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

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

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

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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USA BIOGRAPHICAL SKETCHES

RICHARD W. ARDEN, P.E.

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.

MR. ARDEN HAS CONSIDERABLE EXPERIENCE IN THE EVALUATION OF FEASIBILITY OF LARGE PROJECTS AS WELL AS SITE SELECTION AND PLANNING.

MR. ARDEN IS A GRADUATE OF THE UNIVERSITY OF NEVADA WITH A B.S. AND M.S. DEGREE IN CIVIL ENGINEERING. HE IS A REGISTERED PROFESSIONAL ENGINEER IN NEVADA, CALIFORNIA, AND WASHINGTON. HE IS PAST PRESIDENT OF THE NEVADA SOCIETY OF PROFESSIONAL ENGINEERS AND NEVADA SECTION A.S.C.E. HE WAS NAMED "ENGINEER OF THE YEAR" BY THE RENO CHAPTER AND NEVADA SOCIETY OF PROFESSIONAL ENGINEERS AND IS LISTED IN "WHO'S WHO IN ENGINEERING."

Jim Dodson
Chief Bridge Engineer
Nevada Department of Transportation
1263 S. Stewart Street
Carson City, NV 89712

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
Center for Civil Engineering Earthquake Research
University of Nevada-Reno
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.

Biographical Data

DEPARTMENT OF TRANSPORTATION

1035 PARKWAY AVENUE
TRENTON, NEW JERSEY 08625

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.

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

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."

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.

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.

JAPAN BIOGRAPHICAL SKETCHES

PERSONAL CAREER

NAME : Nobuyuki NARITA

POSITION : Director
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Public Works Research Institute
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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,Faculty 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: Shun-etsu ODAGIRI

POSITION: Senior Engineer
Planning Division
Planning and Administration Department
Public Works Research Institute
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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 : Kazuya OHSHIMA

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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 : Kunio YAMAMOTO

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DATE OF BIRTH : October 15, 1942

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EDUCATION : Master of Civil Engineering
Nagoya University

SPECIALITY : Structural Engineering

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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 : Shoichi SAEKI

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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 : Kazuo SATO

POSITION : Director
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Ministry of Construction

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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.
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MAJOR SUBJECTS : Road Planning
Poor Ground Treatment
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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
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7) "Effects of Damping Ratio on Earthquake Response Spectra", Proc. of JSCE, 1983

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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
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7) "Soil Liquefaction Potential Evaluation with Use of the Simplified Procedure," do.

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1982-	Structure Section of Tokyo Second Construction Bureau Japan Highway Public Corporation

MAJOR SUBJECTS AND SUMMARY:
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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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Bridge Engineering
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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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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

JAPAN PAPERS

J-1 TO J-20

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.

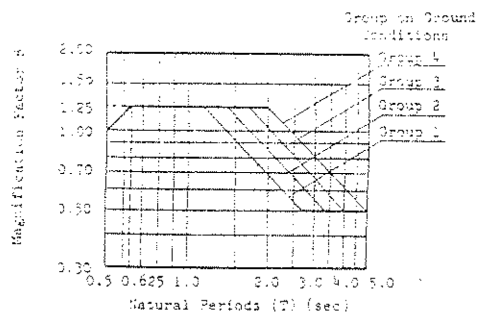

Fig. 1 Magnification Factor for the Modified Seismic Coefficient Method

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

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.

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 – $\frac{1}{2}$	$\frac{1}{2}$ – 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	1 – 2	2 – 3	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)

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.

tral 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)

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

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

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

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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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.

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	G_s	–	2.704
Optimum Moisture Content	w_{opt}	%	15.5
Maximum Dry Density	$\gamma_{d,max}$	gr/cm^3	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

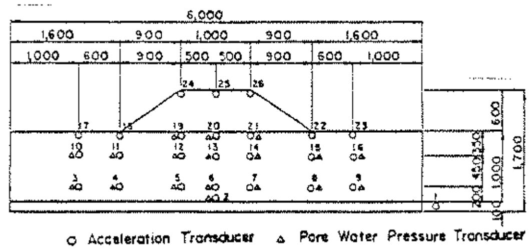

Fig. 1 Ground and Embankment Model

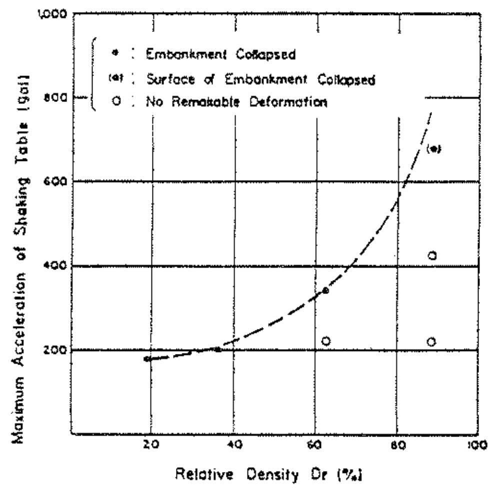

Fig. 2 Relation between Relative Density of Ground and Maximum Acceleration of Shaking Table

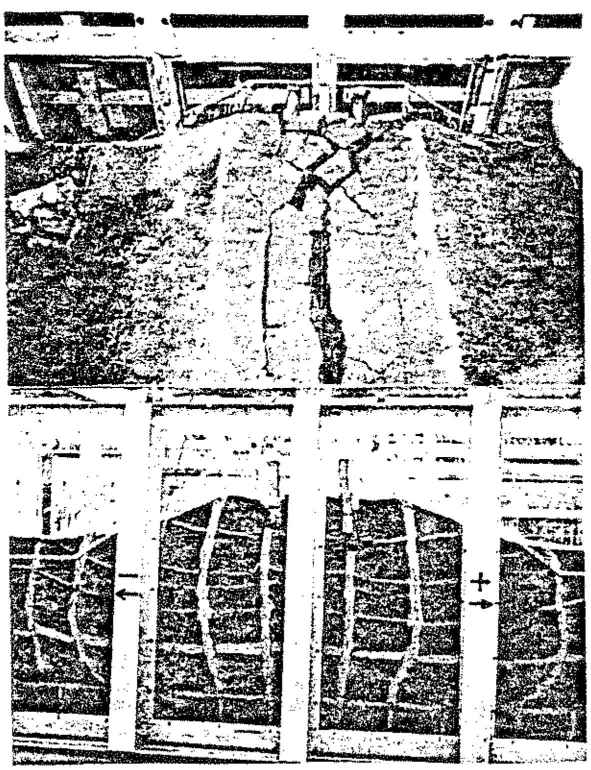

Photo. 1 Failure Mode of Case 1

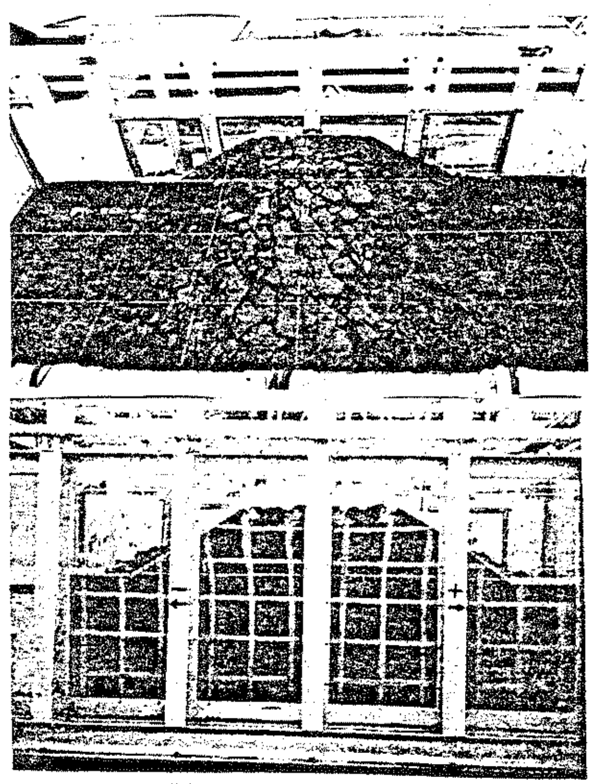

Photo. 2 Failure Mode of Case 4

dynamic response analysis. 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).

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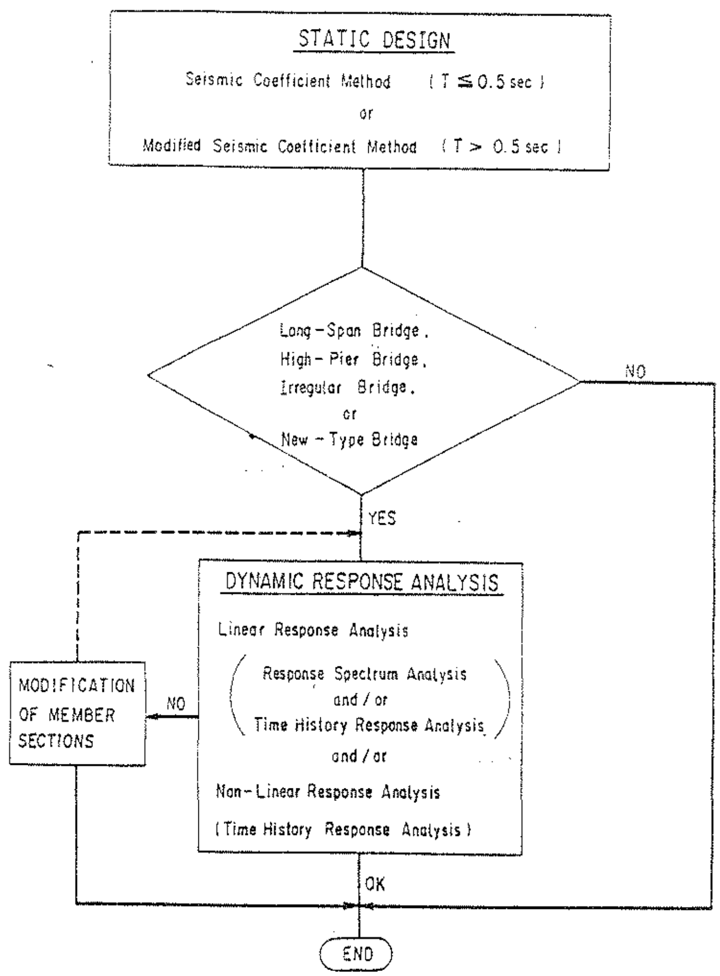

Fig. 1 Seismic Design of Highway Bridges

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 ($\delta_a = 1/3 x \delta_u$) 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 $K_{hd} \geqq 1.3 x K_h$

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

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

where $W=W_u+0.3\times W_p$

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).

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

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, $K_{\mu a}=\delta_a/(10\times\delta_{0.1})$ where $\delta_{0.1}$ is the displacement when holizontal acceleration is 0.1g. If $K_{\mu a}$ is greater than $1.3\times K_h$, 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

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.

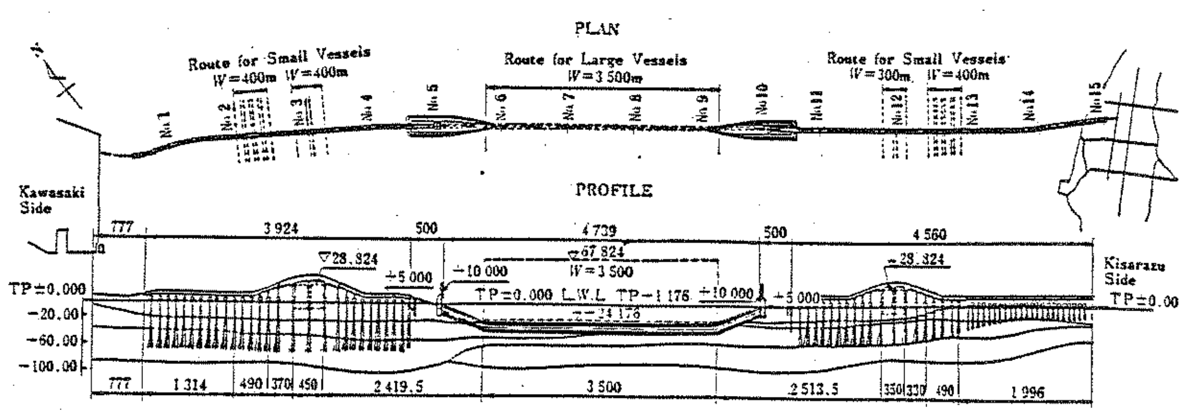

Fig-1. General View of Tokyo Bay Crossing Bridge & Tunnel

J-8

EARTHQUAKE RESISTANCE DESIGN OF THE MEIKO NISHI BRIDGE

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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.).

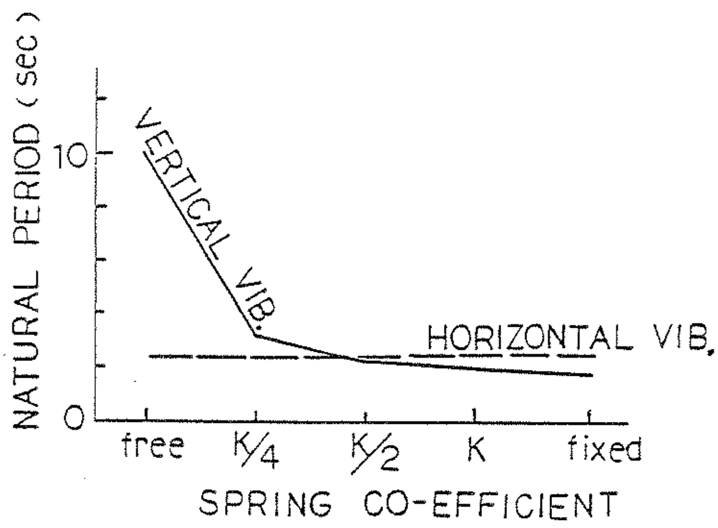


Fig. 1 NATURAL PERIOD OF THE SUPERSTRUCTURE

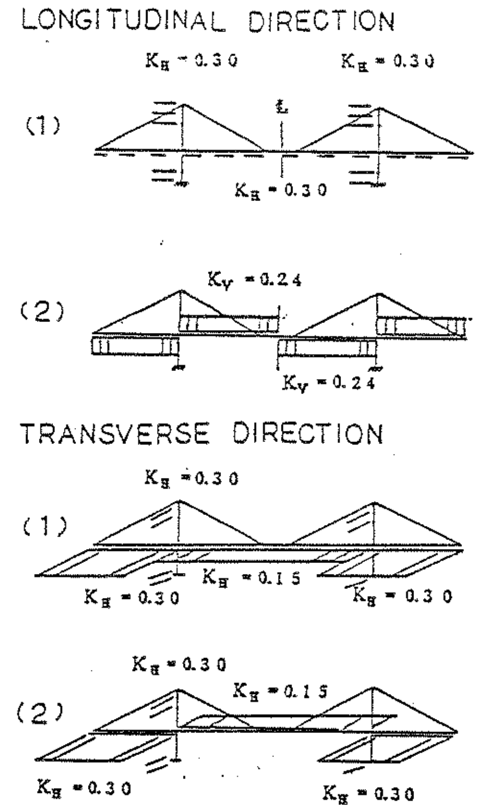


Fig. 3 DESIGN SEISMIC COEFFICIENT

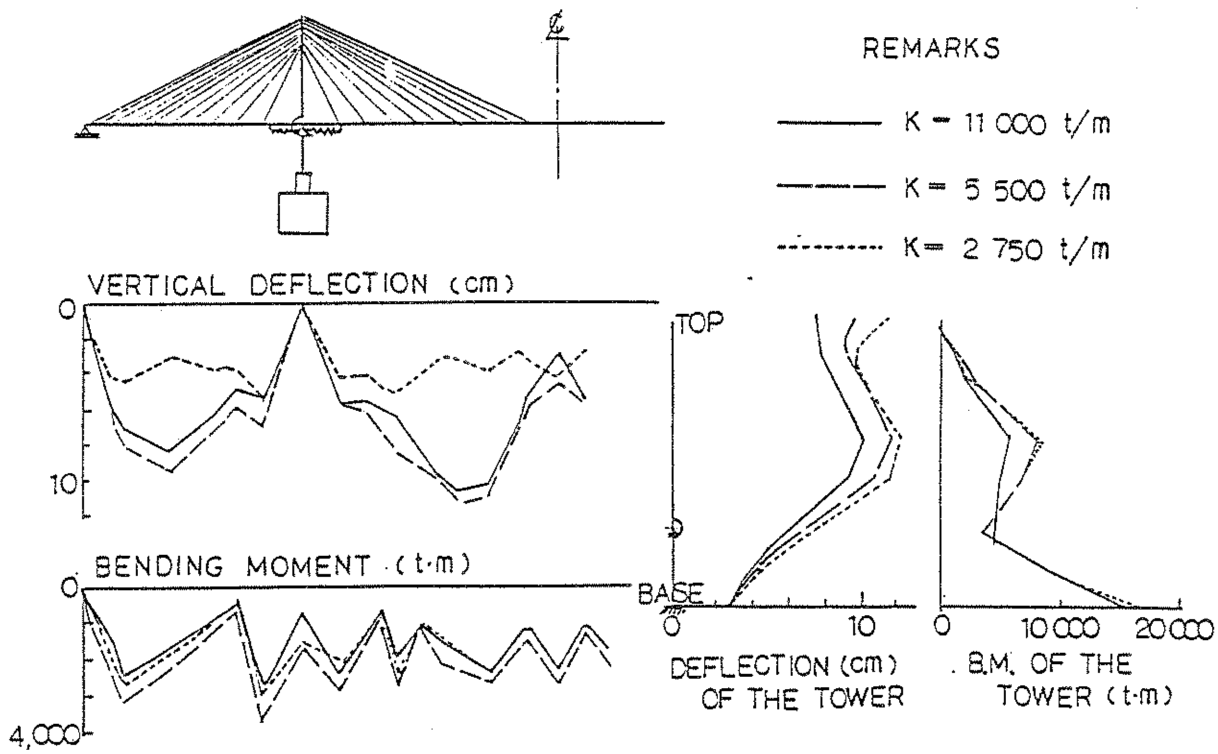


Fig. 2 DYNAMIC RESPONSE OF SPECTRUM ANALYSIS

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.

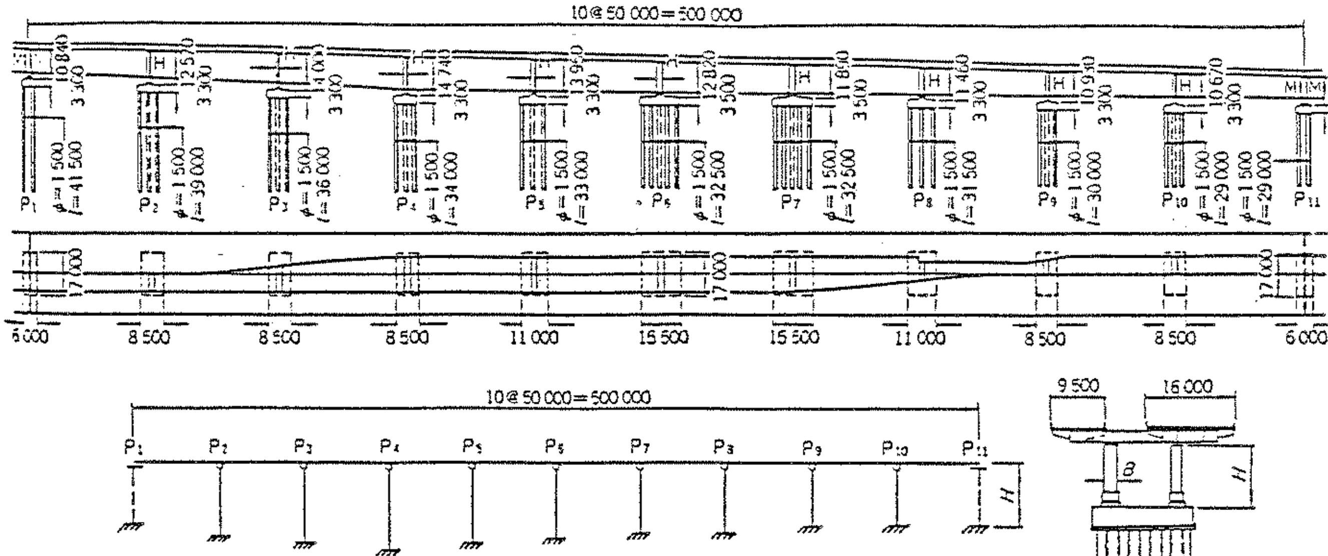
DYNAMIC ANALYSIS RESULTS

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.



A×B	2.5×3.0	1.0×2.0	1.0×2.0	1.5×2.0	2.0×2.0	2.5×2.0	2.5×2.0	2.0×2.0	1.0×2.0	1.0×2.0	2.5×3.0
H	11.25	2.93	14.36	15.10	14.32	13.38	12.45	11.32	11.29	11.03	10.76

A: Longitudinal dimensions (m)
B: Lateral dimensions (m)

Figure 1 : Outline of 10 Span Continuous Bridge

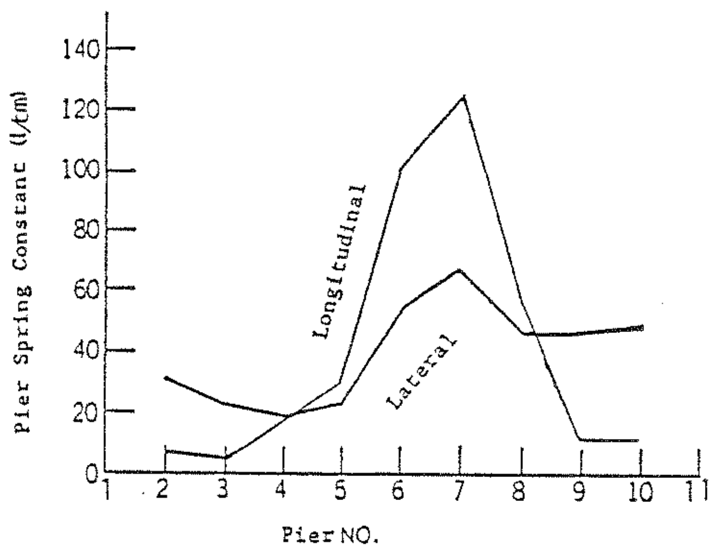


Figure 2 : Distribution of Pier Spring Constant for 10 Span Continuous Girder Bridge

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

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H. Nakajima

J-10

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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 acceleration response spectrum on the top of the footing

existing design response spectra

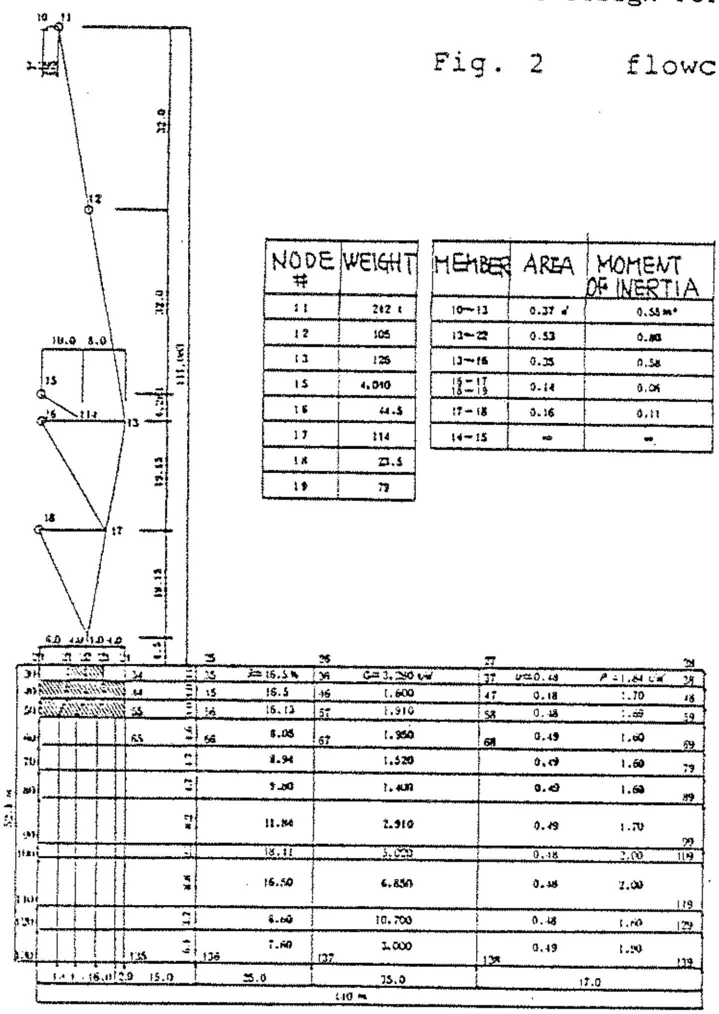
comparison and evaluation

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	4,010
16	44.5
17	114
18	23.5
19	73

MEMBER	AREA	MOMENT OF INERTIA
10~13	0.37 m^2	0.55 m^4
13~22	0.53	0.80
13~16	0.35	0.58
16~17 18~19	0.14	0.06
17~18	0.16	0.11
14~15	∞	∞

Fig. 3

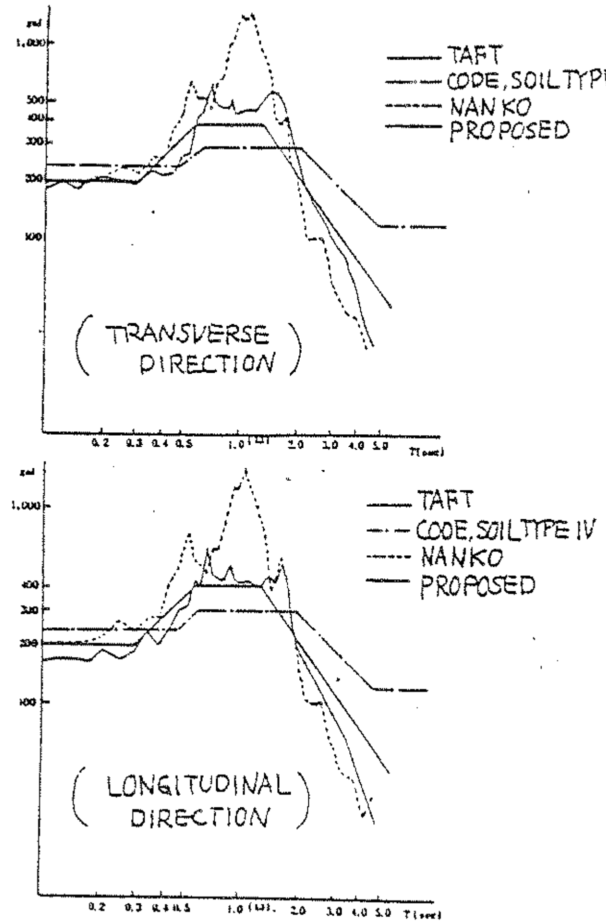


Fig. 4

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1) TAKEMOTO, C & et al, "Design of Cable Stayed Bridge-Ajigawa Bridge", Bridge & Foundation vol. 3, 1980, in Japanese

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

(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/ $4\sqrt{2}$ model of upper part to determine M-Ø curve and hysterisis and I/IO model of lower part to determine the rigidity of I section pier.

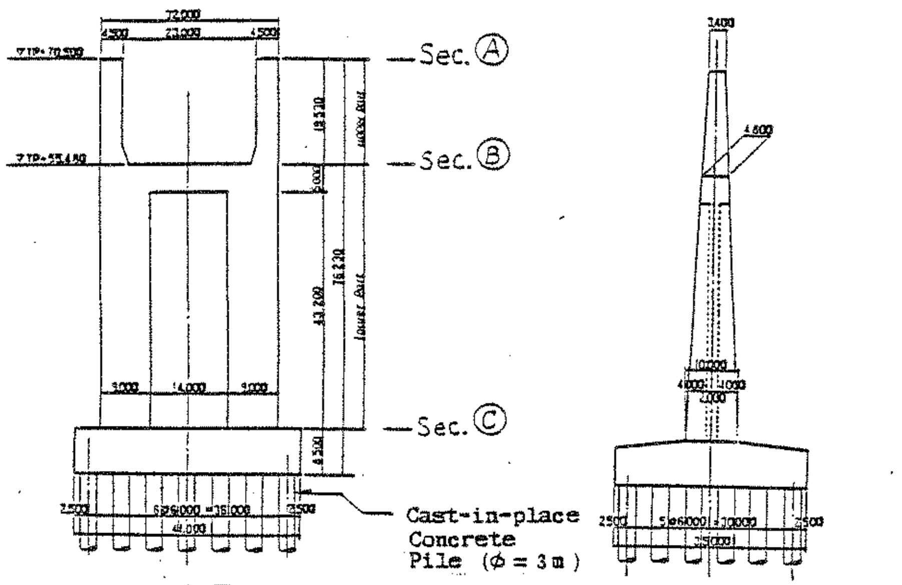

Fig. 1 General View of the Pier of Bannosu Viaduct (4 P)

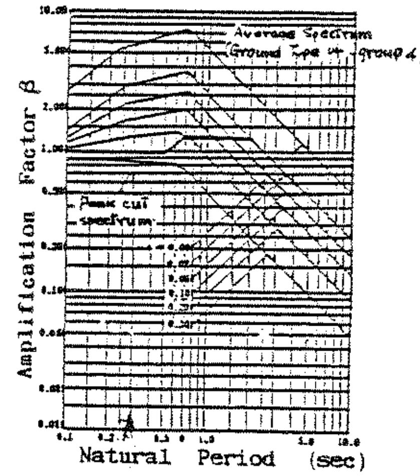

Fig. 2 Response Spectrum Curve

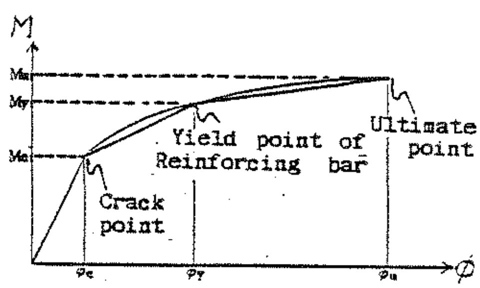

Fig. 3 M-Ø Curve

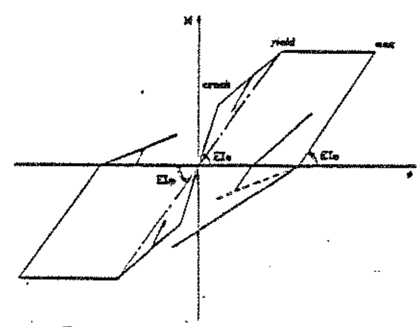

Fig. 4 Hysterisis Curve

(a) Concrete (b) Reinforcing Bars

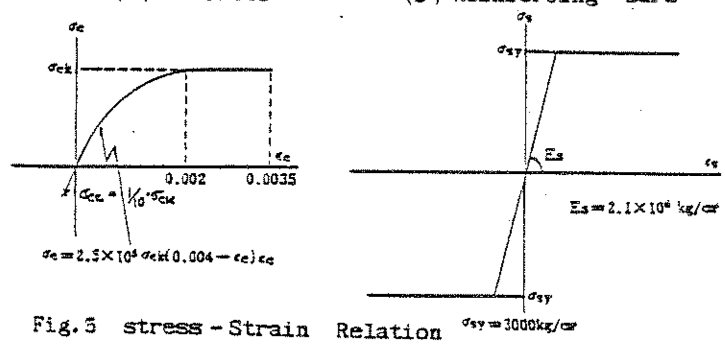

Fig. 5 stress - Strain Relation

FUTURE PROGRAMS AT CONCRETE DIVISION, PWRI

J-12

by Shigetoshi KOBAYASHI

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")

J-13

Research Plan on Earthquake Resistance by Foundation Engineering Division

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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).

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.

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

J-15

FUTURE PROGRAM

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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

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 specifications1), 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.

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.

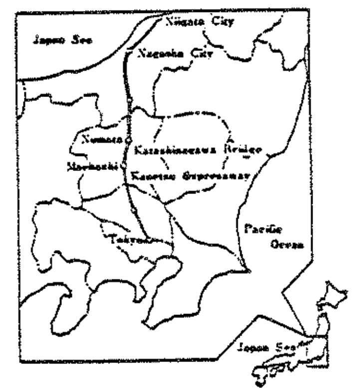

Fig.1 Location

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.

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.

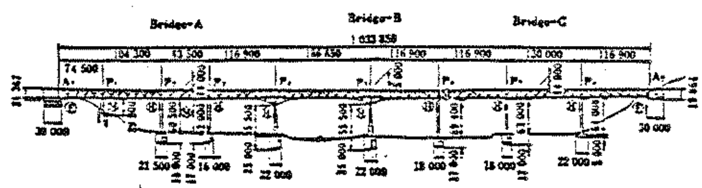

Fig.2 General View

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

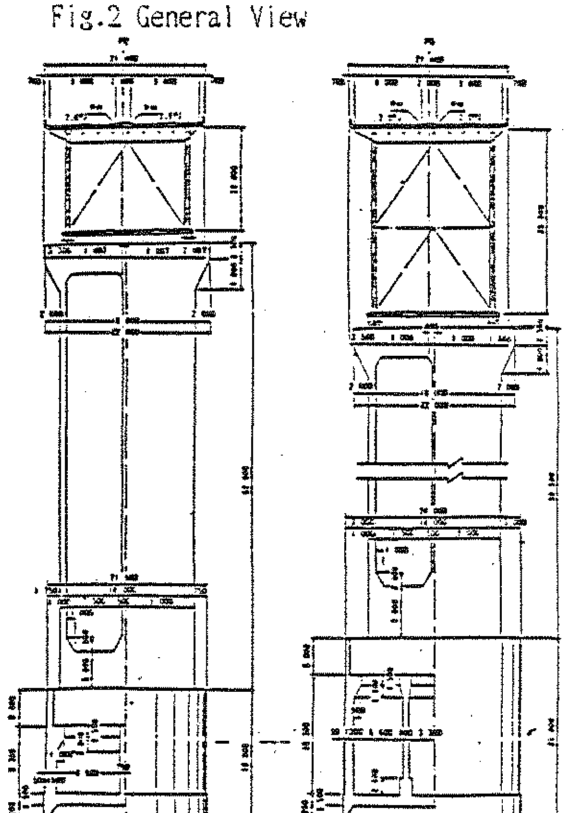

Fig.3 Pier P2 and P5

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.

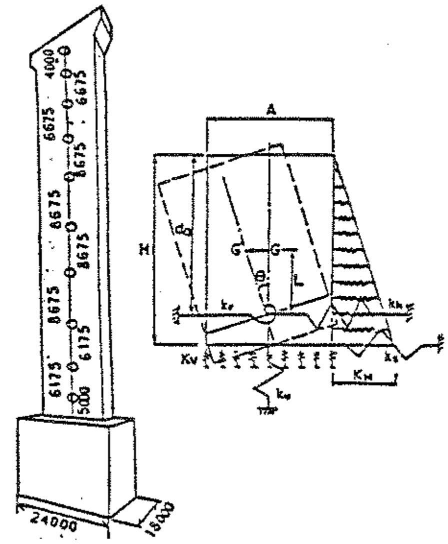

Fig.7 Spring Model

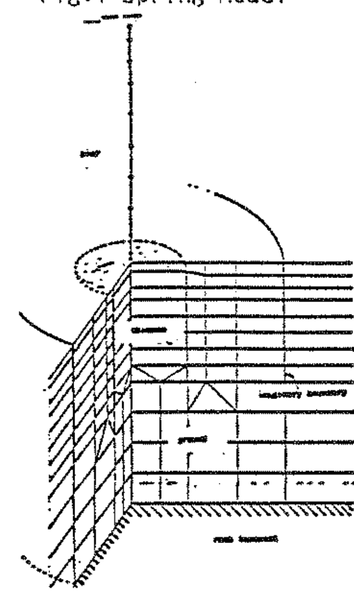

Fig.8 F.E.M. Model

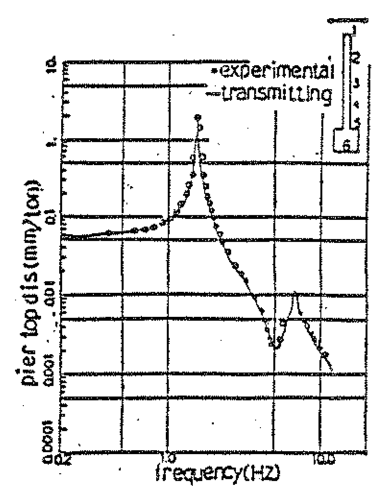

Fig.9 Resonance Curve

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 limited 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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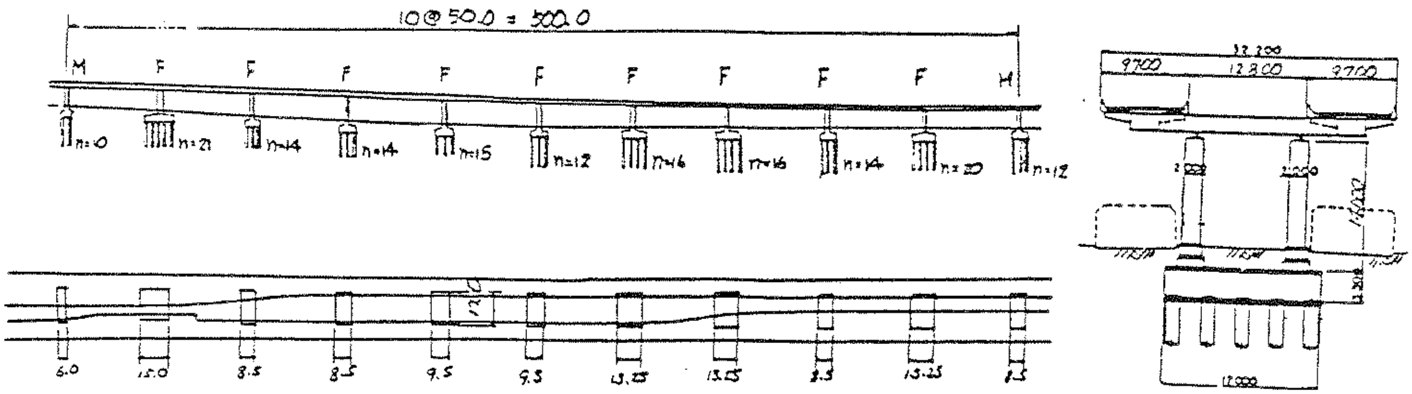
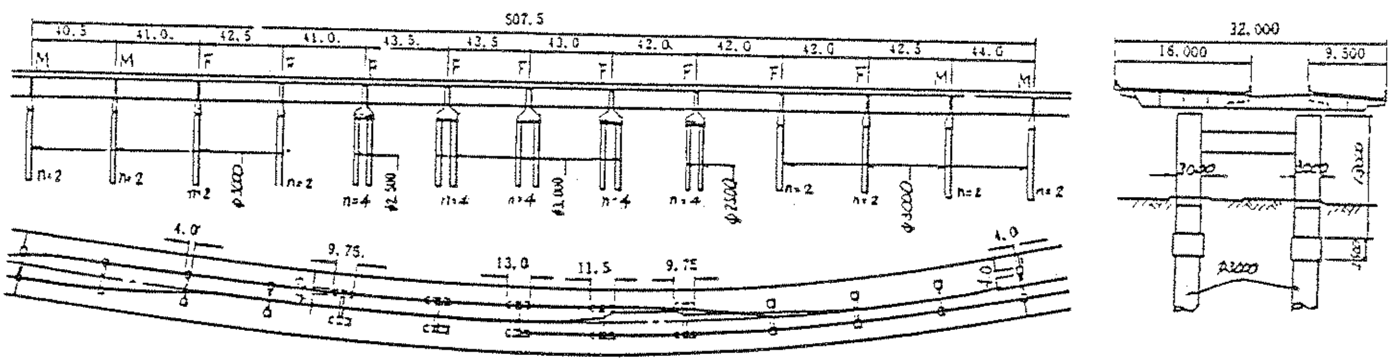
BRIDGE A



BRIDGE B



BRIDGE C

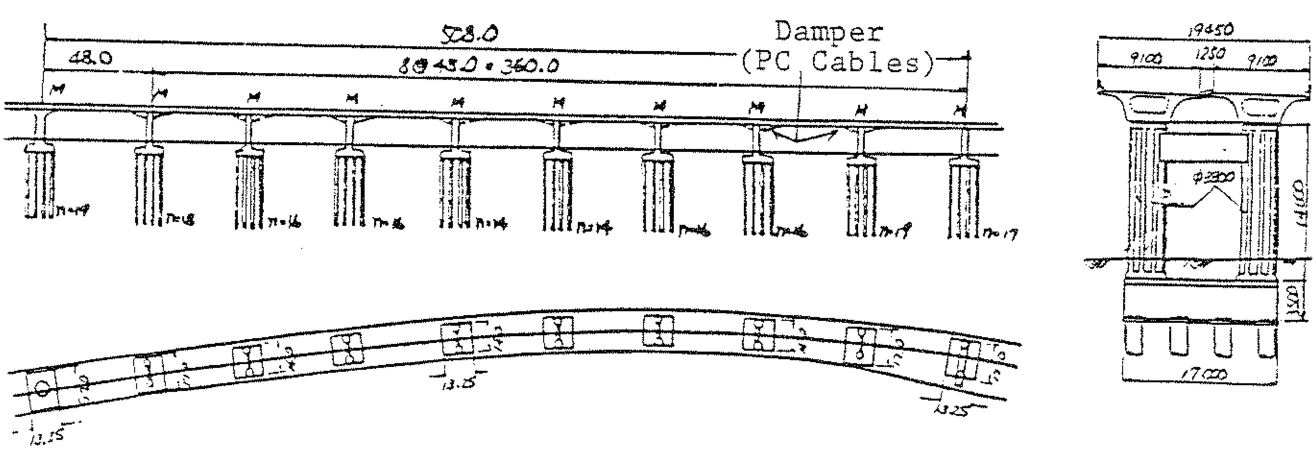


Figure 1: Multi-span Continuous Girder Bridges: General Layout

J-19

Future Programs of Hanshin Expressway Public Corp.

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

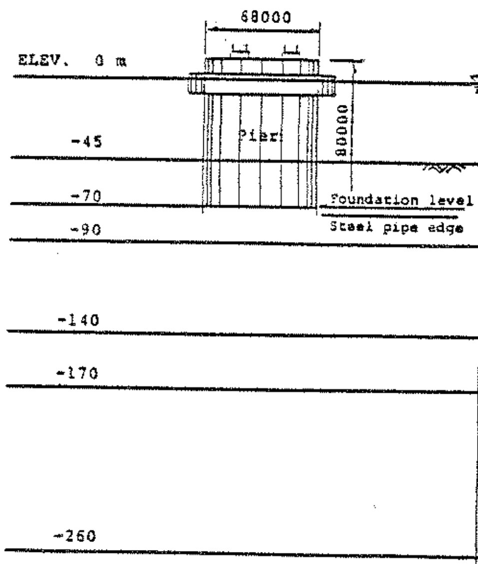
2) Ultimate strength and ductility of RC and steel bridge-structure columns

3) Earthquake resistant design of PC cable stayed bridge

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

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.



Water	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)
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

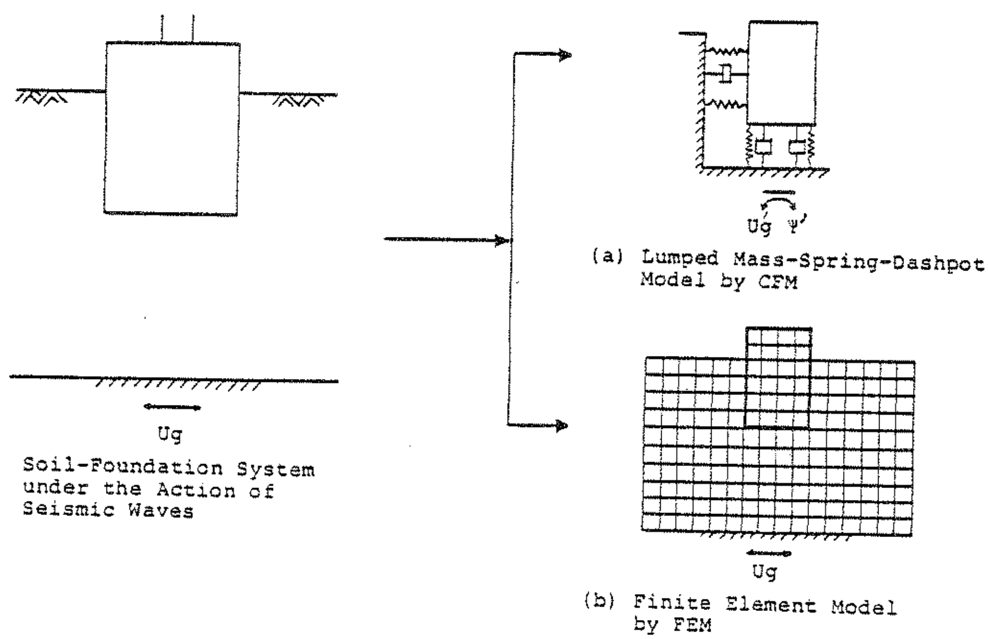


Fig.2 Soil-Foundation Interaction System and Its Models by CFM

USA PAPERS

U-1 TO U-19

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

U-2

January, 1984

SEISMIC RESISTANT BRIDGE DESIGN CRITERIA IN CALIFORNIA

by
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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.

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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:

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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.

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.

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.

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

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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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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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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surface 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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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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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

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."

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

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·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.

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.

CABLE STAYED BRIDGES - STATIC AND DYNAMIC RESPONSE

by

John F. Fleming, Professor of Civil Engineering

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.

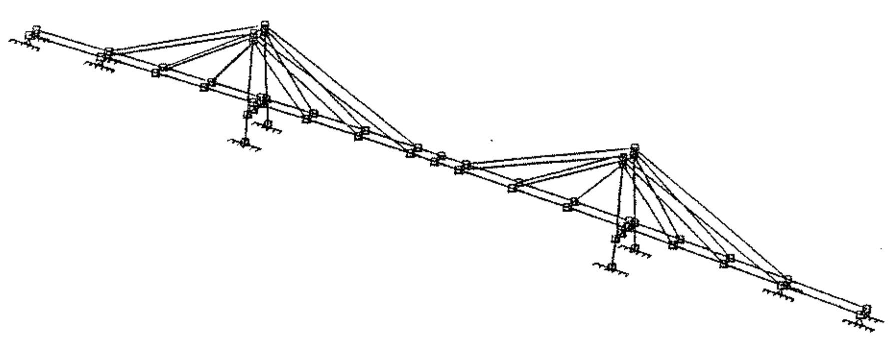


Figure 1 - Mathematical Model

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.

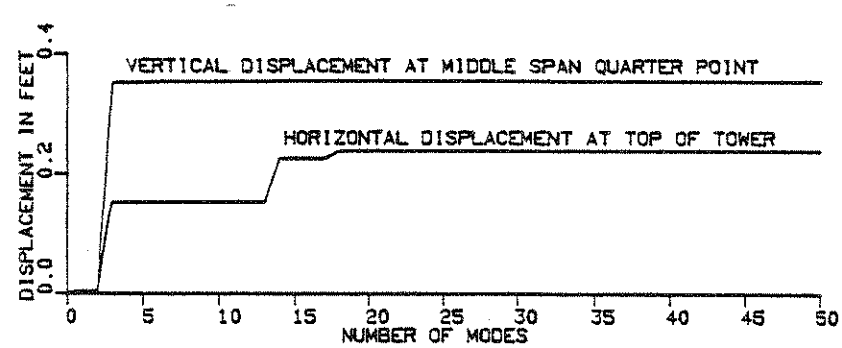


Figure 2 - Response Spectrum Analysis Displacements

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.

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

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.

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, earthquake, 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, natural hazards, 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

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.

RESOLUTIONS

IN ENGLISH

AND JAPANESE

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.

APPENDIX A

THE
MAIN BRIDGES
OF
KAN-ETSU
EXPRESSWAY

STRUCTURE SECTION
TOKYO SECOND CONSTRUCTION BUREAU
JAPAN HIGHWAY PUBLIC CORPORATION

FEBRUARY, 1984

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.

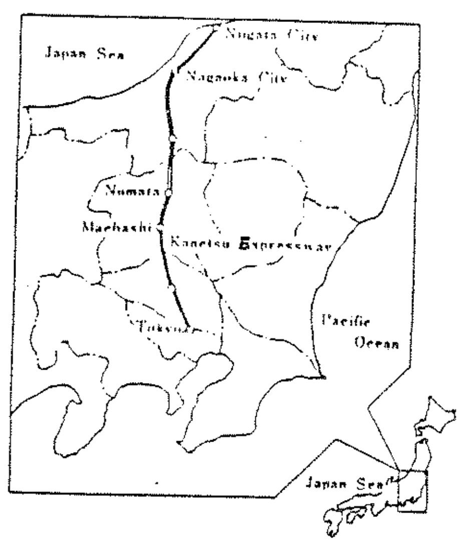


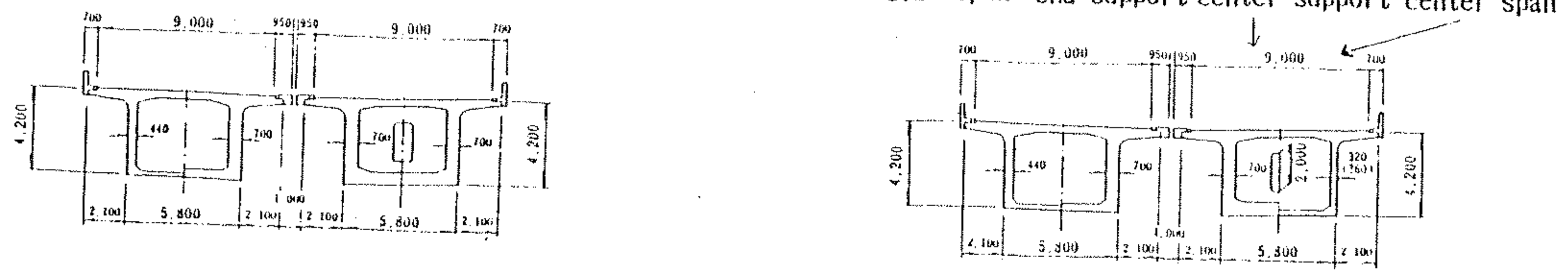
Fig.1 Kan-etsu Expressway

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

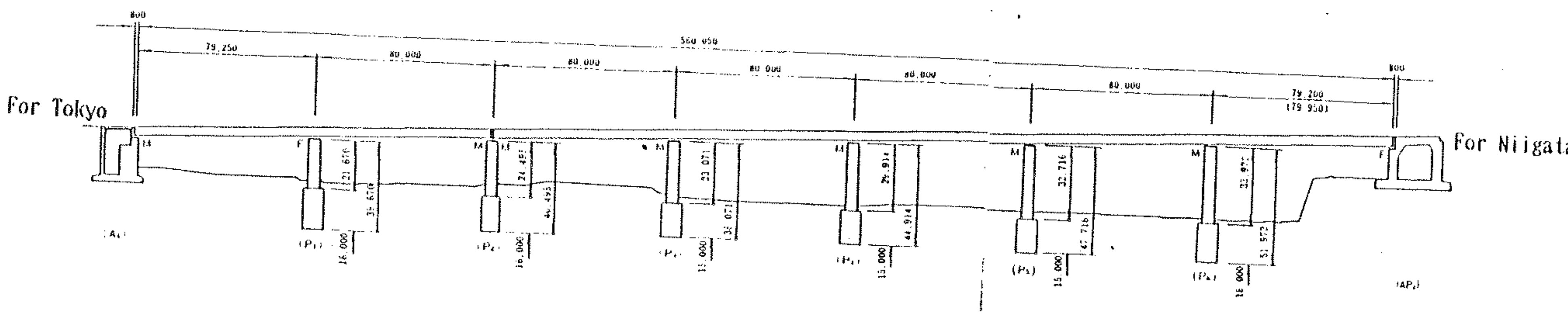


Fig.2 General view of the Tone-gawa Bridge

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

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.

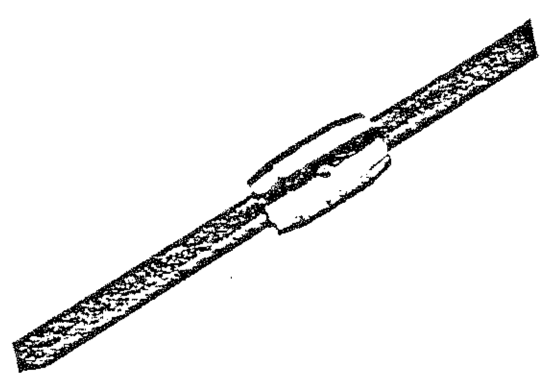
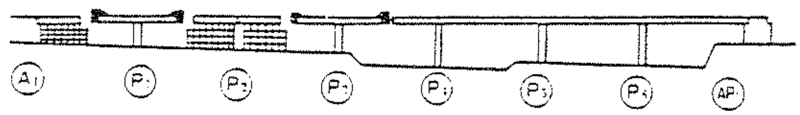
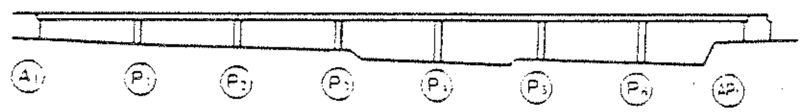


Fig.3 Coupler

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.

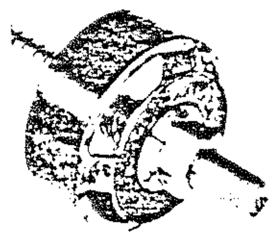
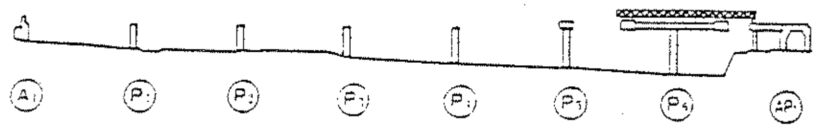
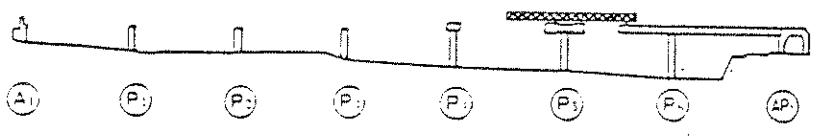


Fig.4 Gloke

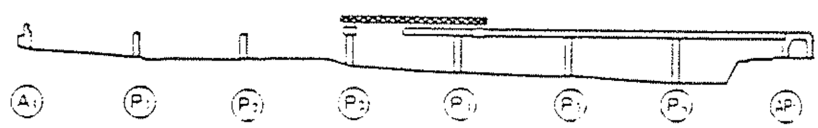
3rd:Stretching is carried out from the first segment to the fourth one in a balanced-cantilever pattern from atop P6 with overhead truss.



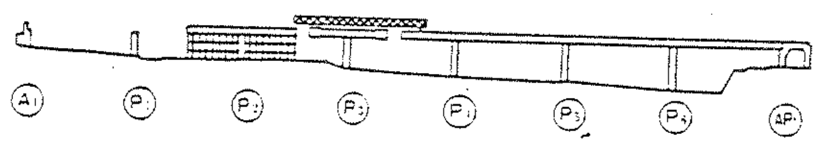
4th:The launching truss is then advanced after the concrete at the closure pour has attained sufficient strength and posttensioning of the continuity tendons.



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.



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.

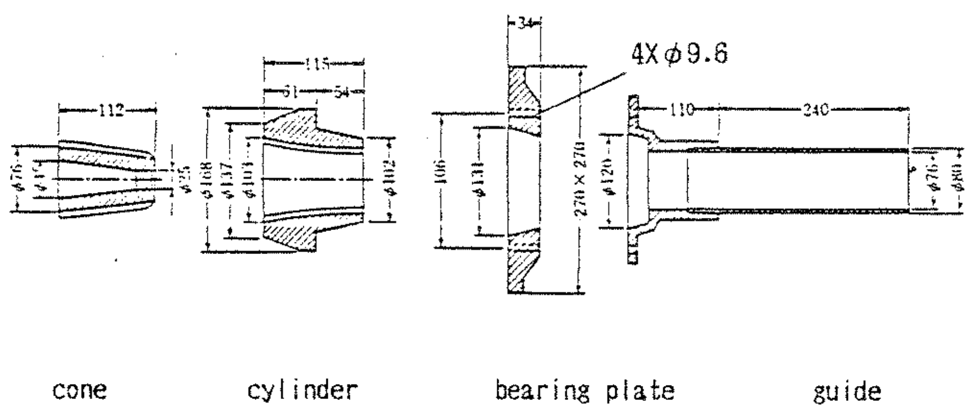


Fig.5 A set of Fressinet anchorage

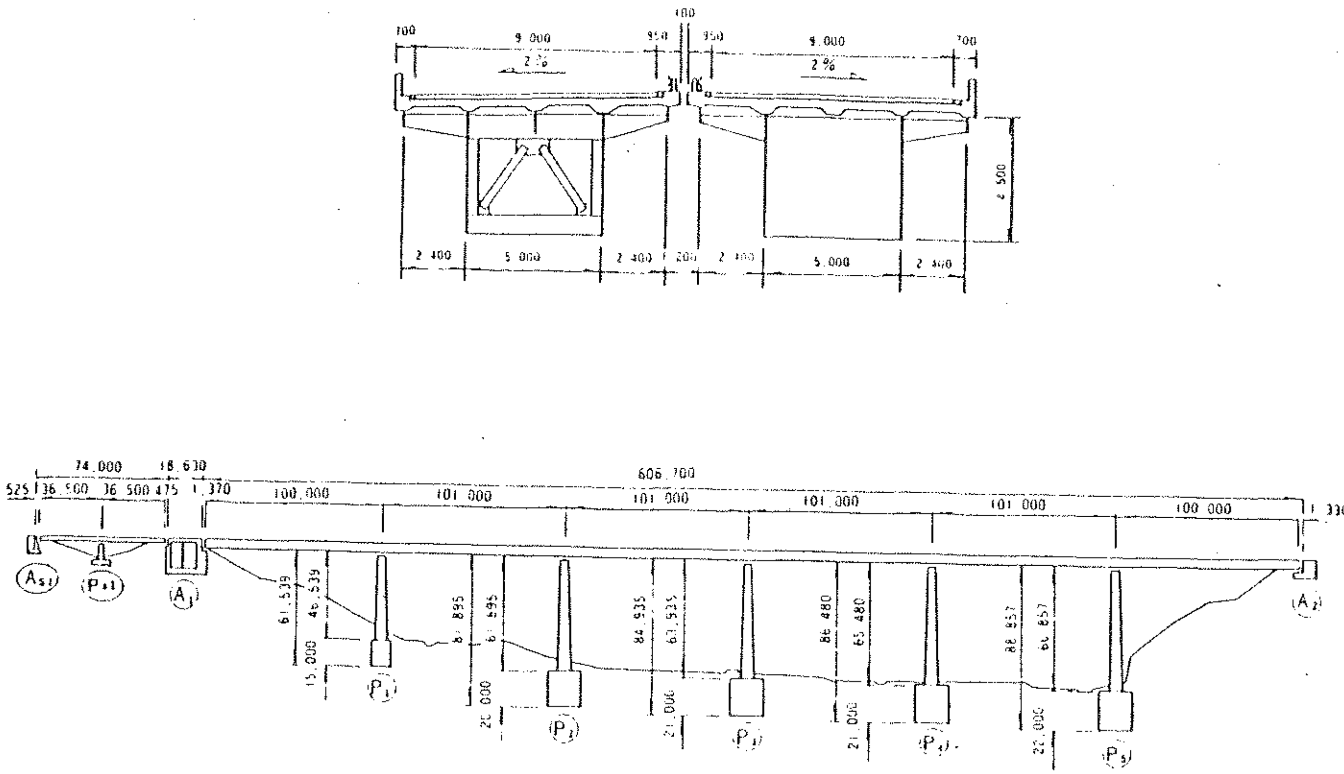


Fig.6 A general view of the Numao-gawa Bridge

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)

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

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.

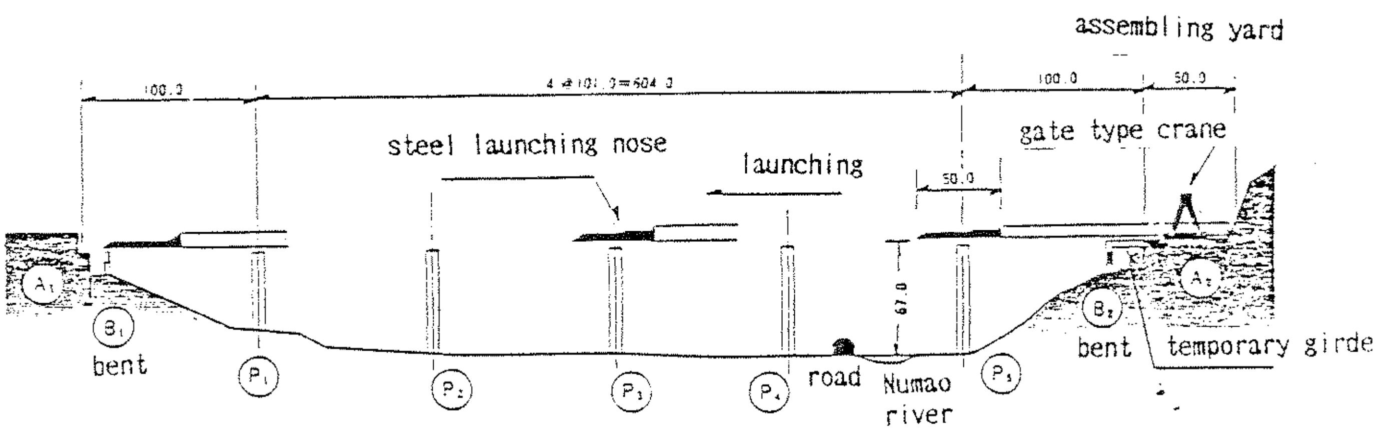


Fig.8 Schematic sequence of incremental launching of box girders

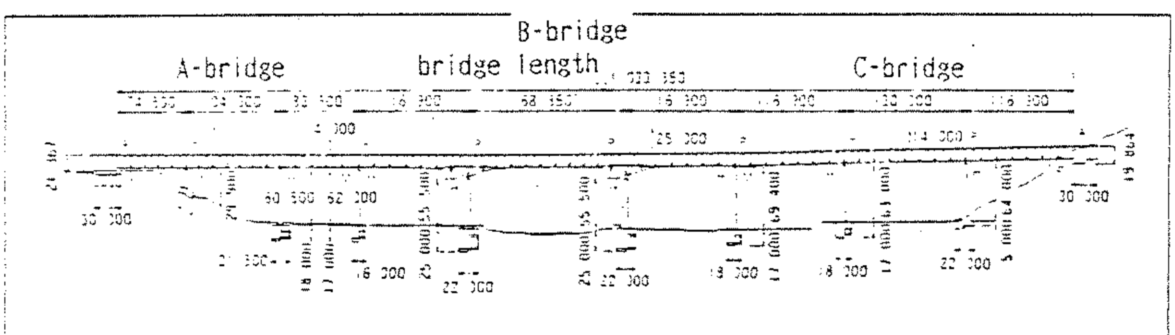
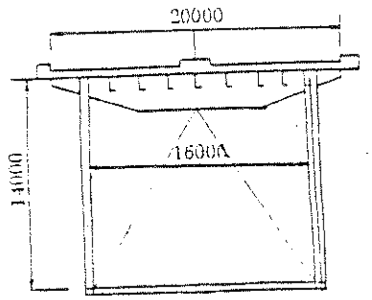
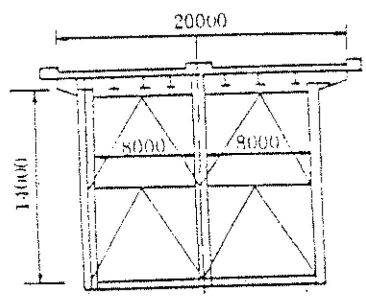


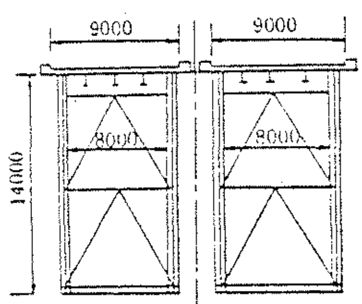
Fig.9 A general view of the Katashina-gawa Bridge



(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

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

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.

ERECTION OF SUPERSTRUCTURE

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.

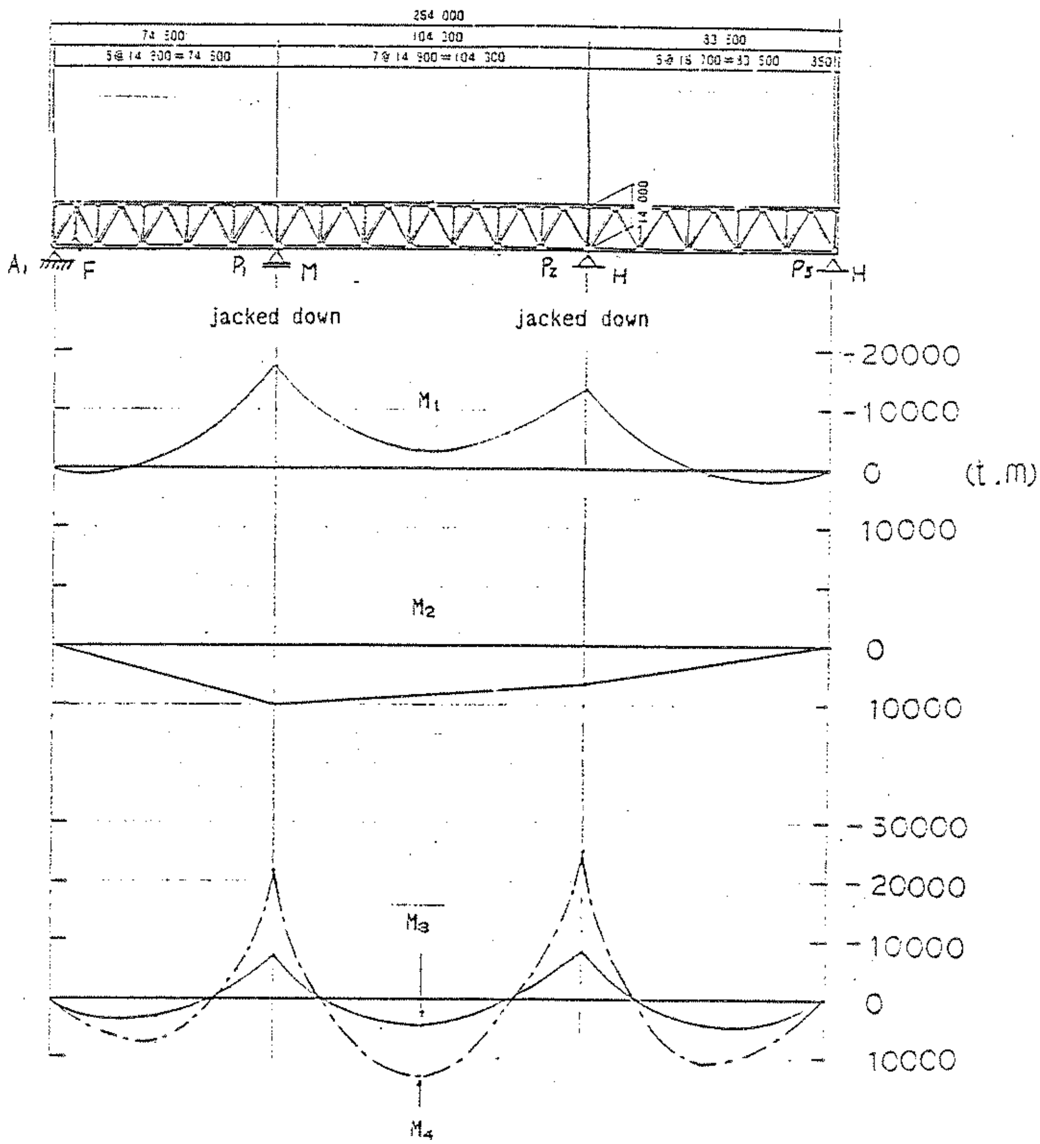
And then, from P_3 pier to P_4 pier (from P_6 pier to P_5 pier), the girder is erected by a large tower and traveller cranes as well as the erection of the girder from P_1 pier to P_3 pier. (Fig.E)

(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



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)

§4 THE NAGAI-GAWA BRIDGE

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)

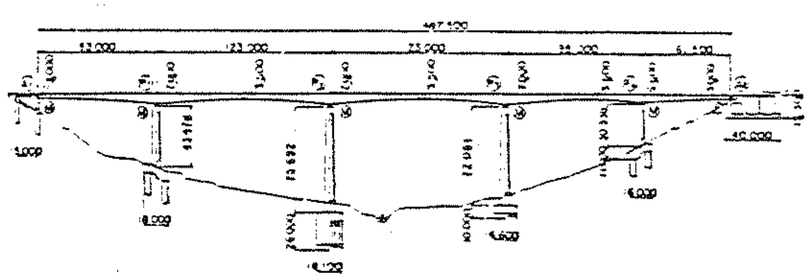


Fig.13 A general view of the Nagai-gawa Bridge

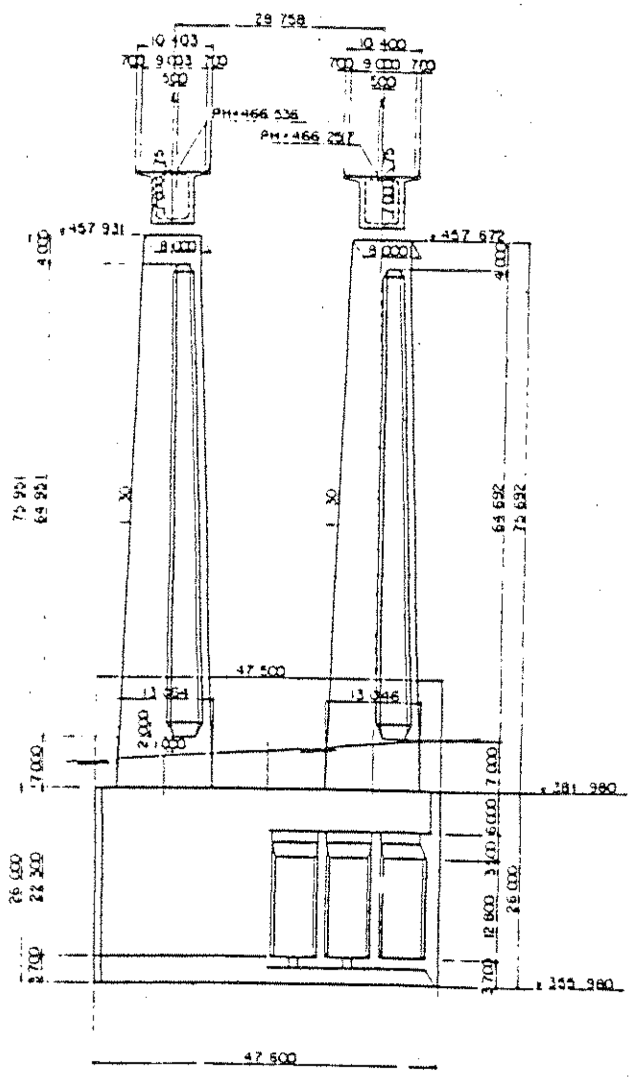


Fig.14 Pier P2 of the Nagai-gawa Bridge
which will be the highest in Japan

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

LIST OF CCEER PUBLICATIONS

Report No.	Publication
CCEER-84-1	Saiidi, M., and R. Lawver, "User's Manual for LZAK-C64, A Computer Program to Implement the Q-Model on Commodore 64," Civil Engineering Department, Report No. CCEER-84-1, University of Nevada, Reno, January 1984.
CCEER-84-1 Reprint	Douglas, B., Norris, G., Saiidi, M., Dodd, L., Richardson, J. and Reid, W., "Simple Bridge Models for Earthquakes and Test Data," Civil Engineering Department, Report No. CCEER-84-1 Reprint, University of Nevada, Reno, January 1984.
CCEER-84-2	Douglas, B. and T. Iwasaki, "Proceedings of the First USA-Japan Bridge Engineering Workshop," held at the Public Works Research Institute, Tsukuba, Japan, Civil Engineering Department, Report No. CCEER-84-2, University of Nevada, Reno, April 1984.