Continuous Monitoring of Isolated Bridges: Parametric Analysis of the Effects of Damper Characteristics

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ABSTRACT
This paper presents preliminary results of a phase of an in progress research project for the definition of an effective health monitoring approach for bridges protected with the most common seismic response modification devices. The research aims at advancing existing health monitoring techniques to the level of accuracy needed to detect variations of performance of the isolation devices and, at the level of overall structural response, to define the state of integrity of the bridge. One phase of the general research plan involves the study of the ranges of performance variation of the devices due to use and damage. As a first case study, the cable-suspension Vincent Thomas Bridge was selected due to the presence of a sensor network and of viscous energy dissipation devices. After a preliminary validation of the numerical model of the bridge, aimed at ensuring the reliability of the FE model, the sensitivity of the bridge seismic response to the variation of the dissipator's characteristics has been analyzed.

INTRODUCTION
After natural events like the 1994 Northridge (USA), the 1995 Kobe (Japan), the 1999 Chi-Chi (Taiwan) and the 1999 Duzce (Turkey) earthquakes it became evident that the demand for bridge structures could greatly benefit from the application of isolation/energy dissipation techniques. A pilot project was initiated after the 1994 Northridge earthquake by the California Department of Transportation that identified the use of isolation solutions as the only economically convenient approach to retrofit particularly complicated structures as the bridges part of the Toll Bridge project (Mellon, 1997). Most of these projects are now in the completion phase and the California applications contributed to increase the confidence of bridge designers in the use of this technology. An increasing number of new bridges are also protected, worldwide, with isolation solutions taking advantage of a continuous research effort in the technological improvement of these devices, testing programs, and code development. Despite the level of maturity achieved in the field of seismic isolation, open questions still remain on the durability of seismic response modification devices (SRMD) under working conditions. Devices are traditionally designed with emphasis on their extreme condition performance, like during a seismic event, but relative motion of the bridge components, traffic service loads, together with different environmental conditions raise concerns about the stability of their performance for the future. The use of more and more advanced material, like composites, for isolators and energy dissipators underlines the importance of this aspect. Presently Departments of Transportation include in their maintenance plan the removal of sample devices from the structures to verify

their performance characteristics. This solution, of course, represents a significant cost, particularly if associated with disruption of the regular traffic. Structural Health Monitoring (SHM) techniques appear a valid alternative, particularly when able to integrate experimental performance results of the isolation components with the overall bridge performance. Applications to isolated bridges are however very limited and do not include experimentally validated performance curves of isolation devices extending in the damaged range.
The research program is aimed at advancing existing SHM techniques to the level of accuracy needed to detect variations of performance of the isolation devices and to relate them to the overall structural response, in order to define a level of significance of the bridge integrity. Ideally the assessment of the system health should not be limited to the identification of the damage but also its location, its severity and its performance impact (Sikorsky et al., 2003). In this perspective, the application of existing SHM algorithms can be successfully applied to the monitoring of isolated bridges, if preliminary additional information on the device performance is gathered. For isolated structures important simplification in the overall damage assessment procedure applies. In this case, in fact, the source and nature of the potential damage is clearly known, since it is related to the non-linear performance of the SRMDs. The "damage localization" stage of any Non-destructive Damage Evaluation (NDE) procedure is therefore greatly bounded. The difficulty of the overall structural simulation is also simplified by the assumption that the structural components perform within the elastic range, until significant level of damage occurs in the SRMDs. The focus of the procedure is, in other words, shifted from the traditional structural elements to the isolation devices. Technical difficulties however still require further research efforts. Uniqueness and observability problems can be associated with the presence of redundant structural members (as additional energy dissipators can be) and limited sensor locations (Agbabian et al 1991). The presence of noise in sensor recordings can reduce the accuracy and reliability of various identification algorithms. Most important, the algorithms must be significantly sensitive to small changes in the performances of the parameters of interest, e.g. the ones characterizing the effective behavior of SRMDs. One of the goals of the proposed research is to improve the required sensitivity of few selected existing approaches. The way to achieve this goal is based on the enrichment of the NDE methods with a major support from experimentally generated data.

General Research Plan

The main objective of this project is the definition of an effective health monitoring approach to be applied to bridges protected with the most common SRMDs (elastomeric bearings, sliding bearings, viscous dampers). This goal will be achieved through the specific contributions schematically visualized in Figure 1.
As part of phase 1, the unique set of data available at UCSD, generated from full-scale testing of isolation and energy dissipation devices is currently under study to provide a database of performance range for different kinds of SRMDs. Based on the existing database, trends of performance variation can be extracted although the devices cannot generally be considered damaged. For this reason an extension of the completed test program is planned in order to widen the range of performance beyond the service conditions of the devices. The first tests are already scheduled for the end of November 2005 and will involve a viscous damper in simulated damaged stages.

The results of the analyses conducted on a FE model of the Vincent Thomas Bridge (VTB) are presented in this paper. The work, as part of phase 2, is aimed at the definition of the structural elements affected by variations of the response characteristics of the energy dissipators.
The general project involves the design of reliable numerical models of the device performance (Phase 3). Some of these models, available in literature, were validated during the prototype and production tests completed at the Caltrans SRMD testing facility on full scale devices. The range of validation however was tailored to the specific application projects. An extension of this validation to a wider range of displacement, load and speed is part of this project and of an additional Caltrans Project ("Effect of the Axial Force Variation on Seismic Response of Isolated Bridges"). The experimental tests, part of Phase 1, will also allow the acquisition of experimental data needed to adapt the numerical model in order to describe the device degradation stages.

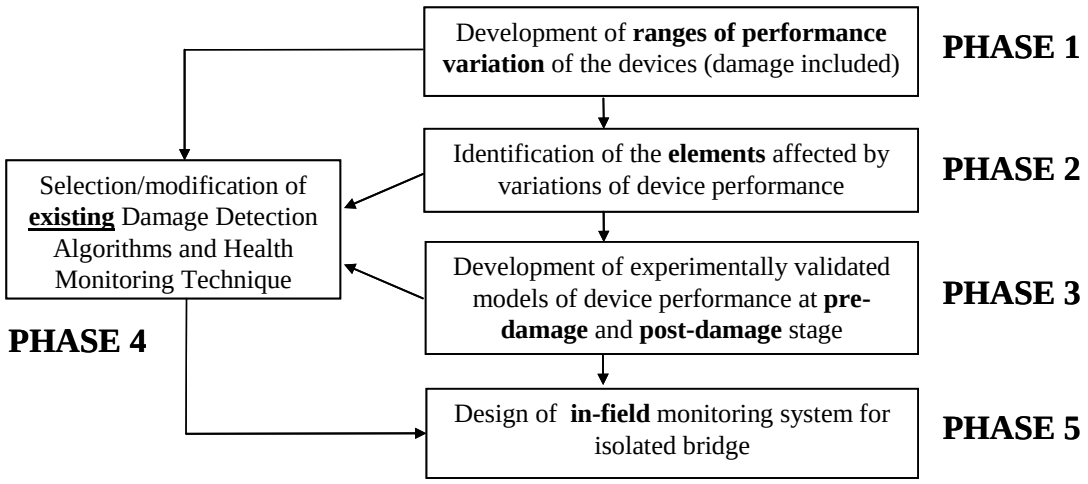

Figure 1. General Project Phases

For the object of phase 4, several techniques are under study. One of them (Stubbs et al, 1996) is based on the monitoring of changes in the structural dynamic properties (i.e. mode shapes and frequencies). The procedure is able to identify if damage has occurred, localize it, estimate the severity of the damage and evaluate the impact of the damage on the structure. It has been implemented on existing bridge structures, although not isolated (Sikorsky et al. 2003). Improvements are presently considered in order to account for the effects generated by the variations of the damping characteristics of the system under consideration.
The final phase of the project (phase 5) will consists of the implementation of the overall procedure on an in-field monitoring system for an isolated bridge.

THE VINCENT THOMAS BRIDGE CASE STUDY

The first case study of this project consists of the Vincent Thomas Bridge, a cable-suspension structure retrofitted in different stages, and lately equipped with 48 viscous dampers. As part of the phase 2 of the general research plan, the sensitivity of the bridge seismic response to the variation of the damper characteristics has been analyzed.
The study has been conducted by means of nonlinear time-history analyses of a detailed three-dimensional FE model of the bridge provided by Caltrans. Such numerical model, including cables, suspenders, suspended structure, towers, cable bents and anchorages, reflects the state of the structure after the last retrofit phase, when dampers and fuses were installed, and towers were

stiffened. The static and the time-history analyses of the bridge were both geometrically nonlinear (large displacement analyses) to account for the geometric stiffness of the cables and suspenders. An exhaustive description of the FE model can be found in Ingham et al. (1997).
The instrumentation network currently installed on the bridge has been triggered during the 1987 Whittier–Narrows earthquake (M = 6.1) and the 1994 Northridge earthquake (M = 6.7). The seismic excitations used in the present work have been chosen among time history records from these earthquakes obtained in the proximity of the bridge.

Validation of the Numerical Model of the VTB

A preliminary validation of the accuracy of the FE model in representing the bridge structural response has been performed. Considering that the Northridge earthquake occurred before the installation of dampers, a configuration without dampers has been implemented, even if the other strengthening interventions are present in the model.
Currently, twenty-six seismic sensors are installed on the bridge to record ambient and seismic behavior (Smyth et al., 2003). The three components of the recorded ground signals were applied both in the form of displacements at the anchorages and towers, and as global ground acceleration. The best agreement with experimental data was found in case of applied displacements, due to the complexity of the foundation system. The use of accelerations at the ground level did not provide matching signals at sensor locations. For this reason, recorded displacements forcedly applied at the base sensor locations represented the most feasible approach.

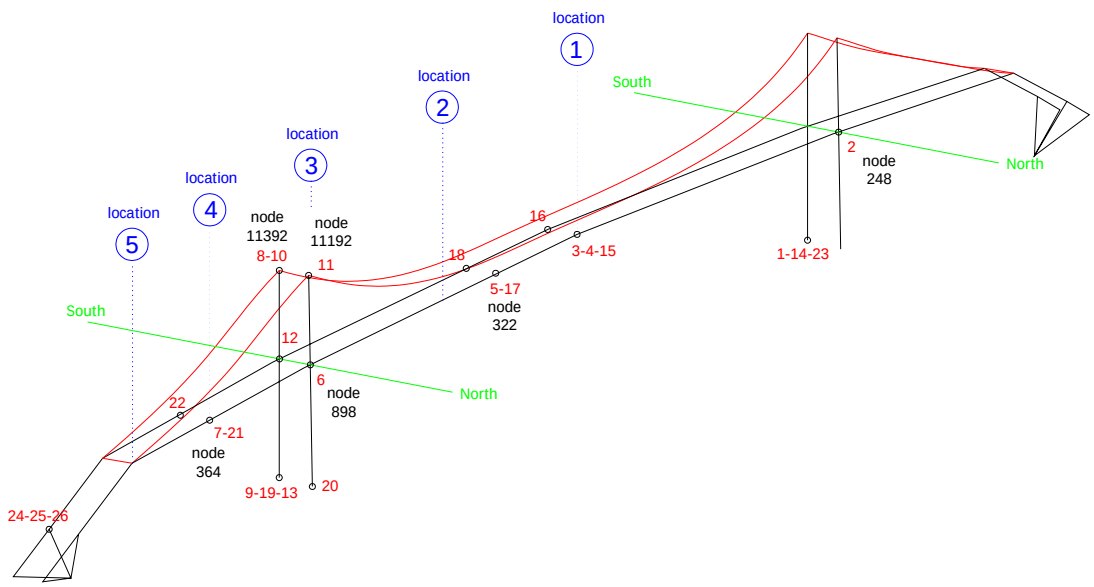

Figure 2. Recording sensors identification numbers on the bridge

Results were compared to the recorded motions in terms of displacements, velocities and accelerations. Figure 3 shows displacement results for sensors at the different positions of the structure identified in Figure 2: the black solid line represents the recorded response, the light solid and dashed lines represent the numerical responses at the same locations on the northern and southern sides, respectively. The comparison between the numerical and recorded bridge responses was also performed by computing transfer functions between the response at the tower

bases and at sensor locations. Additionally, the PSD functions of the same sensor responses have been computed and compared with the ones obtained from the numerical analysis.

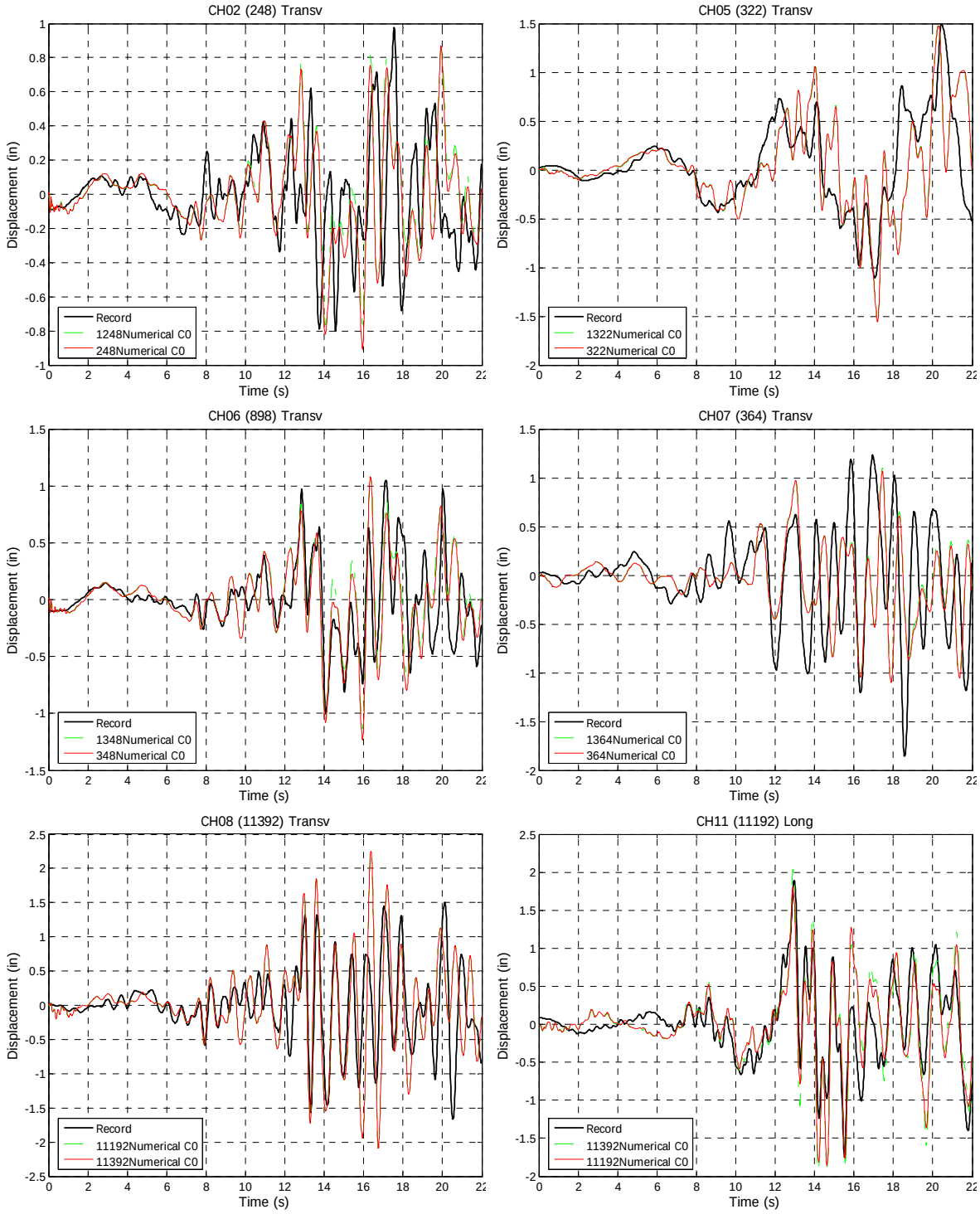

Figure 3. Channels 2, 5, 6, 7, 8 and 11 displacement responses: numerical and recorded

Table 1 shows the main frequencies obtained from the modal analysis of the bridge and recognizable in the transfer functions, both from numerical and recorded signals. Results indicate

also a satisfactory agreement with previous works presented in literature (Conte et al., 2003; Ingham et al., 1997). The frequencies identified in the recorded response are reasonably matched by the ones provided by the present numerical model. However, other significant frequencies are indicated by the modal analysis and the numerical transfer functions. This is attributed to the differences in the configuration of the FE model and of the actual bridge at the time of the earthquake event. Within the scope of the present project, the obtained agreement with the recorded data still indicates a general reliability of the FE model.

Table 1. Location and characteristics of the dampers in the FE model

Mode	Frequencies (Hz)					
Longitudinal	0.13	0.57	0.91	1.07	1.49	2.09
Transversal	0.2*	0.57	0.9	* evident only in the numeric response		

Parametric study of the characteristics of the dampers of the VTB

The following set of analyses was aimed at the evaluation of the effects of the damper characteristics variations on the structural performance, but also of the realistic range of forces and displacements associated with the devices. The latter information is clearly critical to relate the expected and the experimental device performance. It must be noted that, at this stage, laboratory data for such devices are available only through publications of an experimental campaign completed by HITEC (1999), however additional tests are scheduled on new devices provided by Caltrans and designated to replace some existing ones on the bridge. Once removed, the original dampers will also be tested to provide information on their state of degradation.

The dampers are modeled by nonlinear spring elements with assigned stiffness and damping properties represented by a force (F) – velocity (V) relationship of the type $F = CV^a$, where a describes the nonlinear part of the response. The dissipators are located between the suspended structure and the towers, at the side spans coupled with the fuses and at the cable bents. The location and the current characteristics of the dampers are described in Table 2, where the label number refers to the locations in Figure 2.

Table 2. Location and characteristics of the dampers in the FE model

Label	Location	C	a
D5	side span to cable bent bottom chord dampers	5	1
D4	side span hinge damper at fuses	100	0.5
D3s	side span to tower	2.5	1
D3m	main span to tower	4	1

In the parametric analyses, the damper characteristics are varied according to Table 3. In case of structure without dampers, the associated springs are characterized only by their linear stiffness properties. The input records were applied in the form of global ground acceleration. Analyses were performed in the longitudinal and transversal directions, separately. The results were analyzed in terms of effects on damper forces, structural displacements, member forces and global structural reactions. In the undamped configuration, damper forces are measured in the linear spring elements substituting the devices.

Table 3. Ratios of the damper constants employed in the analyses to their nominal values (Table 2)

		a_0 not present	a/a_1 0.25	a/a_1 0.50	a/a_1 1.00
C_0	not present	no dampers			
C/C_1	0.50				Combo 2
C/C_1	1.00		Combo 5	Combo 4	Combo 1
C/C_1	2.00				Combo 3

Longitudinal Response

The fuse locations attract a very high force, shared with the energy dissipators when present. For this reason, the dampers D4 (at fuses) are the most activated devices in both Whittier and Northridge earthquakes, followed by dampers D5 (at side-span to cable bent), which experience a force increase of 20 and 45 times the values obtained in absence of dampers, for Whittier and Northridge earthquakes, respectively. In both the excitation cases, the peak forces in the dampers are obtained for the configuration Combo 3 (C/C_1=2).

The most evident structural response modified by the energy dissipators appears to be the displacement response. For the Northridge record, displacements at mid-span are significantly reduced by the presence of dampers, in the range of 30% (Combo 5) to 60% (Combo 3) with respect to the undamped configuration, while at the side-spans they are not considerably affected. Such uneven distribution of the damper influence suggests a significant variation of the dynamic bridge response. On the contrary, for the Whittier earthquake mid-span displacements do not change appreciably with the presence of dissipators, when instead side-span displacements indicate a reduction in the order of 20% with respect to the undamped case. It must be noted that for both earthquakes, D4 dampers (at fuses) develop a force of the same order of magnitude, while D3 dampers (at tower) develop a significantly smaller force for the Whittier record, thus being less effective in reducing the mid-span displacements.

The presence of dampers does not influence considerably the peak actions in the members. The peak reduction of shear forces due to the dampers is 10% and 45% for Northridge and Whittier, respectively. The structural elements experiencing the highest forces are in most cases at the same locations even after the introduction of dampers. Specifically, the anchorage elements, the beams at cable saddles and truss links groups, the beams at the approaching span and the beam-column joints are the members subjected to the highest stress levels.

For both earthquakes the total base shear is not changed appreciably by the presence of dampers. However, their use implies a total base moment reduction of 20% and 35% for Northridge and Whittier, respectively. Thus, even if global forces are not reduced, it appears that the use of the energy dissipators generates a variation of the overall distribution of the forces on the structure.

Transverse Response

Even for the transverse performance, the fuse elements experience the highest level of force, shared with dampers when present. Dampers at these locations are still the most activated devices under both earthquakes. The second group of dampers with highly increased force is D5 (at side-span to cable bent), with a force increase up to 18 and 43 times the undamped case, for Whittier and Northridge earthquakes, respectively. For both inputs, the peak forces in the dampers are obtained for the configuration Combo 3 (C/C_1=2).

Contrary to what observed for the longitudinal response, the transverse structural displacements are high at mid-span, despite the presence of dampers, as well as at the side-span locations, for the Northridge input. For the Whittier case, all displacements along the deck are slightly reduced by the use of dampers, with a reduction of about 5%-10% compared to the undamped scenario. Dampers do not significantly influence the peak actions in the members, with a maximum reduction of shear forces of 15% in the Northridge case. The members with the maximum stress levels are the beams of the tower shafts, the wind shoes, the superstructure elements at the approaching span and the beam-column joints. As indicated for the longitudinal response, the distribution of the highest forces does not change with the damping configuration.
In terms of total base reactions, for both earthquakes, the total base shear and moment are not significantly influenced by the presence of the energy dissipators, in this response direction.

CONCLUSIONS

An important preliminary phase, in order to ensure the reliability of the FE model of the bridge, consisted in the validation of the numerical model by comparing the numerical response with the recorded signals. Such comparison indicated a reasonable agreement between numerical and recorded responses, even though the actual bridge model includes strengthening interventions not present when data were recorded. Additional and more recent data are needed to further validate the model reflecting the actual state of the bridge.
Based on the results of the parametric study on the variation of damper characteristics, it is observed that the devices mainly reduce displacements and redistribute forces, rather then reducing the global amount of forces in the structure. The degradation of the damper characteristics can result in a significant modification of the structural response. The main effect of the dissipators appears to be more significant in the longitudinal direction, due to the devices orientation. Specifically, displacements across the span-tower expansion joints are limited and relative displacements between the side-spans and the towers are reduced.

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