

**RECENT SELECTED PUBLICATIONS AT
EARTHQUAKE ENGINEERING
DIVISION, PUBLIC WORKS
RESEARCH INSTITUTE (No. 2)**

May, 1993

**Earthquake Engineering Division
Public Works Research Institute
Ministry of Construction
Tsukuba Science City, Ibaraki-ken, 305, Japan**

Copyright © (1993) by P. W. R. I.

All rights reserved. No part of this book may be reproduced by any means, nor transmitted, nor translated into a machine language without the written permission of the Director General of P. W. R. I.

この報告書は、土木研究所長の承認を得て刊行したものである。したがって、本報告書の全部又は一部の転載、複製は、土木研究所長の文書による承認を得ずしてこれを行ってはならない。

**RECENT SELECTED PUBLICATIONS AT EARTHQUAKE
ENGINEERING DIVISION, PUBLIC WORKS
RESEARCH INSTITUTE**

by

**Kazuhiko Kawashima,
Head Earthquake Engineering Division**

Synopsis:

At the Earthquake Engineering Division of the Public Works Research Institute, various researches for aiming to develop rational seismic design methods of bridges and underground structures, seismic evaluation and strengthening methods of existing structures, passive, active and hybrid control, loss estimation caused by earthquake are being made.

This reports presents some of the selected publications in the research on seismic design, seismic evaluation and strengthening of highway bridges, dynamic strength and ductility of structural components, structural response of menshin bridges (seismic isolated bridges), hybrid control, seismic response of embedded pipes, and damping characteristics of cable stayed bridges. They were presented at international conferences and workshops held in the last one years.

Key word: SEISMIC DESIGN, SEISMIC INSPECTION, SEISMIC STRENGTHENING, SEISMIC RETROFIT, BASE ISOLATION, MENSHIN DESIGN, DUCTILITY, DYNAMIC STRENGTHENING, SHIELD TUNNEL, EMBEDDED PIPE, SOIL AMPLIFICATION, BRIDGE, BRIDGE PIER, ECONOMICAL EFFECTS, INDIRECT EFFECTS.

TABLE OF CONTENTS

1. Analysis of Rocking Vibration of Rigid Foundation: Kazuhiko KAWASHIMA, Shigeki UNJOH and Hideyuki SHIMIZU (8th U.S.-Japan Bridges Workshop, Chicago, U.S.A., May, 1992)	1
2. Manual for Menshin Design of Highway Bridges: Kazuhiko KAWASHIMA, Kinji HASEGAWA and Hiroyuki NAGASHIMA (24th Joint Meeting of The U.S.-Japan Cooperative Program in National Resources, Panel on Wind and Seismic Effects, Gaithersburg, MD, U.S.A., May, 1992)	19
3. Guidelines for Seismic Design Methods of Large Underground Structures: Hajime ASAKURA, Kazuhiko KAWASHIMA and Hideki SUGITA (24th Joint Meeting of The U.S.-Japan Cooperative Program in National Resources, Panel on Wind and Seismic Effects, Gaithersburg, MD, U.S.A., May, 1992)	37
4. Seismic Response Control of Highway Bridges by Variable Damper: Kazuhiko KAWASHIMA, Shigeki UNJOH and Hideyuki SHIMIZU (24th Joint Meeting of The U.S.-Japan Cooperative Program in National Resources, Panel on Wind and Seismic Effects, Gaithersburg, MD, U.S.A., May, 1992)	55
5. Seismic Design Method of Underground Parking Garage Structures: Kazuhiko KAWASHIMA, Hideki SUGITA and Tomoru NAKAJIMA (5th U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Tsukuba, Japan, October, 1992)	71
6. Seismic Strengthening and Retrofitting of Highway Bridges in Japan: Kazuhiko KAWASHIMA and Shigeki UNJOH (5th U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Tsukuba, Japan, October, 1992)	91
7. Seismic Behavior of Beried Pipelines Through Field Observation: Kazuhiko KAWASHIMA, Hideki SUGITA and Tomoru NAKAJIMA (5th U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Tsukuba, Japan, October, 1992)	115
8. Effectiveness of the Variable Damper for Reducing Seismic Response of Highway Bridges: Kazuhiko KAWASHIMA, Shigeki UNJOH, Hiroyuki IIDA, and Hidetoshi MUKAI (2nd U.S.-Japan Workshop on Earthquake Protective Systems for Bridges, Tsukuba, Japan, December, 1992)	131

9. Manual for Menshin Design of Highway Bridges: Kazuhiko KAWASHIMA (2nd U.S.-Japan Workshop on Earthquake Protective Systems for Bridges, Tsukuba, Japan, December, 1992) 149
10. Response Analysis of Miyagawa Bridge Based on a Measured Acceleration Record: Kazuhiko KAWASHIMA, Hiroyuki NAGASHIMA, Shuji MASUMOTO and Kohji HARA (2nd U.S.-Japan Workshop on Earthquake Protective Systems for Bridges, Tsukuba, Japan, December, 1992) 177
11. Design Example of a Highway Bridge Based on The Manual for Menshin Design of Highway Bridges: Kazuhiko KAWASHIMA, Mitsuo OKADO and Michihiro HORIKAWA (2nd U.S.-Japan Workshop on Earthquake Protective Systems for Bridges, Tsukuba, Japan, December, 1992) 189
12. Evaluation of Shear Modulus of High Damping Rubber Bearings and Lead Rubber Bearing During Low-Rate Load Reversals: Kazuhiko KAWASHIMA, Masahiro KOSHITOGI, Chiaki SUDO, Yutaka MAKIGUCHI and Kazuo ENDO (2nd U.S.-Japan Workshop on Earthquake Protective Systems for Bridges, Tsukuba, Japan, December, 1992) 209
13. Inelastic Behavior and Seismic Design of a Highway Shield Tunnel: Kazuhiko KAWASHIMA and Yukitake SHIOI (ASCE Structural Congress XI, Irvine, CA, U.S.A., April, 1993) 235
14. Estimation of Damping Ratio of Cable-Stayed Bridges for Seismic Design, Kazuhiko KAWASHIMA, Shigeki UNJOH and Meguru TUNOMOTO (Journal of Structural Engineering, ASCE, Vol. 119, No. 4, April, 1993) 243

Analysis of Rocking Vibration of Rigid Foundation

Kazuhiko KAWASHIMA

Shigeki UNJOH

and

Hideyuki SHIMIZU

8th U.S. -Japan Bridges Workshop

Chicago, U. S. A.

May 1992

THE UNIVERSITY OF CHICAGO PRESS

AMERICAN UNIVERSITY

LIBRARY

1961

CHICAGO, ILL.

THE UNIVERSITY OF CHICAGO PRESS

LIBRARY

1961

ANALYSIS OF ROCKING VIBRATION OF RIGID FOUNDATIONS

by

Kazuhiko KAWASHIMA¹⁾ Shigeki UNJOH²⁾ and Hideyuki SHIMIZU³⁾

- 1) Head, Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan 305
- 2) Research Engineer, ditto
- 3) Assistant Research Engineer, ditto

ABSTRACT

In the current seismic design codes, the seismic stability for a rigid foundation against overturning is checked by means of a static seismic coefficient method. However, it is known that the dynamic characteristics on rocking vibration of a rigid foundation are significantly different from the static ones and the seismic stability against overturning by the static criteria seems to be overestimated in particular for a large-scale foundations. Therefore, it is necessary to develop a rational seismic design method for the check of seismic stability in consideration with the dynamic characteristics of the rigid foundation. This paper presents a dynamic analysis method for the rocking and slip motions of a rigid foundation subjected to an earthquake ground motion. The method was verified by comparing with the shaking table test results for rigid foundation models.

1. INTRODUCTION

The width of a base of a rigid foundation is currently designed so that the seismic stability against overturning, slip and the bearing capacity of subsoil be satisfied under the condition that a static seismic force is applied by means of a seismic coefficient method. As for the overturning, the foundation is checked so that the direction of a resultant force of the gravity load and the static seismic force be in the width of a foundation base or the region considering with a certain safety factor.

According to this method, the ratio between the width, B , and height, H , of a foundation is essential for a check of overturning of the foundation. Therefore, if the ratio, B/H , is the same, such as in a small block with the height of 10cm and a large-scale foundation with the height of 50m, both are judged to be overturned by the same certain seismic force. However, actual dynamic vibrational characteristics of the two foundations are completely different at a view point of required energy for the overturning of the foundations. A large-scale foundation can not easily be overturned even if the static condition for the overturning is not satisfied.

In order to develop a rational seismic design method for a rigid foundation, a dynamic analysis method for a rigid foundation on elastic and plasto-elastic

subsoil is proposed. This paper presents the analysis method proposed and verification of the method by comparing with the shaking table test results for rigid foundation models.

2. ANALYTICAL METHOD FOR ROCKING AND SLIP MOTIONS OF A RIGID FOUNDATION ON A ELASTIC BASE FLOOR

Various analytical methods for the dynamic motion of a rigid foundation on a rigid base floor has been proposed and studied. Mononobe¹⁾ showed that the static condition, that the direction of the resultant force of a seismic force and a gravity load is out of the base of the foundation, is not necessary condition for overturning, but the condition for developing rocking motion. He derived the equations of motion for the rocking motion of a rigid foundation on the rigid base floor and analysed the response subjected to a harmonic base motion. Ohtsuki²⁾ proposed an analytical method for rocking vibration of a rigid foundation considering the energy dissipation induced by the impact between a foundation and a rigid base. Mochizuki et. al.³⁾ proposed a general analytical method by improving the method proposed by Ohtsuki to estimate the earthquake ground motion based on the actual body movement after an earthquake. The analytical method includes rocking, slip and jump motions of the rigid body. Ishiyama⁴⁾ proposed more general analytical method for a rigid body subjected to an earthquake ground motion and the criteria against overturning by means of a limit velocity is proposed. According to these study, the rigid foundation on the rigid base floor makes various types of vibration such as slip, rotation, rotation with slip, jump with translation, and jump with translation and rotation as shown in Fig.1.

On the other hand, analysis methods with considering elastic deformation and/or plastic deformation of subsoil have been also proposed. Toki et. al.⁵⁾ proposed an analytical method by means of a finite element method for slip between a structure and subsoil under seismic load. A joint element was used for modeling separation between the structure and subsoil, and the stability against slip of the foundation was studied⁶⁾. Hara et. al.⁷⁾ also studied rocking, separation and slip between a building and subsoil by means of a finite element method using a joint element.

Although the analytical methods above mentioned considered elastic and plastic deformation of subsoil as well as slip and separation between a structure and subsoil, the following effects should be considered in order to apply for a rational seismic design of an actual rigid foundation.

1) It is necessary to consider the effect of settlement at the edge of the foundation by the rocking vibration on the seismic stability against overturning of the foundation. Since the yielding of a part of subsoil is estimated when a large-scale earthquake occurs, the effect of the bearing capacity of the subsoil on the stability of the foundation should also be considered.

2) If the foundation is buried in the subsoil, the resistance of the side soil should be considered. The side soil has two effects, i.e., resistance and

separation. Therefore, the effect of the side soil on the seismic stability of the foundation should be investigated.

In order to include these effects, a nonlinear dynamic analysis method for a rigid foundation in consideration with the effects of elastic deformation of subsoil and the yielding is proposed. In the proposed analysis method, the boundary conditions around the rigid foundation is assumed as:

1) The rigid foundation is supported at the base by two types of spring elements, i.e., a "base shear spring" and a "base vertical spring" as shown in Fig.3. The base shear spring resists against translational motion of the foundation. The base vertical spring resists against vertical motion as well as the vertical motion caused by rocking vibration. The rigid foundation is also supported at the side by a "side spring" which resists against translational motion of the foundation.

When the rigid foundation is supported by the subsoil modeled in the above and is subjected to an earthquake ground motion as well as considering the effect of static force by the reaction force from the superstructure and cable tension force in case of the anchor foundation of a suspension bridge, the equations of motion can be written as

$$\begin{aligned} M \ddot{X} &= F_X^K + F_X^F + F_X^G \\ M \ddot{Y} &= F_Y^K + F_Y^F + F_Y^G \\ J \ddot{\theta} &= M_Y^K + M_Y^F + M_Y^G - M_X^K - M_X^F - M_X^G \end{aligned} \quad (1)$$

where

M, J : mass and inertia moment of a rigid foundation

X, Y, θ : absolute displacements of a center of gravity of a rigid foundation (translation in the horizontal and the vertical and rotation)

F_X^G, F_Y^G : static forces by cable tension, reaction force from superstructure, etc

F_Y^G : the gravity force of a rigid foundation

F_X^K, F_Y^K : reaction forces of subsoil at base and side in the tangential direction

F_X^F, F_Y^F : reaction forces of subsoil at base and side in the normal direction

M_X^G, M_Y^G : moment by cable tension, reaction force from a superstructure, etc

M_X^K, M_Y^K : moments by reaction forces of subsoil at base and side in the tangential direction

M_X^F, M_Y^F : moments by reaction forces of subsoil at base and side in the normal direction

The spring elements which support the foundation are discretized and modeled as shown in Fig.2.

2) Energy dissipation as well as restoring force is considered in the base vertical spring. The energy dissipation is due to the effect of impact between

the foundation and the subsoil, and it is idealized as a viscous damping. Viscous damping force is assumed as proportional to relative velocity between the foundation and the subsoil. The restoring characteristics of the spring is shown in Fig.2, i.e., the spring resists against compressive force and separates to tension force. In the compression side, the spring is assumed to yield at a certain force level and the stiffness is to be decreased. Therefore, the discretized spring at the base, f_{yi}^K , can be written as

$$f_{yi}^K = \begin{cases} 0 & (y_i > 0) \\ K_{yi}^{B1} \cdot y_i + C_i^B \cdot \dot{y}_i & (y_i^Y \leq y_i \leq 0) \\ K_{yi}^{B1} \cdot y_i^Y + K_{yi}^{B2} \cdot (y_i - y_i^Y) + C_i^B \cdot \dot{y}_i & (y_i^Y < y_i) \end{cases} \quad (2)$$

where

K_{yi}^{B1} : initial stiffness of i-th discretized spring in the compression side

K_{yi}^{B2} : second stiffness of i-th discretized spring in the compression side

y_i^Y : yield displacement of i-th discretized spring

y_i : displacement of i-th discretized spring

C_i^B : damping coefficient which represents energy dissipation due to the impact between a foundation and subsoil

In Eq.(2), the damping coefficient is determined from the relation between the damping coefficient and the impact coefficient of single-degree-of-freedom system. If the velocities of the foundation before and after the impact with the subsoils are assumed as v and v' , the impact coefficient, e , is defined as

$$e = -v'/v \quad (3)$$

In the case of a single-degree-of-freedom system, the relation between damping ratio and velocity after and before the impact can be written as

$$2\pi h / \sqrt{(1-h^2)} = \ln(v'/v) \quad (4)$$

Substituting Eq.(3) into Eq.(4), one can have

$$h = \sqrt{\{(\ln e) / ((\ln e)^2 + (\ln e)^2)\}} \quad (5)$$

The damping coefficient, C_i^B , is assumed as

$$C_i^B = 2h\sqrt{(M \cdot K_{yi}^{B1})} \quad (6)$$

3) The base shear spring is assumed to be a bi-linear type spring as shown in Fig.2. Therefore, the base shear spring, f_{xi}^K , can be written as

$$f_{xi}^K = \begin{cases} -|\mu f_{yi}^K| & (-|\mu f_{yi}^K| > K_{xi}^B \cdot x_i) \\ K_{xi}^B \cdot x_i & (-|\mu f_{yi}^K| > K_{xi}^B \cdot x_i > |\mu f_{yi}^K|) \\ |\mu f_{yi}^K| & (K_{xi}^B \cdot x_i < |\mu f_{yi}^K|) \end{cases} \quad (7)$$

where

K_{xi}^B : stiffness of i-th discretized spring

x_i : displacement of i-th discretized spring

μ : friction coefficient between a foundation and soil

4) The side spring is assumed to be a bi-linear type spring in the compression side and the spring yields at the certain force level. However, the damping force caused by the separation and the impact between the foundation and the side soil is disregarded. Therefore, the discretized side spring, f_{xi}^K , can be written as

$$f_{xi}^K = \begin{cases} 0 & (x_i > 0) \\ K_{xi}^K \cdot x_i & (-x_i^Y \leq x_i \leq 0) \\ K_{xi}^K \cdot x_i^Y & (x_i < -x_i^Y) \end{cases} \quad (8)$$

where

K_{xi}^K : stiffness of i-th discretized spring

x_i^Y : yielding displacement of i-th discretized spring

x_i : displacement of i-th discretized spring

On the other hand, similarly to the base shear spring, the friction force of the side soil, f_{yi}^K , can be written as

$$f_{yi}^K = \begin{cases} -|\mu f_{xi}^K| & (-|\mu f_{xi}^K| > K_{yi}^B \cdot y_i) \\ K_{yi}^B \cdot x_i & (-|\mu f_{xi}^K| > K_{yi}^B \cdot y_i > |\mu f_{xi}^K|) \\ |\mu f_{xi}^K| & (K_{yi}^B \cdot y_i < |\mu f_{xi}^K|) \end{cases} \quad (9)$$

where

K_{yi}^B : stiffness of i-th discretized spring

y_i : displacement of i-th discretized spring

3. SHAKING TABLE TESTS OF RIGID FOUNDATION MODELS

In order to investigate the vibrational characteristics of rigid foundations and to verify the proposed analysis method, the shaking table tests for rigid foundation models were made³⁹. The model used are the concrete blocks which have width-height ratio of 1/3 as shown in Fig.3. The length, volume and weight ratios for the model A, B and C are 1 : 1.65 : 2.5, 1 : 4.5 : 15.6, and 1 : 4.2 : 10.4, respectively. The scale of the models were determined so that the size of the largest model C be able to be shaken within the capacity of the shaking table. The size of model A and B were determined so that the difference from the largest model C be found. In the shaking table tests, concrete base floor was prepared on the shaking table, and the model block was set on it and was subjected to harmonic and earthquake motions as shown in Fig.4. In order to model the effect of deformation of subsoil, a rubber sheet was placed on the concrete base floor.

Fig.5 shows how the rotation angle of the rigid foundation models increases as the peak acceleration of the shaking table increases when the models on the rubber sheet was subjected to harmonic excitation with frequency of 3Hz. According to the tests, the following points were deduced:

1) When the acceleration amplitude of the shaking table exceeds certain level the rigid foundation model starts to rock. The larger foundation model is the smaller the rotation amplitude is. Therefore, if the shape is the same the smaller foundation is more likely to be rotated.

2) Since the width-height ratio of the 3 models is assumed to be 1/3, according to the classical rocking theory the critical acceleration which initiates the foundation to rock is $980/3 = 327 \text{ cm/s}^2$. However, the test results showed that the rigid foundation models initiate to rock from more smaller excitation. This is associated with existence of the natural mode of the rigid foundation which is supported by the rubber sheet.

4. VERIFICATION OF THE ANALYTICAL METHOD PROPOSED IN COMPARING WITH THE SHAKING TABLE TESTS

The shaking tests results were simulated by the analytical method proposed in Section 2. Fig.6 shows the analytical results for the relation between the table acceleration and rotation angle of the rigid foundation models. The rigid models were assumed to be on the rigid base floor in Fig.6. It is interesting to note that the experimental results, that the larger foundation is likely to show smaller rocking oscillation, is well simulated in the analysis. The critical table acceleration for initiating the foundation to rock is 327 cm/s^2 , which is quite accurately coincident with the classical theory.

Fig.7 shows the rocking amplitude vs. peak table acceleration for the foundation resting on the rubber sheet. Stiffness and impact coefficient of the rubber was assumed as shown in Table 1. Because the stiffness and the impact coefficient of the rubber sheet are dependent on the model and the test case, the average values obtained through free oscillation tests of the foundation are used in the analysis. It is clearly shown in Fig.7 that the critical acceleration for initiating the rocking is sensitively affected by the rubber sheet. The critical acceleration is much smaller than that of the foundation resting on rigid base floor.

In order to improve the agreement between the simulation and the test results the stiffness of the rubber sheet was reestimated. The elastic modulus obtained by shear tests of the rubber material was used in the analysis of Fig.7. This value was obtained under the condition that the confined effect at the upper and lower surfaces of the rubber sheet was not considered. In the shaking table tests, the rubber sheet was caught between the rigid foundation and rigid base floor at the upper and lower surfaces. If the upper and lower surfaces of the rubber was confined the resulting elastic modulus becomes 20 times larger than that obtained in the shear tests. Fig.8 shows analytical results using this parameter. The relation between the table acceleration and the rotation angle was obtained with better agreement with the test results as well as the critical acceleration for initiating the rocking vibration.

Fig.9 shows the time history of rotation angle of the model subjected to a table motion. The acceleration obtained on the ground around the Kaihoku Bridge during the Miyagi-ken-oki Earthquake of 1978 was used as the table motion. The acceleration was scaled by 0.4 times (peak acceleration measured on the shaking table was 196 cm/s^2) and 0.5 times (that was 243 cm/s^2). It should be reminded here that the critical acceleration predicted by the classical rocking theory is $980/3 = 327 \text{ cm/s}^2$.

When the foundation was excited with the intensity of 0.4 times the original record, the rocking was found during main excitation. Peak rotation angle and peak translational displacement of the center of gravity of the model was 0.003rad and 2.2mm, respectively. The peak separation of the edge of the model base was 1.5mm. When the table motion was increased to 0.5 times the original record, those are 0.0056rad, 4.2mm and 2.8mm, respectively.

Fig.10 shows the comparison of the time history of rotation angle between the experiments and the analysis. The analytical results agree quite well with the experimental results. It is seen that the critical table acceleration for initiating the rocking vibration becomes smaller when deformation of the subsoil is considered. However, the rocking amplitude of the foundation resting on the rubber sheet is smaller than that of the foundation resting on rigid base floor.

5. CONCLUDING REMARKS

In order to develop a rational seismic design method for a rigid foundation, an analytical method for rocking and slip motions of the rigid foundation was proposed. The method was verified in comparing with shaking table test results of the rigid foundation models. According to the above results the following conclusions may be deduced:

1) By taking the effect of deformation of the subsoil into consideration in the analysis, the foundation is likely to initiate rocking at smaller table acceleration than the acceleration predicted by the classical rocking theory. This is due to the natural mode of the foundation. Therefore, the critical table acceleration for causing the rocking varies with the natural frequency of the rigid foundation and predominant frequency of the table acceleration. Therefore, to correctly estimate the seismic stability against overturning of the rigid foundation it is important to include the effect of subsoil.

2) The shaking table test results can be simulated by the analysis with better agreement by considering the deformation of the subsoil.

3) Although the critical table acceleration for initiating the rocking becomes smaller when the effect of deformation of the subsoil is taken into account, the subsoils do not increase the rocking amplitude of the foundation. This is because that the natural frequency of the foundation varies with the amplitude of rocking.

REFERENCES

1. Mononobe, N. : Vibration Theory on the Up-Down Motion of Earthquakes, Journal of Civil Engineering, Japan Society of Civil Engineers, Vol.10, No.5, October 1921
2. Mochizuki, T. and Kobayashi, K. : Study to estimate the earthquake acceleration based on the body movement (Analysis of Dynamic Behavior of the Body), Proceedings of Architectural Institute of Japan, Vol.248, October 1976
3. Ohtsuki, S. : Seismic Design of Building Structures, Corona Publishing Company, March 1961

4. Ishiyama, Y. : Motion of Rigid Bodies Criteria for Overturning by Earthquake Excitations, Earthquake Engineering and Structural Dynamics, Vol.10, 1982
5. Toki, K., Satoh, T. and Miura, H. : Separation and Slip between the Structures and the Subsoils during Strong Earthquake Motion, Proceedings of Japan Society of Civil Engineers, No.302, October 1980
6. Toki, K. and Miura, H. : Nonlinear Earthquake Response Analysis of Soil-Structure Systems, Proceedings of Japan Society of Civil Engineers, No.317, Jan. 1982
7. Kawashima, K. : A Study on Modeling of the Friction Force for Dynamic Analysis, Proceedings of Japan Society of Civil Engineers, Vol.209, May 1981
8. Kawashima, K. and Unjoh, S. : Analysis of Rocking Vibration of Rigid Foundations subjected to Earthquake Ground Motion, Civil Engineering Journal, Vol.32, No.10, October 1989
9. Kawashima, K. and Unjoh, S. : Analytical Method for the Rocking Vibration of Rigid Foundations considering with the Deformation and the Yielding of the Subsoil, Civil Engineering Journal, Vol.33, No.3, March 1990

**Table 1 Parameters of Elastic Rubber Sheet
Assumed in the Analysis**

Standard Values	Hardness Hs	65
	Elastic Modulus E_0	40 kgf/cm ²
	Poisson's Ratio ν	0.5
	Shear Modulus G	13.3 kgf/cm ²
Elastic Modulus in consideration with Confined Effect		800 kgf/cm ²
Bottom of Models	Friction Coefficient	0.6
~Rubber Sheet	Impact Coefficient	0.7

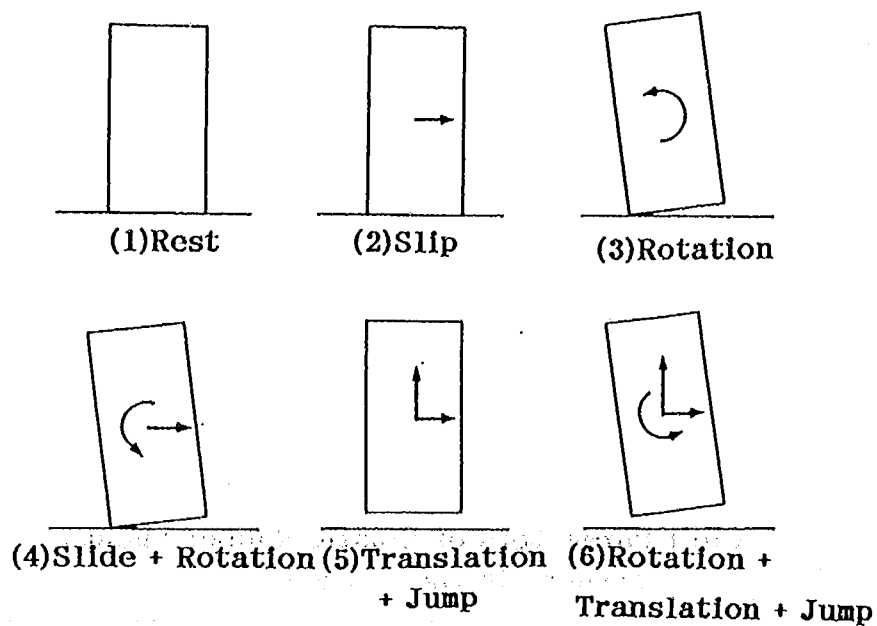


Fig.1 Classification of Motions of a Rigid Body on a Rigid Floor

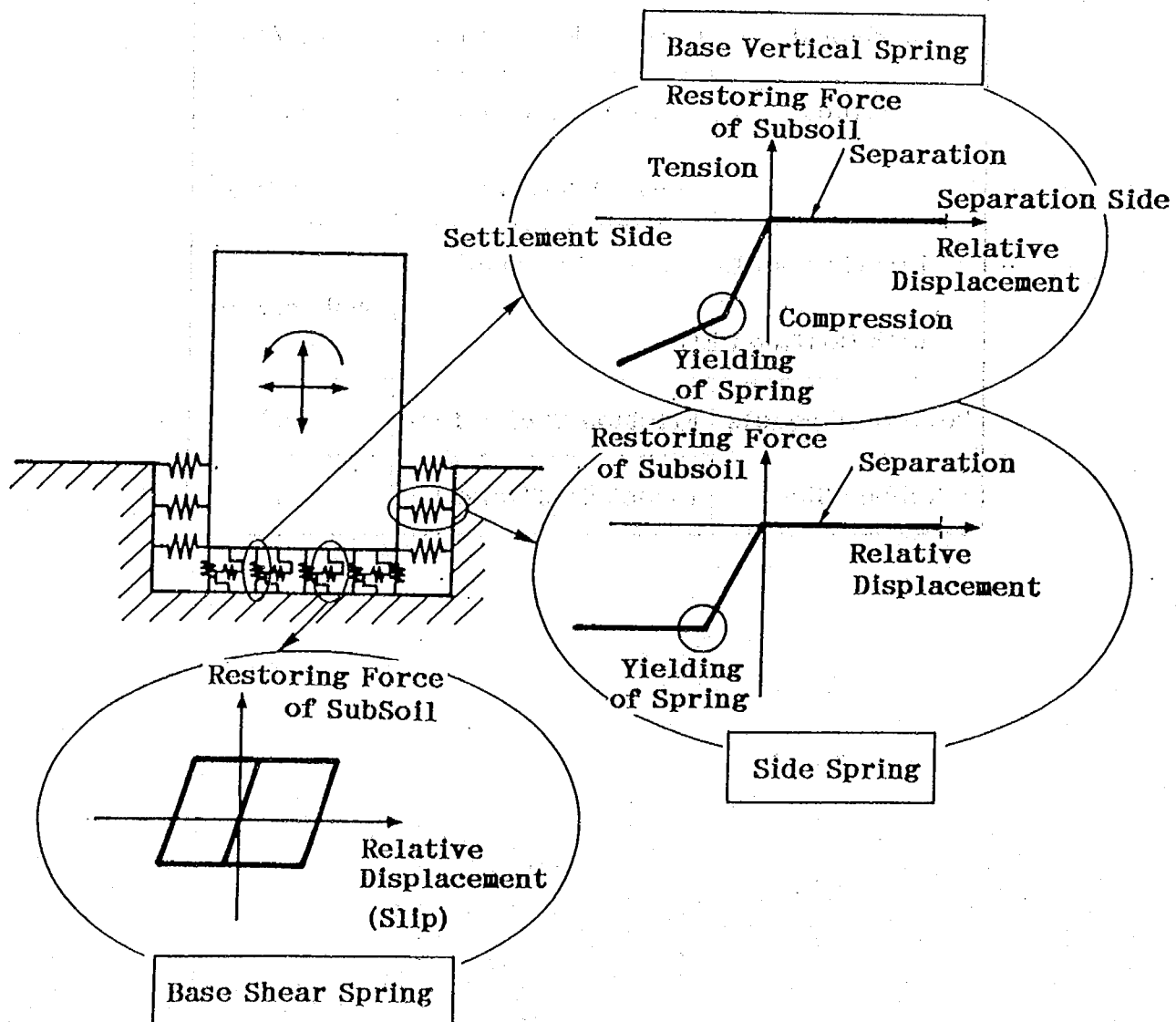


Fig.2 Analytical Idealization for Rocking and Slip of A Rigid Foundation on An Elastic Floor in Consideration with Yielding of Stiffness

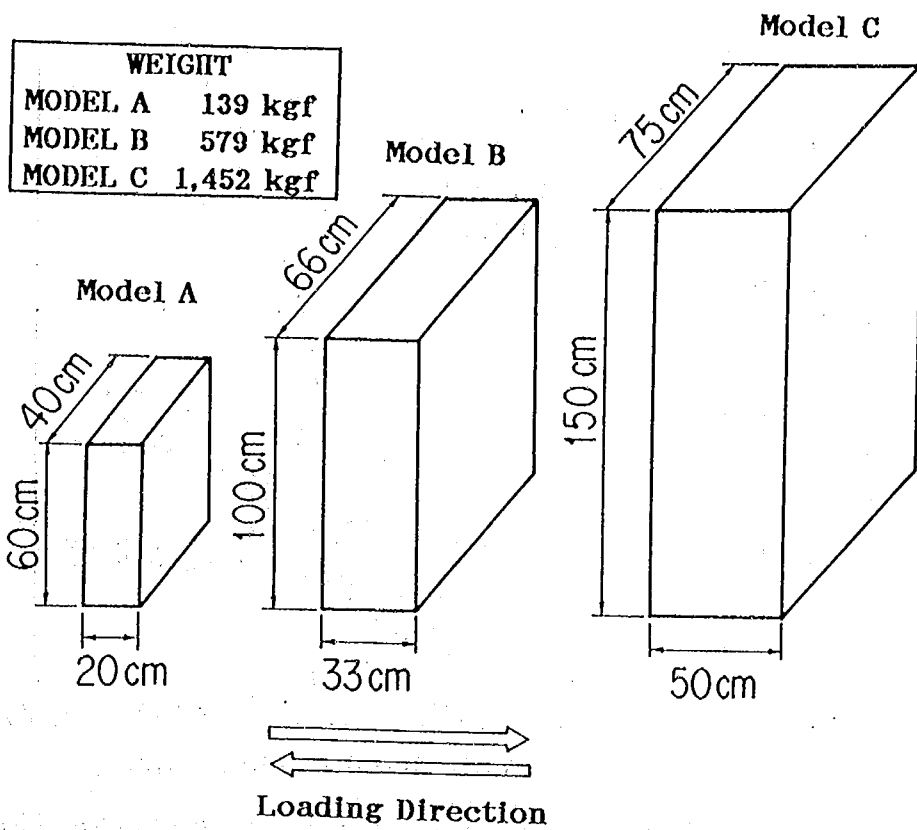


Fig.3 Rigid Foundation Models used for Shaking Table Tests

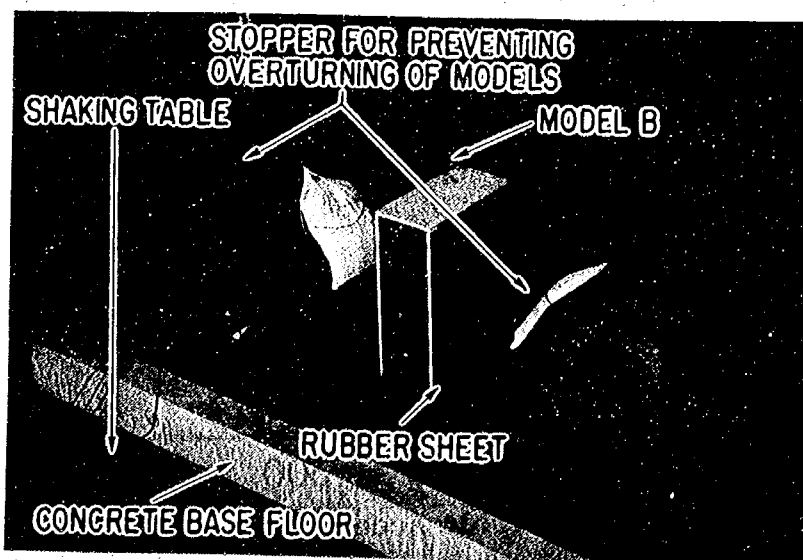


Fig.4 Set up of Shaking Table Tests

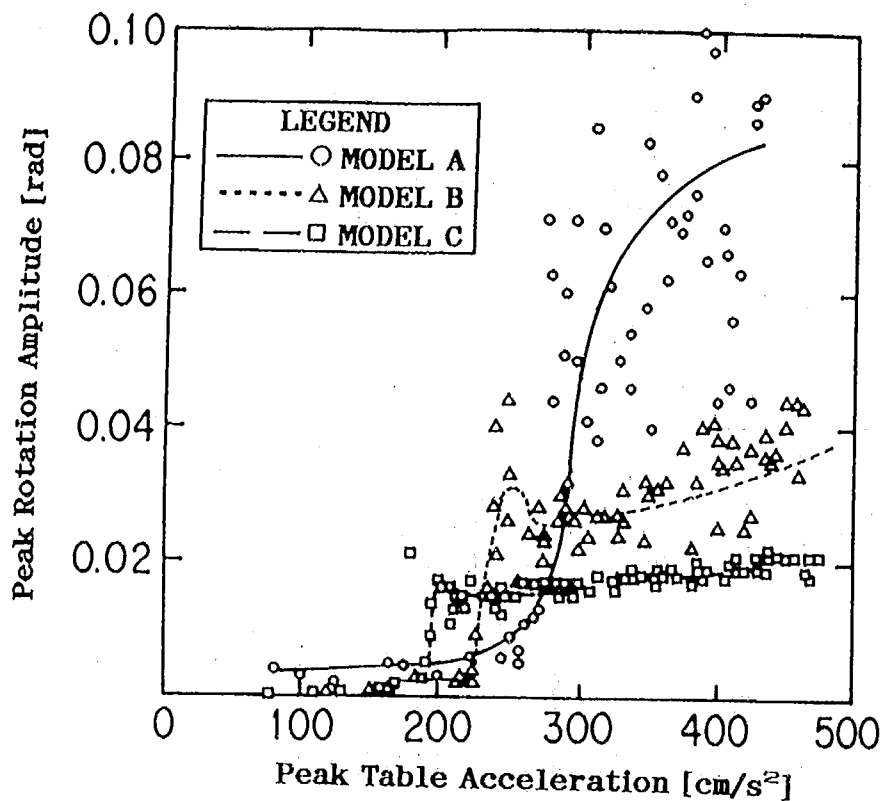


Fig.5 Effect of Model Size on Peak Table Acceleration vs. Rotation Angle of the Rigid Foundation (Harmonic Excitation with Frequency of 3Hz)

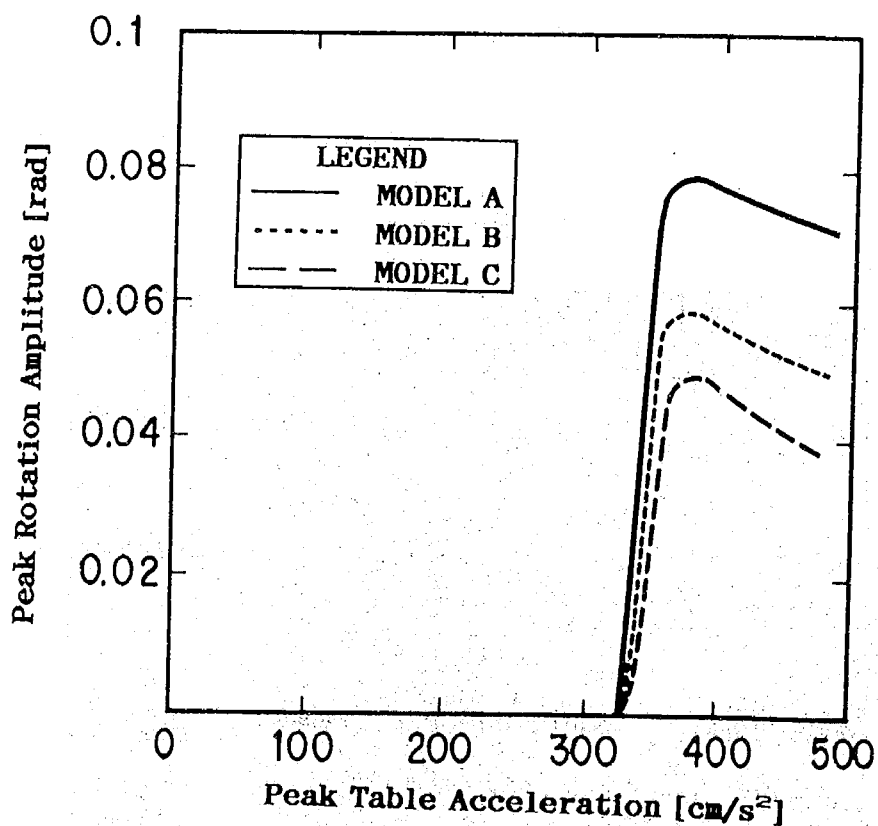


Fig.6 Effect of Model Size on Peak Table Acceleration vs. Rotation Angle of the Rigid Foundation on the Rigid Base Floor (Harmonic Excitation with Frequency of 3Hz)

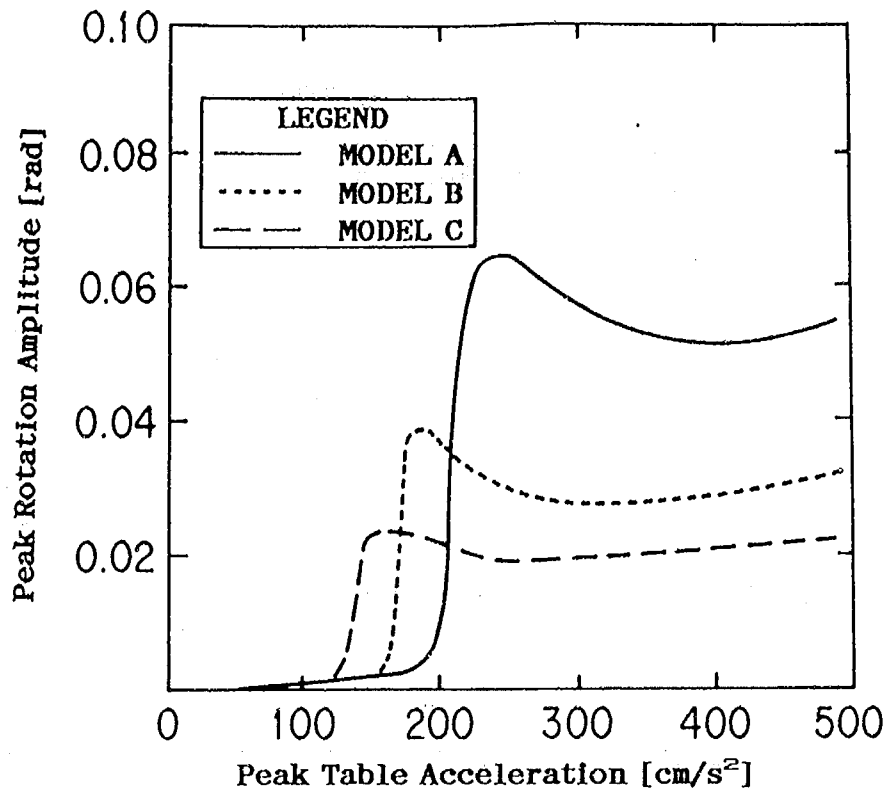


Fig.7 Effect of Model Size on Peak Table Acceleration vs. Rotation Angle of the Rigid Foundation on the rubber Sheet (Harmonic Excitation with Frequency of 3Hz)

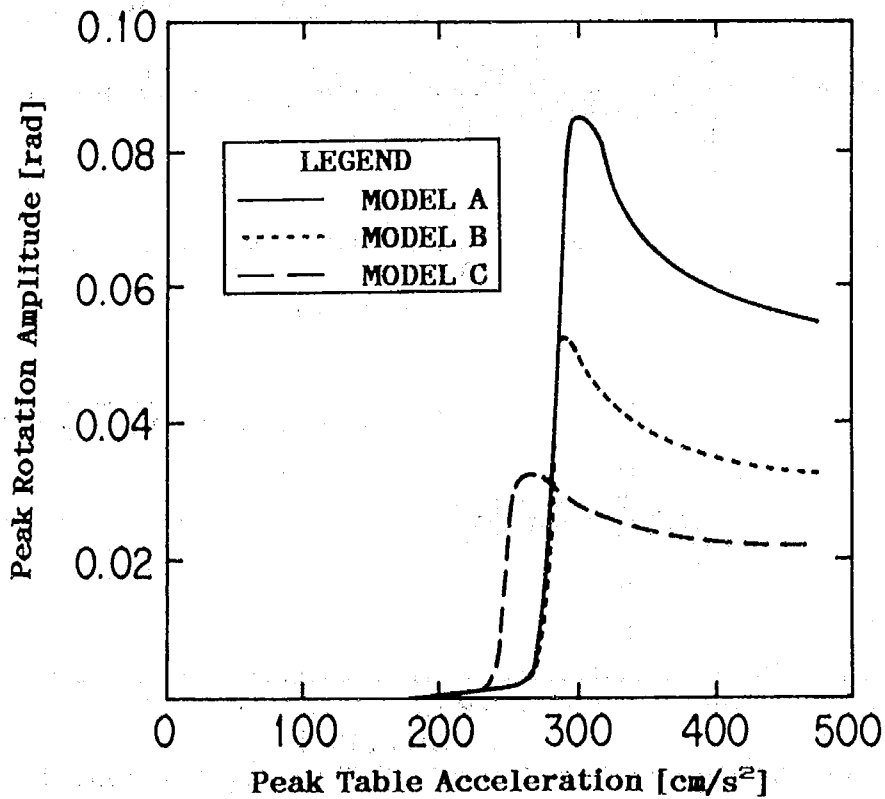
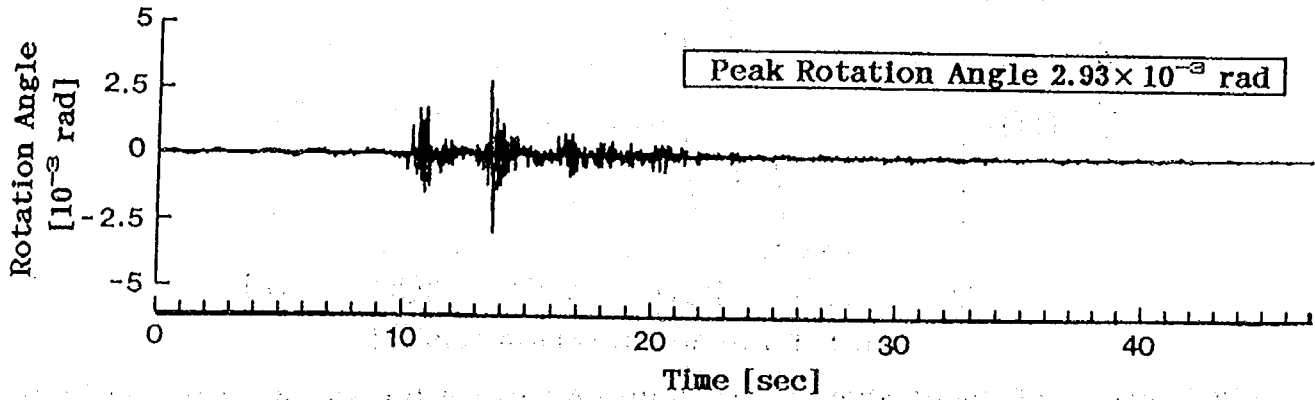
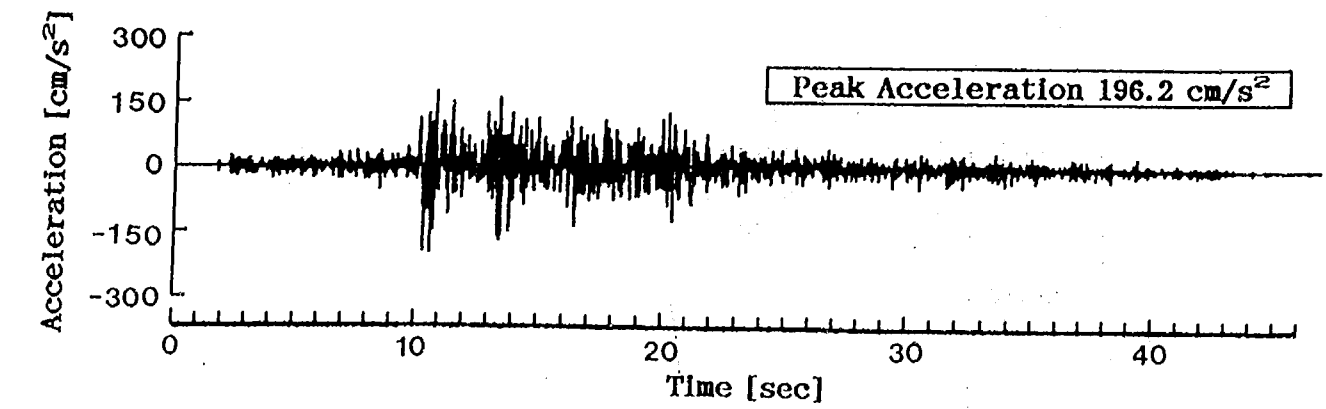
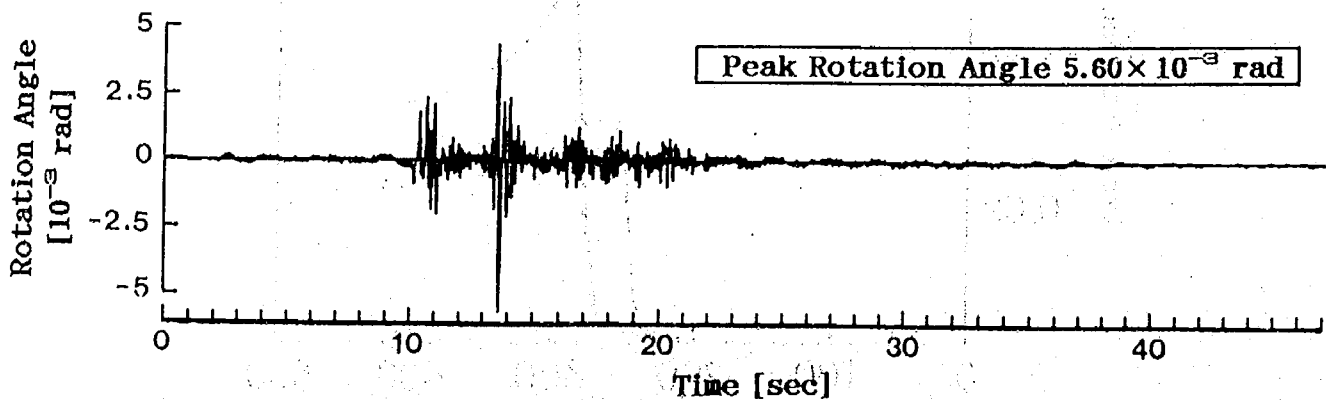
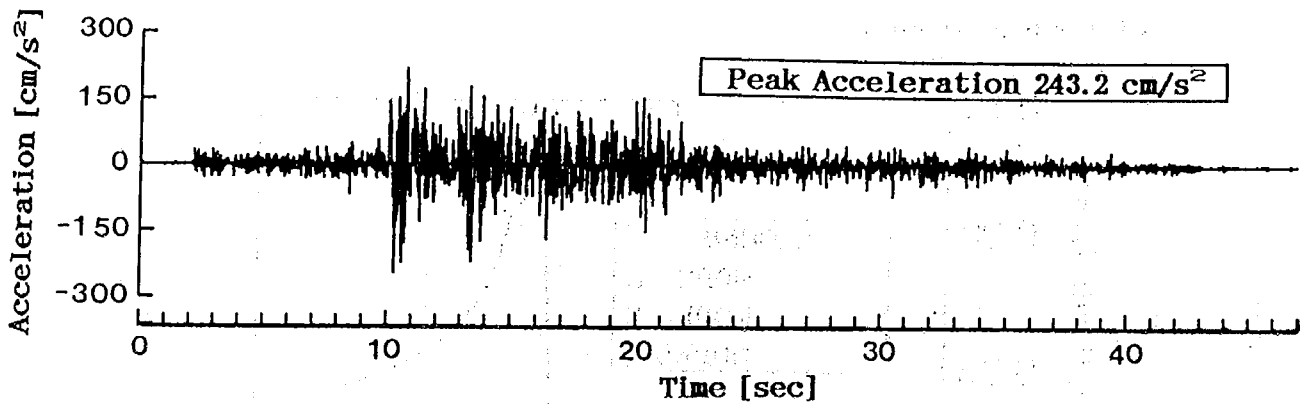


Fig.8 Effect of Model Size on Peak Table Acceleration vs. Rotation Angle of the Rigid Foundation on the rubber Sheet considering Confined Effect (Harmonic Excitation with Frequency of 3Hz)

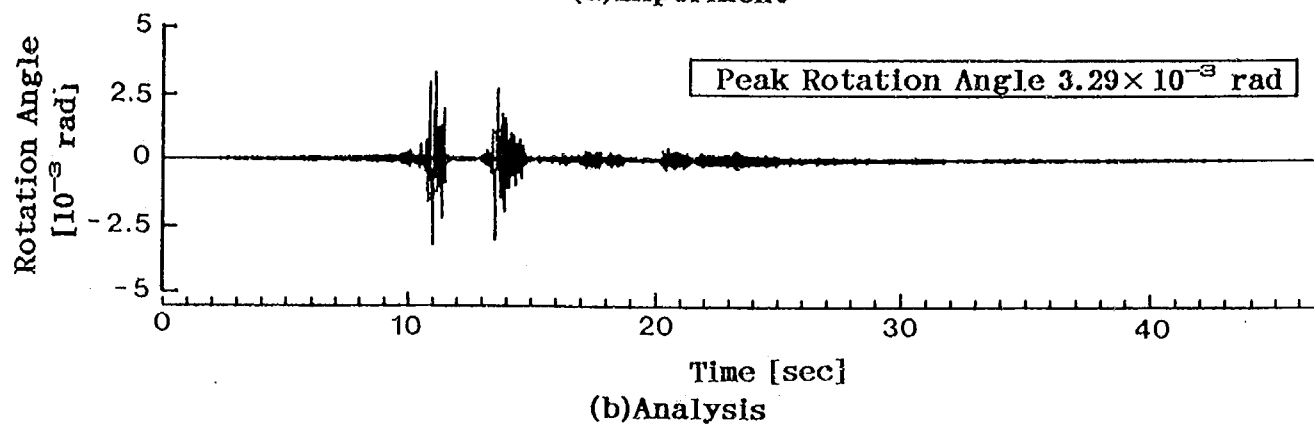
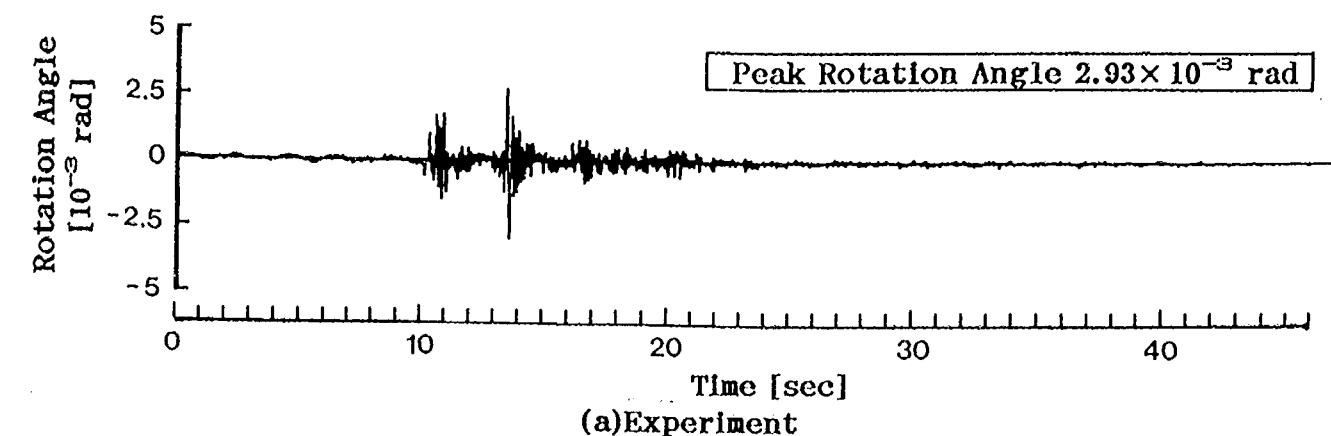


(a) Peak Table Acceleration of 0.4 Times the Original Record

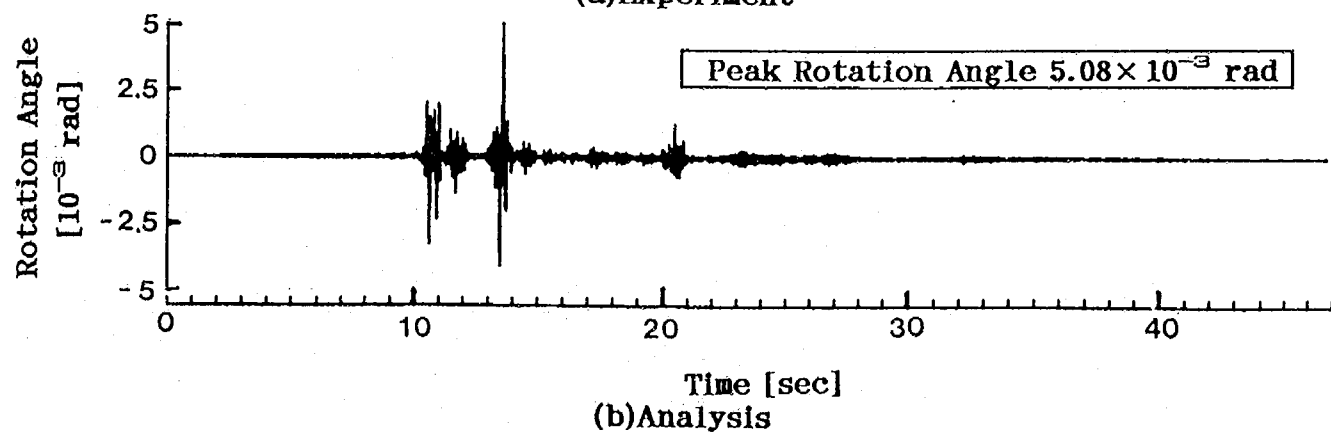
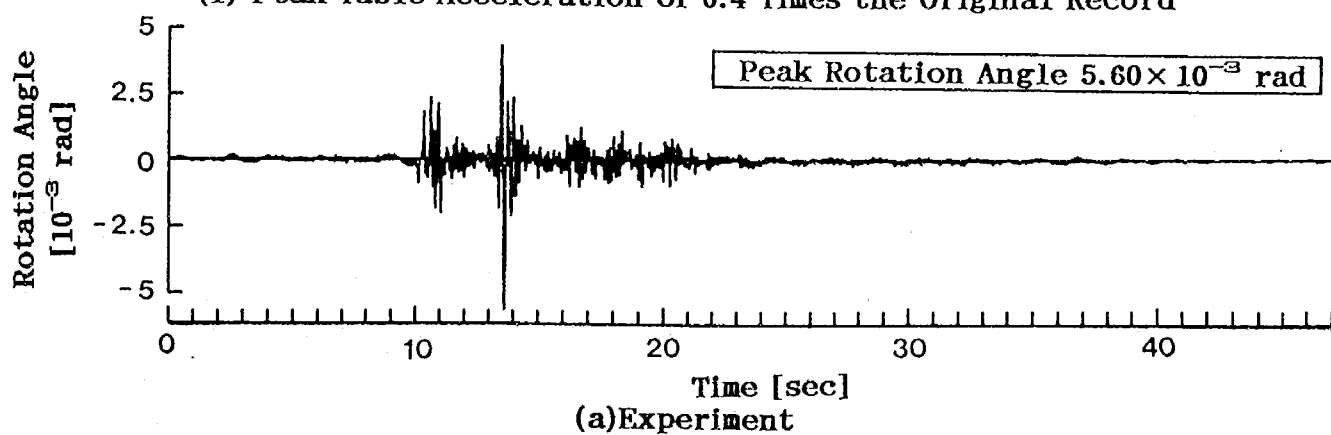


(b) Peak Table Acceleration of 0.5 Times the Original Record

Fig.9 Time History of Table Acceleration and Rotation Angle of the Model C



(1) Peak Table Acceleration of 0.4 Times the Original Record



(2) Peak Table Acceleration of 0.5 Times the Original Record

Fig.10 Comparison of Time History between Experiment and Analysis

Manual for Menshin Design of Highway Bridges

Kazuhiko KAWASHIMA

Kinji HASEGAWA

and

Hiroyuki NAGASHIMA

24th Joint Meeting of The U.S. -Japan Cooperative Program
in National Resources, Panel on Wind and Seismic Effects

Gaithersburg, MD, U. S. A.

May 1992

THE UNIVERSITY OF CHICAGO

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

PHILOSOPHY

DEPARTMENT

MANUAL FOR MENSIN DESIGN OF HIGHWAY BRIDGES

by

Kazuhiko KAWASHIMA¹⁾, Kinji HASEGAWA²⁾ and Hiroyuki NAGASHIMA³⁾

ABSTRACT

A three-year research program on "Development of Menshin Design of Highway Bridges" was made from April 1989 to March 1992. Concentrated efforts were paid to develop energy dissipating devices and falling-off prevention devices for bridges. An appropriate design method of highway bridges with energy dissipating devices and favorable application of the menshin design to highway bridges were studied. Final accomplishment of the three-year research was compiled in the form of "Manual for Menshin Design of Highway Bridges" in March 1992. This paper outlines the Manuals.

KEY WORDS

Menshin Design, Highway Bridge, Joint Research Program, Manual for Menshin Design

1. INTRODUCTION

A three-year Joint Research Program between the Public Works Research Institute and 26 groups consisting of 28 private firms on "Development of Menshin Design of Highway Bridges" was made from April 1989 to March 1992. This program intended to develop a rational seismic design method of highway bridges with energy dissipating devices. The highway bridges with span length from 20 m to 100 m were considered as the major target for this Joint Research Program.

As will be discussed later, "menshin" means "reduction of response" in Japanese. Although the menshin design is close with the base-isolation, natural period of bridge is not forcibly elongated in the menshin design, because there are various restrictions for increasing the natural period. Instead of elongating the natural period, emphasis is placed in the menshin design for increasing energy dissipating capability and distribution of lateral force to as many substructures as possible for decreasing lateral force for design of substructures.

As shown in Table 1, concentrated efforts were paid to 1) development of menshin (energy dissipating) devices for highway bridges, 2) development of falling-off prevention devices and expansion joints appropriate for the menshin bridges, 3) development of rational and simple menshin design method and 4) favorable application of menshin design to highway bridges.

In the research project, four high damping rubber bearings, 2 sliding friction dampers, a steel damper, a roller menshin bearing a link bearing and a viscous damper were newly

developed for bridges (refer to Fig. 1). The menshin devices for highway bridges have to be more compact and more weather-proof than the base-isolation devices for buildings since the menshin devices are installed at narrow and exposed crest of bridge columns.

The knock-off mechanism at an abutment to ease the impact force induced by the collision between the deck and the abutment (refer to Photo 1), and a finger type expansion joint (refer to Photo 2) which is distinguished from regular finger joints by the transverse movement as well as the longitudinal movement were also developed.

In the research project, a simple but rational menshin design method was developed. A series of shaking table tests were made to study the response of menshin bridges, and to provide realistic data of the seismic response of menshin bridges (refer to Photo 3). Analysis of strong motion records measured at a menshin bridge was also made.

From the study to investigate the favorable application of menshin design, it was found that the menshin design is effective for constructing super-multi-span continuous bridge with total deck length over 1 km. Application of the menshin design for seismic retrofit of existing bridges and deck connection for making existing simply supported girder bridges continuous was also studied (refer to Photo 4).

The reports of the project were compiled and published by the Public Works Research Institute in the form of the Joint Research Report. As well as these reports, as the final accomplishment of the three-year research program, the Manual for Menshin Design of Highway Bridges was compiled.

This paper presents the outline of the Manual.

2. SCOPES AND CONTENTS OF THE MANUAL

The Manual presents the seismic design method of highway bridges including design details when "the Structures Expecting Reduction of Inertia Force", which is specified in the "Part V Seismic Design" of the "Specifications of Design of Highway Bridges", is adopted. Application of the

- 1) Head of Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works Research Institute, Ministry of Construction
- 2) Assistant Manager of Planning Division, Road Bureau, Ministry of Construction Proper
- 3) Technical Research Division, Kanto Regional Construction Bureau, Ministry of Construction

menhshn design to seismc strengthening of substructures and connection of adjacent decks for making existing simply supported girder bridges continuous is also presented.

It should be noted that although the description of the Manual is given in the format of the specifications, it merely represents the accomplishments of the three-year research program, and it is not the mandate specifications.

The table of contents of the Manual is as follows:

1. General
 - 1.1 Scope of Application
 - 1.2 Definition of Terms
2. Basic Principle of Menhshn Design
3. Menhshn Design
 - 3.1 General
 - 3.2 Lateral Force Coefficient for Seismic Coefficient Method
 - 3.3 Lateral Force Coefficient and Equivalent Lateral Force Coefficient for Bearing Capacity Method
 - 3.4 Characteristics value for Menhshn Devices
 - 3.5 Computation of Natural Period of Bridge
 - 3.6 Computation of Modal Damping Ratio of Bridges
4. Design of Menhshn Devices
 - 4.1 Basic Principle for Design of Menhshn Devices
 - 4.2 Design of Laminated Rubber Bearings
 - 4.3 Design of Lead Rubber Bearings
 - 4.4 Design of High Damping Rubber Bearings
 - 4.5 Design of Steel Dampers
 - 4.6 Design of Friction Dampers
 - 4.7 Design of Viscous Dampers
 - 4.8 Design of Roller Dampers
5. Dynamic Response Analysis
 - 5.1 General
 - 5.2 Dynamic Response Analysis Method
 - 5.3 Analytical Models
 - 5.4 Design Seismic Forces
 - 5.5 Check of Safety
6. Design Details
 - 6.1 General
 - 6.2 Gap between Structures
 - 6.3 Falling-off Prevention Devices
 - 6.4 Knock-off Devices
 - 6.5 Details of Expansion Joints
7. Approving Tests of Menhshn Devices
 - 7.1 General
 - 7.2 Approving Tests for Confirming Requirements for Dynamic Loads
 - 7.3 Approving Tests for Confirming Requirements for Static Loads
8. Seismic Retrofit of Existing Bridges with Use of Menhshn Design
9. Application of Menhshn Design for Making Existing Simply Supported Girder Bridges Continuous

3. References for Dynamic Response Analysis
4. Expansion Joints and Falling-off Prevention Devices for Highway Bridges
5. Design Example of Prestressed Concrete Bridges
6. Design Examples of Steel Girder Bridges
7. Design Examples of Super-multi-span Continuous Bridges
8. Design Examples for Seismic Retrofit
9. Design Examples for Making Existing Simply Supported Girders Continuous
10. Standard Acceleration Ground Motions Recommended for Dynamic Response Analysis

3. BASIC PRINCIPLE OF MENSHPN DESIGN

Whether the menhshn design should be adopted or not has to be decided based on the advantages of increasing energy dissipating capability from not only seismic safety but function at normal time.

In general, menhshn design can be adopted with advantage for the following conditions:

- 1) bridges on stiff and stable soils
- 2) bridges with stiff substructures with short natural period

It is recommended that the natural period of the bridge designed by the menhshn design is at least 1.5 times longer than that of the bridge without menhshn devices. Such elongation of the natural period of the menhshn bridge makes the coupling vibration so small that the deformation of the bridge be concentrated at the menhshn devices.

The most important decision of the menhshn design is how much increase of the natural period be made. In Japan soils are generally very weak at the bridge construction sites. Significant earthquakes with magnitude over 8, which produce long period ground motions, occur at shorter recurrence period than other countries. Taking account such evidences into account, the design seismic force level is take very high for design of highway bridges in Japan in comparison with the seismic force level adopted in U.S.A., New Zealand and Italy. The natural period at which the design acceleration decrease with increasing the natural period is taken longer, i.e., 1.1 second for type I soils (stiff site), 1.3 second for type II soils (medium site) and 1.5 second for type III soils (soft soils). Therefore, the advantage for reducing deck response can be realized only by elongating the natural period longer than 1.1 ~ 1.5 second.

However elongation of the natural period brings the increase of the deck response displacement, and this requires the adoption of joints with long legs. This is, however, a crucial requirements difficult to adopt, in particular, for overcrossing in city area because it causes noise and vibration pollution, which is the most serious concern in city area. Furthermore it will have the disadvantage for maintenance because replacement of the damaged expansion joint due to traffic load as well as the pavement adjacent

Appendices

1. Procedure of Menhshn Design
2. Menhshn Devices for Highway Bridges

to the joint has to be frequently made.

Based on these reasons, it is not adopted to forcibly elongate the natural period in order to decrease the deck response acceleration. It is therefore more appropriate to adopt the concept of base isolation not for elongating the natural period but for increasing energy dissipating capability. Distribution of the inertia force to as many substructures as possible, which is the practice adopted in Japan for long time, is also significantly advantageous.

Therefore basic principle recommended for introducing the base isolation to highway bridges in Japan is as follows :

- 1) Distribute the inertia force to many substructures so intentionally that the inertia force be distributed equally by selecting the spring stiffness of the isolators. Simultaneously, reduce deck response by increasing energy dissipating capability by means of damper.
- 2) Do not forcibly elongate the natural period, but select the natural period so that the resonance of the bridge with soils be avoided.

Such design criteria is no more the "isolation" of the "base". Therefore it is proposed to call the design concept as "menshin". The "menshin" means "reduction of seismic response" in Japanese.

In menshin design, bridges can be designed by following the standard static design method (static frame analysis). More precise approve of the seismic safety can be made by the dynamic response analysis, if required. In such analyses, the menshin devices (isolators and dampers) are idealized by as a set of equivalent linear springs. Equivalent stiffness and the equivalent damping ratio of the isolator and damper are the major parameters used in the analyses. In the static frame analysis, natural period of the bridge can be computed for each seismic design structural unit as :

$$T = 2.01\sqrt{\delta} \quad (1)$$

$$\delta = \frac{\int w_i \cdot u_i^2}{\int w_i \cdot u_i} \quad (2)$$

where

T : natural period (sec)

w_i : dead weight (tf/m) of the seismic design structural unit (super structure and substructure above the ground surface assumed in seismic design) at point "I"

u_i : lateral displacement (m) developed in the seismic design structural unit at point "I" when subjected to w_i in the direction considered in design

Damping ratio of the bridges is computed as

$$h = \frac{\sum K_{BI} \cdot u_{BI}^2 \cdot c_{BI}}{\sum K_{BI} \cdot u_{BI}^2 \cdot c_i} \quad (3)$$

$$c_{BI} = h_{BI} + \frac{h_{PI}}{K_{PI}} + \frac{h_{FUI}}{K_{FUI}} + \frac{h_{F\theta I} \cdot H^2}{K_{F\theta I}}$$

$$c_i = 1 + \frac{K_{BI}}{K_{PI}} + \frac{K_{BI}}{K_{FUI}} + \frac{K_{BI} \cdot H^2}{K_{F\theta I}}$$

where

h : Modal damping ratio of bridge

h_{BI} : Damping ratio of i-th damper

h_{PI} : Damping ratio of i-th pier/abutment

h_{FUI} : Damping ratio of i-th foundation associated with translational movement

$h_{F\theta I}$: Damping ratio of i-th foundation associated with rotation

K_{PI} : Equivalent stiffness of i-th pier/abutment

K_{FUI} : Translational stiffness of i-th foundation

$K_{F\theta I}$: Rotational stiffness of i-th foundation

u_{BI} : Design displacement of i-th menshin device

H : Height from the bottom of pier to the gravity center of deck

Eq.(3) give the approximate estimation of the damping ratio of bridge assuming that translation and rotation of the foundation are developed only by the inertia force of the deck. When the effect of the inertia force of piers and abutments for evaluating the translation and rotation of the foundation can not be disregarded, it is appropriate to compute the modal damping ratio as

$$h = \frac{\sum_{j=1}^n \phi_{ij}^T \cdot h_j \cdot K_j \cdot \phi_{ij}}{\phi_i^T \cdot K \cdot \phi_i} \quad (4)$$

where

ϕ_{ij} : Mode vector of j-th structural component for i-th mode

h_j : Damping ratio of j-th structural component

K_j : Stiffness matrix of j-th structural component

ϕ_i^T : Mode vector of bridge for i-th mode

K : Stiffness matrix of bridge

Table 2 shows the damping ratio recommended for structural components for Eqs.(3) and (4).

4. DESIGN FORCE FOR MENSHTN DESIGN

In menshin design, the menshin devices are designed by the Seismic Coefficient Method and the Bearing Capacity Method. In both method, the lateral force is statically applied to the bridge, and the seismic safety is examined based on the allowable stress design approach in the Seismic Coefficient Method and bearing capacity basis considering ductility in the Bearing Capacity Method. Bridges are designed by the Seismic

Coefficient Method, and then the ductility is checked for reinforced concrete piers by the Bearing Capacity Method.

In the Seismic Coefficient Method, the design lateral force coefficient k_h is given as

$$k_h = c_z \cdot c_g \cdot c_i \cdot c_T \cdot c_E \cdot k_{ho} \geq 0.1$$

where

$$c_T \cdot c_E \geq 0.8 \quad (5)$$

where

- k_h : Lateral force coefficient
- c_z : Modification factor for zone (refer to Fig. 2)
- c_g : Modification factor for ground condition (refer to Table 3)
- c_i : Modification factor for importance (refer to Table 4)
- c_T : Modification factor for structural response (refer to Table 5)
- c_E : Modification factor for energy dissipation capability (refer to Table 6)
- k_{ho} : Standard design horizontal seismic coefficient (= 0.2)

Those factors excluding c_E are the ones specified in the Specifications of Design of Highway Bridges. The modification coefficient c_E takes a value as shown in Table 6 depending on the first modal damping ratio of the bridge. Reduction of the design lateral force as large as 20 % is proposed in the Manual.

In the Bearing Capacity Method, the lateral force coefficient k_{he} and the equivalent lateral force coefficient k_{ho} are given as

$$k_{he} = \frac{k_{ho}}{\sqrt{2\mu - 1}} \geq 0.3 \quad (6)$$

$$k_{ho} = c_z \cdot c_i \cdot c_R \cdot c_E \cdot k_{hoo} \quad (7)$$

where

- k_{he} : Equivalent lateral force coefficient for Bearing Capacity Method
- k_{ho} : Lateral force coefficient for Bearing Capacity Method
- c_z : Modification factor for zone (refer to Fig. 2)
- c_i : Modification factor for importance (refer to Table 4)
- c_R : Modification factor for structural response (refer to Table 7)
- c_E : Modification factor for energy dissipation capability
- k_{hoo} : Standard lateral force coefficient for Bearing Capacity Method (=1.0)
- μ : Allowable ductility factor of reinforced concrete piers

The modification factors excluding c_E are the ones specified in the Specifications of Design of Highway Bridges. The modification factor c_E depends on the modal damping ratio of the

bridge, and takes a value of Table 8. Reduction of the design force as large as 30 % is proposed.

5. DESIGN OF MENSIN DEVICES

5.1 Characteristics Value of Menshin Devices

In the menshin design, the displacement assumed in design for the devices u_B (design displacement of menshin device), the equivalent stiffness K_B and the equivalent damping ratio h_B of the menshin device are the key factors. They are evaluated as:

(1) Design Displacement of Menshin Device

The design displacement of menshin device is evaluated as

$$u_B = \frac{k_h \cdot W_u}{K_B} \quad (\text{S. C. Method}) \quad (8)$$

$$u_B = \frac{k_{ho} \cdot W_u}{K_B} \quad (\text{B. C. Method}) \quad (9)$$

where

- u_B : Design displacement (m) of menshin device
- k_h : Lateral force coefficient by Eq.(5) for Seismic Coefficient Method
- k_{ho} : Lateral force coefficient by Eq.(6) for Bearing Capacity Method
- K_B : Equivalent stiffness (tf/m) of menshin device
- W_u : Weight of superstructure (tf) supported by the menshin device

(2) Equivalent Stiffness and Equivalent Damping Ratio

The equivalent stiffness and the equivalent damping ratio of menshin device are evaluated from the hysteresis loops as

$$K_B = \frac{F(u_{BE}) - F(-u_{BE})}{2u_{BE}} \quad (10)$$

$$h_B = \frac{\Delta W}{2\pi W} \quad (11)$$

where

- K_B : Equivalent stiffness (tf/m) of menshin device
- h_B : Equivalent damping ratio of menshin device
- u_{BE} : Effective design displacement (m) of menshin device, and is given as
- $u_{BE} = c_B \cdot u_B$ (12)
- u_B : Design displacement (m) of menshin device
- c_B : Modification coefficient for evaluating the effective design displacement (=0.7)
- $F(u)$: Lateral force required to produce u for the menshin device
- W : Strain energy induced in menshin device associated with c_{BE} displacement (refer to Fig. 3)
- ΔW : Energy dissipated in menshin device per

cycle (refer to Fig. 3)

5.2 Requirements for Dynamic Load

(1) Menshin devices have to be designed and fabricated so that their equivalent stiffness K_B and equivalent damping ratio h_B be within $\pm 20\%$ of the design values. The equivalent stiffness K_B and equivalent damping ratio h_B for this requirement have to be evaluated by Eqs. (10) and (11), and they shall be the ones averaged over K_B and h_B from 4th to 10th loading hysteresis among 10 cyclic loading reversals with harmonic displacement of c_{BE} . This is because deck acceleration and displacement are within tolerable range when the scatter of equivalent stiffness and the equivalent damping ratio are within $\pm 20\%$ of the design values. However, the devices which produce unstable hysteretic behavior should not be adopted.

(2) Menshin devices have to be stable against 50 cycles of harmonic loading with displacement of u_B . The number of 50 was determined from the fact that number of load reversals developed during a major earthquake may be about 30. About 50% tolerance was included in 50.

(3) Deck should return to the rest position even after it was subjected to a large earthquake. Therefore the residual displacement developed in menshin devices after smoothly releasing from the deformed displacement of u_B by Eq. (9) has to satisfy

$$u_{BR} \leq 0.1 \cdot u_B \quad (13)$$

where

u_{BR} : Residual displacement (cm)

u_B :

Design displacement(cm) of menshin device by Eq. (9)

(4) The equivalent stiffness and the equivalent damping ratio of menshin devices have to be stable against the change of load condition at normal time, change of natural environment such as the temperature change and cyclic loading developed by an earthquake. Stability has to be examined against the following requirements:

- 1) cyclic loading due to elongation and shrinkage of deck due to the temperature change and the active load
- 2) effect of loading hysteresis
- 3) variation of vertical loading
- 4) effect of loading rate
- 5) effect of pre-deformation due to creep and shrinkage
- 6) direction of excitation
- 7) change of the equivalent stiffness and the equivalent damping ratio depending on the temperature change

5.3 Requirements for Static Load

(1) Materials and mechanism of menshin devices have to give credit to long term use. They have to be stable against cyclic elongation and

shrinkage of deck due to temperature change.

(2) Menshin devices have to be stable against local shear strain. Check of the local shear strain has to be made in accordance of "3.6 Design of Rubber Bearings" of the "Design Guidelines of Bearings".

(3) For rubber-type menshin devices, the creep of rubber in vertical direction due to dead weight of superstructure shall not exceed 5% of total thickness of the rubber.

(4) The equivalent stiffness of menshin device at -10°C normalized by the equivalent stiffness at 40°C shall not exceed 1.5. Because the temperatures of -10°C and 40°C is for moderate climate area, at cold area they have to be decided based on the appropriate site condition.

(5) Menshin devices have to have appropriate initial stiffness so that harmful displacement of deck due to non-seismic lateral force such as wind effects be avoided.

6. APPROVING TESTS OF MENSIN DEVICES

6.1 General

Because the menshin device is one of the the important structural components of menshin bridges, their requirements have to be well approved by the tests. There are various types of menshin devices for use of bridges. Based on the mechanism for producing the energy dissipation, the devices may be classified into three groups as

- (a) displacement dependent type,
- (b) friction force type, and
- (c) velocity dependent (viscous) type.

The approving tests for the menshin devices are presented in the Manual for each of the three types.

6.2 Approving Tests for Dynamic Loads

For approving the requirements for dynamic loads presented in 5.2, the following approving tests are provided in the Manual.

- (1) Approving test for confirming the equivalent stiffness and the equivalent damping ratio
- (2) Approving test for confirming stableness against 50 cycles harmonic load reversals with u_B displacement
- (3) Approving test for confirming residual displacement requirement
- (4) Approving tests for confirming stableness against variation of load condition at normal time, change of natural environment such as the temperature change and cyclic loading developed by an earthquake. These tests include :

- (a) Stableness against Cyclic Load Reversals

Some menshin devices have the dependence of the equivalent stiffness and the equivalent damping ratio on number of cyclic loading. Because considerable change of these parameters during an earthquake produces bridge response significantly different from the one assumed in

design, it is not desirable.

(b) Stableness against Load Hysteresis

In some menshin devices, the equivalent stiffness and the equivalent damping ratio depend on the past experience, in particular, the largest deformation ever experienced. Such dependence of K_B and h_B on the hysteresis ever experienced is significant, it produces different bridge response during an earthquake. Therefore, effect of load hysteresis has to be examined.

(c) Stableness against Change of Compression Force

Compression force applied to the menshin device due to dead weight of superstructure may change associated with the error involved in construction stage and settlement of substructure. If the menshin device shows significantly different stiffness and damping properties due to such change of compression force, it brings different structural response. Therefore, stableness of the equivalent stiffness and the equivalent damping ratio of menshin devices have to be checked.

(d) Stableness against Change of Loading Rate

Because loading rate to the menshin devices is not the same during excitation, dependence of the equivalent stiffness and the equivalent damping ratio on the loading rate has to be examined.

(e) Stableness against Pre-deformation

Menshin devices often start to deform from the displacement drifted to the rest point. This is actually developed if an earthquake occurs when the menshin devices deform due to the deck elongation associated with temperature change and shrinkage of concrete. Because the equivalent stiffness and the equivalent damping ratio depend on the amount of such pre-deformation, their properties have to be checked.

(f) Stableness against Temperature Change

Menshin device have to be stable against daily and yearly change of temperature.

6.3 Approving Tests for Static Loads

For approving the requirements for static loads presented in 5-3, the following tests are required.

(1) Approving test for confirming durability and stableness against cyclic loading associated with the daily and yearly elongation and shrinkage of deck.

(2) Test for evaluating the stiffness of menshin devices subjected to extremely low-rate deformation such as the one encountered due to yearly temperature change and concrete creep. The stiffness of the menshin device during such low-rate deformation is quite important in static design for computing the lateral force developed in substructures due to the temperature change and shrinkage of concrete.

(3) Approving test for confirming the dependence of the equivalent stiffness of menshin device on

temperature.

6.4 Two examples of the Approving Test

Approving test methods presented in 6.2 and 6.3 are precisely described in the Manual for each of the displacement dependent type devices, friction force type devices and the velocity dependent type devices. As the example of such approving tests, the test method for confirming stableness against cyclic loading and the test method for confirming the effect of loading rate are briefly described for the displacement dependent type devices in the following.

(1) Approving Test for Confirming Stableness against Cyclic Loads

At the room temperature of 20°C apply 10 cycles of harmonic lateral load to the menshin device with loading displacement of u_B computed by Eq. (9) or smaller and with the frequency of 0.5 Hz. The vertical load equivalent to the design dead load shall be simultaneously applied. The equivalent stiffness and the equivalent damping ratio from the test have to satisfy Eqs. (14) and (15).

$$R_{KC} = \frac{|K_{BJ} - K_{Bm}|}{K_{Bm}} \begin{matrix} \leq 0.3 \text{ (j=1,2,3)} \\ \leq 0.1 \text{ (j=4,5,\dots,10)} \end{matrix} \quad (14)$$

$$R_{hC} = \frac{|h_{BJ} - h_{Bm}|}{h_{Bm}} \begin{matrix} \leq 0.3 \text{ (j=1,2,3)} \\ \leq 0.1 \text{ (j=4,5,\dots,10)} \end{matrix} \quad (15)$$

where,

R_{KC} : Variation ratio of equivalent stiffness

R_{hC} : Variation ratio of equivalent damping ratio

K_{BJ} : Equivalent stiffness at j-th load reversal(tf/m)

K_{Bm} : Averaged equivalent stiffness by Eq. (16)

h_{BJ} : Equivalent damping ratio at j-th load reversal

h_{Bm} : Averaged damping ratio by Eq. (17)

$$K_{Bm} = \frac{1}{7} \sum_{j=1}^{10} K_{BJ} \quad (16)$$

$$h_{Bm} = \frac{1}{7} \sum_{j=1}^{10} h_{BJ} \quad (17)$$

Being different with Eqs. (10) and (11), the equivalent stiffness and the equivalent damping ratio for the approving tests are defined as

$$K_B = \frac{F(u_B) - F(-u_B)}{2u_B} \quad (18)$$

$$h_B = \frac{\Delta W}{2\pi W} \quad (19)$$

where

K_B : Equivalent stiffness (tf/m) of menshin device

h_B : Equivalent damping ratio of menshin device

u_B : Design displacement (m) of menshin device

$F(u)$: Lateral force (tf) required to produce u displacement for the menshin device

W : Strain energy (tfm) induced in menshin device associated with u_B displacement (refer to Fig. 4)

ΔW : energy dissipated in menshin device per cycle (refer to Fig. 4)

(2) Approving Test for Confirming the Stableness against Loading Rate

At the temperature of 20°C apply 10 cycles of harmonic load with the displacement equivalent to either 100 % of the total thickness of rubber or ± 15 cm. Frequency of the load reversal shall be 0.1 Hz, 0.5 Hz and 1.0 Hz. The equivalent stiffness and the equivalent damping ratio have to satisfy Eqs. (20) and (21).

$$R_{K_T} = \frac{|K_{Bm1} - K_{Bm2}|}{K_{Bm1}} \leq 0.2 \quad (j=1,2) \quad (20)$$

$$R_{h_T} = \frac{|h_{Bm1} - h_{Bm2}|}{h_{Bm1}} \leq 0.2 \quad (j=1,2) \quad (21)$$

where,

R_{K_T} : Variation ratio of equivalent stiffness

R_{h_T} : Variation ratio of equivalent damping ratio

$K_{Bm1}, K_{Bm2}, K_{Bm3}$: Equivalent stiffness by Eq. (16) at frequency of 0.5 Hz, 0.1 Hz and 1.0 Hz, respectively

$h_{Bm1}, h_{Bm2}, h_{Bm3}$: Equivalent damping ratio by Eq. (17) at frequency of 0.5 Hz, 0.1 Hz and 1.0 Hz, respectively

7. CONCLUDING REMARKS

Excluding specially long-span bridges, it is not easily adopted for highway bridges to reduce the lateral force by forcibly elongating the natural period, because seismic force level is very high even at long natural period, and because the increase of deck response displacement brings various problems such as environmental pollution. Therefore the menshin design, which places emphasis not on the forcible elongation of the natural period but on the increase of energy dissipating capability and distribution of inertia force of superstructure to as many substructures as possible, is proposed. By the menshin design, it is proposed to decrease the lateral force as large as 20 % for the Seismic Coefficient Method and 30 % for the Bearing Capacity Method.

ACKNOWLEDGEMENTS

For executing the three-year joint research

project, various supports and encouragement were obtained from many organizations including the Head-quarters of the Ministry of Construction. Without endeavour of more than 100 participants from the 28 private firms, this project could not be successfully made. The authors express their sincere thanks for all who participated in this Joint Research Program.

REFERENCES

- 1) Public Works Research Institute and 28 Private Firms : Report of the Joint Research on Menshin Systems for Highway Bridges, No. 1, July 1990, No. 2, July 1991 and No. 3, March 1992, Joint Research Report, No. 44, No. 60 and No. 75, Public Works Research Institute (In Japanese)
- 2) Public Works Research Institute and 28 Private Firms : Manual for Menshin Design of Highway Bridges (Draft), March 1992, Joint Research Report, Public Works Research Institute (In Japanese)
- 3) Japan Road Association : Part V Seismic Design of the Specifications of the Design of Highway Bridges, February 1990 (In Japanese)
- 4) Technology Research Center for National Land Development : Guidelines for Base Isolation of Highway Bridges, March 1989 (In Japanese)
- 5) Japan Road Association : Guide-Specifications of Design of Bearings, July 1991 (In Japanese)
- 6) Buckle, I. G. and Mayes, R. L. : Seismic Isolation : History, Application and Performance - A World View, Earthquake Spectra, Vol. 6, No. 2, May 1990
- 7) AASHTO : Guide Specifications for Seismic Isolation Design, June 1991
- 8) McKay, G. R., Chapman, H. E. and Kirkcaldie, D. K. : Seismic Isolation : New Zealand Application, Earthquake Spectra, Vol. 6, No. 2, May 1990
- 9) Guidelines for Seismic Design of Bridges with Isolation/Dissipator Devices, Antiseismic Protection of Structures on the Autostrade S.p.A/ Network, October 1991
- 10) Kawashima, K., Hasegawa, K. and Nagashima, H. : A Perspective of Menshin Design for Highway Bridges in Japan, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991
- 11) Kawashima, K., Hasegawa, K. and Nagashima, H. : Experiment and Analysis on Seismic Response of Menshin Bridges, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991

Table 1 Research Theme and Organizations of Joint Research Program
between the Public Works Research Institute and 28 Private
Firms for Developing Menshin Design Method of Highway Bridges

Research Theme	P	Ka	Si	Ob	Ku	Tn	H	Ni	Su	M	G	OR	Ti	I	Nk	Ko	Ns	Oi	Y	To	Bs	Bb	Se	Sh	Pc	J	N	Chief	Sub-Chief	
1. Development of Device for Isolation																														
1.1 High Energy Absorbing Rubber Bearing								○											○	○	○		○					Ol	Bs	
1.2 Friction Damper								○					○									○	○							
1.3 Steel Damper								○																						
1.4 Link Bearing Develop of								○							○															
1.5 Viscous Damper																	○		○											
1.6 Test Method	○							○							○		○	◎	○	○	○	○	○							
2. Development of Expansion Joint and Falling-off Prevention Device for Isolated Bridge																												Tc	—	
2.1 Expansion Joint				○									○																	
2.2 Falling-off Prevention Device	○												○					○												
3. Development of Design Method for Isolated Bridge																														
3.1 Design Philosophy	○	○	○	○		○						○	◎					○								○	○	○	Ka	Pc
3.2 Dynamic Responce Analysis Method	○		○		○	◎		○					◎	○				○								○	○			
3.3 Design Method of Device for Isolation	○						○	○						○				○	○	○	○	○	○	○						
3.4 Simplified Design Method	○	○	◎		○	○						○	○				○	○	○	○	○	○	○							
3.5 Design Method of Expansion Joint and Fall-ing-off Prevention Device	○			○			○								○			○							◎					
4. Application of Base Isolation to Bridge																														
4.1 Application to Prestressed Concrete Bridge	○	◎							○	○			○														○			
4.2 Application to Steel Bridge	○													○	○	○											◎	Ob	J	
4.3 Application to Multiple Super-long Bridge	○			◎					○							○									○	○	○			
4.4 Application to Seismic Retrofit	○									○				○				○								◎				

P: Public Works Research Institute, Ka: Kajima, Si: Shimizu, Ob: Ohbayashi, Ku: Kumagai, Tn: Takenaka Doboku + Takenaka, H: Hazama, Ni: Nishimatsu, Su: Sumitomo, M: Mitsui, G: Goyoh, Ok: Okumura, Ti: Taisei + Tokyo Fablic + Nippon Chuzo, I: Ishikawajima Harima, Nk: NKK + Nippon Chuzo, Ko: Kobe Steel, Ns: Nippon Seiko, Oe: Oiles, Y: Yokohama Rubber, To: Toyo Rubber, Bs: Bridgestone, Bb: BBM, Se: Seibu Polymer, Sh: Showa Densen, Pc: Pacific Consultants, J: Japan Engineering Consultant, N: New Structural Engineering Consultants.

Table 2 Damping Ratio Recommended for Structural Components

Structural Components	Steel	Concrete
Superstructures	0.02~0.03	0.03~0.05
Menshin Device	Damping Ratio by Eq.(11)	
Pier/Columns	0.03~0.05	0.05~0.1
Footing	0.1~0.3	

Table 3 Modification Factor for Ground Condition c_g

Ground Group	I	II	III
c_g	0.8	1.0	1.2

Table 4 Modification Factor for Importance c_i

Group	c_i	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road. Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 5 Modification Factor for Structural Response c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$	$0.1 \leq T \leq 1.1$	$1.1 < T$
	$c_T = 2.69T^{1/3} \geq 1.00$	$c_T = 1.25$	$c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$	$0.2 \leq T \leq 1.3$	$1.3 < T$
	$c_T = 2.15T^{1/3} \geq 1.00$	$c_T = 1.25$	$c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$	$0.34 \leq T \leq 1.5$	$1.5 < T$
	$c_T = 1.80T^{1/3} \geq 1.00$	$c_T = 1.25$	$c_T = 1.64T^{-2/3}$

Table 6 Modification Factor for Energy Dissipation Capability c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$h \geq 0.1$	0.9

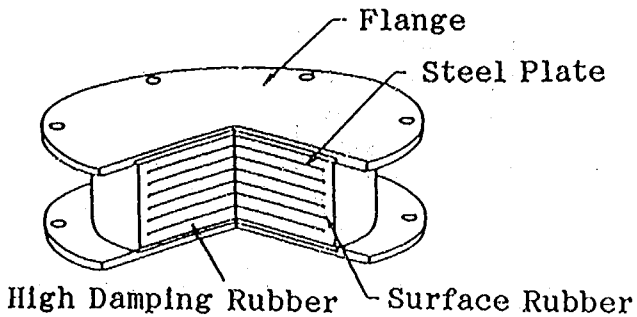
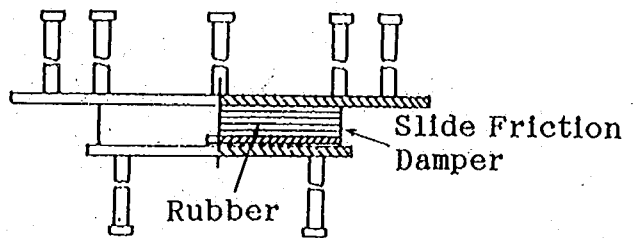
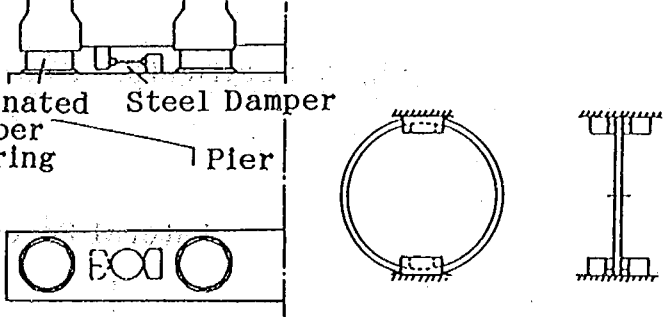
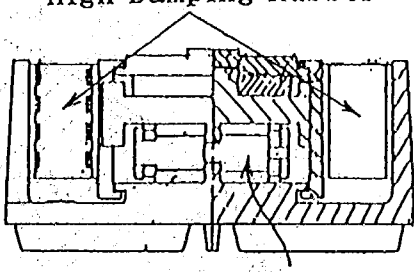
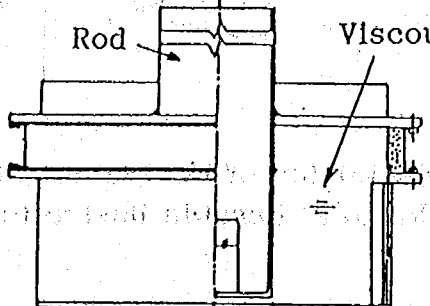
Table 7 Modification factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leq 1.4$ $c_R = 0.7$	$1.4 < T_{EQ}$ $c_R = 0.876T_{EQ}^{-2/3}$	
Group II	$T_{EQ} < 0.18$ $c_R = 1.51 T_{EQ}^{1/3} \geq 0.7$	$0.18 \leq T_{EQ} \leq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51T_{EQ}^{1/3} \geq 0.7$	$0.29 \leq T_{EQ} \leq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59T_{EQ}^{-2/3}$

Table 8 Modification Factor for Energy Dissipating Capability

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$0.1 \leq h < 0.12$	0.9
$0.12 \leq h < 0.15$	0.8
$0.15 \leq h$	0.7

Fig. 1 Menshin Devices Developed

TYPE	OUTLINE OF MENSHIN DEVICE
HIGH DAMPING RUBBER BEARING	 Flange Steel Plate High Damping Rubber Surface Rubber
SLIDE FRICTION -RUBBER BEARING	 Slide Friction Damper Rubber
STEEL DAMPER	 Laminated Rubber Bearing Steel Damper Pier
ROLLER MENSHIN BEARING	 High Damping Rubber Roller
VISCOUS	 Rod Viscous Material

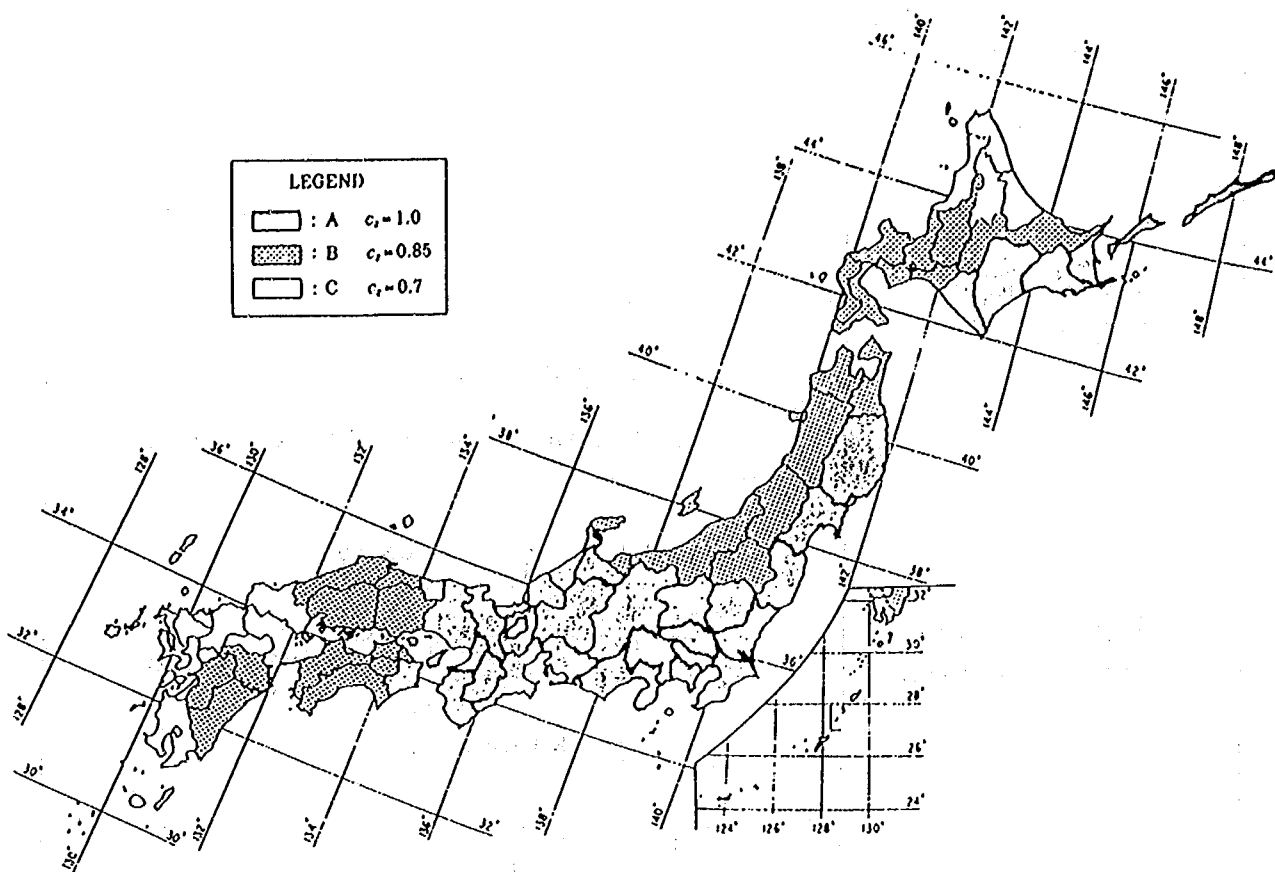


Fig. 2 Modification Factor for Zone c_z

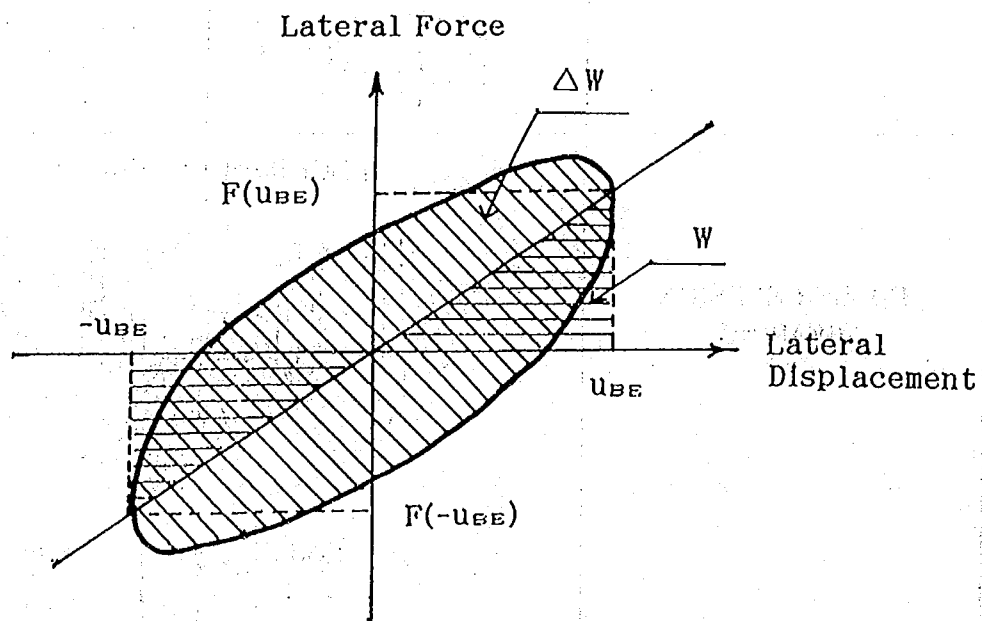


Fig. 3 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menhin Device for Design of Menhin Device

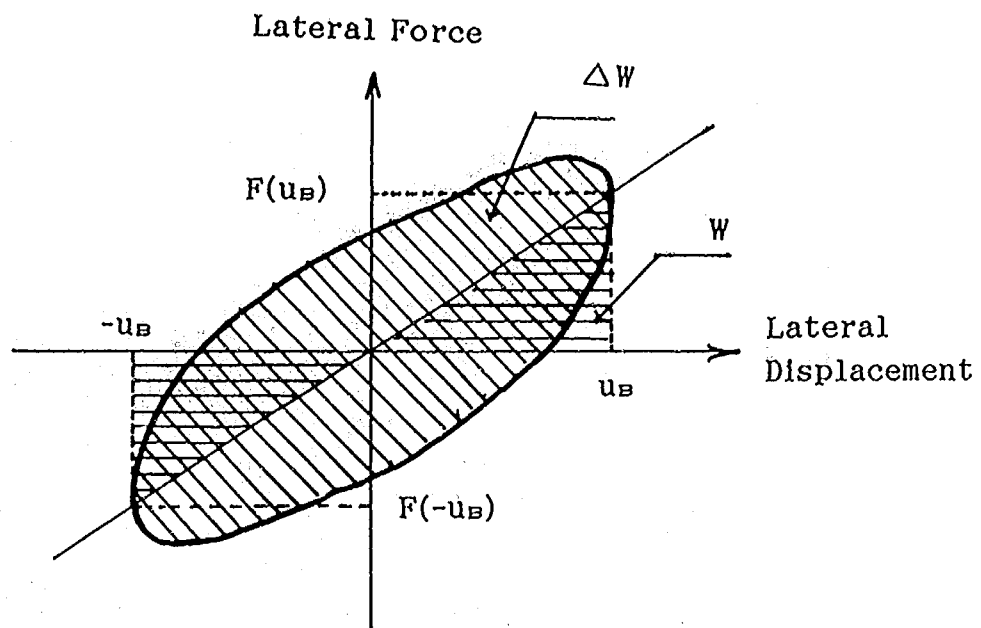


Fig. 4 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menshin Device for Approving Tests

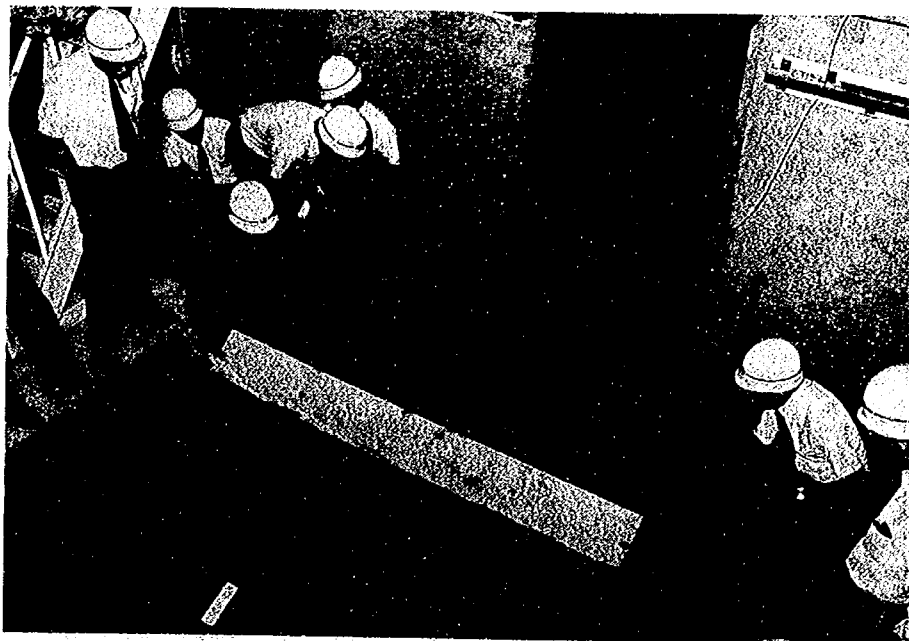


Photo 1 Knock-off Abutment

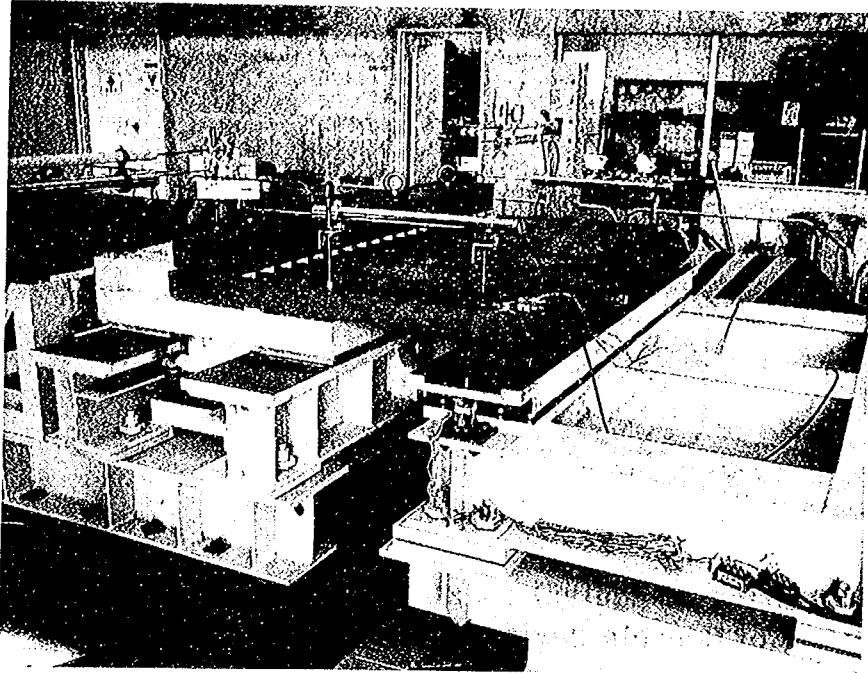


Photo 2 Expansion Joint Movable In Longitudinal and Transverse Directions

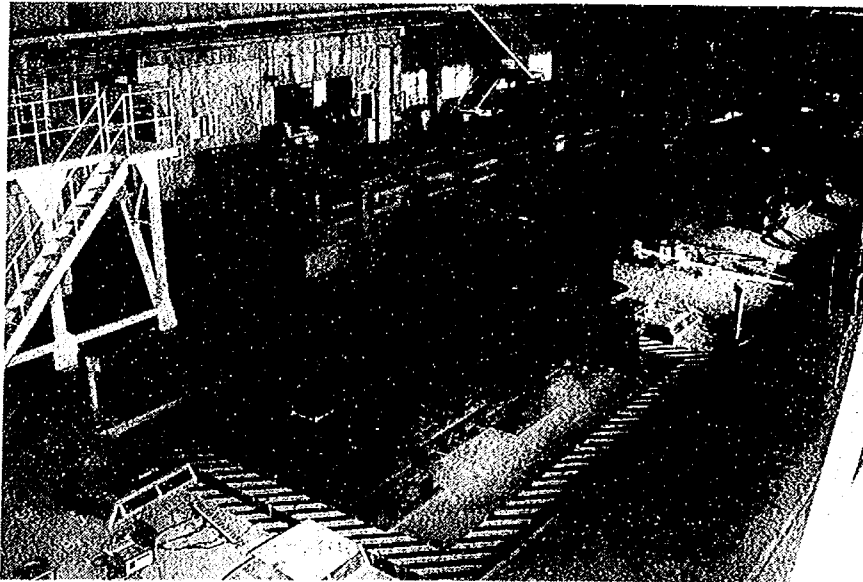


Photo 3 Shaking Table Tests for Studying Structural Response of Menshin Bridges

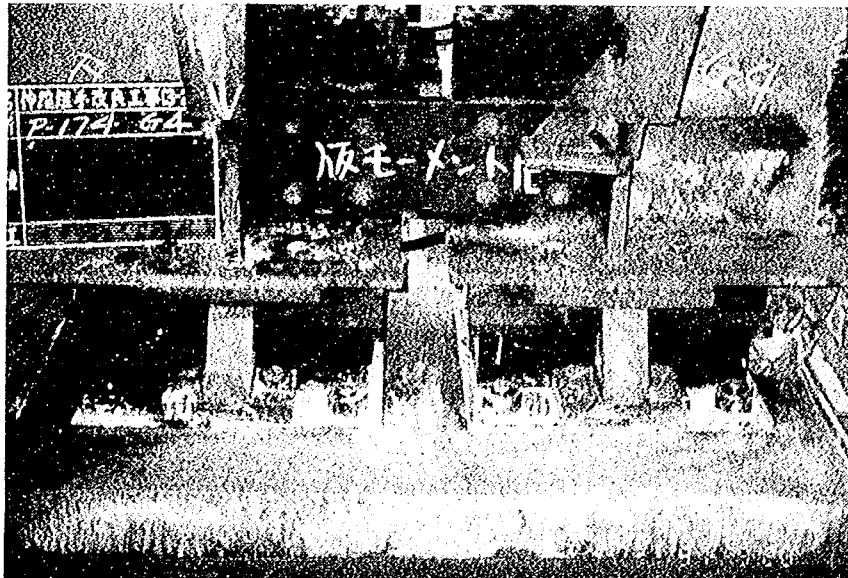
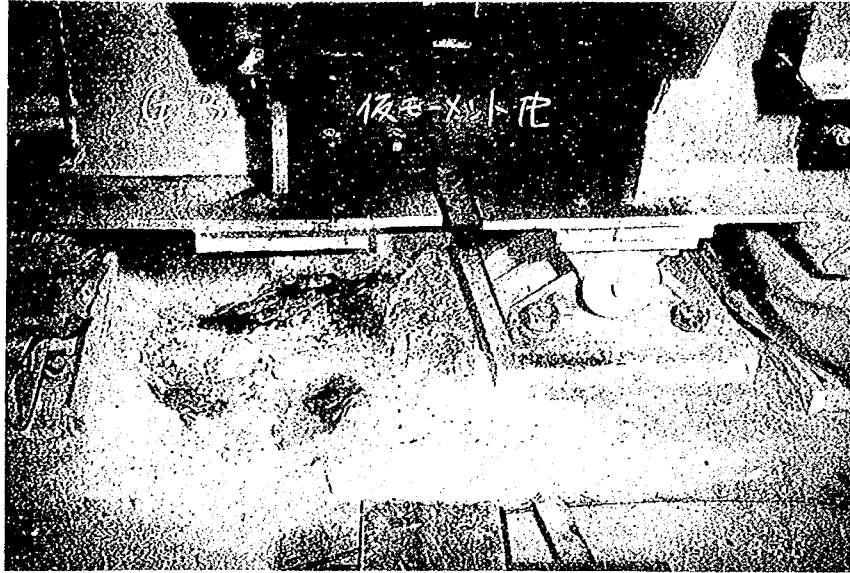


Photo 4 Deck Connection of Existing Bridge

Guidelines for Seismic Design Methods
of Large Underground Structures

Hajime ASAKURA
Kazuhiko KAWASHIMA
and
Hideki SUGITA

24th Joint Meeting of The U.S. -Japan Cooperative Program
in National Resources, Panel on Wind and Seismic Effects

Gaithersburg, MD, U. S. A.

May 1992

GUIDELINES FOR SEISMIC DESIGN METHODS OF LARGE UNDERGROUND STRUCTURES

Hajime ASAKURA¹⁾, Kazuhiko KAWASHIMA²⁾ and Hideki SUGITA³⁾

ABSTRACT

Outline of the Guidelines for Seismic Design Methods of Large Underground Structures is presented. This is one of the final accomplishments of the 5-year Ministerial Research Project on Development of Utilization of Underground Space which was made between 1987 and 1992. Description is given to seismic design methods proposed for the Laterally Long Underground Structures, Laterally Wide Underground Structures and Vertically Long Underground Structures.

KEY WORDS

Underground Structure, Seismic Design, Seismic Deformation Method, Shield Tunnel, Disaster Prevention Systems

1 INTRODUCTION

Utilization of underground space is becoming significant importance in urban area in Japan because of various restrictions for getting open space for new construction. The Ministry of Construction initiated a 5-year Research Project on "Development of Utilization Technic of Underground Space" in 1987. The Public Works Research Institute, Building Research Institute and Geographical Survey Institute worked together in the Research Projects. One of the main research targets of the Project was to develop rational seismic design methods of large underground structures embedded in soft ground.

For executing the Research Projects, joint researches were made between the Public Works Research Institute and Advanced Construction Technology Center. As one of such joint researches, the Joint Research on Development of Seismic Design Method of Underground Structures was conducted between the Earthquake Engineering Division of the Public Works Research Institute, Advanced Construction Technology Center and 8 private firms (Fujita, Kajima, Taisei, Tobishima, Hazama, Okumura, Kounolke and Shimizu).

Including the research results obtained through the joint research¹⁾ with Advanced Construction Technology Center, the final accomplishments of the 5-year Research Project was compiled in March 1992. The results of

investigations on the seismic design methods of underground structures were compiled in the form of "Guidelines for Seismic Design Methods of Underground Structures"²⁾. Because the seismic design methods for the underground structures with small diameter such as the petroleum pipes³⁾ and water pipes⁴⁾, and common utility ducts⁵⁾ were already in use, the Guidelines show the seismic design methods for larger underground structures than those facilities.

This paper outlines the Guidelines.

2 SCOPES AND CONTENTS OF THE GUIDELINES

The Guidelines present the seismic design methods for large underground structures constructed in soft ground in city area. As shown in Fig. 1, underground structures may be classified, based on their structural response characteristics, into three groups as

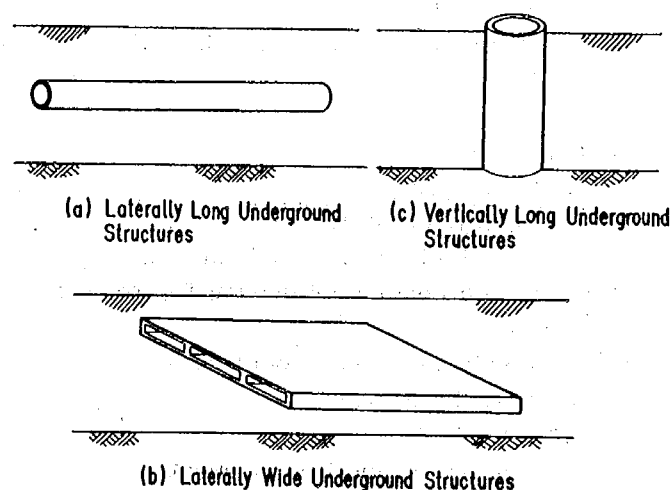


Fig.1 Classification of Underground Structures

1) Laterally Long Underground Structures

Long underground structures embedded laterally along ground surface. Seismic force along their axes is the major effect of an earthquake for such structures.

2) Laterally Wide Underground Structures

Widely spread underground structures along

1) Director, 1st Research Department, Advanced Construction Technology Center

2) Head, Earthquake Engineering Division, Public Works Research Institute

3) Research Engineer, ditto

the ground surface. Shear stress along the surface of structures associated with shear deformation of ground due to an earthquake is the major seismic effect.

3) Vertically Long Underground Structures

Long underground structure in vertical direction. Shear stress along the surface of structures associated with shear deformation of ground due to an earthquake is the major seismic effect.

The seismic safety required for design depends on the type, utilization and importance of the structures. Therefore, seismic design methods which are required for underground structures are presented for the three types in general sense in the Guidelines.

Underground highways, underground parking space and common utility ducts were imaged for developing the Guidelines.

It should be noted that although the description is given in the format of the specifications, it is not the mandate specifications. It merely describes the research results.

The table of contents of the Guidelines is as follows:

- 1 General
 - 1.1 Scopes
 - 1.2 Definition of Terms
- 2 Basic Principle of Seismic Design
- 3 Loads and Loads Combination for Seismic Design
 - 3.1 Loads and Loads Combination
 - 3.2 Seismic Effects
 - 3.3 Inertia Force
 - 3.4 Design Ground Displacement
 - 3.5 Dynamic Earth Pressure
 - 3.6 Design Shear Stress around Surface
 - 3.7 Classification of Soil Condition
 - 3.8 Input Ground Motion for Dynamic Response Analysis
 - 3.9 Soil Spring Stiffness
- 4 Seismic Design Method of Shield Tunnels
 - 4.1 General
 - 4.2 Seismic Design Method in Axial Direction
 - 4.2.1 Basic Principle of Seismic Design
 - 4.2.2 Seismic Design by Seismic Deformation Method
 - 4.2.3 Equivalent Stiffness of Shield Tunnel without Second Lining
 - 4.2.4 Equivalent Stiffness of Shield Tunnel with Second Lining

- 4.2.5 Evaluation of Seismic Safety
- 4.2.6 Dynamic Response Analysis
- 4.2.7 Minimum Reinforcement of Second Lining in Axial Direction

4.3 Seismic Design Method in Transverse Direction

- 4.3.1 Basic Principle of Seismic Design
- 4.3.2 Seismic Design by Seismic Deformation Method
- 4.3.3 Dynamic Response Analysis
- 4.3.4 Evaluation of Seismic Safety

5 Seismic Design Method of Laterally Wide Underground Structures

- 5.1 General
- 5.2 Seismic Design by Seismic Deformation Method
- 5.3 Dynamic Response Analysis

6 Seismic Design of Vertically Long Underground Structures

- 6.1 Basic Principle of Seismic Design
- 6.2 Seismic Design by Seismic Deformation Method
- 6.3 Dynamic Response Analysis
- 6.4 Evaluation of Seismic Safety

7 Structures Expecting Reduction of Seismic Effects

- 7.1 General
- 7.2 Flexible Joints
- 7.3 Structures Reducing Stiffness of Ring Joint of Shield Tunnel
- 7.4 Isolation of Underground Structures from Surround Subsoils

8 Earthquake Disaster Prevention Systems for Underground Highways

- 8.1 General
- 8.2 Fear and Anxiety of Drivers against an Earthquake
- 8.3 Factors Causing Fear and Anxiety to Drivers at Underground Highways
- 8.4 Basic Countermeasure against Earthquake Disaster

3 BASIC PRINCIPLE OF SEISMIC DESIGN

Seismic design of underground structures is recommended to be made by the Seismic Deformation Method⁶⁾. In the Seismic Deformation Method, the displacement and deformation developed in the subsurface ground during an earthquake are taken as the major seismic effects to underground structures. The seismic response of underground structures is distinguished by the small mass effect and the large damping ratio from regular structures on ground, i.e.,

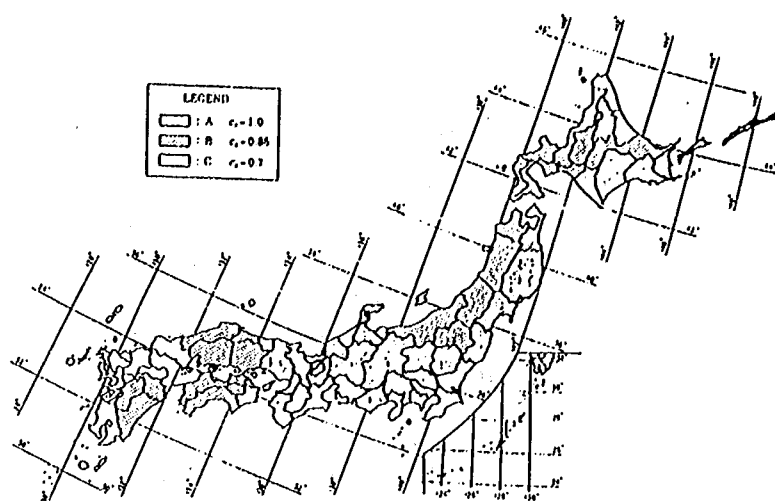


Fig.2 Seismic Zoning Map and Modification Coefficient c_s

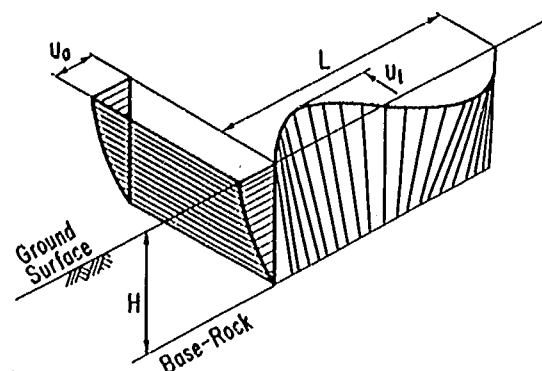


Fig.3 Deformation of Subsurface Ground Assumed in Seismic Deformation Method

a) mass effect is small because unit weight of underground structures is in most cases smaller than that of subsurface ground, and

b) damping is very large because radiational damping due to energy dissipation from underground structures to subsols is very large.

Therefore, it is assumed in the Seismic Deformation method that during an earthquake underground structures deform following the deformation of subsurface ground. Modelling of underground structures has to be made so that structural response of the underground structures be correctly represented. Because the increase of stress and deformation of underground structures due to seismic effects are evaluated by the Seismic Deformation Method, the stress and deformation actually developed during an earthquake have to be computed by adding them with those developed due to the static loads such as the dead weight.

Evaluation of seismic safety of underground structures can be made by either the Allowable Stress Design Method or the Bearing Capacity Design Method. In the Allowable Stress Design Method the stress and the displacement developed in structural components are required to be less than the allowable stress and allowable displacement. In the Bearing Capacity Design Method the sectional forces developed in structural components are required smaller than those of dynamic bearing capacity of structural systems considering ductility. Displacement of structures are also required to be smaller than the critical displacement.

Table 1 Classification of Ground Condition

GROUND CONDITION	DEFINITION	APPROXIMATE ESTIMATION
GROUP I	$T_0 < 0.2 \text{ SEC}$	TERTIARY OR OLDER
GROUP II	$0.2 \leq T_0 < 0.6 \text{ SEC}$	ALLUVIUM AND DILUVIUM
GROUP III	$0.6 \text{ SEC} \leq T_0$	SOFT ALLUVIUM

After the design by means of the Seismic Deformation Analysis, dynamic response analysis is recommended for the underground structures with complex structural response.

4 LOADS CONSIDERED FOR SEISMIC DESIGN

In seismic design of underground structures, the dead weight, the earth pressure, the water pressure, the uplift force, the ground settlement and the seismic effects are to be considered. The loads have to be combined so that the most critical stress or displacement be developed. As the seismic effects, the following loads have to be considered:

(1) Inertia Force

The lateral force coefficient has to be evaluated as

$$k_h = c_z \cdot c_a \cdot c_u \cdot k_{h0} \quad (1)$$

where,

k_h : lateral force coefficient

c_z : modification factor for zone (refer to Fig. 2)

c_a : modification factor for ground condition (refer to Table 1)

c_u : modification factor for depth, and is evaluated as

$$c_u = 1.0 - 0.015z \quad (2)$$

z : depth from ground surface (m)

k_{h0} : standard lateral force coefficient

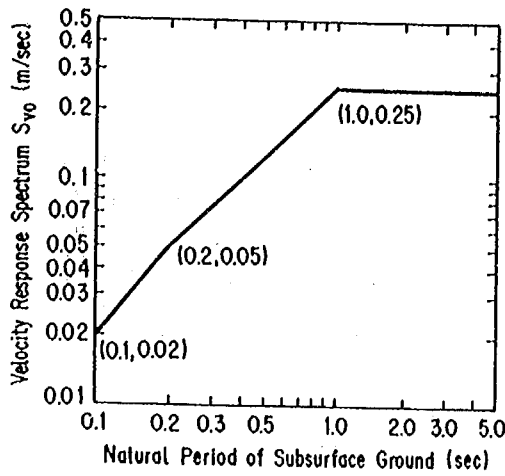


Fig.4 Standard Velocity Response Spectrum (Allowable Stress Design Method)

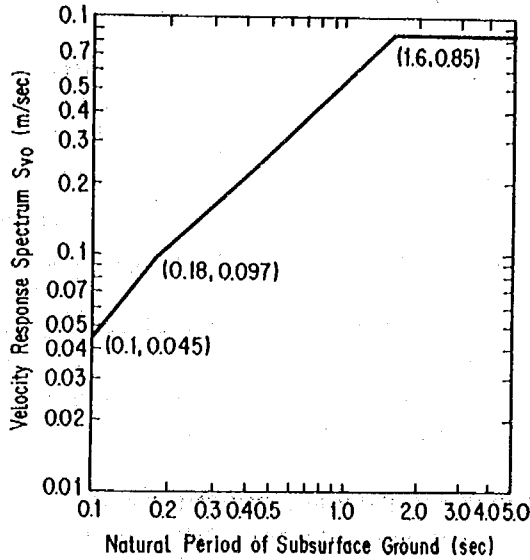


Fig.5 Standard Velocity Response Spectrum (Bearing Capacity Design Method) ($\alpha=0.2$)

(2) Design Ground Displacement

The design ground displacement is evaluated depending on the depth and the distance as (refer to Fig. 3) :

$$u(z, x) = \frac{1}{\pi} S_v \cdot T_s \cdot \cos\left(\frac{\pi z}{2H}\right) \cdot \sin\left(\frac{\pi x}{2L}\right) \quad (3)$$

where,

$u(z, x)$: Design ground displacement (m) at depth z (m) and at distance x (m)

S_v : Design velocity response spectrum at design base rock (cm/sec), and is determined as

$$S_v = c_z \cdot S_{v0} \quad (4)$$

c_z : Modification factor for zone (refer to Fig. 2)

S_{v0} : Standard velocity response spectrum (cm/sec) at design base rock, and is given by Fig. 4 for the Allowable Stress Design Method and by Fig. 5 for the Bearing Capacity Design Method

T_s : Natural period of subsurface ground

(sec), and is evaluated as

$$T_s = 1.25 \cdot T_0 \quad (5)$$

$$T_0 = 4 \sum \frac{H_i}{V_{s1}} \quad (6)$$

H_i : Thickness of i -th soil layer (m)

V_{s1} : Shear wave velocity of i -th soil layer (m/sec)

L : Design wave length (m) of ground motion

$$L = \frac{2L_1 L_2}{L_1 + L_2} \quad (7)$$

L_1, L_2 : Design wave length (m) at subsurface ground and base rock, respectively, and are determined as

$$L_1 = T_s \cdot V_{SD} \quad (8)$$

$$L_2 = T_s \cdot V_{SDB} \quad (9)$$

V_{SD} : Averaged shear wave velocity of subsurface ground (m/sec)

V_{SDB} : Shear wave velocity at design base rock (m/sec)

(3) Design Shear Stress around Surface

At the surface of underground structures contacting with soils, the design shear stress around surface has to be applied to the underground structures as shown in Fig. 6.

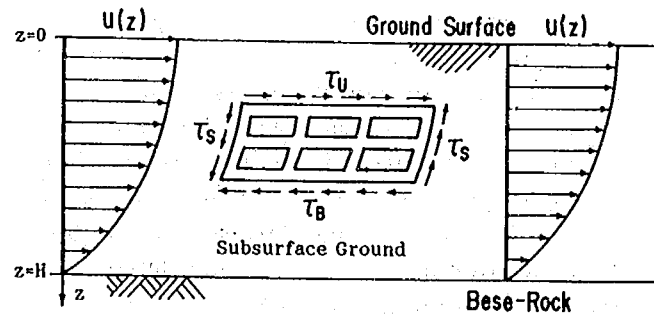


Fig.6 Design Shear Stress around Surface

The design shear stress around surface is given by Eq. (10). If the design shear stress around surface by Eq. (10) is larger than the strength of soils around the underground structure, the design shear stress around surface shall be the strength of soils.

$$\tau = \frac{G_D}{\pi H} S_v \cdot T_s \cdot \sin\left(\frac{\pi z}{2H}\right) \quad (10)$$

where,

τ : Design shear stress around surface (tf/m²)

S_v : Design velocity response spectrum at design base rock (cm/sec), and is given by Eq. (4)

G_D : Shear modulus of subsurface ground (tf/m²)

T_s : Natural period of subsurface ground

(sec), and is computed by Eq. (5)
 H : Thickness of subsurface ground (m)

(4) Dynamic Earth Pressure

Dynamic earth pressure by Eq. (11) is applied to side wall of underground structures as shown in Fig. 7.

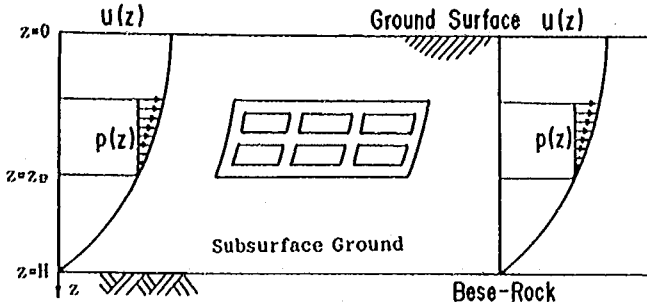


Fig.7 Dynamic Earth Pressure

$$p(z) = k_H \cdot \{u(z) - u(z_B)\} \quad (11)$$

where,

$p(z)$: dynamic earth pressure per unit area (tf/m^2) at depth z (m)
 k_H : spring stiffness of soils per unit area (tf/m^2)
 $u(z)$: Design ground displacement (m), and is given by Eq. (3)
 z_B : Depth of base of underground structure (m)

(5) Input Ground Motions for Dynamic Response Analysis

Design acceleration response spectrum at design base rock is given as

$$S = C_Z \cdot C_D \cdot S_0 \quad (12)$$

where,

S : Design acceleration response spectrum (cm/sec^2) at design base rock
 C_Z : Modification factor for zone (refer to Fig. 2)
 C_D : Modification factor of damping ratio, and is given as

$$C_D = \frac{1.5}{40h_i + 1} + 0.5 \quad (13)$$

h_i : modal damping ratio of critical
 S_0 : Standard acceleration response spectrum at design base rock, and is given by Fig. 8 for the Allowable Stress Design Method and by Fig. 9 for the Bearing Capacity Design Method

Input accelerations for time response analysis shall be either selected from strong motion records which have the close response spectral characteristics with S by Eq. (12) or

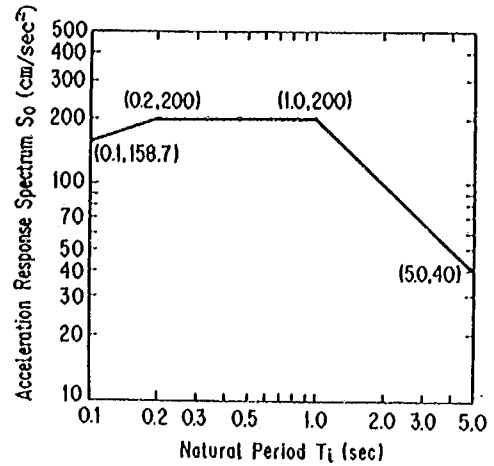


Fig.8 Standard Acceleration Response Spectrum (Allowable Stress Design Method)

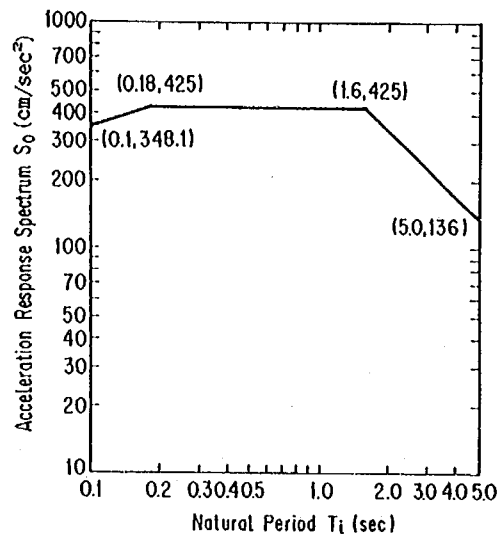


Fig.9 Standard Acceleration Response Spectrum (Bearing Capacity Design Method)

obtained by adjusting the strong motion records in frequency domain so that the acceleration response spectrum be matched with S by Eq. (12).

5 SEISMIC DESIGN METHOD OF LARGE SHIELD TUNNELS

5-1 Seismic Design Method in Axial Direction

(1) Seismic Design by Seismic Deformation Method

Shield tunnels are constructed by assembling a number of segments, ring bolts and segment bolts as shown in Fig. 10. Although there may be various idealizations of shield tunnels as shown in Fig. 11, the idealization of shield tunnels as a beam with equivalent stiffness may be the most appropriate for seismic design. Assuming a shield tunnel as a beam, elastically supported by subsurface ground, with equivalent stiffness as shown in Fig. 12, the seismic forces developed in the shield tunnel are evaluated

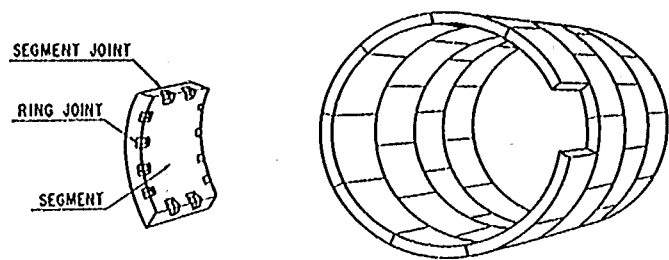


Fig. 10 Segments and Joints of Shield Tunnel Lining

by the Seismic Deformation Method as²⁷ :

$$P_h^C = \alpha_c^1 \frac{\pi u_h}{L} (EA)_{eq}^C \quad (14)$$

$$P_h^T = \alpha_t^1 \frac{\pi u_h}{L} (EA)_{eq}^T \quad (15)$$

$$P_v^C = \alpha_c^1 \frac{\pi (u_h + u_v)}{L} (EA)_{eq}^C \quad (16)$$

$$P_v^T = \alpha_t^1 \frac{\pi (u_h + u_v)}{L} (EA)_{eq}^T \quad (17)$$

$$M_h = \alpha_z^1 \frac{4 \pi u_h}{L^2} (EI)_{eq} \quad (18)$$

$$M_v = \alpha_z^1 \frac{4 \pi u_v}{L^2} (EI)_{eq} \quad (19)$$

$$Q_h = \alpha_z^1 \frac{8 \pi u_h}{L^3} (EI)_{eq} \quad (20)$$

$$Q_v = \alpha_z^1 \frac{8 \pi u_v}{L^3} (EI)_{eq} \quad (21)$$

where,

P_h^C, P_h^T : Compression and tension force (tf), respectively, developed due to the design ground displacement in

lateral direction

P_v^C, P_v^T : Compression and tension force (tf), respectively, developed due to the design ground displacement in vertical direction

M_h, M_v : Bending moment in horizontal plane and vertical plane (tf·m), respectively

Q_h, Q_v : Shearing force in horizontal plane and vertical plane (tf), respectively

$(EA)_{eq}^C, (EA)_{eq}^T$: Equivalent compression rigidity (tf) and equivalent tension rigidity (tf) of shield tunnel, and is given by Eqs.(31) and (32)

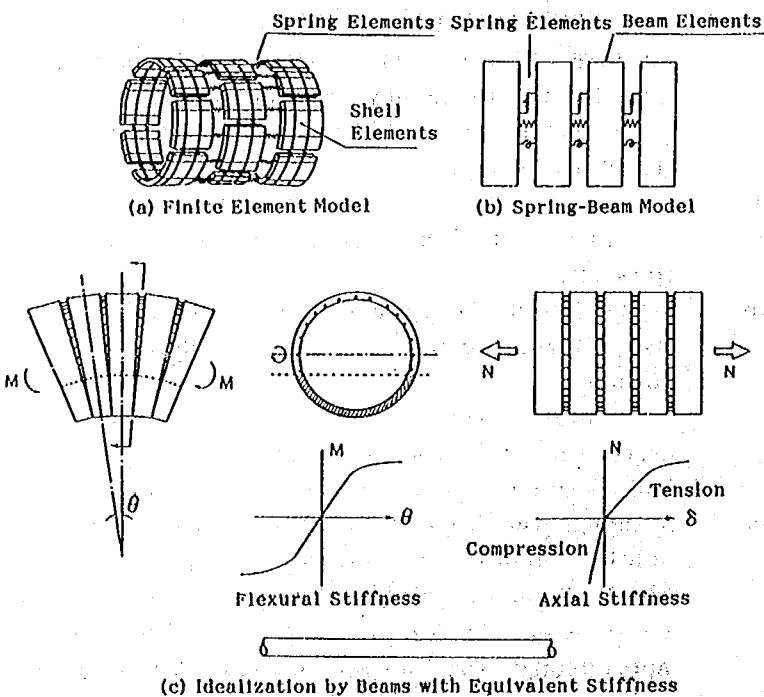
$(EI)_{eq}$: Equivalent flexural rigidity (tf), and is given by Eq. (35)

u_h : Design lateral ground displacement (m) at depth of gravity center of shield tunnel, and is given by Eq. (3)

u_v : Design vertical ground displacement (m) at depth of gravity center of shield tunnel, and is given as 1/2 of u_h

L : Design wave length (m), and is give by Eq. (7)

$\alpha_c^1, \alpha_t^1, \alpha_z^1, \alpha_z^2$: Coefficients to represent the degree of transmission of response displacement from the subsurface ground to shield tunnel, and is given by Eqs. (22) ~ (25).



(c) Idealization by Beams with Equivalent Stiffness

Fig. 11 Idealization of Shield Tunnel

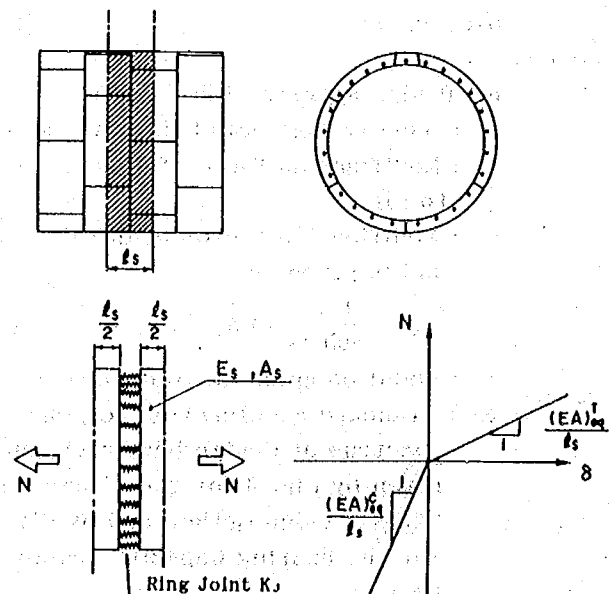


Fig. 13 Axial Stiffness of Segments-Ring Joints

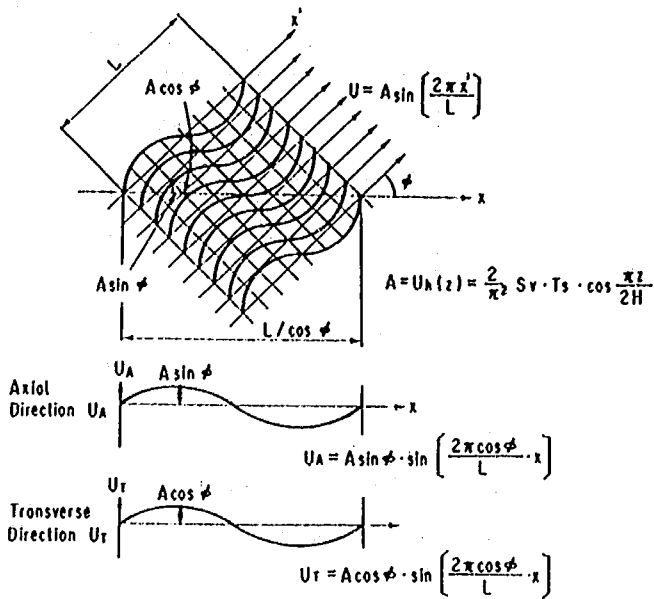


Fig. 12 Ground Displacement Assumed in Seismic Deformation Method

$$\alpha_i = \frac{1}{1 + (2\pi / \lambda_i^2 L_i)^2} \quad (22)$$

$$\alpha_T = \frac{1}{1 + (2\pi / \lambda_T^2 L_i)^2} \quad (23)$$

$$\alpha_z = \frac{1}{1 + (2\pi / \lambda_z^2 L)^4} \quad (24)$$

$$\alpha_{\theta} = \frac{1}{1 + (2\pi / \lambda_{\theta}^2 L)^4} \quad (25)$$

where,

$$\lambda_i = \sqrt{\frac{K_{qi}}{(EA)_{\theta q}}} \quad (26)$$

$$\lambda_T = \sqrt{\frac{K_{qt}}{(EA)_{Tq}}} \quad (27)$$

$$\lambda_z = \sqrt[4]{\frac{K_{qz}}{(EI)_{\theta q}}} \quad (28)$$

$$\lambda_{\theta} = \sqrt[4]{\frac{K_{q\theta}}{(EI)_{\theta q}}} \quad (29)$$

$$L_i = \sqrt{2} \cdot L \quad (30)$$

K_{qi} , K_{qz} , $K_{q\theta}$: soil spring in

longitudinal, transverse and vertical direction, respectively (tf/m²)

(2) Equivalent Stiffness of Shield Tunnel without Second Lining

1) The equivalent stiffness of shield tunnel is different between tension and compression as shown in Fig. 13. The equivalent rigidity is computed by Eqs. (31) and (32).

$$(EA)_{\theta q}^c = E_s A_s \quad (31)$$

$$(EA)_{\theta q}^T = \frac{1}{(K_s/K_J) + 1} \cdot E_s A_s \quad (32)$$

where,

$(EA)_{\theta q}^c$: Equivalent compression rigidity (tf)

$(EA)_{\theta q}^T$: Equivalent tension rigidity (tf)

E_s : Elastic modulus of shield segments (tf/m²)

A_s : Sectional area of segment-ring (m²)

l_s : Length of segment (m) in axial direction

K_s : Stiffness of segment-ring (tf/m), and is given by Eq. (33)

$$K_s = \frac{E_s A_s}{l_s} \quad (33)$$

K_J : Tension stiffness of total ring-joints (tf/m), and is given by Eq. (34)

$$K_J = n \cdot k_J \quad (34)$$

k_J : Tension stiffness of a ring-joint (tf/m)

n : number of ring-joint

2) The equivalent flexural rigidity of shield tunnel is computed as shown in Fig. 14, and is given by Eq. (35)²⁹

$$(EI)_{\theta q} = \frac{\cos^3 \phi}{\cos \phi + (\pi/2 + \phi) \sin \phi} \cdot E_s \cdot I_s \quad (35)$$

where,

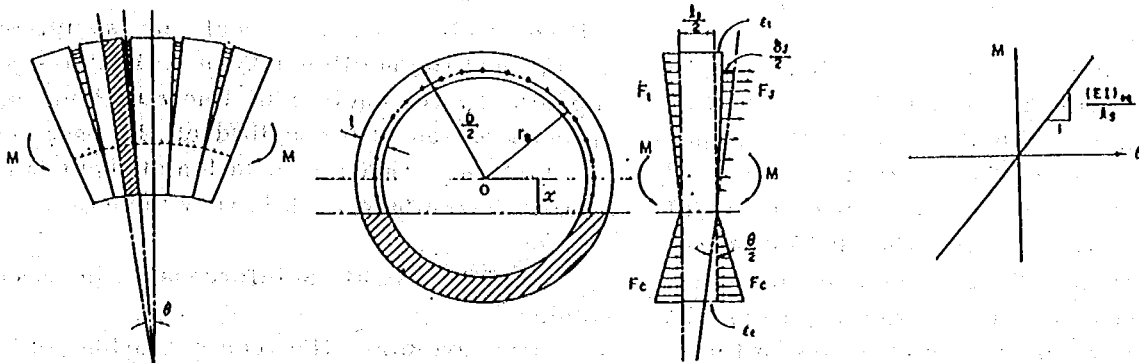
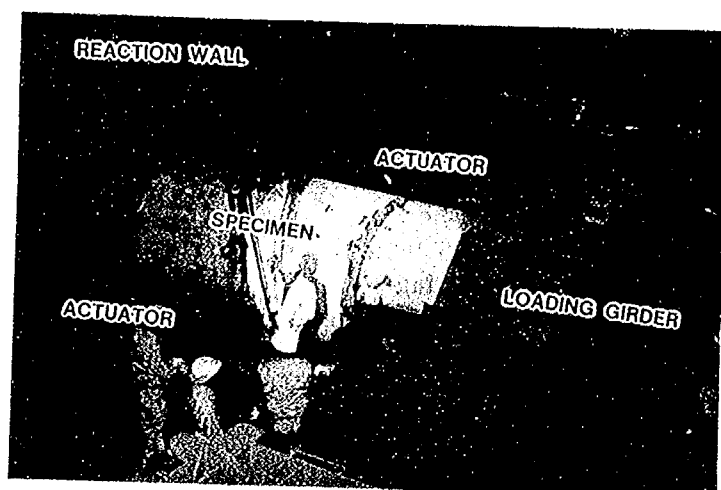
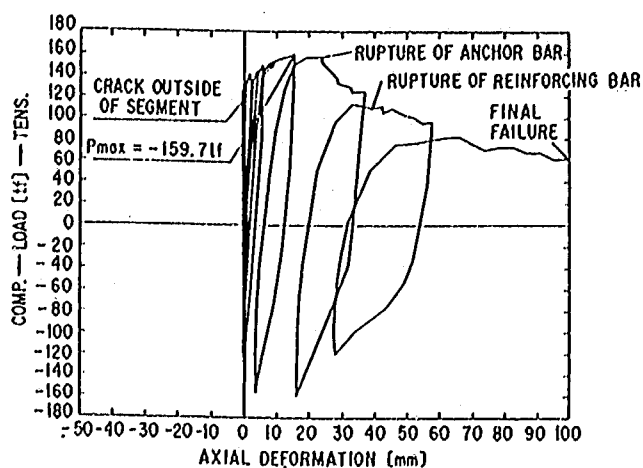


Fig. 14 Flexural Stiffness of Shield Tunnel



(a) Experimental Set-up



(b) Loading Hysteresis Loop

Fig.15 Effect of Second Lining for Ductility of Flexural Deformation

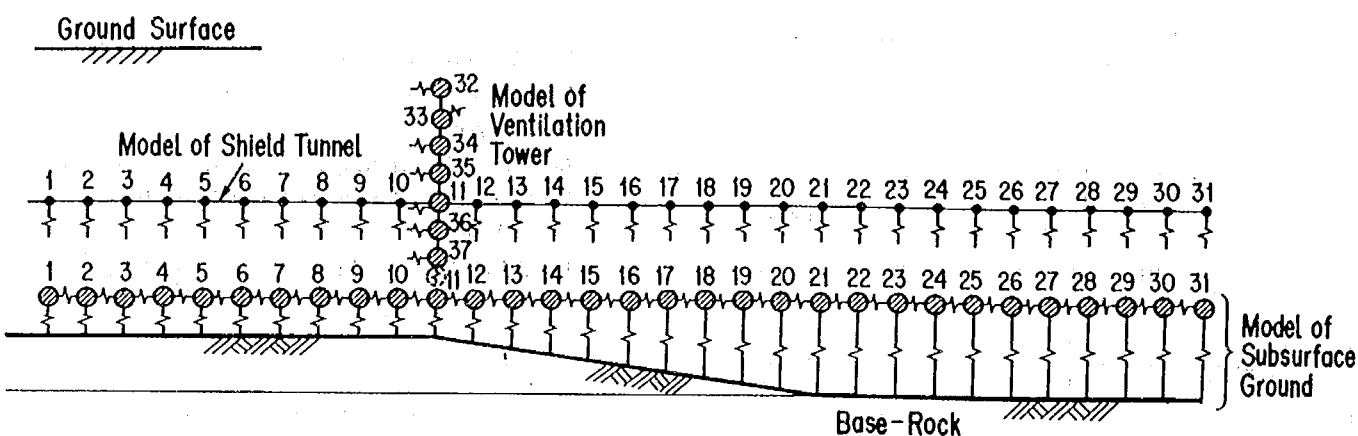


Fig.16 Model of Shield Tunnel, Ventilation Tower and Subsurface ground

$(EI)_{eq}$: Equivalent flexural rigidity of shield tunnel (tfm²)

E_s : Elastic modulus of segment (tf/m²)

I_s : Moment of Inertia of segment (m⁴)

ϕ : angle measured from center line to neutral line (rad), and is given by solving the following equation :

$$\phi + \cot \phi = \pi \left(\frac{K_1}{2 E_s A_s / I_s} \right) \quad (36)$$

(3) Equivalent Stiffness of Shield Tunnel with Second Lining

General description for evaluating the equivalent stiffness of shield tunnel with second lining is difficult because it depends on the spring action of axial reinforcements of the second lining. Based on a series of model loading test results at the Public Works Research Institute⁽²⁰⁾⁽²¹⁾ as shown in Fig. 15, it is known that as well as the segments the second lining resists to the compression force. However, to the tension force the interaction

between the segments and the second lining depends significantly on the damage developed in the second lining and at the ring-joints. It may be classified into four steps as shown in Table 2.

(4) Evaluation of Seismic Safety by Dynamic Response Analysis

In the dynamic response analysis, shield tunnel is generally idealized by a elastic beam with the equivalent stiffness as shown in Fig. 16. The subsurface ground is idealized by either finite element model or spring-mass model, and the nonlinearity of soil stiffness is modeled by the equivalent linear method. Input ground motion is prescribed at the base-rock. Evaluation on spatial variation of input motion at the design base-rock is very important.

(5) Minimum Axial Reinforcement in Second Lining

For avoiding the concentration of the damage at only selected ring-joints as shown in

SECOND LINING

Table 2 Damage Degree of Shield Tunnel with Second Lining

Step	Damage Degree
I	All sections in 1st and 2nd linings are in elastic. No damage.
II	Cracks in 2nd lining around ring only at ring joints.
III	Cracks in 2nd lining around ring occur not only at ring joint but at other sections.
IV	Contact between the 1st lining and 2nd lining is lost. Final failure is to be developed.

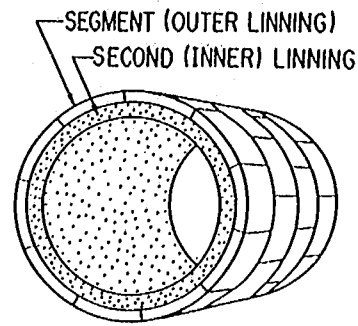


Fig. 17, It is required to provide the minimum reinforcement in the second lining in tunnel axis as

$$A_s = (A_L \cdot \sigma_L - P_R) / \sigma_{LS} \quad (37)$$

where,

A_s : Minimum reinforcement required for second lining in longitudinal direction (cm^2)

A_L : Sectional area of second lining (cm^2)

σ_L : Tension strength of second lining (kgf/cm^2)

σ_{LS} : Tension strength of reinforcement in second lining (kgf/cm^2)

P_R : Tension strength of ring joint (kgf), and takes the smaller value between P_A by Eq.(38) and P_E by Eq.(39).

$$P_A = n \cdot p_A \quad (38)$$

$$P_E = m \cdot p_E \quad (39)$$

where,

P_A : Tension strength of anchor reinforcement (kgf)

P_E : Tension strength of ring bolts (kgf)

n : Number of anchor reinforcements per ring joint

m : Number of ring bolts per ring joint

p_A : Tension strength of an anchor bolt (kgf)

p_E : Tension strength of a ring bolt (kgf)

5-2 Seismic Design Method in Transverse Direction

The seismic design of shield tunnel in transverse direction is generally made as :

- 1) Idealize the shield tunnel as a ring with equivalent stiffness,
- 2) Consider soil springs, around the tunnel ring, which represent the soil-structure interaction,
- 3) Evaluate the design ground displacement,

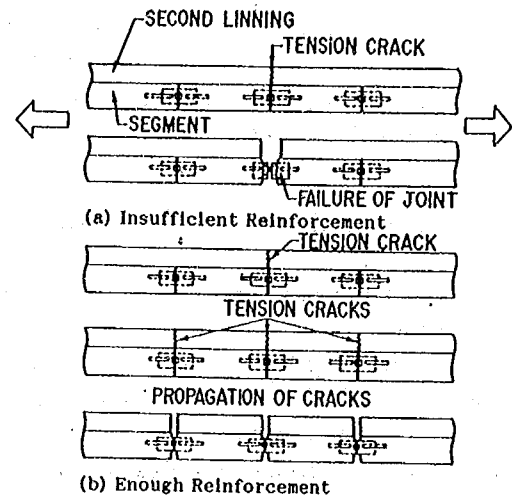


Fig.17 Minimum Reinforcement Required in Second Lining for Preventing Concentration of Damage at only Selected Ring Joints

the design shear stress around surface and the inertia force at the center of gravity of the shield tunnel,

4) Divide the design shear stress around surface and the inertia force into the components in tangential and normal directions, and apply them to the shield tunnel. The design ground displacement is also divided into the components in tangential and normal directions, and apply them to the shield tunnel through the soil springs,

5) Compute the increase of force, due to seismic effects, developed in the tunnel rings, and

6) Evaluate the force, developed in the tunnel rings, during an earthquake by adding the force of 4) with the force developed due to static loads

When the following assumptions can be made for the shield tunnel and subsurface ground,

Elastic and Homogeneous

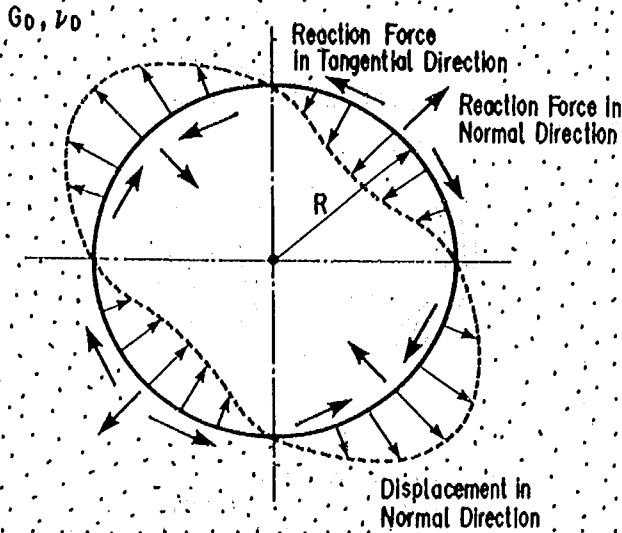


Fig.18 2nd Model Reformation of Fourier Series assumed for Tunnel Ring

the force increase, due to the seismic effects, developed in the tunnel ring can be easily computed.

- 1) The subsurface ground is elastic and homogeneous,
- 2) The fundamental mode is significantly predominant in the subsurface ground,
- 3) Tunnel ring is elastic,
- 4) The inertia force to the tunnel ring is negligibly small,
- 5) Depth of soil to the shield tunnel is large enough, and the shield tunnel is not in close proximity of the design base-rock,
- 6) Saturation of the design shear stress around surface at the shear strength of soils is disregarded, and
- 7) The soil spring can be evaluated based on force vs. displacement relation of an elastic ring placed in infinite elastic homogeneous material as shown in Fig. 18.

The assumption 7) require more explanation. Consider an elastic ring which is placed in the infinite elastic material. From the displacement of the ring by applying a distributed load, the soil spring may be evaluated by Eqs. (40) and (41).

$$k_n = \frac{2G_D}{R} \cdot C_n \quad (40)$$

$$k_n' = \frac{2G_D}{R} \cdot C_n' \quad (41)$$

$$C_n = \begin{cases} \frac{1}{2} & : n=0 \\ \frac{2n+1-2\nu_D(n+1)}{3-4\nu_D} & : n \geq 1 \end{cases} \quad (42)$$

$$C_n' = \begin{cases} \frac{n+2-2\nu_D(n+1)}{3-4\nu_D} & : n \geq 2 \end{cases} \quad (43)$$

where,

k_n : Spring stiffness associated with the displacement in normal (or tangential) direction due to the force in normal (or tangential) direction(tf/m³)

k_n' : Spring stiffness associated with the displacement in normal (or tangential) direction due to the force in tangential (or normal) direction(tf/m³)

G_D : Elastic modulus of subsurface soils(tf/m²)

R : Radius of shield tunnel ring(m)

ν_D : Poisson's ratio of subsurface ground

n : mode number in terms of Fourier series

When the soil spring can be represented by Eqs. (40) and (41), the increase of force, developed in shield tunnel ring, due to the seismic effects may be represented as

$$M = \beta \cdot \frac{3\pi \cdot EI}{2R \cdot H_D} \cdot U_n \cdot \sin\left(\frac{\pi \cdot H}{2H_D}\right) \cdot C \cdot \sin 2\theta \quad (45)$$

$$Q = -\beta \cdot \frac{3\pi \cdot EI}{R^2 \cdot H_D} \cdot U_n \cdot \sin\left(\frac{\pi \cdot H}{2H_D}\right) \cdot C \cdot \cos 2\theta \quad (46)$$

$$N = -\beta \cdot \frac{3\pi \cdot EI}{R^2 \cdot H_D} \cdot U_n \cdot \sin\left(\frac{\pi \cdot H}{2H_D}\right) \cdot \left(1 + \frac{G_D \cdot R^3}{6EI}\right) \cdot C \cdot \sin 2\theta \quad (47)$$

$$C = \frac{4(1-\nu_D) \cdot G_D \cdot R^3}{(3-2\nu_D) \cdot G_D \cdot R^3 + 6(3-4\nu_D) \cdot EI} \quad (48)$$

where,

M : Bending moment of tunnel ring per unit length(tf·m/m)

Q : Shear force developed in tunnel ring per unit length(tf/m)

N : Axial force developed in tunnel ring per unit length(tf/m)

θ : Angle measured from crest of tunnel ring(rad)

H_D : Thickness of subsurface soils(m)

H : Depth of the gravity center of shield tunnel from the ground surface(m)

R : Radius of tunnel ring(m)

EI : Flexural rigidity of tunnel ring per unit length(tf·m²/m)

G_D : Shear modulus of subsurface ground(tf/m²)

ν_D : Poisson's ratio of subsurface ground

U_n : Design ground displacement at the ground surface, and is given by Eq. (3)

β : Modification factor for higher order modes (= 1.3)

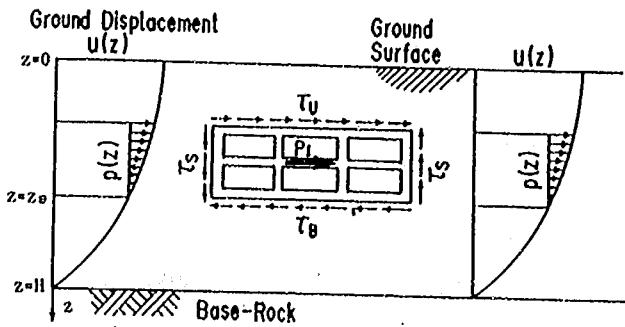


Fig.19 Shear Stress around Surface, Dynamic Earth Pressure and Inertia Force for Seismic Design of Laterally Wide Underground Structures

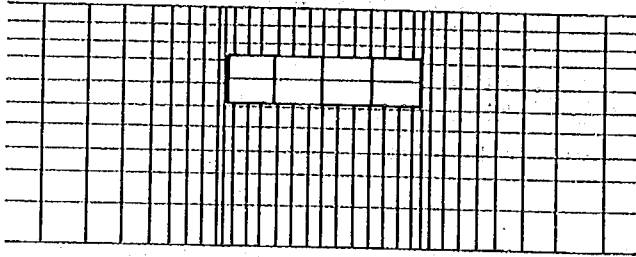


Fig.20 Two Dimensional Finite Element Idealization

6 SEISMIC DESIGN METHOD OF LATERALLY WIDE UNDERGROUND STRUCTURES

The shear stress around surface is significant in seismic design of the Laterally Wide Underground Structures. The increase of force, developed in the structures, due to seismic effects are computed by applying the design shear stress around surface, the dynamic earth pressure and the inertia force as shown in Fig. 19.

Dynamic response analysis is recommended for the Laterally Wide Underground Structures with the following conditions:

- 1) For the underground structures constructed in subsurface ground with significantly irregular soil condition in either vertical or lateral direction. The condition where irregularity of the surface or the design base-rock is significant is also the one recommended for the dynamic response analysis. Two dimensional finite element idealization as shown in Fig. 20 is recommended for such analyses.
- 2) For the underground structures with significant out-of-plane deformation. Analytical models as shown in Fig. 21 is recommended for such structures. In the models, the subsurface ground is recommended to be idealized by the mass-spring model with one or two layers in vertical direction and the Laterally Wide Underground Structure is idealized by plate elements, shell and solid elements.

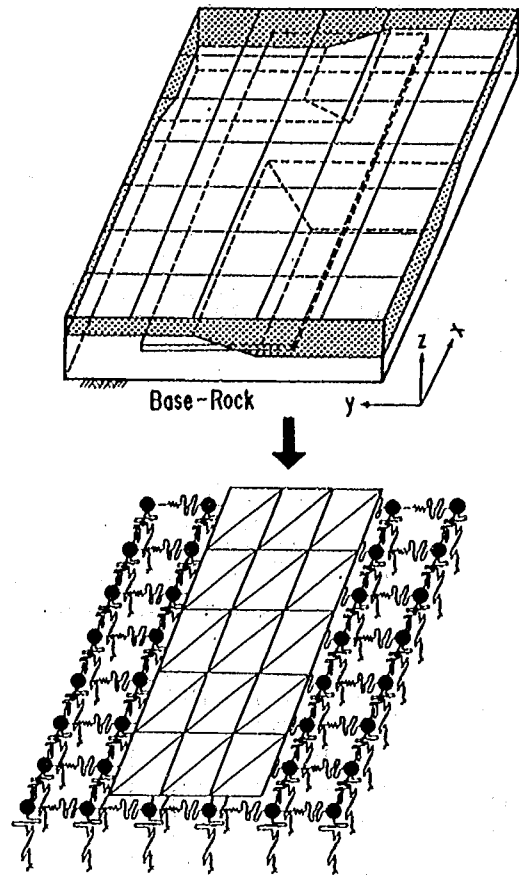


Fig.21 Idealization by Plate Elements and Shell Elements

7 SEISMIC DESIGN METHOD OF VERTICALLY LONG UNDERGROUND STRUCTURES

Because the design shear stress around surface is predominant in the Vertically Long Underground Structures, it is important to evaluate it carefully. Fig. 22 represents how the seismic forces are applied to the structures. There are structures with large section, which shows significant deformation in lateral direction as well. Shell elements and solid elements are recommended for the

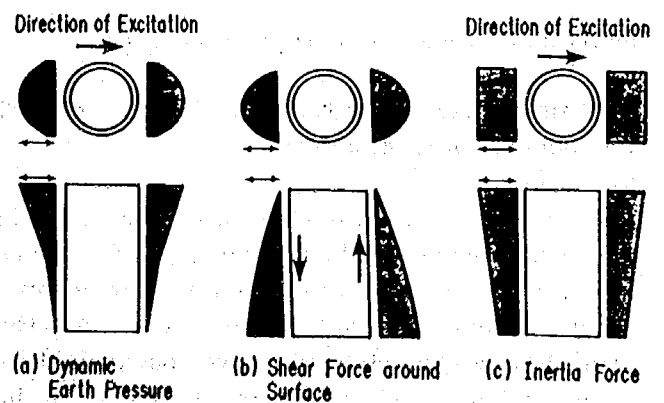


Fig.22 Seismic Effects to Vertically Long Underground Structures

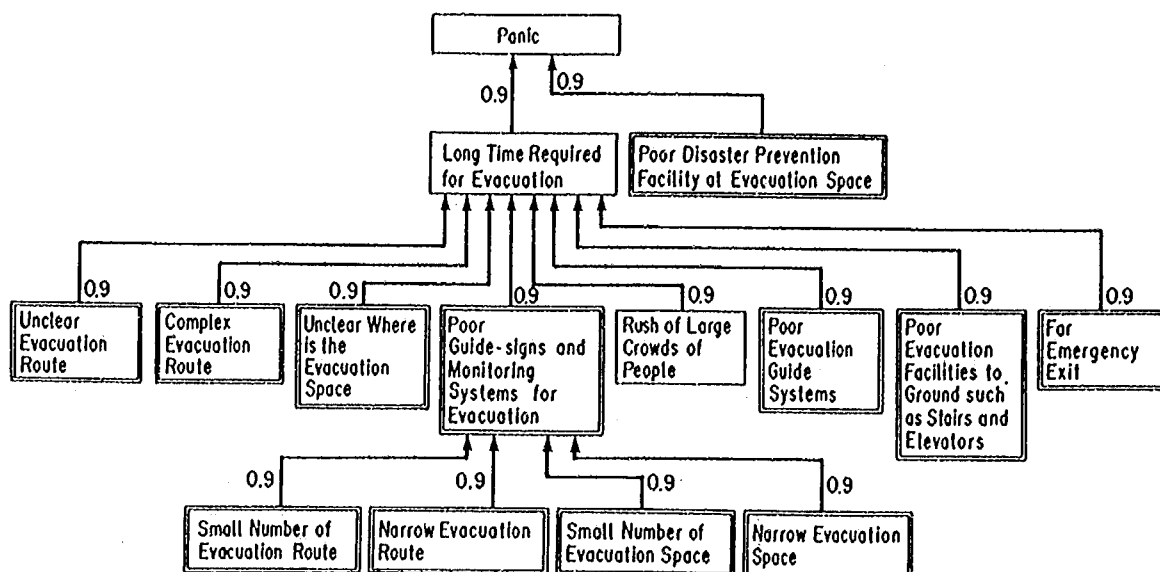


Fig. 23 Structural Model predicted by FSM for Fear and Anxiety against Evacuation

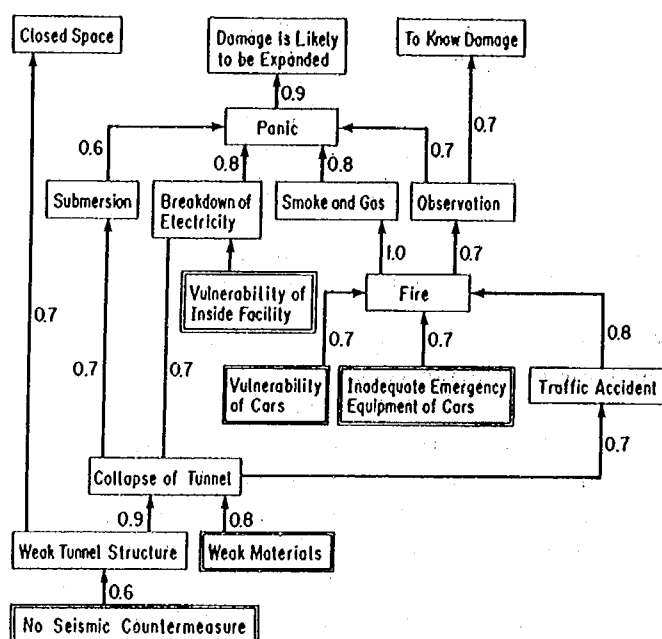


Fig. 24 Structural Model Predicted by FSM for Possible Influence of Damage

analysis of such underground structures.

8 EARTHQUAKE DISASTER PREVENTION SYSTEMS FOR UNDERGROUND HIGHWAYS

8-1 General

Although seismic design is, of course, made for constructing underground highways, there are always considerable anxiety for seismic safety. When an earthquake occurs at the situation where drivers have much anxiety about seismic safety, fear is likely to be amplified to cause panic. Therefore although seismic design is of course essential importance for reducing seismic damage, it is

also important to mitigate fears and anxiety of drivers against safety during an earthquake.

8-2 Fear and Anxiety of Drivers against an Earthquake

Fear and anxiety which are likely to be induced in drivers when they imagine "underground highways" and "earthquakes" were analyzed by the Value Engineering Method. It was found from the analysis that there are two types of fear and anxiety. The first is the fear and anxiety which are always possessed by drivers against the image of "underground highways", and the second is the fear and anxiety against the accidents which are likely to be developed during an earthquake. Although some of them are infantile imaginations from the professional sense, it is important to know how the drivers feel against "underground highways" and "earthquakes".

(1) Fear and Anxiety Always Possessed by Drivers

a) Fear and anxiety from latent sense to the underground structures

Fear and anxiety come from various latent sense to the underground highways such as darkness, narrowness and dirtiness in comparison with the on-ground highways. They also come from unexperienced driving on underground, unfamiliar sense and insensibility to time and direction.

b) Fear and anxiety against driving

Fear and anxiety are sometimes developed even only anticipating to drive on underground highways. The typical senses are

the difficulty to know where they are driving, limited visibility and possible bad manner of drivers.

c) Fear and anxiety against environment

They come from anticipation that the environment may be worse at underground highway than at on-ground highways, such as insufficient facilities for services and rest room and difficulty for access to other facilities. Little change of scenery and circumstance and possible insufficient maintenance for service facilities also cause the fear and anxiety. Sometimes they are induced even from imaging that there may be few drivers at the underground highways.

(2) Fear and Anxiety against Possible Accidents during an Earthquake

a) Fear and anxiety associated with possible influence of damage

Fear and anxiety are developed against possible influence to drivers due to structural damage, damage of non-structural components and crash of cars. Submergence and fire are the most typical scenario for such fear and anxiety. They are also induced by looking at the occurrence of panic, break-down of electricity and accidents.

b) Fear and anxiety against emergency countermeasures

Some drivers have fear and anxiety because they do not know how to respond to emergent situation, where disaster prevention facilities are and how to operate them due to their inexperience.

c) Fear and anxiety against evacuation

Fear and anxiety are induced by imaging unsuccessful evacuation from underground highways. For example, they have the fear and anxiety against the possible difficulty to know where the evacuation and refuge space are, possible long time required for evacuation, insufficient sign for direction and rush of refugee.

d) Fear and anxiety for insufficient information for damage and refuge

They come from the anxiety for not be able to get enough information on damage situation and evacuation. Drivers at underground highways anticipate that they could not get the information on what damage was developed on ground, and that the information may be incorrect even if they could get the information.

e) Fear and anxiety against traffic accidents

This is not necessarily unique to the underground highways. Fear and anxiety are induced against snake-like driving¹⁾ and collision of cars due to sudden brake.

8-3 Factors Causing Fear and Anxiety to Drivers at Underground Highways

There are various factors causing fear and anxiety to drivers during an earthquake. Because the factors are complicatedly coupled, their structure was analyzed by the Interpretive Structural Modeling Method and the Fuzzy Structural Modeling Method. Figs. 23 and 24 show some of such analyses. Based on the analyses, the most basic factors (the lowest factors identified from the analyses) causing fear and anxiety to drivers during an earthquake may be represented as :

(1) Fear and anxiety always possessed by drivers

- a) sense of closing and isolation
- b) complex routes
- c) little separation between automobiles and pedestrians
- d) insufficient education for traffic morals
- e) insufficient environmental control
- d) difficulty for access to other facilities

(2) Fear and anxiety against possible accidents during an earthquake

- a) vulnerability of facilities
- b) vulnerability of cars
- c) unexperienced response
- d) insufficient evacuation facilities
- e) insufficient evacuation control systems
- f) insufficient disaster information systems
- g) unexpected obstacles on highways
- h) vibration of tunnel

8-4 Basic Countermeasures against Earthquake Disaster

It is the most important to take countermeasures against the most fundamental factors causing fear and anxiety of drivers during an earthquake. Because the contribution of each factor is different, the countermeasure have to be taken considering their effectiveness. The Analytical Hierarchy Process Analyses were made to identify the predominant factors for causing fear and anxiety of drivers, and it was found from such analyses that the following measures may be effective :

(1) Simulation of environments of on-ground

highways

It is basic to simulate the environments of on-ground highways so as to prevent drivers to have strong feeling to be driving on the underground highways. It is effective to make the space large enough so that mental pressure due to the sense of closing and isolation be small. It is effective to have sun-light at underground.

(2) Positive utilization of advantage of underground

Because ground shaking at underground is smaller than that at the ground surface, positive education of this feature may be important. It is also important to educate that the evacuation to the ground surface is not the only one choice for drivers during an earthquake, because underground structures are sometimes more safe than the on-ground structures.

(3) Disaster education and disaster prevention training

Because there are various fear and anxiety which come from unawareness of disaster prevention systems such as the unawareness of where the disaster prevention facilities are, more efforts are required to educate drivers on disaster prevention systems. Distribution of pamphlet for educating the disaster prevention systems may be essential. Construction of disaster prevention center, which is in most cases constructed at the corner of the underground space difficult to see from drivers, at the place where drivers can see easily may be an effective way for obtaining the trust from drivers. Positive advertisement of such disaster prevention systems may be effective.

(4) Traffic control systems

Nice traffic control systems are important. It is required to shut-down the traffic to enter the underground highways after a significant earthquake.

(5) Emergency Facilities

Reliable access to the ground surface which can be used even after a significant earthquake is required. However as was pointed out in (2), it is not the only one choice for drivers to evacuate to the ground surface. It may be more reliable to construct stable shelter in underground so that drivers could evacuate into the shelter in case of emergency.

(6) Formulation of disaster information system

It is required to provide the disaster

information system for showing what are happening on at on-ground soon after the occurrence of an earthquake. Such monitoring system of the on-ground scenery may be also effective at normal time to mitigate the fear and anxiety to be in underground. It should be noted however that the information systems which can provide information by only letters and voice are not sufficient enough, because insufficient amount of information is likely to amplify the fear and anxiety of drivers at underground. Visible monitoring systems is the most effective way.

9 CONCLUDING REMARKS

The preceding pages presented the outline of the Guidelines for Seismic Design Methods of Large Underground Structures. Because it is anticipated that more use of underground space for various purposes such as underground highways and parking is going to be made due to various restrictions of utilization of on-ground land, active use of the Guidelines for designing large underground structures is expected.

ACKNOWLEDGEMENTS

Various supports and guidance were obtained for compiling the Guidelines. The authors express their sincere appreciations to the participants of the Joint Research Program on Development of Seismic Design Method of Underground Structures between the Public Works Research Institute, ACTEC and 8 private firms. The authors also express their appreciation to Professor Kenzo Toki, Disaster Prevention Research Institute of the Kyoto University for his invaluable suggestions throughout the research.

REFERENCES

- 1) Public Works research Institute and Advanced Construction Technology Center: Investigation on Seismic Design Methods of Large Underground Structures, 1989, 1990, 1991, 1992 (In Japanese)
- 2) Public Works Research Institute : Guidelines for Seismic Design Methods of Underground Structures, Technical Note, March 1992 (In Japanese)
- 3) Japan Road Association : Technical Standard for Petroleum Pipeline, June 1971 (In Japanese)
- 4) Japan Water Supply Association : Seismin

Design Specification for Water Supply Facilities, December 1979 (In Japanese)

5) Japan Road Association : Design Method of Common Utility Ducts, March 1986 (In Japanese)

6) Ministry of Construction : New Seismic Design Methods, Technical Note, No. 1185, Public Works Research Institute, March 1977 (In Japanese)

7) Shiba, Y., Kawashima, K., Obinata, N. and Kanoh, T. : Evaluation Procedure for Seismic Stress Developed in Shield Tunnel Based on Seismic Deformation Method, Proc. Japan Society of Civil Engineers, Vol. 404, April 1989 (In Japanese)

8) Shiba, Y., Kawashima, K., Obinata, N. and Kanoh, T. : An Evaluation Method of Longitudinal Stiffness of Shield Tunnel Lining for Application to Seismic Response Analysis, Proc. Japan Society of Civil Engineers, Vol. 398, October 1988 (In Japanese)

9) Kawashima, K., Sugita, H. and Kanoh, T. : Loading Test of Shield Tunnel Specimens subjected to Alternative Axial Loading, 4th U.S. National Conference on Earthquake Engineering, Palm Springs, CA., U.S.A., May 1990

10) Kawashima, K., Hasegawa, K. and Nagashima, H. : Seismic Behavior of Buried Pipelines through Field Observation, 3rd U.S. National Conference on Lifeline Earthquake Engineering, Los Angeles, CA., U.S.A., August 1991

11) Kawashima, K., Sugita, H. and Kanoh, T. : Effect of Earthquake on Driving of Vehicles Based on Questionnaire Survey, Proc. Japan Society of Civil Engineers, Structural Eng./Earthquake Engineering, Vol. 6, No. 2, October 1989

Seismic Response Control of Highway Bridges

by Variable Damper

Kazuhiko KAWASHIMA

Shigeki UNJOH

and

Hideyuki SHIMIZU

24th Joint Meeting of The U.S. -Japan Cooperative Program
in National Resources, Panel on Wind and Seismic Effects

Gaithersburg, MD, U.S.A.

May 1992

THE JOURNAL OF THE AMERICAN MEDICAL ASSOCIATION

PUBLISHED WEEKLY

535 N. Dearborn Ave.

CHICAGO, ILL.

606

Subscription Price

Published for the American Medical Association

Subscription Price, \$5.00 per Annum in Advance

Entered as Second-Class Matter, May 2, 1912

Postpaid

SEISMIC RESPONSE CONTROL OF HIGHWAY BRIDGES BY VARIABLE DAMPER

by

Kazuhiko Kawashima¹⁾, Shigeki Unjoh²⁾ and Hideyuki Shimizu³⁾

ABSTRACT

This paper presents analytical and experimental studies on the dynamic characteristics of a variable damper developed at the Public Works Research Institute. Effectiveness of the variable damper for seismic response control of highway bridges is analytically investigated. A small-size model of the variable damper is developed and the dynamic characteristics through the dynamic loading tests is also presented.

KEY WORDS

Seismic Response Control, Highway Bridges, Variable Damper, Small-Size Model, Dynamic Loading Tests

1. INTRODUCTION

The variable damper¹⁾²⁾ is a viscous damper in which the viscous damping force is variable depending on the response of highway bridges as shown in Fig.1. That is:

1) The damping coefficient of the variable damper is taken large during a small deck vibration so that the damper has the same function as a fixed bearing support against the braking load of vehicles. It is movable against actions with low-velocity such as an elongation of deck by temperature change.

2) When the amplitude of deck vibration becomes larger up to a certain level during an earthquake, the damping coefficient is decreased so that the energy dissipation be optimum and the inertia force of a superstructure to substructures be decreased appropriately.

3) Furthermore, when the vibration amplitude of the deck becomes excessive, the damping coefficient is gradually increased in order to suppress the excessive amplitude. Therefore, the damper has a function as a stopper with a shock absorber. This is to prevent the impact response by the sudden operation of the stopper⁴⁾.

Therefore, the variable damper has the advantages of an usual viscous damper stopper^{5)~7)}, a passive energy dissipator and a stopper with a shock absorber function.

The concept of the variable damper, in which the damping characteristics can be variable dependent on the situation, has been effectively adopted in a field of mechanical engineering, in particular in a suspension system for an aircraft and an automobile. An active suspension to improve driving comfortability and stability of the automobile is one of the examples⁸⁾⁹⁾. However, it is a new idea to apply the variable

damper for highway bridges based on the concept of distributing the inertia force of a superstructure to substructures.

Although various types of the variable damper can be made, the simplest one may be as shown in Fig.2. The basic component is an cylinder-type viscous damper. A by-pass pipe is installed between the cylinder-cells divided by the piston. The damping characteristics of the damper can be controlled by varying the amount of flow of viscous material which passes through the by-pass. Since various technology for controlling such material flow has been developed, the variable damper is most promising and near to the practical use. Furthermore, the external energy required for the operation is significantly smaller in the variable damper than in active control devices¹⁰⁾.

This paper presents analytical and experimental studies on the dynamic characteristics of a variable damper developed. The effectiveness of the variable damper for seismic response control of highway bridges is analytically investigated. A small-size model of the variable damper is developed and the dynamic characteristics through the dynamic loading tests is also presented.

2. SEISMIC RESPONSE ANALYSES OF HIGHWAY BRIDGES WITH VARIABLE DAMPERS

2.1 Seismic Response Analysis Method of Multi-Degree-of-Freedom System with Variable Dampers

The equations of motion for a linear multi-degree-of-freedom system with the variable dampers may be written as

$$\mathbf{M} \ddot{\mathbf{x}} + (\mathbf{C} + \mathbf{C}_v) \dot{\mathbf{x}} + \mathbf{K} \mathbf{x} = -\mathbf{M} \ddot{\mathbf{x}}_0 \quad (1)$$

In which $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ represent mass, damping and stiffness matrices of the system, respectively. $\mathbf{x}$ and $\mathbf{x}_0$ denote displacement vector of the system and an earthquake ground motion, respectively. $\mathbf{C}_v$ denotes a damping matrix of the variable dampers and is assumed as

- 1) Head of Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works Research Institute, Ministry of Construction
- 2) Research Engineer, ditto
- 3) First Research Division, Fukushima Construction Office, Tohoku Regional Construction Bureau, Ministry of Construction (Formerly Assistant Research Engineer of the Earthquake Engineering Division, Public Works Research Institute)

$$\underline{C}_v = \sum_{k=1}^{n_v} \underline{c}_{ij}^v \quad (2)$$

where

$$\underline{c}_{ij}^v = \begin{bmatrix} V & -V \\ -V & V \end{bmatrix} \quad (3)$$

In which $\underline{c}_{ij}^v$ and n_v represent damping matrix of the variable damper, which is installed between the i -th node and the j -th node, and a number of the variable dampers installed, respectively. V is the damping coefficient of an each variable damper and is given as a function of relative displacement and relative velocity between the nodes where the variable dampers are installed.

Although there are various methods to prescribe the damping matrix $\underline{C}$, it is assumed here to be written as Eq.(4) assuming that the damping matrix $\underline{C}$ can be diagonalized by the modal matrix.

$$\underline{C} = (\underline{\Phi})^{-1} \text{diag}[2h_k \omega_k] (\underline{\Phi}^T)^{-1} \quad (4)$$

In which, $\underline{\Phi}$ denotes a modal matrix of the system and $\text{diag}[2h_k \omega_k]$ denotes a diagonal matrix with elements of $2h_k \omega_k$ ($k=1, 2, 3, \dots$; mode number).

Since $\underline{C}_v$ is time-varying, Eq.(1) has to be solved by a direct integration method. According to the above analytical method, a computer program "VDAM" which can analyze earthquake response of a multi-degree-of-freedom system with the variable dampers was developed.

2.2 Highway Bridge Analyzed and Analytical Conditions

In order to investigate the effectiveness of the variable damper, a simple span girder bridge, as shown in Fig.3, with a span length of 30m is analyzed. The response in longitudinal direction is considered. Although the variable damper has better applicability for multi-span continuous girder bridges, the simple span girder bridge is taken as an analytical model here for simplicity. The superstructure of the model is supported by elastic isolators in longitudinal direction and the variable dampers are assumed to be installed between the superstructure and the pier top.

The model can be assumed as a single-degree-of-freedom oscillator if it is assumed that the deformation of the superstructure is negligibly small and that the piers are idealized as a linear spring. Therefore, the damping coefficient of the variable damper, V , can be written as

$$V = \sqrt{m_s K_p} h_v \quad (5)$$

In which, m_s , K_p and h_v represent mass of the superstructure, a combined spring stiffness of elastic isolators and piers, and required

damping ratio for the variable damper, respectively. The weight of the superstructure and spring stiffness of the isolator are assumed as 241.5tf and 640tf/m, respectively. The flexural stiffness of piers is assumed so that the fundamental natural period of the model be 0.5sec when the both bearing supports are hinged. The stiffness of isolators result in the fundamental natural period of 1 sec for the model with elastic isolators.

Damping ratio of the variable damper h_v defined by Eq.(5) can be variable dependent on both relative velocity and relative displacement between the superstructure and the pier top. Only analytical cases in which the damping ratio is varied with the relative deck displacement are shown here for representing the effectiveness of the variable damper although the velocity-dependent variable damper was also studied.

Six cases in total are analysed as shown in Fig.4. In Case 1, the variable damper is not installed, i.e., no control. In Case 2, the damping coefficient of the damper h_1 is assumed independent on the relative deck displacement. The damping ratio h_1 is varied as a parameter to be investigated. Cases 3 and 4 are cases for investigating the effectiveness of the stopper with a shock absorber for preventing excessive deck displacement. The damping ratio increased from h_1 to h_3 dependent on relative deck displacement. In Case 3, a weak stopper, i.e., a stopper with a shock absorber is assumed and the damping ratio h_3 is assumed to be 3. In Case 4, the strong stopper is assumed and the damping ratio h_3 is assumed to be 500. The displacement at which the stopper initiates to work is assumed as 2/3 of the peak relative deck displacement d_u of Case 2. This displacement is defined hereafter as a design displacement of the variable damper. The design displacement of the variable damper d_u is 7.7cm as will be shown in Case 2. Case 5 is a case for investigating the effectiveness of the stopper for suppressing small deck displacement caused by vehicle loads. The damping ratio is increased from h_1 to h_2 in the range of the relative displacement from $-1/6d_u$ to $1/6d_u$. Case 6 is a combination of Cases 3, 4 and 5.

An acceleration ground motion shown in Fig.5 is used as an input acceleration. The acceleration, recorded on the ground around the Tsugaru Bridge during the Nihonkai-Chubu Earthquake of 1983, was modified so that the response spectrum matches with the target spectrum. The acceleration spectrum, which is specified in "Part V Seismic Design" of the "Design Specifications for Highway Bridges", for the check of bearing capacity of reinforced concrete piers on soft soil site (ground condition III), is assumed as the target spectra.

When the variable damper is not provided, the modal damping ratio of the model bridge is assumed to be 2% of critical in the above cases.

2.3 Effectiveness of the Variable Dampers for Seismic Response Control of Highway Bridges

Fig.6 shows how the peak deck response and force developed at the pier vary in accordance with damping ratio h_1 for constant damping (Case 2). The peak deck displacement monotonously decreases with increasing the damping ratio h_1 and become almost stable for the damping ratio equal or greater than 2. The peak deck acceleration takes minimum at the damping ratio h_1 of about 0.5. This is because the function as a damper stopper becomes predominant rather than an energy dissipator. The shear force and bending moment at the pier bottom with increase of the damping ratio h_1 in a similar manner with the peak deck acceleration.

On the other hand, as the damping ratio increases the maximum damping force developed in the variable damper increases, while stroke of the piston decreases. Total energy dissipated during the excitation takes maximum value at the damping ratio of about 0.2.

Based on the analysis, the most appropriate damping ratio h_1 is considered as 0.5, which makes peak deck displacement and maximum bending moment at the pier bottom decrease to 31% and 37% of those with no control, respectively. The damping force and peak stroke of the piston required for this control are $34.7\text{tf} \times 2 = 69.4\text{tf}$ and 7.7cm, respectively.

Assuming h_1 as 0.5, the effectiveness of the stopper with a shock absorber was analysed. Fig.7 compares the deck displacement and acceleration as well as damping force of the variable damper for Cases 2 - 4. In Case 4, the pulse-type response developed by strong operation of the stoppers is seen in the damping force of the variable damper. Fig.8 compares the peak response and peak damping force for Case 2 - 4. Comparing Case 3 with Case 2, the peak deck response slightly decreases while maximum forces at the pier bottom increase only 7%. The maximum damping force of Case 3 developed in the variable damper becomes 1.73 times that of Case 2, and the peak stroke decreases by about 10%. The decrease of the stroke of the variable damper means that the function of the stopper operates effectively.

On the other hand, when the stopper operates strongly (Case 4) the responses become larger, and deck acceleration and forces at the pier bottom become about 2 times those in Case 3. Although the stroke of the variable damper decrease to the displacement at which the strong stopper operates, the peak damping force increase to about 4.3 times that in Case 3.

Fig.9 shows the effect of the stopper at the small deck displacement. The damping ratio of the damper, h_1 , is assumed to be 0.5 and h_2 is varied as a parameter. Although the peak deck displacement decrease monotonously as the damping ratio h_2 of the variable damper increases up to 8, it increases for the damping ratio h_2 greater than 8. This is because that the function

as a damper stopper becomes more predominant rather than an energy dissipator when the damping ratio h_2 becomes larger. The peak deck acceleration takes a minimum value when the damping ratio h_2 is about 1, and it increases monotonously for the damping ratio h_2 greater than 1. The forces at the pier bottom vary with h_2 in a similar manner with the deck acceleration.

On the other hand, as the damping ratio h_2 increases the damping force of the variable damper increases and the stroke and the total energy absorbed decrease. Therefore, damper at the small deck displacement which is for suppressing the deck motion caused by the vehicle loads has contrary effects to make seismic response smaller. Therefore, it is necessary to determine damping ratio h_2 in consideration with the function of bridges for normal loads.

Table 1 summarizes the peak responses in Case 1 to 6. If the variable damper with $h_1 = 0.5$ and $h_2 = h_3 = 3$ is provided, peak deck displacement and peak deck acceleration decrease 25% and 43% of those with no control, respectively. The maximum bending moment at the pier bottom decreases to 46% of that. The required damping force and peak stroke of the variable damper are $62.6\text{tf} \times 2 = 125\text{tf}$ and 5.78cm, respectively. The variable damper with this specification can be designed within the current scope of damper technology.

3. DEVELOPMENT OF A SMALL-SIZE MODEL OF THE VARIABLE DAMPER

3.1 Outline of A Small-Size Model of the Variable Damper

Fig.10 shows a design plan of the variable damper developed. The model is designed so that the damping force be variable in the range of 20kgf to 200kgf, maximum relative displacement of damper piston be within $\pm 2.5\text{cm}$. The control of the damping ratio is arbitrarily made by a personal computer dependent on relative displacement and velocity developed between deck and substructures.

The piston-cylinder has the total length of 394mm, the pressure area of the piston is 12.56cm^2 and the stroke of the piston is $\pm 2.5\text{cm}$. A steel pipe by-pass is installed between the cylinder-cells divided by the piston and the oil tank through two servo-valves. The servo-valve is called as a D.C. proportional magnetic valve, and the width of the valve is adjusted by the magnetic force of the D.C. proportional solenoid. The return spring is installed at the spool as shown in Fig.11. The differential transformer is installed at the servo-valve for the position detection of the spool so that the accurate control can be made. The input voltage from a servo amplifier to the servo valve is set in the range of 0 to 5V. The valve is closed at the input voltage of 0V and the valve is fully opened at

5V. The viscous material used for the variable damper is an usual oil for a dynamic loading actuator.

Fig.12 shows the flow of the viscous material in the variable damper. The damper piston can move in the left and right directions accompanying with the deck response. When the piston moves in the left side the viscous material flows from the left cell of the cylinder to the oil tank through the servo-valve 1, and from the tank to the right cell through the check valve 2 which can pass the oil only in one direction. At this time the servo-valve 2 is closed. When the piston moves in the opposite direction the oil passes through from cylinder to the tank through the servo-valve 2 and check valve 1. Therefore, the damping characteristics of the variable damper is controlled by the servo valve 1 when the piston moves in the left side, and by the servo-valve 2 when the piston moves in the right side.

3.2 Dynamic Loading Tests of the Small-Size Model of the Variable Damper

The variable damper can vary the damping characteristics depending on the displacement and/or velocity of the structures. Therefore, in order to provide the variable damper with the required damping characteristics it is necessary to know the basic dynamic characteristics of the variable damper. The damping characteristics of the variable damper is controlled by the amount of opening of the servo-valve and it is controlled by the input voltage to the servo-valve. Therefore, to know the characteristics of the variable damper developed, the relation between the input voltage to the servo-valve and damping characteristics of the variable damper is essential.

Fig.13 and Photo 1 show the set-up of the dynamic loading tests. The opening of the servo valve is kept to be constant under the condition that the input voltage is set to be a constant value, i.e., the damping ratio of the variable damper is set to be constant, the load is applied to the model by a dynamic actuator under the displacement control. The input voltage to the servo amplifier, reaction force and stroke of the piston are measured. The loading is harmonic and 10 cycle were repeated for each loading displacement.

Table 2 shows the experimental cases. Case 1 is to measure the friction force of the damper piston and static loading was made with loading frequency of 0.05Hz and loading displacement of 20mm. The input voltage of the variable damper is assumed to be 5V so as to open the valve fully. Case 2 is to obtain the input voltage to the servo-valve vs. damping force of the variable damper relation. Loading frequency, loading displacement and input voltage are varied.

3.3 Performance of the Small-Size Model of the Variable Damper

(1) Friction force of the damper-piston

Fig.14 shows a hysteresis loop of the load-displacement relation when the damper is loaded with the loading frequency of 0.05Hz and loading displacement of 20mm to obtain the friction force of the damper-piston. The load-displacement relation shows almost rectangular shape. This represents the typical type energy dissipation. The hysteresis is stable for a number of loading cycles. The average friction force was obtained as about 7kgf.

(2) Hysteresis Loops of the Load-displacement Relation

Fig.15 shows hysteresis loops of the load-displacement relation. In the loading displacement of 2mm, the time-lag was found in the peak damping force for the input voltage lower than 1.5V. This is developed due to compressibility and friction between the pipes and the viscous material. To compensate the effect it is required to make the pressure area greater for obtaining enough flow even if the displacement is small. In the loading displacement of 6mm and 10mm, the hysteresis loop seems as expected.

(3) The relation between the input voltage and damping force

Fig.16 shows the relation between the loading displacement and damping force dependent on the input voltage. The empirical equations obtained by a least square method are also shown in Fig.16. The relation between the damping force and loading velocity shows a function of the power of the loading velocity as an usual cylinder-type viscous damper. Therefore, the empirical equation is assumed as

$$F = F_0 + C(V) \cdot v^{n(V)} \quad (6)$$

where

- F : damping force (kgf)
- F_0 : friction force of the damper-piston (7kgf)
- $C(V)$: damping coefficient (function of the input voltage V)
- v : velocity of the damper-piston
- $n(V)$: power coefficient of velocity term (function of the input voltage V)

In Eq.(6), damping coefficient $C(V)$, and power coefficient of velocity term $n(V)$, are unknown factors. They are computed by a least square method based on the tests results. The damping coefficient $C(V)$ thus obtained are almost constant in the range of the input voltage of 0 to 5V, and averaged value of $C(V)$ is 0.8. The relation between the input voltage and the power coefficient of the velocity term obtained is shown in Fig.17. The power coefficient increases with decrease of the input voltage. When the

Input voltage is larger than 2.5V, the power coefficient becomes almost constant ($n(V) = 1.4$), and when the input voltage is lower than 2.5V the value takes larger. The relation obtained by a linear approximation is shown in Fig.17.

5. CONCLUDING REMARKS

The effectiveness of the variable damper developed for seismic response control of highway bridges was analytically investigated. The small-size model of the variable damper was developed and the performance was studied through the dynamic loading tests. According to the above investigations the following conclusions may be deduced:

1) In order to provide functions as damper stopper, energy dissipator and stopper with shock absorber for the variable damper, the damping ratio of $h_1 = 0.5$ and $h_2 = h_3 = 3$ is the most favorable.

2) By providing the variable damper with damping ratio shown in 1), the peak deck displacement and the peak deck acceleration of the model highway bridge were reduced to 25% and 43%, respectively. The peak bending moment decreased to 46%. The required peak damping force and peak stroke of the variable damper were $62.6\text{tf} \times 2 = 125\text{tf}$ and 5.78cm , respectively. The peak damping force is equivalent with 52% of the deck weight. These capability of the damper can be designed within the current scope of damper technology.

3) Based on the dynamic loading tests of the small-size model of the variable damper, the load-displacement relation was obtained as expected. A proto-type model of the variable damper is now under development for further improving the performance.

REFERENCES

- 1) Taguchi, J., Iwasaki, T., Adachi, Y., Sasaki, Y. and Kawashima, K. : U.S.-Japan Cooperative Research Program on Hybrid Control of Seismic Response of Bridge Structures, 22nd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, Maryland, USA, May, 1990
- 2) Kawashima, K., Unjoh, S., Nagashima, H. and Shimizu, H. : Current Research Efforts in Japan for Passive and Active Control of Highway Bridges Against Earthquakes, 23rd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, May, 1991
- 3) Liu, S., C., Lagorio, H., J. and Chong, K., P. : Status of U.S. Research on Structural Control Systems, 23rd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, May, 1991
- 4) Kawashima, K., Hasegawa, K. and Nagashima, H. : Experiment and Analysis on Seismic Response of Menhshin Bridges, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991
- 5) Fukuoka, S. : Design of Multi-span Continuous Girder Bridge using Viscous Shear Stopper, Bridge, pp.22-29, February, 1980 (In Japanese)
- 6) Izeki, H. : Viscous Shear Stopper, Proposal to the Multi-Continuous Girder Bridge, Bridge, pp.30-33, February, 1980 (In Japanese)
- 7) Matsumura, S., Fukuoka, S., Mizumoto, Y. and Nakata, T. : Seismic Design of Multi-Span Continuous Girder Bridge using Damper Stopper, Bridge and Foundation, pp.31-37, May, 1982 (In Japanese)
- 8) Milliken, Jr., W., F. : Lotus Active Suspension System, Proceedings of 11th International Conference on Experimental Safety Vehicles, pp.467-475, Washington D.C., May, 1987
- 9) Sugawara, F. : Electronically Controlled Shock Absorber System Used as a Road Sensor which Utilizes Super Sonic Waves, Proceedings of Society of Automotive Engineers, pp.6.15-6.25, 1986
- 10) Kawashima, K., Unjoh, S. and Shimizu, H. : Optimum Seismic Response Control of Highway Bridges by Active Mass Damper, Civil Engineering Journal, Vol.33, No.9, pp.61-68, November, 1991 (In Japanese)
- 11) Kawashima, K., Unjoh, S. and Shimizu, H. : Earthquake Response Control of Highway Bridges by Variable Damper, Proceedings of Colloquium on Control of Structures, Japan Society of Civil Engineers, Part. B, pp.221-224, July, 1991 (In Japanese)
- 12) Feng, Q. and Shinozuka, M. : Use of A Variable Damper for Hybrid Control of Bridge Response Under Earthquake, U.S. National Workshop on Structural Control Research, University of Southern California, CA., U.S.A., October, 1990
- 13) Shinozuka, M., Feng, Q. and Kawamura, S. : New Perspectives for Sliding Base Isolation Systems, Proceedings of International Workshop on Recent Developments in Base-Isolation Techniques for Buildings, Tokyo, Japan, 27-30 April, 1992
- 14) Kawashima, K., Unjoh, S. and Shimizu, H. : Earthquake Response Control of Highway Bridges by Variable Damper, Transactions of the Japan National Symposium on Active Structural Response Control, Japan Science Council, pp.311-317, March, 1992 (In Japanese)
- 15) Kawashima, K., Unjoh, S. and Shimizu, H. : Experimental Study on Dynamic Characteristics of Variable Damper, Transactions of the Japan National Symposium on Active Structural Response Control, Japan Science Council, pp.311-317, March, 1992 (In Japanese)

Table 1 Peak Response of the Bridge and the Variable Damper

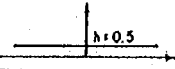
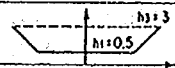
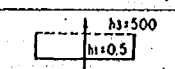
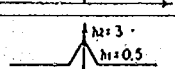
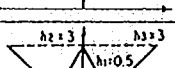
Analytical Cases		Deck			Pier		Variable Damper			
		Displacement (cm)	Velocity (cm/S)	Acceleration (cm/S ²)	Shear Force (tf)	Bending Moment (tf·m)	Damping Force (tf)	Relative Displacement (cm)	Relative Velocity (cm/S)	Total Energy (tf·m)
1	No Control	33.04	189.0	1300	166.9	3 297	—	—	—	—
2	Constant Damping 	10.28	60.2	478	63.4	1 223	34.7	7.67	44.9	93.9
3	Displacement Dependent Damping with Weak Stopper for Excessive Deck Response Displacement 	9.94	54.2	538	72.0	1 395	60.2	6.90	68.1	94.4
4	Displacement Dependent Damping with Strong Stopper for Excessive Deck Response Displacement 	10.90	76.4	965	123.9	2 382	151.3	5.54	112.3	94.9
5	Displacement Dependent Damping with Stopper for Small Deck Response Displacement 	8.50	51.3	567	80.1	1 540	64.7	6.34	55.8	74.5
6	Displacement Dependent Damping (Combination of Case 3 and Case 4) 	8.38	47.5	555	81.2	1 528	62.6	5.78	60.7	77.4

Table 2 Experiment for Small-Size model of Variable Damper

Case	Objective	Loading Frequency (Hz)	Loading Displacement (mm)	Input Voltage (V)
1	Measurement of Friction Force of Damper Piston	0.05	20	5
2	Relation between Damping Force and Input Voltage	1, 2, 3, 4	2, 6, 10, 15	0~5

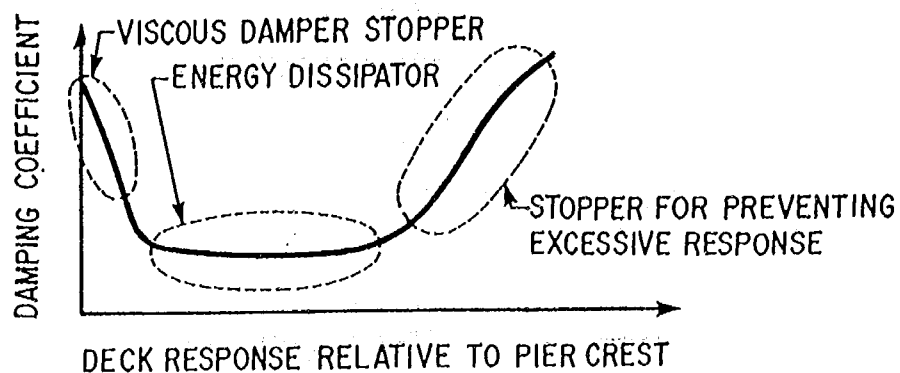


Fig.1 Basic Concept of Variable Dampers

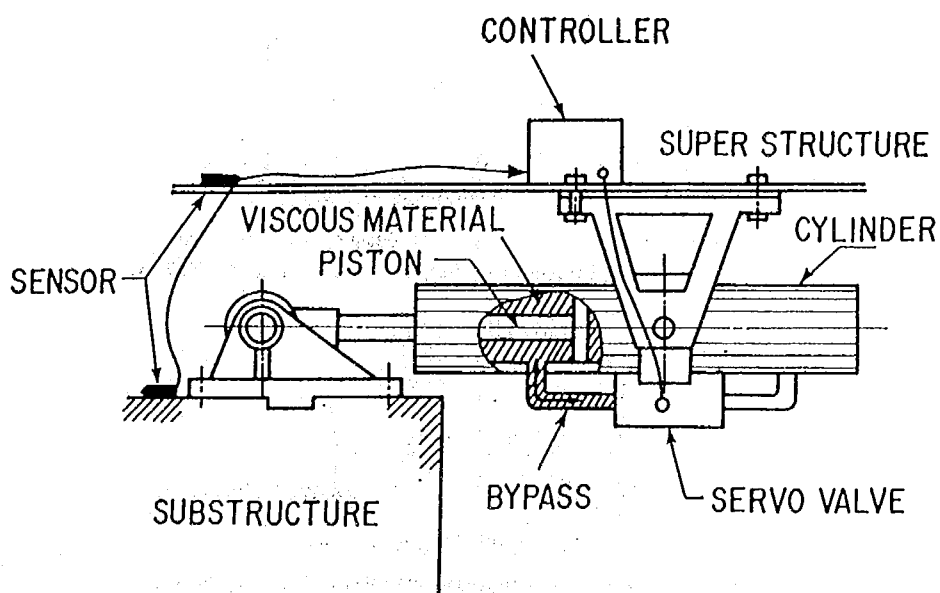


Fig.2 Variable Damper

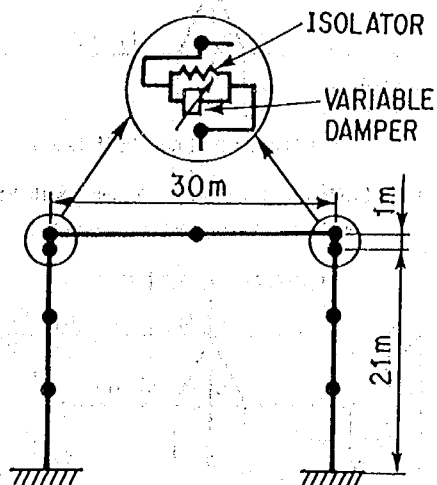
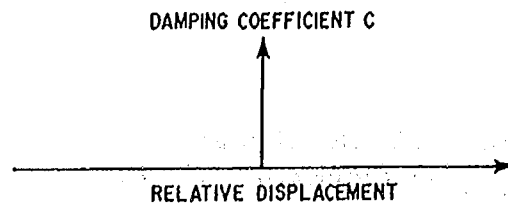
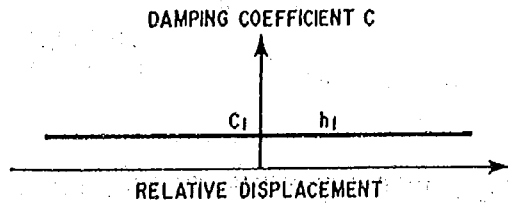


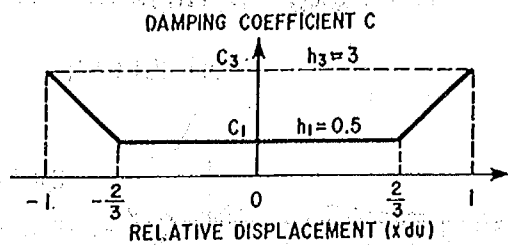
Fig.3 Highway Bridge Analyzed and Analytical Idealization



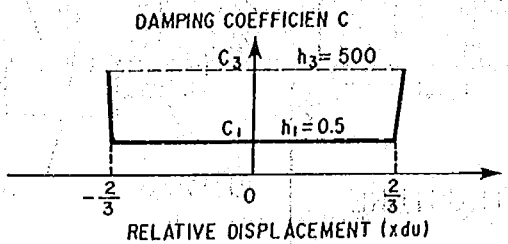
(a) Case 1 : No Control



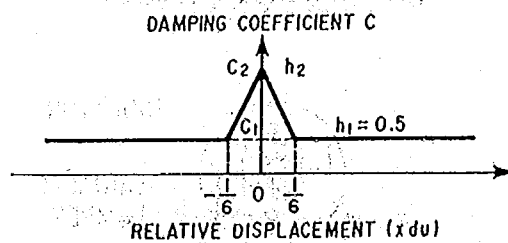
(b) Case 2 : Constant Damping



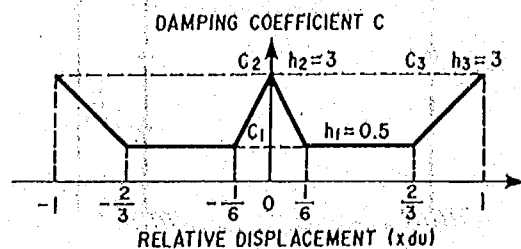
(c) Case 3 : Displacement Dependent Damping with Weak Stopper for Excessive Deck Response Displacement ($du=7.67\text{ cm}$)



(d) Case 4 : Displacement Dependent Damping with Strong Stopper for Excessive Deck Response Displacement ($du=7.67\text{ cm}$)



(e) Case 5 : Displacement Dependent Damping with Stopper for Small Deck Response Displacement ($du=7.67\text{ cm}$)



(f) Case 6 : Displacement Dependent Damping (Combination of Case 3 and Case 5) ($du=6.34\text{ cm}$)

Fig.4 Analytical Cases

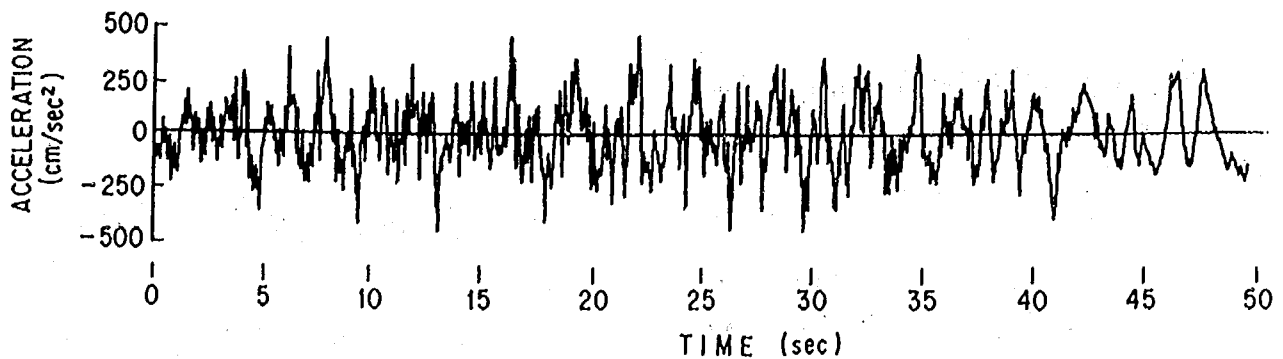
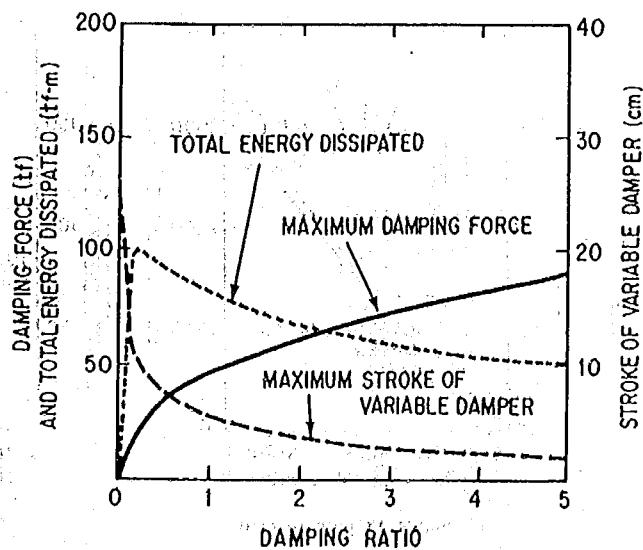
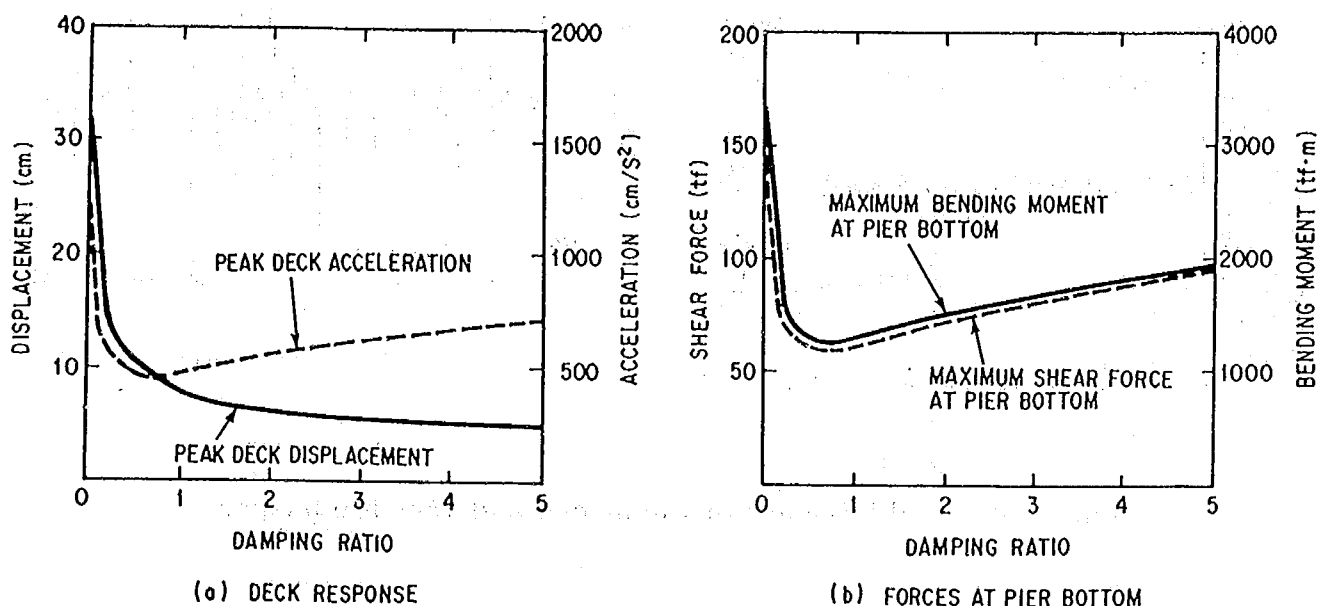


Fig.5 Input Acceleration



(c) RESPONSE OF VARIABLE DAMPER

Fig.6 Peak Response of the Bridge and the Variable Damper (Case 2)

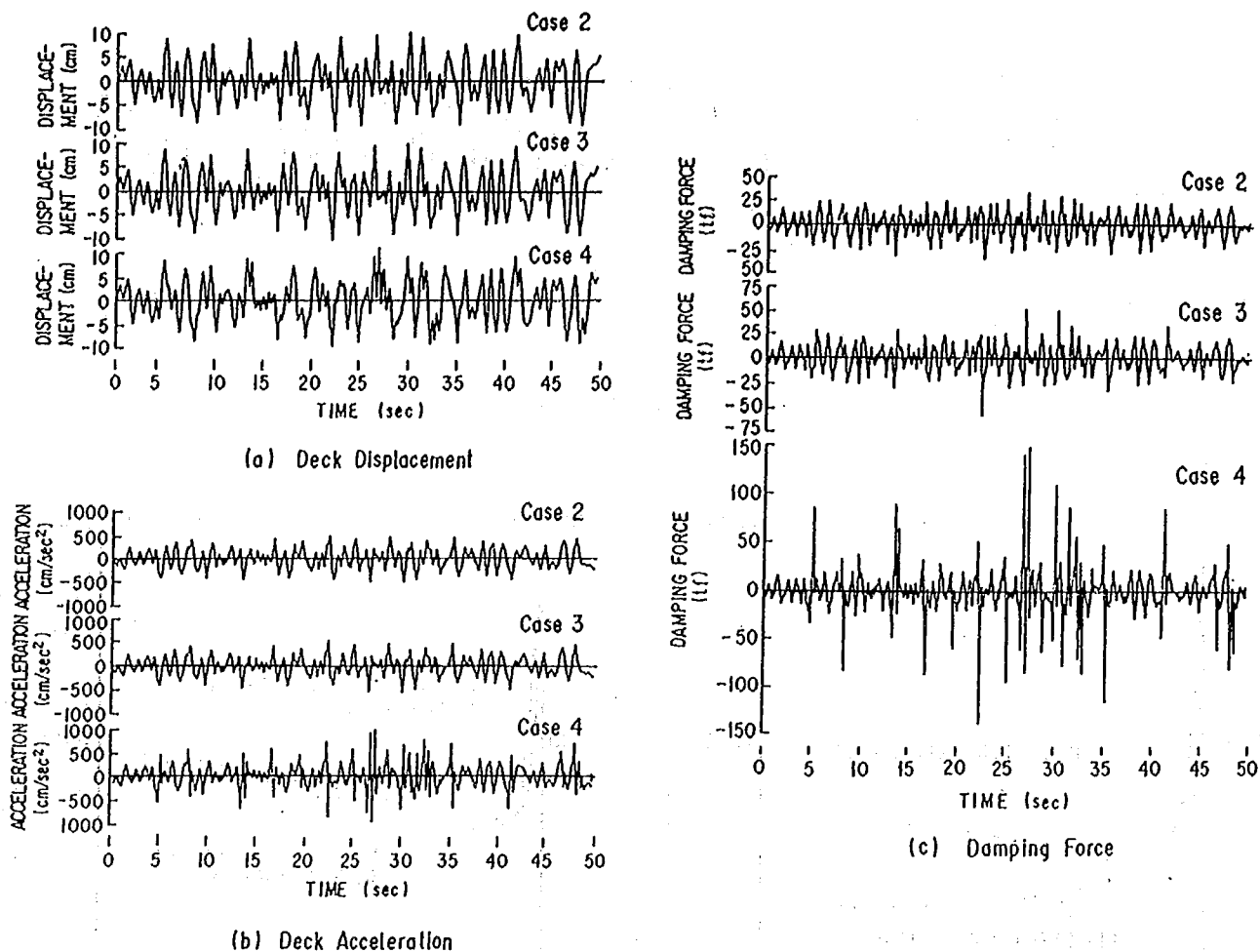


Fig.7 Computed Response of the Bridge and Variable Damper

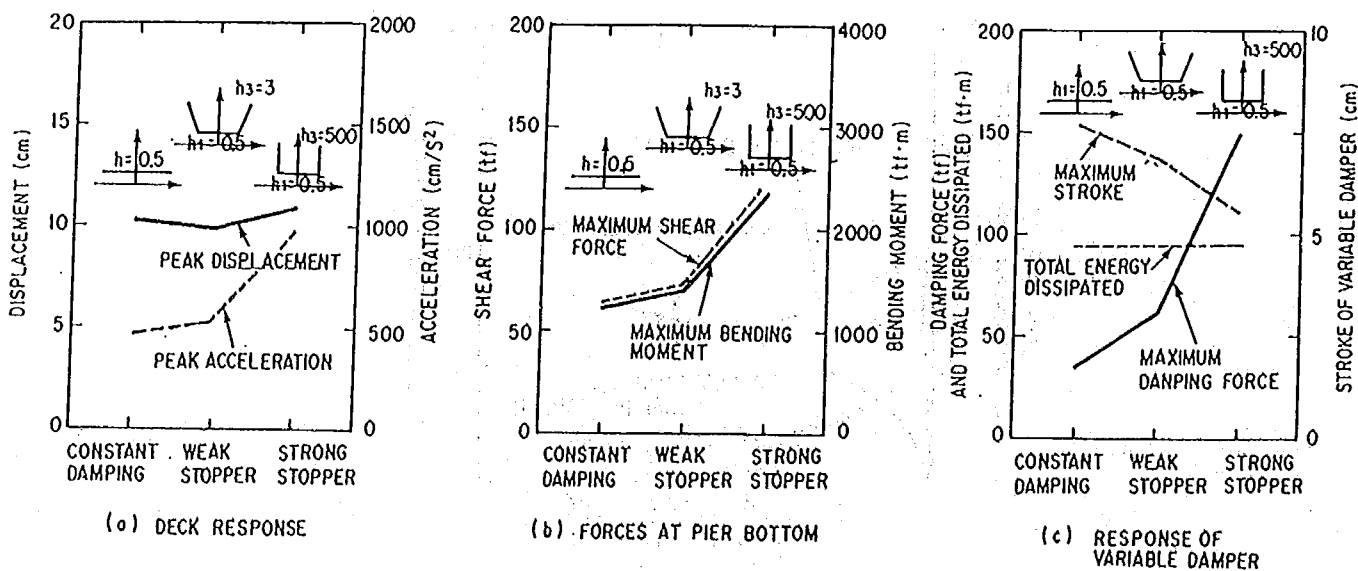
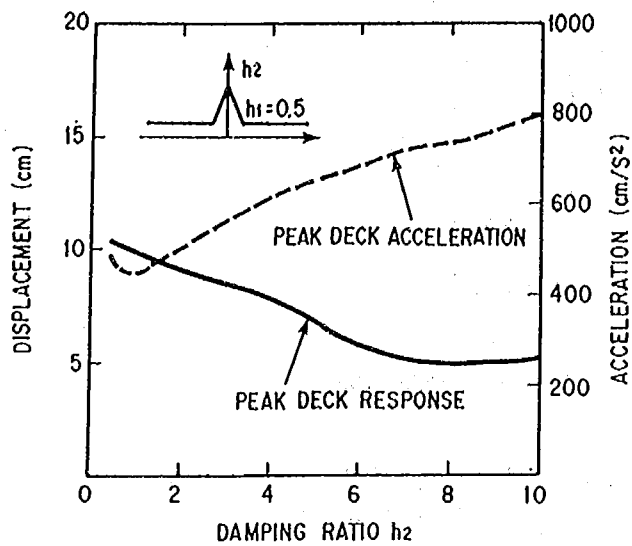
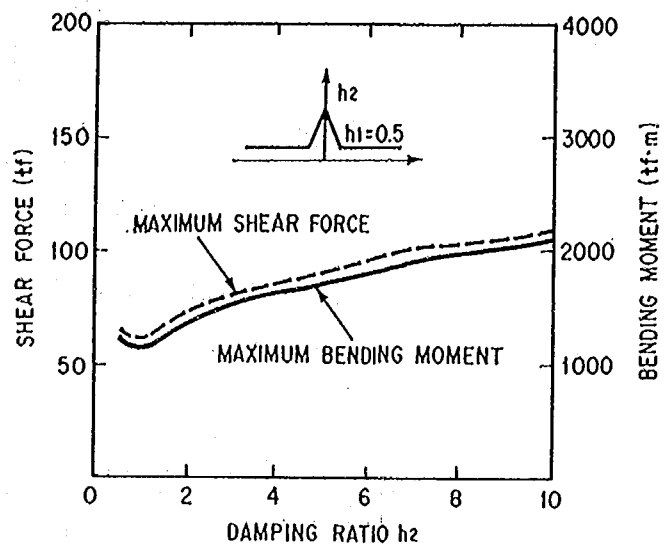


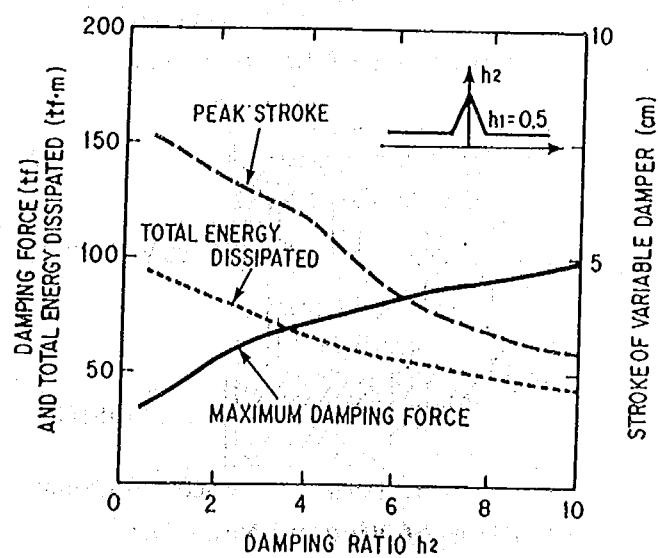
Fig.8 Effect of Weak Stopper (Case 3) and Strong Stopper (Case 4) for Excessive Deck Response Displacement



(a) DECK RESPONSE



(b) FORCES AT PIER BOTTOM



(c) RESPONSE OF VARIABLE DAMPER

Fig.9 Effect of Stopper for Small Deck Response Displacement (Case 5)

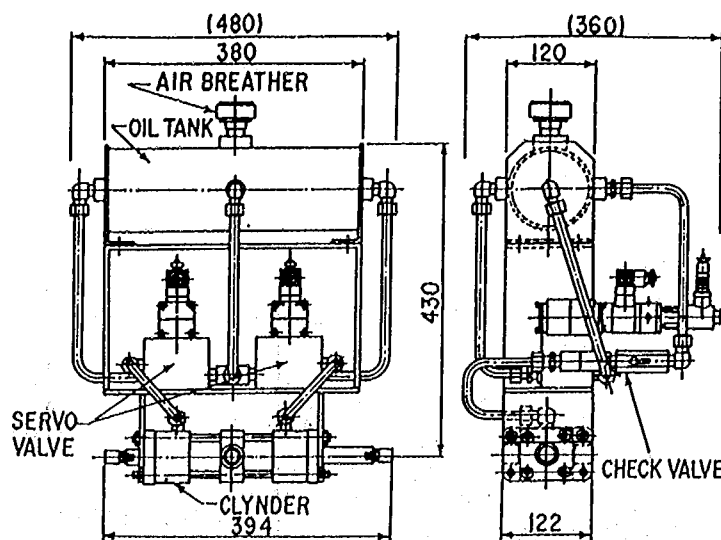


Fig.10 Small-Size Model of Variable Damper

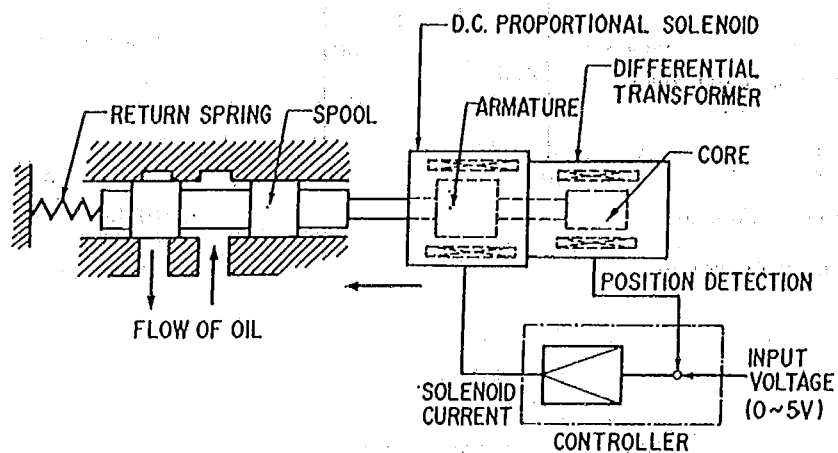


Fig.11 Structure of Servo Valve

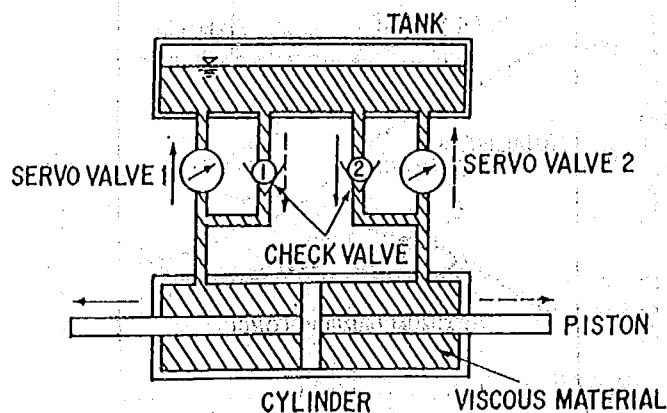


Fig.12 Flow of Viscous Material

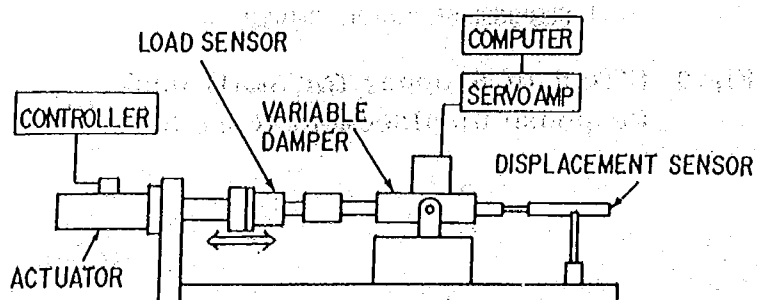


Fig.13 Set Up of Loading Test

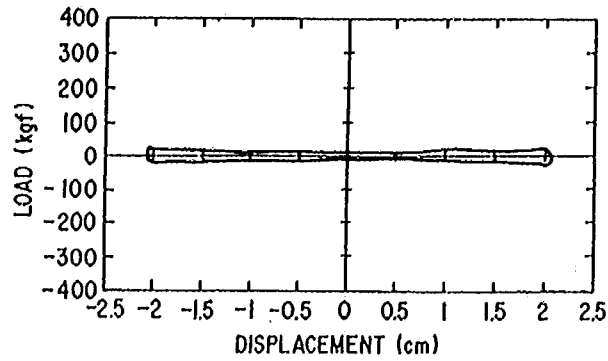
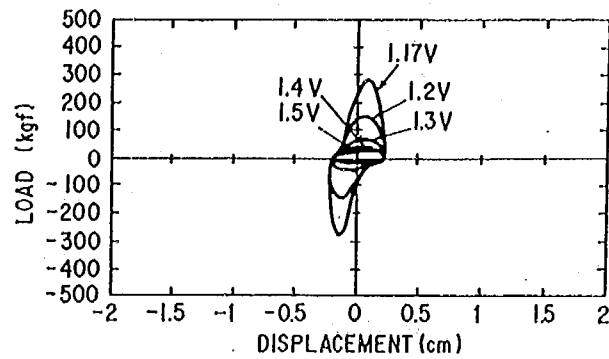
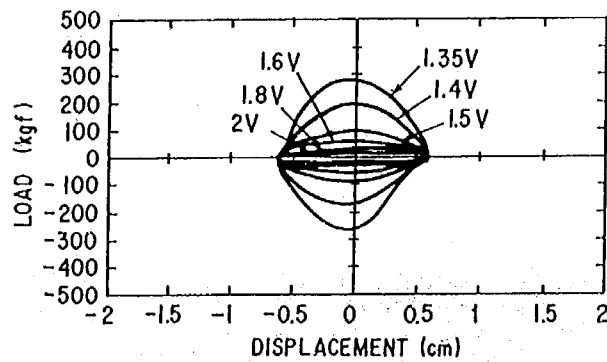


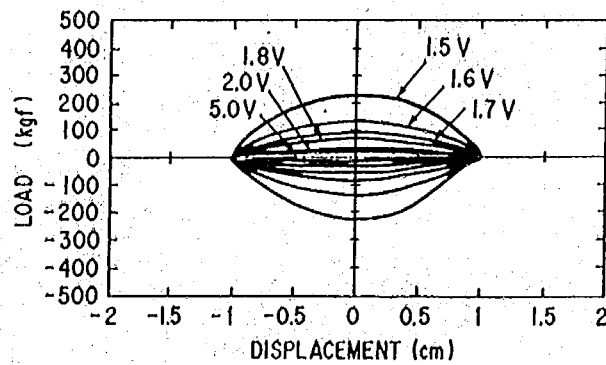
**Fig.14 Hysteresis Loops of Load-Displacement Relation
(Displacement=20mm, Frequency=0.05Hz)**



(a) Displacement 2mm



(b) Displacement 6mm



(c) Displacement 10 mm

**Fig.15 Hysteresis Loops of Load-Displacement Relation
(Frequency=1Hz)**

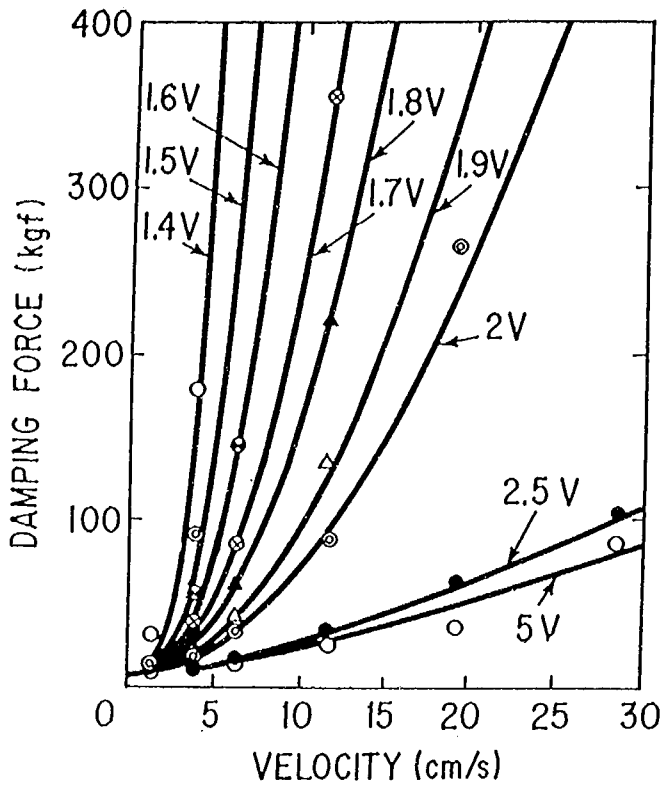


Fig.16 Damping Force vs. Peak Velocity Relation

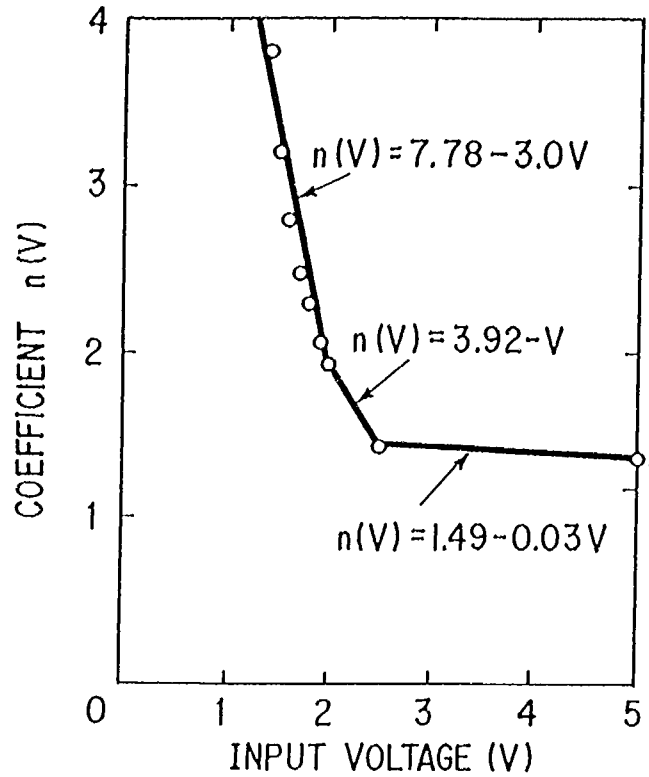


Fig.17 Power Coefficient $n(V)$ vs. Input Voltage Relation

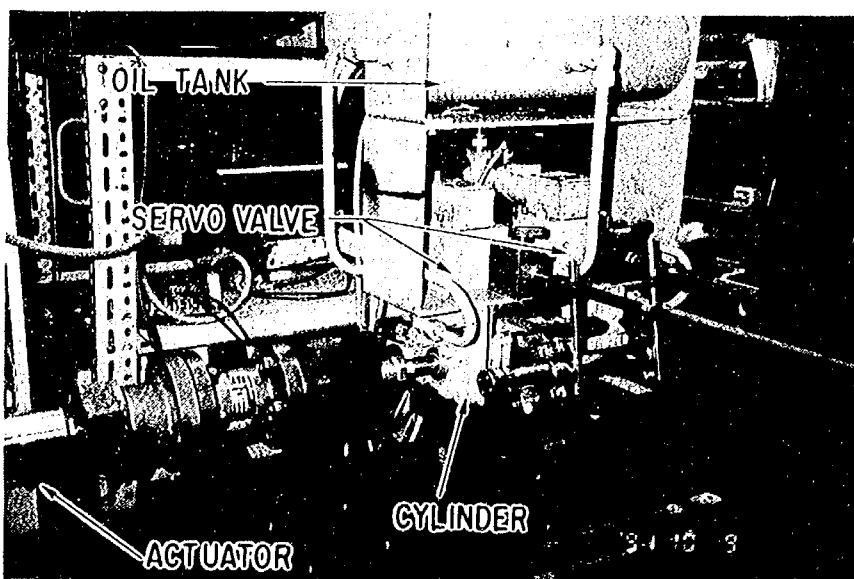


Photo 1 Experimental Set Up

Seismic Design Method of
Underground Parking Garage Structures

Kazuhiko KAWASHIMA

Hideki SUGITA

and

Tomoru NAKAJIMA

5th U. S. -Japan Workshop on
Earthquake Disaster Prevention for Lifeline Systems

Tsukuba, Japan

October 1992

to the 100th Anniversary of the

Establishment of the National

Archives and Records Administration

and the National Historical

Archives

and the National Archives

and the National Archives

and the National Archives

and the National Archives

and the National Archives

SEISMIC DESIGN METHOD OF UNDERGROUND PARKING GARAGE STRUCTURES

by Kazuhiko KAWASHIMA¹, Hideki SUGITA² and Tomoru NAKAJIMA³

ABSTRACT

Presented in this paper is the outline of seismic design method of reinforced concrete underground structures for parking garage based on the Guide Specifications of Design and Construction of Parking Garage issued by the Ministry of Construction in 1992. Background of seismic design by means of the Seismic Deformation Method is described.

INTRODUCTION

Resolving traffic congestion in urban area is a significant importance. Providing enough space for parking is one of the key solutions for decreasing the traffic congestion and traffic accident. For this purpose, the Ministry of Construction revised the Road Law in 1991 so that the bodies who construct the road, such as the Ministry of Construction and Regional Governments, can construct parking garages as the "Supplementary Structures of Road". It is anticipated that many garages would be constructed underground of roads in urban areas because of various limitation of space available and restriction of land use on ground.

For facilitating the construction of parking space and garage, the Guide Specifications of Design and Construction of Parking Garage was notified by the Ministry of Construction in March 1992. Main body of the notification as well as the supplementary explanation was published by the Japan Road Association in

-
- 1 Head, Earthquake Engineering Division, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan
 - 2 Research Engineer, ditto
 - 3 Research Engineer, ditto

November 1992¹⁾. The Guide Specifications cover planning, design and construction of parking space and garage. In the chapter of Planning, the overall procedure of planning, basic considerations for type selection, lay-out, size and height of parking lots are described. In the chapter of Design, description is given to design and construction of reinforced concrete underground structures for parking garage, because they are the major type adopted in urban area and they require specific design and construction considerations. When other type of structures such as prestressed concrete structures are used, appropriate modifications are required to adopt the Guide Specifications. General design procedure of main body of the structures including seismic design and design of facilities such as electricity, lightning and ventilation are described. Construction procedure with emphasis on the fact that the construction of underground parking garage structures is made in highly populated city areas are described in the chapter of Construction.

Table of contents of the Guide Specifications is as :

Chapter 1 General

Purpose

Scope

Chapter 2 Planning

1 Survey

2 Basic Planning

Chapter 3 Design

1 General

1.1 Scope

1.2 Definition of terms

2 Design of Main Body of Parking Garage Structures

2.1 Loads

2.2 Materials

2.3 Allowable stress

2.4 Soil properties for design

2.5 General provisions for design of structural members

2.6 Design of structural members

2.7 Structural details

2.8 Design of foundation

2.9 Special consideration for wall and floor

3 Seismic Design

3.1 Basic principle for seismic design

3.2 Loads and load combination

- 3.3 Evaluation of soil liquefaction
 - 3.4 Seismic design
 - 3.5 Evaluation for uplift due to liquefaction and countermeasures
 - 3.6 Evaluation of safety by dynamic response analysis
 - 4 Design of Temporary Works for Construction
- Chapter 4. Construction
- 1 General
 - 2 Planning of Construction
 - 3 Notes for Construction

This paper outlines the seismic design method of the underground parking garage structures based on the Guide Specifications.

STRUCTURAL TYPE OF UNDERGROUND PARKING GARAGE

Structural type of underground parking garage may be classified into three groups, i.e., the Slab and Beam Type Structure, Beam and Column Type Structure and Flat Slab Type Structure as shown in Fig. 1.

The Slab and Beam Type Structure which is the most often adopted is of longitudinal beams, slabs, side walls and columns. Lateral beams are not provided. Because two dimensional frame idealization is the most popular in design analysis, the structure is idealized as shown in Fig. 2 for design purpose. The design is made for two perpendicular directions. The side walls are assumed as the plates which resist for earth pressure, and are assumed to be supported by the lateral beams, columns and slabs. Contribution of the slabs is considered in evaluating the equivalent stiffness of longitudinal beams. The upper slab, intermediate slab and lower slab are assumed as beams with the equivalent stiffness. Thus, the structure is idealized as an elastic frame consisting of beams (slabs) and columns as shown in Fig. 2.

The Beam and Column Type Structure is of longitudinal beams, transverse beams, lateral beams, columns, slabs and side walls. The side walls are assumed as the plates which resist for earth pressure, and are assumed to be supported by the lateral beams, transverse beams and columns. The slabs are assumed as the plates which resist to either soil pressure (upper slab and lower slab) or vehicle load (intermediate slab). Contribution of the slabs and the side walls is considered in evaluating the equivalent stiffness of beams and columns. The structure is idealized as shown in Fig. 3.

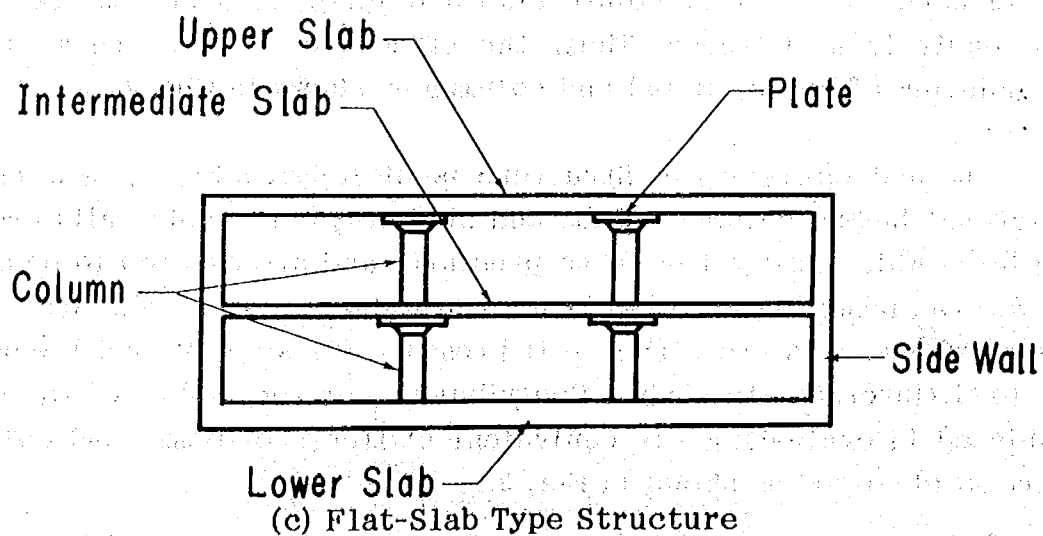
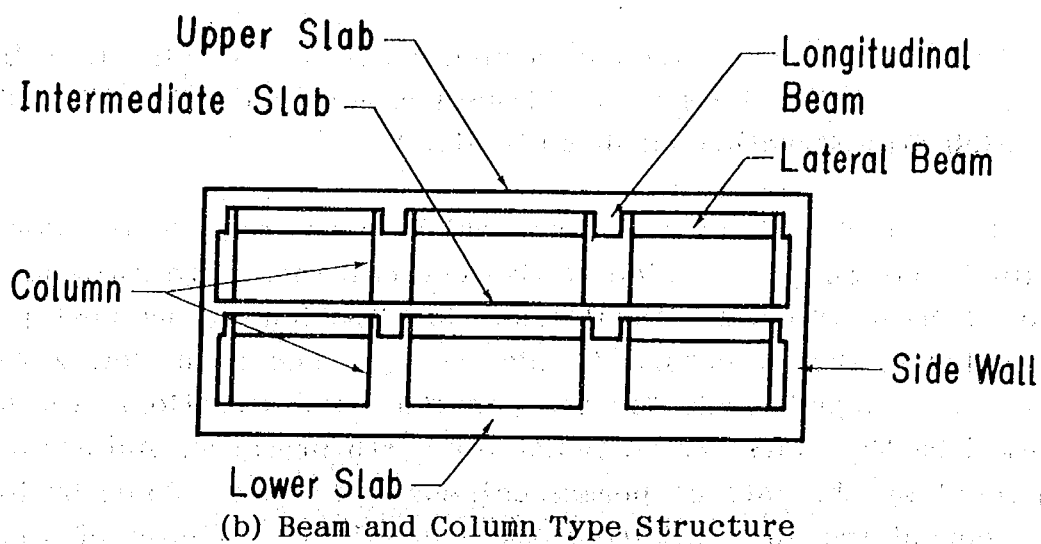
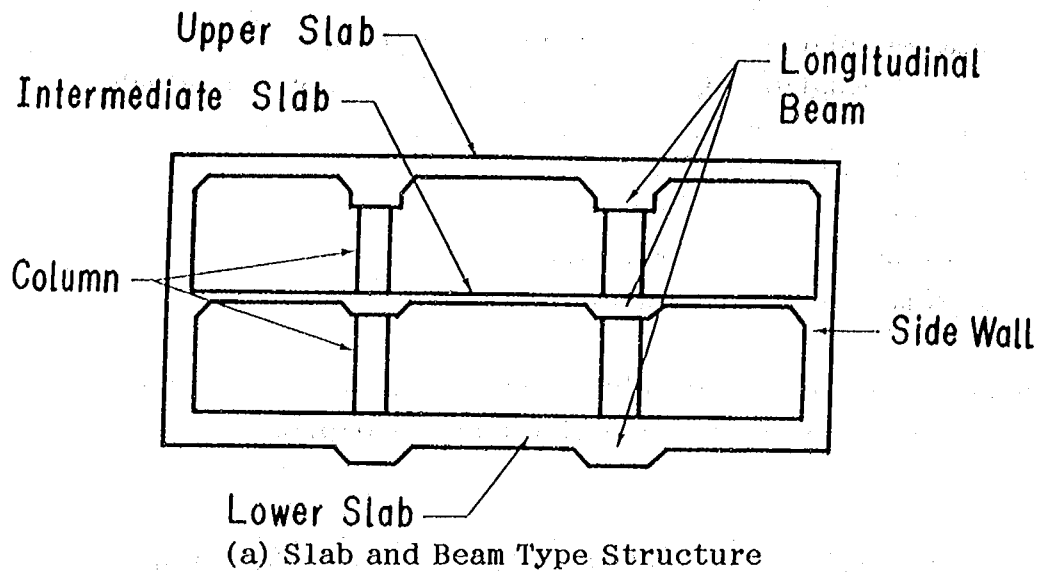
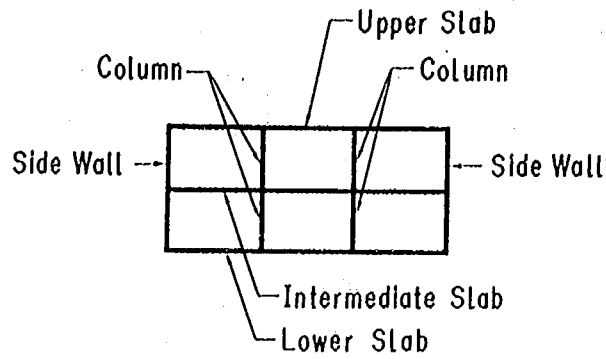
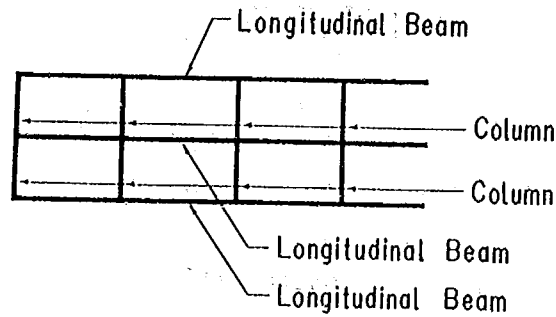


Fig. 1 Major Structural Type of Underground Parking Garage (2 Story)

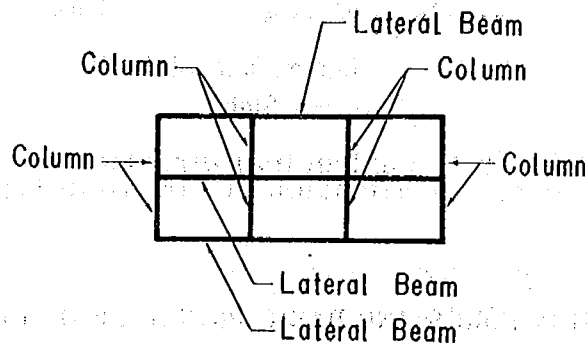


(a) Lateral Direction

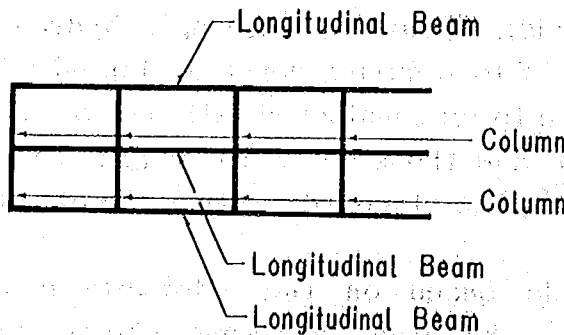


(b) Longitudinal Direction

Fig. 2 Analytical Idealization of Slab and Beam Type Structure



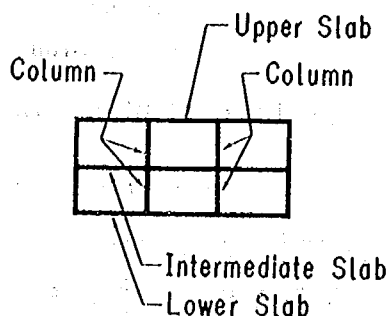
(a) Lateral Direction



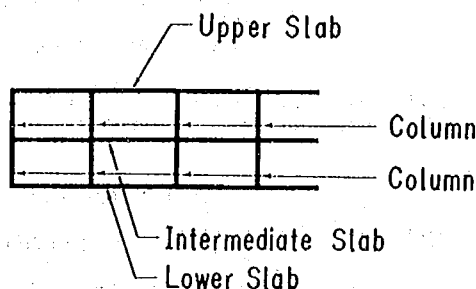
(b) Longitudinal Direction

Fig. 3 Analytical Idealization of Beam and Column Type Structure

The Flat-Slab Type Structure is of slabs, side walls and columns. Because no lateral beams and longitudinal beams are provided, slab-column connections require special attention in their strength. The slabs are assumed as beams with the equivalent stiffness. The side walls are assumed as the plates which resist for earth pressure, and are assumed to be supported by the columns and slabs. The idealization of the Flat Slab Type Structure is shown in Fig. 4.



(a) Lateral Direction



(b) Longitudinal Direction

Fig. 4 Analytical Idealization of Flat Slab Type Structure

DESIGN FORCE AND BASIC PRINCIPLE OF DESIGN

In design of underground parking garage structures, ① dead weight, ② active load (vehicle load), ③ earth pressure, ④ hydraulic pressure, ⑤ uplift force, ⑥ seismic effects, ⑦ load during construction, ⑧ effect of soil settlement and ⑨ other effects have to be considered. All the loads are not always need to be considered in design, and those which are required to be considered have to be selected. Table 1 shows the standard combination of the loads.

The design is made based on the allowable design approach. Design calculations are therefore made assuming the elastic behavior of the

Table 1 Standard Combination of Loads

Loads	Static	Construction Stage	Seismic
Dead Weight	○	○	○
Active Load	○	△	○
Earth Pressure	○	○	○
Hydraulic Pressure	○	○	○
Uplift Force	○		○
Seismic Effects			○
Temporary Load during Construction		○	
Effect of Ground Settlement	△		
Other Loads	△		△

○ has to be considered
△ considered if required

structures. The calculation of forces and stress have to be principally made based on the frame analysis, in which the structure is assumed as a two dimensional frame consisting of beams and columns. Safety has to be examined in the two perpendicular directions.

SEISMIC DESIGN

Basic Principle of Seismic Design

Seismic design is made by the Seismic Deformation Method²⁾. In the Seismic Deformation Method, the displacement and deformation developed in the subsurface ground during an earthquake are taken as the major seismic effects to underground structures. The seismic response of underground structures is distinguished by the small mass effect and the large damping ratio from regular structures on ground, i.e.,

- mass effect is small because unit weight of underground structures is smaller than that of subsurface ground, and
- damping is very large because radiational damping due to energy dissipation from underground structures to subsoils is very large.

Therefore, it is assumed in the Seismic Deformation Method that underground structures deform following the deformation of subsurface ground during an earthquake. Modelling of underground structures has to be made so that structural response of the underground structures be correctly represented.

Because only the increase of stress and deformation of underground structures due to seismic effects are evaluated by the Seismic Deformation Method, the stress and deformation actually developed during an earthquake have to be computed by adding them with those developed due to the static loads such as the dead weight.

When liquefaction of sandy soil layers is developed around the underground parking garage, the uplift associated with the increase of pore water pressure has to be examined.

Dynamic response analysis is recommended for the underground parking garages with complex structural response.

Loads Considered for Seismic Design

In seismic design, dead weight, earth pressure, water pressure, uplift force, ground settlement and seismic effects are to be considered. The loads have to be combined so that the most critical stress or displacement be developed. As the seismic effects, the following loads have to be considered^(3) 4):

(a) Inertia Force

The lateral force coefficient has to be evaluated as

$$k_h = c_z \cdot c_g \cdot c_d \cdot k_{h0} \quad (1)$$

where,

k_h : lateral force coefficient

c_z : modification factor for zone (refer to Fig. 5)

c_g : modification factor for ground condition (refer to Table 2)

c_d : modification factor for depth, and is evaluated as

$$c_d = 1.0 - 0.015z \quad (2)$$

z : depth from ground surface (m)

k_{h0} : standard lateral force coefficient (= 0.2)

Fig. 5 Modification Factor for Zone

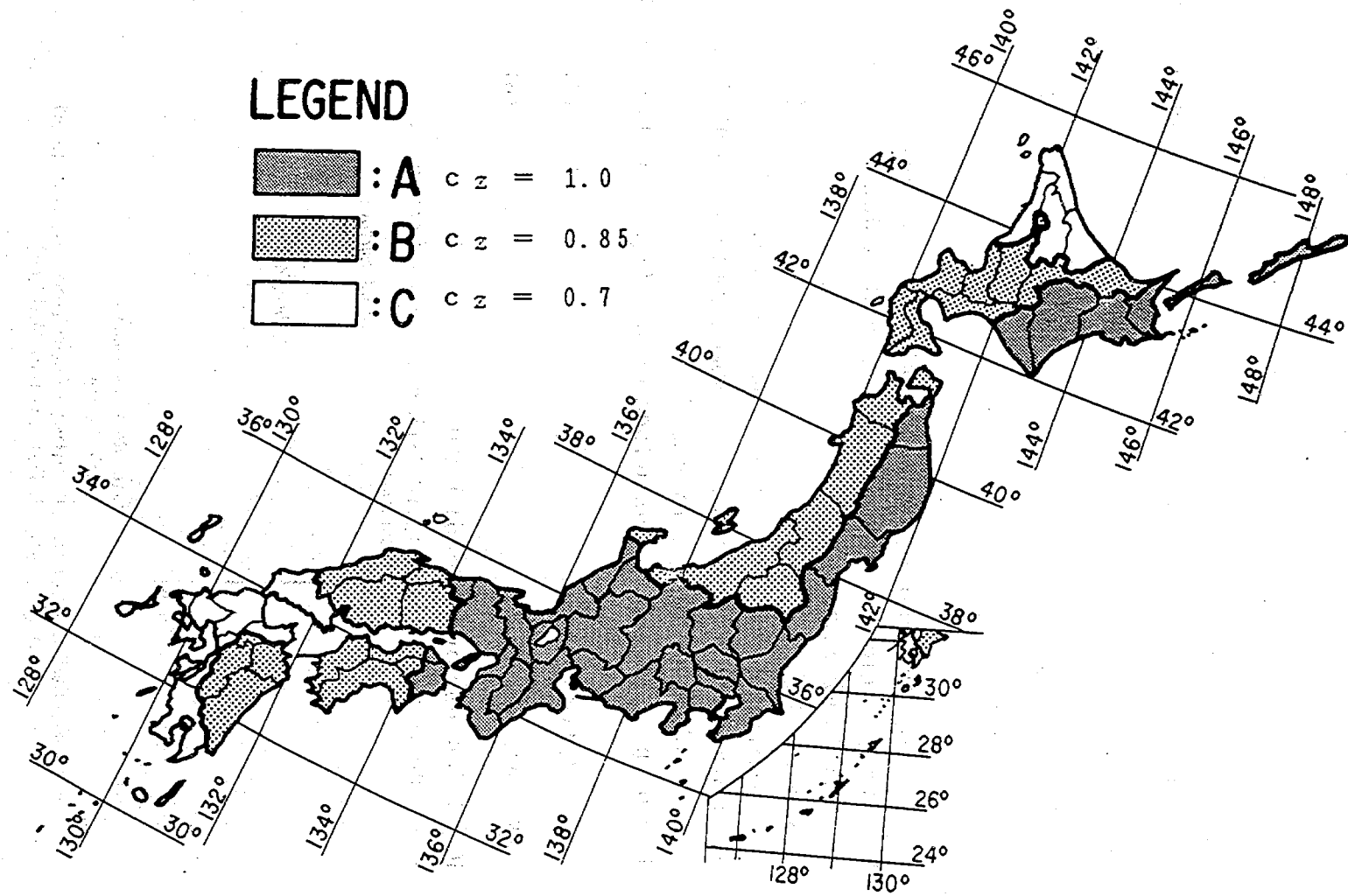


Table 2 Modification Factor for ground Condition

GROUND CONDITION	DEFINITION	APPROXIMATE ESTIMATION
GROUP I	$T_G < 0.2\text{SEC}$	TERTIARY OR OLDER
GROUP II	$0.2 \leq T_G < 0.6\text{SEC}$	ALLUVIUM AND DILUVIUM
GROUP III	$0.6\text{SEC} \leq T_G$	SOFT ALLUVIUM

(b) Design Ground Displacement

The design ground displacement is evaluated depending on the depth as (refer to Fig. 6) :

$$u(z) = \frac{1}{\pi^2} S_v \cdot T_s \cdot \cos\left(\frac{\pi z}{2H}\right) \quad (3)$$

where,

$u(z)$: design ground displacement (m) at depth z (m)

S_v : design velocity response spectrum at design base rock (cm/sec), and is given as

$$S_v = c_z \cdot S_{v0} \quad (4)$$

c_z : modification factor for zone (refer to Fig. 5)

S_{v0} : standard velocity response spectrum (cm/sec) at design base rock, and is given by Fig. 7

T_s : natural period of subsurface ground (sec), and is evaluated as

$$T_s = 1.25 \cdot T_G \quad (5)$$

$$T_G = 4 \sum \frac{H_i}{V_{si}} \quad (6)$$

H_i : thickness of i -th soil layer (m)

V_{si} : shear wave velocity of i -th soil layer (m/sec)

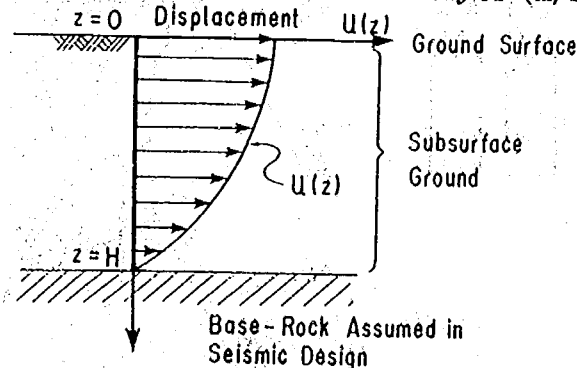


Fig. 6 Deformation of Subsurface Ground Assumed In Seismic Deformation Method

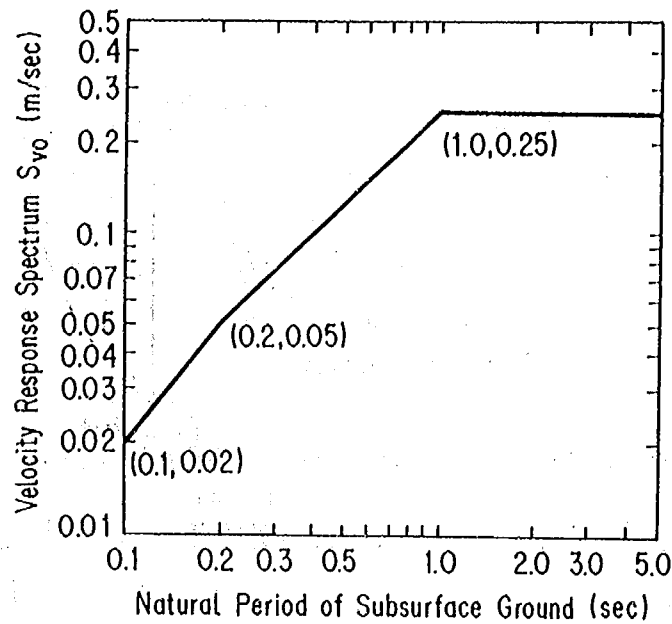


Fig. 7 Standard Velocity Response Spectra

(c) Design Shear Stress around Surface

At the surface of underground parking garage structures contacting with soils, the design shear stress around surface has to be applied to the structure as shown in Fig. 8. The design shear stress around surface is given by Eq. (7). If the design shear stress around surface by Eq. (7) is larger than the strength of soils, the design shear stress around surface shall be the strength of soils.

$$\tau = \frac{G_D}{\pi H} S_{v0} T_s \sin\left(\frac{\pi z}{2H}\right) \quad (7)$$

where,

τ : design shear stress around surface (tf/m²)

S_{v0} : design velocity response spectrum at design base rock (cm/sec), and is given by Eq. (4)

G_D : shear modulus of subsurface ground (tf/m²)

T_s : natural period of subsurface ground (sec), and is computed by Eq. (5)

H : thickness of subsurface ground (m)

(d) Dynamic Earth Pressure

Dynamic earth pressure by Eq. (8) is applied to side walls of underground parking garage structure as shown in Fig. 9.

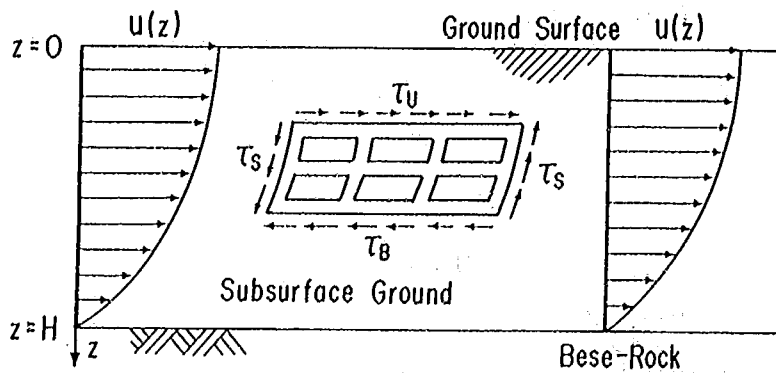


Fig. 8 Design Shear Stress around Surface

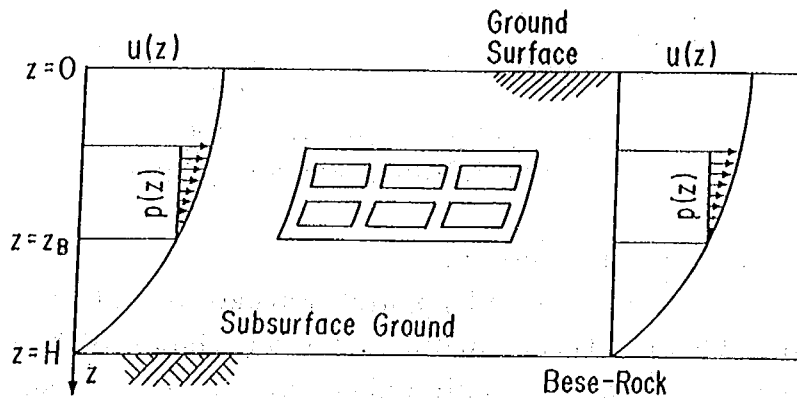


Fig. 9 Dynamic Earth Pressure

$$p(z) = k_H \cdot \{u(z) - u(z_B)\} \quad (8)$$

where,

$p(z)$: dynamic earth pressure per unit area (tf/m^2) at depth z (m)

k_H : spring stiffness of soils per unit area (tf/m^2)

$u(z)$: design ground displacement (m), and is given by Eq. (3)

z_B : depth of base of structure (m)

Effect of Liquefaction

(a) Estimation of Liquefaction

Saturated alluvial sandy layers which have the water table within 10 m from the ground surface and have D_{50} -values on the grain size accumulation curve between 0.02 and 20. mm are vulnerable to liquefaction up to a depth of 20 m below the ground surface.

For those soil layers which were judged to be vulnerable to liquefaction, liquefaction potential shall be checked based on the liquefaction resistance factor $F_L^{(5)}$ defined as

$$F_L = R / L \quad (9)$$

where,

F_L : liquefaction resistance factor

R : resistance of soil elements against dynamic load defined as

$$R = R_1 + R_2 + R_3 \quad (10)$$

$$R_1 = 0.0882 \sqrt{\frac{N}{\sigma_v' + 0.7}} \quad (11)$$

$$R_2 = \begin{cases} 0.19 & (0.02 \text{ mm} \leq D_{50} \leq 0.05 \text{ mm}) \\ 0.225 \log_{10}(0.35/D_{50}) & (0.05 \text{ mm} < D_{50} \leq 0.6 \text{ mm}) \\ -0.05 & (0.6 \text{ mm} < D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (12)$$

$$R_3 = \begin{cases} 0.0 & (0 \% \leq F_C \leq 40 \%) \\ 0.004 F_C - 0.16 & (40 \% < F_C \leq 100 \%) \end{cases} \quad (13)$$

N : N-value of standard penetration test

D_{50} : averaged grain size on grain size accumulation curve

F_C : fine sand (grain size less than $74 \mu\text{m}$) content

L : dynamic load induced in soil elements during an earthquake defined as

$$L = k_s \frac{\sigma_v}{\sigma_v'} \quad (14)$$

k_s : seismic coefficient for evaluating liquefaction, and shall be determined as

$$k_s = c_z \cdot c_g \cdot c_u \cdot k_{s0} \quad (15)$$

c_z and c_g : modification factors for zone and ground condition (refer to Fig. 5 and Table 2)

c_u : modification factor for depth, and is given by Eq. (2)

k_{s0} : standard design horizontal seismic coefficient for check of liquefaction (= 0.15)

σ_v : total overburden pressure (kgf/cm^2), and

$$\sigma_v = \{ \gamma_t h_w + \gamma_{t2} (z - h_w) \} / 10 \quad (16)$$

σ_v' : effective overburden pressure (kgf/cm^2), and

$$\sigma_v' = \{ \gamma_{t1} h_w + \gamma_{t2}' (z - h_w) \} / 10 \quad (17)$$

z : depth from ground surface (m)

h_w : depth of water table (m)

γ_{t1} : unit weight of soils (tf/m³) on water table

γ_{t2} : unit weight of soils (tf/m³) under water table

γ_{t2}' : effective unit weight of soils (tf/m³) under water table, and is given as

$$\gamma_{t2}' = \gamma_{t2} - 1.0 \quad (18)$$

Soil layers having the liquefaction resistance factors, F_L , smaller than 1.0 shall be judged to liquefy during an earthquake.

(c) Treatment of Liquefaction

If the soils underneath the bottom of underground parking garage structure is judged to develop liquefaction, countermeasures have to be considered regardless of the degree of uplift. There are basically three measures for the liquefaction as shown in Table 3. The first is to excavate the soils which liquefy during an earthquake for placing the bottom of the structure on deeper stable soil layers. The second method is to strengthen the foundation of the structure. The foundation should be designed strong enough for uplift force. Consideration should be paid to the settlement after an earthquake due to liquefaction. The third is to prevent the liquefaction by either replacing or strengthening the soils. The appropriate countermeasures have to be selected considering effectiveness and cost.

Table 3 Countermeasures for Soil Liquefaction

Place the bottom at deeper stable soil layer		
Strengthening of Foundation		Use of Piles
Prevention of Liquefaction	Increase of Soil Density	Vibro Floatation, Sand Compaction Rod Compaction
	Improvement of Grain Size or Solidification	Replacement, Injection, Deep Mixing

When there are soils which have the liquefaction resistance factor $F_L < 1.0$ at side of the underground parking garage as shown in Fig. 10, the uplift of the structure has to be checked even if the soil layers under the bottom of the structure are stable against liquefaction. This is because the pore water pressure at the side of the structure tends to propagate to result in the

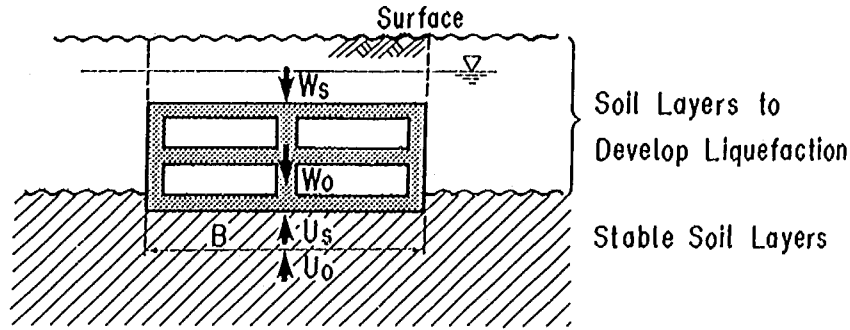


Fig. 10 Check against Uplift due to Soil Liquefaction developed at Side of Structure

propagate to result in the increase of the pore water pressure at the bottom of the structure. It is usual to have water-path due to disturbance during excavation. The safety against uplift shall be examined as

$$F_s = \frac{W_s + W_B}{U_s + U_D} \quad (19)$$

where,

F_s : safety factor against uplift

W_s : weight of overburden soils (tf)

W_B : weight of underground parking garage (tf)

U_s : uplift force due to static hydraulic pressure (tf)

U_D : uplift force due to increase of pore water pressure associated with liquefaction (tf), and shall be evaluated as

$$U_D = \Delta u \cdot A = L_u \cdot \sigma_v' \cdot A \quad (20)$$

Δu : pore water pressure (tf/m²)

A : sectional area of bottom of structure (m²)

L_u : pore water pressure ratio ($L_u = \Delta u / \sigma_v'$)

σ_v' : effective overburden pressure (tf/m²) at surface of stable soil layer

If the safety factor F_s is less than 1.0, measures against the uplift have to be taken.

Dynamic Response Analysis

Dynamic response analysis is recommended for the underground parking garage structure with complex structural response. In the dynamic response

analysis, design acceleration response spectrum at the design base rock is given as

$$S = c_z \cdot c_D \cdot S_0 \quad (21)$$

where,

S : design acceleration response spectrum (cm/sec) at design base rock

c_z : modification factor for zone (refer to Fig. 5)

c_D : modification factor for damping ratio, and is given as

$$c_D = \frac{1.5}{40h_i + 1} + 0.5 \quad (22)$$

h_i : modal damping ratio of critical for i-th mode

S_0 : standard acceleration response spectrum at design base rock, and is given by Fig. 11

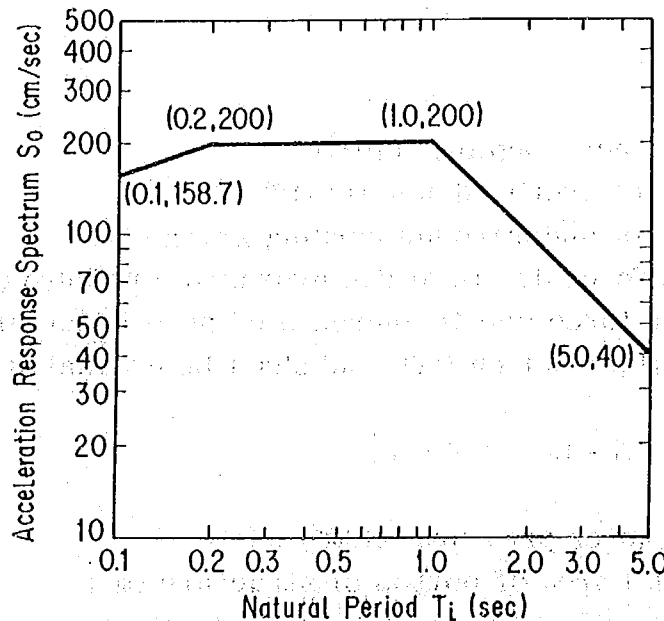


Fig. 11 Standard Acceleration Response Spectrum for Dynamic Response Analysis

Input accelerations for time response analysis shall be either selected from strong motion records which have the close response spectral characteristics with S by Eq. (22) or obtained by adjusting the strong motion records in frequency domain so that the acceleration response spectrum be matched with S by Eq. (22).

STRUCTURAL DETAILS

In addition to design calculation, attention should be paid to structural details. Table 4 summarizes the major structural details. Special attention is placed on the amount of tie and stirrup reinforcement.

Table 4 Major Structural Details

Beam	① $A_s \geq 0.005bd$ where A_s: reinforcement in tension (cm²) b: height of beam (cm) d: effective width of beam (cm) ② Interval of stirrup should be less than $1/2d$ or 25cm. ③ Stirrup ratio $\geq 0.2\%$
Column	① Diameter of main reinforcement $\geq 13\text{mm}$ ② Number of main reinforcement ≥ 4 ③ Main reinforcement ratio $\geq 0.8\%$ ④ Diameter of tie reinforcement $\geq 13\text{mm}$ ⑤ Interval of tie reinforcement $\leq 1/2d$ $\leq 15\text{mm}$ $\leq 12\phi$ where d: smaller size (width) of column ϕ: diameter of main reinforcement ⑥ Tie reinforcement ratio $\geq 0.2\%$ ⑦ Minimum width of column $\geq 30\text{cm}$
Beam-Column Connection	① At column between ± 2 times larger width of column from the beam-column connection a) Interval of tie reinforcement $\leq 10\text{cm}$ b) Tie reinforcement ratio $\geq 0.3\%$ ② At beam between ± 2 times height of beam from the beam-column connection a) Interval of stirrup $\leq 20\text{cm}$ b) Stirrup ratio $\geq 0.3\%$
Slab	① Thickness $\geq 20\text{cm}$ ② Diameter of reinforcement $\geq 13\text{mm}$ ③ Interval of main reinforcement $\geq 125\text{mm}$ ④ Interval of sub reinforcement $\geq 200\text{mm}$
Structural Wall	① Thickness $\geq 20\text{cm}$ $\geq 1/30 \times \text{Height}$ ② Double-sided reinforcement ③ Reinforcement ratio $\geq 0.25\%$ ④ Interval of reinforcement $\geq 30\text{cm}$

CONCLUDING REMARKS

The preceding pages presented the outline of the seismic design method of

reinforced concrete underground structure for parking garage based on the Guide Specifications of Design and Construction of Park Garage issued by the Ministry of Construction in March 1992. Although there are various specifications which describe seismic design method of underground structures of road, most of them are for specific projects. This is the third specifications, following the petroleum pipe line⁶⁾ and common utility ducts⁷⁾, which comprehensively specifies seismic design method of underground facilities of road. Because seismic safety is one of the most critical issues, it is expected that the specifications contribute for design and construction of underground parking garage with enough seismic safety.

ACKNOWLEDGEMENT

Drafting of the Guide Specifications was made in a Working Group formulated in the Subcommittee of Parking Garage, Japan Road Association.

REFERENCES

- 1) Japan Road association : Guide Specifications of Design and Construction of Parking Garage, 1992 (in Japanese)
- 2) Ministry of Construction : New Seismic Design Methods, Technical Note, No. 1185, Public Works Research Institute, March 1977 (in Japanese)
- 3) Public Works Research Institute : Guidelines for Seismic Design Methods of Underground Structures, Technical Note, March 1992 (in Japanese)
- 4) Asakura, H., Kawashima, K. and Sugita, H. : Guidelines for Seismic Design Methods of Large Underground Structures, Proc. 24th Joint Meeting, Panel on Wind and Seismic Effects, UJNR, Gaithersburg, Maryland, U.S.A., May 1992
- 5) Japan Road Association : Part V Seismic Design , Specifications of Design of Highway Bridges, 1990
- 6) Japan Road Association : Technical Standard for Petroleum Pipeline, June 1971 (in Japanese)
- 7) Japan Road Association : Design Method of Common Utility Ducts, March 1986 (in Japanese)

Seismic Strengthening and Retrofitting
of Highway Bridges in Japan

Kazuhiko KAWASHIMA

and

Shigeki UNJOH

5th U. S. -Japan Workshop on
Earthquake Disaster Prevention for Lifeline Systems

Tsukuba, Japan

October 1992

THE UNIVERSITY OF CHICAGO

DEPARTMENT OF CHEMISTRY

RESEARCH REPORT

1960

NO. 100

RESEARCH REPORT

DEPARTMENT OF CHEMISTRY

1960

NO. 100

SEISMIC STRENGTHENING AND RETROFITTING OF HIGHWAY BRIDGES IN JAPAN

Kazuhiko KAWASHIMA and Shigeki UNJOH

Earthquake Engineering Division, Public Works Research Institute

SUMMARY

Presented are current technical developments for seismic strengthening and retrofitting of highway bridges in Japan. Seismic inspections, history of provision of seismic design codes, and seismic strengthening methods for highway bridges and the applications are described with emphasis on practical applications of strengthening and retrofitting.

INTRODUCTION

Japan is one of the most seismically disastrous countries in the world and has often suffered significant damages from large earthquakes. More than 3,000 highway bridges have suffered damages in the past earthquakes since the Kanto Earthquake of 1926. It should be noted here that although there were many bridges which suffered damages due to earthquakes, number of bridges which fell down was only 15 except number of bridges which fell down by fires. Since the damage to highway bridges loses functions as an emergency evacuation route and a transportation route for emergency relief goods and it takes a long time to be repaired, it significantly affects social and economical activities in the area. Therefore, it is important to provide appropriate seismic resistance for highway bridges.

This paper presents the factors which affect seismic vulnerability of highway bridges and the seismic strengthening and retrofitting for the substructures with emphasis on the practical applications.

DETAILS OF THE PAST SEISMIC INSPECTIONS OF HIGHWAY BRIDGES IN JAPAN

The Ministry of Construction has made several seismic inspections of highway bridges throughout the country since 1971 as a part of comprehensive earthquake disaster prevention measures for highway facilities. Seismic strengthening work for vulnerable highway bridges has been successively made according to these seismic inspections. Table 1 shows the history of past seismic inspections of highway bridges in Japan.¹⁾⁻²⁾

Table 1 Details of the Past Seismic Inspections of Highway Bridges in Japan

Year	Highways Inspected	Inspection Items	Number of Bridges		
			Inspected	Require Strengthening	Strengthened
1971	All Sections of National Expressways and National Highways, and Sections of the Others (Bridge Length ≥ 5 m)	① Deterioration ② Bearing Seat Length S for Bridges supported by Bent Piles	18,000	3,200	1,500
1976	All Sections of National Expressways and National Highways, and Sections of the Others (Bridge Length ≥ 15 m or Overpass Bridges)	① Deterioration of Substructures, Bearing Supports and Girders/Slabs ② Bearing Seat Length S and Devices for Preventing Falling-off of Superstructure	25,000	7,000	2,500
1979	All Sections of National Expressways, National Highways and Principal Local Highways, and Sections of the Others (Bridge Length ≥ 15 m or Overpass Bridges)	① Deterioration of Substructures and Bearing Supports ② Devices for Preventing Falling-off of Superstructure ③ Effect of Soil Liquefaction ④ Bearing Capacity of Soils and Piles ⑤ Strength of RC Piers ⑥ Vulnerable Foundations (Bent Pile and RC Frame on Two Independent Caisson Foundation)	35,000	16,000	13,000
1986	All Sections of National Expressways, National Highways and Principal Local Highways, and Sections of the Others (Bridge Length ≥ 15 m or Overpass Bridges)	① Deterioration of Substructures, Bearing Supports and Concrete Girders ② Devices for Preventing Falling-off of Superstructure ③ Effect of Soil Liquefaction ④ Strength of RC Piers (Bottom of Piers and Termination Zone of Main Reinforcement) ⑤ Bearing Capacity of Piles ⑥ Vulnerable Foundations (Bent Piles and RC Frame on Two Independent Caisson Foundation)	40,000	11,800	6,000 (As of the End of 1989 Fiscal Year)

Note) Number of bridges inspected, number of bridges that required strengthening and number of bridges strengthened are approximate numbers.

The first seismic inspection was made in 1971 in order to strengthen and promote earthquake disaster prevention measures in Japan. The damage experiences of highway bridges caused by the San Fernando Earthquake (Magnitude = 6.6) close to Los Angeles, USA, in February, 1971 was a turning point of the initiation of the inspection. Highway bridges with span length greater than or equal to 5m on all sections of national expressways and national highways, and sections of the others were inspected in which attention was paid to detect deterioration such as cracks of reinforced concrete structures, tilting, sliding, settlement and scouring of foundations, etc. Approximately 18,000 highway bridges in total were inspected and approximately 3,200 bridges were found to require retrofitting.

Following the first seismic inspection, it has been subsequently made in 1976, 1979 and 1986, with gradually expanding highways inspected and inspection items. The latest inspection of 1986 was made with the increase of social needs to insure seismic safety of highway traffic after the damage experiences caused by the Urakawa-oki Earthquake of 1982 and the Nihon-kai-chubu Earthquake in 1983. The highway bridges with span length greater than or equal to 15m on all sections of national expressways, national highways and principal local highways, and sections of the other, and overpass bridges were inspected. The inspection items included deterioration, devices for preventing falling-off of superstructure, strength of substructures and stability of foundations.

A series of these seismic strengthening works, approximately 23,000 highway bridges were successively strengthened by the end of 1989 fiscal year.

SEISMIC DAMAGE TO HIGHWAY BRIDGES AND PROVISIONS OF SEISMIC DESIGN CODES

It was in 1927, which was the next year of the Kanto Earthquake in 1926, to initiate introducing the seismic effect in the design of highway bridges. The Civil Engineering Bureau of the Ministry of Internal Affairs in those days notified "the method of seismic design of abutments and piers". The seismic design method has been developed and improved with progress of comprehensive technology on earthquake engineering and a number of damage experiences. In particular, the seismic design technology was integrated and summarized by the establishment of "Specifications for Seismic Design of Highway Bridges" in 1971, which independently provided seismic design issues. Furthermore, it was revised in 1980 and integrated in "Design Specifications of Highway Bridges" as "Part V : Seismic Design." And the "Part V : Seismic Design" was revised in 1990 and is currently in practice. In the latest specification for seismic design, design seismic loads, bearing capacity of reinforced concrete pier for lateral force, soil liquefaction, design details in seismic design and so on are prescribed and it may be an unique specification for seismic design in the world. It should be noted here that the remarkable attentions were paid to the seismic design of highway bridges after 1971.

Table 2 shows the provisions of design issues for the seismic design method of highway bridges. In this table, the design issues for seismic design of substructures are shown.

Table 2 History of Seismic Design Methods for Highway Bridges in Japan

		1926 Details of Road Structure (draft), Road Law, MIA	1939 Design Specifications of Steel Highway Bridges (draft), MIA	1956 Design Specifications of Steel Highway Bridges, MOC	1964 Design Specifications of Substructures (Pile Foundations), MOC	1964 Design Specifications of Steel Highway Bridges, MOC	1966 Design Specifications of Substructures (Survey and Design), MOC	1968 Design Specifications of Substructures (Piers and Direct Foundations), MOC	1970 Design Specifications of Substructures (Caisson Foundations), MOC	1971 Specifications for Seismic Design of Highway Bridges, MOC	1972 Design Specifications of Substructures (Cast-in-Piles), MOC	1975 Design Specifications of Substructures (Pile Foundations), MOC	1980 Design Specifications of Highway Bridges, MOC	1990 Design Specifications of Highway Bridges, MOC			
Seismic Loads	Seismic Coefficient	Largest Seismic $k_s = 0.2$		$k_s = 0.1 \sim 0.35$		$k_s = 0.1 \sim 0.3$											
		varied dependent on the site		varied dependent on the site and ground Condition		Standardization of Seismic Coefficient Provision of Modified Seismic Coefficient Method									Revision of Application range of Modified Seismic Coefficient Method	$k_s = 0.1 \sim 0.3$ Integration of Seismic Coefficient Method and Modified one.	
	Dynamic Earth Pressure	Equations proposed by Mononobe and Okabe were supposed to be used.					Provision of Dynamic Earth Pressure										
	Dynamic Hydraulic Pressure	Less Effect on Piers except High Piers in Deep Water.					Provision of Hydraulic Pressure										
							Provision of Dynamic Hydraulic Pressure.										
Reinforced Concrete Column	Bending at Bottom	Supposed to be designed in a similar way provided in current Design Specifications					Provisions of Definite Design Method										
	Shear	Less Effect on RC Piers except those with Smaller section area such as RC Frame and Hollow Section					Check of Shear Strength									Provision of Definite Design Method, Decreasing of Allowable Shear Stress	
	Termination of Main Reinforcement at Mid-Height															Elongation of Anchorage Length of Terminated Reinforcement at Mid-Height	
	Bearing Capacity for Lateral Fore						Less Effect on RC Piers with Larger Section Area									Ductility Check	Check of Bearing Capacity for Lateral Force
Footing							Provisions of Definite Design Method (Designed as a Cantilever Plate)									Provisions of Effective width and Check of Shear Strength	
Pile Foundation		Bearing Capacity in vertical direction was supposed to be checked.		Provisions of Definite Design Method (Bearing Capacity in vertical and horizontal directions)							Provisions of Design Details for Pile Head						
Direct Foundation		Stability (Overturning and Slip) was supposed to be checked.		Provisions of Definite Design Method (Bearing Capacity, Stability Analysis)							Special Condition (Foundation on Slope, Consolidation Settlement, Lateral Movement)						
Caisson Foundation		Supposed to be designed in a similar way provided in Design Specification of Caisson Foundation of 1969					Provisions of Definite Design Method										
Soil Liquefaction							Provisions of Soil Layers of which Bearing Capacity shall be ignored in seismic design					Provisions of Evaluation Method of Soil Liquefaction and the treatment in seismic design					Consideration of effect of fine sand content
Bearing Support	Bearing Support	Provisions of Design Methods for Steel Bearing Supports (Bearing, Roller, Anchor Bolt)					Provision of Transmitting Method of Seismic Load at Bearing										
	Devices Preventing Falling-off of Superstructure						Provision of Bearing Seat Length S		Provisions of Stopper at Movable Bearings, Devices for Preventing Superstructure from Falling (Seat Length S , Connection of Adjacent Decks)			Provisions of Stopper at Movable Bearings, Devices for Preventing Superstructure from Falling (Seat Length S , Devices)					

FACTORS WHICH AFFECT SEISMIC VULNERABILITY OF HIGHWAY BRIDGES AND FEASIBILITY OF SEISMIC STRENGTHENING

Table 3 summarizes the factors which affect seismic vulnerability of highway bridges and feasibility of seismic strengthening which was obtained through the statistical analyses of the damaged bridges in the past earthquakes in Japan.^{10) 11)} According to the results, 16 factors in total are significantly associated with the damages of highway bridges, in which 8 factors of ① design specifications, ⑥ devices for preventing falling-off of superstructure, ⑨ ground condition, ⑫ effect of scouring, ⑬ materials of substructures, ⑭ type of foundations, ⑮ intensity of ground motion, and ⑯ effect of termination of main reinforcement at mid-height significantly affect seismic performance of highway bridges.

From the viewpoint of seismic strengthening, these 8 factors can be categorized as follows :

- 1) ⑥ devices for preventing falling-off of superstructure
→ Installation of devices
- 2) ⑦ type of substructure, ⑩ effect of soil liquefaction, and ⑫ type of foundation
→ Improvement of seismic strength of foundations
- 3) ⑬ type of substructure, ⑭ materials of substructure, and ⑯ effect of termination of main reinforcement at mid-height
→ Improvement of seismic strength of substructures
- 4) ⑮ intensity of ground motion
→ Distribution and decrease of seismic loads

Fig.1 shows the typical and effective seismic strengthening measures for the above categories.

SEISMIC STRENGTHENING METHODS OF FOUNDATIONS AND THE APPLICATIONS

Seismic strengthening of foundations are required in cases of 1) weak bearing capacity of soils induced by soil liquefaction, 2) weak bearing capacity of soils by scouring, and 3) vulnerable foundations which were built of bricks and masonry without sufficient consideration for seismic design.

In the above items, seismic strengthening measures to soil liquefaction is becoming the most important a seismic strengthening one. There are two measures to mitigate the effect of soil liquefaction, which include measures to prevent or mitigate occurrence of soil liquefaction itself, and measures to resist the effect of soil liquefaction by strength of foundations. The former measures include increase of strength of soils against soil liquefaction and replacement of liquefiable soils by non-liquefiable ones. On the other hand, in order to resist the effect of soil liquefaction by the latter measures, foundations have to be supported by non-liquefiable layer so as to prevent the settlement and tilting.

**Table 3 Factors which Affect Seismic Vulnerability of Highway Bridges
and Feasibility of the Seismic Strengthening**

Items	Seismic Vulnerability	Feasibility of Seismic Strengthening	Principles of Countermeasures
① Design Specifications	Those designed in accordance with 1926 or 1939 Specifications have higher vulnerability	○	Strengthening of inadequate members from the viewpoint of current Design Level
② Type of Superstructure	· Gerber or simply supported girders with 2 or more spans have higher vulnerability · Arch, frame, continuous girders, cable-stayed bridges or suspension bridges have lower vulnerability	×	_____
③ Shape of Superstructure	Skewed or curved bridges do not necessarily have higher vulnerability than straight bridges	×	_____
④ Materials of Superstructure	Reinforced concrete bridges or prestressed concrete bridges have lower vulnerability than steel bridges although the difference is small	×	_____
⑤ Slope in Bridge Axis	Bridges with slope in bridge axis have higher vulnerability	×	_____
⑥ Device for Preventing Falling-off of Superstructure	Bridges with devices for preventing falling-off of superstructure have lower vulnerability	○	Installation of Devices
⑦ Type of Substructure	Bridges supported by single-line bent piles or by reinforced concrete frame placed on two separate caisson foundations have higher vulnerability	○	Strengthening of Substructures
⑧ Height of Piers	Bridges supported by higher piers have higher vulnerability	×	_____
⑨ Ground Condition	Bridges constructed on soft soil have higher vulnerability	×	_____
⑩ Effect of Soil Liquefaction	Bridges constructed on sandy soil layers susceptible to liquefaction have higher vulnerability	○	Strengthening of Foundations or Strengthening of Surrounding Soils
⑪ Irregularity of Supporting Soil Condition	Bridges constructed on soils with irregularity of supporting conditions have higher vulnerability	×	_____
⑫ Effect of Scouring	Bridges where the surface soils are scoured have higher vulnerability	○	Treatment for Prevention of Scouring or Strengthening of Foundations
⑬ Materials of Substructures	Bridges supported by plane-concrete substructures designed in accordance with 1926 or 1939 specifications have higher vulnerability	○	Strengthening Substructures, Renewal of Substructures
⑭ Type of Foundation	Bridges supported by timber, brick, masonry or other old unknown type substructures have higher vulnerability	○	Strengthening of Foundations
⑮ Intensity of Ground Motion	Bridges subjected to higher intensity of ground acceleration have higher vulnerability. In particular, vulnerability becomes quite high when the bridges are subjected to peak ground acceleration larger than 400 gal (0.4g)	○	Application of Base-Isolation
⑯ Effect of Termination of Main Reinforcement at Mid-height	Bridges with RC Piers which do not have enough anchorage length for terminated reinforcement have higher vulnerability	○	Strengthening of Substructures

Note ○ Items for which seismic strengthening is feasible
 × Items for which seismic strengthening is not feasible

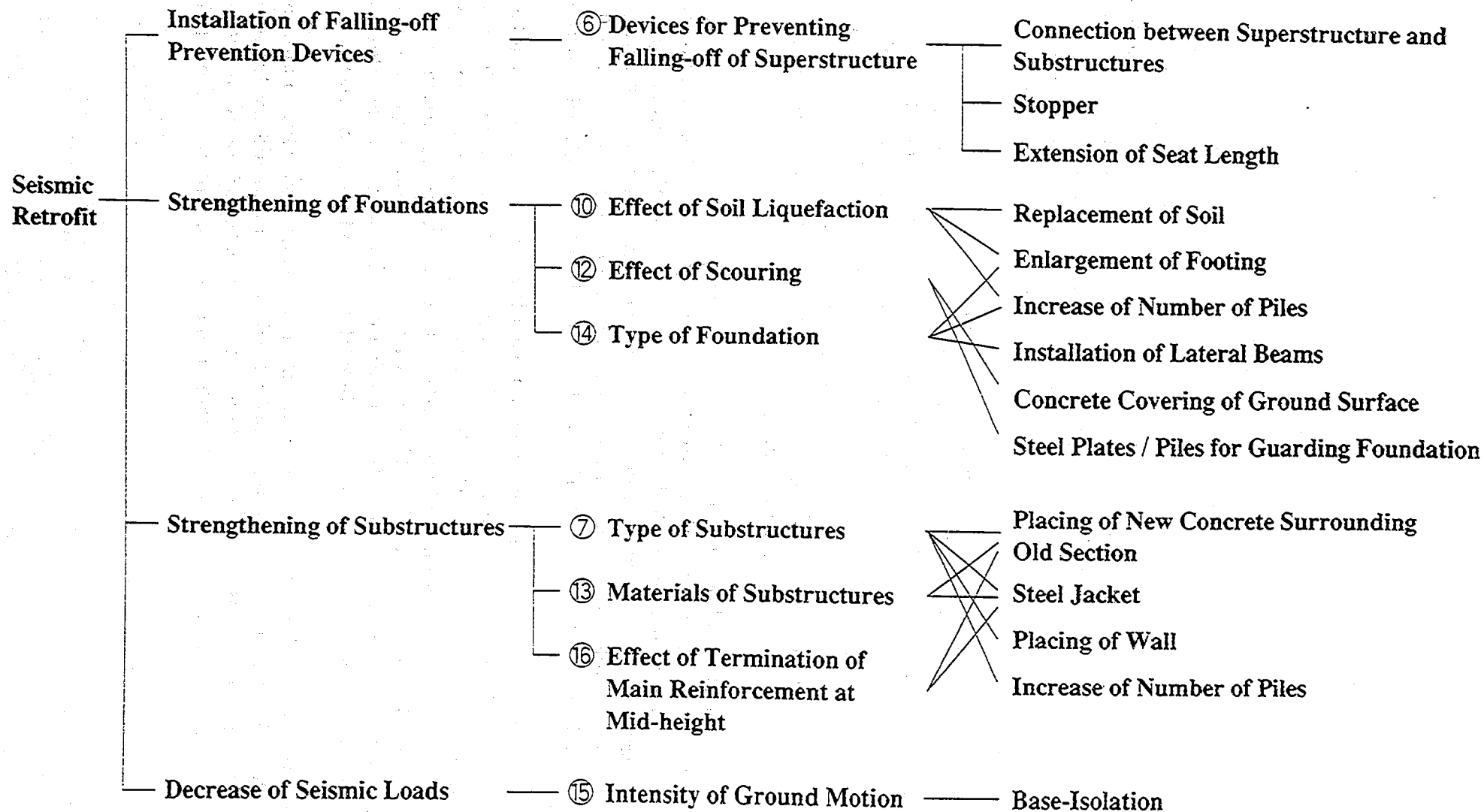


Fig.1 Selection of Seismic Strengthening Measures for Highway Bridges

Although the selection of the above two measures depends on the ground conditions and structural conditions of each site, the strengthening measures by increasing strength of foundations may generally be reliable and economical.

Table 4 and Table 5 show the seismic strengthening methods for abutment foundations and pier foundations, respectively.¹²⁾

Two strengthening practices to soil liquefaction are shown as follows:¹³⁾

A SEISMIC STRENGTHENING PRACTICE BY INCREASING NUMBER OF PILES

The strength of piers and piles as shown in Fig.2 were found to be sufficient, but the bearing capacity of soils became vulnerable when the soil liquefaction occurred. Three strengthening methods by ①ground improvement by soil cement, ②increase of number of piles in the transverse direction by using steel pipe piles, and ③covering of the existing foundations by steel sheet piles were investigated and compared as a strengthening measure. Since Method ① was relatively expensive, Method ② was finally adopted as a result of comparison of Methods ② and ③ with consideration of construction method, economy and reliability.

In the strengthening design, it was assumed that the existing piles resisted seismic load up to the designed level and that the force beyond the design force was supported by newly constructed piles. Since the pier to be strengthened was on the high water channel, the temporary road to the pier was provided and steel pipe pile was placed by using a vibro hammer. The new footing was constructed by shutting-up by steel sheet piles from water around the pier. Integration between existing and new footings were made by anchoring reinforcing bars and injecting resin mortar.

A SEISMIC STRENGTHENING PRACTICE BY CONNECTING BEAMS

Fig.3 shows a strengthening practice of 11-span simply-supported pretension concrete slab bridge. Reinforced concrete bent piles were used for piers and abutments. It was found that the liquefaction resistance factor, F_L , of the layer with thickness of 3.2m under the river bed was less than 1.0 and pile length which was supported by the stable soil layer was 1/3 of total pile length of 12m.

Three strengthening measures by ①strengthening of all 12 substructures, ②strengthening of one substructure per two spans, and ③strengthening of both abutments and connecting all pier crests by continuous beams were investigated as a strengthening method. Alternative methods by ④replacement of liquefiable soil layer by non-liquefiable soils and ⑤chemical grouting were also investigated. As a result of the investigation of these measures, Method ③ was selected with consideration of reliability, construction method and economy.

In the strengthening design, it was assumed that existing bent piles supported the vertical load and that seismic loads was supported by integrated structure of new and existing ones. Since pier crests of bent piles were strengthened by connecting beams, it was required to insure appropriate

Table 4 Seismic Strengthening Methods for Abutment Foundations

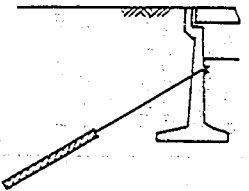
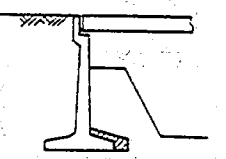
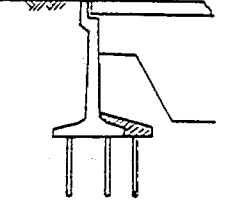
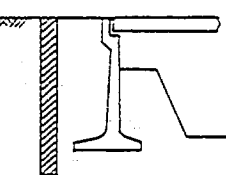
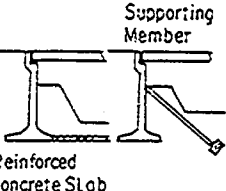
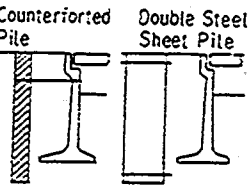
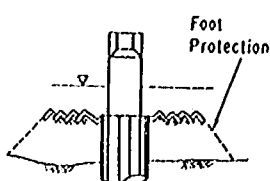
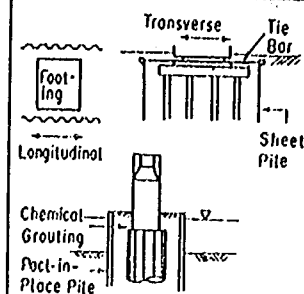
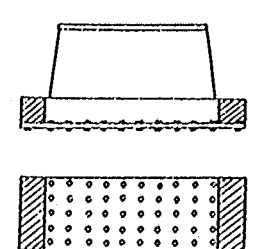
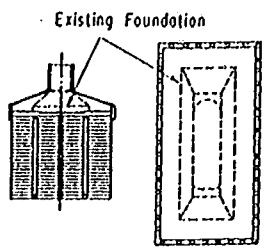
Method		① Earth Anchor	② Enlargement of Footing	③ Increase of Number of Piles	④ Piles for decreasing earth pressure	⑤ Others (Measures in the front of Foundation)	⑥ Others (Measures in the back of Foundation)
Illustration							
Outline		Increasing resistance to sliding and overturning by earth anchor constructed from the abutment wall to the ground.	Excavating ground in the front of footing and enlarging the footing.	Excavating ground in the front of footing, constructing piles and enlarging the footing.	Constructing piles in the back-filling ground and decreasing earth pressure to the foundation.	Increasing resistance to sliding and overturning by reinforced concrete slab or supporting member.	Decreasing earth pressure by counterforted pile, double steel sheet pile, improvement of ground, etc.
Strengthening Item		· Bearing Capacity of Soils. · Displacement of Foundation.	· Bearing Capacity of Soils	· Bearing Capacity of Soils · Bearing Capacity of Piles	· Displacement of Foundation	· Bearing Capacity of Soils	· Displacement of foundation
Points to be considered in the Design		Share of loads between earth anchor and existing footing	Connection of new and existing footing	Share of loads between new and existing piles. Connection of new and existing footing	· Share of earth pressure between piles and abutment	· Connection of new and existing structures	· Share of earth pressure between new and existing structures.
Points to be considered in the Construction	Traffic Regulation, Construction Space	Construction can be made by relatively small-scale construction machines. Clear headway under the girder at least up to 3m is necessary.	Construction space can be small.	Construction method for placing piles is restricted because of limited construction space. Cast-in-place pile (reverse circulation drill method) and steel pipe pile with joints can be applicable.	Traffic regulation is necessary during construction	Construction space can be small.	Traffic regulation is necessary during construction.
	Cofferdam	Construction of cofferdam is necessary but may be small-scale.	Construction of cofferdam by sheet pile is necessary. Special construction method under the girder using short sheet piles with joints is necessary.	same as ②	not necessary	same as ②	not necessary
	Effect on Existing Structure	Attention should be paid to cut of reinforcing bars of abutment wall. Strength of fixing section of earth anchor should be checked.	Stability of foundation should be checked during construction.	Stability of foundation during construction should be checked. Placing point of piles is limited if battered pile is used in the existing piles.	Decrease of bearing capacity of soils by soil relaxation should be checked in case of constructing cast-in-place piles	same as ① and ②	same as ④
	Effect on Surrounding Structures	Attention should be paid to underground structures in the back of abutment.	—————	—————	same as ①	—————	same as ①
Commentary		· Earth anchor method has been applied in many practices since the construction cost is cheaper. · Careful consideration to effectiveness of strengthening should be given because the back ground may move during earthquake. · Anchorage portion should be leveled under the bottom of footing.	· Reliable construction for connecting new and existing footings is necessary. · By installation of projection in the bottom of footing increase of bearing capacity to lateral loading is possible.	· Reliable construction for connecting new and existing footings is necessary.	—————	· Although increase of resistance to lateral force is reliable, this method can be applicable only for short span bridges.	—————

Table 5 Seismic Strengthening Methods for Pier Foundations

Method	① Foot Protection	② Cofferdam	③ Increase of Number of Piles	④ Cast-In-Site Diaphragm Wall
Illustration				
Outline	Protecting foundation from scouring by blocks, cover concrete and mattress shaped gabion	Constructing cofferdam by sheet pile or pact-in-place pile around foundation and filling soils inside. Chemical grouting and prepacked concrete can be used for soil filling.	Increasing number of piles and enlarging the footing	Constructing cast-in-site diaphragms wall and integrating new structure with existing foundation
Strengthening Item	• Scouring	• Scouring • Bearing Capacity of Piles	• Bearing Capacity of Soils • Bearing Capacity of Piles	• Bearing Capacity of Soils • Scouring
Points to be considered in the Design	• Strengthening Area • Can not be a permanent strengthening method	• Blocking of drainage area of river	• Reliable effectiveness of strengthening • Share of loads between new and existing structures • Connection between new and existing structures	• Reliable effectiveness of strengthening • Blocking of drainage area of river • Effect of water flow pressure
Points to be considered in the Construction	• Relatively easy construction	• Construction machine for sheet piles may be restricted by limited construction space.	• Relatively easy for constructing piles in transverse direction • Construction machine is restricted for constructing piles under the girder • Cofferdam and dry-up construction is necessary for connection of new and existing footings.	• Appropriate construction method for ground condition should be selected.

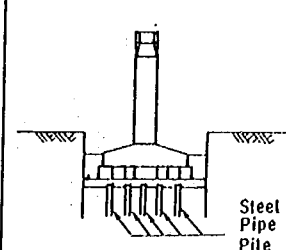
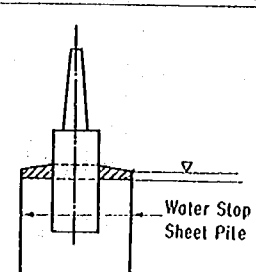
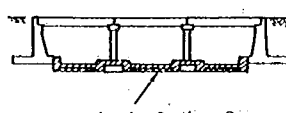
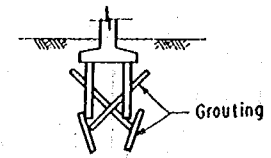
Method	⑤ Strengthening by Steel Pipe Pile	⑥ Enlargement of Footing	⑦ Cast-in-site Beam, Reinforced Concrete Plate	⑧ Foundation Improvement
Illustration				
Outline	Strengthening by constructing steel pipe piles under existing footing	Constructing footing at the design ground surface and resisting to moment and lateral loads	Increasing stability of foundations by connecting adjacent foundations by cast-in-site beam or reinforced concrete plate. Applicable for short span bridges.	Increasing bearing capacity of soft soils by placing quicklime pile, sand compaction pile, gravel drain, chemical grouting, soil cement, etc.
Strengthening Item	• Bearing Capacity of Foundation • Bearing Capacity of Soils	• Bearing Capacity of Soils • Scouring	• Bearing Capacity of Soils • Displacement of Foundation.	• Bearing Capacity of Soils • Effect of Soil Liquefaction
Points to be considered in the Design	_____	• Share of loads between new and existing structures. • Connection of new and existing structures	_____	• Evaluation of effectiveness of foundation improvement • Improvement area
Points to be considered in the Construction	• Construction of cofferdam and placement of pile under foundation is generally difficult. • Construction cost is relatively expensive.	• Construction of cofferdam is necessary	• Construction of cofferdam under the girder is necessary. • In case of river crossing bridge, transference of river course is necessary.	_____

Fig.2 Seismic Strengthening of A Foundation on Liquefiable Soils by Increasing Number of Piles

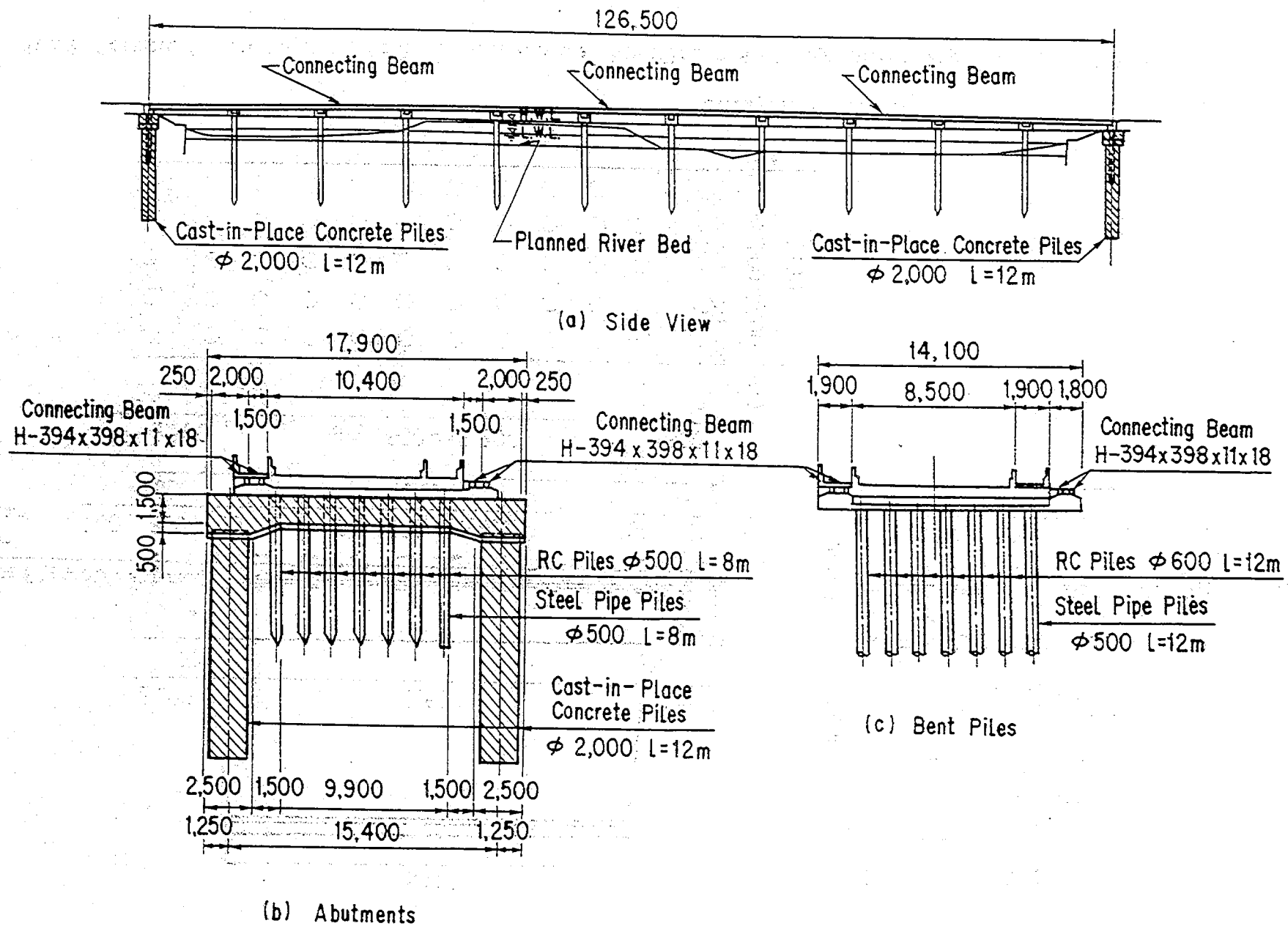


Fig.3 Seismic Strengthening of A Bent Pile on Liquefiable Soils by Connecting Beams

strength for the connecting beams. Therefore, the connecting beams were designed as a device for preventing falling-off of superstructure. The connecting beams and fix bearing on the lateral beams were designed using two times horizontal seismic coefficient. It should be noted here that the connecting beams were also used as girders of pedestrian decks.

SEISMIC STRENGTHENING METHODS OF PIERS AND THE APPLICATIONS

There are two cases that required strengthening of substructures, which include vulnerable type and materials of substructures, and weak strength of reinforced concrete piers at the bottom and termination zone of main reinforcement at mid-height.

Type of foundations vulnerable to seismic damage includes single-line bent piles and reinforced concrete frame on two independent caisson foundation. Substructures made of plain concrete, brick and masonry which were constructed in the old ages are also vulnerable to brittle failure.

On the other hand, substructures which have weak strength at the bottom include the cases in which the design seismic force is smaller than current design level or arrangement of the reinforcement is not adequate. The strength at the termination zone of main reinforcement at mid-height had not generally checked before 1980. When the anchorage length of terminated reinforcement is not enough long, brittle failure at the zone is anticipated similarly to the damage of the piers of the Shizunai Bridge during the Urakawa-oki Earthquake of 1982.

There are two strengthening methods for such weak reinforced concrete piers, which include placement of new reinforced concrete or steel jackets. In both cases, it is important to integrate new and existing structures. Recently, jacketing method by using new materials such as carbon fiber and fiber reinforced plastic are also being studied.

Table 6 and Table 7 show the seismic strengthening methods for vulnerable bent piles and reinforced concrete piers.¹²⁾

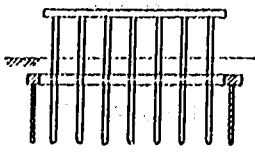
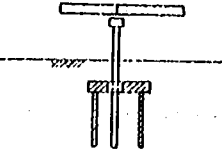
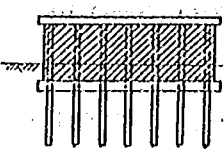
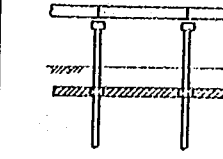
The seismic strengthening practices of substructures are shown as follows.¹³⁾

A SEISMIC STRENGTHENING PRACTICE BY PLACING NEW REINFORCED CONCRETE (CASE 1)

Fig.4 shows a seismic strengthening practice for increasing ductility of a reinforced concrete pier by placing new reinforced concrete. The cover concrete of the existing pier was chipped-out up to the surface of longitudinal reinforcing bars, anchoring reinforcing bars were newly installed and new reinforced concrete with thickness of 17cm was placed.

The longitudinal reinforcing bars were anchored with necessary anchorage length and anchorage portion was inclined toward the pier center in order to anchor them to the existing footing.

Table 6 Seismic Strengthening Methods for Bent Piles

Method		① Increase of Number of Piles in Transverse Direction	② Increase of Number of Piles in Longitudinal Direction	③ New Reinforced Concrete Section	④ Connection of Adjacent Piers by Cast-in-slab Beam
Illustration					
Outline		Constructing piles outside of deck	Constructing piles in longitudinal direction under the girder and connecting new and existing piles by new footing	Placing new reinforced concrete and integrating new and existing structures	Connecting adjacent piers and/or abutment by cast-in-slab beam
Strengthening Item		Type of Substructure (Bent Pile) (Bearing Capacity of Pile)	Type of Substructure (Bent Pile) (Bearing Capacity of Pile)	Type of Substructure (Bent Pile) (Strength of Pile, Stiffness of Pile)	Type of Substructure (Bent Pile) (Bearing Capacity of Pile)
Points to be considered in the Design		Connection of new and existing section	same as ①	Check of pile stress caused by increase of weight	Connection of beam and existing piles
Points to be considered in the Construction	Traffic Regulation, Construction Space	not necessary in general. When construction of piles is made from the deck, partial traffic regulation is necessary.	Construction method is restricted for placing piles under the girder.	Construction space is relatively small.	same as ①
	Cofferdam	Cofferdam is necessary for constructing footing. Special construction method is required for constructing sheet pile under the girder.	same as ①	Construction of cofferdam under the girder is necessary. same as ①	Construction of cofferdam under the girder is necessary. In case of river crossing bridge, transference of river course is necessary.

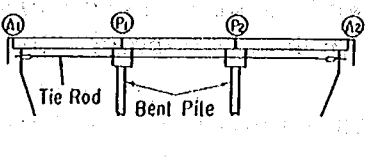
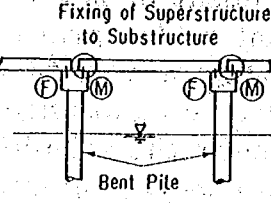
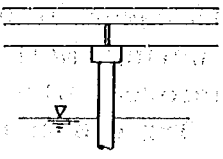
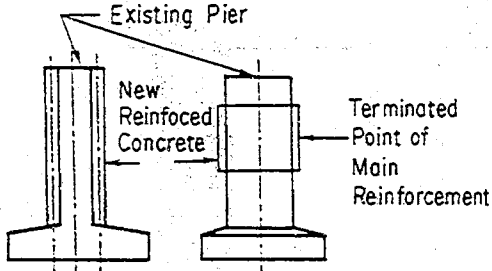
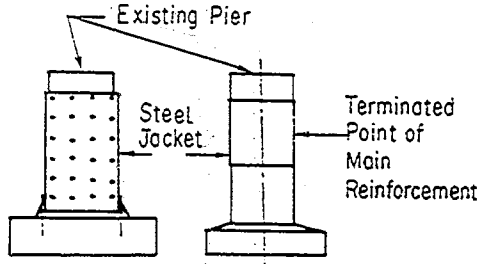
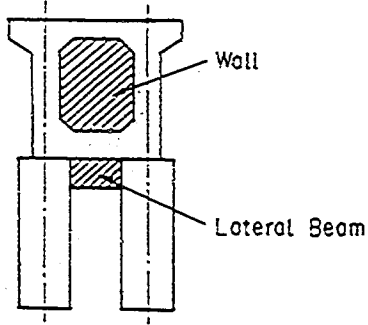
Method		⑤ Connection of Piers and Abutments by Tie-Rod	⑥ Replacement of Movable Bearing by Fixed Bearing	⑦ Connection of Deck Slab
Illustration				
Outline		Connecting piers and abutments by tie-rod and increasing stiffness in the longitudinal direction.	Fixing superstructure to substructure, and increasing stiffness in the longitudinal direction.	Connecting adjacent deck slabs and increasing drivability.
Strengthening Item		Type of Foundation (Bent Pile) (Bearing Capacity of Pile Stiffness of Pile)	Type of Foundation (Bent Pile) (Bearing Capacity of Stiffness of Pile)	Type of Foundation (Bent Pile) (Stiffness of Pile)
Points to be considered in the Design		• Design method for devices for preventing falling-off of superstructure is applicable. • Anchorage of tie-rod and anti-corrosive coating of tie-rod	• Connection between superstructure and substructure • Strengthening of fixing portion of pier crest and girder	Strengthening of connection portion of slab is necessary.
Points to be considered in the Construction	Traffic Regulation, Construction Space	not necessary	Partial traffic regulation is necessary.	Traffic regulation is necessary for removing expansion joint and replacing a part of deck slab.
	Cofferdam	not necessary	not necessary	not necessary

Table 7 Seismic Strengthening Methods for Piers

Method	① New Reinforced Concrete Section	② Steel Jacket	③ Wall and Lateral Beam
Illustration			
Outline	- Placing new reinforced concrete on existing piers - A number of practices for repairing damaged pier	Placing steel jacket on existing piers	- Construction of wall to increase lateral strength of reinforced concrete frame. - Connecting two independent caisson foundation by lateral beam for reducing the relative displacement
Strengthening Item	Strength of Pier	Strength of Pier	- Strength of Reinforced Concrete Frame - Two Independent Caisson Foundation
Points to be considered in the Design	- Anchorage of reinforcing bars to footing, chipping at cover concrete of existing pier and anchoring reinforcing bars for integration of new and existing structures - Thickness and width of new reinforced concrete - Blocking of drainage area of river	- Anchoring method of steel jacket to existing pier and footing - Thickness, width and anchor bolt density - Future maintenance (painting, etc)	Connection between new and existing structures
Points to be considered in the Construction	In case of a river crossing bridge, construction of cofferdam is necessary.	same as ①	same as ①

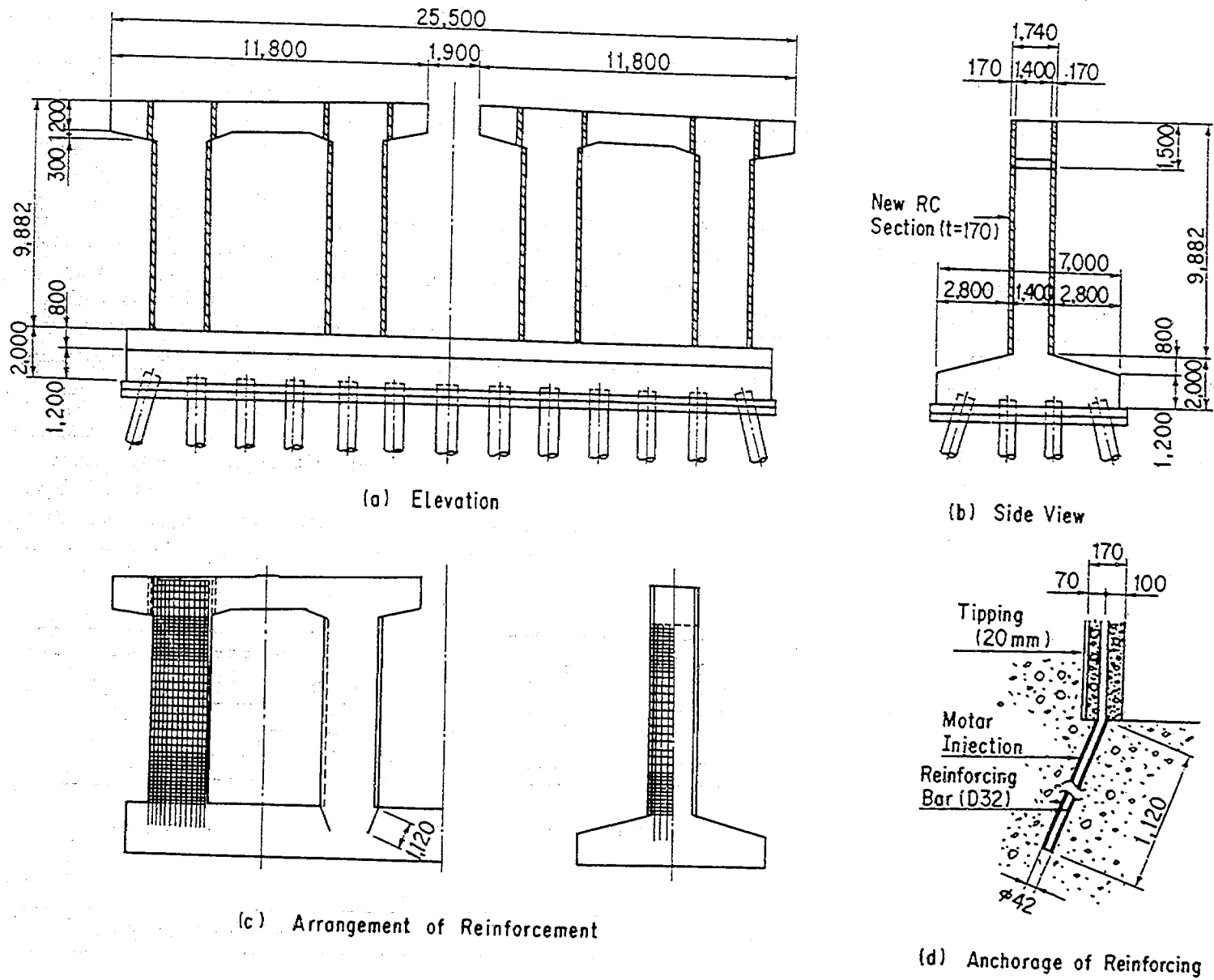


Fig.4 Seismic Strengthening of A Reinforced Concrete Frame by Placing New Reinforced Concrete

A SEISMIC STRENGTHENING PRACTICE BY PLACING NEW REINFORCED CONCRETE (CASE 2)

Fig.5 shows a seismic strengthening practice of a multi-column reinforced concrete frame pier. Three strengthening methods by ① replacement of pier, ② placement of new lateral beam, and ③ placement of new reinforced concrete so as to change a structure-type from multi-column to oval section type were investigated. Method ③ was finally selected as a result of investigation with consideration of the viewpoint of river administration.

In the strengthening design, it was assumed that the seismic force was supported by the integrated structure of new and existing ones. All of cover concrete of the existing structure was chipped-out in order to insure integration between existing and new structures. Although the weight of substructure increased, strengthening of foundation was not required because the foundation was on the stiff ground with N-value of more than 50.

A SEISMIC STRENGTHENING PRACTICE OF REINFORCED CONCRETE FRAME BY PLACING REINFORCED CONCRETE WALL

Fig.6 shows a strengthening practice of a reinforced concrete frame to increase lateral strength in the transverse direction by placing new reinforced concrete wall. Thickness of the wall was designed as 40 cm, and it was assumed that seismic loads was supported by the integrated structure of new and existing ones. Cover concrete of the connection portion of the existing structure was chipped-out and anchoring reinforcing bars were installed for reliable integration between new and existing structures.

A SEISMIC STRENGTHENING PRACTICE OF BENT PILE BY INCREASING NUMBER OF PILES

Fig.7 shows a seismic strengthening practice of a bent pile by increasing number of piles. Since the existing steel pipe piles were corroded and heavily damaged, the bent piles were strengthened by placing new reinforced concrete and the structure-type was changed to wall-type. Since the deck width was enlarged for increasing the number of traffic lanes, enlargement of pier width and construction of new piles were required.

In the strengthening design, it was assume that seismic loads was supported by the integrated structure of new and existing ones. In order to insure reliable integration between new and existing structures cover concrete of existing structure was chipped-out and anchoring reinforcing bars were installed. Since necessary pile length was 42m long, a steel pipe pile with larger diameter of 1.2m was selected. In the strengthening construction, a steel shell frame, which was a part of mold for concrete was used. The construction of piles and setting of the steel shell frame was made from the ship.

A SEISMIC STRENGTHENING PRACTICE OF REINFORCED CONCRETE PIER BY STEEL JACKETS

Fig.8 shows a seismic strengthening practice of a reinforced concrete pier at the termination zone of main reinforcement at mid-height. Instead of placing new reinforced concrete, steel jackets were used for covering the existing reinforced concrete pier. This was to mitigate the environmental effects such

Technical drawing of a composite material specimen, showing front, side, and detail views.

Front View Dimensions:

- Overall Width: 10800
- Overall Height: 1600
- Top Section Widths: 2800, 200, 16@300=4800, 200, 2800
- Central Grid Widths: 1600, 900, 3400
- Grid Height: 17@300 = 5100
- Left Section Widths: 400, 600, 1600
- Right Section Widths: 400, 600, 1600

Side View Dimensions:

- Overall Width: 800
- Overall Height: 800
- Central Core Width: 400
- End Section Widths: 400, 800
- End Section Labels: R-893

Detail View Dimensions:

- Overall Width: 26
- Overall Height: 240
- Overall Length: 560
- Chipping (10mm)

Labels and Annotations:

- D16 (Fiber Diameter)
- D13 (Fiber Diameter)
- Wall
- Epoxy Resin Injection
- R-893 (End Section Label)
- Chipping (10mm)

Fig.6 Seismic Strengthening of A Reinforced Concrete Frame by Placing New Reinforced Concrete Wall

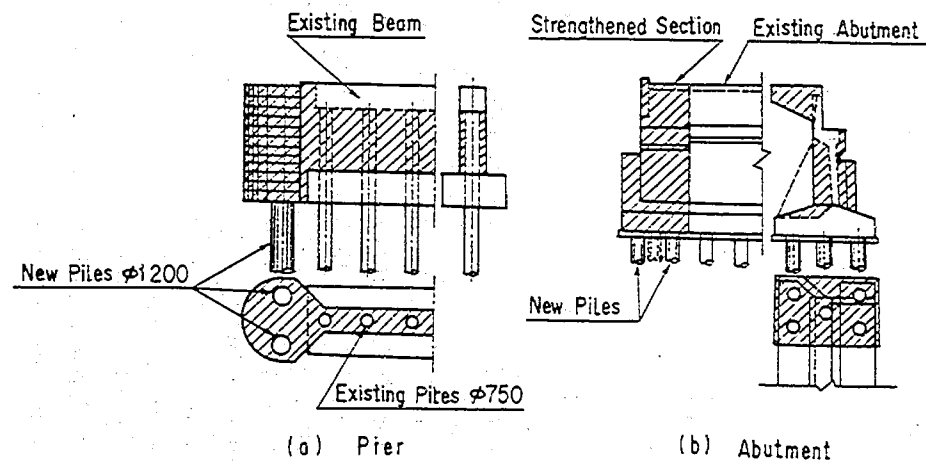


Fig.7 Seismic Strengthening of A Bent Pile by Increasing Number of Piles

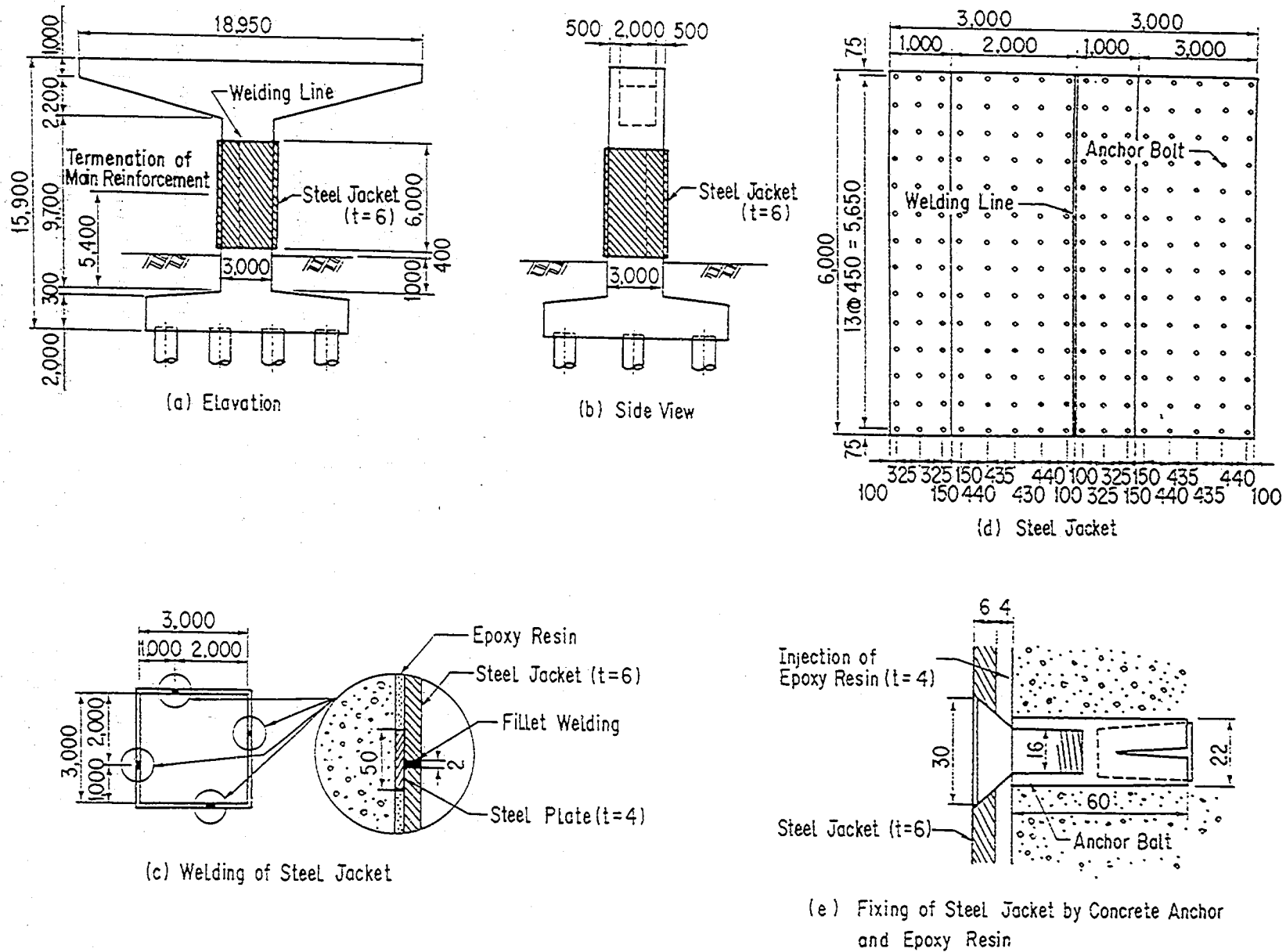


Fig.8 Seismic Strengthening of A Reinforced Concrete Pier at Termination Zone of Main Reinforcement by A Steel Jacket

as noise and air pollution caused by chipping-out of cover concrete of existing piers during construction, and to prevent increase of weight of pier because residual bearing capacity of the foundation was not enough. The minimum width of a steel jacket was determined to be more than two times section width through the dynamic loading tests conducted at the Public Works Research Institute. In the strengthening construction, the pier was covered by steel jackets from the bottom to crest of the pier for better outside appearance.

CONCLUDING REMARKS

This paper presents seismic strengthening methods for highway bridges with emphasis on practical applications. In the past seismic strengthening work in Japan, installation of devices for preventing falling-off of superstructure has successively been made. However strengthening of substructures has had difficulties such as cost problem and lack of reliable strengthening methods, so promotion of the strengthening work for substructures is now being required. Therefore, it is significantly important to develop rational and effective seismic strengthening methods with application of new materials and construction methods. It should be noted here that strengthening methods by connecting girders of a multi-span simply-supported bridge and base-isolation design are also being developed and applied.

REFERENCES

- 1) Edited by Planning Division, Road Bureau, Ministry of Construction : Notifications on Technical Standard for Road Facilities, Gyosei Publishing Co., March, 1978
- 2) Edited by Planning Division, Road Bureau, Ministry of Construction : Notifications on Technical Standard for Road Facilities, 3rd edition, Gyosei Publishing Co., April, 1978
- 3) Headquarters Office for Earthquake Disaster prevention Countermeasures, Ministry of Construction, Countermeasures against Large-scale Earthquakes, Monthly Journal of Construction, Vol.25, No.1, January, 1972
- 4) Fujii, T. : Road Disaster Prevention Measures, Road Journal, September, 1978
- 5) Fujii, T. : Road Service and Road Disaster Prevention Measures, Road Journal, June, 1980
- 6) Yamauchi, Y. : Present Status of Road Disaster Prevention Measures, Road Journal, December, 1980
- 7) Yoshikoshi, H. : Present Status and Subjects of Road Disaster Prevention Measures, Road Journal, September, 1982
- 8) Miyaji, A. : Road Disaster Prevention Measures, Road Journal, September, 1990
- 9) Edited by Road Bureau, Ministry of Construction : Road Traffic and Economy Survey, 1992 Version, December, 1990
- 10) Kawashima, K. and Unjoh, S. : An Inspection Method of Seismic Vulnerability

- of Existing Highway Bridges, Proceedings of Japan Society of Civil Engineers, Vol.7, No.1, April, 1990
- 11) Japan Road Association : Manual for Earthquake Disaster Prevention of Road Facilities (Pre-earthquake Countermeasures), February, 1988
 - 12) Road Bureau and Public Works Research Institute, Ministry of Construction : Investigation on Assessment of Durability and Improvement Technology for Existing Highway Bridges, Proceedings of the 42nd Meeting on Construction Technology, Ministry of Construction, November, 1989
 - 13) Earthquake Engineering Division, Public Works Research Institute : Case Survey of Seismic Strengthening Practices of Existing Highway Bridges Based on Seismic Inspection of 1979, Technical Memorandum of Public Works Research Institute, No.2674, October, 1989
 - 14) Kawashima, K. : Seismic Design and Earthquake Disaster Countermeasures for Highway Bridges, Road Journal, September, 1990

Seismic Behavior of Buried Pipelines
Through Field Observation

Kazuhiko KAWASHIMA

Hideki SUGITA

and

Tomoru NAKAJIMA

5th U. S. - Japan Workshop on
Earthquake Disaster Prevention for Lifeline Systems

Tsukuba, Japan

October 1992

THE FIRST PART OF THE HISTORY OF THE

REIGN OF KING CHARLES THE FIRST

IN THE YEAR 1625

BY JOHN BURNET

OF THE UNIVERSITY OF OXFORD

PRINTED BY J. STURGEON

THE SECOND PART OF THE HISTORY OF THE

REIGN OF KING CHARLES THE FIRST

IN THE YEAR 1625

BY JOHN BURNET

SEISMIC BEHAVIOR OF EMBEDDED PIPELINES THROUGH FIELD OBSERVATION

By Kazuhiko KAWASHIMA¹, Hideki SUGITA² and Tomoru NAKAJIMA²

ABSTRACT

This paper presents seismic response characteristics of embedded pipes based on field measurements. Strains induced in steel pipe and ductile pipes with expansion joints are studied. Special attention is placed for effect of bends on pipe response. Soil springs which are important in seismic design by means of the seismic deformation method is also studied based on the measured data.

INTRODUCTION

Seismic design of embedded pipes is generally made by the seismic deformation method⁴⁾ in which embedded pipes are assumed to respond in accordance with strain induced in ground during earthquakes. Pipes are considered to be beams elastically supported by soils when the stiffness of the soil is modelled by springs. Although the seismic deformation method has been adopted for seismic design of embedded lifeline facilities, measured data available for evaluation pipe response during earthquakes is very limited. Strong motion observation was initiated for measuring pipe response, because precise evaluating of pipe response in ground is essential for improving seismic design method. Special attention was paid to obtain response data for pipe with bends and expansion joints.

This paper presents an analysis of the measured data of pipe response, and soil spring stiffness to be used in the seismic deformation method is also presented.

1 Head, Earthquake Engineering Division, Public Works Research Institute,
Ministry of Construction, Tsukuba Science City, Japan

2 Research Engineer, ditto

OBSERVATION SYSTEMS

Fig.1 and Photo 1,2 shows the observation systems placed at Sodegaura city, about 50 km east of Tokyo, for measuring seismic behavior of embedded pipes. Continuous steel pipe with a diameter of 15 cm and ductile pipes with diameters of 15 cm and 30 cm are embedded 1 m below the ground surface as shown in Photo 3-7. The length of the continuous steel pipe is 120 m and there is a bend at 20 m from the right end. The length of one element of the ductile pipe is 5 m in the 15 cm diameter pipe and 6 m in the 30 cm diameter pipe. They were jointed together by expansion joints which are capable of absorbing a relative deformation of $\pm 10\%$ of pipe length in the axial direction and a rotation of ± 6 degrees. Stoppers for preventing excessive pulling-out are provided at the expansion joints.

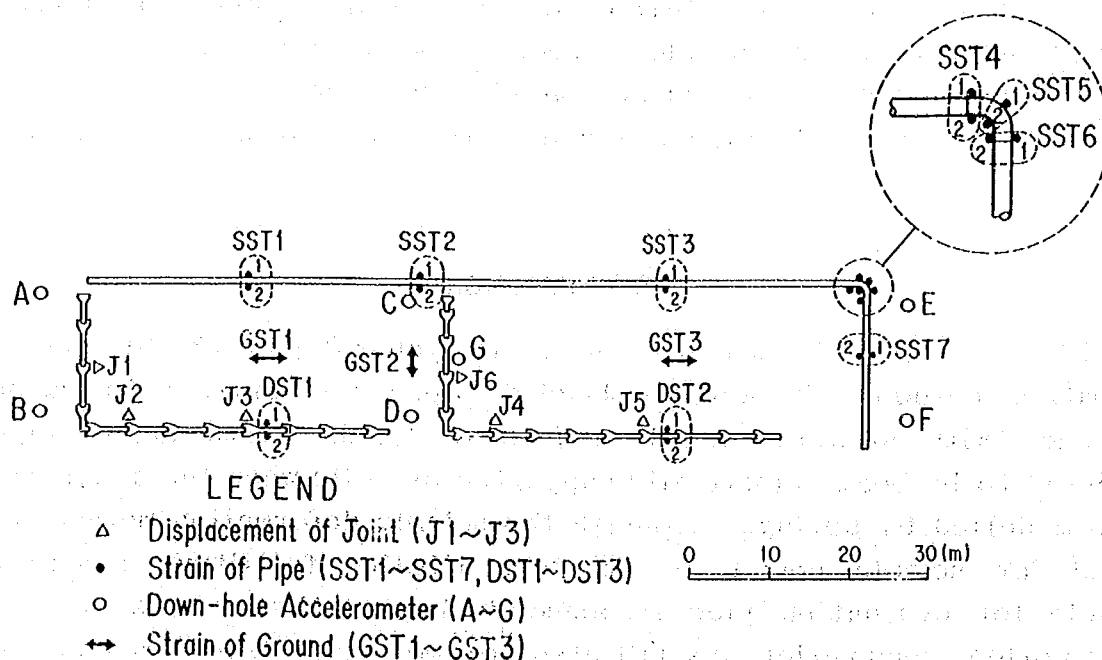


Figure 1. Observation Systems

Dense instrumentation is made to measure the pipe response. The strain induced in the steel and ductile pipes as well as relative displacements developed between two adjacent ductile pipe sections were measured. Strain gauges with coating against water are placed on the both sides of the pipes so that the flexural strain as well as the axial strain could be detected.

The ground acceleration and ground strain were measured. Three-components down-hole accelerometers were installed at six points to measure ground acceleration. The measured data was used to compute the averaged ground

strain induced at the site. Special devices were installed 1 m below the ground surface to directly measure ground strain. The devices consist of two 80 cm diameter circular plates and a 6 cm diameter double tubed steel rod. The circular plates were connected to the double tubed steel rod.

Movement of the circular steel plates in accordance with soils vibration during earthquakes would develop small relative displacements between the two circular plates. Ground strain may be evaluated by dividing the relative displacement by the length of the double tubed steel rod. This device has been successfully used for measuring ground strain at several observation sites¹⁾³⁾.

All the measured signals are digitized with an interval of 1/100 second and recorded with a digital acquisition systems. Pre-event memory of about 10 seconds are provided to get full records from their start.

MEASURED DATA

Observation was initiated in January 1989 and many records have been already obtained. In this report the pipe response was analyzed based on the measured data obtained after October 1989. Record which are large enough for analysis were measured by thirteen earthquakes as shown in Table 1.

Table 1. Major Earthquakes Triggered

NO. OF EQ.	DATE	EPICENTER			MAGNITUDE (M)	FOCAL DEPTH (km)
		EPICENTRAL REGION	LONGITUDE	LATITUDE		
EQ-1	1989. 10. 14	NEAR IZU-OHSHIMA	135° 03'	34° 54'	5.7	25
EQ-2	1990. 6. 1	EAST OF CHIBA-KEN	140° 40'	35° 38'	6.0	70
EQ-3	1990. 6. 5	MIDDLE OF KANAGAWA-KEN	139° 20'	35° 50'	5.3	120
EQ-4	1990. 6. 27	SAGAMI-BAY	139° 07'	35° 00'	5.4	148
EQ-5	1990. 8. 23	MIDDLE OF CHIBA-KEN	140° 24'	35° 21'	5.4	50
EQ-6	1990. 8. 23	MIDDLE OF CHIBA-KEN	140° 24'	35° 21'	5.2	49
EQ-7	1990. 12. 11	MIDDLE OF CHIBA-KEN	140° 19'	35° 19'	4.0	35
EQ-8	1990. 12. 16	MIDDLE OF CHIBA-KEN	140° 13'	35° 37'	4.6	77
EQ-9	1991. 6. 28	TOKYO-BAY	139° 52'	35° 29'	4.6	68
EQ-10	1991. 7. 14	EAST OF NAGANO-KEN	138° 31'	36° 25'	5.4	187
EQ-11	1991. 9. 29	NORTH OF CHIBA-KEN	140° 06'	35° 45'	4.9	80
EQ-12	1991. 11. 19	TOKYO-BAY	140° 02'	35° 36'	4.9	81
EQ-13	1992. 2. 2	TOKYO-BAY	139° 08'	35° 20'	5.7	90

Fig. 2 shows the measured axial strain induced in the steel pipe and ground accelerations developed by an earthquake with epicentral distance of 75 km and an earthquake magnitude of 5.7. This earthquake is designated hereinafter as EQ 3 earthquake. The peak ground acceleration was about 65 to 70 cm/sec² and the peak pipe strain was about 7.2 to 10.1 μ . It is interesting to note in Fig. 2 that although there are many similarities of the pipe strains between the three locations, the pipe strain are considerably different each other. This clearly shows that the pipe response in axial direction depends considerably on the response of nearby soil.

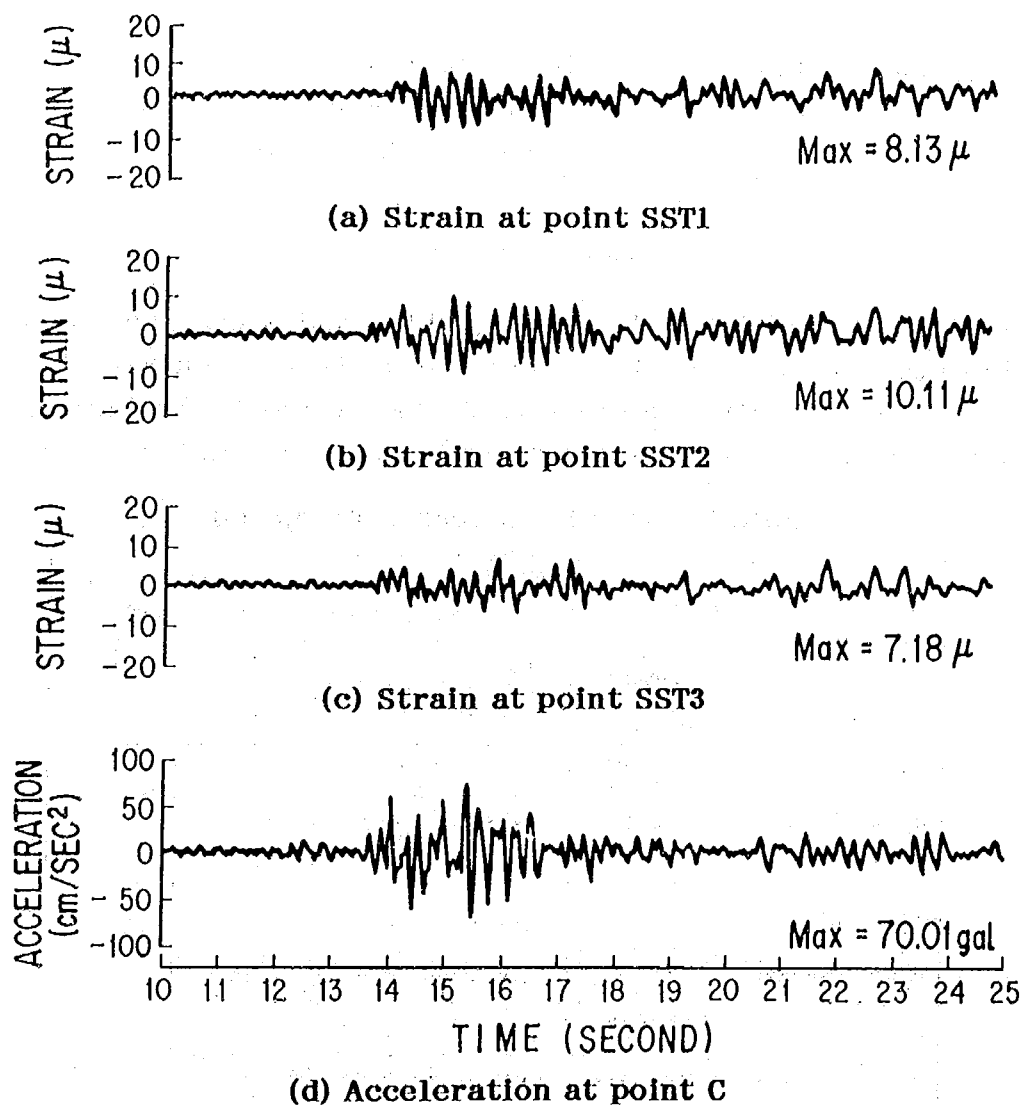


Figure 2. Measured Axial Strain of Steel Pipe and Ground Acceleration C Point during EQ 13

Because the ground motion was small, the devices for measuring ground strain did not directly detect ground strain. Fig. 3 shows ground strains

computed for the EQ 13 earthquake from measured ground accelerations. The measured ground accelerations were double integrated to obtain the ground displacements⁷⁾. Assuming triangular elements such as ΔABC and ΔBCD , the ground strain along pipe axis was computed with use of the standard finite element method⁵⁾. Linear interpolation function for displacements was assumed in each finite element. If the ground strain computed for ΔABC , which is the averaged ground strain between points A and C, is compared with the axial pipe strain at point SST1, in Fig. 2, it is found that the strains obtained are quite similar.

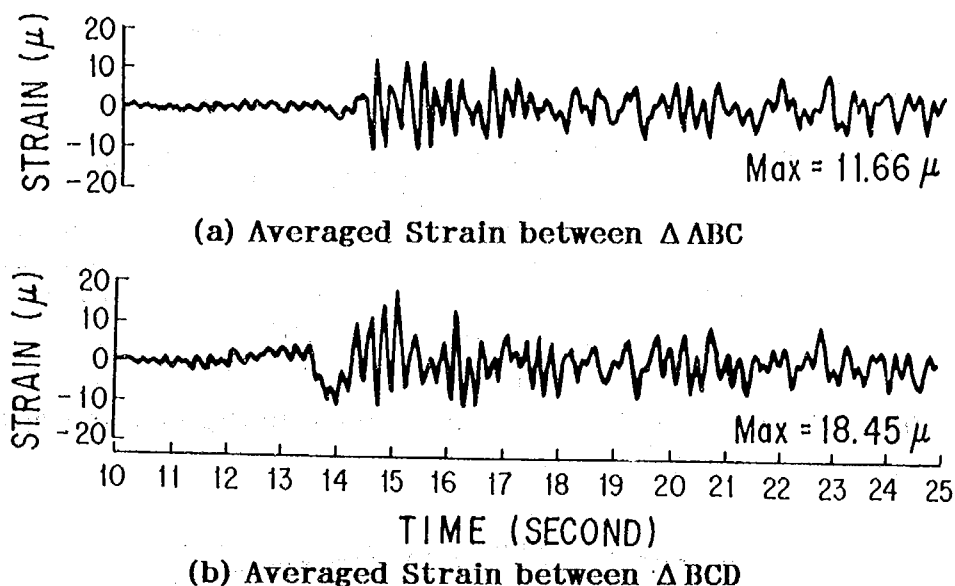
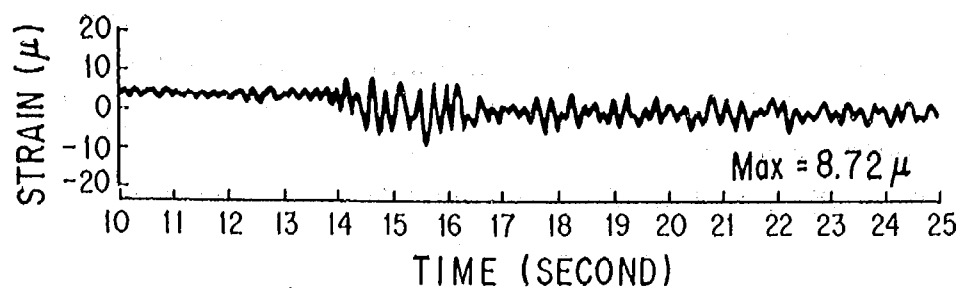


Figure 3. Ground Strain Computed for EQ 13 from Measured Ground Acceleration

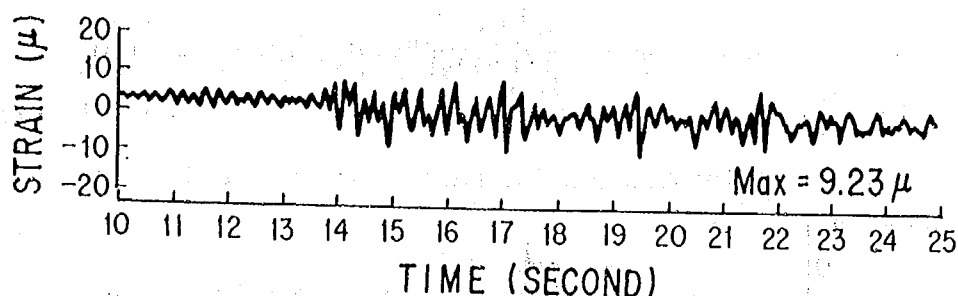
Fig. 4 shows axial strain induced during the EQ 13 earthquake in the ductile pipes. Almost no relative displacement was developed at the joints. It is interesting to note that the strain of the ductile pipe at the point DST-1 is quite similar with the strain of the steel pipe at the point SST-1. The strain of the ductile pipe is not necessarily smaller than the strain of the steel pipe. This implies that reduction of axial force induced in the ductile pipe due to the expansion joints was quite small during the EQ 13. The forces developed at the expansion joints during the EQ 13 were so small that friction at the expansion joints prevented slippage to occur between the adjacent pipe sections.

Fig. 5 shows a distribution of peak strain along the steel pipe for the EQ 13 earthquake. Both axial and flexural strains are presented. It should be noted that although axial strain is predominant in the straight section, flexural

strain becomes larger than the axial strain at the bends. Flexural strain has to be therefore taken into account in seismic design of pipes at bend sections.

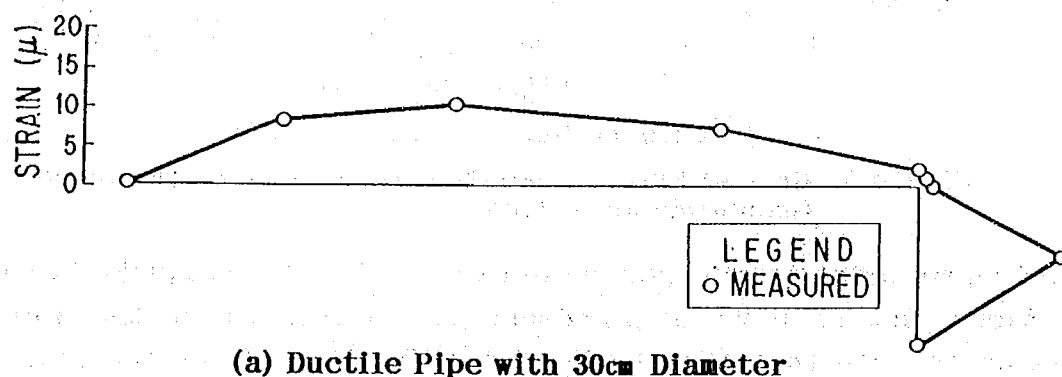


(a) Ductile Pipe with 30cm Diameter

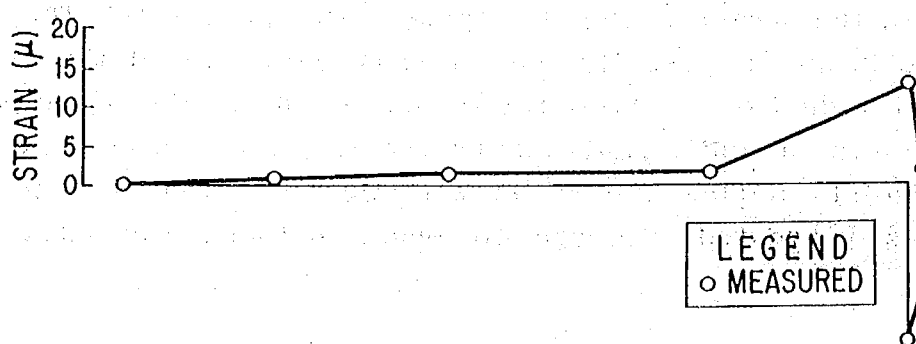


(b) Ductile Pipe with 15cm Diameter

Figure 4. Measured Axial Strain of Ductile Pipe during EQ13



(a) Ductile Pipe with 30cm Diameter



(b) Ductile Pipe with 15cm Diameter

Figure 5. Peak Strain of Steel Pipe during EQ 13

Fig. 6 shows the axial strain of steel pipe vs. peak ground acceleration relation for the data obtained by the eight earthquakes presented in Table 1. Although number of the data obtained is still small, clear correlation between the pipe strain and the peak ground acceleration is observed. By increasing the number of data, this is expected to provide a practical estimation for the pipe strain induced during earthquakes.

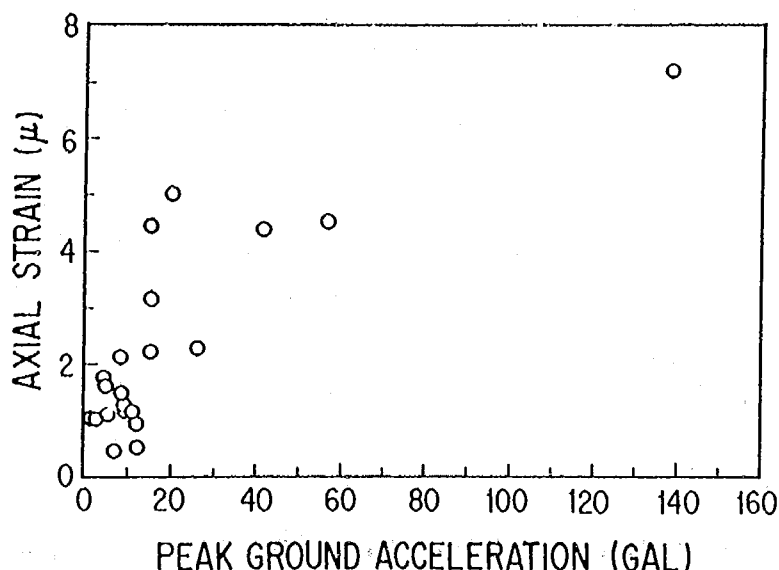


Figure 6. Axial Strain of Steel Pipe vs. Peak Ground Acceleration Relation

EVALUATION OF SOIL SPRING STIFFNESS

An analytical model presented in Fig. 7 is often adopted to estimate dynamic response analysis of embedded pipes⁶⁹. The pipe is idealized by a beam elastically supported by springs which represent pipe-soil interaction. Soil response is computed by prescribing the input ground motion at the base rock.

In the analytical model presented in Fig. 7, it is essential to determine the stiffness of the soil springs which elastically support correctly the pipe. Various methods have been proposed from analytical and experimental studies to estimate the soil springs. However due to limitation of measured data available for estimating soil springs based on measured seismic response data of pipes, few studies have been made for soil springs. Therefore, with use of the analytical model presented in Fig. 7, the soil springs stiffness were studied.

The response displacements of soils which were estimated from the measured accelerations were used for computing the pipe responses. The steel pipe was

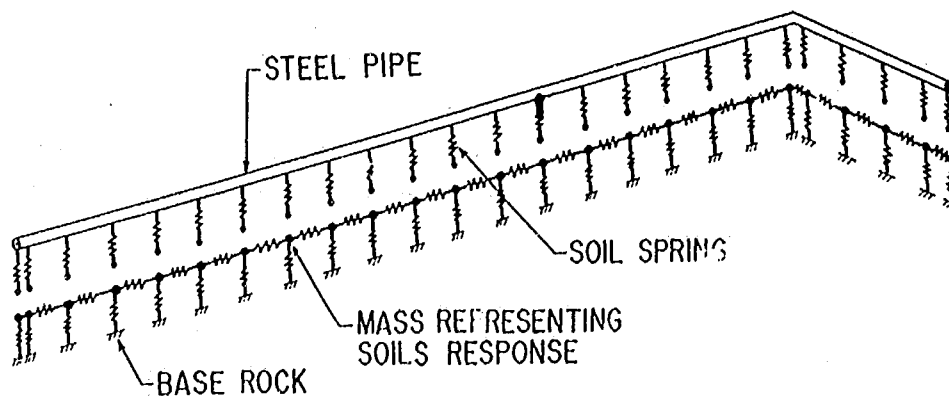


Figure 7. Analytical Model for Dynamic Response Analysis of Embedded Pipes

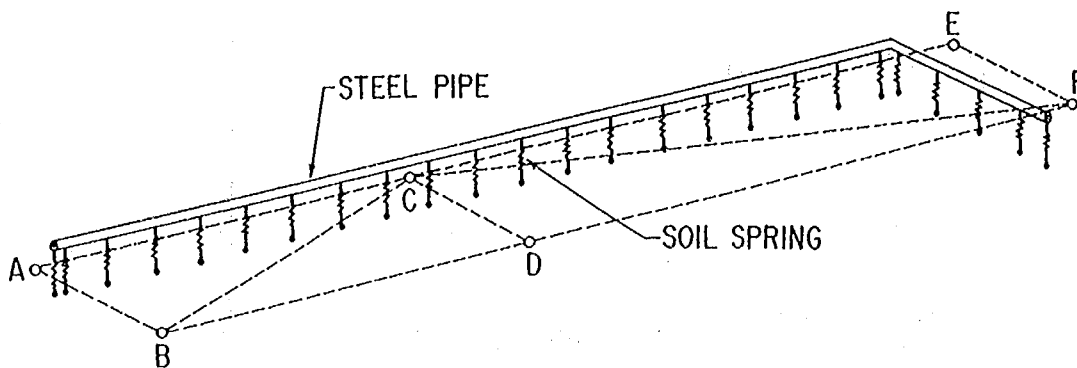


Figure 8. Analytical Model Used for Pipe Response

idealized by a discrete analytical model with 24 beam elements as shown in Fig. 8. Because the displacements which were computed from the measured accelerations were available at only six points, the displacements at each nodal point of the analytical model were computed from either linear interpolation or extrapolation of the displacements from the six points.

In the Design Specifications of Petroleum Pipelines²⁾, the soil spring stiffness is stipulated to be estimated as

$$K = c \times G_s \quad (1)$$

where K : stiffness of soil spring per unit length, c : a coefficient, and G_s : shear moduli of soils. The shear moduli of soils are estimated as

$$G_s = \gamma_t \times V_s^2 / g \quad (2)$$

where γ_t : unit weight of soils, V_s : shear wave velocity of soils, and g : acceleration of gravity. In the specifications the coefficient c is stipulated to be determined by appropriate tests, and a value of 3.0 is recommended when such test data is not available.

The soil spring which gives realistic estimation for the measured pipe strain was therefore studied. Fig. 9 shows the approximation of the computed pipe strain assuming a soil spring stiffness of 100 tf/m^2 and 300 tf/m^2 . The computed axial and flexural strain approximates quite well the measured pipe strain. Because shear wave velocity which was measured at three points in the observation site was about 150 m/sec at the depth between the ground surface and 15 meters below the ground surface, the soil spring stiffness of 100 tf/m^2 and 300 tf/m^2 are predicted by assuming a coefficient "c" of about 0.05 and 0.16 respectively in Eq.(1). From such evidence it may be said that the recommended value of 3.0 for the coefficient "c" in Eq.(1) is excessively conservative. It is important to precisely study the coefficient "c" which gives a realistic soil spring stiffness.

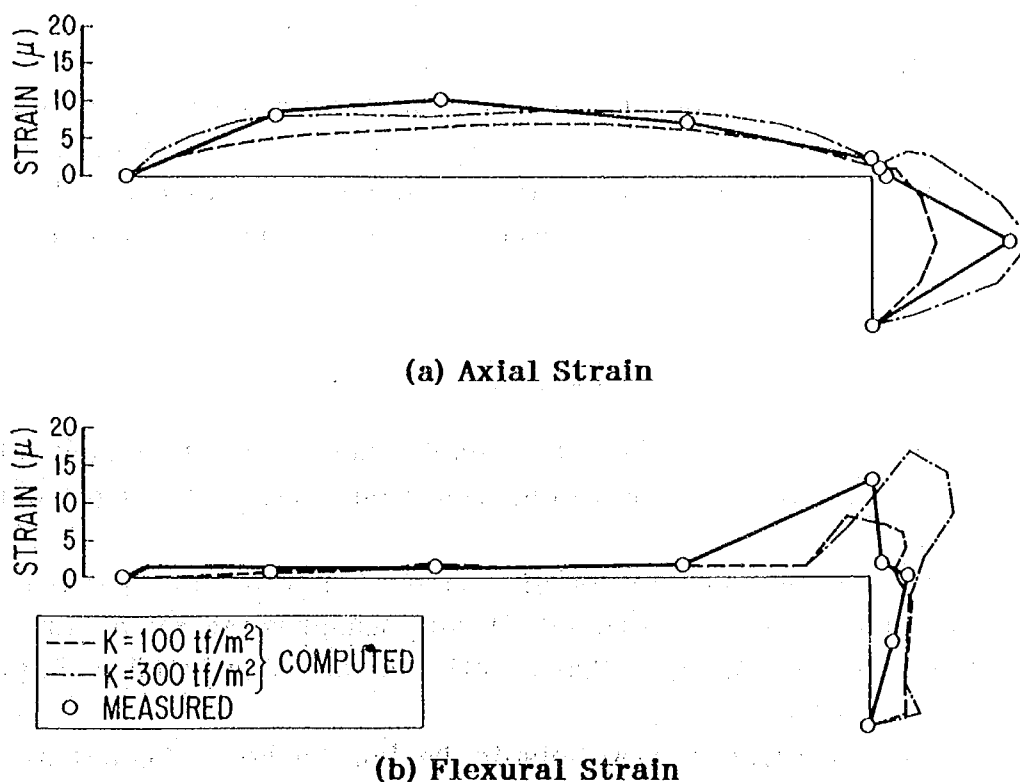


Figure 9. Best Correlation of Pipe Strain for EQ 13 CONCLUSIONS

For accumulating actual seismic behavior of embedded pipes during earthquakes, seismic observation of two types of pipes was initiated. Based on the analyses of the measured data on the strain of embedded pipes, the following conclusions may be deduced:

- 1) Although axial strain was predominant in the straight section, flexural

strain is more predominant than axial strain at the bend section of the pipe. Flexural strain should be considered in seismic design at bend sections.

- 2) Considerably good correlation was observed between the measured axial strain of the pipe and the peak ground acceleration. Accumulation of measured data is expected to provide a realistic estimation method for the pipe strain based on the peak ground acceleration.
- 3) The computed pipe strain assuming Eqs.(1) and (2) was excessively larger than the measured pipe strain. It is thought that the spring stiffness predicted by Eq.(1) assuming the coefficient "c" of 3.0 is too conservative. Coefficient "c" has to be precisely studied by analyzing more measured data.

FUTURE DIRECTIONS

Accumulation of measured data is required for providing a realistic estimation method for the pipe strain based on the peak ground acceleration, and for studying the coefficient "c" which gives a realistic soil spring stiffness. Because the observation site is located at one of the highest seismicity area in Japan, it is anticipated to have a number of measured data in a short time.

REFERENCES

- 1) Iawamoto, T.et.al.: Observation of Dynamic Behavior of Buried Pipelines during Earthquakes, Eighth World Conference on Earthquake Engineering, San Francisco, U.S.A., 1984
- 2) Japan Road Association : Design Specifications of Petroleum Pipelines, 1974
- 3) Katayama, T. and Sato, N. : Ground Strain Measurement by a Very Densely Located Seismometer Array, Proc. 6th Japan Earthquake Engineering Symposium, Tokyo, 1982
- 4) Ministry of Construction : New Seismic Design Methods, Technical Note of PWRI, No. 1185, Public Works Research Institute, 1977
- 5) Okubo, T., Arakawa, T. and Kawashima, K. : Dense Instrument Array Program of the Public Works Research Institute and Preliminary Analysis of the Records, Eighth World Conference on Earthquake Engineering, San Francisco, U.S.A., 1984
- 6) Tamura, C., Okamoto, S. and Hamada, M. : Dynamic Behavior of a Submerged Tunnel during Earthquakes, Report of the Institute of Industrial Science, University of Tokyo, Vol.24, No.5, 1975
- 7) Tamura, K., Kawashima, K., Aizawa, K. and Takahashi, K. : Recording Accuracy

- of Digital Strong Motion Accelerographs with Independent Triggering and Recording Systems for Analysis of Finite Ground Strains Induced during Earthquakes, Proc. Japan Society of Civil Engineering, Vol. 392/I-9, 1988
- 8) Kawashima, K., Hasegawa, K. and Nagashima, H.: Seismic Behavior of Buried Pipelines through Field Observation, Proc. of 3rd U.S. National Conference on Lifeline Earthquake Engineering, Los Angeles, CA, August 1991

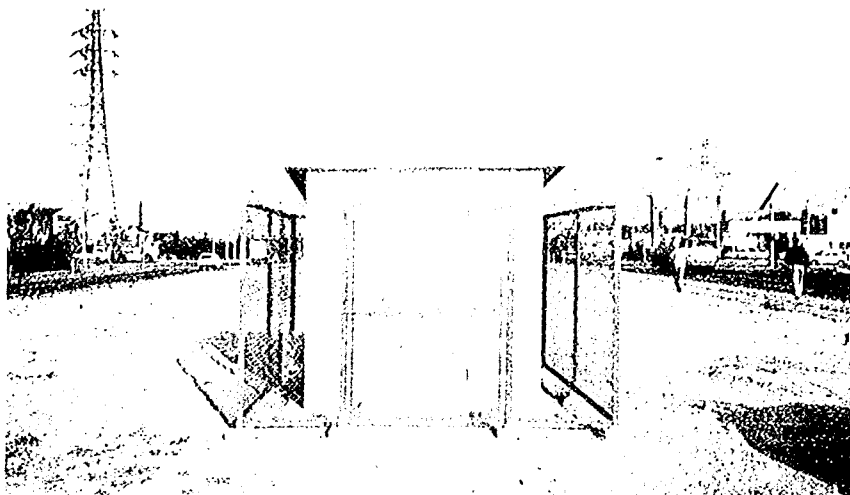


Photo 1. Observation Station

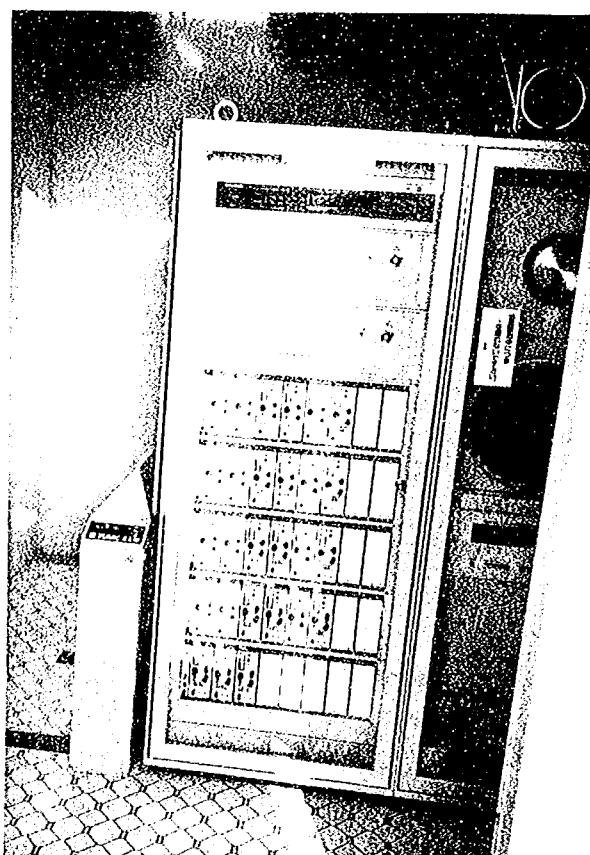


Photo 2. Data Acquisition System

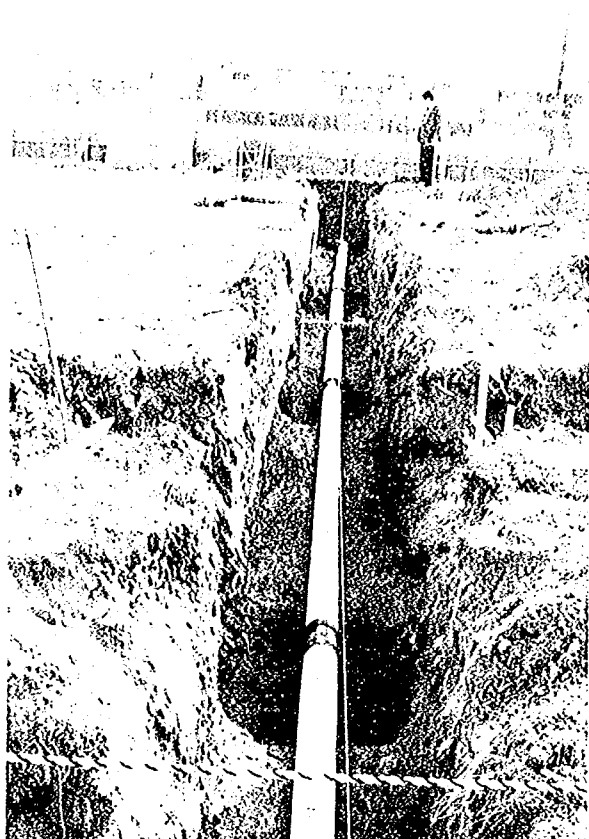


Photo 3. Steel Continuous Pipe (Straight Section)

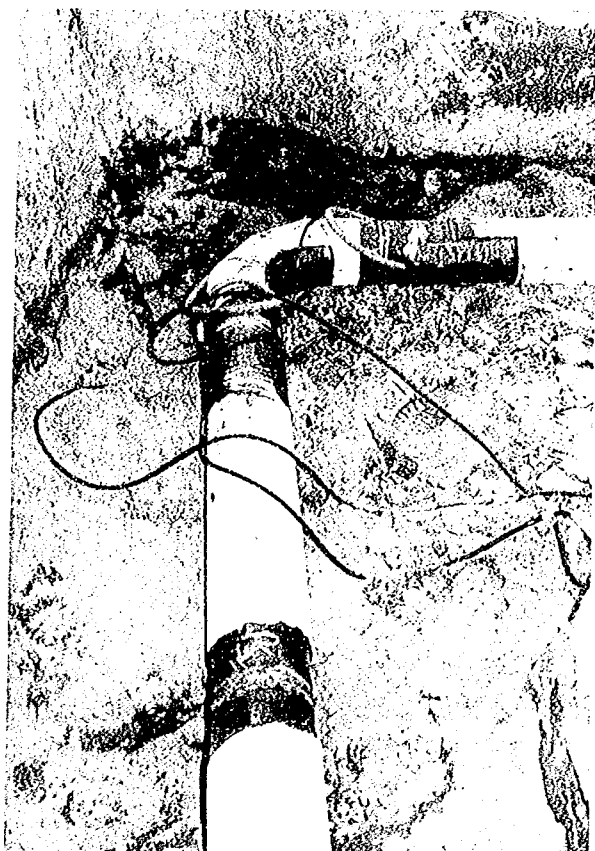


Photo 4. Steel Continuous Pipe (Bend Section)

Effectiveness of the Variable Damper
for Reducing Seismic Response of Highway Bridges

Kazuhiko KAWASHIMA

Shigeki UNJOH

Hiroyuki IIDA

and

Hidetoshi MUKAI

2nd U. S. -Japan Workshop on
Earthquake Protective Systems for Bridges

Tsukuba, Japan

December 1992

Mr. C. J. Smith, 1200 1st St. N.W.

Washington, D. C. 20004

Dear Mr. Smith:

I am writing to you

to inform you of

the results of

our investigation

into the matter of

the alleged activities of

the subject of this

report.

EFFECTIVENESS OF THE VARIABLE DAMPER FOR REDUCING SEISMIC RESPONSE OF HIGHWAY BRIDGES

Kazuhiko KAWASHIMA, Shigeki UNJOH, Hiroyuki IIDA
and Hidetoshi MUKAI

Earthquake Engineering Division, Public Works Research Institute

SUMMARY

Presented are analytical and experimental studies on the dynamic characteristics of a variable damper developed at the Public Works Research Institute. Effectiveness of the variable damper for reducing seismic response of highway bridges is analytically investigated. A prototype model of the variable damper is developed and the dynamic characteristics obtained through the dynamic loading tests is also presented.

INTRODUCTION

Because integration of functions of a damper-stopper and a energy dissipator is effective for increasing seismic safety of highway bridges, the applicability and practicability of the variable damper, in which the damping characteristics is variable depending on the response of highway bridges, has been studied at the Public Works Research Institute¹⁾⁻³⁾.

This paper presents analytical and experimental studies on the dynamic characteristics of the variable damper. Simple analytical results are described for presenting the effectiveness of the variable damper for reducing seismic response of highway bridges. A prototype model of the variable damper is developed and the load-displacement characteristics and the controllability obtained through the dynamic loading tests are also presented.

FUNCTIONS OF THE VARIABLE DAMPER

The variable damper is a viscous damper in which the viscous damping force is variable depending on the response of highway bridges as shown in

Fig.1. That is:

1) The damping coefficient of the variable damper is taken large during a small deck vibration so that the damper have the same function as a fixed bearing support against the braking load of vehicles. It is movable against actions with low-velocity such as an elongation of deck by temperature change.

2) When the amplitude of deck vibration becomes larger up to a certain level during an earthquake, the damping coefficient is decreased so that the energy dissipation be optimum and the inertia force of a superstructure to substructures be decreased appropriately.

3) Furthermore, when the vibration amplitude of the deck becomes excessive, the damping coefficient is gradually increased in order to suppress the excessive amplitude. Therefore, the damper has a function as a stopper with a shock absorber function. This is to prevent the impact response by the sudden operation of the stopper⁵⁾.

Therefore, the variable damper has the advantages of an usual viscous damper^{6)~8)}, a passive energy dissipator and a stopper with a shock absorber function.

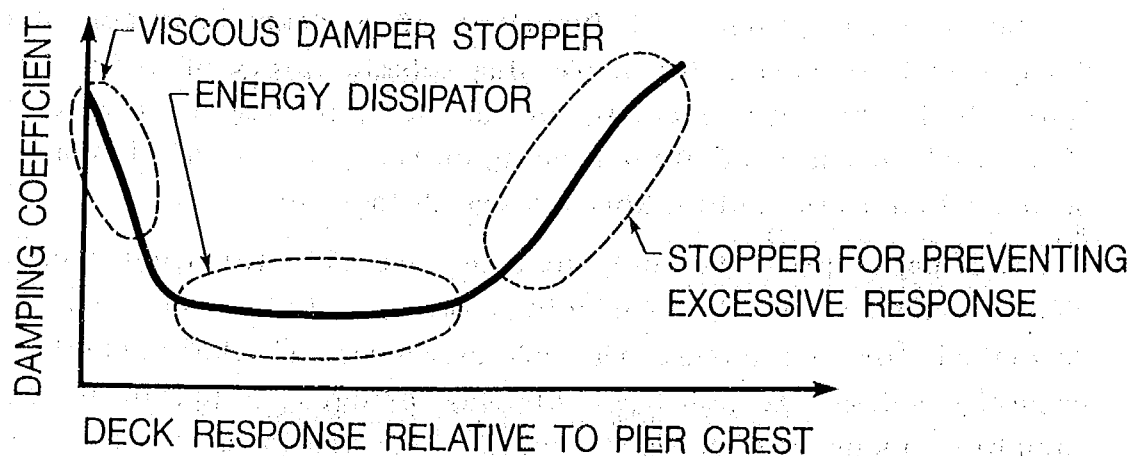


Fig.1 Basic Concept of Variable Dampers

The concept of the variable damper, in which the damping characteristics can be variable depending on the situation, has been effectively adopted in a field of mechanical engineering, in particular in a suspension system for an aircraft and an automobile. An active suspension to improve driving comfortability and stability of the automobile is one of the examples^{9)~10)}.

However, it is a new idea to apply the variable damper for highway bridges based on the concept of distributing the inertia force of a superstructure to substructures.

Although various types of the variable damper can be made, the simplest one may be as shown in Fig.2. The basic component is an cylinder-type viscous damper. A by-pass pipe is installed between the cylinder-cells divided by the piston. The damping characteristics can be controlled by varying the amount of flow of viscous material which passes through the by-pass. Since various technology for controlling such material flow has been developed, the variable damper is most promising and near to the practical use. Furthermore, the external energy required for the operation is significantly smaller in the variable damper than in active control devices¹¹⁾.

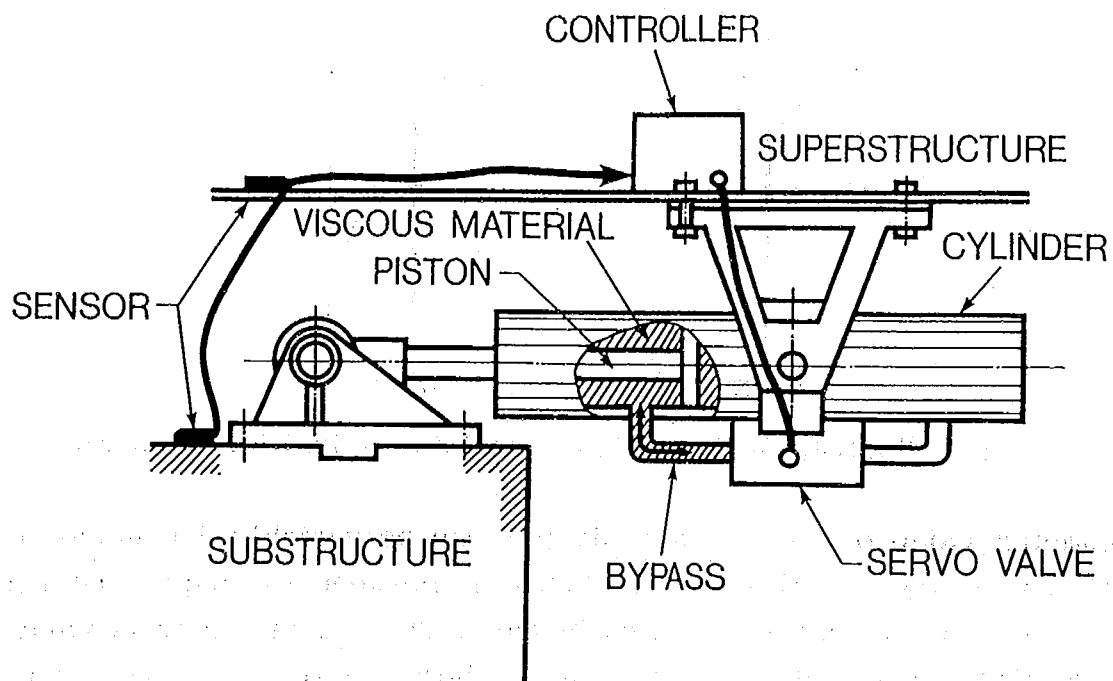


Fig.2 Variable Damper

SEISMIC RESPONSE ANALYSES OF HIGHWAY BRIDGES WITH VARIABLE DAMPERS

In order to investigate the effectiveness of the variable damper, a simple span girder bridge, as shown in Fig.3, with a span length of 30m is analyzed²⁾. The response in longitudinal direction is considered. Although the variable damper has better applicability for multi-span continuous girder bridges, the simple span girder bridge is taken as an analytical model here for simplicity.

The superstructure of the model is supported by elastic isolators in longitudinal direction and the variable dampers are assumed to be installed between the superstructure and the pier top. The weight of the superstructure and spring stiffness of elastic isolator are assumed as 241.5tf and 640tf/m, respectively. The flexural stiffness of piers is assumed so that the fundamental natural period of the model be 0.5sec when the both bearing supports are hinged. The stiffness of isolators results in the fundamental natural period of 1sec for the model with elastic isolators.

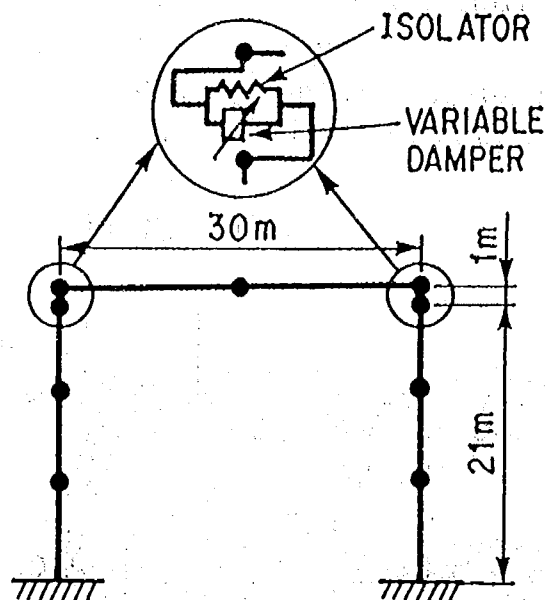


Fig.3 Highway Bridge Analysed and Analytical Idealization

Damping ratio of the variable damper can be variable dependent on both relative velocity and relative displacement between the superstructure and the pier top. Only analytical cases in which the damping ratio is varied with the relative deck displacement are shown here for presenting the effectiveness of the variable damper.

Fig.4 shows the control method of the damping coefficient of the variable damper so that it has the functions as a damper-stopper, a energy dissipipator and a stopper with a shock absorber function. The displacement, at which the stopper function for preventing excessive displacement initiates to work, is assumed as $2/3$ of the peak relative displacement d_u and the damping ratio is increased from h_1 to h_3 . The displacement d_u is defined hereafter as a design displacement of the variable damper. For suppressing small deck displacement

the damping ratio of the variable damper is increased from h_1 to h_2 in the range of the relative displacement from $-1/6d_u$ to $1/6d_u$. The design displacement d_u , damping ratio h_1 , h_2 and h_3 are assumed here as 6.34cm, 0.5, 3 and 3, respectively. These values are determined through the parametric analyses to obtain appropriate ones for the model bridge with the variable damper³⁾.

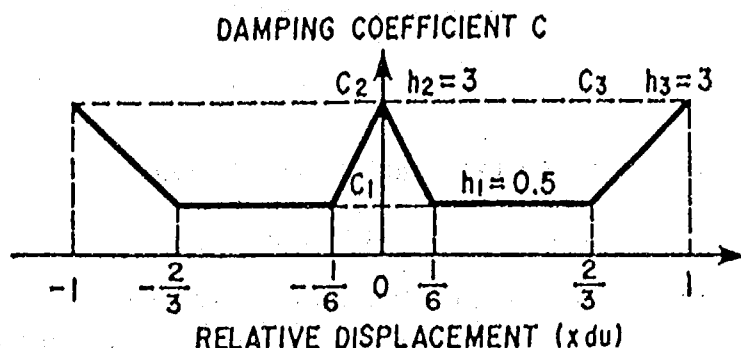


Fig.4 Displacement Dependent Damping ($d_u=6.34\text{cm}$)

The acceleration recorded on the ground around the Tsugaru Bridge during the Nihonkai-Chubu Earthquake of 1983 was used as an input acceleration. The acceleration was modified so that the response spectrum matches with the target spectrum. The acceleration spectrum, which is specified in "Part V Seismic Design" of the "Design Specifications for Highway Bridges", is assumed as the target spectra. The target spectra is used for the check of bearing capacity of reinforced concrete piers on soft soil site (ground condition III).

When the variable damper is not provided, the modal damping ratio of the model bridge is assumed to be 2% of critical.

Table 1 compares the peak responses of the model bridge with and without control. If the variable damper with $h_1 = 0.5$ and $h_2 = h_3 = 3$ is provided, peak deck displacement and peak deck acceleration decrease to 25% and 43% of those with no control, respectively. The maximum bending moment at the pier bottom decreases to 46% of that. The required damping force and peak stroke of the variable damper are $62.6\text{tf} \times 2 = 125\text{tf}$ and 5.78cm, respectively. The variable damper with this specification can be designed within the current scope of damper technology.

Table 1 Peak Response of the Bridge and the Variable Damper

Analytical Cases	Deck		Pier		Variable Damper		
	Displacement (cm)	Acceleration (cm/s ²)	Shear Force (tf)	Bending Moment (tf·m)	Damping Force (tf)	Relative Displacement (cm)	Total Energy (tf·m)
No Control	33.04	1,300	166.9	3,297	---	---	---
Displacement Dependent Damping	8.38	555	81.2	1,528	62.6	5.78	77.4

DEVELOPMENT OF A PROTOTYPE MODEL OF THE VARIABLE DAMPER

Outline of A Prototype Model of the Variable Damper

Fig.5 and Photo.1 show a design plan and a view of the variable damper developed. The model is designed so that the maximum damping force be 20tf, maximum relative displacement of damper piston be within ± 10 cm. The control of the damping ratio is arbitrarily made by a personal computer depending on relative displacement and velocity developed between deck and substructures.

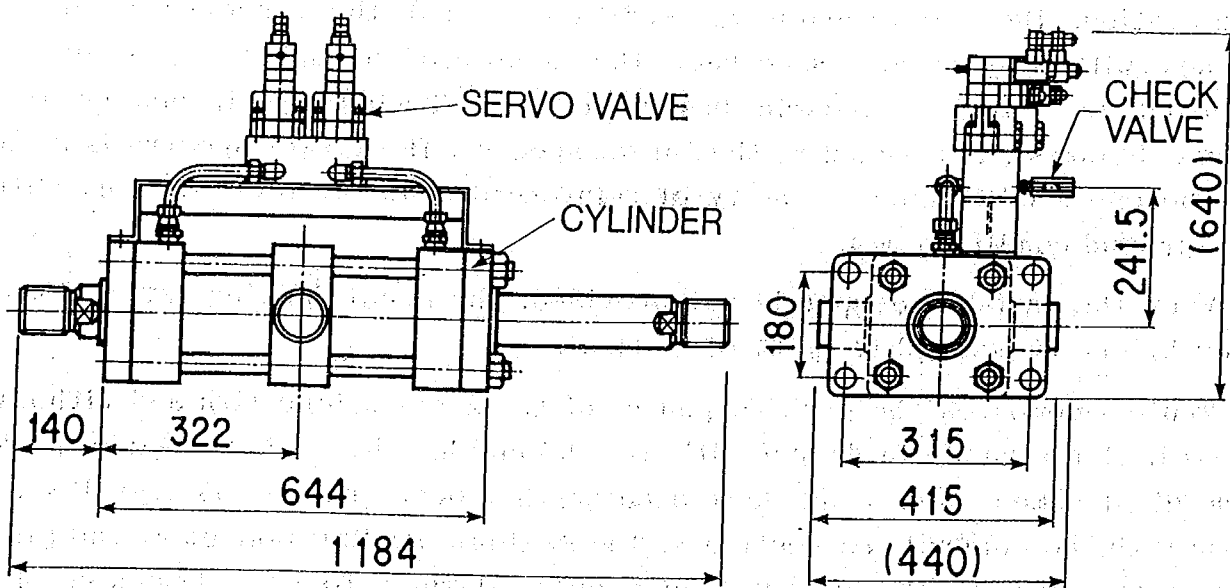


Fig.5 Prototype Model of Variable Damper

The piston-cylinder has the total length of 1,184mm, the pressure area of the piston is 137.4cm² and the maximum stroke of the piston is ± 13 cm. A steel pipe by-pass with servo valves is installed between the cylinder-cells

divided by the piston. The servo valve is a D.C. proportional relief valve. When the oil pressure through the valve increases to specified one, the pore oil pressure is relieved. This pressure is defined hereafter as control pressure. Therefore, damping force of the piston is developed by the difference of the pressure between two cells in the cylinder. The damping force increases up to when the pressure reaches the specified one, then becomes constant after the pressure exceeds the specified one. Therefore, if the control pressure of the servo valve is assumed to be constant, the damping force is dependent on neither displacement nor velocity of the piston. Therefore, friction type damping force is developed.

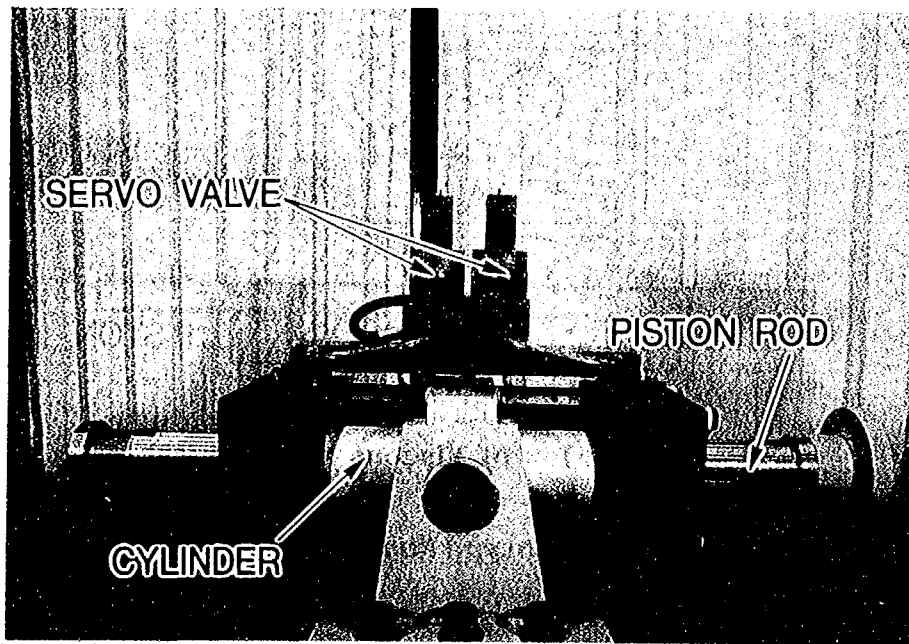


Photo 1 Prototype Model of Variable Damper

The input voltage from a servo amplifier to the servo valve is designed in the range of 0 to 5V. The control pressure is 0 at the input voltage of 0V and increases with increase of the input voltage. The viscous material used for the variable damper is an usual oil for a dynamic loading actuator. The variable damper also has a fail-safe function for an emergency condition such as power failure. The control pressure can be fixed at arbitrary one for such cases.

Fig.6 shows the flow of the viscous material in the variable damper. The damper piston can move in the left and right directions accompanying with the deck response. When the piston moves in the left side, the viscous material

flows from the left cell of the cylinder to the right one through the by-pass and the servo valves. The difference of the pressure in the two cells is adjusted as specified one by servo valves. When the piston moves in the opposite direction, damping force is controlled in the same manner.

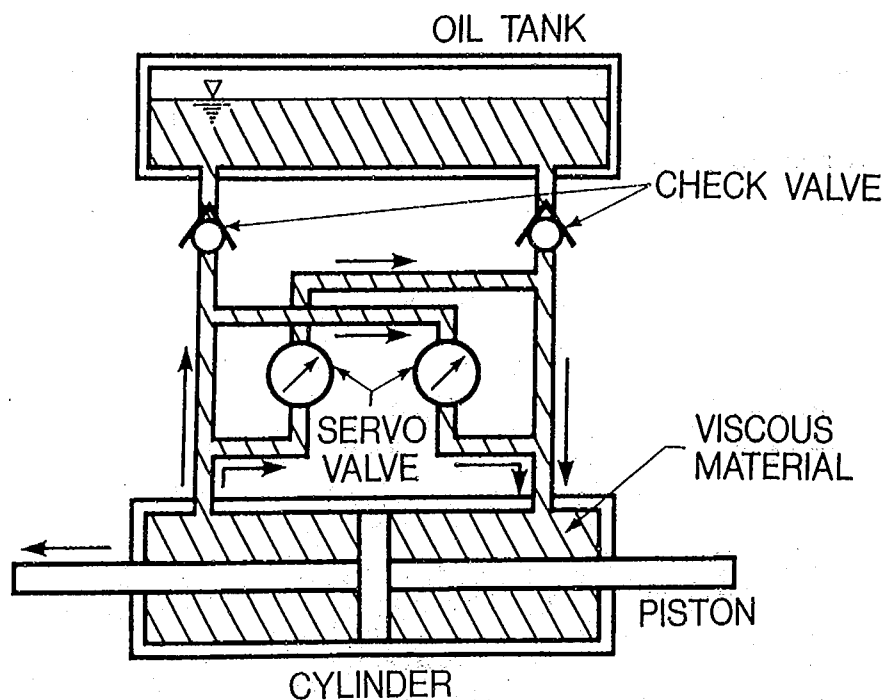


Fig.6 Flow of Viscous Material

Dynamic Loading Tests of the Prototype Model of the Variable Damper

The variable damper can vary the damping characteristics depending on the displacement and/or velocity of the structures. Therefore, in order to provide the variable damper with the required damping characteristics it is necessary to know the basic dynamic characteristics. The damping characteristics of the variable damper is controlled by the control pressure of the servo valve and it is controlled only by the input voltage to the servo valve from the servo amplifier. Therefore, in order to arbitrarily control the damping force of the variable damper developed, the relation between the input voltage to the servo valve and damping characteristics of the variable damper is essential.

Fig.7 shows the set-up of the dynamic loading tests. Under the condition that the control pressure of the servo valve is kept to be constant, the loading was applied to the piston by a dynamic actuator under the

displacement control. The input voltage to the servo valve, reaction force and stroke of the piston are measured. The loading is assumed as harmonic and repeated by 10 cycles for each loading displacement.

Table 2 shows the experimental cases. Case 1 is to measure the friction force of the damper piston. Static loading was made with loading frequency of 0.05Hz and loading displacement of $\pm 100\text{mm}$. The input voltage of the variable damper is assumed to be 0V so that the control pressure be 0. Case 2 is to obtain the relation between input voltage to the servo valve and damping force of the variable damper. Loading frequency, loading displacement and input voltage are varied. It should be noted here that the loading, in which the maximum loading velocity exceeded 7cm/sec, was not made because of the capability of a loading apparatus.

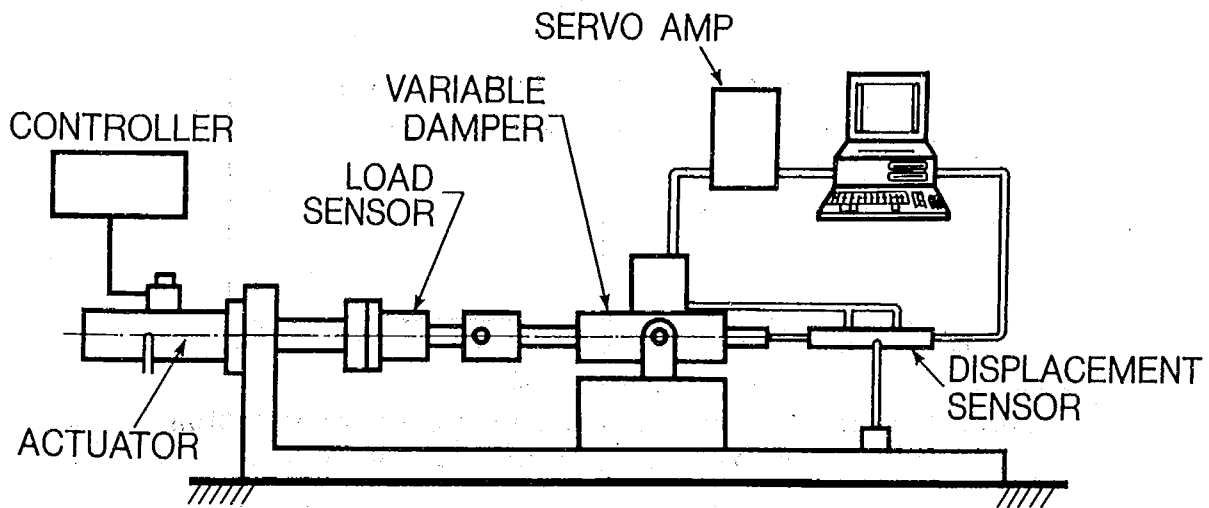


Fig.7 Set Up of Loading Test

Table 2 Experimental Cases

Case	Objective	Loading Frequency (Hz)	Loading Displacement (cm)	Input Voltage (Volt)
1	Measurement of Friction Force of Damper-stopper	0.05	± 100	0
2	Relation between Damping Force and Input Voltage	0.1~1.0	$\pm 2 \sim \pm 100$	0~5

Performance of the Prototype Model of the Variable Damper

(1) Friction force of the damper-piston

Fig.8 shows hysteresis loops of the load-displacement relation when the damper is loaded with the loading frequency of 0.05Hz and loading displacement of $\pm 100\text{mm}$ to obtain the friction force of the damper-piston. The load-displacement relation shows almost rectangular shape. It means that the typical friction type load-displacement relation is obtained. The hysteresis is stable for a number of loading cycles. The average friction force is about 1tf.

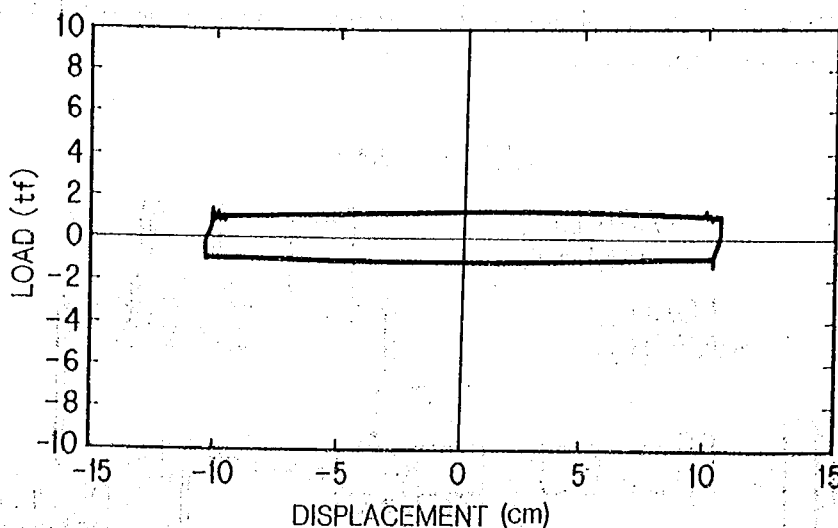
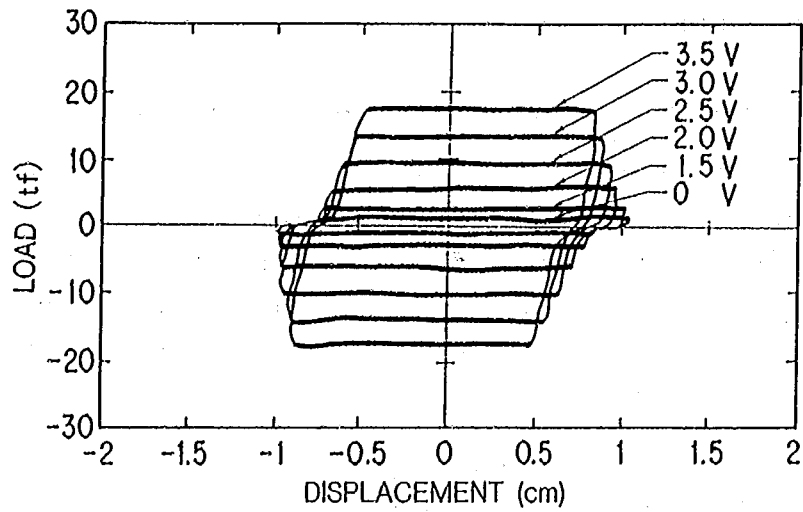


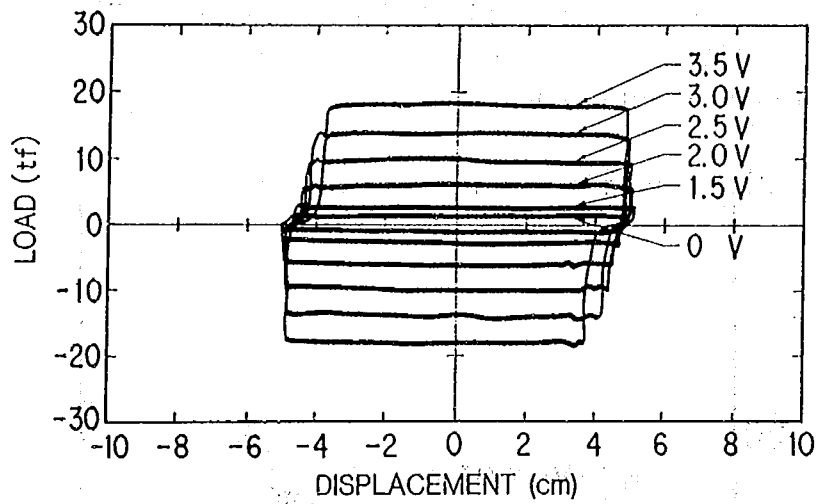
Fig.8 Hysteresis Loops of Load-Displacement Relation
(Displacement = 10cm, Frequency = 0.05Hz)

(2) Hysteresis Loops of the Load-displacement Relation

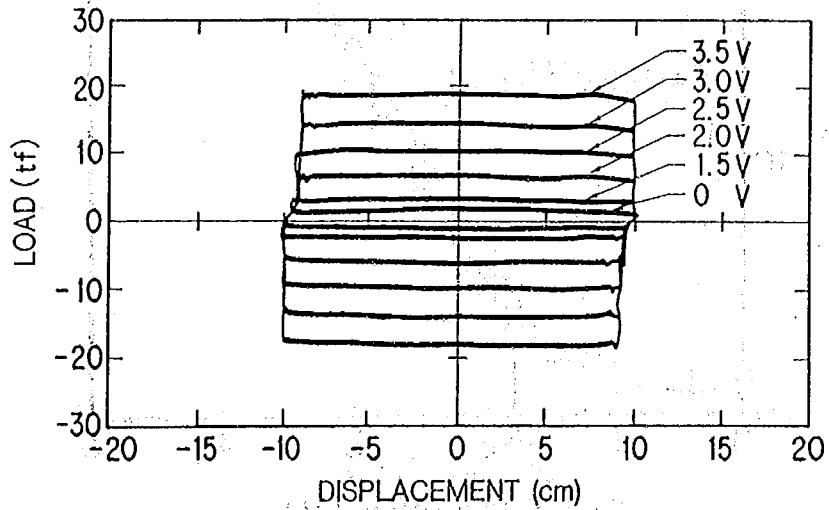
Fig.9 shows some of hysteresis loops of the load-displacement relation obtained. The hysteresis loops are shown for the loading frequency is 0.1Hz and loading displacement of $\pm 1\text{cm}$, $\pm 5\text{cm}$ and $\pm 10\text{cm}$. The variation of hysteresis loops with the input voltage are also shown in Fig.9. The hysteresis loops have the rectangular shape which means that the variable damper has the typical friction type hysteresis loops as designed. Although the damping force instantaneously decreases from specified one to 0 at the instance when the direction of movement of the piston changes at maximum displacement, the time-lag is found at the standing of the damping force. This is because increase of the oil pressure from 0 to specified one takes some time. Turbulence is also found at the instance when the damping force reaches specified one. This may be because the pressure wave motion is developed by



(a) Displacement 1cm



(b) Displacement 5cm



(c) Displacement 10cm

Fig.9 Hysteresis Loops of Load-Displacement Relation (Frequency = 0.1Hz)

sudden standing of the pressure.

(3) The relation between the input voltage and damping force

Fig.10 shows the relation between the damping force and the input voltage. The empirical equation obtained by a least square method is also shown in Fig.10. The damping force varies as a power function of the input voltage when the input voltage is lower than 1.8V. When the input voltage is greater than 1.8V the relation becomes almost linear. Therefore, the empirical equation is assumed as

$$F = \begin{cases} 0.42V^{3.7} + 1.0 & (0 \leq V \leq 1.8) \\ 8.14V - 9.95 & (V > 1.8) \end{cases} \quad (1)$$

where, F and V represent damping force (tf) and input voltage (Volt), respectively. In Eq.(1), the constants are determined as averaged values of those obtained in each loading case.

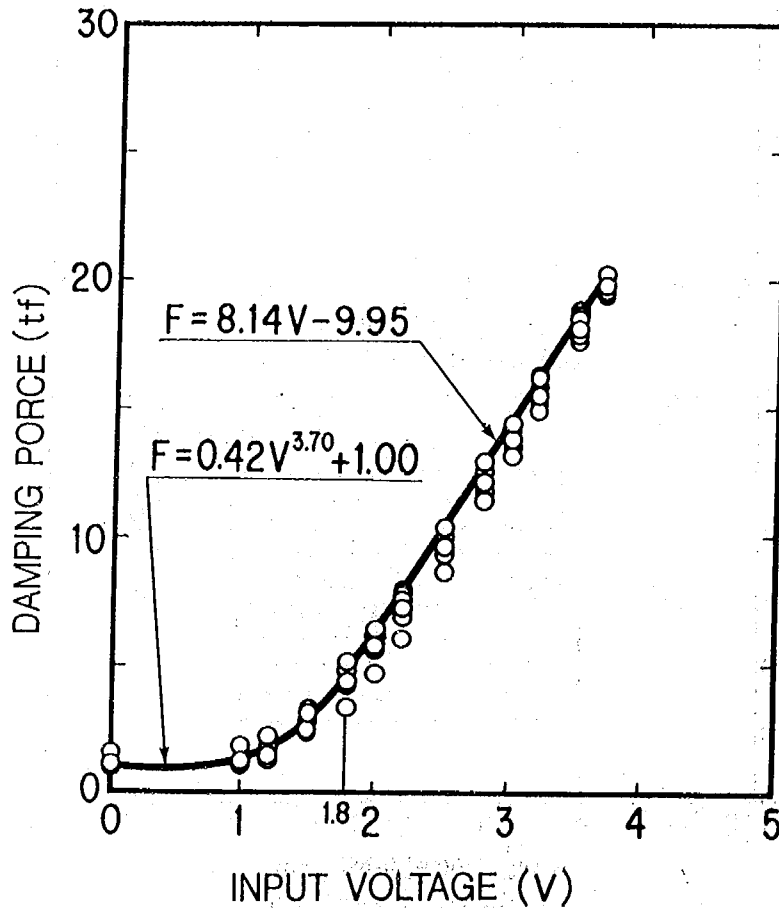
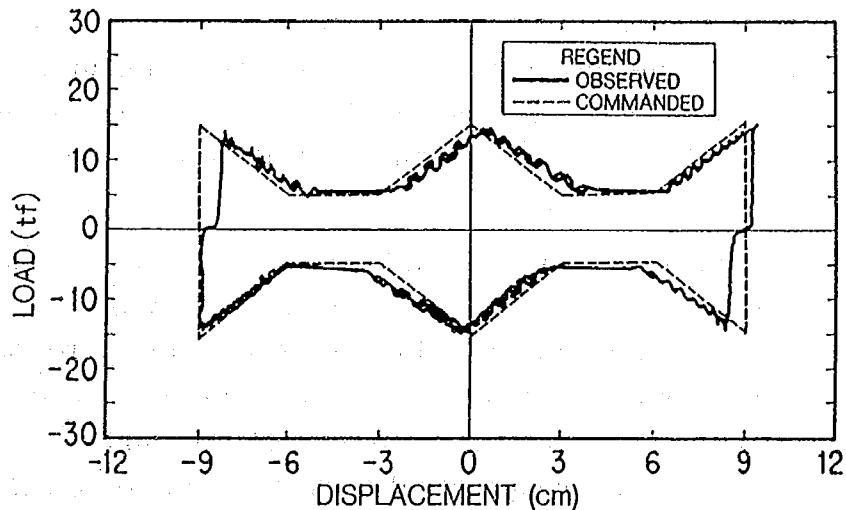


Fig.10 Damping Force vs. Input Voltage Relation

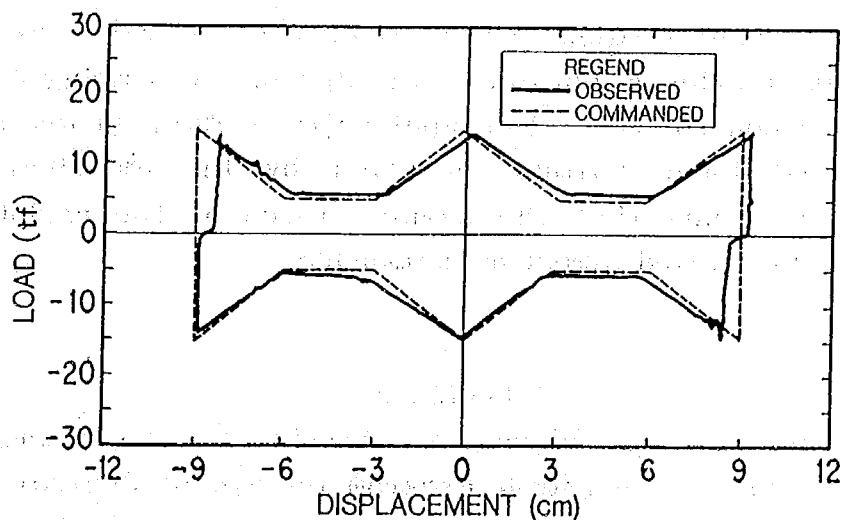
(4) Controllability of the Variable Damper

Based on the above experimental results, the damping force is controlled by the input voltage using Eq.(1). The controllability of the variable damper so as to control damping force as shown in Fig.1 is tested. The control of the variable damper is made by control signal from a personal computer according to the displacement and/or velocity obtained by sensors.

Fig.11 shows hysteresis loops of load-displacement relation to show the controllability of the variable damper. The commanded damping force and the



(a) Sampling Time 0.1sec



(b) Sampling Time 0.01sec

Fig.11 Controllability of the Variable Damper Developed

measured one are shown in Fig.11. Fig.11(a) and (b) show the cases when the sampling time of displacement is assumed as 0.1sec and 0.01sec, respectively. The damping force is found to be controlled almost as commanded. When the sampling time is 0.1 sec, corrugation is found when the damping force is varied with the displacement. Therefore, it is required to use the appropriate sampling time in order to precisely control in consideration with the loading velocity.

CONCLUDING REMARKS

The applicability and controllability of the variable damper developed for reducing seismic response of highway bridges are analytically and experimentally studied. The prototype model of the variable damper was developed and the performance was studied through the dynamic loading tests. According to the above investigations the following conclusions may be deduced:

1) By providing the variable damper with functions of a damper-stopper, an energy dissipator and a stopper, the peak deck displacement and the peak deck acceleration of the model highway bridge were reduced to 25% and 43%, respectively. The peak bending moment at the pier bottom decreased to 46%. The required peak damping force and peak stroke of the variable damper were $62.6\text{tf} \times 2 = 125\text{tf}$ and 5.78cm, respectively. The peak damping force is equivalent with 52% of the deck weight. These capability of the damper can be designed within the current scope of damper technology.

2) Based on the dynamic loading tests of the prototype model developed, the load-displacement relation is obtained as designed. The damping force of the variable damper is controlled by the input voltage. The relation between the damping force and input voltage is given by the empirical equation. Furthermore, it is found that the damping force of the variable damper developed can be controlled almost as commanded.

REFERENCES

- 1) Taguchi, J., Iwasaki, T., Adachi, Y., Sasaki, Y. and Kawashima, K. : U.S.-Japan Cooperative Research Program on Hybrid Control of Seismic Response of Bridge Structures, 22nd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, Maryland, USA, May, 1990
- 2) Kawashima, K., Unjoh, S., Nagashima, H. and Shimizu, H. : Current Research

- Efforts in Japan for Passive and Active Control of Highway Bridges Against Earthquakes, 23rd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, May, 1991
- 2) Kawashima, K., Unjoh, S. and Shimizu, H. : Seismic Response of Highway Bridges by Variable Damper, 24th Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, Maryland, May, 1992
 - 3) Liu, S., C., Lagorio, H., J. and Chong, K., P. : Status of U.S. Research on Structural Control Systems, 23rd Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, May, 1991
 - 4) Kawashima, K., Hasegawa, K. and Nagashima, H. : Experiment and Analysis on Seismic Response of Menshin Bridges, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991
 - 5) Fukuoka, S. : Design of Multi-span Continuous Girder Bridge using Viscous Shear Stopper, Bridge, pp.22-29, February, 1980 (In Japanese)
 - 6) Izeki, H. : Viscous Shear Stopper, Proposal to the Multi-Continuous Girder Bridge, Bridge, pp.30-33, February, 1980 (In Japanese)
 - 7) Matsumura, S., Fukuoka, S., Mizumoto, Y. and Nakata, T. : Seismic Design of Multi-Span Continuous Girder Bridge using Damper Stopper, Bridge and Foundation, pp.31-37, May, 1982 (In Japanese)
 - 8) Milliken, Jr., W., F. : Lotus Active Suspension System, Proceedings of 11th International Conference on Experimental Safety Vehicles, pp.467-475, Washington D.C., May, 1987
 - 9) Sugawara, F. : Electronically Controlled Shock Absorber System Used as a Road Sensor which Utilizes Super Sonic Waves, Proceedings of Society of Automotive Engineers, pp.6.15-6.25, 1986
 - 10) Kawashima, K., Unjoh, S. and Shimizu, H. : Optimum Seismic Response Control of Highway Bridges by Active Mass Damper, Civil Engineering Journal, Vol.33, No.9, pp.61-68, November, 1991 (In Japanese)
 - 11) Kawashima, K., Unjoh, S. and Shimizu, H. : Earthquake Response Control of Highway Bridges by Variable Damper, Proceedings of Colloquium on Control of Structures, Japan Society of Civil Engineers, Part. B, pp.221-224, July, 1991 (In Japanese)
 - 12) Feng, Q. and Shinozuka, M. : Use of A Variable Damper for Hybrid Control of Bridge Response Under Earthquake, U.S. National Workshop on Structural Control Research, University of Southern California, CA., U.S.A., October, 1990
 - 13) Shinozuka, M., Feng, Q. and Kawamura, S. : New Perspectives for Sliding

Base Isolation Systems, Proceedings of International Workshop on Recent Developments in Base-Isolation Techniques for Buildings, Tokyo, Japan, 27-30 April, 1992

- 14) Kawashima, K., Unjoh, S. and Shimizu, H. : Earthquake Response Control of Highway Bridges by Variable Damper, Transactions of the Japan National Symposium on Active Structural Response Control, Japan Science Council, pp.311-317, March, 1992 (In Japanese)
- 15) Kawashima, K., Unjoh, S. and Shimizu, H. : Experimental Study on Dynamic Characteristics of Variable Damper, Transactions of the Japan National Symposium on Active Structural Response Control, Japan Science Council, pp.311-317, March, 1992 (In Japanese)

Manual for Menshin Design of Highway Bridges

Kazuhiko KAWASHIMA

2nd U. S. -Japan Workshop on
Earthquake Protective Systems for Bridges

Tsukuba, Japan

December 1992

1944-1945

1946-1947

1948-1949

1950-1951

1952-1953

1954-1955

MANUAL FOR MENSHEIN DESIGN OF HIGHWAY BRIDGES

Kazuhiko KAWASHIMA

Head, Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan

ABSTRACT

A three-year research program on "Development of Menshin Design of Highway Bridges" was made from April 1989 to March 1992. Concentrated efforts were paid to develop energy dissipating devices and falling-off prevention devices for bridges. An appropriate design method of highway bridges with energy dissipating devices and favorable application of the menshin design to highway bridges were studied. Final accomplishment of the three-year research was compiled in the form of "Manual for Menshin Design of Highway Bridges" in March 1992. This paper outlines the Manuals.

INTRODUCTION

A three-year Joint Research Program between the Public Works Research Institute and 26 groups consisting of 28 private firms on "Development of Menshin Design of Highway Bridges" was made from April 1989 to March 1992. This program intended to develop a rational seismic design method of highway bridges with energy dissipating devices. The highway bridges with span length from 20 m to 100 m were considered as the major target for this Joint Research Program.

As will be discussed later, "menshin" means "reduction of response" in Japanese. Although the menshin design is close with the base-isolation, natural period of bridge is not forcibly elongated in the menshin design, because there are various restrictions for increasing the natural period. Instead of elongating the natural period, emphasis is placed in the menshin design for increasing energy dissipating capability and distribution of lateral force to as many substructures as possible for decreasing lateral force for design of substructures.

As shown in **Table 1**, concentrated efforts were paid to 1) development of

menshin (energy dissipating) devices for highway bridges, 2) development of falling-off prevention devices and expansion joints appropriate for the menshin bridges, 3) development of rational and simple menshin design method and 4) favorable application of menshin design to highway bridges.

In the research project, four high damping rubber bearings, 2 sliding friction dampers, a steel damper, a roller menshin bearing a link bearing and a viscous damper were newly developed for bridges (refer to Fig. 1). The menshin devices for highway bridges have to be more compact and more weather-proof than the base-isolation devices for buildings since the menshin devices are installed at narrow and exposed crest of bridge columns.

The knock-off mechanism at an abutment to ease the impact force induced by the collision between the deck and the abutment (refer to Photo 1), and a finger type expansion joint (refer to Photo 2) which is distinguished from regular finger joints by the transverse movement as well as the longitudinal movement were also developed.

In the research project, a simple but rational menshin design method was developed. A series of shaking table tests were made to study the response of menshin bridges, and to provide realistic data of the seismic response of menshin bridges (refer to Photo 3). Analysis of strong motion records measured at a menshin bridge was also made.

From the study to investigate the favorable application of menshin design, it was found that the menshin design is effective for constructing super-multi-span continuous bridge with total deck length over 1 km. Application of the menshin design for seismic retrofit of existing bridges and deck connection for making existing simply supported girder bridges continuous was also studied (refer to Photo 4).

The reports of the project were compiled and published by the Public Works Research Institute in the form of the Joint Research Report. As well as these reports, as the final accomplishment of the three-year research program, the Manual for Menshin Design of Highway Bridges was compiled.

This paper presents the outline of the Manual.

SCOPES AND CONTENTS OF THE MANUAL

The Manual presents the seismic design method of highway bridges including

design details when "the Structures Expecting Reduction of Inertia Force", which is specified in the "Part V Seismic Design" of the "Specifications of Design of Highway Bridges", is adopted. Application of the menshin design to seismic strengthening of substructures and connection of adjacent decks for making existing simply supported girder bridges continuous is also presented.

It should be noted that although the description of the Manual is given in the format of the specifications, it merely represents the accomplishments of the three-year research program, and it is not the mandate specifications.

The table of contents of the Manual is as follows:

1. General
 - 1.1 Scope of Application
 - 1.2 Definition of Terms
2. Basic Principle of Menshin Design
3. Menshin Design
 - 3.1 General
 - 3.2 Lateral Force Coefficient for Seismic Coefficient Method
 - 3.3 Lateral Force Coefficient and Equivalent Lateral Force Coefficient for Bearing Capacity Method
 - 3.4 Characteristics value for Menshin Devices
 - 3.5 Computation of Natural Period of Bridge
 - 3.6 Computation of Modal Damping Ratio of Bridges
4. Design of Menshin Devices
 - 4.1 Basic Principle for Design of Menshin Devices
 - 4.2 Design of Laminated Rubber Bearings
 - 4.3 Design of Lead Rubber Bearings
 - 4.4 Design of High Damping Rubber Bearings
 - 4.5 Design of Steel Dampers
 - 4.6 Design of Friction Dampers
 - 4.7 Design of Viscous Dampers
 - 4.8 Design of Roller Dampers
5. Dynamic Response Analysis
 - 5.1 General
 - 5.2 Dynamic Response Analysis Method
 - 5.3 Analytical Models
 - 5.4 Design Seismic Forces

- 5.5 Check of Safety
6. Design Details
 - 6.1 General
 - 6.2 Gap between Structures
 - 6.3 Falling-off Prevention Devices
 - 6.4 Knock-off Devices
 - 6.5 Details of Expansion Joints
7. Approving Tests of Menshin Devices
 - 7.1 General
 - 7.2 Approving Tests for Confirming Requirements for Dynamic Loads
 - 7.3 Approving Tests for Confirming Requirements for Static Loads
8. Seismic Retrofit of Existing Bridges with Use of Menshin Design
9. Application of Menshin Design for Making Existing Simply Supported Girder Bridges Continuous

Appendices

1. Procedure of Menshin Design
2. Menshin Devices for Highway Bridges
3. References for Dynamic Response Analysis
4. Expansion Joints and Falling-off Prevention Devices for Highway Bridges
5. Design Example of Prestressed Concrete Bridges
6. Design Examples of Steel Girder Bridges
7. Design Examples of Super-multi-span Continuous Bridges
8. Design Examples for Seismic Retrofit
9. Design Examples for Making Existing Simply Supported Girders Continuous
10. Standard Acceleration Ground Motions Recommended for Dynamic Response Analysis

BASIC PRINCIPLE OF MENSIN DESIGN

Whether the menshin design should be adopted or not has to be decided based on the advantages of increasing energy dissipating capability from not only seismic safety but function at normal time.

In general, menshin design can be adopted with advantage for the following conditions:

- 1) bridges on stiff and stable soils
- 2) bridges with stiff substructures with short natural period

It is recommended that the natural period of the bridge designed by the menshin design is at least 1.5 times longer than that of the bridge without menshin devices. Such elongation of the natural period of the menshin bridge makes the coupling vibration so small that the deformation of the bridge be concentrated at the menshin devices.

The most important decision of the menshin design is how much increase of the natural period be made. In Japan soils are generally very weak at the bridge construction sites. Significant earthquakes with magnitude over 8, which produce long period ground motions, occur at shorter recurrence period than other countries. Taking account such evidences into account, the design seismic force level is take very high for design of highway bridges in Japan in comparison with the seismic force level adopted in U.S.A., New Zealand and Italy. The natural period at which the design acceleration decrease with increasing the natural period is taken longer, i.e., 1.1 second for type I soils (stiff site), 1.3 second for type II soils (medium site) and 1.5 second for type III soils (soft soils). Therefore, the advantage for reducing deck response can be realized only by elongating the natural period longer than 1.1 ~ 1.5 second.

However elongation of the natural period brings the increase of the deck response displacement, and this requires the adoption of joints with long legs. This is, however, a crucial requirements difficult to adopt, in particular, for overcrossing in city area because it causes noise and vibration pollution, which is the most serious concern in city area. Furthermore it will have the disadvantage for maintenance because replacement of the damaged expansion joint due to traffic load as well as the pavement adjacent to the joint has to be frequently made.

Based on these reasons, it is not adopted to forcibly elongate the natural period in order to decrease the deck response acceleration. It is therefore more appropriate to adopt the concept of base isolation not for elongating the natural period but for increasing energy dissipating capability. Distribution of the inertia force to as many substructures as possible, which is the practice adopted in Japan for long time, is also significantly advantageous.

Therefore basic principle recommended for introducing the base isolation to highway bridges in Japan is as follows :

- 1) Distribute the inertia force to many substructures so intentionally that the inertia force be distributed equally by selecting the spring stiffness of the isolators. Simultaneously, reduce deck response by increasing energy dissipating capability by means of damper.
- 2) Do not forcibly elongate the natural period, but select the natural period so that the resonance of the bridge with soils be avoided.

Such design criteria is no more the "isolation" of the "base". Therefore it is proposed to call the design concept as "menshin". The "menshin" means "reduction of seismic response" in Japanese.

In menshin design, bridges can be designed by following the standard static design method (static frame analysis). More precise approve of the seismic safety can be made by the dynamic response analysis, if required. In such analyses, the menshin devices (isolators and dampers) are idealized by as a set of equivalent linear springs. Equivalent stiffness and the equivalent damping ratio of the isolator and damper are the major parameters used in the analyses. In the static frame analysis, natural period of the bridge can be computed for each seismic design structural unit as :

$$T = 2.01 \sqrt{\delta} \quad (1)$$

$$\delta = \frac{\sum w_i \cdot u_i^2}{\sum w_i \cdot u_i} \quad (2)$$

where

T : natural period (sec)

w_i : dead weight (tf/m) of the seismic design structural unit (super structure and substructure above the ground surface assumed in seismic design) at point "i"

u_i : lateral displacement (m) developed in the seismic design structural unit at point "i" when subjected to w_i in the direction considered in design

Damping ratio of the bridges is computed as

$$h = \frac{\sum K_{Bi} \cdot u_{Bi}^2 \cdot c_{hi}}{\sum K_{Bi} \cdot u_{Bi}^2 \cdot c_i} \quad (3)$$

$$c_{hi} = h_{Bi} + \frac{h_{Pi}}{K_{Pi}} + \frac{h_{Fui}}{K_{Fui}} + \frac{h_F \theta_i \cdot H^2}{K_F \theta_i}$$

$$c_i = 1 + \frac{K_{BI}}{K_{PI}} + \frac{K_{BI}}{K_{FUI}} + \frac{K_{BI} \cdot H^2}{K_{FOI}}$$

where

h :Modal damping ratio of bridge

h_{BI} :Damping ratio of i-th damper

h_{PI} :Damping ratio of i-th pier/abutment

h_{FUI} :Damping ratio of i-th foundation associated with translational movement

h_{FOI} :Damping ratio of i-th foundation associated with rotation

K_{PI} :Equivalent stiffness of i-th pier/abutment

K_{FUI} :Translational stiffness of i-th foundation

K_{FOI} :Rotational stiffness of i-th foundation

u_{BI} :Design displacement of i-th menshin device

H :Height from the bottom of pier to the gravity center of deck

Eq.(3) give the approximate estimation of the damping ratio of bridge assuming that translation and rotation of the foundation are developed only by the inertia force of the deck. When the effect of the inertia force of piers and abutments for evaluating the translation and rotation of the foundation can not be disregarded, it is appropriate to compute the modal damping ratio as

$$h = \frac{\sum_{j=1}^n \phi_{ij}^T \cdot h_j \cdot k_j \cdot \phi_{ij}}{\phi_i^T \cdot K \cdot \phi_i} \quad (4)$$

where

ϕ_{ij} :Mode vector of j-th structural component for i-th mode

h_j :Damping ratio of j-th structural component

k_j :Stiffness matrix of j-th structural component

ϕ_i^T :Mode vector of bridge for i-th mode

K :Stiffness matrix of bridge

Table 2 shows the damping ratio recommended for structural components for Eqs.(3) and (4).

DESIGN FORCE FOR MENSIN DESIGN

In menshin design, the menshin devices are designed by the Seismic Coefficient Method and the Bearing Capacity Method. In both method, the lateral force is

statically applied to the bridge, and the seismic safety is examined based on the allowable stress design approach in the Seismic Coefficient Method and bearing capacity basis considering ductility in the Bearing Capacity Method. Bridges are designed by the Seismic Coefficient Method, and then the ductility is checked for reinforced concrete piers by the Bearing Capacity Method.

In the Seismic Coefficient Method, the design lateral force coefficient k_h is given as

$$k_h = c_z \cdot c_g \cdot c_i \cdot c_T \cdot c_E \cdot k_{h0} \geq 0.1$$

where

$$c_T \cdot c_E \geq 0.8 \quad (5)$$

where

k_h : Lateral force coefficient

c_z : Modification factor for zone (refer to Fig. 2)

c_g : Modification factor for ground condition (refer to Table 3)

c_i : Modification factor for importance (refer to Table 4)

c_T : Modification factor for structural response (refer to Table 5)

c_E : Modification factor for energy dissipation capability (refer to Table 6)

k_{h0} : Standard design horizontal seismic coefficient (= 0.2)

Those factors excluding c_E are the ones specified in the Specifications of Design of Highway Bridges. The modification coefficient c_E takes a value as shown in Table 6 depending on the first modal damping ratio of the bridge. Reduction of the design lateral force as large as 20 % is proposed in the Manual.

In the Bearing Capacity Method, the lateral force coefficient k_{h0} and the equivalent lateral force coefficient k_{he} are given as

$$k_{he} = \frac{k_{h0}}{\sqrt{(2\mu - 1)}} \geq 0.3 \quad (6)$$

$$k_{h0} = c_z \cdot c_i \cdot c_R \cdot c_E \cdot k_{h00} \quad (7)$$

where

k_{he} : Equivalent lateral force coefficient for Bearing Capacity Method

k_{h0} : Lateral force coefficient for Bearing Capacity Method

- c_z :Modification factor for zone (refer to Fig. 2)
 c_i :Modification factor for Importance (refer to Table 4)
 c_R :Modification factor for structural response (refer to Table 7)
 c_E :Modification factor for energy dissipation capability
 k_{ho} :Standard lateral force coefficient for Bearing Capacity Method (=1.0)
 μ :Allowable ductility factor of reinforced concrete piers

The modification factors excluding c_E are the ones specified in the Specifications of Design of Highway Bridges. The modification factor c_E depends on the modal damping ratio of the bridge, and takes a value of Table 8. Reduction of the design force as large as 30 % is proposed.

DESIGN OF MENSIN DEVICES

Characteristics Value of Menshin Devices

In the menshin design, the displacement assumed in design for the devices u_B (design displacement of menshin device), the equivalent stiffness K_B and the equivalent damping ratio h_B of the menshin device are the key factors. They are evaluated as:

(1) Design Displacement of Menshin Device

The design displacement of menshin device is evaluated as

$$u_B = \frac{k_h \cdot W_u}{K_B} \quad (\text{S. C. Method}) \quad (8)$$

$$u_B = \frac{k_{ho} \cdot W_u}{K_B} \quad (\text{B. C. Method}) \quad (9)$$

where

- u_B :Design displacement (m) of menshin device
 k_h :Lateral force coefficient by Eq.(5) for Seismic Coefficient Method
 k_{ho} :Lateral force coefficient by Eq.(6) for Bearing Capacity Method
 K_B :Equivalent stiffness (tf/m) of menshin device
 W_u :Weight of superstructure (tf) supported by the menshin device

(2) Equivalent Stiffness and Equivalent Damping Ratio

The equivalent stiffness and the equivalent damping ratio of menshin device are evaluated from the hysteresis loops as

$$K_E = \frac{F(u_{BE}) - F(-u_{BE})}{2u_{BE}} \quad (10)$$

$$h_E = \frac{\Delta W}{2\pi W} \quad (11)$$

where

K_E :Equivalent stiffness (tf/m) of menshin device

h_E :Equivalent damping ratio of menshin device

u_{BE} :Effective design displacement (m) of menshin device, and is given as

$$u_{BE} = c_E \cdot u_E \quad (12)$$

u_E :Design displacement (m) of menshin device

c_E :Modification coefficient for evaluating the effective design displacement (=0.7)

$F(u)$ Lateral force required to produce u for the menshin device

W :Strain energy induced in menshin device associated with c_E displacement (refer to Fig. 3)

ΔW :Energy dissipated in menshin device per cycle (refer to Fig. 3)

Requirements for Dynamic Load

(1) Menshin devices have to be designed and fabricated so that their equivalent stiffness K_E and equivalent damping ratio h_E be within $\pm 20\%$ of the design values. The equivalent stiffness K_E and equivalent damping ratio h_E for this requirement have to be evaluated by Eqs. (10) and (11), and they shall be the ones averaged over K_E and h_E from 4th to 10th loading hysteresis among 10 cyclic loading reversals with harmonic displacement of c_E . This is because deck acceleration and displacement are within tolerable range when the scatter of equivalent stiffness and the equivalent damping ratio are within $\pm 20\%$ of the design values. However, the devices which produce unstable hysteretic behavior should not be adopted.

(2) Menshin devices have to be stable against 50 cycles of harmonic loading with displacement of u_E . The number of 50 was determined from the fact that number of load reversals developed during a major earthquake may be about 30. About 50% tolerance was included in 50.

(3) Deck should return to the rest position even after it was subjected to a large earthquake. Therefore the residual displacement developed in menshin

devices after smoothly releasing from the deformed displacement of u_B by Eq. (9) has to satisfy

$$u_{ER} \leq 0.1 \cdot u_B \quad (13)$$

where

u_{ER} : Residual displacement (cm)

u_B : Design displacement (cm) of menshin device by Eq. (9)

(4) The equivalent stiffness and the equivalent damping ratio of menshin devices have to be stable against the change of load condition at normal time, change of natural environment such as the temperature change and cyclic loading developed by an earthquake. Stability has to be examined against the following requirements:

- 1) cyclic loading due to elongation and shrinkage of deck due to the temperature change and the active load
- 2) effect of loading hysteresis
- 3) variation of vertical loading
- 4) effect of loading rate
- 5) effect of pre-deformation due to creep and shrinkage
- 6) direction of excitation
- 7) change of the equivalent stiffness and the equivalent damping ratio depending on the temperature change

Requirements for Static Load

- (1) Materials and mechanism of menshin devices have to give credit to long term use. They have to be stable against cyclic elongation and shrinkage of deck due to temperature change.
- (2) Menshin devices have to be stable against local shear strain. Check of the local shear strain has to be made in accordance of "3.6 Design of Rubber Bearings" of the "Design Guidelines of Bearings".
- (3) For rubber-type menshin devices, the creep of rubber in vertical direction due to dead weight of superstructure shall not exceed 5 % of total thickness of the rubber.
- (4) The equivalent stiffness of menshin device at -10°C normalized by the equivalent stiffness at 40°C shall not exceed 1.5. Because the temperatures of -10°C and 40°C is for moderate climate area, at cold area they have to be

decided based on the appropriate site condition.

(5) Menshin devices have to have appropriate initial stiffness so that harmful displacement of deck due to non-seismic lateral force such as wind effects be avoided.

APPROVING TESTS OF MENSIN DEVICES

General

Because the menshin device is one of the the important structural components of menshin bridges, their requirements have to be well approved by the tests. There are various types of menshin devices for use of bridges. Based on the mechanism for producing the energy dissipation, the devices may be classified into three groups as

- (a) displacement dependent type,
- (b) friction force type, and
- (c) velocity dependent (viscous) type.

The approving tests for the menshin devices are presented in the Manual for each of the three types.

Approving Tests for Dynamic Loads

For approving the requirements for dynamic loads presented in 5.2, the following approving tests are provided in the Manual.

- (1) Approving test for confirming the equivalent stiffness and the equivalent damping ratio
- (2) Approving test for confirming stableness against 50 cycles harmonic load reversals with u_E displacement
- (3) Approving test for confirming residual displacement requirement
- (4) Approving tests for confirming stableness against variation of load condition at normal time, change of natural environment such as the temperature change and cyclic loading developed by an earthquake. These tests include :

- (a) Stableness against Cyclic Load Reversals

Some menshin devices have the dependence of the equivalent stiffness and the

equivalent damping ratio on number of cyclic loading. Because considerable change of these parameters during an earthquake produces bridge response significantly different from the one assumed in design, it is not desirable.

(b) Stableness against Load Hysteresis

In some menshin devices, the equivalent stiffness and the equivalent damping ratio depend on the past experience, in particular, the largest deformation ever experienced. Such dependence of K_B and h_B on the hysteresis ever experienced is significant, it produces different bridge response during an earthquake. Therefore, effect of load hysteresis has to be examined.

(c) Stableness against Change of Compression Force

Compression force applied to the menshin device due to dead weight of superstructure may change associated with the error involved in construction stage and settlement of substructure. If the menshin device shows significantly different stiffness and damping properties due to such change of compression force, it brings different structural response. Therefore, stableness of the equivalent stiffness and the equivalent damping ratio of menshin devices have to be checked.

(d) Stableness against Change of Loading Rate

Because loading rate to the menshin devices is not the same during excitation, dependence of the equivalent stiffness and the equivalent damping ratio on the loading rate has to be examined.

(e) Stableness against Pre-deformation

Menshin devices often start to deform from the displacement drifted to the rest point. This is actually developed if an earthquake occurs when the menshin devices deform due to the deck elongation associated with temperature change and shrinkage of concrete. Because the equivalent stiffness and the equivalent damping ratio depend on the amount of such pre-deformation, their properties have to be checked.

(f) Stableness against Temperature Change

Menshin device have to be stable against dally and yearly change of temperature.

Approving Tests for Static Loads

For approving the requirements for static loads presented in 5-3, the following tests are required.

(1) Approving test for confirming durability and stableness against cyclic loading associated with the daily and yearly elongation and shrinkage of deck.

(2) Test for evaluating the stiffness of menshin devices subjected to extremely low-rate deformation such as the one encountered due to yearly temperature change and concrete creep. The stiffness of the menshin device during such low-rate deformation is quite important in static design for computing the lateral force developed in substructures due to the temperature change and shrinkage of concrete.

(3) Approving test for confirming the dependence of the equivalent stiffness of menshin device on temperature.

Two examples of the Approving Test

Approving test methods presented in 6.2 and 6.3 are precisely described in the Manual for each of the displacement dependent type devices, friction force type devices and the velocity dependent type devices. As the example of such approving tests, the test method for confirming stableness against cyclic loading and the test method for confirming the effect of loading rate are briefly described for the displacement dependent type devices in the following.

(1) Approving Test for Confirming Stableness against Cyclic Loads

At the room temperature of 20°C apply 10 cycles of harmonic lateral load to the menshin device with loading displacement of u_B computed by Eq. (9) or smaller and with the frequency of 0.5 Hz. The vertical load equivalent to the design dead load shall be simultaneously applied. The equivalent stiffness and the equivalent damping ratio from the test have to satisfy Eqs. (14) and (15).

$$R_{KC} = \frac{|K_{BJ} - K_{Bm}|}{K_{Bm}} \begin{cases} \leq 0.3 \quad (j=1,2,3) \\ \leq 0.1 \quad (j=4,5 \dots 10) \end{cases} \quad (14)$$

$$R_{hC} = \frac{|h_{BJ} - h_{Bm}|}{h_{Bm}} \begin{cases} \leq 0.3 \quad (j=1,2,3) \\ \leq 0.1 \quad (j=4,5 \dots 10) \end{cases} \quad (15)$$

where

R_{KC} :Variation ratio of equivalent stiffness

R_{hC} :Variation ratio of equivalent damping ratio

K_{BJ} :Equivalent stiffness at j-th load reversal(tf/m)

K_{Em} :Averaged equivalent stiffness by Eq. (16)

h_{BJ} :Equivalent damping ratio at j-th load reversal

h_{Em} :Averaged damping ratio by Eq. (17)

$$K_{Em} = \frac{1}{7} \sum_{j=4}^{10} K_{BJ} \quad (16)$$

$$h_{Em} = \frac{1}{7} \sum_{j=4}^{10} h_{BJ} \quad (17)$$

Being different with Eqs. (10) and (11), the equivalent stiffness and the equivalent damping ratio for the approving tests are defined as

$$K_E = \frac{F(u_E) - F(-u_E)}{2u_E} \quad (18)$$

$$h_E = \frac{\Delta W}{2\pi W} \quad (19)$$

where

K_E :Equivalent stiffness (tf/m) of menshin device

h_E :Equivalent damping ratio of menshin device

u_E :Design displacement (m) of menshin device

$F(u)$ Lateral force (tf) required to produce u displacement for the menshin device

W :Strain energy (tfm) induced in menshin device associated with u_E displacement (refer to Fig. 4)

ΔW :energy dissipated in menshin device per cycle (refer to Fig. 4)

(2) Approving Test for Confirming the Stableness against Loading Rate

At the temperature of 20 °C apply 10 cycles of harmonic load with the displacement equivalent to either 100 % of the total thickness of rubber or ± 15 cm. Frequency of the load reversal shall be 0.1 Hz, 0.5 Hz and 1.0 Hz. The equivalent stiffness and the equivalent damping ratio have to satisfy Eqs. (20) and (21).

$$R_{Kt} = \frac{|K_{Em1} - K_{Emj}|}{K_{Em1}} \leq 0.2 \quad (j=1,2) \quad (20)$$

$$R_{kr} = \frac{|h_{Bm1} - h_{Bmj}|}{h_{Bm1}} \leq 0.2 \quad (j=1,2) \quad (21)$$

where

R_{kr} : Variation ratio of equivalent stiffness

R_{hr} : Variation ratio of equivalent damping ratio

$K_{Bm1}, K_{Bm2}, K_{Bm3}$: Equivalent stiffness by Eq. (16) at frequency of 0.5 Hz, 0.1 Hz and 1.0 Hz, respectively

$h_{Bm1}, h_{Bm2}, h_{Bm3}$: Equivalent damping ratio by Eq. (17) at frequency of 0.5 Hz, 0.1 Hz and 1.0 Hz, respectively

CONCLUDING REMARKS

Excluding specially long-span bridges, it is not easily adopted for highway bridges to reduce the lateral force by forcibly elongating the natural period, because seismic force level is very high even at long natural period, and because the increase of deck response displacement brings various problems such as environmental pollution. Therefore the menshin design, which places emphasis not on the forcible elongation of the natural period but on the increase of energy dissipating capability and distribution of inertia force of superstructure to as many substructures as possible, is proposed. By the menshin design, it is proposed to decrease the lateral force as large as 20 % for the Seismic Coefficient Method and 30 % for the Bearing Capacity Method.

ACKNOWLEDGEMENTS

For executing the three-year joint research project, various supports and encouragement were obtained from many organizations including the Head-quarters of the Ministry of Construction. Without endeavour of more than 130 participants from the Public Works research Institute and 28 private firms, this project could not be successfully made. The author expresses his sincere thanks for all who participated in this Joint Research Program.

REFERENCES

- 1) Public Works Research Institute and 28 Private Firms : Report of the Joint Research on Menshin Systems for Highway Bridges, No. 1, July 1990, No. 2,

- July 1991 and No. 3, March 1992, Joint Research Report, No. 44, No. 60 and No. 75, Public Works Research Institute (In Japanese)
- 2) Public Works Research Institute and 28 Private Firms : Manual for Menshin Design of Highway Bridges (Draft), March 1992, Joint Research Report, Public Works Research Institute (In Japanese)
 - 3) Japan Road Association : Part V Seismic Design of the Specifications of the Design of Highway Bridges, February 1990 (In Japanese)
 - 4) Technology Research Center for National Land Development : Guidelines for Base Isolation of Highway Bridges, March 1989 (In Japanese)
 - 5) Japan Road Association : Guide-Specifications of Design of Bearings, July 1991 (In Japanese)
 - 6) Buckle, I. G. and Mayes, R. L. : Seismic Isolation : History, Application and Performance - A World View, Earthquake Spectra, Vol. 6, No. 2, May 1990
 - 7) AASHTO : Guide Specifications for Seismic Isolation Design, June 1991
 - 8) McKay, G. R., Chapman, H. E. and Kirkcaldie, D. K. : Seismic Isolation : New Zealand Application, Earthquake Spectra, Vol. 6, No. 2, May 1990
 - 9) Guidelines for Seismic Design of Bridges with Isolation/Dissipator Devices, Antiseismic Protection of Structures on the Autostrade S.p.A/ Network, October 1991
 - 10) Kawashima, K., Hasegawa, K. and Nagashima, H. : A Perspective of Menshin Design for Highway Bridges in Japan, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991
 - 11) Kawashima, K., Hasegawa, K. and Nagashima, H. : Experiment and Analysis on Seismic Response of Menshin Bridges, Proc. 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991

**Table 1 Research Theme and Organizations of Joint Research Program
between the Public Works Research Institute and 28 Private
Firms for Developing Menshin Design Method of Highway Bridges**

Research Theme	P	Ka	Si	Ob	Ku	Tn	H	Ni	Su	M	G	On	Ti	I	Nk	Ko	Ns	Ol	Y	To	Bs	Bb	Se	Sh	Pc	J	N	Chief	Sub-Chief
1. Development of Device for Isolation																													
1.1 High Energy Absorbing Rubber Bearing								○											○	○	○		○					Ol	Bs
1.2 Friction Damper								○					○									○	○						
1.3 Steel Damper								○							○														
1.4 Link Bearing Develop of								○									○												
1.5 Viscous Damper								○									○	○											
1.6 Test Method	○							○							○		○	○	○	○	○	○	○	○					
2. Development of Expansion Joint and Falling-off Prevention Device for Isolated Bridge																											Tc	—	
2.1 Expansion Joint				○									○																
2.2 Falling-off Prevention Device	○												○					○											
3. Development of Design Method for Isolated Bridge																													
3.1 Design Philosophy	○	○	○	○		○						○	○					○							○	○	○	Ka	Pc
3.2 Dynamic Responce Analysis Method	○		○		○	○		○			○		○	○				○											
3.3 Design Method of Device for Isolation	○						○	○				○	○		○		○	○	○	○	○	○	○	○					
3.4 Simplified Design Method	○	○	○		○	○					○	○						○											
3.5 Design Method of Expansion Joint and Fall-ing-off Prevention Device	○			○			○							○			○							○					
4. Application of Base Isolation to Bridge																													
4.1 Application to Prestressed Concrete Bridge	○	○							○	○			○													○			
4.2 Application to Steel Bridge	○													○	○	○									○	○	Ob	J	
4.3 Application to Multiple Super-long Bridge	○			○					○							○									○	○			
4.4 Application to Seismic Retrofit	○									○				○				○							○	○			

P: Public Works Research Institute, Ka: Kajima, Si: Shimizu, Ob: Ohbayashi, Ku: Kumagai, Tn: Takenaka Doboku + Takenaka, H: Hazama, Ni: Nishimatsu, Su: Sumitomo, M: Mitsui, G: Goyoh, Ok: Okamura, Ti: Taisei + Tokyo Fablic + Nippon Chuza, I: Ishikawajima Harima, Nk: NKK + Nippon Chuza, Ko: Kobe Steel, Ns: Nippon Seiko, Oe: Oiles, Y: Yokohama Rubber, To: Toyo Rubber, Bs: Bridgestone, Bb: BBM, Se: Seibu Polymer, Sh: Showa Densen, Pc: Pacific Consultants, J: Japan Engineering Consultant, N: New Structural Engineering Consultants.

Table 2 Damping Ratio Recommended for Structural Components

Structural Components	Steel	Concrete
Superstructures	0.02~0.03	0.03~0.05
Menshin Device	Damping Ratio by Eq.(11)	
Pier/Columns	0.03~0.05	0.05~0.1
Footing	0.1~0.3	

Table 3 Modification Factor for Ground Condition c_g

Ground Group	I	II	III
c_g	0.8	1.0	1.2

Table 4 Modification Factor for Importance c_i

Group	c_i	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road, Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 5 Modification Factor for Structural Response c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$ $c_T = 2.69T^{1/3} \geq 1.00$	$0.1 \leq T \leq 1.1$ $c_T = 1.25$	$1.1 < T$ $c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$ $c_T = 2.15T^{1/3} \geq 1.00$	$0.2 \leq T \leq 1.3$ $c_T = 1.25$	$1.3 < T$ $c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$ $c_T = 1.80T^{1/3} \geq 1.00$	$0.34 \leq T \leq 1.5$ $c_T = 1.25$	$1.5 < T$ $c_T = 1.64T^{-2/3}$

Table 6 Modification Factor for Energy Dissipation Capability c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$h \geq 0.1$	0.9

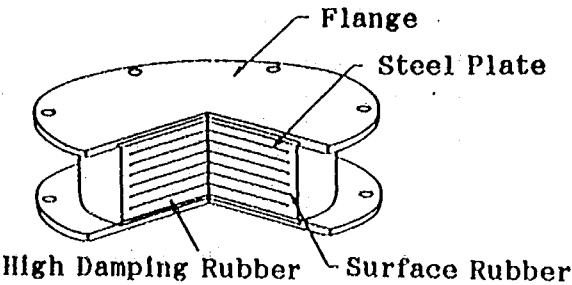
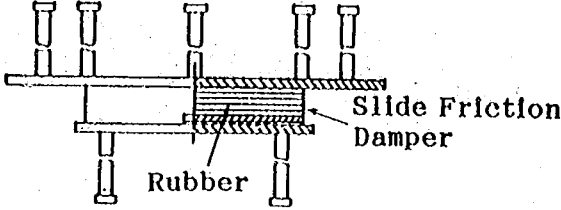
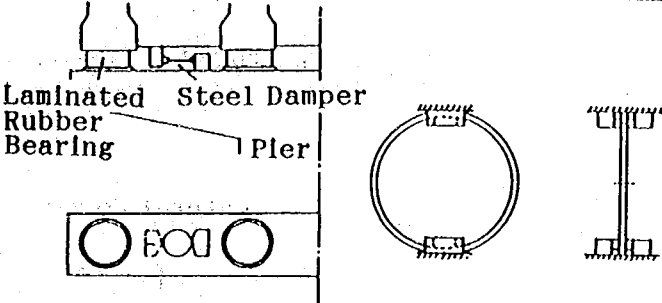
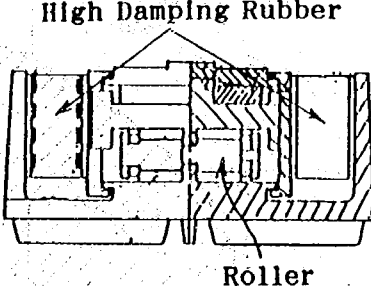
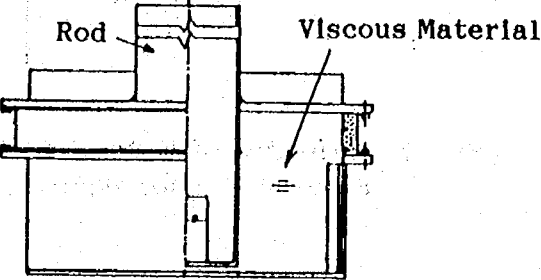
Table 7 Modification factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leq 1.4$ $c_R = 0.7$		$1.4 < T_{EQ}$ $c_R = 0.876T_{EQ}^{-2/3}$
Group II	$T_{EQ} < 0.18$ $c_R = 1.51 T_{EQ}^{1/3} \geq 0.7$	$0.18 \leq T_{EQ} \leq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51T_{EQ}^{1/3} \geq 0.7$	$0.29 \leq T_{EQ} \leq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59T_{EQ}^{-2/3}$

Table 8 Modification Factor for Energy Dissipating Capability

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$0.1 \leq h < 0.12$	0.9
$0.12 \leq h < 0.15$	0.8
$0.15 \leq h$	0.7

Fig. 1 Menshin Devices Developed

TYPE	OUTLINE OF MENSIN DEVICE
HIGH DAMPING RUBBER BEARING	
SLIDE FRICTION -RUBBER BEARING	
STEEL DAMPER	
ROLLER MENSIN BEARING	
VISCIOUS	

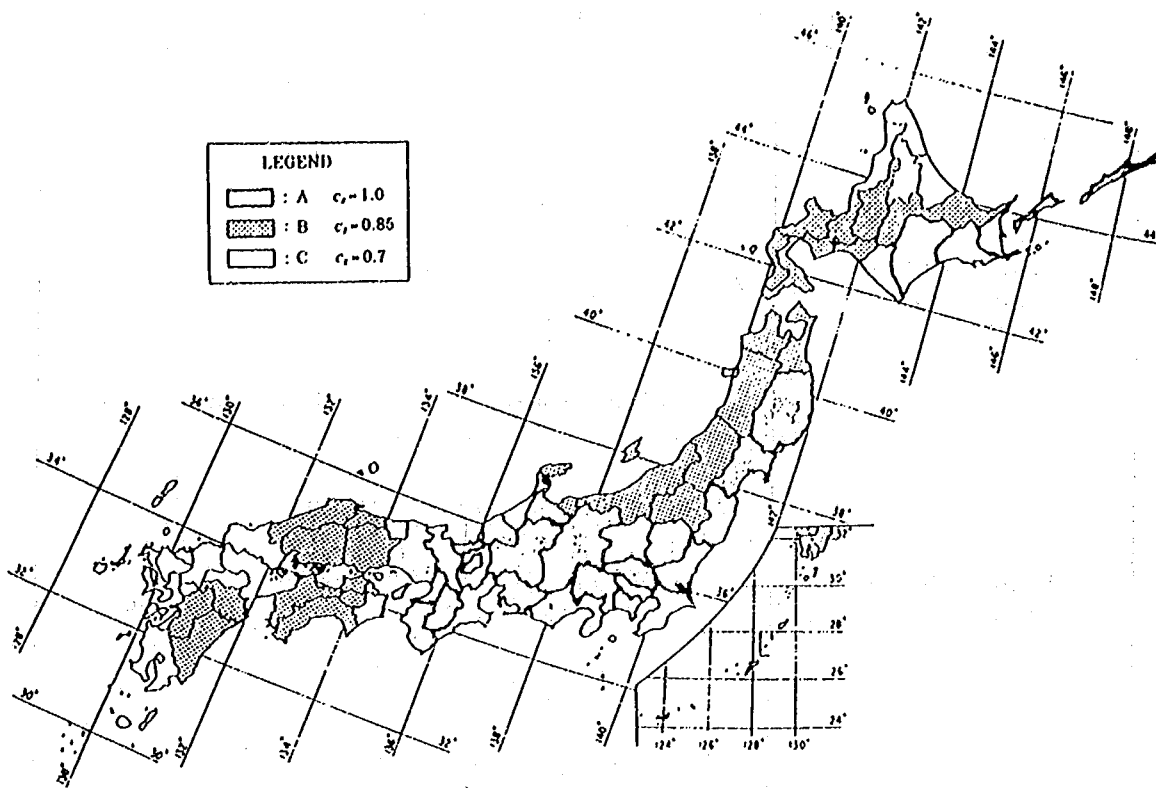


Fig. 2 Modification Factor for Zone c z

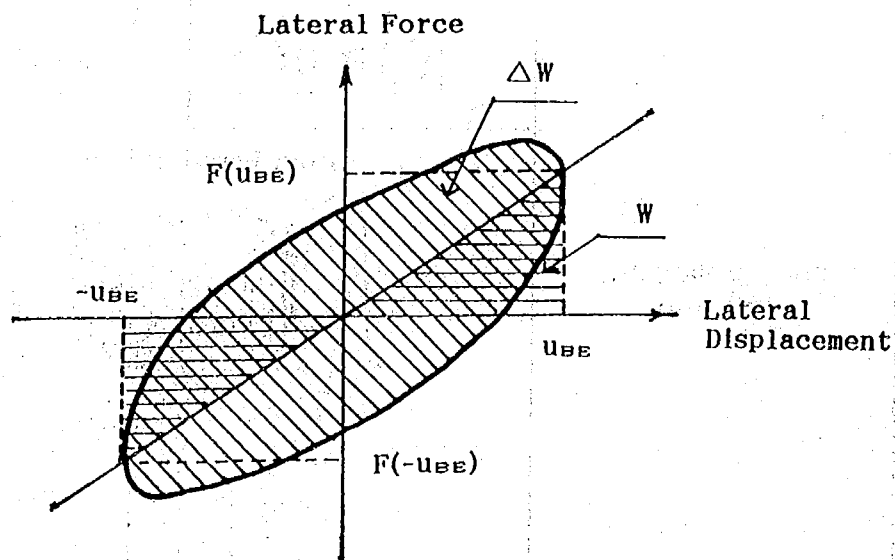


Fig. 3 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menshin Device for Design of Menshin Device

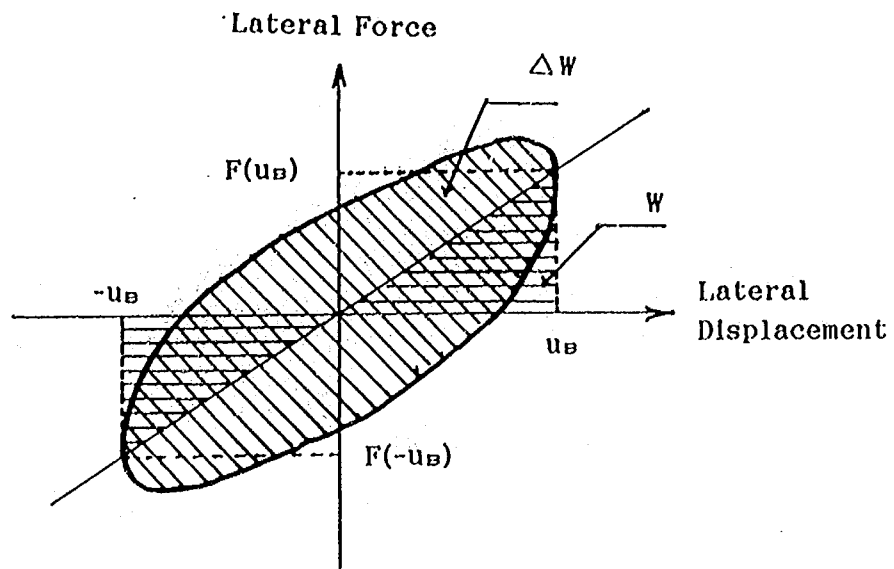


Fig. 4 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menshin Device for Approving Tests

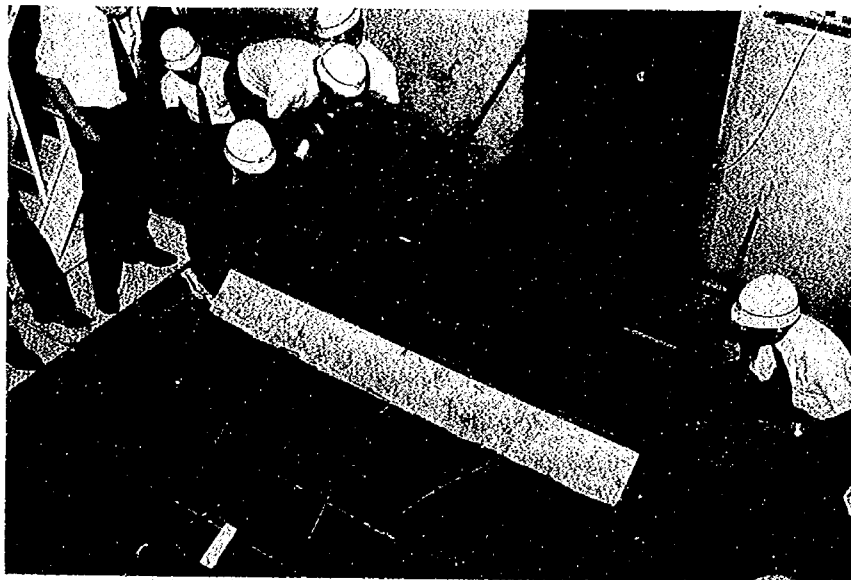


Photo 1 Knock-off Abutment

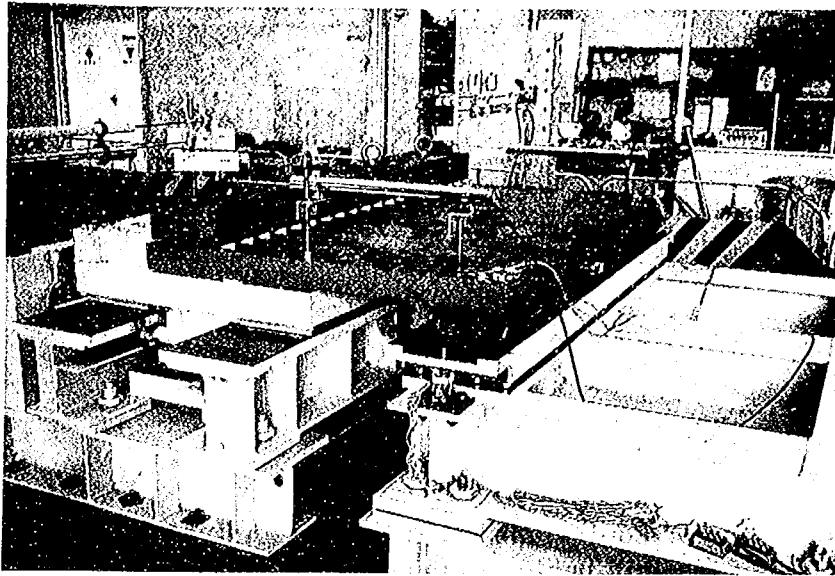


Photo 2 Expansion Joint Movable In Longitudinal and Transverse Directions

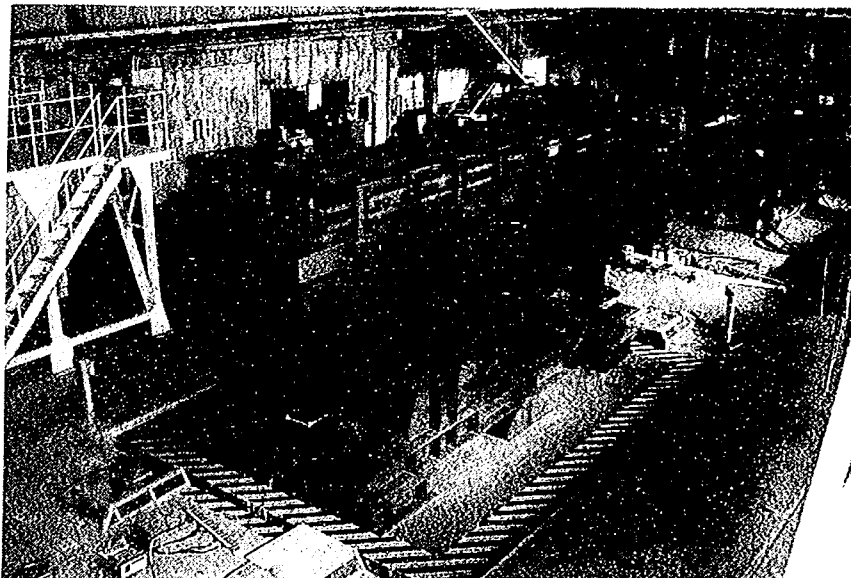
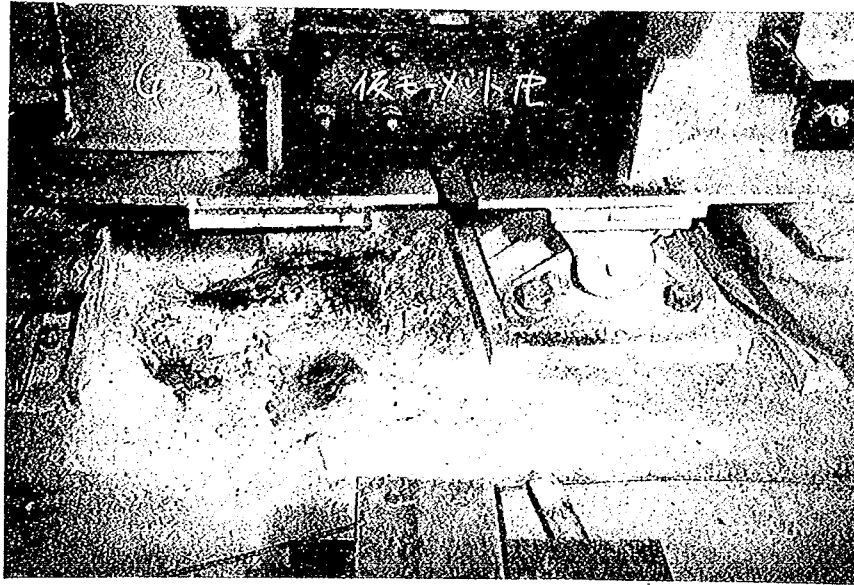


Photo 3 Shaking Table Tests for Studying Structural Response of Menshin Bridges



(a) Replacement of Existing Steel Bearings



(b) Setting of Menshin Bridges

Photo 4 Deck Connection of Existing Bridge with use of Menshin Bridges

Response Analysis of Miyagawa Bridge Based
on a Measured Acceleration Record

Kazuhiko KAWASHIMA

Hiroyuki NAGASHIMA

Shuji MASUMOTO

and

Kohji HARA

2nd U. S. -Japan Workshop on
Earthquake Protective Systems for Bridges

Tsukuba, Japan

December 1992

THE UNIVERSITY OF CHICAGO

DEPARTMENT OF CHEMISTRY

RESEARCH REPORT

NO. 1000

1954

1954

1954

RESEARCH REPORT

NO. 1000

1954

1954

RESPONSE ANALYSIS OF MIYAGAWA BRIDGE BASED ON A MEASURED ACCELERATION RECORD

Kazuhiko Kawashima^I, Hiroyuki Nagashima^{II}, Shuji Masumoto^{III} and Kohji Hara^{IV}

I Head, Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works research Institute, Ministry of Construction

II Planning Department, Kanto Regional Construction Bureau, Ministry of Construction

(Formerly, Research Engineer, Earthquake Engineering Division, ditto)

III Director, Road Construction Department, Shizuoka Ken

IV Second Design Office, Numazu Construction Office, Shizuoka Ken

ABSTRACT

This paper presents an analysis of a measured acceleration record at Miyagawa Bridge, the first seismically isolated bridge in Japan. Amplification of the peak accelerations is examined between the deck, pier and ground motions. Seismic isolation effect can be clearly observed. Dynamic response analysis using the analytical model adopted for seismic design is made assuming the acceleration recorded at underground as the input ground motion. Response acceleration computed at the deck is quite similar with the measured acceleration although peak acceleration is larger in the analysis.

INTRODUCTION

Miyagawa Bridge is the first seismically isolated bridge in Japan, and is located on National Highway No. 362 in Shizuoka ken¹⁾²⁾. It completed in March 1991. Strong motion observation has been made since the completion. Because the first record was obtained by an earthquake with magnitude 4.9, which took place close to the site, an analysis was made for verifying the effectiveness of Menshin Design³⁾. This paper presents the seismic behavior of Miyagawa Bridge based on the analysis of the recorded accelerations.

STRONG MOTION OBSERVATION AT MIYAGAWA BRIDGE

As was reported in the previous paper¹⁾²⁾, the Menshin Design (seismic isolation design) of Miyagawa Bridge was made in accordance with the

Guidelines of Menshin Design of Highway Bridges²⁾. In the Menshin Design, the Menshin devices are designed by both the Seismic Coefficient Method and the Bearing Capacity Method. In both methods, the lateral force is statically applied to the bridge, and the seismic safety is examined based on the allowable stress design approach in the Seismic Coefficient Method and the bearing capacity considering ductility in the Bearing Capacity Method.

The lateral force coefficient was not, however, reduced at the Miyagawa Bridge from the design value specified in the Specifications of Design of Highway Bridges⁴⁾, because it was the pilot project for verifying the effectiveness of the Menshin Design. The lateral force coefficient is therefore ? in both longitudinal and transverse directions.

Lead rubber bearings⁵⁾ were adopted so that damping ratio of 0.12 can be obtained for the first mode of the bridge. Natural period of the bridge in longitudinal direction was assumed as 0.76 second in the Seismic Coefficient Method and 1.0 second in the Bearing Capacity Method. Because of the deformation dependence of the stiffness of the lead rubber bearings, the natural period is not the same for the Seismic Coefficient and the Bearing Capacity Method.

Soil condition at the site is I-group (stiff) based on the classification of ground condition specified in the Seismic Design Specifications of Highway Bridges⁴⁾.

Three 3-components feed-back servo type strong motion accelerometers were placed on the deck and pier crest at pier 1 (P1) and 10 m below the ground surface 10 m apart from the abutment 1 (A1) as shown in Fig. 1. The data are recorded on the analog magnetic type recording system when the ground acceleration larger than the triggering level is developed.

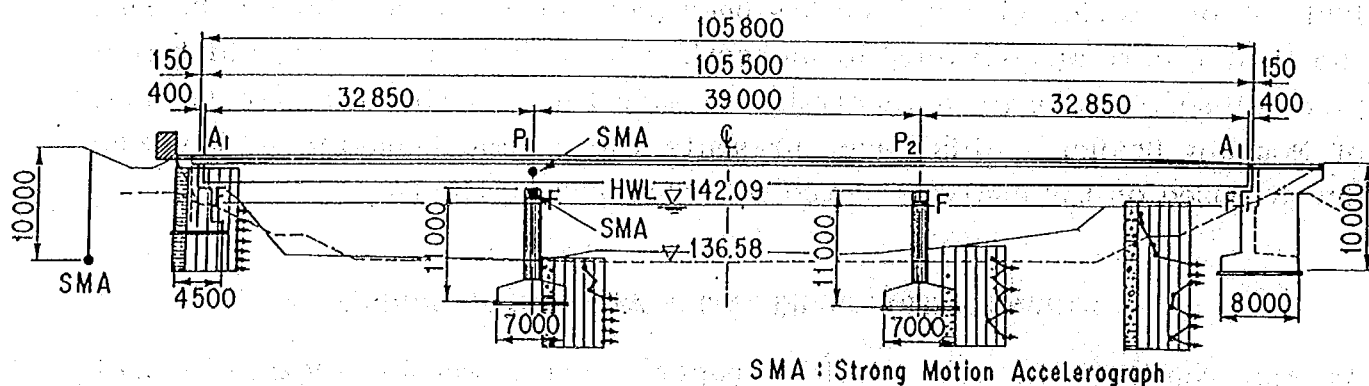
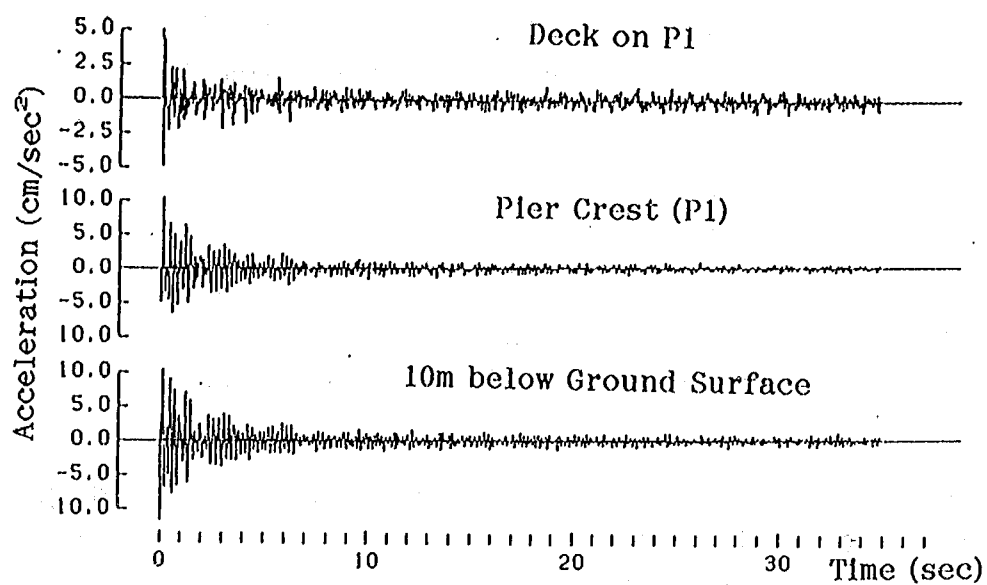
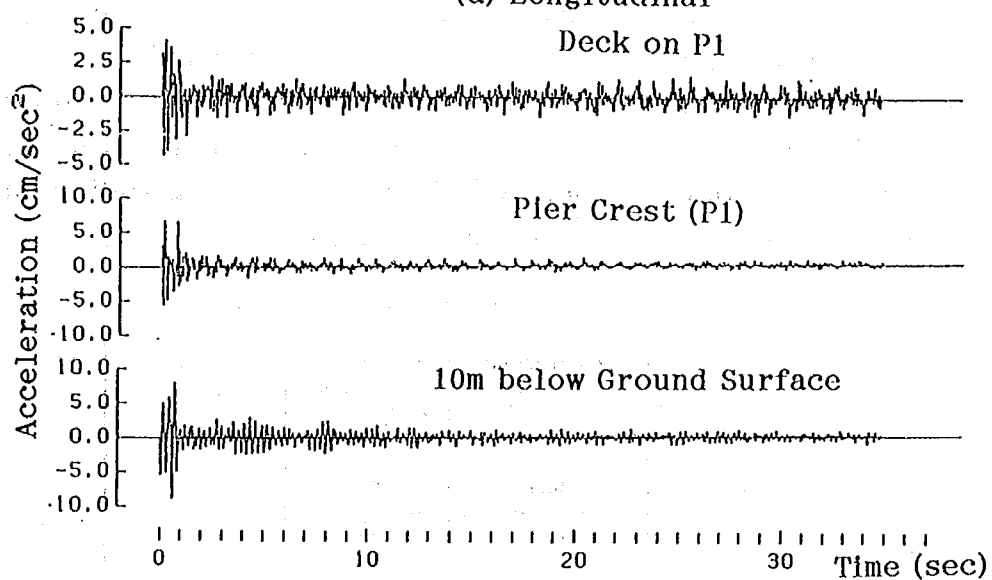


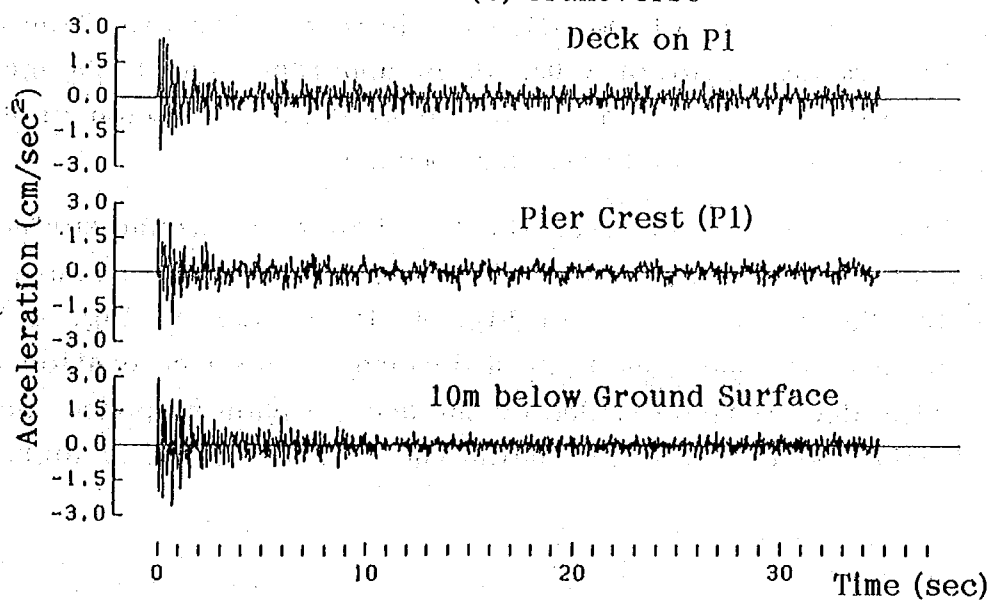
Fig. 1 Miyagawa Bridge and Location of Accelerometers



(a) Longitudinal



(b) Transverse



(c) Vertical

Fig. 2 Measured Accelerations

The first record was obtained by an earthquake with magnitude 4.9 which took place about 30 km north east from the site. Seismic Intensity of Japanese Methodological Agency was II.

ANALYSIS OF MEASURED RECORDS

Fig. 2 shows the accelerations recorded at the deck, pier crest and 10m below the ground surface. Because intensity was so small, first part of the response could not be recorded. But main part could be successfully recorded.

Table 1 shows the peak acceleration. At 10m below the ground surface, the peak acceleration was 11.6 cm/sec^2 in longitudinal direction and 8.9 cm/sec^2 in transverse direction. This decreased to 10.4 cm/sec^2 in longitudinal direction and 6.6 cm/sec^2 in transverse direction on the pier crest. Because the amplification of the peak acceleration from the ground to pier crest is generally about 2.5 for this type of bridges³⁾, the decrease of pier response in comparison with the ground acceleration at 10 m below the surface is apparent. Furthermore, it decreased to 5.0 cm/sec^2 in longitudinal direction and 4.3 cm/sec^2 in transverse direction on the deck.

Table 1 Peak Accelerations Measured

Location	Longitudinal Direction	Transverse Direction	Vertical Direction
Deck on P1	5.0	4.3	2.6
Pier Crest of P1	10.4	6.6	2.5
10 m underground	11.6	8.9	2.9

Fig. 3 shows such decrease of amplification in terms of acceleration response spectra with damping ratio of 0.05. It is apparent that although ground motion at 10 m below the surface had the predominant period of 0.2 ~ 0.3 second, it is not predominant in the deck motion.

For studying the amplification from the 10 m below the ground surface to the pier crest and the deck, ratios of the acceleration response spectra presented in Fig. 3 are shown in Fig. 4. It is seen in Fig. 4 that the acceleration response spectrum ratio R defined as the acceleration response spectrum of the deck motion divided by the acceleration response spectrum of the ground motion measured at 10 m below the ground surface is approximately

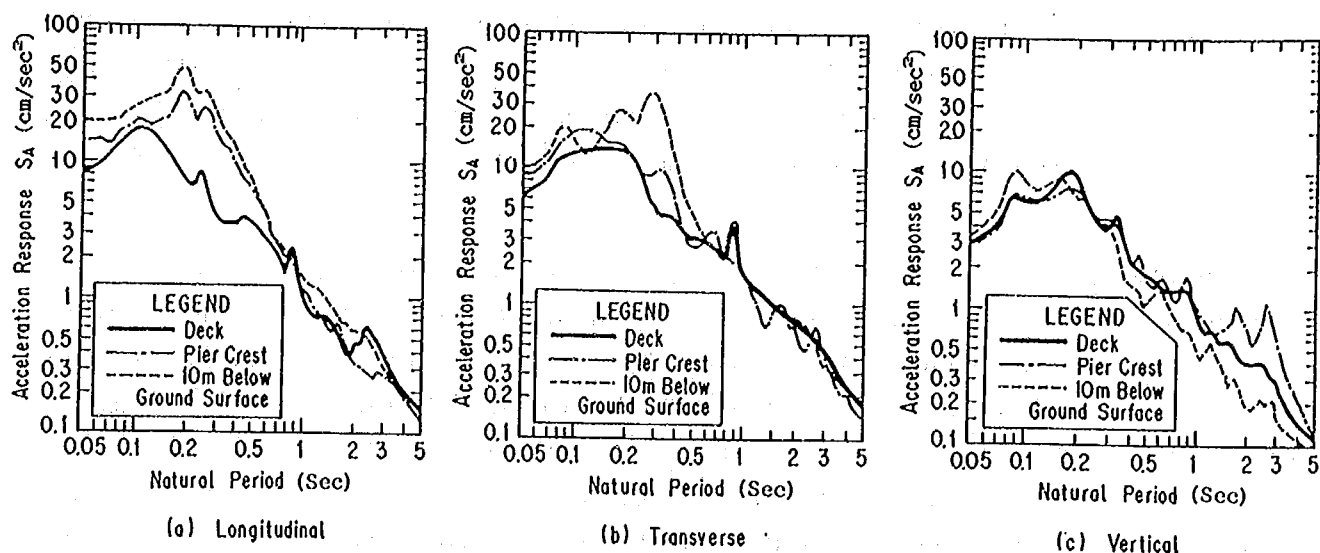


Fig. 3 Acceleration Response Spectra of Measured Acceleration Record (Damping Ratio of 0.05)

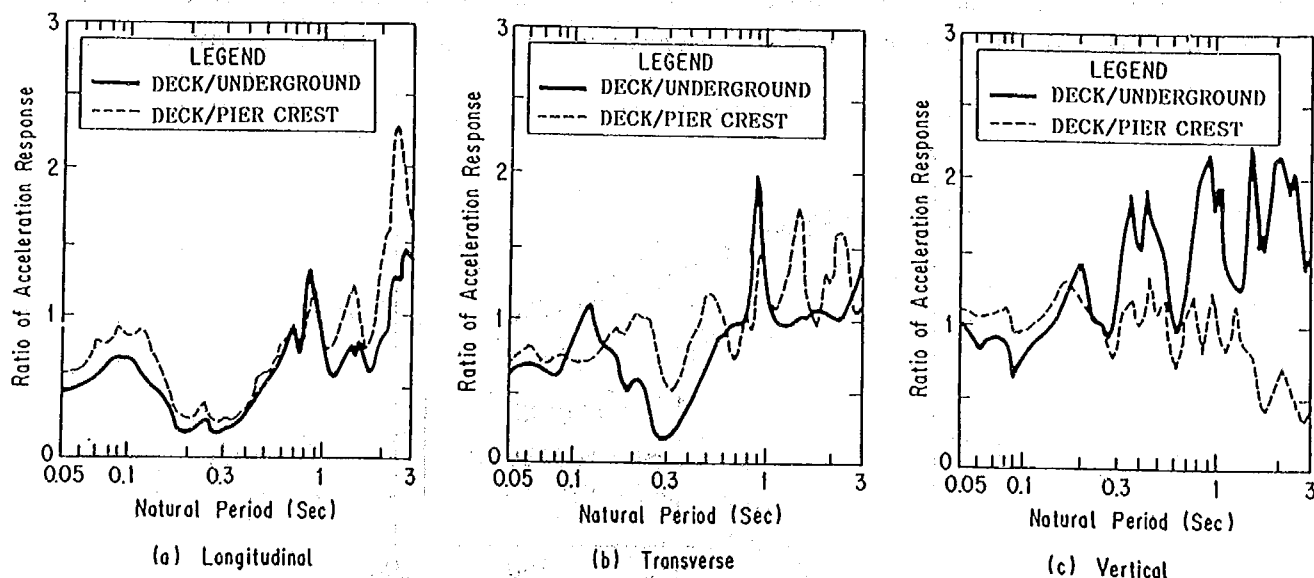


Fig. 4 Ratios of Acceleration Response Spectra

0.2 at natural period of 0.2 ~ 0.3 second. The motion at this period is filtered out at the deck due to the effect of seismic isolation.

On the other hand, the ratio R defined as the acceleration response spectrum of the deck motion divided by the acceleration response spectrum of the ground motion measured 10 m below the ground surface increases to 1.4 in longitudinal direction and 2.0 in transverse direction at natural period of

about 0.85 second. Reminding the fact that the natural period assumed in the Seismic Coefficient Method was 0.76 second, this represents the amplification of deck response due to resonance of the first mode. Reflecting the fact that the deformation of LRBs developed during the earthquake is much smaller than that assumed in the design by means of the Seismic Coefficient Method, stiffness of LRBs is approximately 25 % higher than that assumed in the design.

The LRBs were designed so that yielding of the lead plug be developed for the lateral force coefficient over 0.12. Of course, because the response during the earthquake was much smaller than this level, bilinear behavior of LRBs seems not significant. However it should be noted that LRBs show high energy dissipation capability from small displacement. Fig. 5 shows the equivalent damping ratio of the LRBs obtained from the full size test⁷⁾. Although the equivalent damping ratio for shear strain smaller than 20 % is not presented here, it is known that the equivalent damping ratio for smaller shear range can be approximately evaluated by extrapolating the relation presented in Fig. 5. Therefore it may be assumed that the equivalent damping ratio of LRBs was approximately 30 % for the shear strain developed during the earthquake.

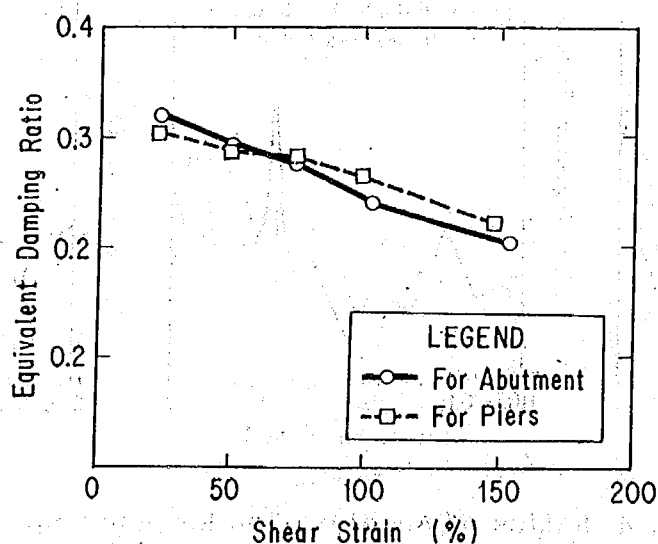


Fig. 5 Equivalent Damping Ratio of Lead Rubber Bearings Based on Full Model Tests

STUDY OF THE RECORD BY DYNAMIC RESPONSE ANALYSIS

Because dynamic response analysis was made at the design stage for Miyagawa Bridge, it was decided to adopt the same analytical model as shown in Fig. 6 for studying the record. The acceleration recorded at 10 m below the ground

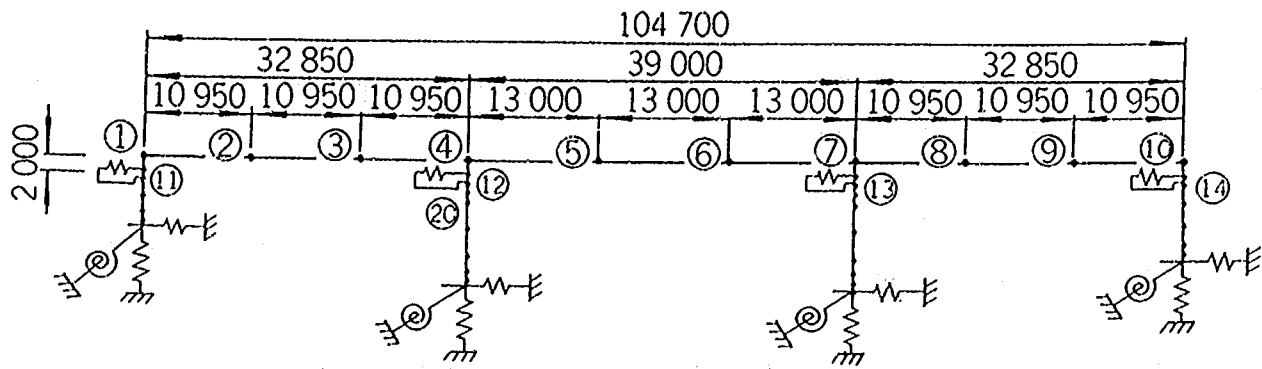


Fig. 6 Dynamic Response Analysis Model of Miyagawa Bridge for Design Purpose

surface was regarded as the input ground motion and the response of the deck and substructure were computed and compared with the recorded accelerations. Because the footings of P1 and P2 are almost on the same level with the down-hole accelerometer placed 10 m below the ground surface (refer to Fig. 1), it was considered appropriate to apply the acceleration measured at 10 m below the ground surface to the footings. Although the footing of two abutments, A1 and A2, is slightly higher in level than the down-hole accelerometer, it was disregarded in the analysis because it would cause only minor difference.

Analysis was made by the equivalent linear analysis. Because the deformation of the LRBs was quite small during the earthquake, the first stiffness k_1 was assumed as the equivalent stiffness of LRBs as

$$k_1 = 6.5 k_2 \quad (1)$$

$$k_2 = \frac{F - Q_d}{u_B} \quad (2)$$

$$Q_d = A_P \cdot q_0 \quad (3)$$

$$F = A_R \cdot G \cdot \gamma + A_P \cdot q \quad (4)$$

$$q = -283.6 \gamma^2 + 183.8 \gamma + 85.0 \quad (5)$$

where

k_1 : 1st stiffness of LRB (kgf/cm)

k_2 : 2nd stiffness of LRB (kgf/cm)

Q_d : yielding force of lead (kgf)

q_0 : shear stress of yielding of lead plug (= 85.0 kgf/cm²)

G : shear modulus of rubber (kgf/cm²)

F : shear force of LRB (kgf)

u_B : design displacement (cm)

γ :shear strain

Modal damping ratio h of the bridge was computed by means of the energy-proportional damping method as⁴⁾

$$h = \frac{\sum_{j=1}^n \phi_{ij}^T h_j k_j \phi_{ij}}{\Phi_i^T K \Phi_i} \quad (6)$$

where

ϕ_{ij} :mode vector of j -th structural component for i -th mode

h_j :damping ratio of j -th structural component

K_j :stiffness matrix of j -th structural component

Φ_i :mode vector of bridge for i -th mode

K :stiffness matrix of bridge

Eq. (6) is effective to evaluate the modal damping ratio of bridges based on an experimental study⁵⁾. Damping ratio of structural components was assumed in design as shown in Table 2. Because the deformation developed in LRBs is small during the design earthquake assumed in the Seismic Coefficient Method, it was assumed in the design that the damping ratio of LRBs is zero in the Seismic Coefficient Method. It may be therefore good to assume zero damping ratio for LRBs for the consistency with the seismic design. However it became apparent that zero damping ratio for LRBs gives appreciably larger bridge response than the measured values. Therefore it was assumed as 0.3 based on the above described full model test results on the damping characteristics of LRB.

Table 2 Damping Ratio Assumed in Seismic Design of Miyagawa Bridge

Structural Components		For Design		For this Analysis
		Seismic Coefficient Method	Bearing Capacity Method	
Super structure		0.03	0.03	0.03
Piers and Abutments		0.05	0.05	0.05
Foundations		0.1	0.1	0.1
Menshin Bearings	A1	0	0.177	0.3
	P1	0	0.137	0.3
	P2	0	0.139	0.3
	A2	0	0.199	0.3

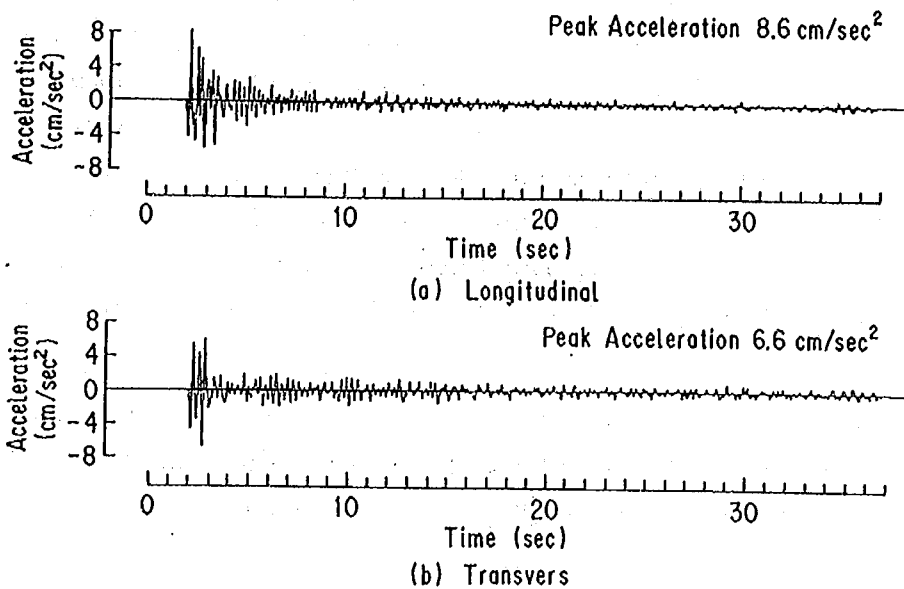


Fig. 7 Computed Deck Acceleration

Fig. 7 shows the response acceleration of the deck thus computed. Although the computed acceleration is 1.5 ~ 1.7 times larger in amplitude than the measured response, overall characteristics of the computed response is similar with the measured acceleration. Overestimation of response acceleration of the deck may be attributed to the fact that the damping ratio of the foundation is underestimated in the dynamic analysis. This is for the sake of giving safe-side result for design purpose.

CONCLUSION

The first record obtained at the Miyagawa Bridge, the first Menshin bridge in Japan, was analyzed to study the seismic behavior of Menshin bridges. The following conclusions may be deduced based on the analysis presented herein :

- 1) The deck acceleration is about 50 % in amplitude of the underground acceleration at 10 m below the ground surface. This clearly shows the effectiveness of the seismic isolation.
- 2) When amplification of the response from the 10 m below the ground surface to the deck is represented in terms of the acceleration response ratio with damping ratio of 0.05, the acceleration response ratio R defined as the response acceleration of the deck motion divided as the response

acceleration of ground motion measured at 10 m below the ground surface is only 0.25 at natural period of 0.2 ~ 0.3 second which is the predominant period of the ground motion. However the acceleration response ratio R defined as the response acceleration of the deck motion divided by the response acceleration of ground motion measured at 10 m below the ground surface is 1.4 ~ 2.0 at 0.85 second which is the fundamental natural period of the bridge. Therefore selection of the natural period in consideration with the predominant period of ground motion is very important in the Menshin Design.

3) The dynamic response analysis model which was adopted for the seismic design of Miyagawa Bridge can realistically simulate the response of the deck although the analysis overestimates the measured acceleration.

REFERENCES

- 1) Matsuo, Oishi, Hara, K. and Yamashita, : Design and Construction of Miyagawa Bridge, Bridges and Foundations, Vol. 91-2, 1991
- 2) Matsuo, Y and Hara, K. : Design and Construction of Miyagawa Bridge -First Menshin Bridge in Japan-, 1st U.S.-Japan Workshop on Earthquake Protective Systems of Highway Bridges, National Center for Earthquake Engineering Research, Buffalo, N.Y., U.S.A., September 1991
- 3) Technology Research Center for National Land Development : Guidelines for Base Isolation of Highway Bridges, 1989
- 4) Japan Road Association : Part V Seismic Design of Design Specifications of Highway Bridges, 1990
- 5) Robinson, W.H. : Lead Rubber Hysteretic Bearings Suited for Protecting Structures during Earthquakes, Earthquake Engineering and Structural Dynamics, Vol. 10, 1982
- 6) Kawashima, K. : A State of the Arts of Earthquake Engineering Research for Application to Seismic Design of Bridge Foundations, Technical Report, No. 2385, Public Works Research Institute, July 1986
- 7) Makiguchi, Y., Matsuo, and Hara, H. : Performance Test of Lead Rubber Bearings for Miyagawa Bridge, Annual Meeting of the Japan Society of Civil Engineers, Part I, 1991
- 8) Kawashima, K., Nagashima, H. and Iwasaki, H. : Effectiveness of the Energy Proportional Method for Evaluating Modal Damping Ratio of Menshin Bridges, Civil Engineering Journal, 1992

Design Example of a Highway Bridge Based on
The Manual for Menshin Design of Highway Bridges

Kazuhiko KAWASHIMA

Mitsuo OKADO

and

Michihiro HORIKAWA

2nd U. S. -Japan Workshop on
Earthquake Protective Systems for Bridges

Tsukuba, Japan

December 1992

THE UNITED STATES OF AMERICA

DOCTOR OF THE UNIVERSITY OF CALIFORNIA

IN THE DEPARTMENT OF MEDICINE

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

OF THE UNIVERSITY OF CALIFORNIA

DESIGN EXAMPLE OF A HIGHWAY BRIDGE BASED ON THE MANUAL FOR MENSHIN DESIGN OF HIGHWAY BRIDGES

Kazuhiko KAWASHIMA¹⁾, Mitsuo OKADO²⁾ and Michihiro HORIKAWA³⁾

- 1) Head, Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works research Institute, Ministry of Construction, Tsukuba, Japan
- 2) Assistant Manager, Design Department, New Structural Engineering Ltd., Chiyoda-ku, Tokyo
- 3) Chief, Design Department, New Structural Engineering Ltd., Chiyoda-ku, Tokyo

ABSTRACT

Presented is a design example of a base isolated bridge with a span length of 40 m and a clear height of 10 m. Design was made in accordance with the Manual of Menshin Design of Highway Bridges. Effect of Menshin design is discussed.

INTRODUCTION

A design was made for the same design condition with the U.S. group for comparing the seismic isolation design of bridges. Two types of Menshin devices were considered in this example calculation. First is the lead rubber bearings¹⁾ (LRB) and the other is the high damping rubber bearing (HDR). Design was made in accordance with the Manual for Menshin Design of Highway Bridges²⁾ which was compiled in March 1992 as the final accomplishment of the Five Year Research Program between the Public Works Research Institute and 28 private firms on the "Development of Menshin Systems of Bridges". The design is not the detailed design but the rough estimation of approximate size and reinforcement.

DESIGN CONDITION

A bridge with multi-span continuous prestressed concrete girder with a weight of 200 kN/m (20.4 tf/m) as shown in Fig. 1 is considered. Only a single span segment is designed. The span length and the width of the superstructure are assumed as 40 m and 14 m, respectively. The girder depth is 2 m. The clear height is assumed as 10m. Soil condition is assumed as a stiff clay with N-value of the Standard Penetration Test of 30 or more. Thickness of this layer is about 60 m. Those are the design conditions specified for the common condition between U.S. and Japan. Table 1 summarizes those conditions.

Because the soil is of stiff clay with N-value of 30 or more, it may be diluvial clay layer. The soil condition is classified by the characteristics value as defined by

$$T_G = \sum \frac{4 H_i}{V_{s1}} \quad (1)$$

where,

- T_G : characteristics value (sec)
 H_i : thickness of i-th sub layer (m)
 V_{Si} : shear wave velocity of i-th sub layer (m/sec)

Because the shear wave velocity of the diluvial clayey layer may be over 400 m/sec, the soil condition may be classified as the 1st group in accordance with the "Part V Seismic Design" of the "Design Specifications of Highway Bridges"²⁾.

The foundation type is therefore assumed as a spread foundation (direct foundation). The bearing capacity, which is the most critical in design of foundation, is assumed as 60 tf/m². In actual design, the detailed soil survey is required. Table 2 shows the design condition of the foundation based on the Design Specifications of Highway Bridges. Assuming that the covering depth is 1.5 m, the pier height from the surface of the footing to the pier crest is 11.5 m.

It is assumed that the bridge is located in the seismically active area. And also it is assumed that the bridge is categorized as the "Important" bridge.

Then based on the Manual the design lateral force coefficient in the Seismic Coefficient Method is as²⁾

$$K_h = C_z \cdot C_G \cdot C_I \cdot C_T \cdot C_E \cdot K_{HO} \geq 0.1 \quad (2)$$

where

$$C_T \cdot C_E \geq 0.8 \quad (3)$$

where,

- K_h : seismic coefficient in the Seismic Coefficient Method
 C_z : modification factor for zone (= 1.0, seismically active area)
 C_G : modification coefficient for ground condition (= 0.8, 1st group)
 C_I : modification factor for importance (= 1.0, important bridge)
 C_T : modification of structural response, and shall be computed by Table 3
 C_E : modification factor for damping ratio of bridge, and shall be determined by Table 4
 K_{HO} : standard design lateral force coefficient for Seismic Coefficient Method(= 0.2)

In the Bearing Capacity Method, the lateral force coefficient shall be as²⁾

$$K_{he} = \frac{K_{HO}}{\sqrt{2\mu - 1}} \geq 0.3 \quad (4)$$

$$K_{HO} = C_z \cdot C_I \cdot C_R \cdot C_E \cdot K_{HOO} \quad (5)$$

where,

- K_{he} : equivalent lateral force coefficient for Bearing Capacity Method
 K_{HOO} : lateral force coefficient in the Bearing Capacity Method

- c_z : modification factor for zone (= 1.0, seismically active area)
 c_i : modification factor for importance (= 1.0 , important bridge)
 c_R : modification factor for structural response, and shall be computed by Table 5
 c_E : modification factor for damping ratio of bridge, and shall be obtained by Table 6
 k_{ho} : standard lateral force coefficient for Bearing Capacity Method (= 1.0)
 μ : allowable ductility for reinforced concrete piers

The modification coefficients c_E , c_T and c_R have to be determined based on the natural period and damping ratio of the bridge.

Because in seismic design the bridge is critical in longitudinal (bridge axis) direction, major attention was paid in seismic design in longitudinal direction.

DESIGN OF MENSHIN DEVICES

Design Requirements

The design displacement of the Menshin device is an important parameter for designing the Menshin devices, and is defined as

$$u_B = \frac{k_h \cdot W_u}{K_B} \quad (\text{Seismic Coefficient Method}) \quad (6)$$

$$u_B = \frac{k_{ho} \cdot W_u}{K_B} \quad (\text{Bearing Capacity Method}) \quad (7)$$

where,

- u_B : design displacement of Menshin device (cm)
 K_B : equivalent stiffness (kgf/cm²)
 k_h : seismic coefficient in the Seismic Coefficient Method, and shall be computed by Eq. (2)
 k_{ho} : lateral force coefficient in the Bearing Capacity Method, and shall be computed by Eq. (5)
 W_u : weight of superstructure supported by the Menshin device (kgf)

It should be noted in Eq. (7) that the lateral force coefficient by Eq. (5) is used for design of the Menshin device. Therefore the reduction of lateral force considering the ductility of the pier is not considered for evaluating the lateral force for design of the Menshin device.

Following items are required to be checked for the Menshin devices :

(a) Check against Vertical Pressure

$$\sigma_{\max} \leq \sigma_{\max, a} \quad (8)$$

$$\sigma_{\max} - \sigma_{\min} \leq \Delta \sigma_a \quad (9)$$

$$\sigma_{\max} = R_{\max} / A_{RO} \quad (10)$$

$$\sigma_{\min} = R_{\min} / A_R \quad (11)$$

where,

$\sigma_{\max}$: maximum vertical stress (kgf/cm²)

$\sigma_{\max, a}$: allowable maximum vertical stress (= 80.0 kgf/cm²)

$\sigma_{\min}$: minimum vertical stress (kgf/cm²)

$\Delta \sigma_a$: allowable amplitude of compression stress (= 50 kgf/cm²)

$R_{\max}$: maximum vertical force (dead weight + maximum active load)

$R_{\min}$: minimum vertical force (dead weight + minimum active load)

A_R : sectional area of Menshin device (cm²)

A_{RO} : effective sectional area of deformed Menshin device (cm²)

(b) Check against Lateral Displacement

$$u_O \leq \gamma_{Oa} \cdot \Sigma t_e \quad (12)$$

$$u_B \leq \gamma_{Ba} \cdot \Sigma t_e \quad (13)$$

where,

u_O : design displacement of Menshin device against normal load such as creep, shrinkage and temperature change (cm)

γ_{Oa} : allowable shear strain of rubber against normal load (= 0.7)

u_B : design displacement of Menshin device against normal load and seismic effects (cm)

γ_{Ba} : allowable shear strain of rubber against normal load and seismic effects, and shall be 1.5 in the Seismic Coefficient Method and 2.5 in the Bearing Capacity Method

Σt_e : total thickness of rubber (cm)

(c) Check against Buckling

$$\sigma_c \leq \sigma_{cRa} \quad (14)$$

where,

σ_c : averaged vertical stress (kgf/cm²)

σ_{cRa} : allowable vertical stress for safety against buckling (kgf/cm²), and shall be determined as

$$\sigma_{cRa} = G \cdot S \cdot \alpha / f_B \quad (15)$$

where,

G : shear modulus of rubber (kgf/cm²)

S : coefficient depending on shape, and shall be obtained in the square bearings as

$$S = \frac{A_R}{2(a+b)t_e} \quad (16)$$

A_R : sectional area of rubber (cm²)

a, b : effective length and width of rubber (cm)

t_e : thickness of one rubber layer (cm)

$\alpha = a / \Sigma t_e$ or $b / \Sigma t_e$

Σt_o : total thickness of rubber (cm)
 f_s : safety factor (= 2.5)

(d) Check against Local Shear Strain

$$(\gamma_o + \gamma_R + \gamma_s) \leq \gamma_u / f_s \quad (17)$$

where,

γ_o : local shear strain due to vertical load
 γ_R : local shear strain due to rotation of bearing
 γ_s : local shear strain due to lateral displacement
 γ_u : rupture strain of rubber and shall be determined by Table 7
 f_s : safety factor for local shear strain, and is 2.5 for normal load and 1.8 and 1.2 for seismic design by means of the Seismic Coefficient Method and Bearing Capacity Method, respectively

(e) Check against Stress of Internal Steel Plate

$$\sigma_s \leq \sigma_{sa} \quad (18)$$

$$\sigma_s = \sigma_o \frac{t_o}{t_s} \quad (19)$$

where,

σ_s : stress in tension of the plate (kgf/cm²)
 σ_{sa} : allowable stress of steel in tension (kgf/cm²)
 t_s : thickness of steel plate (cm)
 σ_o : maximum vertical stress induced in Menshin device (kgf/cm²)
 t_o : thickness of one rubber layer (cm)

Design Requirement of LRB

In the LRB, natural rubber with the shear modulus of 8.0 kgf/cm² was adopted. The equivalent stiffness and the equivalent damping ratio is obtained as

$$K_B = \frac{A_R \cdot G \cdot \gamma + A_P \cdot q}{u_{Be}} \quad (20)$$

$$h_B = \frac{2 Q_d \{ u_{Be} + Q_d / (K_2 - K_1) \}}{\pi u_{Be} (Q_d + u_{Be} K_2)} \quad (21)$$

where,

K_B : equivalent stiffness (kgf/cm²)
 h_B : equivalent damping ratio
 u_{Be} : effective design displacement of Menshin device (cm)
 Q_d : yield force of lead, and can be determined as

$$Q_d = A_P \cdot q_o \quad (22)$$

A_P : sectional area of lead plug (cm²)
 q_o : yielding shear stress of lead (85 kgf/cm²)

A_R : sectional area of rubber (cm^2)
 G : shear modulus of rubber (kgf/cm^2)
 γ : shear strain
 q : shear stress of lead, and shall be obtained as

$$q = \begin{cases} -283.6\gamma^2 + 183.8\gamma + 85.0 & (0 \leq \gamma \leq 0.5) \\ 28.3\gamma^2 - 128.1\gamma + 163.0 & (0.5 < \gamma \leq 2.0) \\ 20 & (2.0 < \gamma \leq 2.5) \end{cases} \quad (23)$$

K_1, K_2 : first and second stiffness of LRB (kgf/cm^2)

The shape of the lead plug needs to be as

$$1.25 \leq \frac{H_P}{D_P} \leq 5.1 \quad (24)$$

where,

H_P : height of lead plug (cm)
 D_P : diameter of lead plug (cm)

Design Requirements of HDR

In the HDR, the equivalent stiffness and the equivalent damping ratio are evaluated as

$$K_B = \frac{A_R \cdot G(\gamma_e)}{\sum t_e} \quad (25)$$

$$G(\gamma_e) = \begin{cases} 45.3 - 72.7\gamma_e + 57.3\gamma_e^2 - 19.0\gamma_e^3 + 2.18\gamma_e^4 & (0 < \gamma_e \leq 2.0) \\ 12.2 & (2.0 < \gamma_e \leq 3.0) \end{cases} \quad (26)$$

$$h_B = 0.154 + 0.0383\gamma_e - 0.0445\gamma_e^2 + 0.00858\gamma_e^3 \quad (0 < \gamma_e \leq 3.0) \quad (27)$$

where,

K_B : equivalent stiffness (kgf/cm^2)
 h_B : equivalent damping ratio
 G : shear modulus of the high damping rubber (kgf/cm^2)
 A_R : sectional area of rubber (cm^2)
 γ_e : effective shear strain
 $\sum t_e$: total thickness of rubber

Design of LRB and HDR

The natural period of the Menshin bridge is recommended in the Manual to set about twice as long as the one of the bridge with usual bearing condition (non-Menshin bridge). Because the natural period of the non-Menshin bridge was estimated as 0.58 second, it was aimed to design LRB and HDR so that the natural period of the Menshin bridge becomes approximately 1.1 second. It should be noted that effect of soils was considered in computing the natural period by a set of soil springs as shown in Table 8. Because the soil is stiff, effect of soil was insensitive for the natural period.

Because the equivalent stiffness of LRB and HDR depends on the deformation developed in the device, the natural period of the bridge is not the same depending on the device and the Menshin design method. The natural period of the bridge was evaluated as 1.23 second (LRB) and 0.95 second in the Seismic Coefficient Method, and 1.91 second (LRB) and 1.44 second (HDR) in the Bearing Capacity Method. The damping ratio was evaluated as

$$h = \frac{\sum K_{Bi} \cdot u_{Bi}^2 \cdot c_{hi}}{\sum K_{Bi} \cdot u_{Bi}^2 \cdot c_i} \quad (28)$$

$$c_{hi} = h_{Bi} + \frac{h_{Pi}}{K_{Pi}} + \frac{h_{Fui}}{K_{Fui}} + \frac{h_{F\theta i} \cdot H^2}{K_{F\theta i}} \quad (29)$$

$$c_i = 1 + \frac{K_{Bi}}{K_{Pi}} + \frac{K_{Bi}}{K_{Fui}} + \frac{K_{Bi} \cdot H^2}{K_{F\theta i}} \quad (30)$$

where

- h : Modal damping ratio of bridge
- h_{Bi} : Damping ratio of i -th damper
- h_{Pi} : Damping ratio of i -th pier/abutment
- h_{Fui} : Damping ratio of i -th foundation associated with translational movement
- $h_{F\theta i}$: Damping ratio of i -th foundation associated with rotation
- K_{Pi} : Equivalent stiffness of i -th pier/abutment
- K_{Fui} : Translational stiffness of i -th foundation
- $K_{F\theta i}$: Rotational stiffness of i -th foundation
- u_{Bi} : Design displacement of i -th menshin device
- H : Height from the bottom of pier to the gravity center of deck

The damping ratio of structural components including the LRB and HDR was assumed as shown in Table 9. The damping ratio of the bridge for the first mode was evaluated by Eq. (28) as 0.21 (LRB) and 0.13 (HDR) in the Seismic Coefficient Method and 0.12 (LRB) and 0.11 (HDR) in the Bearing Capacity Method. The design displacement u_B was 3.1 cm (LRB) and 2.3 cm (HDR) in the Seismic Coefficient Method and 33.0 cm (LRB) and 18.5 cm (HDR) in the Bearing Capacity Method.

The modification factor for damping ratio of bridge c_E , which was evaluated from Tables 4 and 6 based on the above described damping ratio of the bridge, is shown in Table 10. Table 11 shows the modification factor for structural response c_T (Seismic Coefficient Method) and c_R (Bearing Capacity Method). The lateral force coefficient becomes 0.17 (LRB) and 0.18 (HDR) in the Seismic Coefficient, and 0.45 (LRB) and 0.55 (HDR) in the Bearing Capacity Method.

It should be noted that if the non-Menshin bridge has the natural period of 0.58 second, the modification factor for structural response c_T and c_R is

1.25, and that the reduction of the lateral force coefficient by means of the modification factor for damping ratio of bridge c_E can not be considered. A comparison of the lateral force coefficient between the Menshin bridge and the non-Menshin bridge is shown in Table 12. In the Menshin bridge the lateral force coefficient is 15 % (LRB) and 10 % (HDR) in the Seismic Coefficient Method and 36 % (LRB) and 21 % (HDR) in the Bearing Capacity Method less than the non-Menshin bridge.

Fig. 2 shows the LRB and HDR designed based on those conditions. Although there was some interaction between the design of Menshin device and the design of substructure which will be described later, the final shape of the devices determined is presented here. The most critical factor for determining the dimension of the Menshin devices was the check against the lateral displacement in the Bearing Capacity Method by Eq. (13). The shear strain of the rubber is 250 % and 231 % in LRB and HDR, respectively, while the allowable shear strain γ_{B0} in Eq. (13) is 250 %.

Table 13 summarizes the design of LRB and HDR.

SEISMIC DESIGN OF SUBSTRUCTURES

Seismic Design of Pier

Seismic design of pier was made by the Seismic Coefficient Method. Fig. 3 shows the analytical model for the pier and the foundation. For designing the pier, the distance in longitudinal direction from the edge of the pier to the front edge of the bearing has to be larger than the following seat length to avoid the spalling of the concrete at the pier crest during an earthquake :

$$S = 20 + 0.5l \quad (31)$$

where,

- S : seat length from the edge of pier to the edge of bearing (cm)
 l : span length (m)

Because the span length is 40 m, 40 cm is required for the seat length S . The seat length has to be provided both side of the bearing in longitudinal direction. Because the length of the Menshin device is 95 cm (LRB) and 89 cm (HDR), two times of the seat length S plus the length of the Menshin device is 175 cm (LRB) and 169 cm (HDR). Therefore, the depth of the pier (length in longitudinal direction) was determined as about 2 m. The width of the pier (length in transverse direction) was determined so that the stress of the concrete and that of the reinforcement become in good proportion.

The strength of the concrete is 240 kgf/cm², and the grade of the reinforcement is the SD 345 (deformed bars). As shown in Table 2, the allowable stress is 120 kgf/cm² in concrete and 3,000 kgf/cm² in the reinforcement.

Table 14 shows the force used for seismic design of pier and the stress computed. Because the bottom of the pier is the most critical in seismic design, the design calculation was made for this section. It can be said that the selected section is in good proportion, because the stress developed in the concrete and the reinforcement is close with their allowable stress and both the concrete and the reinforcements are effectively used.

After the dimension of the pier and the reinforcement was determined, the check of bearing capacity was made by the Bearing Capacity Method. Some increase of the depth and the reinforcement was required.

Fig. 4 shows the section thus determined. The size of the column is 2.2 m in longitudinal direction and 3 m in transverse direction. When LRB is adopted, 38 reinforcements with the diameter of 29 mm are required along the face in axial direction, and 24 reinforcements with diameter of 19 mm along the face in transverse direction are required. The total area of the reinforcements is 313.0 cm^2 , and this corresponds to 0.47 % of the concrete section (main reinforcement ratio = 0.47 %). On the other hand, when HDR was adopted, the diameter of main reinforcement along the face in longitudinal direction were required to increase one grade, and 38 reinforcements with diameter of 32 mm were provided. The reinforcement along the face in transverse direction was the same with the previous case. The total area of the reinforcement is 370.6 cm^2 , and the main reinforcement ratio is 0.56 %.

Seismic Design of Foundation

Design of foundation was made by the Seismic Coefficient Method. Safety against sliding, overturning and bearing capacity of supporting soil was checked. Table 15 summarizes the design of the foundation. The size of the footing is 7.5 m x 7.5 m when the LRB was adopted and 7.7 m x 7.7 m when the HDR was adopted.

CONCLUDING REMARKS

The preceding pages presented a design example of a bridge supported by LRB and HDR. By adopting the Menshin devices, 10 % ~ 15 % decrease and 21 % ~ 36 % decrease of the lateral force could be made in the Seismic Coefficient Method and the Bearing Capacity Method, respectively. The bridge designed seems in good shape and reasonable.

REFERENCES

- 1) Robinson, W.H. : Lead-rubber Hysteretic Bearings Suitable for Protecting Structures during Earthquakes, Earthquake Engineering and Structural Dynamics, Vol. 10, 1982
- 2) Manual for Menshin Design of Highway Bridges, Public Works Research Institute and 28 Private Firms, Bulletin of PWRI, Vol. 59, Public Works Research Institute, March 1992
- 3) Japan Road Association : Design Specifications of Highway Bridges, 1990

Table 1 Design Conditions

Structure	Multi-span Continuous Prestressed Concrete Girder
Span Length	40m
Width	14m
Clear Height	10m
Dead Weight of Superstructure	20.4tf/m (200KN/m)
Soile Condition	N=30, Depth=60m

Table 2 Design Condition for Foundation

Allowable Stress	Steel	σ_{sa}	3,000kgf/cm ² (SD345)
	Concrete	σ_{ca}	120kgf/cm ² (σ_{ck} =240kgf/cm ²)
Allowable Bearing Capacity q_a			60tf/m ²
Safy Factor for Sliding			1.2
Safy for Overturning			Combined force of Axial force and lateral force should be applied within 1/3 of the width of footing

Table 3 Modification Factor for Structural Response c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$ $c_T = 2.69T^{1/3} \geq 1.00$	$0.1 \leq T \leq 1.1$ $c_T = 1.25$	$1.1 < T$ $c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$ $c_T = 2.15T^{1/3} \geq 1.00$	$0.2 \leq T \leq 1.3$ $c_T = 1.25$	$1.3 < T$ $c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$ $c_T = 1.80T^{1/3} \geq 1.00$	$0.34 \leq T \leq 1.5$ $c_T = 1.25$	$1.5 < T$ $c_T = 1.64T^{-2/3}$

Table 4 Modification Factor for Damping Ratio c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$h \geq 0.1$	0.9

Table 5 Modification Factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leq 1.4$ $c_R = 0.7$	$1.4 < T_{EQ}$ $c_R = 0.876T_{EQ}^{-2/3}$	
Group II	$T_{EQ} < 0.18$ $c_R = 1.51T_{EQ}^{1/3} \geq 0.7$	$0.18 \leq T_{EQ} \leq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51T_{EQ}^{1/3} \geq 0.7$	$0.29 \leq T_{EQ} \leq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59T_{EQ}^{-2/3}$

Table 6 Modification Factor for Damping Ratio c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$0.1 \leq h < 0.12$	0.9
$0.12 \leq h < 0.15$	0.8
$0.15 \leq h$	0.7

Table 7 Rupture Strain of Rubber

Type of Rubber	Shear Modulus $G(\text{kgf/cm}^2)$	Rupture Strain $\gamma_u(\%)$
Natural Rubber	8.0	500
	10.0	500
	12.0	400
Chloroprene Rubber	8.0	400
	10.0	400

Table 8 Spring Stiffness of Soils

	LRB	HDR
Stiffness for Translation	$2.88 \times 10^5 \text{tf/m}$	$2.99 \times 10^5 \text{tf/m}$
Stiffness for Rotation	$4.04 \times 10^5 \text{tf/m}$	$4.51 \times 10^5 \text{tf/m}$

Table 9 Damping Ratio Assumed in Design for Structural Components

Superstructures			0.03
Menshin Device	SCM	LRB	0.28
		HDR	0.16
	BCM	LRB	0.15
		HDR	0.14
Pier/Columns			0.05
Footing			0.1

Table 10 Modification Factor c_E for the Bridge Designed

	LRB	HDR
SCM	0.9	0.9
BCM	0.8	0.8

Table 11 Modification Factor c_T and c_R for the Bridge Designed

	LRB	HDR
SCM	0.9	1.0
BCM	0.8	0.8

Table 12 Comparison of Lateral Force Coefficient between Menshin Bridge and Non-Menshin Bridge

Design Method	SCM		BCM	
	LRB	HDR	LRB	HDR
① Menshin Design	0.17	0.18	0.45	0.55
② Non-Menshin Design	0.20		0.7	
① / ②	85 %	90 %	64 %	79 %

Table 13 Design of LRB and HDR

Characteristic Values		LRB		HDR	
		SCM	BCM	SCM	BCM
Lateral Force	k_h	0.17	0.45	0.18	0.55
Natural Period	$T(s)$	1.23	1.91	0.95	1.44
Design Displacement	$u_B(cm)$	3.1	33.0	2.3	18.5
Effective Displacement	$u_{Be}(cm)$	2.1	23.1	1.6	13.0
Damping Ratio	h_T	0.21	0.12	0.14	0.12
Equivalent Stiffness	$K_B(tf/m)$	4,775	1,132	6,403	2,374
Equivalent Damping Ratio	h_B	0.28	0.15	0.16	0.14
Lateral Force	$F(tf)$	105.9	261.4	144.0	448.0
Local Shear Strain	Vertical Strain γ_v	0.24	1.09	0.60	1.62
	Lateral Strain γ_s	0.75	2.50	0.28	2.31
	Total Strain γ_T	0.99	3.59	0.88	3.93

Table 14 Design Force and Check of Seismic Safety of Pier

Characteristics		LRB	HDR
Force Developed at Pier Bottom	Axial Force F	1,062tf	1,062tf
	Bending Moment M	1,564tfm	1,656tfm
	Shear Force Q	136tf	144tf
Diameter of Main Reinforcement and Interval		D29 @ 150mm	D32 @ 150mm
Seismic Design Method	Stress of Concrete σ_c	108kgf/cm ²	106kgf/cm ²
	Stress of Reinforcement σ_s	2,838kgf/cm ²	2,643kgf/cm ²
Bearing Capacity Method	Bearing Capacity considering Ductility	186tf	219tf
	Lateral Force considering Ductility $k_h \cdot W$	165tf	203tf

Table 15 Design of Foundation

	LRB	HDR	Allowable Values
Bearing Capacity	54.6tf/m ²	53.2tf/m ²	60.0tf/m ²
Safety Factor for Sliding	6.8	6.6	≥ 1.2
Safety for Overturning (Eccentricity from Center of Footing)	1.23m	1.28m	$\leq 2.5 \sim 2.6$

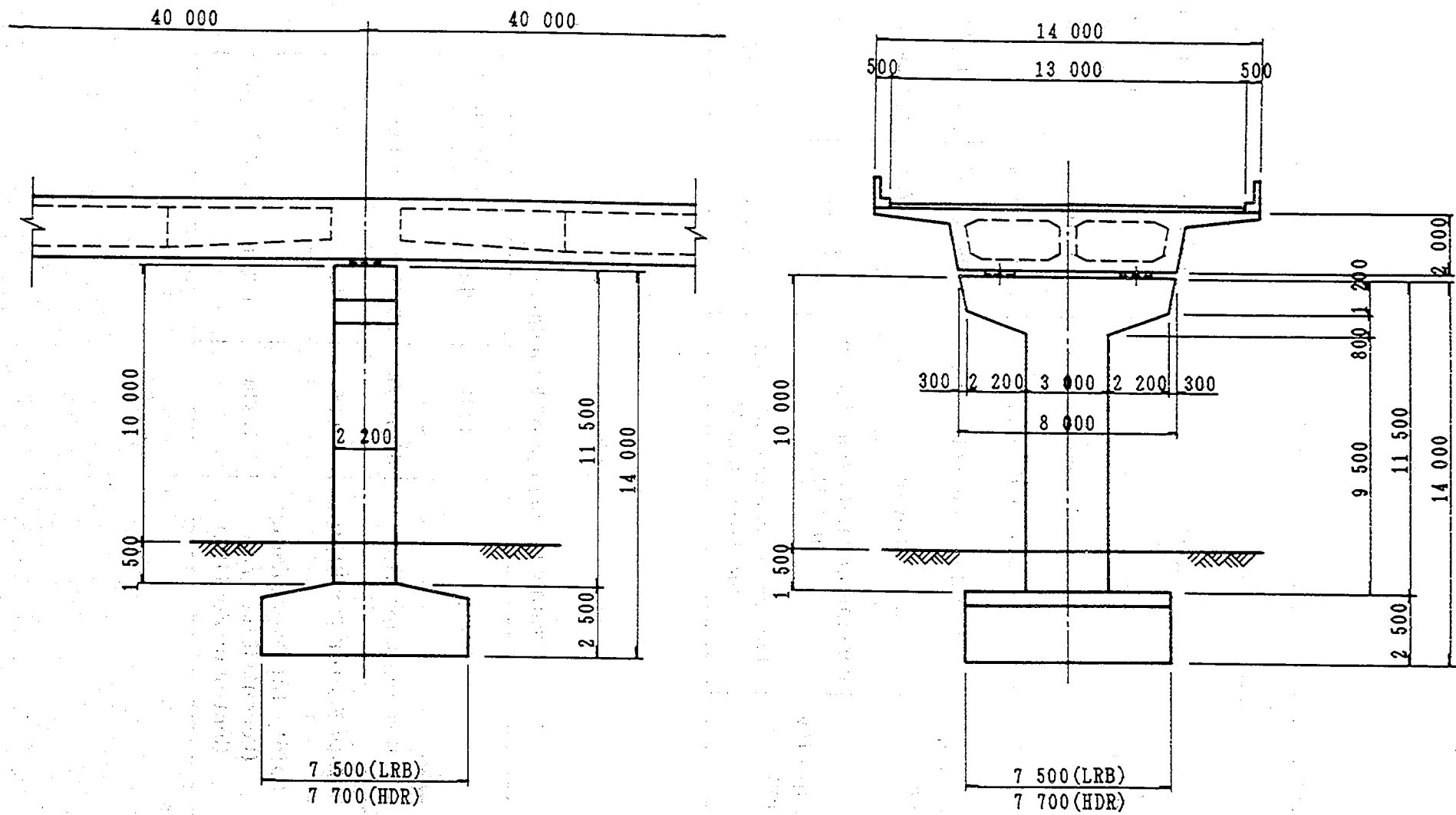
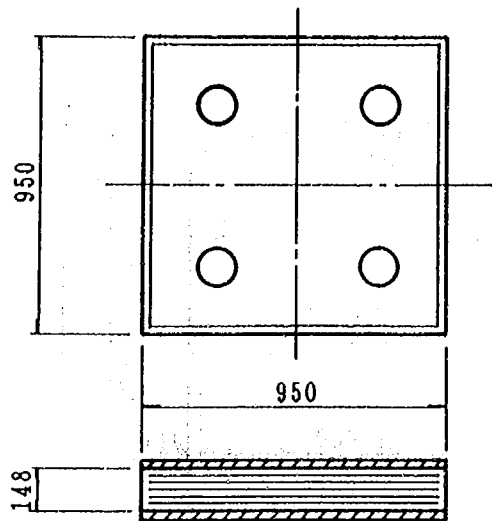
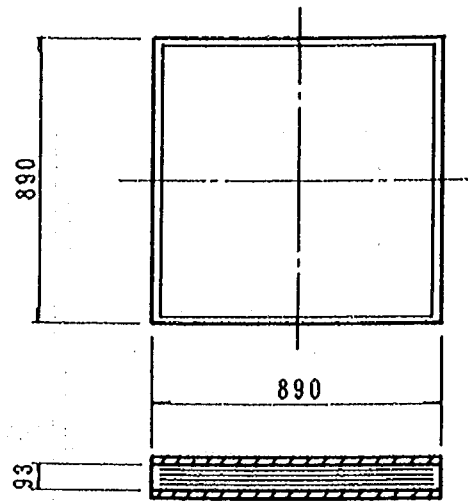


Fig.1 Bridge Considered in Design



(a) LRB



(b) HDR

Fig. 2 LRB and HDR

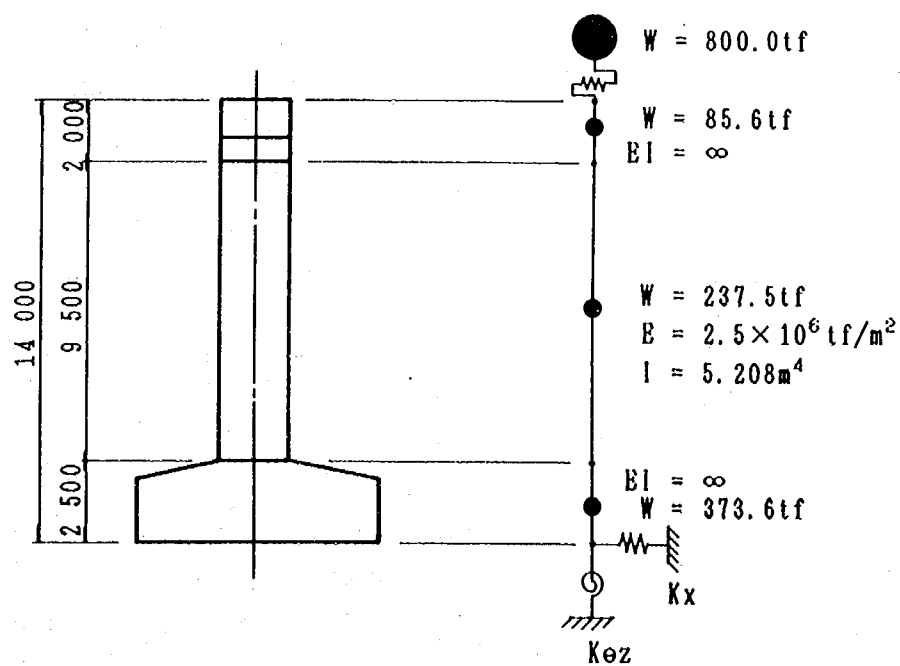


Fig. 3 Analytical Model in Design

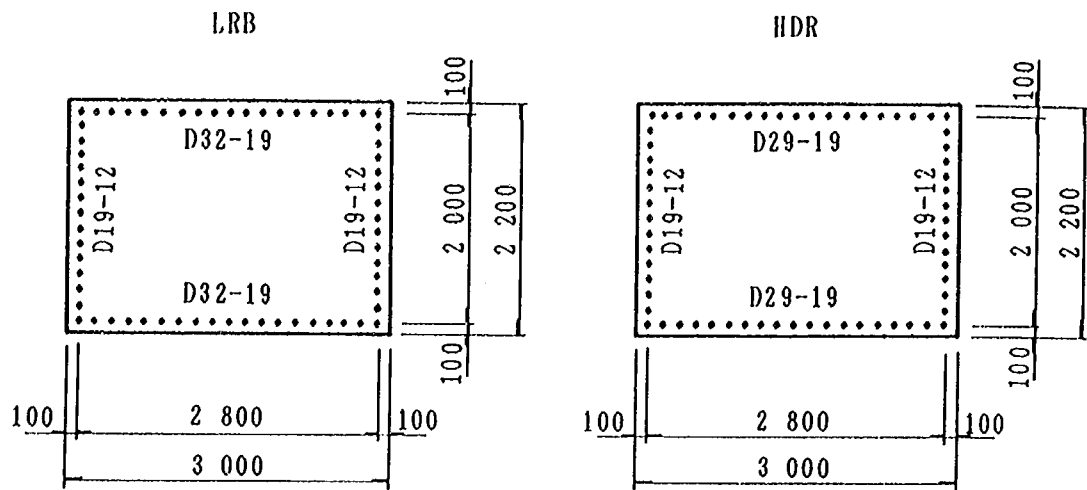


Fig.4 Section of Pier and Main Reinforcement

Evaluation of Shear Modulus of High Damping Rubber Bearings
and Lead Rubber Bearing During Low-Rate Load Reversals

Kazuhiko KAWASHIMA

Masahiro KOSHITOGI

Chiaki SUDO

Yutaka MAKIGUCHI

and

Kazuo ENDO

2nd U. S. -Japan Workshop on
Earthquake Protective Systems for Bridges

Tsukuba, Japan

December 1992

1. The first part of the report is devoted to a general survey of the situation in the country.

2. The second part of the report is devoted to a detailed analysis of the economic situation.

3. The third part of the report is devoted to a detailed analysis of the social situation.

4. The fourth part of the report is devoted to a detailed analysis of the political situation.

5. The fifth part of the report is devoted to a detailed analysis of the cultural situation.

6. The sixth part of the report is devoted to a detailed analysis of the environmental situation.

7. The seventh part of the report is devoted to a detailed analysis of the international situation.

8. The eighth part of the report is devoted to a detailed analysis of the future prospects.

9. The ninth part of the report is devoted to a detailed analysis of the conclusions.

10. The tenth part of the report is devoted to a detailed analysis of the recommendations.

11. The eleventh part of the report is devoted to a detailed analysis of the annexes.

12. The twelfth part of the report is devoted to a detailed analysis of the bibliography.

13. The thirteenth part of the report is devoted to a detailed analysis of the index.

14. The fourteenth part of the report is devoted to a detailed analysis of the appendices.

15. The fifteenth part of the report is devoted to a detailed analysis of the conclusions.

EVALUATION OF SHEAR MODULUS OF HIGH DAMPING RUBBER BEARINGS AND LEAD RUBBER BEARING DURING LOW-RATE LOAD REVERSALS

Kazuhiko KAWASHIMA¹⁾, Masahiro KOSHITOGI²⁾, Chiaki SUDO³⁾,
Yutaka MAKIGUCHI⁴⁾ and Kazuo ENDO⁵⁾

- 1) Head, Earthquake Engineering Division, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan
- 2) Engineer, Testing Division, MB Material Development Department, The Yokohama Rubber Co., Ltd, Hiratsuka City, Kanagawa-ken, Japan
- 3) Chief Engineer, New Products Department (Seismic Isolation), Bridgestone Corporation, Yokohama City, Kanagawa-ken, Japan
- 4) Staff Manager, Second Technical Development Department, Oiles Corporation, Fujisawa City, Kanagawa-ken, Japan
- 5) Chief Engineer, Technical Development Division, The Yokohama Rubber Co., Ltd, Hiratsuka City, Kanagawa-ken, Japan

ABSTRACT

Presented are two loading schemes proposed for evaluating the equivalent stiffness of Menshin devices against low rate loading developed due to the elongation and shrinkage of a bridge deck associated with the daily change of temperature. Although it is important for properly designing the Menshin devices against such deformation, it is generally difficult to evaluate it because the test requires a special loading facility which can produce the loading with extremely low frequency. Therefore the Extrapolation Method and the Relaxation Method are proposed for such purpose. The validity of the two methods is examined through the tests to two high damping rubber bearings and a lead rubber bearing.

INTRODUCTION

Menshin devices are subjected to various loadings developed by not only seismic effects but creep, shrinkage and temperature change effect. It is one of the main feature in bridge application that the elongation and shrinkage of a deck becomes important as the total deck length becomes long. Menshin devices are designed so that the lateral force developed in the devices by not only seismic effects but such temperature change, creep and shrinkage be properly distributed to as many piers as possible. For distributing the lateral force to piers, it is important to correctly evaluate the equivalent stiffness of the devices.

Some Menshin devices have the loading rate dependence of their equivalent stiffness. For example, some high damping rubber bearings (HDR) have lower equivalent stiffness against low rate loading such as the one developed due to the daily temperature change than the equivalent stiffness against seismic loading. Such characteristics are favorable for constructing super multi-span continuous bridge with the deck length over 1 km, because the

lateral force developed due to shrinkage and elongation of the deck associated with temperature change becomes quite large in such bridge. Therefore, if the equivalent stiffness against the low rate loading is small, it is favorable to reduce the lateral force caused by the deformation of the deck associated with temperature change.

For studying the equivalent stiffness against the low rate loading, a special loading facility which can produce the loading with extremely low frequency is required. However because the loading rate required is very low, it is not always easy to produce such low rate loading. Therefore special loading procedures are required for studying the equivalent stiffness against the low rate loading.

Two methods are proposed for such purpose in this paper. The validity of the procedures is presented through the implementation of the methods to two high damping rubber bearings and a lead rubber bearings.

EVALUATION METHOD OF SHEAR MODULUS AGAINST LOW RATE LOADING

Low Loading Rate Expected due to Daily Temperature Change

Associated with the deck elongation and shrinkage due to the daily temperature change, the Menshin devices are subjected to cyclic loading reversals with very low loading rate. The deck elongation and shrinkage is evaluated as

$$\Delta L = \varepsilon \cdot \Delta t \cdot L \quad (1)$$

where,

ΔL : elongation and shrinkage of deck (m)

ε : coefficient of elongation per unit length and unit temperature (1/m/°C)

Δt : increase of temperature (°C)

L : effective length of deck (m) against temperature change

The coefficient ε of elongation per unit length and unit temperature is 1.0×10^{-5} in concrete.

It is required to design the devices against such temperature change effect as¹⁾

$$u_D \leq \gamma_{DA} \cdot \sum t_e \quad (2)$$

$$u_B \leq \gamma_{BA} \cdot \sum t_e \quad (3)$$

where,

u_D : design displacement of Menshin device against normal load such as creep, shrinkage and temperature change effect (cm)

γ_{DA} : allowable shear strain of rubber against normal load (= 0.7)

u_B : design displacement of Menshin device against normal load and seismic effects

γ_{BA} : allowable shear strain of rubber against normal load and seismic

effects, and shall be 1.5 in the Seismic Coefficient Method and 2.5 in the Bearing Capacity Method
 Σt : total thickness of rubber (cm)

Because the Menshin devices are designed so that Eqs. (2) and (3) are satisfied, it can be considered that the shear strain induced in the Menshin devices due to the temperature change effect should be less than $\gamma_{0.0}$, i.e., 70 %. There are other contribution by the creep and shrinkage in concrete bridges. Therefore the shear strain induced in the Menshin devices against the temperature change effect may be between 50 % and 70 %.

Assuming that the daily temperature changes in a harmonic way as shown in Fig. 1 with the period of 24 hours, the averaged shear strain rate may be approximated as

$$\dot{\gamma} = \frac{\gamma}{t^*} = \frac{50\% \sim 70\%}{6 \text{ hours}} = 2.3 \times 10^{-5} / \text{sec} \sim 3.2 \times 10^{-5} / \text{sec} \quad (4)$$

in which γ and $\dot{\gamma}$ represent the shear strain and shear strain rate, and t^* represents a quarter of a day (6hours).

It should be noted that the shear strain rate during an earthquake is much larger than those values. Because the Menshin devices are designed against normal load and seismic load so that Eq. (3) be satisfied, one can consider that the shear strain developed in the Menshin device during an earthquake is at maximum 150 % in the Seismic Coefficient Method. Therefore assuming that the first natural period of the bridge is 2 second, the shear strain rate during an earthquake may be estimated as

$$\dot{\gamma} = \frac{\gamma}{t^*} = \frac{150 \%}{1/4 \text{ second}} = 3 / \text{sec} \quad (5)$$

Extrapolation Method

If the loading rate vs. the equivalent stiffness of the Menshin devices can be evaluated for a set of loading rates which can be produced by the testing machine available, the equivalent stiffness for the desired loading rate may be estimated by extrapolating the relation. This method is not special but may be used for most type of Menshin devices.

In the Manual for the Menshin Design of Highway Bridges, it is recommended to conduct the test for loading rate of 0.002 cm/sec, 0.005 cm/sec, 0.01 cm/sec, 0.05 cm/sec, 0.1 cm/sec, 0.5 cm/sec, 1.0 cm/sec and 5.0 cm/sec. The Menshin devices are subjected to a harmonic excitation with either strain amplitude of $\pm 50\%$ or displacement amplitude of ± 7.5 cm under the vertical load equivalent to the dead weight of superstructure and at the temperature of 20°C.

Relaxation Method

Although the mechanism causing the loading rate dependence of the shear modulus of rubber (high damping rubber) is not well known, it may be related to the relaxation of the rubber. Therefore the loading rate dependence of the shear modulus of rubber may be evaluated by the relaxation method.

When a rubber specimen is subjected to a constant shear strain for a certain period, the stress induced in the specimen decreases as shown in Fig. 2 (a) and (b). Because shear modulus of the rubber is defined as

$$G_s = \frac{\tau}{\gamma} \quad (6)$$

where,

G_s : shear modulus of rubber (kgf/cm²)

τ : shear stress (kgf/cm²)

γ : shear strain

shear modulus G_s vs. period of relaxation t^* relation can be written as shown in Fig. 2(c). The averaged shear strain rate for a period of relaxation t^* may be evaluated from Eq. (4) as

$$\dot{\gamma} = \frac{\gamma}{t^*} \quad (7)$$

Therefore, the shear modulus G_s vs. shear strain rate $\dot{\gamma}$ relation may be written as shown in Fig. 2(d). It should be noted that this relation is similar in nature with the loading frequency dependence of the shear modulus of rubber subjected to a harmonic excitation. Therefore, by specifying an appropriate shear strain rate $\dot{\gamma}$, one can estimate the shear modulus for $\dot{\gamma}$. It should be noted that the equivalent stiffness of the Menshin devices can be obtained from the shear modulus of the rubber as

$$K_E = \frac{A_R \cdot G_s}{\Sigma t_s} \quad (8)$$

where,

K_E : Equivalent Stiffness of Menshin device (kgf/cm)

A_R : Sectional area of Menshin device (cm²)

G_s : Shear modulus of rubber (kgf/cm²)

Σt_s : Total thickness of rubber (cm)

This method is proposed hereinafter as the "Relaxation Method".

However various consideration has to be made for applying the Relaxation Method for evaluating the equivalent stiffness of the Menshin devices subjected to the daily temperature change, because the shear strain developed in the Menshin devices by the daily temperature effect changes with time as shown in Fig. 1.

As the first approximation for such change, a constant strain may be applied to the devices as shown in Fig. 3(a). But more accurate result may be obtained by applying the strain in several steps as shown in Fig. 3 (b) and (c). It is expected that the accuracy would be improved by increasing the number of load step.

In such a stepwise loading, the load need to be applied as follows. Consider the case where the shear strain is applied to the devices in three steps. The shear strain for which the low loading rate stiffness is required (target shear strain γ_T) is divided into three steps as shown in Fig. 4(a). The load needs to be applied to the specimen until the shear strain induced in the specimen reaches to 1/3 of the target shear strain. Then hold this shear strain for two hours so that relaxation be developed. The stress of the specimen decreases as shown in Fig. 4(b) due to relaxation. Then, apply the load until the shear strain induced in the specimen reaches to 2/3 of the target shear strain and hold it for two hours. Finally the same procedure needs to be repeated for the target shear strain. The stress developed in the specimen after the relaxation for two hours at this shear strain level is the shear stress required. This approximates the shear strain developed against a quarter cycle of harmonic loading with the period of 24 hours. Because actuator can be held for two hours after each stepwise loading, a special loading facility which is capable to produce the extremely low frequency loading is not required.

The shear stress vs. shear strain relation of Fig. 4 (a) and (b) can be obtained as shown in Fig. 4(c).

Although it was proposed to apply the shear strain as shown in Fig. 4 (a), it may be applied as shown in Fig. 5 (Loading Scheme B) as an alternative scheme. The Loading Scheme A which was proposed in Fig. 4 (a) would give higher equivalent stiffness than the Loading Scheme B, because relaxation time at the target shear strain is longer. Loading scheme for approximating the actual change of the daily temperature needs to be carefully set.

TEST MODELS

Three types of test specimens as shown in Table 1 were used to evaluate the stiffness of the devices against low strain rate load reversals. The HDR-A and LRB are specimens fabricated for this test. The HDR-B is the specimen with the same size and characteristics with the one adopted for the Yama-age Bridge²⁾. Figs. 6,7 and 8 as well as photos 1, 2 and 3 show the specimens.

LOW LOADING RATE SHEAR MODULUS OF HDR-A

Extrapolation Method

In the Extrapolation Method, the HDR-A was subjected to a harmonic loading with the displacement amplitude equivalent to 50% shear strain of the rubber. The loading frequency was varied as 0.05 Hz, 0.1 Hz, 0.05 Hz, 0.01 Hz,

0.005 Hz, 0.001 Hz and 0.00025 Hz. Because the strain rate may be evaluated as

$$\dot{\gamma} = 4 \cdot \gamma \cdot f \quad (9)$$

where,

$\dot{\gamma}$: strain rate (1/sec)

γ : shear strain

f : frequency (Hz = 1/sec)

the strain rate in the tests was from 1/sec to 0.00041/sec.

The vertical load of 14.7 tf, which corresponds to the vertical stress of 60 kgf/cm², was applied to the specimen for representing the dead weight of the deck. A special loading facility at the Yokohama Rubber Co. was used for the test.

Fig. 9 shows the hysteresis of shear strain vs. shear stress relation for the shear strain rate of 1/sec, 0.021/sec and 0.00041 /sec. It is apparent that the equivalent stiffness decreases as the shear strain rate decreases. Fig. 10 and Table 2 show how the equivalent stiffness in terms of shear modulus decreases as the shear strain rate decreases. Because the test results for various strain rates are consistent, they may be approximated as

$$G_s = 5.83 \dot{\gamma}^{0.204} + 7.0 \quad (10)$$

where,

G_s : shear modulus of HDR-A (kgf/cm²)

$\dot{\gamma}$: shear strain rate (1/sec)

By substituting $\dot{\gamma} = 2.3 \times 10^{-5}$ /sec from Eq. (4), one can estimate the shear modulus of 7.7 kgf/cm². Because the shear strain during an earthquake is about 3/sec by Eq. (5), the shear modulus at the shear strain rate of 3/sec is evaluated by Eq. (10) as 14.3. The 7.7 kgf/cm² is about 46 % smaller than 14.3.

Relaxation Method

After the Extrapolation Method was made, the same specimen was used for the Relaxation Method. In the Relaxation Method, the target shear strain of 50 % was applied in the period of 6 hours. The loading step was set as 1, 2 and 4. In case of loading step of 1, the specimen was subjected to a load with the loading rate of 1 mm/sec (shear strain rate of 2.1×10^{-2} /sec) until 50 % shear strain. Then holding of the shear strain was made for 6 hours to have the relaxation. In the case of loading step of 4, the specimen was firstly loaded until 12.5 % shear strain, and the the shear strain was kept constant for 1.5 hour. The load was then increased so that 25% shear strain was developed, and the holding of the shear strain was made for 1.5 hour. Similarly, the load was stepwisely increased until the shear strain reached to 50%. It should be noted that because the shear strain was increased from 0 to 50% for 6 hours, the shear strain rate is 2.3×10^{-5} /sec for all the three tests.

Fig. 11(a) shows the shear strain vs. relaxation time relation thus used for the three types of loading steps. The relaxation of shear force of the specimen was developed as shown in Fig. 11(b). It is seen that the shear force at the shear strain of 50% slightly increases as the number of load step increases from 1 to 4. It is expected that the case of 4 loading step would give more accurate result. Fig. 11 (c) shows the shear force vs. shear strain relation.

From those results, the shear modulus of the HDR-A was evaluated as shown in Table 3. The shear modulus was 7.0 kgf/cm^2 in case of 4 load step. Reminding that for the shear strain rate of $2.3 \times 10^{-5}/\text{sec}$ the shear modulus of the HDR-A evaluated by the Extrapolation Method was 7.7 kgf/cm^2 , the shear modulus evaluated by the Relaxation Method is very close with that evaluated by the Extrapolation Method.

LOW LOADING RATE SHEAR MODULUS OF LRB

Extrapolation Method

The specimen was subjected to a harmonic loading with the displacement amplitude equivalent to 70% shear strain of the rubber. The loading frequency was varied as 1 Hz, 0.023 Hz, 0.0023 Hz, 0.00023 Hz and 0.000012 Hz. Because a specially designed facility which is capable for evaluating the low rate loading at Oiles Corporation was used, loading test could be made for the target shear strain (70 % / 6 hours = $3.2 \times 10^{-5}/\text{sec}$). The shear strain rate by Eq. (9) is from 4.4/sec to 0.00005/sec in this series of tests. The vertical load of 30.2 tf, which corresponds to the vertical stress of 60 kgf/cm^2 , was applied to the specimen.

Fig. 12 shows the hysteresis of shear strain vs. shear force relation for the shear strain rate of 4.4/sec, 0.01/sec and 0.0002/sec. From these results, shear strain rate dependence of the shear modulus was obtained as shown in Fig. 13 and Table 4. The shear modulus G_s vs. shear strain rate $\dot{\gamma}$ relation may be approximated as

$$G_s = 12.1 \dot{\gamma}^{0.0406} \quad (11)$$

From Eq. (11) the shear modulus for the shear strain rate of $3.2 \times 10^{-5}/\text{sec}$ is estimated as 7.9 kgf/cm^2 . On the other hand, the shear strain rate developed during an earthquake is estimated by Eq. (5) as 3/sec. The shear modulus corresponding to this shear strain rate is evaluated by Eq. (11) as 12.7 kgf/cm^2 . Therefore the above shear modulus for the daily temperature change is approximately 38 % smaller than the one developed during an earthquake.

Relaxation Method

The same specimen was used for the Relaxation Method. The target shear

strain was assumed as 70 %, and the number of load step was 1. Therefore the load was increased until the shear strain of the specimen reached 70 %, and then holding of this shear strain was made for 6 hours.

Fig. 14 shows the decrease of stress developed in the specimen and the shear force vs. shear strain relation. From these results, the shear modulus of the specimen was obtained as 7.7 kgf/cm² for the loading rate of 3.2×10^{-5} /sec. Because the shear modulus evaluated from the Extrapolation Method is 7.9 kgf/cm², this value is quite close with the one evaluated by the Extrapolation Method.

LOW LOADING RATE SHEAR MODULUS OF HDR-B

Only Extrapolation Method was applied to the HDR-B. The specimen was subjected to a harmonic loading with the displacement amplitude of 70 % of the shear strain of the rubber. The loading frequency was varied as 0.005 Hz, 0.0011 Hz, 0.00011 Hz and 0.000012 Hz. The vertical load of 238 tf, which corresponds to the vertical stress of 40 kgf/cm², was applied to the specimen beside the lateral loading.

Fig. 15 shows the hysteresis of shear force vs. shear strain. From these results, the shear strain rate dependence of the shear modulus was evaluated as shown in Fig. 16 and Table 5. This can be approximated as

$$G_s = 11.1 \cdot \dot{\gamma}^{0.0418} \quad (12)$$

Based on Eq. (12), the shear modulus corresponding to the shear strain rate of 3.2×10^{-5} /sec and 3/sec is evaluated as 7.2 kgf/cm² and 11.6 kgf/cm². Therefore, the shear modulus developed during the daily temperature change is 38 % smaller than that developed during an earthquake.

CONCLUSIONS

For evaluating the equivalent stiffness of the Menshin devices subjected to low shear strain rate loading, two methods were proposed. In both the Extrapolation Method and the Relaxation Method, the test can be made even if a loading facility which is capable for producing extremely low frequency loading is not available. A series of loading tests were made for verifying the test methods proposed with use of two high damping rubber bearings and a lead rubber bearing. From the results presented in, the following conclusions may be deduced :

1) The shear modulus of HDR-A evaluated by the Extrapolation Method for the shear strain rate of 2.3×10^{-5} /sec (= 50 % / 6 hours) is 7.7 kgf/cm², while the value evaluated by the Relaxation Method is 7.0 kgf/cm².

2) The shear modulus of LRB evaluated by the Extrapolation Method for the shear strain rate of 3.2×10^{-5} /sec (= 70 % / 6 hours) is 7.9 kgf/cm², while the value evaluated by the Relaxation Method is 7.7 kgf/cm².

3) Based on 1) and 2), it can be said that the Relaxation Method gives the close shear modulus by the Extrapolation Method. Therefore, the Relaxation Method can be used as well as the Extrapolation Method for estimating the equivalent stiffness of Menshin devices for low rate loading developed by the temperature change.

4) Based on the Extrapolation Method, the shear strain rate dependence of the shear modulus is given by Eq. (10) for HDR-A and by Eq. (11) for LRB.

5) Assuming that the shear strain rate developed during an earthquake is 3/sec and that the shear strain rate due to the daily temperature change is 2.3×10^{-5} for HDR-A and 3.2×10^{-5} for LRB and HDR-B, the shear modulus against the daily temperature change is 46 % (HDR-A), 38 % (HDR-B) and 38 % (LRB) smaller than the one developed during an earthquake.

REFERENCES

- 1) Public Works research Institute and 28 Private Firms : Manual for Menshin Design of Highway Bridges, Bulletin of PWRI, Vol. 59, Public Works research Institute, March 1992
- 2) Ikeda, T., Kumakura, K, Oozeki, K. and Abe, N. : Design of Karasuyama (Yama-Age), Bridge, Bridges and Foundation, Vol. 91-6, 1991

Table 1 Specimens Used for Loading Tests

Specimen	Natural Period in Design (sec)	Dead Weight with Live Load		Design Displacement of Menshin Device (cm)
		Vertical Load (tf)	Vertical Stress (kgf/cm ²)	
HDR-A	1.2	15	60	7
HDR-B	1.4	328	56	33
LRB	0.9	30	60	6

Table 2 Shear Strain Rate Dependence of Shear Modulus of HDR-A evaluated by Relaxation Method

Frequency Hz	Strain Rate (1/sec)	Shear Modulus (kgf/cm ²)
2.5×10^{-4}	4.1×10^{-4}	8.3
1.0×10^{-3}	2.1×10^{-3}	8.5
5.0×10^{-3}	1.1×10^{-2}	9.3
1.0×10^{-2}	2.1×10^{-2}	9.7
5.0×10^{-2}	1.1×10^{-1}	10.6
1.0×10^{-1}	2.1×10^{-1}	11.2
5.0×10^{-1}	1.0	12.9

Table 3 Effect of Number of Load Step by Relaxation Method (HDR-A)

Number of Load Step	Shear Modulus G_s (kgf/cm ²)
1	6.1
2	6.5
4	7.0

Table 4 Shear Strain Dependence of Shear Modulus of LRB by Extrapolation Method

Frequency Hz	Strain Rate (1/sec)	Shear Modulus (kgf/cm ²)	
		No.1 Facilities	No.2 Facilities
4.6×10^{-5}	2.0×10^{-4}	—	8.7
2.0×10^{-4}	8.8×10^{-4}	9.0	9.2
2.0×10^{-3}	8.8×10^{-3}	9.8	10.1
2.0×10^{-2}	8.8×10^{-2}	10.7	—
1.0	4.4	13.1	—

Table 5 Shear Strain Dependence of Shear Modulus of HDR-B by Extrapolation Method

Frequency Hz	Strain Rate (1/sec)	Shear Modulus (kgf/cm ²)
1.2×10^{-5}	3.3×10^{-5}	7.2
1.1×10^{-4}	3.2×10^{-4}	7.9
1.1×10^{-3}	3.2×10^{-3}	8.8
5.0×10^{-3}	1.4×10^{-2}	9.3

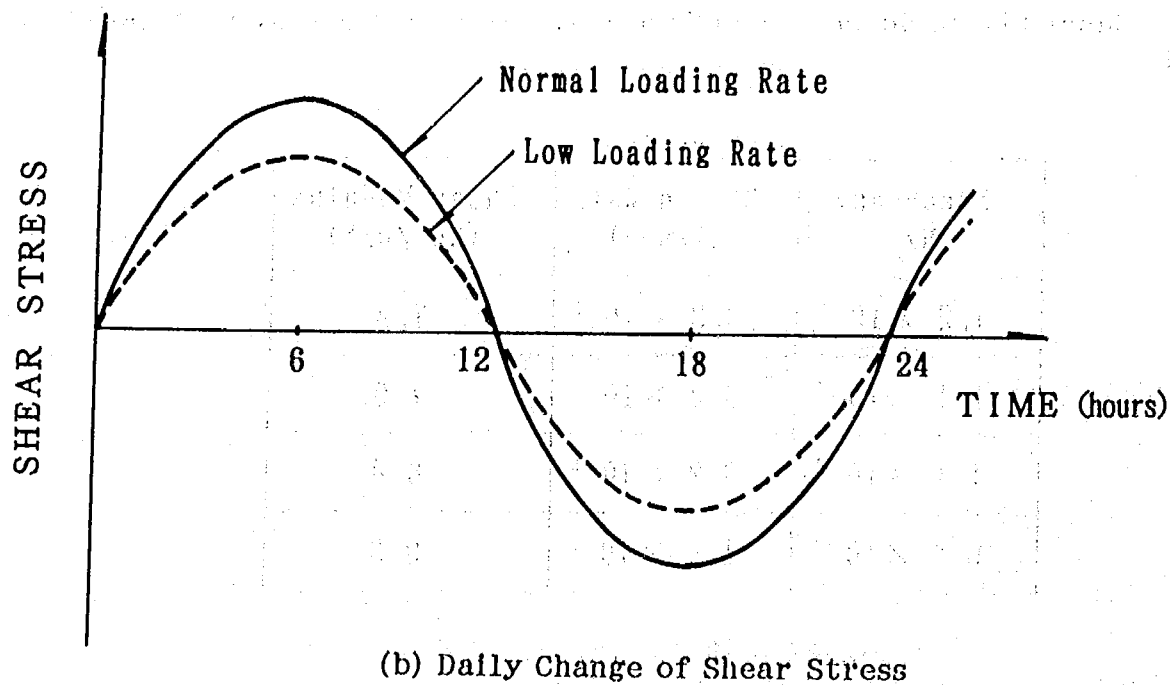
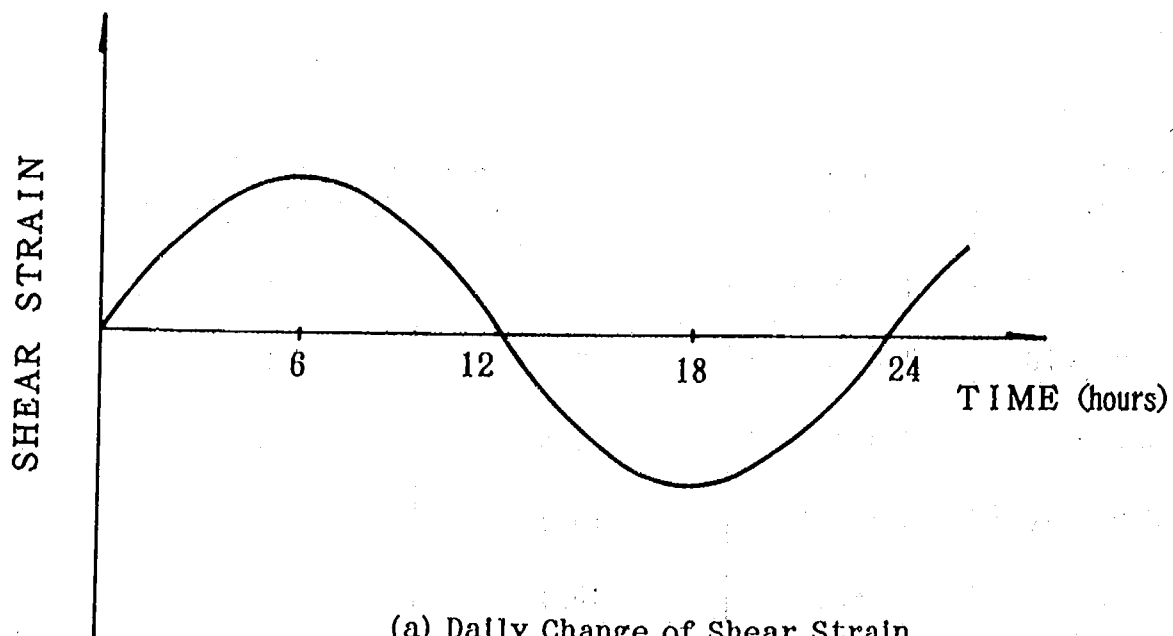
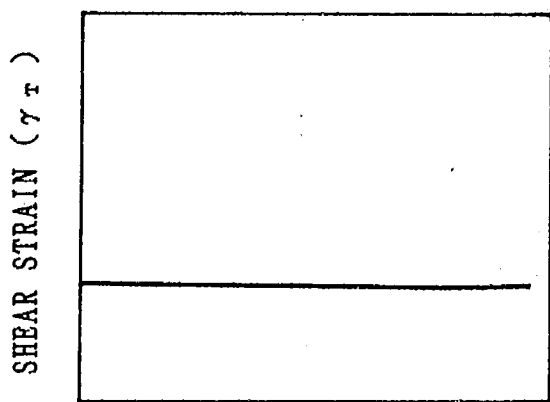
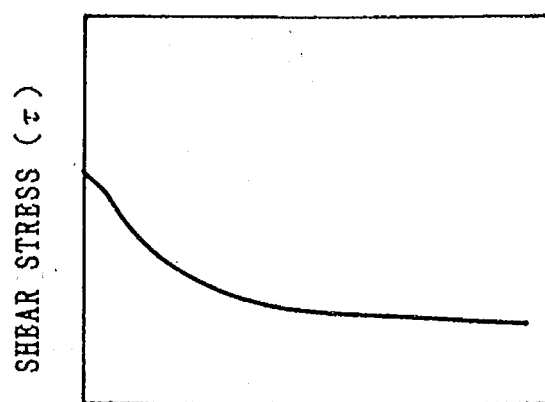


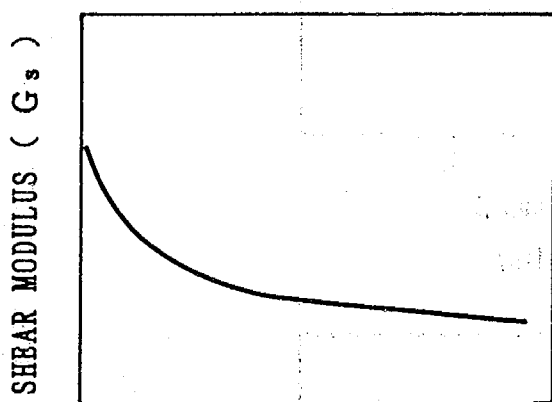
Fig. 1 Change of Shear Stress Shear Strain Induced in Menshin devices by Daily Change of Temperature



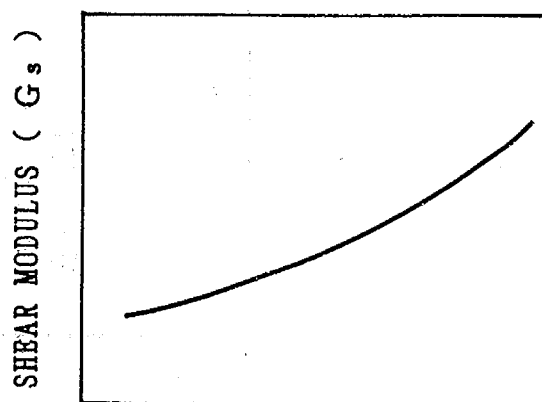
(a) Constant Strain Applied to Specimen



(b) Relaxation of Stress

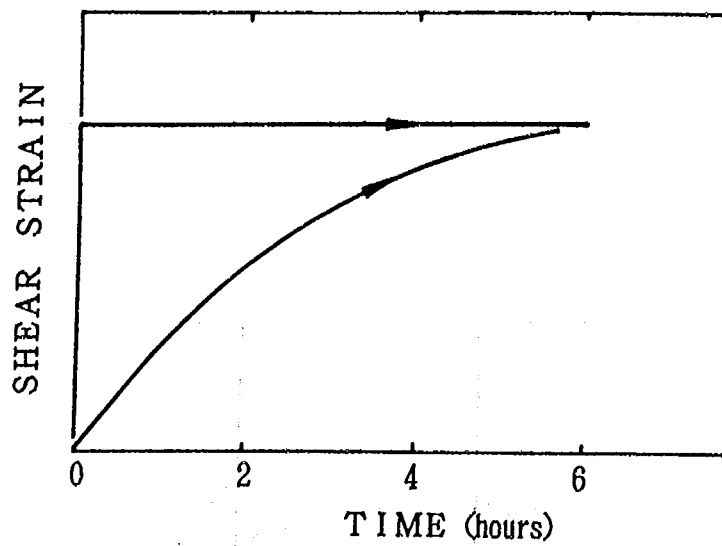


(c) Relaxation of Shear Modulus

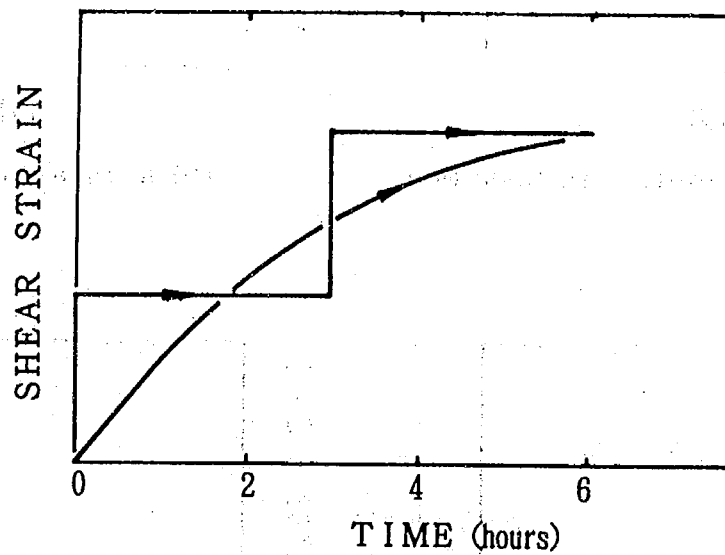


(d) Shear Modulus vs. Shear Strain Rate

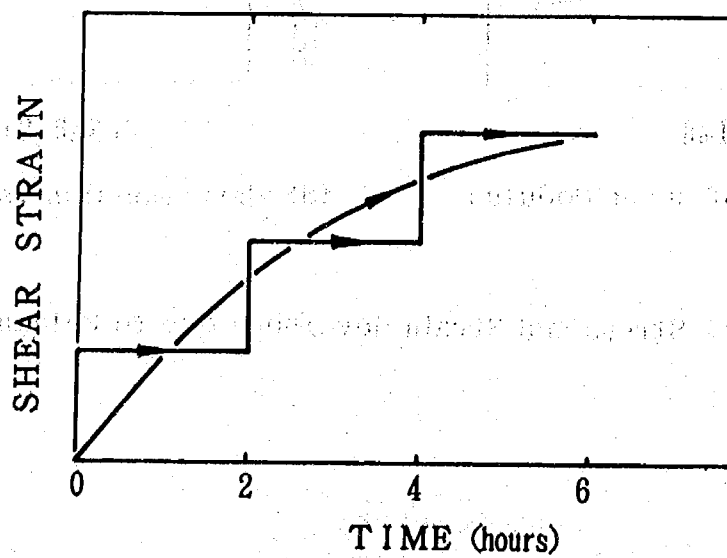
Fig. 2 Stress and Strain developed due to Relaxation



(a) 1 Step Loading

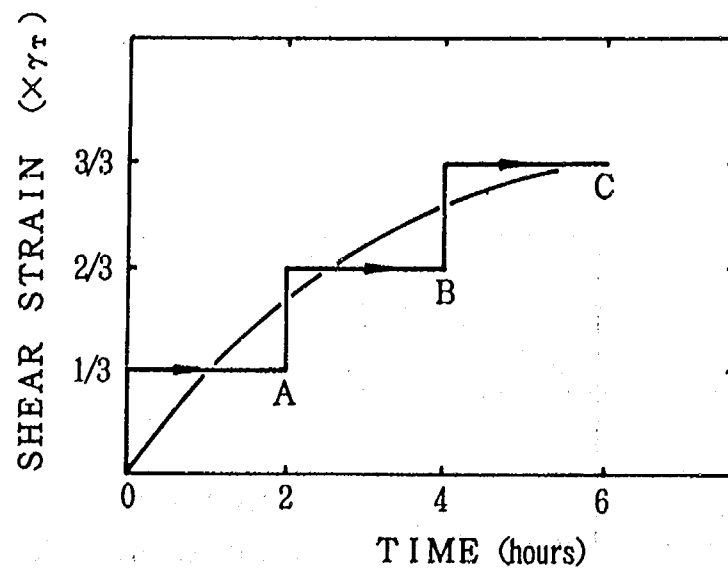


(b) 2 Step Loading

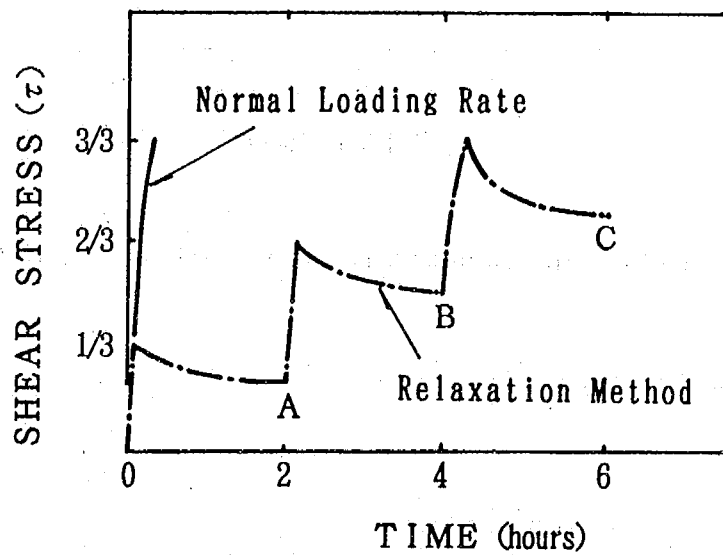


(c) 3 Step Loading

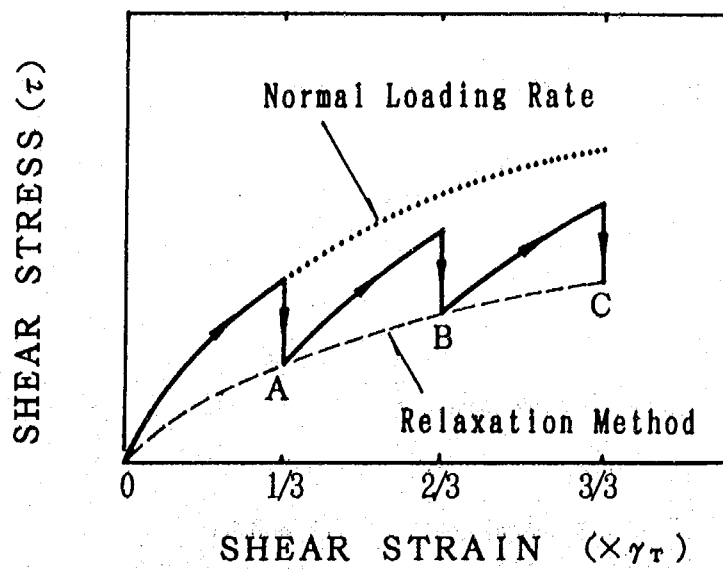
Fig. 3 Approximation by Stepwise Loading in Relaxation Method



(a) Shear Strain Applied to Specimen



(b) Shear Stress Hysteresis



(c) Shear Stress vs. Shear Strain Rate Relation

Fig. 4 Relaxation of Shear Stress vs. Shear Strain Rate in Relaxation Method for 3 Step Loading

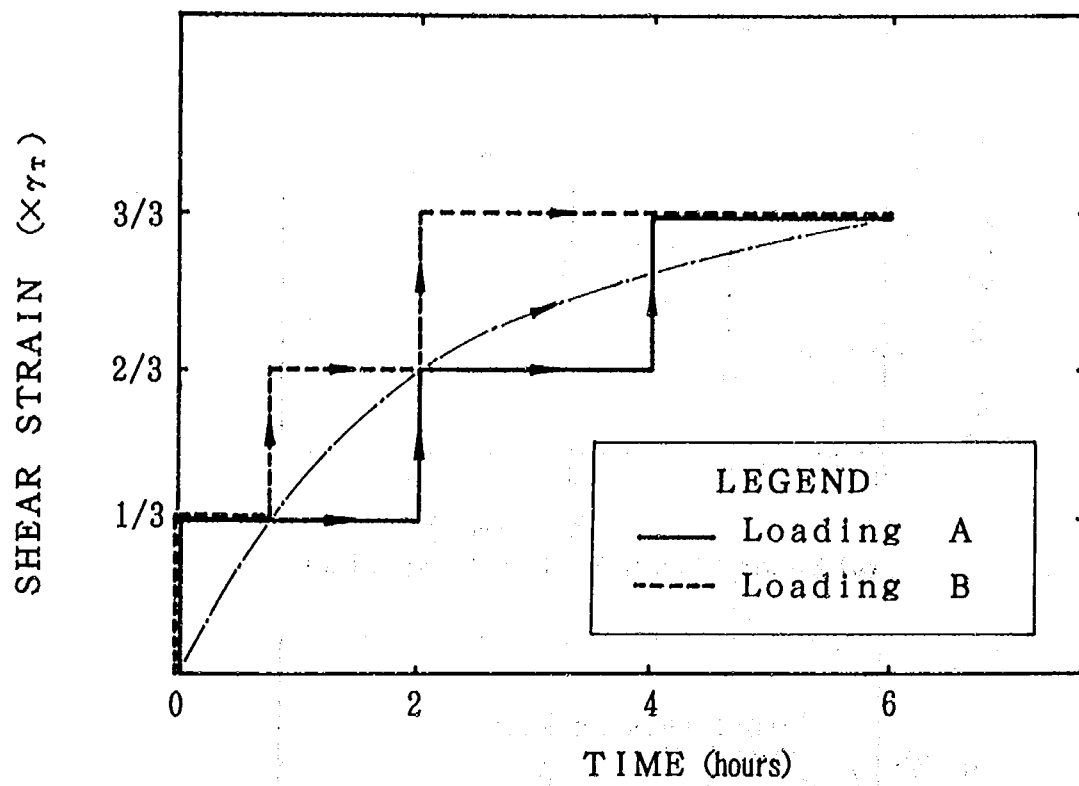
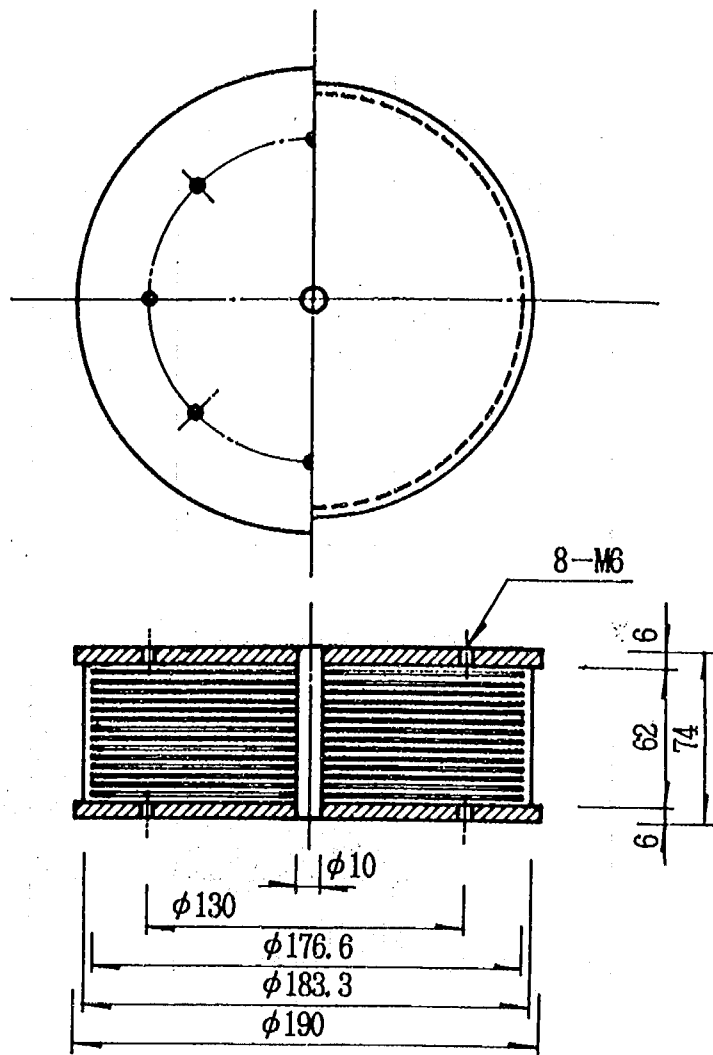


Fig. 5 Two Loading Schemes for Controlling Shear Strain



Rubber Layer $3.17\text{mm} \times 15 = 47.5\text{mm}$

Steel Plate $1.0\text{mm} \times 14 = 14.0\text{mm}$

Fig. 6 Test Specimen of HDR-A

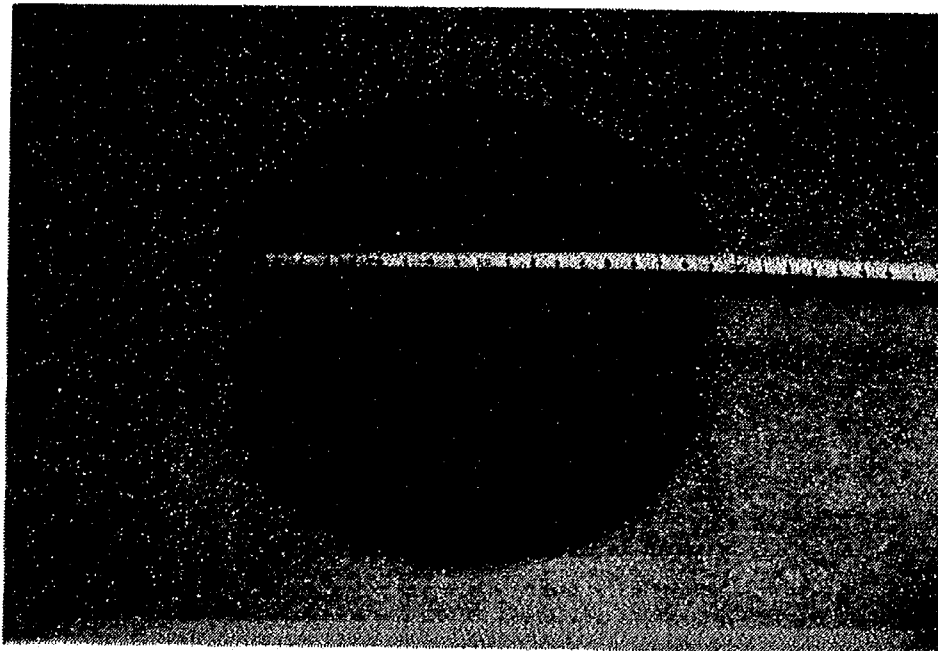
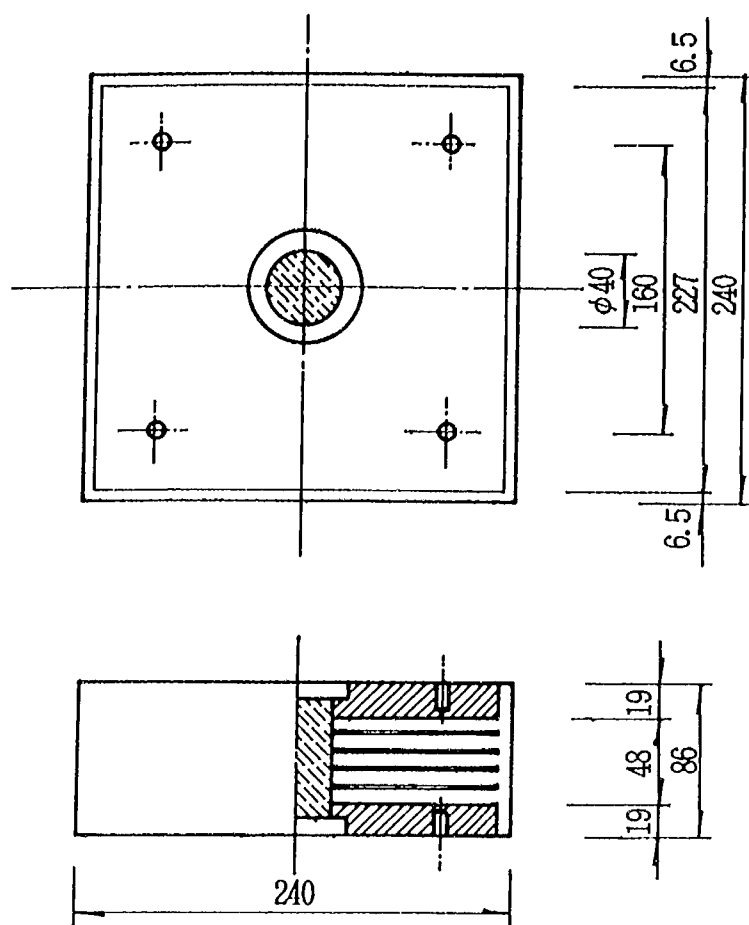


Photo 1 Test Specimen of HDR-A



Rubber Layer $8.0\text{mm} \times 5 = 40.0\text{mm}$

Steel Plate $2.0\text{mm} \times 4 = 8.0\text{mm}$

Fig. 7 Test Specimen of LRB

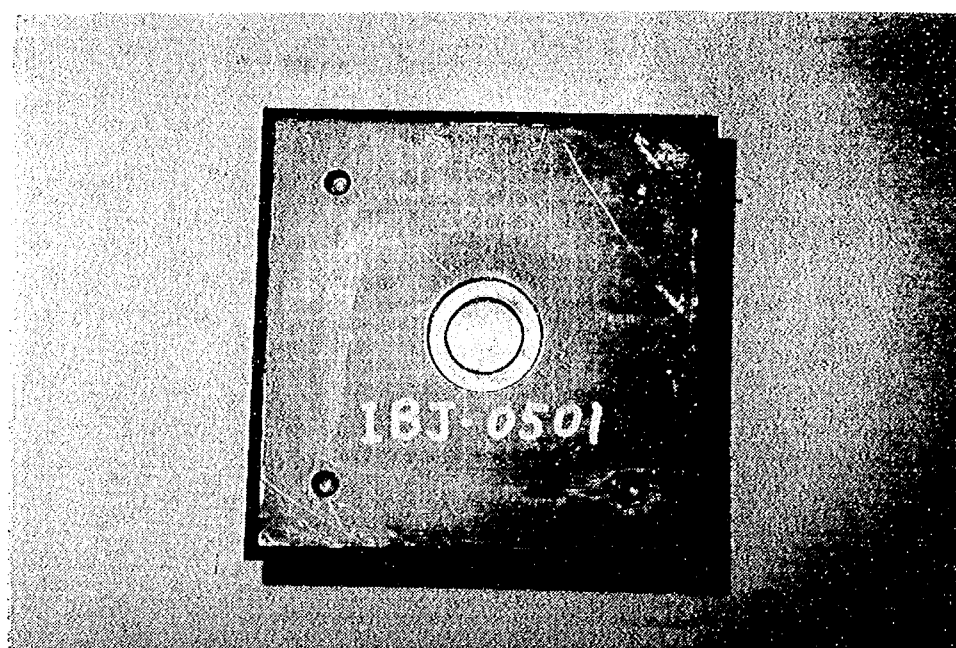
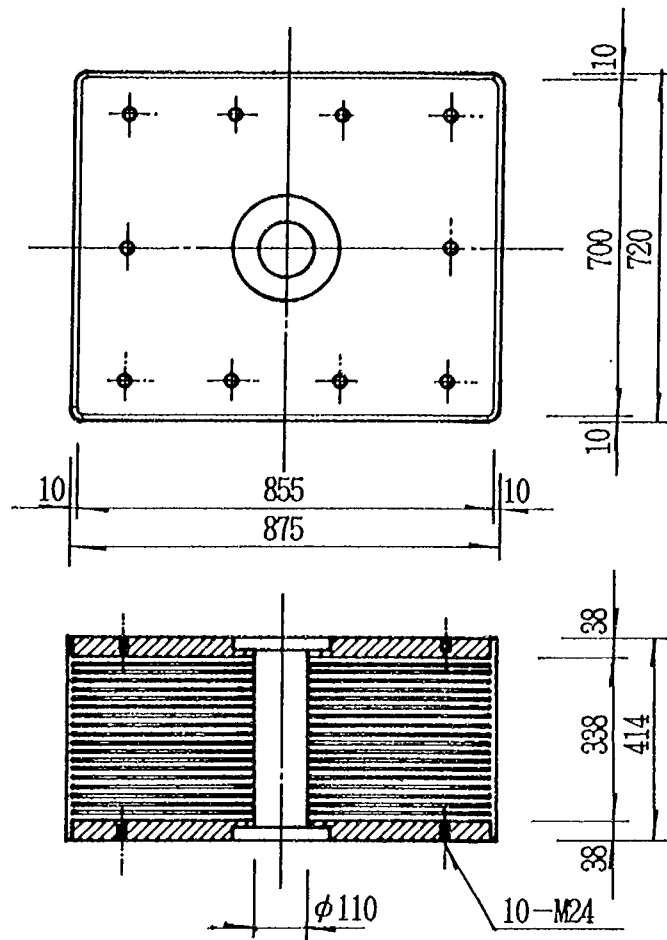


Photo 2 Test Specimen of LRB



Rubber Layer $13.8\text{mm} \times 19 = 262.2\text{mm}$

Steel Plate $4.2\text{mm} \times 18 = 75.6\text{mm}$

Fig. 8 Test Specimen of HDR-B

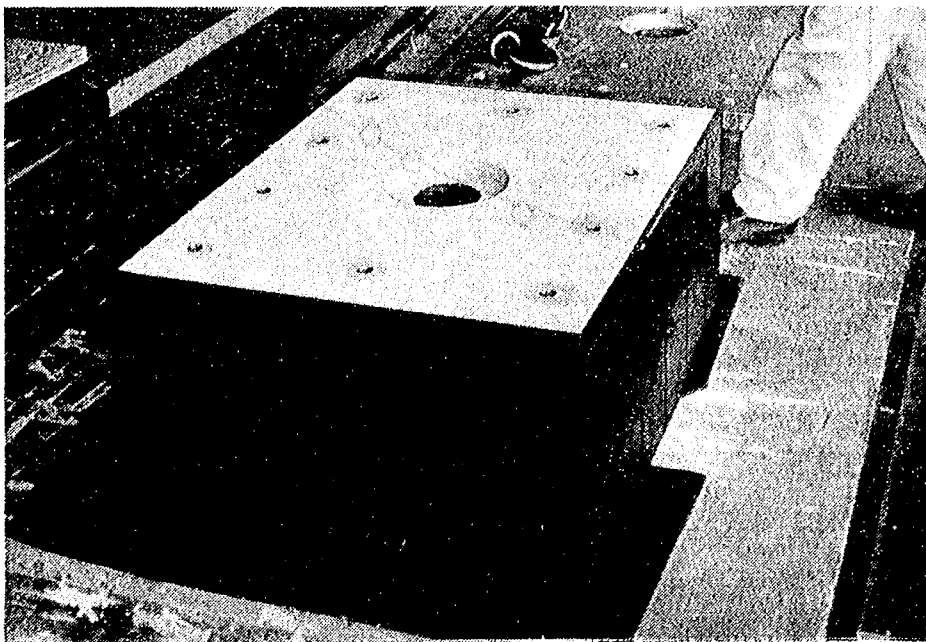


Photo 3 Test Specimen of HDR-B

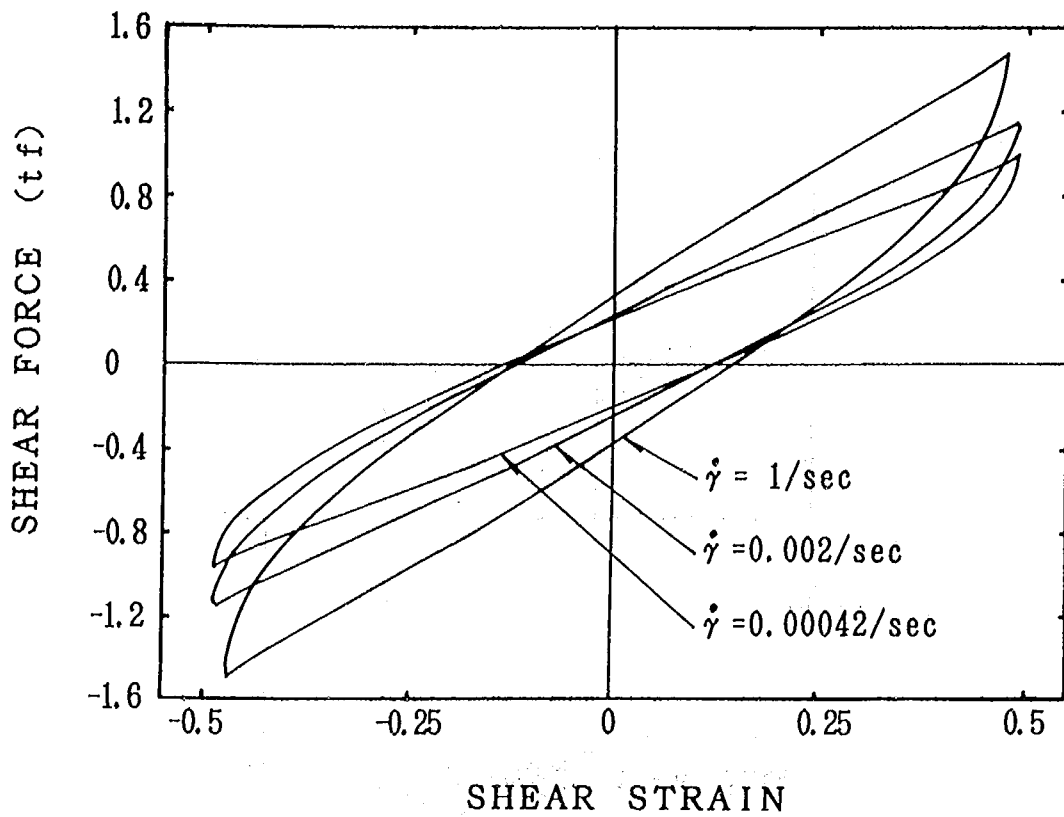


Fig. 9 Hysteresis Loops of HDR-A Subjected to a Harmonic Excitation with Displacement Amplitude of 50% Shear Strain of Rubber

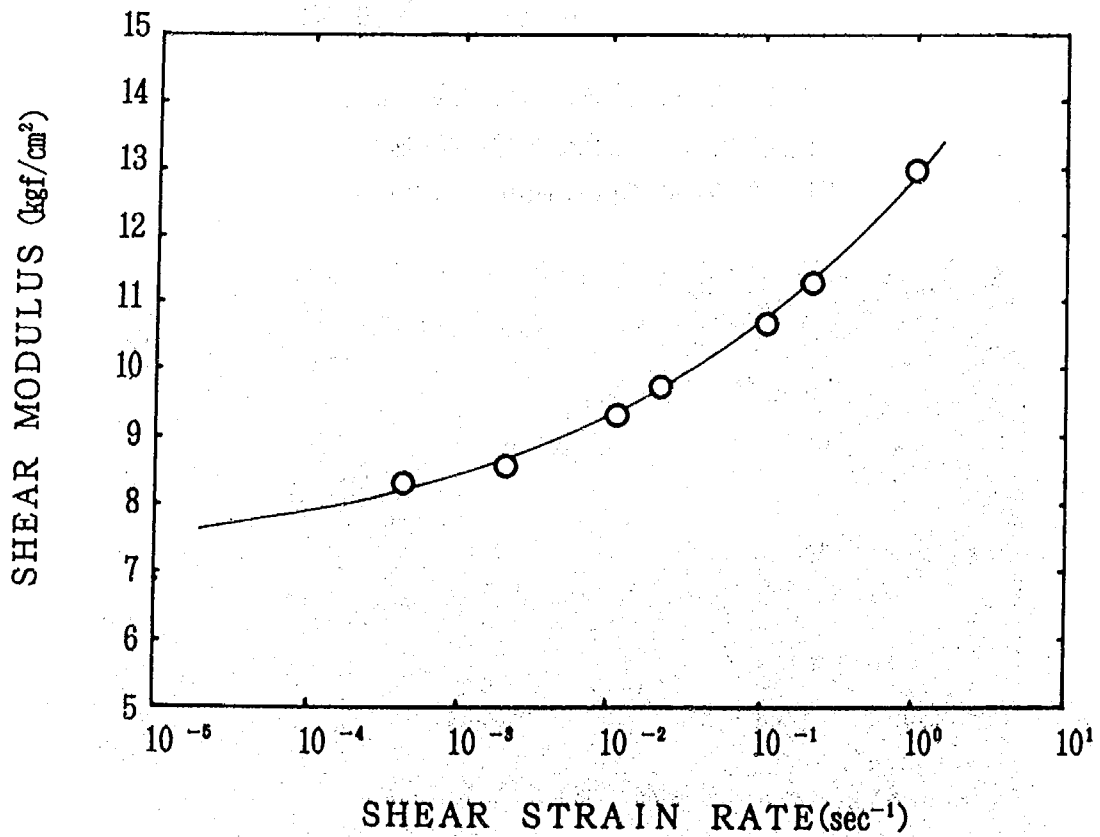
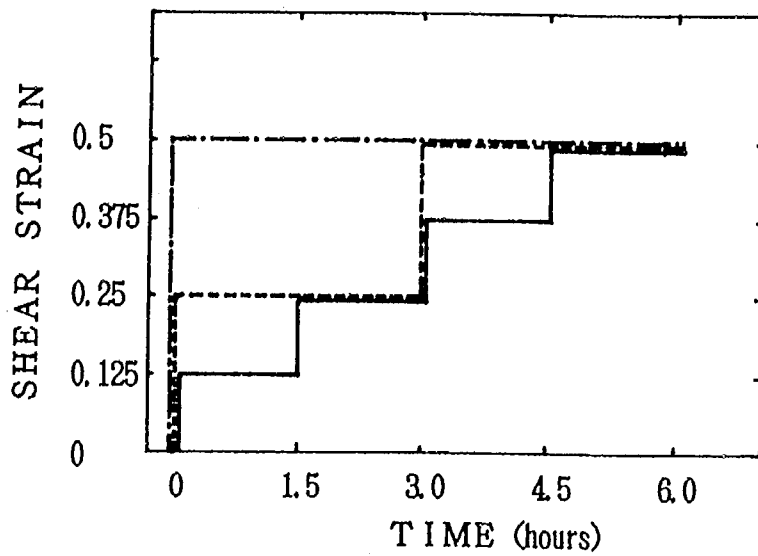
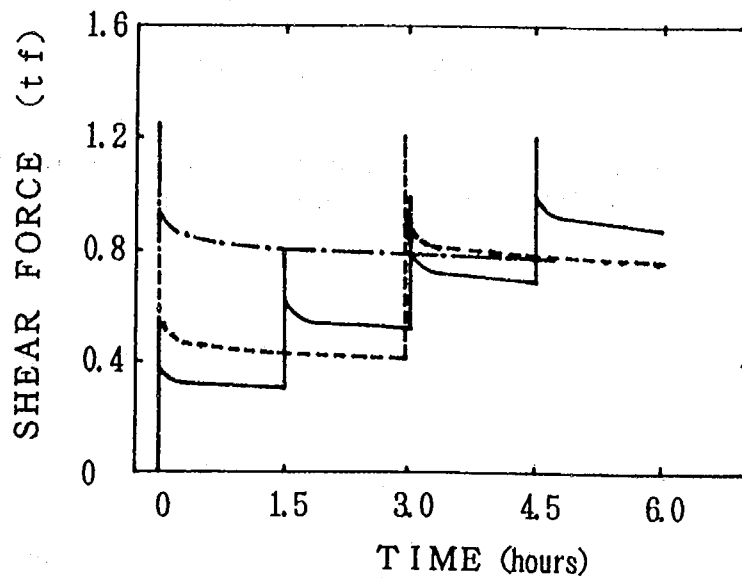


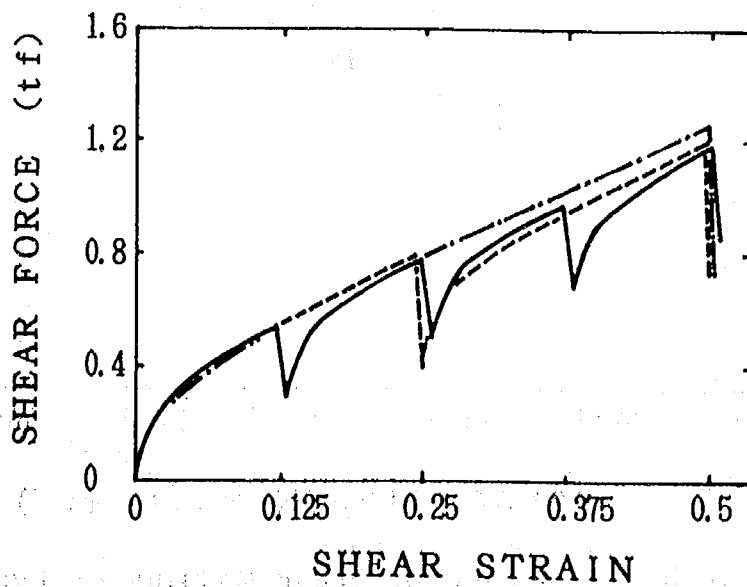
Fig. 10 Shear Strain Dependence of Shear Modulus of HDR-A Evaluated by Extrapolation Method



(a) Shear Strain Applied to Specimen



(b) Shear Stress Hysteresis



(c) Shear Strain Rate Dependence of Shear Strain

Fig. 11 Relaxation Method for HDR-A

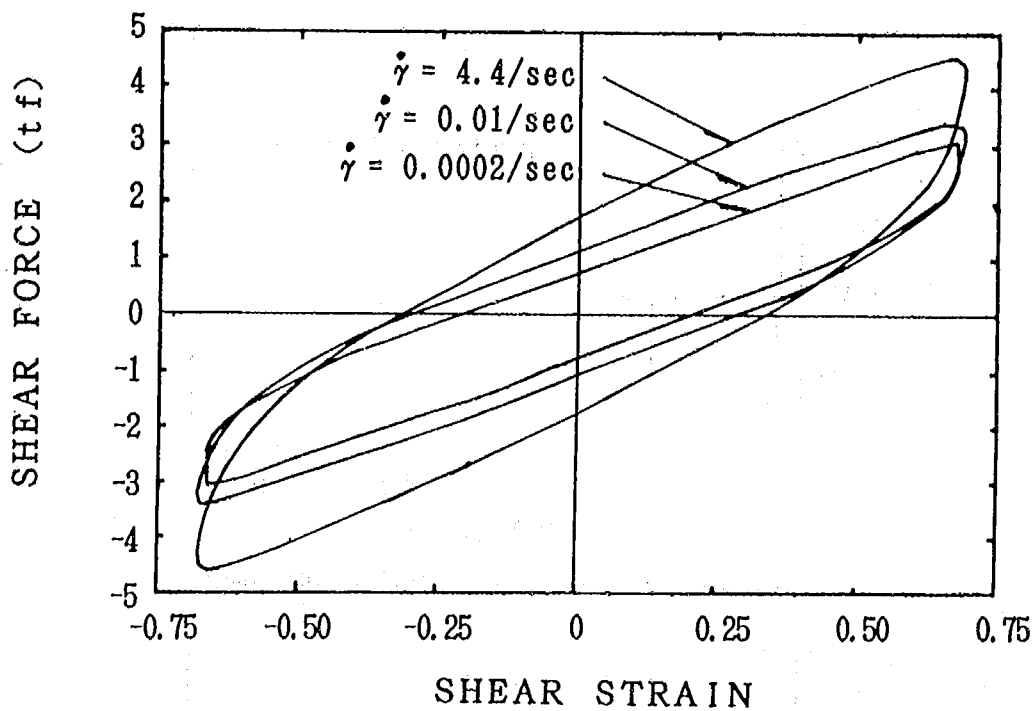


Fig. 12 Hysteresis Loops of LRB Subjected to a Harmonic Excitation with Displacement Amplitude of 50% Shear Strain of Rubber

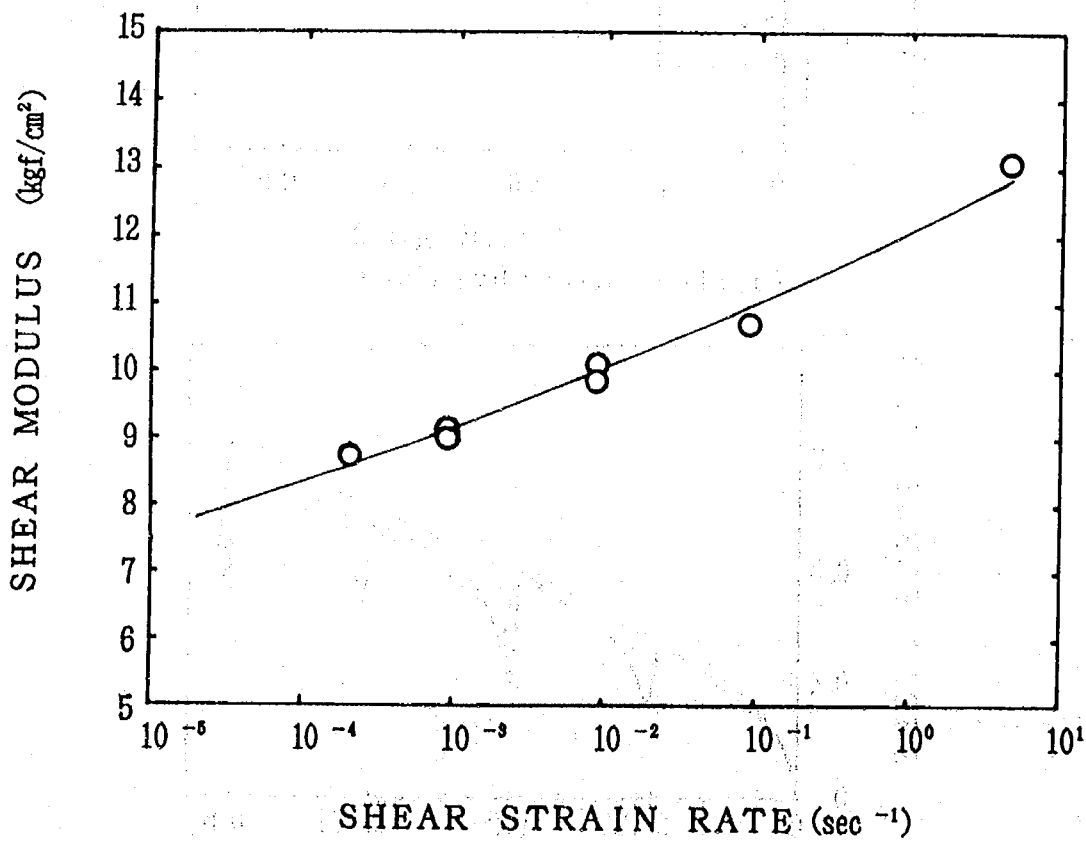
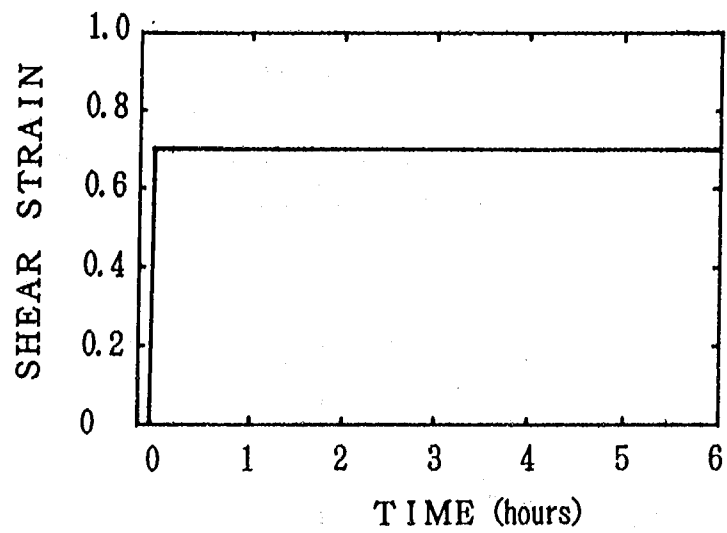
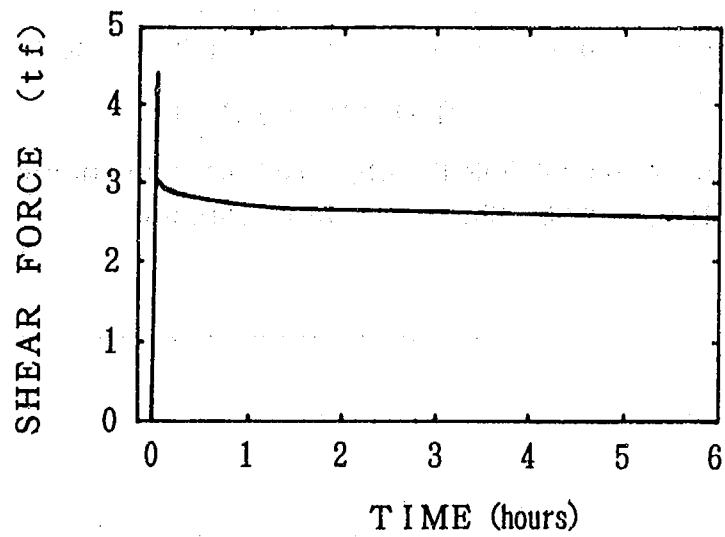


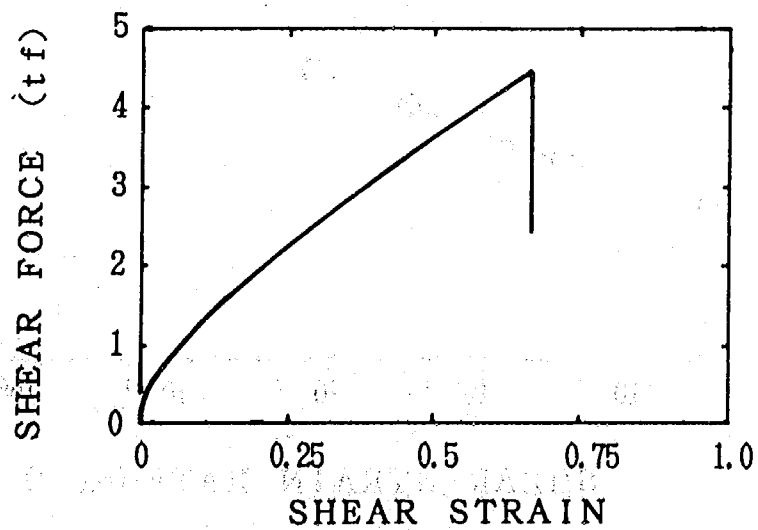
Fig. 13 Shear Strain Dependence of Shear Modulus of LRB Evaluated by Extrapolation Method



(a) Shear Strain Applied to Specimen



(b) Shear Stress Hysteresis



(c) Shear Strain Rate Dependence of Shear Strain

Fig. 14 Relaxation Method for LRB

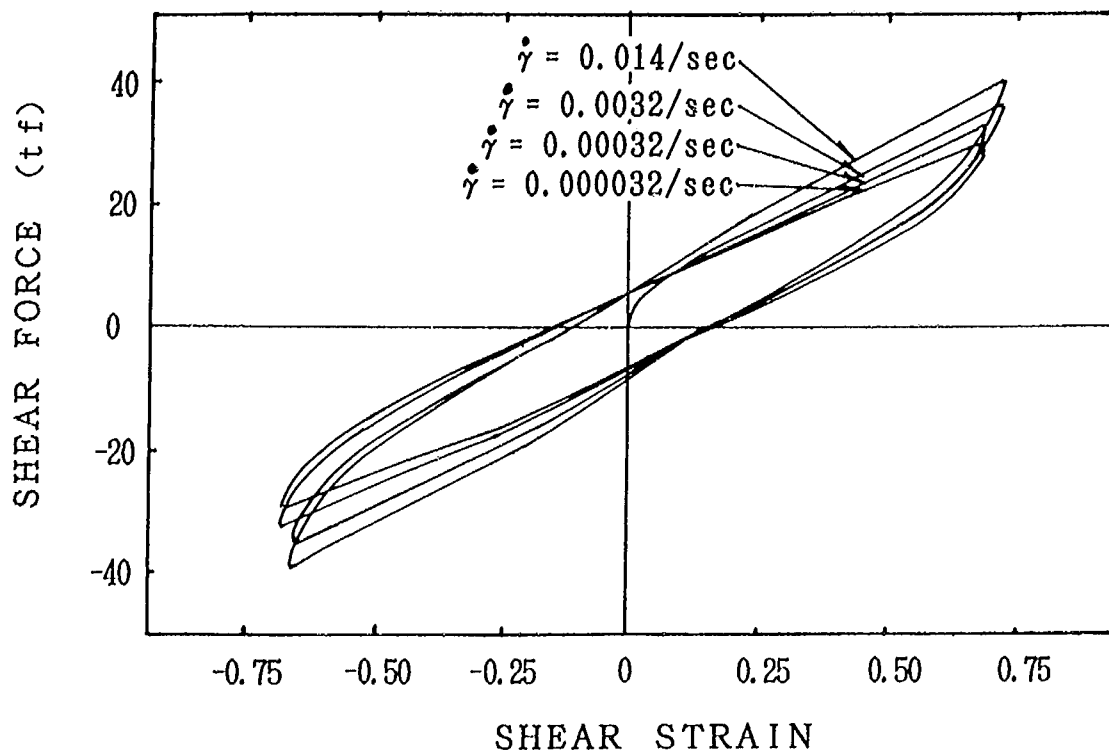


Fig. 15 Hysteresis Loops of HDR-B Subjected to a Harmonic Excitation with Displacement Amplitude of 50% Shear Strain of Rubber

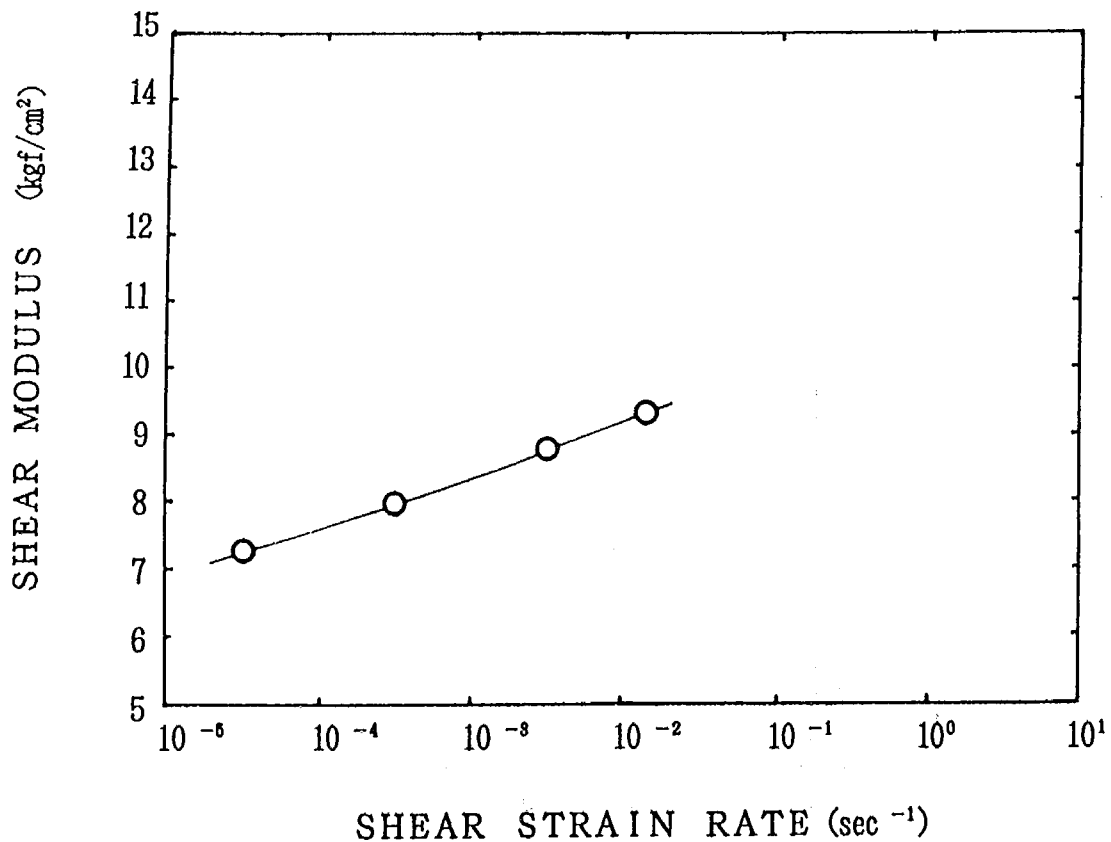


Fig. 16 Shear Strain Dependence of Shear Modulus of HDR-B Evaluated by Extrapolation Method

Inelastic Behavior and Seismic Design of
a Highway Shield Tunnel

Kazuhiko Kawashima

and

Yukitake Shioi

ASCE Structural Congress XI

Irvine, CA, U. S. A.

April 1993

Inelastic Behavior and Seismic Design of a Highway Shield Tunnel

Kazuhiko Kawashima¹ and Yukitake Shioi²

Abstract

Nonlinear behavior of shield tunnels required for seismic design by means of Seismic Deformation Method is presented based on dynamic loading tests. Application of the test results to the seismic design of Trans-Tokyo Bay Highway Tunnel is also described.

Introduction

Shield tunnels have been widely adopted in city areas where open-cut tunnels cannot be constructed due to various restrictions from land use and existing structures. Because the shield tunnels are constructed by bolting a number of segments, it shows a significant discontinuous behavior in their stiffness and failure modes. A series of dynamic loading tests were made at the Public Works Research Institute to clarify the stiffness and failure modes of the shield tunnels. The result of investigation was reflected to the seismic design of Trans-Tokyo Bay Highway.

This paper briefly summarizes the concept of seismic design of the shield tunnels by means of the seismic deformation method and dynamic response analysis, test results of the dynamic loading tests and the seismic design of Trans-Tokyo Bay Highway.

1) Head, Earthquake Engineering Division, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Ibaraki-ken, Japan.

2) General Manager, Design and Engineering Department, Trans-Tokyo Bay Highway Corporation, Ichi-bancho, Chiyoda-ku, Tokyo

Seismic Deformation Method

Seismic design of underground structures is made by the Seismic Deformation Method. The underground structure is idealized by an elastically supported beam subjected to a assumed ground deformation as shown in Fig. 1. The ground deformation assumed in design depends on the engineering assumption on the ground strain induced during an earthquake. Tunnels are in most cases critical when tension force is induced in their axis due to the ground deformation.

Dynamic response analysis as shown in Fig. 2 is generally made to study overall dynamic behavior of the underground structure. In the analysis by the Seismic Deformation Method and dynamic response analysis, it is essential importance to accurately evaluate the equivalent stiffness and the failure modes of shield tunnels.

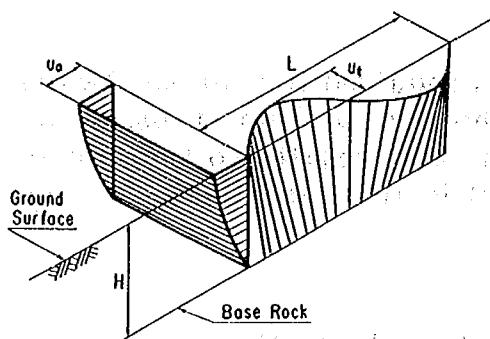


Fig. 1 Ground Deformation Assumed in Seismic Deformation Method

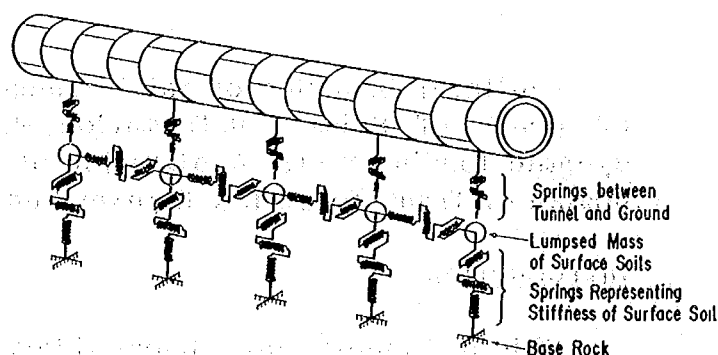


Fig. 2 Analytical Model for Dynamic Response Analysis

Inelastic Behavior of Shield Tunnels

A series of dynamic loading tests were conducted at the Public Works Research Institute to study the nonlinear behavior of shield tunnels subjected to alternative tension and compression in axial direction. Tests were also made for the tunnel subjected to pure bending moment and rotational moment. Because the behavior of the shield tunnel subjected to the axial tension and compression force is the fundamental importance, test results for this case is described hereinafter. Two types of specimens as shown in Fig. 3 were used. The first specimen was made by assembling 16 pieces of concrete segments with 900 mm long and 125 mm thick. The second specimen is the same with the first specimen, but reinforced concrete lining (second lining) with the thickness of 200 mm was placed inside the concrete segments (first segment).

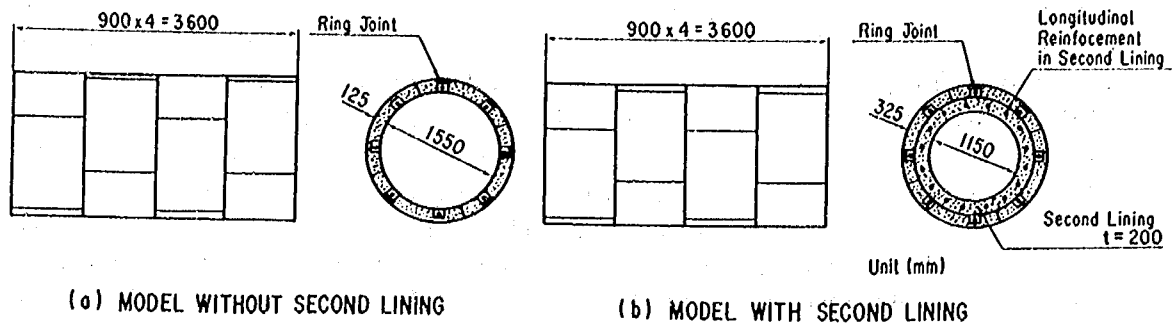


Fig. 3 Shield Tunnel Models for Dynamic Loading Tests

The specimens were loaded as shown in Fig. 4. One of the two ends of the specimens was fixed to a reaction wall, and the other end was subjected to alternative tension and compression by two dynamic actuators.

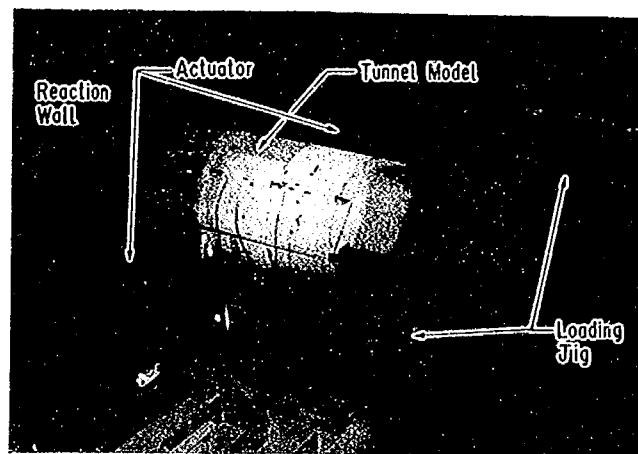


Fig. 4 Set-up of Dynamic Loading Test

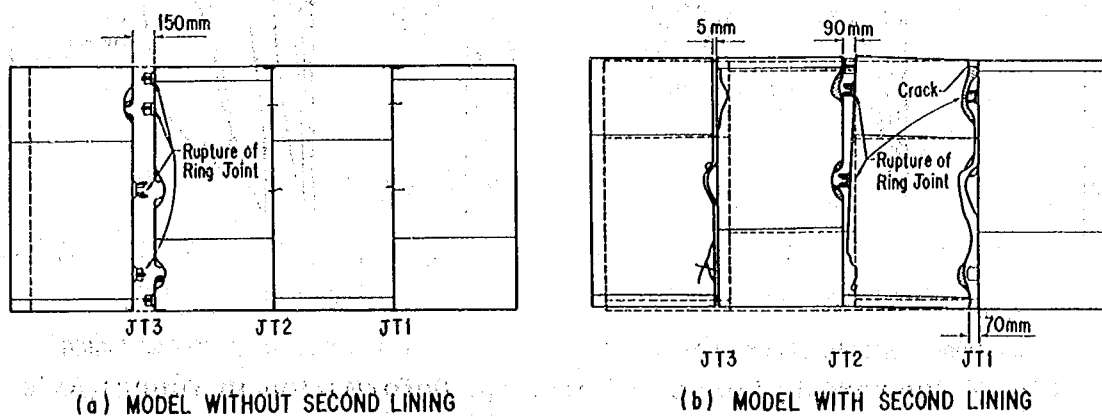


Fig. 5 Failure Modes

Fig. 5 compares the failure mode of the specimens. Failure occurred only at the ring joints with segments being free from damage. It should be noted that failure concentrated only at one of the three ring joints when the second lining was not provided, while the failure equally distributed at three ring joints when the second lining was provided. In the specimen without second lining, once cracks occurred at one ring joint, then the damage due to further loading tended to concentrate to this ring joint. Fig. 6 shows the failure mode of the ring joints. As the tension force increased the face plates deformed so that the anchor bars deformed only at the end of the side plate. This resulted in rupture of the anchor bars at this point, and developed pulling out of one side of the face plate from the other.

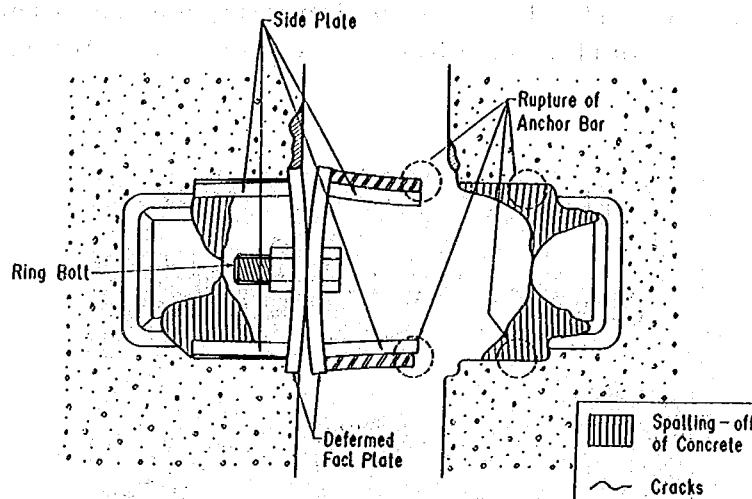


Fig. 6 Failure of Ring Joint due to Rupture of Anchor Bars

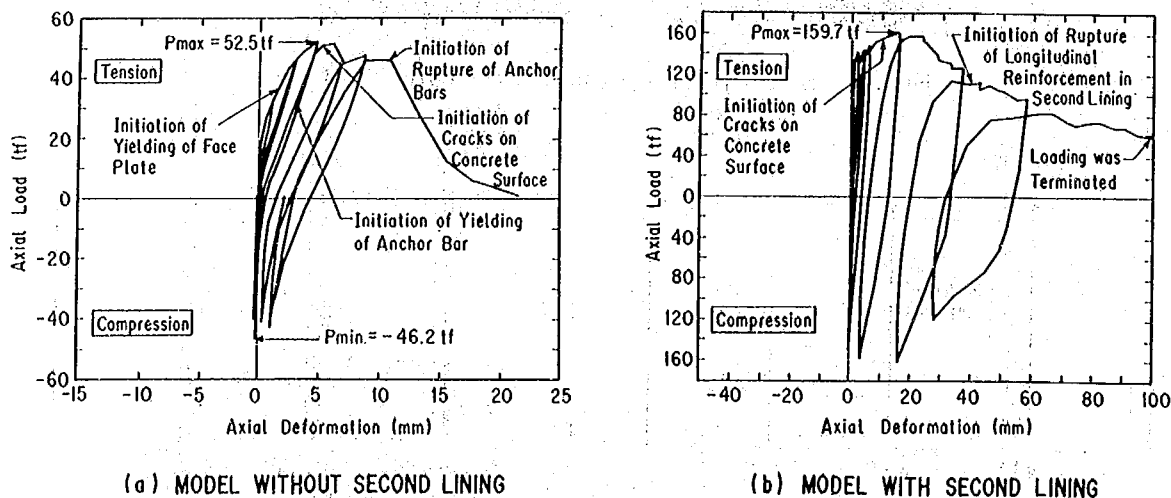


Fig. 7 Hysteresis Loops of Force vs. Deformation in Tunnel Axis

Fig. 7 compares the hysteresis loops for the two specimens. Although the stiffness of the specimen is very high for compression,

it is small in tension. The stiffness in tension decreases as the failure of the ring joints increases. In the specimen without second lining the restoring force was lost at displacement of 22 mm in tension, while in the specimen with second lining the restoring force of 60 tf was still preserved at displacement of 100 mm. The displacement of 22 mm and 100 mm corresponds to 0.6 % and 2.8 % of the total length of the specimen. They can be recognized as the ductility of the shield tunnels. The second lining is very effective to increase ductility as well as the increase of restoring force capacity. For making the reinforcement in second lining effective for increasing the ductility, the following reinforcing bars are required in the tunnel axis.

$$A_{smin} > A_o \sigma_o / \sigma_s \quad (1)$$

where A_{smin} : minimum reinforcement in second lining (cm^2), A_o : section of second lining (cm^2), σ_o : tension strength of concrete of second lining, and σ_s : strength of reinforcing bars (kgf/cm^2).

Seismic Design of Trans-Tokyo Bay Highway Tunnel

Advantage of the second lining was reflected to the seismic design of Trans-Tokyo Bay Highway. The Highway, under construction, is of shield tunnel with length of 10 km and bridges with length of 5

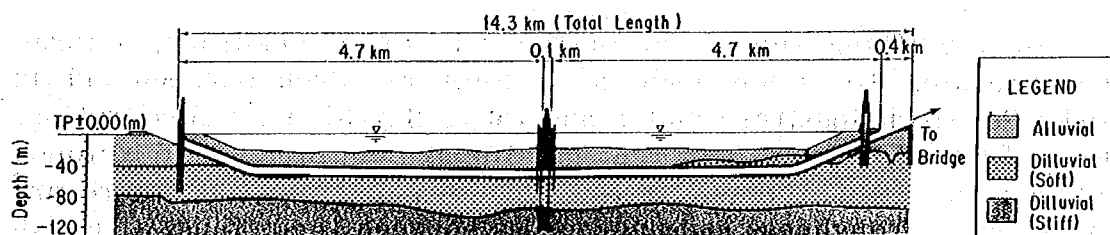


Fig. 8 Shield Tunnel of Trans-Tokyo Bay Highway

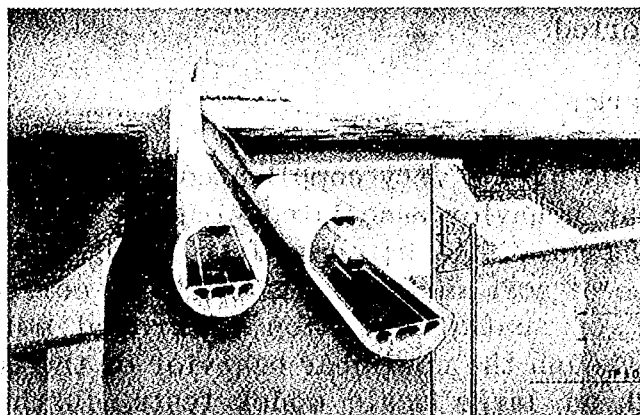


Fig. 9 Shield Tunnels and Ventilation Tower

km, and links the Kawasaki city and Kisarazu city around the Tokyo Bay as shown in Fig. 8. Fig. 9 shows the shield tunnels at the approach from Kawasaki City side near Haneda International Airport. Two shield tunnels with outer diameter of 13.9 m are being constructed in a very soft cohesive soils, 20 -30 m below the sea bottom. The depth from sea-water level to the bottom of tunnel is about 60m.

In the tunnels, the second lining was decided to be placed for increasing water tightness, ductility against seismic effect and dead weight. Because impermeability of the tunnel is very important, it was also decided to put waterproof membrane between the first and second lining. The waterproof membrane made the effectiveness of the second lining against seismic effect very complicated. The waterproof membrane might isolate the second lining from the first lining when the shield tunnels are subjected to tension force due to the ground deformation induced during an earthquake. It should be noted that the tension force is developed firstly in the first lining and then transmitted to the second lining because the first lining directly contacts with the soils. If so, the waterproof membrane might be effective to decrease the reinforcement in the second lining. Such isolation effect was at first expected to the waterproof membrane. But this decreases the effectiveness of the second lining for increasing ductility against seismic effect.

For clarifying the effect of the waterproof membrane, a large scale loading tests was made as a joint research between Public Works Research Institute and Trans-Tokyo Bay highway Corporation. It was found from the tests that no slippage would not be developed between the first lining and the second lining, because there are considerable roughness and unevenness between them. Thus, although the isolation effect by the waterproof membrane cannot be expected, the second lining was fully approved for its effectiveness for increasing ductility against seismic effect even if the waterproof membrane is inserted.

Concluding Remarks

Shield tunnels have a very complicated structural response due to the nonlinear behavior associated with the interaction between segment and ring joint and between first lining and second lining. Seismic response of soft surface soils has to be correctly evaluated taking account of spatial variation of ground motion along the tunnel axis. Although the nonlinear behavior of the tunnels is being clarified based on large scale model tests, further studies are required for tunnel and soil interaction when the tunnel is subjected to a large scale earthquake.

Estimation of Damping Ratio of Cable-Stayed Bridges
for Seismic Design

Kazuhiko KAWASHIMA

Shigeki UNJOH

and

Meguru TUNOMOTO

Journal of Structural Engineering, ASCE, Vol.119, No.4

April 1993

THE UNIVERSITY OF CHICAGO

LIBRARY

1917

1917

1917

THE UNIVERSITY OF CHICAGO

1917

ESTIMATION OF DAMPING RATIO OF CABLE-STAYED BRIDGES FOR SEISMIC DESIGN

By Kazuhiko Kawashima,¹ Shigeki Unjoh,² and Meguru Tunomoto³

ABSTRACT: This paper proposes a method to evaluate the structural damping ratio of cable-stayed bridges, based on energy dissipation. As demonstrated by many field forced-excitation tests, the damping ratio of cable-stayed bridges varies from bridge to bridge. This is due to the fact that the energy mechanisms predominant in the bridges studied are different. Therefore an analytical approach capable of evaluating the damping ratio from structural components is desirable. By dividing a cable-stayed bridge into several substructures in which the energy dissipation mechanism can be regarded as the same, it is proposed for each substructure to evaluate the energy dissipation function, which relates the amount of energy dissipation, with either the strain energy or the displacement at specific points in the substructures. By evaluating the overall energy dissipation and strain energy in the whole bridge, the damping ratio for the desired mode shape can be obtained. The proposed method is applied to a cable-stayed bridge model. Dependence of the damping ratio on oscillation amplitude and cable type of the bridge model is predicted with reasonable accuracy by the proposed method.

INTRODUCTION

According to a number of full-scale tests conducted for cable-stayed bridges, it is well known that natural frequencies and natural mode shapes can be predicted with acceptable accuracy by means of linear analyses that assume appropriate mass and stiffness distribution (Kawashima et al. 1986; Wilson and Liu 1991). However, few investigations have been made for damping ratio. It is pointed out from the field forced-excitation tests that quite smaller damping values than those usually adopted in seismic design were obtained from the tests (Kawashima et al. 1986; Stierner et al. 1988; Ohlsson 1986). Dependence of damping ratio on natural mode shapes is also pointed out, based on analysis of strong motion records of a cable-stayed bridge (Kawashima et al. 1990). Because response of cable-stayed bridges significantly depends on damping ratio, it is of great importance to correctly evaluate the damping ratio for seismic design.

For developing an empirical method for estimating damping ratio of cable-stayed bridges, various attempts for correlating measured damping ratio with structural characteristics such as natural frequency, span length, and pier height have been made. However, because factors contributing to damping ratio vary from bridge to bridge, they show considerable scatter in most cases. Analytical approaches capable of providing more accurate evaluation of the damping ratio of cable-stayed bridges are therefore desirable for improving seismic design.

¹Head, Earthquake Engrg. Div., Public Works Res. Inst., Ministry of Constr., Tsukuba Science City, Japan, 305.

²Res. Engrg., Earthquake Engrg. Div., Public Works Res. Inst., Ministry of Constr., Tsukuba Science City, Japan, 305.

³Visiting Res. Engrg. at Earthquake Engrg. Div., Public Works Res. Inst., Ministry of Constr., Tsukuba Science City, Japan, 305.

Note. Discussion open until September 1, 1993. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on June 4, 1991. This paper is part of the *Journal of Structural Engineering*, Vol. 119, No. 4, April, 1993. ©ASCE, ISSN 0733-9445/93/0004-1015/\$1.00 + \$.15 per page. Paper No. 1997.

The present paper describes a method for estimating the damping ratio of cable-stayed bridges based on energy dissipation, and investigates its validity through a comparison with free-oscillation tests of a scaled model.

FREE-OSCILLATION TEST OF CABLE-STAYED BRIDGE MODEL

Experimental Model

To study damping characteristics of cable-stayed bridges and to provide realistic response data that can be used to verify an analytical procedure for evaluating damping ratio, a series of model oscillation tests were made. Fig. 1 shows an experimental model that was accurately fabricated for simulating the Meiko-nishi Bridge (Kato et al. 1983) as a prototype. The rigidity and mass of the model were determined assuming scales of length, density, and time of $1/150$, $1/1$, and $1/\sqrt{150}$, respectively. Although the damping ratio is nondimensional, the similarity law for the damping ratio is quite difficult to satisfy because there are various mechanisms for causing energy dissipation. Therefore, instead of paying special attention to similarity of the damping ratio, consideration was given for selection of the materials of the model. It should not be therefore considered that the damping ratio of the model fully represents that of the prototype. In the present study, only qualitative characteristics of the damping ratio are useful for the analysis.

Table 1 shows comparisons of the length, rigidity, and mass between the prototype and the model. EI_y and EI_z represent flexural rigidity around transverse axis y and vertical axis z , respectively. Considering total balance of the model, the ratio of the center span and total deck length was reduced by 6% from prototype; and, in order to reduce the effect of viscous damping with air, the weight and rigidity of the deck were made approximately 8 times and 3 times greater, respectively, than the values determined by the similitude ratio. Although the number of cables is 22 on each side in the prototype, these were simplified to 3 in the model.

For modeling the deck, lateral beams of 12.7×3.0 mm were attached at 50 mm intervals. Lead weights were placed, as shown in Fig. 2, to match the mass ratio. The tower was made of two 5.5×5.0 mm steel members, which were rigidly fixed to a rigid steel base.

Piano wires of 0.5 mm diameter were used as cables. Special mounting devices as shown in Fig. 3 were used to anchor the cables to the tower so that cable tension could be adjusted. The cable tension due to dead-weight was adjusted to match with the computed values within $\pm 10\%$.

The deck was supported by only cables so that friction between the deck

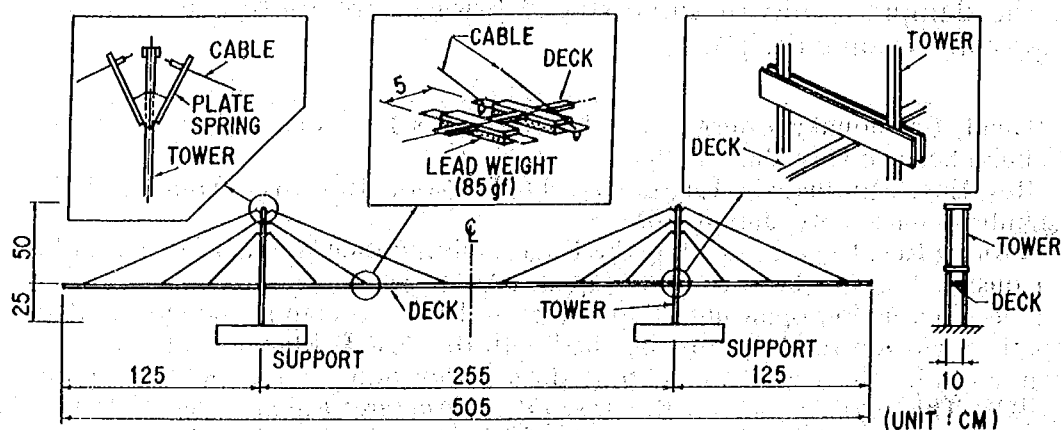


FIG. 1. Experimental Bridge Model

TABLE 1. Scale of Bridge Model

[illegible]

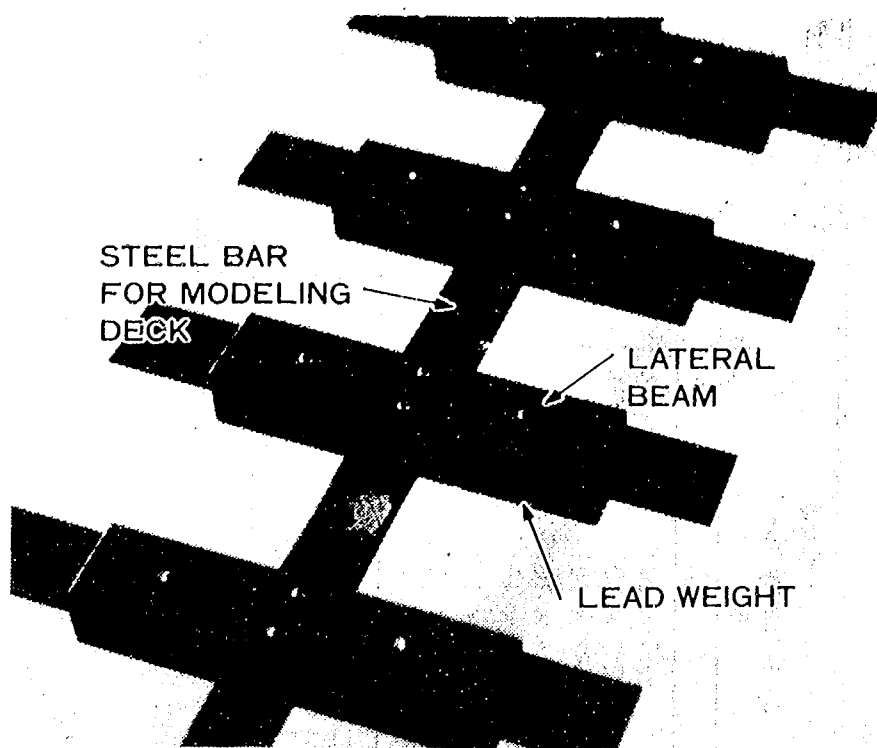


FIG. 2. Model of Deck

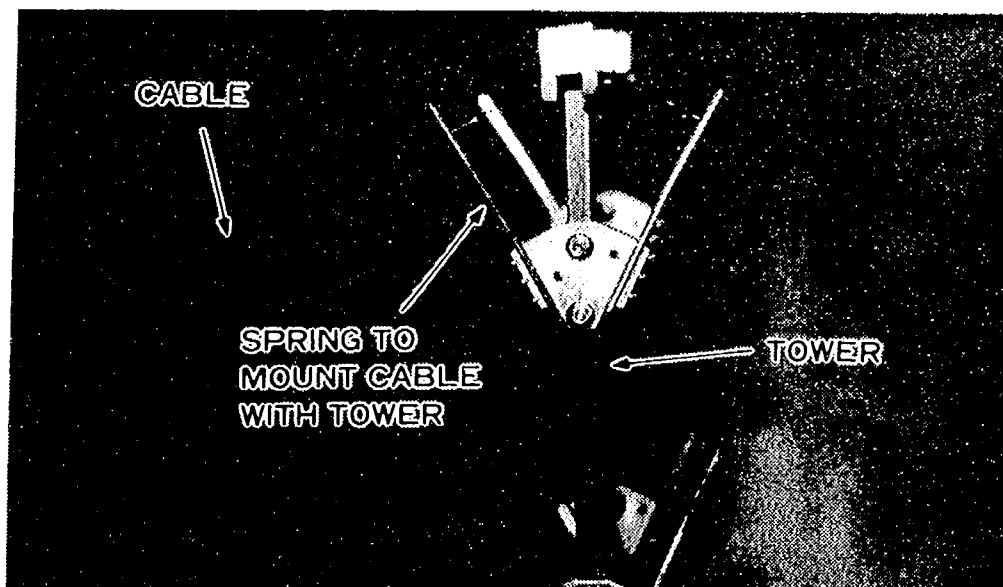


FIG. 3. Device to Mount Cables with Tower

and the tower due to relative movement of the deck in longitudinal direction can be disregarded. Cable arrangements were studied, as shown in Fig. 4, varying from fan type (type 3A) to harp type (type 3E).

Fig. 5 shows the fundamental natural frequencies and natural mode shapes of the model predicted with a standard linear modal analysis. Geometrical stiffness associated with the axial force induced in the tower and the deck was included.

The fundamental mode is an antisymmetric mode in the longitudinal direction and symmetric mode in the vertical direction. In the longitudinal

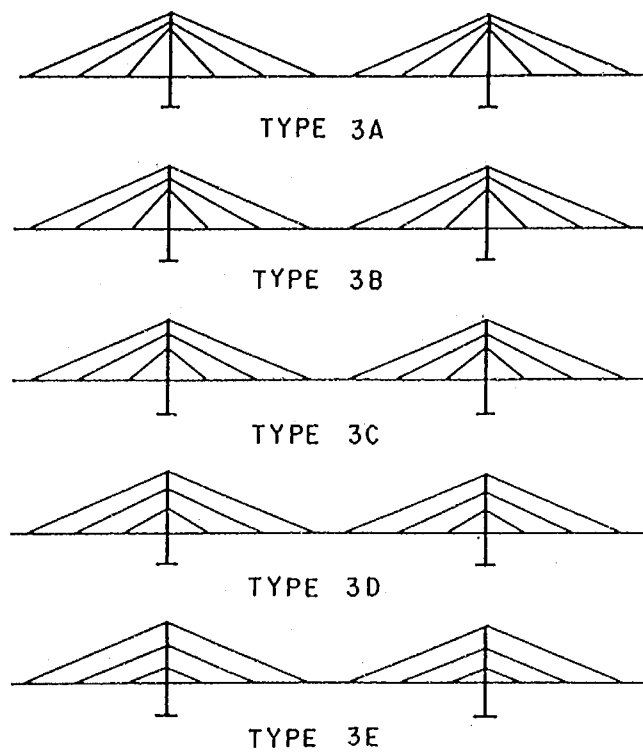


FIG. 4. Cable Arrangement of Bridge Model

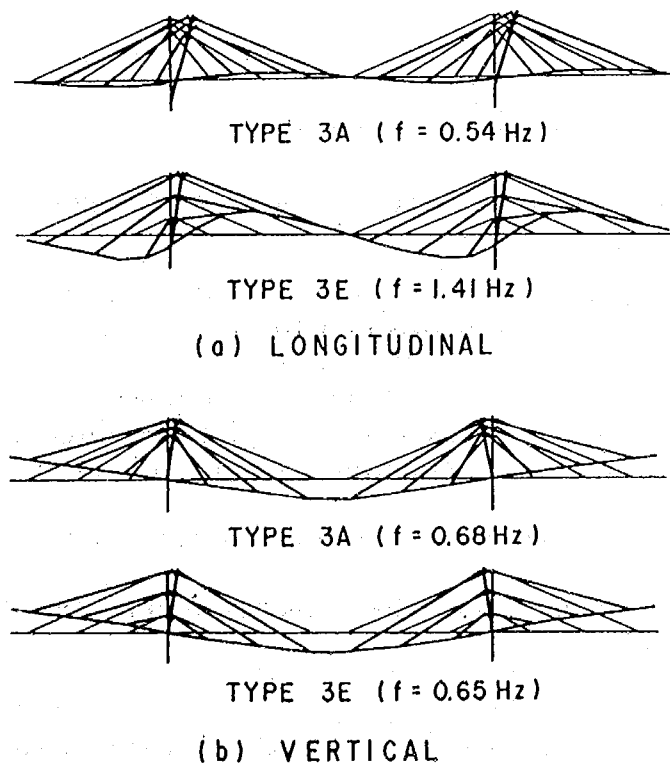


FIG. 5. Predicted Mode Shapes for Bridge Model

oscillation, the natural frequencies increase as the cable type changes from the fan type to the harp type. It should be noted that vertical deformation of the deck associated with a unit displacement of the deck in this direction increases as the cable type changes from the fan type to the harp type. The

shortest cable acts in an important role for producing the deck deformation in the vertical direction. As will be described later, this is closely related with energy dissipation of the deck. On the other hand, for flexural oscillations in the vertical direction, few changes of the natural frequencies and the mode shapes are observed for the various cable types.

Free-Oscillation Test

The deck was statically displaced so as to create an initial deformation close to the target mode shape presented in Fig. 5. It was then smoothly released to result in a free oscillation. In the longitudinal direction, one of the ends of the deck was laterally displaced so as to produce an initial deformation close to the first mode shape. For developing the first flexural oscillation mode in the vertical direction, the center of the deck was displaced.

The free oscillation of the model was measured at the center of the deck for the vertical oscillation and at one of the ends of the deck for the longitudinal oscillation. The displacement at these points was regarded as a reference value for measuring the amplitude of free oscillation. For measuring the displacement, a noncontact displacement sensor was used to prevent the damping ratio of the model from being affected by cables of the sensor. The damping ratio was computed from the averaged decay over $(2n + 1)$ cycles of free oscillation as

$$\delta_m = \frac{2\pi h_m}{\sqrt{1 - h_m^2}} = \frac{1}{n} \ln \frac{a_{m-n}}{a_{m+n}} \dots \dots \dots (1)$$

in which δ_m and h_m = logarithmic damping ratio and damping ratio of critical, respectively; and a_{m-n} and a_{m+n} represent the amplitude at $(m - n)$ th and $(m + n)$ th oscillation, respectively. n is assumed as 7.

EFFECT OF MODE SHAPE ON DAMPING RATIO FROM FREE OSCILLATION TEST

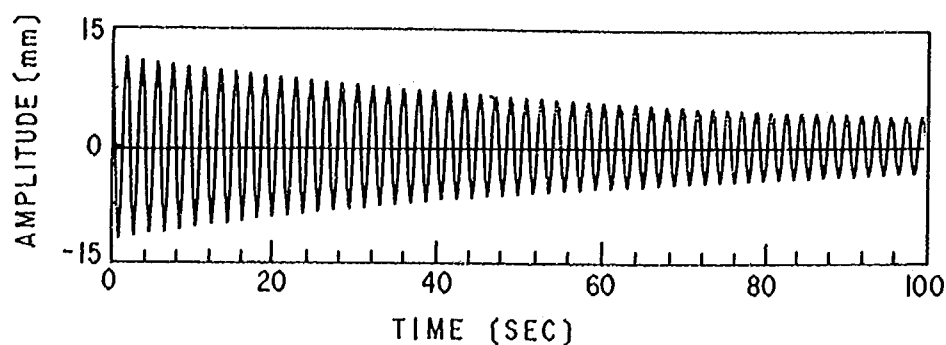
Table 2 shows the natural frequencies of the model measured from the free-oscillation tests, in comparison with the predicted values. The natural frequencies predicted agree well with the experimental values.

As an example of the free oscillation, Fig. 6 shows the displacement decay of the type 3A and type 3E excited in longitudinal direction. It is apparent that the decay of the free oscillation is significantly different depending on the cable type.

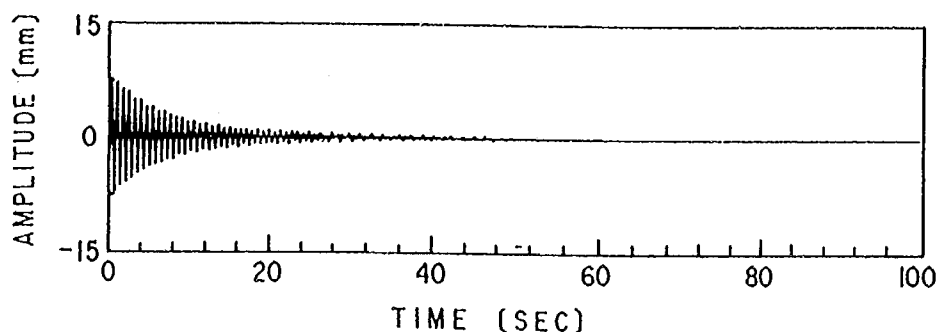
Because the damping ratio thus estimated from the free oscillation varies

TABLE 2. Comparison of Natural Frequencies (Hz) between Predicted and Experimental Results

Cable type (1)	Longitudinal Mode		Vertical Mode	
	Experimental (2)	Predicted (3)	Experimental (4)	Predicted (5)
3A	0.54	0.54	0.66	0.68
3B	0.61	0.61	0.65	0.67
3C	0.76	0.76	0.65	0.66
3D	0.96	0.97	0.64	0.65
3E	1.38	1.41	0.65	0.65



(a) TYPE 3A



(b) TYPE 3E

FIG. 6. Decay of Free Oscillation Amplitude

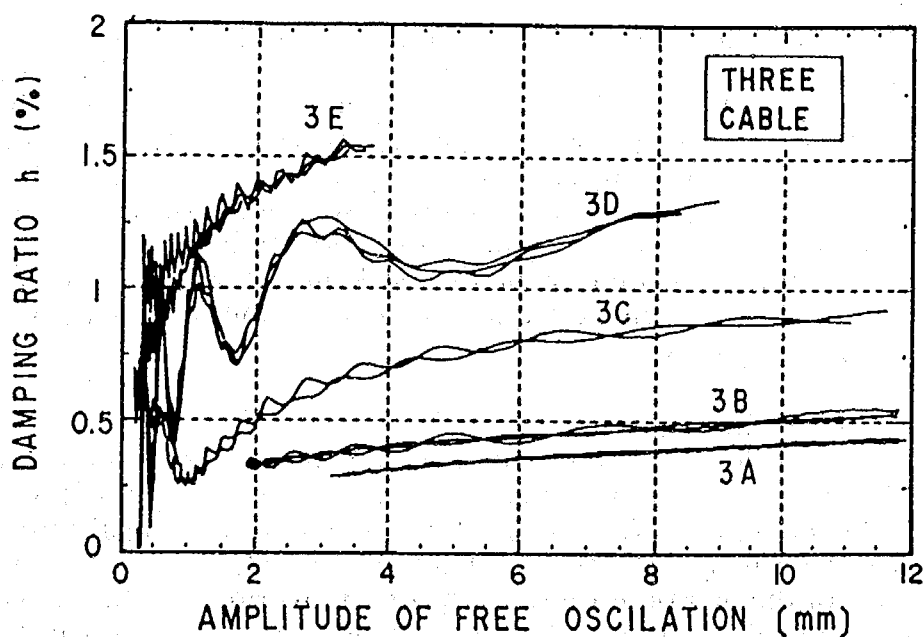


FIG. 7. Damping Ratio versus Oscillation Amplitude for Longitudinal Oscillation

in accordance with the amplitude of oscillation, the damping ratio was evaluated as a function of the oscillation amplitude as shown in Figs. 7 and 8. Two or three tests were made for each cable type, and this accounts for the multiple lines for each cable type in Figs. 7 and 8.

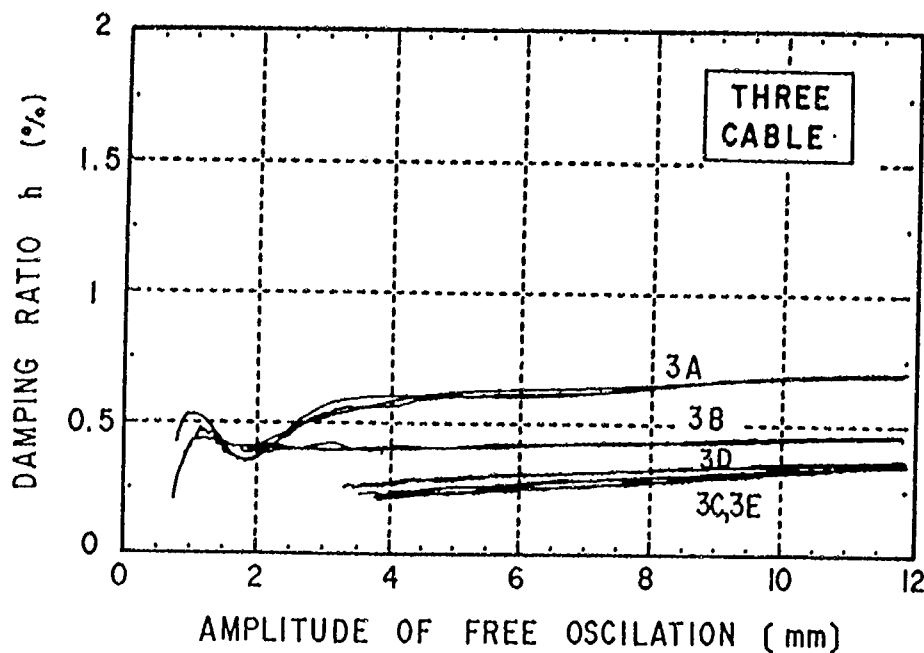


FIG. 8. Damping Ratio versus Oscillation Amplitude for Vertical Flexural Oscillation

Oscillation in Longitudinal Direction

The dependence of the damping ratio on the cable type is considerable, as shown in Fig. 7. Some scatter of the damping ratio against the amplitude was developed in type 3D and type 3C in the amplitude range from 0.5 mm to 4 mm, probably associated with slightly distorted initial deformation of the model.

The damping ratio at the same amplitude increases as the cable type changes from the fan type (type 3A) to the harp type (type 3E). For example, the damping ratio of type 3A is 0.4% at an amplitude of 8 mm, while the damping ratio of type 3D is approximately 3 times larger (1.2%) at the same amplitude. Such a significant cable-type dependence of the damping ratio may be clearly related to the flexural deformation of the deck in the vertical direction per unit deck displacement in longitudinal direction (refer to Fig. 5). Larger flexural deformation of the deck in the vertical direction, as associated with the harp type, dissipates a greater amount of energy resulting in a larger damping ratio.

Oscillation in Vertical Direction

In the vertical direction, the damping ratio decreases as the cable type changes from the fan type (type 3A) to the harp type (type 3E) as shown in Fig. 8. Damping ratio for the flexural oscillation in vertical direction is generally smaller than that for the longitudinal oscillation. Such cable-type dependence of the damping ratio is opposite in its order to that developed in the longitudinal oscillation, but the reason for this reversal in trend is not clear.

METHOD FOR EVALUATION OF DAMPING RATIO

There are various factors causing energy dissipation in cable-stayed bridges. Energy dissipation is generally developed by material nonlinearity, structural damping such as friction at movable bearings (Kawashima and Unjoh 1989), radiation of energy from foundations to ground (Kawashima et al.

1988), and friction with air. Predominant factors contributing to damping ratio of cable-stayed bridges varies with structural types, however an important point in estimating the damping ratio of cable-stayed bridges is to evaluate total energy dissipation. Because the energy dissipation capability may not be the same in all parts of the bridge, it is necessary to evaluate the energy dissipation for each structural segment. Once the energy dissipation in individual structural segments is determined, the total energy dissipation in the whole structural system can be evaluated by summing up the energy dissipation developed in each structural segment.

Based on such considerations, it is proposed to evaluate energy dissipation of a cable-stayed bridge as follows:

1. Divide the cable-stayed bridge into several structural segments in which energy dissipation capability can be considered practically the same. Such a structural segment is referred hereinafter as a substructure. For instance, a cable-stayed bridge may be divided into deck(s), tower(s), and cables. Material nonlinearity is considered to be the prevailing mechanism for energy dissipation in such substructures. Bearing supports may also be considered as a substructure because energy dissipation due to friction occurs at the bearings.

2. When the i th substructure is idealized by a n th degree of freedom discrete system, the strain energy E_j^i of the i th substructure for j th mode may be represented as

$$E_j^i = \frac{1}{2} (\mathbf{u}_j^i)^T \mathbf{k}^i \mathbf{u}_j^i \quad \dots \dots \dots (2)$$

in which $\mathbf{u}_j^i$ = amplitude vector of i th substructure for j th mode; and $\mathbf{k}^i$ = stiffness matrix of the i th substructure.

Strain energy E_j of the whole structural system for j th mode can then be evaluated as

$$E_j = \sum_i E_j^i \quad \dots \dots \dots (3)$$

3. For the i th substructure where energy dissipation due to material nonlinearity is predominant, the energy dissipation associated with the j th mode may be written as

$$\delta E_j^i = f_j^i(E_j^i) \quad \dots \dots \dots (4)$$

in which δE_j^i = energy dissipation in i th substructure for j th mode; and f_j^i = function relating δE_j^i to E_j^i , referred to hereafter as energy dissipation function. In most cases, it is quite difficult to directly evaluate the energy dissipation function f_j^i from analytical studies, and usually it has to be determined based on appropriate experiments.

4. For the i th substructure where the energy dissipation function f_j^i is such a form that it can be represented in terms of a displacement at a specific point u_j^{ik} within the i th substructure, the energy dissipation function may be represented as

$$\delta E_j^i = f_j^i(u_j^{ik}) \quad \dots \dots \dots (5)$$

in which u_j^{ik} = displacement at point k in i th substructure for j th mode.

5. Energy dissipation δE_j in the whole structural system for the j th mode can be obtained by summing up the energy dissipation developed in individual substructures determined by (4) and/or (5) as

$$\delta E_j = \sum_i \delta E_j^i \dots\dots\dots (6)$$

6. The damping ratio of the whole structure for the j th mode may then be obtained from (4) and (6) as (Clough and Penzien 1975)

$$h_j = \frac{\delta E_j}{4\pi E_j} \dots\dots\dots (7)$$

EVALUATION OF ENERGY DISSIPATION FUNCTIONS

To demonstrate the application of the proposed method, the cable-stayed bridge model was analyzed. This is because full-scale test data of damping ratio for the substructures are not presently available, although damping ratios of the complete bridge structures have been measured. Because the model bridge was supported by only cables, friction between the deck and the tower can be neglected. Effect of radiation of energy from the foundation to subsoils can also be neglected, because the bottom of the tower was rigidly fixed by a steel base plate. Therefore, it was assumed that the energy dissipation of the model bridge occurs due to material nonlinearity of the deck and the tower and friction at the anchoring portion of the cables to the deck and the tower. Therefore, the tower, the deck, and the cable-anchoring portion are considered here as substructures.

Energy Dissipation Function of Tower

For estimating the energy dissipation function of the tower, a simple free-oscillation test was made for the tower. Only the tower was taken out from the whole model, and it was supported as a cantilevered beam with one of the ends being fixed and the other being free. To avoid energy dissipation from the bottom of the cantilevered tower, special attention was paid to fix it rigidly to a rigid steel member. A weight was placed on the free end of the tower to study the effect of axial force induced in the tower. The weight placed on the free end of the tower was varied from 8.34 N (0.85 kgf) to 50.0 N (5.1 kgf). Because the axial force induced at the bottom of the tower due to dead weight of the deck is 67.4 N (6.87 kgf) in the whole model, the weight of the mass corresponds to 17% to 74% of the axial force of the tower.

By smoothly releasing the tower top from a displaced position, a free oscillation simulating the first mode was developed. Amplitude of the tower was measured at 33 cm from the bottom by means of a noncontact displacement sensor. The damping ratio of the tower was estimated from decay of the free oscillation. Fig. 9 shows that the damping ratio of the tower increases as both the oscillation amplitude and the added mass increase. The predicted damping ratio, which will be described later, is also presented for comparison.

The total energy E_j of the structural system oscillating in j th mode shape comes from two contributions as

$$E_j = E_{sj} + E_{ej} \dots\dots\dots (8)$$

in which E_{sj} = strain energy associated with the deformation of the structure; and E_{ej} = strain energy associated with the geometrical stiffness due to the compression force induced in the tower and the deck. Because the energy dissipation during oscillation is developed only by the strain energy E_{sj} , the energy dissipating function f_j may be represented as

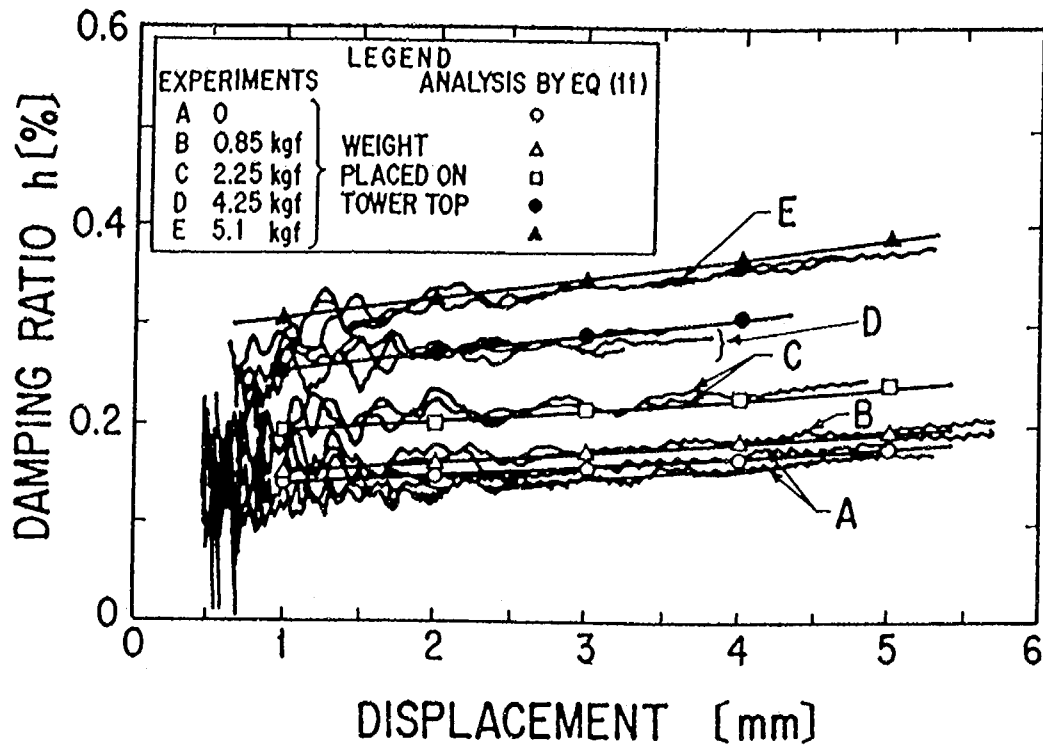


FIG. 9. Damping Ratio versus Oscillation Amplitude for Tower

$$\delta E_j = f_j(E_{sj}) \quad \dots\dots\dots (9)$$

in which δE_j = energy dissipation during the oscillation. Therefore, the damping ratio of the tower may be obtained by substituting (8) and (9) into (7) as

$$h_j = \frac{\delta E_j}{4\pi E_j} = \frac{f_j(E_{sj})}{4\pi(E_{sj} + E_{ej})} \quad \dots\dots\dots (10)$$

Because E_{ej} is negative when the axial force of the tower is positive (compressive), the damping ratio presented by (10) increases as the axial force becomes larger.

Because the damping ratio h_j of the tower can be determined through the decay of the free oscillation and the total energy E_j can be computed from (8), one can determine the energy dissipation δE_j by substituting h_j and E_j into (10).

Fig. 10 shows the relation between δE_1 and E_{s1} for the first mode. The relation seems almost independent of the weight of the mass placed on the tower top. It is approximated by a least-square fit as

$$\delta E_{s1} = 0.016E_{s1} + 0.0021E_{s1}^{3/2} \quad \dots\dots\dots (11)$$

Based on (11), the damping ratio versus the oscillation amplitude of the tower is evaluated as shown in Fig. 9. It is seen that the predicted damping ratio agrees reasonably well with the experimental result.

Energy Dissipation Function of Deck

A segment of the deck was taken out from the whole model and was supported as a cantilever beam with a weight placed at the free end. The length of the cantilevered deck was varied from 40 cm to 60 cm so that the effect of the deck length on the damping ratio could be studied.

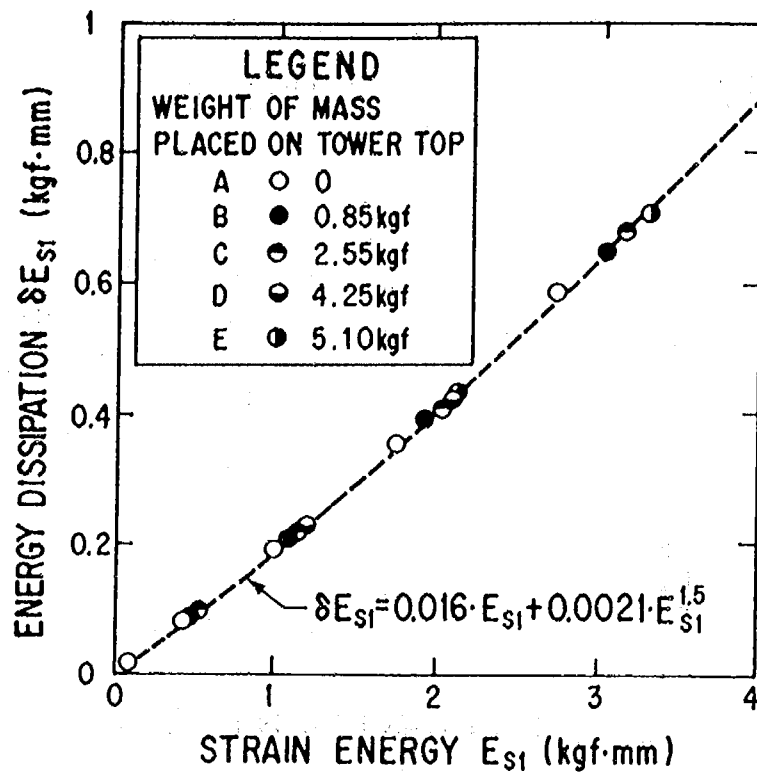


FIG. 10. Energy Dissipation δE_{s1} versus Strain Energy E_{s1} for Tower (1 kgf = 9.81 N)

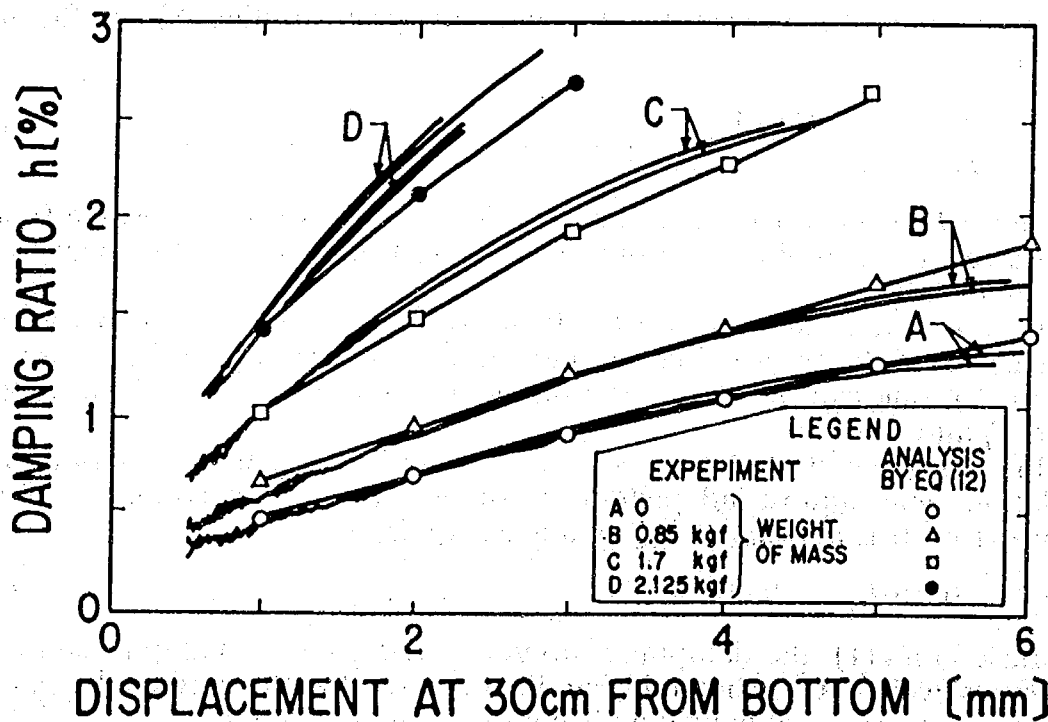


FIG. 11. Damping Ratio versus Oscillation Amplitude for Deck

Fig. 11 shows how the damping ratio estimated from the decay of free oscillations depends on the oscillation amplitude measured at 30 cm from the bottom. Only the result for the deck length of 60 cm is presented because the other case shows similar results. The predicted damping ratio, which will be described later, is also presented for comparison.

As is the case for the tower, the damping ratio of the cantilevered deck increases as the oscillation amplitude and the weight of the mass increase. It is interesting to note that the damping ratio of the deck is appreciably larger than the damping ratio of the tower. The tower consists of only a steel frame, while the deck is fabricated by assembling the main beam, lateral beam, and lead weight. The larger damping ratio of the deck may be attributed to energy dissipation associated with the lateral beam and lead weight attached to the main beam. Therefore the larger damping ratio of the deck is only for the model.

Fig. 12 shows the relation between δE_{s1} and E_{s1} for the cantilevered deck. Because the effect of the deck length on δE_{s1} versus E_{s1} relation is small, its effect can be neglected, and the relation between δE_{s1} and E_{s1} may be approximated as

$$\delta E_{s1} = 0.016E_{s1} + 0.083E_{s1}^{1.37} \quad \dots\dots\dots (12)$$

The damping ratio predicted by (12) agrees fairly well with the experimental result as shown in Fig. 11.

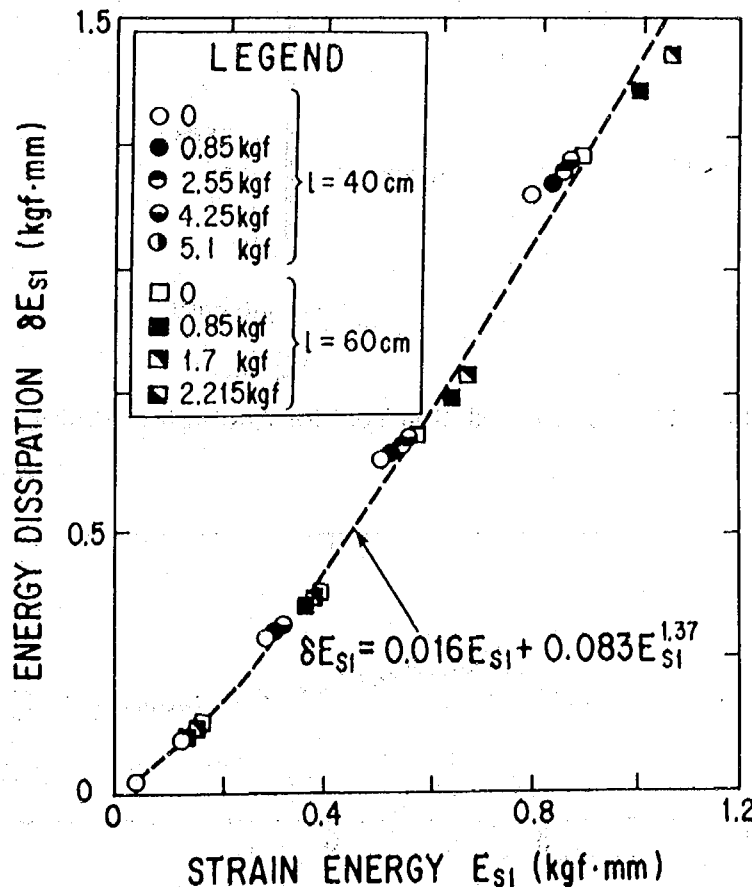


FIG. 12. Energy Dissipation δE_{s1} versus Strain Energy E_{s1} for Deck (1 kgf = 9.81 N)

Energy Dissipation Function of Anchoring Portion of Cables

To measure energy dissipation at the cable-anchoring portion, this substructure was isolated from the whole model and a free oscillation test of this substructure was made as shown in Fig. 13. To isolate the energy dissipation at the cable-anchoring portion, both ends of the tower were fixed so that the energy dissipation associated with the tower deformation was as small as possible. Furthermore, the deck was supported by two cables so that flexural deformation of the deck in the vertical direction was small during the free oscillation. To study the effect of the tension force developed in the cables, two masses were placed on the deck.

Fig. 14 shows a relation between the damping ratio and the oscillation amplitude of the deck in the longitudinal direction. Being different from the tower and deck, the amplitude dependence of the damping ratio varies in a complicated manner depending on the oscillation amplitude. This seems to be developed probably because the friction between the cables and the anchoring devices delicately changes depending on the oscillation amplitude.

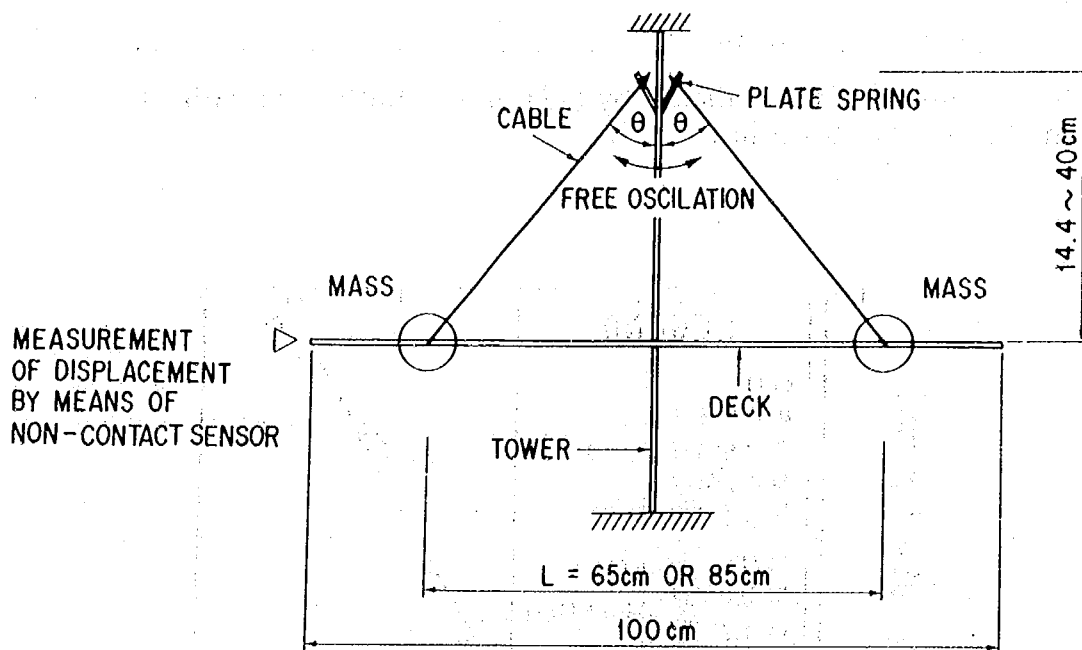


FIG. 13. Free Oscillation Test for Anchoring Portion

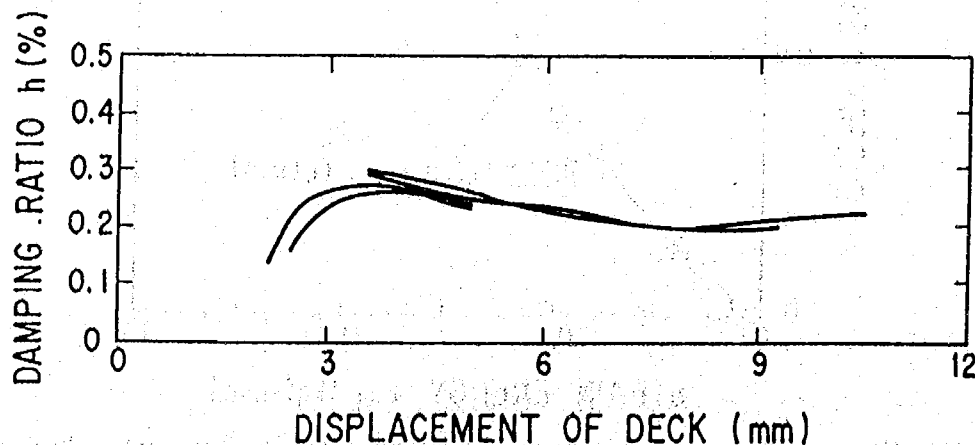


FIG. 14. Damping Ratio versus Oscillation Amplitude for Anchoring Portion

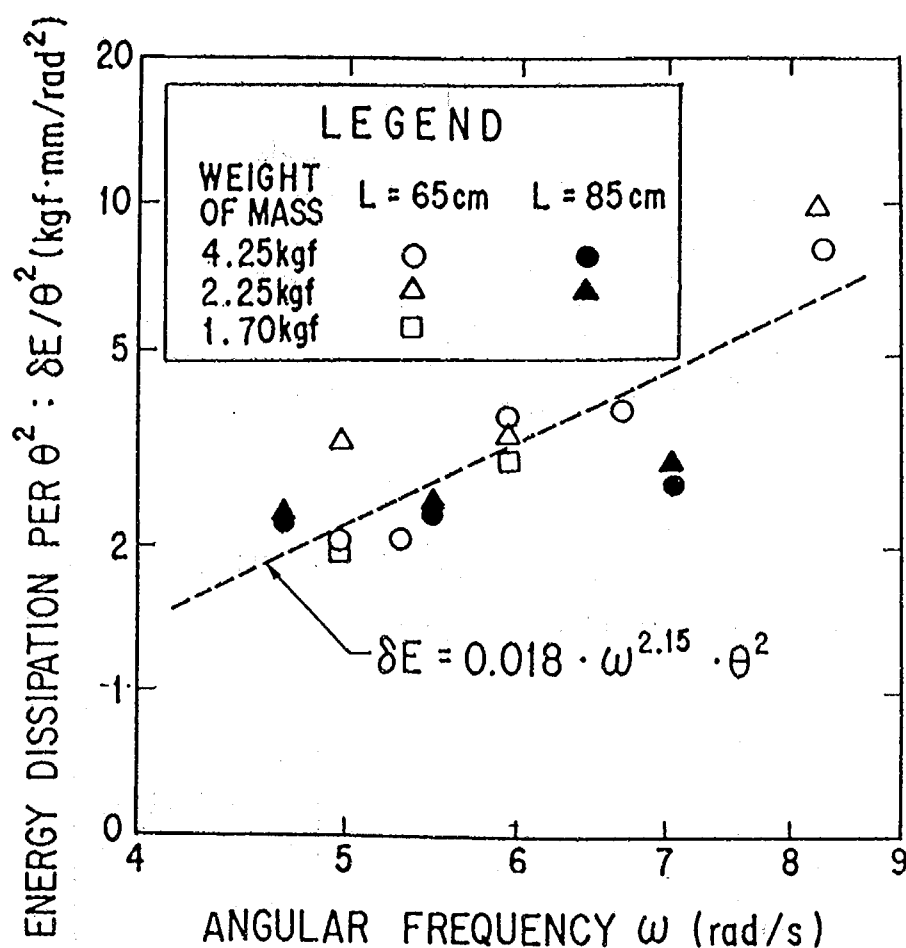


FIG. 15. Energy Dissipation, Angular Frequency, and Oscillation Amplitude for Anchoring Portion (1 kgf = 9.81 N)

Fig. 15 shows a relation between the energy dissipation, angular frequency of the free oscillation, and the oscillation amplitude at the cable-anchoring portion, in which θ and ω represent an angle between the tower and the cable (refer to Fig. 13) and angular frequency of the free oscillation, respectively. The effect of the tension force induced in the cables seems less significant.

By approximating the relation, the energy dissipation at the cable anchoring portion may be written as

$$\delta E = 0.018 \omega^{2.15} \theta^2 \quad \dots \dots \dots (13)$$

EVALUATION OF DAMPING RATIO OF CABLE-STAYED BRIDGE MODEL

The damping ratio predicted by (7) assuming the energy dissipation functions of the substructures as given by (11), (12), and (13) is presented in Figs. 16 and 17 in comparison with the experimental results.

The predicted damping ratio of the model in the longitudinal oscillation increases, in the same manner as in the test, as the cable type changes from the fan type (type 3A) to the harp type (type 3E). The dependence of the predicted damping ratio on the oscillation amplitude increases in the same order with the experiments. Such general trends are the same as the experiments, although the predicted damping ratios are underestimated as the cable type approaches the harp type.

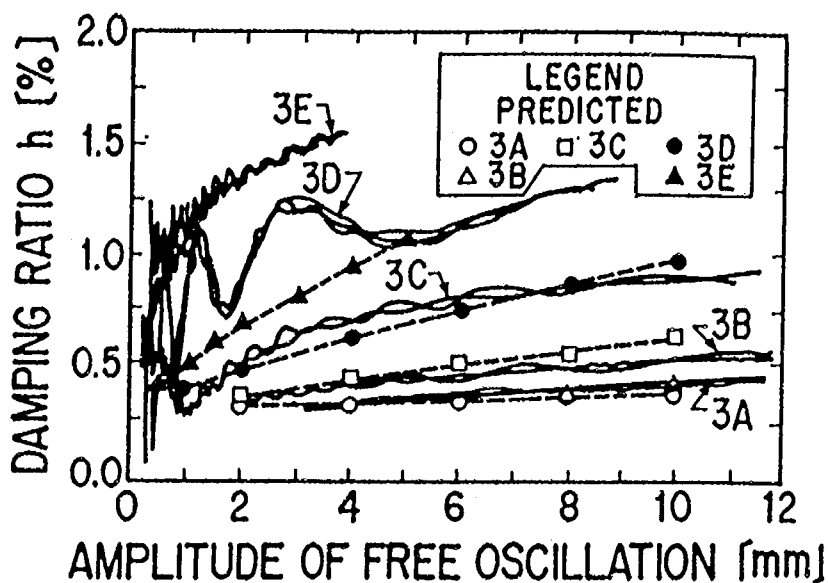


FIG. 16. Comparison of Damping Ratio versus Oscillation Amplitude Relation for Longitudinal Oscillation

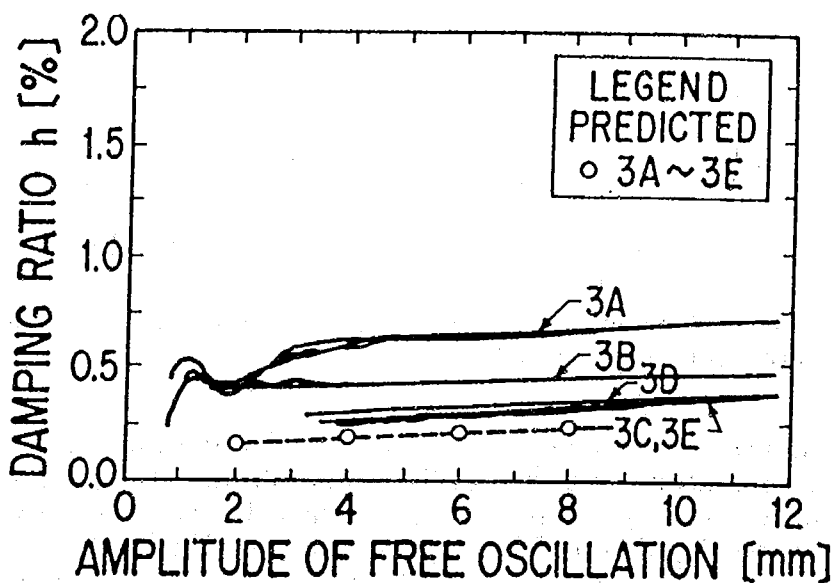


FIG. 17. Comparison of Damping Ratio versus Oscillation Amplitude Relation for Vertical Flexural Oscillation

The predicted damping ratio of the model for the vertical flexural oscillation is almost independent of the cable type. Although the damping ratio estimated from the experiments slightly decreases as the cable type changes from the fan type to the harp type, the overall characteristics of the predicted damping ratio seem reasonably close to the experimental results.

The damping ratio predicted is smaller than the experimental results probably because other sources of energy dissipation that are not considered in this analysis would contribute to the total energy dissipation. If the energy dissipation at each portion is evaluated in more detail, it is expected that the accuracy of the preceding estimation could be improved.

CONCLUSIONS

A method for evaluating the damping ratio of cable-stayed bridges by determining energy dissipation functions for individual substructures was

proposed, and the method was demonstrated by application to free-oscillation test results from a model of a cable-stayed bridge.

Through the analysis presented herein, the following conclusions may be deduced:

1. In the model free-oscillation test, the damping ratio depends on the cable type. As the cable type changes from the fan type to the harp type, the damping ratio for oscillation in the longitudinal direction increases. The damping ratio for the longitudinal oscillation is larger than that for the vertical flexural oscillation. In both directions, the damping ratio increases as the oscillation amplitude increases.

2. To clarify complicated damping characteristics of cable-stayed bridges, it is important to accurately evaluate energy dissipation in individual substructures. Special attention should be paid to the mechanism involved in the energy-dissipation process.

3. Dependence of damping ratio on the cable shape and oscillation amplitude, which is observed in the model free oscillation test, can be realistically predicted by evaluating the energy-dissipation functions in the proposed method.

4. Once the energy-dissipating functions are determined based on full-scale model tests for substructures, the proposed method is expected to be widely used for evaluating the damping ratio of cable-stayed bridges with arbitrary shape and proportion. Further effort for collecting full-scale test data on the energy-dissipating functions would be valuable.

APPENDIX. REFERENCES

- Clough, R. W., and Penzien, J. (1975). *Dynamics of structures*. McGraw-Hill Inc., New York, N.Y.
- Kato, N., Iioka, Y., and Kawahito, T. (1983). "Design of Meiko-nishi bridge." *Bridges and Found.*, 17(12), Tokyo, Japan, 18–26 (in Japanese).
- Kawashima, K., Unjoh, S., and Azuta, Y. (1986). "Seismic design method of cable stayed bridges—Part I: Dynamic characteristics of cable stayed bridges based on field forced excitation tests results." *Technical Report No. 2388*, Public Works Research Institute, Tsukuba, Japan (in Japanese).
- Kawashima, K., Unjoh, S., and Azuta, Y. (1988). "Damping characteristics of cable stayed bridges." *Proc., 9th World Conf. on Earthquake Engrg.*, International Association for Earthquake Engineering, Tokyo, Japan, (VI), 471–476.
- Kawashima, K., and Unjoh, S. (1989). "Damping characteristics of cable stayed bridges associated with energy dissipation at movable supports." *Proc., Japan Soc. of Civ. Engrs., Struct. Engrg./Earthquake Engrg.*, 6(1), 145–152.
- Kawashima, K., Unjoh, S., and Azuta, Y. (1990). "Analysis of damping characteristics of a cable stayed bridge based on strong motion records." *Proc., Japan Soc. of Civ. Engrs., Struct. Engrg./Earthquake Engrg.*, 7(1), 181–190.
- Ohlsson, S. (1986). "Model testing of the Tjörn bridge." *Int. Model Anal. Conf.*, Union College, New York, N.Y., 599–605.
- Stiemer, S. F., Taylor, P., and Vincent, D. H. C. (1988). "Full scale dynamic testing of the Annacis bridge." *IABSE Proc.*, P-122/88, 1–16.
- Wilson, J. C., and Liu, T. (1991). "Ambient vibration measurements on a cable-stayed bridge." *Earthquake Engrg. and Struct. Dyn.*, 20, 723–747.

RECENT SELECTED PUBLICATIONS AT
EARTHQUAKE ENGINEERING DIVISION,
PUBLIC WORKS RESEARCH INSTITUTE
(No. 2)

ISSN 0386-5878
Technical Memorandum
of PWRI No. 3183
May, 1993
Kazuhiko Kawashima, Head
Earthquake Engineering Division

This report presents some of the selected publications in the research on seismic design, seismic evaluation and strengthening of highway bridges, dynamic strength and ductility of structural components, structural response of menhir bridges (seismic isolated bridges), hybrid control, seismic response of embedded pipes, and damping characteristics of cable stayed bridges. They were presented at international conferences and workshops held in the last one years.



郵便はがき

Postage
stamp

305-□□

建設省土木研究所地震防災部耐震研究室
〒305 茨城県つくば市大字旭1番地

Earthquake Engineering Division,
Public Works Research Institute
Tsukuba Science City
Japan 305

No. 033

No. 033

配布先氏名

配布先住所



Receipt of

Technical Memorandum of PWRI No. 3183

Name _____

Address _____

