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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

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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.

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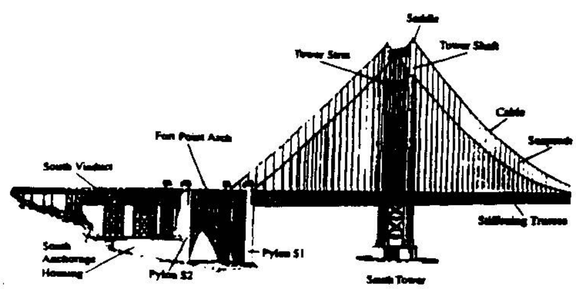
Mode	1	2	3	4	5	6	7	8	9	10
Period	1.10	.768	.566	.558	.445	.402	.375	.369	.312	.309

Table. 1 Golden Gate Bridge - Fort Point Arch Eigenvalues



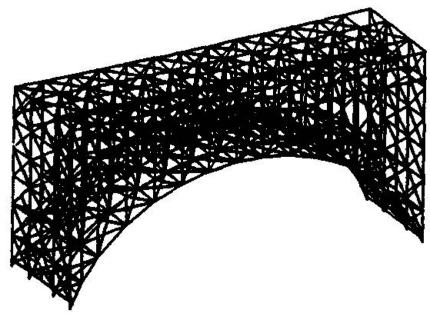
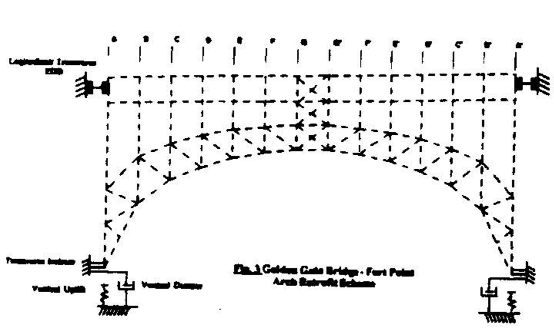


Fig. 2 Golden Gate Bridge - Fort Point Arch - Nonlinear KARMA Model.



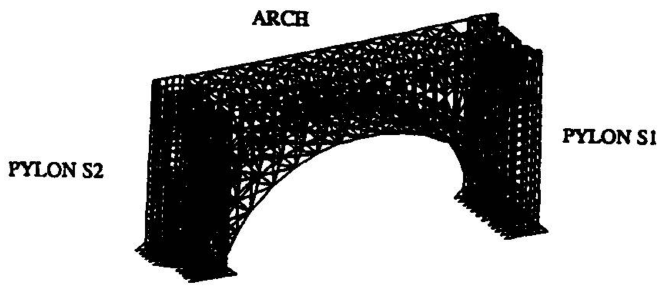


Fig. 4 Golden Gate Bridge - Fort Point Arch/Pylons - Nonlinear KARMA Model.