11 WCEE

Copyright © 1996 Elsevier Science Ltd
***Paper No. 1396. (quote when citing this article)***
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

# SEISMIC RETROFIT OF THE GOLDEN GATE BRIDGE

TIM J. INGHAM, SANTIAGO RODRIGUEZ, MARWAN NADER, AND FABIO TAUCER

T.Y. Lin International
825 Battery Street
San Francisco CA 94111

## ABSTRACT

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 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. It also includes strengthening of the bridge piers, strengthening of the saddles that support the cables on the tops of the towers, strengthening of the wind-locks connecting the suspended structure and the towers, and strengthening of the pedestals supporting the orthotropic deck of the bridge.

## KEYWORDS

Retrofit; multiple-support excitation; viscous dampers; local buckling; rocking; stiffeners; orthotropic deck.

## 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.

The seismic retrofit of the suspension bridge will be its fourth major retrofit since its completion in 1937. The previous retrofits were the addition of a bottom lateral bracing system in the 1950s to improve the flutter stability of the bridge, the replacement of the bridge suspenders in the 1970s, and the replacement of the original reinforced concrete deck with a steel orthotropic deck in the 1980s. These retrofits are a testimony to the diligence of the Bridge District in maintaining the bridge.

The bridge is shown in elevation in Fig. 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. It carries six lanes of traffic.

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

Fig. 1. Bridge elevation

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 orthotropic deck is supported on floorbeams spanning between the trusses at every 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 towers in cast steel saddles. At the ends of the bridge, the cables pass through the pylons, where the elevation of the cables is fixed by steel rope tie-downs.

The bridge towers consist of slender, multi-cellular shafts braced together by portal struts above the roadway, and by double-diagonal struts below the roadway. They are supported on reinforced concrete piers that extend down to bedrock.

## 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 (Geospectra, 1992).

The analysis of the bridge was for multiple-support excitation. The ground motions were developed considering possible fault rupture scenarios, and the propagation of the ground motions from the fault to the bridge site. The motions include the wave-passage and extended source effects, and the effect of ray-path incoherency. The peak relative displacement between the bridge towers is about 30 cm.

A study was made of the response of the bridge to multiple-support excitation, versus the response to rigid base excitation. The only systematic trend observed in the study was that vertical displacements of the stiffening trusses were larger for multiple-support excitation, probably because differential movement between the bridge supports straightens the cables and lifts the spans. In other respects, the differences in the response to multiple-support and rigid base excitation were small, and somewhat random over the three design earthquakes.

## 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.

## 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, each of these aspects of the bridge response is discussed in a subsequent section of the paper.

## 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 (Ingham *et al.*, 1994).

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).

Table 1. Effectiveness of dampers

| | Dampers, No Impact | Dampers, Impact | No Dampers, No Impact | No Dampers, Impact | Capacity |
|---|---|---|---|---|---|
| Damper Force, kN | 21,500 | 23,500 | 0 | 0 | |
| 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 | |

## 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 over-stressed by about 50% in both tension and compression. Because of the contribution of higher modes of vibration to the response of the bridge, the over-stress occurs over a large proportion of the length of the bridge, and for a large percentage of members. The over-stress occurs in both the top and bottom lateral bracing systems.

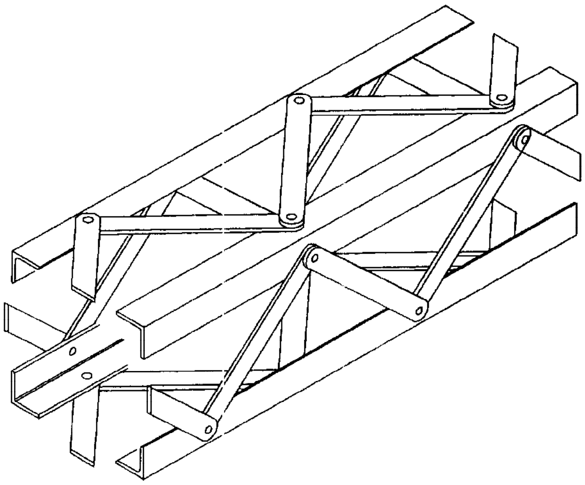

Fig. 2. Typical lateral brace

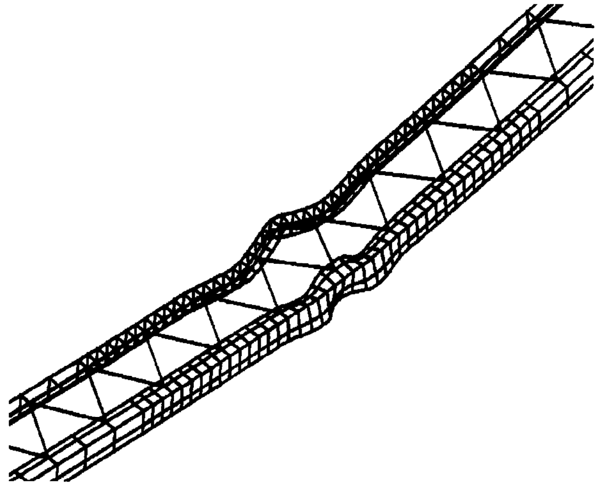

Fig. 3. Local buckling of lateral brace

Unfortunately, the existing braces are of non-ductile construction; they only consist of four angles laced together into a box, as shown in Fig. 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 Fig. 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.

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 (Rodriguez and Ingham, 1995).

The retrofit to eliminate the over-stress of the lateral braces is shown in Fig. 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.

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 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, 1987): 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.

## 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 Fig. 5, the bases of the towers will rock during an earthquake; the magnitude of the uplift is about 45 mm at the extreme fibers of the base. As shown in Fig. 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

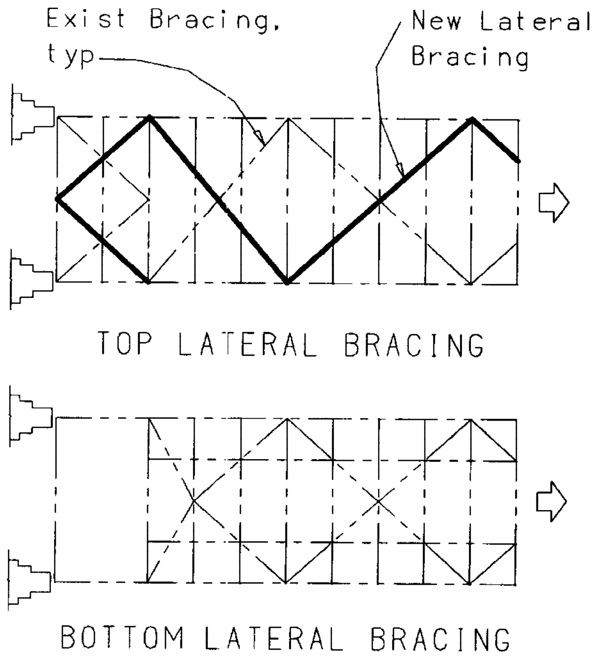


Fig. 4. Lateral bracing retrofit

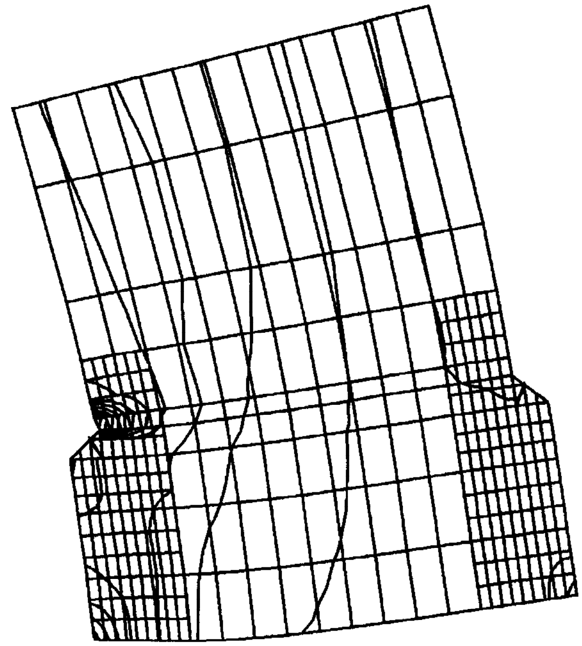

Fig. 5. Uplift of tower base

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).

Strains of this magnitude can be but, unfortunately, the tower base is cellular construction; it consists of angles. At the base, the cross-section mm square (just large enough to work giving a width-to-thickness ratio of 48. after yielding, with a significant loss of typical cell showed the corner angles restraining the buckling of the plates, of the rivets connecting the two ele-

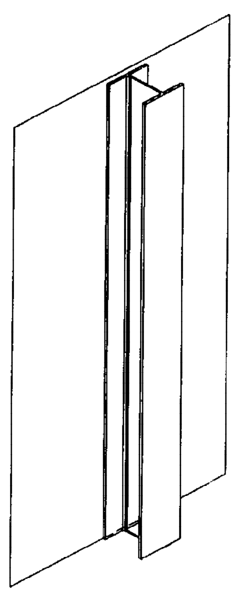

Fig. 6. Plate stiffener

accommodated by compact sections, not compact. The tower is of multi-plates riveted together with corner consists of 103 cells, each 1070×1070 inside). The plates are 22 mm thick, Plates of this dimension buckle shortly strength. A finite element analysis of a to be only minimally effective in because of the large spacing (180 mm) ments (Nader and Ingham, 1995).

Buckling of the plates at the location because, in a sense, the tower vertical the extreme fibers of the cross-section. base suggested that the buckling would cross-section. This will be prevented stiffener is added along the vertical diaphragms). The stiffeners will delay displacement ductility of four is buckling will then be prevented, so

suggested in Fig. 5 is undesirable load is being carried in *compression* on The finite element study of the tower propagate towards the center of the by the retrofit shown in Fig. 6, where a centerline of the plate (between buckling of the tower plates until after a reached. The propagation of the 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.

## RETROFIT OF THE PIERS

The towers of the bridge are supported on reinforced concrete piers extending down to bedrock, as shown schematically in Fig. 7. Although the piers are massive and very stable, they will be severely loaded when the towers rock onto one edge during an earthquake.

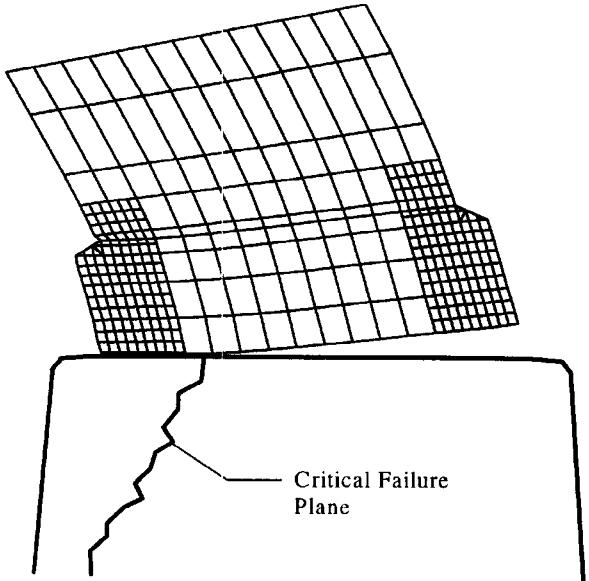


Fig. 7. Loading of pier

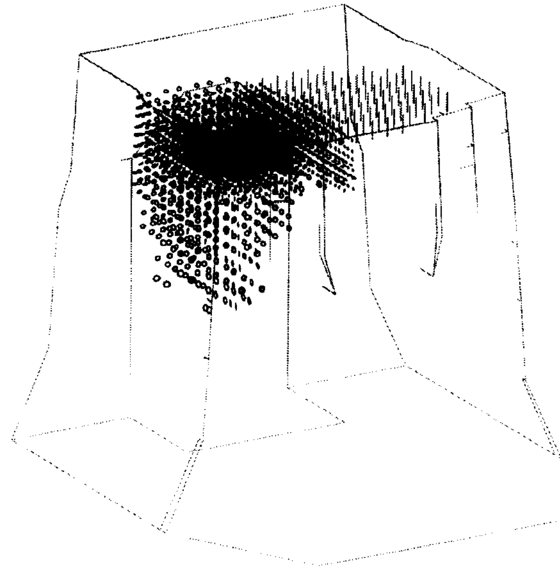

Fig. 8. Cracking of pier

When the towers rock onto one edge, a vertical force of 410,000 kN per tower shaft is carried on a footprint only a couple of tower cells wide. The average stress over this footprint is 36 MPa. This stress could be supported quite easily if the loaded area were well confined by surrounding concrete, but, unfortunately, the towers sit close to the edges of the piers. The loaded area is only 1.9 m from the edge of the pier. The edges of the piers might spall during an earthquake.

The response of the piers to edge loading was determined by a nonlinear finite element analysis (Anatech, 1994). Loads corresponding to the maximum rocking of the tower base were applied to the model; including the vertical force of 410,000 kN per tower shaft, and a longitudinal shear force of 36,000 kN acting outwards from the pier.

The analysis included a material model that considered the tension strength of the concrete, cracking and crushing of the concrete, and other nonlinear phenomena. The analysis also included the reinforcing steel in the pier; with yielding of the reinforcing and tension stiffening of the reinforcing by the concrete.

Figure 8 shows the calculated crack pattern in the corner of the pier. The planes of the circles define the surfaces of the cracks. The cracks formed on planes inclined from about 20° to 40° from the vertical, along both the longitudinal and transverse edges of the pier. The maximum strain in the pier (smeared over the cracks) was 0.006000 (compare with the cracking strain of the concrete–about 0.000125). The cracks are restrained from opening by the reinforcing in the pier. The crack width can be estimated if the crack spacing is taken to be equal to the spacing of the reinforcing at the pier surface. This gives a crack spacing of $0.006 \times 450 = 2.7$ mm. The maximum stress in the reinforcing was 320 MPa, which exceeds the assumed yield strength of 275 MPa. This indicates that some of the reinforcing was strained into the hardening range.

The retrofit of the pier is to install post-tensioned, high-strength threaded bars; from the surface of the pier underneath the tower base, as shown in Fig. 9. These bars will prevent a shear failure along the critical failure plane in the pier, beneath the loaded edge of the tower. The efficacy of the retrofit was demonstrated by a nonlinear analysis similar to that described above. The retrofit reduced the cracking of the pier considerably, and reduced the crack width to 1.0 mm.

## RETROFIT OF THE CABLE SADDLES

The cables are supported on the tops of the towers in cast steel saddles, as shown in Fig. 10. The saddles are supported on beds of 31 200 mm diameter steel rollers. The rollers bear on six inch thick steel plates bolted to the tower cells. The rollers allowed adjustment of the cables during erection of the suspended structure.

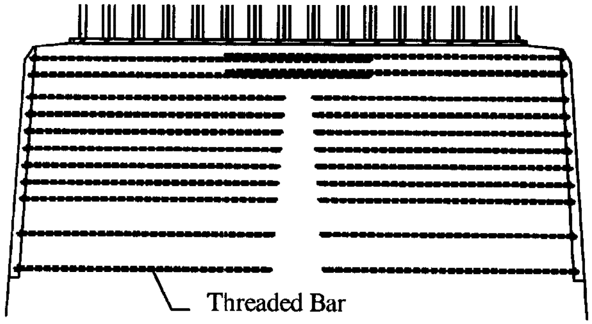


Fig. 9. Retrofit of pier

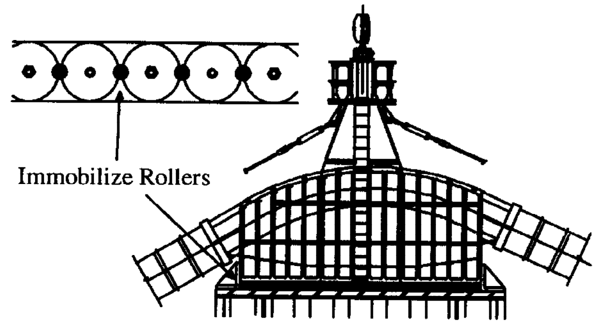


Fig. 10. Cable saddle

Subsequent to the erection, the rollers were grouted into a solid mass. Two cast steel "key blocks" fix each saddle to the tower, but these are not strong enough to resist the longitudinal shear force from the cables. Also, the grout between the rollers is discounted.

The maximum longitudinal shear force acting between the cables and the towers is 25,000 kN per saddle. The minimum vertical reaction of the cables on the towers is 175,000 kN per saddle, which does not necessarily occur at the same time as the maximum shear force. Combining these forces anyway, a "coefficient-of-friction" between the saddles and the towers of $\mu \geq 25{,}000/175{,}000 = 0.14$ is needed to transfer the shear force. If the coefficient of friction between steel surfaces, with clean mill scale, is taken to be 0.33, it is only necessary to immobilize the rollers in order to transfer the shear force in friction.

Figure 10 shows how the rollers will be immobilized by locking them with steel dowels. The dowels will be inserted into holes drilled into the rollers. With this retrofit, the shear force between the cables and the towers will be resisted by friction between the saddles and the rollers and between the rollers and the steel plates.

## RETROFIT OF THE WIND-LOCKS

Another retrofit is to the wind-locks that connect the side spans to the towers of the bridge. As shown in Fig. 11 (which shows a retrofitted wind-lock), each wind-lock consists of a rectangular pin block, connected to the top lateral bracing, moving in a longitudinal slot in the top and bottom plates of the wind-lock. The top and bottom plates are framed back to the tower shafts by the diagonal members shown in Fig. 11. The deficiencies of the wind-locks are two in number. They are not strong enough to carry the transverse shear forces from the lateral bracing, and the slots are not long enough to accommodate the calculated displacements of the suspended structure–even with the damper retrofit described above.

The retrofit measures include enlarging the longitudinal slot in the top and bottom plates of each wind-lock to accommodate the calculated displacement demand. Enlarging the slot requires replacing the casting that lines the sides of the slot. This will be replaced by two, stronger, weldments. The strengthening of each wind-lock also includes the replacement of rivets with high-strength bolts throughout the body of the wind-lock.

## RETROFIT OF THE ORTHOTROPIC DECK

Yet another retrofit is to the orthotropic deck of the bridge, which can be seen, in part, in Fig. 12. The orthotropic deck consists of 15 m long ribbed deck panels, supported by pedestals on top of the bridge floorbeams. A finite element analysis of the combined orthotropic deck / stiffening truss / lateral bracing system was made in order to investigate the behavior of the orthotropic deck. The model was subjected to the motion of the suspended structure calculated from the global analysis of the bridge.

The conclusion of the analysis was that the shear forces from the deck panels will concentrate into those supporting pedestals near "hard points," where the top lateral braces are connected to the floorbeams. As

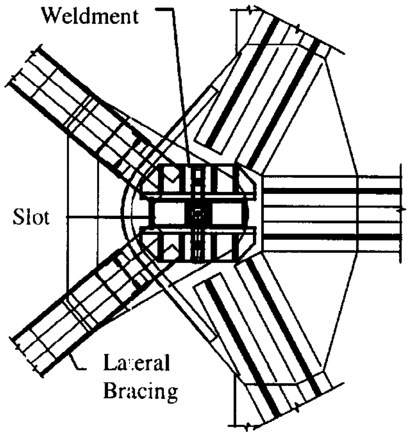


Fig. 11. Side span wind-lock

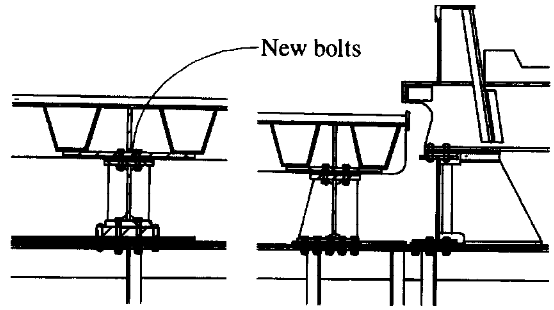


Fig. 12. Retrofit of orthotropic deck

shown, in Fig. 12, the necessary retrofit is to strengthen the connections of these critical pedestals to the deck plates and floorbeams by the addition of new bolts into voids in the existing bolt patterns.

## 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 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. They also include strengthening of the bridge piers, strengthening of the saddles that support the cables on the tops of the towers, strengthening of the wind-locks connecting the suspended structure and the towers, and strengthening of the pedestals supporting the orthotropic deck of the bridge. Complete details of the retrofit may be found in (Ingham *et al.*, 1994).

## ACKNOWLEDGEMENTS

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

## REFERENCES

Anatech Applications Corp. (1994). *Nonlinear Finite Element Analysis of the As-Built South Tower Pier of the Golden Gate Bridge.*

Dowrick, D.J. (1987), *Earthquake Resistant Design.* John Wiley & Sons.

Geospectra Incorporated (1992). *Geological, Geotechnical and Ground Motion Studies for Seismic Retrofit of the Golden Gate Bridge.* Golden Gate Bridge, Highway and Transportation District, San Francisco, CA.

Ingham, T.J., S. Rodriguez, M. Nader, F. Taucer, and C. Seim (1994). *Golden Gate Bridge Seismic Retrofit Design, Suspension Bridge Strategy Report.* Golden Gate Bridge, Highway and Transportation District, San Francisco, CA.

Nader, M. and T.J. Ingham (1995). Seismic Retrofit of the Towers of the Golden Gate Bridge. In: *Proceedings of the National Seismic Conference on Bridges and Highways.* San Diego, CA.

Rodriguez, S. and T.J. Ingham (1995). Seismic Protective Systems for the Stiffening Trusses of the Golden Gate Bridge. In: *Proceedings of the National Seismic Conference on Bridges and Highways.* San Diego, CA.