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State University of New York at Buffalo

NCEER-Taisei Corporation Research Program on Sliding Seismic Isolation Systems for Bridges:

Experimental and Analytical Study of a System Consisting of Lubricated PTFE Sliding Bearings and Mild Steel Dampers

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Experimental and Analytical Study of a System Consisting of Lubricated PTFE Sliding Bearings and Mild Steel Dampers

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P. Tsopelas[1] and M.C. Constantinou[2]

July 22, 1994

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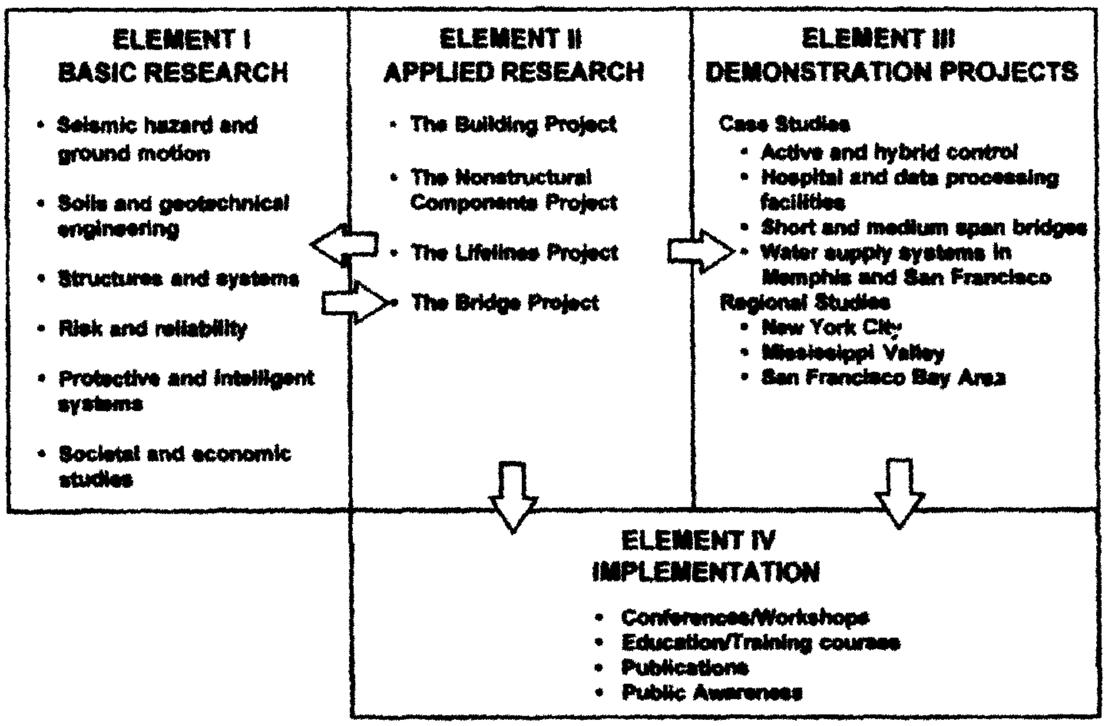
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research tasks in the **Bridge Project** expand current work in the retrofit of existing bridges and develop basic seismic design criteria for eastern bridges in low-to-moderate risk zones. This research parallels an extensive multi-year research program on the evaluation of gravity-load design concrete buildings. Specifically, tasks are being performed to:

1. Determine the seismic vulnerability of bridge structures in regions of low-to-medium seismicity, and in particular of those bridges in the eastern and central United States.
2. Develop concepts for retrofitting vulnerable bridge systems, particularly for typical bridges found in the eastern and central United States.
3. Develop improved design and evaluation methodologies for bridges, with particular emphasis on soil-structure mechanics and its influence on bridge response.
4. Review seismic design criteria for new bridges in the eastern and central United States.

The end product of the **Bridge Project** will be a collection of design manuals, pre-standards and design aids which will focus on typical eastern and central United States highway bridges. Work begun in the **Bridge Project** has now been incorporated into the **Highway Project.**

The **protective and intelligent systems program** constitutes one of the important areas of research in the **Bridge Project**. Current tasks include the following:

1. Evaluate the performance of full-scale active bracing and active mass dampers already in place in terms of performance, power requirements, maintenance, reliability and cost.
2. Compare passive and active control strategies in terms of structural type, degree of effectiveness, cost and long-term reliability.
3. Perform fundamental studies of hybrid control.
4. Develop and test hybrid control systems.

As one part of the NCEER-Taisei Cooperative Research Program on Sliding Seismic Isolation Systems for Bridges, this report presents results from an experimental study on the seismic response of an isolated bridge model and a comparison of its response to that of a comparable non-isolated bridge. The isolation system consisted of lubricated sliding bearings and E-shaped mild steel devices. The bridge model, at quarter length scale, was tested on the University at Buffalo shaking table which provided input motions to the model. The experimental results show that the isolation system is capable of maintaining the forces transmitted to the substructure at a preset limit. This was accomplished, however, at the expense of significant permanent displacements. An analytical model was developed to predict the dynamic response of the system and produced results that are in good agreement with those obtained from the experiments.

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ABSTRACT

This report describes the results of an experimental study of the behavior of a bridge seismic isolation system consisting of lubricated flat sliding bearings and mild steel dampers. Earthquake simulator tests have been performed on a model bridge structure both isolated with this system and non-isolated. The experimental results demonstrate that the system is capable of maintaining the forces transmitted to the substructure at a preset limit, however at the expense of significant permanent displacements. Analytical techniques are used to predict the dynamic response of the system and the obtained results are in good agreement with the experimental results.

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The isolation bearings used in the study were manufactured by Alga SpA. of Milano, Italy.

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SECTION 1

INTRODUCTION

Seismic isolation systems are typified by the use of either elastomeric or sliding bearings. Elastomeric isolation systems have been used in the seismic isolation of buildings in Japan and the United States (Buckle 1990, Soong 1992, Kelly 1993). Several other countries, such as New Zealand and Italy among others, have a number of applications of elastomeric isolation systems in buildings (Buckle 1990, Martelli 1993).

Sliding isolation systems in buildings have been widely used in the former Soviet Union, where over 200 buildings are now seismically isolated (Constantinou 1991a, Eisenberg 1992). In Japan, Taisei Corporation constructed three buildings on the TASS sliding isolation system (Kawamura 1988, Constantinou 1991a). In the United States, sliding isolation systems have recently been selected for the retrofit of three buildings (Soong 1992, Kelly 1993). In particular, spherical sliding or FPS bearings (Zayas 1987, Mokha 1990 and 1991) have been selected for the retrofit of the U.S. Court of Appeals building in San Francisco. This historic structure with a floor area of $31500m^2$, will be, when completed, the largest base-isolated structure in the U.S. and one of the largest in the world (Soong 1992, Palfalvi 1993).

Seismic isolation of bridge structures has been widely implemented in New Zealand and Italy (Buckle 1990, Medeot 1991, Martelli 1993). While in New Zealand the application is exclusively with elastomeric systems, in Italy the application is primarily with sliding systems. Over 150 km of isolated bridge deck in Italy is supported by sliding bearings together with various forms of restoring force and energy dissipation devices (Medeot 1991, Constantinou 1991a).

Japan has over 100 concrete railway bridges of the Shinkansen supported by sliding bearings together with viscous fluid devices, called the KP-stoppers, for restricting displacements within acceptable limits (Buckle 1990, Constantinou 1991a). This system is regarded as an early form of sliding isolation system.

More recently, Japan moved towards a cautious implementation of modern seismic isolation systems in bridges. So far, the application is restricted to only longitudinal isolation using elastomeric systems (Kawashima 1991).

The application of seismic isolation to bridges in the U.S. followed an interesting development. Until 1989, only six bridges were isolated, of which five were retrofit projects in California and one was a new construction in Illinois (Buckle 1990). While the 1989 Loma Prieta earthquake resulted in an accelerated implementation of seismic isolation systems to buildings, this has not been the case in bridges. Rather, we observe a renewed interest and new applications of bridge seismic isolation following the development of specifications for seismic isolation design (ICBO 1991, AASHTO 1991) and the adoption of seismic design guidelines for bridges in the entire U.S. The lack of specifications for the design of seismic isolated structures was regarded as an impediment to the application of the technology (Mayes 1990). Today (March 1994), 57 isolated bridges of total deck length exceeding 11 km are opened to traffic or they are in either the construction or in the design process in the U.S. The isolation system of these bridges consists of either lead-rubber bearings or sliding bearings with restoring force devices and sliding bearings with yielding steel devices. Interestingly, the majority of these bridges are located in the Eastern United States. A recent account of these bridges may be found in Soong and Constantinou, 1994.

While seismic isolation systems found application to over 200 bridges, large scale testing of bridge isolation systems has been so far limited to three studies which concentrated on elastomeric systems (Kelly 1986, Kawashima 1991) and one specific sliding system (Constantinou 1991a). All three studies were restricted to models with rigid piers or abutments and rigid decks. The effects of pier flexibility, pier strength, deck flexibility and distribution of isolation elements could not be studied in these experimental programs. Rather, these effects were studied by analytical techniques and found to be significant (Constantinou 1991a, Kartoum 1992).

The study reported herein was carried out as part of the NCEER-Taisei Corporation research project on bridge seismic isolation systems. This project included the development of advanced sliding isolation systems for bridges and a comprehensive testing program utilizing a flexible pier model. This report concentrates on one of these systems, which consists of lubricated PTFE sliding bearings and E-shaped mild steel dampers. This system and other similar in behavior systems found a number of applications in bridges in Italy. Results for other sliding isolation systems studied under this project have been reported by Constantinou 1993, Tsopelas 1994a and Tsopelas 1994b.

SECTION 2

NCEER-TAISEI CORPORATION RESEARCH PROJECT ON BRIDGE SLIDING SEISMIC ISOLATION SYSTEMS

In 1991, the National Center for Earthquake Engineering Research and Taisei Corporation began a collaborative research project on the development and verification of advanced sliding seismic isolation systems for bridges (Constantinou 1992). The project included also the study of established sliding isolation systems such as the Friction Pendulum (or FPS) system (Zayas 1987, Mokha 1990 and 1991, Constantinou 1993) and the lubricated sliding bearing/hysteretic steel damper system used in a large number of bridges in Italy (Medeot 1991, Marioni 1991).

The project had two portions: one concentrated on active systems and was carried out at Taisei Corporation and Princeton University, and the other concentrated on passive systems and was carried out at the University at Buffalo and Taisei Corporation. The Buffalo/Taisei portion of the project had the objective of producing a class of advanced passive sliding seismic isolation systems by modifying and/or adapting existing technology. Particular emphasis has been given to the adaptation and use of aerospace and military hardware in either the form of restoring force and damping devices or in the form of high performance composite materials in the construction of sliding bearings. The following systems were experimentally studied:

(1) Flat sliding bearings consisting of PTFE or PTFE-based composites in contact with polished stainless steel (coefficient of sliding friction at high velocity of sliding in the range of 0.07 to 0.15) and in combination with

  (a) Rubber restoring force devices,

  (b) Rubber restoring force devices and fluid viscous dampers,

  (c) Wire rope restoring force devices, and

(d) Fluid restoring force/damping devices.

(2) Spherically shaped FPS sliding bearings.

(3) Flat lubricated PTFE-stainless steel sliding bearings in combination with yielding E-shaped mild steel devices.

This report contains the results of the experimental study, interpretation of the results and analytical modeling of systems consisting of lubricated PTFE sliding bearings and E-shaped mild steel dampers.

SECTION 3

ISOLATION SYSTEM

3.1 Description of Isolation System

The tested bearings were scaled versions of bridge isolation bearings developed by Italian engineers and used in a number of bridges in Italy. Approximately 150 bridges in Italy employ some form of sliding system for seismic isolation. An account of these bridges and some details of the isolation systems have been presented by Medeot (1991) and Martelli (1993). Typically, bridge isolation systems in Italy consisted of lubricated PTFE sliding bearings together with some form of energy dissipating device such as fluid dampers or yielding mild steel elements.

One of the most interesting applications is that of the Mortaiolo viaduct on the Livorno-Civitavecchia highway. The tested bearings were scaled versions of bearings used in this structure. Constructed in 1990, the viaduct has an isolated total length of 8 km. It consists of isolated continuous sections, each one of which has length of about 426 m and it is divided into 10 spans of length equal to either 33m or 45m. Each end of the span is supported by piers (Marioni 1991).

A combination of bearings is used in the Mortaiolo viaduct as shown in Figure 3-1. The four end bearings in each 426m long section are multidirectional lubricated PTFE sliding bearings. The two central bearings are also multidirectional lubricated bearings, however equipped with E-shaped mild steel dampers. The bearings in-between the end and central bearings are also equipped with E-shaped steel dampers, however they employ shock transmission units (seismic snappers) which activate the dampers only in motions with velocity exceeding about 1 mm/s. Thus under service loading conditions, each section behaves as a standard bridge which allows thermal expansion about the central fixed bearings. The action of the bearings with shock transmission units is illustrated in Figure 3-2. Under slow longitudinal motion, as in thermal expansion, the E-shaped dampers do not deform and allow for unrestricted movement. Under seismic excitation, the shock

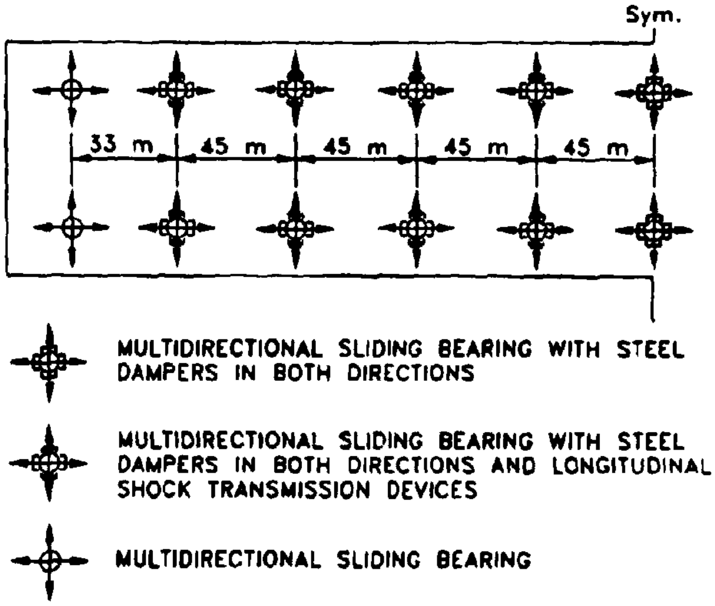


Figure 3-1 Layout of Bearing in Mortaiolo Viaduct.

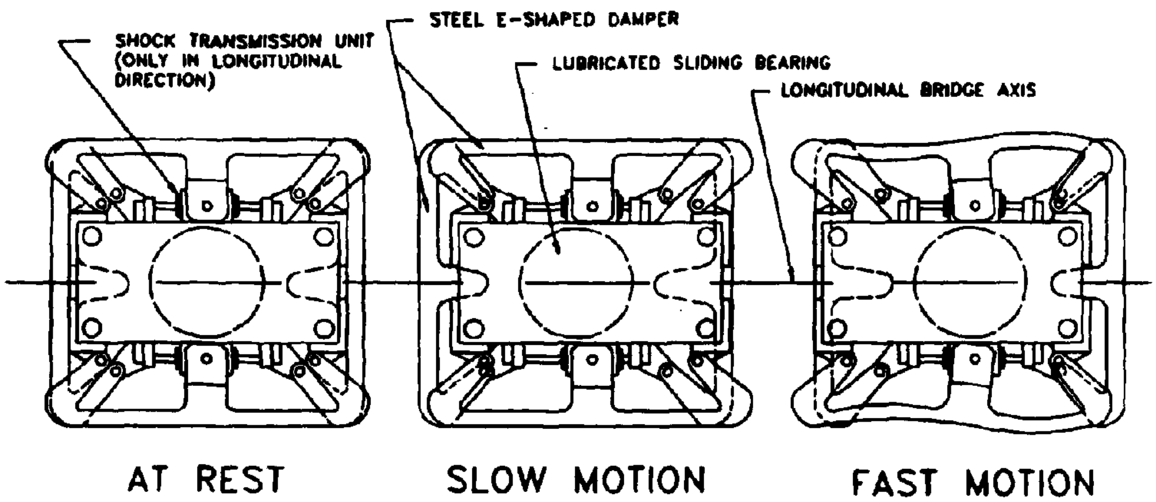


Figure 3-2 Operation of E-shaped Damper under Slow and Dynamic Longitudinal Motion.

transmission units lock and the E-shaped dampers deform as illustrated in Figure 3-2. It should be noted that E-shaped dampers are also installed in the transverse direction, so that isolation is multidirectional (Marioni 1991).

The action of E-shaped damper is double. First is to provide rigidity against service loads at selected locations. Second is to yield and dissipate energy in seismic excitation. Deformation of the E-shaped damper (see Figure 3-2, the central leg moves with respect to the two exterior legs) induces constant bending moment over the entire length of the beam. Plasticization occurs simultaneously throughout the entire volume of the material in the beam, assuming of course that the beam has constant cross section throughout its length. Flexural moments and axial forces have opposite sings in the two portals of the E-shaped damper. These are desirable properties, since effects from geometric changes and accumulation of axial strains are minimized, extending the low cycle fatigue life of the device. The columns of the E-shaped damper are designed such that they remain elastic at all times and to essentially work as lever arms (Ciampi 1991).

The behavior of the E-shaped device is a function of its geometry and material properties. This behavior is nearly elastoplastic with very small post-yielding stiffness. Ciampi 1991 established the principles of operation of the device. Figure 3-3 depicts the geometry of the device and its idealized behavior. The yield force P_Y and yield displacement δ_Y are given by

$$P_Y = \frac{1}{3}\sigma_Y \frac{sb^2}{h} \tag{3-1}$$

$$\delta_Y = \frac{2hl}{b}\epsilon_Y(1 + \frac{a}{3}\frac{h}{l}) \tag{3-2}$$

where $a = 2(\frac{b}{b_1})^3 + (\frac{b}{b_2})^3$ (3-3)

The elastic stiffness K is then obtained as

$$K = \frac{P_Y}{\delta_Y} = \frac{Esb^3}{6h^2 l(1 + \frac{ah}{3l})} \tag{3-4}$$

where σ_Y = yield stress of material, ε_Y = yield strain of material and E = modulus of elasticity.

The plastic moment of the beam of the device is

$$M_p = \sigma_Y \frac{sb^2}{4} \tag{3-5}$$

from where the corresponding force P at the central leg (see Fig. 3-3) is

$$P_{\max} = \frac{\sigma_Y s b^2}{2h} \tag{3-6}$$

Furthermore, the displacement δ of the leg of the device may be related to the surface strain ε of the beam under conditions of full plasticization

$$\delta = \frac{2hl\varepsilon}{b}\left(1 + \frac{ah\varepsilon_Y}{2l\varepsilon}\right) \tag{3-7}$$

The maximum allowed displacement δ_{max} is derived from Equation (3-7) by substituting the allowable maximum value of surface strain ε_{max} for ε. Typically, ε_{max}= 0.03 for the steel used in these devices.

Other parameters of interest are the local ductility, μ_l , and the global ductility, μ , defined by

$$\mu_l = \frac{\varepsilon_{\max}}{\varepsilon_Y} \quad , \qquad \mu = \frac{\delta_{\max}}{Y} \tag{3-8}$$

where Y is the theoretical yield displacement, assumed to be equal to $1.5\ \delta_y$. Chiampi, 1991 showed that

$$\mu = \frac{2}{3}\mu_l \frac{1 + \frac{ah\varepsilon_y}{2l\varepsilon_{max}}}{1 + \frac{ah}{3l}} \approx \frac{2\mu_l}{3\left(1 + \frac{ah}{3l}\right)} \tag{3-9}$$

Typically, the steel used in these devices has local ductility in the range of 15 to 20.

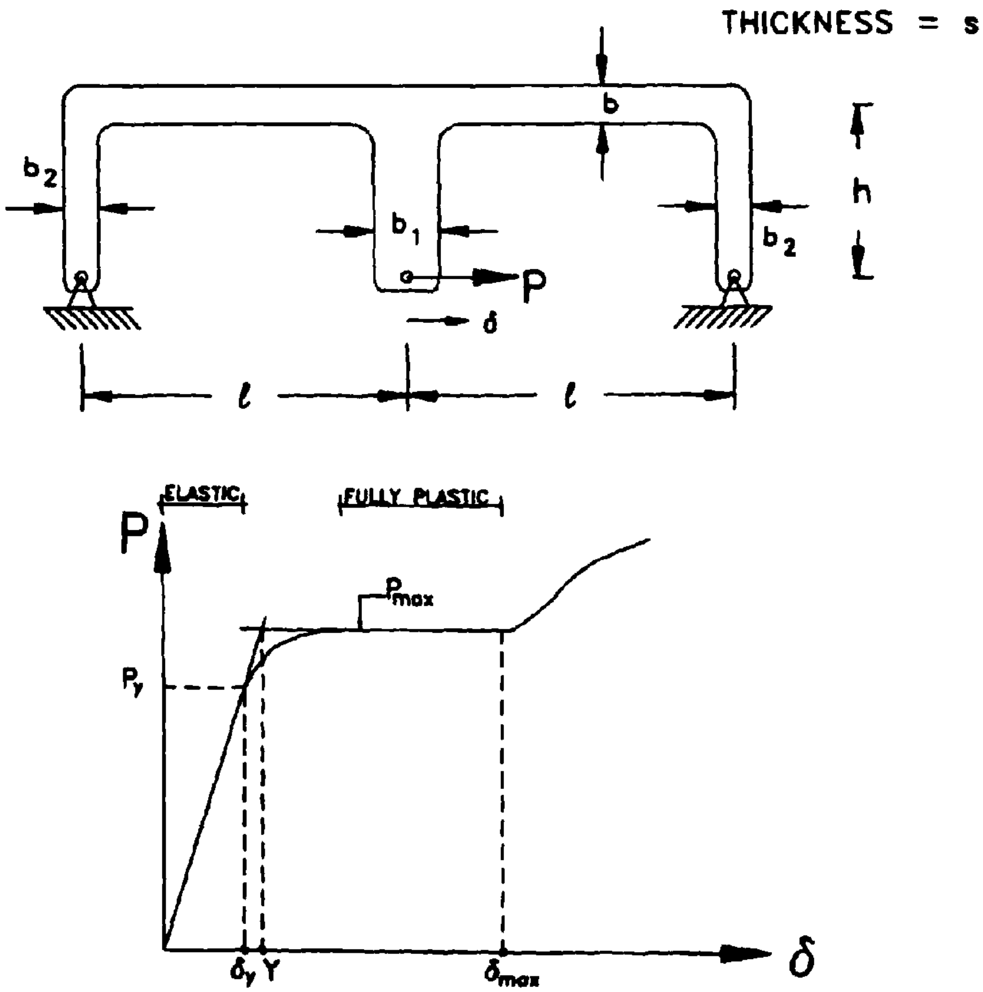


Figure 3-3 Geometry and Idealized Behavior of E-shaped Device.

3.2 Tested Isolation System and Design Requirements

The tested isolation system consisted of the basic elements of the system described in Section 3.1. That is, the system consisted of

(1) Lubricated flat sliding bearings to support the weight of the deck.

(2) Mild steel E-shaped devices for providing the mechanism for energy dissipation.

The specific design requirements of the isolation system were to minimize the transmission of force to the substructure, that is piers and foundation, while bearing displacements in the scale of the model (length scale factor equal to 4) did not exceed 50 mm. These requirements were to be met for seismic motions representative of bridge design spectra in California (CalTrans) (Gates 1979) and in Japan (Level 2) (CERC 1992) for all ground conditions. Furthermore, the performance of the isolated bridge should be better, in terms of transmission of force to the substructure, than a comparable non-isolated bridge under weak seismic excitation, such as the Japanese Level 1 motions (CERC 1992).

The severe requirement on the maximum bearing displacement (50 mm in the scaled model or 200 mm in prototype scale) under strong seismic excitation reflects some design and economic considerations in bridge seismic isolation. A maximum bearing displacement of 200 mm allows the use of short multidirectional expansion joints and eliminates the need for knock-off elements. Short expansion joints are less expensive, require less maintenance and produce less noise on automobile crossing than long ones.

Preliminary analyses showed that these requirements could be met with a design having for each bearing a characteristic strength P_{max} (see Fig. 3-3) equal to 0.16 times the carried weight. This combined with the coefficient of friction in the lubricated bearings would give approximately a characteristic strength of 0.18 times the weight.

The isolation system consisted of four isolators of the design shown in Figure 3-4. Each isolator consisted of two E-shaped dampers and a lubricated sliding pot bearing. The two exterior columns of the E-shaped device were mounted (pin connections) on the top plate of the isolator. The interior column of the device was connected to a slider which was attached to the bottom plate through a piston. The piston contained a rubber disc which provided rotational capability to the isolator. The top plate was faced with a polished stainless steel plate. This plate was in contact with a lubricated PTFE sheet on top of the pot. Continuous lubrication was provided to the sliding interface by grease confined in dimples in the PTFE sheet.

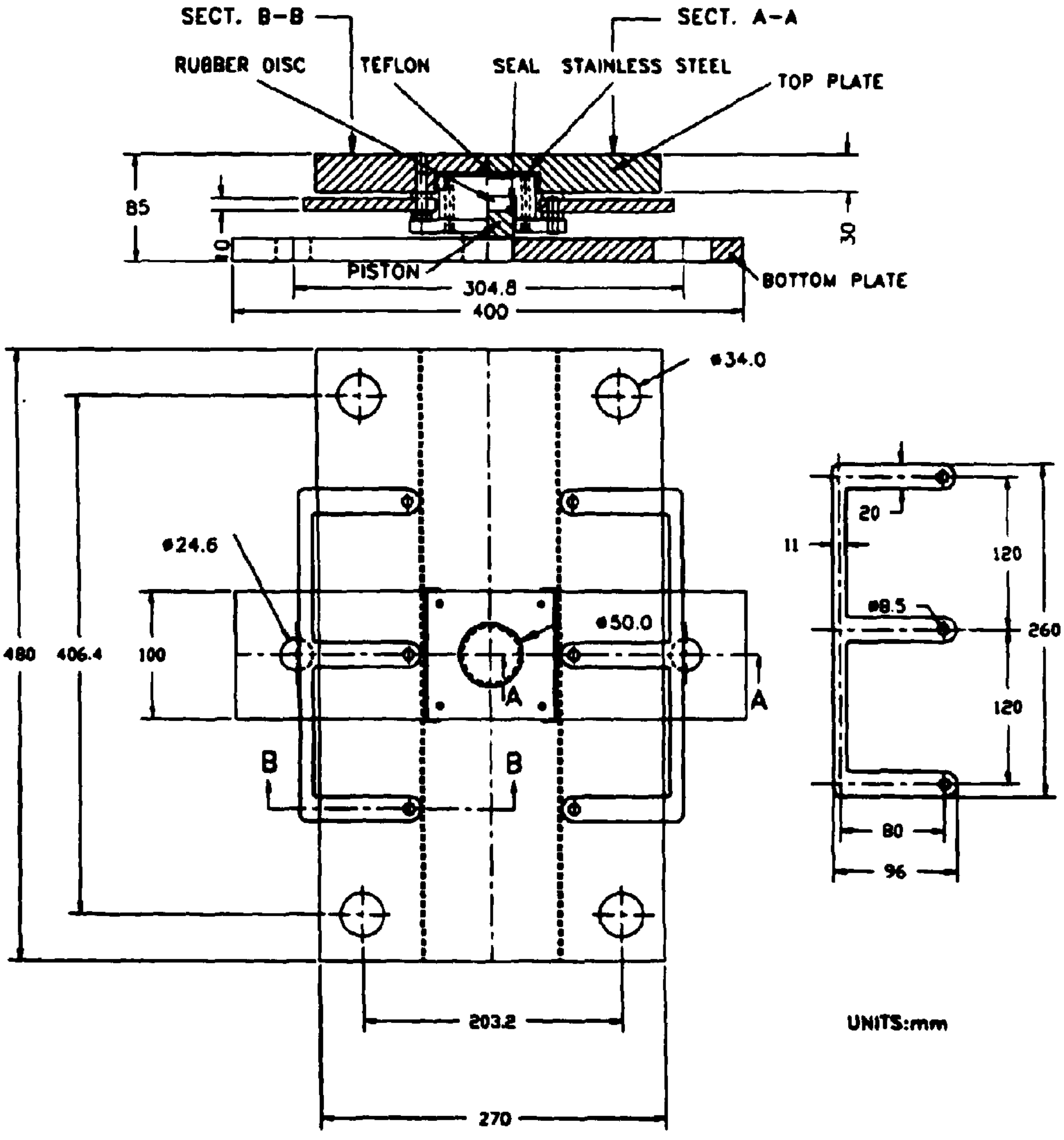


Figure 3-4 Design of Tested Bearing with E-shaped Dampers.

Figure 3-5 shows views of parts of a disassembled bearing. The top view shows the top plate of the bearing with two E-shaped devices. The two exterior legs of each device are pin connected to the plate. The other view shows the slider which is faced with circular recessed PTFE sheet. The sheet has dimples which contain grease.

Figure 3-6 shows views of the installed bearing. The top view shows the sliding interface installed with the stainless steel plate facing down. This is common practice for preventing contamination of the interface. The other view shows the side of the bearing during deformation. The deformation pattern of the E-shaped device is visible.

It should be noted that the tested bearings were not exact replicas of the full-size bearings of Figure 3-2. Rather the tested bearings were uni-directional and were not equipped with shock transmission units.

3.3 Behavior of Isolation System

Identification tests were conducted prior to seismic tests in order to determine the force-displacement characteristics of the isolation system. Furthermore, one identification test was conducted at the end of the seismic test program in order to determine the capability of the bearing to sustain a large number of cycles of motion. For this purpose the piers of the bridge model were braced for increasing their stiffness and the deck was connected to a nearby erected reaction frame, while on the shake table. Load cells monitored the force transmitted by the connection of the deck to the reaction frame (total force transmitted through the isolation system), while the shake table below was driven at specified sinusoidal motion. Furthermore, load cells which supported the isolators monitored the force transmitted through each bearing.

Utilizing experience gained in previous tests using the same apparatus (Constantinou 1993 and Tsopelas 1994a and 1994b), the identification tests were conducted under low frequency motion of 0.03 and 0.1 Hz. In higher frequency testing, the flexibilities of the testing arrangement induced additional high frequency components on the imposed sinusoidal motion, which caused an irregular wavy form in the recorded force-displacement loops.

Figure 3-5 Views of Disassembled Bearing.

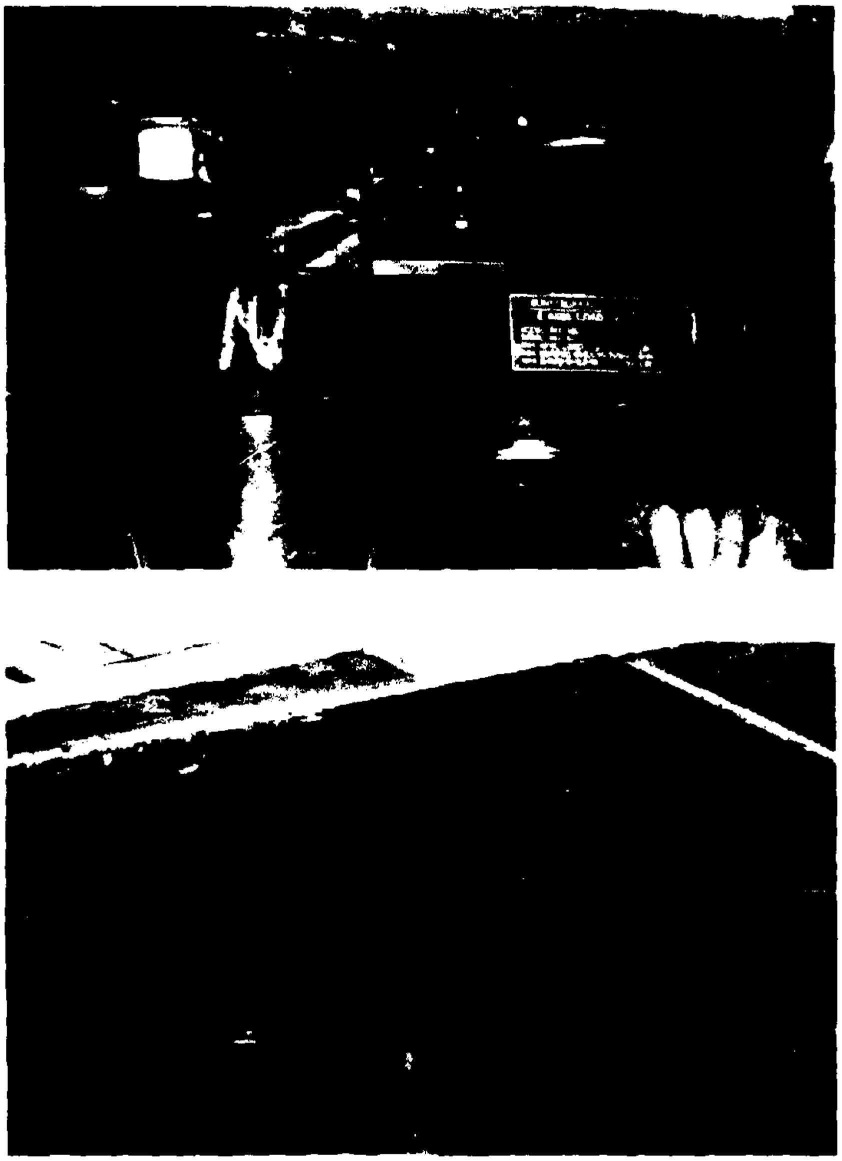


Figure 3-6 Views of Installed Bearing under Deformation.

Figures 3-7 and 3-8 show recorded loops of the force (normalized by the carried weight) vs displacement of the four isolators under sinusoidal excitation at frequency 0.03 Hz and amplitudes of 12.7 and 38.1 mm, respectively. The four bearings exhibit very similar but not identical behavior. The tests indicate that the global yield displacement (Y in Figure 3-3) is approximately equal to 7.5 mm. The maximum force at full plasticization of each bearing is in the range of 6.5 kN to 7 kN, thus in the range of 0.185 to 0.2 of the carried weight. Considering a friction coefficient of 0.01 to 0.02, which is consistent with the experimental results (observe that friction is different at north and south pier location - Fig. 3-8), the force P_{max} at full plasticization (see Section 3.1) is in the range of 0.175 to 0.18 of the carried weight, or 6.1 kN to 6.3 kN. Considering a yield stress of 400 MPa (58 Ksi) and using (3-6), the theoretical value of P_{max} is 6.05 kN (for two E-shaped devices). This is in very good agreement with the experimental results.

Figure 3-9 shows force-displacement loop of the four bearings as recorded in a test conducted after the end of the seismic tests. The amplitude of motion is 50 mm and 25 cycles of motion at 0.1 Hz frequency were imposed. The dampers sustained 25 cycles of motion and exhibited stable properties up to the last cycle in which failure occurred in one of the E-shaped devices of one of the bearings. This was thought to be very good performance. At the displacement of 50 mm the surface strain in the damper is approximately 0.03 (Eq. 3-7). Considering a yield strain $\varepsilon_Y = 0.002$, the local ductility is equal 15 (Eq. 3-8) and the global ductility is about equal to 9 (Eq. 3-9).

It should be noted that the loops exhibit elastoplastic characteristics with some mild slippage. The slippage was caused by a gap at the connection of the central leg of the E-shaped dampers to the sliding part of the isolator. This behavior is not typical of full size isolators. It was rather a manufacturing oversight.

Overall the bearings maintained their properties for the total of about 50 cycles at global ductility of about 10. The bearings exhibit a characteristic strength including friction force of about 7 kN or 0.2 times the carried weight. They had insignificant post-yielding stiffness. The friction coefficient mobilized by the lubricated PTFE sliders was deduced from the identification tests of the isolators. The coefficient of friction at the two south bearings was estimated to be 0.02 and at the two north bearings to be 0.01.

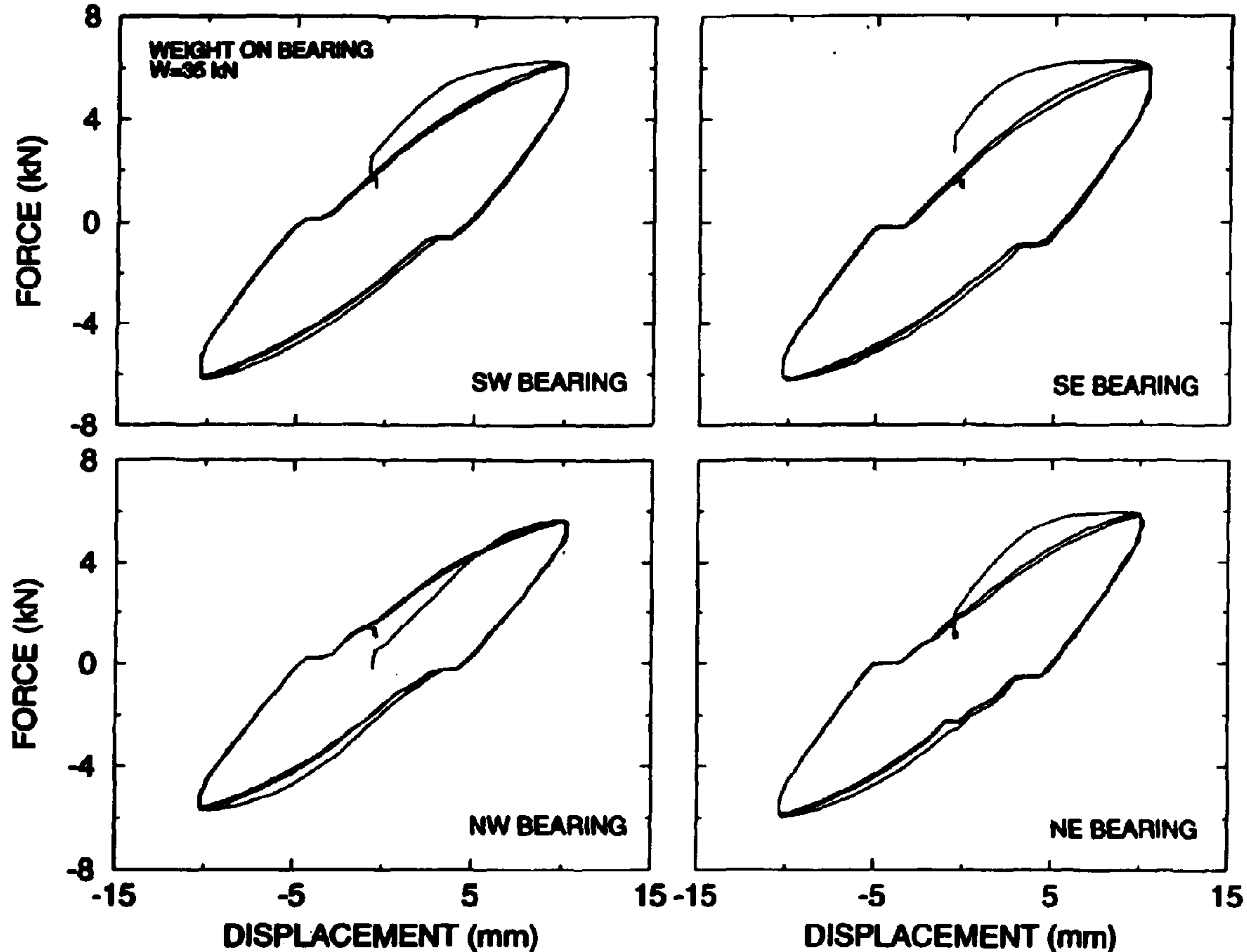


Figure 3-7 Recorded Force-Displacement Loops of Four Bearings under Sinusoidal Motion of 0.03 Hz Frequency and Small Amplitude.

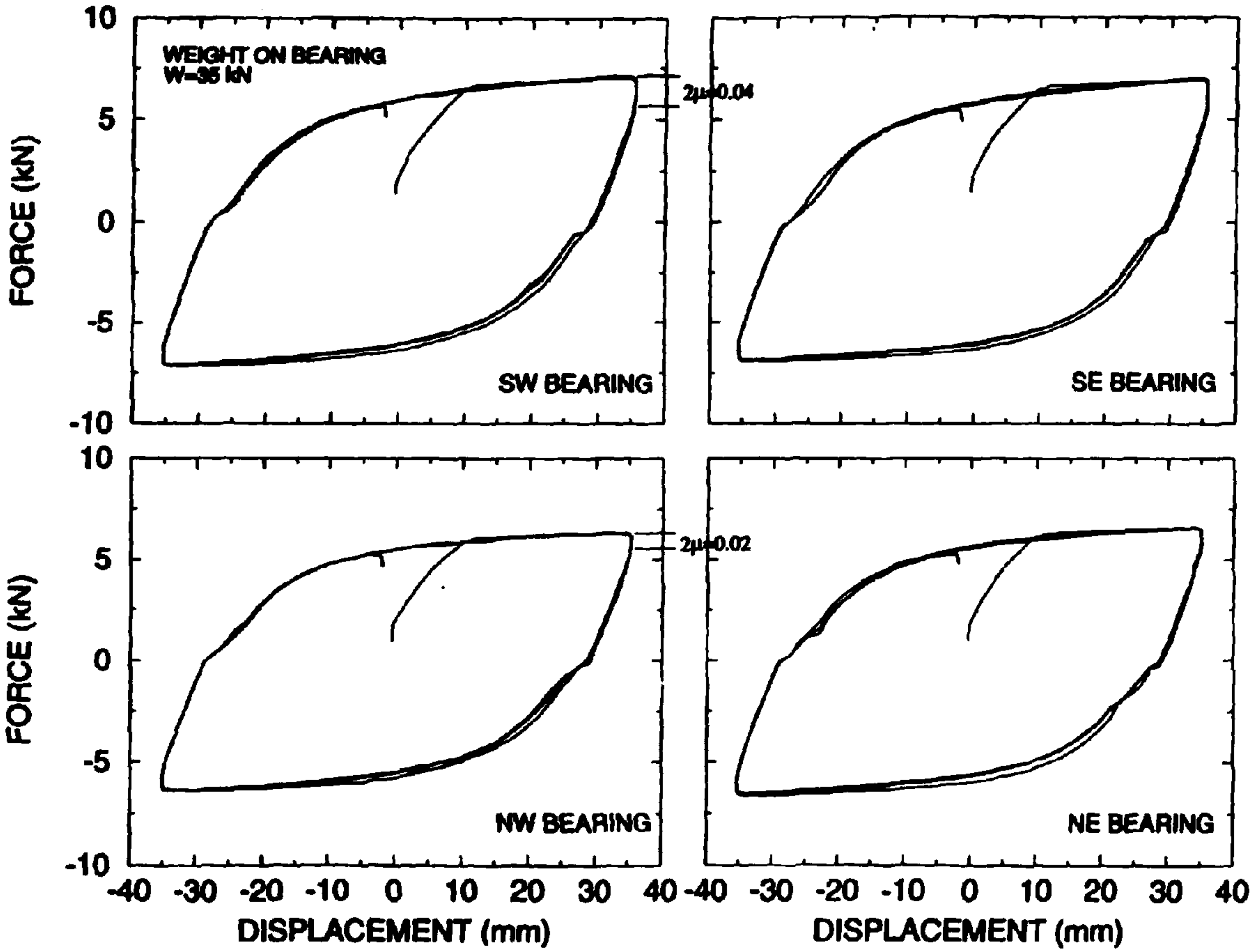


Figure 3-8 Recorded Force-Displacement Loops of Four Bearings under Sinusoidal Motion of 0.03 Hz Frequency and Large Amplitude.

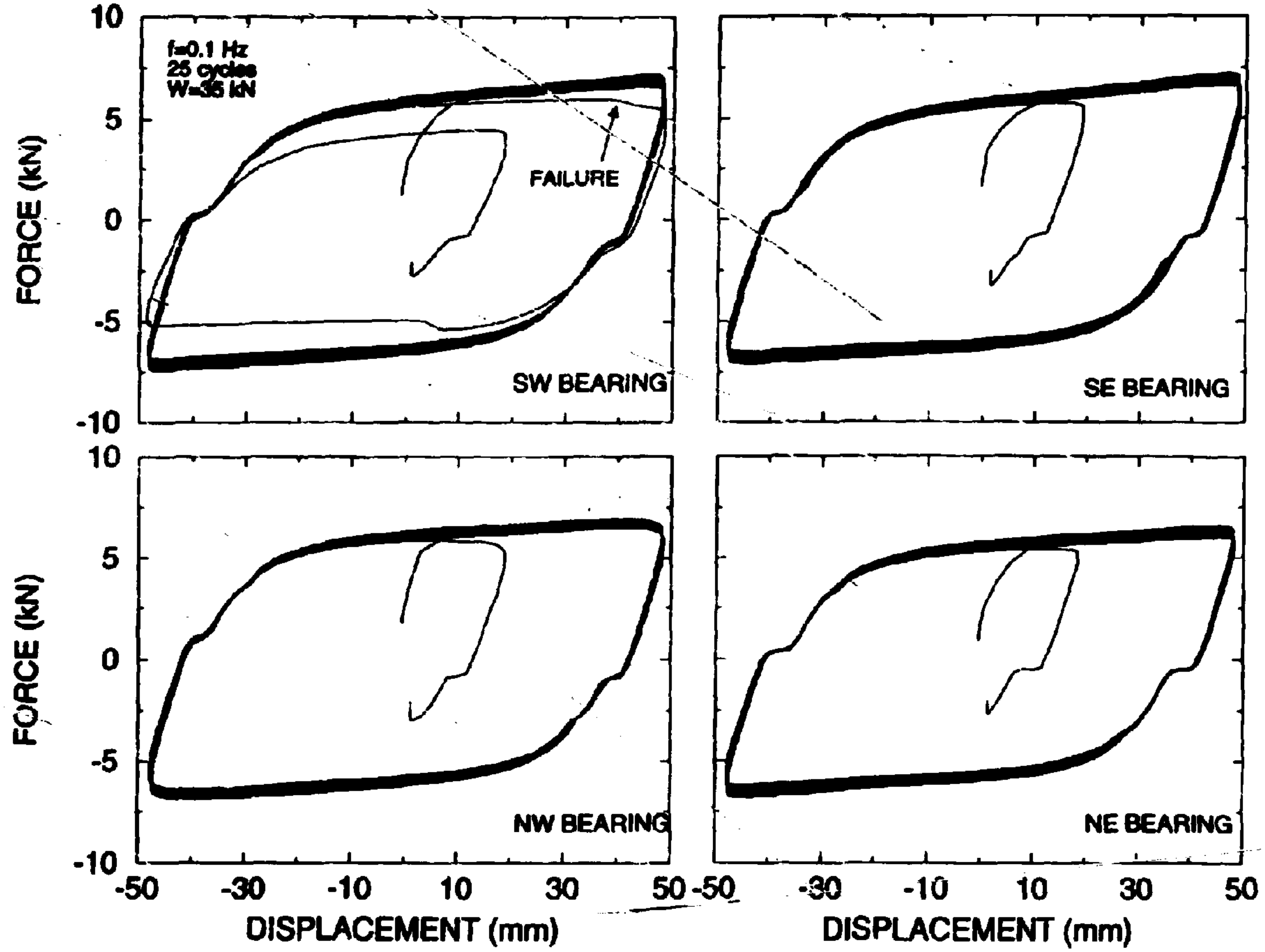


Figure 3-9 Recorded Force-Displacement Loops of Four Bearings in Destructive Test.

SECTION 4

MODEL FOR EARTHQUAKE SIMULATOR TESTING

4.1 Bridge Model

The bridge model was designed to have flexible piers so that under non-isolated conditions the fundamental period of the model in the longitudinal direction is 0.25s (or 0.5s in prototype scale).

The bridge model is shown in Figure 4-1. At quarter length scale, it had a clear span of 4.8m (15.7 feet), height of 2.53m (8.3 feet) and total weight of 157.8 kN (35.5 kips). The deck consisted of two AISC W14x90 sections which were transversely connected by beams. Additional steel and lead weights were added to reach the model deck weight of 140 kN (31.5 kips), as determined by the similitude requirements. Each pier consisted of two AISC TS 6 x 6 x 5/16 columns with a top made of a channel section which was detailed to have sufficient torsional rigidity. The tube columns were connected to beams which were bolted to a concrete extension of the shake table. In this configuration the column loads were transferred at a point located 0.57 m (1.87 ft) beyond the edge of the shake table. While the overhangs of the concrete shake table extension could safely carry the column load of over 80 kN (18 kips), they had some limited vertical flexibility which during seismic testing resulted in vertical motion of the piers and the supported deck.

The piers were designed to have in their free standing cantilever position a period of 0.1 s (0.2 s in prototype scale) when fully loaded (load cells and bottom part of bearings). Furthermore, the piers were detailed to yield under the combined effects of gravity load (40 kN each column) and 50 percent of the gravity load applied as horizontal load at each bearing location. The stiffness of each pier was verified by pulling the piers against each other on the shake table. During the test the piers were also proof-loaded to their rated capacity and the results were used to calibrate the strain gage load cell of each column.

Identification of the model was conducted by exciting the shake table with a 0-20 Hz banded white noise of 0.03g peak acceleration. Acceleration transfer functions of each

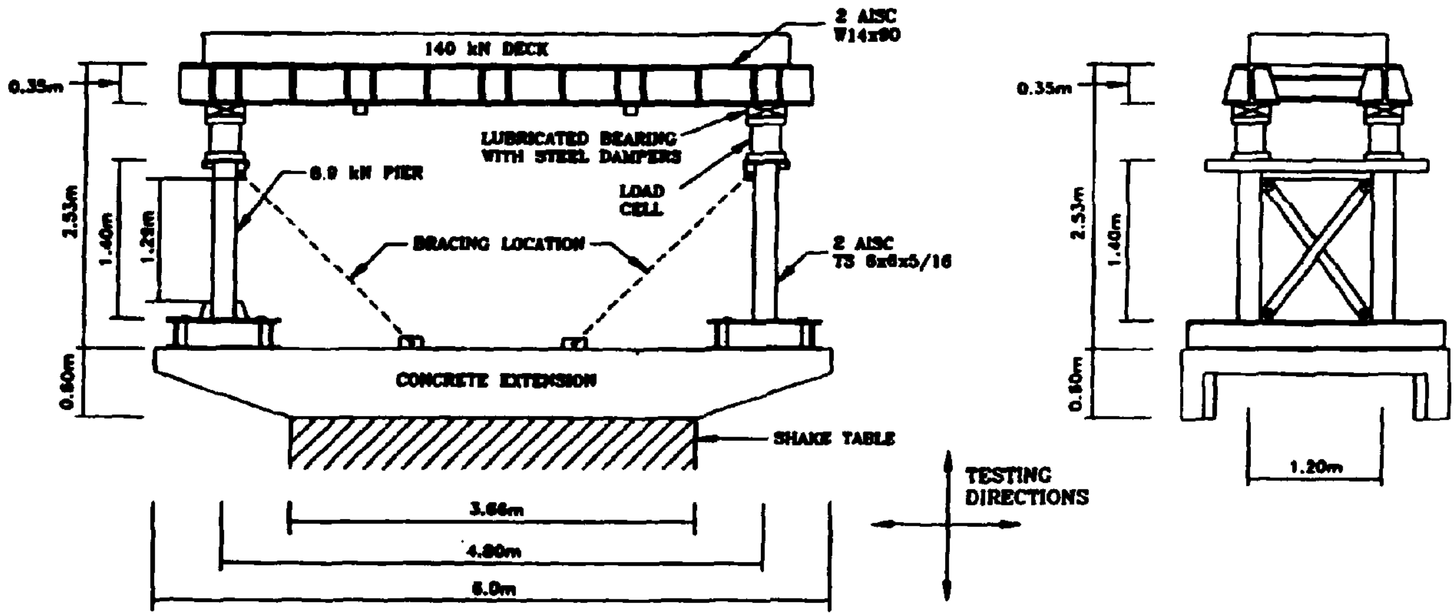


Figure 4-1 Schematic of Quarter Scale Bridge Model.

free standing pier and of the assembled bridge model with all bearings fixed against translational movement (but not rotation) revealed the following properties: fundamental period of free standing pier equal to 0.096s and fundamental period of non-isolated bridge in the longitudinal direction equal to 0.26s. These values are in excellent agreement with the design values of 0.1s and 0.25s, respectively.

Damping in the model was estimated to be 0.015 of critical for the free standing piers and 0.02 of critical for the entire model in its non-isolated condition. Identification tests of the model were also conducted with white noise input of 0.1g peak table acceleration to obtain a fundamental period of 0.25s and corresponding damping ratio of 0.04 of critical. The increased damping was the result of hysteretic action, not in the columns of the model but in the overhangs of the concrete extension of the shake table. During shake table testing of the non-isolated model, the recorded loops of shear force versus displacement of the piers displayed hysteretic action (see Section 5). Estimates of damping ratio from these loops were in the range of 0.04 to 0.08 of critical. Thus while the columns of the piers remained elastic, the pier system displayed realistic hysteretic action with equivalent damping ratio of at least 5 percent of critical.

The design of the model bridge was based in the similitude laws for artificial mass simulation (Sabnis 1983). A summary of the scale factors in the model is presented in Table 4-I.

4.2 Instrumentation

The instrumentation consisted of load cells, accelerometers and displacement transducers. Figure 4-2 shows the overall instrumentation diagram, whereas Figures 4-3 and 4-4 show the instrumentation diagrams for accelerometers and displacement transducers, respectively. A list of monitored channels and their corresponding descriptions are given in Table 4-II. A total of 51 channels were monitored.

Table 4-1 : Summary of Scale Factors in Bridge Model

QUANTITY	DIMENSION	SCALE FACTOR[1]
Linear Dimension	L	4
Displacement	L	4
Velocity	LT^{-1}	2
Acceleration	LT^{-2}	1
Time	T	2
Frequency	T^{-1}	0.5
Force	F	16
Pressure	FL^{-2}	1
Strain	---	1

[1] PROTOTYPE/MODEL

4.3 Test Configurations

Testing of the bridge model was performed in four different bridge configurations. Figure 4-5 shows the four bridge configurations. They were :

(1) The sliding bearings were locked by side plates to represent a non-isolated bridge. In this configuration, the structure was identified in tests with banded white noise table motion. Furthermore, a selected number of seismic tests was conducted.

(2) Braces were installed to stiffen the piers and the deck was connected by stiff rods to a nearby reaction frame. In this configuration, the shake table was driven in displacement-controlled mode with specified frequency and amplitude of harmonic motion. This motion was nearly the motion experienced by the bearings. Loops of bearing horizontal force versus bearing displacement were recorded and used to extract the properties of the bearings.

(3) Both piers were stiffened by braces so that they represented stiff abutments In this configuration, the model resembled a single span isolated bridge (see Figure 4-6).

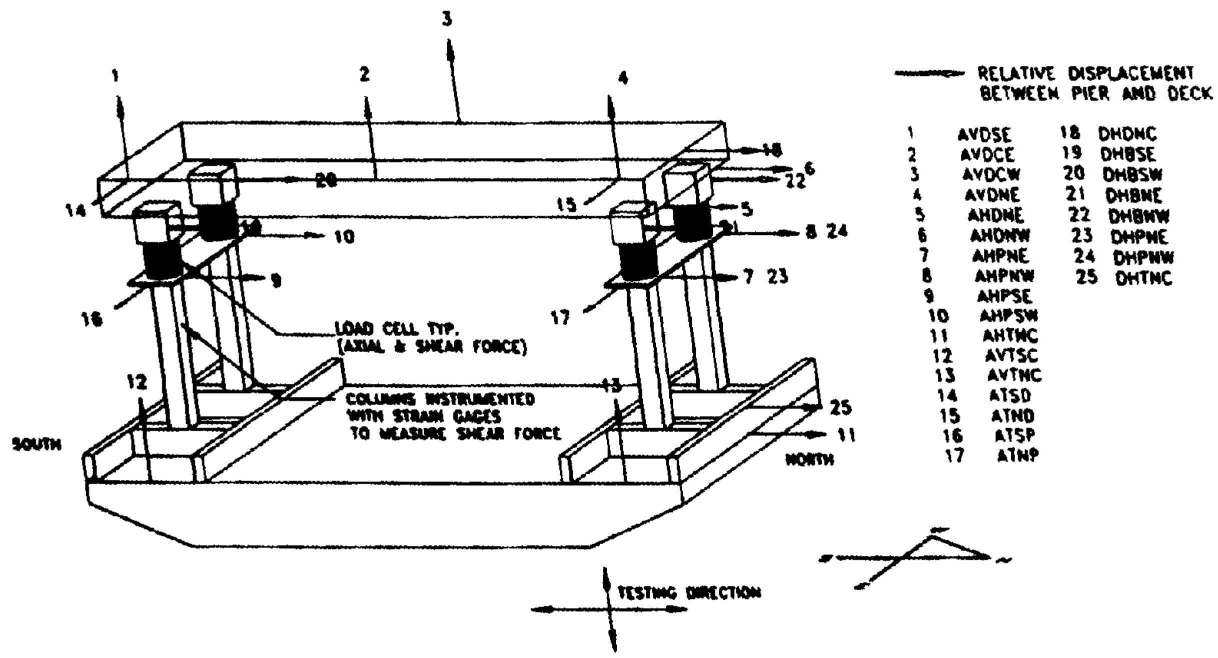


Figure 4-2 Overall Instrumentation Diagram.

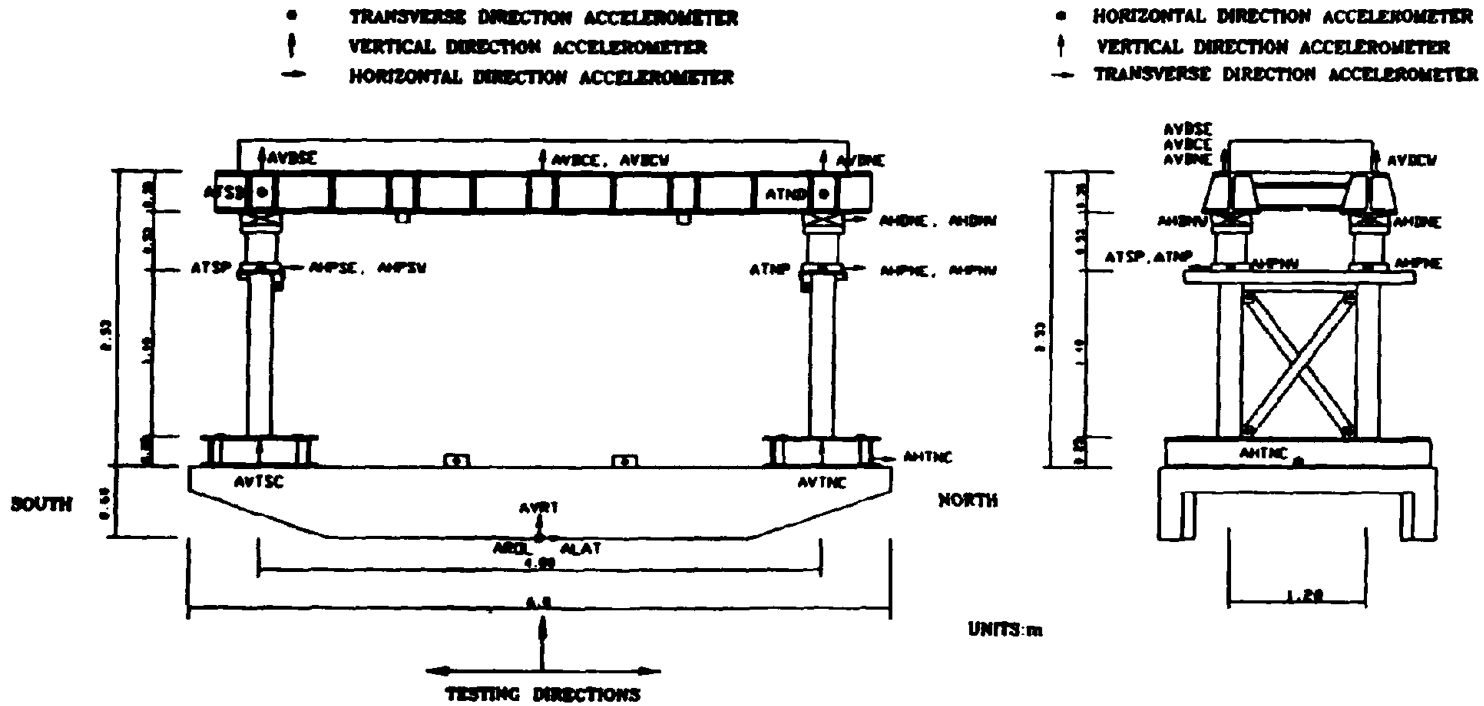


Figure 4-3 Location of Accelerometers (Units:m)

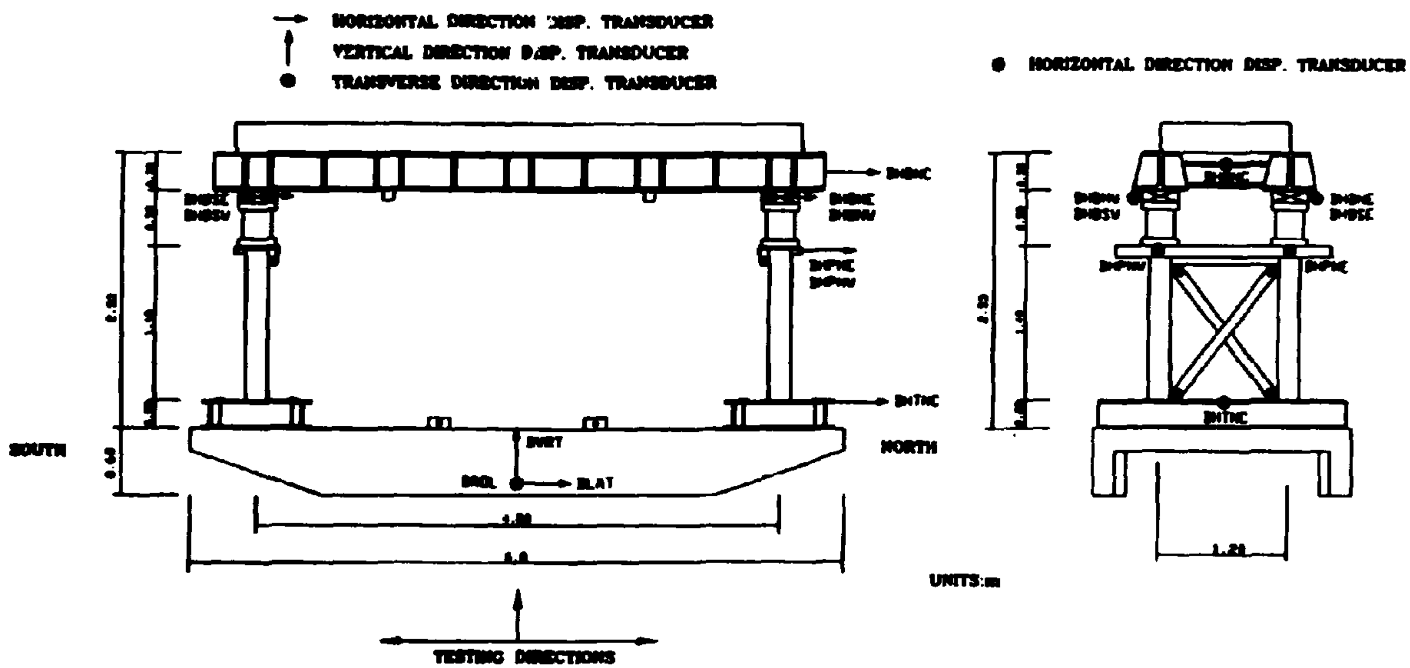


Figure 4-4 Location of Displacement Transducers (Units:m)

Table 4-II List of Channels (with reference to Figures 4-2 to 4-4)

CHANNEL	NOTATION	INSTRUMENT	RESPONSE MEASURED
1	AVDSE	ACCL	Deck Vertical Accel.-South East Corner
2	AVDCE	ACCL	Deck Vertical Accel.-East Side at Center
3	AVDCW	ACCL	Deck Vertical Accel.-West Side at Center
4	AVDNE	ACCL	Deck Vertical Accel.-North East Corner
5	AHDNE	ACCL	Deck Horizontal Accel.-North East Corner
6	AHDNW	ACCL	Deck Horizontal Accel.-North West Corner
7	AHPNE	ACCL	Pier Horizontal Accel.-North East
8	AHPNW	ACCL	Pier Horizontal Accel.-North West
9	AHPSE	ACCL	Pier Horizontal Accel.-South East
10	AHPSW	ACCL	Pier Horizontal Accel.-South West
11	AHTNC	ACCL	Table Horizontal Accel.-North Side at Center
12	AVTSC	ACCL	Table Vertical Accel.-South Side at Center
13	AVTNC	ACCL	Table Vertical Accel.-North Side at Center
14	ATSD	ACCL	Deck Transverse Accel.-South Side
15	ATND	ACCL	Deck Transverse Accel.-North Side
16	ATSP	ACCL	Pier Transverse Accel.-South
17	ATNP	ACCL	Pier Transverse Accel.-North
18	DHDNC	DT	Deck Total Horizontal Displ.-North Side Center
19	DHBSE	DT	Bearing Horizontal Displ.-South East
20	DHBSW	DT	Bearing Horizontal Displ.-South West
21	DHBNE	DT	Bearing Horizontal Displ.-North East
22	DHBNW	DT	Bearing Horizontal Displ.-North West
23	DHPNE	DT	Pier Total Horizontal Displ.-North East
24	DHPNW	DT	Pier Total Horizontal Displ.-North West
25	DHTNC	DT	Table Horizontal Displ.-North Side at Center
26	DHBAV	DT	Bearing Horizontal Average Displ.
27	DLAT	DT	Table Horizontal Displ.
28	ALAT	ACCL	Table Horizontal Accel.
29	DVRT	DT	Table Vertical Displ.
30	AVRT	ACCL	Table Vertical Accel.
31	DROL	DT	Table Rolling Displ.
32	AROL	ACCL	Table Rolling Accel.
33	SX1	LOAD CELL	Shear Bearing Force-South West
34	SX2	LOAD CELL	Shear Bearing Force-South East

ACCEL=Accelerometer, DT=Displacement Transducer

Table 4-II (Cont'd)

CHANNEL	NOTATION	INSTRUMENT	RESPONSE MEASURED
35	SX3	LOAD CELL	Shear Bearing Force-North West
36	SX4	LOAD CELL	Shear Bearing Force-North East
37	SCNE	LOAD CELL	Column Shear Force-North East
38	SCSE	LOAD CELL	Column Shear Force-South East
39	SCNW	LOAD CELL	Column Shear Force-North West
40	SCSW	LOAD CELL	Column Shear Force-South West
41	N1SW	LOAD CELL	Axial Bearing Force-South West
42	N2SE	LOAD CELL	Axial Bearing Force-South East
43	N3NW	LOAD CELL	Axial Bearing Force-North West
44	N4NE	LOAD CELL	Axial Bearing Force-North East
45	SCN	LOAD CELL	Average Column Shear Force-North
46	SCS	LOAD CELL	Average Column Shear Force-South
47	DHDSW	DT	Deck Total Horizontal Displ.-South West Corner
48	DHDSE	DT	Deck Total Horizontal Displ.-South East Corner
49	LCNE	LOAD CELL	East Friction Force-North East Corner(ID-test)
50	LCNW	LOAD CELL	West Friction Force-North West Corner(ID-Test)
51	LCTOT	LOAD CELL	Average Friction Force(ID-Test)

ACCEL=Accelerometer, DT=Displacement Transducer

(4) A configuration with two flexible piers which resembled portion of a multiple span bridge between expansion joints. A view of this configuration on the shake table is shown in Figure 4-7.

A total of 16 seismic tests were conducted in the isolated bridge configurations listed in Table 4-III.

Table 4-III Bridge and Isolation System Configurations

TEST No.	NUMBER OF TESTS	PIER CONDITION		SLIDING BEARINGS (Type)		E - SHAPED DAMPING DEVICES (Number)	
		SOUTH	NORTH	SOUTH PIER	NORTH PIER	SOUTH PIER	NORTH PIER
ITBR01-07	7	STIFF	STIFF	Lubr. PTFE	Lubr. PTFE	4	4
ITBR08-16	9	FLEXIBLE	FLEXIBLE	Lubr. PTFE	Lubr. PTFE	4	4

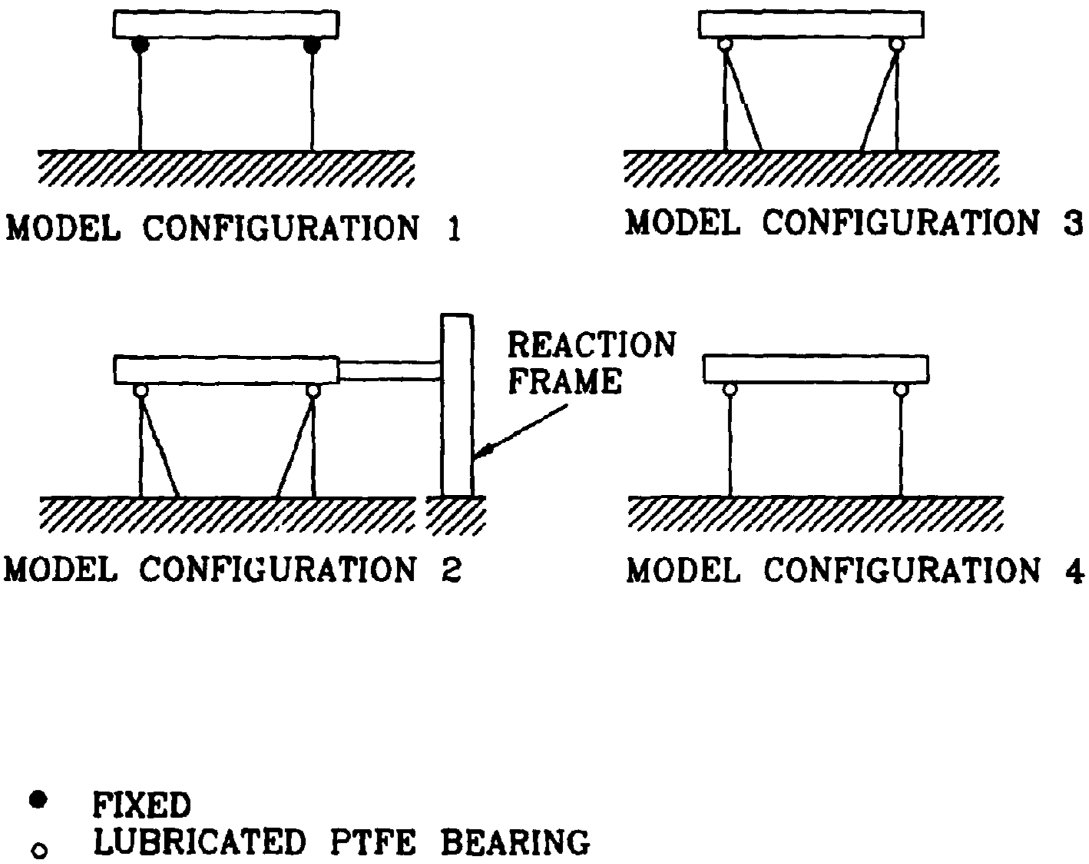


Figure 4-5 Model Configurations in Testing (1:Non-isolated Bridge, 2:Identification of Bearing Properties, 3:Single Span Model, 4:Multiple Span Model).

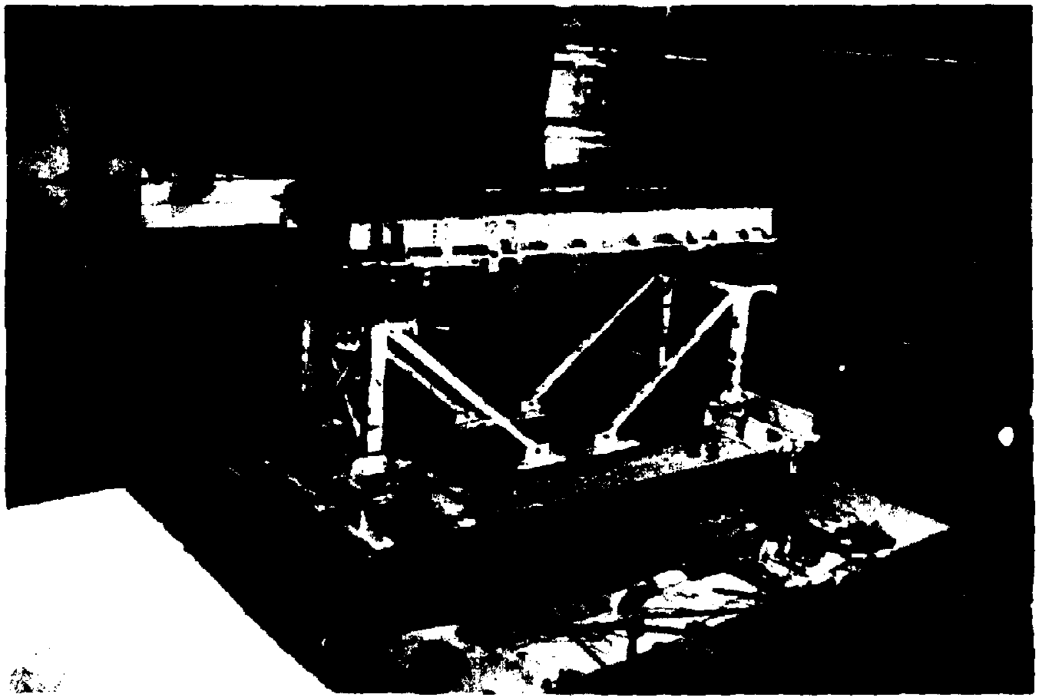


Figure 4-6 View of Bridge Model in Configuration with Two Stiff Piers

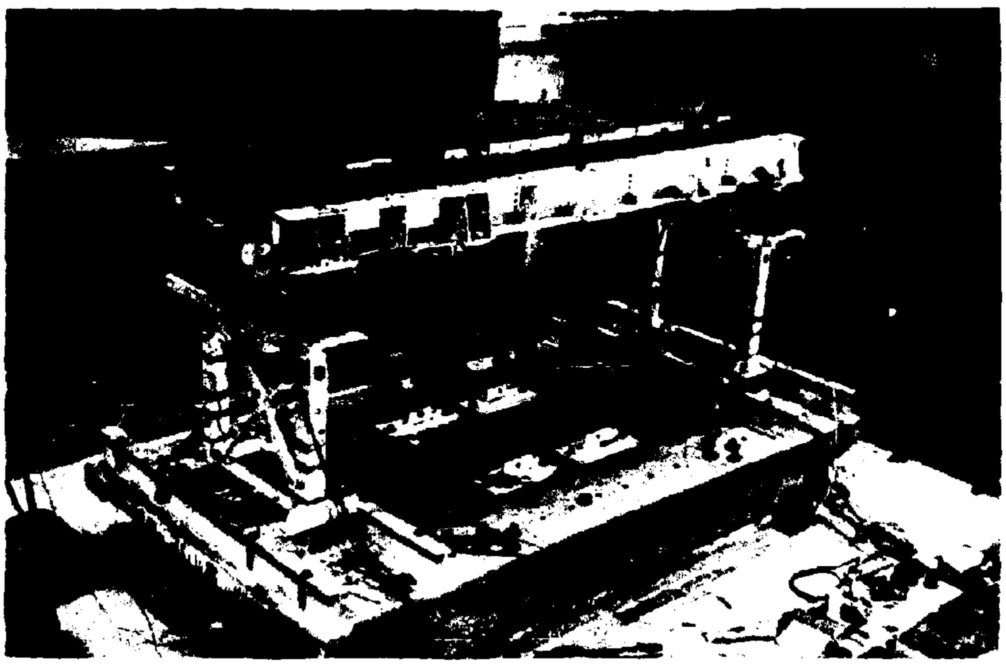


Figure 4-7 View of Bridge Model in Configuration with two Flexible Piers

4.4 Test Program

A total of 16 earthquake simulation tests were performed on the isolated model bridge and another 18 tests on the non-isolated bridge. Tests were conducted with only horizontal input. The earthquake signals and their characteristics are listed in Table 4-IV. The earthquake signals consisted of historic earthquakes and artificial motions compatible with:

(a) The Japanese bridge design spectra for Level 1 and 2 and ground conditions 1 (rock), 2 (alluvium) and 3 (deep alluvium) (CERC 1992). In Japan, it is required that bridges are designed for two levels of seismic loading. In Level 1 seismic loading, it is required that the bridge remains undamaged and fully elastic. In Level 2 seismic loading, inelastic behavior is permitted. Tables 4-V and 4-VI describe the shapes of the 5%-damped acceleration spectra of the Japanese Level 1 and 2 motions.

(b) The California Department of Transportation (CalTrans) bridge spectra (Gates 1979). These motions were identical to those used in the testing of another bridge model by Constantinou, 1991a.

Each record was compressed in time by a factor of two to satisfy the similitude requirements. Figure 4-8 to 4-14 show recorded time histories of the table motion in tests with input being those earthquake signals of Table 4-IV which were used in the testing of the isolated bridge. The acceleration and displacement records were directly measured, whereas the velocity record was obtained by numerical differentiation of the displacement record. It may be observed that the peak ground motion was reproduced well, but not exactly, by the table generated motion.

Figures 4-8 to 4-14 also show the response spectra of acceleration of the table motions. The 5% damped acceleration spectrum is compared to the spectrum of the target record to demonstrate the good reproduction of the motion by the table.

Table 4-IV Earthquake Motions Used in Test Program and Characteristics in Prototype Scale

NOTATION	RECORD	PEAK ACC. (g)	PEAK VEL. (mm/sec)	PEAK DIS. (mm)
EL CENTRO S00E	Imperial Valley, May 18 1940, Component S00E	0.34	334.5	108.7
TAFT N21E	Kern County, July 21,1952, Component N21E	0.16	157.2	67.1
MEXICO N90W	Mexico City, September 19, 1985 SCT building, Component N90W	0.17	605.0	212.0
PACOIMA S16E	San Fernando, February 9, 1971, Component S16E	1.17	1132.3	365.3
PACOIMA S74W	San Fernando, February 9, 1971, Component S74E	1.08	568.2	108.2
HACHINOHE N-S	Tokachi, Japan, May 16, 1968 Hachinohe, Component N-S	0.23	357.1	118.9
MIYAGIKEN OKI	Miyaki, Japan, June 12, 1978 Ofunato-Bochi, Component E-W	0.16	141.0	50.8
AKITA N-S	Nihonkai Chuubu, Japan, May 23, 1983 Component N-S	0.19	292.0	146.0
JP. L1G1	Artificial Compatible with Japanese Level 1 Ground Condition 1	0.10	215.0	90.0
JP. L1G2	Artificial Compatible with Japanese Level 1 Ground Condition 2	0.12	251.0	69.0
JP. L1G3	Artificial Compatible with Japanese Level 1 Ground Condition 3	0.14	274.0	132.0
JP. L2G1	Artificial Compatible with Japanese Level 2 Ground Condition 1	0.37	864.0	526.0
JP. L2G2	Artificial Compatible with Japanese Level 2 Ground Condition 2	0.43	998.0	527.0
JP. L2G3	Artificial Compatible with Japanese Level 2 Ground Condition 3	0.45	1121.0	700.0
CALTRANS 0.6g A2	Artificial Compatible with CalTrans 0.6g 80'-150'Alluvium Spectrum, No.2	0.60	836.4	282.9
CALTRANS 0.6g S3	Artificial Compatible with CalTrans 0.6g 10'-80'Alluvium Spectrum, No.3	0.60	778.0	438.9
CALTRANS 0.6g R1	Artificial Compatible with CalTrans 0.6g Rock Spectrum, No.1	0.60	571.0	342.4
CALTRANS 0.6g R3	Artificial Compatible with CalTrans 0.6g Rock Spectrum, No.3	0.60	571.0	342.4

Table 4-V Spectral Acceleration of Japanese Bridge Design Spectra, Level 1

G.C.	Spectral Acceleration (S_{10}) in units of cm/sec^2 as Function of Period T_i in units of seconds		
1	$T_i < 0.1$ $S_{10} = 431T_i^{1/3}$ $S_{10} \geq 160$	$0.1 \leq T_i \leq 1.1$ $S_{10}=200$	$1.1 < T_i$ $S_{10}=220/T_i$
2	$T_i < 0.2$ $S_{10} = 427T_i^{1/3}$ $S_{10} \geq 200$	$0.2 \leq T_i \leq 1.3$ $S_{10}=250$	$1.3 < T_i$ $S_{10}=325/T_i$
3	$T_i < 0.34$ $S_{10} = 430T_i^{1/3}$ $S_{10} \geq 240$	$0.34 \leq T_i \leq 1.5$ $S_{10}=300$	$1.5 < T_i$ $S_{10}=450/T_i$

Table 4-VI Spectral Acceleration of Japanese Bridge Design Spectra, Level 2

G.C.	Spectral Acceleration (S_{20}) in units of cm/sec^2 as Function of Period T_i in units of seconds		
1	$T_i \leq 1.4$ $S_{20}=700$		$1.4 < T_i$ $S_{20}=980/T_i$
2	$T_i < 0.18$ $S_{20} = 1506T_i^{1/3}$ $S_{20} \geq 700$	$0.18 \leq T_i \leq 1.6$ $S_{20}=850$	$1.6 < T_i$ $S_{20}=1360/T_i$
3	$T_i < 0.29$ $S_{20} = 1511T_i^{1/3}$ $S_{20} \geq 700$	$0.29 \leq T_i \leq 2.0$ $S_{20}=1000$	$2.0 < T_i$ $S_{20}=2000/T_i$

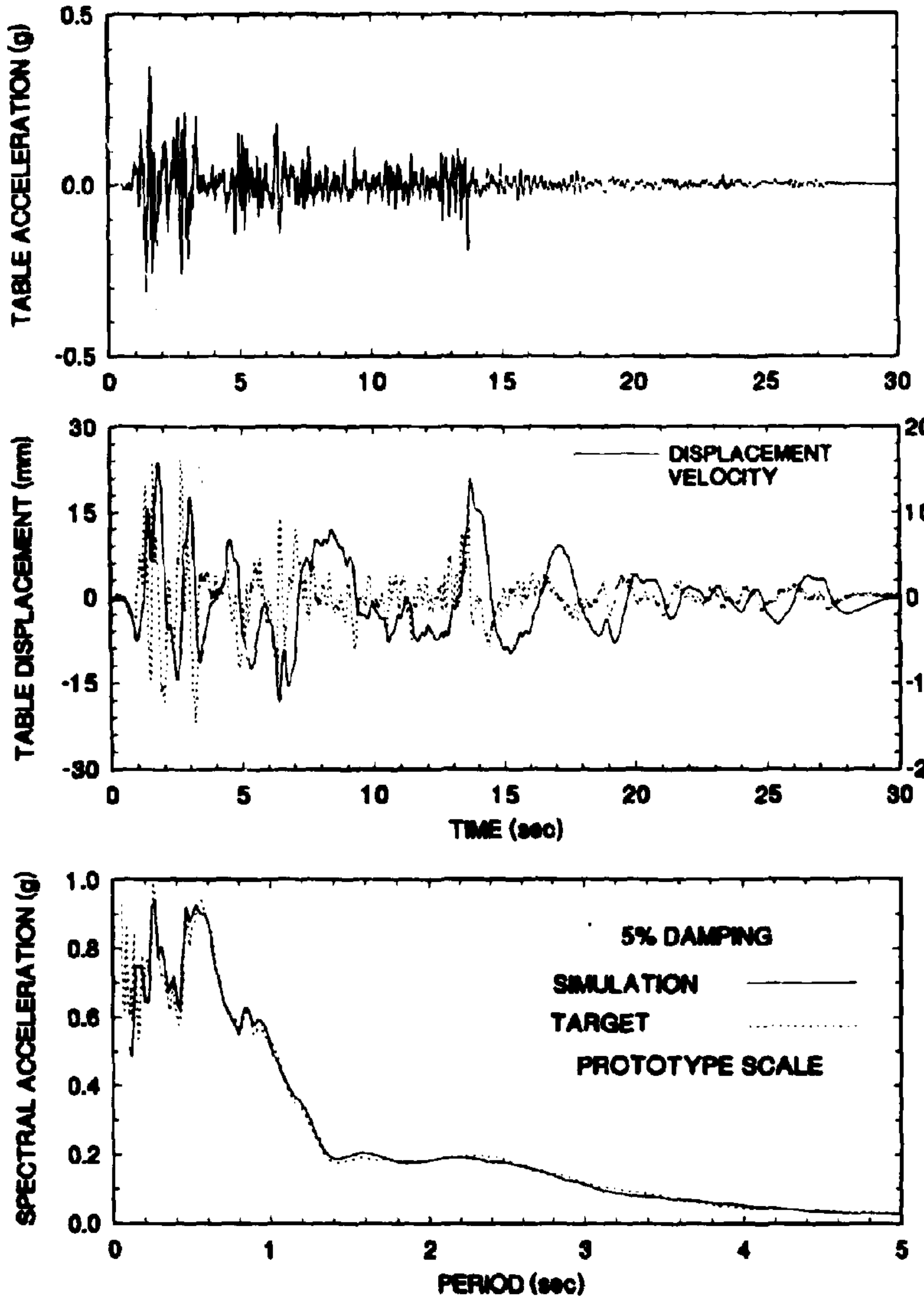


Figure 4-8 **Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with El Centro S00E 100% Motion.**

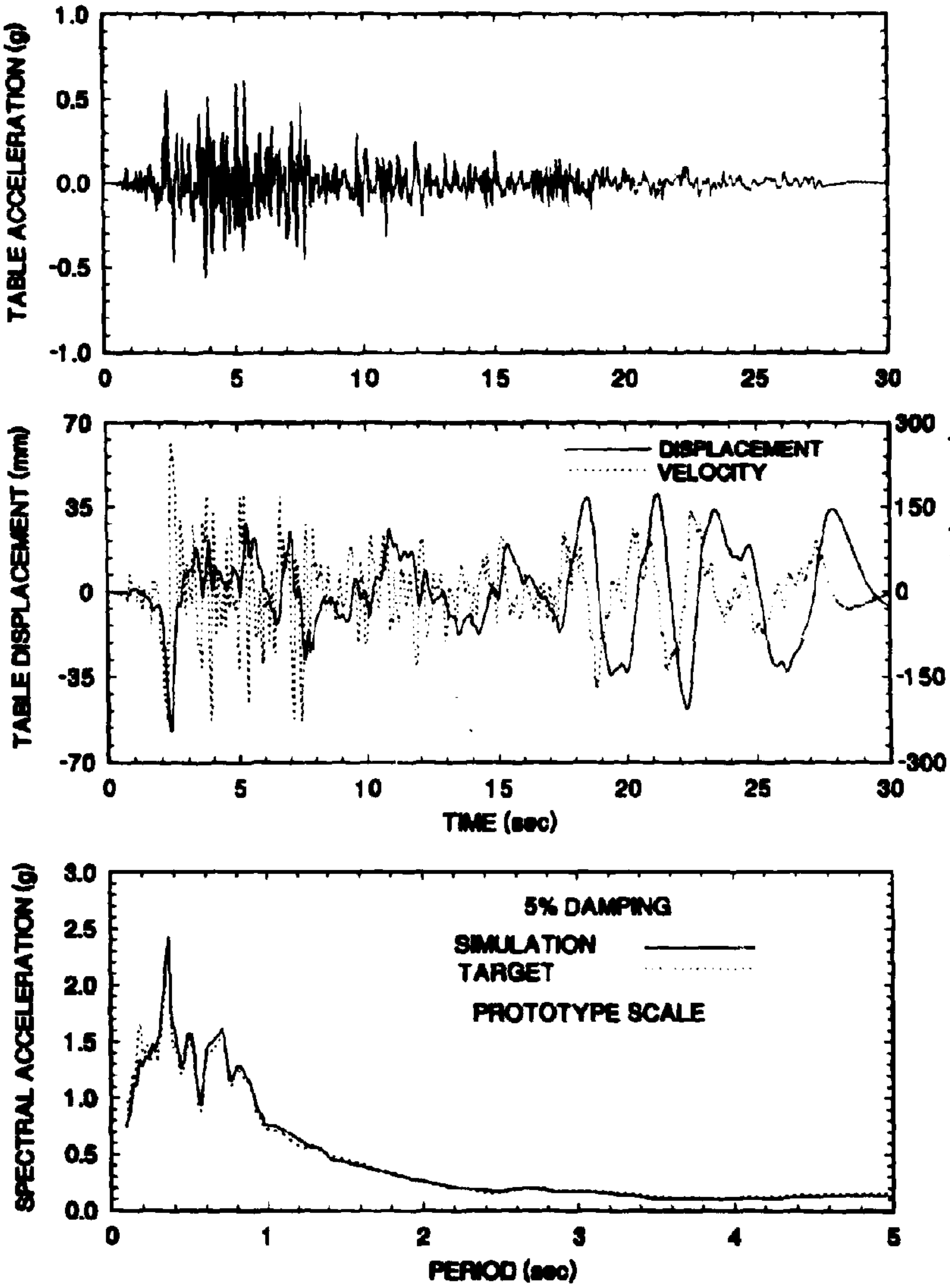


Figure 4-9 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with Taft N21E 400% Motion.

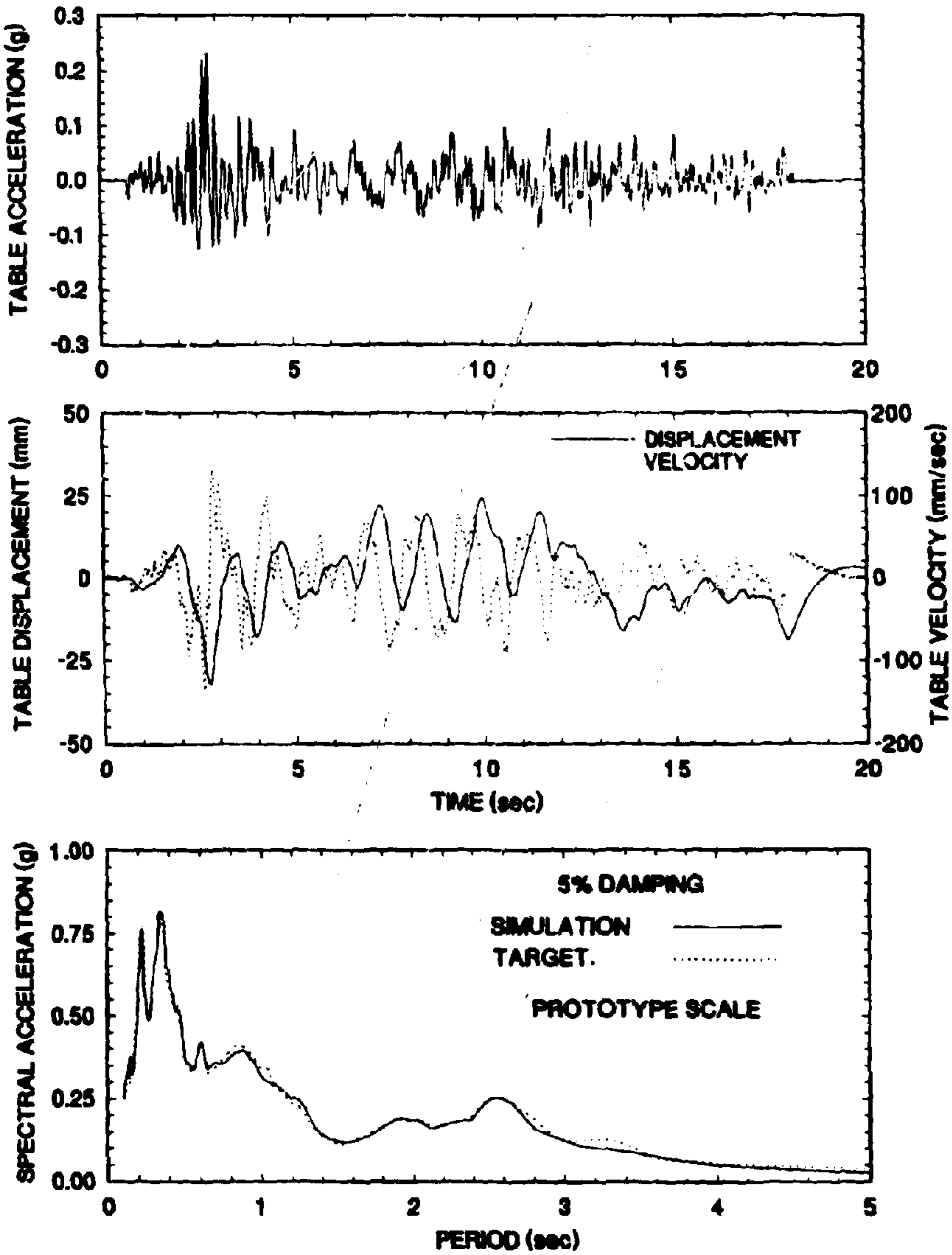


Figure 4-10 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with Hachinohe N-S 100% Motion.

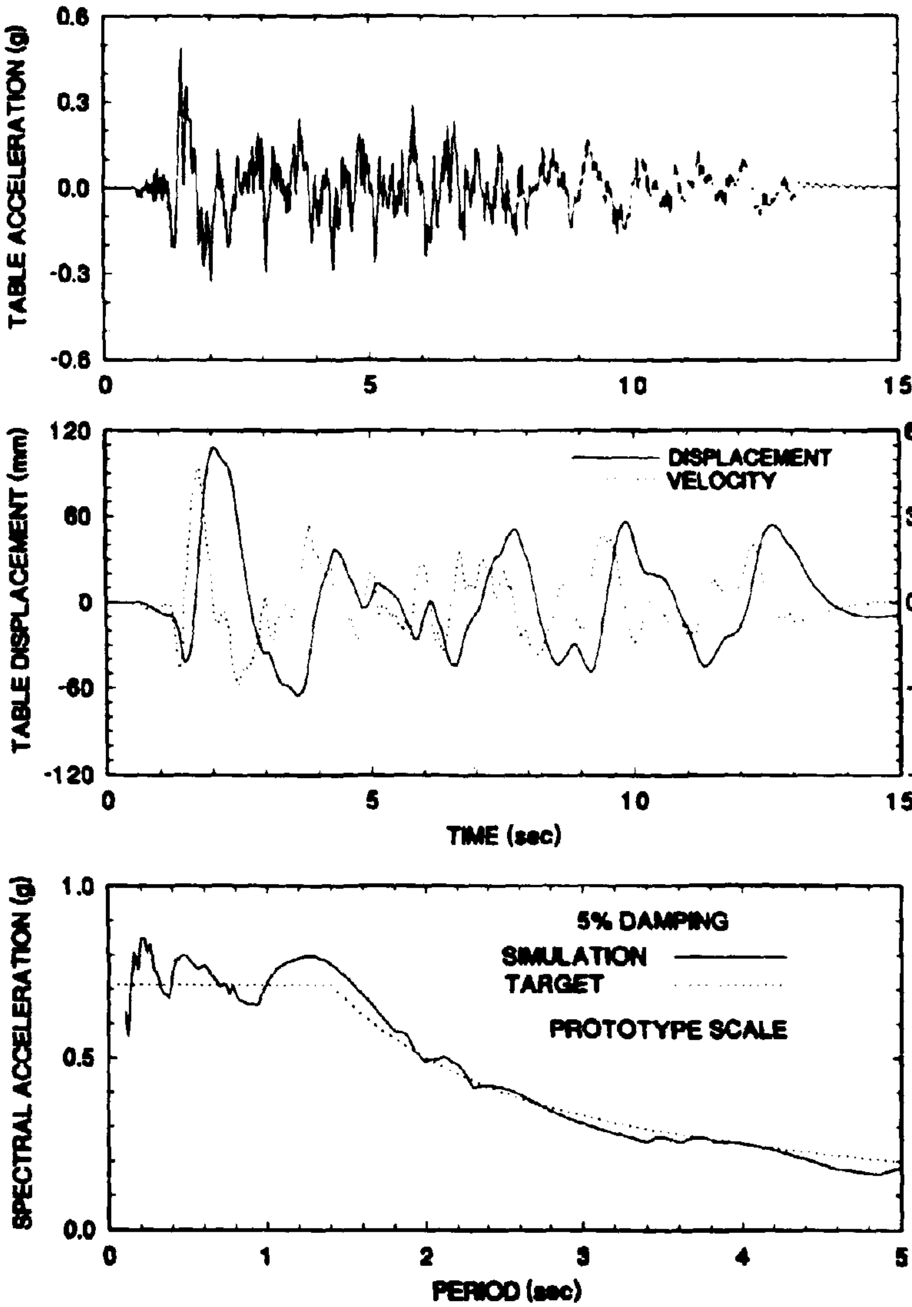


Figure 4-11 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with JP. Level 2 G.C.1 100% Motion.

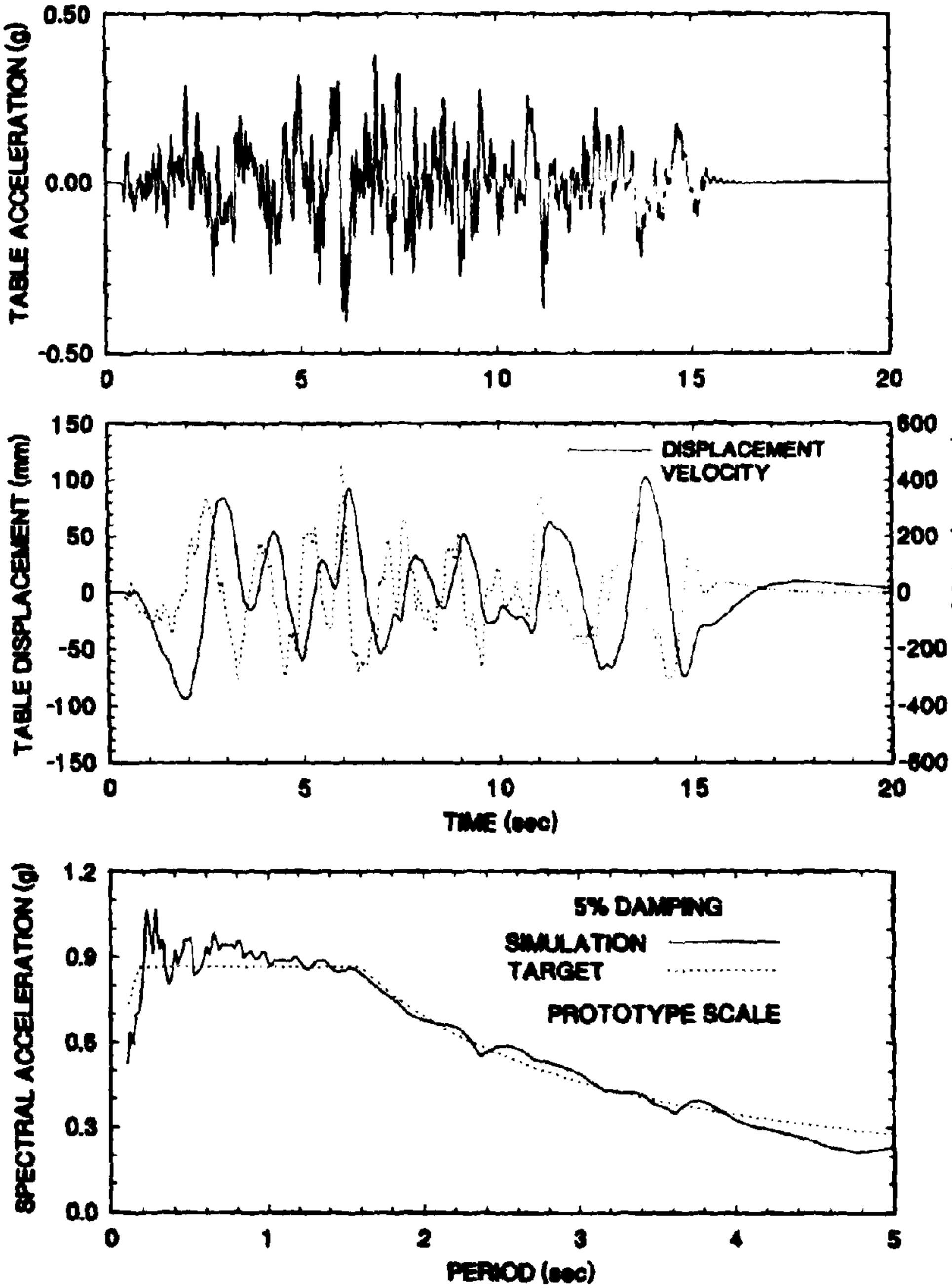


Figure 4-12 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with JP. Level 2 G.C.2 100% Motion.

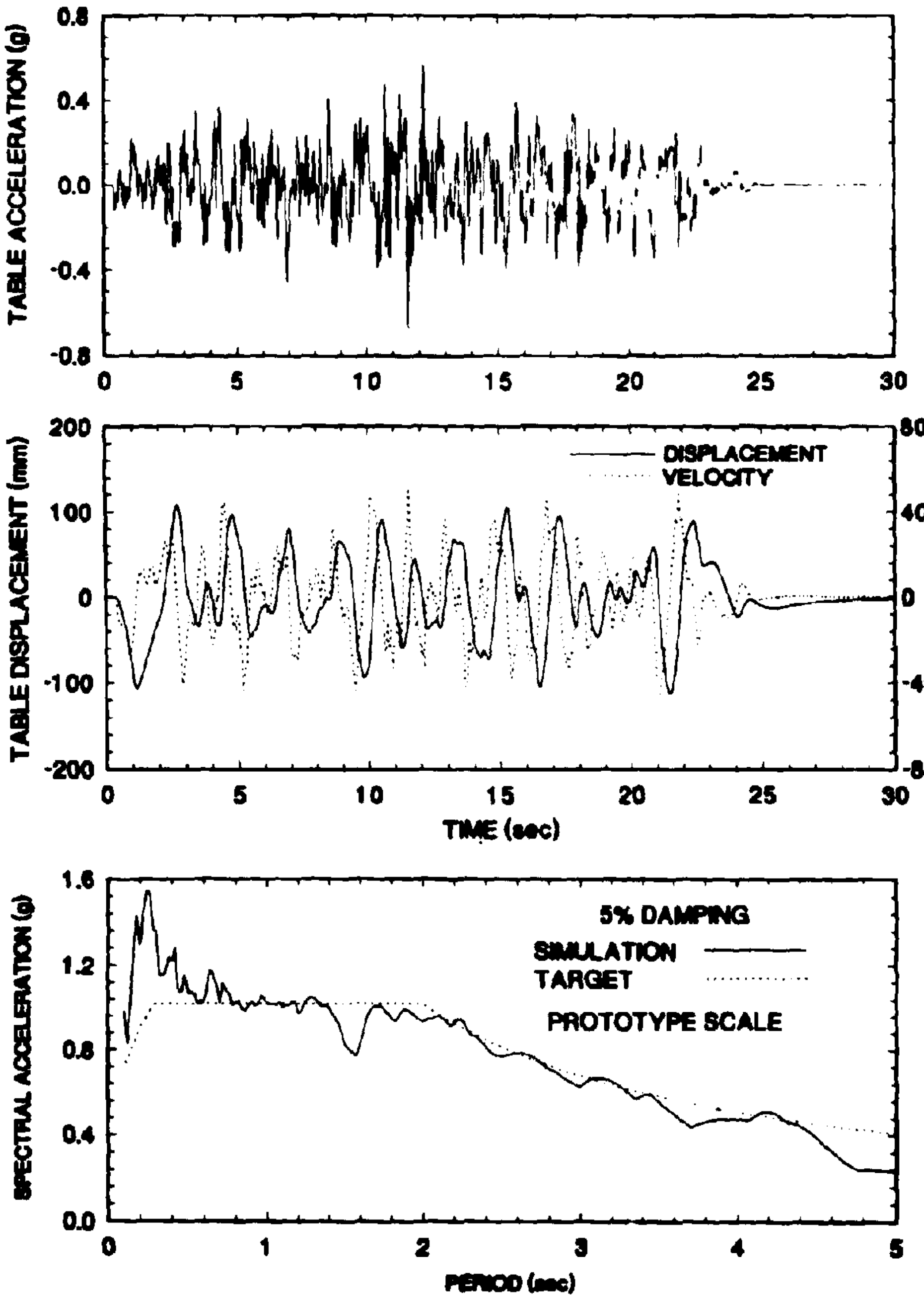


Figure 4-13 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with JP. Level 2 G.C.3 100% Motion.

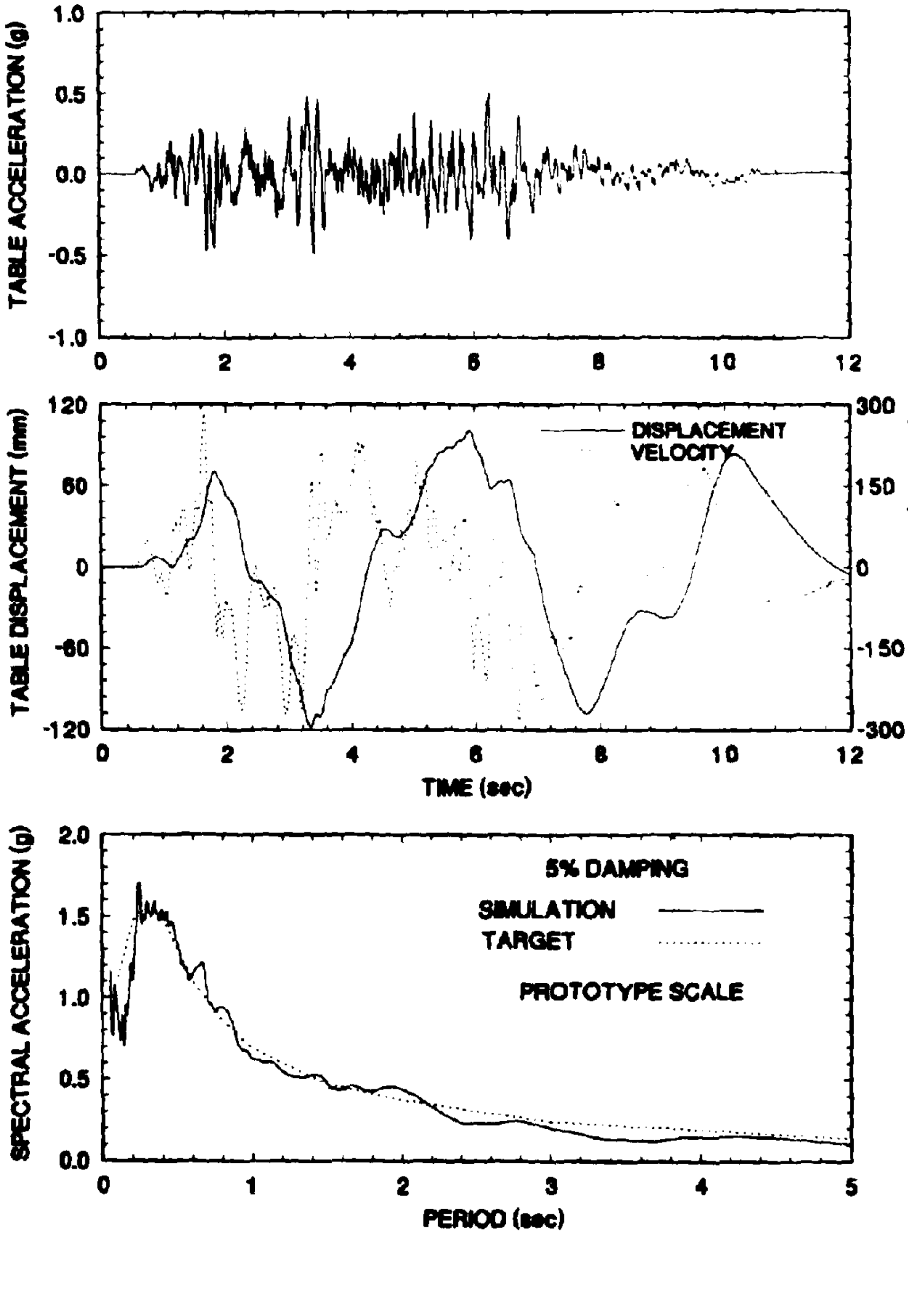


Figure 4-14 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Shaking Table Motion Excited with CalTrans Rock No.1 0.6g 100% Motion.

SECTION 5

EARTHQUAKE SIMULATOR TEST RESULTS

5.1 Results for Non-isolated Bridge

Testing of the non-isolated bridge (see Figure 4-5, configuration 1) was conducted with only horizontal excitation. The experimental results for the bridge in its non-isolated configuration are presented in Appendix A and are summarized in Table 5-I. For each test the peak values of the table motion in the horizontal direction are given. The displacement and acceleration were directly measured whereas the velocity was determined by numerical differentiation of the displacement record. The peak pier drift is given as a percentage of the pier height which was 1290.3mm. This is the length of the column excluding the stiffeners at the ends (see Figure 4-1). The peak shear force is given as a fraction of the axial load carried by the pier (70 kN each pier).

5.2 Results for Isolated Bridge

Tables 5-II list the earthquake simulation tests and model conditions in the tests of the isolated bridge. The excitation in these tables is identified with a percentage figure which represents a scaling factor on the acceleration, velocity and displacement of the actual record. For example, the figure 200% denotes a motion scaled up by a factor of two in comparison to the actual record.

Appendix A presents recorded time histories of response in each test, whereas Table 5-III presents a summary of the experimental results. The table includes the following results:

(a) Displacement of bearings located at the south pier (see Figures 4-2 to 4-4). The transducers monitoring the south bearing displacement were continuously monitored and not initialized prior to each test. Thus, the instruments recorded correctly the initial and permanent bearing displacements. Figure 5-1 shows an example of bearing displacement time history. The initial displacement is the permanent displacement in the previous test and the initial displacement in the current test.

Table 5-I Summary of Experimental Results of Non-Isolated Bridge

TEST No.	EXCITATION	PEAK TABLE MOTION			DECK ACCEL.	PIER SHEAR / AXIAL LOAD		PIER DRIFT RATIO (%)	
		DISP. (mm)	VEL. (mm/sec)	ACCEL. (g)	(g)	SOUTH	NORTH	SOUTH	NORTH
FRUN05	EL CENTRO S00E 25%	5.8	40.0	0.095	0.25	0.266	0.271	N/A	0.381
FRUN06	TAFT N21E 50%	7.0	32.7	0.069	0.21	0.230	0.234	N/A	0.315
FRUN07	TAFT N21E 75%	10.5	47.7	0.102	0.25	0.273	0.278	N/A	0.385
FRUN08	JP LEVEL 1 G.C.1 100%	16.6	96.0	0.109	0.21	0.231	0.222	N/A	0.346
FRUN09	JP LEVEL 1 G.C.2 100%	17.3	113.6	0.110	0.26	0.280	0.269	N/A	0.414
FRUN10	JP LEVEL 1 G.C.3 100%	33.7	158.3	0.130	0.33	0.353	0.354	N/A	0.623
FRUN11	AKITA N-S 75%	25.1	108.4	0.138	0.26	0.284	0.283	N/A	0.474
FRUN12	HACHINOHE N-S 50%	15.8	66.0	0.103	0.18	0.200	0.198	N/A	0.311
FRUN13	MIYAGIKEN OKI E-W 75%	8.0	38.0	0.080	0.22	0.242	0.235	N/A	0.384
FRUN14	MEXICO N90W 100%	51.7	303.1	0.169	0.26	0.286	0.284	N/A	0.522
FRUN15	JP LEVEL 2 G.C.1 25%	26.7	114.1	0.104	0.17	0.189	0.181	N/A	0.301
FRUN16	JP LEVEL 2 G.C.2 25%	25.0	109.8	0.098	0.21	0.232	0.225	N/A	0.365
FRUN17	JP LEVEL 2 G.C.3 25%	27.6	116.6	0.117	0.26	0.285	0.283	N/A	0.497
FRUN18	PACOIMA S74W 13%	4.0	36.4	0.103	0.2	0.221	0.214	N/A	0.346
FRUN19	PACOIMA S16E 13%	10.4	63.9	0.095	0.17	0.187	0.186	N/A	0.275
FRUN20	CALTRANS R3 0.6g 20%	23.5	124.8	0.101	0.22	0.227	0.234	N/A	0.389
FRUN21	CALTRANS S3 0.6g 20%	32.1	102.4	0.112	0.31	0.320	0.345	N/A	0.565
FRUN22	CALTRANS A2 0.6g 20%	47.2	128.3	0.104	0.27	0.278	0.298	N/A	0.475

(b) Maximum travel of bearings located at the north pier. The transducers monitoring the north bearing displacements were initialized prior to each test so that the initial displacement appeared always as zero. Thus, only the maximum travel (MAX.-INIT. in Figure 5-1) could be accurately obtained and not the initial and permanent displacements.

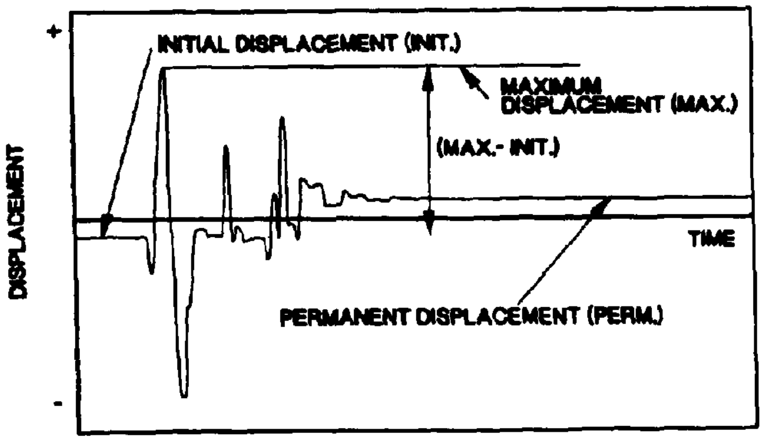


Figure 5-1 Example of Bearing Displacement History.

(c) Isolation system shear force normalized by the carried weight (140 kN for total shear force and 70 kN for shear force at each pier). The isolation system shear force is the sum of the horizontal components of bearing forces and was measured directly from the load cells supporting the bearings. The recorded values were corrected for the effect of the acceleration of the load cells and bearings (upper part of load cells and bottom part of the bearings). That is, the isolation system shear force at the south pier, V_S, was obtained from

$$V_S = F_{LCS} + W_{PS} a_{PS} / g \tag{5-1}$$

where F_{LCS} is the recorded force from the load cells, W_{PS} is the weight of the accelerating part of the south load cells and bearings and a_{PS} is the acceleration of the load cells. A similar equation is valid for the isolation system force at the north pier location, V_N. The total isolation system force, V, was then derived from

$$V = V_S + V_N \tag{5-2}$$

Equations (5-1) and (5-2) were used to obtain time histories of forces V_S, V_N and V, from which the peak values were extracted and included in Table 5-III.

It should be noted that for a rigid deck the isolation system force could be directly obtained from the deck acceleration measurement :

$$V = \frac{W_d \, a_d}{g} \tag{5-3}$$

where W_d = 140 kN and a_d is the recorded deck acceleration. However, the deck had some flexibility which caused amplification of the recorded deck acceleration. When Equation (5-3) was used, the loops of isolation system force (as obtained from the deck acceleration) versus bearing displacement were wavy. Since the recorded loops of bearing force versus displacement did not exhibit a similar wavy form, it was concluded that the recorded acceleration of the deck contained additional components caused by the deck's flexibility.

(d) Pier acceleration. The peak accelerations of the top of the south and north piers are reported.

(e) Deck horizontal acceleration.

(f) Pier shear force normalized by axial load. Each column was instrumented with strain gages to measure the shear force. The reported quantity is the sum of the shear forces in the two columns of each pier divided by the axial load on each pier (140/2=70 kN). The pier shear force is, in general, different than the isolation system shear force. The two forces differ by the inertia force of the accelerating part of the pier between the sliding interface and the location of the strain gages. The pier shear force in the case of stiff piers could not be measured and is not reported in the tables. It should be noted that in the case of stiff piers the columns were braced (see Figures 4-1, 4-5 and 4-6), so that the force measured by the strain gage load cells of the columns represented only part of the total pier shear force.

(g) Pier drift ratio. This is the displacement of the top of the pier relative to the shake table, divided by the length of the column (1290.3 mm).

During testing of the model bridge in its isolated condition it was observed that the overhangs of the shake table extension, which supported the piers (see Figure 4-1), underwent significant vertical motion even when only horizontal table motion was imposed. The two overhangs did not move vertically in unison. Rather, the motion of the two overhangs was anti-symmetric with the two sides moving with different amplitude and content in frequency. It was concluded that this vertical motion of the overhangs was the combined result of table-structure interaction, vertical flexibility of the overhangs and differences in the vertical stiffness of the overhangs (it was later found that on one side of the concrete table extension the reinforcement was misplaced).

The implications of this phenomenon were to increase the severity of the testing. In effect, in all tests the piers experienced out-of-phase vertical input at their bases. This caused changes in the vertical load carried by the sliding bearings, which in turn affected the friction force of the bearings.

Table 5-II List of Earthquake Simulation Tests and Model Conditions in Tests with Lubricated Bearings and Mild Steel Energy Dissipating Devices

TEST No.	EXCITATION	PEAK TABLE MOTION			PIER CONDITION		BEARING PRESSURE(MPa)		FRICTION COEFFICIENT	
		DIS. (mm)	VEL. (mm/s)	ACC. (g)	SOUTH	NORTH	SOUTH	NORTH	SOUTH	NORTH
ITBR01	EL CENTRO S00E 100%	23.7	180.1	0.275	STIFF	STIFF	17.8	17.8	2%	1%
ITBR02	EL CENTRO S00E 150%	35.5	240.2	0.399	STIFF	STIFF	17.8	17.8	2%	1%
ITBR03	HACHINOHE N-S 100%	32.0	134.2	0.231	STIFF	STIFF	17.8	17.8	2%	1%
ITBR04	TAFT N21E 200%	28.6	129.8	0.303	STIFF	STIFF	17.8	17.8	2%	1%
ITBR05	TAFT N21E 400%	57.3	276.6	0.613	STIFF	STIFF	17.8	17.8	2%	1%
ITBR06	TAFT N21E 200%	28.7	130.0	0.307	STIFF	STIFF	17.8	17.8	2%	1%
ITBR07	CALTRANS R1 0.6g 100%	117.9	286.0	0.515	STIFF	STIFF	17.8	17.8	2%	1%
ITBR08	CALTRANS R1 0.6g 100%	118.2	285.5	0.504	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR09	EL CENTRO S00E 100%	23.6	180.3	0.311	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR10	EL CENTRO S00E 150%	35.5	237.7	0.467	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR11	TAFT N21E 200%	28.7	131.6	0.337	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR12	HACHINOHE N-S 100%	31.9	136.2	0.232	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR13	JP LEVEL 2 G.C.1 50%	54.0	234.8	0.189	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR14	JP LEVEL 2 G.C.2 50%	50.6	228.2	0.211	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR15	JP LEVEL 2 G.C.3 50%	55.8	243.6	0.205	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%
ITBR16	JP LEVEL 2 G.C.3 75%	83.5	367.7	0.313	FLEXIBLE	FLEXIBLE	17.8	17.8	2%	1%

Table 5-III Summary of Experimental Results of Isolated Bridge with Lubricated Bearings and Mild Steel Energy Dissipating Devices

TEST No.	BEARING DISPLACEMENT (mm)				ISOLATION SYSTEM SHEAR/WEIGHT			DECK ACC. (g)	PIER ACC. (g)		PIER DRIFT (%)		PIER SHEAR/ AXIAL LOAD	
	SOUTH			NORTH										
	INIT.	MAX.	PERM.	MAX.- INIT.	SOUTH	NORTH	TOTAL		SOUTH	NORTH	SOUTH	NORTH	SOUTH	NORTH
ITBR01	-0.6	19.3	-2.4	19.8	0.182	0.180	0.181	0.191	0.349	0.362	0.05	0.04	N/A	N/A
ITBR02	-2.4	32.3	-7.1	34.5	0.200	0.200	0.200	0.213	0.467	0.443	0.05	0.04	N/A	N/A
ITBR03	-7.1	-23.7	-9.9	16.7	0.182	0.183	0.182	0.197	0.261	0.254	0.04	0.03	N/A	N/A
ITBR04	-9.9	-28.1	-6.6	18.4	0.198	0.192	0.190	0.194	0.335	0.344	0.04	0.04	N/A	N/A
ITBR05	-6.6	-58.8	-30.7	52.0	0.220	0.212	0.216	0.237	0.688	0.638	0.05	0.25	N/A	N/A
ITBR06	-30.2	-48.9	-30.4	18.7	0.185	0.181	0.183	0.188	0.323	0.344	0.04	0.04	N/A	N/A
ITBR07	-30.7	-4.1	-28.2	26.7	0.200	0.195	0.197	0.220	0.604	0.525	0.06	0.06	N/A	N/A
ITBR08	-28.3	0.0	-13.9	29.0	0.198	0.204	0.200	0.234	0.748	0.721	0.34	0.36	0.228	0.237
ITBR09	-13.9	-27.6	-22.9	18.8	0.178	0.183	0.181	0.189	0.605	0.564	0.28	0.28	0.178	0.184
ITBR10	-22.9	9.0	-27.2	32.0	0.193	0.216	0.203	0.212	0.867	0.811	0.30	0.32	0.213	0.227
ITBR11	-27.3	-44.1	-29.2	17.0	0.195	0.195	0.195	0.204	0.674	0.611	0.27	0.30	0.181	0.197
ITBR12	-29.2	-41.7	-29.5	12.6	0.184	0.189	0.186	0.193	0.414	0.409	0.27	0.29	0.176	0.194
ITBR13	-29.5	-50.7	-24.9	21.4	0.203	0.209	0.206	0.212	0.452	0.428	0.30	0.32	0.204	0.209
ITBR14	-24.9	12.0	-2.0	37.0	0.195	0.204	0.199	0.215	0.523	0.469	0.31	0.30	0.195	0.210
ITBR15	-2.0	-34.0	-4.8	32.3	0.193	0.200	0.196	0.207	0.583	0.599	0.31	0.32	0.200	0.213
ITBR16	-4.8	-55.6	-2.2	50.8	0.214	0.209	0.212	0.216	0.620	0.633	0.34	0.35	0.228	0.231

SECTION 6

INTERPRETATION OF EXPERIMENTAL RESULTS

6.1 Behavior of Isolation System

The experimental results of Table 5-III demonstrate a significant property of the tested isolation system. The isolation system force, that is the force transmitted to the substructure, is nearly constant in the range of 0.18 to 0.21 of the deck weight, regardless of the level of seismic excitation and its content in frequency. This desirable property is the result of the nearly elastoplastic behavior of the isolation bearings. However, this behavior is accompanied by the development of large permanent displacements. This issue is further discussed in Section 6.2.

Figure 6-1 presents the peak response of the tested isolated and non-isolated bridges with flexible piers as function of the peak table acceleration. It is clear that the shear force and drift in the piers are nearly constant. The pier shear force is marginally higher than the isolation system force as a result of inertia forces due to pier acceleration. In general, the recorded isolation system force is consistent with the characteristic strength of the isolation system as determined in the identification tests. These tests (see Section 3.3) revealed a combined (friction plus force from steel devices at full plasticization) strength of 6.5 kN at each of the south pier bearings and 7.0 kN at each of the north pier bearings. The total strength is 27 kN or 0.193 times the deck weight.

The result of Figure 6-1 demonstrate significant differences between the response of the isolated and the non-isolated bridges. A different illustration of these differences is presented in Figure 6-2. Both the isolated and the non-isolated bridges are subjected to the Japanese level 2 bridge design motions. The substructure response of the isolated bridge is elastic and completely insensitive to the input. The reader should observe the isolation system force-displacement loops in these tests, which are presented in Appendix A. For the ground condition 3 motion, the bearings undergo a significant number of inelastic cycles, whereas for the ground conditions 1 and 2 motions they undergo only a single inelastic cycle. In contrast the non-isolated bridge shows a marked sensitivity on

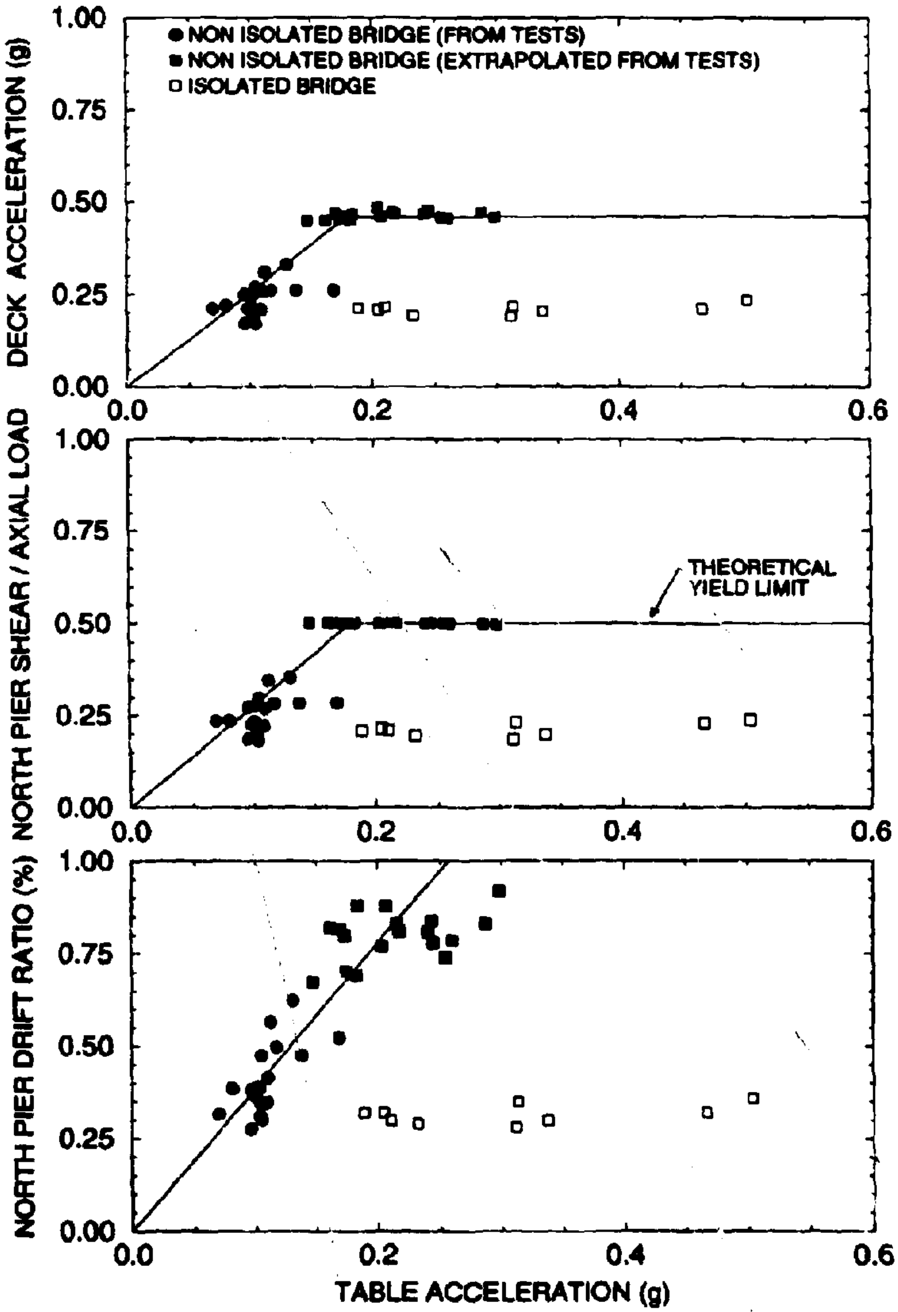


Figure 6-1 Comparison of Response of Non-Isolated and Isolated Bridges.

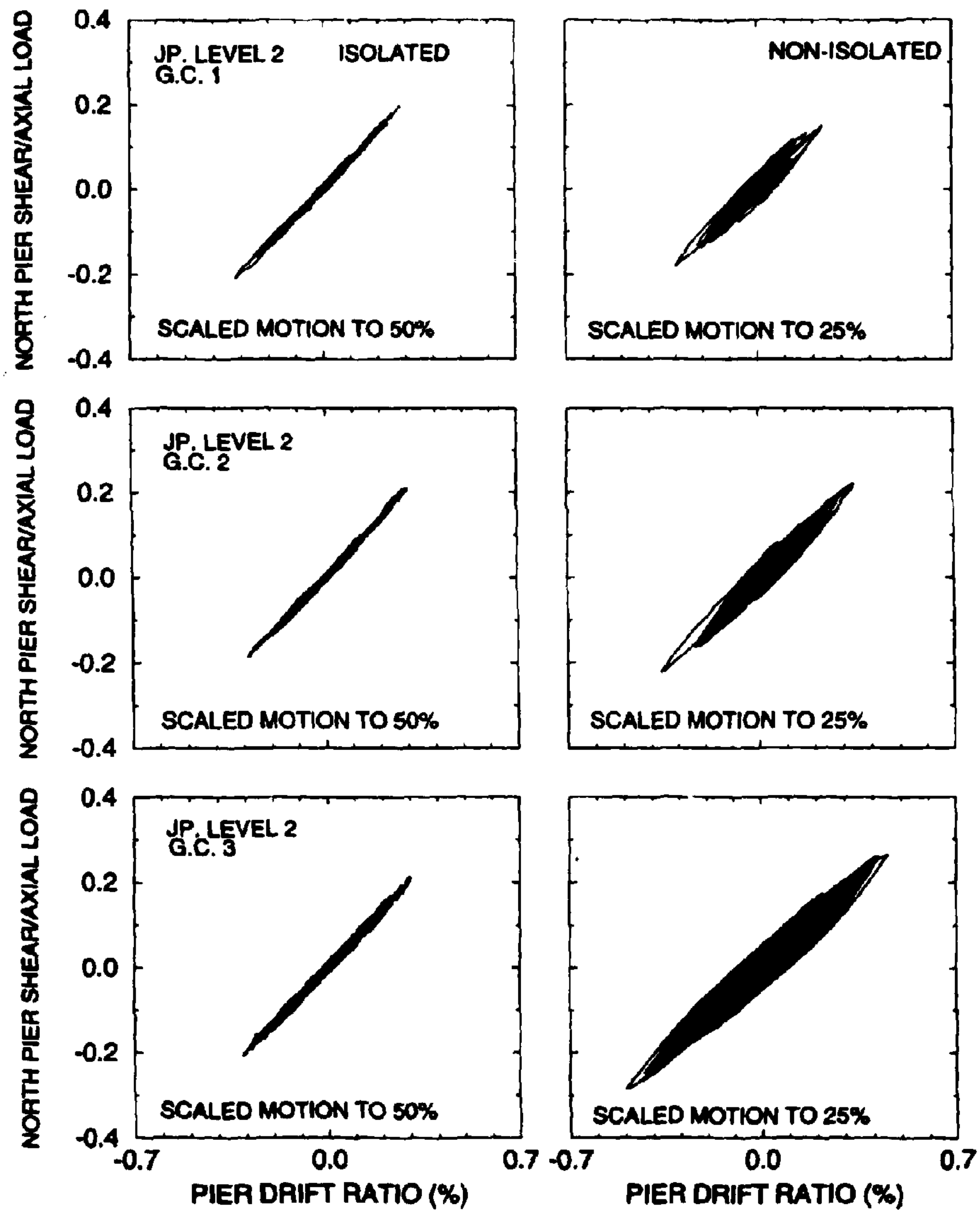


Figure 6-2 Comparison of Pier Response of Isolated and Non-Isolated Bridges in Reduced Japanese Level 2 Motions.

the ground conditions and it undergoes some inelastic action in the substructure. One should note that the seismic input in the non-isolated bridge is only half as strong as that of the isolated bridge.

The effects of increasing intensity of seismic excitation on the response of the isolated bridge are illustrated in Figure 6-3 to 6-4, which depict the response of the isolated bridge with flexible and stiff piers, respectively, as a function of increasing intensity of earthquake input. The intensity of the excitation is represented by the peak table velocity, which is regarded as a better single measure of intensity of input than the peak table acceleration. This is because the response of isolated structures is primarily influenced by the amplitude and frequency content of the velocity domain of the response spectrum of the input. It may be observed that the acceleration and force responses of the isolated bridge are not affected by the intensity of the input. Rather, we observe a noticeable effect of input intensity on the bearing displacement. This displacement is actually the peak bearing travel (maximum displacement minus initial displacement), and not the peak bearing displacement. We note that the peak bearing travel is less than the peak table displacement.

Finally, Figure 6-5 compares the peak response of the isolated with flexible piers to that with stiff piers for specific seismic inputs. Clearly, the bearing travel and isolation system force are unaffected by the stiffness of the piers. This is a result of the elastoplastic behavior of the system. However, the pier acceleration is higher in the flexible pier system. This behavior is common to all isolation system (Constantinou 1993, Tsopelas 1994a and 1994b).

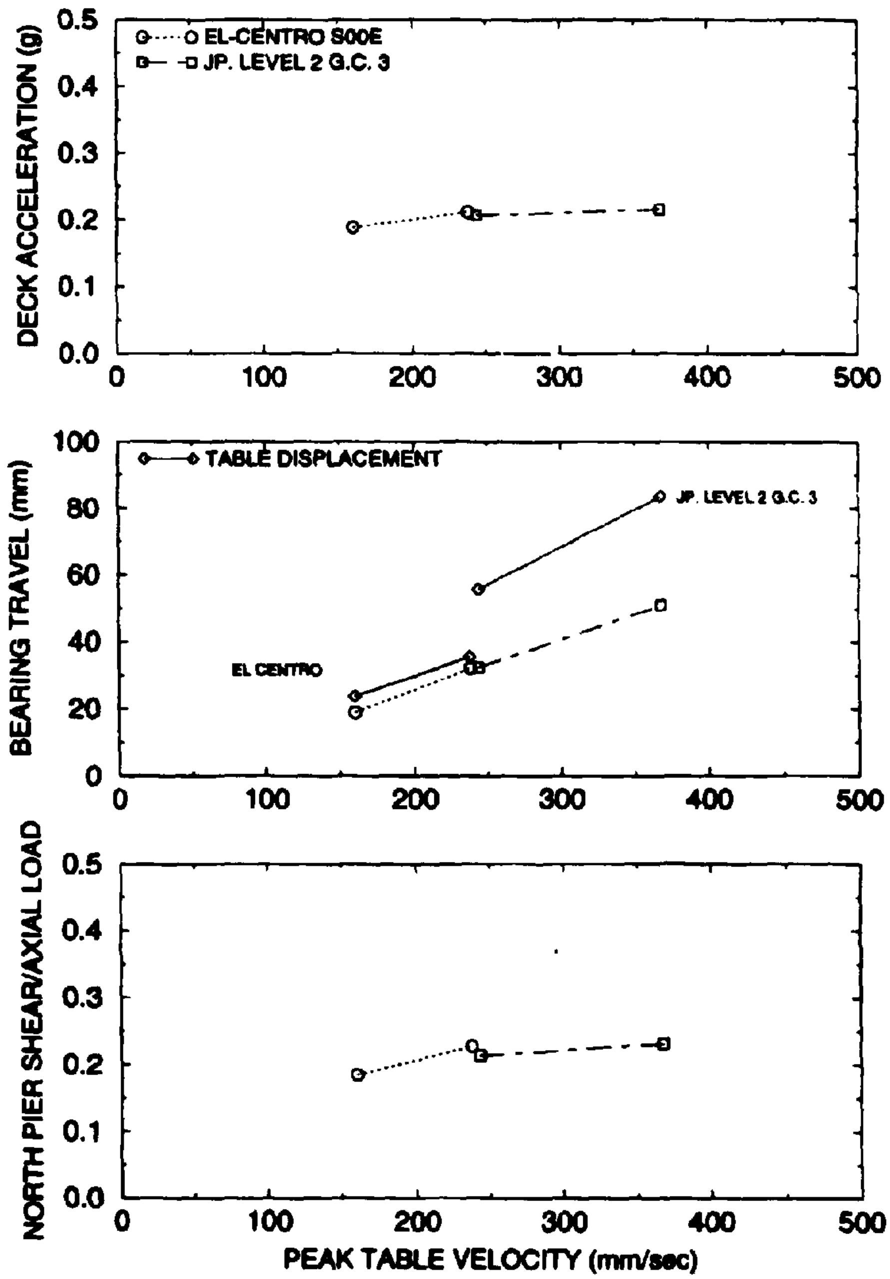


Figure 6-3 Response of Isolated Bridge with Flexible Piers under Increasing Intensity of Input (Represented by Peak Table Velocity).

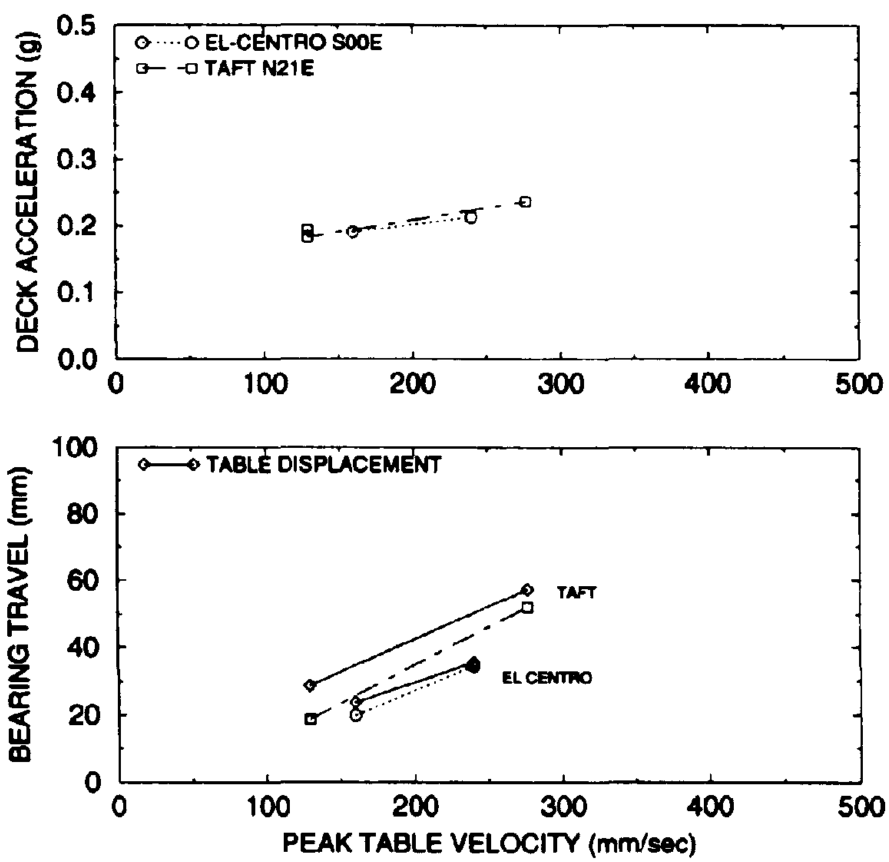
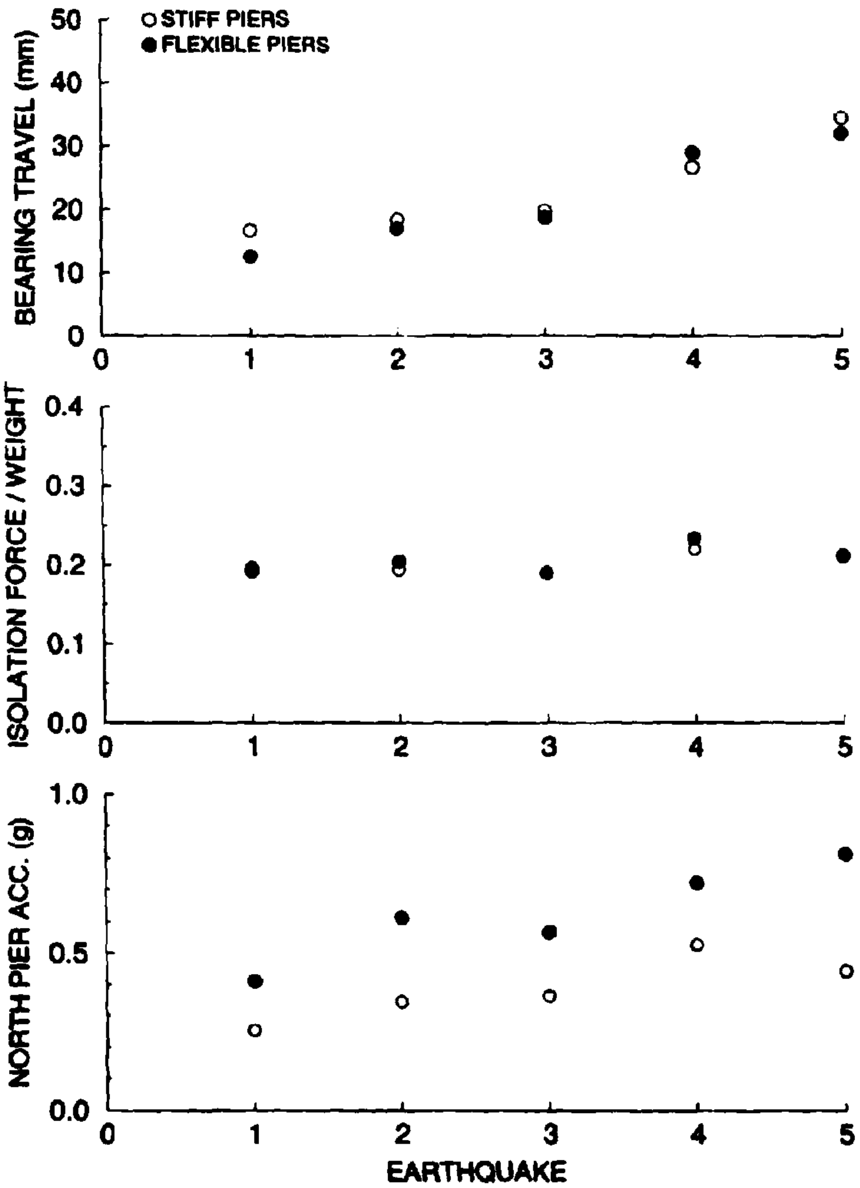


Figure 6-4 Response of Isolated Bridge with Stiff Piers under Increasing Intensity of Input (Represented by Peak Table Velocity).



1 HACHINOHE N-S 100% ; 2 TAFT N21E 200% ; 3 EL CENTRO S00E 100% ;
4 CALTRANS ROCK 0.6g 100% ; 5 EL CENTRO S00E 150%

Figure 6-5 Comparison of Response of Isolated Bridge with Stiff and Flexible Piers.

6.2 Permanent Displacements

The initial and final (permanent) bearing displacements in each test were recorded and reported in Table 5-III. A more detailed presentation is given in Table 6-I. This table lists for each test the initial and final bearing displacements, the permanent displacement in each test (that is, final minus initial displacement) and the accumulated permanent displacement. It may be noted in Table 6-I that the initial displacement in the first seismic test was -0.6 mm. This small permanent displacement occurred at the conclusion of the identification tests prior to conducting the seismic tests.

The results demonstrate that significant permanent displacements developed. Particularly, test No. ITBR05 with the Taft motion 400% level resulted in a permanent displacement of 30.7 mm, for which the surface steel strain was (Eq. 3-7, $\varepsilon_Y = 0.002$) equal to 0.0172. In the same test the peak bearing displacement was equal to 58.8 mm, which corresponds to a surface steel strain of 0.0334. At the time of testing we thought that the displacements which resulted in the ITBR05 test were large. Accounting for the possibility of accumulation of even large permanent displacement, it was decided at that time to reduce the scope of the experimental program and not test the isolated bridge with stronger motions.

The large permanent displacement recorded in the Taft 400%-level motion has been the result of the particular nature of this motion. The effect of this earthquake has been primarily a single large amplitude displacement cycle of the bearings (see details of response in Appendix A), as if the earthquake consisted of a single strong shock. Following this strong shock, the earthquake lacked sufficient acceleration to develop the required inertia forces at the deck level for recentering of the bridge. This may be confirmed by observing that in motions with large number of cycles (e.g. the Japanese level 2, G.C. 3), there is very small difference between the initial and final permanent bearing displacements.

The observed behavior of the isolation system is not unique to the Taft motion. A number of other motions have been identified, which may result in large permanent displacements.

Table 6-I List of Permanent Bearing Displacements

Pier Condition	Earthquake	Initial Displacement (mm)	Final Displacement (mm)	Final-Initial Displ. (mm)	Accumulated Permanent Displacement (mm)
Stiff	El Centro S00E 100%	-0.6	-2.4	-1.8	-2.4
	El Centro S00E 150%	-2.4	-7.1	-4.7	-7.1
	Hachinohe N-S 100%	-7.1	-9.9	-2.8	-9.9
	Taft N21E 200%	-9.9	-6.6	3.3	-6.6
	Taft N21E 400%	-6.6	-30.7	-24.1	-30.7
	Taft N21E 200%	-30.2	-30.4	-0.2	-30.4
	Caltrans R1 0.6 g 100%	-30.7	-28.2	2.5	-28.2
Flexible	Caltrans R1 0.6 g 100%	-28.3	-13.9	14.4	-13.9
	El Centro S00E 100%	-13.9	-22.9	-9.9	-22.9
	El Centro S00E 150%	-22.9	-27.2	-4.6	-27.2
	Taft N21E 200%	-27.3	-29.2	-1.9	-29.2
	Hachinohe N-S 100%	-29.2	-29.5	-0.3	-29.5
	JP Level 2 G.C. 1 50%	-29.5	-24.9	4.6	-24.9
	JP Level 2 G.C. 2 50%	-24.9	-2.0	22.9	-2.0
	JP Level 2 G.C. 3 50%	-2.0	-4.8	-2.8	-4.8
	JP Level 2 G.C. 3 75%	-4.8	-2.2	2.6	-2.2

These include the Japanese bridge design motions of the level 2 and ground conditions 1 and 2 (100% strength), the Pacoima Dam records of the 1971 San Fernando earthquake and several records of the 1994 Northridge earthquake (California Department of Conservation, 1994).

To demonstrate the behavior, analyses of the tested isolated bridge have been performed using the analytical model described in Section 7. Table 6-II lists the characteristics in prototype scale of the input motions used. Figures 6-6 to 6-9 present time histories of the ground acceleration and the response spectra of these motions. However for the analysis, the records were scaled to the scale of the experimental model. Furthermore, the Pacoima Dam motion was applied with a peak acceleration of only 0.59g (reduced to 50%). Dynamic analysis results are presented in Figures 6-10 to 6-15. Evidently, these motions result in large permanent displacements, as observed in the test with the Taft motion. Of interest is to note that these motions are representative of a wide range of soil conditions.

Table 6-II Motions Used in Analytical Study

Earthquake	Station	Component	Peak Ground Accel. (g)	Site Geology	Epicentral Distance (Km)	Scale Factor for Analysis
1971 San Fernando	Pacoima Dam	S16E	1.17	Rock	3	50%
1994 Northridge	Sylmar- Parking Lot	90°	0.61	Alluvium	16	100%
1994 Northridge	Newhall-LA County Fire Station	360°	0.61	Alluvium	20	100%
1994 Northridge	Newhall-LA County Fire Station	90°	0.63	Alluvium	20	100%
Jap. Level 2 G.C. 1	Artificial	--	0.71	Stiff Soil	--	100%
Jap. Level 2 G.C. 2	Artificial	--	0.71	Soft Soil/ Alluvium	--	100%

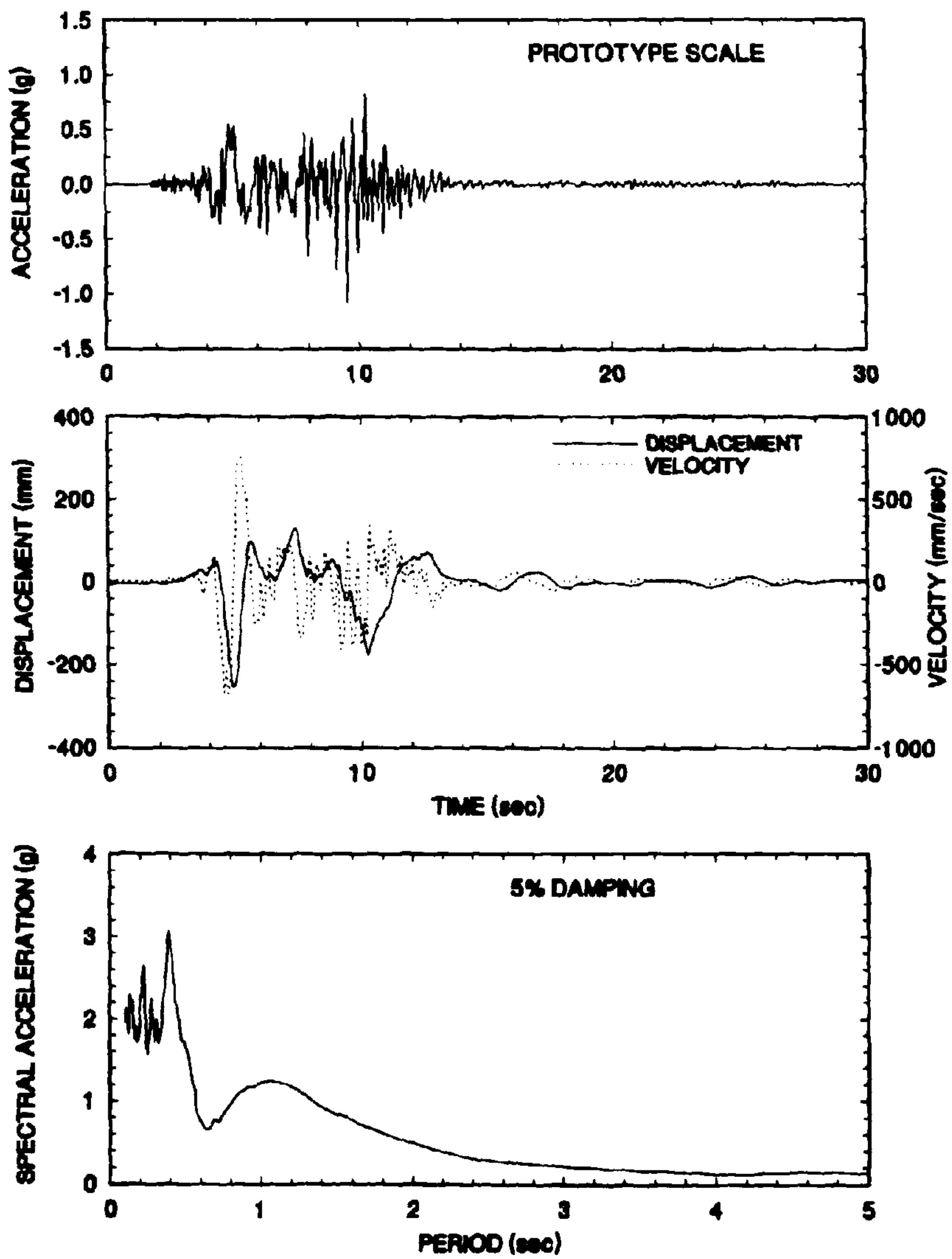


Figure 6-6 **Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Pacoima Dam S16E Motion.**

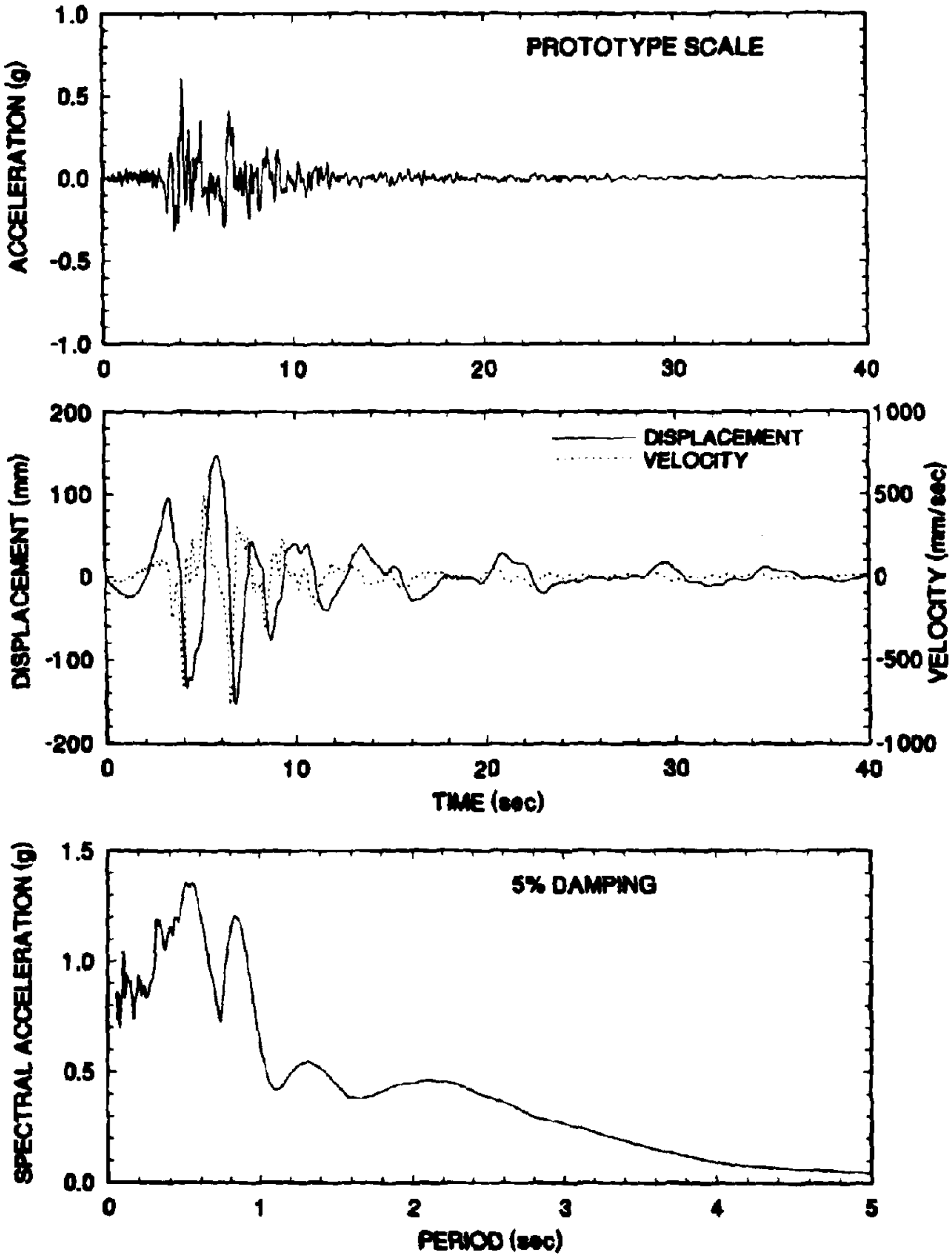


Figure 6-7 **Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Sylmar-Parking Lot, Component 90° Motion of 1994 Northridge Earthquake.**

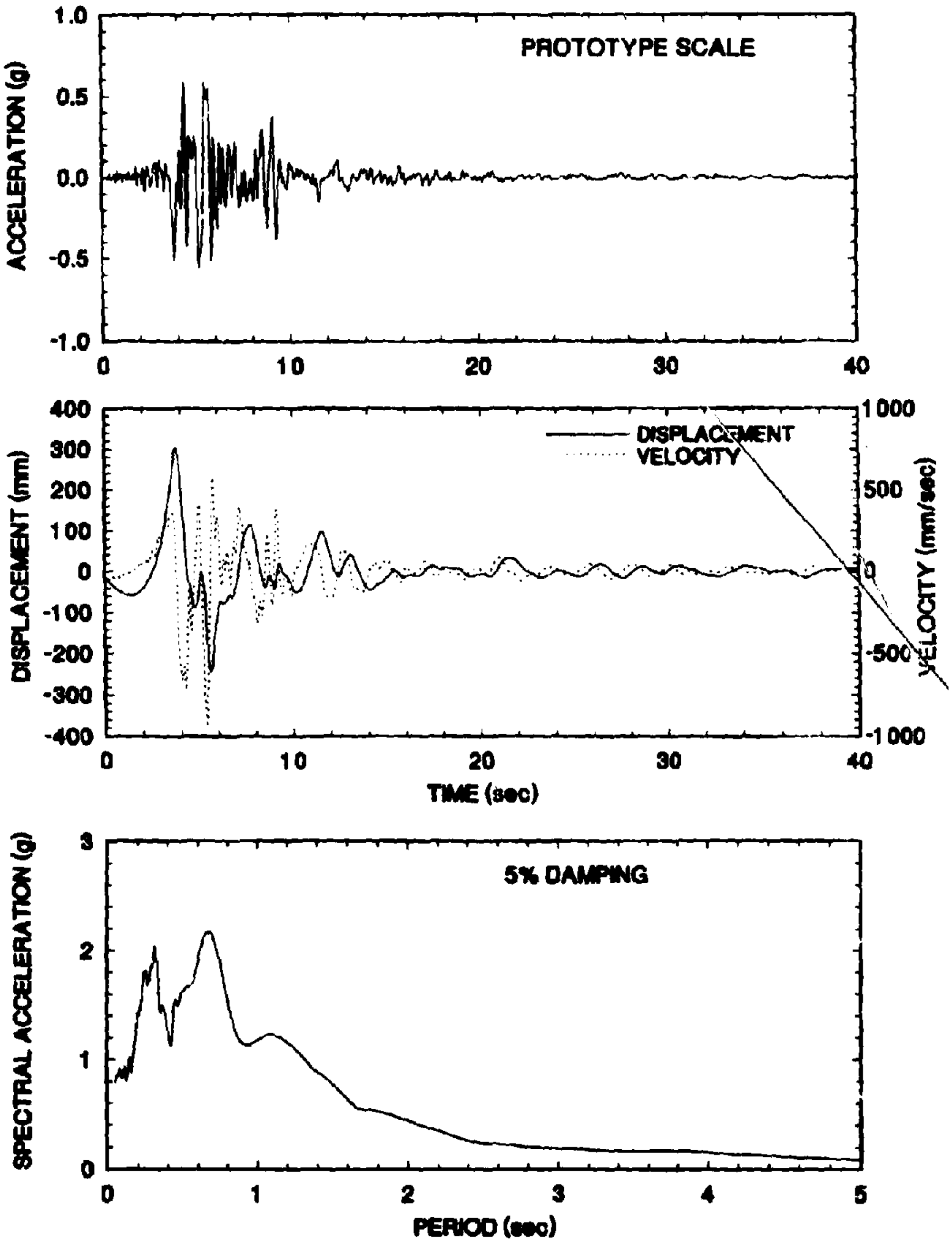


Figure 6-8 Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Newhall-Fire Station, Component 360° Motion of 1994 Northridge Earthquake.

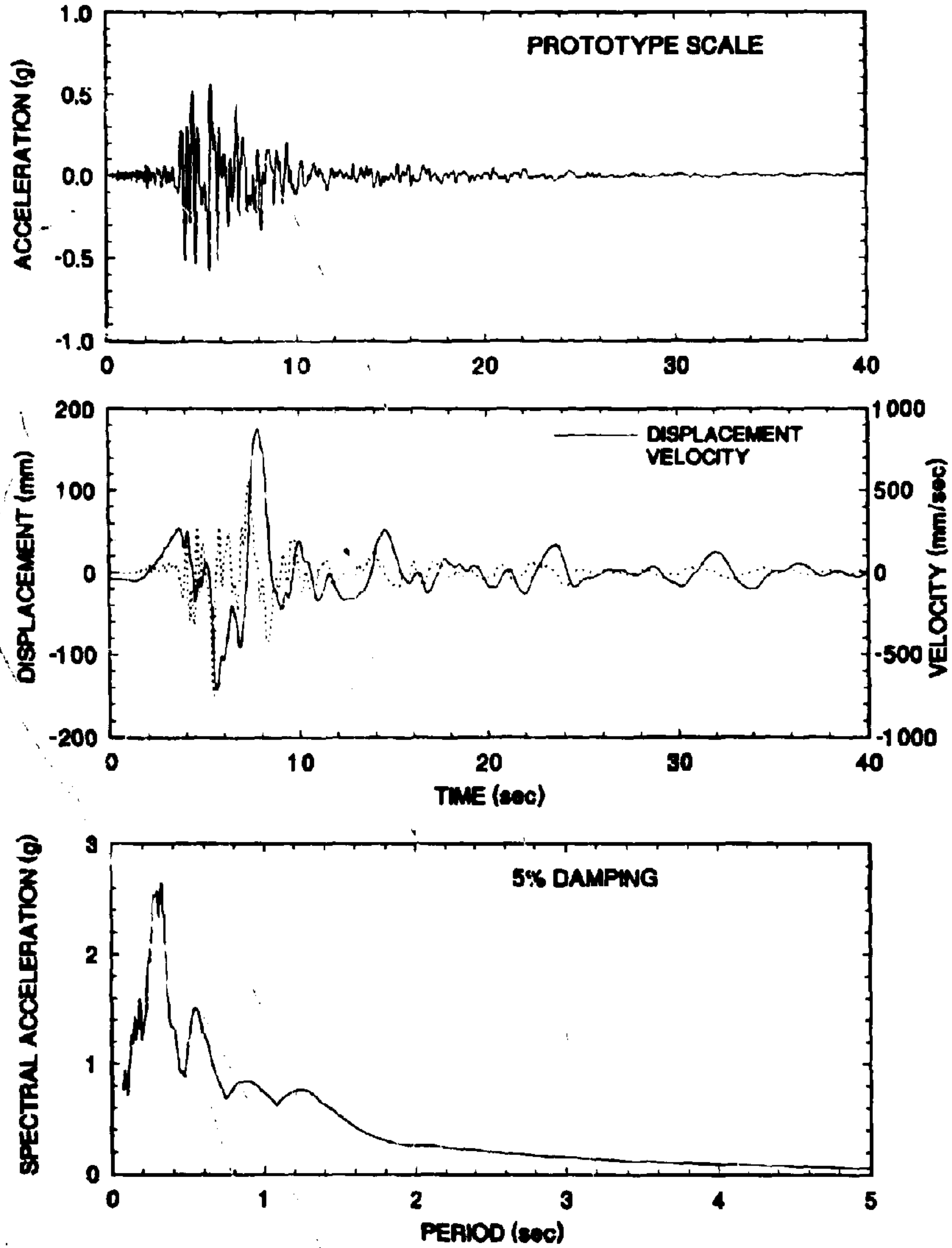


Figure 6-9 **Time Histories of Displacement, Velocity and Acceleration and Acceleration Response Spectrum of Newhall-Fire Station, Component 90° Motion of 1994 Northridge Earthquake.**

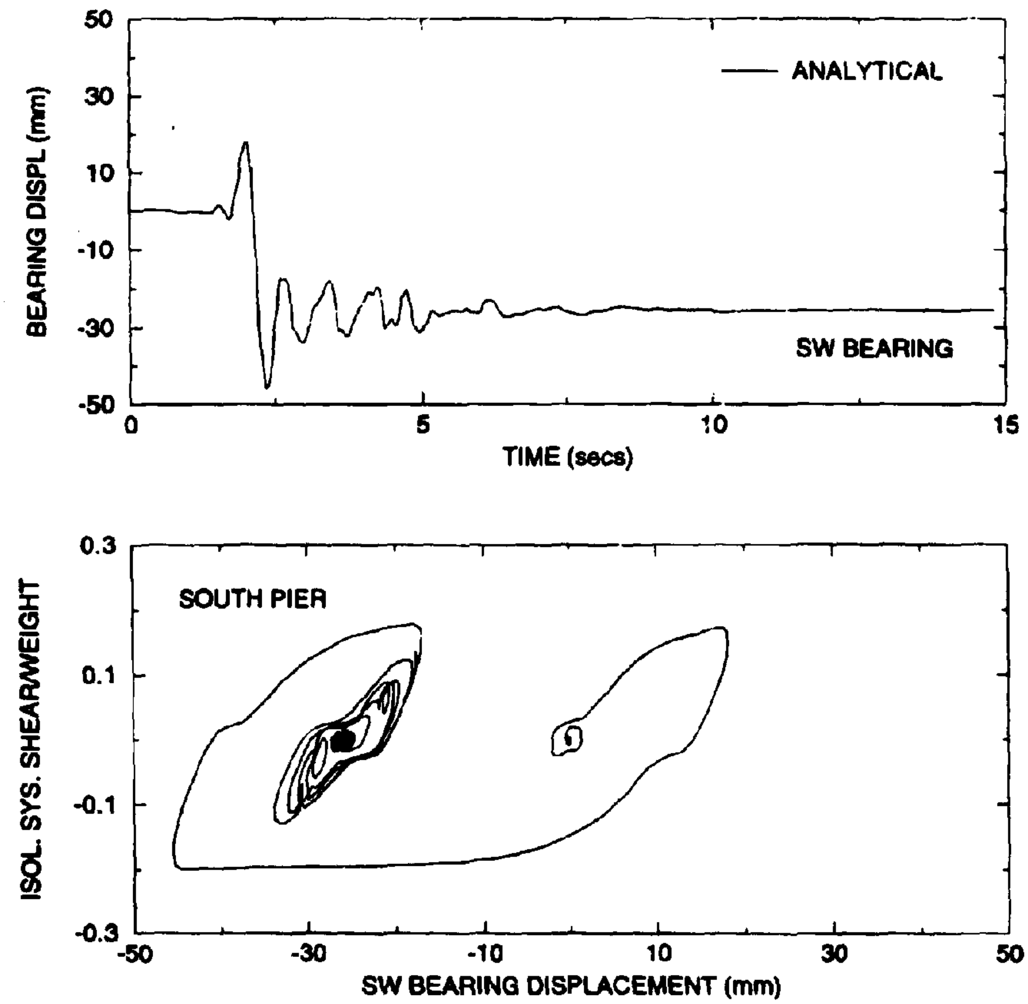


Figure 6-10 Analytical Response of Bridge Model with Flexible Piers to Pacoima S16E 50% Input Motion.

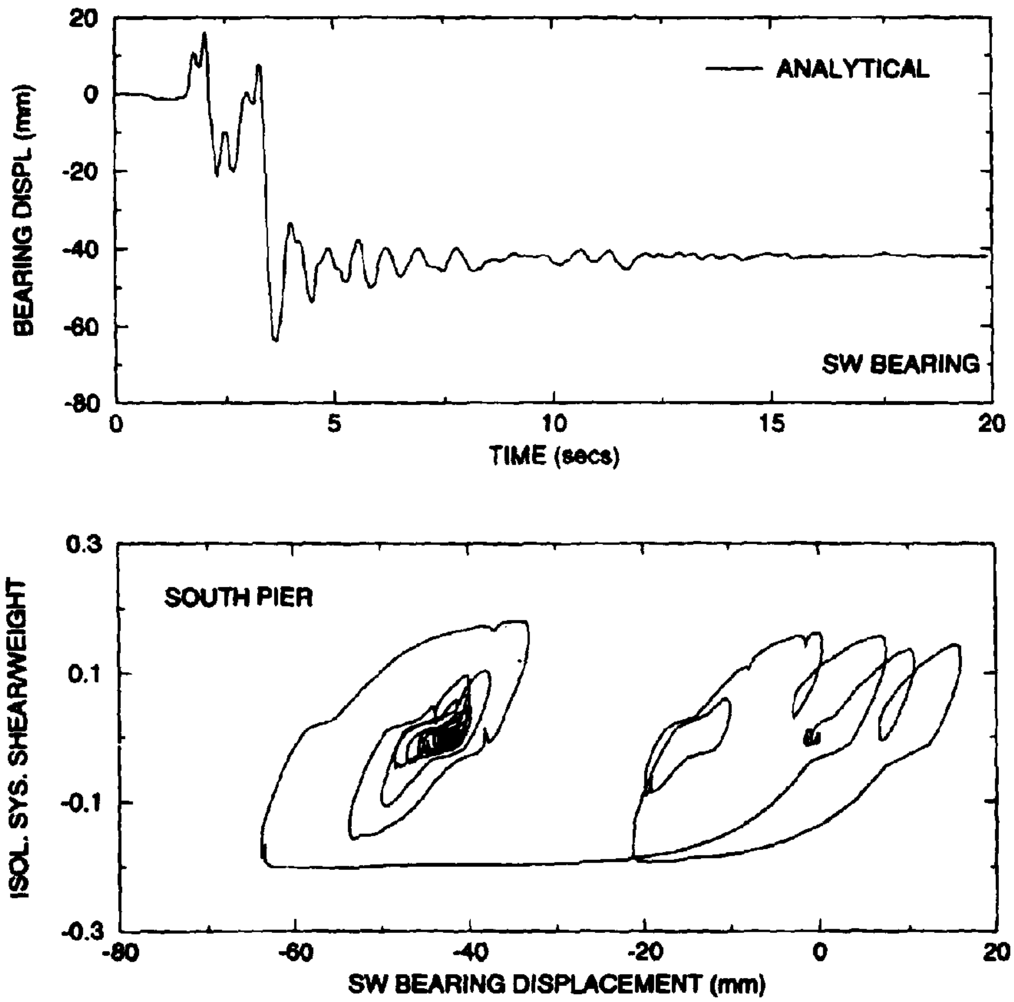


Figure 6-11 Analytical Response of Bridge Model with Flexible Piers to Sylmar 90° 100% Input Motion.

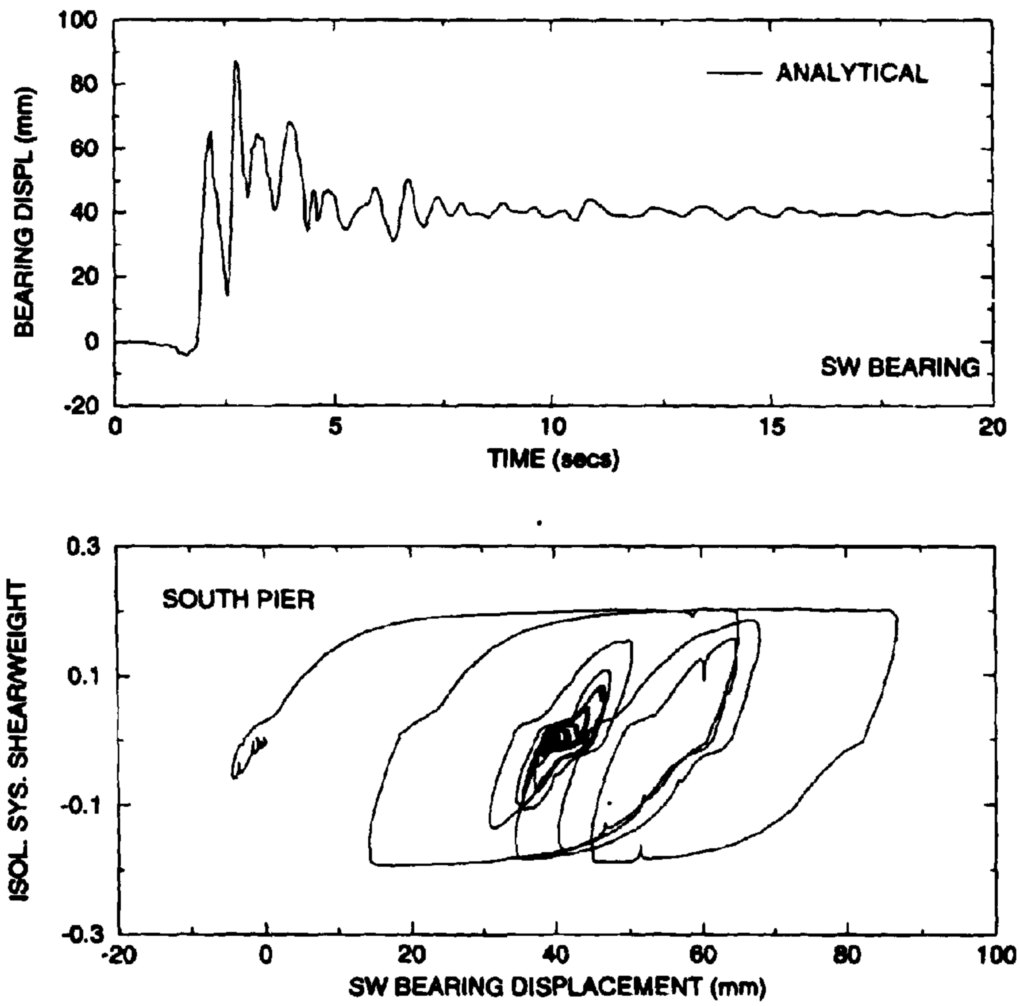


Figure 6-12 Analytical Response of Bridge Model with Flexible Piers to Newhall 360° 100% Input Motion.

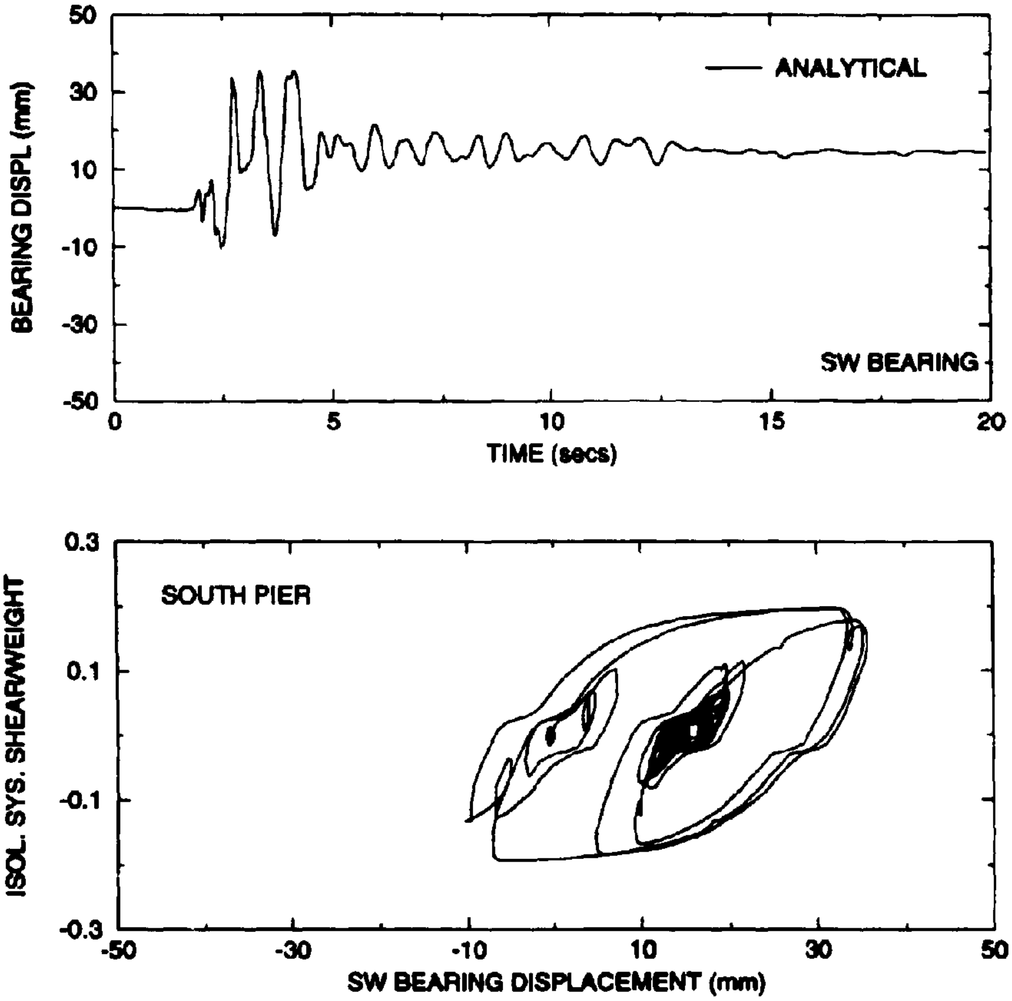


Figure 6-13 Analytical Response of Bridge Model with Flexible Piers to Newhall 90° 100% Input Motion.

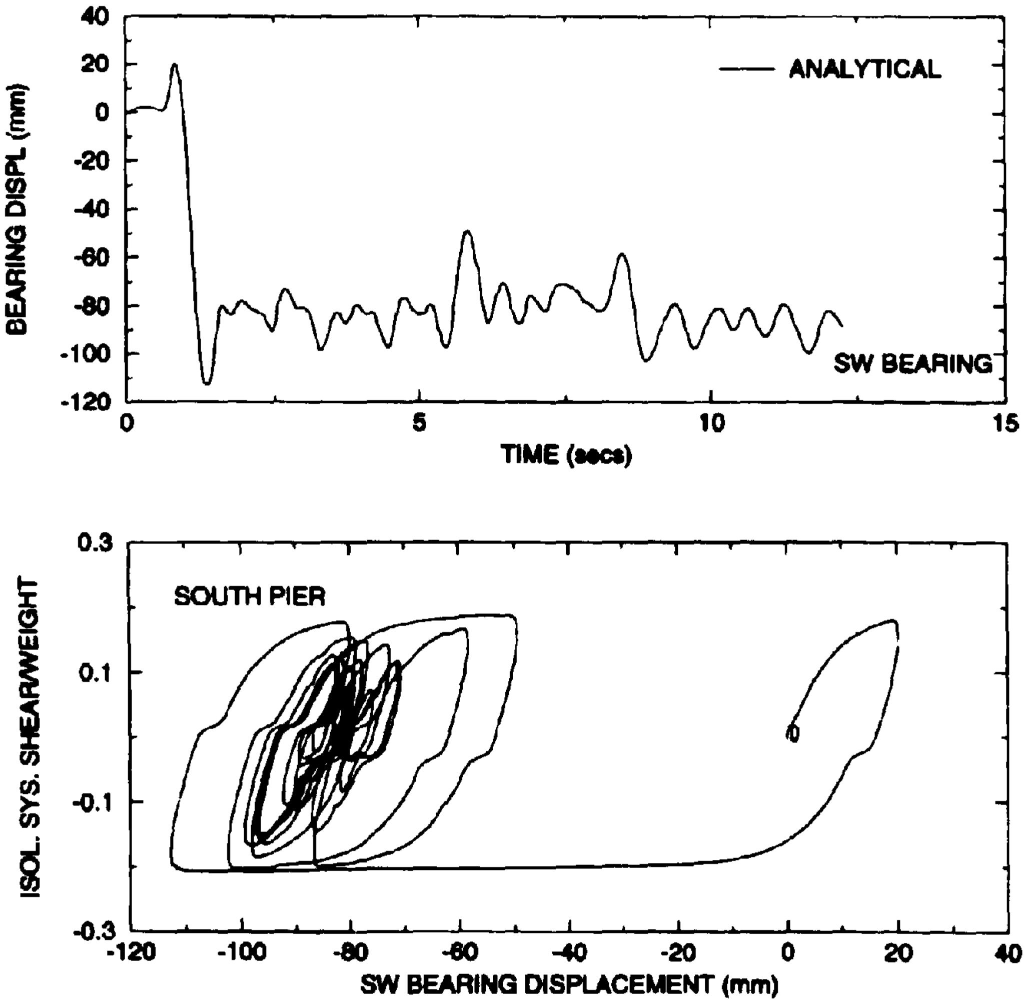


Figure 6-14 Analytical Response of Bridge Model with Flexible Piers to Japanese Level 2 G.C. 1 100% Input Motion.

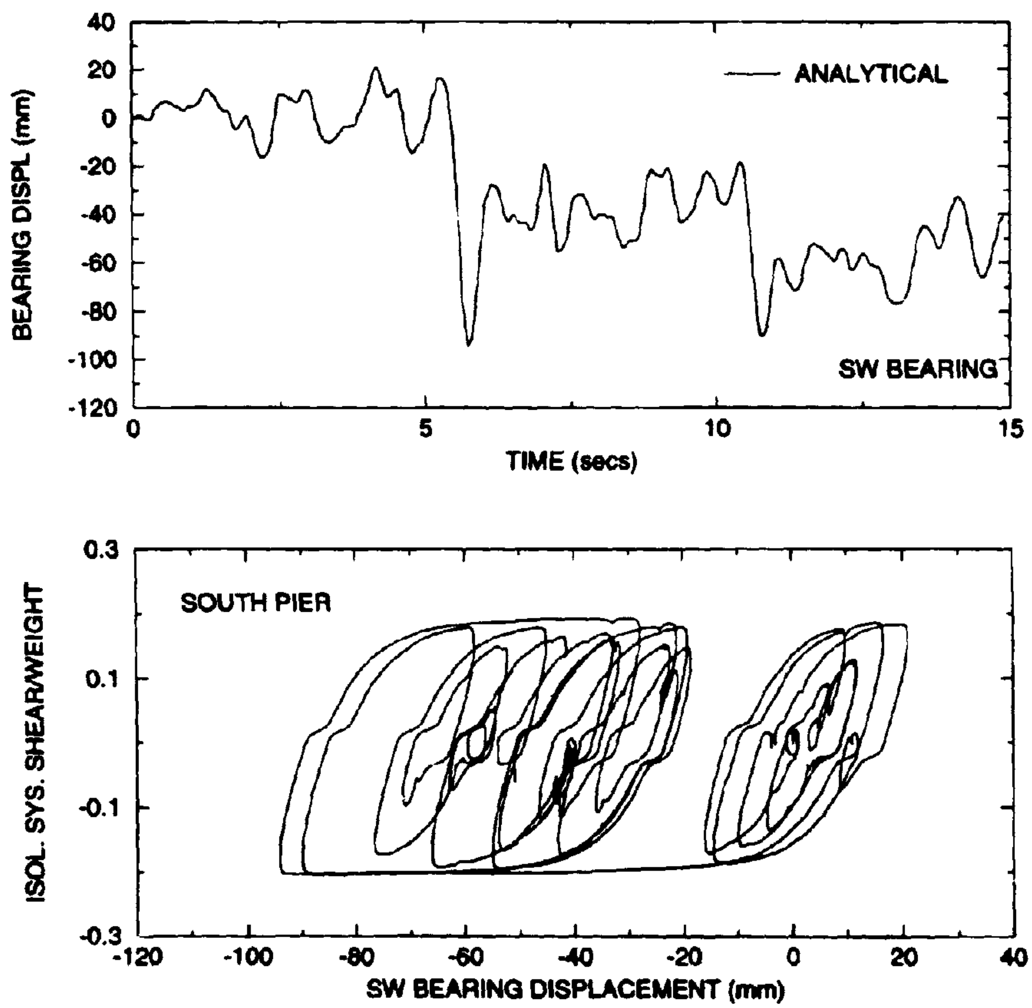


Figure 6-15 Analytical Response of Bridge Model with Flexible Piers to Japanese Level 2 G.C. 2 100% Input Motion.

It is apparent that the development of permanent displacements in systems which lack restoring force is dependent almost entirely on the details of the seismic excitation. Such systems are vulnerable to excitations which contain a single strong shock, but appear to perform well in excitations with a large number of cycles of motion. For the tested isolation system and sequence of seismic testing, it appears that permanent displacements are cumulative up to a certain limiting value. From Table 6-1, this limiting value is approximately equal to 30 mm, which corresponds to a surface steel strain of 0.017. This approximately equal to one half of the maximum allowable strain in the steel dampers. This implies that permanent displacements of up to approximately one half of the bearing displacement capacity may develop.

The lack of restoring force in this system represents a drawback which in the current design philosophy in the U.S. is penalized (AASHTO 1991). If the scaled Taft 400% motion represented the design earthquake for an isolated bridge with characteristics of the tested model, the design displacement should have been 4 X 60 = 240 mm (from test results after extrapolation to prototype scale). However, the 1991 AASHTO would require that the bearings be designed for a displacement capacity of 3 X 240 mm or 720 mm. With such requirement on the displacement capacity, the bearings would have been excessively large in comparison to other isolation systems which could be easily designed to have an isolation system force of 0.2 W with design displacement not exceeding 200 mm.

It is not clear what is the basis of the AASHTO, 1991 requirement for providing a displacement capacity of three times the peak dynamic displacement for systems without sufficient restoring force. The requirement may may have been based on the assumptions that the permanent displacement is equal to the peak dynamic displacement, that the design earthquake consists of three consecutive events and that permanent displacements are cumulative. These assumptions appear to be very conservative.

To investigate this we repeated the analyses of the tested bridge with the Sylmar 90° and Japanese Level 2, G.C. 2 input motions (see Figure 6-11 and 6-15), except that we applied the motions in three consecutive events. The time histories of bearing displacement are shown in Figure 6-16. The permanent displacements appear to accumulate to large values, however the peak bearing displacement is significantly less than three times the peak bearing displacement of the first event. We should note that the analytical model assumes infinite bearing displacement capacity. Actually, the bearings have limited displacement capacity. At steel strains beyond about 0.03, the dampers will exhibit stiffening behavior. This will prevent the accumulation of further permanent displacement, although at the expense of damage to the dampers and reduced fatigue life.

Thus, we may conclude that the 1991 AASHTO requirement for displacement capacity equal to three times the peak bearing displacement is very conservative.

6.3 Comparison to other Isolation Systems

A number of bridge isolation systems have been tested in the NCEER-Taisei Corporation bridge isolation project. All systems have been tested with the same bridge model and seismic excitations. A comparison of experimental response of the tested systems is instructive. For this purpose we compare the recorded response of the six systems described in Table 6-III in the Taft and El Centro motions. The results for the bridge with stiff piers are graphically presented as functions of increasing peak table velocity and compared in Figures 6-17 and 6-18. The comparison shows that all systems have comparable deck accelerations, and thus forces transmitted to the substructure. However, bearing displacements are larger in the elastoplastic system. This is entirely a result of the elastoplastic nature of the system. On this we note that the other systems had less characteristic strength than the elastoplastic system, yet they developed less bearing displacement. This is true even for the T2-No.1 system which has very weak restoring force and does not meet the 1991 AASHTO minimum requirements for restoring force (Tsopelas 1994a). This comparison should demonstrate the significance of restoring force.

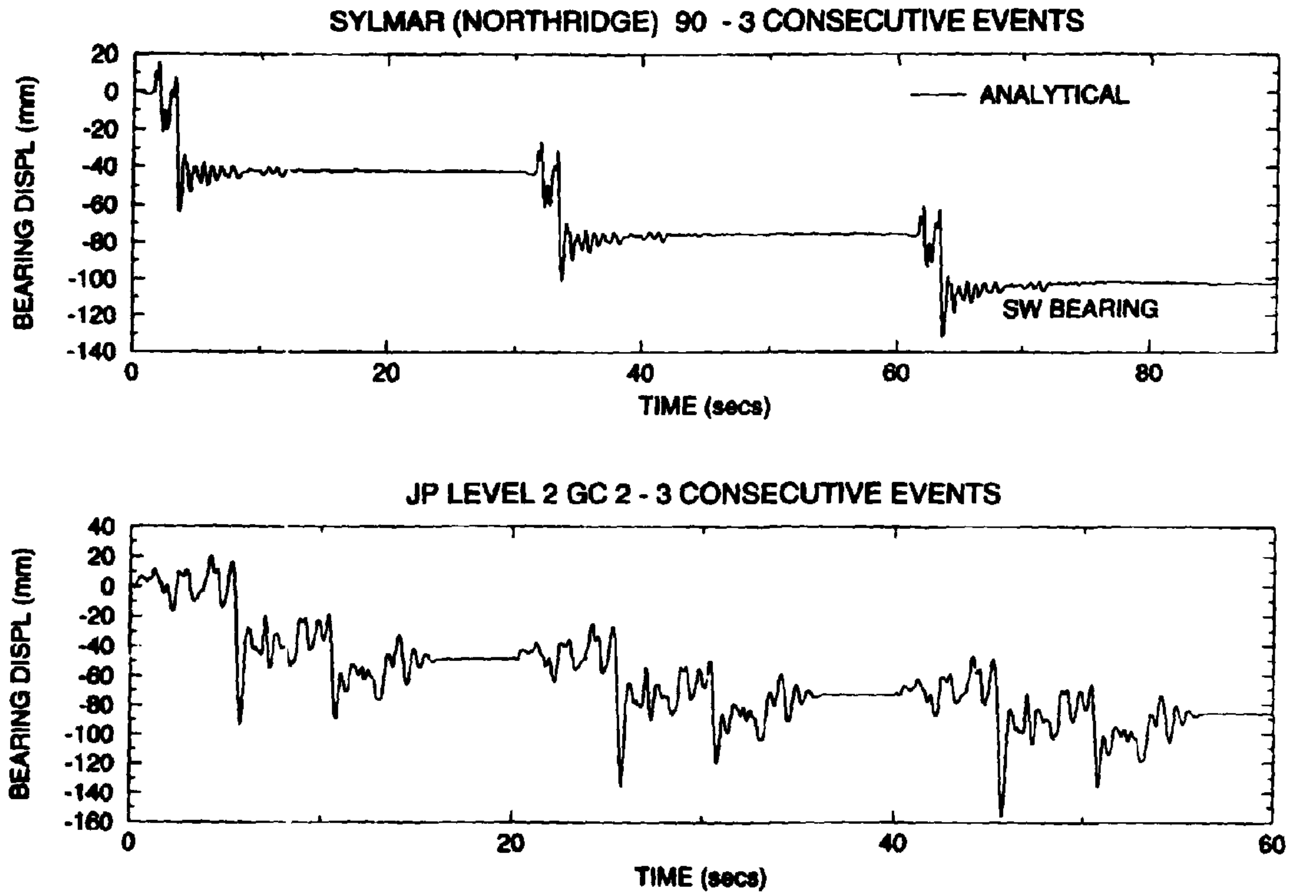


Figure 6-16 Calculated Bearing Displacement Histories of Tested Bridge in Input Consisting of Three Consecutive Events.

Table 6-III Description of Compared Isolation Systems

Isolation System	Designation	Description	Characteristic Strength/Deck Weight	Isolation Period at Quarter Length Scale (secs)	Reference
Friction Pendulum System	FPS1	Spherical Sliding Bearing	0.104	1.5	Constantinou 1993
Friction Pendulum System	FPS2	Spherical Sliding Bearing	0.120	1.5	Constantinou 1993
Sliding with Restoring Force	T2-No.1	Flat PTFE Sliding Bearing and Rubber Restoring Force Device	0.138	2.47	Tsopelas 1994a
Sliding with Restoring Force	T2-No.2	Flat PTFE Sliding Bearing and Rubber Restoring Force Device	0.138	1.6	Tsopelas 1994a
Sliding with Restoring Force	T2-No.3	Flat PTFE Sliding Bearing and Rubber Restoring Force Device	0.138	1.33	Tsopelas 1994a
Elastoplastic	E-DAMPERS	Lubricated PTFE Sliding Bearing with E-shaped Steel Dampers	0.200	Theoretically Infinite	This Report

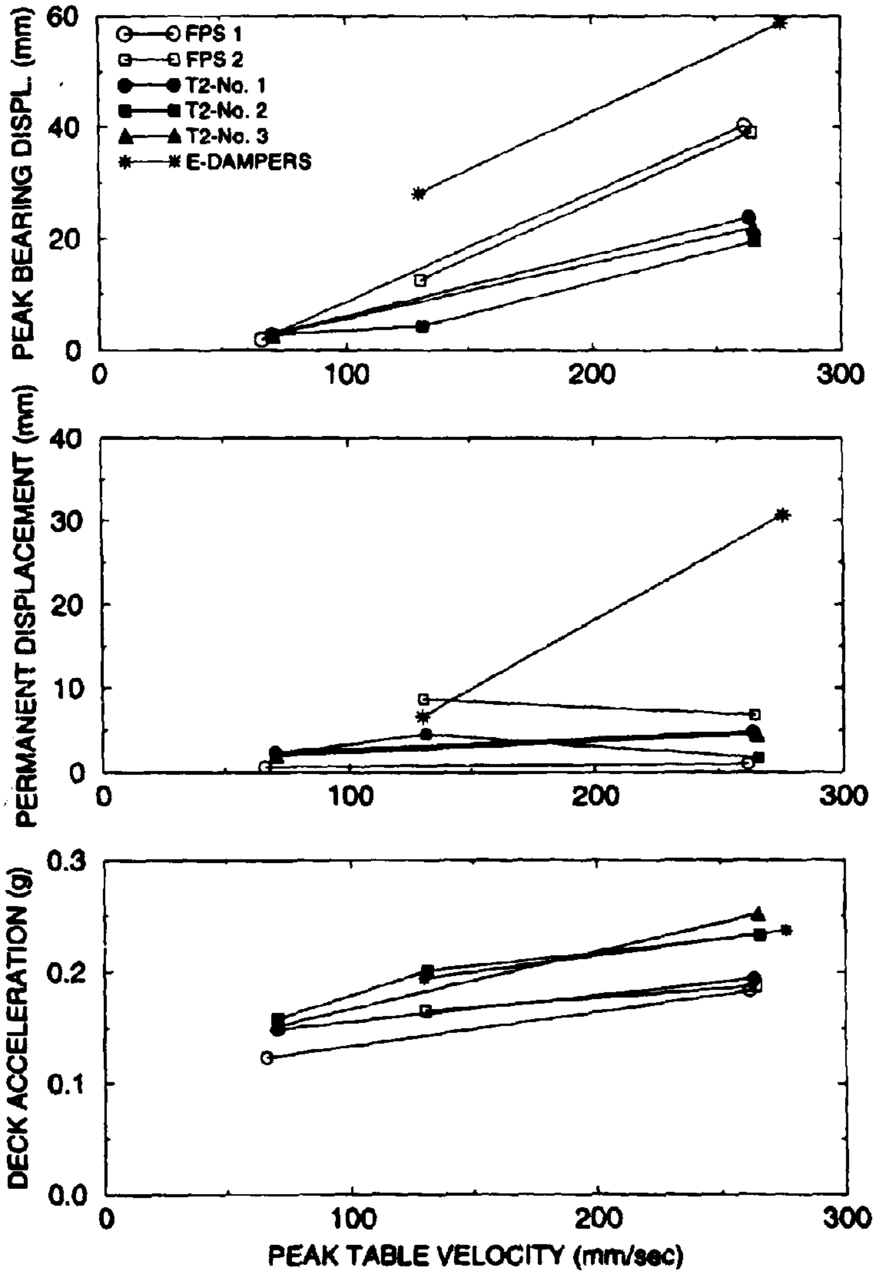


Figure 6-17 Comparison of Response of Isolated Bridge with Stiff Piers and Different Isolation Systems as Function of Increasing Intensity for the Taft N21E Input Motion.

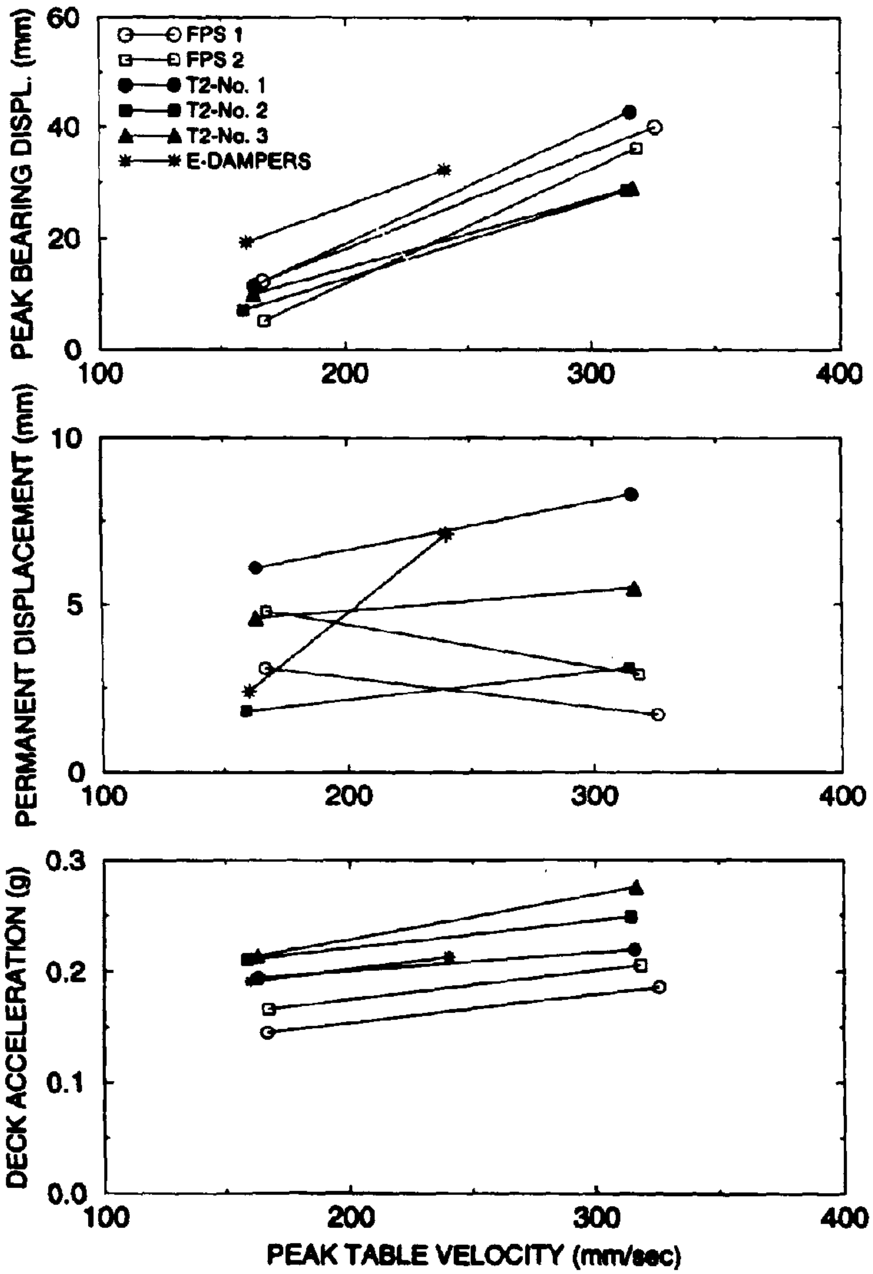


Figure 6-18 Comparison of Response of Isolated Bridge with Stiff Piers and Different Isolation Systems as Function of Increasing Intensity for the El Centro S00E Input Motion.

SECTION 7

ANALYTICAL PREDICTION OF RESPONSE

7.1 Introduction

Analytical techniques for predicting the dynamic response of sliding isolation systems are available (Mokha 1988, 1990 and 1991; Constantinou 1990a, 1990b, 1991a 1991b and 1993, Tsopelas 1994a and 1994b). These analytical techniques have been modified herein for the prediction of the response of the tested bridge model. The analytical model accounts for the pier flexibility, pier top rotation, and nonlinear hysteretic characteristics of the isolators.

7.2 Analytical Model

Figure 7-1 shows the analytical model in the case of the bridge with flexible piers. The degrees of freedom are selected to be the deck displacement with respect to the table, U_d, the pier displacements with respect to the table, U_{p1} and U_{p2}, and the pier rotations, ϕ_{p1} and ϕ_{p2}.

Each pier is modeled by a beam element of length L_i, moment of inertia I_i and modulus of elasticity E_i (i=1 or 2). The beam element is fixed to the table and connected at its top to a rigid block of height h, mass m_{pi} and mass moment of inertia about the center of mass (C.M.) I_{pi}. The center of mass is located at distance h_i from the bottom of the block. This block represents the pier top.

Free body diagrams of the deck and pier tops of the bridge model are shown in Figure 7-2. It should be noted that it was assumed that there is no transfer of moment between the deck and the supporting pier top. In reality, there is some transfer of moment due to the rotational stiffness of the supporting pot with rubber of the sliding bearings. The equations of motion are derived by consideration of dynamic equilibrium of the deck and piers in the horizontal direction and of the piers in the rotational direction :

$$m_d(\ddot{U}_d + \ddot{U}_g) + F_{b1} + F_{b2} = 0 \tag{7-1}$$

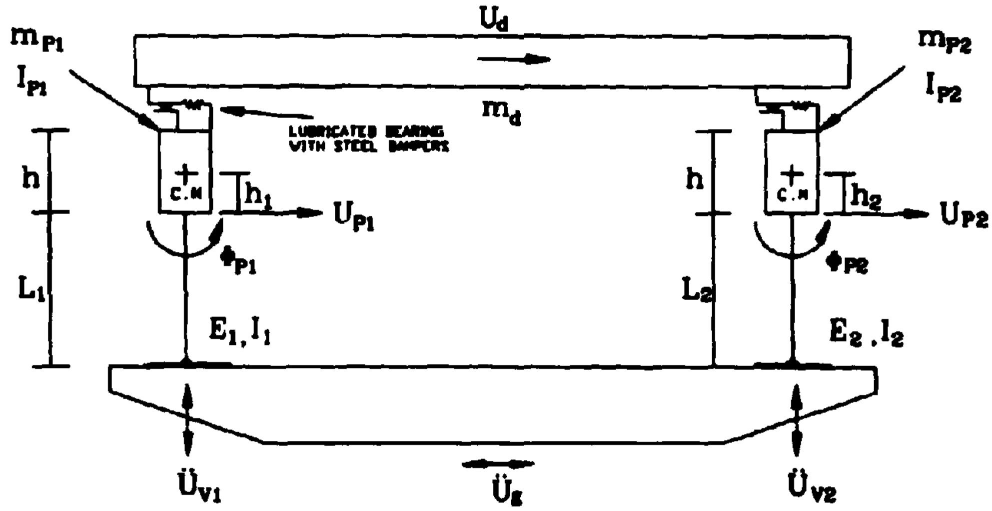


Figure 7-1 Longitudinal Direction Model of Isolated Bridge.

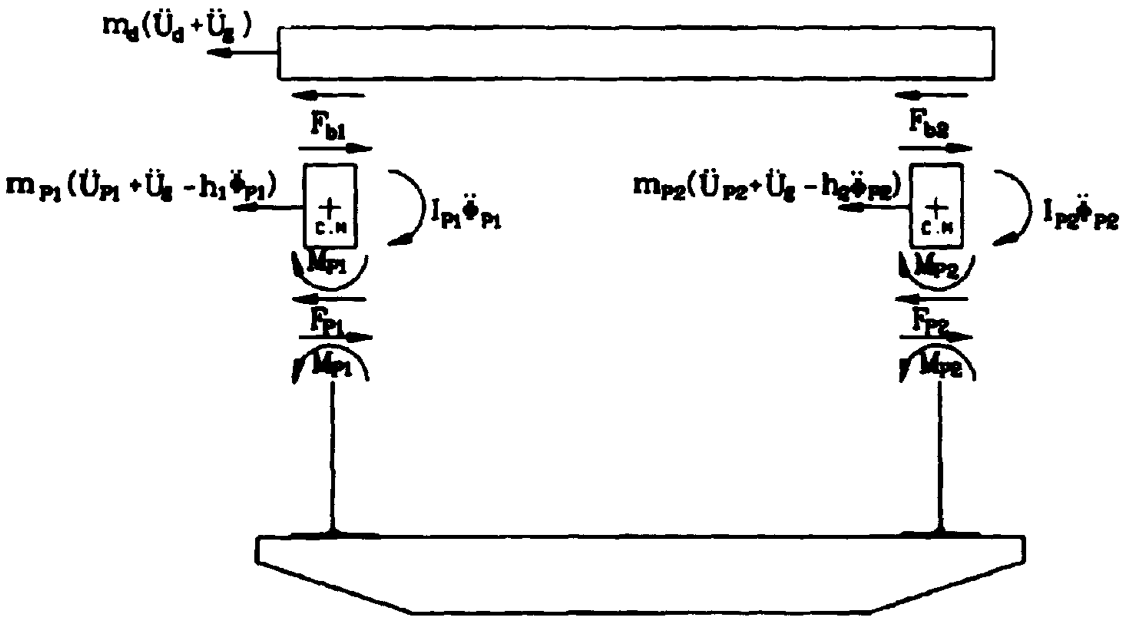


Figure 7-2 Free Body Diagram of Bridge Model.

$$m_{p1}(\ddot{U}_{p1}+\ddot{U}_g-h_1\ddot{\phi}_{p1})+F_{p1}-F_{b1}=0 \tag{7-2}$$

$$m_{p2}(\ddot{U}_{p2}+\ddot{U}_g-h_2\ddot{\phi}_{p2})+F_{p2}-F_{b2}=0 \tag{7-3}$$

$$I_{p1}\ddot{\phi}_{p1}+M_{p1}+F_{p1}h_1+F_{b1}(h-h_1)=0 \tag{7-4}$$

$$I_{p2}\ddot{\phi}_{p2}+M_{p2}+F_{p2}h_2+F_{b2}(h-h_2)=0 \tag{7-5}$$

where $\ddot{U}_g$ is the horizontal table (ground) acceleration, F_{b1} and F_{b2} are the lateral forces in the isolation system (sliding bearings, E-shaped dampers), and F_{pi} and M_{pi} are the lateral force and bending moment at the connection of the pier top to the end of the column:

$$\begin{Bmatrix} F_{pi} \\ M_{pi} \end{Bmatrix} = E_iI_i \begin{bmatrix} \frac{12}{L_i^3} & \frac{6}{L_i^2} \\ \frac{6}{L_i^2} & \frac{4}{L_i} \end{bmatrix} \begin{Bmatrix} U_{pi} \\ \phi_{pi} \end{Bmatrix} + \begin{bmatrix} C_{pi}^1 & 0 \\ 0 & C_{pi}^2 \end{bmatrix} \begin{Bmatrix} \dot{U}_{pi} \\ \dot{\phi}_{pi} \end{Bmatrix} \tag{7-6}$$

The first part of Equation (7-6) describes the elastic forces, whereas the second part is used to account for linear viscous energy dissipation in the piers.

Forces F_{bi} (i=1,2) include a component from friction in the sliding bearings and a component from the E-shaped steel damping devices. These forces are described as follows:

$$F_{bi}=\pm\mu_iW_i+F_{Ei} \tag{7-7}$$

where μ_i = coefficient of sliding friction at pier i (assuming Coulomb friction). W_i = normal load on two sliding interfaces at pier i and F_{Ei} = force from four E-shaped steel dampers at pier i. Furthermore, U_{bi} is the bearing displacement at pier i:

$$U_{bi}=U_d-U_{pi}+h\phi_{pi} \tag{7-8}$$

7.3 Analytical Model for E-Shaped Mild Steel Damper

An E-shaped steel damper has essentially elastoplastic behavior with very small post-yielding stiffness. The tested dampers had also slip-lock behavior due to slippage at the joint connecting the central leg of the damper to the sliding part of the bearing. This connection become loose with repeated testing so that a total slip of about 3.6 mm was possible.

Modeling of the slip-lock behavior of scaled dampers appeared to be of significance. The behavior of the damper was modeled by a smooth bilinear hysteretic element in series with a slip-lock element as described by Baber 1988. The model is depicted in Figure 7-3. The resultant displacement of the element is the sum of the displacements from the hysteretic and the slip-lock elements. A mathematical expression relating force to displacement is given by

$$F_{Ei} = \alpha_i \frac{F_{yi}}{Y_i} U_{bi} + (1 - \alpha_i) F_{yi} Z_i \tag{7-9}$$

where F_{yi} is the yield force of the damper, α_i is the post to pre-yielding stiffness ratio, Y_i is the yield displacement and U_{bi} is the displacement of the interior leg of the E-shaped damper with respect to the exterior legs. This displacement may be expressed as

$$U_{bi} = U_{bi1} + U_{bi2} \tag{7-10}$$

where U_{bi1} is the displacement of the hysteretic element and U_{bi2} is the displacement of the slip-lock element. The latter can be described by a rate equation of the form proposed by Baber 1988 :

$$\dot{U}_{bi2} = \sqrt{\frac{2}{\pi}} \frac{a}{\sigma} \exp\left[-\frac{Z_i^2}{2\sigma^2} \right] \dot{Z}_i \tag{7-11}$$

This expression will convey a slip of $2a$ when Z_i changes sign. σ is a constant that controls the transition between slip to lock phases. Furthermore, variable Z_i in Equations (7-9) and (7-11) satisfies the following equation, which was proposed by Bouc 1971 and modified by Wen 1976 :

$$Y_i \dot{Z}_i + \gamma |\dot{U}_{bi}| Z_i |Z_i| + \beta \dot{U}_{bi} Z_i^2 - \dot{U}_{bi} = 0 \tag{7-12}$$

In this equation, β and γ are parameters satisfying the condition $\beta+\gamma=1$.

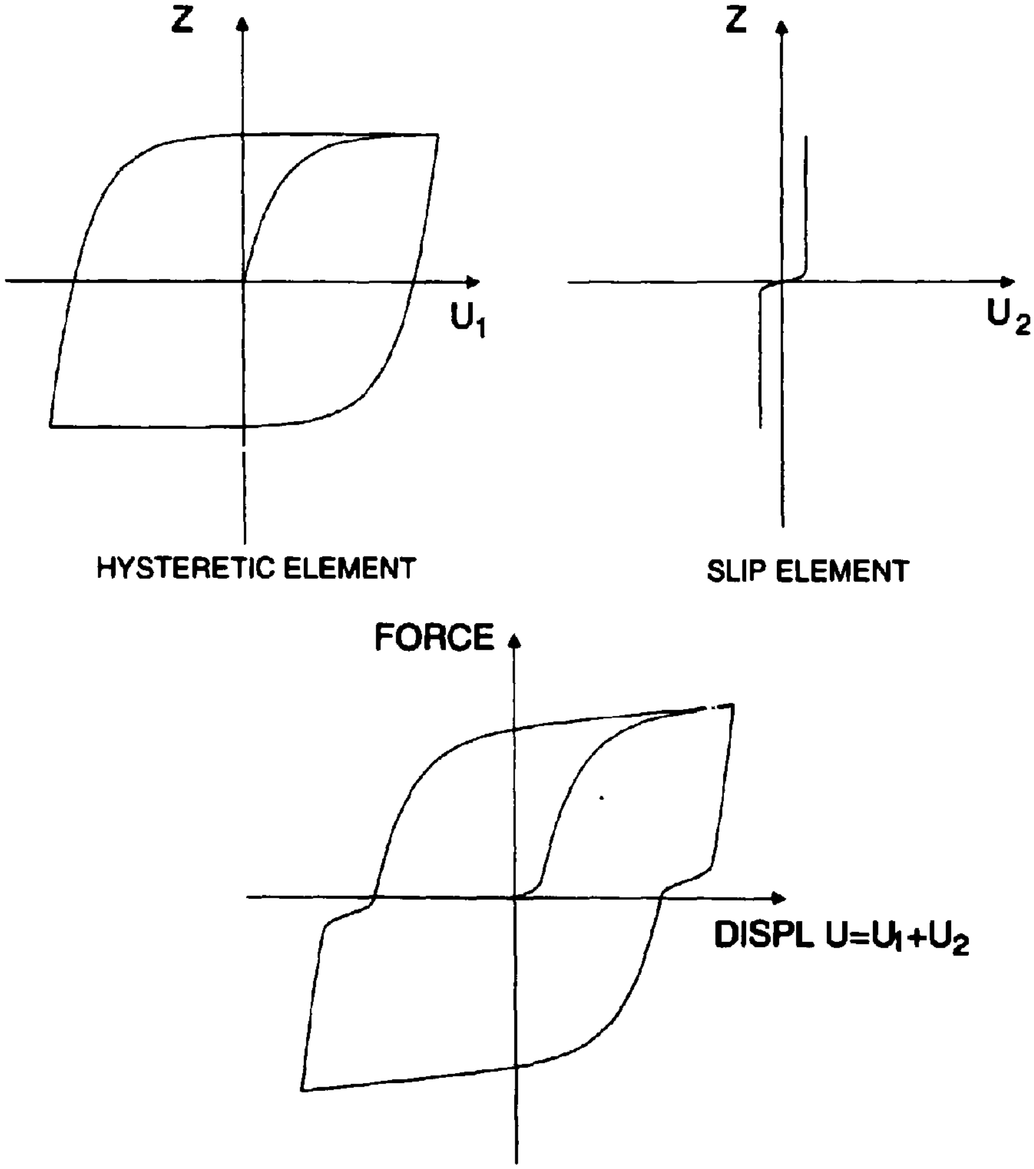


Figure 7-3 Hysteretic Slip-Lock Element Resulting from Combination of Hysteretic and Slip Elements.

Table 7-I presents values of the parameters in the model of the E-shaped damping device (Equations 7-9 to 7-11). It should be noted that the parameters presented in Table 7-I correspond to properties of two isolators, thus 4 E-shaped mild steel dampers. The parameters were determined from analysis of experimental results over the entire range of displacements (0 to 50 mm). The calibration was based on results of dynamic sinusoidal tests at specified frequency and amplitude. Figures 7-4 and 7-5 compare experimental loops of the isolators to the predictions of the calibrated model of Equations (7-9) to (7-12). The analytical prediction is seen to be good at large amplitude motions, whereas discrepancies between analytical and experimental results are evident in the small amplitude motions. Particularly, the model does not properly represent the ascending branch of the initial cycle of motion.

Table 7-I Parameters in Calibrated Model of E-Shaped Mild Steel Damping Device

Parameter	South Pier	North Pier
F_y (kN)	11.56	11.12
Y (mm)	7.62	7.62
α	0.0045	0.0045
σ	0.03	0.03
a (mm)	1.8	1.8

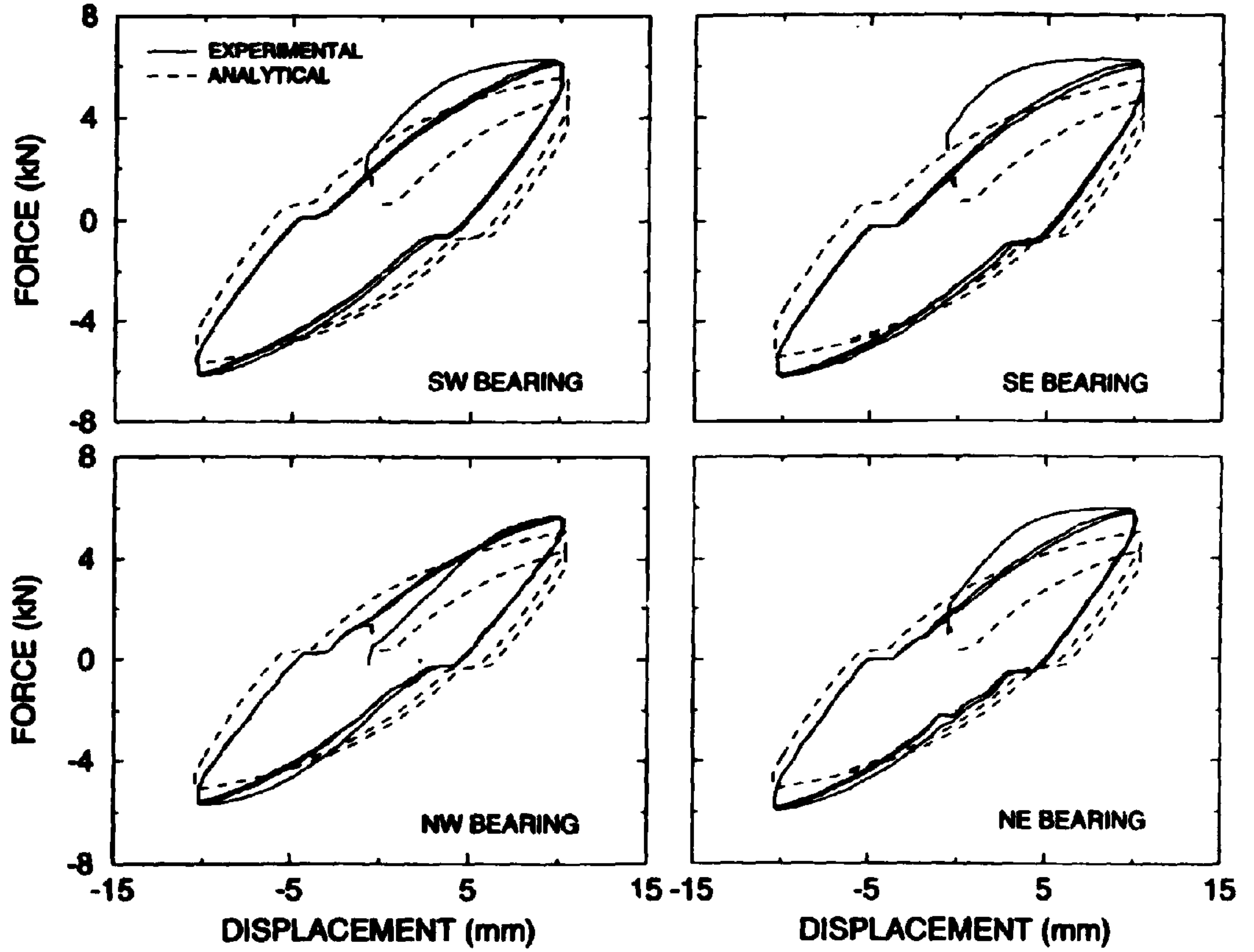


Figure 7-4 **Comparison of Experimental and Analytical Force-Displacement Loops of Isolators. Sinusoidal Motion of Frequency 0.03 Hz and Amplitude of 12.45 mm.**

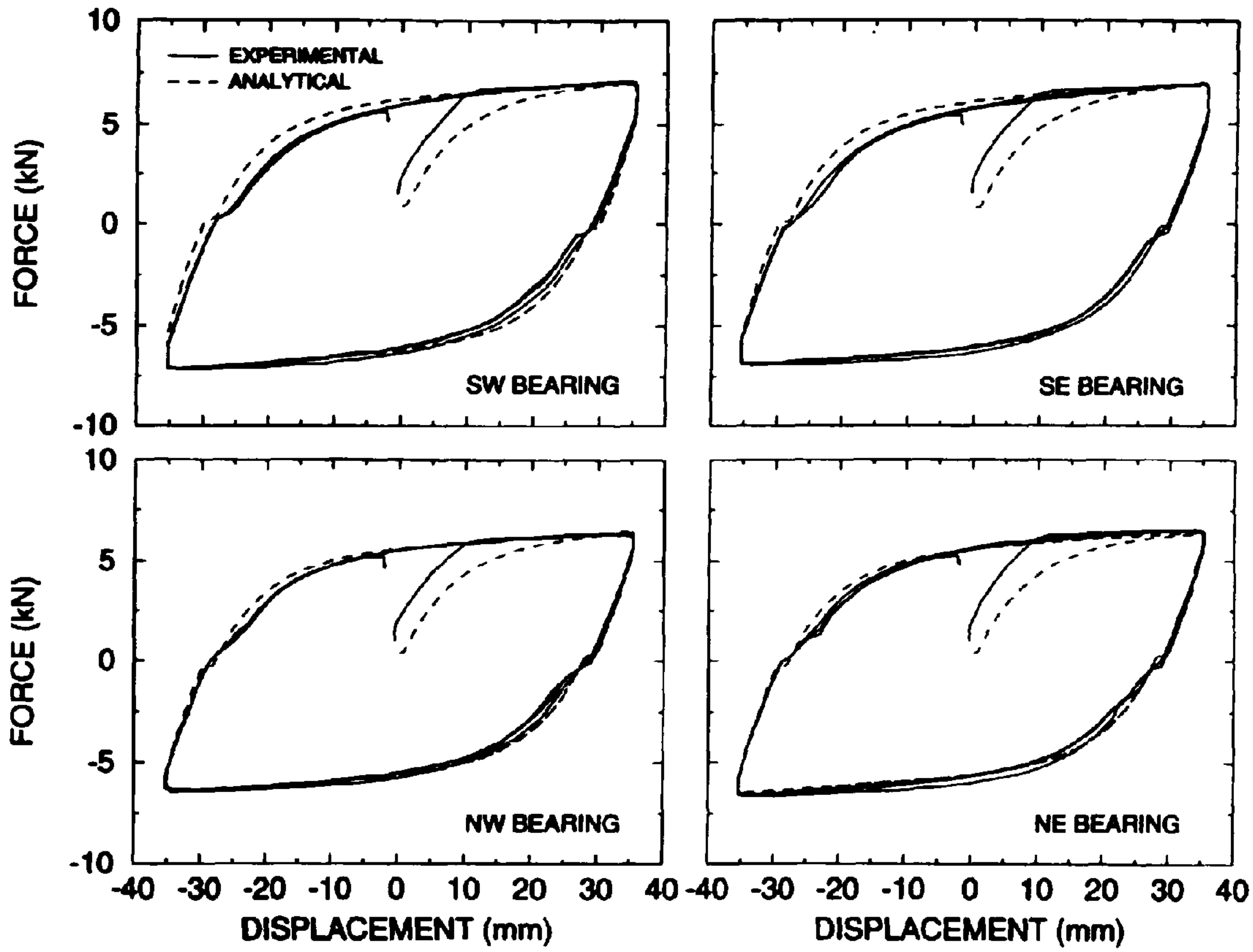


Figure 7-5 **Comparison of Experimental and Analytical Force-Displacement Loops of Isolators. Sinusoidal Motion of Frequency 0.03 Hz and Amplitude of 38.1 mm.**

7.4 Comparison of Analytical and Experimental Results

The equations of motion of the isolated bridge model are Equations (7-1) to (7-8) with force F_{Bi} described by Equations (7-9) to (7-12). Furthermore, the condition $sgn(U_{bi2})=sgn(U_{bi})$ has to be imposed for the numerical solution of the equations.

Solution of the governing Equations (7-1) through (7-12) was obtained by first reducing the equations to a system of first order differential equations and then numerically integrating the system by using an adaptive integration scheme with truncation error control (Gear 1971). The initial conditions included the initial displacement (that is permanent displacement from previous test) and the associated friction force. The former required the specification of the initial value of Z_i (Equations 7-9 to 7-12). Since at start $F_{Bi} = 0$, it follows that

$$Z_i = -\frac{\alpha \frac{F_{yi}}{Y_i} U_{oi} \pm \mu_i W_i}{(1-\alpha) F_{yi}} \tag{7-13}$$

where U_{oi} = initial displacement.

The data used in the analytical model were : deck weight $m_d g$ = 140 kN, pier weight $m_p g$ = 8.9 kN, $L_1 = L_2$ = 1600 mm, $h_1 = h_2$ = 98 mm, h = 413 mm, $I_{p1} = I_{p2}$ = 38.22 kN s^2 mm, $E_1 = E_2$ = 200000 MPa, $I_1 = I_2$ = 3.022x10^{-5} m^4 (2 AISC tubes Ts 6x6x5/16). Based on these data the fundamental period of each pier, in its cantilever position, was calculated to be 0.092s. This is in close agreement with the experimentally determined value of 0.096s. The second mode of the cantilever pier had a calculated frequency of 102 Hz. This frequency could neither be detected in the tests nor have any significance in the analysis.

Damping in the piers was described by the second term in Equation (7-6). The fact that the calculated second frequency of the cantilever pier is much larger than the first frequency indicates that the second mode of the pier may be neglected. Accordingly, constant C^2_{pi} in Equation (7-6) was set equal to zero and constant C'_{pi} was assigned a value equal to 0.0062 kNs/mm. Based on this value, the damping ratio in the fundamental mode of the cantilever pier was calculated to be 5% of critical. This is consisted with the experimental data.

The coefficient of friction in the bearings was specified as 0.01 at the north pier and 0.02 at the south pier locations, in accordance with the experimental result (see Figure 3-8).

Comparisons of analytical and experimental results are presented in Figures 7-6 to 7-11. The analytical results are in relatively good agreement with the experimental results. Permanent displacements, isolation system forces and pier shear forces are predicted with good accuracy. The details of bearing displacement histories are also predicted well, but peak bearing displacements are typically overpredicted.

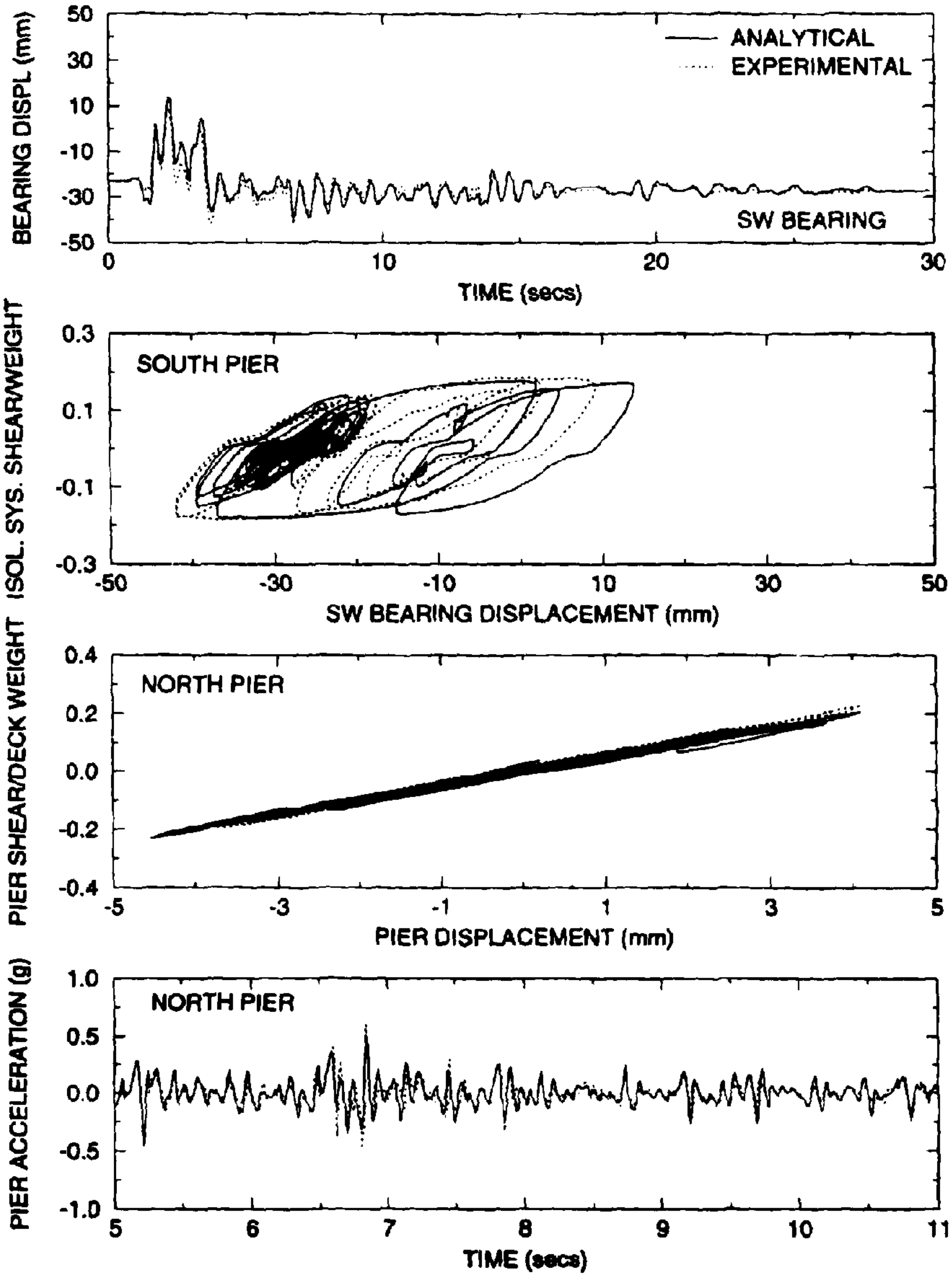


Figure 7-6 Comparison of Experimental and Analytical Results in Test with El Centro S00E 150% Input (Test No.ITBR10).

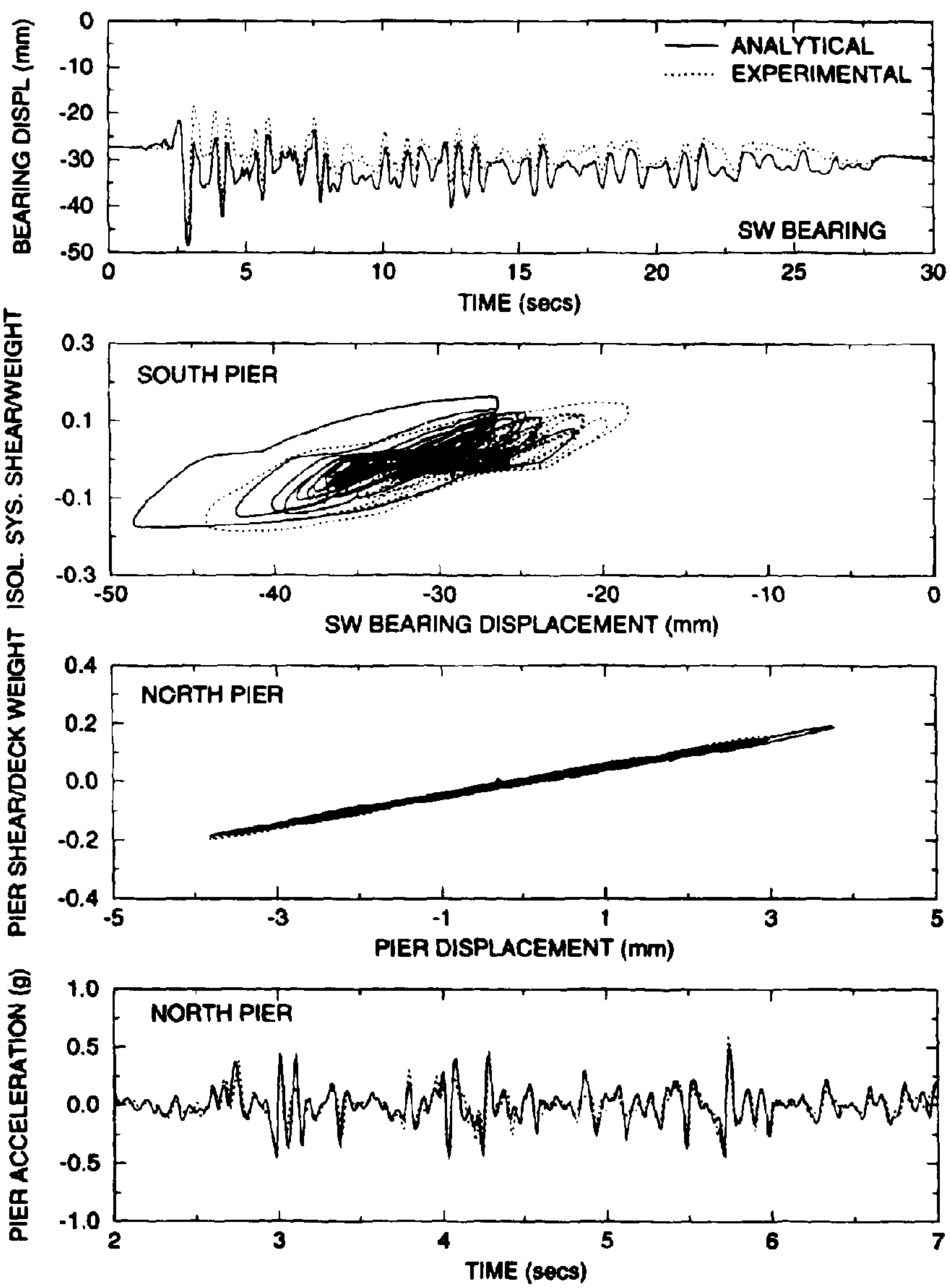


Figure 7-7 Comparison of Experimental and Analytical Results in Test with Taft N21E 200% Input (Test No.ITBR11).

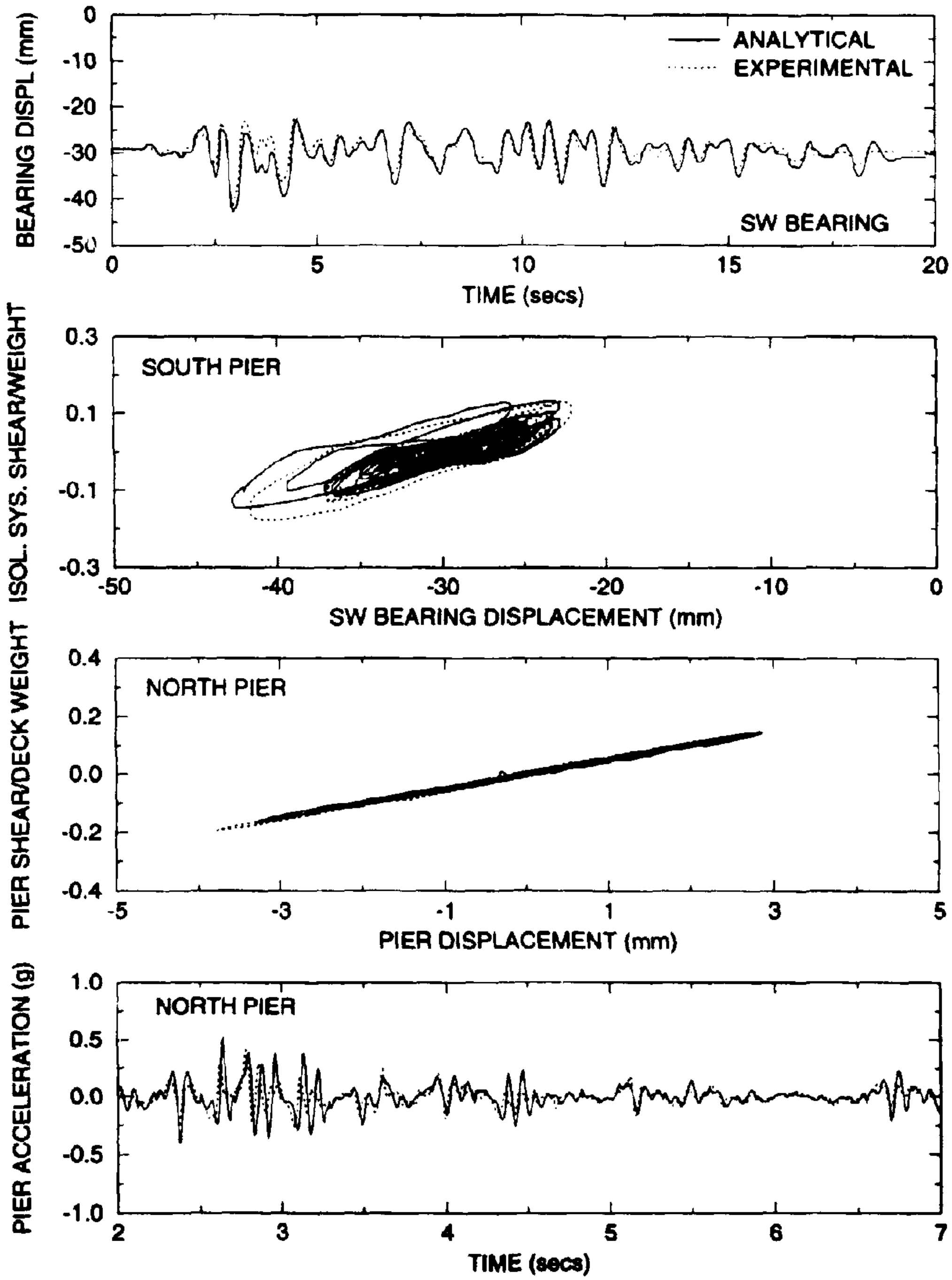


Figure 7-8 Comparison of Experimental and Analytical Results in Test with Hachinohe N-S 100% Input (Test No.ITBR12).

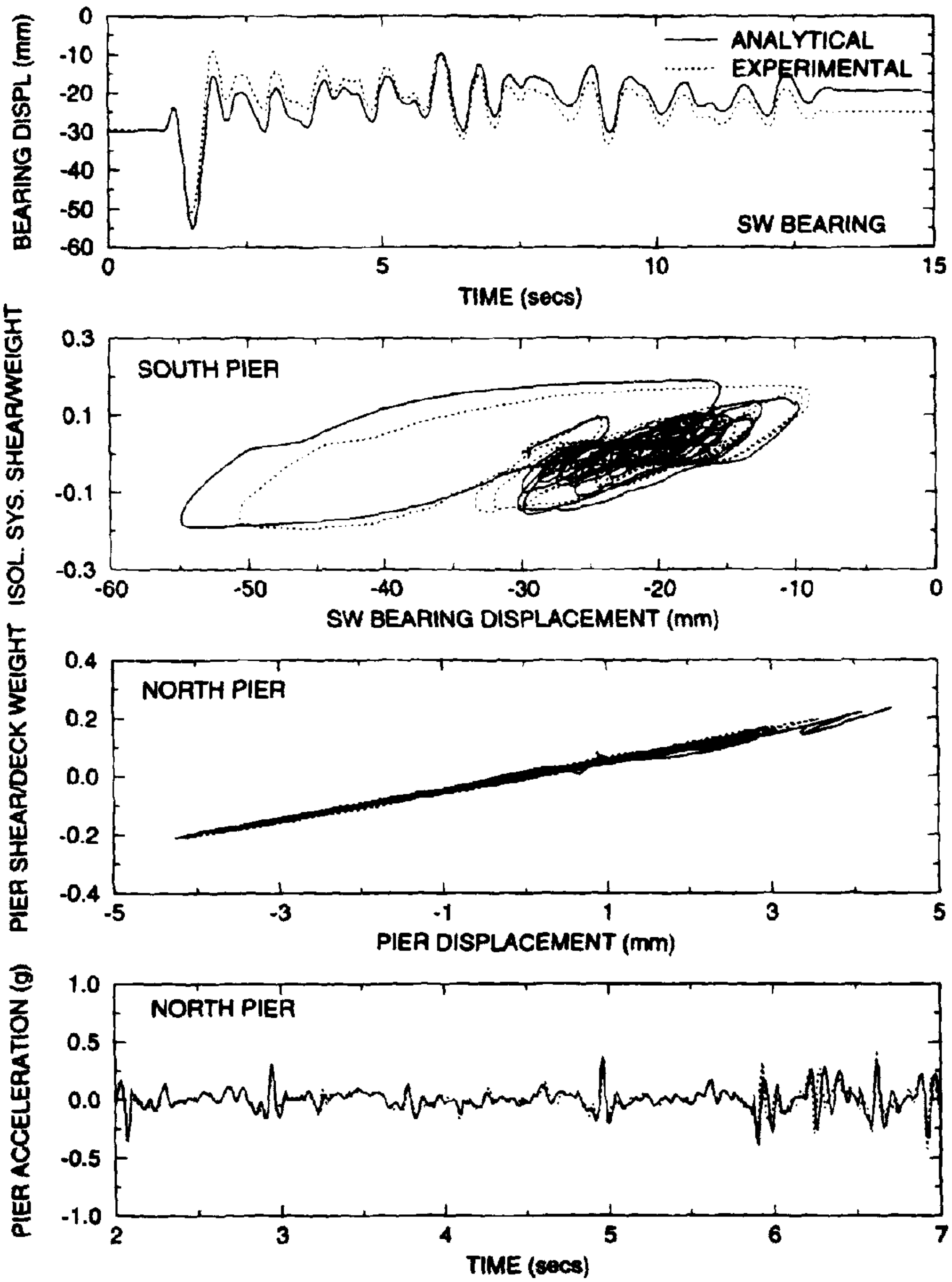


Figure 7-9 Comparison of Experimental and Analytical Results in Test with Japanese Level 2 G.C.1 50% Input (Test No.ITBR13).

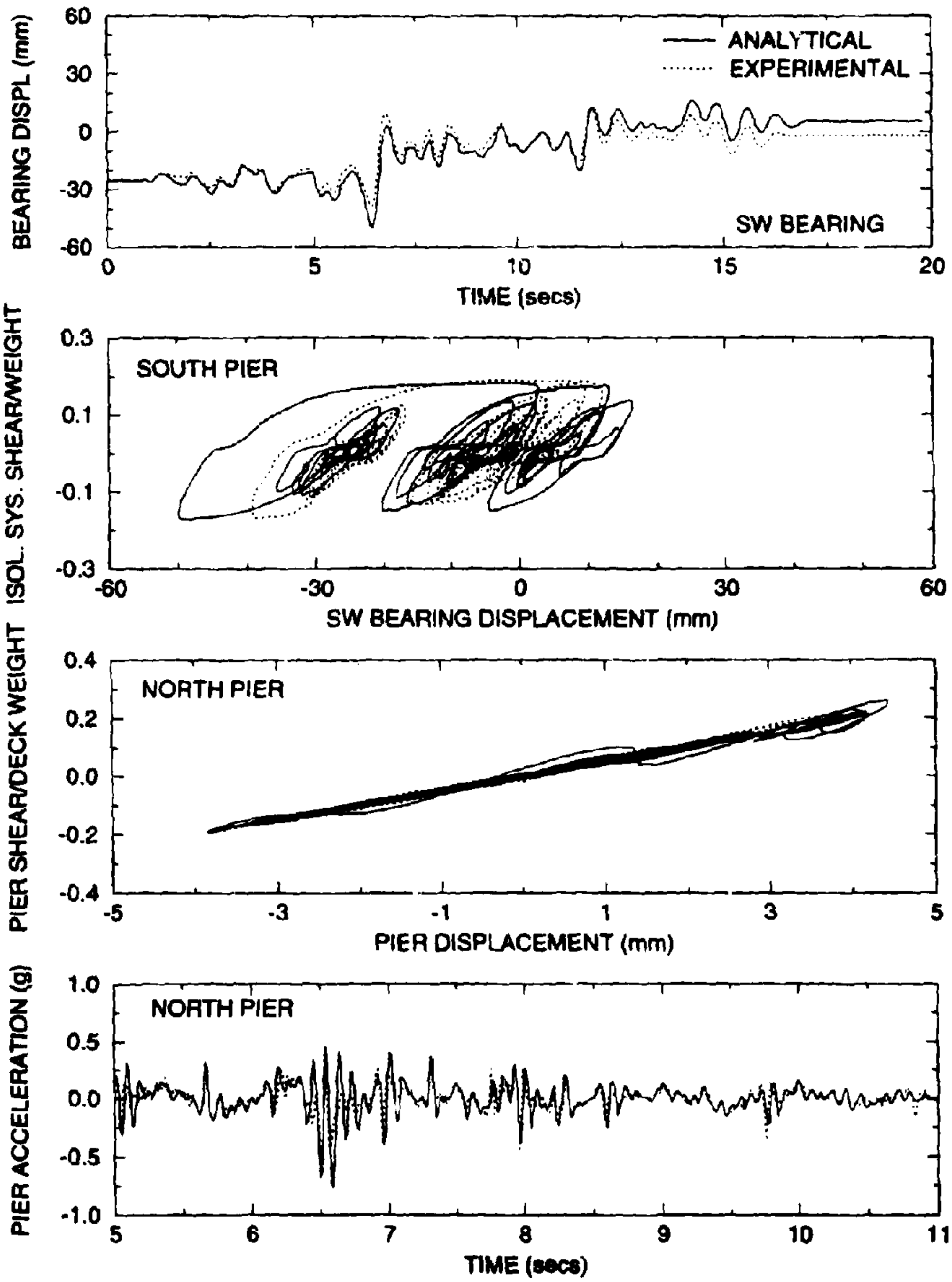


Figure 7-10 Comparison of Experimental and Analytical Results in Test with Japanese Level 2 G.C.2 50% Input (Test No.ITBR14).

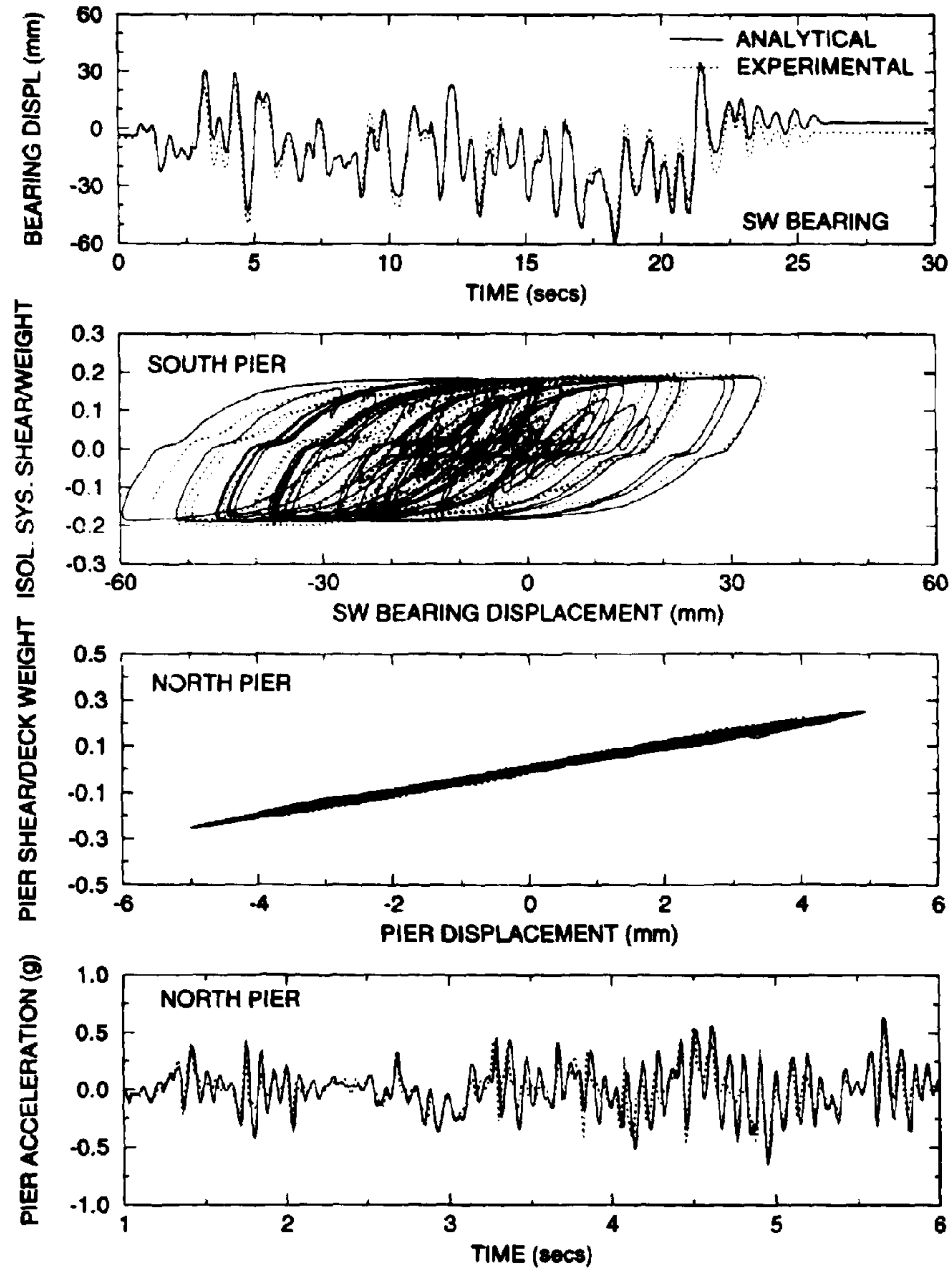


Figure 7-11 Comparison of Experimental and Analytical Results in Test with Japanese Level 2 G.C.3 75% Input (Test No.ITBR16).

SECTION 8

CONCLUSIONS

This report presented an experimental study of the seismic response of an isolated bridge and a comparison of its response to that of a comparable non-isolated bridge. The isolation system consisted of lubricated sliding bearings and E-shaped mild steel devices. The lubricated sliding bearings carried the weight of the deck. Energy dissipation capability was primarily provided by the E-shaped devices. The system had insignificant restoring force. The system was designed for strong seismic excitation.

The conclusions of the study are :

(1) The E-shaped mild steel dampers exhibited stable hysteretic characteristics over a large number of cycles. Specifically, after a number of identification and seismic tests the isolators were subjected to 25 cycles of sinusoidal motion at peak displacement of nearly 50mm, which corresponds to a peak steel surface strain of 0.03, local ductility of 15 and global ductility of about 9. The dampers maintained their properties up to the last cycle, at which one of them failed. This was thought to be very good performance.

(2) The isolation system limited the force transmitted to the substructure to a value approximately equal to design characteristic strength of the system, which was equal to 0.2 times the deck weight. Thus, the system showed insensitivity to the content in frequency and intensity of excitation, provided that bearing displacements were within its capacity.

(3) The isolation system developed significant permanent displacements, which for some earthquakes exceeded 50% of the bearing's displacement capacity. These earthquakes typically caused response containing primarily a single cycle of motion, as if they consisted of a single strong shock. Recorded motions such as the 1952 Taft, 1971 Pacoima Dam and several of the 1994 Northridge earthquake exhibited this behavior. In contrast, in earthquake motions with a large number of

cycles, such as in the simulated deep soil (ground condition 3) Japanese level 2 motions, the isolation system developed small permanent displacements. The recentering of the isolated bridge in these cases was entirely accomplished by the action of the deck inertia forces.

(4) The permanent displacements were found to be cumulative up to a certain limit, which for the tested system and sequence of testing was equal to approximately one half of the bearing displacement capacity. Analytical studies with input consisting of three identical consecutive events showed that the peak bearing displacement is significantly less than three times the peak displacement in the first event.

(5) According to 1991 AASHTO provisions, the tested isolation system would have been penalized for the lack of restoring force by requiring to have a displacement capacity equal to three times the peak dynamic displacement. However, this study indicates that this requirement is very conservative and is in need of revision.

(6) Restoring force capability, even when limited, is important in reducing permanent displacements. This has been demonstrated by comparison of the tested isolation system to other sliding isolation systems which have been previously tested with the same bridge model and seismic excitations (Constantinou 1993, Tsopelas 1994a).

(7) An analytical model consisting of a smooth bilinear element in series with a slip-lock element is adequate in describing the behavior of the isolators with acceptable accuracy. Analyses of the dynamic response of the tested isolated bridge showed reasonably good agreement of experimental and analytical results.

SECTION 9

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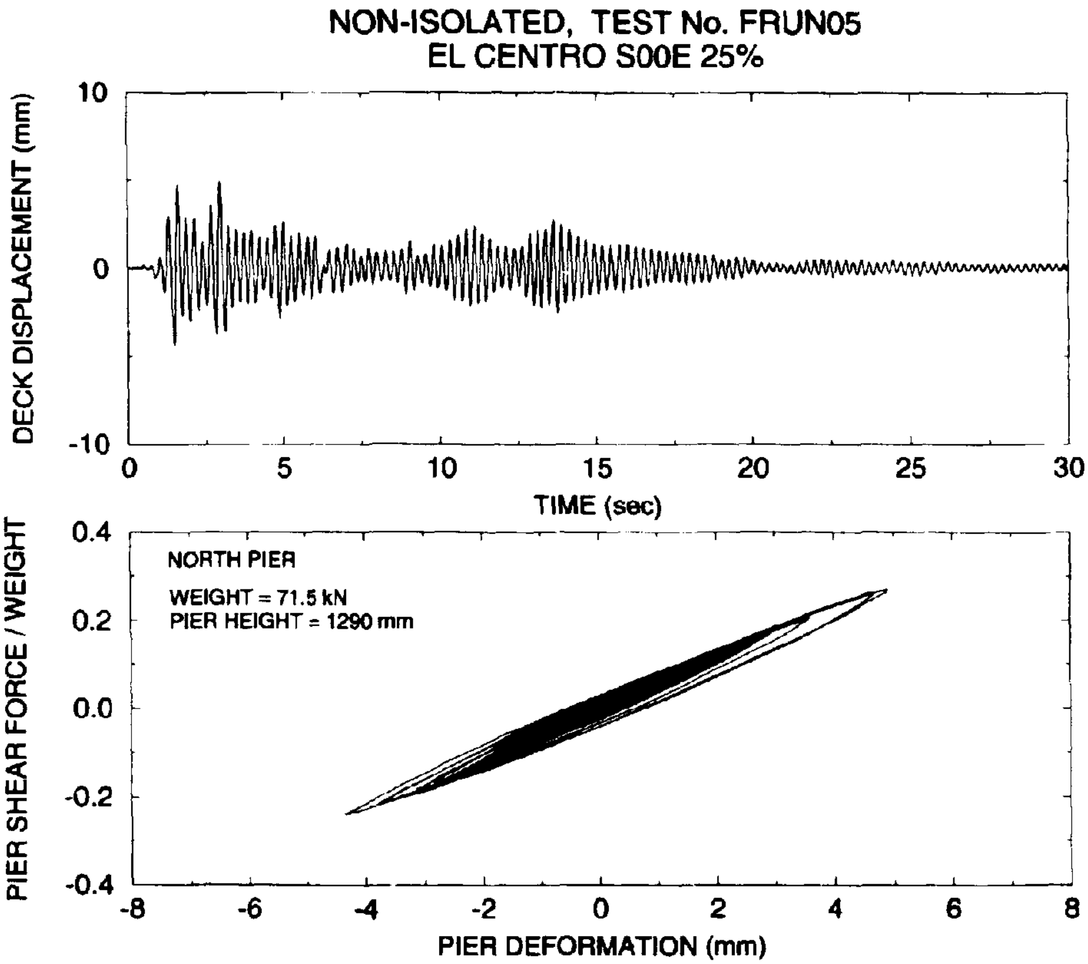
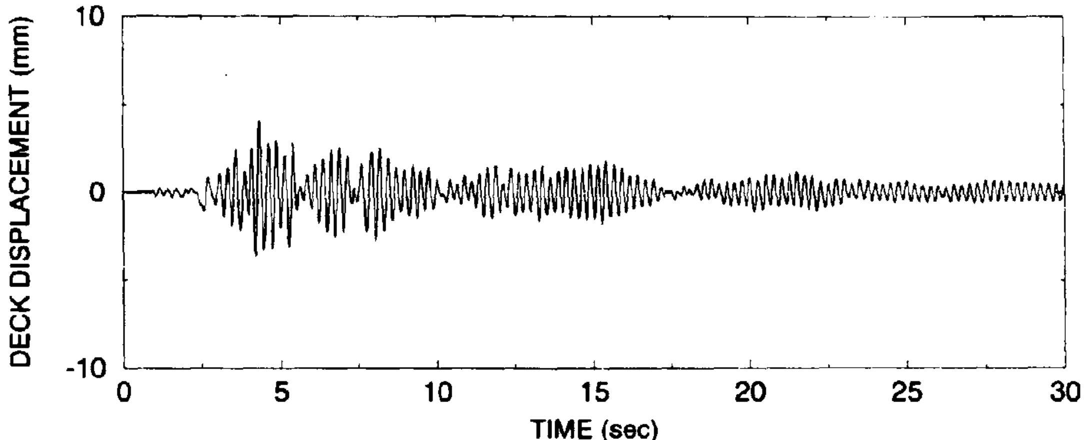
APPENDIX A

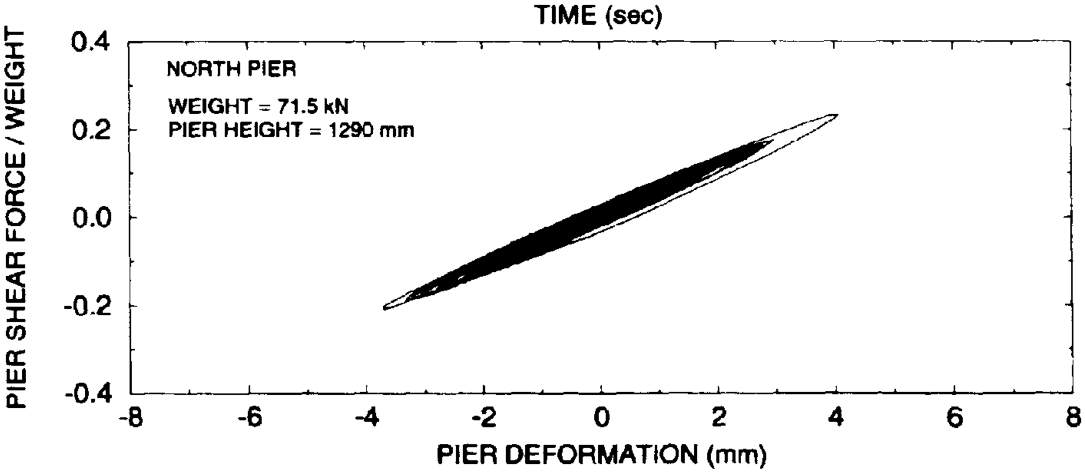
EXPERIMENTAL RESULTS

This Appendix contains experimental results of the tested bridge model in the non-isolated and the isolated configuration with either two stiff or two flexible piers. In the case of the non-isolated bridge (test No. FRUN05 to FRUN22), the recorded time history of the deck displacement with respect to the table and the loops of the shear force versus deformation of the north pier are presented. In the case of the isolated bridge with stiff piers (tests No ITBR01 to ITBR16), the recorded SW bearing displacement history and the loops of isolation system force versus SW bearing displacement are presented. The isolation system force was obtained as the sum of the forces recorded by the four load cells supporting the bearings. In the case of the isolated bridge with flexible piers (tests No. ITBR07 to ITBR16), the recorded SW bearing displacement history, the loops of isolation system force versus SW bearing displacement and the loops of shear force versus deformation of the north pier are presented. The test number and excitation are identified at the top of each page.

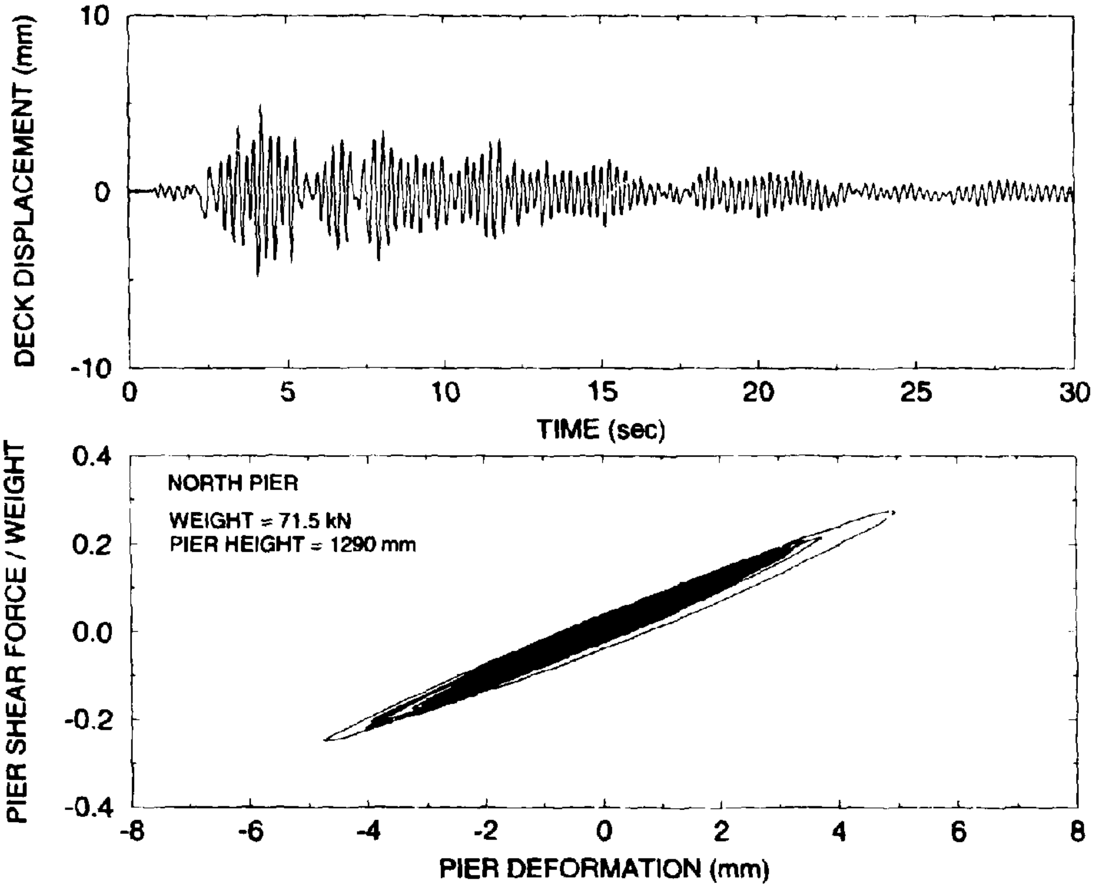

NON-ISOLATED, TEST No. FRUN05
EL CENTRO S00E 25%
DECK DISPLACEMENT (mm)
10
0
-10
0
5
10
15
20
25
30
TIME (sec)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
-8
-6
-4
-2
0
2
4
6
8
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm
PIER DEFORMATION (mm)

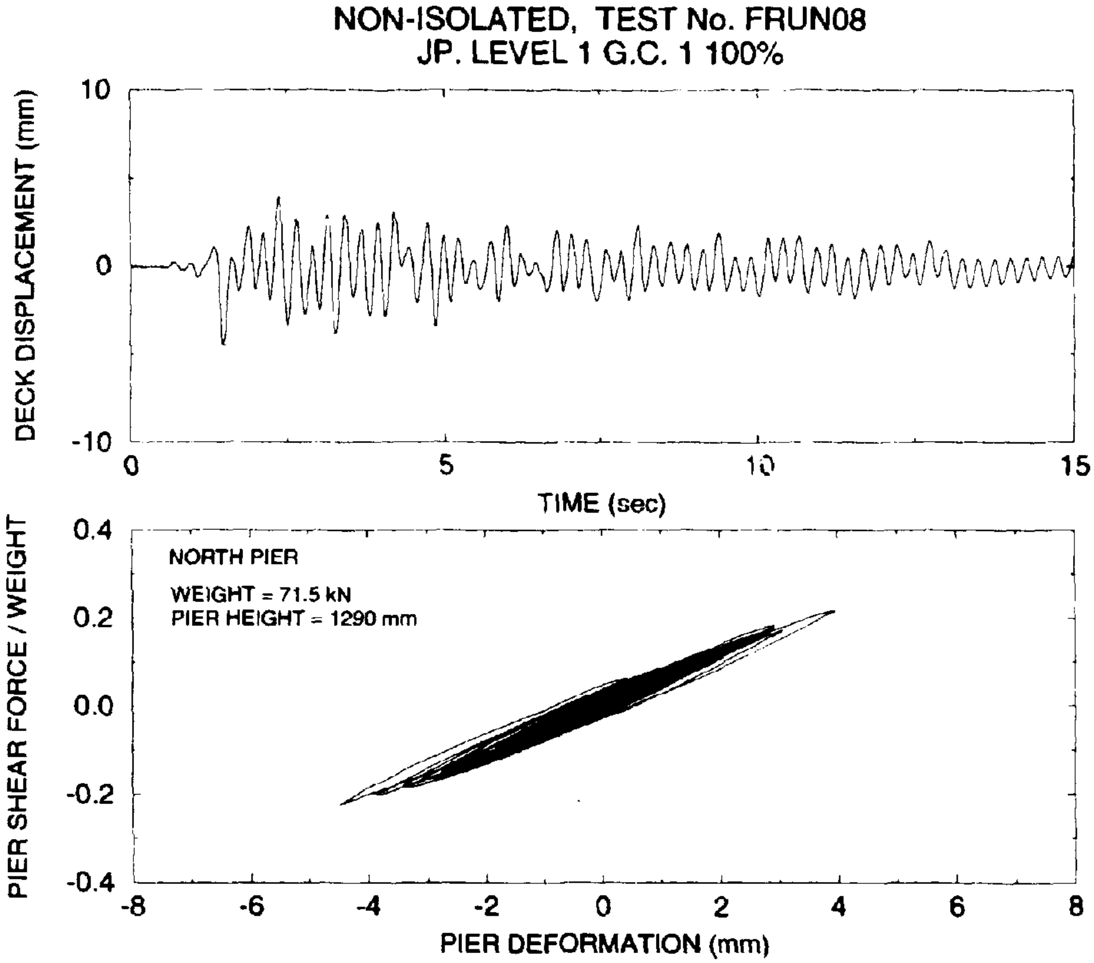
NON-ISOLATED, TEST No. FRUN06
TAFT N21E 50%



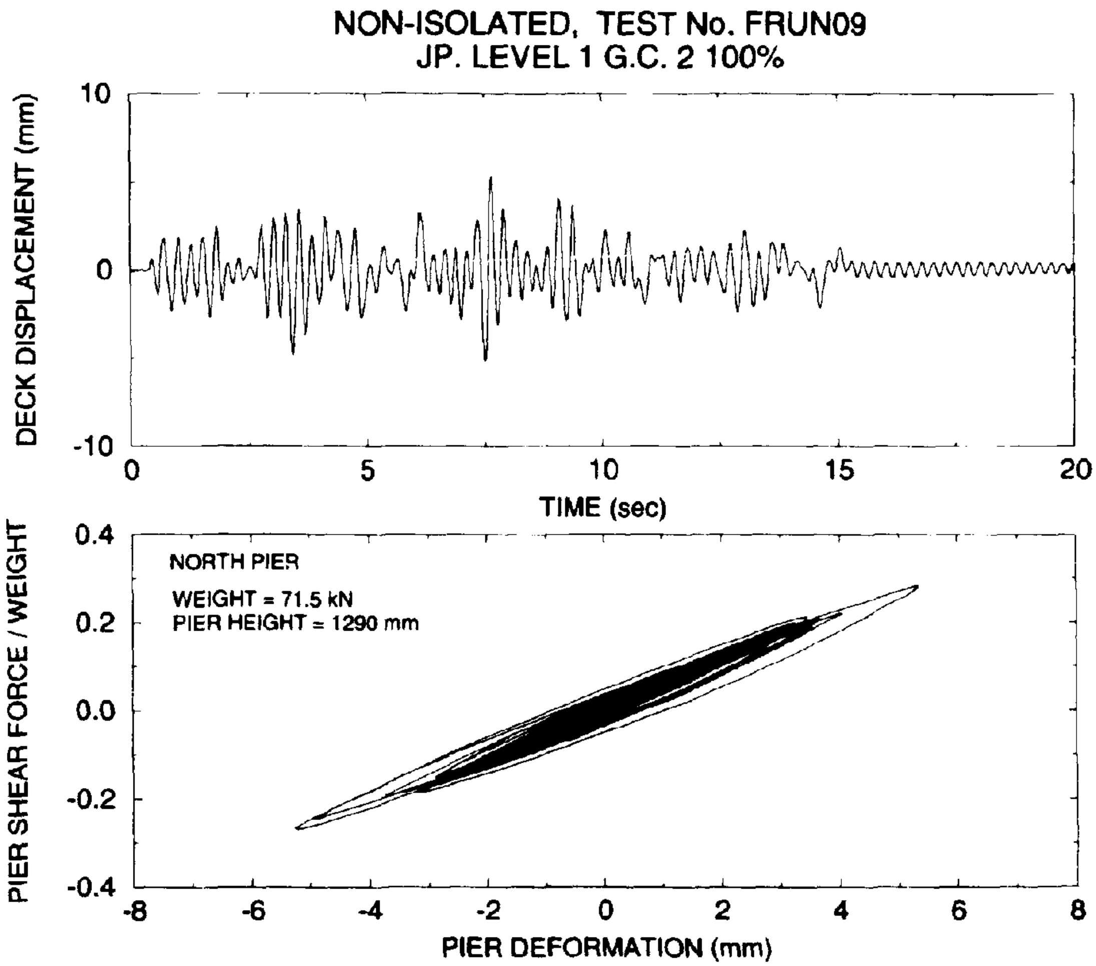


NON-ISOLATED, TEST No. FRUN07
TAFT N21E 75%

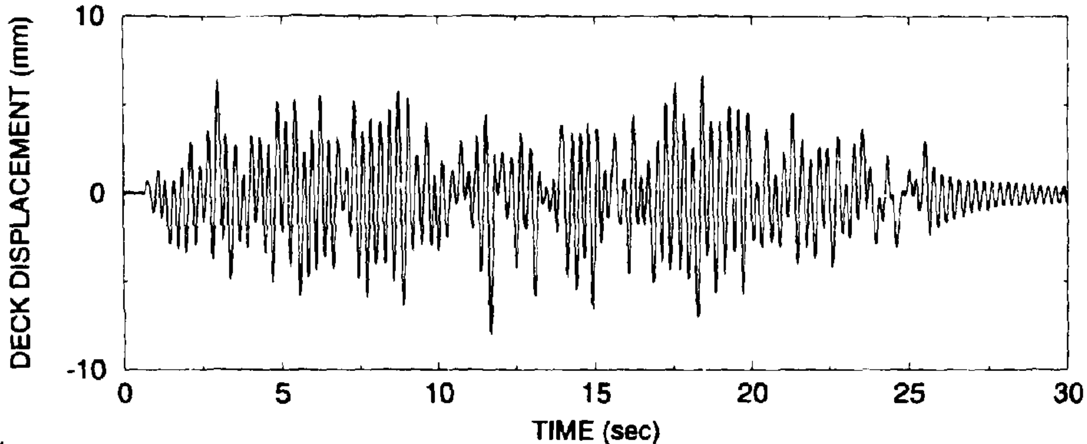


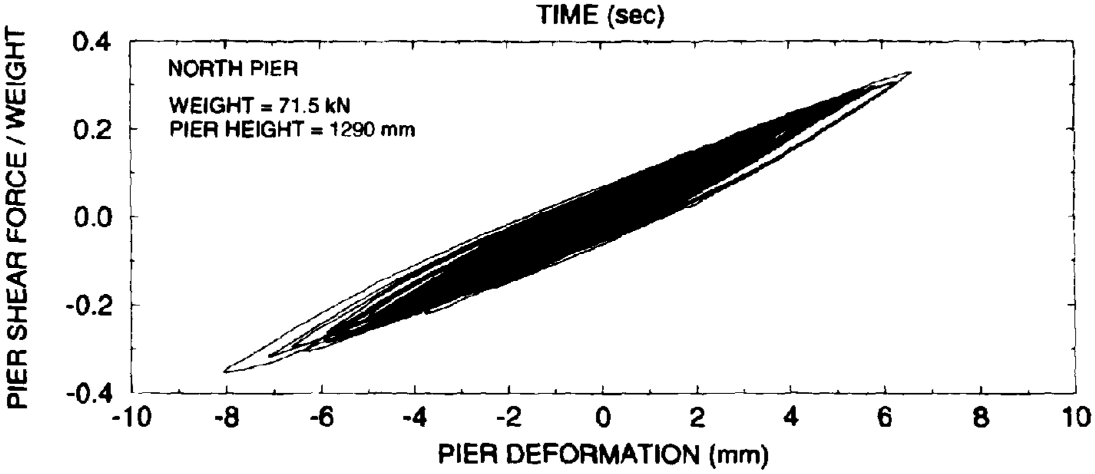


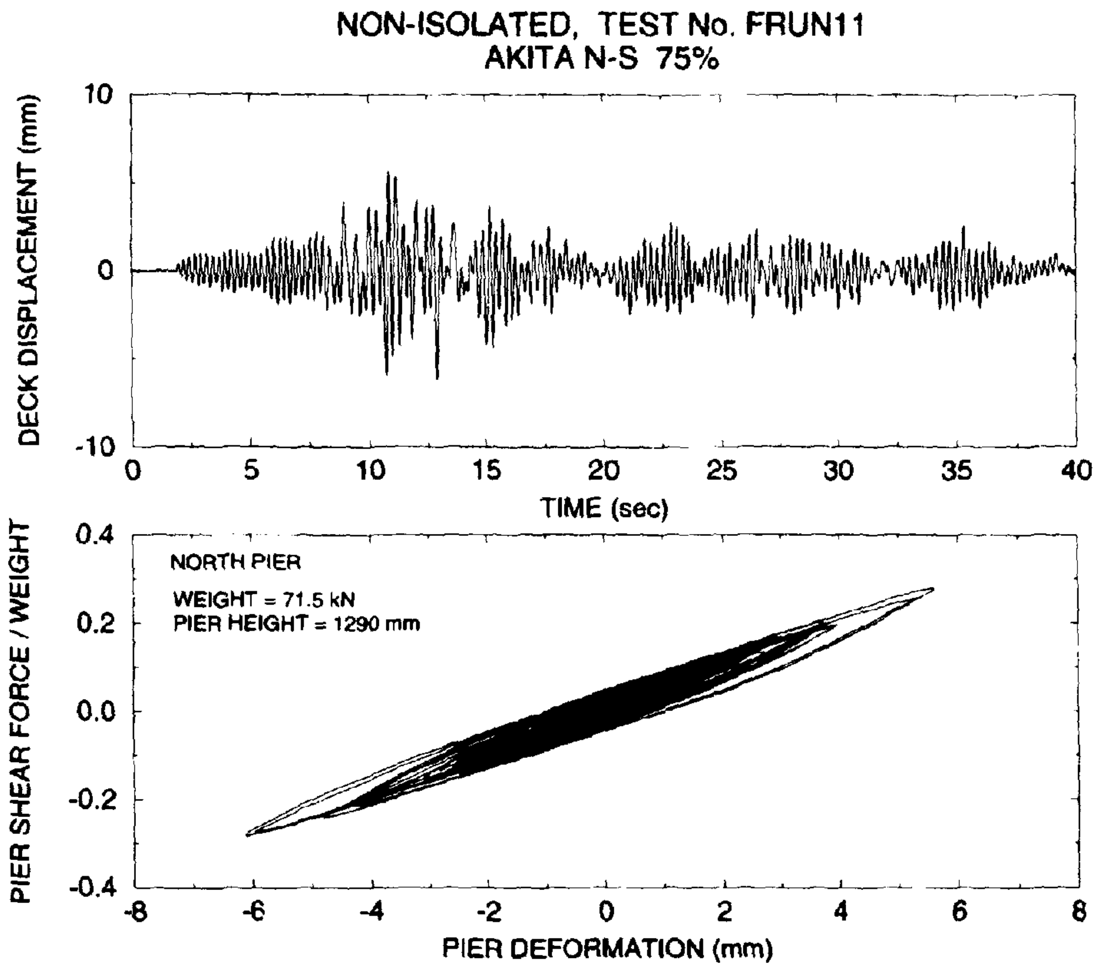
NON-ISOLATED, TEST No. FRUN08
JP. LEVEL 1 G.C. 1 100%
DECK DISPLACEMENT (mm)
10
0
-10
0
5
10
15
TIME (sec)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)


NON-ISOLATED, TEST No. FRUN09
JP. LEVEL 1 G.C. 2 100%
DECK DISPLACEMENT (mm)
TIME (sec)
PIER SHEAR FORCE / WEIGHT
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm
PIER DEFORMATION (mm)

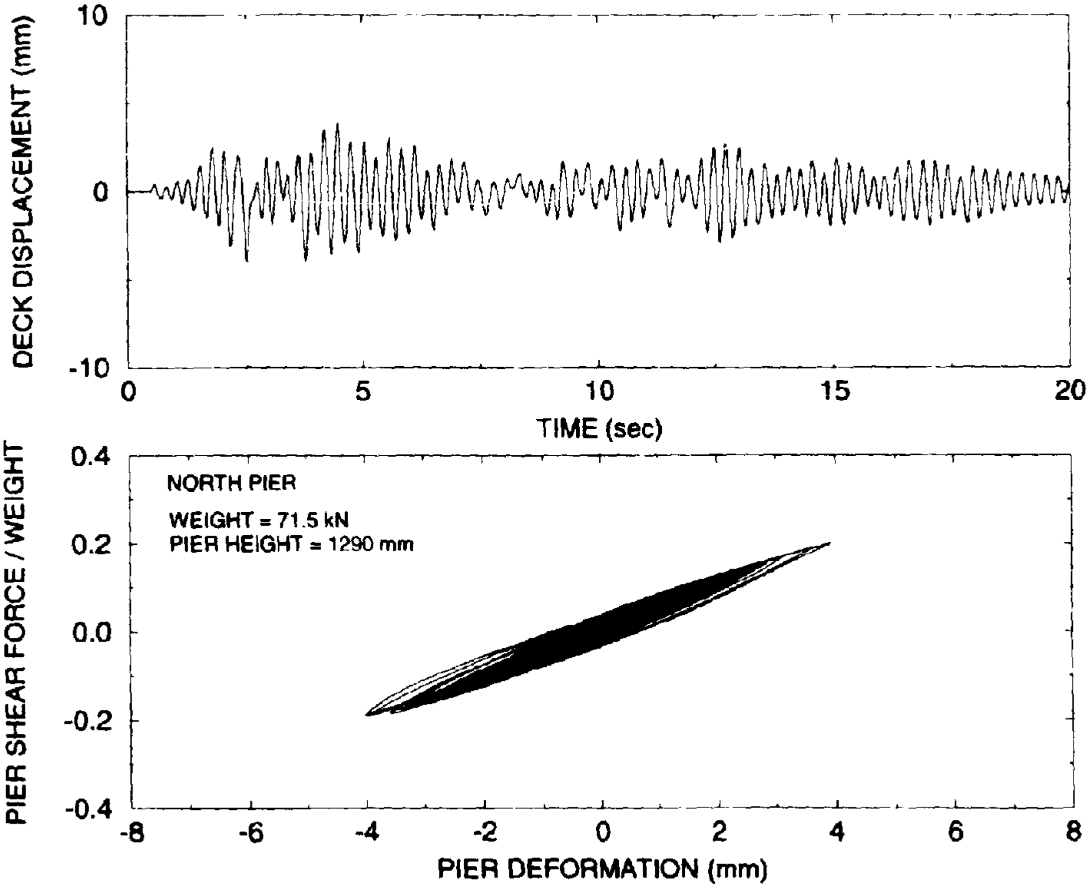
NON-ISOLATED, TEST No. FRUN10
JP. LEVEL 1 G.C. 3 100%

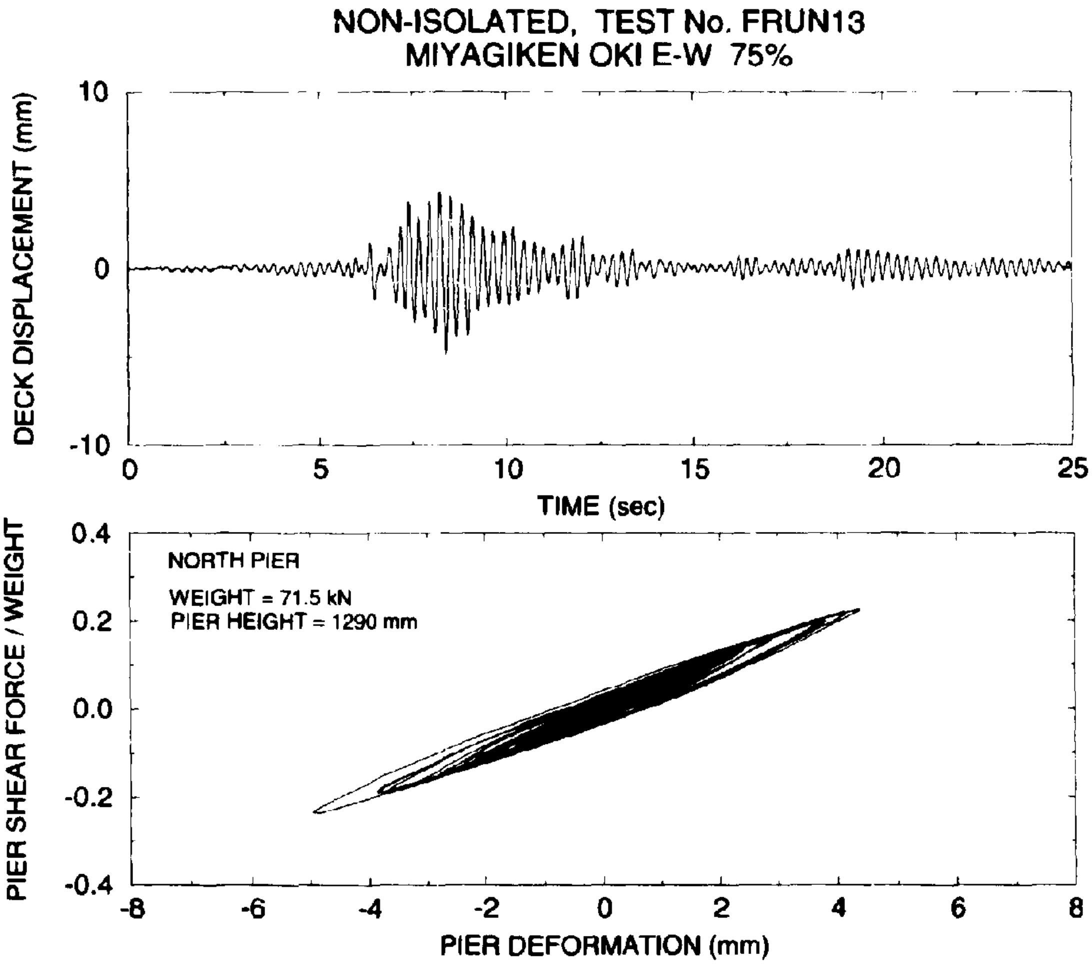




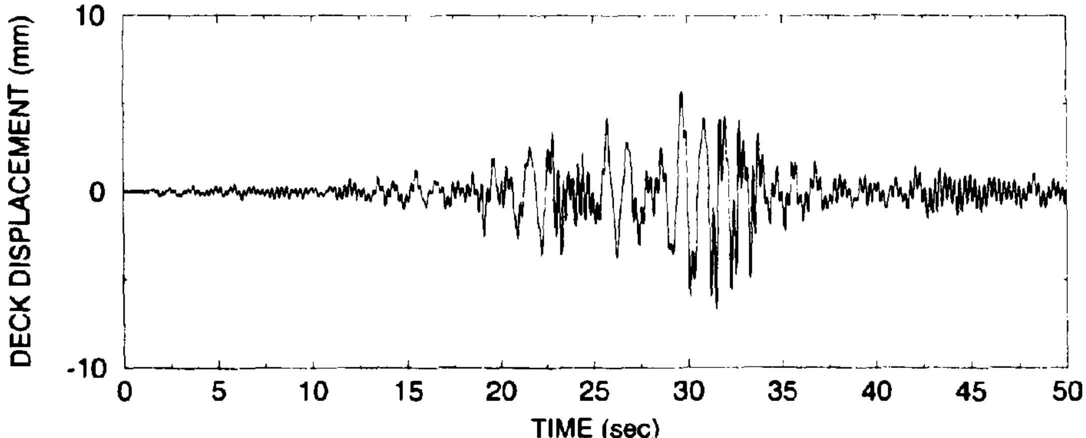

NON-ISOLATED, TEST No. FRUN11
AKITA N-S 75%
DECK DISPLACEMENT (mm)
10
0
-10
0
5
10
15
20
25
30
35
40
TIME (sec)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm

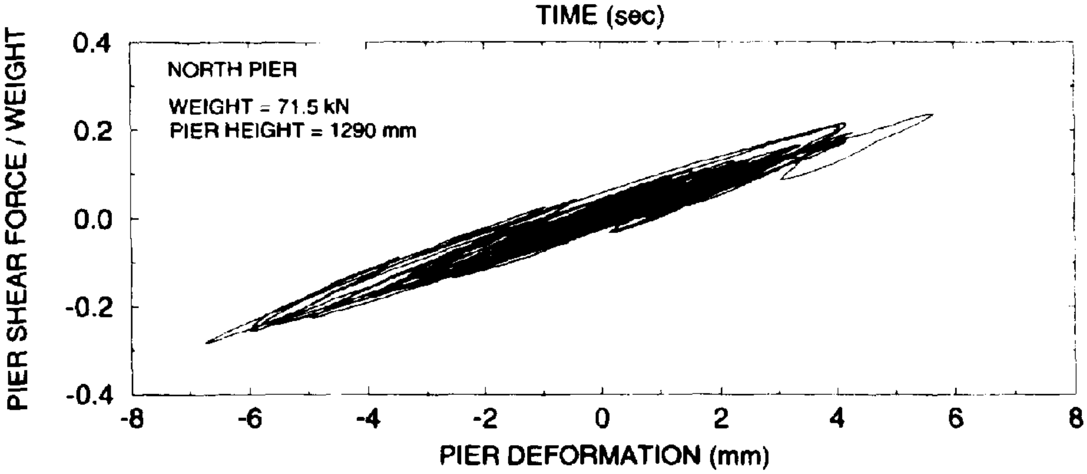
NON-ISOLATED, TEST No. FRUN12
HACHINOHE N-S 50%



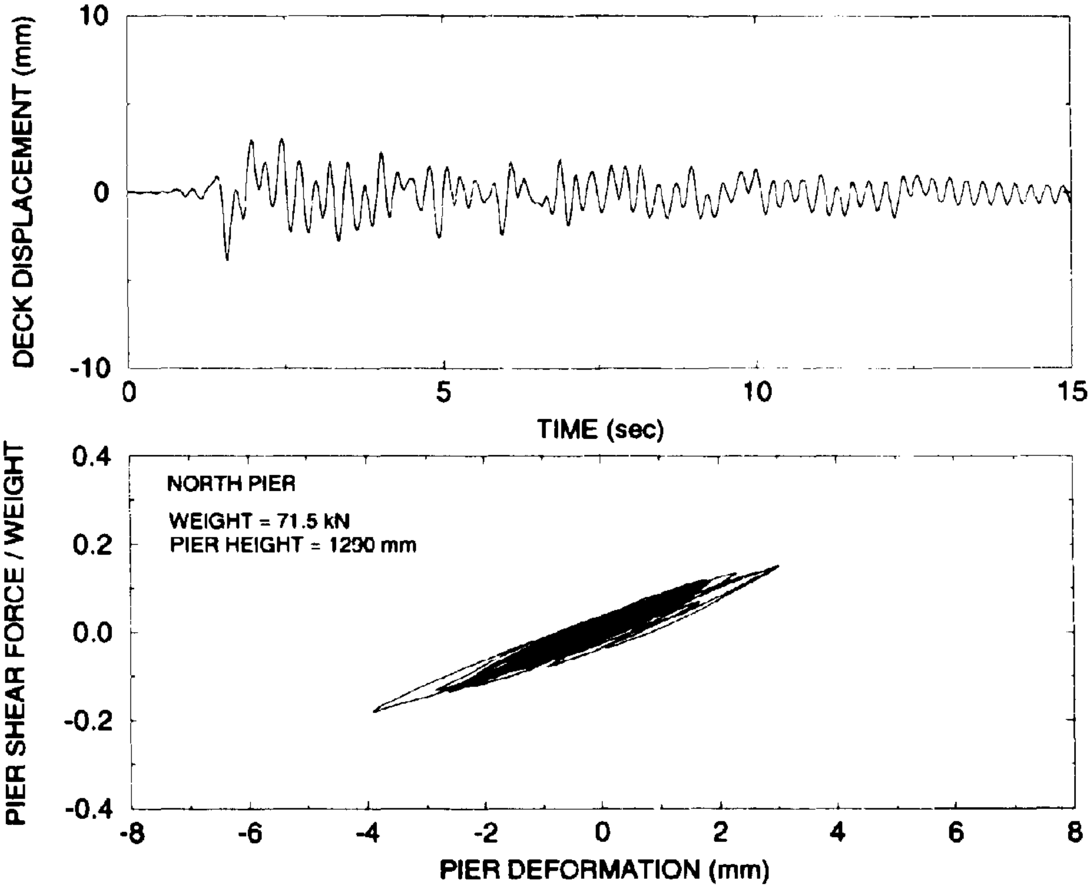

NON-ISOLATED, TEST No. FRUN13
MIYAGIKEN OKI E-W 75%
DECK DISPLACEMENT (mm)
10
0
-10
0
5
10
15
20
25
TIME (sec)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)

NON-ISOLATED, TEST No. FRUN14
MEXICO N90W 100%

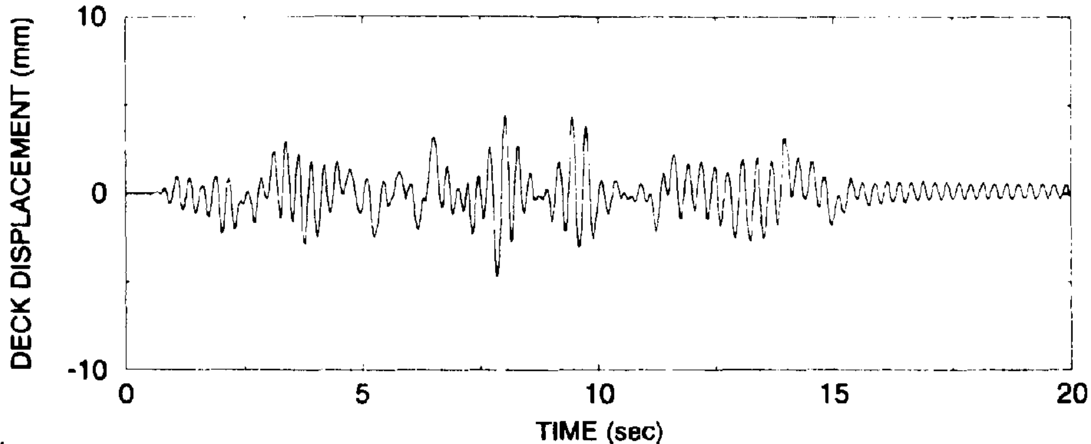


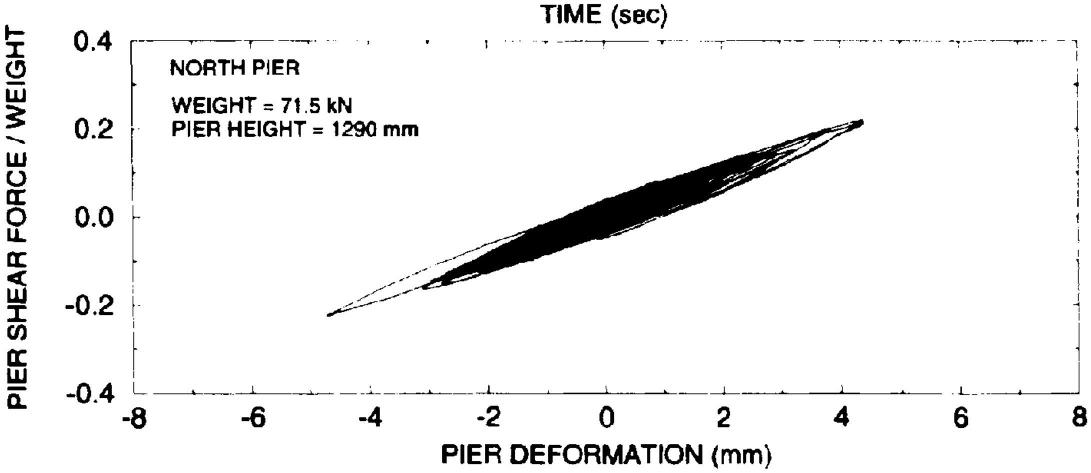


NON-ISOLATED, TEST No. FRUN15
JP. LEVEL 2 G.C. 1 25%

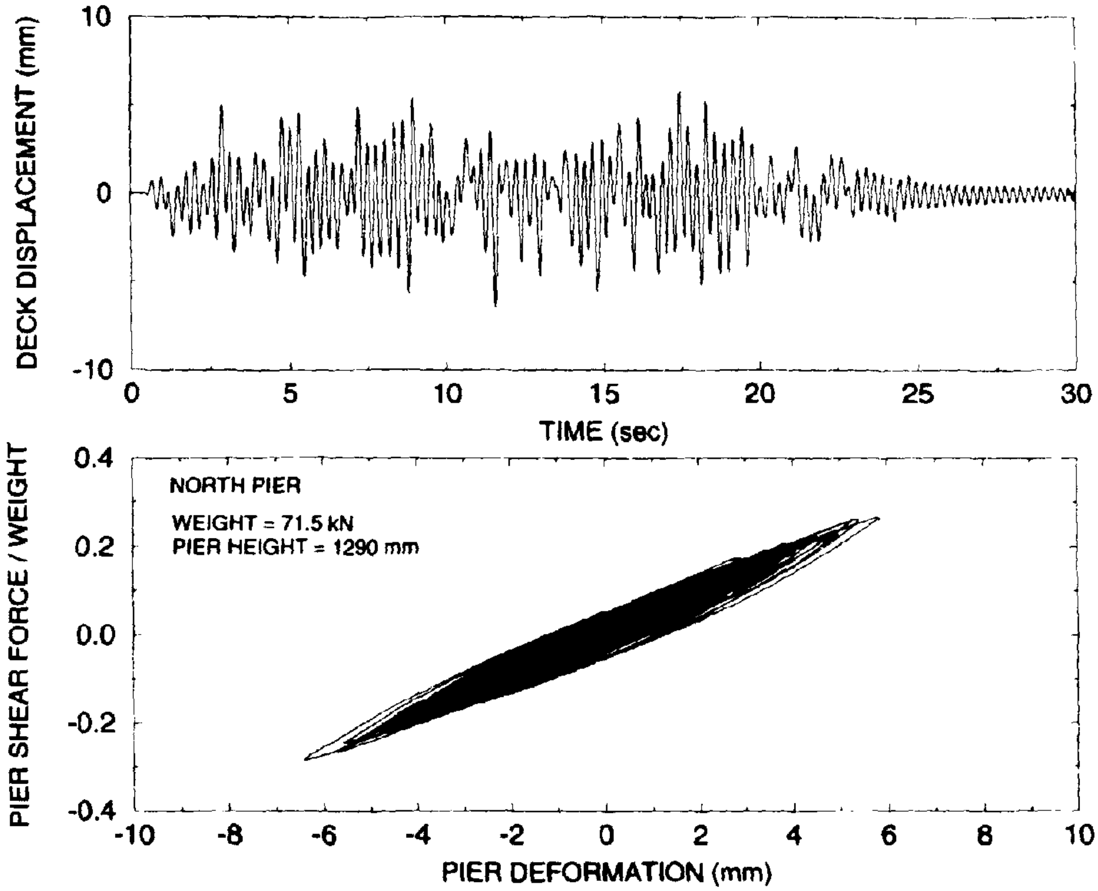


NON-ISOLATED, TEST No. FRUN16
JP. LEVEL 2 G.C. 2 25%

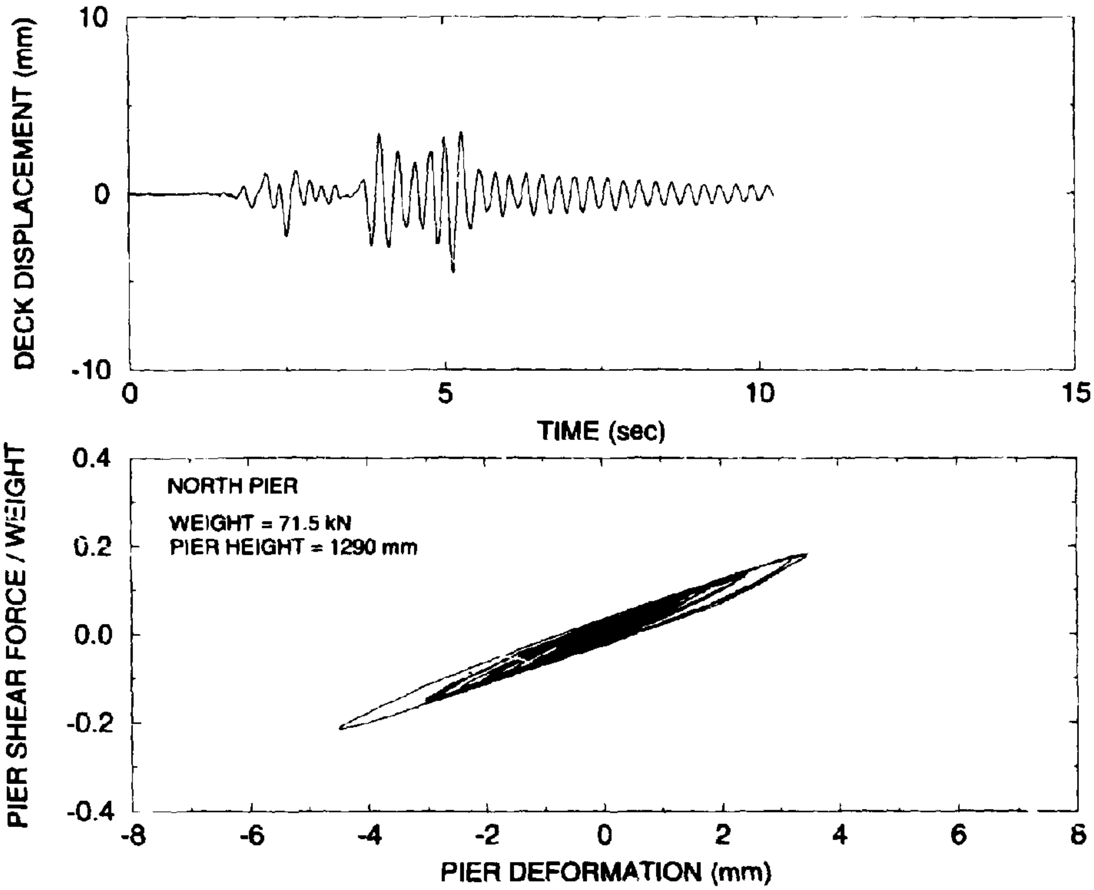




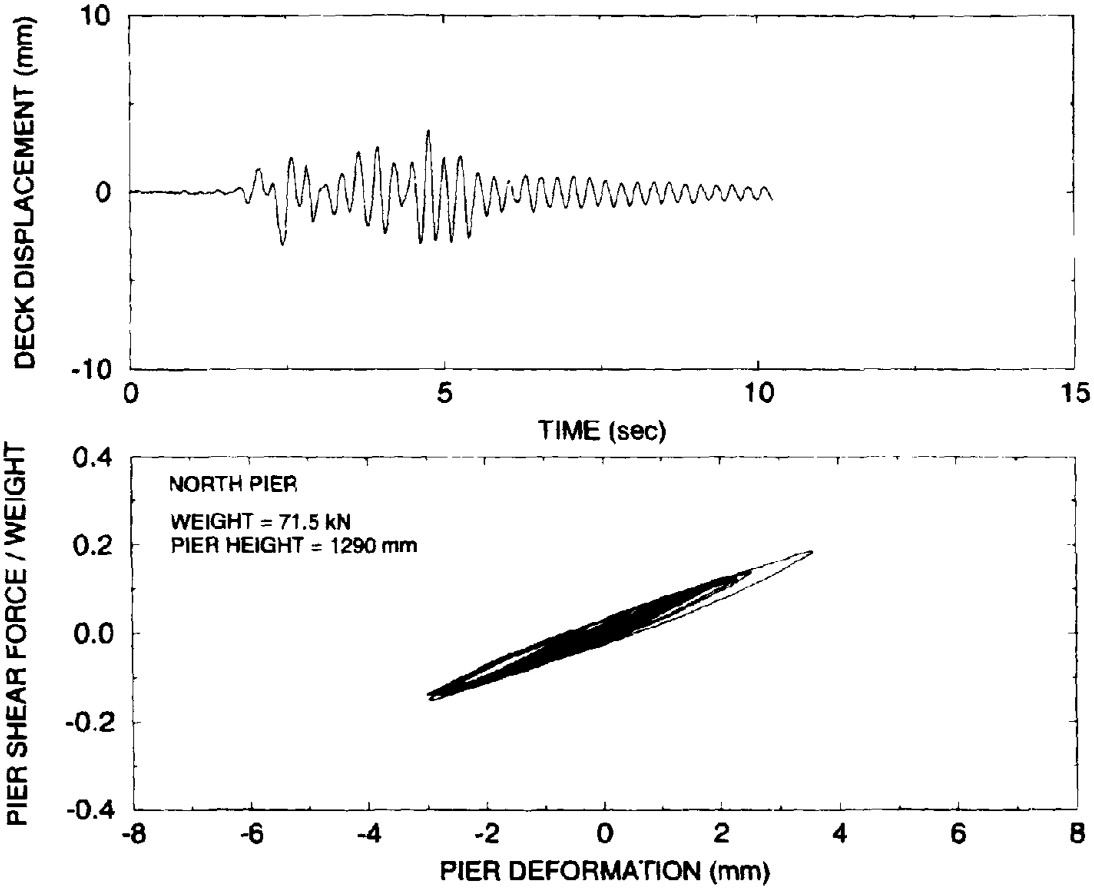
NON-ISOLATED, TEST No. FRUN17
JP. LEVEL 2 G.C. 3 25%



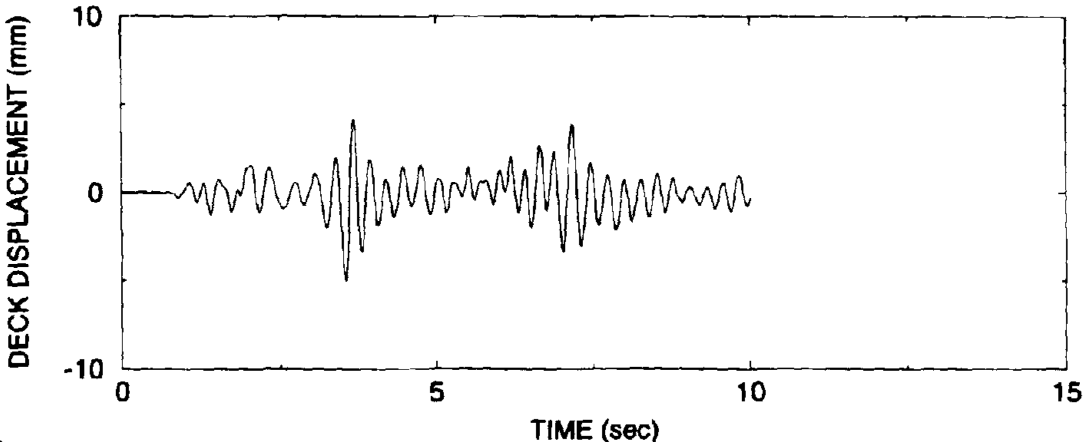
NON-ISOLATED, TEST No. FRUN18
PACOIMA S74W 13%

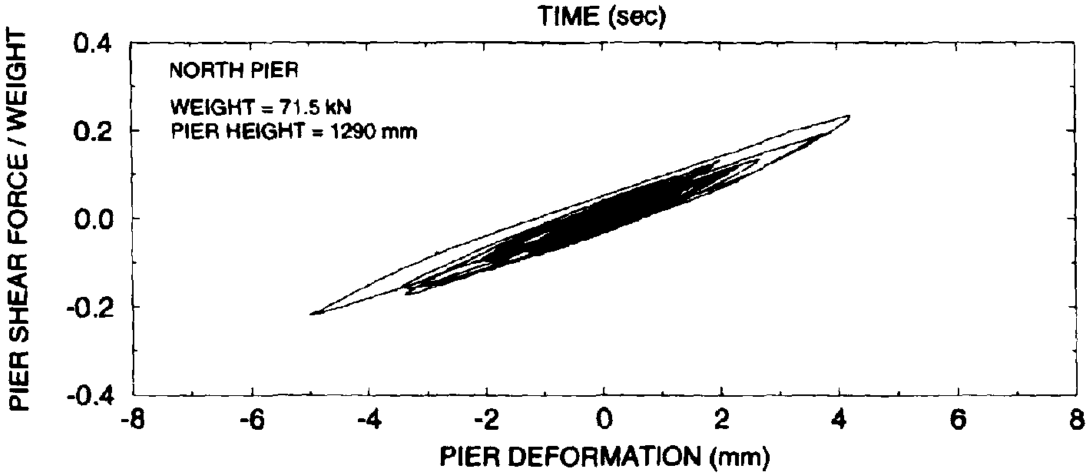


NON-ISOLATED, TEST No. FRUN19
PACOIMA S16E 13%

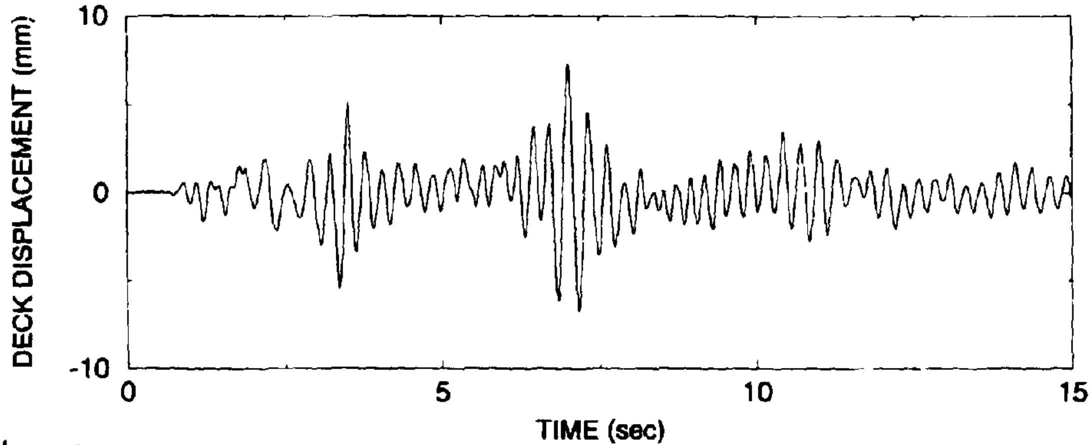


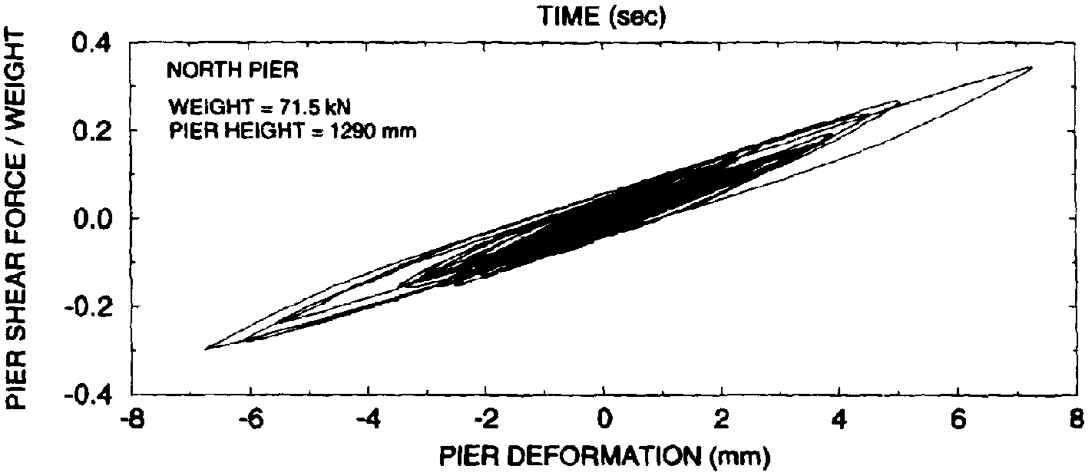
NON-ISOLATED, TEST No. FRUN20
CALTRANS ROCK 0.6g SPEC. 3 20%

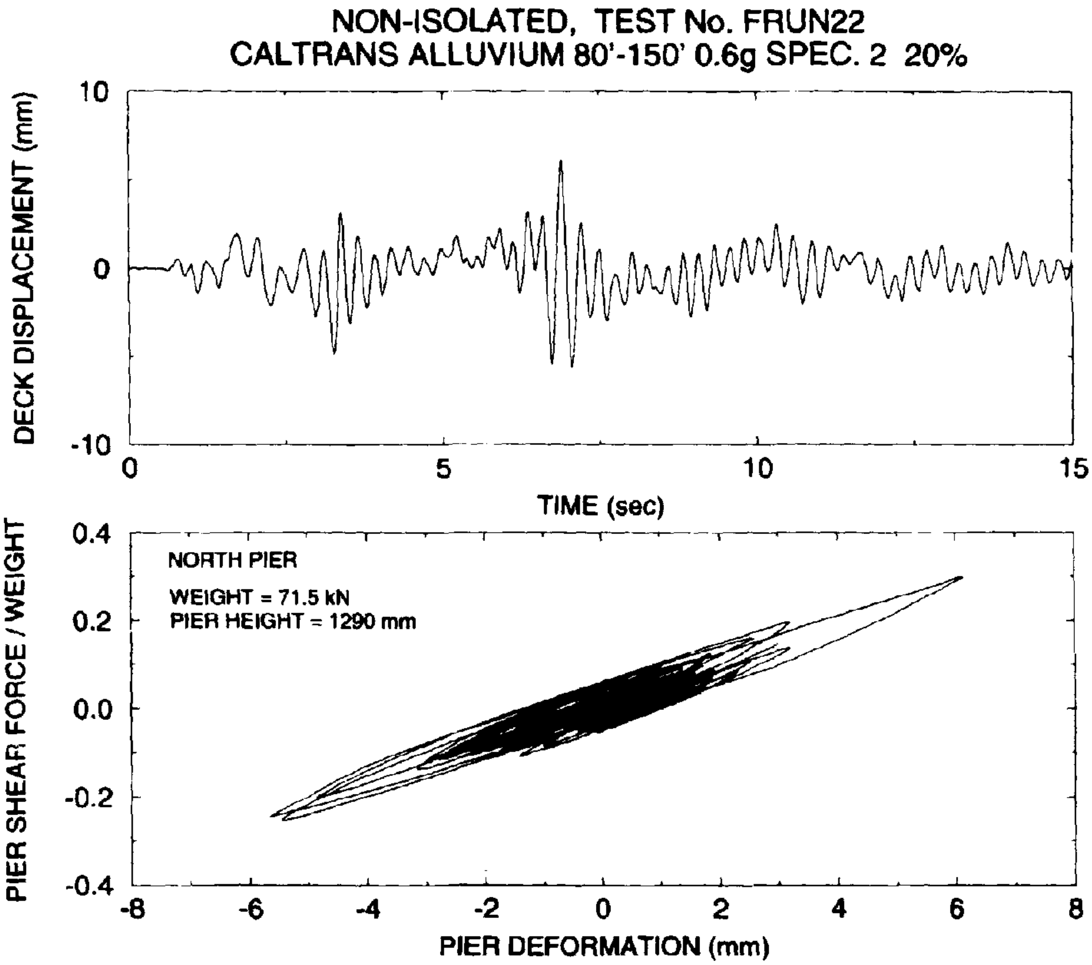


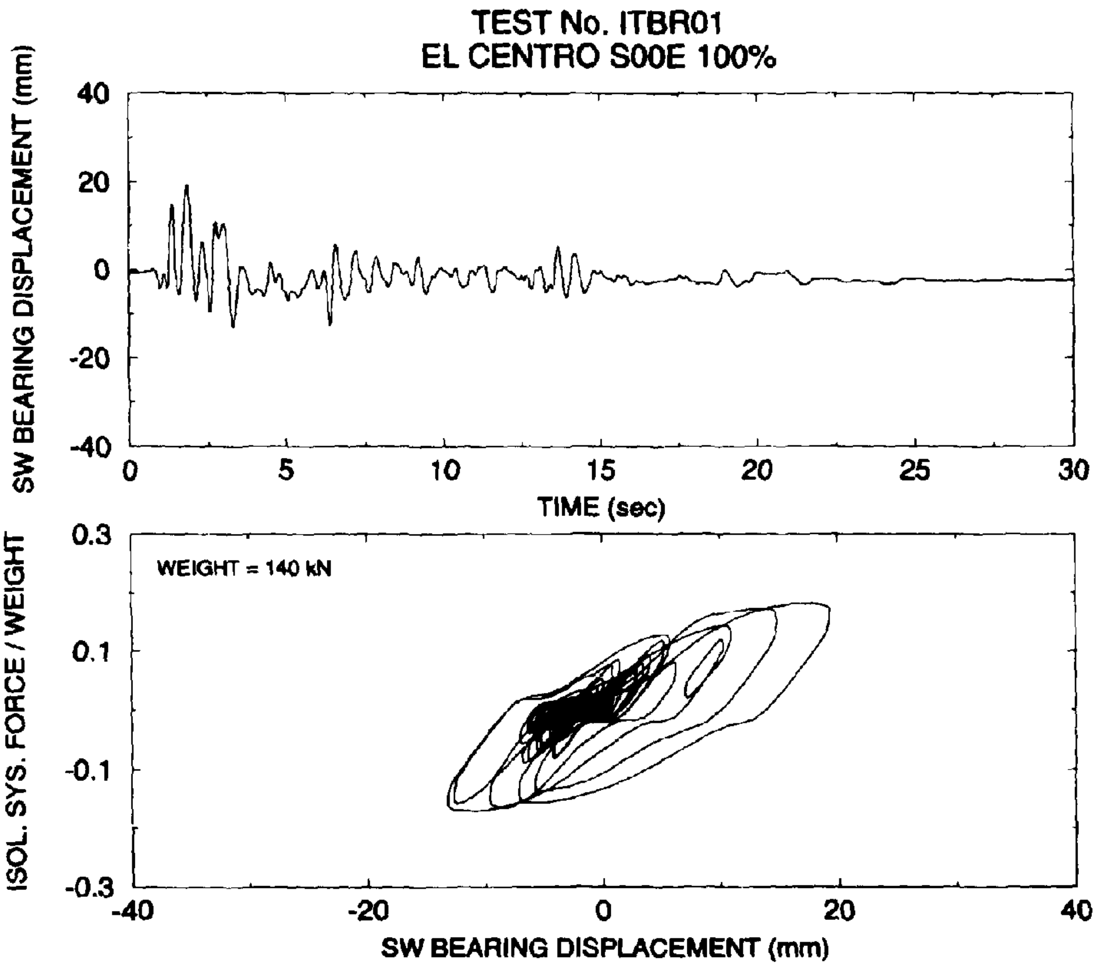


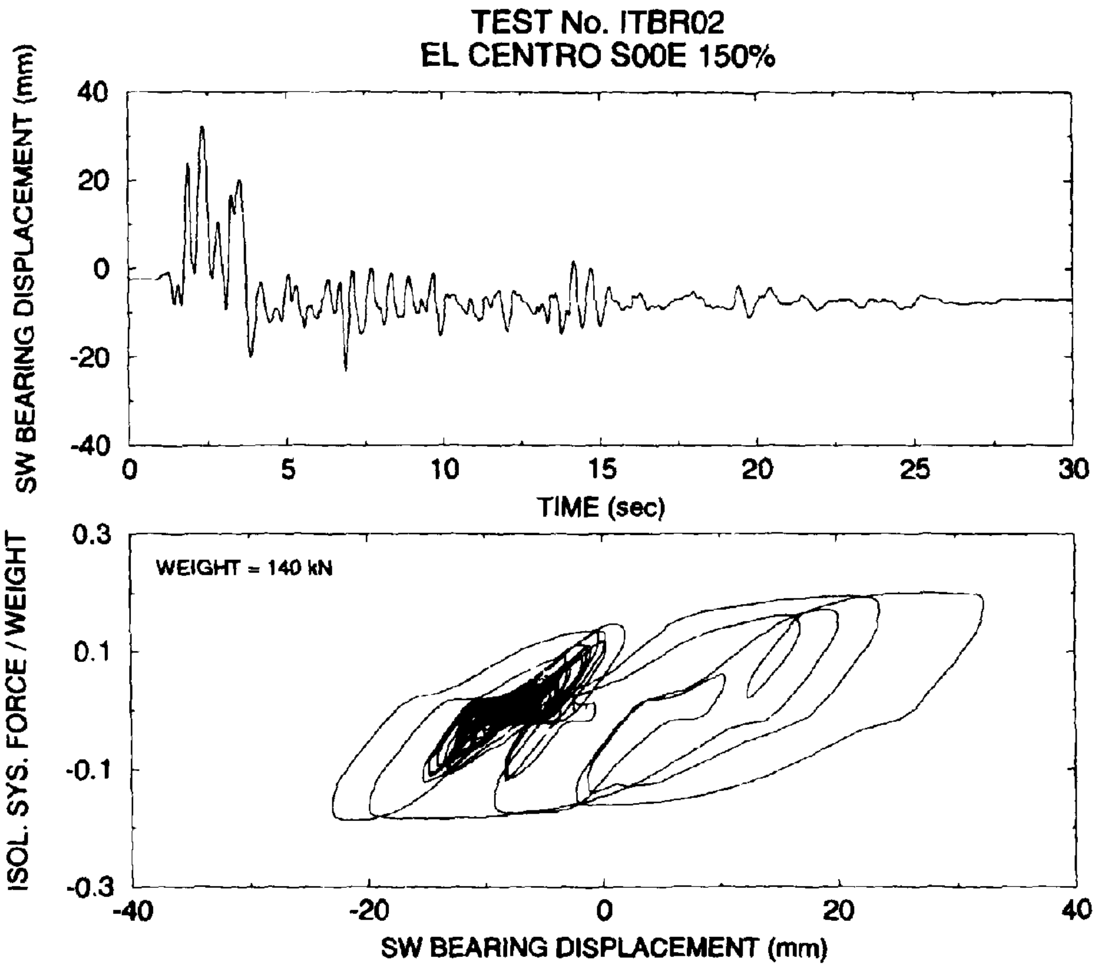
NON-ISOLATED, TEST No. FRUN21
CALTRANS ALLUVIUM 10'-80' 0.6g SPEC. 3 20%



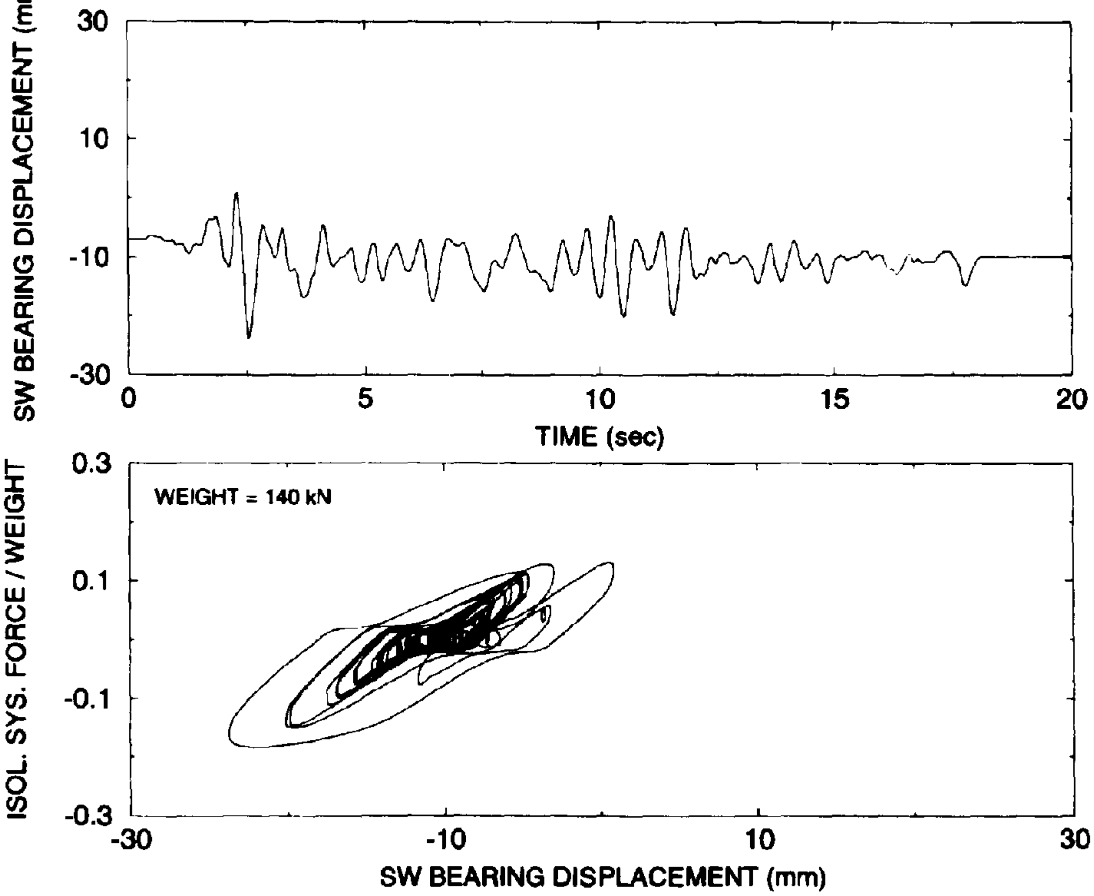


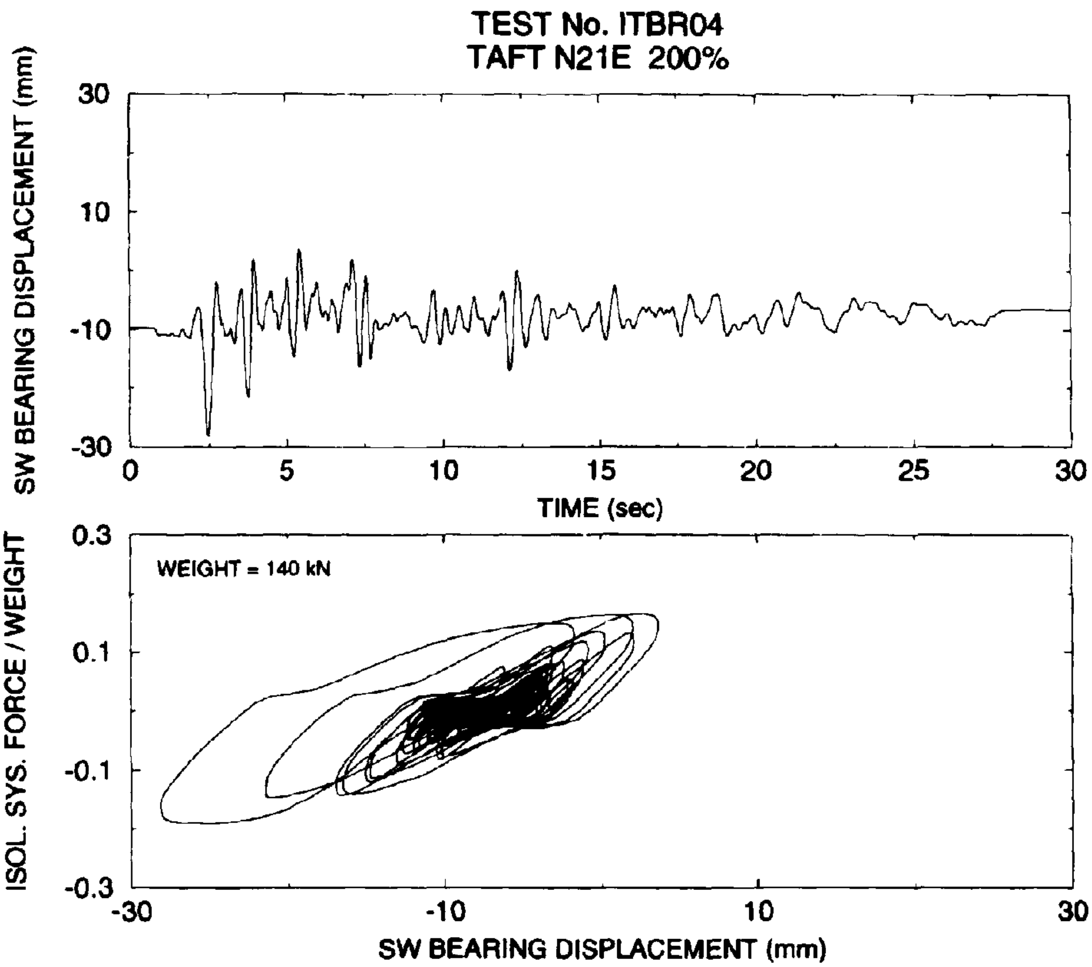

NON-ISOLATED, TEST No. FRUN22
CALTRANS ALLUVIUM 80'-150' 0.6g SPEC. 2 20%
DECK DISPLACEMENT (mm)
10
0
-10
0
5
10
15
TIME (sec)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
NORTH PIER
WEIGHT = 71.5 kN
PIER HEIGHT = 1290 mm
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)

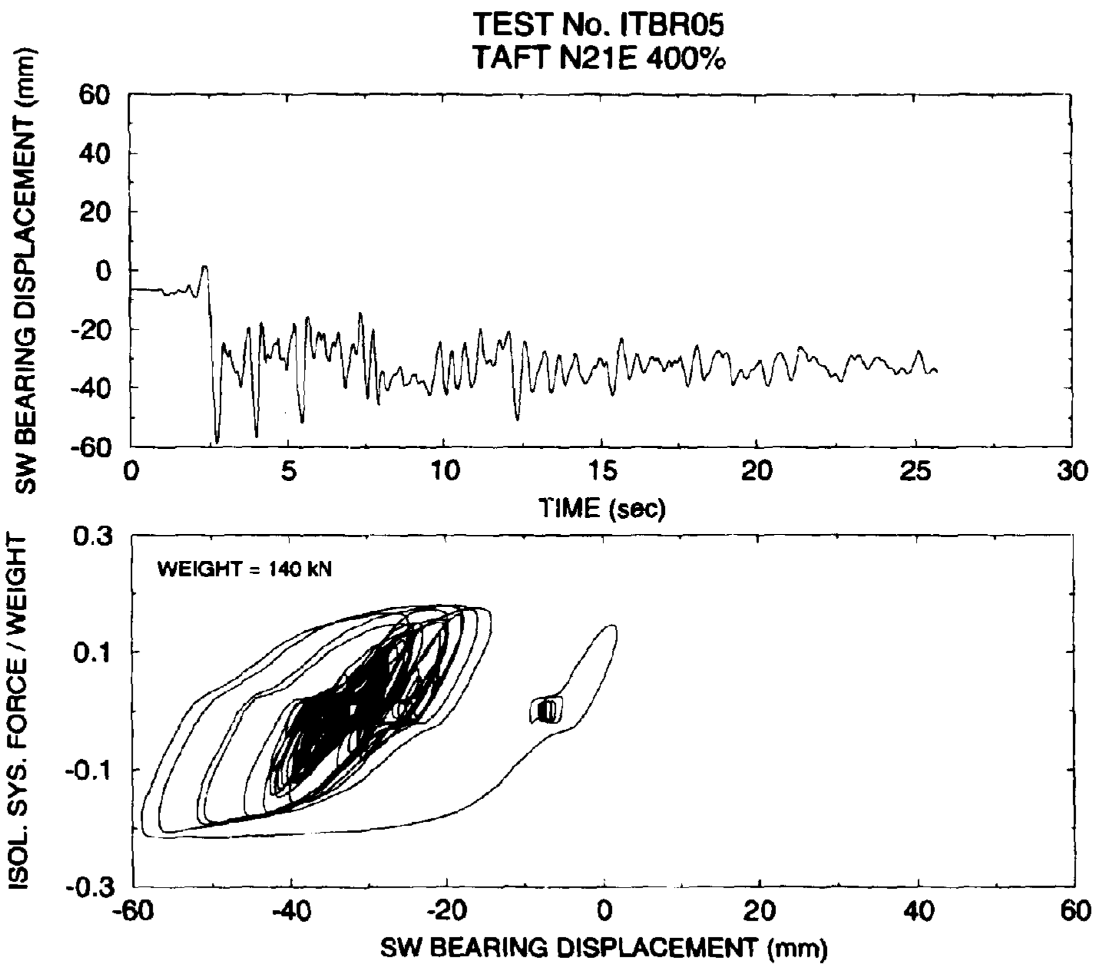

TEST No. ITBR01
EL CENTRO S00E 100%
SW BEARING DISPLACEMENT (mm)
40
20
0
-20
-40
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-40
-20
0
20
40
SW BEARING DISPLACEMENT (mm)

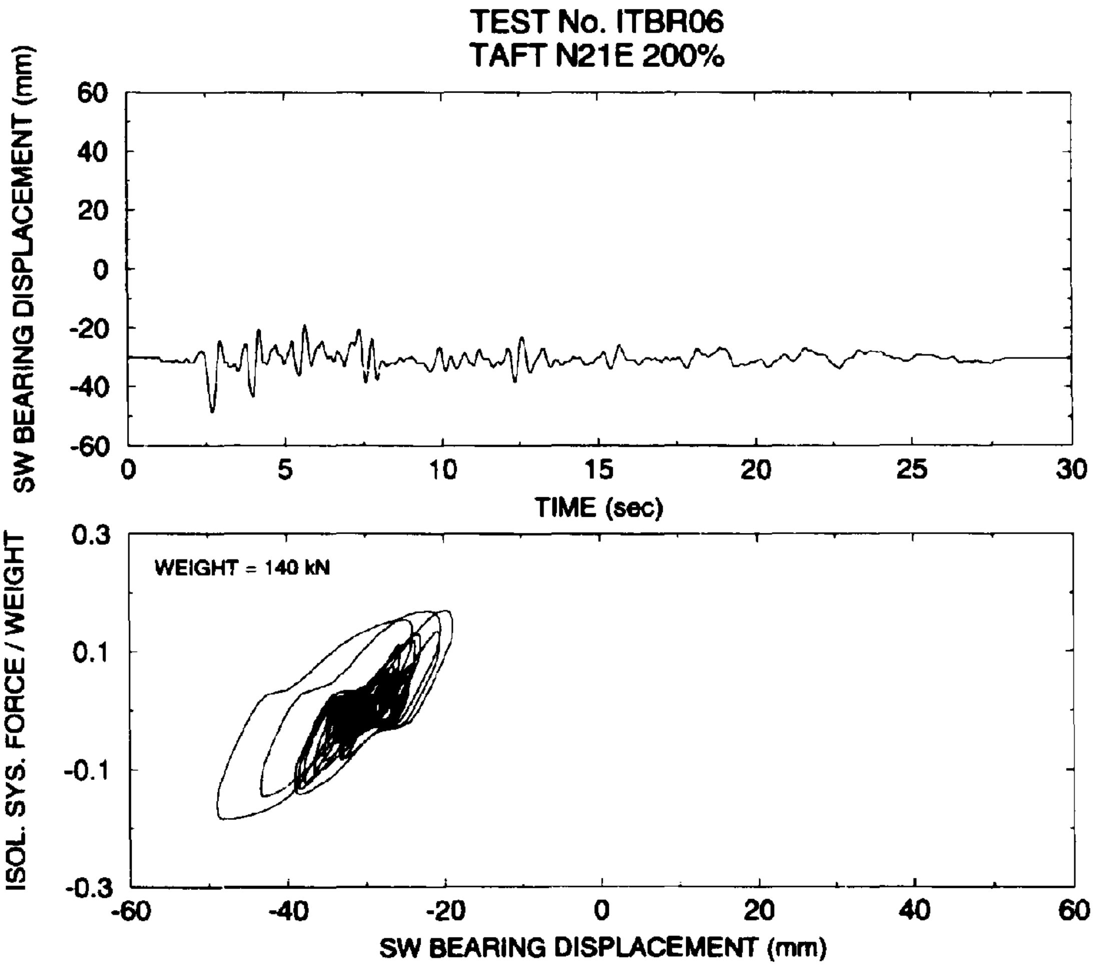

TEST No. ITBR02
EL CENTRO S00E 150%
SW BEARING DISPLACEMENT (mm)
40
20
0
-20
-40
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-40
-20
0
20
40
SW BEARING DISPLACEMENT (mm)

TEST No. ITBR03
HACHINOHE N-S 100%

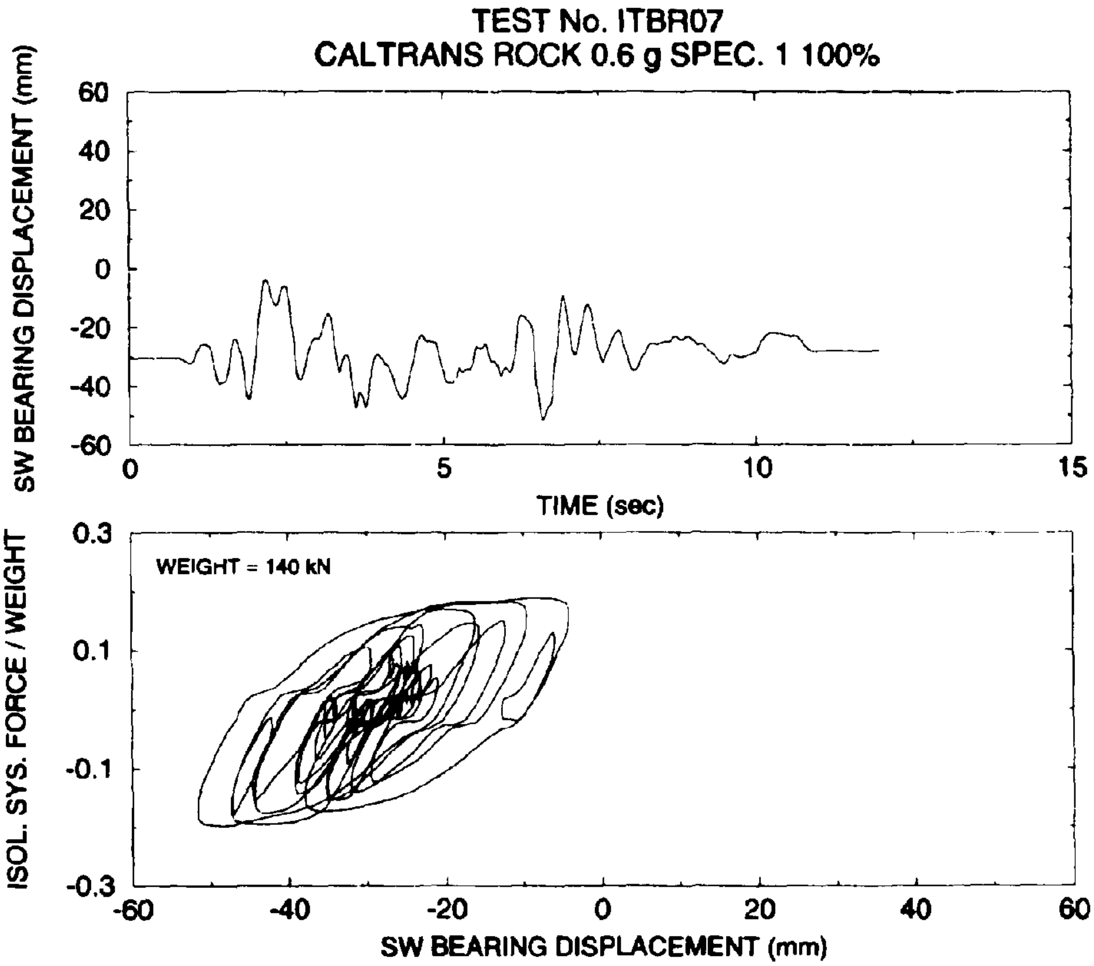


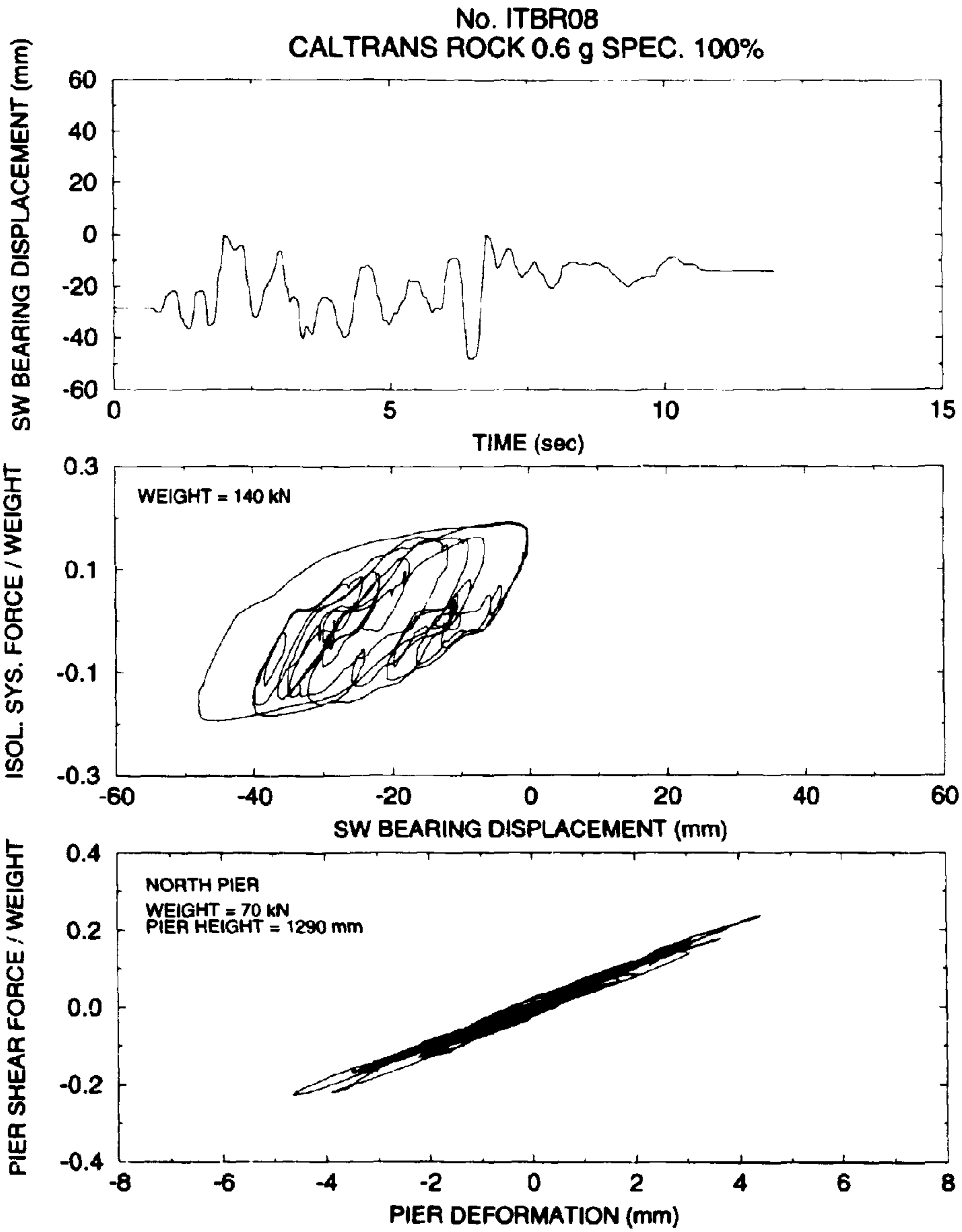

TEST No. ITBR04
TAFT N21E 200%
SW BEARING DISPLACEMENT (mm)
30
10
-10
-30
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-30
-10
10
30
SW BEARING DISPLACEMENT (mm)


TEST No. ITBR05
TAFT N21E 400%
SW BEARING DISPLACEMENT (mm)
60
40
20
0
-20
-40
-60
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-60
-40
-20
0
20
40
60
SW BEARING DISPLACEMENT (mm)

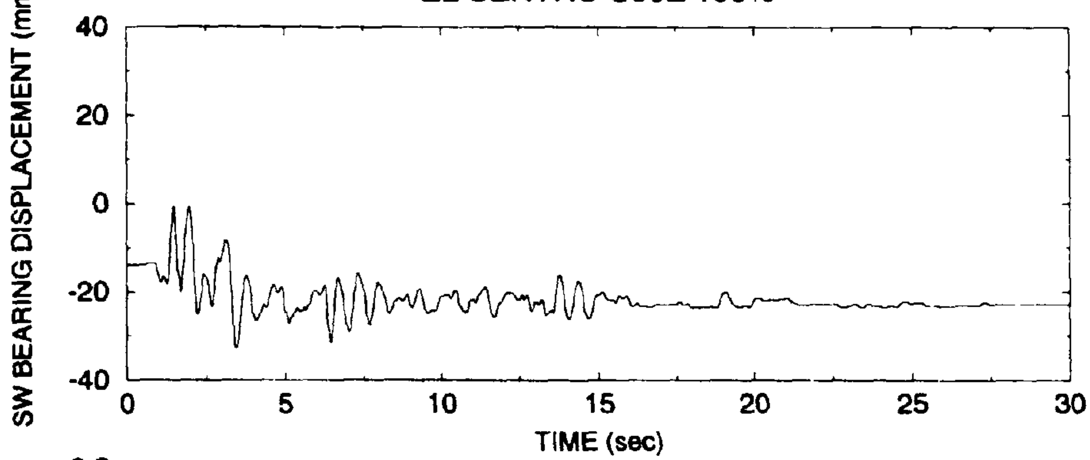


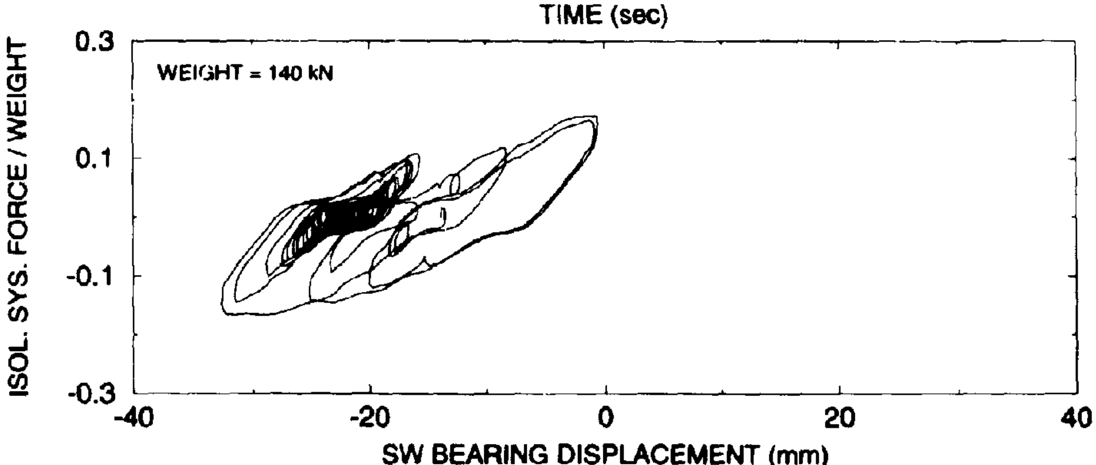
TEST No. ITBR06
TAFT N21E 200%
SW BEARING DISPLACEMENT (mm)
60
40
20
0
-20
-40
-60
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-60
-40
-20
0
20
40
60
SW BEARING DISPLACEMENT (mm)

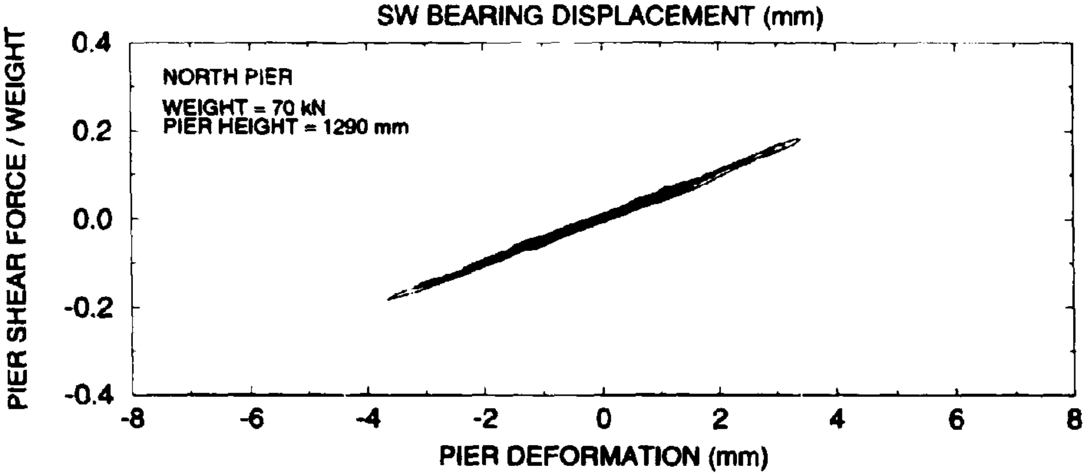

TEST No. ITBR07
CALTRANS ROCK 0.6 g SPEC. 1 100%
SW BEARING DISPLACEMENT (mm)
60
40
20
0
-20
-40
-60
0
5
10
15
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-60
-40
-20
0
20
40
60
SW BEARING DISPLACEMENT (mm)


No. ITBR08
CALTRANS ROCK 0.6 g SPEC. 100%
SW BEARING DISPLACEMENT (mm)
60
40
20
0
-20
-40
-60
0
5
10
15
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-60
-40
-20
0
20
40
60
SW BEARING DISPLACEMENT (mm)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
NORTH PIER
WEIGHT = 70 kN
PIER HEIGHT = 1290 mm
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)

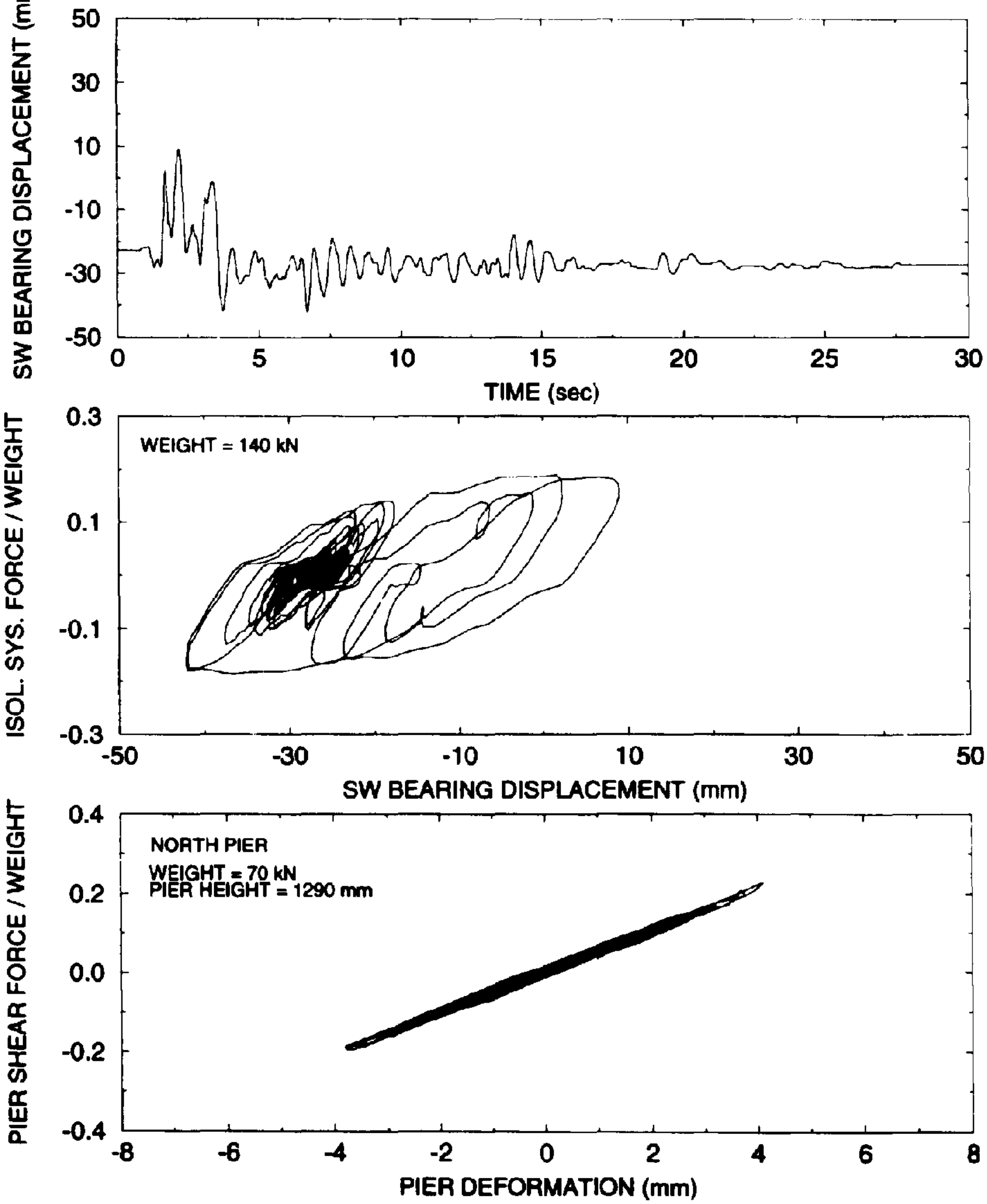
No. ITBR09
EL CENTRO S00E 100%



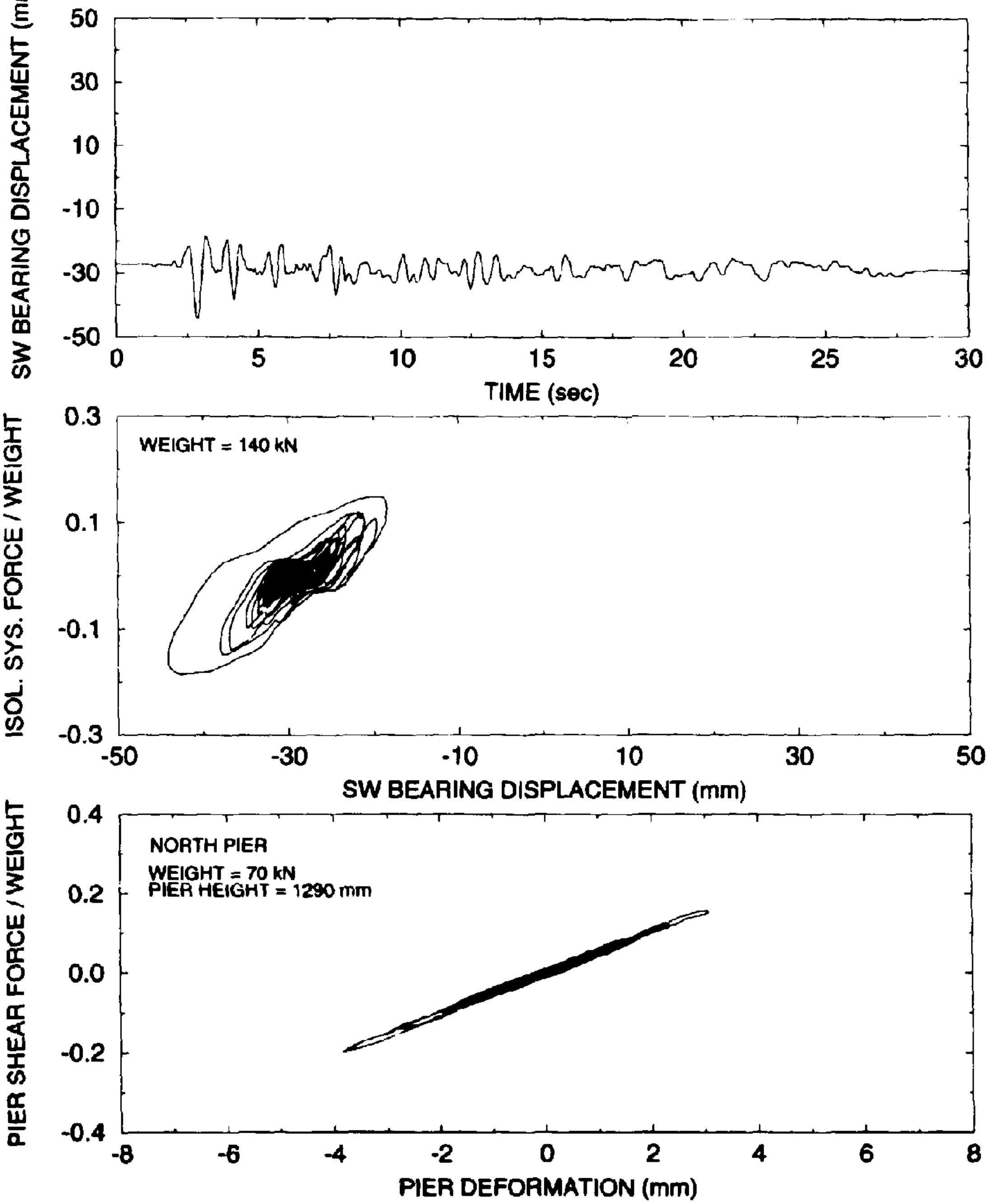


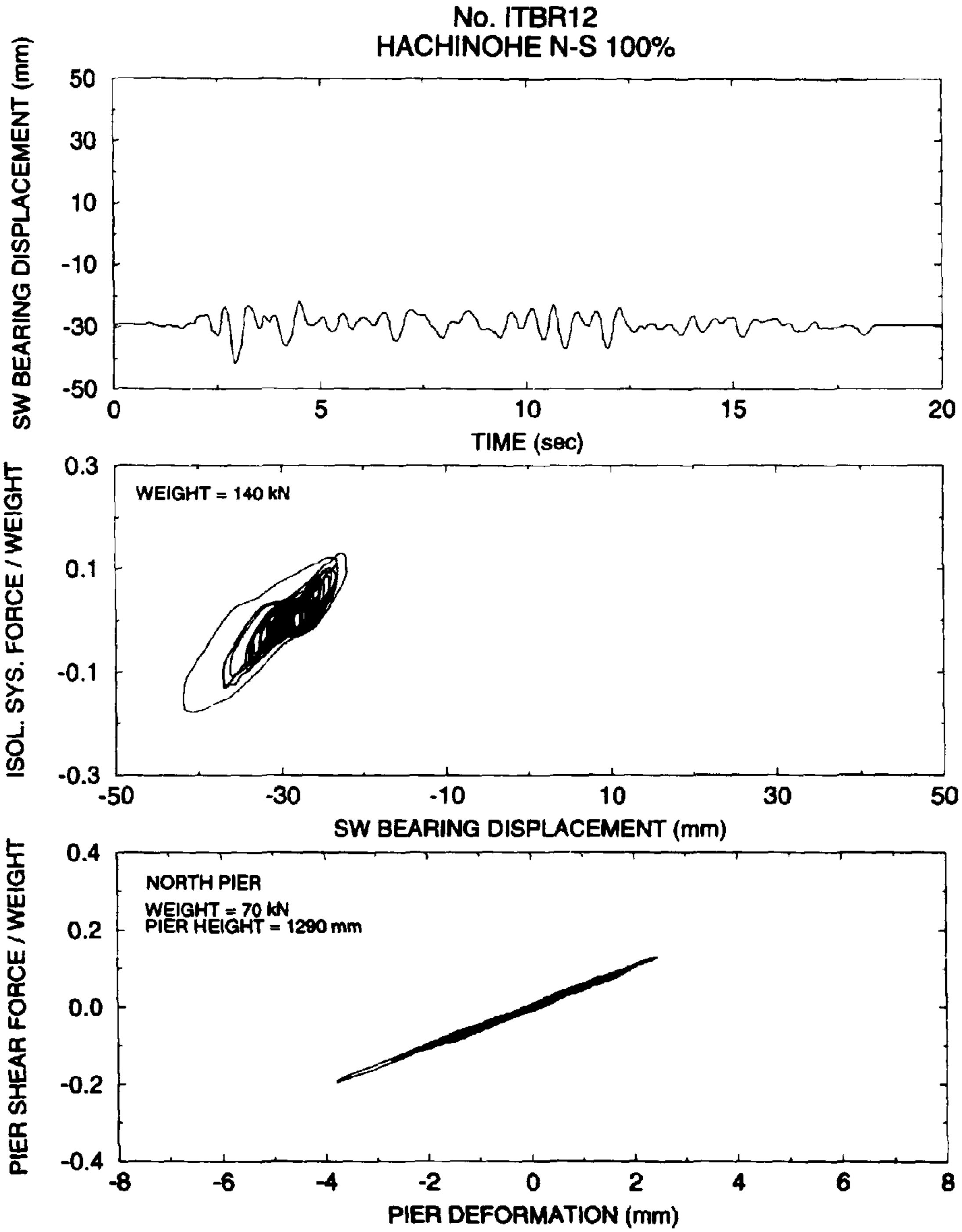


No. ITBR10
EL CENTRO S00E 150%

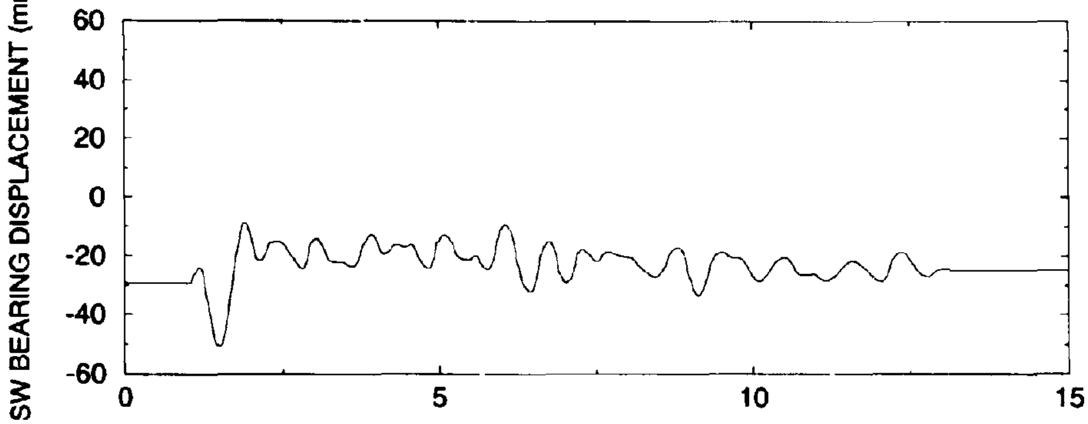


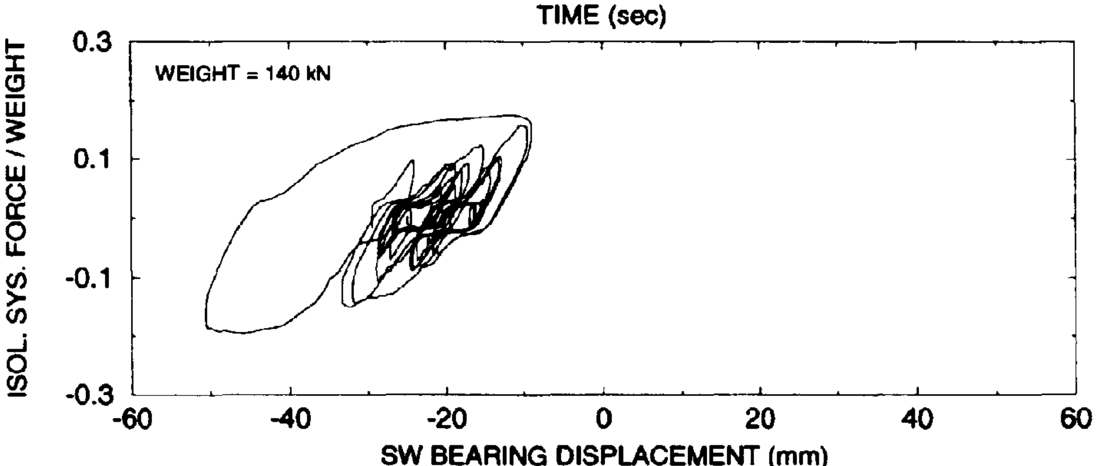
No. ITBR11
TAFT N21E 200%

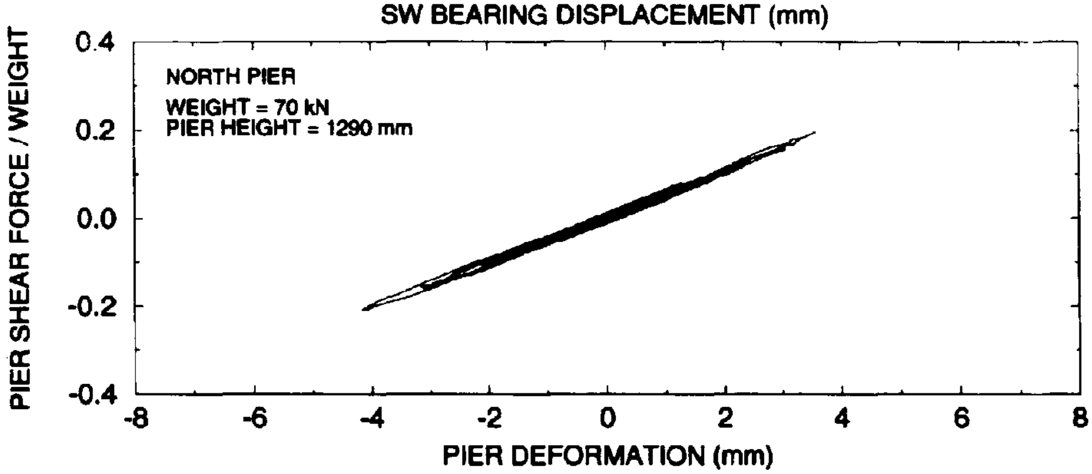



No. ITBR12
HACHINOHE N-S 100%
SW BEARING DISPLACEMENT (mm)
50
30
10
-10
-30
-50
0
5
10
15
20
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
0.3
0.1
-0.1
-0.3
WEIGHT = 140 kN
-50
-30
-10
10
30
50
SW BEARING DISPLACEMENT (mm)
PIER SHEAR FORCE / WEIGHT
0.4
0.2
0.0
-0.2
-0.4
NORTH PIER
WEIGHT = 70 kN
PIER HEIGHT = 1290 mm
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)

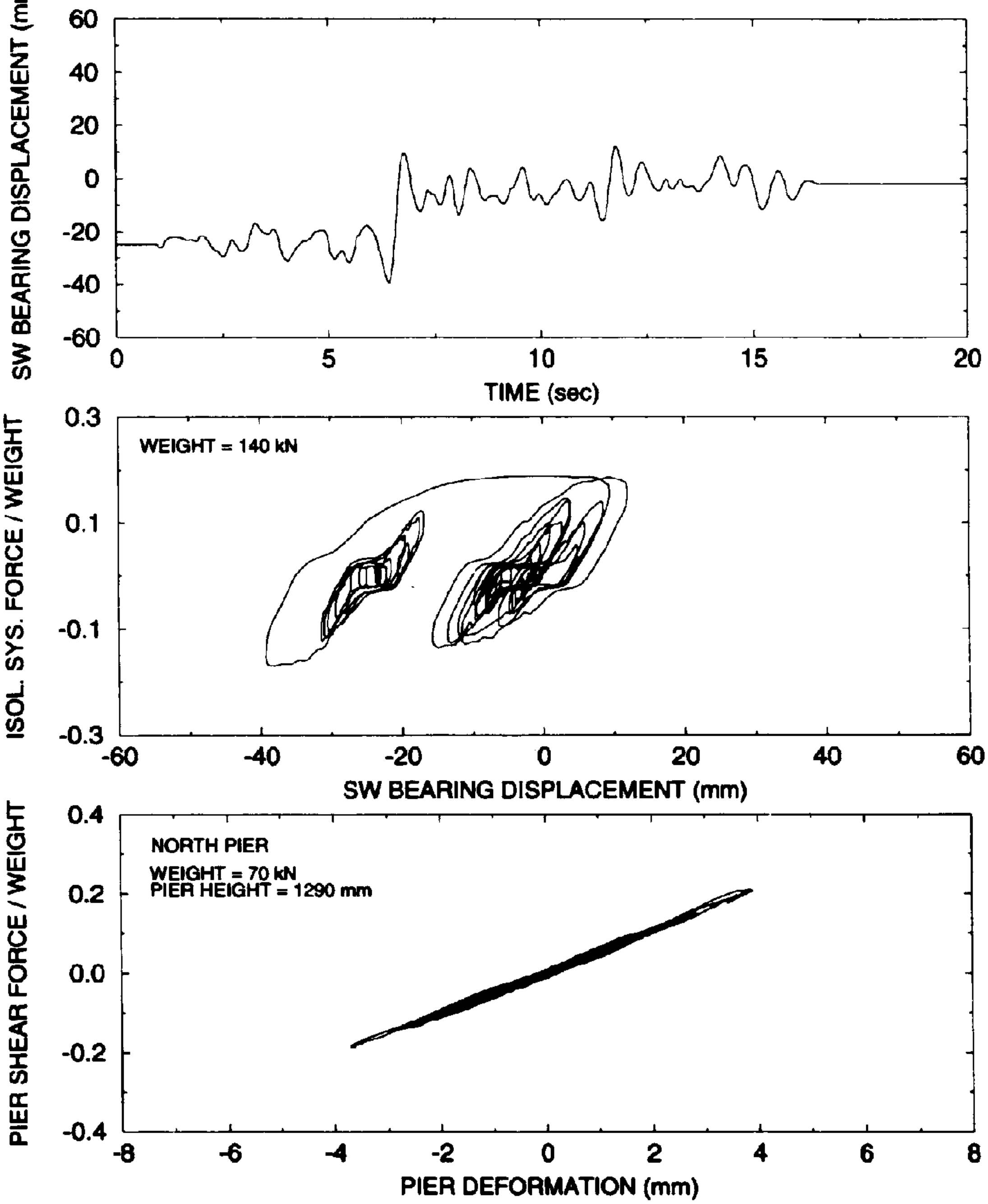
No. ITBR13
JP. LEVEL 2 G.C. 1 50%

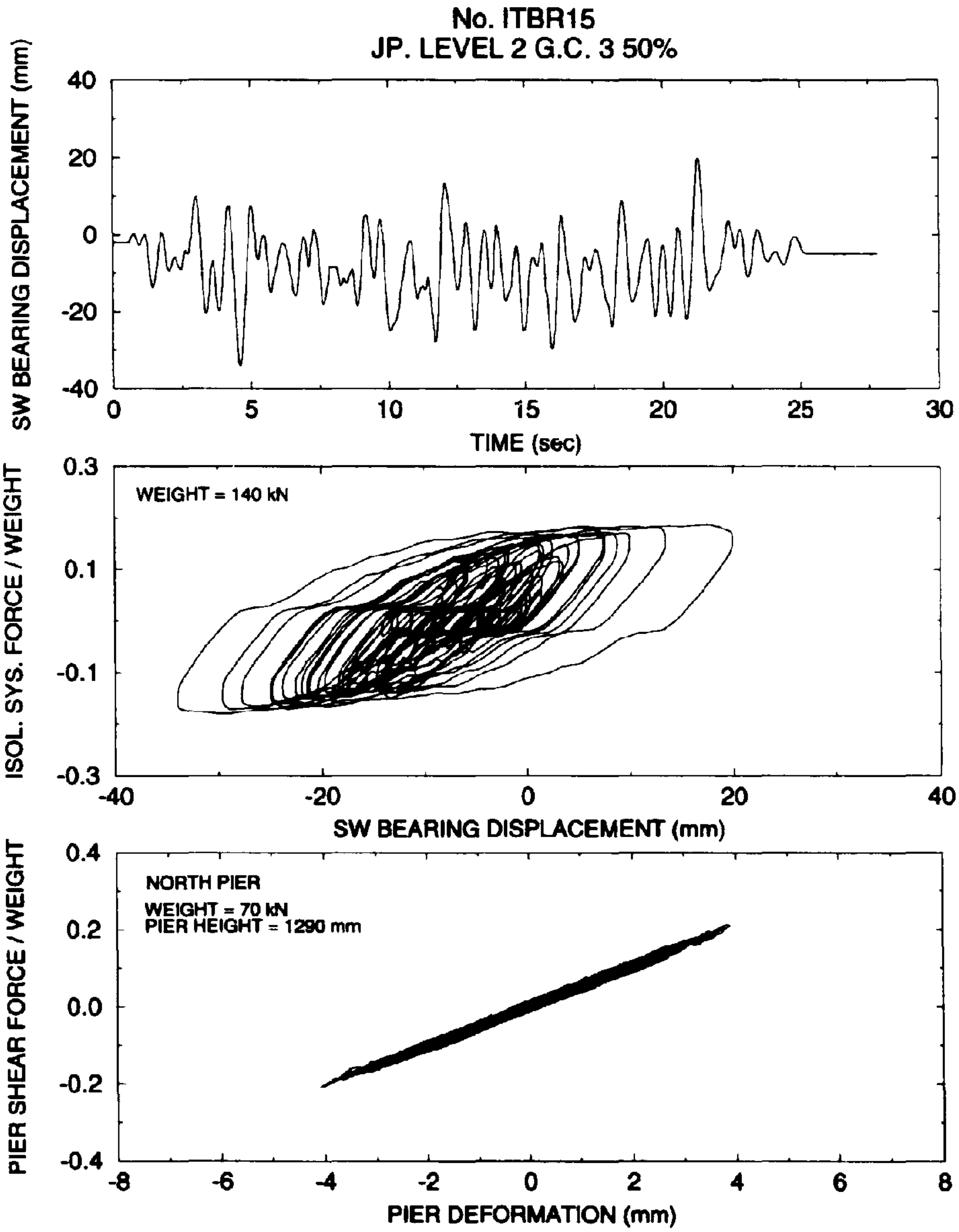






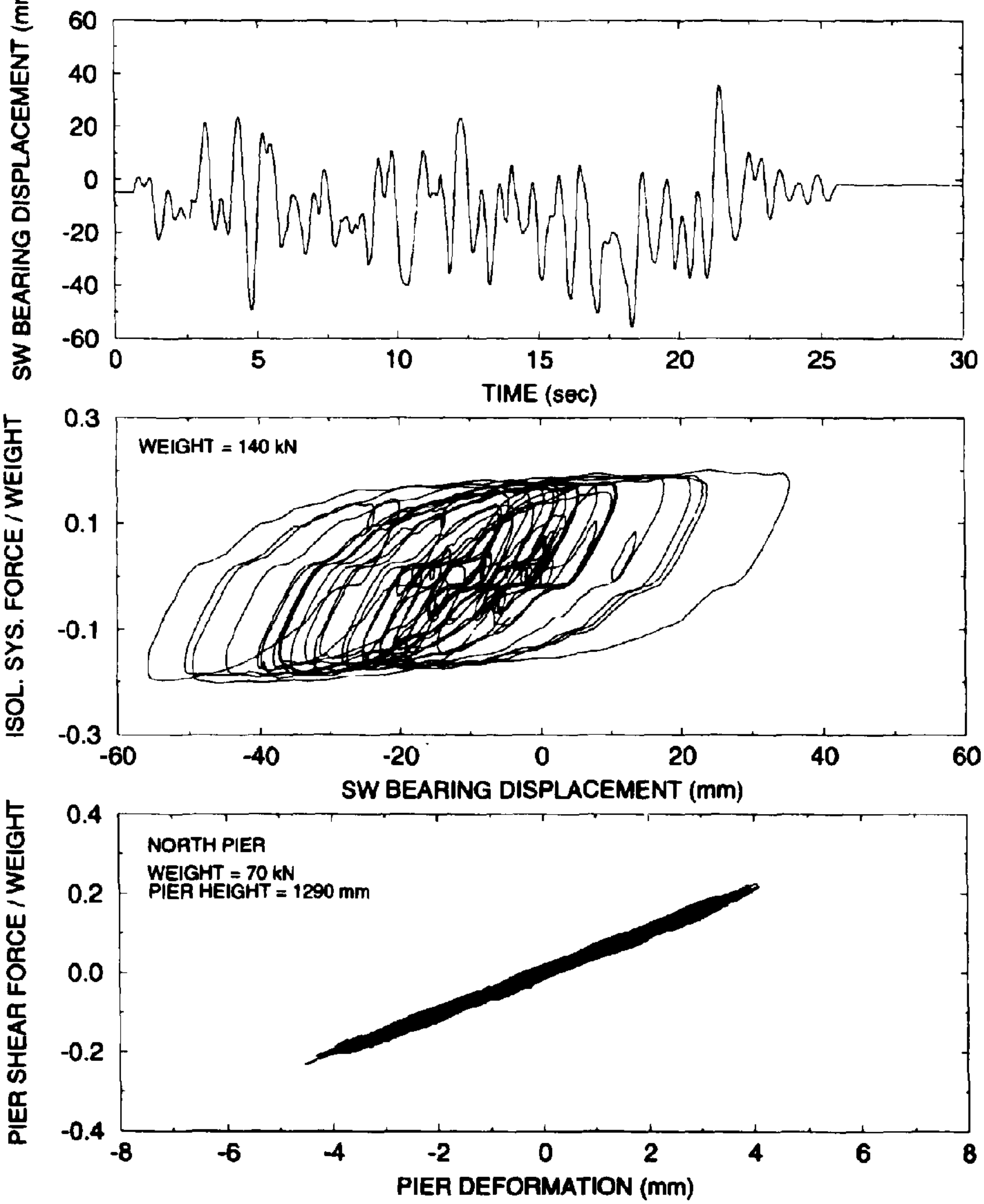
No. ITBR14
JP. LEVEL 2 G.C. 2 50%





No. ITBR15
JP. LEVEL 2 G.C. 3 50%
SW BEARING DISPLACEMENT (mm)
40
20
0
-20
-40
0
5
10
15
20
25
30
TIME (sec)
ISOL. SYS. FORCE / WEIGHT
WEIGHT = 140 kN
0.3
0.1
-0.1
-0.3
-40
-20
0
20
40
SW BEARING DISPLACEMENT (mm)
PIER SHEAR FORCE / WEIGHT
NORTH PIER
WEIGHT = 70 kN
PIER HEIGHT = 1290 mm
0.4
0.2
0.0
-0.2
-0.4
-8
-6
-4
-2
0
2
4
6
8
PIER DEFORMATION (mm)

No. ITBR16
JP. LEVEL 2 G.C. 3 75%



NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
LIST OF TECHNICAL REPORTS

The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER's Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259). This report is available only through NTIS (see address given above).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-218522).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333). This report is only available through NTIS (see address given above).

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