be the most beneficial, bridge designers must be able to realistically analyze the true non-linear behavior of various types of bridge structures employing many different design and retrofitting strategies.

Nonlinear dynamic-analysis capabilities would also facilitate much of the research recommended in recent workshops on the seismic aspects of bridge design. The objectives of much of this recommended research could be accomplished more efficiently if such analytical capabilities were readily available in a form that practicing bridge engineers could use.

The nonlinear program, SEISAB-II (SEISmic Analysis of Bridges-II), will consider, along with the nonlinear behavior of bridge bearings, the effects of column flexural yielding and the formation of plastic hinges. An efficient method for considering column yielding in a finite-element computer program is to use nonlinear beam elements in which flexural yielding can occur at the ends of each element. An axial bending load and biaxial moment interaction yield surface can be described by using the conventional ultimate strength, reinforced concrete theory. By assuming a transition from ideally elastic behavior to ideally plastic behavior at the yield surface, engineers can write (and have written) algorithms that include the nonlinear behavior of reinforced bridge columns (4, 5, 6).

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SEISMIC RESPONSE OF MELOLAND ROAD OVERPASS

By
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U-7

INTRODUCTION

The Meloland Road Overpass (MRO), located east of El Centro, California, is a reinforced concrete bridge comprised of two continuous 104-ft spans with monolithic abutments and a central single-column pier. Because of its close proximity to the Imperial Fault (0.5 km away), the MRO was instrumented with an array of 26 strong motion accelerographs (Fig. 1). These instruments were triggered during the 1979 Imperial Valley earthquake, providing the most extensive set of seismic response measurements yet obtained for bridges in the United States. It is noted that the MRO was virtually undamaged by this earthquake, despite the intense shaking to which it was subjected (0.51 g peak acceleration).

A project has been implemented which uses the strong motion measurements from the MRO to evaluate its seismic response characteristics during the Imperial Valley earthquake. The project has had 3 main tasks: (1) strong motion data reprocessing to circumvent recorder stall and nonsynchronization difficulties; (2) extraction of normal mode and pseudostatic response parameters from the reprocessed data; and (3) identification and application of an optimized finite element model of the MRO. This paper summarizes results from Tasks 2 and 3; Task 1 procedures and results are described elsewhere (Ref. 1).

NORMAL MODE AND PSEUDOSTATIC RESPONSE PARAMETER ESTIMATION

Methodology. It is well known that the response of an elastic structure subjected to different input motions at each of several support points can be fully characterized in terms of its pseudostatic influence matrix and fixed-base normal modes (e.g., Ref. 2). To estimate these parameters for the MRO from its measured seismic response, a new methodology (PAREST) has been developed that is applicable to structures with any number of multiple input and response measurements. PAREST applies optimization techniques to the structure's pseudostatic and modal equations of motion to obtain parameters that minimize the square of the difference between the measured response and the estimated response. This minimization process uses a modal sweep algorithm that has been shown to be numerically efficient and to have reliable convergence properties (Ref. 3).

Application. PAREST has been applied to the two cases shown in Table 1. Results are presented here only for Case 1, in which the input motions are the motions measured at the two abutments and at the base of the central pier, and the response corresponds to the motions measured along the road deck. The normal modes for this case are those of the

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bridge superstructure, whereas the effects of the abutments and surrounding soil medium are characterized through the pseudostatic influence matrix. The parameter estimation results for this case are summarized as follows: (1) the only two modes excited significantly by the ground shaking are a transverse response mode and a vertical response mode, which are both symmetric about the midspan of the bridge (Fig. 2a); (2) the estimated pseudostatic and modal response parameters provide an excellent fit to the measured response of the MRO (Fig. 2b); and (3) the pseudostatic response parameters contribute substantially to the bridge's transverse response, but not to its vertical response.

FINITE ELEMENT MODEL IDENTIFICATION

Methodology. A new methodology (IDENT) has been developed to identify a three-dimensional equivalent-linear finite element model of the MRO that best fits the strong motion data. IDENT employs quasi-Newton type optimization techniques to obtain model parameters that minimize the weighted mean-square difference between the measured bridge response and the model response. In this, the bridge response can be represented using motion histories at the accelerograph locations, normal mode parameters, or any other appropriate quantities.

Application. IDENT has been used to identify MRO finite element models that best match the Case 1 and Case 2 normal modes obtained from PAREST; results are summarized here for Case 1 only. Under this case, the following parameters have been optimized: (1) Young's modulus of the concrete; (2) moments of inertia of the road deck in bending (two directions) and in torsion; and (3) bending moment of inertia of the central pier. The Case 1 results of this process indicate that: (1) the optimized section properties of the road deck are close to their gross values; and (2) the column's optimized section properties are dependent on its restraint by the underlying soil medium, and therefore must be identified from the IDENT application to the Case 2 modes in which the soil springs at the bridge supports are optimized together with the column's section properties. Applications of the resulting Case 1 model to estimate dynamic states of stress in the bridge indicate low stress levels in the road deck, but possibly higher short-time stresses in the column.

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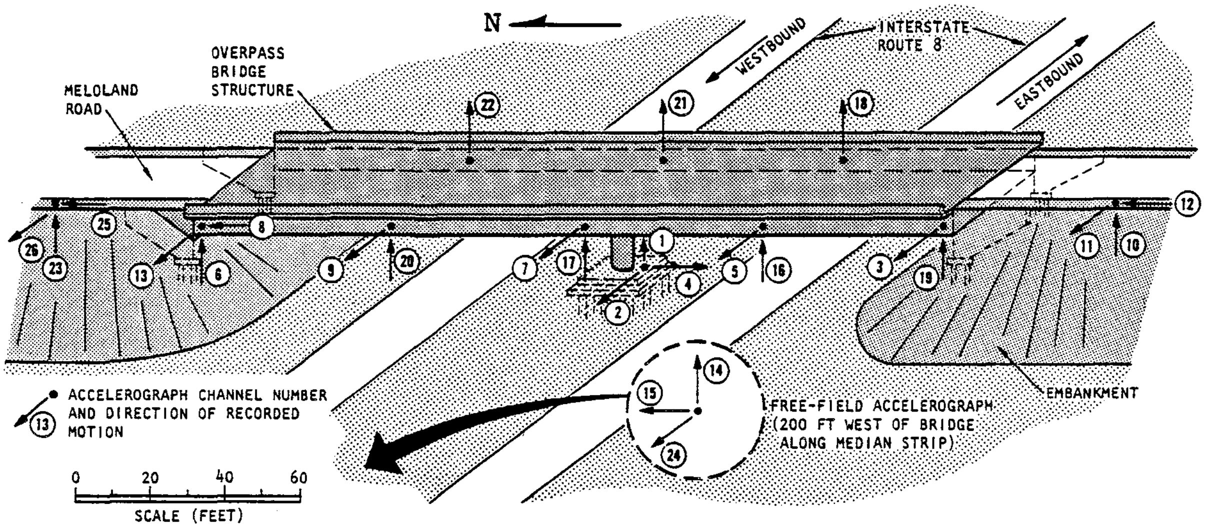
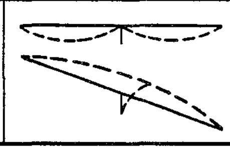


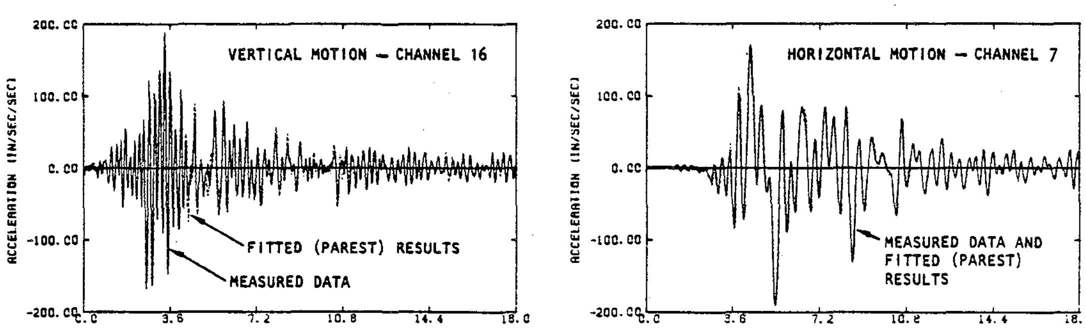
FIGURE 1. MELOLAND ROAD OVERPASS

Case No.	Input Channels	Response Channels
1	At abutments (Ch 3, 19, 6, 13) and at base of central pier (Ch 1, 2)	Along road deck (Ch 5, 7, 9, 16, 17, 18, 20, 21, 22)
2	At embankments (Ch 10, 11, 23, 26) and at base of central pier (Ch 1, 2)	At abutments (Ch 3, 19, 6, 13) and along road deck (Ch 5, 7, 9, 16, 17, 18, 20, 21, 22)

TABLE 1. DATA EVALUATION CASES

MODE	FREQUENCY (Hz)	DAMPING RATIO	MODE SHAPE
VERTICAL-SYMMETRIC	4.74	0.066	
TRANSVERSE-SYMMETRIC	3.72	0.066	

a) Modes excited by ground shaking



b) Measurements vs. fitted results

FIGURE 2. CASE 1 PAREST RESULTS

BRIDGE FOUNDATION PROBLEMS

RICHARD W. ARDEN [1]

U-8

Our firm has designed bridges; however, our emphasis is on the geotechnical evaluation of bridge foundations. These bridges were all designed under the current AASHTO code prior to the advent of the newly adopted seismic design guidelines (1).

The support for all the bridge foundations we have investigated have been spread footing, pile or cast-in-place concrete. The first choice is a spread footing; however, if the shallow soils do not have adequate bearing capacity then alternative foundation support has to be investigated. Depending upon soil type, location of ground water, loads and anticipated driving problems, timber, steel H, cast-in-place concrete, pile bents or single pile column extension are some possible solutions to the foundation problem.

The analysis of the vertical load is readily available to the designer. An important point which needs to be made relative to bridge foundation design is that firms such as ours especially those in states of moderate to low seismicity, will need guidance in the form of continuing education regarding the implications of the new seismic design guidelines (1).

Upon reviewing the foundation section of these guidelines for bridges, it is clear as a practicing consulting engineer, that the guidelines are somewhat vague concerning how to deal with the foundation problem. From the full scale experimental work being conducted (3) we know that the soil-structure interaction phenomenon can contribute very significantly to the way seismic loads are distributed. Yet from our perspective, we are not aware of reasonably simple reliable methods for predicting the foundation stiffnesses and strengths required by the bridge designer for seismic design. This strikes us as an area which requires much additional research.

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In fact, I would like to give another example related to bridge pile foundations in which more research is needed to clarify the apparently simple problem of driving H piles. One of the main concerns with foundation is to make sure the piles have been driven to the proper depth to provide the design load. There have been many attempts to determine the bearing capacity of piles from their driving record. Although a design length is specified it is always correlated to the driving record during construction to make sure proper bearing capacity has been obtained. ASCE Committee on the Bearing Value of Pile Foundations (2) had two views which were presented as Reports A and B.Report A recognizes the many limitations of pile driving formulas but indicates that the rational use of the formula can be used in practice if proper judgement is used. Report B made the following statement: "All dynamic pile-driving formula is nothing more than a yardstick to help the engineer secure reasonably safe and uniform results over the entire job. The use of complicated formula is not recommended since such formulas has no greater claim to accuracy than the more simple ones." The two reports indicate the need for the field engineer to have a good understanding of the limitations of the pile driving formulas. The present state of the art requires as much experience with pile driving as on methods of design and use of pile driving formulas.

Each type of pile has different driving characteristics. Pile driving formulas cannot be relied on in soils that could be subject to liquefaction. Saturated sands tend to lose their shear strength during driving of certain types of piles. Under these conditions if the field engineer uses the pile driving formulas as the only criteria for bearing capacity, the pile lengths may greatly exceed the design length.

The example case relates to a bridge which was constructed with steel H piles as foundation support. The subsurface investigation revealed that the underlying soils were saturated poorly graded sands. The relative density increased with depth, so that liquefaction was not an apparent problem.Based on the subsurface investigation, it was estimated that the steel H piles would support 60 tons with a length of 40 to 50 feet. However, during construction, the field inspector used the ENGINEERING NEW RECORD driving formula to determine bearing capacity. After driving the pile for more than 90 feet, the 60 ton capacity was not achieved using the formula. What may have happened was that during the driving of the steel H pile the saturated sand liquefied in a localized area around the pile which allowed the pile to be driven without much resistance. The next pile was driven its design length, approximately 50 feet and test loaded the next day to twice the design load. The load test verified the design length and the remaining piles were driven to the design depth.

The load carrying capacity of the pile was achieved because the sand regained its strength after driving was completed and the pile was allowed to rest. In this case the design vertical load was verified by load testing; however, these are expensive tests. As a practicing engineer, if a better correlation between design value and pile driving record for saturated sands could be obtained more economical foundations could be achieved without load testing.

I do not want to give the impression that driving formulas cannot be used; however, the point needs to be made that under certain conditions a good deal of judgement has to be used.

Soils are as much part of the structural design as the superstructure of the bridge. During a seismic event, the interaction of the foundation and the superstructure play an important part in evaluating how the structure will react.

The design profession needs assistance in the form of research and continuing education opportunities to better understand the implications of the Seismic Design Guidelines (1).

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FULL-SCALE DYNAMIC TESTINGS AND EARTHQUAKE RESPONSE CHARACTERISTICS OF SUSPENSION BRIDGES

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U-9

1. INTRODUCTION

This overview paper summarizes two phases of extensive research that was concerned with: (1) full-scale ambient vibration measurements of suspension bridges, and (2) the earthquake response analysis of these long-span structures when subjected to multiple-support seismic excitations.

2. FULL-SCALE AMBIENT VIBRATION MEASUREMENTS

Performing tests on full-scale structures is the only sure way of assessing the reliability of the various assumptions employed in formulating mathematical or finite-element models of structures. It is also a very reliable way of determining the parameters of major interest in structural dynamic, wind, and earthquake engineering problems.

Extensive experimental investigations were conducted on the Golden Gate Bridge (San Francisco, California, USA) to determine, using ambient vibration data, the mode shapes, the associated frequencies, and the damping of the bridge vibration. The ambient vibration tests involved the simultaneous measurement of vertical, lateral, and longitudinal vibrations of the suspended span (Figs. 1 and 2) and the measurement of longitudinal and lateral vibration of the main tower (Fig. 3). During each testing session of the suspended span, six accelerometers were mounted at one of the stations (1-18) indicated in Fig. 1. The positions and orientations of these six accelerometers at a typical station are shown in Fig. 2. Six reference accelerometers were located at the cross section indicated by R (Station 8 in Fig. 1), where they remained throughout the tests on both spans. Summing the outputs of accelerometers C and E (Fig. 2) gives the purely vertical motion while subtracting their outputs gives the torsional motion. Similarly, summing the outputs of accelerometers A and B gives the purely lateral motion while subtracting their outputs provides information on the torsional motion. Purely longitudinal tower vibration was obtained by summing the outputs of accelerometers A and C (Fig. 3) while torsional motion was obtained by subtracting their outputs. A total of 91 modal frequencies and mode shapes of the suspended span and a total of 46 modal frequencies and modes of vibration of the tower were determined. Comparison with previously computed mode shapes and frequencies showed good agreement with the experimental results. For more details on this experimental research see Ref. 1.

3. EARTHQUAKE RESPONSE OF LONG-SPAN SUSPENSION BRIDGES

Both theoretical dynamic analyses and full-scale ambient vibration tests of suspension bridges have indicated that modes of vibration of the structure can be separated into two groups. In one group, the displacements of the stiffening structures and cables are predominant, and in the other group, the displacements of the towers are predominant. Consequently, the multiple-support seismic response of the suspension bridge cable-suspended structure was investigated for vertical, torsional, and lateral vibrations separately from the longitudinal vibration investigation of the tower-pier system. Calculations were performed in both the time and frequency domains, and these methods were compared for consistency. Ground motion inputs were taken from existing ground motion records recorded at time-synchronized closely-spaced stations. In addition, artificially generated earthquake records were used for comparison purposes. The behavior of three bridges, chosen to represent the whole spectrum of possible suspension bridge dimensions, was investigated in terms of their response displacements, stresses, and shear forces to multiple-support seismic excitations.

It was found that a relatively large number of modes, closely-spaced in the frequency domain, participate in the earthquake response of a suspension bridge. Uniform ground motion for such a long-span structure is not a good assumption since it results in nonconservative responses. The vibrational stresses induced in the cable-suspended structure under multiple-support seismic excitation are significant live loads and may come close to or exceed design yield stresses. Also, the additional cable tensions experienced by the bridge under earthquake excitation are significant live load contributions.

In the analysis of longitudinal tower-pier vibration, the effect of the soil flexibility underlying and surrounding the pier upon the mode shapes and natural frequencies as well as the response displacements, stresses, and shear forces is very important. Thus the estimation of soil properties underlying the foundation is essential in design. The response stresses in the tower-pier system under earthquake excitation are significant but are still below their yield values.

Finally, it should be emphasized that assurance of the aerodynamic stability of a suspension bridge does not in any way imply the safety of these structures during earthquake loading. Both the inputs and the responses, as well as the possible modes of failure, are different for the two kinds of excitation. A multiple-support analysis methodology is essential in the earthquake resistant design of such a long-span structure. More details on this phase of research can be found in Ref. 2.

ACKNOWLEDGMENT

The two phases of this research were supported by grants from the National Science Foundation and the U.S. Department of Transportation; this support is greatly appreciated. The author is also grateful to the people of the Golden Gate Bridge, Highway and Transportation District for their assistance.

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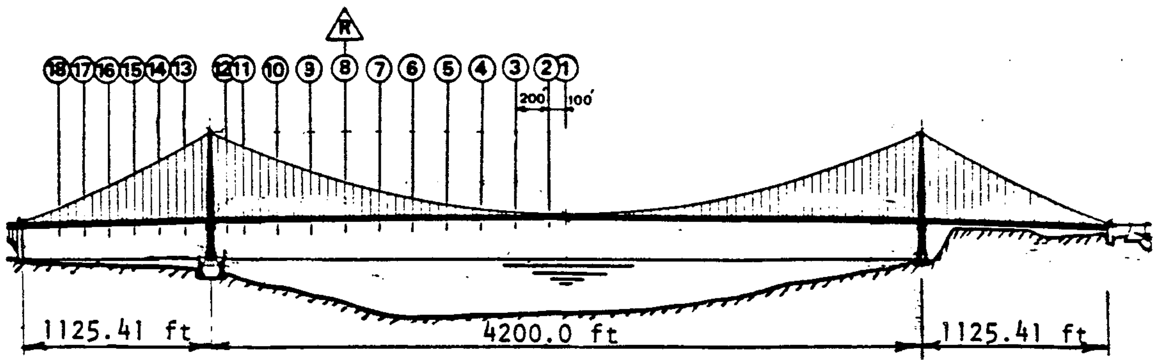


Fig. 1 **The Golden Gate Suspension Bridge: Measurement Stations**

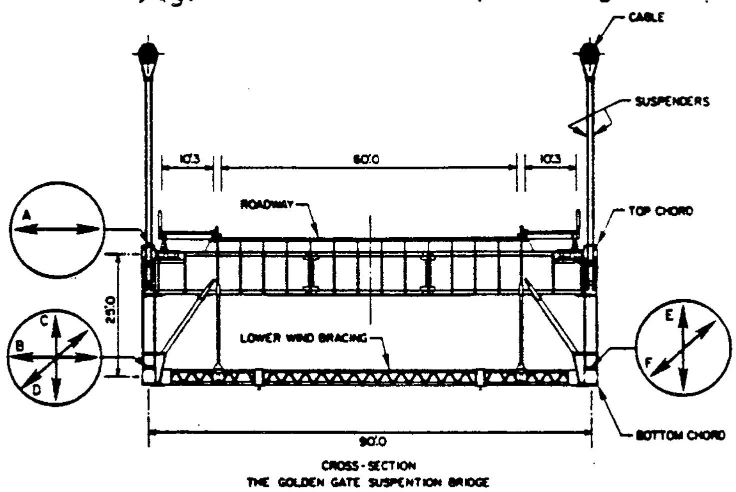


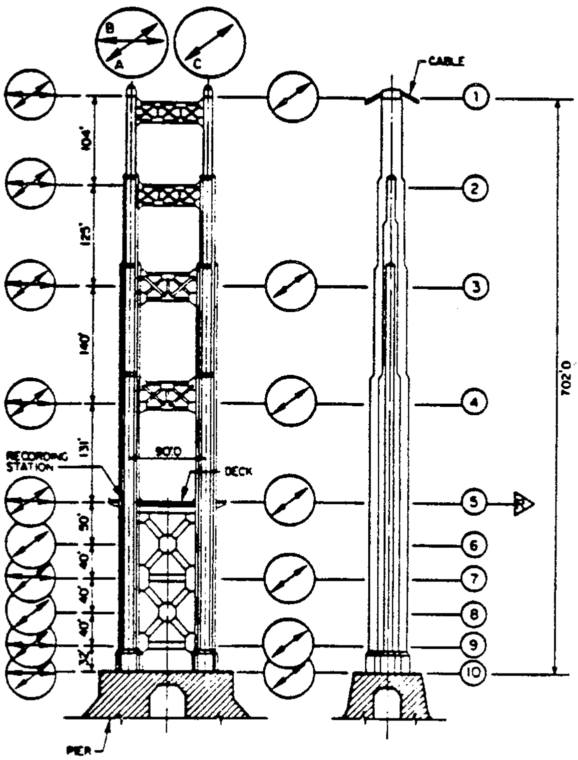
Fig.2 Deployment of accelerometers on typical cross section.

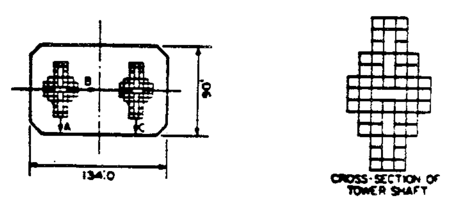
A,B: Lateral motion

C,E: Vertical motion

D,F:Longitudinal motion

THE RECORDED MOTION AT DIFFERENT POINTS OF A TYPICAL CROSS-SECTION





MEASUREMENT STATIONS OF THE GOLDEN GATE BRIDGE TOWER

Fig. 3 Measurements of the tower

A,C: Longitudinal motion

B : Transverse (lateral) motion

U-10

AASHTO AND BRIDGE EARTHQUAKE ENGINEERING

Jack Freidenrich

My interest in bridge earthquake engineering is prompted primarily as a result of my chairing the American Association of State Highway and Transportation Officials Subcommittee on Bridges and Structures.

The American Association of State Highway and Transportation Officials (AASHTO) is an association whose members are the officers of the 50 states, Puerto Rico and the District of Columbia Departments in which the official highway responsibility is lodged as well as the United States Department of Transportation. The stated purpose for which the association is organized is to foster the development, operation and maintenance of a nationwide integrated transportation system and to cooperate with other appropriate agencies in considering matters of mutual interest in serving the public need. To this end, the Member Departments have pledged their cooperation to develop and improve methods of administration, planning, research, design, construction, maintenance and operation of facilities to provide the efficient and effective transportation of persons and goods in support of national goals and objectives.

The AASHTO Committee structure consists of a Policy Committee, an Executive Committee, various Standing Committees, each of which may have several Subcommittees. The Subcommittee on Bridges and Structures, whose membership includes the Chief Bridge Engineer of each of the Member Departments, reports to the Standing Committee on Highways.

The Bridge Committee meets each year in a series of Regional Meetings to conduct the business of the Committee. The stated charge to the Subcommittee on Bridges and Structures is to develop and keep current all major engineering standards, specifications and problems pertaining to the methods and procedures of bridge and structural design, fabrication, erection and maintenance including geometric standards and aesthetics as appropriate for such structures; to make recommendations for testing and investigating existing and new materials of construction and to determine areas of needed study and research in the area of bridge engineering; and to develop and maintain such standards and procedures as are appropriate for rating and evaluating existing bridges in service. Each Regional Meeting consists of two days of intensive technical discussions and presentations, and each meeting utilizes an identical agenda which historically has consisted of between 40 and 60 items. Each agenda item deals with a specific bridge design specification or other matter which bears directly on the bridge design criteria utilized by all of the Member Departments Chief Bridge Engineers.

As a result of the Regional Meetings, a consensus opinion is developed which yields a series of ballot items for formal consideration by each of the Chief Bridge Engineers. Generally speaking, each ballot item will place before the Bridge Committee for their consideration, the addition, deletion or modification of a specific bridge design specification. Any item which receives the approval of at least 2/3 of the Committee Members triggers the annual promulgation of an interim specification which subsequently becomes a part of the "Standard Specifications for Highway Bridges."

An agenda item for the 1982 Bridge Committee Regional Meetings provided for the discussion of "Seismic Design Guidelines for Highway Bridges." This document was prepared by the Applied Technology Council, Berkeley, California, for the Federal Highway Administration, Office of Research. The guidelines were the recommendations of a team of nationally recognized experts which included consulting engineers, academicians, state highway and federal agency representatives from throughout the United States.

Subsequently, as a result of the balloting process described above, the Bridge Committee formally approved "Seismic Design Guidelines for Highway Bridges" for publication as a guide specification.

The agenda for the 1983 Bridge Committee Regional Meetings provided for a presentation by representatives of the Applied Technology Council on a study entitled, "Seismic Retrofitting Guidelines for Highway Bridges." While this presentation generated much interest and discussion, the Bridge Committee took no further formal action concerning the matter at that time.

The AASHTO Subcommittee on Bridges and Structures remains extremely interested in establishing design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes, and will welcome the results of the National Science Foundation project, "Workshop to Review and Identify the Earthquake Engineering Research Needs for Bridges and to Identify the Required Experimental Facilities."

"SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES" - A BRIEF OVERVIEW CONCERNING THEIR USE

by

Jim Dodson

U-11

In 1982, the American Association of State Highway and Transportation Officials (AASHTO) adopted Report No. FHWA/RD-81/081, "Seismic Design Guidelines for Highway Bridges", as a guide specification. The purpose of this paper is to report on the extent of use of the guide specifications and to identify any needs stemming from problems associated with their use. Findings reported are based on informal interviews conducted with bridge engineers of seven state highway departments and two private consulting firms which have done bridge design work for state highway departments. States interviewed included some of which would have occasion to design for all seismic performance categories.

Most of the states interviewed had some experience using the guidelines, although two did not. Reasons given for not using the guidelines were generally expressed as a lack of time and manpower to properly evaluate their incorporation into the state's design practice. Some states were using the guidelines for the less rigorous requirements of seismic performance categories A and B but had not employed their use for bridges requiring category C or D design.

Of the states that had used the guidelines, the general impression given was that they were well presented. This is not to say that there were not problems associated with their use. As might be expected, the problems reported concerned mainly the more complex design procedures of seismic performance categories C and D.

One of the problems mentioned was that some states felt they did not have computer software to adequately perform the required analysis called for in the guidelines. Difficulties reported in this area varied. Some states do not have computer programs available with space frame analysis capability. One state reported that it had a program of this nature available, but had not attempted to use it. Another state has to employ the use of an outside consultant when a space frame analysis is necessary.

Another problem that surfaced during the interviews might be described as a general hesitancy among the states to become involved in performing a multimode spectral analysis. This is called for in the guidelines for certain types of bridges in certain performance categories. The analysis must be performed with a suitable linear dynamic analysis computer program. Of the states contacted, there was very little use of this analysis procedure reported. Partial attribution for this lies in the fact that several of the states contacted are in regions where an analysis of this type would not be necessary to fulfill the minimum requirements of the guidelines. In those states where this was not the case, the expertise to properly model the structure and insufficient computer software were generally given as reasons for not performing a multimode spectural analysis. Some confusion was also manifested as to when a dynamic analysis is necessary. The guidelines require multimode spectural analysis for bridges in seismic performance categories C and D only if the bridge is defined as "irregular". The problem felt by some is that the distinction made in the guidelines between a "regular" and an "irregular" structure is somewhat vague.

Other problems mentioned dealt with interpretation of certain specific aspects of the guidelines. Engineers from the states that mentioned these problems in initial application had discussed the problems with individuals responsible for the development of the guidelines and had received sufficient clarification to complete their analyses.

The design procedures presented in the guidelines appear to be a rather radical departure from those used by AASHTO prior to their introduction. Problems arising in application may be further compounded by the fact that earthquake engineering in itself requires bridge engineers in many areas to deal with design aspects which they may have little or no training. To insure that bridge engineers are properly trained and are correctly applying the guidelines is an issue of obvious importance. The recent completion of guidelines for seismic retrofitting of existing structures will invoke further demand for educational programs for bridge engineers.

HIGHWAY BRIDGE DESIGN SPECIFICATIONS

FOR SEISMIC LOADS IN U.S.

VELDO M. GOINS, P.E.

U-12

As Bridge Engineer for the Oklahoma Department of Transportation, my interest in Earthquake Engineering is primarily as it relates to bridges and the "AASHTO Standard Specifications for Highway Bridges", the bridge design code routinely used in the United States. The "Subcommittee on Bridges and Structures" under the "AASHTO Highway Committee", is formed by participation of the engineer in charge of bridge design from each of the fifty states, and is responsible for the writing and annual updating of these specifications. There are eighteen technical committees formed to deal specifically with the main subjects of bridge design. One of these technical committees is the "Technical Committee for Loads and Load Distribution" of which I have acted as Chairman for the past twelve years. The responsibility for the Earthquake Engineering Specification falls mainly under this Technical Committee.

As late as 1974, the section of the "AASHTO Standard Specifications for Highway Bridges" dealing with earthquake stresses was covered in a very brief five line paragraph with the formula $E.Q = C \cdot D$ where C is a constant and D is the deadload of the structure. The San Fernando earthquake in the state of California in 1971 caused significant loss of life and severe damage and collapse of many bridges in that state. An in-house study conducted by the California Department of Transportation pointed up the deficiencies in the earthquake engineering design procedures that were being used at that time. In 1973, based on the best available research, Caltrans adopted a new code "California Earthquake Design Criteria" which was later approved by the "AASHTO Subcommittee on Bridges and Structures" and included in the "AASHTO Interim Specifications Bridges 1975." This criteria was specifically developed for California but was adapted for use in all areas of the United States. The adoption by AASHTO was intended to be a temporary measure until a more comprehensive criteria could be developed for use in all states taking into account the complete range of earthquake probability from the most to the least severe areas.

In 1976, a research proposal by the Applied Technology Council (ATC-6) was funded by the Federal Highway Administration to develop new and improved seismic design guidelines for highway bridges applicable to all regions of the United States. A project engineering panel, was formed to guide the development of the criteria which included AASHTO representatives from four state bridge departments, four private design firms, three university researchers, two FHWA engineers and three ATC representatives. Also consulting engineers were used as needed for specific technical phases of the project. Three principles used for the development of the criteria were: 1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. 2. Realistic seismic ground motion intensities and forces are used in the design procedures. 3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge, and damage that does occur should be readily detectable and accessible for repair. A basic premise for the development of these guidelines was that they be applicable to all parts of the United States because the seismic risk varies from very small to high across the country. Therefore, for purposes of design, four Seismic Performance Categories (SPC) A, B, C, and D were defined on the basis of an Acceleration Coefficient (A) for the site, determined from the map provided in the criteria and the Importance Classification (I.C.)

Different degrees of complexity and sophistication of seismic analysis and design are specified for the four seismic performance categories. A detailed seismic analysis is not required for single span bridges; however, the connections between the bridge span and the abutments are designed both longitudinally and transversely to resist the gravity reaction force at the abutment multiplied by the Acceleration Coefficient of the site. Also minimum support lengths are specified. The State of Oklahoma is in a low and moderate seismic risk area. Those areas with an Acceleration Coefficient less than .09 are in SPC A requiring no special analysis procedure but requiring minimum connection details of the superstructure to the substructure to resist a horizontal seismic force of .20 times the deadload reaction force in the restrained directions. Also minimum supports lengths are specified.

Those areas with an Acceleration Coefficient of .10 to .19 are in SPC B requiring Analysis Procedures, a single-mode spectral method to determine elastic member forces. Design forces for each component are obtained by dividing the elastic forces by a response modification factor R. For connections at abutments, columns and expansion joints, the R factor is either 1.0 or .8; therefore, these components are designed for expected or greater elastic forces. For columns and piers, the R factor varies between 2 and 5 resulting in design forces lower than predicted by elastic analysis. Therefore, the columns are expected to yield when subjected to the forces of the design earthquake. The yielding in turn implies relative distortions of the structural system that must be considered in assessing the adequacy of the final bridge design. Design requirements to ensure reasonable ductility capacity of columns for bridges classified as SPC B are specified, but they are not as stringent as those for bridges classified as SPC C and D. Foundations are designed for twice the seismic design forces of the column or pier to ensure yielding in the column.

Those areas of the country with an Acceleration Coefficient of .20 and greater are in SPC C or D requiring a similar approach to that of SPC B; however, several additional requirements are included. Single-mode spectral analysis is specified for a bridge defined as a regular bridge with Multimode Spectral Analysis specified for a bridge defined as an irregular bridge. A regular bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (Abutments excluded). An irregular bridge is any bridge that does not satisfy the definition of a regular bridge. For columns, additional requirements are specified to ensure that they are capable of developing reasonable ductility capacities. For connections and foundations, the recommended design forces are based on maximum shears and moments that can be developed by column yielding. Horizontal linkage and tie-down requirements at connections are also provided.

The computer program SEISAB (SEISmic Analysis of Bridges) developed by Roy Imbsen specifically for use by bridge engineers is an excellent analysis program and will make it much easier for our designers to adapt seismic design analysis to their every-day design procedure.

This project was concluded in October, 1981, with the published report "ATC-6 Seismic Design Guidelines for Highway Bridges." The criteria has now been adopted by AASHTO and will soon be published as "AASHTO Guide Specifications for Seismic Design of Highway Bridges." A follow up project "ATC-6-2 Recommended Guidelines for Seismic Retrofit of Highway Bridges" is also complete. These projects and Workshops in San Diego, California, and New Zealand on research needs for seismic design of bridges have contributed greatly to a better understanding of the state of the art.

With so many research organizations involved in this specific problem in the U. S. and in other countries, it is important that the design engineers solving the everyday problems be kept abreast of the technology. Keeping our design codes up-to-date is extremely critical in this regard. A goal as a member of the AASHTO Bridge Committee is to keep the technology in our design specification as current as possible, but at the same time keeping the analysis and design procedures simple enough to enable a designer to understand and adapt the procedures without attaining a PhD. This workshop should enable us to develop the most meaningful research for durability of bridges for seismic forces.

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CABLE STAYED BRIDGES - STATIC AND DYNAMIC RESPONSE

by

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University of Pittsburgh, Pittsburgh, Pennsylvania, U.S.A.

U-13

INTRODUCTION

In a cable-stayed bridge, the roadway is supported elastically at specific points by inclined cable stays which are attached directly to tall towers. One of the main differences encountered in the analysis of a cable-stayed bridge, compared to more conventional structures, such as continuous girder bridges or rectangular framed buildings, is the possibility of nonlinear behavior under normal working loads.

NONLINEAR STATIC ANALYSIS

For normal static live loads, the material in a cable-stayed bridge can be considered to remain elastic, however, the overall load-deformation relationship can still be nonlinear. Three primary sources of nonlinear behavior have been proposed by previous investigators. These are: the geometry changes caused by the large displacements which can occur in this type of structure under normal design loads; the interaction of the bending deformations and high axial forces in the towers and longitudinal deck members; and the nonlinear axial force-elongation relationship for the inclined cable stays.

In order to investigate the importance of each of these possible sources of nonlinear behavior, a number of static analyses were performed, at the University of Pittsburgh, on mathematical models which represented several different actual or proposed bridges. The analyses were performed by the Stiffness Method using a combined incremental and iterative approach, in which the stiffness of the structure was recomputed after each load increment was applied. Iterations were continued until the unbalanced joint loads were within an acceptable limit. Figure 1 shows the geometry of a typical three dimensional mathematical model which was considered in the study.

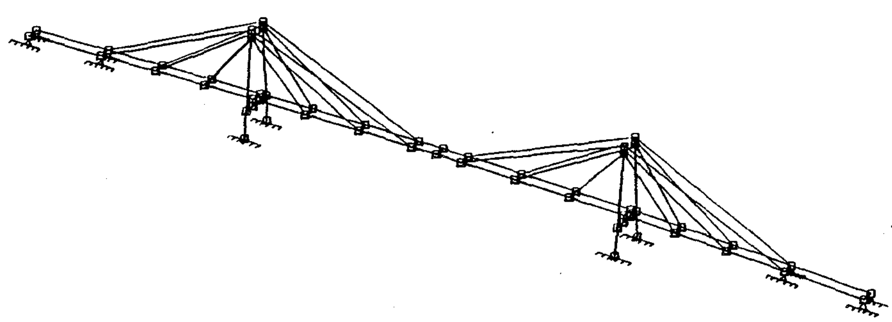

Figure 1 - Mathematical Model

The results of the static analyses showed that, for normal static design loads, both the effects of the overall change in geometry, of the structure, and the effects of the interaction of the bending deformations and high axial forces, in the towers and longitudinal deck members, are small. These effects can be neglected when computing the overall response of the structure. The analyses also showed that the axial stiffness of the cables can vary significantly as the tension in the cables changes. This variation in stiffness in the cables will have an effect upon the overall stiffness of the structure. For small loads, the overall stiffness can vary significantly as the load is changed, however, as the load is increased, the load-deformation relationship becomes more linear. For loads equal to the full dead load of the structure, or greater, the relationship becomes essentially linear, since the cables have achieved a tension at which their axial stiffness does not change.

The results, of these static analyses, lead to the conclusion that a cable-stayed bridge structure does behave in a nonlinear manner for low loads, however, after the full dead load deformed position has been reached, the structure can be considered to behave linearly. Therefore, the stiffness of the structure can be considered to remain constant during the application of the live load. This indicates that linear static and dynamic analysis techniques are applicable to this type of structure, starting at the dead load deformed position, as long as the material remains elastic.

NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

In order to verify that a linear dynamic analysis is adequate, a number of time history analysis were performed, for several different types of dynamic loads, including seismic ground motion, a simulated wind loading and moving traffic loads. Three distinct types of analyses were performed, consisting of the following combinations of static analysis and dynamic analysis: linear static analysis, to compute the structure stiffness in the static dead load deformed position, and linear dynamic analysis, in which the stiffness was assumed to remain constant as the structure deformed due to the dynamic loads, hereafter denoted as a Linear-Linear analysis; nonlinear static analysis, using the combined incremental and iterative analysis procedure described previously, and linear dynamic analysis, hereafter denoted as Nonlinear-Linear; and nonlinear static analysis and nonlinear dynamic analysis, in which the stiffness of the structure was changed, corresponding to the cable tensions and member loads at the end of each dynamic time step, hereafter denoted as Nonlinear-Nonlinear.

The results of the Nonlinear-Linear and Nonlinear-Nonlinear analyses were almost identical, and varied considerably from the results of the Linear-Linear analyses. This verifies that, although a nonlinear static analysis is necessary to obtain the stiffness of the structure in the dead load deformed position, a linear dynamic analysis will suffice starting at this position. This is an important conclusion since a linear time history dynamic analysis is much simpler and more economical to perform than a nonlinear analysis. In addition, this suggests that linear dynamic analysis techniques, such as the Response Spectrum Method, are applicable to this type of structure.

RESPONSE SPECTRUM ANALYSIS

The next step in the investigation was to use the Response Spectrum Method to analyze the bridge represented by the mathematical model shown in Figure 1. This mathematical model has properties very similar to the Luling Bridge, in Louisiana. The specific ground motion which was used was the May 18, 1940 El Centro California Earthquake. The Response Spectra, for the three measured components of this earthquake, have been developed by the Earthquake Engineering Research Laboratory of the California Institute of Technology.

The natural frequencies and mode shapes of the lumped mass mathematical model were computed using a standard eigenvalue procedure. The fundamental mode has a freqency of 0.321 cycles per second and consists primarily of vertical translations of the bridge deck with very little movement of the tops of the towers. The first mode to exhibit any significant bending in the towers is the eighth mode. The primary movements for all lower modes was either vertical or transverse translation of the deck due to bending and/or torsion. After approximately the fifteenth mode, the movements become localized and are primarily due to axial deformation in a particular member.

Due to lack of space, it is not possible to show all of the results obtained from the Response Spectrum analyses. As a typical example, Figure 2 shows the horizontal translation at the top of one of the towers and the vertical translation of a point on the middle span deck, which were obtained for the N-S El Centro Earthquake component, acting along the longitudinal axis of the bridge. The plot shows that the deck vertical displacement is primarily due to the contribution of the third mode while the displacement at the top of the towers is affected mainly by the third and fourteenth modes. The primary displacements in the third mode are vertical movements of the deck with an anti-symmetrical mode shape. It would be expected that this mode would be excited by longitudinal support motion.

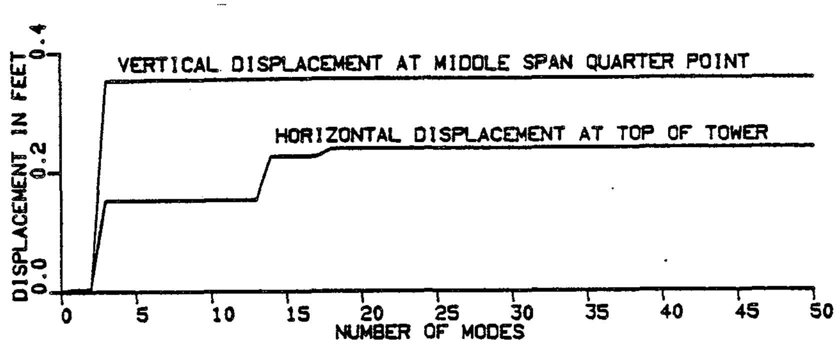

Figure 2 - Response Spectrum Analysis Displacements

The Response Spectrum analyses, which have been performed, show that extreme care must be exercised when applying this procedure to cable-stayed bridges. For some responses, only the first few modes must be considered, while for other responses, the higher modes make the greatest contribution.

DYNAMIC FIELD TESTING OF FULL SCALE HIGHWAY BRIDGES

U-14

Bruce M. Douglas [1]

Introduction: In 1974, the Civil Engineering Department at the University of Nevada at Reno established a dynamic field testing program for full scale highway bridges (1,2,3,4). The principal thrust of this work has been the lateral dynamic testing of typical overcrossing structures to identify the in situ properties of these bridges which significantly affect the distribution of seismic forces. However, to test the validity of using simple analytic models of bridges suggested by field test results in a seismic environment, the strong motion accelerogram data obtained from the Meloland Road Overcrossing during the 1979 Imperial Valley earthquake was also studied in detail (1). The purpose of this summary paper will be to: 1) describe and summarize the principal results from the various field dynamic tests, and 2) present the principal conclusions from the Meloland Road Overcrossing study.

First Rose Creek Tests: In May of 1979 a fairly extensive series of moderate amplitude lateral dynamic tests (1,2) were conducted on the Rose Creek Interchange near Winnemucca, Nevada. This bridge is a four hundred foot long, five span reinforced concrete box girder bridge supported on single column piers. The deck was designed to be isolated from the abutments by supporting it on elastomeric bearing pads at each abutment, and all foundations were pile (one foot diameter steel shells, 25 feet long, filled with reinforced concrete) supported to bypass a layer of relatively soft clay. The lateral dynamic tests were conducted by the pull back and quick release method by using two D-8 tractors to deform the structure with cables. The largest loads used during the course of the experiments was about one quarter of the design earthquake loads. In this series of tests, the foundations were not excavated to allow direct response measurements to be made on the bridge's foundations, but detailed dynamic response measurements were made on the bridge deck allowing four mode shapes and six natural frequencies to be extracted from the data (1,2). System identification methods were used to find the important structural parameters including values for the lateral soil/structure interaction springs for the pile foundations. The significant results from this study included: 1) the lateral flexibility of all of the bridge's piers was significantly affected by the soil/structure interaction phenomenon. The soil/structure interaction phenomenon contributed between 48 and 79 percent of the total lateral pier flexibility. This, of course, greatly affects the distribution of seismic loads, and 2) only the addition of simple boundary element springs in the analytical model were required to characterize the soil/structure interaction effects observed in field test data.

High Amplitude Tests at Rose Creek: Analysis of the data from the 1979 Rose Creek tests weakly suggested that the pile foundations had a rotational flexiblity (1,2). In order to clarify this question and to obtain dynamic response data at the highest possible amplitudes a second series of tests were conducted in 1982 (3,4). The main differences relative to the first series of tests were that: 1) the pile foundations of the four piers were all excavated such that direct modal rotations and lateral modal displacements could be measured, and 2) the test amplitudes were up to six times larger up to 1.5 times the design earthquake loads) than in the 1979 tests. Dynamic excitation was achieved by using four hydraulic rams to

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deform the structure and quick releasing the hydraulic fluid from them simultaneously. System identification methods were again used to obtain the important in situ dynamic properties of the bridge from the field data (4). The main results from this study were: 1) the rotation of the pile foundations contributed much more (about five times more) to the lateral flexibility of the bridge piers than the lateral flexibility of the pile foundations, 2) only simple rotation and lateral boundary element springs were required in the analytical model to characterize the observed soil/-structure interaction effects, and the values for these foundation springs obtained from the field tests correlate reasonably well with values obtained from independent geotechnical analyses (1), 3) the neoprene elastomeric bearing pads had stiffened substantially (about a factor of three or four) with age, and 4) no evidence of a cyclic degredation of foundation stiffness was observed over the very large number of significant cycles of vibration involved in this series of high amplitude tests.

Meloland Road Overcrossing: The previous field test correlation studies indicate that simple boundary element springs may be used in conjunction with linear bridge models to explain the dynamic response of this class of highway bridge providing appropriate adjustments are made in these soil/structure interaction springs to account for the amplitude dependent nature of the foundation soils (1). The fact that these models can be used to explain bridge test data over a wide range of amplitudes does not necessarily mean that these same simple models will predict the seismic response of bridges with the same precision. To test this and other questions the Meloland Road Overcrossing (which has 26 permanent channels of strong motion instrumentation) that was subjected to the 1979 Imperial Valley Earthquake was studied. Fig. 1 shows a plan view of this bridge and most of the channels of instrumentation. This reinforced concrete bridge has abutments which are monolithic with the approach fills and a single column which divides the bridge into two equal spans. All foundations were pile (timber) supported.

System identification methods were again used in the time series domain to optimally fit the parameters of a very simple bending beam model with simple boundary element soil/structure interaction springs to the data. Allowance was made in this model to accept different accelerogram inputs (inputs can be out of phase) at the two abutments and the base of the column. Optimization of this very simple model with respect to only four parameters (two soil/structure interaction springs, overall effective damping, and lateral deck stiffness) produced the correlation shown in Fig. 2.

This study (1) led to a number of conclusions. Among them are: 1) the experimentally determined rotational stiffness of the single columns pile foundation was abnormally low. It was so low as to stimulate a detailed analysis of the liquefaction potential of the subsurface soils (1) which indicated that this bridge probably came perilously close to collapse during this earthquake due to liquifaction. A complete confirmation of this explanation, however, will have to await a more detailed study of the subsurface soil conditions at this site, 2) linear structural models using simple soil/structure interaction springs (suitably adjusted to account for the amplitude dependent nature of the foundation soils) will provide sufficiently accurate seismic response predictions providing the bridge structure itself does not yield. During this earthquake, the peak ground accelerations were on the order of 30 percent g while the peak structural accelerations were about 20 percent g and no observable damage was done to the structure, and 3) if system identification methods are to be used in conjunction with either field test data or earthquake response measurements to estimate the in situ soil/structure interaction springs it is essential that detailed response measurements be made on the foundations. In the case of field tests, this means excavating the foundations and in the case of earthquake strong motion studies it means having permanent detailed instrumentation packages on the foundations.

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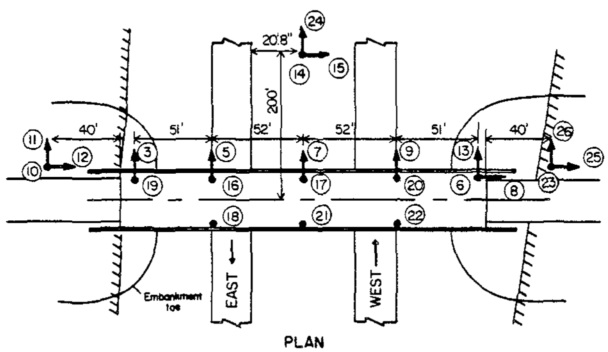


Figure 1

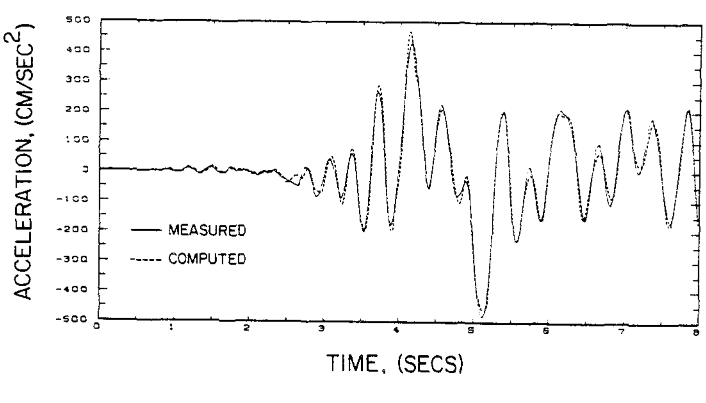


Figure 2

NSF PROGRAM

John B. Scalzi

U-15

presented by B. Douglas

NSF is research funding agency, primarily to universities. Although under special conditions, professional organizations, professional consulting firms, non-profit research laboratories, industry, and other federal agencies may also be funded.

The mode of operation is by the submission of unsolicited proposals from the researcher. These are reviewed by their peers, but are declined or awarded by NSF.

At present there are approximately 10 current projects on bridges ranging from field experiments, laboratory experiments, analytical methods, and computer model simulations. The approximate funding is $1,200,000.

NSF expects to continue supporting bridge projects every year, at about the same number and funding support as is currently being provided. NSF cannot predict the exact number of awards in bridge projects, but expects the number to increase slightly because of the necessity to build earthquake resistant bridges in the U.S.

Several projects which are currently under review involve coordinated research by U.S. researchers and researchers in New Zealand. Dr. Scalzi expresses the hope that similar coordination of projects will be generated by this meeting. There are many types of bridges of common interest.

NSF has supported research on the seismic response of suspension bridges which you will (or have) heard about. Possibly more work needs to be done on obtaining seismic records on these bridges in Japan. The same is true for cable-stayed bridges. Our U.S. researchers are in need of experimental data to verify the matematical concepts. The same is true for other types of bridges.

Repair and strengthening of bridges is an important part of the U.S. program and may be a fruitful area to explore for coordinated projects.

This meeting may develop several topics of interest to both countries.

PROSPECTIVE EXPERIMENTAL RESEARCH ON BRIDGE STRUCTURES AT THE NATIONAL BUREAU OF STANDARDS

U-16

- Ductile Capacity of Concrete Bridge Columns and Piers

    a. Inelastic flexural response of circular and non-circular bridge columns under axial and lateral loads.

    b. Shear strength of columns.

    c. Columns subjected to lateral loads in two orthogonal directions.

- Anchorage and Splice Requirements Under Cyclic Loads

    a. Development of large diameter reinforcing bars.

    b. Anchorage of large diameter reinforcing bars into footings and bent caps.

- Concrete Filled Steel Tubular Columns

    a. Load transfer from steel tube to concrete.

    b. Connection between column and deck slab.

FHWA Program of Earthquake Engineering Research

James D. Cooper
Federal Highway Administration
Washington, D.C.

U-17

Earthquake engineering research is being conducted as part of the Federally Coordinated Program of Research and Development FCP Project 5A - "Bridge Loading and Design Criteria." Research under this project includes a comprehensive review of all types of bridge loadings including those generated by both traffic and environment and time dependent sources. Where data and design deficiencies exist, analytical, laboratory and field studies will be conducted to provide a data base on which design criteria and guidelines can be developed to provide bridge safety and increased life expectancy. Specifically, research will be conducted to develop bridge loading factors, to improve life expectancy, to evaluate and enhance structural systems and components, and to develop new design criteria.

Since highway bridges are being subjected to ever increasing taffic and exceptional live loads, much more data, confirmed by research and/or experience are needed in order to raise the confidence limits for the design of new structures. Bridge design standards and specifications determine, in principle, the load carrying capacity of bridges ensuring that they can safely carry the anticipated vehicle traffic. Codes and regulations limit wheel and axle loads as well as gross vehicle weights and they impose a number of size limits and standards.

An analysis of the following characteristics is required: statistical distribution of gross vehicle weights, axle loads and speed of heavy vehicles in motion; dynamic action of loaded and unloaded freight vehicles on the bridge decks and joints; and stresses and strains, induced by heavy vehicles, in the substructure and foundations.

Other external loading and environmental factors such as wind, <u>earthquake</u>, high water and moving ice, temperature variations, corrosion, and chemical and biological attack will be researched prior to incorporation into a reliable design process.

The major outputs from this research will include the quantification of the effects of traffic induced loads, <u>natural hazards</u>, and ambient and time dependent loads on the design, construction, and maintenance of a highway bridge.

The following earthquake related studies will be continued or initiated in the FHWA research program.

1. Foundation Design to Resist Earthquake Motion

   This study will advance the state-of-the-practice in the earthquake resistant design of bridge foundations and where appropriate will supplement the foundation design guidance specified in the AASHTO

Guide Specification for Earthquake Design of Highway Bridges. Analytical data and existing field data will be used to identify dynamic force levels and displacements which foundation systems must resist.

2. Large Scale Bridge Column Testing

Research will continue at the National Bureau of Standards to conduct tests of full scale and reduced scale reinforced concrete bridge columns designed by recently developed seismic design guidelines. The tests will provide data on the effects of scaling factor on the interpretation of results obtained from small scale testing, column performance and ductility, and performance of damaged columns which have been repaired.

3. Bridge Seismic Instrumentation

The FHWA is participating with the US Geological Survey to incorporate instrumented bridges into the USGS National Strong Motion Network to record structural response to earthquake motion. Records retrieved from instrumented bridge sites are being used to verify assumptions which have been made in the development of several bridge computer programs. The data is important because analyses made to date assume simultaneous earthquake input motion of the base of all piers.

In reality an earthquake wave travels from the source of a finite velocity and does not "load" each foundation or pier simultaneously. It is believed that existing computer programs are adequate to predict response due to the travelling wave theory, but the data is needed to verify the structural response programs and ultimately design assumptions.

4. Seismic Design and Retrofit Structures Manual

This implementation study will result in a comprehensive, well illustrated seismic design and retrofit manual on highway bridges. The manual will incorporate all research conducted over the past decade and be based on the AASHTO design guide specification.

U-18 J. Gates, Feb., 1984

RESEARCH AT THE CALIFORNIA DEPARTMENT OF TRANSPORTATION

1984-85 FY

The research at CALTRANS for the 1984-85 fiscal year can be divided into three general categories:

FIXED COSTS	$ 1.25 million
ON GOING PROJECTS	2.00 million
NEW PROJECTS	0.75 million
TOTAL	$ 4.00 million

Fixed costs include support to TRB and NCHRP as well as CALTRANS administrative costs.

A few on going projects related to bridges include:

A $604,000, 7 year project with the University of California at Berkeley to improve the non-linear analytical capabilities for bridge structures. This project has developed the non-linear program called NEABS. Current work is aimed at improving the soil-structure interaction modeling capabilities of the program.

A 193,000, 3 year project funded cooperatively with the National Bureau of Standards, the National Science Foundation, the FHWA (Washington) and CALTRANS to determine design parameters and ductility capacities of large bridge columns by use of full scale testing procedures.

A $248,000, 4 year project to determine the horizontal resistance of cast-in-drilled-hole piling in sloping ground. Results from both full scale and model tests are being compared to current design procedures. Results will include the development of design formulas.

A $160,000, 2 year project with UCLA to verfiy and/or determine the design parameters and capacities of seismic retrofit componants using full scale testing procedures.

A few new projects related to bridges include:

A 200,000, 4 year project to develop computer programs incorporating new design methods. This project will investigate and implement on-line interactive computer systems for bridge design and drafting to increase productivity.

A 60,000, 2 year project to investigate the shear capacity of multi-spiral reinforced oblong columns.

A 25,000, 1 year project to evaluate current deck reinforcement practice and determine the feasability of reduction in reinforcement.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

U-19

L. F. Spaine, Engineer of Design

The National Cooperative Highway Research Program is supported on a continuing basis by funds from participating departments of the American Association of State Highway and Transportation Officials with the full cooperation and support of the Federal Highway Administration of the United States Department of Transportation. The Transportation Research Board administers the program for the sponsors. Following is a tabulation of projects that have been approved for research in this year's program:

Project Number	Title	Funds Available	Expected Start
4-15	Corrosion Protection of Prestressing Systems in Concrete Bridges (Phase II)	100,000	Late 1984
10-20	Elastomeric Bearings - Design, Construction, and Materials (Phase III)	150,000	Early 1985
10-22	The Performance of Weathering Steel in Bridge (Phase II)	250,000	Late 1984
12-18	Development of an Integrated Bridge Design System (Phase II)	150,000	Mid 1985
12-25	Fatigue and Fracture Evaluation for Rating Steel Bridges	200,000	Mid 1984
12-26	Distribution of Wheel Loads on Highway Bridges	300,000	Late 1984
12-27	Welded Repair of Cracks in Steel Bridge Members	375,000	Mid 1984
12-28	Load Capacity of Bridges	1,000,000	Late 1984
12-29	Design of Simple-Span Precast Prestressed Girders Made Continuous	250,000	Late 1984
15-10	Development of a General Design Graphics System	500,000	Early 1985
20-5	Synthesis of Information Related to Highway Problems Topic 16-01, Bridge Inspection Equipment, Staffing and Safety Topic 16-04, Microcomputer Software for Highway and Structural Engineering Topic 16-10, Bridge Expansion Devices		

RESOLUTIONS IN ENGLISH AND JAPANESE

RESOLUTIONS OF THE US-JAPAN BRIDGE WORKSHOP

TASK COMMITTEE J - WIND AND EARTHQUAKE

ENGINEERING FOR TRANSPORTATION SYSTEMS

US-JAPAN PANEL ON WIND AND SEISMIC EFFECTS

TSUKUBA SCIENCE CITY, JAPAN

FEBRUARY 20 - 22, 1984

1. The US-Japan Bridge Workshop was held under the auspices of Task Committee J of the UJNR Panel on Wind and Seismic Effects, at the Public Works Research Institute, Tsukuba Science City, Japan. The Workshop emphasized earthquake topics relative to highway bridges including design specifications, seismic forces and bridge response.
2. Workshop presentations on future studies to be conducted or sponsored by agencies in both countries indicated the future trends of earthquake engineering research in bridge related areas. Comprehensive discussion was held on future coordinated research between both countries. In the near future, coordinated research between the US and Japan is desired in the following areas:
    1) Laboratory experiments on the seismic behavior of reinforced concrete piers and columns including concrete-filled steel tubes.
    2) Repair procedures for seismically damaged bridge structures.
    3) Experimental field testing of bridges including forced and ambient vibration measurements.
    4) Correlation between experimental and theoretical studies.
    5) Experimental and analytical studies on the effects of soils on the behavior of bridge structures.

6) Strengthening procedures for existing bridges.

7) Strong-motion instrumentation of bridges, corresponding data-processing and parameter identification.

3. The US-Japan Bridge Workshop provided an extremely valuable exchange of technical information which was beneficial to both countries. In view of the importance of cooperative programs on the subject of bridge engineering, the continuation of Joint Bridge Workshop and personnel exchange is considered essential.

   Such future exchanges are strongly encouraged to enhance the effectiveness of the UJNR cooperative program and will be discussed at the Sixteenth (16th) Joint Meeting of the Panel on Wind and Seismic Effects in Washington, D.C., May, 1984.

4. At the Sixteenth Joint Meeting, the Chairmen of Task Committee J will request the endorsement of the full panel to proceed with implementation of the specific recommendations resulting from this Workshop.

5. In the interest of exchange of information, a proceedings of the papers presented at the US-Japan Bridge Workshop will be developed, published and distributed by the US side to all members of the UJNR Panel on Wind and Seismic Effects and to participants of the Workshop.

天然資源の開発利用に関する日米会議
耐風・耐震構造専門部会
作業部会Ｊ．交通システムの耐風耐震技術

日米橋梁ワークショップ．最終結論

（１９８４．２．２０－２２，筑波研究学園都市）

１．　ＵＪＮＲ耐風・耐震構造専門部会の作業部会Ｊの主催のもと，日米橋梁ワークショップが１９８４年2月２０日～２２日の間，日本の筑波研究学園都市にある土木研究所で開催された。本ワークショップは，設計基準，地震力及び橋の応答を含む道路橋の地震問題に焦点をしぼって行われた。

２．　今後，両国の関係機関で実施する研究計画が発表され，橋梁の耐震性に関する方向が示唆された。将来における両国間の協力研究について幅広い討議が行われ，特に近い将来の日米間の協力研究の課題として，下記の課題が指摘された。

１）コンクリート充塡鋼管を含む鉄筋コンクリート橋脚柱の地震時挙動に関する室内実験

２）地震により被災した橋梁構造の復旧方法

３）強制振動及び常時振動の測定を含む橋梁の現場実験

４）実験研究と理論研究の相関性

５）橋梁構造の挙動に与える地盤の影響に関する実験的・解析的研究

６）既設橋梁の耐震補強方法

７）橋梁に対する強震観測法，データ処理法，及び解析法

３．　本ワークショップは，両国関係者にとって極めて有益な技術情報交換の場を与えた。橋梁工学の課題に関する協力研究計画の重要性に鑑みると，合同ワークショップ及び人材交換を引続き行う必要がある。このことは，ＵＪＮＲの研究協力計画を効果的かつ強力に推進させることになるので，１９８４年5月ワシントンＤ．Ｃ．にて開催予定の第１６回耐風・耐震構造専門部会合同部会の場において討議されることとなろう。

４．　作業部会Ｊの両作業部会長は，第１６回合同部会について，本ワークショップで得られた結果を報告し，全体部会の合意を求めるものとする。

５．　情報交換のため，今回の日米橋梁ワークショップに提出された論文の会議録を米側が作成，印刷し，ＵＪＮＲ耐風・耐震構造専門部会の全委員及び本ワークショップの参加者に配布するものとする。

APPENDIX A

THE
MAIN BRIDGES
OF
KAN-ETSU
EXPRESSWAY

STRUCTURE SECTION
TOKYO SECOND CONSTRUCTION BUREAU
JAPAN HIGHWAY PUBLIC CORPORATION

FEBRUARY, 1984

CONTENTS

PREFACE

The Kan-etsu Expressway extending approximately 300 kilometers in length links Tokyo, capital of Japan, with Niigata, the largest city on the Japan Sea, for speedy automobile transportation.

Since some portions under construction pass through well-known mountainous regions and rivers in Japan, the expressway calls for big-scale bridges and long tunnels. The bridges shown here are some of the biggest in Japan.

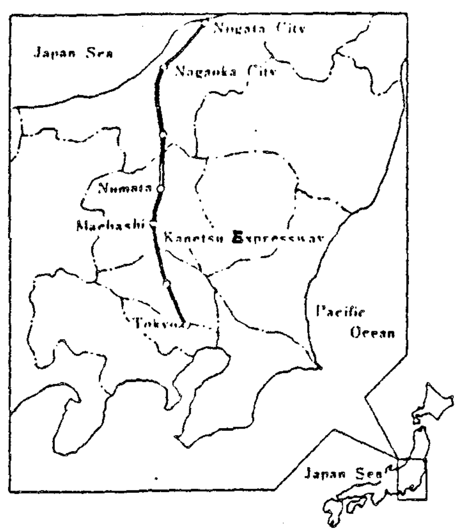


Fig.1 Kan-etsu Expressway

§1 THE TONE-GAWA BRIDGE

INTRODUCTION

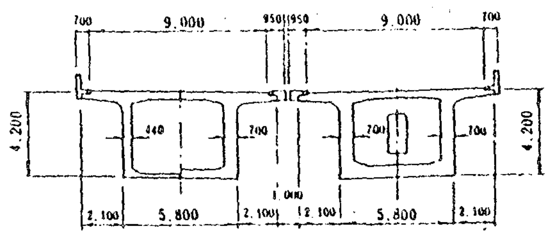
Since the Tone river has had a large flux(10000ton/sec) at a snow-melting period as well as at a typhoon season, the spans are determined to make the obstacle by piers in the river as small as possible. The spans are 80 meters throughout the bridge.

This bridge is also designed to keep the same cross-section and the straight longitudinal gradient to get a good view because that is in the well-known sightseeing area.

The bridge had to be parted into two continuous girders mechanically because the abutment AP1 would have been much larger if a seven continuous type had been chosen. Considering possible high water level as mentioned before, two stretching erection methods, that is, balanced cantilever methods, are chosen for major parts. P&Z method and Dywidag method explained later are applied to the up line and down one respectively.

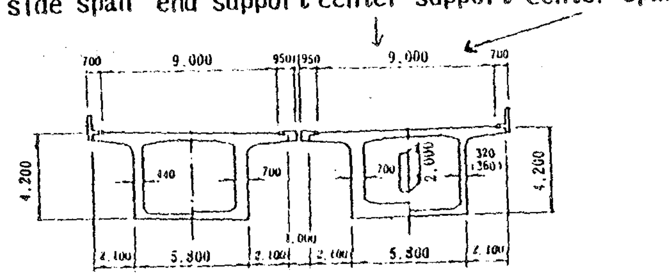
2-Span PC continuous girder

side span end support center support



5-span PC continuous girder

side span end support center support center span



( ):for up line

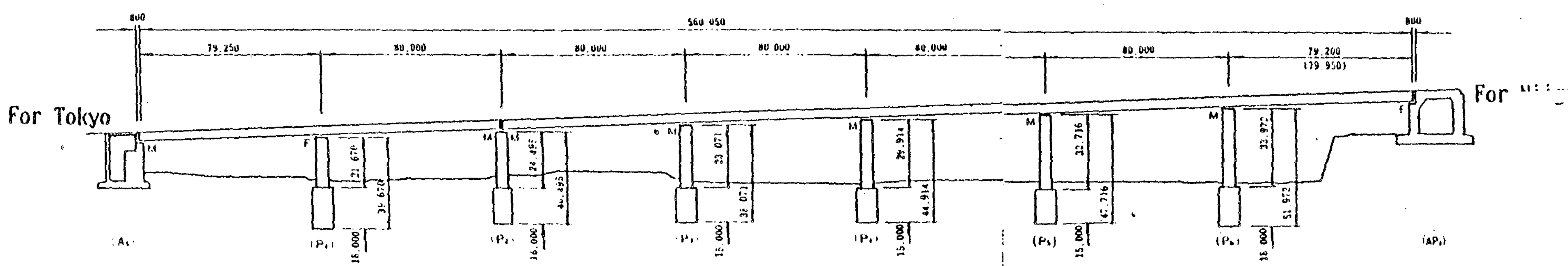


Fig.2 General view of the Tone-gawa Bridge

DESCRIPTION OF THE PROJECT(SUPERSTRUCTURE)

Name of the Highway:Kan-etsu Expressway

Name of the Project:Kan-etsu Expressway Tone-Gawa Bridge Prestressed Concrete Superstructure Project

Location of the Project:

From Shibukawa City, Gumma Prefecture

To Seta County, Gumma Prefecture

Period of the Project

Construction Beginning: June 12, 1982

Completion: May 31, 1984

Bid:2,450,000,000 YEN($10,208,000)

Bridgere type:Prestressed concrete road bridge

Structure type:Two-span continuous box girder + five-span continuous box girder

Classification of the Bridge:1st(live load 20 tons, 43 tons)

Bridge Length:Up line 560.838 meters =(2x80.000)+(4x80.000+80.838)

Down line 559.162 meters =(2x80.000)+(4x80.000+79.162)

Effective Width:Twin of 9.0 meters(four lanes)

Gradient:Longitudinal 3.5%

Transversal 2.15~-2.096%

Prestressing Materials:Up line Fressinet Cable 12T12.4

PC steel bar for Dywidag system 32 mm in diameter

Down line PC steel bar for Dywidag system 32 mm in diameter

Erection Method:Up line Stretching erection method with launching truss girder(P&Z method)

Down line Stretching erection method by traveller (Dywidag method)

Table 1 Main materials of the Tone-gawa Bridge(superstructure)

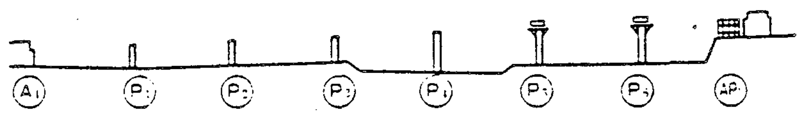
	up line	down line
concrete	5,659 cubic meters	5,593 cubic meters
reinforcing bar	672 tons	674 tons
prestressing steel		
longitudinal	236.2 tons	361.62 tons
transversal, etc	102.7 tons	99.56 tons
shoes	97.48 tons	97.26 tons

BRIEF EXPLANATION OF DYWIDAG METHOD

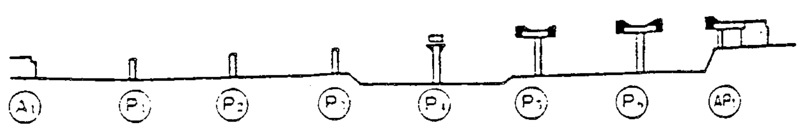
The Dywidag method developed by the Dycherhoff & Widmann Company in West Germany was introduced to Japan by the Sumitomo Electric Industries through technical cooperation. After the Arashiyama bridge of span 51 meters was constructed by the stretching method of Dywidag in 1959, the method was applied to over 350 bridges in Japan. Especially, using this method, we could construct long spanned PC bridges such as Hamana bridge of span 240 meters, one of the world's longest.

The method is briefly explained as follows for the construction of Tone-gawa bridge.

1st:The segments on the top of the pier P5 and P6 are temporarily fixed by prestressing bars. The pier plays a role in the bridge erection by supporting cantilever arms that progress outward from the left and right of the pier step by step. Some part of the span adjacent to the abutment AP1 is built with a staging which means a supporting structure from the ground.



2nd:The segment on the top of P4 is built. Movable working platforms called travellers are assembled on the pier P5 and P6. New segments on the left and right are cantilevered from the previously constructed segments keeping the balance against each other.



One cycle in the Dywidag method is as follows:

1) The PC bars which progress outward from atop of the pier are lengthened using couplers(Fig.3).

2) The PC bars are arranged and seaths are used to protect PC bars from concrete-casting.

3) Once the PC bars with seaths are in position, concrete casting begins.

4) After the concrete is cured, the PC bars are tensioned by the Dywidag hydraulic jack.

5) In this way, a segment is prestressed and joined with the previously constructed segment.

6) The traveller is launched to make next new block by advancing over the prestressed segment. Thus, segments are gradually added keeping the balance between the left portion and right one of the pier.

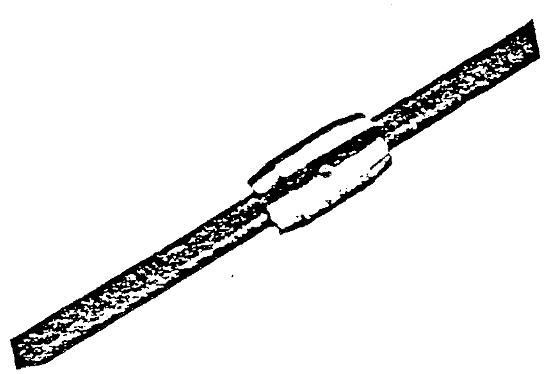
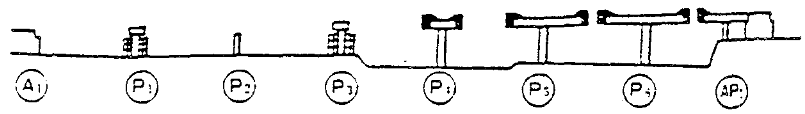
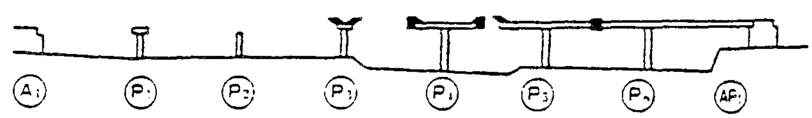


Fig.3 Coupler

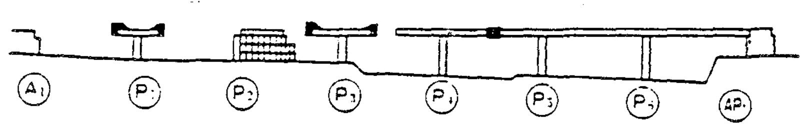
3rd:The segments on the top of P1 and P3 are built. After the assembly of travellers over pier P4, P5, P6 and the temporary bent close to AP1, the girders begin to be stretched just like 2nd step.



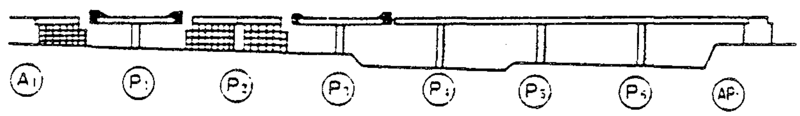
4th:The segment on the top of P1 is built. After the assembly of travellers on pier P3 and P4, stretching begins. P5-P6 span and P6-AP1 span are completed after the ends of opposite cantilevers are joined.



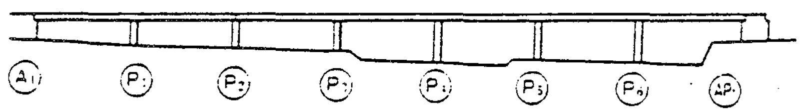
5th:Travellers are assembled on pier P1 and P3. Cantilever work begins. The girder on the right of P2 is constructed by the staging. The span P4-P5 is finished after adjacent cantilevers are linked with an integrated closure.



6th:The girders adjacent to A1 and P2 are constructed by the staging.



7th:Completed.



CHARACTERISTICS OF DYWIDAG METHOD

1)Since no support under the girder is needed, the longer the span is, the more economical the cost is. Especially, if a bridge has to cross over deep valley, river with large flux and places with large traffic volume on sea or land, this method is favorable.

2)Since the construction of 3 to 4 meter segment is repeated, protection and control are easy.

3)Since the construction work is carried out in the traveller, wearther condition doesn't govern the progress of work.

4)Since the major work such as forming, concrete casting and prestressing is repetitive, workers can easily get accustomed. It makes the process simple and efficient.

5)An anchor is fixed with nut, prestressing is simple and secure. Furthermore, it is simple to join any number of prestressing bars because couplers are used.

CHARACTERISTICS OF PRESTRESSING STEEL BAR FOR DYWIDAG SYSTEM

The prestressing steel bars for Dywidag system of 32 mm in diameter are used for the construction of the Tone-gawa bridge. The thread of PC bar is made by rolling a round bar instead of cutting it. That makes the thread of PC bar stronger than original material because steel fibers are still continuous after rolling. The threads of Dywidag system are characterised by asymmetric screw that is superior to symmetric one in fatigue strength. The PC steel bar for Dywidag system is fixed with special anchorage called Glocke(Fig.4)which enables earlier prestressing work than usual anchor plate.

Table 2 Specification of Dywidag PC steel bar in diameter 32 mm
for the Tone-gawa Bridge

place used	grade	tensile strength kg/sq.mm	yield point kg/sq.mm	elongation %	relaxation %
trans-versal and diago-nal	SBPR 95/120	120 (94.72 tons /32mm-bar) (min)	95 (74.98 tons /32mm-bar) (min)	6.0 (min)	1.5 (max)
longi tudin-al	SBPR 80/105	105 (82.88 tons /32 mm-bar)	80 (63.14 tons /32 mm-bar)	7.0 (min)	1.5 (max)

cf. Usually, allowable stress is minimum one of 0.6x(ultimate stress) and 0.75x(yield stress).

Fig.4 Gloke

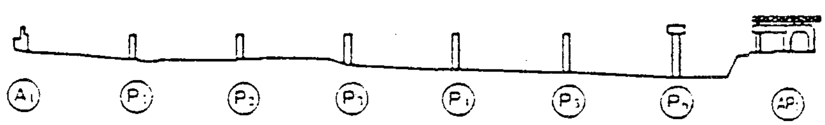
BRIEF EXPLANATION OF P&Z METHOD

The P&Z method developed by Polensky & Zollner Company in West Germany is one of the erection methods of prestressed concrete bridge. In this method, metal forms are supported by an overhead launching truss installed on the PC girder of the bridge. Then cantilever arms progress outward from atop the pier keeping the balance.

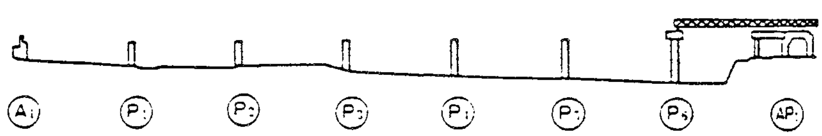
This method has been applied in Europe since the first bridge by the method was constructed in West Germany. In Japan, this was adopted to erect the Tsukiyono-ohashi Bridge(bridge length=306 meters, span=84.5 meters) for the first time in 1982, and the Tone-gawa Bridge is the first one for express-ways using this method.

The method is briefly explained as follows for the construction of the Tone-gawa Bridge.

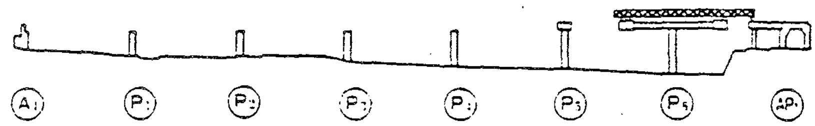
1st:Some part adjacent to abutment AP1 is constructed by the staging. An temporary support is used. Then an overhead launching truss is assembled on the prestressed concrete girder.



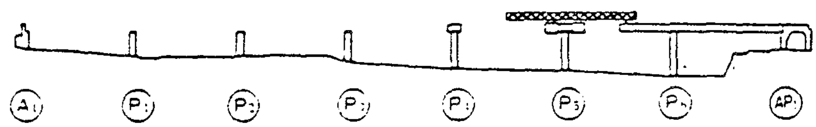
2nd:Launch the overhead truss.



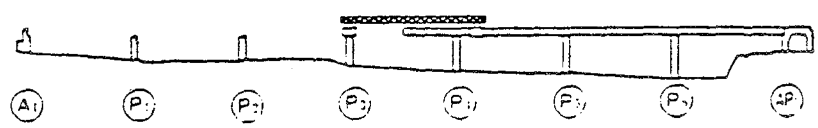
3rd:Stretching is carried out from the first segment to the fourth one in a balanced-cantilever pattern from atop P6 with overhead truss.



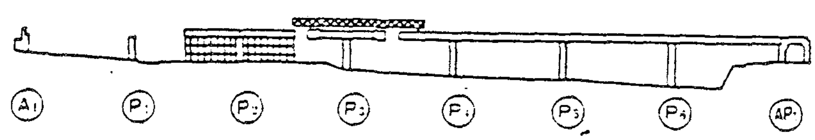
4th:The launching truss is then advanced after the concrete at the closure pour has attained sufficient strength and posttensioning of the continuity tendons.



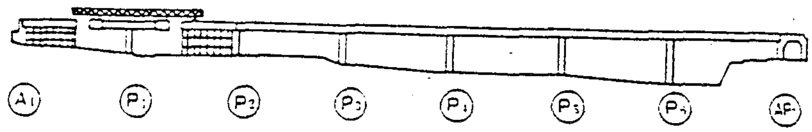
5th:A temporary fixed segment on P3 is constructed with overhead truss.



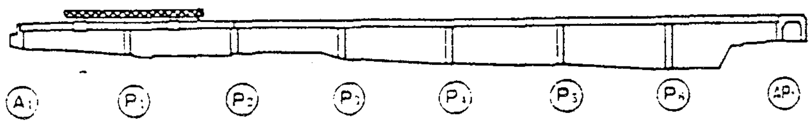
6th:Build the segments, the first to the fourth, from atop P3. The PC girder adjacent to P2 is constructed by the staging.



7th:Build the segments from the first to the fourth on the both sides of P1. The girder adjacent to A1 is constructed by the staging.



8th:Completed.



CHARACTERISTICS OF P&Z METHOD

1)Segments, 10 meter each, for the both ends of cantilevers are simultaneously constructed in ten days. Therefore, the progress of work is so efficient that a construction period can be reduced.

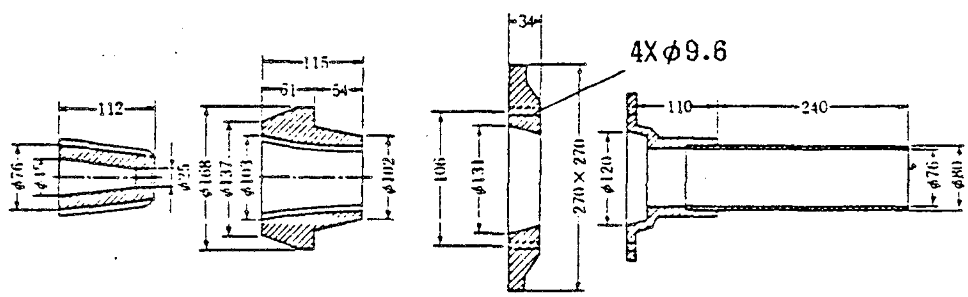
2)Since overhead launching truss spans from the finished part of the bridge to the next pier where the new cantilevers are to be started, no access roads to the intermediate piers are needed. Therefore, the methods is efficient and safe in case a bridge has to pass over a river with large flux or deep valley.

3)The P&Z can be applied to bridges with clothoid or variable cross section.

4)Since the method can adjust unbalanced moment at a pier under construction, that can adjust the reactions at piers to make desirable moment destribution in the bridge when the ends of opposite cantilevers are joined.

THE SELECTION AND DISTRIBUTION OF MAIN CABLES

Since a ten meter segment in the P&Z method is longer than that of usual cantilever methods, the amount of PC steel in one segment is 3 to 4 times larger than that of the usual methods suppose PC steel round bars are used. To avoid distribution problem on the section and complicated PC bar arragement, a multi-prestressing strand called Fressinet 12T12.4 is adopted. A tendon of 12T12.4 consists of twelve strands, 12.4 mm in diameter each, which are fixed by a set of Fressinet anchorage made of steel shown in Fig.5. The ultimate strength and yield one of the Fressinet 12T12.4 are 175 kg/sq.mm (195.6 tons/tendon) and 150 kg/sq.mm(166.8 tons/tendon) respectively.



cone cylinder bearing plate guide

Fig.5 A set of Fressinet anchorage

§2 THE NUMAO-GAWA BRIDGE

INTRODUCTION

The Numao-gawa Bridge shown in Fig.6 has to cross over a typical U-shape valley developed by the Numao river. It needs 700 meters in length and 47 to 67 meter high piers as well as an abutment on a steep slope.

A steel box type bridge is adopted by the following reasons:

(1) Since the bridge has to stand on the ground accumulated by the Numao river, a light weight structure is desirable.

(2) An infra-sound problem could occur according to bridge types since a residential area is close to the bridge.

More details on the box girder are as follows:

(a) A six continuous box girder is adopted considering driving condition, expansions and degree of indeterminacy for earthquake resistant design.

(b) A one box girder for each line is taken due to a simpler construction work.

(c) The depth of each girder is kept constant because of an incremental launching erection.

For the substructure, hollow piers with two cells each are used for high piers to make the weight as light as possible. High piers are built with reinforced concrete mixed with steel frame that develops a better ultimate strength, ductility and workabity.

The pier-foundation are built by pneumatic caissons of circle cross sections by the following reason. Since the caisson top stress in the longitudinal direction of the bridge is almost same as in the transversal direction, either square or circle is considered as a cross section. Considering amount of excavation, friction at sinking and possible rock stones, a circle cross section is adopted.

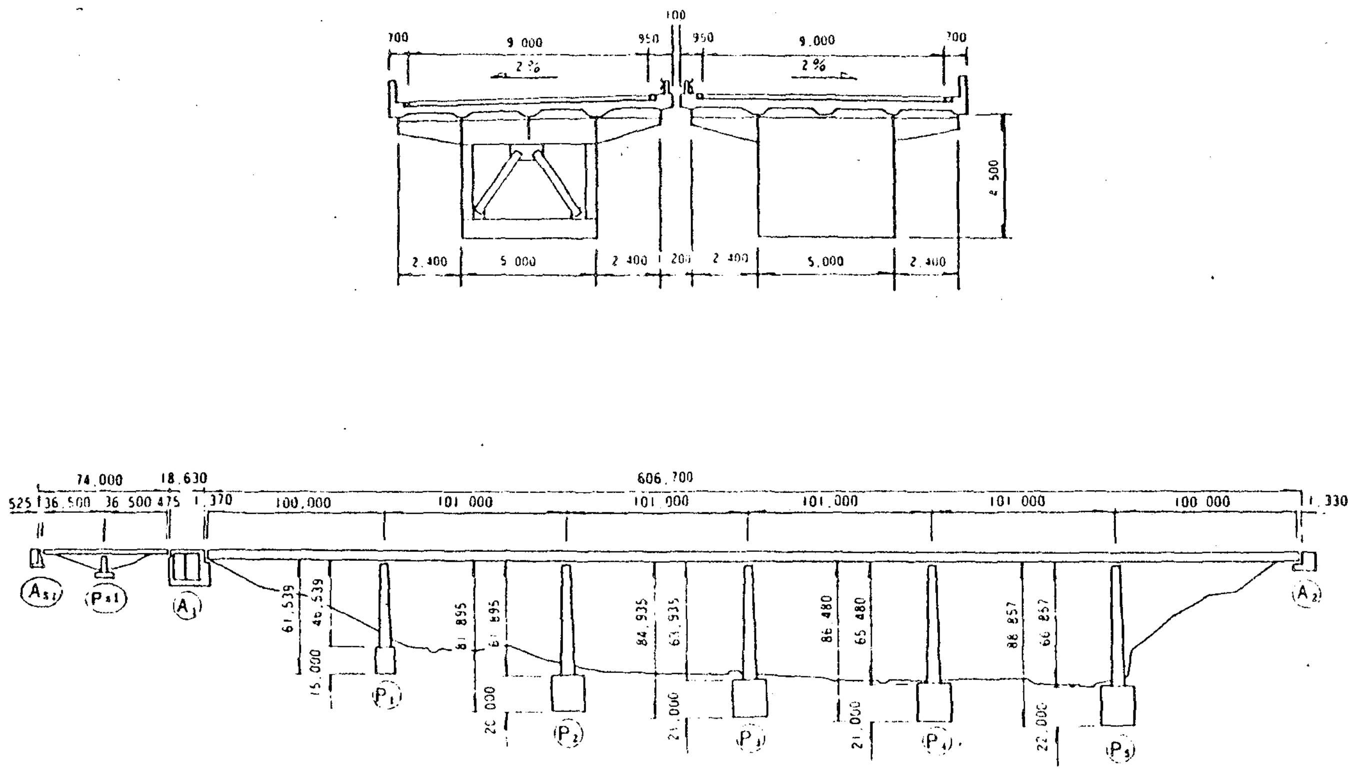


Fig.6 A general view of the Numao-gawa Bridge

DESCRIPTION OF THE PROJECT(SUBSTRUCTURE)

Name of the Highway:Kan-etsu Expressway

Name of the Project:Kan-etsu Expressway Numao-gawa Bridge Substructure Project

Location of the Project:

Seta County, Gumma Prefecture

Period of the Project

Construction Beginning: Feb 27, 1980

Completion: May 16, 1982

Total Cost:3,231,317,000YEN($13,464,000)

Table 3 Main materials of the Numao-gawa Bridge(substructure)

concrete	41,475 cubic m
reinforcing bar	4,266 tons
steel frame	1,338 tons

DESCRIPTION OF THE PROJECT(SUPERSTRUCTURE)

Name of the Highway:Kan-etsu Expressway

Name of the Project:Kan-etsu Expressway Numao-gawa Bridge Superstructure Project

Location of the Project:

Akagi Village, Seta County, Gumma Prefecture

Period of the Project

Construction Beginning: January 13,1982

Completion: December 27,1984 (1080 days)

Bid:3,930,000,000 YEN($16,375,000)

Materials supplied by the Corporation(reinforcing bar and steel plate):

648,804,000 YEN($2,703,350)

Total Cost:4,578,804,000 YEN($19,078,350)

Bridgere type:Steel road bridge

Structure type:Six-span continuous non-composite box girder + two-span continuous non-composite plate girder

Classification of the Bridge:1st(live load 20 tons, 43 tons)

Bridge Length: 677 meters=(box)+(plate)=(100+101x4+100)+(36.5x2)

Effective Width:Twin of 9.0 meters(four lanes)

Gradient:Longitudinal 0.95%

Transversal 2.0%

Slab:Reiforced concrete slab of 23 cm in thickness

Erection Method:Incremental launching method with steel nose(box girder)

Truck crane(plate girder)

Table 4 Main materials of the Numao-gawa Bridge(superstructure)

	box	plate	others	total
steel plate, etc	6,018 tons	247 tons		6,265 tons
high tensile bolt	458,000units	11,200units		469,200units
concrete	3,900cubic m	493cubic m	741cubic m	5,134cubic m
reinforcing bar	880 tons	106 tons	41 tons	1,027 tons
painting area	33,600 sq.m	4,400 sq.m		38,000 sq.m

EARTHQUAKE RESISTANT DESIGN

Since Japan is a typical country in the seismic zone with the addition of severe topography, all considerations for the countermeasures against earthquake must be taken in the planning and designing of bridges.

The simplest earthquake resistant design method is a seismic coefficient method, which determines an equivalent lateral earthquake force by a static horizontal inertia. The factors to determine the magnitude of the horizontal earthquake force are the seismic zone factor, ground condition factor and important factor of the structure.

When a structure has a relatively long natural period larger than 0.5 sec, we multiply the horizontal inertia force by the modifying coefficient in response to the natural period of the structure. It is called the modified seismic coefficient method shown in Fig.6-1.

This modified coefficient method is basically applied to the design of the Numao-gawa Bridge. The natural period for the modified seismic coeeficient method is obtained by solving a lumped mass model for the entire structure. Furthermore, the results are checked by the dynamic response analysis.

Since the piers have nearly the same heights except the pier No.1, a multi-fixed support system is adopted in order to make a structural system more flexible than a conventional one-fixed support system. Due to the fxexibility of the natural period over 2 sec, the design seismic force is able to be reduced considerably as follows:

Design seismic coefficient

longitudinal 0.18 (0.24 for shoes)

transversal 0.25

CONSTRUCTION OF PIERS

In order to construct very high piers over 60 meters, a large scale panel form with working platform is used not only because it does not need a conventional staging from the ground, but also it can take advantage of the steel frame in the pier. The panel form and working platforms can jointly climb along the wall to construct next segment. This is called a self-climbing form.

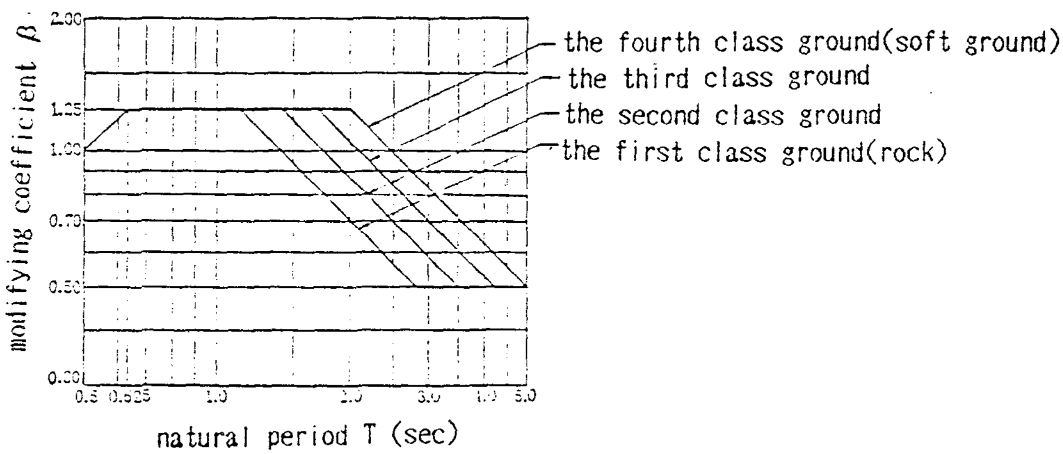


Fig.6-1 Modifying coefficient of design seismic coefficient

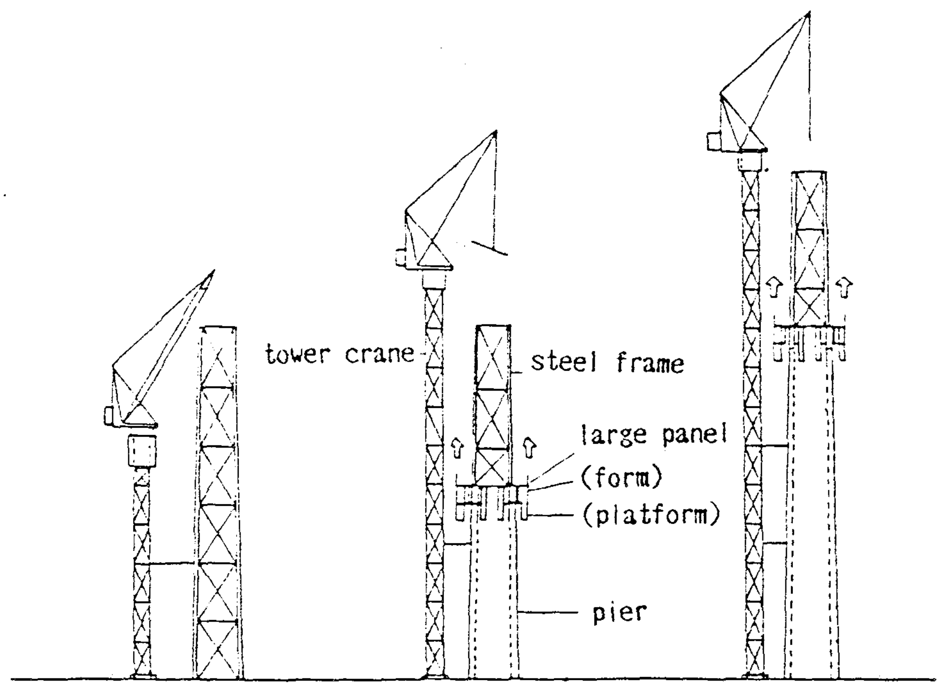


Fig.7 Framing work of the Numao-gawa Bridge

ERECTION BY INCREMENTAL LAUNCHING

The Numao-gawa is erected by launching the girder since other methods such as staging, crane and cable erection are not appropriate because of severe geographcal condition.

The girder is pushed out from the assembling yard incrementally with special jacks installed on the top of each pier. A steel-made launching nose is attached to the leading edge of the bridge girder to reduce the cantilever bending moment.

1) The cross section of one segment is subdivided into four parts, 12 to 13 meters long each, to be assembled by a gate type crane.
2) Since an assembling yard is 50 meters long, launching is carried out after three segments are joined with high tensile bolts.
3) The girder is launched 20 to 30 meters per one cycle. It takes 25 cycles and six months until a 600 meter girder reaches the opposite abutment.

After that, the entire girder of 3,000 tons for the down line is transfered transversally from the up line to the down line. Then, the girder for up line starts to be launched. Thus, we need the assembing yard only for the up line.

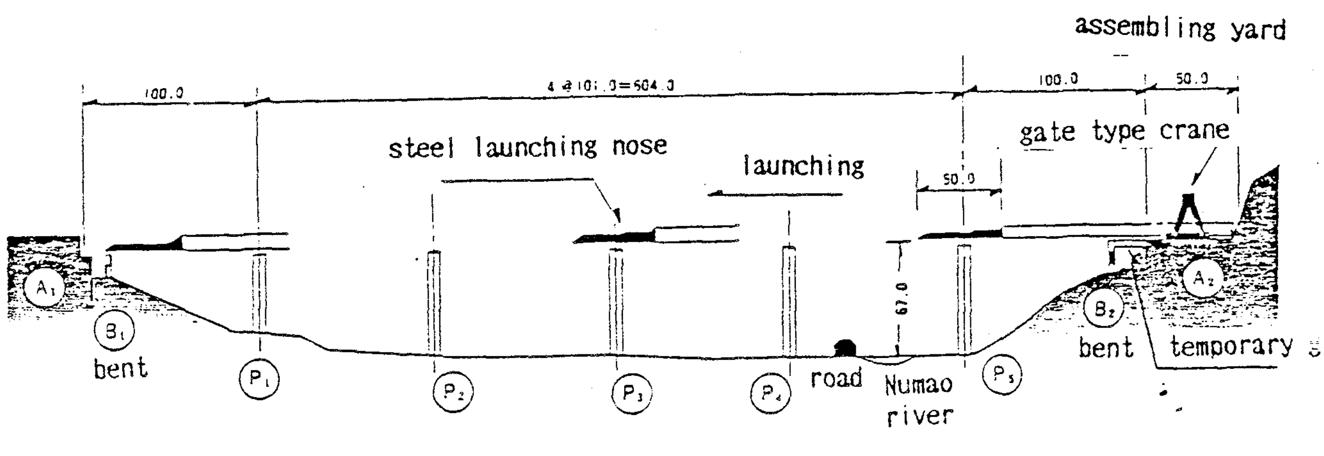


Fig.8 Schematic sequence of incremental launching of box girders

§3 THE KATASHINA-GAWA BRIDGE

INTRODUCTION

The Katashina-gawa Bridge shown in Fig.9 has to cross over a huge U-shaped valley with a wide and flat bottom 100 meters lower than the river terrace. It, therefore, needs very high piers whose maximum height is 70 meters, as well as long spans the maximum of which is 168 meter long. That is one of the longest bridges ever built in Japan.

The truss type steel bridge is determined by the following reasons:

(1) Since the bridge has to stand on the ground accumulated by the Katashina river, a light weight structure is desirable.

(2) As far as truss members are concerned, the transportation is favorable compaired with the other types because simpler and smaller members can be used for the construction.

(3) It comparatively needs less steel.

(4) Since a truss girder usually needs a larger depth, a pier height can be reduced.

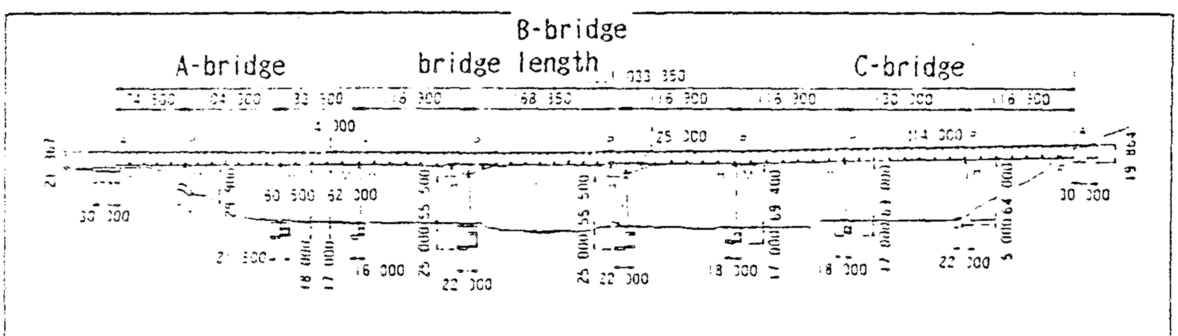


Fig.9 A general view of the Katashina-gawa Bridge

Furthermore, an arrangement of main girders is studied on the following types as shown in Fig.10.

The two main girder type to support up line and down one shown in Fig.10(a) has the following characteristics:(actually adopted in the design)

(1) A stress destribution can be clarified because total loads are supported by the two main girders.

(2) It needs the least steel among the three types in Fig.10.

(3) A camber adjustment is simple during construction and makes the erection period shorter.

(4) It is stable against lateral loads, however, needs larger lateral members.

The three main girder type to support both up line and down one shown in Fig.10(b) is as follows:

(1) A load distribution is not so clear.

(2) A camber adjustment is complicated and needs a longer erection period.

(3) It demands more steel than two truss girder case shown in Fig.10(a).

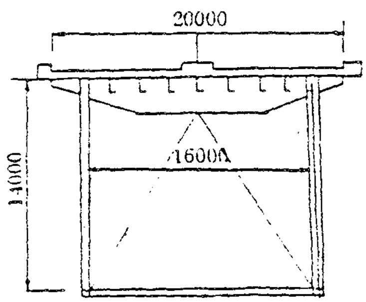
(4) Smaller members can be used compaired with the two truss girder case in Fig.10(a).

The two main girder type to support up line and down one independently (Fig.10(c)) is as follows:

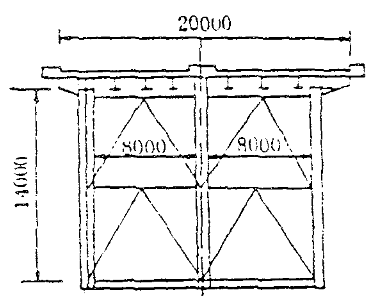
(1) It requires more steel than the former two cases.

(2) Since space between the truss girders is not enough compaired with the spans, it need special countermeasures against lateral loads.

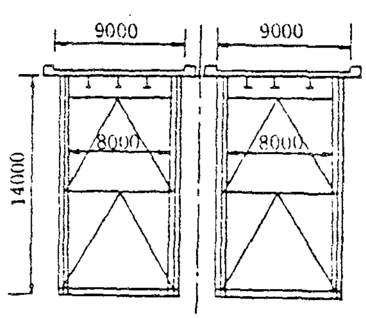
(3) An erection is comparatively simple, however it must be made for two lines and takes a longer time.



(a) Two main girders for up line and down one



(b) Three main girders for up line and down one



(c) Two main girders for each line

Fig.10 Types of arrangements of girders

DESCRIPTION OF THE PROJECT

Name of the Highway:Kan-etsu Expressway

Name of the Project:

Kan-etsu Expressway Katashina-gawa Bridge Left Bank Side Substructure Project

Kan-etsu Expressway Katashina-gawa Bridge Right Bank Side Substructure Project

Kan-etsu Expressway Katashina-gawa Bridge Superstructure Project

Location of the Project:

From Showa Village, Gumma Prefecture

To Numata City, Gumma Prefecture

Period of the Project

The Left Bank Side Substructure: September, 1981 ~ November, 1983 (27 months)

The Right Bank Side Substructure:March, 1981 ~ June, 1983 (27 months)

The Superstructure:Nomvember, 1981 ~ March, 1985 (41 months)

Construction Cost

Superstructure: 6,760,000,000 YEN($28,167,000)

327,000 YEN/sq.m($1,363/sq.m)

Substructure: 6,400,000,000 YEN($26,667,000)

310,000 YEN/sq.m($1,292/sq.m)

Bridge length:1,033.85 meters

Effective width:Twin of 9.0 meters(four lanes)

Minimum radius of curvature:R=1,000 meters

Longitudinal gradient:2.70% ~ -2.27%

Type of the superstructure:

Three sets of three span continuous steel truss bridge (from Tokyo, A bridge, B bridge and C bridge)

Type of the substructure

Box abutment for A1 and A2 abutment

Wall type hollow pier with two cells for P1～P8

(reinforced concrete structure mixed with steel frame)

Cast-in-place concrete pile of 4 meters in diameter for A1 and P1

Pneumatic caisson foundation for P2～P7

Direct foundation for P8 and A2

Span

A bridge 262.3 m = 74.50+104.30+83.50

B bridge 402.65 m = 116.90+168.85+116.90

C bridge 363.80 m = 116.90+130.00+116.90

Depth of main girder

Parallel parts H=14.0 m

At support of P4 and P5 H=25.0 m

Space between main girders B=16.0 m

Table 5 Main materials of the Katashina-gawa Bridge

	superstructure	substructure	total
concrete	6,600 cubic m	83,000 cubic m	89,600 cubic m
reinforcing bar	1,500 tons	7,600 tons	9,100 tons
steel	10,290 tons	1,600 tons	11,890 tons

EARTHQUAKE RESISTANT DESIGN

The modified seismic coefficient method explained in the section of the Numao-gawa Bridge is basically applied to the design of the Katashina-gawa Bridge. Natural periods are obtained solving an eigen-value problem for lumped mass model. For the analysis of the longitudinal direction, A, B and C bridge are analized independently, however in the transversal direction, the entire bridge which consists of A, B and C bridge is considered to solve an eigen-value problem.

Design natural period and modified seismic coefficient

longitudinal			
	A bridge	0.893 sec	0.25
	B bridge	1.848 sec	0.20
	C bridge	1.216 sec	0.25
Transversal			
		1.55 sec	0.25

Since it could behave unlike the modified seismic coefficient method, it is reviewed by the dynamic response analysis as well as the vibration test of 1/100 experimental model. Furthermore, a vibration test by the oscillator was carried out for pier P5 and P6 in order to check the theoretical results and design parameters.

DESIGN OF SUPERSTRUCTURE

Since the truss girder is not straight in the longitudinal direction, in other words, changes the direction at piers, the structure is analized three-dimentionally for the system under construction as well as completed system.

ON PIERS

Piers are connected to the superstructure by hinges. A hollow type with two cells is adopted to make the structure flexible. Since piers from P2 to P8 are 55 to 70 meters high, reinforced concrete structure mixed with steel frame is applied. The structural analysis of steel frame is carried out by the theory of equivalent amount of reiforcing bar. The maximum displacement at the top of a pier is 16 cm. Pier P2 and P5 are shown in Fig.11.

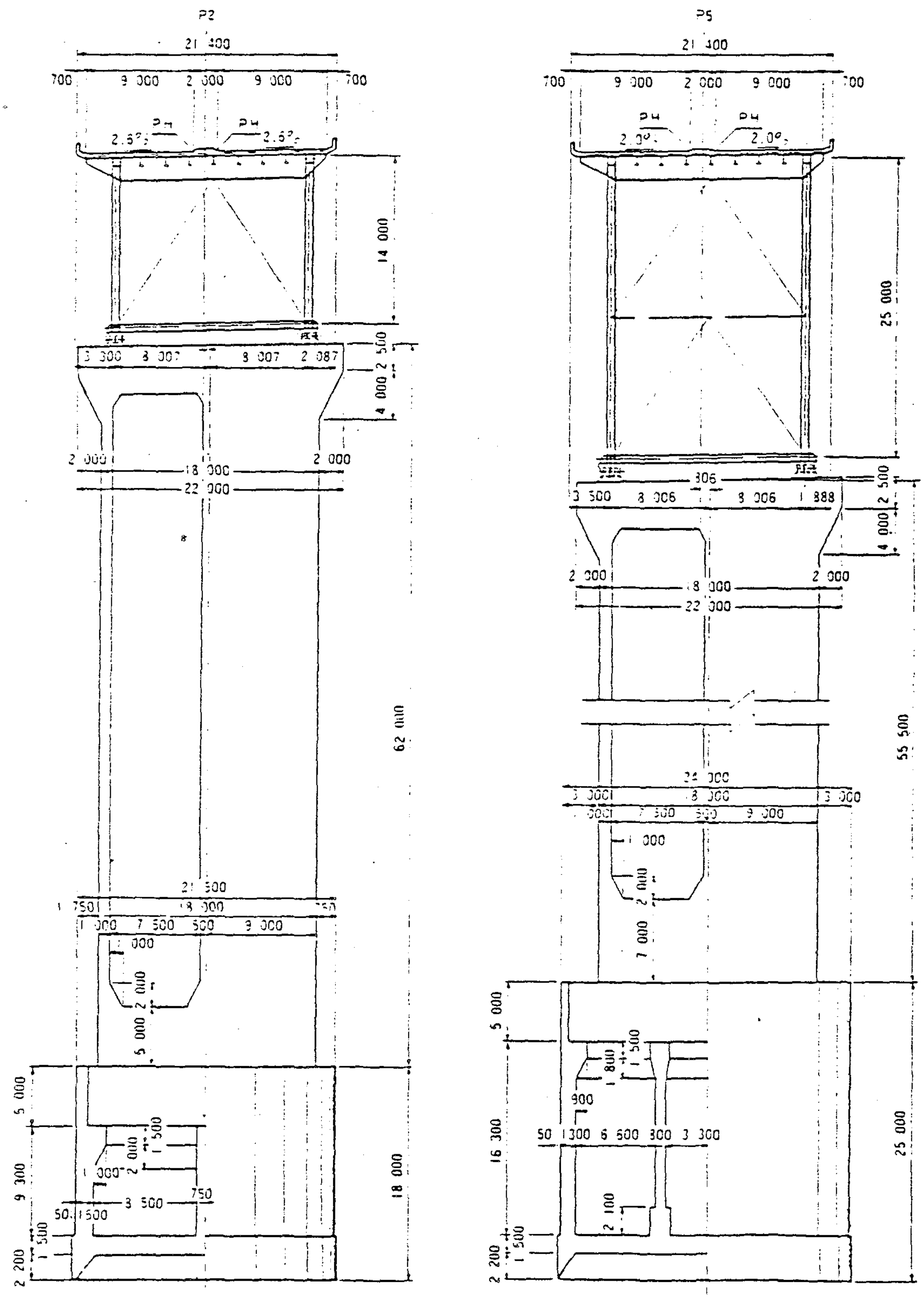


Fig.11 Pier P2 and P5

A cantilever erection by traveller cranes and large supports is adopted not only because the girder is 55 to 70 meters from the ground but also spans are over 100 meters.

The procedure of erection is shown as follows:

(1) From A_1 abutment to P_1 pier (From A_2 abutment to P_8 pier), the erection is carried out by staging and traveller cranes, so that the members of truss is free from the stresses during erection. (Fig.A)

(2) From P_1 pier to P_3 pier (From P_8 pier to P_6 pier), a cantilever erection is carried out by one large tower which supports the girder at the center of each span. (Fig.B, Fig.C)

During cantilever erection, the girder is uplifted at each support so that the girder can reach the top of the next pier.

(3) After the girder has reached P_3 pier (P_6 pier), the girder is jacked down so that the stresses, which the members of girder have got during cantilever erection, are reduced to those of all staging state. (Fig.D)

(4) On P_3 pier (P_6 pier), a member of B bridge which is shown in Fig.9 is connected with a member of A bridge (C bridge), which has already been constructed.

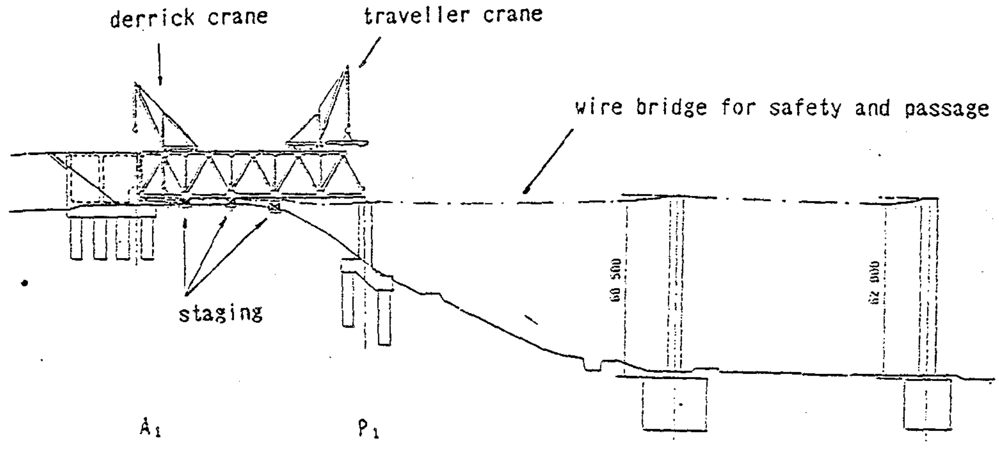
And then, from P_3 pier to P_4 pier (from P_6 pier to P_5 pier), the girder is erected by a large tower and traveller cranes as well as the erection of the girder from P_1 pier to P_3 pier. (Fig.E)

(5) After the girder has reached P_4 pier (P_5 pier), the connections of B bridge with A bridge (C bridge) is released.

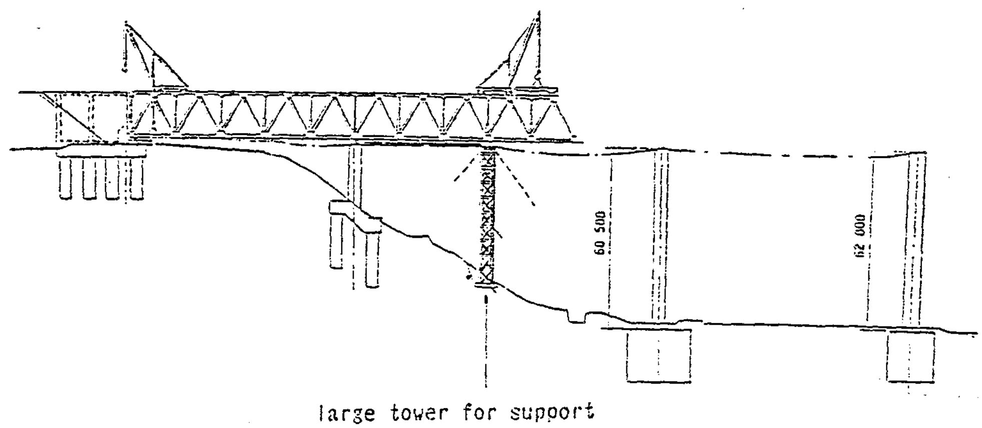
And then the girder is erected in cantilever state with no support.

The girder in cantilever state is balanced with the part of the girder between P_3 pier and P_4 pier (P_6 pier and P_5 pier), which has already been erected. (Fig.F)

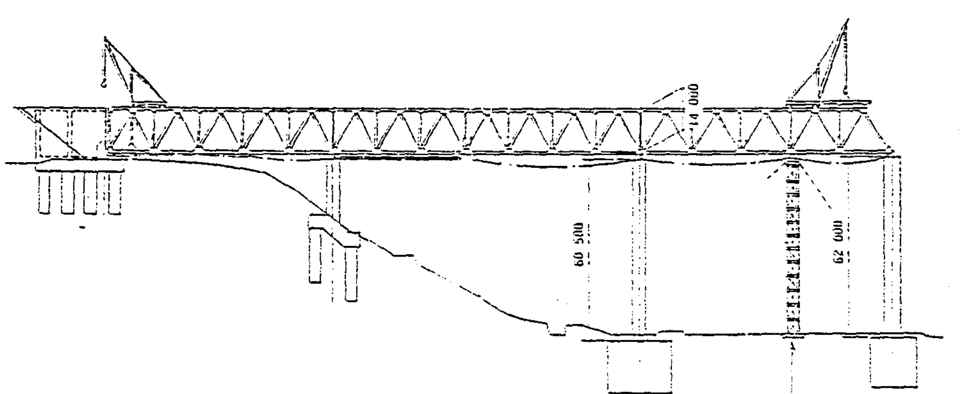
A bridge



(Fig.A)

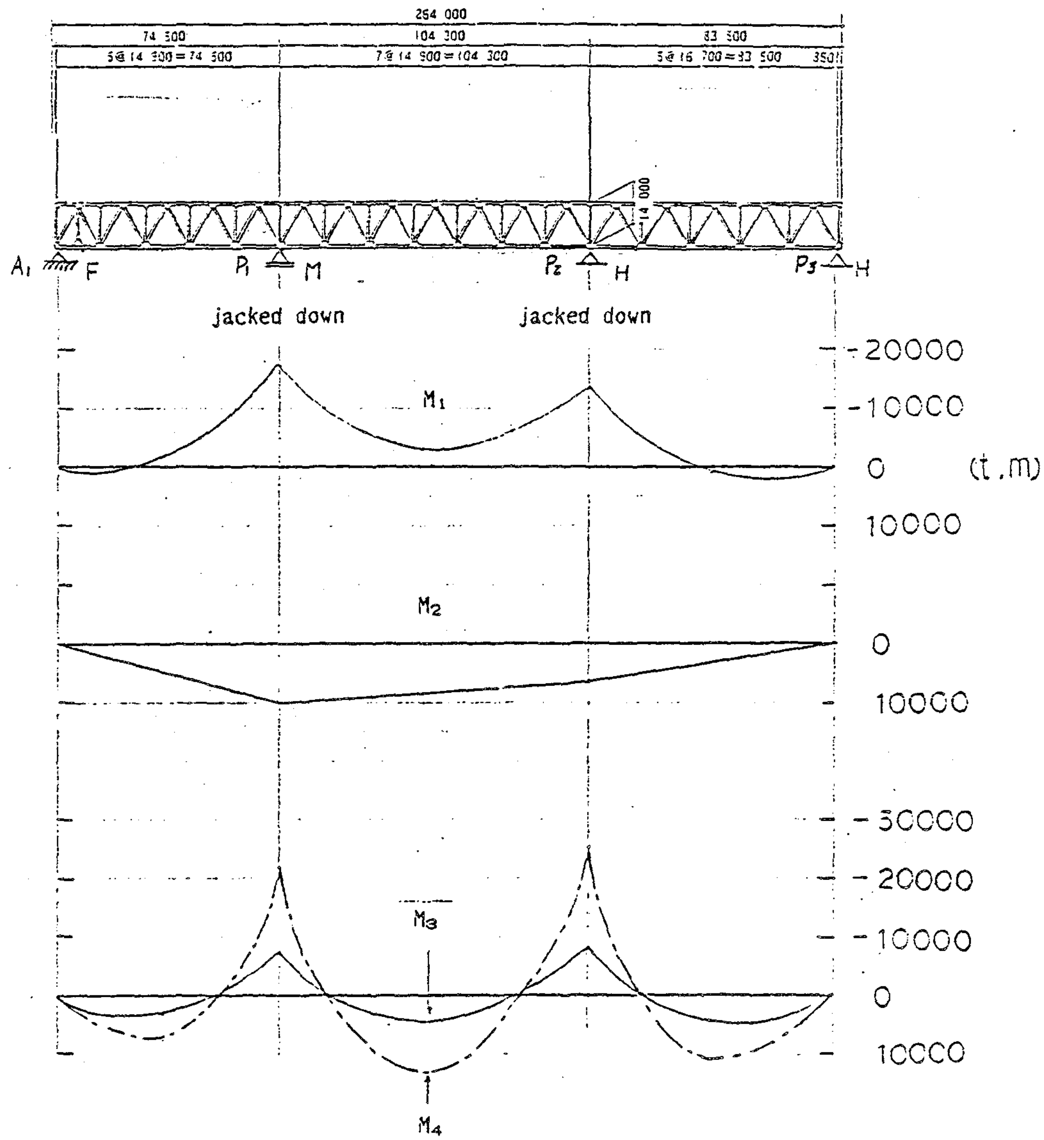


(Fig.B)



(Fig.C)

A bridge



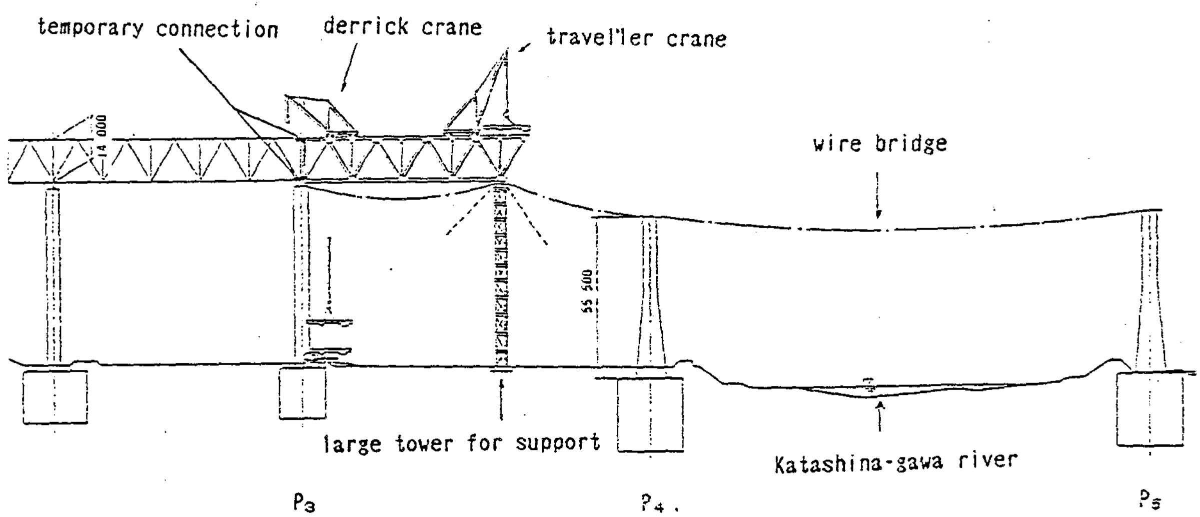
M_1;The moment just after the girder has reached P_3

M_2;The moment which is reduced by jack down

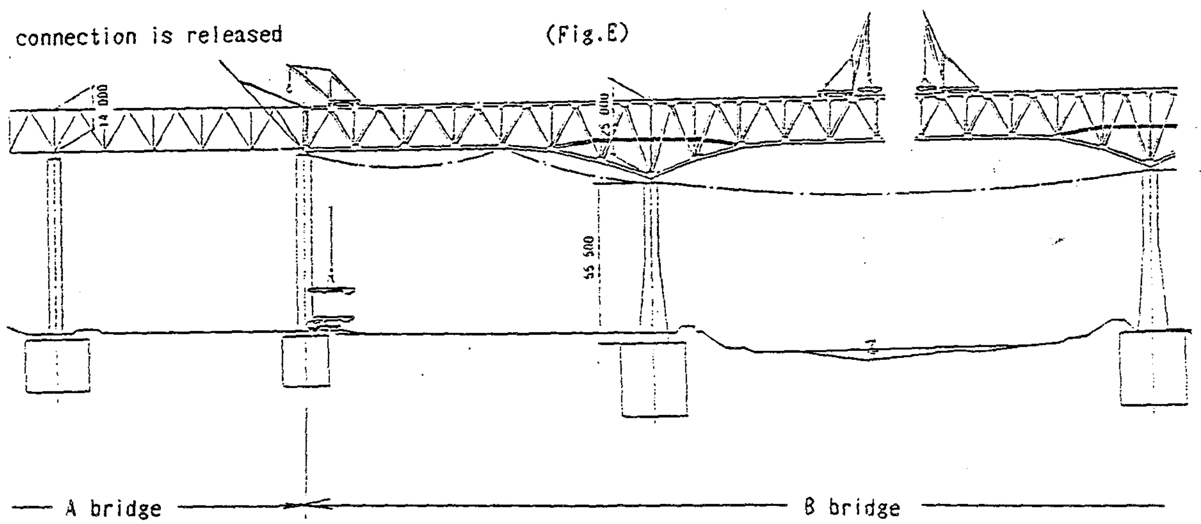
M_3;The moment just after the girder has been jacked down

M_4;The moment just after slab-concrete has been cast

(Fig.D)


temporary connection
derrick crane
traveller crane
wire bridge
14 000
55 500
large tower for support
Katashina-gawa river
P3
P4
P5

(Fig.E)


connection is released
14 000
25 000
55 500
A bridge
B bridge

(Fig.F)

INTRODUCTION

The Nagai-gawa Bridge shown in Fig.13 will pass over a huge and steep V-shaped valley, 500 meters wide and 100 meters deep, developed by the Nagai River. This will make pier P2 shown in Fig.14 the highest in Japan.

DESCRIPTION OF THE PROJECT

Name of the Highway:Kan-etsu Expressway

Name of the Project:

Kan-etsu Expressway Nagai-gawa Bridge Substructure Project

Kan-etsu Expressway Nagai-gawa Bridge Superstructure Project

Location of the Project:

Showa Village, Tone County, Gumma Prefecture

Period of the Project

The substructure:January, 1982 ~ July, 1984

The superstructure:March, 1983 ~ July, 1985

Construction Cost

The substructure:4,757,066,980 YEN($19,821,000)

The superstructure:2,839,500,000 YEN($11,831,000)

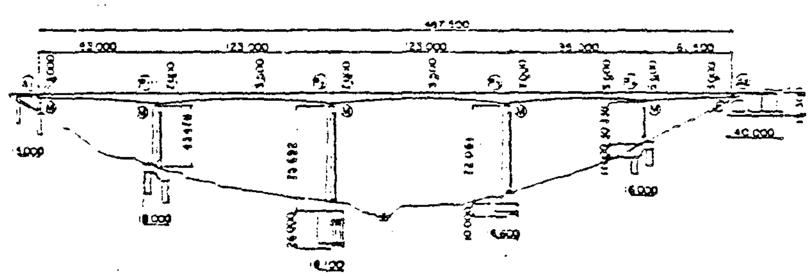


Fig.13 A general view of the Nagai-gawa Bridge

Table 6 Main materials of the Nagai-gawa Bridge

	superstructure	substructure
concrete	11,800 cubic m	57,600 cubic m
prestressing bar	924 tons	
reiforcing bar	940 tons	5,500 tons
Steel frame		1,240 tons

SUPERSTRUCTURE

A prestressed concrete five span continuous box girder is selected not only because the ground foundation can stand a heavier concrete structure but also because an infra-sound problem could occur according to the bridge type.

An erection will be conducted by a balanced cantilever method called Dywidag Method in Fig.14 which extends a prestressed concrete girder block from the central support to keep the balance between left portion and right one.

SUBSTRUCTURE

Hollow piers with two cells in each pier are used for high piers to reduce the weight. They are being built with reiforced concrete with steel frame to assure excellent ultimate strength, ductility and workability. The foundations for pier P1 and P4 are constructed by cast-in-place concrete piles on the steep slopes. The foundation for pier P2 is built by a huge pneumatic caisson whose area is 860 sq.m, on the other hand the foundation for pier P3 is made by an open caisson method since no water is expected.

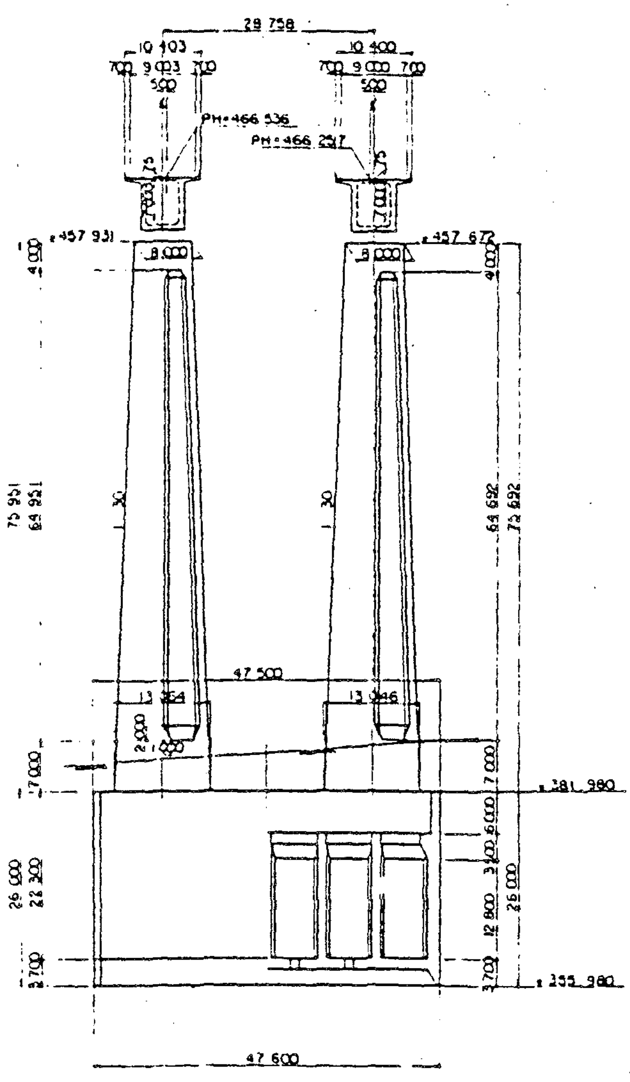


Fig.14 Pier P2 of the Nagai-gawa Bridge

which will be the highest in Japan

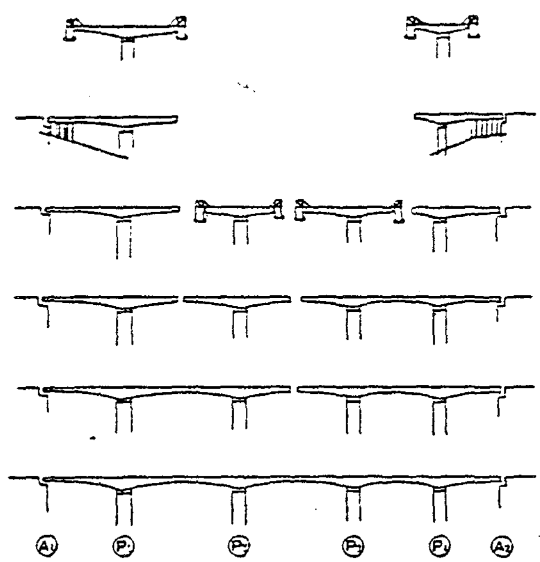


Fig.15 Dywidag Method(Balanced Cantilever Method)

for the Nagai-gawa Bridge

Itinerary of U. J. N. R. Site Inspection for Japan Highway Public Corporation

2/23 (THU)

8:00 —————— 10:30 ——(Yukemuri 3rd)—— 11:27
Tsukuba — Omiya — Takasaki (East exit)

12:00 —————— 13:00 —————————— 17:30 Minakami

Kan-etsu Expressway Construction Sites

(Lunch) (Dinner)

13:00	Tonegawa Bridge
	Kurinokigawa Bridge
14:00	Numaogawa Bridge
15:00	Nagaigawa Bridge
30	Katashinagawa Bridge
	Usunegawa Bridge
	Okutone Bridge
17:30	Minakami

HOTEL:Minakamikan
(02787-2-3221)

2/24 (FRI)

8:00 —————— 9:16 ——Jo-etsu Shinkansen —— 10:10
Hotel — Jyomokogen — Omiya