RETROFIT OF SUSPENSION BRIDGES SUBJECTED TO LONG PERIOD EARTHQUAKE MOTION

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SUMMARY

This paper presents the results of preliminary analytical investigations into the reduction of suspension bridge response to long period rich ground motions by the use of distributed supplemental damping elements. Recent retrofits of suspension bridges in the United States have included the use of large damping elements only at a few locations within the superstructure. The use of a larger number of smaller damping devices distributed throughout the superstructure may reduce the overall response and reduce the need for other retrofit measures.

INTRODUCTION

Recent retrofit strategies for suspension bridges located in the earthquake prone West Coast of the United States have included large damping elements placed between the suspended spans and the towers. The primary reason for this measure is to reduce the relative displacement between the suspended spans and the tower and to prevent the span from battering into the tower. These dampers are not intended to reduce the vibrations within the suspended span, and as a result many of the members of the stiffening truss are to be replaced because of the large inelastic demands caused by an earthquake [Ingham et al. 1998]. Suspension bridges are characterised by having many narrowly spaced modes. The response is rarely dominated by only a small number of modes; often the response includes substantial contributions by as many as 75 different modes. Placing damping elements at only a few locations will therefore only reduce vibration in a few of the important modes.

The ground motions utilised in the design of these strategies were typical of West Coast motions in that a majority of the earthquake energy is contained in short period motion. Recent work done by seismologists indicate that ground motions produced by an earthquake in the Central and Eastern portions of the United States may contain significant energy at much longer periods than those typical of West Coast earthquakes [Jost 1989]. Because suspension bridges normally contain dynamic modes of vibration with periods in excess of 2.0 seconds, ground motions containing significant energies in the period range of 2.0 to 15.0 seconds may have a greater impact on suspension bridges than has previously been anticipated.

The present study examines the response of suspension bridges to ground motions containing significant energy in the long period range. A retrofit strategy is investigated in which a large number of supplemental damping devices are distributed throughout the suspended spans to reduce response of many different modes of vibration. This investigation consists of computer simulations of suspension bridges subjected to earthquake records. Several different configurations of the suspension bridge are considered. Both non-linear and linear transient dynamic analyses are utilised. The input earthquake motions are simulated records of the 1811-1812 New Madrid events in Missouri, USA, and real recordings of the 1971 San Fernando event are used for comparison.

MODELING PARAMETERS

The present investigation utilises linear and non-linear transient 3D dynamic analyses to determine the response of a suspension bridge with varying damper configurations subjected to earthquake ground motions.

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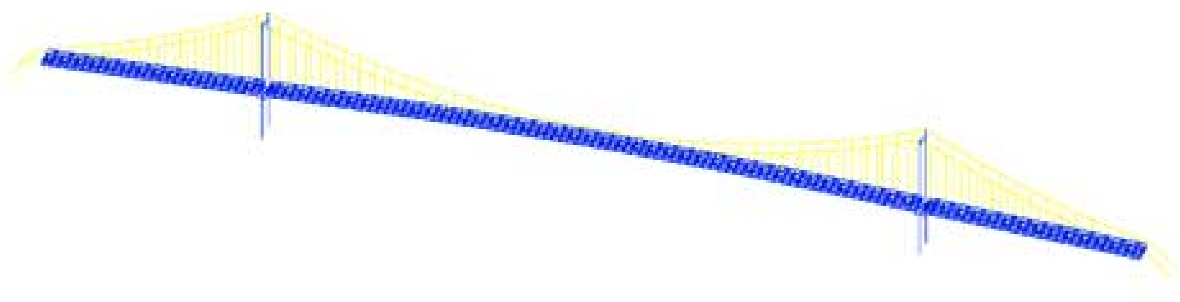


Figure 1: Finite element model of suspension bridge.

Bridge model

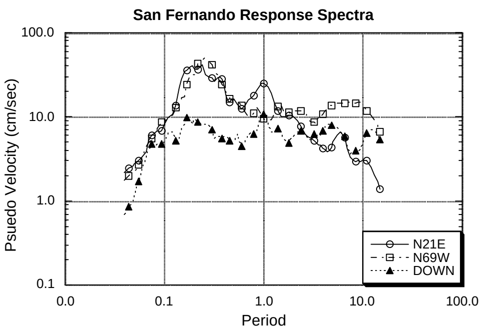
A finite element model of a suspension bridge was used in this analysis. The commercial finite element code ANSYS version 5.4 was used. The bridge is typical of the types of suspension bridges found in the United States. It has 3 suspended spans (229 m, 655 m, 229 m) which are non continuous and supported at the towers and anchorages, a stiffening truss rather than a box girder, cellular steel towers, and a deck slab supported by stringer-floor beam structures. The stringers are fixed longitudinally at only one support.

All bridge elements are modeled by beam elements, except the cables which are modelled as link elements. Initial strains equivalent to the strains induced by dead load only are applied to all cable and suspender elements. The model has approximately 14,000 degrees of freedom and 8055 elements. A general perspective view of the model is shown in Figure 1. For the configurations of the bridge including damping devices, the damping elements were connected between nodes, alongside the truss elements of the bridge.

Ground Motions

Two 3-dimensional sets of ground motions were used in this investigation. The first was that of the San Fernando earthquake recorded and Lake Hughes, Array Station 12. These motions are used to represent the short period rich motions expected in the West Coast region of the United States. Figure 2 shows the pseudo velocity response spectrum of these motions.

The second set of ground motions used in this investigation is a simulated set based on work done by Herrmann & Jost [1988]. These time histories were developed to simulate the New Madrid events of 1811-1812. Because of the geologic and seismic conditions of the central United States, the ground motions from an earthquake in this region are thought to have significant energies in the period range T > 2sec. Because suspension bridges are very flexible, they are vulnerable to excitations in this period range.



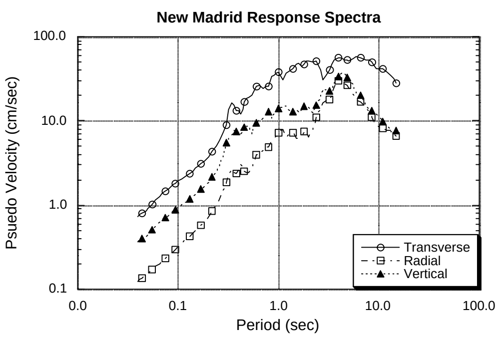


Figure 2: Response spectra of San Fernando E.Q. **Figure 3: Response spectra of New Madrid E.Q.**

Unfortunately, only the response spectra of the simulated motions were available. Therefore, spectrum compatible acceleration time histories were created using the spectral shapes proposed by Jost [1989]. Figure 3 shows the pseudo velocity response spectra of the New Madrid time histories. It is clear from a comparison of Figures 2 and 3 that the New Madrid motions contain significantly more earthquake energy in the longer periods than the San Fernando motions.

These acceleration time histories were integrated to obtain displacement time histories that were used as support displacements for the ANSYS model of the suspension bridge. The San Fernando records have a duration of 37 seconds while the simulated New Madrid motions had a duration of 60 seconds; however, because the excited period of the bridge can be up to 12 seconds, the bridge continues to respond with large amplitudes after the ground motion has stopped. Therefore the response was monitored for a duration of 5 seconds following the end of the San Fernando time histories and 6 seconds following the end of the New Madrid event.

Damping devices

In this investigation, viscous dampers were utilised. The devices modelled were based on those available commercially from Taylor Devices, Inc. [Taylor 1996]. These are the damping devices currently being used in the retrofit of several suspension bridges on the West Coast of the United States [Ingham et al. 1998, Reno & Pohll 1997]. They are fluid viscous devices that dissipate energy by forcing a silicon fluid through orifices at high velocity. The behavior of the devices can be described by the following equation:

$$F = CV^{\alpha} \quad [1]$$

where F is the damping force, V is the velocity across the device; C is a damping coefficient and α is an exponent. These damping devices can be manufactured with a wide range of C and α values. The cost of the devices is generally proportional to the maximum damping force required. Figure 4 shows the location of damping devices in the main span stiffening truss for the cases that include distributed dampers.

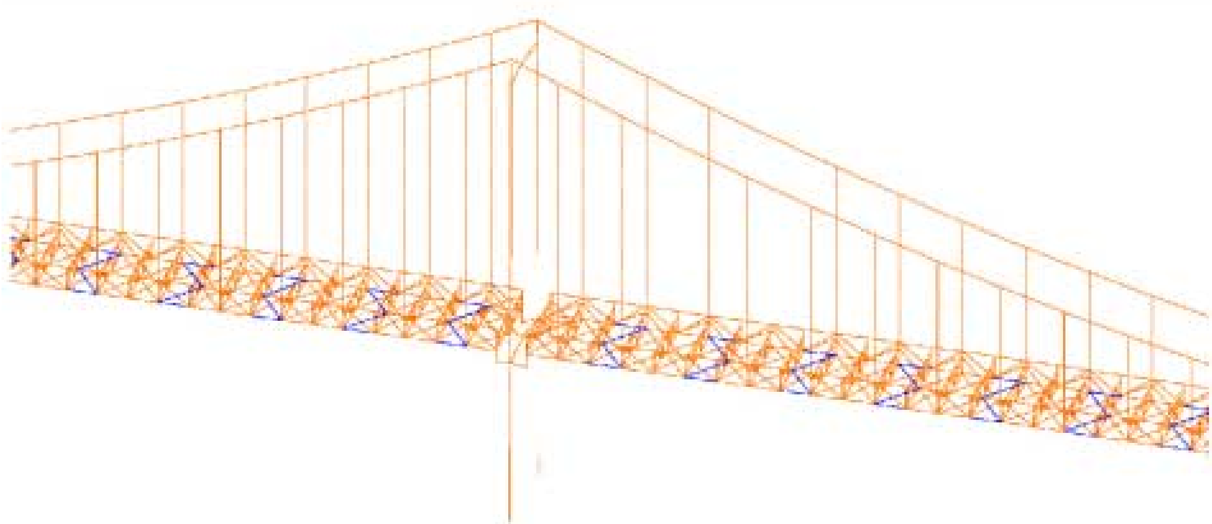


Figure 4: Damping element locations.

RESULTS

The results from a transient dynamic analysis can be quite voluminous. It is important to identify relatively few parameters that will still represent the total behavior of the structure. For these analyses, the maximum and minimum axial forces in a representative number of elements were monitored. Top and bottom chord members, top and bottom lateral diagonal members, vertical diagonal members, and damping members were all represented in the group of monitored elements.

Baseline analysis

There are several sources of non-linear behavior in suspension bridges. Firstly, the very large tensile forces present in the suspension cables add to the stiffness of the cables. This is accounted for by including non-linear terms in the strain-displacement relationships. This is often called the stress-stiffening effect. This stress-stiffening effect can be including in either a pseudo-non-linear or fully linear manner. In the pseudo-non-linear analysis, a stress-stiffness matrix is calculated based on an initial solution of the linear problem. This stress-stiffness matrix is then added to the normal elastic stiffness matrix, and is not updated to capture the effects of changing axial load. In the fully non-linear method, the stress-stiffness matrix is updated at every load step to account for changes in the axial forces.

Secondly, because the stiffness matrices are formed based on the initial geometry, large changes in the geometry under load will create errors in the stiffness matrix formulation. A full geometrically non-linear analysis updates the stiffness matrix based on the deformed geometry at every load step.

For this investigation, it was found that the fully non-linear stress-stiffening effect was very important in capturing the realistic response of a suspension bridge. A full geometrically non-linear analysis was performed on the baseline structure and the deflections were found to be nearly identical to the analysis including only stress-stiffening non-linearities. Therefore for the remainder of the analyses only the stress-stiffening non-linearity was included.

The suspension bridge in its existing configuration without dampers was subjected to the ground motions to obtain a benchmark for evaluating the effectiveness of the damping elements. Figures 5 and 6 show the difference in response of a suspension bridge to the New Madrid and San Fernando events. Both are plots the maximum and minimum force in the bottom chord members of the stiffening truss during the earthquake. The New Madrid event caused a significantly higher force level throughout the stiffening truss bottom chord. This relationship between the responses caused by the two earthquakes was fairly constant across all response parameters investigated.

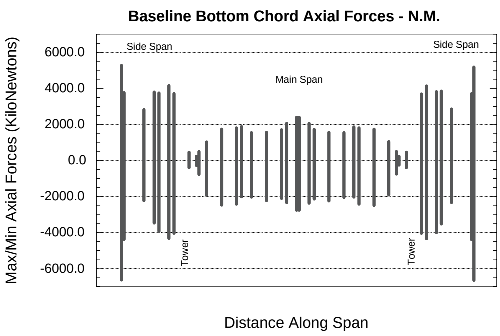


Figure 5: Baseline bot. chord summary – N. M.

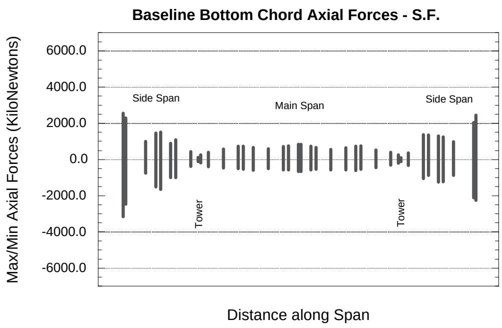


Figure 6: Baseline bot. chord summary – S. F.

Analyses with distributed damping

Five different configurations of the damping devices were utilised. The dampers were located at the same positions in the stiffening truss for all cases, but the stiffness of the truss members alongside the dampers and the parameters of the dampers themselves were varied.

Case 1: The truss members alongside the dampers were reduced to one half of the original stiffness values. The damping coefficient for the devices was 2190 kN-sec/m. Alpha was 0.5.

Case 2: The truss members alongside the dampers were reduced to one half of the original stiffness values. The damping coefficient for the devices was 4380 kN-sec/m. Alpha was 0.5.

Case 3: The truss members alongside the dampers were reduced to one half of the original stiffness values. The damping coefficient for the devices was 4380 kN-sec/m. Alpha was 1.0.

Case 4: The truss members alongside the dampers were reduced to one half of the original stiffness values. The damping coefficient for the devices was 4380 kN-sec/m. Alpha was 1.5.

Case 5: The stiffness of the truss members alongside the dampers was nearly eliminated, the damping coefficient was 4380 kN-sec/m and alpha was 0.5.

Cases 2 through 4 have identical damping coefficients and stiffnesses, but varying values of alpha; Cases 2 and 5 have identical damping parameters but different stiffnesses.

Stiffness variations

Because the damper force is dependent on the relative velocity between its end nodes, the stiffness of the truss element that is also present is important. An element with significant stiffness will prevent large relative velocities, which will require damping devices with very large damping coefficients in order to significantly impact the bridge response. The relative stiffness of the truss member alongside the damping elements was varied to determine its effect. Two values of stiffness were used: (1) one half of the initial stiffness, and (2) a very low stiffness value. A member with zero stiffness was not utilised because of the convergence problems encountered in the analysis.

Cases 2 and 5 have identical parametric values except for the stiffness of the truss members alongside the dampers. In Case 2, the stiffnesses of the truss members alongside the dampers were reduced to one half of their initial values. In case 5, the stiffness was nearly eliminated.

Figures 5 through 11 show different response parameters for the Baseline and Cases 2 and 5. These parameters are the maximum and minimum (tension and compression) forces developed in the bottom chord members and the bottom lateral bracing (wind diagonal) members of the stiffening truss. The reduction in stiffness had the expected result of increasing the relative velocity across the damping devices, which increased damper forces and overall effectiveness. The maximum dynamic forces in the main span chord and lateral diagonals were reduced in comparison with the baseline case.

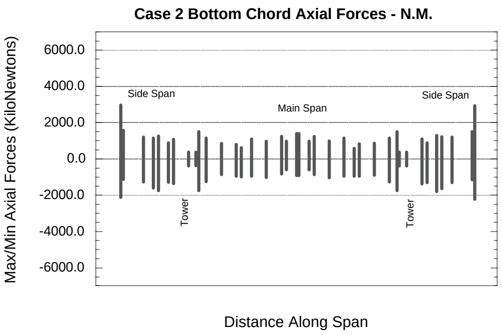


Figure 7: Case 2 bot. chord summary – N. M.

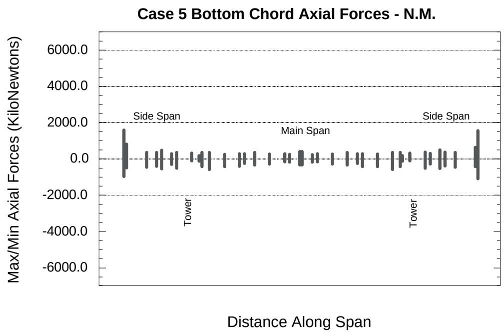


Figure 8: Case 5 bot. chord summary – N. M.

Considering the bottom chord forces only (Figures 5, 7, and 8). Case 2 has significantly smaller forces than the baseline. Even with significant stiffness in the truss members, the response of the bridge is greatly reduced. Case 5, however, shows a dramatic reduction in force levels. It can clearly be seen that the large reduction in stiffness has allowed the damping elements to function more effectively and forces in the bottom chord members are correspondingly reduced.

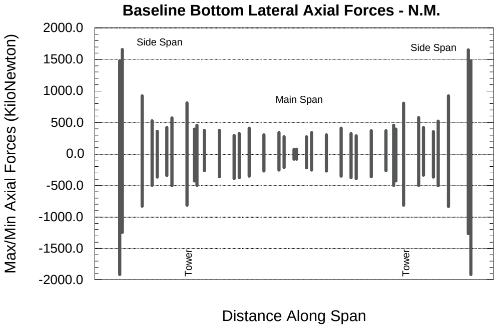


Figure 9: Baseline bot. lateral summary – N. M.

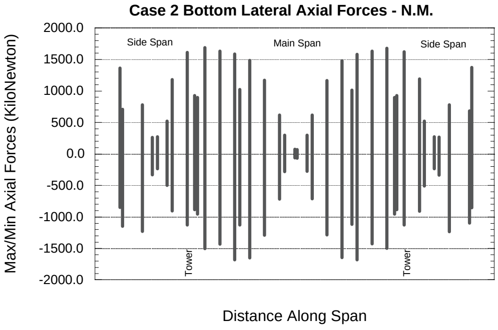
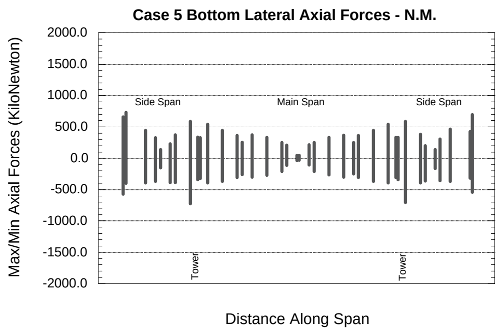


Figure 10: Case 2 bot. lateral summary – N. M.



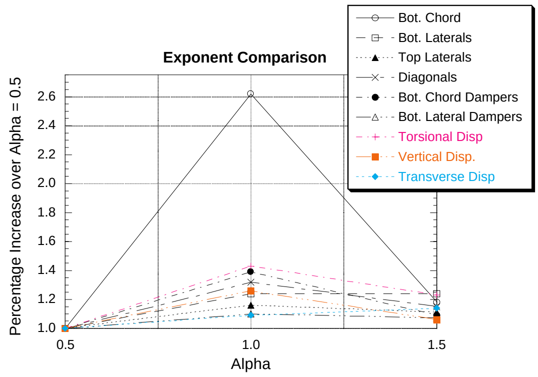


Figure 11: Case 5 bot. lateral summary – N. M. **Figure 12: Variation of response with changing alpha**

The results for the bottom lateral members show a slightly different effect. Considering the Baseline and Case 2 (Figures 9 and 10) it is apparent that while Case 2 shows a mild reduction in the maximum force, the average force level has greatly increased. Thus, a shift in the behavior of the bridge has occurred wherein the lateral members are now called upon to carry larger loads. When Case 5 is considered (Figure 11) all the member forces are shown to have been reduced to a level below that of the baseline.

Damper parameter variations

The value of α in equation 1 was varied to determine its effect on response and on maximum damper force. Cases 2 through 4 have identical parameters except for the value of alpha. The value of alpha changes from 0.5 in Case 2, to 1.0 in Case 3, and 1.5 in Case 4. Figure 12 shows the percent change in numerous response parameters with changing alpha.

In both Case 3 and Case 4, the response is higher than in Case 2. This is specially true for the forces in the bottom chord members of Case 3. This displays a decline in the ability to decrease the response of the bridge with increasing alpha. In addition, it is important to note that the maximum damper forces, and hence the cost of the damping devices, also increases between Case 2 and Cases 3 and 4. Thus, even though the dampers are producing more force, the reduction in response is less. This decrease in damper efficiency appears to indicate that a value of 0.5 for alpha is more beneficial than higher values.

CONCLUSIONS

Although this investigation has been far from thorough, the following conclusions can be drawn:

Placing damping devices throughout the stiffening truss of a suspension bridge can reduce the response of the bridge to earthquake excitations.

Earthquakes in the Central and Eastern regions of the United States could have a greater impact on suspension bridges due to the higher energy content expected at longer periods.

Reducing the stiffness of the truss members alongside the damping devices increases the effectiveness of the dampers.

Dampers exhibiting a damper force proportional to the square root of the velocity are efficient in reducing bridge response.

Aspects of this technology which require further investigation include the effects of large intra-truss displacements on the deck structure, the behavior of a bridge with reduced stiffness under static and wind loads, and a wider variety of damping device types and capacities.

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