CURRENT RESEARCH EFFORTS IN JAPAN FOR PASSIVE AND ACTIVE CONTROL OF HIGHWAY BRIDGES AGAINST EARTHQUAKE

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ABSTRACT

This paper presents current research activities on passive, active and hybrid control for application to bridge structures conducted at the Public Works Research Institute.

KEY WORDS : Highway Bridges, Menshin Systems, Active Control, Hybrid Control

1. INTRODUCTION

Menshin systems (Base-Isolation), which is being introduced for application to highway bridges in Japan[1)2)], is to reduce earthquake response of highway bridges by "Passive Means". On the other hand, active control is to control it by "Active Means." Since external energy supply required to reduce the bridge response to satisfactory level is significant during a significant earthquake in the active control, it is still in a laboratory. Therefore, "Hybrid" control, which is a combination of advantages of passive and active control, is promising for reducing the earthquake response of structures with small amount of external energy. The development of hybrid control for improving the seismic safety of highway bridges, which can not be achieved by extending current seismic design method, is expected[1)].

A series of researches, and developments on passive, active and hybrid control for highway bridges have been made at the Public Works Research Institute since 1987. This paper briefly presents current research activities on passive, active and hybrid control for application to bridge structures conducted at the Public Works Research Institute.

2. DEVELOPMENT OF MENSHIN SYSTEMS

2-1 Joint Research Program on Menshin Systems for Highway Bridges

For studying the application of the menshin systems (passive control) to the seismic design of highway bridges, a committee chaired by Professor Tsuneo Katayama, University of Tokyo, was formulated through 1986 to 1989 at the Technology Research Center for National Land Development. Three programs were studied in the committee, i.e., 1) a survey of menshin devices which can be used for highway bridges, 2) a study on the key points of the design of menshin highway bridges, and 3) designs of menshin highway bridges. As the final accomplishment of the study, "Guidelines for Design of Menshin Bridges (draft)" was published in 1989[3)].

Based on the study above, the three-year joint research program on the menshin systems for highway bridges is now underway between Public Works Research Institute and twenty eight private firms since the year of 1989. The goal of the program is to develop the menshin design method and the new menshin devices for highway bridges in order to improve the seismic performance of new and existing bridges with less cost by the menshin systems. **Fig.1** shows the research items of the program, which are outlined in the follows[4)].

2-2 Development of New Menshin Devices

A lot of base isolation devices which are similar to the menshin device have been ever developed for building structures. However, the menshin devices for highway bridges have to be more compact and more weather-proof than the devices for building structures since the menshin devices would be installed at narrow and exposed crests of bridge columns. The new menshin devices should be developed exclusively for menshin highway bridges to be effectively constructed in Japan.

The following ten new devices in the six types are now being developed under the research program.

1) High Damping Rubber Bearing----- 4 devices
2) Sliding Friction Damper---------- 2 devices
3) Steel Damper -------------------- 1 device
4) Roller Menshin Bearing ---------- 1 device
5) Link Bearing -------------------- 1 device
6) Viscous Damper ------------------ 1 device

All developed menshin devices but the link bearing were tested by the loading experiment system of PWRI under the same loading conditions to verify their performance. In the tests, the cyclic lateral force was given to the devices by the actuator under the constant axial force, which is equivalent to the dead load of the superstructure, as shown in **Photo 1**. Since the performance of some devices could be affected by the temperature, the tests were conducted in the room shown in **Photo 2**, where the temperature was kept at 20 ℃.

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Fig.2 shows the lateral force vs. displacement hysteresis loops of a high damping rubber bearing and a sliding friction damper. It is said that the damping effect of every tested devices is satisfactory for highway bridges and the dynamic character is also stable with the smooth hysteresis loop.

The standard of the test procedure for menshin devices are being specified under the research program in order to certify the other devices which would be developed for highway bridges in future. There are two main items in the procedure, one of which is related to the performance as a support such as the check of the vertical stiffness, and the other is related to the dynamic horizontal performance as a menshin device such as the check of the stability under the repeated cyclic lateral loading, the check of the dependency on the loading velocity or frequency, the check of the dependency on the axial force, the check of the dependency on the lateral loading direction and the check of the dependency on the temperature. The test condition for each item and the evaluation of the test result have been formulated. The size and shape of the specimens, the loading test apparatus and the loading process will be standardized.

2-3 Development of Expansion Joints and Restrainers for Menshin Bridges

The knock-off mechanism and the finger expansion joint which is distinguished from the regular finger joints by the transverse movement, are being developed under the research program. The restrainer which consists of the steel bar installed in the crest of the substructure and the steel casing with the high damping rubber inside installed under the superstructure, is also being developed.

The knock-off mechanism is illustrated in **Fig.3**[5]. If the excessive displacement of the girder toward the abutment were brought by earthquakes, the apron of the slab would collide with the top portion of the abutment. Since the portion is not actually fixed to the abutment itself, it would be squeezed into the backfill behind the abutment to ease the impact induced by the collision.

The mechanism was tested as shown in **Fig.4**, in which the top portion was pressed back into the backfill by the actuator, in order to observe the situation of the squeeze and the damage of the backfill, and measure the resistant force of the backfill[6]. **Photos 3** and **4** show the specimen and the situation after the test, respectively. **Fig.5** shows the lateral force vs. displacement curve of the mechanism. It is seen that the impact force could be mitigated and the extensive displacement of the girder would be controlled by the mechanism.

2-4 Development of Menshin Design Method

Fig.6 illustrates the flow of the menshin design method, in which the two levels of the design earthquake loading are considered as limited states. The lower level, Level 1, is equivalent to the design seismic force of the seismic coefficient method regulated in the Design Specifications of Highway Bridges - Part V Seismic Design. This represents the seismic force for moderate earthquakes which are supposed to take place with higher probability. The higher level, Level 2, is equivalent to the design seismic force of the check of the bearing capacity of the reinforced concrete columns for the lateral force regulated in the specification, for larger earthquakes, such as Kanto earthquake in 1923, which are supposed to take place rarely[7][8]. The design procedure will be formulated, being based on the flow.

2-5 Application of Menshin Design

Since expansion joints make annoying noise and are vulnerable to the traffic load, continuous bridges tend to be constructed recently with the demand of lowering the number of joints. Therefore, one target of the menshin systems is to construct continuous bridges. The multi-span continuous bridge with the girder length more than 1 km, which is called in the research program as a super multi-span continuous bridge, is highlighted as a crucial research item. It was concluded that the super multi-span continuous bridge with 26 spans as shown in **Fig.7** could be constructed with the menshin system.

Existing simple girders would be connected one another in order to also decrease expansion joints between adjacent simple girders. Since the inertia force of the girder transmitted to each substructure might be varied by the connecting, the strengthening of some substructures would be required due to the lack of the bearing capacity. It was deduced that the distribution of the inertia force could be intentionally adjusted with proportion to the capacity of each substructure, by the replacement of the existing ordinary fixed or movable bearings with the menshin devices. The construction method and the structural detail of the connecting and the replacement will be examined under the research program.

2-6 Draft Manual for Menshin Design of Highway Bridges

The program will be accomplished in March, 1992. The draft manual of the menshin design method, the contents of which are shown in the follows, is to be composed as the fruits of the research program.

Chapter 1 INTRODUCTION

1.1 Purpose of Manual

1.2 Application of Manual
1.3 Definitions of Terms

Chapter 2 BASIC CONCEPTS
- 2.1 Purpose of Menshin Design
- 2.2 Earthquake Levels and Limited States
- 2.3 Seismic Design Force of Menshin Design
- 2.4 Design Method and Criterion of Safety Check
- 2.5 Basic Flow Charts of Menshin Design
- 2.6 Application of Menshin Design

Chapter 3 DESIGN AND CHECK OF SUPERSTRUCTURE

Chapter 4 DESIGN AND CHECK OF SUBSTRUCTURE
- 4.1 General
- 4.2 Design Acceleration for Substructure

Chapter 5 DESIGN METHOD OF MENSHIN DEVICES
- 5.1 Basic Concept of Design
- 5.2 Design Acceleration for Menshin Devices
- 5.3 Rubber Bearing
- 5.4 Lead Rubber Bearing
- 5.5 High Damping Rubber Bearing
- 5.6 Steel Damper
- 5.7 Friction Damper
- 5.8 Viscous Damper

Chapter 6 CHECK OF MENSHIN BRIDGES BY DYNAMIC ANALYSIS
- 6.1 General
- 6.2 Input Motion of Dynamic Analysis
- 6.3 Dynamic Analysis Method
- 6.4 Safety Check by Dynamic Analysis

Chapter 7 STRUCTURAL DETAILS OF MENSHIN BRIDGES
- 7.1 General
- 7.2 Restrainers
- 7.3 Structural Details of Menshin Devices
- 7.4 Structural Details of Expansion Joints

Chapter 8 CERTIFICATION TESTS OF MENSHIN DEVICES
- 8.1 General
- 8.2 Dynamic Loading Test
- 8.3 Static Loading Test
- 8.4 Ultimate State Test

Chapter 9 RETROFITTING BY MENSHIN DESIGN
- 9.1 General
- 9.2 Application of Menshin Design for Retrofitting
- 9.3 Design Method of Menshin Devices for Retrofitting
- 9.4 Construction of Menshin Devices for Retrofitting
- 9.5 Structural Details of Retrofitting

Appendix I References for Design
Appendix II Design Examples

3. CONSTRUCTION OF MENSHIN HIGHWAY BRIDGES

3-1 Pilot Construction of Menshin Highway Bridges

Five pilot menshin highway bridges as shown in **Table 1** are under construction with the supervision of Ministry of Construction, in order to verify the performance of the menshin system. One of them, Miyagawa Bridge in Shizuoka-ken, was completed and opened for the public traffic in March 15, 1991 as the first menshin highway bridge in Japan.

3-2 Miyagawa Bridge - First Menshin Highway Bridge -

1)Outline of Miyagawa Bridge

The Miyagawa bridge located at Haruno-machi, Shu-ti-gun, Shizuoka-ken, is a three-span continuous steel plate girder bridge with the total length of 105.8 m and the width of 10.5 m as shown in **Fig.8**, which spans over the Ketagawa river as a bridge of the national highway No. 362[20]. The rubber bearings with the lead plug inserted at the center shown in **Photo 5**, which were developed in New Zealand and are usually designated as lead rubber bearings [10][11], were used as the menshin devices. Although the lead rubber bearing is originally movable to any lateral direction, the restrainers were installed to prevent the bearing from the transverse movement so that the menshin system works only in longitudinal direction.

2)Menshin Design

The bridge was designed in accordance with the Design Specifications of Highway Bridges. **Table 2** summarizes the natural period, the design horizontal acceleration and the design relative displacement of the bearing for both the seismic coefficient method and the check of the bearing capacity of the reinforced concrete columns for the lateral force. The design acceleration is 0.2 g with the natural period of 0.76 sec in the seismic coefficient method. It is 0.54 g with the natural period of 1.01 sec in the check of the bearing capacity, when energy dissipation effect of the base-isolated devices is taken into account. However, since the bridge is one of the first menshin bridges in Japan, the damping effect of the devices was not taken into account in the check of the bearing capacity. Therefore, the design acceleration of 0.7 g which was obtained by assuming that the energy dissipation of the devices is zero, was used in the check of the bearing capacity.

Since the menshin system triggers off the larger displacement of the superstructure relative to the substructure than the ordinary bridges, the sufficient gap should be kept between the abutment and the girder, and at the expansion joint. On the other hand, the larger gap at the expansion joint would bring the vulnerability to the traffic load. Therefore, the following concept was adopted that the gap between the abutment and the girder should be decided as the girder would not collide to the abutment when it is subject to earthquakes for the check of the bearing capacity, and the gap at the expansion joint should be made equivalent to the relative displacement developed at the expansion joint by earthquakes of the seismic coefficient method. It was followed that the gaps between the abutment and the girder were 150 mm and the gaps at the expansion joints were 50 mm.

3) Field Vibration Test

The free vibration tests were conducted under the joint research program between the Earthquake Engineering Division of PWRI and the Shizuoka-ken government prior the opening in order to verify the effect of the menshin system. The three hydraulic static jacks owned by the Research Institute of Japan Highway Public Corporation were placed between the temporary steel frames assembled under the girder and the A2 abutment as shown in **Photo 6.** The force capacity of each jack is 150 tf and the maximum stroke is 150 mm. The force created by the jack can be released more rapidly than the regular static jack to develop the free vibration. The girder was forced by the jack to move against the abutment in the longitudinal direction to a certain displacement. The maximum displacement was decided as 8 cm so that the total force of the jacks should not exceed the capacity of the abutment.

The accelerations on the girder at each substructure (8 points), the accelerations on the crests of the substructures (8 points) and the displacement of the girder relative to each substructure (8 points) were recorded during the free vibration tests.

Fig.9 shows the relative displacement of the girder on the A2 abutment when the force of the jack was released at 8 cm. It is observed in the figure that the free vibration of the girder was developed in 0.75 sec after the release and the girder came nearly to a stop at 4 sec, remaining the residual displacement of 5.4 cm by the plastic resistant force of the lead plug.

Fig.10(a) shows the general model for the lateral load vs. displacement relation of the lead rubber bearing. Since the load consists of the linear restoring force of the rubber and the plastic restoring force of the lead plug, the relation can be expressed as a combination of the linear spring model and the friction model shown in **Figs.10(b)** and **(c)**, respectively. The natural period and the damping effect was analyzed with a simple single degree of freedom oscillation system as shown in **Fig.11**, which is a model for the girder and the lead rubber bearings.

The spring constant and the friction force necessary for the analysis were obtained from the result of the loading tests, which had measured the lateral load vs. displacement relation of the lead rubber bearing prior to the construction, as 2,351 tf/m and 173.8 tf, respectively. The result of the analysis is also shown with a dotted line in the **Fig.9.**

The analysis evaluates the test very well in the first 0.75 sec after the release. It can be derived that the dynamic performance of the menshin bridge could be assessed with the parameters obtained from the loading test of the bearing itself. The gradual decrease of the relative displacement after 0.75 sec observed in the test, could have to do with the creep of the lead plug.

The traffic vibration tests were also conducted by running the trucks under the joint research program between the Structure Division of PWRI and the Shizuoka-ken government. No unusual vibration due to the menshin system was monitored around the bearing.

The long-term measurement of the residual relative displacement after the release at 8 cm will be done since the end of a series of tests. The displacement was recovered to 0.9 cm at about one month later. It is said that the residual displacement of the bearing cause no practical problem.

At the Miyagawa bridge, the strong motion observation is now being made by the Shizuoka-ken government and Ground Vibration Division of PWRI.

4. ACTIVE CONTROL FOR SEISMIC RESPONSE OF HIGHWAY BRIDGES

4-1 Introduction

Applicability of an Active Mass Damper (refers as AMD in the following) for controlling earthquake response of highway bridges was investigated by using a single-degree-of freedom model and reported in the last joint meeting. The results showed that a great deal of external energy is required in order to control earthquake response in appropriate level by means of AMD. The mass and/or stroke required are excessively large, and it seems not practical to control seismic response of bridge structures against a significant earthquake [12)13)]. Hence, it is of great importance to develop hybrid control by combination of passive and active means.[13)]

Development of hybrid control technology was initiated at the Public Works Research Institute since 1989. Preliminary study on control of seismic response of highway bridges using multiple AMDs, which is basis for hybrid control is presented here.

The cooperative research program is being executed as a U.S.-Japan cooperative research program through the the Panel on Wind and Seismic Effects, UJNR. The program is being conducted at the Public Works Research Institute in Japan side, and at the National Institute for Standards and Technology, and at National Center for Earthquake Engineering Research[14] with supports of National Science Foundation in U.S. side. Major subjects of the program in Japan side includes investigations on ① passive control method and ② active control method for application to hybrid control as well as ③ optimum hybrid control which combines them appropriately

4-2 Highway Bridge Analyzed and Analytical Conditions

For studying where is the best point for control and effect of multiple control by means of AMD, the transverse response of three-span continuous girder bridge with length of 150m as shown in Fig.12 is analyzed. The bridge is modeled as a two-dimensional spring-mass model. Fundamental natural frequency is 1.67Hz, and this is a first flexural mode of the deck. The first mode is most predominant in the transverse response of the bridge.

For controlling the deck response by means of one AMD, the weight of a driving mass is assumed as 180tf, which corresponds to about 1/10 of the deck weight (1,950tf). Control by means of two AMDs was also studied, in which two AMDs were assumed to be installed on the girder at the P_1 and P_2 piers, respectively. Three cases are analyzed, that is, their weights are assumed to be 180tf each, 90tf each, and 120tf and 60tf (180tf in total). The spring coefficients, which supports driving masses, are assumed so that fundamental natural frequency of the mass of AMD be 1/4 of fundamental natural frequency of the bridge analyzed. Structural damping ratio of the bridge is assumed as 5%. A standard ground acceleration, which is specified in "Part V Seismic Design[7,8]" of the "Design Specifications for Highway Bridges", is used as an input motion.

Control algorithm of AMD follows the classical optimum feedback control method. The control force driving the mass of AMD is assumed as a linear combination of the values proportional to displacement and velocity of the structure and AMD. Proportional coefficients are called as control gain, and the optimum gain obtained by solving Riccatti's Algebraic Equation[15,16] is used.

4-3 Effect of Controlling Point by AMD

In order to investigate an optimum control point, the earthquake response analyses are conducted. AMD is assumed to be placed at the some points on the girder. **Fig.13** shows peak response displacement and peak response acceleration of the girder. It is more effective in reducing seismic response of the girder when the AMD is installed near the center of the girder. Comparing with uncontrolled one, peak displacement of the girder decreases to 1/4 and peak acceleration decreases to about half. On the other hand, effectiveness is smaller when AMD is placed near the abutments. For example, when AMD is placed on Abutment A_2, although the response displacement becomes smaller by 15% than uncontrolled one, displacement response near the ends of girder becomes larger by about 13 times the uncontrolled displacement.

Therefore, it is more effective to install AMD at the point where the mode value of predominant mode shape is largest.

4-4 Effect of Multiple Control

In order to investigate effectiveness of multiple control, the earthquake response is analyzed, in which AMDs are assumed to be installed on the P_1 and P_2 piers. **Fig.14** shows peak displacement and peak acceleration of the girder controlled at the two points. Results controlled by only one AMD is also shown in **Fig.14** for comparison. Two-point control makes possible to decrease the seismic response of girder more significantly than the control by only one AMD. Effectiveness of control of the response and control force required by AMDs with the weight of masses of 180tf is almost same as those of 90tf both. Effect of the weight of driving masses appears only in stroke lengths of driving masses. The strokes of driving mass when the weight of driving masses is 90tf both is twice those when the weight is 180tf. Therefore, the weight of driving masses affects the stroke lengths of driving masses, but does not affect the effectiveness of control. The similar results can be obtained when the weight of driving masses are 60tf and 120tf each.

4-5 Concluding Remarks up to the Present

1) It is most effective that the AMD is installed at the point where the mode shape value of a predominant mode is largest for controlling the seismic response of continuous girder bridges.

2) Two-point control using AMDs is more effective than one-point control in reducing the displacement response and acceleration response of the girder. Since multiple control makes it possible to control the deck response against any mode shapes, it is required to further investigate the effect of the multiple-point control in detail.

5. DEVELOPMENT OF A VARIABLE DAMPER

5.1 Basic Concept of A Variable Damper for Highway Bridges

The variable damper is basically a viscous damper and the viscous damping force is variable depending on the response of structures (highway bridges) as shown in **Fig.15**. For instance, the damping coefficient of the damper is larger during the vibration within a small amplitude and the damper has the same function as a fixed bearing support against braking load of vehicles. It is, however, movable against the load with low velocity, such as elongation of girder by temperature change. Once an earthquake occurs and amplitude of vibration of girder becomes larger up to a certain level, the damping coefficient of damper decreases so that energy dissipation be optimum and inertia force to the substructure be adjusted appropriately. Furthermore, when the vibrational amplitude becomes excessive, the damping coefficient increases gradually in order to suppress the amplitude and so that the damper have a function as a stopper. Therefore, the variable damper has the advantages of an usual viscous damper-stopper, a passive energy dissipator and a stopper with a shock absorber. The required external energy by the variable damper for altering the damping coefficient is greatly smaller than usual active control devices because the width of the orifice of the damper can be changeable with a small amount of energy.

5.2 Earthquake Response Analysis of Multi-Degree-of-Freedom System with Variable Dampers

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

$$\underline{M}\ \underline{x} + (\underline{C} + \underline{C_v})\ \underline{x} + \underline{K}\ \underline{x} = -\ \underline{M}\ x_0 \qquad (1)$$

in which $\underline{M}$, $\underline{C}$ and $\underline{K}$ represent mass, damping and stiffness matrices of the structure, respectively. $\underline{x}$ and x_0 denote displacement vector and ground displacement, respectively. $\underline{C_v}$ denotes a damping matrix of the variable dampers and is given by a function of relative displacement and relative velocity between the nodes where the variable dampers are installed. Since $\underline{C_v}$ is time-varying, Eq.(1) has to be solved by a direct integration method. According to the Newmark β method, velocity and displacement are assumed as

$$x_i = x_{i-1} + (1-\alpha)\Delta t \cdot x_{i-1} + \alpha \cdot \Delta t\ x_i \qquad (2)$$

$$x_i = x_{i-1} + \Delta t \cdot x_{i-1} + (1/2 - \beta)(\Delta t)^2\ x_{i-1} + \beta(\Delta t)^2 \cdot x_{i-1} \qquad (3)$$

in which α and β are parameters of the Newmark β method and if a constant acceleration method is assumed α and β have to be taken 0.5 and 0.25, respectively.

Using the above analytical method, a computer program, named as "VDAM", which can analyse earthquake response of multi-degree-of-freedom system with the variable dampers was developed.

5-3 Highway Bridge Analyzed and Analytical Conditions

In order to investigate effectiveness of the variable damper, a simple span girder bridge, as shown in **Fig.16**, with a span length of 30m is analysed in the longitudinal direction. The model bridge has elastic isolator and the variable dampers are assumed to be installed between the superstructure and the top of substructure. The spring constant of elastic isolator is defined so that a fundamental natural period of the bridge be 1 second.

Fourteen cases in total are analysed as shown in **Figs.17** and **18**. In Case 1 the variable damper is not installed, in Cases 2 and 3 the damping coefficient of the damper is in dependent of displacement and/or velocity. The damping coefficients of Cases 2 and 3 are given so that damping ratio of critical be 5% (Case 2) and 100% (Case 3). In Cases 4 and 5, the damping coefficient is dependent on relative displacement as shown in **Fig.17**. Peak relative displacement d_u is defined as a peak relative displacement computed in Case 2 and it was assumed as 19.4cm. In Cases 6 and 7, the damping coefficient is dependent on relative velocity as shown in Fig.17. Peak velocity v_u is defined as a peak value (113 cm/s) computed in Case 2. Case 8 is a combination of Cases 4 and 6 and the damping coefficient of the damper is dependent on relative displacement and relative velocity.

A ground motion acceleration which was modified so that the response spectra matches with the target spectra is used as an input ground acceleration. The acceleration response 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, is assumed as the target spectra.

Damping ratio of critical of a model bridge is assumed 2% as a modal damping.

5-4 Effectiveness of Variable Dampers on Earthquake Response of Highway Bridges

Table 3 shows the peak response of girder and the variable damper, where the peak values of displacement, velocity and acceleration of the girder and damping force, relative displacement and relative velocity of the damper are presented. **Figs.19** and **20** compare time histories of displacement and acceleration of the girder and damping force of the variable damper between Case 1 (without the variable damper), Case 5 (displacement dependent) and Case 7 (velocity dependent).

According to these results, the following remarks may be pointed out:

1) If the damping coefficient is dependent on relative displacement of the damper, it is effective to set larger damping coefficient around zero displacement (Case 5). The peak damping force in Case 5 becomes 58.2× 2tf which corresponds to 48.2% of the weight of the deck (241.5tf). The damping force of 96.4tf makes reduction of acceleration of girder to 566gal/1300gal = 43.5%, and relative displacement of the girder to 13.25cm/33.04cm = 40.1% comparing with Case 1 in which the variable damper is not installed.

2) The damping force required to control the response up to the same level is smaller in displacement dependent dampers (cases 4 and 5) than in velocity dependent dampers (cases 6 and 7). This is because pulse-type large damping forces are required in the velocity dependent dampers. Large damping coefficient at the time when large velocity is developed causes large damping force. On the other hand, since the damping force is dispersed as time large damping forces are not required in the displacement dependent dampers.

3) The time history displacement shows that the variable dampers operate satisfactorily as a stopper which also has a function as a shock absorber without an impact response.

5-5 Development of A Pilot Model of the Variable Damper

Photos 7 - 9 show a pilot model of the variable damper developed. The model is designed so that the damping force be variable within 20kgf to 200kgf, maximum relative displacement of damper piston be ±5cm, and the weight be about 40kgf. The variable damper consists of an usual viscous damper and a control system. Damping characteristics of the variable damper is now being studied. Model experiments are planed.

5-6 Concluding Remarks up to the Present

1) In the displacement dependent dampers, it is effective to set larger damping coefficient around zero displacement.

2) The damping force required to control the response up to the same level is smaller in the displacement dependent dampers than in the velocity dependent dampers.

3) The time history response shows that the variable dampers operate satisfactorily as a stopper which also has a function as a shock absorber without an impact response.

4) The pilot model of the variable damper was developed. Experimental study on the characteristics and control method of the variable damper is being studied.

5) The variable damper seems to be near a practical use.

6. CONCLUSIONS

This paper briefly presents current research and development activities on passive, active and hybrid control for application to bridges structures conducted at Public Works Research Institute. It is thought that these technologies may become very important in future for constructing super-multiple continuous girder bridges, long-span bridges, bridges with high-rise piers, and bridges in urban area with strict land use restrictions.

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Table 1 Pilot Program for Construction of Menshin Highway Bridges

Owner	Name of Bridge	Type of Superstructure	Total Length
Hokkaido developing Bureau	On-netoh Bridge	Steel Girder	456 m
Tohoku Regional Construction Bureau, MOC	Nagakigawa Bridge	Steel Girder	97 m
Iwate-ken	Maruki Bridge	Prestressed Concrete	92 m
Tochigi-ken	Daiichi Karasuyama Bridge	Prestressed Concrete	250 m
Shizuoka-ken	Miyagawa Bridge	Steel Girder	110 m

Table 2 Natural Period, Design Acceleration and Design Relative Displacement

Design Method	Item	A1	P1	P2	A2
Seismic Coefficient Method	Natural Period (sec)	0.764			
	Design Acceleration (g)	0.20			
	Design Relative Displacement (mm)	28.2	21.8	20.2	28.0
Check of Bearing Capacity	Natural Period (sec)	1.01			
	Design Acceleration (g)	0.54			
	Design Relative Displacement(mm)	133	109	105	130

Table 3 Peak Response of Deck and Variable Damper

(a) Full Control

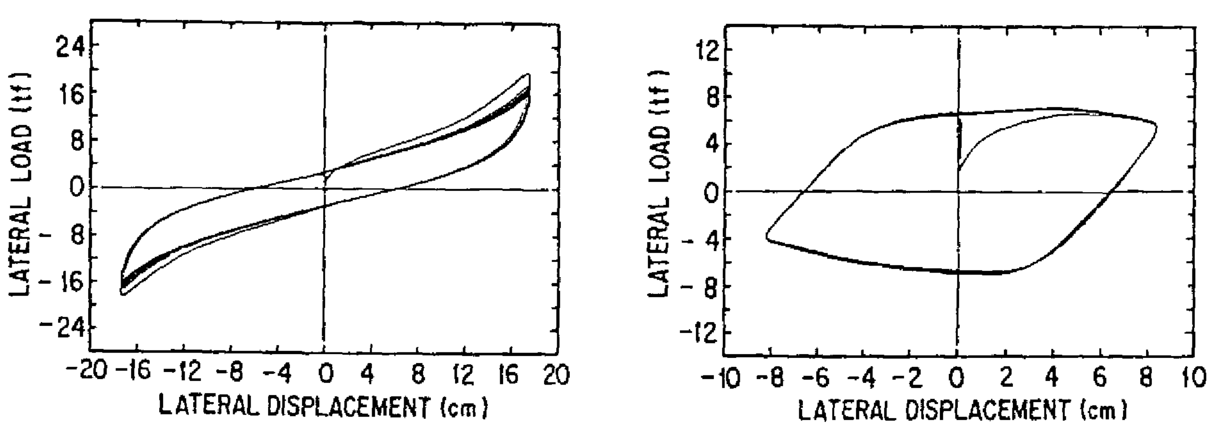
Analytical Cases		Deck			Variable Damper			
		Displace-ment (cm)	Velocity (cm/sec)	Acele-ration (cm/sec^2)	Damping Force (tf)	Relative Displace-ment (cm)	Relative Velocity (cm/sec)	Total Energy Absorption (tf·m)
1	No Control	33.04	189.0	1300	—	—	—	—
2	Constant Damping ($h_1 = 0.05$)	25.63	148.7	1012	8.8	19.41	113.1	65.41
3	Constant Damping ($h_2 = 1.0$)	7.97	45.1	481	47.1	5.73	30.4	81.51
4	Displacement Dependent Damping	23.33	140.7	1043	39.2	17.32	142.2	70.70
5	Displacement Dependent Damping	13.25	84.8	566	58.8	10.18	123.6	93.62
6	Velocity Dependent Damping	21.10	131.3	1117	140.3	16.05	169.4	78.41
7	Velocity Dependent Damping	20.35	126.7	1029	109.8	15.44	147.4	76.38
8	Displacement and Velocity Dependent Damping	20.96	122.7	1136	129.3	15.72	168.1	80.52

(b) Partial Control

Analytical Cases		Deck			Variable Damper			
		Displace-ment (cm)	Velocity (cm/sec)	Acele-ration (cm/sec^2)	Damping Force (tf)	Relative Displace-ment (cm)	Relative Velocity (cm/sec)	Total Energy Absorption (tf·m)
5	Displacement Dependent Damping	13.25	84.8	566	58.8	10.18	123.6	93.62
9	Control When $v > 0$	19.05	113.7	765	70.2	14.72	177.4	94.30
10	Control When $v < 0$	19.22	115.5	776	71.8	14.94	181.8	96.21
11	Control When $d \cdot v > 0$	20.53	127.9	909	67.6	15.40	167.9	92.20
12	Control When $d \cdot v < 0$	20.34	114.9	826	65.2	15.95	230.4	82.58
13	Control When $d > 0$	18.87	114.7	953	104.6	16.29	231.3	80.25
14	Control When $d < 0$	20.28	117.8	886	73.1	17.11	226.7	77.49

- Development of Menshin Devices for High way Bridges
  - High Damping Rubber Bearing
  - Sliding Friction Damper
  - Steel Damper
  - Roller Type Menshin Bearing
  - Link Bearing
  - Viscous Damper
  - Certification Test of Menshin Devices
- Development of Expansion Joints and Restrainers for Menshin Bridges
  - Expansion Joints for Menshin Bridges
  - Restrainers for Menshin Bridges
- Development of Menshin Design Method
  - Basic Concept of Menshin Design
  - Dynamic Analysis Method of Menshin Bridges
  - Design Method of Menshin Devices
  - Simplified Design Method of Menshin Bridges
  - Design Method of Restrainers and Expansion Joints for Menshin Bridges
  - Evaluation of Model Tests by Dynamic Analysis
- Application of Menshin Design
  - Application for Prestressed Concrete Bridges
  - Application for Steel Bridges
  - Application for Super Multi-span Continuous Bridges
  - Application for Seismic Retrofitting

Fig.1 Research Items of Joint Research Program



(a) High Damping Rubber Bearing

(b) Sliding Friction Damper

Fig.2 Lateral Load vs. Displacement Hysteresis Loop

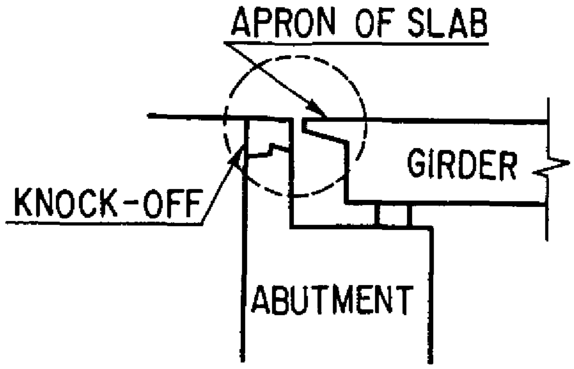


Fig.3 Knock-off Mechanism

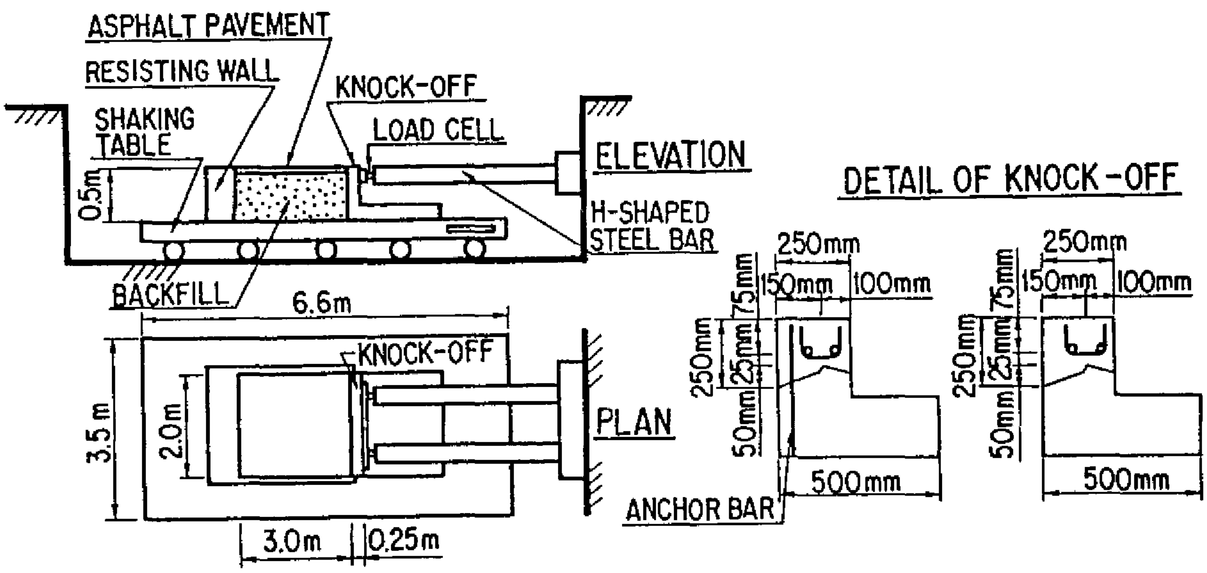


Fig.4 Experiment of Knock-off Mechanism[5]

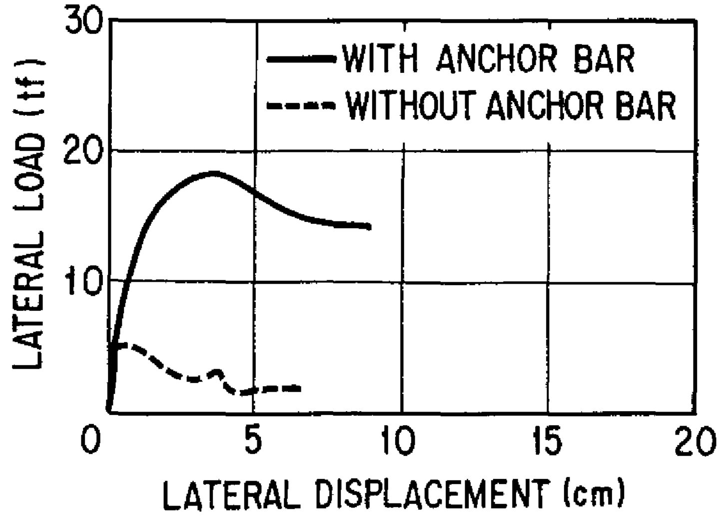


Fig.5 Lateral Load vs. Displacement Relation of Knock-off Mechanism[6]

[Item]	[Object]	[Analysis Method]	[Design Force Level]
Assumption of Initial Dimension	Substructure	Static Analysis	Level 1
Design	Menshin Device	Static Analysis	Level 2
	Substructure		Level 1
Check	Menshin Device Substrcture	Static Analysis	Level 2
		Dynamic Analysis	Level 1
			Level 2
Structural Detail	Expansion Joint		(Level 2)
	Restrainer		(Level 3)

Fig.6 Basic Flow Chart of Menshin Design

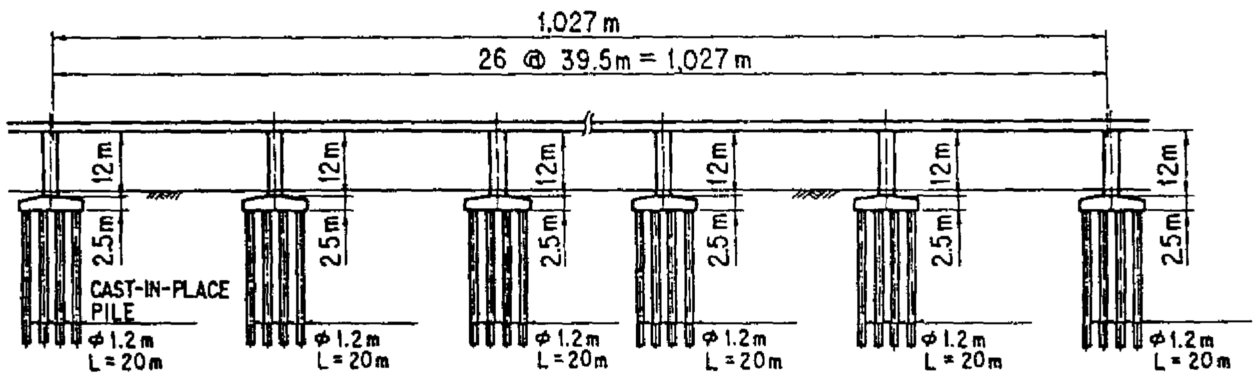


Fig.7 Super Multi-span Continuous Bridge

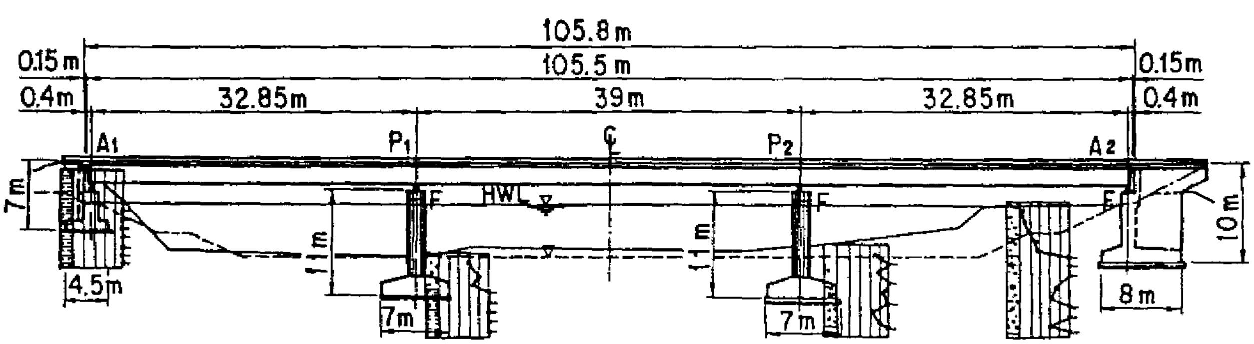


Fig.8 Miyagawa Bridge

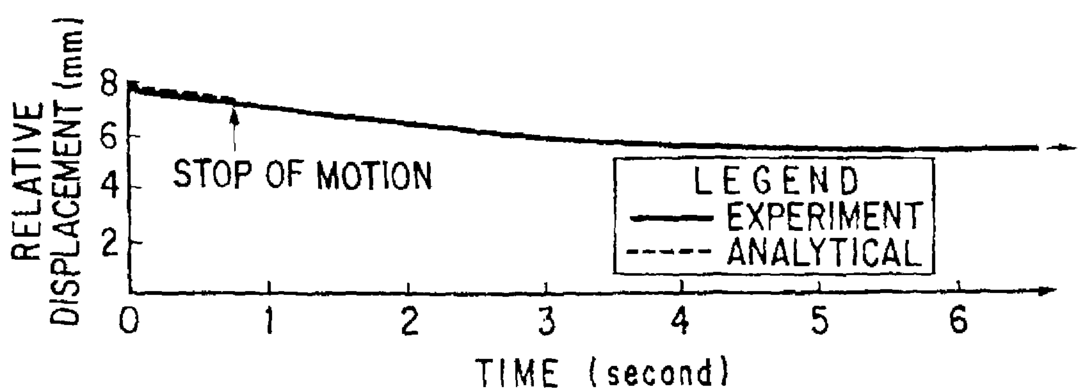
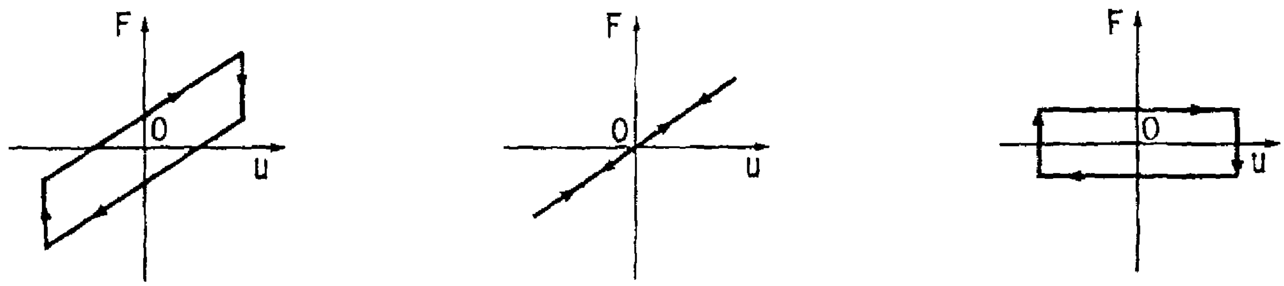


Fig.9 Relative Displacement after Release



(a) Lateral Load vs Lateral Displacement Relation of LRB

(b) Elastic Spring

(c) Friction Force

Fig.10 Load vs. Displacement Relation of LRB

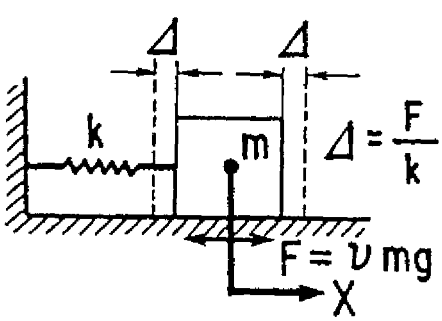
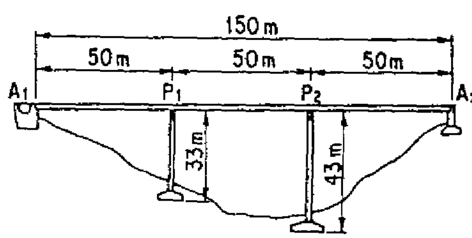
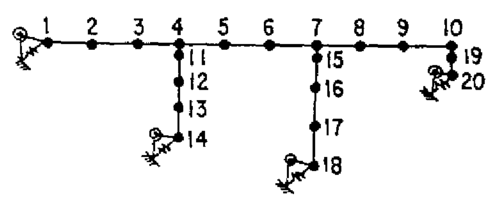


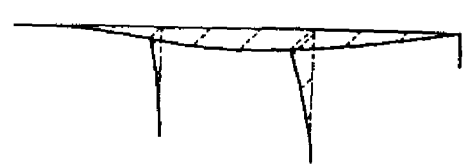
Fig.11 Single Degree of Freedom



(a) Highway Bridge Analyzed

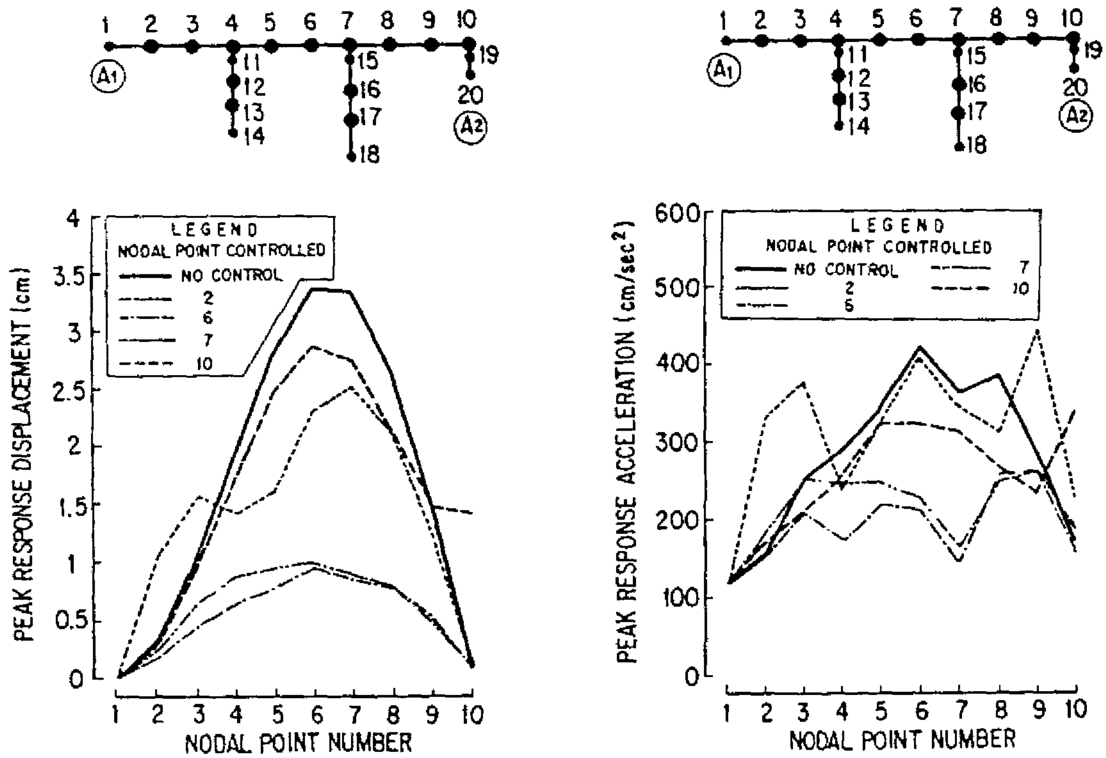


(b) Analytical Idealization



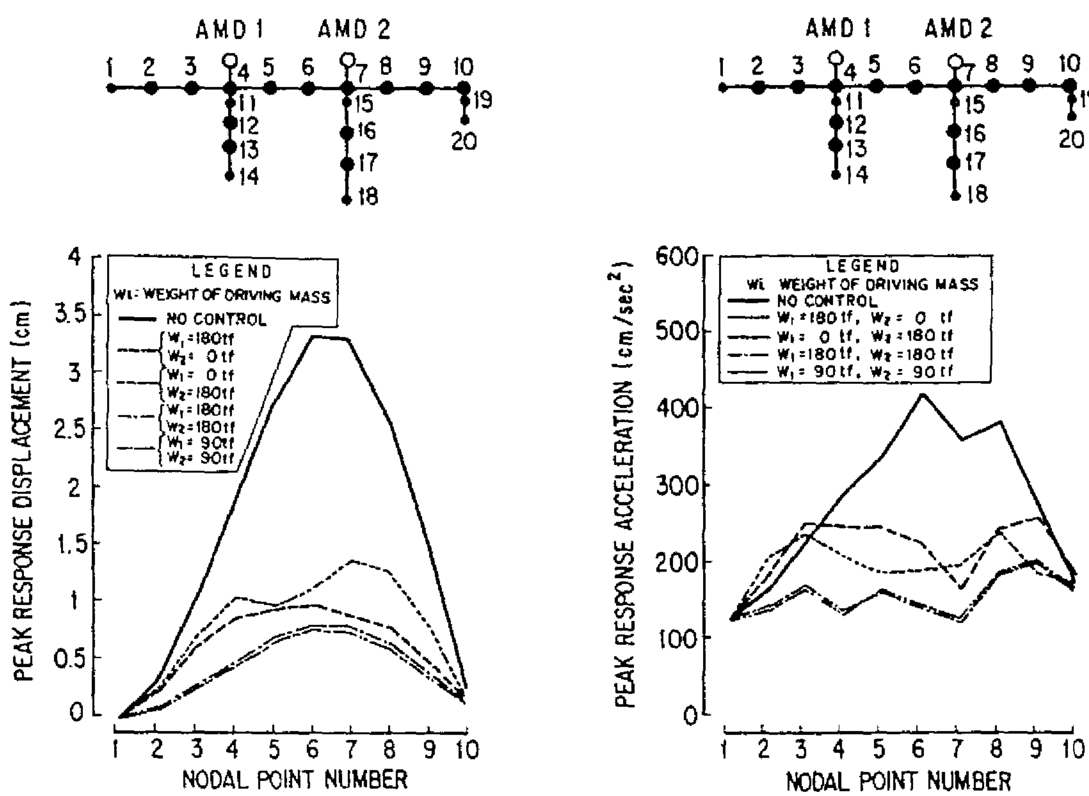
(c) Fundamental Natural Mode of Vibration (f_1=1.67Hz)

Fig.12 Highway Bridge Analyzed and Analytical Idealization



(a) Peak Response Displacement (b) Peak Response Acceleration

Fig.13 Effect of Controlling Point by AMD



(a) Peak Response Displacement (b) Peak Response Acceleration

Fig.14 Effect of Multiple Control by AMDs

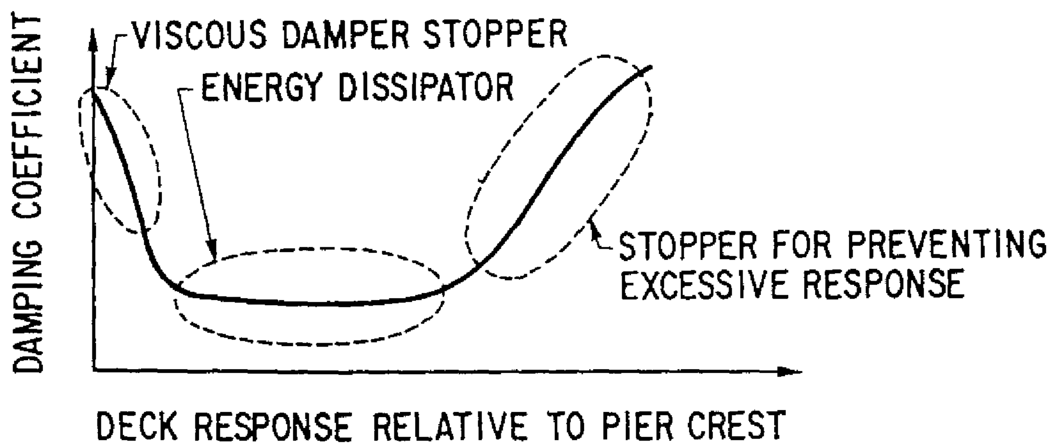


Fig.15 Basic Concept of Variable Dampers

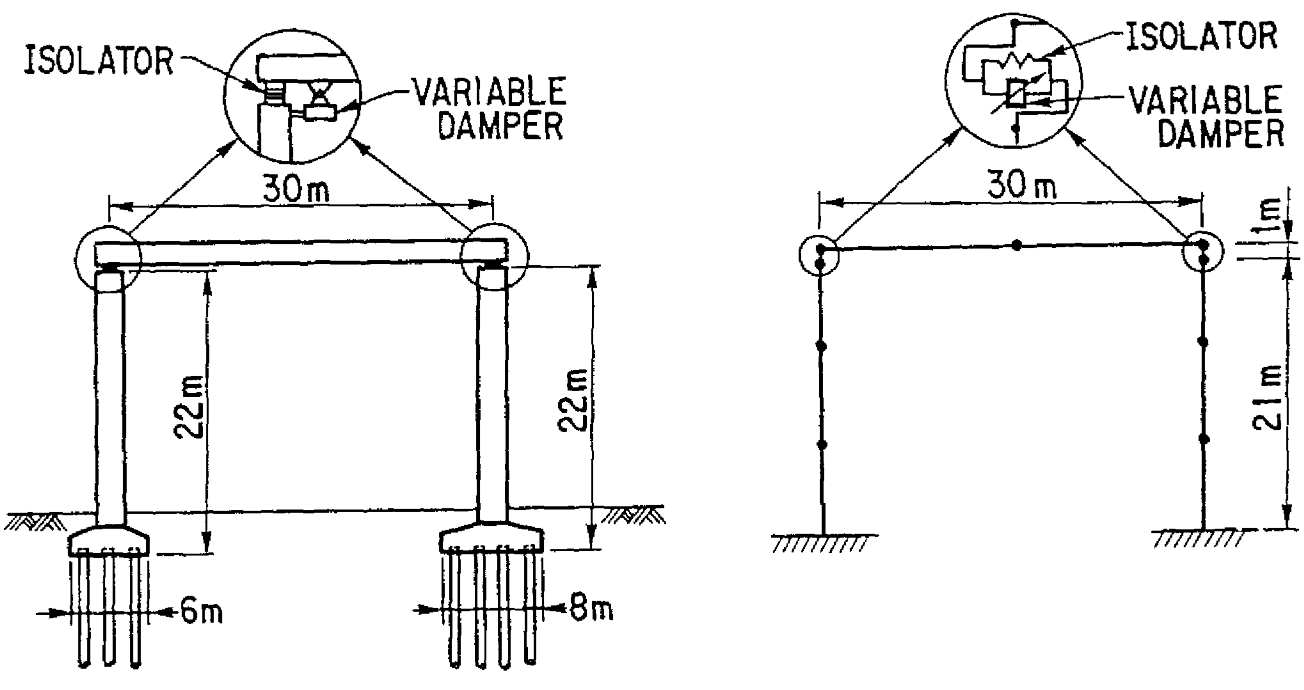
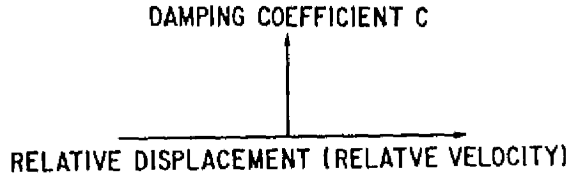
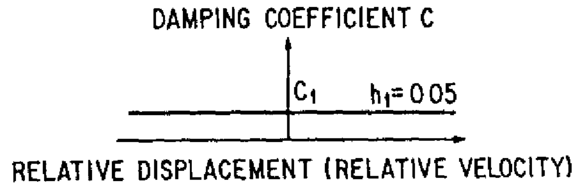


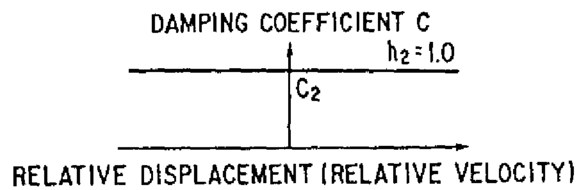
Fig.16 Highway Bridge Analyzed and Analytical Idealization



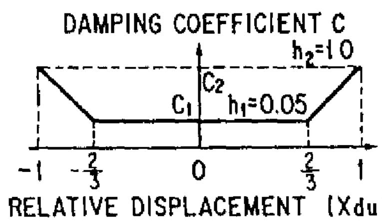
(a) Case 1: No Control



(b) Case 2: Constant Damping (h_1 = 0.05)

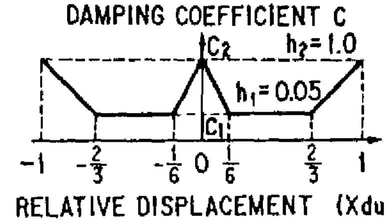


(c) Case 3: Constant Damping (h_2 = 1.0)

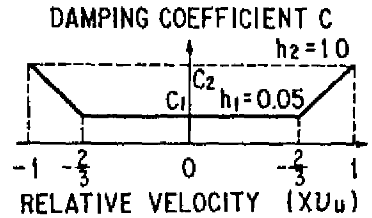


(d) Case 4: Displacement Dependent Damping

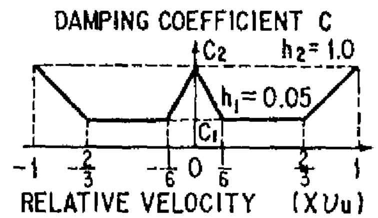
(du = 19.41 cm)



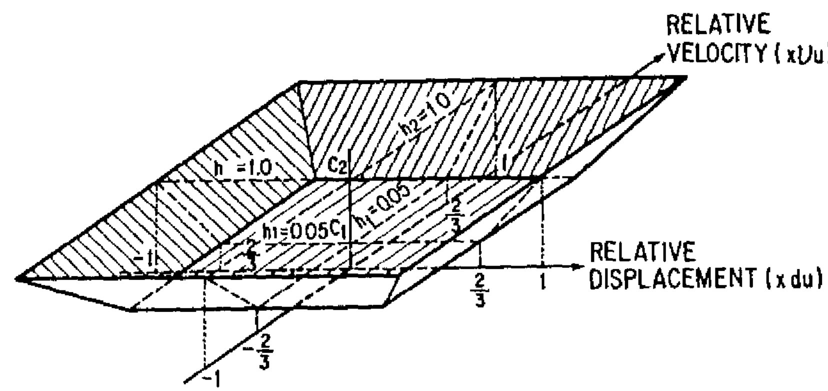
(e) Case 5: Displacement Dependent Damping (du = 19.41 cm)



(f) Case 6: Velocity Dependent Damping (Vu = 113.1 cm/sec)



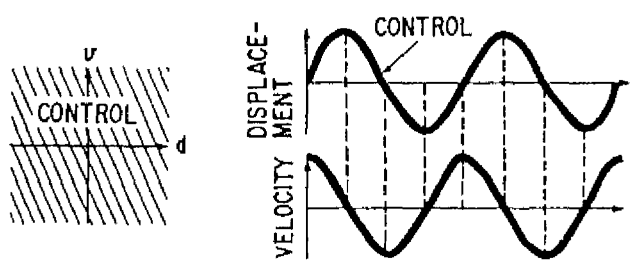
(g) Case 7: Velocity Dependent Damping Constant Damping (Vu = 113.1 cm/sec)



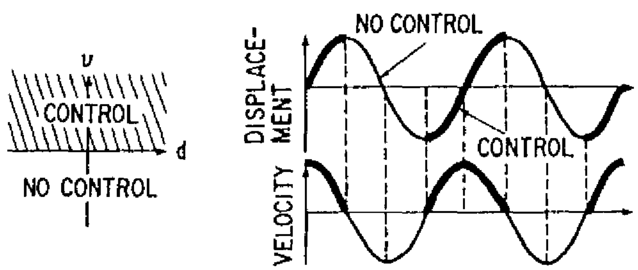
(h) Case 8: Displacement and Velocity Dependent Damping

(du =19.41 cm, Vu = 113.1 cm/sec)

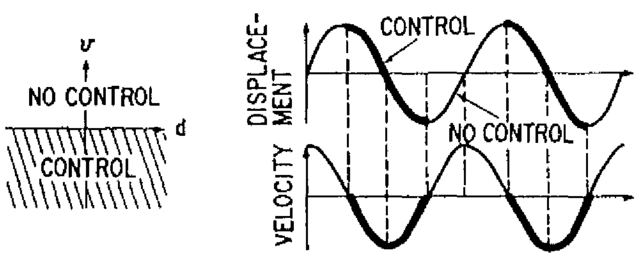
Fig.17 Analytical Cases for Full Control



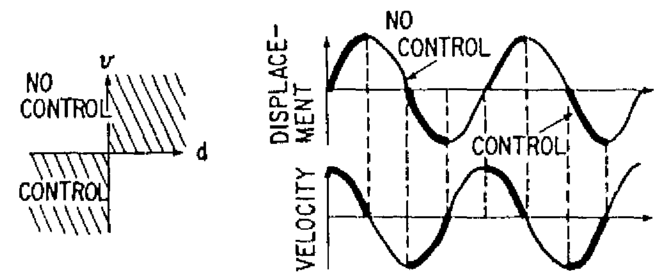
(a) Case 5



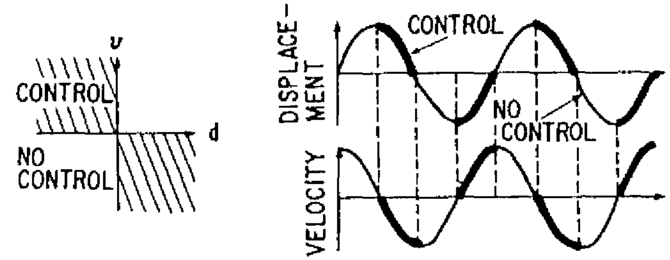
(b) Case 9 (Control when v > 0)



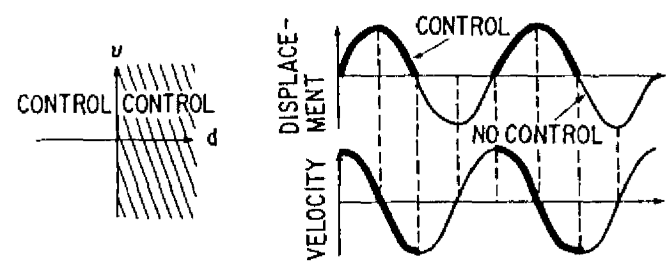
(c) Case 10 (Control when v < 0)



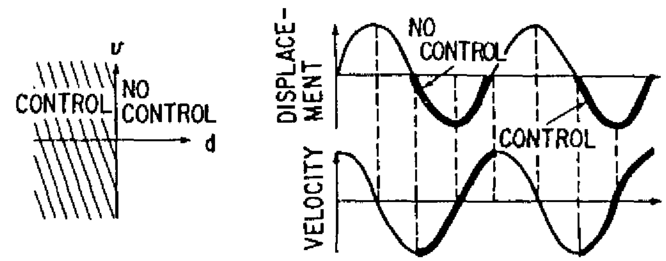
(d) Case 11 (Control when dv > 0)



(e) Case 12 (Control when dv < 0)

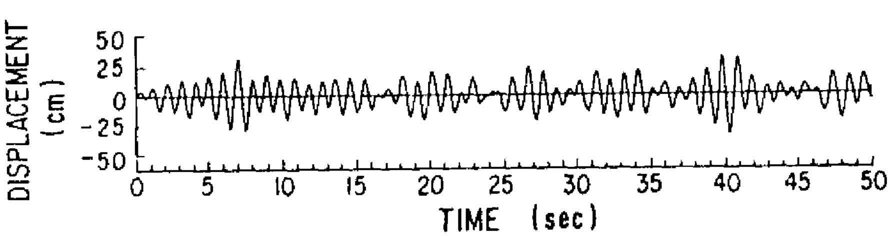


(f) Case 13 (Control when d < 0)

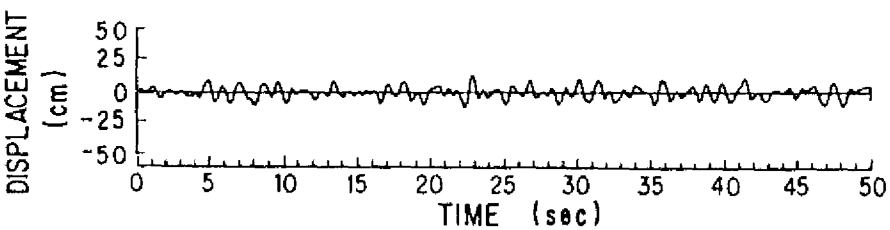


(g) Case 14 (Control when d < 0)

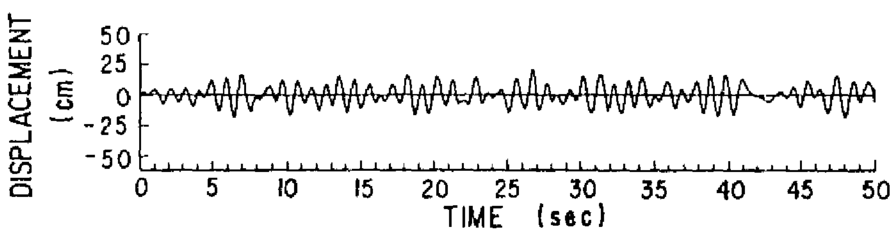
Fig.18 Analytical Cases for Partial Control



(a) Case 1· No Control

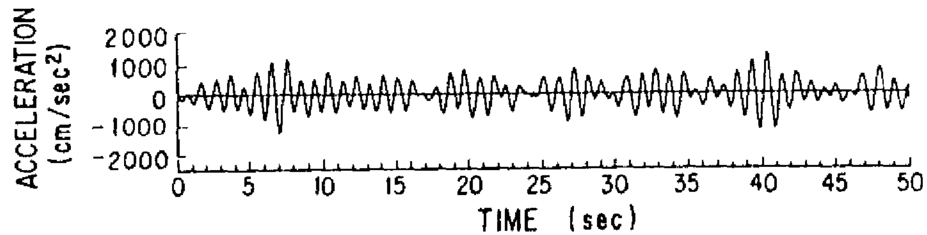


(b) Case 5: Displacement Dependent Damping

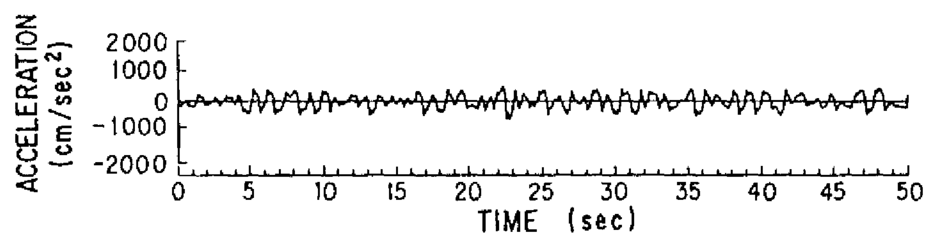


(c) Case 7: Velocity Dependent Damping

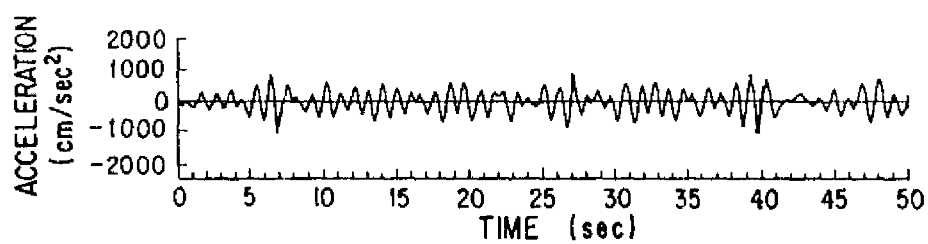
(a) Displacement



(a) Case 1: No Control



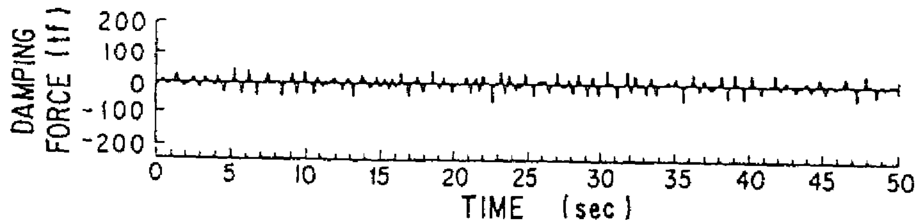
(b) Case 5: Displacement Dependent Damping



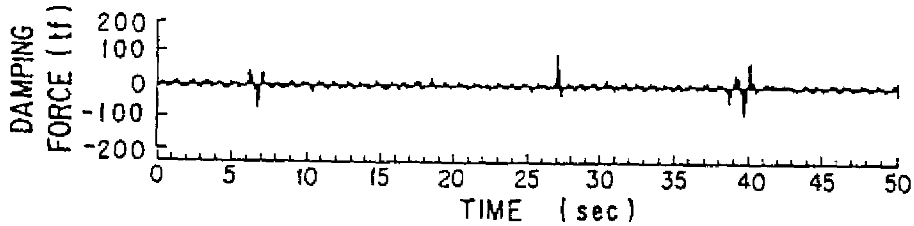
(c) Case 7: Velocity Dependent Damping

(b) Acceleration

Fig.19 Computed Response of Deck

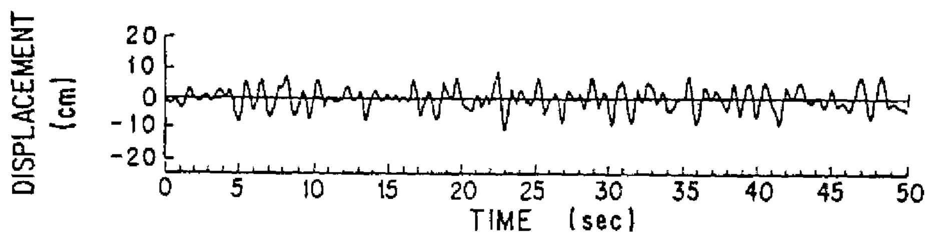


(a) Case 5. Displacement Dependent Damping

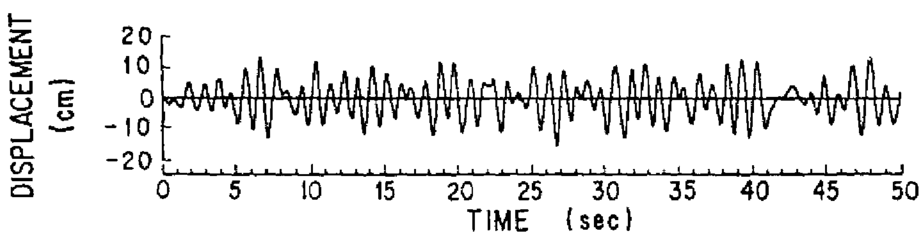


(b) Case 7. Velocity Dependent Damping

(1) Damping Force

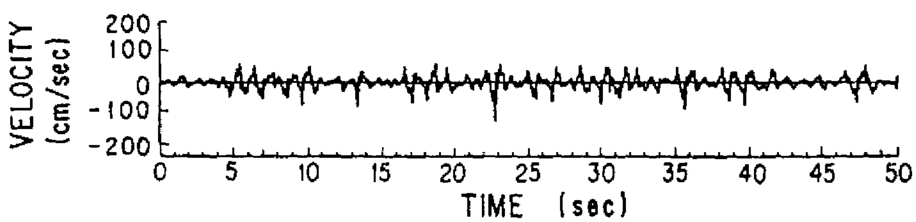


(a) Case 5· Displacement Dependent Damping

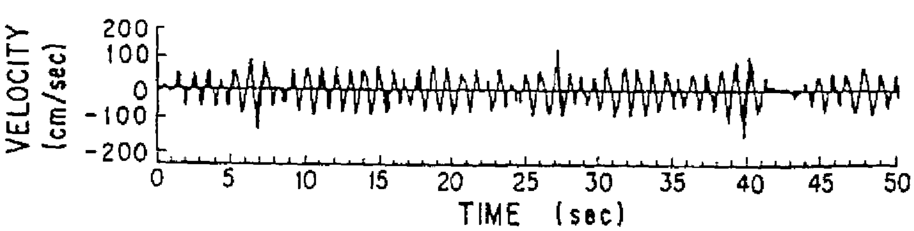


(b) Case 7 Velocity Dependent Damping

(2) Displacement



(a) Case 5: Displacement Dependent Damping



(b) Case 7: Velocity Dependent Damping

(3) Velocity

Fig.20 Computed Response of Variable Dampers