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Seismic Retrofit of the Suspension Spans of the Golden Gate Bridge

Consolidation parasismique des travées suspendues du pont de Golden Gate

Erdbebenverstärkung der Hängespannen der Golden-Gate-Brücke

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SUMMARY

Since 1937 the Golden Gate Bridge has served as a vital transportation link connecting San Francisco with the counties to its north. Prompted by the Loma Prieta Earthquake of 1989, the Golden Gate Bridge District initiated a series of studies of the bridge, culminating in the retrofit design described in this paper. The retrofit of the suspension bridge includes the installation of dampers between the stiffening trusses and the towers of the bridge, replacement of one-quarter of the stiffening truss lateral braces with new ductile members, and stiffening of the bridge towers to prevent undesirable plate buckling.

RÉSUMÉ

Depuis 1937, le pont de Golden Gate sert de liaison vitale entre San Francisco et les régions du nord. La décision de consolidation du pont a été prise à la suite du tremblement de terre Loma Prieta en 1989. Depuis cette date, de nombreuses études ont été menées. L'examen des différentes solutions envisagées a conduit au choix de l'étude présentée ici. La rénovation du pont suspendu inclut l'installation d'amortisseurs entre les treillis métalliques raidissants et les pylônes, le remplacement d'un quart des membres des raidisseurs latéraux avec des éléments ductiles, et finalement l'augmentation de la rigidité des pylônes pour éviter le flambement des tôles.

ZUSAMMENFASSUNG

Seit 1937 funktioniert die Golden-Gate-Brücke als eine lebenswichtige Verkehrsverbindung zwischen San Francisco und Besiedlungen im Norden. Erschüttert vom Loma Prieta Erdbeben 1989 hat der Golden Gate Distrikt mehrere Brückenanalysen eingeleitet, welche in diesem Artikel zusammengefasst sind. Die Erdbebenverstärkung der Hängebrücke umfasst die Installation von Dämpfern, die Ersetzung eines der seitlichen Versteifungen mit duktilen Elementen und schliesslich die Erhöhung der Steifheit der Pylone, um unerwünschte Plattenverformungen zu verhindern.

1. INTRODUCTION

Since 1937 the Golden Gate Bridge has served as a vital transportation link connecting San Francisco with the counties to its north. Prompted by the Loma Prieta earthquake of October 1989, the Golden Gate Bridge District engaged T.Y. Lin International and Imbsen & Associates to study the seismic vulnerabilities of the bridge and design a seismic retrofit.

Fig. 1 Bridge Elevation

The bridge is shown in elevation in Figure 1. The suspension bridge has a center span of 1,280 m and side spans 343 m long, for a total length of 1966 m. It is supported at the ends by reinforced concrete pylons, and flanked by steel viaduct and steel arch approach structures.

The suspended structure consists of parallel 7620 mm deep stiffening trusses, spaced 27.4 m apart in the planes of the cables. The trusses are connected by a top lateral bracing system that was a part of the original bridge, and by a bottom lateral bracing system constructed in the 1950s. The stiffening trusses are suspended from the cables at every other panel point. The suspended structure is connected to the towers and pylons through wind-locks that transfer lateral forces. The main span wind-locks allow longitudinal movement and rotation about transverse and vertical axes. The side spans are longitudinally restrained to the towers. The cables are supported on the bridge towers in cast steel saddles. The towers consist of slender, multi-cellular shafts braced together by portal struts above the roadway, and by double-diagonal struts below the roadway.

2. GROUND MOTIONS

The Golden Gate Bridge lies 10 km to the east of the San Andreas fault, which caused the M 8.3 San Francisco earthquake of 1906. Three “maximum credible” design earthquakes were developed to be representative of a major earthquake on this fault, based on recordings of the 1952 Kern County (M 7.2), 1985 Mexico City (M 8.1), and 1992 Landers (M 7.3) earthquakes. The design earthquakes have peak ground accelerations of about 0.65 g, peak velocities of about 110 cm/sec, peak displacements of about 55 cm, and durations of 60-90 seconds. Details of the design earthquakes are given in [1].

The analysis of the suspension bridge was for multiple-support excitation. The multiple-support motions include the wave-passage and extended source effects and the effect of ray-path incoherency. A study was made of the response of the bridge to multiple-support excitation, versus the response to rigid base excitation. The effects of multiple-support excitation were found to be small, and random over the three design earthquakes.

3. DESIGN CRITERIA

The technical criteria for the retrofit of the bridge are derived from performance criteria established by the Bridge District. These require the bridge to be opened to traffic within 24 hours after an earthquake, and repairable to fully operational status within one month.

Since the retrofit design is based on inelastic analysis of the bridge, the technical criteria limit the displacement ductility demands on bridge members. For instance, the ductility demand on *existing* bracing members is limited to two, in compression; and the number of cycles of inelastic deformation is limited to between one and three, depending of the quality of the member and the amount of empirical data available regarding its inelastic behavior. All of the existing members are of riveted construction, for which only very limited empirical data are available.

4. ANALYSIS METHODOLOGY

The bridge was evaluated by inelastic time history analysis of a three-dimensional finite element model, subjected to multiple-support excitation. Besides the "stress-stiffening" effect needed for the analysis of suspension bridges, the analysis included the following nonlinear effects:

- Nonlinear action of the dampers between the stiffening trusses and the towers and pylons
- Impact between the stiffening trusses and the towers
- Uplift of the bases of the towers
- Buckling of the lateral braces

With the exception of impact between the stiffening trusses and the towers, which will be eliminated by the retrofit of the bridge, each of these aspects of the bridge response is discussed in a subsequent section of the paper.

5. RETROFIT WITH VISCOUS DAMPERS

Installation of viscous dampers between the stiffening trusses and the towers is one part of the bridge retrofit. Viscous dampers were chosen for the retrofit because they won't restrain the thermal expansion of the bridge, and because they can be built with the large capacity needed. Dampers with a total relationship at each cross-section, of $F = (1{,}670 \cdot \mathrm{kN} \cdot \mathrm{sec}^{1/2} / \mathrm{cm}^{1/2}) \cdot V^{1/2}$ were chosen. At a calculated peak velocity of 190 cm/sec, the dampers will produce a peak force of 23,000 kN between the stiffening trusses and the towers, at each location.

The beneficial effect of the dampers is illustrated in Table 1, which shows the results of analyses made with and without the dampers, and with and without impact considered inside the wind-locks connecting the stiffening trusses and the towers. The dampers dramatically reduce the displacement demands on the bridge wind-locks and expansion joints, and eliminate actual impact between the stiffening trusses and the towers. They also reduce the peak stresses in the stiffening truss chords and the towers, and reduce the tower base shear forces and uplift (see below).

Parameter\Analysis	Dampers, No Impact (Retrofit)	Dampers, Impact	No Dampers, No Impact	No Dampers, Impact (Existing)	Capacity
Damper Force, kN	21,500	23,500	0	0	23,100
Wind-Lock Displacement, mm	570	530	1460	1230	460
Wind-Lock Impact Force, kN	0	11,100	0	92,000	13,100
Chord Demand/Capacity Ratio	0.84	0.87	1.05	5.22	
Tower Stress, MPa	390	360	530	470	
Tower Base Long. Shear, kN	55,700	54,000	77,400	60,700	
Tower Uplift, mm	46	56	140	81	

Table 1 Effectiveness of Dampers

6. RETROFIT OF THE LATERAL BRACING

Replacement of one-quarter of the lateral braces in the suspended structure is another part of the bridge retrofit. The existing braces are overstressed by about 50% in both tension and compression. Because of the contribution of higher modes of vibration to the response of the bridge, the overstress occurs over a large proportion of the length of the bridge, and for a large percentage of members. The overstress occurs in both the top and bottom lateral bracing systems.

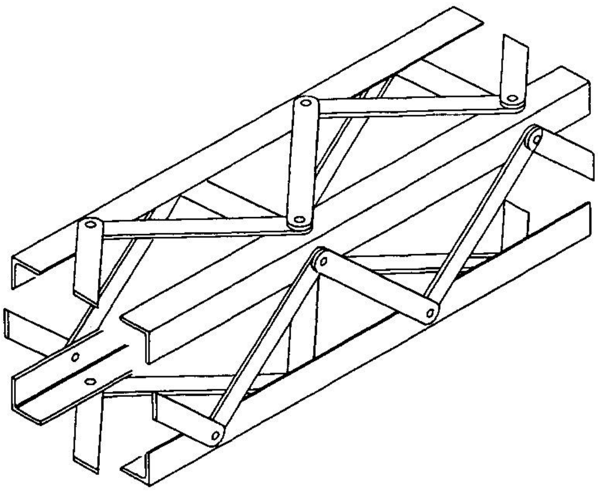

Fig. 2 Typical Lateral Brace

Unfortunately, the existing braces are of non-ductile construction; they only consist of four angles laced together into a box, as shown in Figure 2. A finite element analysis of a typical lateral brace was made in order to determine its inelastic behavior. The model was subjected to progressively increasing axial displacements in compression. As shown in Figure 3, the corner angles of the brace buckled locally, at an overall ductility demand of 1.15. This represents the limit of usefulness of the member; rapid strength and stiffness degradation occur after local buckling.

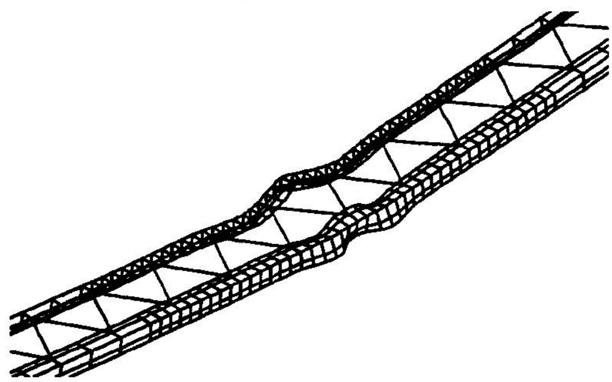

Fig. 3 Local Buckling of Lateral Brace

An inelastic time history analysis of the bridge was made, using the results of the finite element study as a guide in modeling the inelastic behavior of the lateral braces. The analysis showed that the deformation demands on the lateral braces were concentrated into those members which yielded first. The ductility demands on those members were considerably larger than the force demand/capacity ratios calculated from the elastic analysis of the bridge. The peak ductility demands from the inelastic analysis were about five, in excess of the design criteria limit of two.

The retrofit to eliminate the overstress of the lateral braces is shown in Figure 4, for that portion of the main span near the tower. The retrofit consists of replacing one-half of the top lateral braces with new members. These will be *ductile*, compact members of tubular cross-section. The installation of dampers into the top and bottom lateral bracing systems was considered as a retrofit measure also, but this solution was considered to be both more expensive and less reliable than the alternative chosen [2].

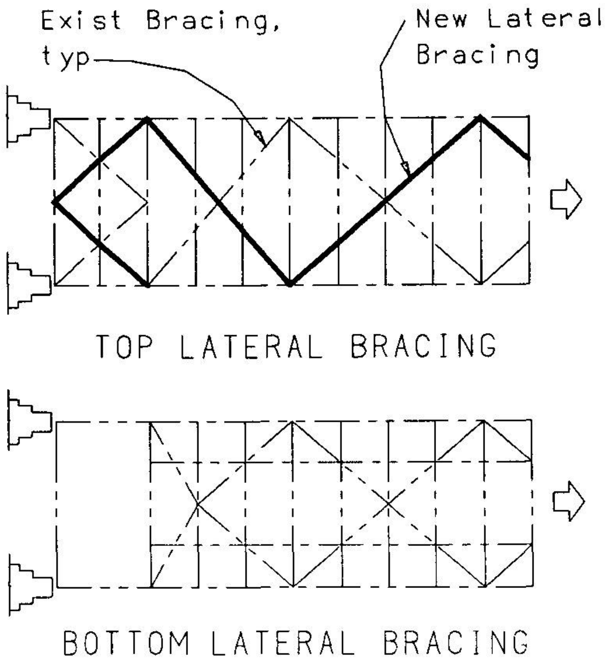

Fig. 4 Lateral Bracing Retrofit

The decision to replace one-half of the top lateral braces was a difficult one, since the bridge would be able to carry traffic even if the lateral braces were damaged. But, the lateral bracing systems are the primary means of resistance of the bridge to both aftershocks and wind, and these loads must be provided for. In the final analysis, the designers felt that the bridge was deserving of a ductile lateral bracing system, made from members of higher quality than the existing members. After retrofit, the bridge will satisfy some of the basic principles of aseismic design, as put forward by Dowrick [3]: that a structure have a "uniform and continuous distribution of strength and stiffness," (even after inelastic deformation) and that "brittle" modes of failure be avoided. Eliminating damage to the lateral braces also avoids collateral damage to the bridge floorbeams and other secondary members, which would occur in the areas of concentrated deformation of the lateral bracing systems.

7. RETROFIT OF THE TOWERS

Stiffening of critical locations of the towers to prevent plate buckling is another part of the bridge retrofit. As shown in Figure 5, the bases of the towers will uplift during an earthquake; the magnitude of the uplift is about 45 mm at the extreme fibers of the base. As shown in Figure 5, the uplift causes concentrations of stress (and strain) on the opposite side of the tower, both at the base and above the set-back in the tower elevation. In a finite element study of the base of the tower, the peak strains were found to be about four times the yield strain (assuming elastic-plastic behavior).

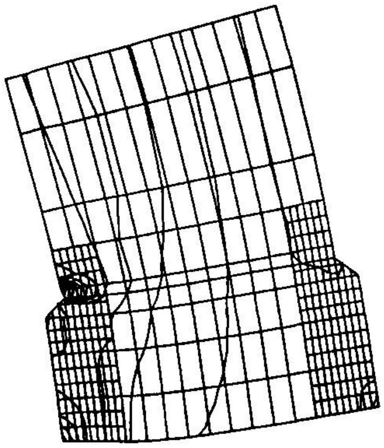

Fig. 5 Uplift of Tower Base

Strains of this magnitude can be accommodated by compact sections, but, unfortunately, the tower base is not compact. The tower is of multi-cellular construction; it consists of plates riveted together with corner angles. At the base, the cross-section consists of 103 cells, each 1070×1070 mm square (just large enough to work inside). The plates are 22 mm thick, giving a width-to-thickness ratio of 48. Plates of this dimension buckle shortly after yielding, with a significant loss of strength. A finite element analysis of a typical cell showed the corner angles to be only minimally effective in restraining the buckling of the plates, because of the large spacing (180 mm) of the rivets connecting the two elements.

Buckling of the plates at the location suggested in Figure 5 is undesirable because, in a sense, the tower vertical load is being carried in *compression* on the extreme fibers of the cross-section. The finite element study of the tower base suggested that the buckling would propagate towards the center of the cross-section. This will be prevented by the retrofit shown in Figure 6, where a stiffener is added along the vertical centerline of the plate (between diaphragms). The stiffeners will delay buckling of the tower plates until after a displacement ductility of four is reached. The propagation of the buckling will then be prevented, so that the base of the tower remains stable.

Fixing the bases of the towers was found to be undesirable because it caused higher stresses than did uplift of the towers, and because it would be very difficult to achieve in practice.

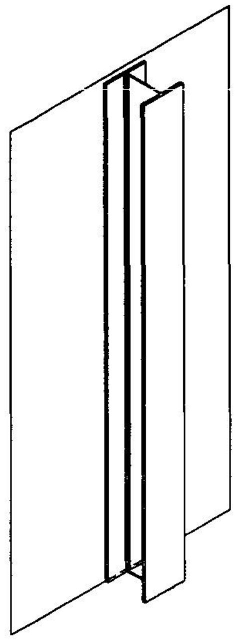

Fig. 6 Plate Stiffener

8. SUMMARY

The seismic retrofit of the bridge is intended to eliminate fundamental weaknesses resulting from the original design of the bridge to an equivalent lateral force of only 5% of gravity. The retrofit measures described herein include installation of dampers between the stiffening trusses and the towers of the bridge, replacement of one-quarter of the stiffening truss lateral braces with new ductile members, and stiffening of the bridge towers to prevent undesirable plate buckling. Other retrofit measures include strengthening of the cable saddles where they are connected to the tops of the towers and strengthening of the reinforced concrete piers that support the towers.

The authors wish to acknowledge the many helpful suggestions of Charles Seim and Fabio Taucer, and Drs. Roy Imbsen, David Liu, and Jerry Kao.

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