SEISMIC RETROFIT OF THE VINCENT-THOMAS SUSPENSION BRIDGE

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This paper evaluates some seismic retrofit schemes that had been proposed during a study completed in 1997 for the Vincent-Thomas Suspension Bridge in southern California, USA. A rigorous nonlinear seismic-response analysis of the as-built bridge and the retrofitted bridge is performed using 3-D analytical models. The retrofit schemes evaluated in this study include using longitudinal dampers in the suspended side spans to dampen the axial forces in the stiffening truss members, in addition to other dampers placed between the stiffening truss and the towers, and between the truss and the cable bents. Another retrofit measure is to allow the formation of plastic hinges at the base of the tower shafts during severe seismic events. The design ground motion used in this study was especially developed for the project to represent a site-specific multiple-support excitation. The study concludes by emphasizing the efficiency of the proposed seismic retrofit schemes in reducing the seismic demand on the bridge.

INTRODUCTION

The seismic risk to long-span bridges in California was forcefully brought to the attention of the California Department of Transportation (Caltrans) by the Loma Prieta earthquake of October 1989 in northern California, and further stressed by the Northridge earthquake of January 1994 in southern California. As a result of these two major earthquakes, Caltrans contracted several projects in 1995 to private consulting firms to seismically upgrade six toll bridges in the Golden State. Among these bridges, was the Vincent-Thomas Suspension Bridge, which was contracted to Moffatt & Nichol Engineers of Santa Ana, California. Several technical groups participated in the retrofit study of that bridge, which was completed in 1997, with Moffatt & Nichol serving as the main contractor with Caltrans. Several retrofit schemes were developed during the study based on the experience of T.Y. Lin International of San Francisco, California, with the Golden Gate Bridge, and were discussed among the participating technical groups. This paper evaluates three of the proposed retrofit schemes in order to examine their effectiveness.

There are generally two main strategies for seismic retrofit of structures; strengthening, to increase the capacity for resisting the seismic demand, or using isolation or energy dissipation devices to reduce the seismic demand. Both approaches could also be combined to achieve an optimal solution. It is typical in retrofitting old steel suspension bridges to strengthen their towers, stiffening trusses and, more importantly, the connection of the deck system to the towers. At expansion joints, cable restrainers could be used to reduce the earthquake movements and, in some cases where their use is not enough to resist unseating leading to potential collapse, catcher blocks are used to avoid collapse. Additionally, sacrificial expansion joints, also known as structural fuses, have been used on a limited scale to provide additional movements during earthquakes and may potentially be used for a broader range of bridges if design details and guidelines are developed for their use. Energy dissipation devices (dampers) have also been used on a limited scale as a seismic retrofit measure for long-span suspension bridges by absorbing energy from the structural system and then dissipating it, usually as heat. For such long-span bridges, these devices need to be large, able to perform in extreme environments, reliable, and durable with low maintenance.
Figure 1. The Vincent-Thomas Suspension Bridge.

(a) General view of the bridge.

(b) Some structural details of the suspended structure and tower.
The plan proposed for the seismic upgrade of the Vincent-Thomas Suspension Bridge included both tuning the bridge to reduce the violent actions caused by the ground motion of a strong earthquake, and strengthening the bridge to minimize the damage caused by these actions. Among the major seismic retrofit measures proposed were providing longitudinal viscous dampers in the suspended side spans to dampen the axial forces in the stiffening truss members, in addition to other dampers placed between the bridge stiffening truss and the towers in both the center and side spans, and between the truss and the cable bents in the side spans. Another retrofit measure was to allow the formation of plastic hinges at the base of the tower shafts during severe seismic events in order to limit the longitudinal bending moment at the tower base to its plastic moment capacity. These proposed retrofit schemes have been evaluated in this investigation by performing a rigorous nonlinear seismic-response analysis of the as-built bridge and the retrofitted bridge, using a three-dimensional analytical model of the bridge.

DESCRIPTION OF THE BRIDGE

The Vincent-Thomas Suspension bridge, shown in Figure 1, was constructed in the early 1960s to connect San Pedro with the Terminal Island at the Port of Long Beach, to the south of Los Angeles, California. The bridge superstructure consists of a 1500 ft suspended center span and two 506.5 ft suspended side spans, with a 52 ft wide four-lane roadway. The suspended structure consists of two stiffening trusses, floor trussed beams and a lower chord wind bracing of K-truss type. The two stiffening trusses with the deck structure and lower chord bracing form a box system with relatively high torsional rigidity. Longitudinal stringers, 7 ft apart center to center, are supported by the transverse top chords of the floor truss. Lightweight concrete was utilized for the deck slabs. The 335 ft high steel tower legs have cruciform cross sections made of four welded box sections. The cable, which has a vertical sag of 150 ft at the center of the main span, consists of 4,028 cold drawn, galvanized, 6 gauge steel wires of ultimate strength of 225 ksi, providing a cable strength of 27,337 kips. The suspenders are made of small diameter high strength wires. A more detailed description of the bridge can be found in Abdel-Ghaffar.4

BRIDGE DYNAMIC CHARACTERISTICS AND THE NONLINEAR ANALYSIS PROCEDURE

It has been shown by several investigators that long-span bridges, in general, are prone to high degree of geometric nonlinearity and three-dimensionality in their dynamic behavior.5-7 For suspension bridges, in particular, the bridge stiffness is very sensitive to the cable geometry. As the applied load changes, the cable shape changes to adjust itself to the applied load causing a change in the overall structural stiffness. Therefore, a geometrically nonlinear analysis that considers large displacement effects is required to capture the full behavior under dynamic excitation.

In the present investigation, a geometrically nonlinear static analysis was first performed on the bridge under the dead load effect to compute its tangent stiffness matrix in the dead-load deformed state. This matrix is then utilized in solving the eigenvalue problem to determine the first 100 natural mode shapes of free vibration and their associated modal frequencies and periods. Figure 2 shows the classification of all 100 modes according to the type of dominant motion (in case of coupled motion, the most dominant one is mentioned first), while Figure 3 shows some selected computed modes of vibration. These figures reflect the unique dynamic characteristics of the bridge. Strong coupling in the three orthogonal directions is observed in a number of modes, e.g. there is coupled torsional and lateral motion of the bridge deck, tower longitudinal motion coupled with vertical and longitudinal motion of the deck, and tower lateral motion coupled with torsional and lateral motion of the deck. Almost all vertical, lateral, and torsional modes of the bridge deck were coupled with cable vibrations, while a large number of pure cable modes was also observed. The coupled motion cannot be captured by simple 2-D modeling, and demonstrates the necessity of using 3-D modeling of the bridge.
Figure 2 also shows the modal damping ratios used in the present study. As an exception from the shown values, all pure cable modes were given a damping ratio of 0.005%, and all modes that involved tower motion were given a damping ratio of 3%.

In this study, a nonlinear time-history seismic-response analysis was performed using step-by-step direct integration of the nonlinear equations of motion, and a tangent stiffness iterative procedure for computational efficiency. The formulation developed by Nazmy and Abdel-Ghaffar for the nonlinear seismic analysis of cable-stayed bridges under multiple-support excitations was utilized in this study, and the computer programs developed by Nazmy for cable-stayed bridges were modified to accommodate the special features of suspension bridges, and then used in the present investigation. Although the linear modal superposition method was not used, the normal mode shapes of the bridge were utilized as a set of orthogonal bases for coordinate transformation, to transform the analysis from the real displacement coordinate space into the modal coordinate space thus reducing the size of the matrices and leading to cost-effective computations. The modal damping ratios were also needed to compute the coefficients of the damping matrix used in the nonlinear analysis.

DESIGN GROUND MOTION FOR THE RETROFIT STUDY

In long-span bridges, it becomes more realistic to assume non-uniform, or multiple-support, seismic excitation, as opposed to uniform excitation, due to their extended length nature. For suspension bridges, the multiple-support excitation imposes relative displacements between the towers and the anchorage blocks, which induce both quasi-static and dynamic stresses and displacements. Due to the high degree of flexibility of suspension bridges, the out-of-phase motions usually give only a minor increase, or even sometimes a decrease, in the stresses in the structure. However, the demands on the expansion joints are generally larger with multiple-support excitation.

The design ground motions time histories used in this study were especially developed for the project by Earth Mechanics Inc. to represent a site-specific multiple-support excitation. The development of ground motion was based on evaluation of the bridge site geology, the relative location of nearby faults, the reoccurrence interval of the seismic event, and computer simulation of ruptures on nearby faults. The peak ground acceleration for the design ground motion was taken as 0.97g in the vertical direction, 1.03g in the longitudinal direction, and 0.96g in the lateral direction of the bridge, where “g” is the gravitational acceleration.

EVALUATION OF THE PROPOSED SEISMIC RETROFIT SCHEMES

The bridge model was analyzed under the effect of the design earthquake discussed above, using the nonlinear time-history analysis procedure described earlier. Three orthogonal components of ground motion were applied simultaneously at all the bridge supports. The analysis was performed on the as-built bridge as well as the retrofitted bridge. This section describes the proposed seismic retrofit measures that have been evaluated in this study, and presents the results of the study.

Using Side Span Chord Dampers at the 1/3 points:

These longitudinal dampers are proposed to be installed in the suspended side spans at the third points from the cable bent end, in order to dampen the axial forces in the stiffening truss members. These dampers will replace the bottom chord member where installed, and structural fuses with a capacity equal to one-half that of the removed chord members will be put in parallel with the dampers to carry live load. The fuses will break early in the seismic excitation. End releases, simulating physical hinges, will be inserted at the ends of the top chord and diagonal members at the same panel where the damper is installed. These “hinges” will be effective for vertical bending only. The force-velocity relationship of these dampers will be:

\[ F = 250 \ V^{1/2} \text{ kips, but } \leq 1200 \text{ kips} \]  

where \( V \) is the relative velocity between the damper ends in inch/sec.

Figure 4 shows the hysterisis loops of the left side span damper during seismic excitation. It clearly shows that a considerable amount of energy can be dissipated by this damper.

Using Tower-Deck and Cable Bent-Deck Dampers

These longitudinal dampers are proposed to be installed between the bridge stiffening trusses and the towers, on both sides of each tower, and between the stiffening truss and the cable bents in the side spans. One damper is placed on each chord of the bridge (i.e. top and bottom chords), at each of the six expansion joints between the spans and the towers and cable bents. These dampers can provide energy dissipation and control of the relative displacement between the spans and the towers or cable bents. They significantly reduce the impact forces between the stiffening truss and the towers, and between the stiffening truss and cable bents. The force-velocity relationship of these dampers will be:

\[ F = 75 \ V^{1/2} \text{ kips, at the towers} \]

\[ F = 1 \ V^{1/2} \text{ kips, at the cable bents} \]
Figure 3. Some selected natural modes of free vibration.
Tower Base Plastic Hinge Formation
Another retrofit measure was to allow the formation of plastic hinges at the base of the tower shafts during severe seismic events, which limits the longitudinal bending moment at the tower base to its plastic moment capacity. The tower becomes pinned at the bottom under any additional seismic forces, and its longitudinal period of vibration increases. The increased longitudinal displacement of the towers can be controlled by longitudinal dampers.

Results and Discussion
The performance of structural components was determined by calculating the demand over capacity ratio (D/C) for every proposed retrofit scheme. Figure 5 shows the variation in D/C for axial force in the top chord of the bridge stiffening truss for the as-built bridge, the case of side-span dampers, the case of tower-deck and cable bent-deck dampers, and the case of plastic hinge formation at the tower base. Figure 6 shows a similar comparison for the axial force in the stiffening truss diagonals. The demand for each member is the absolute maximum force (tension or compression) computed for that member during the nonlinear time-history analysis. This means that these maximum values for different members did not occur at the same time. The capacity was computed for each member once as a tension member and once as a compression member. Therefore, in each plot the D/C values are plotted once assuming that the seismic force in the member is compression, and again assuming the seismic force as tension.

It can be observed in these figures that the installation of dampers at the one-third points in the side spans has considerably reduced the demand on both the chord members and the diagonals of the stiffening truss, especially in the side spans. Furthermore, the installation of dampers between the bridge stiffening truss and the towers and between the stiffening truss and the cable bents has further reduced the seismic demand on the structure, especially in the vicinity of these connections. However, the east side span members still need some strengthening, especially near the east cable bent. The formation of plastic hinge at the tower base has considerably reduced the seismic forces in the stiffening truss diagonals, and marginally reduced them in the truss chord members. Allowing these plastic hinges to form has also limited the maximum longitudinal bending moment at the tower base to its plastic moment capacity.

CONCLUSIONS
This paper has demonstrated the salient dynamic characteristics of suspension bridges. The flexibility and three-dimensionality of these special structures render them susceptible to a complex class of vibrational problems in which cable vibrations and coupled motions of various components of the bridge cannot be captured in a simple linear 2-D analysis. It also discussed different proposed schemes for the seismic retrofit of these bridges using the Vincent-Thomas suspension bridge in California as a case study. The vulnerabilities associated with longitudinal and vertical vibrations of the bridge deck can be mitigated by installing longitudinal dampers at the stiffening truss expansion joints at the towers and cable bent locations, and allowing plastic hinge formation at the tower base. Also, installation of dampers in the side spans proved to be a very efficient scheme in reducing the seismic demand on the truss members in these spans.
Figure 5. D/C curves for the axial force in the top chord of the stiffening truss.
Figure 6. D/C curves for the axial force in the stiffening truss diagonals.
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