Session B1

Seismic Retrofit of the Golden Gate Bridge
Renforcement parasismque du pont de Golden Gate
Verstärkung bezüglich Erdbeben der Golden Gate Brücke

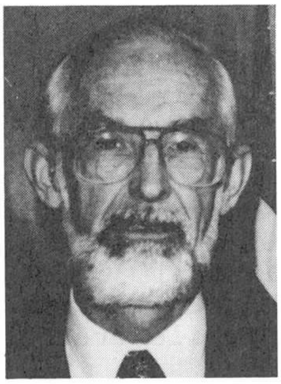
Leere Seite
Blank page
Page vide

Introduction to the Golden Gate Bridge Retrofit Project

Introduction au projet de consolidation du pont de Golden Gate

Einführung in das Ertüchtigungsprogramm für die Golden-Gate-Brücke

Mervin C. GIACOMINI
District Engineer
Golden Gate Bridge HTD
San Francisco, CA, USA



Mervin C. Giacomini is District Engineer for the Golden Gate Bridge, Highway and Transportation District since January 1, 1995. He has 21 years of experience with the maintenance and repair of the Golden Gate Bridge. He has been Project Manager for major projects related to the Golden Gate Bridge.

SUMMARY

The Golden Gate Bridge and Highway District was formed in 1928 to design, construct and finance the bridge, which was opened to traffic in 1937. Today, tolls fund the operation and maintenance of the bridge and also subsidise a bus and ferry transportation system from the northern counties to San Francisco. Over 100'000 vehicles cross the bridge daily. After the Loma Prieta earthquake in 1989, the ongoing comprehensive maintenance program was completed with state-of-the-art seismic evaluation of the bridge. The retrofit project is well advanced and many measures are ready to be undertaken.

RÉSUMÉ

L'autorité du "Golden Gate Bridge and Highway District" a été créée en 1928 afin de concevoir, réaliser et financer la construction du pont, lequel a été ouvert au trafic en 1937. Actuellement, le péage permet de financer l'exploitation et l'entretien du pont, et subsidie également un système de transport par autocar et bateau entre les comtés du nord et San Francisco. Plus de 100'000 véhicules traversent le pont chaque jour. A la suite du tremblement de terre de Loma Prieta en 1989, le programme, global et continu, de maintenance a été complété par une évaluation parasismique détaillée du pont. Le projet de consolidation est en bonne voie et de nombreuses mesures vont être entreprises bientôt.

ZUSAMMENFASSUNG

Der "Golden Gate Bridge and Highway District" wurde 1928 für den Entwurf, den Bau und die Finanzierung der Brücke gegründet, die 1937 dem Verkehr übergeben wurde. Heutzutage finanzieren Brückenzölle den Betrieb und Unterhalt der Brücke und subventionieren einen Bus- und Fährebetrieb davon den Nordbezirken in die Stadt. Ueber 100.000 Fahrzeuge benutzen die Brücke täglich. Nach dem Loma Prieta Erdbeben 1989 wurde das laufende umfangreiche Unterhaltsprogramm mit einer Erdbebenbeurteilung nach dem heutigen Stand der Technik abgeschlossen. Das Ertüchtigungsprojekt ist weit fortgeschritten, und etliche Massnahmen sind vorbereitet.

On behalf of the Golden Gate Bridge, Highway and Transportation District, I am pleased and honored to introduce this session of the 1995 IABSE Symposium, "The Seismic Retrofit of the Golden Gate Bridge." The Bridge District has completed some extraordinary work in the field of seismic retrofit engineering: groundbreaking work that has laid the foundation for similar retrofit projects worldwide. This symposium is the first opportunity we have had to share our findings in detail with the international community. But first, I want to briefly outline the history of the Bridge and the District that runs it, and give some background on the seismic retrofit project to date.

On December 4, 1928, the Golden Gate Bridge and Highway District was formed to design, construct and finance the Bridge. The District consists of San Francisco County and five counties to the north. Voters within the District put up their homes, their farms and their business properties as collateral for a $35 million bond issue to finance the Bridge. After four years of construction, it was completed and opened to automobile traffic on May 28, 1937.

For 58 years, the District has successfully fulfilled its primary mission of maintaining and operating the Golden Gate Bridge. In 1971, the last of the construction bonds were retired. The bonds were financed entirely by tolls. Today, Bridge tolls completely fund the operation and maintenance of the Bridge.

But tolls support more than the Bridge itself. They also subsidize a bus and ferry transportation system from the northern counties to San Francisco.

Due to increasing traffic congestion, the California State Legislature, in 1969, authorized the District to develop a mass transportation system in the Golden Gate Corridor consisting of buses and ferries. The legislative mandate was clear: Reduce traffic congestion. However, the District was not given the authority to levy taxes to support transit. All intercounty service had to be subsidized by Bridge tolls.

Traffic growth in the Golden Gate Corridor has been held to a manageable level. Before Golden Gate Transit, approximately 30,000 people in 20,000 vehicles crossed the Bridge during each morning commute. Today, 38,000 people commute to San Francisco each morning while vehicle traffic had grown to only 22,000. In total, over 40 million vehicles crossed the span last year.

And with so many people depending upon the structure, its failure during a seismic event would deal a devastating blow to the region. As you know, in 1989, the Loma Prieta Earthquake caused severe damage to many structures in the Bay Area. The Richter magnitude 7.1 quake caused no significant damage to the Golden Gate Bridge, however, thanks to earlier seismic upgrade work in 1982 and an ongoing, comprehensive maintenance program.

For the Bridge District, however, the Loma Prieta was a call to action. Right after the quake, the District contracted for a state-of-the-art seismic evaluation of the Bridge by T.Y. Lin International of San Francisco. The firm reported that, although the Bridge had performed well in previous quakes, it is vulnerable to damage in a Richter magnitude 7 or greater with a nearby epicenter and could be closed for an extensive period of time after such an earthquake.

The Loma Prieta occurred over sixty miles south of the Golden Gate Bridge. Ground movement during that event brought the Bridge near its current earthquake design limits. If an earthquake the size of Loma Prieta occurred on the San Andreas or Hayward faults, 7 miles west and 10 miles east of the Bridge respectively, the results could be catastrophic.

As the District conducted its initial seismic evaluation, the California Governor's Board of Inquiry on the Loma Prieta Earthquake issued a 1990 report directing all important transportation structures in the state to be seismically retrofit to assure their function following a major earthquake. Accordingly the District, in concert with leading seismologists and seismic engineers, recommended a retrofit on the Golden Gate Bridge.

Since then, the Bridge District has completed geotechnical studies, preliminary environmental assessments and preliminary design concepts. The District spent over $2.3 million to ensure that all necessary, preliminary work was complete to provide the foundation for the final design.

In January 1993, the final design phase for the seismic retrofit began. Final design was placed on a fast track and conducted by two engineering firms simultaneously, Sverdrup Corporation, from Walnut Creek, California, and T.Y. Lin International/Imbsen & Associates, a Joint Venture from San Francisco. The final design phase is nearing completion and construction could begin as early as September 1995.

Just as Joseph Strauss found a challenge in funding the original Bridge construction, so it is with seismic strengthening. In 1991, understanding the regional and economic significance of a retrofit, the District developed a funding strategy to generate local match revenues for the project. The strategy called for a 50 percent increase in bridge tolls and adjustments to bus and ferry fares. These increases have already been implemented.

To date, bridge tolls have funded over $7.4 million of the seismic design effort. A federal grant financed an additional $5.9 million under the Intermodal Surface Transportation Efficiency Act of 1991, or ISTEA.

Further, between 1991 and 1996, Bridge tolls will generate a 20 percent local match for the estimated $165 to $175 million construction cost. The earthquake retrofit expense is significant, but represents only one-tenth of the replacement cost of the bridge, estimated to be over $1.4 billion.

As of this writing, the District is actively seeking federal funding. The District has generated the 20 percent local match for this project.

The Golden Gate Bridge itself is made up of seven distinct structures. The retrofit project includes both tuning the structures to reduce the violent actions caused by ground motion, and strengthening the structures to reduce the damage caused by these actions.

For example, starting from the San Francisco end, the approach foundations will be strengthened and the towers of the approach will be replaced altogether. Also, each of the separate spans of the approach will be connected so they move together during an earthquake rather than moving separately and potentially collapsing. Seismic motion isolators and dampers, or shock absorbers, are also being applied to this structure in innovative new ways. Vulnerability studies show the north and south approaches are most vulnerable to failure in an earthquake.

Other notable retrofit measures include the reinforcement of the Fort Point Arch. Advanced, computer-assisted modeling has shown that during a seismic event, the arch could actually jump off its support bearing at the base of the structure. Rather than securing the arch at the base, we've learned to let the arch move during an earthquake but control the movement so that when it does lift up, it will be guided back down into its proper place. Also, each of these pylons, equal in height to a 20-story building, will be strengthened.

Modeling has also shown that during a quake, the main span can act as a battering ram, slamming against each tower. As a result, connections between the towers and the roadway will be reinforced and dampers, or shock absorbers, will be installed. Also the tower legs, which have shown the potential to buckle, will be reinforced internally.

These are just a few of the many measures we are ready to undertake, and some of what will be covered in the following symposium. Even though most work will occur where motorists and visitors will not see, the District has gone to great lengths to preserve the existing, historical appearance of the structure. The retrofit is designed not to change the appearance of the Golden Gate Bridge.

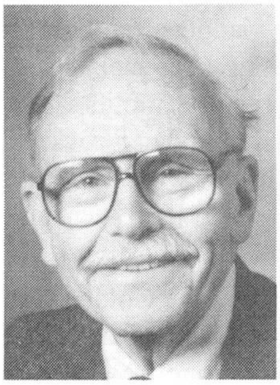
As a result of our efforts, the Golden Gate Bridge, Highway and Transportation District has become what many consider an international leader in the seismic retrofit design field, applying state-of-the-art techniques to surpass any retrofit efforts to date. The District's design work and advanced seismic technology will assist similar seismic retrofit projects around the world.

Architectural Heritage and Seismic Retrofit of the Golden Gate Bridge

Héritage architectural et consolidation du pont de Golden Gate vis-à-vis de séismes

Denkmalschutz und Erdbebenertüchtigung der Golden-Gate-Brücke

Charles SEIM
Senior Principal
T.Y. Lin International
San Francisco, CA, USA



Charles Seim has been with T.Y. Lin for 15 years. He has been active in the design of large bridges, seismic investigations and is the Project Manager for the seismic retrofit of the suspension span of the Golden Gate Bridge. Prior to joining T.Y. Lin, he participated in the design, construction and maintenance of California toll bridges.

SUMMARY

The 58 year old Golden Gate Bridge is world renown as an example of the beauty that engineering can achieve. The crossing consists of seven structural types, including the 1280 m suspension span. All are in need of seismic retrofit to upgrade the structures to withstand a magnitude 8 event. The retrofit measures must conform to performance, design and architectural criteria to preserve the bridge's historical and architectural heritage. The paper reviews the original design and the collaboration between the engineer and the architect and presents the criteria to be used by designers to safeguard this structure.

RÉSUMÉ

Le Pont de Golden Gate, construit il y a 58 ans, a une réputation mondiale, due à sa beauté et au succès de sa réalisation technique. La traversé du Golden Gate peut être décomposé en sept parties structurales, comprenant entre autres, la partie suspendue de 1280 m. Toutes ces parties doivent être reprises et consolidées, du point de vue sismique, afin de résister à un événement de magnitude 8. Les mesures de consolidation doivent satisfaire des critères de performance, de projet et d'aspects architecturaux, afin de préserver les qualités historiques et architecturales du pont. L'article traite du projet original et de la collaboration entre l'ingénieur et l'architecte. Il présente les critères que doivent respecter les ingénieurs afin d'assurer la pérennité de cette construction.

ZUSAMMENFASSUNG

Die 58 Jahre alte Golden-Gate-Brücke ist ein weltbekanntes Beispiel für die im Ingenieurbau erreichbare Schönheit. Der Brückenzug besteht aus sieben Tragsystemen, darunter der 1280 m langen Hängebrücke. Sie alle benötigen eine Tragwerksverstärkung, um einem Erdbeben der Magnitude 8 widerstehen zu können. Die Ertüchtigungsmassnahmen müssen Leistungs-, Entwurfs- und architektonische Kriterien erfüllen, um die bauhistorische Bedeutung der Brücke zu erhalten. Der Beitrag behandelt den ursprünglichen Entwurf und die Zusammenarbeit zwischen Ingenieur und Architekt mit den denkmalschützerischen Vorgaben.

1. INTRODUCTION

The Golden Gate Bridge is one of the most famous, historical and enduring structural achievement in the world. The start of its construction culminated a decade of bridge designs that extended the world record for span length five times (1) in the United States. The Golden Gate Bridge opened on May 28, 1937 and held the title of the world's longest bridge for 27 years when it lost by only 60 ft to the Verrazzano-Narrows Bridge in New York.

Although most people think of the Golden Gate Bridge as a single structure, the 2790m overall length of the bridge actually consists of seven different structure types. The bridge's major components are the North & South steel truss approach viaducts, the Fort Point steel arch, the steel cable's concrete anchorages and concrete anchorage housings, the main span steel suspension bridge, and the art deco concrete pylons which are purely architectural motifs. All of the foundations for these structures except the northern viaduct are supported directly on rock. The design and construction of the bridge has been well documented in the Report of the Chief Engineer (2).

2. BRIDGE LIFE

The question is often asked, "How long will the Golden Gate Bridge last?". The answer, of course, is "hundreds of years", if it is properly maintained. Eliminating obsolesces, what will bring the Golden Gate Bridge down? Corrosion, fatigue, scour, wind, and earthquakes are the enemy of all bridges. But these are natural events; perhaps more bridges have been destroyed by the most evil and horrible of all - man-made events - war.

The owners and operators, The Golden Gate Bridge Highway and Transportation District's policy is to maintain the bridge in first class conditions; corrosion is not a factor. Live load stresses are low; fatigue is unlikely. The bridge is founded on rocks - scour cannot happen. But the bridge was damaged by a wind storm in 1951. The damage was repaired and a lower lateral bracing system installed in 1954.

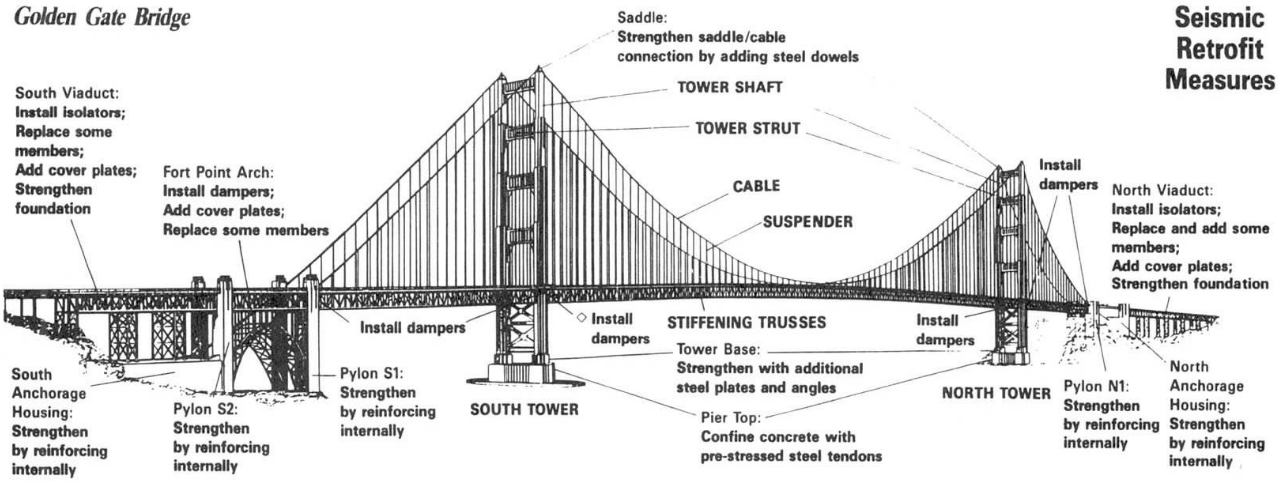
The seismic risk to the bridge was brought startling to the attention of the engineering profession by the October 17, 1989, Loma Prieta Earthquake. The Golden Gate Bridge was not damaged by this moderate and distant earthquake. The maximum acceleration near the bridge site was a modest eight percent of gravity, about its design value of seven and one half percent.

Shortly after the earthquake, the District employed T.Y. Lin International (TYLI) to perform a seismic evaluation of the entire crossing. That investigation (3) found that in a magnitude 7 event, severe damage could close the bridge for an extended period of time. In an event similar to the 1906 disaster, considered equal to the maximum credible earthquake of magnitude eight plus, portions of the bridge could be in risk of collapse.

A follow-up report (4) developed design criteria, a seismic retrofit concept for each of the 7 structures comprising the crossing, and a cost estimate for seismic retrofit of the crossing.

3. PERFORMANCE CRITERIA

Performance Criteria that controls the service and use of the bridge after a major event, were developed by the District to meet the following four requirements in a maximum credible earthquake defined as a return period of 1000 to 2000 years.



- The bridge shall not be totally closed to the public for more than 24 hours after the earthquake.
- The bridge shall provide emergency vehicle access immediately after the earthquake.
- The bridge shall be available for limited vehicular access (e.g., public transportation) within a few days after the earthquake.
- The bridge should be repairable to fully operational pre-earthquake service levels within one month after an earthquake.

Limited, repairable damage to the bridge, consistent with these four requirements, is acceptable. This criteria will allow the bridge to serve as an emergency entrance and exit for San Francisco and, within a few days, to serve the public and reduce the cost of damage repairs.

4. ARCHITECTURAL HERITAGE

The aesthetic impact of the Golden Gate Bridge was the result of a fruitful collaboration between an engineer, Joseph B. Strauss and an architect, Irving F. Morrow. Strauss wanted to create the most beautiful bridge in the world. Perhaps the considerable criticism he received for his original proposed monstrosity of a hybrid cantilever/suspension bridge convinced him to stress beauty of design for this crossing..

The project was both his and Morrow's opus magnum, an unprecedented technical challenge and achievement at a breathtaking site. Strauss wrote in (2) " It is a truism that every great bridge project in its consummation has contributed notably to the science of structural design and the technique of the builder; and in these respects the Golden Gate Bridge has been no exception. Nevertheless, its outstanding contribution has not been to these alone, but to architectonics as well, for the structure since its completion has received notable recognition because of its majestic beauty and size."

" We have seen that the design adopted was one in which the essential beauty and elemental simplicity of a conventional suspension design was obtained, a design with symmetrical shore spans supported by the cables. To this simplicity of line was added the dignity of the well-proportioned portal-braced towers, all in accordance with the original studies. A happily-selected color scheme dominated by orange-vermilion completes the picture, blending perfectly with the changing seasonal tints of the natural setting of the bridge and the surrounding land masses, sea and sky. The effect is as highly pleasing as it is unusual in the realm of engineering structures."

Within this statement by Strauss, we see his feelings of the importance of aesthetics (architectonics) to the overall impact of the bridge on the viewer. He mentions simplicity of line, well proportions, and color scheme, all of which contributes to the beautiful appearance of the structure.

Morrow's major contribution is that he viewed the Bridge as a whole, and insisted that all elements form an integral design. As Morrow himself put it: "The architectural design of the bridge is properly a single, all-inclusive problem embracing its appearance in every possible aspect. Form, texture, color, illumination, etc., are each and every one only integral parts of one general conception. To isolate as a separate detail any one of these aspects of appearance would result in disharmony, or at best in failure to realize to the full the original intention of the design. In view of the tremendous scale and dignity of the Golden Gate Bridge, the preservation of unity is of prime importance. Small effects cleverness, trickiness will prove disintegrating and unworthy. All treatment must aim at the utmost breadth and simplicity of effect."

We can see that Morrow's viewpoint harmonizes with Strauss' and emphasizes the importance of respecting and preserving the structural heritage that is so admired by the public.

Clifford Paine, the Principal Assistant Engineer to Strauss throughout the construction of the Bridge, wrote: "The architectural treatment of the bridge was carefully studied during the early stages of the design, the towers being given special attention as they constitute such a prominent feature. The size and spacing of the struts above the floor, the treatment of the strut enclosures, the location of the offsets in the shafts, the number and position of the diagonals...all received careful consideration."

Paine stresses his concern for proportion of size and spacing particularly for the two main towers which are the dominant features of the bridge, and by simple measurements, determines the size and spacing of the struts follows a logical profession. The towers are 227m high with struts above the roadway and the more structurally efficient cross-bracing used below the roadway. The shafts of the towers are tapered both transversely and longitudinally by setbacks along the height.

It is interesting to view the proportions of the towers as they were finally constructed. The four struts above the roadway vary in depth and spacing, being thinner and closer together with height. The two top struts are each approximately 6.7m deep and the lower two are approximately 9.1 ft deep, giving a ratio of about 1 to 1.36. From the tower top down to the roadway, the vertical strut spacing ratios are 1, 1.1, 1.3 and 1.4, respectively.

The importance of the architectural design and historical heritage of the Golden Gate Bridge is well documented in (5). The bridge, by every measure, is eligible for listing on the National Register of Historic Places and is a Civil Engineering Landmark. In 1994, the American Society of Civil Engineers elected the bridge as one of the "Modern Wonders of Civil Engineering."

5. ARCHITECTURAL CRITERIA

According to the U.S. Historic Preservation Act of 1966 and the Secretary of the Interior's Standards for Rehabilitation, special consideration must be given to any changes to the bridge that may affect the defining characteristics of the structure. For the Golden Gate Bridge, these include distinctive features, such as the steel arch over Fort Point, the flanking concrete pylons, and finishes such as the International orange color or concrete form marks.

If new work is required, it should not destroy historic materials that characterize the property and should be compatible with mass, size, scale, and architectural features to protect the historic integrity of the property and its environment. These issues are not binding on the retrofit measures at this time, but were used as strong guidelines for the seismic retrofit design.

In recognition of these issues, the retrofitting measures developed to upgrade the seismic performance of the bridge will meet the following hierarchical guidelines:

1. First priority shall be to meet the seismic retrofit design criteria presented in the Design Criteria.
2. Second priority shall be to maintain the current architectural appearance of the bridge and to follow as much as possible the guidelines of the U.S. Preservation Act of 1966. Care shall be taken not to radically change the structural systems and structural features of the existing bridge. Seismic retrofit measures shall preserve as much as possible the scale of member and proportions of solids to voids of the existing bridge.
3. Third priority shall be to respect as much as possible the architectural vocabulary established for each of the structural types comprising the bridge. Care shall be taken not to radically change the character defining features, materials, finishes and color of the existing bridge.
4. Fourth priority shall be to retain as much as possible of the original material that is now constructed into the structure.

6. DESIGN CRITERIA

Since no design documents existed for seismic retrofit design of long span bridges, TYLI was engaged to develop a Design Criteria for Seismic Retrofit Measures (5). The criteria was developed as a guide for designers and was printed in loose leaf notebooks so that revisions could easily be inserted. In lieu of using the criteria, the designer is permitted two alternatives (1) analysis methods which consider actual material properties and behavior, or (2) physical models or testing.

The technical issues that the Design Criteria addresses are based on meeting the Performance Criteria and Architectural Requirements noted above. Limited repairable damage that does not threaten structural safety and that can be repaired without interrupting traffic is acceptable. This does allow portions of the bridge to respond to limited inelastic action but the primary response should be essentially elastic where possible.

7. SEISMIC RETROFTI METHODS

The figure shows the final seismic retrofit methods developed for the seismic retrofit of the seven structural types of the Golden Gate Bridge crossing. Several consultants are working on the construction documents with completion schedule for mid-summer. The District plans to start construction during the summer of 1995. The total estimated cost of the construction, engineering and administration is about $165 to $175 million. This is approximately 10 to 15 percent of the replacement costs of the entire crossing.

8. CONCLUSION

The seismic retrofit methods developed for the seven structural types comprising the Golden Gate crossing has shown that all seven can be structurally upgraded to meet the Performance Criteria, the Design Criteria and the Architectural Requirements of the Golden Gate Bridge.

The design criteria and retrofit methods developed during the design of the seismic retrofit of the Golden Gate Bridge provides a methodology that can be applied to the seismic retrofit design of other major bridges and also honor the historical heritage of the structure.

9. ACKNOWLEDGMENTS

All of the Golden Gate Bridge studies summarized in this paper were made under the direction of the Golden Gate Bridge Highway and Transportation District. The authors wish to thank the General Manager Carney Campion, District Engineer Emeritus Daniel Mohn, and District Engineer Mervin Giacomini for their support.

10. REFERENCES

1. Encyclopedia Britannia, Latest Edition.

2. STRAUSS, J., The Golden Gate Bridge: Report of the Chief Engineer to the Board of Directors of the Golden Gate Bridge and Highway District, California, 1937, San Francisco, California, Golden Gate Bridge and Highway District, 1938.

3. KETCHUM, M.A and SEIM, C., Golden Gate Bridge Seismic Evaluation, T.Y. Lin International, Golden Gate Bridge, Highway and Transportation District, San Francisco, CA, November 1990.

4. KETCHUM, M.A and SEIM, C., Golden Gate Bridge Seismic Retrofit Studies, T.Y. Lin International, Golden Gate Bridge, Highway and Transportation District, San Francisco, CA, July 1991.

5. MAC DONALD, D.W., and MOL, C., Findings of No Adverse Effects, MacDonald Architects, San Francisco, CA, January, 1995

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

Tim INGHAM
Principal
T.Y. Lin International
San Francisco, CA, USA

Tim Ingham, born in 1955, obtained his Ph.D. from the University of California, Berkeley, in 1982. Since then he has worked on bridge projects throughout the US, including the design of segmental and cable-stayed bridges, and the aseismic design of structures.

Santiago RODRIGUEZ
Project Engineer
T.Y. Lin International
San Francisco, CA, USA

Santiago Rodriguez, born in 1962, obtained his M.S. degrees from the Polytechnic University of Madrid in 1986, and from Princeton University in 1990. Since then he has worked on the analysis and design of segmental, cable-stayed, and suspension bridges.

Marwan NADER
Project Engineer
T.Y. Lin International
San Francisco, CA, USA

Marwan Nader, born in 1964, obtained his Ph.D. from the University of California, Berkeley, in 1990, where he researched the aseismic design of steel structures. Since then he has worked on the seismic retrofit of major San Francisco Bay crossings.

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.

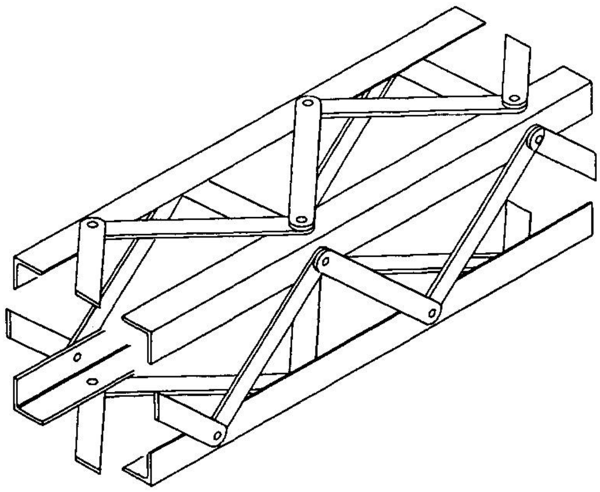


Fig. 2 Typical Lateral Brace

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.

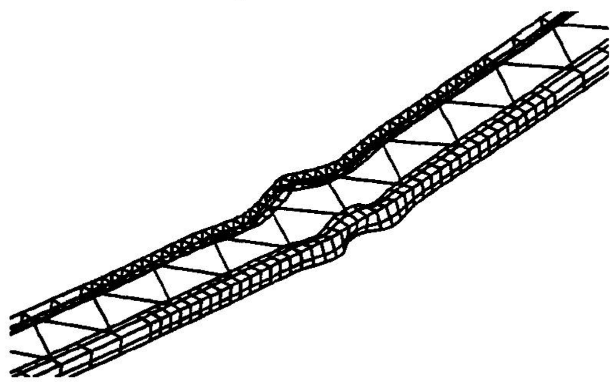


Fig. 3 Local Buckling of Lateral Brace

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].

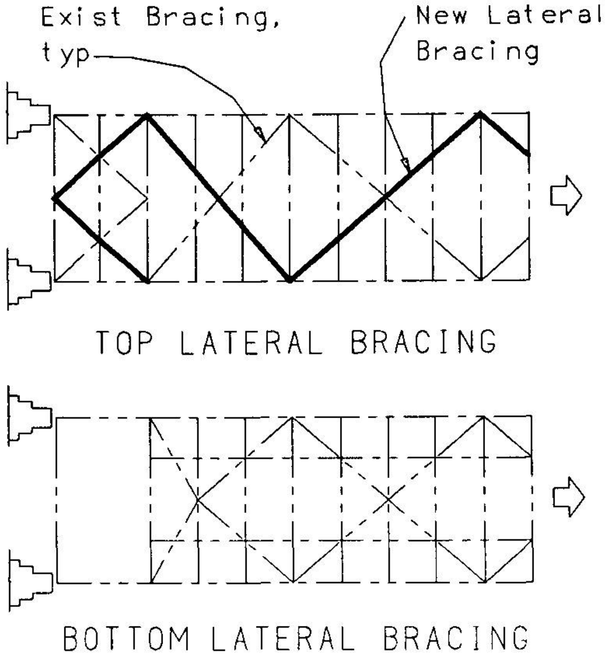


Fig. 4 Lateral Bracing Retrofit

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).

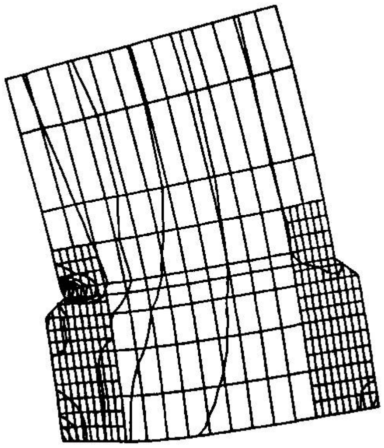


Fig. 5 Uplift of Tower Base

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.

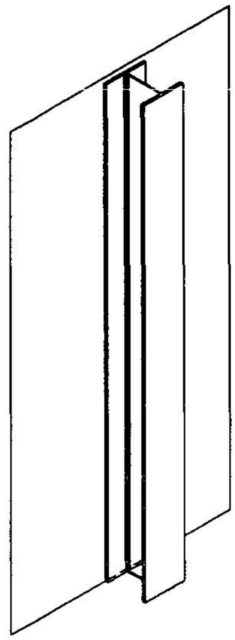


Fig. 6 Plate Stiffener

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.

REFERENCES

1. GEOSPECTRA INCORPORATED, Geological, Geotechnical and Ground Motion Studies for Seismic Retrofit of the Golden Gate Bridge, Golden Gate Bridge, Highway and Transportation District, San Francisco, CA, July 2, 1992.

2. RODRIGUEZ, S., SEIM, C., and INGHAM, T.J., 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, NCEER-94-0009, March 31, 1994.

3. DOWRICK, D.J., Earthquake Resistant Design, John Wiley & Sons, 1987.

Seismic Retrofit of the South Pylons for the Golden Gate Bridge

Consolidation parasismique des pylônes sud du pont de Golden Gate

Erdbebenertüchtigung der Südpylone der Golden-Gate-Brücke

Thomas C. JEE
Chief Bridge Engineer
Sverdrup Civil, Inc.
Walnut Creek, CA, USA

Kuang Y. LIM, Ph.D.
Project Engineer
Sverdrup Civil, Inc.
Walnut Creek, CA, USA

Mervin C. GIACOMINI
District Engineer
Golden Gate Bridge District
San Francisco, CA, USA

Ewa Z. BAUER
Deputy District Engineer
Golden Gate Bridge District
San Francisco, CA, USA

SUMMARY

This paper presents the rationale governing the seismic retrofit design of the Golden Gate Bridge South Pylons, from identifying deficiencies in seismic performance to possible strengthening strategies and final strategy selection. Comprehensive investigations performed on various possible retrofit options are presented. This paper also illustrates the importance of integrating analysis, design and detailing with other design criteria such as reliability, aesthetics, constructibility, serviceability, and economics.

RÉSUMÉ

L'article présente les considérations qui ont conduit au concept de renforcement du pylône sud du pont de Golden Gate, allant de l'identification des faiblesses du comportement sismique jusqu'aux stratégies de renforcement possibles et au choix final de la stratégie. Des études globales ont été réalisées sur différentes possibilités de consolidation. L'article illustre l'importance d'une conception globale de l'analyse du projet et des détails constructifs en fonction d'autres critères de projet tels que fiabilité, esthétique, possibilités de réalisation, aptitude aux service et aspects financiers.

ZUSAMMENFASSUNG

Der Beitrag zeigt die Ueberlegungen auf, die das Verstärkungskonzept für die Südpylone der Golden-Gate-Brücke bestimmen, von der Identifizierung von Mängeln im Erdbebenverhalten bis zur definitiven Konzeptwahl. Dazu wurden umfangreiche Untersuchungen an verschiedenen möglichen Ertüchtigungsoptionen vorgenommen. Die Bedeutung einer integralen Lösung, die ausser den Tragwerksproblemen auch die Zuverlässigkeit, Aesthetik, Bauverfahren, Gebrauchstüchtigkeit und Wirtschaftlichkeit einbezieht, wird deutlich.

1. INTRODUCTION

The two South Pylons, S1 and S2, of the Golden Gate Bridge are two massive under-reinforced concrete bent type structures (see Figure 1). Pylons S1, the northern most of the two structures, is located between the Fort Point Arch and the main Suspension Span. Pylon S2 is located between the South Viaduct and the Fort Point Arch. Pylon S1 is approximately 250 feet tall and is made up of walls of different thicknesses varying from 24 to 36 inches which form both of the approximately 32 feet by 43 feet double cell hollow legs of the Pylon. The legs are joined at the top by a transverse cross beam that consists of two walls, each 48 inches thick and about 30 feet deep. The top of the Pylon supports a 32-foot roadway. In addition to supporting 32 feet of roadway, Pylon S1 provides a tie-down for each of the main suspension cables. It also provides wind lock coonection for the side suspension span. Pylon S2 is somewhat similar in dimension to S1, however, it has less demand placed on it from adjacent structures, therefore its wall thicknesses vary from 18 to 30 inches and the legs are single cell hollow structures. The existing Pylons S1 and S2 weigh approximately 40,000 kips and 30,000 kips, respectively. Both Pylons are supported on spread footings founded on rock and provide support for the Fort Point Arch structure near their base. The existing reinforcement in the walls of the Pylons consists of two curtains of 0.75 inch diameter plain bars at 24 inches spacing, totalling less than 1 % steel, which is significantly less than the minimum required by modern design codes. The corrosive environment due to ocean wave spraying at the bridge has resulted in serious deterioration and spalling of the west walls of both Pylons.

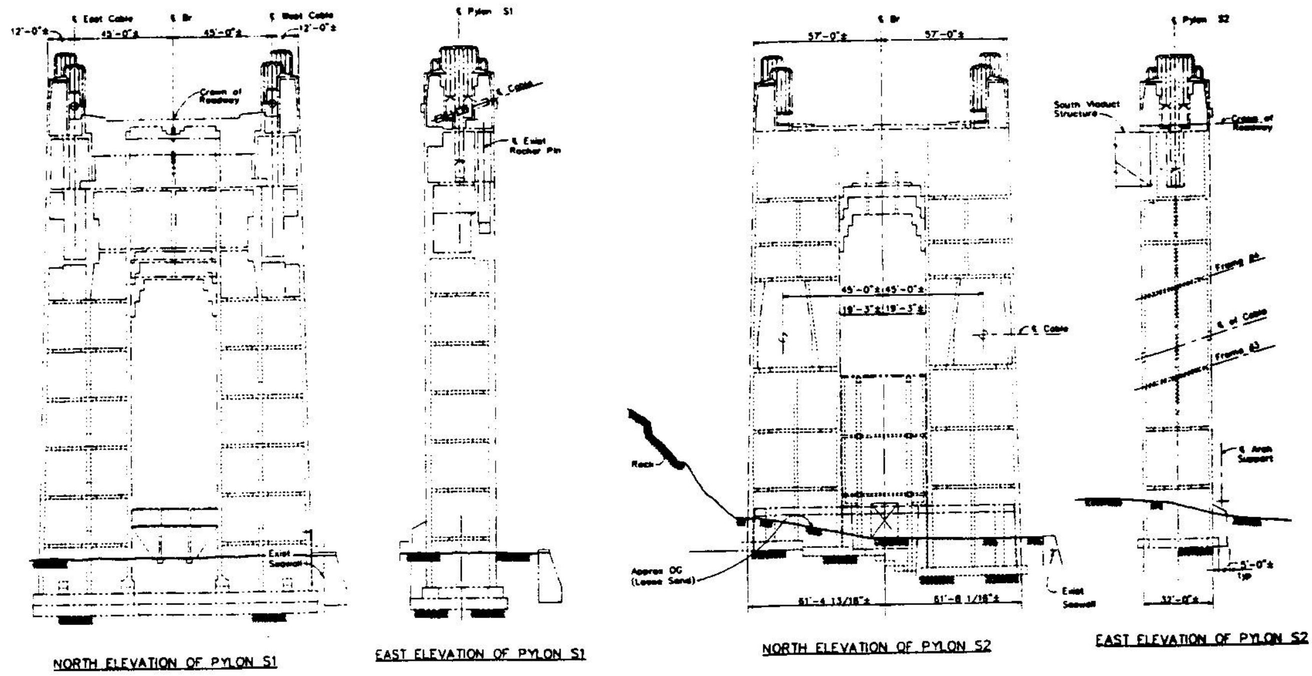


Figure 1: Elevations of Pylons S1 and S2

2. DESIGN CRITERIA AND GROUND MOTIONS:

The goal of the proposed seismic retrofit of the Golden Gate Bridge is to strengthen the bridge so that it will maintain its function after sustaining a maximum credible earthquake (MCE). The importance of the structure and its proximity to two major active faults (the San Andreas and Hayward faults) required 16386Dthe development of a comprehensive project specific design criteria [1].

For seismic evaluation purposes, site specific studies generated three sets of ground motion time histories for the bridge site based on the rupture scenario of the San Andreas fault [2]. These time histories depict acceleration, velocity and displacement in three orthogonal directions at all support points of the bridge, and they also incorporated the effects of seismic source, wave attenuation and passage, and local wave scattering resulting from the peculiar site topography and geology. The magnitude of the time histories were for a maximum credible design event based on 1000 to 2500 year return period.

3. DIAGNOSTIC ANALYSES AND RESULTS:

Comprehensive methodical three-dimensional linear and non-linear time-history analyses were conducted on each pylon to identify the structure's vulnerabilities, damage scenarios and failure modes [3]. Since the height to width ratio of the pylons are large and their foundations are founded on spread footings, preliminary analysis results with a fixed base indicated that overturning would be a problem. The structures would be subjected to forces that would cause uplift early on in the earthquake record during a maximum credible event. Thus, to better predict the response of the structures, uplift phenomenon was explicitly considered by using nonlinear support boundary conditions in the models. The assumption used at the supports was that only vertical movement would be permitted. In addition to the demands placed on the Pylons due to their own self weight combined with seismic loads, the interaction between the Pylons and the adjacent structures were also included in the analysis. These interactions were modeled as time histories of force reactions from the adjacent structures applied to the Pylons.

The seismic vulnerabilities identified from the analysis indicated that the existing Pylons would uplift and since the Pylon walls were very lightly reinforced, severe tension cracking occurred, which under repeated cyclic loads caused severe degradation and subsequent failure.

4. PRELIMINARY RETROFIT SCHEME INVESTIGATIONS:

The diagnostic analyses showed that the existing lightly reinforced concrete wall of the Pylons was not able to resist the tension demands due to seismic loads and that a major retrofit was needed to satisfy the design criteria. The maximum compressive stresses before failure were approximately 30% of the ultimate stress indicating that the Pylon had sufficient compressive capacity. The lack of tension capacity is what caused the Pylon to fail. The results also indicated that the response of the Pylon legs is not like that of a typical column in bending, but instead the walls act like a membrane in either tension or compression. In addition, allowing rocking or uplift at the Pylon base significantly reduced the forces on the structure.

Prior to the development of the final retrofit strategy, several possible concepts for retrofitting the Pylons were methodically investigated in detail. The development process of the most promising options comprises two essential considerations:

1. Strengthening the walls while attempting to minimize the added mass.

2. Displacement compatibility with adjacent structures. The interaction between the Pylons and the Arch, as well as the interaction between Pylon S1 and the Suspension Bridge and between Pylon S2 and the South Viaduct limits the acceptable displacements at the top of the Pylons. Besides retrofitting the structure for seismic resistance, there are other important aesthetics considerations as the Golden Gate Bridge is classified as a historical landmark. The Historic Preservation Act of 1966 and the Secretary of Interior's Standard of Rehabilitation dictates that the defining characteristics of any historical landmark

shall be preserved, which include distinctive features and finishes.

The numerous strategies generated during the preliminary analytical investigations include one or a combination of base isolation, foundation rocking, thickening of concrete walls, steel plate encasement, wall posttensioning, construction of new inner ductile frame, and complete structure replacement.

The retrofit scheme as suggested by the previous studies [4] by adding new interior reinforced concrete walls within each Pylon leg and providing complete base fixity by installing posttensioned rock anchors was found to be impractical and inadequate for the following reasons: First, completely fixing the base invariably attracts excessive seismic base shears and overturning moments which requires an impractical large number of rock anchors. Second, strengthening the Pylon walls on the inside face alone does not help the structure seismically as the outside face of the walls would crack and spall due to the high tension stresses. Third, the mass of the structure will increase by such retrofit scheme which in turn would generate higher seismic demands and thereby reduce the effectiveness of the strength to mass ratio.

Most of the preliminary investigations attempted to maintain the architecture of the Pylons by placing the retrofit on the inside. As the process evolved, however, the high tension and compression stresses on the outside face of the existing walls required strengthening outside as well. As a result, modifying the exterior of the Pylons was necessary. However, any exterior modifications would be architecturally reviewed and approved.

5. FINAL RETROFIT STRATEGY SELECTION:

The need for adequate wall strength is obvious for the Pylons which have very low tensile and ductility capacities due to the minimal reinforcement levels and the absence of confinement steel. Due to the extremely high tension demand on the Pylon walls, a scheme which includes the steel plate "sandwiching" the existing walls while allowing uplift of the foundation was finally accepted (see Figure 2). The scheme requires through ties to provide composite action between the existing Pylon walls and the steel plates. The "sandwich" steel plating scheme was adopted because of its higher strength to mass ratio; its superior strength against two dimensional membrane stresses; its inherent ductility; its more predictable structural behavior; and lastly its relative ease and simplicity in construction.

The foundation will be partially tied down with partially unbonded rock anchors. This was chosen for two reasons. First, allowing the foundation to rock freely results in excessive displacements at the top of the Pylons. Second, completely fixing the base requires an impractical number of rock anchors. The level of tiedown required is determined by sliding resistance to the rocking base shear. A large monolithic combined footing linking both the Pylons legs will be provided (see Figure 3). Not only does this combined footing enhance rocking stability by ensuring that the center of gravity of the rocking mass falls within the base, but it also spreads the stresses more uniformly across the foundation-soil interface to prevent abrupt soil failure beneath the Pylon's foundations. Special detailing attention is given to the interface between the walls and the foundations of the Pylons to ensure structural integrity so that the Pylon walls would not break away from the combined footing during the rocking motion.

The architectural criteria for the Pylon retrofits are focused on retaining most of the dimensional characteristics and the concrete exterior. The basic retrofit concept for the Pylon walls consists of a continuous structural steel plate on both exterior and interior faces of the Pylons with a shotcrete veneer on the exterior face of the walls as shown in Figure 4. Extensive effort was taken to address the constructibility and corrosion protection and structural durability of the steel plates against the highly corrosive environment at the bridge site [5], in addition to preserving the original concrete texture.

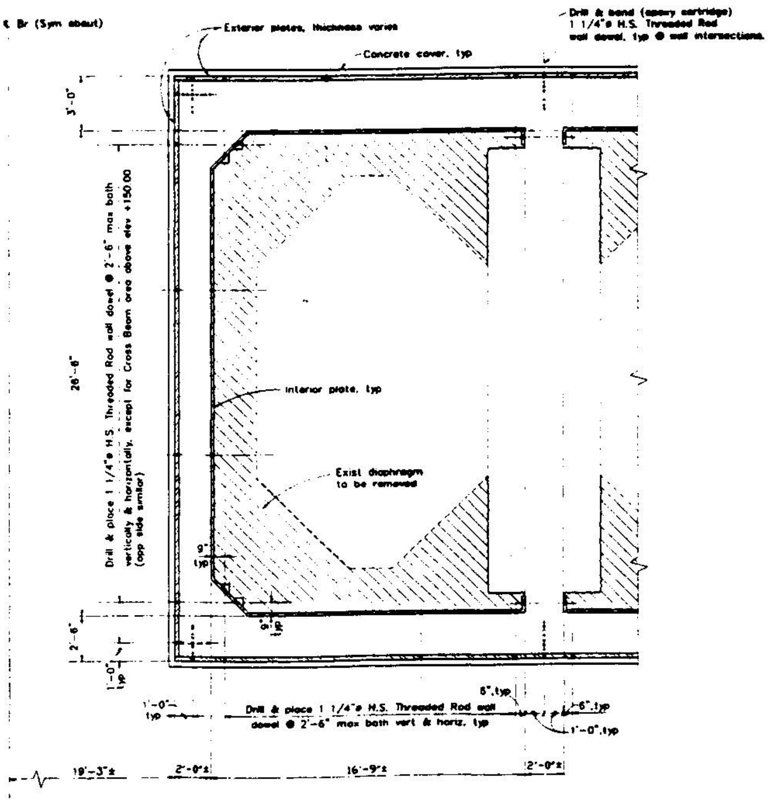


Figure 2: Part Typical Wall Section Showing Steel Plating Arrangement

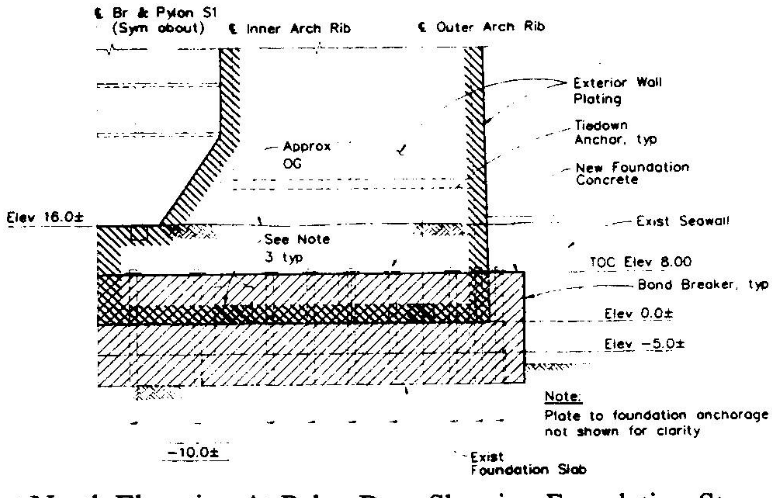


Figure 3: Part North Elevation At Pylon Base Showing Foundation Strengthening

The envisioned construction begins with the removal of the existing cover concrete on the exterior face of the Pylons, including any unsound concrete. The exposed concrete and reinforcement are sandblasted to remove any traces of impurities. The exterior and interior steel plates, in manageable sizes, are then placed and tied together by through ties in stages. Each smaller individual steel plate panels will be field welded together and grout is then pressure injected to seal any void between the steel plates and the existing concrete. A four-inch new concrete cover in two coats is placed on the exterior face. A three inch inner coat which comprises steel fiber reinforced silica fume shotcrete will be placed first with one-inch second coat of shotcrete on top to simulate the original concrete texture. To ensure proper bonding and cracking control, the concrete cover will be reinforced with shear studs and reinforcing mesh.

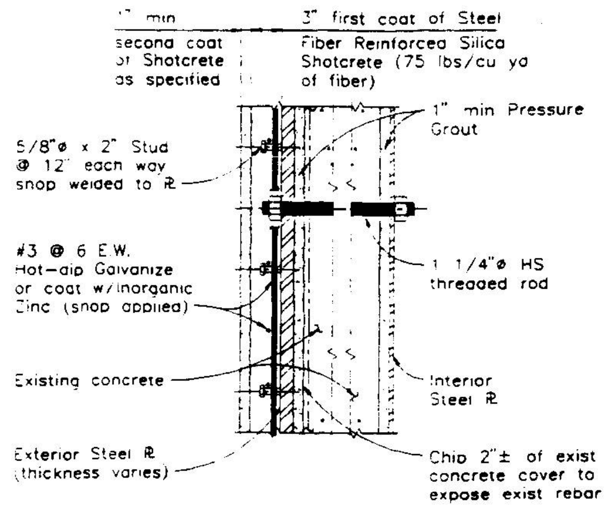


Figure 4: Retrofit Details of Pylon Wall

6. CONCLUSIONS:

The goal of the proposed seismic retrofit of the Golden Gate Bridge is to strengthen the Bridge so that it will maintain its function after sustaining a next major earthquake. The highly under-reinforced nature of the concrete combined with the mass and geometry of the existing Pylons make them very vulnerable to the next earthquake. Extensive linear and nonlinear computer models and analyses were created to determine various retrofit options for the Pylons. Due to the extremely high tension demand on the Pylon walls and the relatively poor condition of the existing concrete, a scheme which includes the steel plate encasement while allowing a certain amount of uplift for foundation rocking was adopted. To meet the corrosion protection and architectural aesthetic project requirements, the exterior steel plating is covered with high density concrete to maintain the original structure appearance. The Sverdrup team is confident that the final retrofit scheme adopted will improve the Pylons' overall behavior, prevent collapse, and maintain serviceability.

REFERENCES

1. "Golden Gate Bridge Design Criteria for Seismic Retrofit Measures", T.Y.Lin International in Association with Imbsen and Associates, Inc., Geospectra, Inc., and MacDonald Architects, November 2, 1992.

2. "Incoherency and Spectra Compatible Ground Motions for San Andreas Events SA1, SA2, and SA3", Geospectra, Inc., November 22, 1993.

3. "Golden Gate Bridge South Pylons S1 and S2 Retrofit Strategies Report", Sverdrup Corporation in Association with Isec, Inc., and Buckland and Taylor, Ltd., April 27, 1994.

4. "Golden Gate Bridge Seismic Retrofit Studies", T.Y.Lin International in Association with Imbsen and Associates, Inc., and Geospectra, Inc., July 10, 1991.

5. "Supplemental Report on Golden Gate Bridge South Pylons S1 and S2 Retrofit Strategies", Sverdrup Corporation in Association with Isec, Inc., and Buckland and Taylor, Ltd., September 15, 1994.

Seismic Analysis of the Fort Point Arch of the Golden Gate Bridge

Analyse parasismique de l'arc Fort Point du pont de Golden Gate

Erdbebenberechnung des Fort Point Arch der Golden-Gate-Brücke

Jawahar M. GIDWANI
President
ISEC, Inc.
San Francisco, CA, USA

Thomas C. JEE
Project Manager
Sverdrup Civil Inc.
Walnut Creek, CA, USA

Noel A. STAMFLI
Senior Civil Engineer
Golden Gate Bridge H&T District
San Francisco, CA, USA

SUMMARY

This paper describes the three-dimensional linear and nonlinear dynamic time history analyses used to evaluate the response and vulnerabilities of the as-built arch over the historic Fort Point of the Golden Gate Bridge. Comprehensive investigations conducted on possible retrofit schemes are presented. Comparison of results from linear and non-linear analyses are discussed. All results presented are from step-by-step time integration of equations of motion. The results of nonlinear analyses were instrumental in developing a simple, efficient, and cost-effective retrofit scheme that did not alter the appearance of the bridge.

RÉSUMÉ

L'article décrit les analyses tridimensionnelles, linéaires et non linéaires du comportement historique et dynamique utilisées pour évaluer le comportement et les faiblesses de l'arc actuel de Fort Point du Pont de Golden Gate. Les études générales de variantes de consolidation sont présentées. La comparaison des résultats d'analyses linéaires et non linéaires est discutée. Tous les résultats présentés résultent d'une intégration continue des équations du mouvement. Les résultats de l'analyse non linéaire ont été utilisés pour réaliser un projet de consolidation simple, efficace et économique, qui n'altère pas l'apparence du pont.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die linearen und nichtlinearen Zeitverlaufsberechnungen zum linearen und nichtlinearen dynamischen Antwortverhalten und der Verwundbarkeit der bestehenden Bogenbrücke über das historische Fort Point. Es werden umfangreiche Vergleichsuntersuchungen zu möglichen Verstärkungsmassnahmen vorgestellt und die linearen und nichtlinearen Berechnungsergebnisse mit direkter Integration der Bewegungsgleichungen diskutiert. Dank der nichtlinearen Berechnungen konnte ein einfaches, effizientes und wirtschaftliches Verstärkungskonzept entwickelt werden, das das Aussehen der Brücke nicht beeinträchtigt.

1. INTRODUCTION

Recent earthquakes in California and Japan have reminded us all of the urgency to retrofit our existing structures for seismic safety. The Golden Gate Bridge, close to the San Andreas fault, is one such vital transportation structure that has been evaluated for seismic loads. Appropriate retrofit schemes to upgrade the system have been developed. Specifically, key results from the methodical comprehensive Fort Point Arch study are presented in this paper.

The Arch of the Golden Gate Bridge spans approximately 320 feet over the historical Fort Point. It comprises four ribs (Outer East, Inner East, Inner West and Outer West) that are supported on pins. The pins are on shoes fixed to the Pylon S1 on the north end and Pylon S2 on the south end (see figure 1). Spandrel columns on these arch ribs support a truss system. The roadway is on the truss system.

The project was conducted in three stages: (a) evaluation of the vulnerabilities of the as-built structure; (b) investigation of preliminary retrofit schemes; and (c) selection of a final detailed retrofit scheme.

All results presented in this paper are from linear and state-of-the-art nonlinear time history analyses. Ground motion time histories in three orthogonal directions (longitudinal, transverse and vertical) were simultaneously used as input for the analyses. Program KARMA® [1] was used for all linear and nonlinear analyses on SUN's desktop multi-processor workstations. Finally, a real time motions video for the retrofitted system was created using program SWAMI® [2].

2. DESIGN CRITERIA AND GROUND MOTION

Comprehensive project specific design criteria were developed which addressed policy and performance issues [3]. In essence, any retrofit scheme should be such that under a maximum credible earthquake (MCE), the retrofitted Golden Gate Bridge must satisfy the following performance criteria: (a) severe structural damage that can cause the bridge to be closed to traffic for more than 24 hours is NOT acceptable; (b) damage to bridge members is acceptable if such damage does not compromise the structural integrity of the bridge and if it can be repaired without interrupting traffic; (c) the bridge shall be capable of providing emergency access immediately after the earthquake; (d) within a few days of the earthquake, the bridge shall be available for limited public access;(e) the bridge shall be repairable to fully operational pre-earthquake levels within one month of the earthquake; (f) and any retrofit scheme shall NOT significantly alter the appearance of the bridge.

Though these criteria were comprehensive, they did not address key issues of inelastic analyses and behavior, or qualify and quantify repairable damage. Hence, a comprehensive set of guidelines for inelastic analyses were developed and adopted in this study [4].

Site specific studies generated three sets of ground motion time histories, each set representing a rupture scenario of the San Andreas fault [5]. The magnitude of time histories were for a maximum credible design event based on 1000 to 2500 year return period.

3. ANALYSES OF THE AS-BUILT STRUCTURE

The main purposes of this exercise were to identify the system's vulnerabilities, damage scenarios and failure modes. Since, the arch sits on shoes, it is possible that the arch would uplift from its supports during a maximum credible earthquake. Such an uplift phenomenon was explicitly considered in the models. Two separate three-dimensional mathematical models were created: (a) a linear elastic model of the as-built structure where the material properties of the arch members were considered linear elastic, but uplift of the arch from its supports was considered (i.e. the support boundary conditions were nonlinear); and (b) a fully nonlinear model where the inelastic behavior of all arch members was modeled. Again, the support boundary conditions were nonlinear (i.e. uplift was considered).

3.1 Three-Dimensional Linear Elastic Model and Analyses Results.

All members in the model were modeled as linear elastic members. The arch was allowed to uplift from its supports while retaining its horizontal thrust. The section properties of individual members were determined from shop drawings. Interaction between the pylons and the arch was neglected.

To verify the validity of the model, a dead load analysis using the 1930's concrete deck weight was performed. The analytical results were compared to the stress sheets. The member forces were within 6% of the stress sheets results.

Next, the concrete deck was replaced by the new orthotropic deck which was installed in the 1980s. Dead and eigensolution were performed on this model. The first ten eigenvalues are listed in Table 1.

Once the model was verified, two analyses were performed: a response spectrum analysis using the CQC method and 50 modes to capture more than 90% of the mass and (b) linear time history analyses using the ground motion time histories for San Andreas Event 1. Results from both analyses were similar. The time history analysis indicated a total of 415 members had Demand/Capacity (D/C) ratios greater than 1.0 (approximately 35% of the total members). Extensive overstressing was caused in all the arch ribs, spandrel columns, top truss members and vertical braces. Specifically the inner arch ribs were severely overstressed. The uplift displacement of the outer ribs was approximately 400 mm. Also, the analysis showed that the transverse arch motions were more critical than the longitudinal motions. However, it was difficult to identify the failure mode and the "true" extent of damage.

3.2 Three-Dimensional Fully Nonlinear Model and Analyses Results.

Once the linear elastic model was verified, it was converted to a fully nonlinear 3-D KARMA model (see figure 2). A material yield stress of 250 Mpa was used. The strain hardening behavior was assumed to be 2% of the elastic material modulus of elasticity of 200 GPa.

All columns, and arch ribs were modeled as 3-D large displacement inelastic beam-column elements with distributed plasticity capable of resisting axial loads and bending moments. The geometric nonlinearity was explicitly included by (a) including the geometric stiffness effects and (b) determining equilibrium in the deformed configuration (i.e. large displacement effects). A four-dimensional yield surface defined the interaction between axial forces, in-plane bending, out-of-plane bending and torsion.

All vertical and horizontal bracing members, were modeled using 3-D large displacement post-buckling elements capable of resisting axial loads only. These elements degrade in strength and stiffness due to cyclic loads. The post-buckling behavior was appropriately modeled (based on the member kl/r ratio, and available test data). The compressive capacity of the members was ignored after one full inelastic cycle, i.e., if a brace experienced one full cycle of inelastic response, then its subsequent hysteretic response was of a tension only brace with zero compressive load carrying capacity. It was assumed that modeling the inelastic behavior in this fashion would yield a conservative response.

In the full nonlinear analysis for San Andreas Event 1, a total of 173 members were identified as inelastic members (approximately 15 percent of the total members). Essentially most of the arch ribs (specifically the top chords) were now linear elastic. Flexural hinges were formed in the spandrel columns of the frames. Since the bending capacity of these columns was less than the arch rib axial capacities, formation of flexural hinges in these columns limited the forces transferred from these columns to the arch ribs. The uplift displacement of the outer ribs was approximately 250 mm. This analysis showed that the flexural hinges in the spandrel columns were formed predominantly due to longitudinal motions.

3.3 Linear Versus Nonlinear Analyses Results Comparison.

The following differences are summarized.

1. The number and location of damaged members are different. A total of 415 members were overstressed in the linear analysis, while only 173 members experienced inelastic behavior in the nonlinear analysis.

2. The most important distinction was the type of damage, initiation and propagation of damage. Specifically, in the linear analysis, the arch ribs are overstressed and transverse motions are the most damaging. In the nonlinear analysis, arch ribs are essentially linear. The only damage is localized near the support points. Flexural hinging in the spandrel columns prevents the top chords of the arch ribs from being damaged. Longitudinal motions are more critical.

3. The maximum displacements at the top from a nonlinear analysis are lower than from a linear analysis.

4. The maximum forces on the bottom bearing support points from nonlinear analysis are significantly lower than forces from a linear elastic analysis.

The linear and nonlinear analysis was repeated assuming the arch was tied to its supports (i.e. no uplift was allowed). The number of overstressed members increased significantly in the linear analysis (i.e. 615 members had D/C greater than 1.0). The nonlinear analysis indicated that the arch would fail at around 11.0 seconds into the ground motion record.

4. ANALYTICAL INVESTIGATIONS FOR PRELIMINARY RETROFIT SCHEMES

In the analysis of the as-built system, the nonlinear time history analysis provided a more realistic and detailed insight into the dynamic structural response characteristics. One specific conclusion was that allowing the arch to uplift significantly reduced the forces on the structure. In addition, flexural hinges in the spandrel columns prevent damage to the arch ribs.

Several different retrofit strategies were investigated. Some schemes investigated brute force strengthening. Others considered isolation schemes. However the most promising schemes centered around allowing the arch to uplift.

The nonlinear analysis of the as-built system showed that the arch uplifted about 250 mm from its support. This uplift caused impact forces on the supports during load reversals. The retrofit approach then was to reduce the amount of uplift and minimize or eliminate the impact problem.

One of the schemes investigated included the following (see figure 3).

1. Since the transverse loading predominantly caused uplift, Energy Dissipation Devices (EDDs) were added between the top of the arch and the adjacent pylons. These EDDs could be sacrificial structural members or any device that would dissipate the earthquake energy from transverse motions. In addition, these EDDs would control the amount of uplift and impact forces.

2. Similar to the transverse EDDs, longitudinal EDDs were placed between the pylons and the arch to dissipate earthquake energy due to longitudinal motions.

3. The arch support bearing was designed to allow (a) vertical uplift only while maintaining the arch's horizontal thrust carrying capacity; (b) allow transverse support isolation; and (c) provide a damper to cushion the impact due to uplift.

4. Strengthen some deficient members and connections.

5. Allow flexural hinges in spandrel columns with maximum rotational ductility demands within the Design Criteria limits with corresponding maximum axial forces less than 60% of the yield forces.

A full nonlinear analysis of the above retrofit scheme performed extremely well. The total number of inelastic members was 25. The arch ribs remained linear elastic. The maximum uplift displacement was approximately 25 mm. Some of this inelastic behavior was in the form of flexural hinging in spandrel columns with rotational ductility demands well within the Design Criteria limits.

All analyses performed so far ignored the interaction between the pylons and the arch. In the retrofit described above, a stick model of the pylons was added and a fully nonlinear analysis of the arch interacting with pylons was performed. The results showed that more members (93 members) were inelastic since the arch and pylons move in phase during certain intervals of the earthquake and therefore the EDDs are not fully efficient. However, it provided the basis for a preliminary retrofit strategy.

5. SELECTION OF FINAL RETROFIT STRATEGY

Once the preliminary retrofit strategy was established, a detailed study was conducted to achieve a final design. All these subsequent analyses were based on a full 3-D model of the retrofitted pylons interacting with the arch. In addition, interaction of the side span with pylon S1 and that of the south viaduct with the pylon S2 were also included in the analysis. These interactions were modeled as time histories of force reactions applied to the pylons. These forces were in addition to the ground motions. A full nonlinear arch model with rocking pylons is shown in figure 4.

During the course of analytical investigations, it was established that the transverse isolation and the damper at the arch supports could be eliminated by choosing an appropriate design of the transverse EDDs. This resulted in a simplified bearing design that allowed uplift only while maintaining the arch's horizontal thrust capacity. These investigations also indicated that making the deck rigid reduced the forces on the arch and was therefore included in the final strategy.

A full nonlinear analysis of the system indicated that the arch ribs remain linear elastic. Flexural hinges in spandrel columns with rotational ductility demands within the specified Design Criteria limits protect the arch ribs. The maximum relative longitudinal displacements between the pylons and the arch is approximately 200 mm while the corresponding maximum transverse displacements are about 180 mm.

6.0 CONCLUSIONS

The nonlinear analysis procedures used were essential in developing an efficient, simple, reliable and cost-effective retrofit scheme. Knowledge of the realistic nonlinear response of the as-built system, and the extent and locations of damage facilitated the retrofit process. More importantly, it provided information on what does not need retrofitting. These complex 3-D nonlinear analyses can now be performed on relatively inexpensive desktop workstations. The visualization tools used provided great insight into the structural response. If repairable damage to a structure is permitted, the performance must be evaluated using nonlinear analysis techniques. Use of linear or "equivalent" linear approaches may be inadequate and unsatisfactory.

REFERENCES

1. GIDWANI, JAWAHAR, M., "Program KARMA - 3-D Nonlinear Static and Dynamic Failure Analysis Program Version ISEC-07", ISEC, Inc., San Francisco, CA., 1993.

2. GIDWANI, JAWAHAR, M., "Program SWAMI - 3-D General Purpose Visualization and Animation Program Version 3.2", ISEC, Inc., San Francisco, CA., 1993.

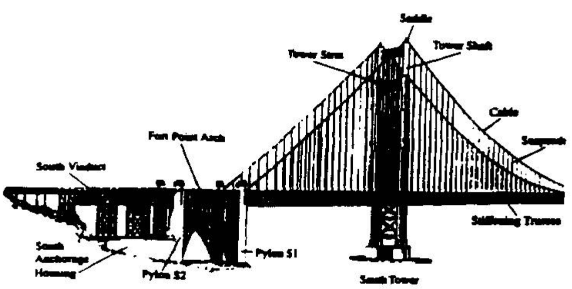
3. "Golden Gate Bridge Design Criteria for Seismic Retrofit Measures", T. Y. Lin International in Association with Imbsen and Associates, Inc., Geospectra, Inc., and MacDonald Architects, November 2, 1992.

4. GIDWANI, JAWAHAR, M., "Chapter 10 - Nonlinear Inelastic Analyses Guidelines for Steel Structures", Addition to Golden Gate Bridge Design Criteria for Seismic Retrofit Measures, January 6, 1994.

5. "Incoherency and Spectra Compatible Ground Motions for San Andreas Events SA1, SA2 and SA3", Geospectra, Inc., November 22, 1993.

Mode	1	2	3	4	5	6	7	8	9	10
Period	1.10	.768	.566	.558	.445	.402	.375	.369	.312	.309

Table. 1 Golden Gate Bridge - Fort Point Arch Eigenvalues



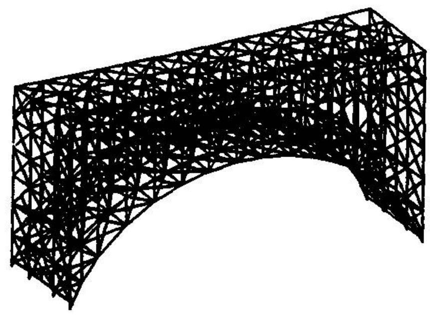
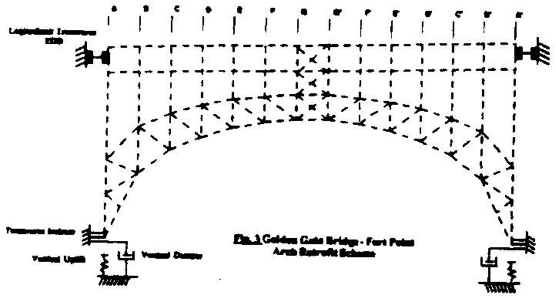


Fig. 2 Golden Gate Bridge - Fort Point Arch - Nonlinear KARMA Model.



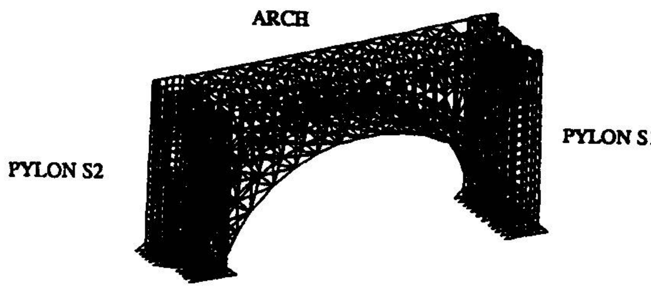


Fig. 4 Golden Gate Bridge - Fort Point Arch/Pylons - Nonlinear KARMA Model.

Seismic Retrofit of the South Approach Viaduct of the Golden Gate Bridge

Consolidation parasismique du viaduc sud du pont de Golden Gate

Erdbebenertüchtigung der Südzufahrtsrampe der Golden-Gate-Brücke

Thomas C. JEE
Chief Bridge Engineer
Sverdrup Civil, Inc.
Walnut Creek, CA, USA

Marvin YOUNG
Bridge Engineer
Sverdrup Civil, Inc.
Walnut Creek, CA, USA

Mervin C. GIACOMINI
District Engineer
Golden Gate Bridge District
San Francisco, CA, USA

Ewa Z. BAUER
Deputy District Engineer
Golden Gate Bridge District
San Francisco, CA, USA

SUMMARY

This paper examines the thinking behind the Golden Gate Bridge South Viaduct seismic retrofit. It discusses the analytical and design issues involved and explains the current retrofit strategy and the reasons behind it. It presents a broad overview of the analysis results and a prediction of the south viaduct's structural behavior. The paper exposes the design bases and assumptions involved. It also illustrates the importance of integrating analysis, design and detailing with other design criteria such as reliability, aesthetics, constructibility, serviceability, and economics.

RÉSUMÉ

L'article présente les réflexions qui ont conduit à la consolidation parasismique du viaduc sud du pont du Golden Gate. Il présente des questions de projet et d'analyse et explique la stratégie actuelle de la consolidation et les raisons qui y ont conduites. Les résultats de l'analyse sont présentés de façon générale, ainsi qu'une prédiction du comportement structural du viaduc du sud. L'article expose les bases du projet et les hypothèses faites. Il illustre également l'importance de la considération globale de l'analyse du projet et des détails constructifs en fonction d'autres critères de projet tels que fiabilité, esthétique, possibilité de réalisation, aptitude au service et aspects financiers.

ZUSAMMENFASSUNG

Der Beitrag untersucht das Konzept für die Erdbebenertüchtigung der südlichen Golden-Gate-Vorlandbrücken im Hinblick auf Berechnungs- und Bemessungsfragen. Nach einem Ueberblick über die Ergebnisse der analytischen Vorhersage des Brückentragwerkverhaltens werden die Bemessungsgrundlagen und -annahmen dargelegt. Dabei wird deutlich, wie wichtig die Integration von Berechnung, Bemessung und konstruktiven Detaillösungen mit anderen Kriterien wie Zuverlässigkeit, Aesthetik, Bauvorgang, Gebrauchstüchtigkeit und Wirtschaftlichkeit ist.

1. INTRODUCTION

The majestic Golden Gate Bridge guards the only entrance to the beautiful San Francisco Bay in Northern California. This historical landmark is bounded by Marin County in the north and San Francisco in the south. The 9151 foot long - six lane bridge is composed of four main structures, namely the South Approach Viaduct, the Fort Point Arch, the Main Suspension Span, and the North Approach Viaduct. The bridge joins San Francisco and Marin County and spans the only entrance into the San Francisco Bay. This paper concentrates on the South Approach Viaduct retrofit strategy from its initial development to final PS&E.

The South Approach Viaduct is a 700 foot long bridge consisting of three 70'-0" long girder spans and three truss spans (2@125'-0" & 175'-0") supported on pin steel bearings (See Figure 1). It lies between the south abutment on the San Francisco side to the south face of Pylon S2. The south viaduct supports a 62'-0" roadway with 11'-6" sidewalks on either side. The girder spans are simply supported by the south abutment, two steel planer bents 9 and 10, and bent 8 which is integral with Tower 1. The three truss spans are simply supported on three steel Towers 1,2, & 3 and Pylon S2. Bents 9 & 10 and Tower 1 are supported on concrete pedestals with no piles. Similarly, towers 2 and 3 rest on concrete frame towers which are housed within the Main Suspension Cable south housing structure. During the 1980's, the original concrete deck was replaced with a lighter and stronger orthotropic steel deck system supported on the original floorbeam pedestals. This new steel orthotropic deck system dramatically reduced the overall weight of the structure as a whole.

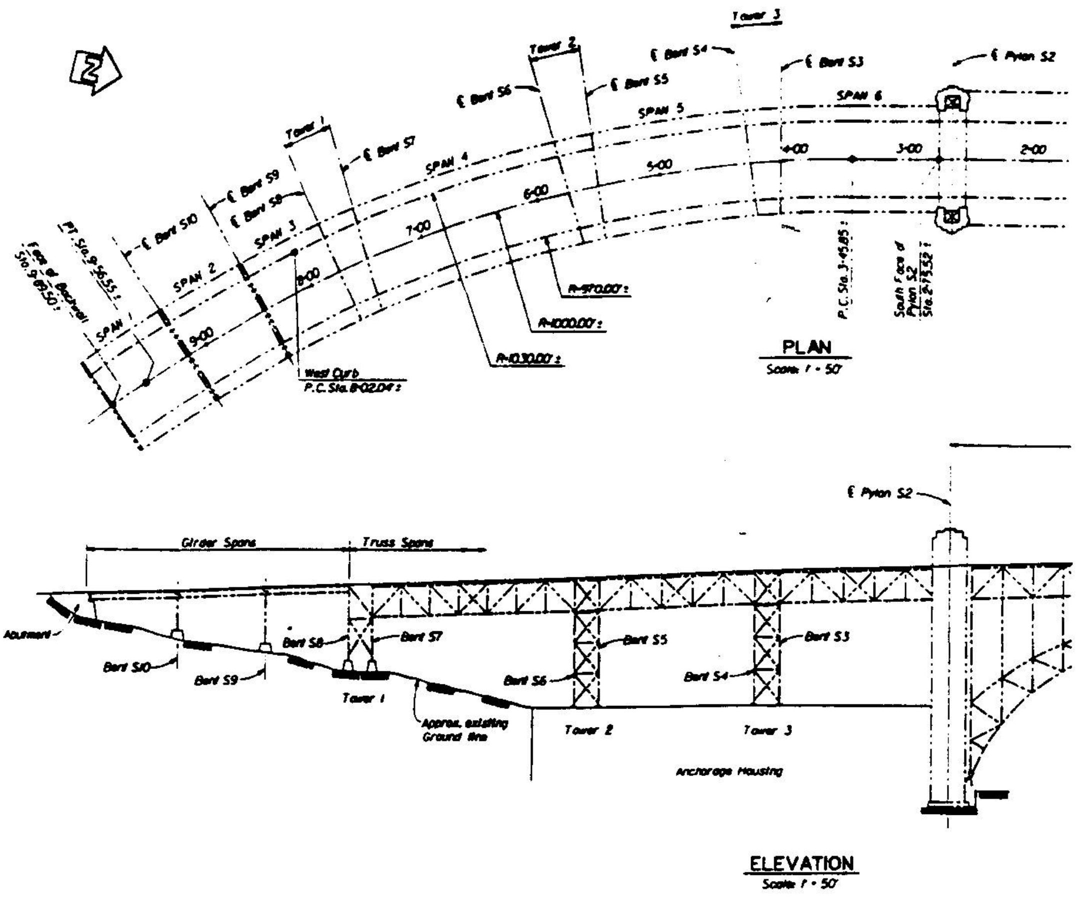


FIGURE 1 - SOUTH VIADUCT GENERAL PLAN

In the early 1980's, the Golden Gate Bridge District performed a preliminary and less thorough seismic retrofit of the Golden Gate bridge which included the installation of uplift tiedown cables at all supports, a supplementary bearing seat extension at the Pylon S2 interface, and tying all adjacent spans together with restrainer rods. These retrofit items proved beneficial as demonstrated during the 1989 Loma Prieta earthquake. However, it is believed that such a simple retrofit would not suffice should the bridge be subjected to an earthquake of much larger magnitude. The lessons learned from this retrofit did however provide some insight into how the structure behaves and what benefits certain retrofit improvements can provide.

The current retrofit scheme involves incorporating a series of improvements to help maintain serviceability before and after a large quake. The scheme includes replacing all the existing pinned steel rocker and fixed bearings with isolation bearings, installing additional shear connectors between the deck and floorbeams, linking all adjacent spans together to provide axial continuity, providing isolation joints at the south abutment and at the Pylon S2 interface, replacing the existing towers 2 and 3, and additional joint and member strengthening as deemed necessary.

2. ANALYSIS AND RETROFIT SCHEME

The intent of the Golden Gate Bridge Seismic Retrofit Project is to ensure that the bridge maintains its function after a major earthquake, termed the maximum credible earthquake, registered as 8.3 on the Richter scale. This is comparable to the infamous 1906 San Francisco earthquake. To gain a more realistic insight into the bridge behavior, site specific ground motion time histories were developed along the bridge based on the maximum credible events at the San Andreas fault.

Extensive linear, longhand, and nonlinear analysis was used to adequately predict the overall behavior of the existing structure. Initial runs showed high demand forces in the superstructure resulting in extensive strengthening and increased retrofit costs. Also with the existing bearings, a large uplift force developed resulting in a global overturning problem. For this reason, the concept of brute force strengthening was abandoned as a possible retrofit scheme. In order to reduce these high demands and the overturning problem, all the existing bearings were replaced

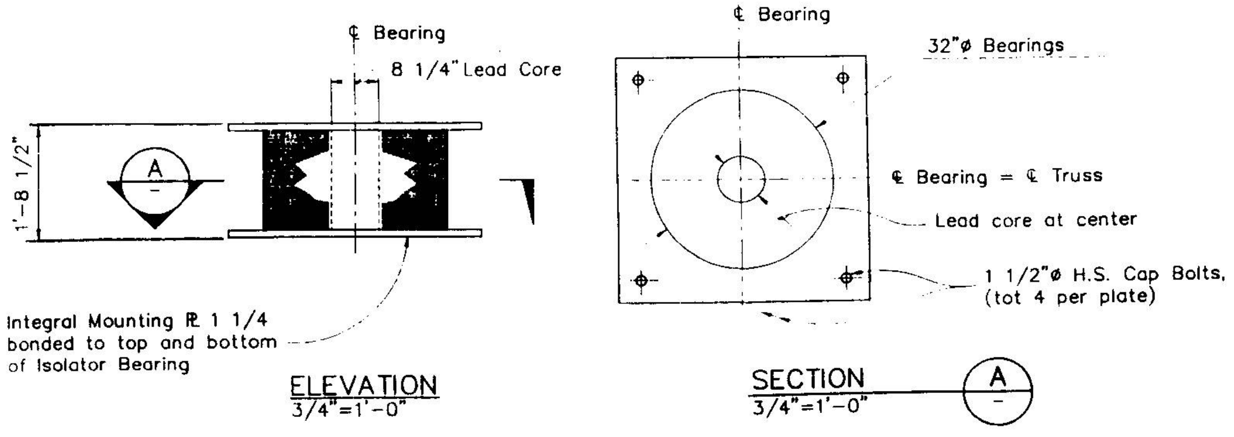


FIGURE 2 - ISOLATION BEARING DETAIL

with isolation bearings (See Figure 2). By doing so, the superstructure force acceleration was dramatically reduced to 0.3g, and the amount of strengthening required was reduced as well. With this lower force acceleration, overturning was no longer a problem. However, the benefits of reduced seismic forces through isolation bearings does have one disadvantage. Large displacements in the order of 15", characteristic of isolator bearings, occurs which will need to be accounted for at the boundary ends. The existing structure currently has only 1" temperature joints at the abutment and at the Pylon S2 interface.

Although the ground motions used in the analysis were thoroughly researched and investigated within the limits of present day technology, it is rather uncertain to predict the type and magnitude of earthquake that will occur and where. In fact, no one can predict with any degree of certainty what type or strength of earthquake will occur or how the structure will actually behave. With this in mind, secondary safety measures were incorporated into the retrofit design to provide backup safety measures in the event the primary system were to fail. A good example of this is the isolation bearings. Although the bearings were designed and sized for the design earthquake, there is a possibility that the bearing will be pushed beyond its ultimate displacement capacity resulting in failure of the bearing and support as a whole. Therefore, supplemental catcher/support blocks are added to the structure at all bearing locations that serve to support the superstructure and prevent complete span failure should the isolators fail. These blocks are positioned to allow free movement of the isolators and provide secondary support in both the transverse and longitudinal directions. This provides a much needed additional level of protection for such an important bridge.

Like all bridge structures, the majority of the structure mass is in the deck. Therefore, it is essential that the deck mass have an adequate load path in order to safely ground the seismic forces. In depth analysis showed that the existing deck supporting pedestals are inadequate to transfer the deck mass shear to the floorbeams. Therefore, a series of shear connectors were added between the deck and the floorbeams to ensure full mass transfer.

Currently, the individual girder and truss spans are independent with no continuity between adjacent spans. Retrofit analysis found that linking all the spans together proved more beneficial from a seismic standpoint. For one, linking adjacent spans together reduces the possibility of a drop span condition similar to what occurred on the San Francisco-Oakland Bay Bridge during the Loma Prieta Earthquake of 1989. In addition, linking spans allows the bridge to behave as one unit resulting in improved behavior during strong out of phase seismic movements. With the use of isolation bearings, temperature movement is accounted for in the bearings. Therefore, linking adjacent spans does not cause a concern from a thermal standpoint.

Interaction between adjacent structures has always been a concern when performing analysis. For this particular situation, the interaction with the adjacent Pylon S2 was a concern. With this in mind, the computer model was modified to include Pylon S2. Pylon S2 is a massive 240 feet tall lightly reinforced concrete hollow structure with a narrow 32'-0" base longitudinally and a 120'-0" base transversely. The current Pylon retrofit strategy calls for partially tying down the base while allowing some minimal amount of uplift to reduce the seismic forces. The approach viaduct rests on pedestals built within Pylon S2 and is therefore influenced by the Pylon behavior. With its narrow base, the pylon longitudinal displacement is relatively large. This displacement coupled with the viaduct's longitudinal displacement warranted the need for a relatively large isolation deck joint since out of phase movements could result in the two structures moving away from each other or together. The joint was designed and detailed for both longitudinal and transverse movements since the isolator bearing acts in both directions (See Figure 3). At the abutment, a similar size and joint type was necessary. These large deck joints will ensure free movement of the structure and thus make full use of the isolation bearings.

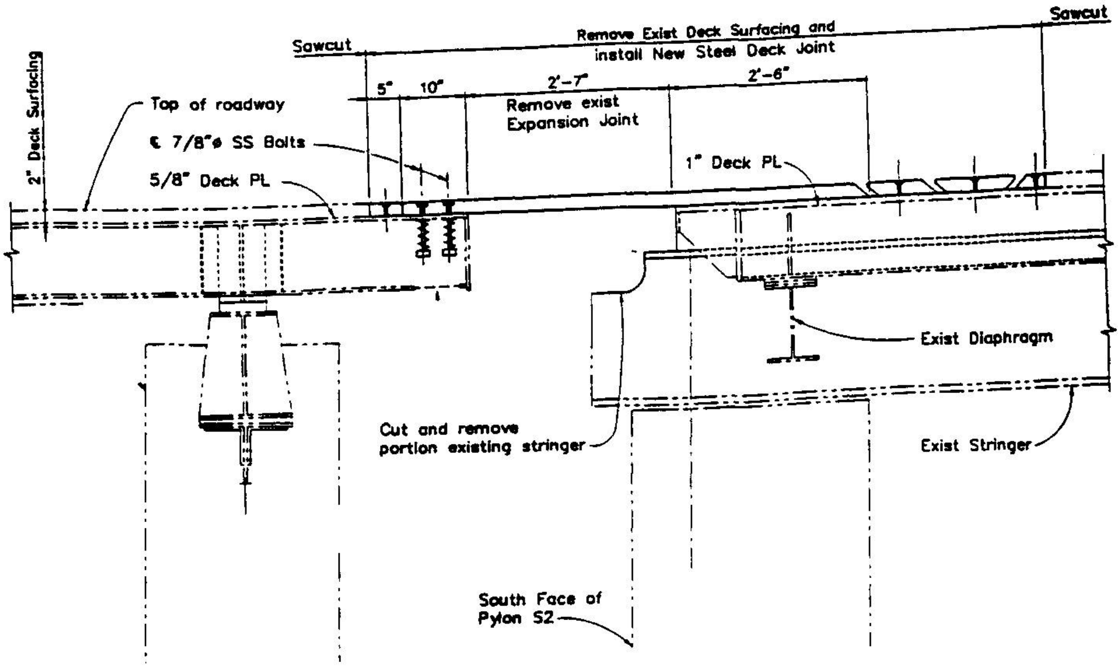


FIGURE 3 - TYPICAL ISOLATION JOINT SECTION

As discussed, the isolation bearings aid in reducing the seismic input into the structure, but there is one limitation. The limitation is that isolation bearings can resist little or no uplift force. The project specific criteria mandates that the vertical acceleration be included in the analysis and accounted for in the design. This creates a dilemma in that the superstructure must be tied down to reduce the vertical seismic force component while at the same time allowing full longitudinal and transverse cyclic isolator displacement. A detail involving standard restrainer cables with lengths that will permit the required horizontal displacement was devised. These cables were prestressed to the required uplift force to ensure their effectiveness immediately without any initial elongation.

Analysis of the viaduct demonstrated that the existing towers 1,2, and 3 remained elastic throughout the design earthquake. With this in mind, it seems logical to keep the existing towers and make strengthening provisions where necessary. However with the close proximity to the harsh salt water spraying environment, the original 1935 constructed towers 2 and 3 are severely corroded with rust. Therefore, the validity of using the original net section was in question. Continued use of these towers would result in on-going steel deterioration as well as a reduction in overall tower strength. A detailed life cycle cost comparison was performed on whether it would be more economical in the future to keep, upgrade, and maintain the existing towers or to replace them altogether. The comparison indicated similar costs with each alternative. Therefore, it was found more beneficial economically and structurally to replace both towers 2 and 3. Tower 1 is in better condition with little or no corrosion. Because of this, it was recommended and approved to keep the existing tower 1 structure. The north approach viaduct retrofit involved a similar scheme with replacement of all the steel towers as well.

Throughout the course of the project, the understanding that the original bridge appearance must not be altered was maintained. With Sverdrup Civil, Inc as the prime consultant on this project, we hired the services of a historical preservationist to oversee, review, and suggest ways in which structural retrofit details could be incorporated into the structure without altering the bridge's original historic appearance. Items such as the use of perforated plates to simulate the original member lacing were scrutinized and reviewed as the issue of whether strength or historical preservation was most important.

3. CONCLUSION

The seismic retrofit of the Golden Gate Bridge offered engineers a chance to use their creativity in analyzing, designing, and detailing ways to aid the structure in surviving the next large earthquake. The key retrofit item was the installation of isolation bearings at all support points. This single item dramatically reduced the forces in the entire structure thus requiring less retrofit throughout the structure. Large isolation joints at the boundaries were one downfall to the isolation bearings but were well worth it in exchange for reduced seismic forces. The replacement of the supporting steel towers demonstrates the true concept of value engineering. Life cycle cost comparisons, as shown here, illustrate the importance of careful investigation of retrofit alternatives in order to save money and provide the required structural aspects. Severe corrosion of the existing towers was a major contributor to the decision to replace them. Linking of adjacent simply supported spans was also a major refinement in that it lessened the possibility of a drop span condition and improved the overall structure behavior for out of phase movements.

The south viaduct is as important a structure as any other portion of the bridge. Without the survival of any one portion of the bridge, the bridge would have to be closed. Therefore, the south viaduct is a critical transportation link to the main suspension span and is equally important. We believe we have investigated and devised a scheme that will improve the viaduct's overall behavior, prevent collapse, and maintain serviceability.

REFERENCES

1. "Golden Gate Bridge Design Criteria for Seismic Retrofit Measures", T.Y.Lin International in Association with Imbsen and Associates, Inc., Geospectra, Inc., and MacDonald Architects, November 2, 1992.

2. "Incoherency and Spectra Compatible Ground Motions for San Andreas Events SA1, SA2, and SA3", Geospectra, Inc., November 22, 1993.

3. "Golden Gate Bridge South Viaduct Retrofit Strategies Report", Sverdrup Corporation in Association with Isec, Inc., and Buckland and Taylor, Ltd., December 14,1993.

Extending the Lifespan of the North Anchorage Housing of the Golden Gate Bridge

Prolongement de la durée de vie de l'ancrage nord du pont de Golden Gate

Verlängerung der Gebrauchsdauer der nördlichen Seilverankerung der Golden-Gate-Brücke

Fay BERNSTEIN
Principal
Faye Bernstein & Assoc.
San Francisco, CA, USA

Faye Bernstein received her architectural engineering degree from the Univ. of Miami, Florida. She has over 25 years of structural engineering experience on major projects including bridges, industrial facilities, high-rise buildings and historic landmark structures. She has been principal of Faye Bernstein and Associates for 11 years.

Michael O'SULLIVAN
Engineering Manager
Faye Bernstein & Assoc.
San Francisco, CA, USA

Michael O'Sullivan received his B.S. and M. Eng. degrees from the University of California, Berkeley. He has over 20 years of Structural Engineering experience on a wide variety of major projects including bridges, marine, heavy construction, both fixed and floating offshore oil platforms, industrial facilities and buildings.

Jerry CAO
Senior Civil Engineer
Golden Gate Bridge District
San Francisco, CA, USA

Jerry Cao received his B.S.C.E. degree from National Taiwan Univ., and M.S.C.E. and Ph.D. degrees from Univ. of Notre Dame, Indiana. He is a Project Engineer on the Golden Gate Bridge Seismic Retrofit Project, responsible for project coordination of the suspension spans and north structures.

SUMMARY

The retrofit process of the North Anchorage Housing began with the study of five structural systems. In addition to the design criteria, several other design constraints were imposed: preserving the historic character, providing an open corridor for a future light rail system, and maintaining continual service of the bridge. The final design provides new lateral and vertical support for the structure within the existing housing shell and includes a new roadway deck. By incorporating the new deck into the seismic retrofit of the Housing, significant savings were realized in the cost of the retrofit work.

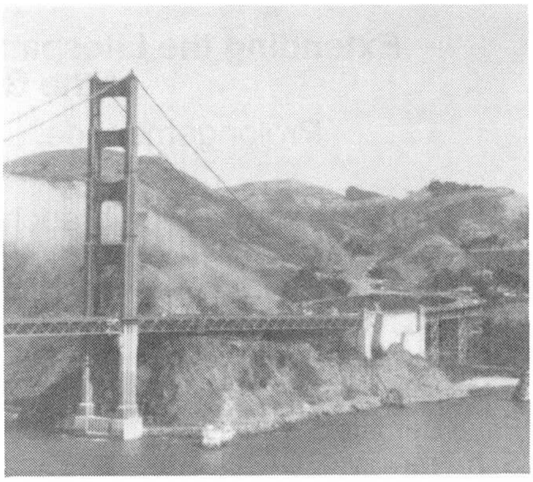
RÉSUMÉ

Le processus de consolidation de l'ancrage nord a débuté par l'étude de cinq systèmes structuraux. Outres les critères de projet, plusieurs autres contraintes étaient imposées, préserver le caractère historique, garder un espace libre pour un futur système léger de transport ferroviaire et maintenir le pont en service pendant les travaux. Le projet final prévoit un support latéral et vertical pour la structure à l'intérieur de l'ancrage actuel et comprend un nouveau tablier routier. En réalisant le nouveau tablier en même temps que la consolidation parasismique de l'ancrage, des économies importantes ont été réalisées dans l'ensemble des travaux de consolidation.

ZUSAMMENFASSUNG

Bei der Ertüchtigung des nördlichen Seilverankerungsblocks waren neben den Bemessungskriterien etliche andere Randbedingungen zu beachten: der historische Charakter des Bauwerks, die Freihaltung eines Korridors für eine zukünftige Leichtschnellbahn und das Bauen unter Verkehr. Von fünf untersuchten Tragsystemen wurde eines gewählt, das innerhalb der bestehenden Hülle zusätzliche seitliche und vertikale Stützung verleiht und eine neue Fahrbahnplatte beinhaltet. Dadurch konnten bedeutende Einsparungen bei der Sanierung realisiert werden.

1.0 BACKGROUND



The North Anchorage Housing is located on the side of a mountain on the Marin side of the bridge's main span. Uphill on the west side lie the Marin headlands of the Golden Gate; downhill on the east side is the shore of the San Francisco Bay. Functionally the housing encloses and protects the main bridge cables where the cable strands splay out to their attachment to mass concrete anchor blocks. In addition, its roof forms the highway roadbed between the north end of the suspension bridge and the north viaduct structure. In contrast to the lightness of the steel main span, the housing is a massive-looking reinforced concrete building-type structure, measuring 107 meters long and 40 meters wide. Its height varies from 18 to 34 meters due to the sloping site on which it is located. Interior support for the roadway is provided by a series of reinforced concrete bent frames in the transverse direction. Perimeter walls of the housing as well as the roadway and walkways are of reinforced concrete construction. The roadway and walkway deck consist of a simple span system of longitudinal stringers with integral slab spanning 7.6 meters between transverse frames. The south side of the housing is supported on spread footings which bear on native rock, while the north is supported on the anchor blocks. The anchor blocks, in turn, are founded on and keyed into the native rock.

2.0 RETROFIT APPROACH

A state of the art seismic retrofit is being performed on the bridge and a project specific retrofit criteria has been developed. The site specific response spectrum shown in figure 1 is being used for the retrofit with a demand/capacity analysis similar to current AASHTO requirements.

A number of retrofit options and structural systems were studied for the anchorage housing and were evaluated on the basis of technical merit and cost. These options included:

A) Base Isolation
B) Ductile Concrete Frames
C) Concrete Shearwalls
D) Hybrid Systems

Initially all of these options considered retrofitting the existing housing and maintaining the concrete deck.

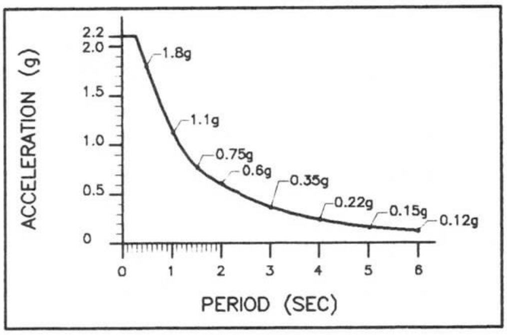
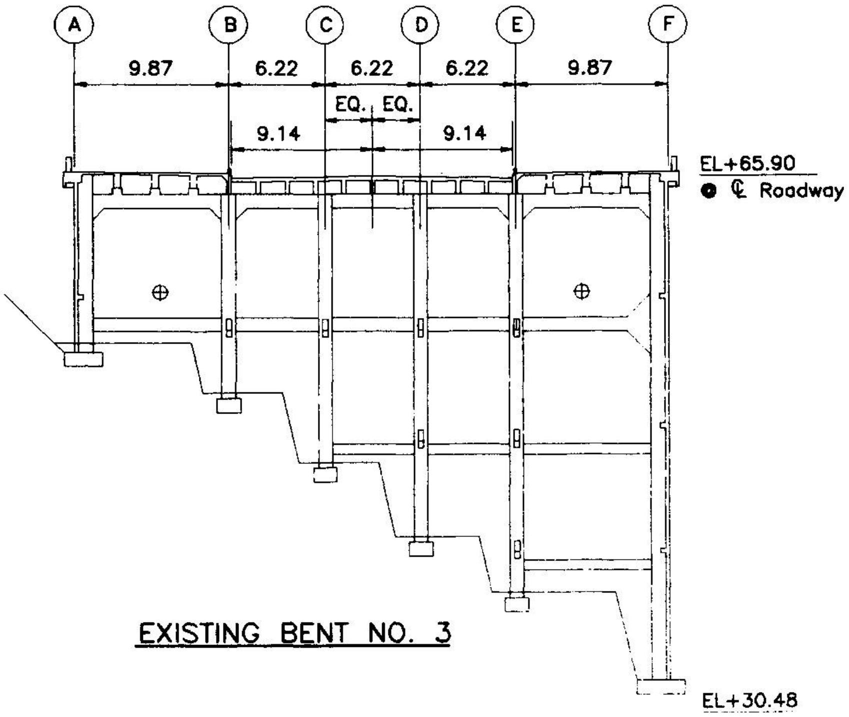


FIGURE 1

3.0 DESIGN CONSTRAINTS

Three major design constraints were imposed on the designer of the retrofit system. First, because of the historic character and appearance of the bridge, the exterior appearance of the housing could not be altered by the structural retrofit work. While appearance was a major constraint, the project criteria required, however, that seismic safety not be compromised. Second, the project criteria requires that the bridge remain serviceable after a maximum credible seismic event with little or minor repairable damage. As a result of this second constraint, combined with the location of the housing between two different structures, both with damping systems, seismic displacements had to be controlled and kept at manageable limits. The third constraint was to provide an open corridor beneath the roadway for a future light rail system.

BENTS ON NATIVE ROCK AT SOUTH SECTION



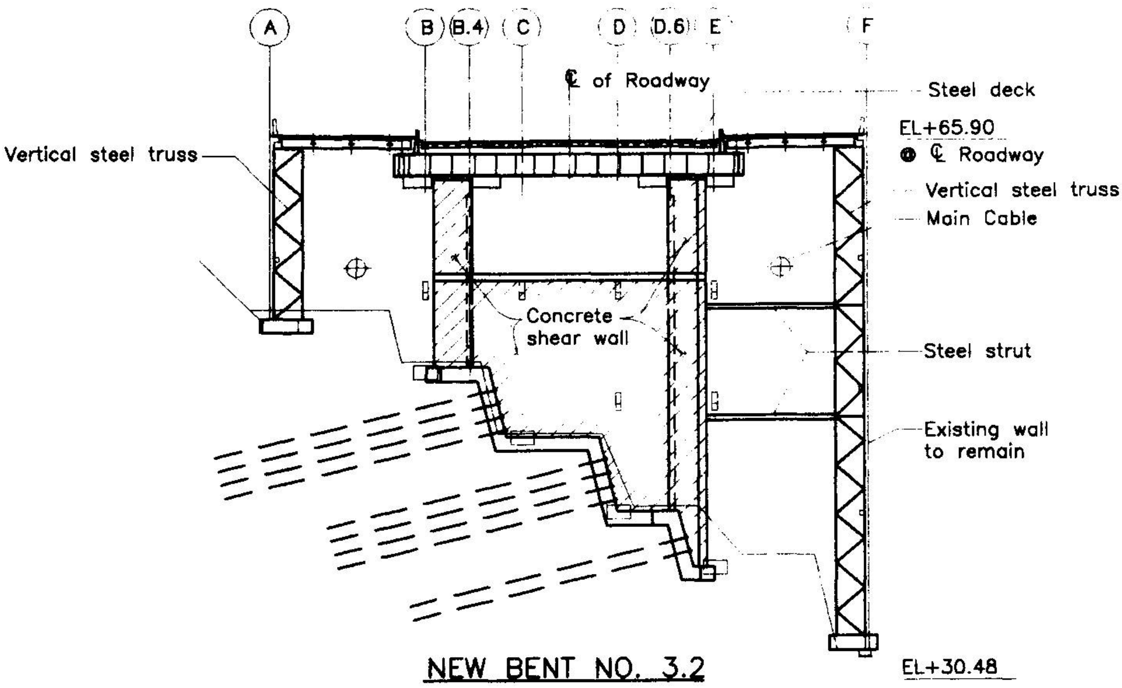


FIGURE 2

4.0 DESIGN CHALLENGE AND COMPLEXITY OF RETROFIT

At project start, retrofit of the housing appeared to require simple straight-forward modifications to a box type structure. But as the design process evolved, the housing retrofit became more complex. One reason for the complexity was the configuration of the housing. Vertical loads are supported on a series of transverse bents. The bents on the south end of the housing rest on the native rock while the ones on the north end are supported on the mass concrete anchor blocks. When the potential for rocking of the anchor blocks was identified, the housing retrofit required division of the housing into two distinct structures; north and south. The south end included the structure founded on native bedrock materials south of the anchor block region. The north end included the structure atop the anchor blocks. Since the east and west anchor blocks are not connected, they are free to rock independently under seismic excitation. As a result of this rocking potential, the north end was further divided into east and west structural sections. The structural separation was to be accomplished by providing new transverse roadway girders spanning between blocks, detailed to allow for differential movement.

Another reason for the complexity of the retrofit was the existing deck. The Bridge District had identified deck replacement as necessary but lacked available funds and understood that construction challenges made replacement impossible. The deck had been repaired several times but was not replaced when the bridge was redecked about 10 years ago. Modifications were performed in the 1950s and 1980s to resupport deck stringers which were cracking at their bearing locations. The modifications included installation of miscellaneous metal at every stringer and every bent. As a result of these repairs, the existing roadway stringers are now supported on a system of suspended bearings outward of the original bearings. In addition, a series of suspended access catwalks had been installed to allow inspection of the under roadway deck stringers. Working the retrofit around these obstacles proved very difficult. Further complexity of the retrofit was caused by the deck stringer expansion bearings which make the roadway deck discontinuous at 25 foot intervals. Numerous collectors, restrainers, and specialized details were required in the retrofit to provide a positive path for seismic forces and to control seismic displacements.

5.0 RETROFIT COSTS ALLOW FOR NEW VERTICAL SUPPORT AND DECK REPLACEMENT

The original retrofit scope included strengthening of the housing to resist lateral loads. Vertical loads were carried by the existing bents and deck replacement was not included. When the potential for rocking of the anchor blocks was identified, it became necessary to replace the transverse deck girders above the anchor blocks. The retrofit concept was then modified to provide one structural system at the south portion of the housing and another in the north.

As the design continued to develop it became increasingly apparent that a high premium was being paid to retain the deck and work around the miscellaneous steel from previous repairs. The cost of deck replacement alone was previously prohibitive; but deck replacement in conjunction with the seismic upgrade of the housing became viable. However, the deck could only be replaced with a construction procedure which would allow use of all traffic lanes during daytime hours and would not increase construction costs.

Faye Bernstein & Associates, working together with Ed K. McNinch & Associates, developed a redecking approach within the established design constraints. The new scheme provides a new vertical support system and a new deck at no additional cost than retrofitting the existing structure alone. Cost reduction was accomplished by elimination of shoring and costly retrofit items such as restrainers needed to retrofit the existing concrete roadway. In addition, the weight reduction in the deck created a cost savings in the supporting structure. For each kilogram of weight the structure had to resist about two kilograms of lateral load. With the new roadway support structure and deck installed, the historic appearance of the structure remains unchanged.

The proposed deck replacement will be done at night. To temporarily retain the existing verticalsupport system, new bents adjacent to existing ones will first be installed. The new bents allow for easy removal of existing deck sections while providing a new modern support structure with simple seismic details. Each night a portion of existing roadway deck will be removed,

BENTS ON ANCHOR BLOCKS AT NORTH SECTION

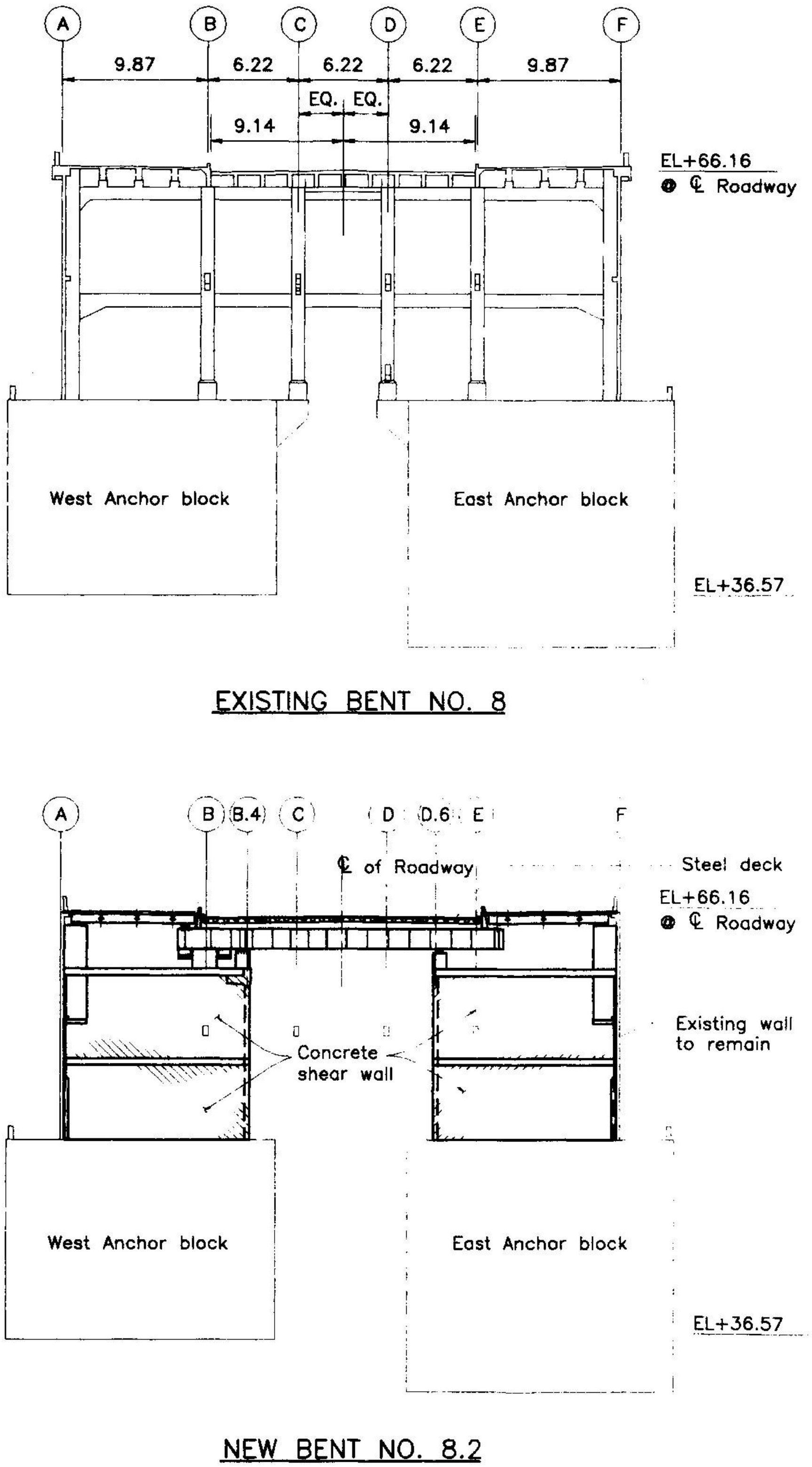


FIGURE 3

unloading the existing bents, and will be replaced with a new segment of steel orthotropic deck panel, supported on the new transverse bents.

6.0 STRUCTURAL SYSTEM

The retrofit design provides new transverse bents offset 1.5 meters from the existing bents. The new bents will ultimately provide both lateral and vertical support for the structure. New shear walls in the longitudinal direction augment the support provided by the existing exterior walls. The exterior walls are retrofitted to increase in-plane and out-of-plane capacity. The structural systems on the north and south portions of the housing differ slightly. The south system, founded on native rock, consists of hybrid frames in the transverse direction centered on the roadway, and shear walls in the longitudinal direction. The north section consists of shear walls in the transverse direction above each anchor block with deep roadway support girders spanning between the walls. Shear walls are used in the longitudinal direction to resist lateral loads.

The new walls terminate in wide horizontal beams about 3.5 meters below the deck. The beams span between perpendicular walls and will be utilized as catwalks. The reasons for the lowered walls include reduction in weight, reduction in lateral loads to be transferred by the new deck, ease of construction before redecking, and open access below the deck during and after construction.

Exterior walls in the longitudinal direction are retrofitted to increase their in-plane and out-of-plane capacity. Vertical steel trusses will be installed to brace the walls out of plane. Some shotcrete will be added to the bottom portion of walls to increase in-place shear capacity.

7.0 CONCLUSION

The seismic retrofit process has produced an upgrade which extends the lifespan, increases seismic safety and reduces maintenance, while maintaining the historic appearance of the Golden Gate Bridge. A cooperative effort between the engineering staff of the Golden Gate Bridge District and structural consultants, has resulted in a design which best serves the long term operational needs of the District. A strength of the design process was maintaining flexibility to incorporate new information as it developed. Two milestone pieces of information which significantly altered the design were the identification of the potential for anchor block rocking and the cost control which identified deck replacement as viable. The flexibility in the design process incorporated the new deck into the seismic retrofit work; and as a result, reduced the cost of the seismic retrofit.

Seismic Retrofit of the North Approach Viaduct of the Golden Gate Bridge

Consolidation parasismique du viaduc nord du pont de Golden Gate

Seismische Nachrüstung des North Approach Viaduct der Golden-Gate-Brücke

Roy A. IMBSEN
Civil Engineer
Imbsen & Assoc., Inc.
Sacramento, CA, USA

Roy A. Imbsen has 32 years of experience in bridge design and structural engineering and is nationally and internationally recognized for his expertise in the field of seismic design of highway bridges.

Robert A. SCHAMBER
Civil Engineer
Imbsen & Assoc., Inc.
Sacramento, CA, USA

Robert A. Schamber has 15 years of experience in bridge design and structural engineering. He has been project manager for several retrofitting jobs in California and other states.

Gerald V. DAVIS
Structural Engineer
Imbsen & Assoc., Inc.
Sacramento, CA, USA

Gerald V. Davis is a Senior Bridge Engineer. He has 28 years of experience in structural engineering. He has been lead designer for several retrofitting jobs.

SUMMARY

This paper describes the seismic retrofit of the North Approach Viaduct of the Golden Gate Bridge. A three-dimensional model of the complete viaduct was developed. Both the response spectrum analysis using 100 modes and time history analysis were carried out for the as-built and retrofitted structure. Three earthquake input components were considered. The retrofit schemes included replacing the existing towers with new towers, replacing steel bearings with isolation bearings, replacing the existing footings with new pedestals and footings with tiedowns, providing an expansion joint device at the Pylon and North Abutment, and replacing and strengthening superstructure bracing members.

RÉSUMÉ

Cet article décrit la rénovation parasismique du viaduc nord du pont de Golden Gate. Un modèle tridimensionnel du viaduc complet a été établi. Trois cas de séismes ont été considérés. Les projets de rénovation comprenaient le remplacement des tours existantes par de nouvelles tours, le remplacement des appuis d'acier par des appuis isolants, le remplacement des plaques de base par de nouveaux socles et fondations avec ancrage, l'introduction d'un assemblage à joint de dilatation au pylône N2 et à la culée nord, et le remplacement ou le renforcement d'éléments de la superstructure.

ZUSAMMENFASSUNG

In diesem Artikel wird die seismische Nachrüstung des North Approach Viaduct der Golden-Gate-Brücke behandelt. Ein dreidimensionales Modell des gesamten Viadukts wurde erstellt. Die Nachrüstungspläne beinhalteten das Auswechseln der vorhandenen Pylonen gegen neue, Austausch der Stahlauflagerungen gegen Isolierungsauflagerungen, Auswechseln der vorhandenen Fundamente gegen neue Sockel und verankerte Fundamente, Erstellung eines Dehnungsfugensystems am Pylon N2 und North Abutment (nördliches Widerlager) sowie den Austausch und die Verstärkung der Oberbauverstrebungsteile.

1. DESCRIPTION OF THE EXISTING NORTH APPROACH VIADUCT

The North Approach Viaduct of the Golden Gate Bridge extends from the north end of the North Anchorage Housing to the North Abutment. It consists of five truss spans of 53.3m each on partially straight and horizontally curved alignment. These truss spans are supported on four intermediate steel braced frames, on Pylon N2 on the south end, and on the North Abutment. The braced frame tower heights are about 15.2 m at Tower 1, 39.6 m at Tower 2, and 45.7 m for Towers 3 and 4. The original construction of the viaduct consists of two parallel trusses spaced 15.2 m apart. Each braced frame tower has 4 vertical legs and plan dimensions of 15.2 by 15.2 m. As part of a 1961 widening project, an additional line of trusses and two additional vertical legs at each tower were added. These new members are located 25 feet west of the original structure. The towers are supported on concrete pedestals and spread footings at the original bridge and caissons at the widening.

In 1982, the viaduct was retrofitted to the then-current Caltrans seismic retrofit guidelines. The following details were used: 1) threaded bar restrainers oriented in the longitudinal direction were installed at bearing supports to prevent excessive relative displacement; 2) transverse restrainers were installed to tie the truss spans to the supporting towers; 3) additional diagonal bracing members were added to the widening portion of the towers and at the lower portion of the original towers; and 4) foundations at the tower legs of the widening section were strengthened to provide stronger anchorage. During the early 1980s, the original concrete deck was replaced with a lightweight orthotropic steel deck system.

2. ANALYTICAL MODEL AND ANALYSIS METHOD

2.1 General

The seismic evaluation and retrofit strategy determination were based on the linear response spectrum method, the linear elastic time history analysis method and the nonlinear time history analysis method. The computer program IAI-NEABS, which is an enhanced version of NEABS (Nonlinear Earthquake Analysis of Bridge Systems), was used for these studies [1].

2.2 Linear Elastic Analysis

A three dimensional model of the complete viaduct was developed to perform a linear elastic time history analysis and linear elastic response spectrum analysis. Several models were used in the linear elastic response spectrum analysis and the linear elastic time history analysis of the as-built structure. The vulnerable components were identified as follows:

1. Restrainers,
2. Abutment connections,
3. Foundations (anchor bolt uplifting),
4. Bearings (rocking response of the tall bearings), and
5. Tower braces and column members.

The behavior of these components were modeled as nonlinear and/or inelastic elements in subsequent runs.

Various retrofit schemes were modeled using equivalent linearized parameters for various components included. The analyses were carried out either by the linear time history analysis method or linear elastic response spectrum analysis method.

2.3 Nonlinear Analysis

A three dimensional nonlinear model of the complete viaduct was developed. The nonlinear time history analysis was performed using the step-by-step direct integration of the coupled equations of motion. At the end of each time step, the equilibrium condition was checked. If the maximum unbalanced residual force was too high, the program automatically reduced the integration time increment by subdivision. Within each subdivision step, further equilibrium iterations were performed to assure accuracy of the solution.

The nonlinear process consisted of an iterative or "piece meal" process using the initial linear analysis and subsequent nonlinear analyses to establish the basis for the model refinement in the next level of the nonlinear model and analysis (i.e., identifying members that should be modeled as nonlinear in the next run). This process allowed for an early retrieval of data and a review of the structural responses of the adjusted model for each run. With a clear understanding of the limitation of each model and the resulting responses, engineering judgments were made regarding the adequacy of the "partial nonlinear" model for the intended purpose of the analysis (i.e., to identify the vulnerability and the potential retrofit scheme).

As the process of model refinement continued, the effect on the structural responses caused by the modification of the model gradually decreased. Eventually, this iterative nonlinear analysis was stopped when either all members were modeled as nonlinear or no additional nonlinear response were expected.

3. SEISMIC DEFICIENCIES

The following is a description of deficiencies in various components throughout the structure.

3.1 Superstructure

Many truss members need to be reinforced or replaced to meet strength (kl)/r and b/t ratio requirements. The capacity of existing longitudinal and transverse restrainers must be increased. Existing deck joints are inadequate for the anticipated lateral displacements. Truss longitudinal connections to Pylon N2 and North Abutment are inadequate.

3.2 Bearings

Existing steel bearing anchor bolts fail in tension creating possible instability and unacceptable vertical displacements. Bearing support widths are inadequate to ensure that trusses will be supported after the earthquake. Transverse restraint is inadequate to handle forces and displacements.

3.3 Towers

Most connections of diagonal braces and horizontal struts are inadequate to develop the computed tensile forces or the yield strength of the member. In many diagonal bracing members the allowable demand/capacity ratios are exceeded. Column elements have a demand/capacity ratio higher than 1. Anchor bolts that connect the tower base to the concrete pedestals will fail in tension and shear.

3.4 Foundations

Concrete pedestals are underreinforced and overloaded and will fail in a brittle manner. The footings provide insignificant resistance to uplift.

3.5 North Abutment and Pylon N2

The North Abutment is inadequate to absorb lateral forces and displacements. The superstructure will impact the North Anchorage Housing during lateral motions at Pylon N2 and existing longitudinal and transverse restrainers are inadequate.

4. SEISMIC RETROFIT DETAILS

Brief descriptions of the retrofit procedures are discussed below.

4.1 Temporary Supports

Temporary supports will be required at each tower to support the structure while bearings and towers are replaced. They will be designed so that the temporary towers can be reused at all four towers. New temporary foundations and strengthening of the superstructure truss will be required.

4.2 Isolation Bearings

Isolation bearings will be installed at the top of the new towers. Bearing performance will be confirmed by a comprehensive series of tests which will be performed over the life of the structure so that the effect of aging will be known. Provisions will be made for bearing replacement should that be needed. Inspection of the bearings will be straightforward, only routine visual inspection need be performed. In particular, inspection of the bearing outer cover to verify that it is free of large cracks, tears, etc. would be performed. The retrofit design will make some provision for replacement of the isolation bearings. The new jacking points were built into the bearing assembly as a part of the retrofit. The new isolation bearings at the top of the towers will serve to dissipate horizontal loads but there are also significant vertical uplift loads which must be transmitted from the superstructure to the tower. Vertical restrainers will be incorporated into the design.

4.3 Tower Foundations

The concrete foundations under each leg of the towers have minimal reinforcement. Originally it was proposed to encase each pedestal on all four sides with a properly reinforced and tied collar; however, it was found that replacement of the pedestals and footings was more cost effective. Also, because the seismic forces are generating large overturning moments, tension tiedowns are required at each tower footing.

These tower overturning moments generate high overstresses and hence large elongations in the existing anchor bolts, which connect the tower legs to the concrete foundation. Additional anchor bolts and shear pins will be required.

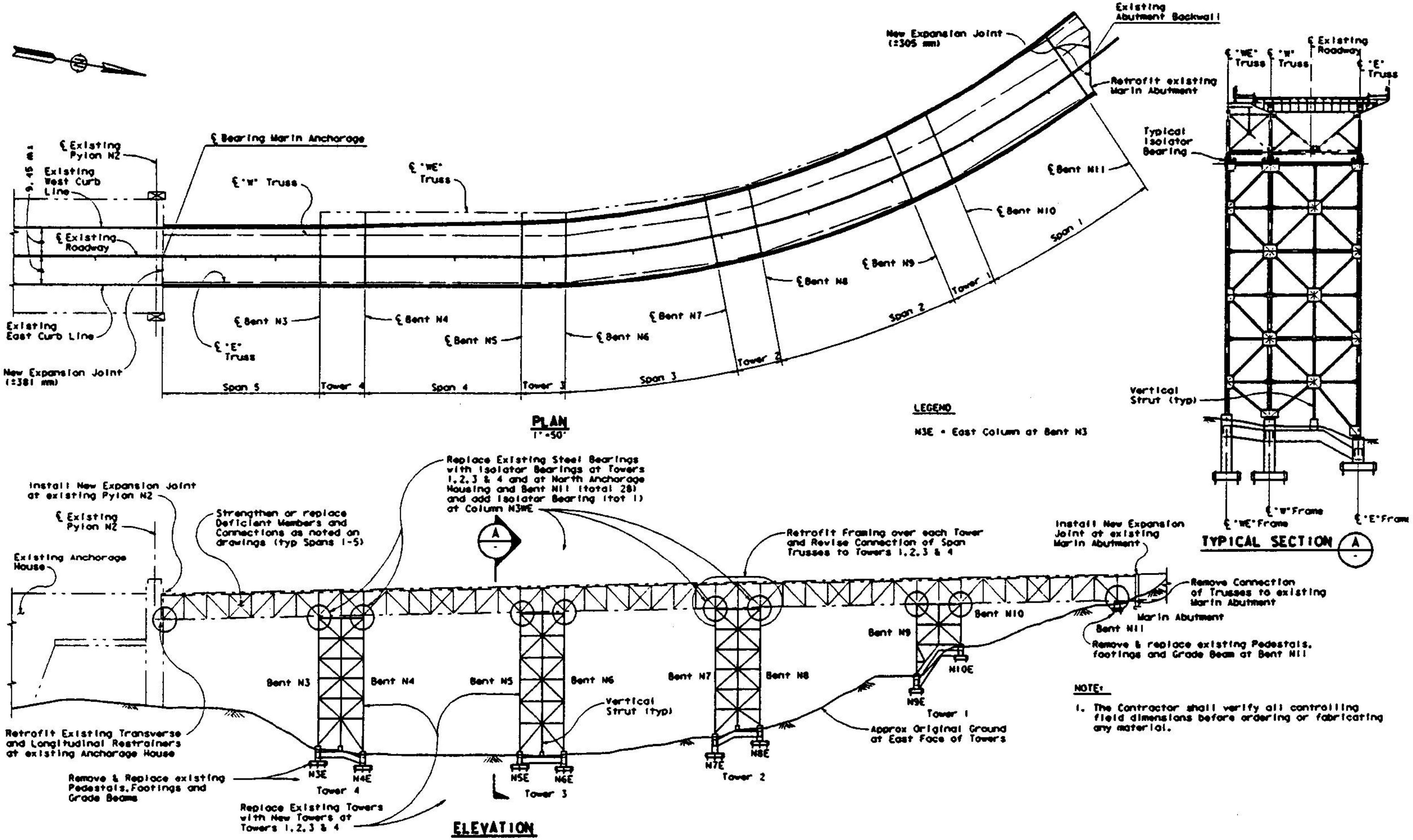


Figure 1 Seismic Retrofit of the North Approach Viaduct

4.4 Superstructure

The lateral seismic loads overstress both the top and bottom lateral bracing systems and if transferred to the main truss members they will be overstressed and require strengthening. The entire bottom lateral system in the tower box, immediately above the towers, will be strengthened to accommodate the transfer of loads between the superstructure and substructure. Other bottom lateral members will be replaced, added, or strengthened. Several top lateral bracing members must be strengthened, added, or replaced. The main truss verticals located directly above the temporary support towers will be strengthened to handle the tower reactions which are temporarily brought into the truss. Also, other vertical truss members in the spans need retrofitting. The sway bracing in the widening and main superstructure will be strengthened and retrofitted.

4.5 End Supports (North Abutment and Pylon N2)

At the North Abutment and Pylon N2 there will be large longitudinal and transverse displacements which cannot be accommodated by the existing joints. Therefore, a large movement joint will be installed.

4.6 Towers

The towers will be replaced; however, had we decided to retrofit the existing towers, the following work would have been needed:

- Rivets at Joints
  Some of the rivets in the joints of the tower members would have to be replaced with high strength bolts.

- Tower Diagonals
  Some diagonal members would have to be strengthened to keep within the design criteria and some diagonals added in the widened portion of the tower.

- Tower Columns
  Column strengthening would be required to maintain a D/C ratio of 1.

- Top of Tower Lateral Bracing
  The horizontal bracing system at the top of the towers would be strengthened to make the top level of the towers as stiff as possible to transfer seismic forces from the superstructure to the towers.

REFERENCES

1. IMBSEN, R. A. and J. PENZIEN (1984), "Evaluation of Energy Absorption Characteristics of Highway Bridges under Seismic Conditions," Earthquake Engineering Research Center, Report No. UCB/EERC-84/17, University of California, Berkeley.