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SEISMIC PROTECTIVE SYSTEMS FOR THE STIFFENING TRUSS OF THE GOLDEN GATE BRIDGE

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ABSTRACT

This paper describes the final seismic retrofit recommended for the stiffening trusses of the Golden Gate Bridge in San Francisco, California. The seismic analysis method and the modeling techniques are summarized. A discussion of the seismic response of the suspension bridge is presented with the objective of explaining the function of the seismic protective systems to be implemented. The inelastic seismic analysis of the existing structure reveals that the most vulnerable elements in the stiffening truss are the lateral braces, the expansion joints and the wind-locks. The retrofit of the bridge includes longitudinal hydraulic dampers to limit relative displacements, transverse dampers to isolate the south side span from the south pylon, and replacement of one half of the top lateral braces. A parametric study was carried out to select the damper type and characteristics as a compromise between peak damper forces and expansion joint displacements.

KEYWORDS

Seismic retrofit; suspension bridges; nonlinear analysis; brace buckling; hydraulic dampers; seismic isolation

Fig. 1. Golden Gate Bridge Elevation

INTRODUCTION

The Golden Gate Bridge, opened to traffic in 1937, provides the only direct highway link between San Francisco and Marin County. With a 1280 m main span and 343 m side spans the suspension bridge was the longest span in the world until 1964 and it is a symbol of the City of San Francisco. The major components of the suspension bridge, shown in Fig. 1, are the steel towers, main cables and suspenders, stiffening truss, concrete anchorage blocks and concrete pylons. The bridge is owned, operated and maintained by the Golden Gate Bridge, Highway and Transportation District.

Immediately after the 1989 Loma Prieta earthquake, the District engaged T.Y. Lin International to perform seismic evaluation and retrofit studies of the Golden Gate Bridge. These studies culminated with the final seismic retrofit design (Ingham *et al.*, 1992; Ingham *et al.*, 1995). The performance criteria for this retrofit require the bridge to be opened to traffic within 24 hours after a maximum credible earthquake similar to the M 8.3 San Francisco earthquake of 1906, and repairable to fully operational status within one month (T.Y. Lin International, 1992). The *Design Criteria* also prevent the designer from significantly altering the appearance of the bridge due to its historic relevance.

One important part of the retrofit of the suspension bridge is the retrofit of the stiffening trusses. This paper describes the design process that lead to a retrofit scheme consisting of the installation of dampers between the main span stiffening truss and the towers, and between the side span stiffening trusses and the pylons. One half of the top lateral braces will also be replaced with new ductile members.

The retrofit also includes stiffening of the bridge towers to prevent undesirable plate buckling (Nader and Ingham, 1995), 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 (Ingham *et al.*, 1995).

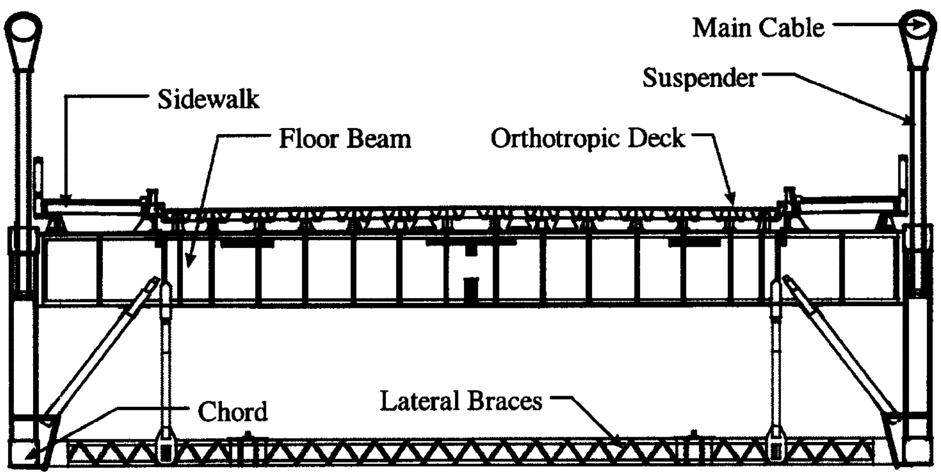


Fig. 2. Cross-section of the Stiffening Truss

DESCRIPTION OF THE STIFFENING TRUSS

The stiffening trusses of the Golden Gate Bridge consist of parallel 7.6 m deep vertical trusses, spaced 27.4 m apart in the planes of the cables (See Fig. 2). These 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 1950's. The stiffening trusses are suspended from the cables at 27.4 m intervals. An orthotropic deck, supported on floor beams spanning between the trusses at 7.6 m intervals, replaced the original concrete deck in the 1980's.

The vertical truss consists of chords, diagonals and verticals (Strauss, 1937). The top lateral bracing system is made up of silicon steel box-members consisting of four angles laced on four sides. The floor beams are 2.6 m deep and are connected to the vertical trusses by knee braces. All the stiffening truss members in the original bridge construction have riveted connections. The bottom lateral bracing system added in the 1950's consists of diagonal lateral braces, transverse struts and longitudinal track girders of A7 steel. All the members in the bottom lateral bracing system are steel box members consisting of four angles laced on four sides. Shop rivets and field bolts were used in the connections of the bottom lateral bracing system.

The stiffening truss is connected to the towers and pylons through rocker-links that transfer vertical loads and wind-locks that transfer lateral forces. The wind-locks allow longitudinal movement and rotation about transverse and vertical axes at both towers and pylons. The main span has expansion joints at both ends and it is longitudinally free to move at the tower wind-locks. The only longitudinal restraint is provided by the cable system. The wind-locks were designed with a displacement capacity of 46 cm towerward and 53 cm channelward. The side spans are longitudinally restrained to the towers by means of a steel pin and free at the pylons. The pylon wind-lock has a displacement capacity of ±41 cm.

SEISMIC ANALYSIS

The seismic response of the Golden Gate Bridge was evaluated by inelastic time history analysis of a three-dimensional finite element model, subjected to multiple-support excitation. Three artificially generated ground motions with duration between 60 and 90 seconds were used for the seismic analysis. The multiple support ground motions were based on earthquake records and were made compatible with a target spectrum with a peak ground acceleration of 0.65 g. The peak ground displacement is 55 cm and the peak relative displacement between supports 43 cm (Ingham *et al.*, 1992).

The analytical models included all the elements of the suspension bridge, such as main cables, suspenders, towers, piers and stiffening truss members. The reinforced concrete pylons were included in the model as substructures. The model was verified by comparing computed natural frequencies with those obtained from ambient vibration measurements (Abdel-Ghaffar and Scanlan, 1985). Large displacement effects were considered by establishing the dynamic equilibrium of the structure in its deformed configuration. Nonlinear elements were included for the modeling of rocking and uplift of the bases of the towers, nonlinear dampers between the stiffening trusses and the towers and pylons, impact between the stiffening trusses and the towers, failure of the pins connecting the side spans to the towers, and buckling of the lateral braces.

The nonlinear dynamic analyses were performed by integrating the coupled equations of motion in the time domain using ABAQUS, a general purpose nonlinear finite element program (Hibbitt Karlsson & Sorensen, Inc., 1993). The ground motion excitation was applied as a time-varying displacement boundary condition at each of the supports.

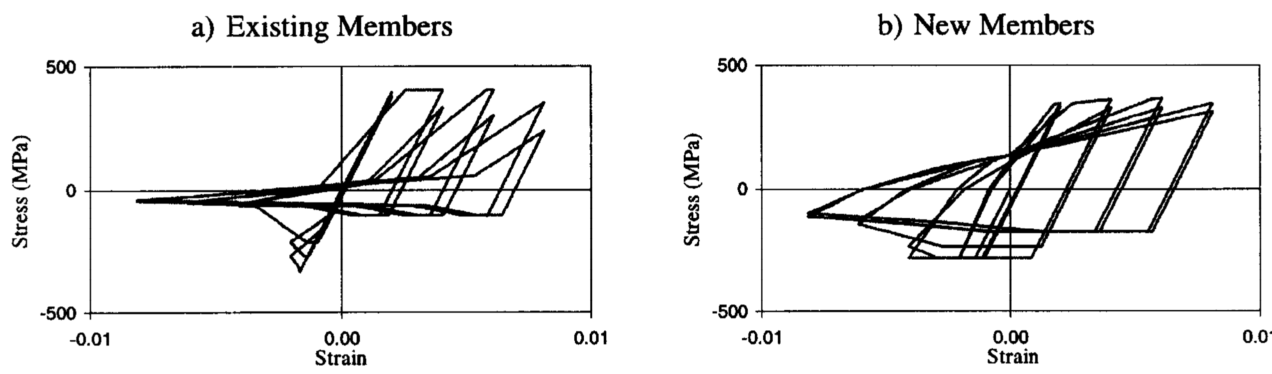


Fig. 3. Hysteretic Behavior for Lateral Braces.

STIFFENING TRUSS MODELING

The stiffening truss was modeled with different levels of detail ranging from super-element models of the truss to fully detailed models including all the members. Elastic beam elements were used for all the members presenting elastic behavior. Both top and bottom lateral braces, expected to have inelastic deformations, were modeled with inelastic truss elements, using the Maison model of brace behavior (Zayas *et al.*, 1981).

This model accounts for yielding and buckling with a hysteretic behavior that includes the strength and stiffness degradation expected of the lateral braces. The assumed hysteretic behavior, shown in Fig. 3 (a) corresponds to a slender brace, with *kl/r* = 120, made from angles (Black *et al.*, 1980). The new tubular members to be used to replace one half of the top lateral braces were also modeled with inelastic truss elements. The Maison model properties were those of a compact squat brace, with *kl/r* = 40 (Black *et al.*, 1980). The assumed hysteretic behavior is shown in Fig. 3 (b).

SEISMIC RESPONSE AND VULNERABILITIES OF THE SUSPENSION BRIDGE

The seismic response of suspension bridges is very complex due to the high participation of secondary vibration modes, coupling between longitudinal, transverse and vertical motion, large displacement effects and the multiple support excitation. Fig. 5 shows one instantaneous deformed shape of the bridge obtained during the dynamic analysis. The analysis of the *existing* structure reveals the seismic vulnerabilities and helps in understanding which modes of behavior contribute to the seismic demands. The retrofit scheme was developed in order to modify the seismic response with energy dissipation and structural fuses. Strengthening requirements were therefore minimized.

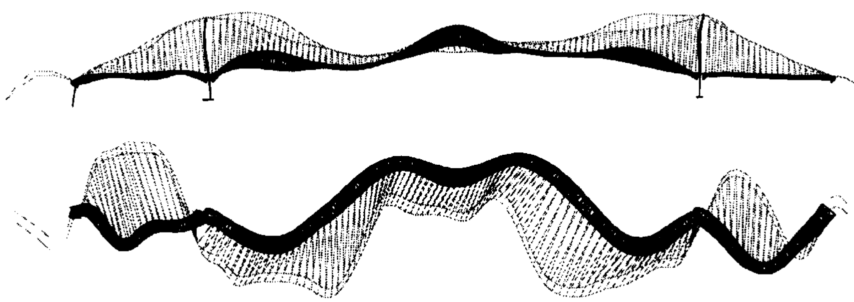


Fig. 4. Displaced shape at 18.5 seconds, at 200 times scale. Elevation and Plan.

Vertical Response

The Golden Gate Bridge was designed to carry vertical loads due to dead load and traffic with a high factor of safety. Because of this, the vertical response to seismic excitation by itself does not create demands exceeding the structural capacity. Table 1 shows mode 19, one of the important modes in the vertical response.

Longitudinal Response

Table 1 shows mode 5 in which the main span is displaced as a pendulum hanging from the suspenders and mode 47, first longitudinal mode of the tower coupled with the side span. These two modes have a remarkable influence in the longitudinal response. The relative displacement between the main span and the tower, calculated assuming an unlimited displacement capacity, is ±130 cm. This relative displacement exceeds the 46 cm capacity of the existing expansion joints and wind locks. Impact forces in excess of 90,000 KN that would damage the wind locks would be the result of this displacement demand.

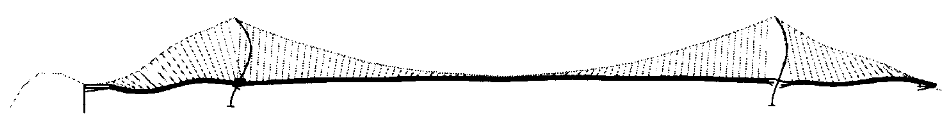
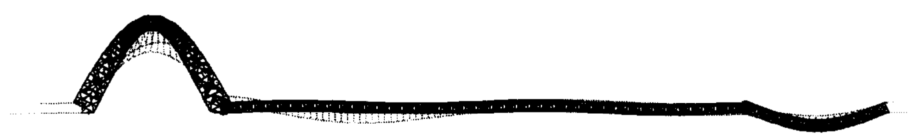
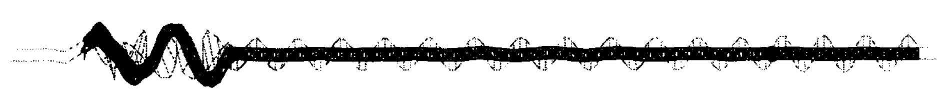
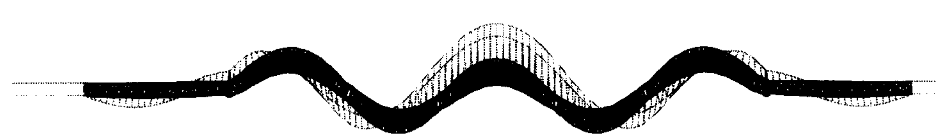
The longitudinal connection between the side span and the tower is also vulnerable. Retrofitting this connection to take the elastically calculated longitudinal force would require a strength of 45,000 KN. Strengthening this connection does not seem appropriate because this connection force greatly contributes to the tower stresses due to longitudinal deflection of the towers. A better approach is to actually weaken the connection by replacing the existing steel pins that link the side spans to the towers. The new pins will act as structural fuses with a capacity of 3,200 KN, enough to take longitudinal wind and breaking loads, but they would fail early during an earthquake. Longitudinal dampers between the side spans and the towers and between the side spans and the pylons will also be installed in order to control the longitudinal relative displacements.

Transverse Response

Modes 17 and 254 in Table 1 define the transverse response of the side spans. In addition to the first transverse mode of the side spans, mode 17, the south side span also has important contributions from a secondary mode, mode 254, in which the side span vibration is coupled with the south pylon. The south pylon has a mass three times larger than that of the side span and really dominates the coupled vibration. Mode 34 in Table 1 is predominant in the transverse and torsional response of the main span. The first transverse mode with a period of 20 seconds has a very small participation.

The top and bottom lateral braces of the stiffening truss are over-stressed due to transverse shear and torsion. Their ductility demand calculated assuming elastic material behavior is 2.2 and reaches 15 when considering inelastic yielding and buckling of the lateral braces. This ductility demand exceeds the *Design Criteria* limit of 4 in tension and 2 in compression (T.Y. Lin International, 1992). Both top and bottom lateral braces are expected to buckle at locations extended over the full length of the main and side spans.

Table 1. Predominant Mode Shapes of de Golden Gate Bridge

Mode	Type	Period (sec)	Displaced Shape
19	Vertical	3.5	
5	Long.	7.1	
47	Long.	1.8	
17	Trans.	3.5	
254	Trans.	0.5	
34	Trans.	2.4	

The retrofit recommended to alleviate the problems in the south side span consists of the installation of transverse dampers between the side span stiffening truss and the south pylon, in combination with a structural fuse with enough capacity to resist transverse wind loads. The side span would be effectively isolated from the vibration of the south pylon after breaking the structural fuse. In addition to the transverse isolation of the south side span, a retrofit consisting of the replacement of one half of the top lateral braces with new ductile members was recommended (Ingham *et al.*, 1995). This retrofit is to be applied to the two side spans and the main span. Other retrofit concepts explored to eliminate the over-stress of the lateral braces were the installation of dampers into the top and bottom lateral bracing systems, and transverse isolation of the main span (Rodriguez *et al.*, 1994).

LONGITUDINAL DAMPERS AT THE STIFFENING TRUSS EXPANSION JOINTS

Longitudinal dampers will be installed at the locations shown in Fig. 5. Two hydraulic dampers will be installed at each of the four truss chords between the main span and the towers, at each of the four chords between the side spans and the towers and at the two bottom chords between the side span and the pylons. The dampers will consist of a piston in a cylinder, with the damping force being generated by a viscous fluid moving through an orifice.

The main objective of this retrofit is to reduce the relative displacements at the expansion joints and wind-locks, thus eliminating the possibility of impact forces being developed. The installation of dampers at the ends of the side spans also makes possible to use *structural fuses* between side spans and towers, in order to uncouple the vibration of these two elements. Longitudinal dampers have other secondary benefits such as a reduction in the tower stresses due to longitudinal displacements, and a reduction in stiffening truss demands.

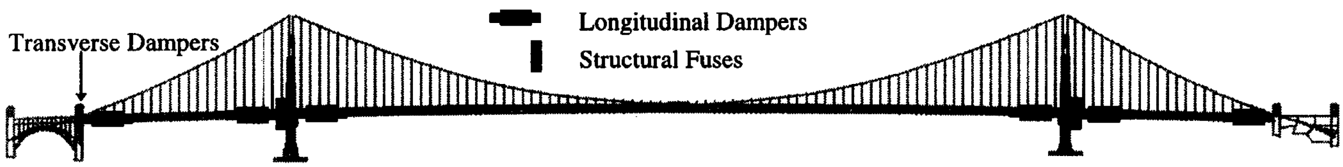


Fig. 5. Location of Longitudinal Dampers

The properties of the dampers were optimized to achieve an appropriate reduction of the relative displacements and stresses, keeping the damper force within reasonable limits. Hydraulic, viscous devices are the best option because they can accommodate slow temperature displacements without forces at the dampers. Viscous devices provide a restoring force (F) which is a function of the relative velocity (V). The constitutive law is:

$$F = C \cdot V^n \tag{1}$$

The exponent n defines the type of device, whereas the coefficient C controls the force that will be developed. Different values for the exponent n were evaluated. Exponents $n = 0, ½, 1$, and 2 were considered. For each of these exponents, several values of C were examined in a parametric study aimed at identifying the optimum dampers. This study was carried out by calculating the seismic response to one of the ground motions with three-dimensional elastic models of the bridge. The models included a super-element representation of the stiffening truss, structural fuses between the side spans and the towers, tower bases able to rock and uplift, and expansion joints with unlimited capacity. The same damper properties were assumed for main and side spans.

Figure 6 shows the total energy dissipated by the dampers as a function of the peak damper force per chord in the main span dampers. Each point in this graph provides the results from one dynamic analysis using a set of values for n and C. Four different *trend* lines were fitted to the available points. The line named *linear* corresponds to $n = 1$ dampers. The line named *constant* corresponds to $n = 0$ dampers, with a constant force. The line named *square root* corresponds to $n = ½$ dampers, and the line named *parabolic* corresponds to $n = 2$ dampers. The trend is quite clear; dampers with a small exponent n are preferable. For the same force capacity, they dissipate more energy than dampers with a large exponent. This advantage in energy dissipation is translated into a larger reduction in displacements and stresses, although this reduction is not necessarily proportional to the amount of energy being dissipated.

Figure 6 also shows the maximum relative displacement at the main span wind-locks as a function of the peak damper force in the main span dampers. Again, dampers with a square root or constant relationship between force and velocity are more *efficient* than those with parabolic force velocity relationship. They achieve the same reduction in relative displacements with a smaller force. Linear dampers are in-between. A damper force of about 6,700 KN per chord would be required to reduce the relative displacement to 53 cm when using *square root* dampers. A larger reduction in the relative displacement will require a much larger force as the slope of the trend lines decreases with an increasing force.

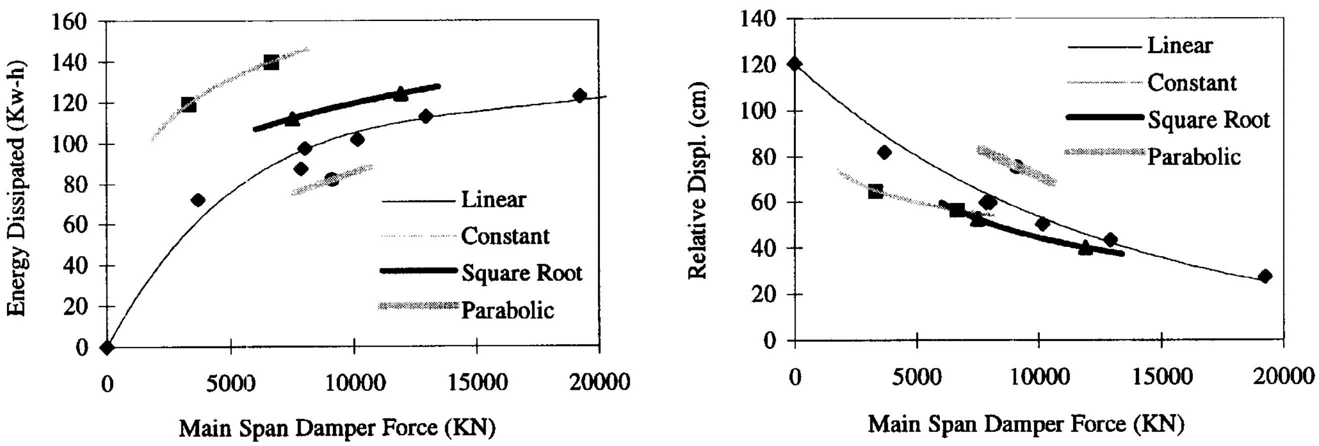


Fig. 6. Energy Dissipated and Relative Displacement at Main Span Dampers

Figure 7 shows the maximum tower base compressive stress as a function of the peak damper force in the main span dampers. The peak compressive stress is caused by rocking of the tower base due to longitudinal displacements. The calculated compressive stress reaches 690 MPa when the side spans are connected to the towers without structural fuses. Introducing the structural fuses in the analytical model reduces the compressive stress to 530 MPa, before considering dampers. The tower plates have a yield stress of 345 Mpa. The dampers reduce the stress to about 450 MPa when providing a force of about 6,700 KN per chord. Dampers providing a larger force may even be counterproductive as the mass of side and main spans becomes coupled with the towers.

Figure 7 shows the maximum side span chord stress as a function of the peak damper force. The side span chord stress is reduced by longitudinal dampers due to coupling between longitudinal and vertical displacements and to the rotational restraint being supplied by the presence of dampers in each of the truss chords. The dampers reduce the stress from 580 to 480 MPa when providing a force of about 6,700 KN per chord with either *constant* or *square root* dampers. Truss chords yield stress is 345 Mpa. Increasing C to provide a larger force is not very effective since the slope of the trend lines decreases with an increasing force.

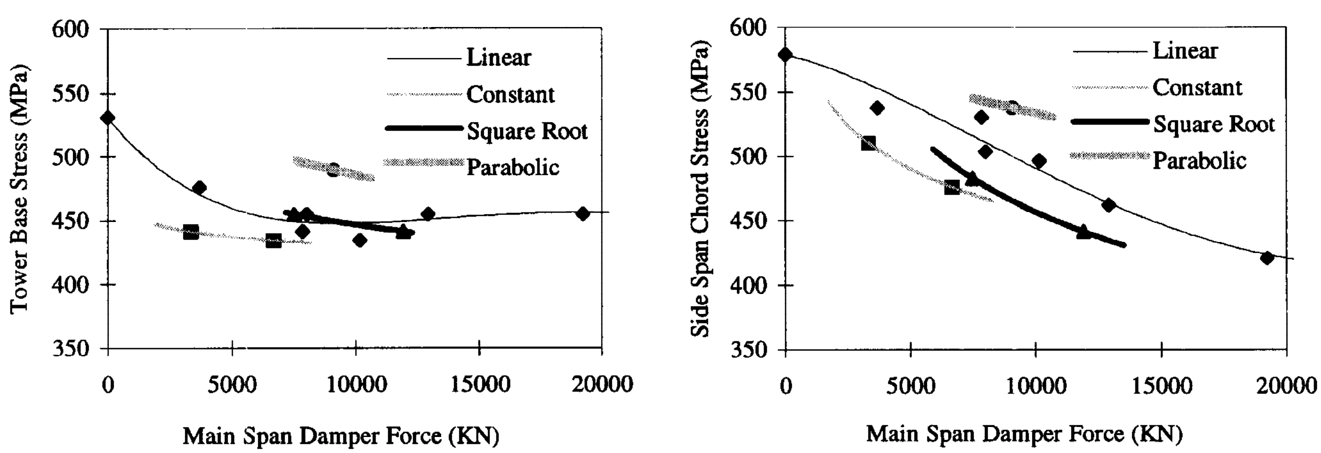


Fig. 7. Tower Base and Side Span Chord Peak Stress

Considering these results, a exponent $n = ½$ was selected because it is nearly equivalent to a exponent $n = 0$ and dampers with these characteristics can be supplied by several damper manufacturers. The optimum damper capacity is in the neighborhood of 6,700 kN per chord. A more detailed parametric study with fully nonlinear analytical models and three different ground motions was performed in order to evaluate the optimum C coefficient as a compromise between peak damper forces and expansion joint displacements. The result was choosing two different coefficients $C = 490$ kN·sec $^{1/2}$ /cm $^{1/2}$ and $C = 560$ kN·sec $^{1/2}$ /cm $^{1/2}$ per chord. The first coefficient is used for the main span and for the south pylon, the second one for the side span-tower location plus the north pylon. The calculated peak force is 6,540 kN per chord that will be divided into two physical dampers.

Besides seismic excitation, the dampers will be subjected to thermal movements, wind excitation and small amplitude, high frequency vibrations caused by traffic on the bridge. The dampers have the beneficial effect of controlling wind oscillations due to buffeting. The final damper design must be able to perform under all these environments. A test program was conducted at the University of California, Berkeley to pre-qualify damper manufactures for bidding on the project. The program included cyclic testing of 445 kN dampers with a peak velocity of 50 cm/sec and a stroke of ±15 cm.

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REFERENCES

Abdel-Ghaffar, A. and Scanlan, R. H. (1985). Ambient Vibration Studies of the Golden Gate Bridge: I Suspended Structure. *Journal of Engineering Mechanics*, ASCE, **111**, No. 4, 463-482.

Black, R.G., Wenger, W.A., and Popov, E. P. (1980). *Inelastic Buckling of Steel Struts Under Cyclic Load Reversals*. Report No. UCB/EERC-80/40. Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, California.

Hibbitt, Karlsson & Sorensen, Inc. (1993). *ABAQUS Version 5.3 User's and Theory Manuals*. Hibbitt, Karlsson & Sorensen, Inc., Pawtucket, Rhode Island.

Ingham, T., Rodriguez, S., Nader, M., Taucer, F. and Seim, C. (1992). *Golden Gate Bridge Seismic Retrofit Design, Suspension Bridge Strategy Report*. Golden Gate Bridge, Highway and Transportation District, San Francisco, California.

Ingham, T.J., Rodriguez, S., Nader, M., Taucer, F. and Seim, C. (1995). Seismic Retrofit of the Golden Gate Bridge. *Proceedings of the National Seismic Conference on Bridges and Highways*. San Diego, California.

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

Rodriguez, S., Seim, C., and Ingham, T.J. (1994). Earthquake Protective Systems for the Seismic Upgrade of the Golden Gate Bridge. *Proceedings of the Third U.S.-Japan Workshop on Earthquake Protective Systems for Bridges*. Report No. NCEER-94-0009. National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, New York.

Strauss, J. B. (1937). *The Golden Gate Bridge, Report of the Chief Engineer to the Board of Directors of the Golden Gate Bridge and Highway District*. Golden Gate Bridge and Highway District, California.

T.Y. Lin International (1992). *Golden Gate Bridge Design Criteria for Seismic Retrofit Measures*. Golden Gate Bridge, Highway and Transportation District, San Francisco, California.

Zayas, V. A., Shing, P.B., Mahin, S.A., and Popov, E. P. (1981). *Inelastic Structural Modeling of Braced Offshore Platforms for Seismic Loading*. Report No. UCB/EERC-81/04, Earthquake Engineering Research Center, University of California, Berkeley, California.