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Carquinez Bridges' Seismic Hazard Assessment and Conceptual Retrofit

Évaluation du risque sismique et renforcement des Ponts de Carquinez

Abschätzung von seismischen Gefahren und der konzeptionellen Verstärkung der Carquinez-Brücken

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SUMMARY

This paper summarises the results of a study performed to evaluate the seismic vulnerabilities and develop a conceptual retrofit for the existing Carquinez Bridges near San Francisco. These two steel truss bridges are classified as important bridges by the state. The study involved evaluating the dynamic behavior of the bridges with three-dimensional computer models using various analytical methods, identifying vulnerabilities in the structures based on calculated structural responses, and developing a conceptual retrofit strategy to address the vulnerabilities.

RÉSUMÉ

L'article résume les résultats d'une étude ayant pour objectif l'évaluation du risque sismique et le développement d'un modèle de renforcement pour les ponts de Carquinez, près de San Francisco. Ces deux ponts à treillis sont considérés comme très importants par l'État. L'étude a consisté en l'évaluation du comportement dynamique des ponts à l'aide de modèles tridimensionnels générés par ordinateur et analysés en utilisant diverses méthodes, permettant d'identifier les faiblesses des structures et préparer un plan de renforcement.

ZUSAMMENFASSUNG

Dieser Artikel fasst die Resultate einer Studie zusammen, die die seismische Anfälligkeit der Carquinez-Brücken in der Nähe von San Francisco evaluiert und eine konzeptionelle Verstärkung entwickelt. Diese zwei Stahlbrücken sind vom Staat als wichtig klassiert worden. Die Studie beinhaltet eine Evaluation des dynamischen Verhaltens der Brücken, welche aus einem dreidimensionalen Computermodell gewonnen wurde. Dazu wurden, gestützt auf berechnetem Bauwerksverhalten, verschiedene analytische Methoden verwendet, um Schwachstellen in Konstruktionen zu identifizieren. Diese Studie erlaubt einen Verstärkungsentwurf.

1. INTRODUCTION

1.1 Scope of Study

There are two bridges across the western end of the Carquinez Strait about 40 km north of San Francisco. The first bridge was erected in 1927, the second in 1958. Both are part of Interstate 80, which is a major interstate highway connecting California with states to the east. Since I-80 is a vital link for interstate commerce and important to the regional economy of the San Francisco Bay area, the Carquinez Strait Bridges are classified as important bridges by the state.

This paper summarizes the results of a study [2] performed for the California Department of Transportation (Caltrans) to assess the seismic vulnerabilities of the bridges when subjected to two intensities of ground motion. The highest intensity ground motion was based on a deterministic assessment of the maximum credible earthquake, and is called the "safety level" event. The lowest intensity ground motion was based on a probabilistic assessment of ground motions which have a 40% chance of occurring during the useful life of the bridge, and is called the "functional level" event. Caltrans performance objectives for important bridges include no structural damage or loss of function for the functional level event, and only limited structural damage without loss of function for the safety level event. Upon determination of the seismic vulnerabilities, a conceptual retrofit strategy was developed to meet these objectives.

1.2 Bridge Description

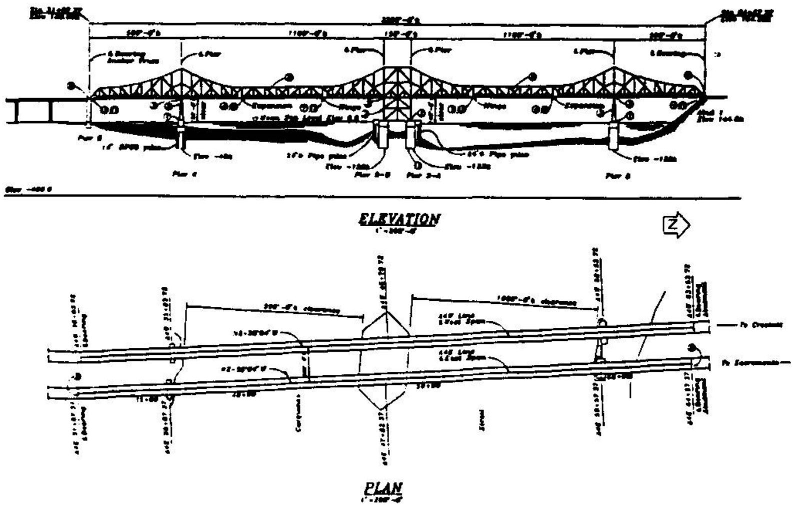

Fig. 1 General Bridge Plan

The two bridges are parallel four-span steel truss structures each with a total length of 1021 m (Fig. 1). Each bridge has two long spans of 335 m composed of suspended spans interconnected to cantilever spans at each end with hinge and expansion joints. The four interior supports for each bridge are tapered steel towers with heights varying from 36 to 41 m. These towers are typically supported by large concrete caissons located in as much as 30 m of water and penetrating 15 m into the soil. The roadway for each bridge is a concrete slab supported by steel floor beams and stringers. Southern approach viaducts for each bridge are approximately 335 m long, and consist of steel girders supported on reinforced concrete piers.

The west bridge was built in 1927 by the American Toll Bridge Company for a total construction cost of $4.6 million. It was the world's longest highway bridge at the time, and was unique since, for the first time, hydraulic buffers were used to provide bottom chord continuity across the two expansion joints during dynamic loading. In addition, the foundation caissons were the deepest-water caissons in the United States when they were constructed. The bridge was sold to the state of California in 1939. The main truss members of the 1927 bridge are built-up laced/riveted steel members. Tension members are typically steel eyebars.

The east bridge was built in 1958 by the state of California. The global geometry of the bridge is almost identical to the 1927 bridge except that it is wider and uses different steel types and member shapes and sizes. Several of the main truss members of the 1958 bridge were constructed using high strength T1 steel to reduce the cost and to minimize secondary stresses. High strength bolted field connections were also used, which was a first for bridge construction.

1.3 Site Soils

The subsurface conditions along the east and west Carquinez Bridge sites consist of a 10 m thick layer of soft to very soft bay mud typically underlain by a 23 m thick layer of stiff alluvium over bedrock. At the north and south ends of the bridges, bedrock is at or near the ground surface.

1.4 Site Seismicity

Two sets of site specific motions were developed by Geomatrix Consultants [3] for the Carquinez bridge site. The first set corresponded to probabilistic equal-hazard spectra for return periods ranging from 100 years to 2000 years. The second set corresponded to the maximum credible earthquakes deterministically defined for the San Andreas (M_w 8), Hayward (M_w 7.3), and Franklin (M_w 6.5) faults, which are as close as 41 km, 13 km, and 1 km, respectively, of the bridge site. For the maximum credible earthquake, free-field motions at each bridge support location were generated at the rock outcrop and at the mudline. The different motions at the support locations account for the variation of the seismic waves (phasing, frequency content, and amplitude) due to the characteristics of the soil profiles and the distance between the supports. The event associated with the Franklin fault dominates the seismic response due to its proximity to the site.

2. ANALYTICAL METHODOLOGY

2.1 Analysis Summary

In order to evaluate seismic vulnerabilities of the as-built bridges, as well as to evaluate the effects of various bridge retrofit strategies, complete three-dimensional computer models were developed for each bridge (Fig. 2). Numerous analyses with varying levels of complexity were performed to study the dynamic behavior of the bridges. Soil modeling was also performed to study the effects of soil-structure interaction. All of the different analytical tools used for this study are described in the sections which follow.

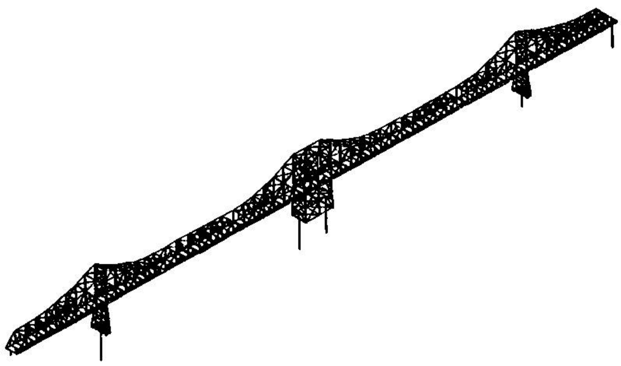

Fig. 2 Finite Element Computer Model

2.2 Soil Modeling

The properties of soil during an earthquake vary with the level of shear strain induced by the earthquake. With increasing shear strain, the shear modulus of the soil decreases and material damping increases. This nonlinear behavior of the soil was simulated by performing a linear unidimensional wave analysis using a computer model of the soil column with the ground motion defined at the rock outcrop. An effective strain equal to 65% of the maximum strain was calculated for each soil layer. New soil properties consistent with this effective strain were then calculated, and the iterative process repeated. The procedure was halted when the soil properties converged to a stable result. The computer program SHAKE [5] was used for the analysis, and the resulting properties included the strain compatible shear modulus, bulk modulus, and damping values for the soil. In addition, free-field time histories compatible with the strain-compatible soil profile were calculated at the mudline at each support location.

The soil-structure interface has frequency dependent characteristics which are represented by complex valued impedance and scattering functions. Impedances describe the load-displacement characteristics of the soil-foundation system, and depend on the soil configuration, material behavior, excitation frequency, and foundation geometry. Scattering functions modify the free-field seismic motion due to the presence of the foundation. The SASSI [4] and CLASSI [6] families of programs were used to calculate these functions at each support location. These functions were used for the soil-structure interaction analyses performed for the bridges.

2.3 Response Spectra Analyses

Response spectra analyses were performed for the safety and functional level events using the computer program SAP90 [1]. This was the most widely used method of analysis in this study because it provided a simple and efficient way to assess the basic dynamic behavior of the structures and consider the relative effects of certain variables on the overall system response.

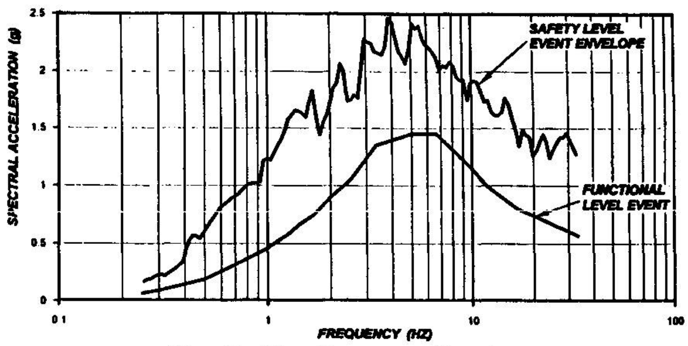

Fig. 3 Site Response Spectra

For the safety level earthquake evaluation, the ground motion time histories were used to calculate longitudinal, transverse, and vertical response spectra for each support location. These spectra for each support were then enveloped to develop a single set of three orthogonal response spectra. For the functional level analysis, the equal-hazard response spectrum for a 300-year event was used. This is consistent with the selected occurrence probability stated in Section 1.1 for a useful bridge life of 150-years. (Fig. 3).

Although all of the response spectra analyses performed were linear elastic, non-linear behavior was simulated in some instances by iteratively reducing member stiffnesses. Response spectra analyses were essential to develop a basic understanding of the bridge behavior and to provide a benchmark to assess the effects of time history loading, soil-structure interaction, and multiple-support excitation.

2.4 Single Input Time History Analysis

Single input time history analyses were performed to study the response of the bridges to time history loading. The safety level ground motion time histories at one support location were applied simultaneously to all of the supports. The structural responses calculated in this analysis were generally within 5 to 20% of the responses calculated in the response spectra analyses. Since the multiple-support excitation analysis described in the next section provided a more accurate representation of ground motion, the single input time history analyses were not used for final vulnerability and conceptual retrofit assessment. The primary benefit in performing this analysis was to obtain a better understanding of the bridge behavior under time history loading.

2.5 Soil-Structure Interaction Analysis Considering Multiple-Support Excitation

Due to the distance between supports and the variation of the soil profile along the bridge alignment, the seismic motion at each support differs in phasing, frequency content, and amplitude. In addition, the interaction between the soil and the structure can have a large impact on the dynamic response of the bridges. Therefore, a soil-structure interaction analysis considering multiple-support excitation was performed to provide the most accurate representation of the dynamic response of the bridges during the seismic event.

The bridge response to multiple-support excitation contains two components. A dynamic component represents the bridge inertial response to the seismic input. A pseudo-static component represents the bridge response to relative foundation motions. The total response is the sum of these components. This analysis was performed in the following manner:

- A modal analysis was performed assuming fixity at the soil-foundation interface. The mode shapes were used to determine the inertial response of the superstructure and foundation elements to the input ground motion.
- Pseudo-static mode shapes were calculated by applying a unit displacement sequentially at each degree of freedom at the soil-foundation interface (6 supports x 6 DOF/support = 36 total shapes). These shapes relate the bridge response to differential foundation motion.

- Frequency dependent impedance matrices were calculated for the soil region at each support.
- Incident wave scattering functions were calculated to consider the effect of motion deconvolution and foundation geometry on the free-field ground motion input.

This information was combined to calculate the response of the coupled soil-structure system using the CLASSI computer programs. Free-field acceleration time-histories were applied to each support at the soil-structure interface and the responses were calculated in the frequency domain using Fourier analysis methods. This was the most advanced analysis performed for this study.

3. AS-BUILT BRIDGE EVALUATION

3.1 Dynamic Behavior

The analyses performed for the as-built bridges described in Section 2 provided valuable information on the behavior of the bridges for the safety and functional seismic events. The following is a partial summary of the expected bridge behavior:

- The natural frequency of each bridge superstructure is 0.3 Hz in the transverse direction and 0.6 Hz in the longitudinal direction.
- Displacements and forces obtained from the soil-structure interaction multiple-support excitation analysis averaged 25% less than response spectra results. Primary reasons for this reduction are the effects of motion phasing and soil radiation damping, and the fact that the response spectra were developed by enveloping the ground motion at the different support locations, resulting in a more conservative definition of motion.
- The concrete caissons which support the steel truss towers are extremely large, and make up almost 85% of the total system mass. Since the majority of the mass is located in the foundation, and the superstructure is relatively flexible, there is a distinct separation between the foundation response and the superstructure response to the ground motion. While the superstructure responds to low frequency excitation due to its flexibility, the foundation responds to high frequency excitation near the peak of the response spectra. The majority of the seismic forces acting on the foundation are generated by inertia of the foundation itself.

3.2 Seismic Vulnerability Assessment

Based on Caltrans' performance objectives for important bridges, a set of assessment criteria was developed to allow a quantitative evaluation of demand to capacity ratios to identify vulnerabilities. Refer to Reference 2 for a description of the criteria used for this study. The following is a partial list of vulnerabilities identified for the bridges:

- Certain primary truss members and critical components and connections, including tower columns and diagonal members, suspended span bottom chord members, and hydraulic buffers, have calculated forces which exceed their capacities. In the 1927 bridge, seismic compression demands during the safety level event are likely to overcome tensile gravity loads in eyebar members, resulting in member buckling.
- At the ends of each bridge, there is an existing 15 cm separation between the steel truss and the concrete abutment. Longitudinal displacements of each bridge are expected to exceed this gap, resulting in large impact forces between the truss and the abutments.
- Due to the large inertial forces which generate in the caissons, caisson rocking and sliding may occur during the safety and functional level events.

4. CONCEPTUAL RETROFIT

4.1 Global Retrofit Strategy

Several global retrofit strategies to address bridge vulnerabilities were studied. The computer models were modified and analyzed to assess the effects of retrofit on the bridge response. The final strategy recommended for each bridge included the following:

- Allow unrestricted longitudinal movement of the bridges by increasing the abutment gap and retrofitting the rocker bearings at each abutment.
- Strengthen selected steel members and connections, and replace hydraulic buffers.
- Interconnect the foundation caissons to control rocking response.

This retrofit strategy strengthens but does not otherwise alter the existing lateral load path. It is important to note that further verification of the foundation retrofit is necessary. Further analyses based on detailed soil borings at the site are recommended for the final design stage to more accurately assess the effect of non-linear foundation response.

4.2 Conceptual Retrofit Details

Conceptual retrofit details were developed for various structural elements and components in accordance with the recommended retrofit strategy. These details include the following:

- As described above, seismic compression demands could overcome gravity tensile loads in eyebar members. To prevent member buckling, new plates connected by high-strength bolts would be added at the top and bottom of the eyebars (Fig. 4) to create a box-shaped section capable of resisting compression.
- To strengthen typical steel truss members, new cover plates would be bolted to the members. Bolted connections were recommended due to potential difficulties in field welding to existing steel.

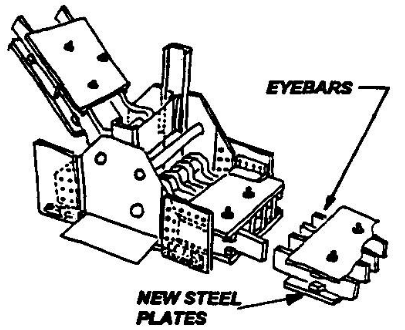

Fig. 4 Conceptual Eyebar Retrofit

4.3 Retrofit Cost Estimate

The total cost to retrofit the 1927 and 1958 Bridges for the safety level event was estimated to be $29 million and $24 million, respectively. This cost includes the superstructure, foundation, and approach structure retrofit for each bridge.

5. ACKNOWLEDGMENTS

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