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SEISMIC EVALUATION AND RETROFIT OF LONG SPAN BRIDGES

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ABSTRACT

Currently, seismic evaluation of a long span bridge is done by establishing realistic ground motions that can reach the bridge site, developing three dimensional elastic models of the super-structure and conducting time-history dynamic analyses of almost elastic model of the super-structure. The nonlinearities are usually in expansion joints, foundations and in energy dissipating devices.

To perform seismic retrofit design of a long span bridge, first, time history analyses of the 3-dimensional almost elastic models of existing as well as retrofitted structure is conducted. The objective is to develop a feasible strategy for seismic retrofit. Then, by developing "partially inelastic" models of the towers and super-structure, performance of the selected retrofit strategies is studied. Finally after seismic retrofit is designed, a model of the bridge with more inelastic elements is developed and the dynamic behavior of the retrofitted bridge under safety and functionality earthquakes is evaluated.

Due to complexity of long span bridges and the large number of members involved, with current technology it is very difficult if not impossible to conduct a meaningful *inelastic* dynamic analysis of such large systems. To mitigate the problem, the concept of using "Mega-truss" is developed and proposed here. By using Mega-truss, the number of members involved in dynamic behavior can be significantly reduced, therefore, making it possible to model all members of the Mega-truss inelastically.

KEYWORDS

Bridges, seismic design, steel structures, seismic evaluation, seismic retrofit, cyclic behavior, seismic behavior, steel bridges, suspension bridges, truss bridges.

INTRODUCTION

The 1989 Loma Prieta earthquake near San Francisco resulted in significant damage to building, bridges and other facilities. A segment of double deck Cypress elevated freeway collapsed and resulted in 38 deaths. Other non-ductile reinforced concrete overpasses also sustained severe damage. There are seven major long span bridges in the San Francisco Bay area including the Golden Gate and the San Francisco Oakland Bay Bridges. Figure 1 shows views of three of the long span bridges in the San Francisco Bay. Damage to the

Oakland Bay bridge. This bridge, one of the most important bridges in United States, was closed for a month due to collapse of a 15 meter long portion of its deck. The collapse of this segment of the Bay bridge and closure of the bridge for a month for repairs, had considerable impact on the transportation of people and goods in the greater San Francisco Bay area.

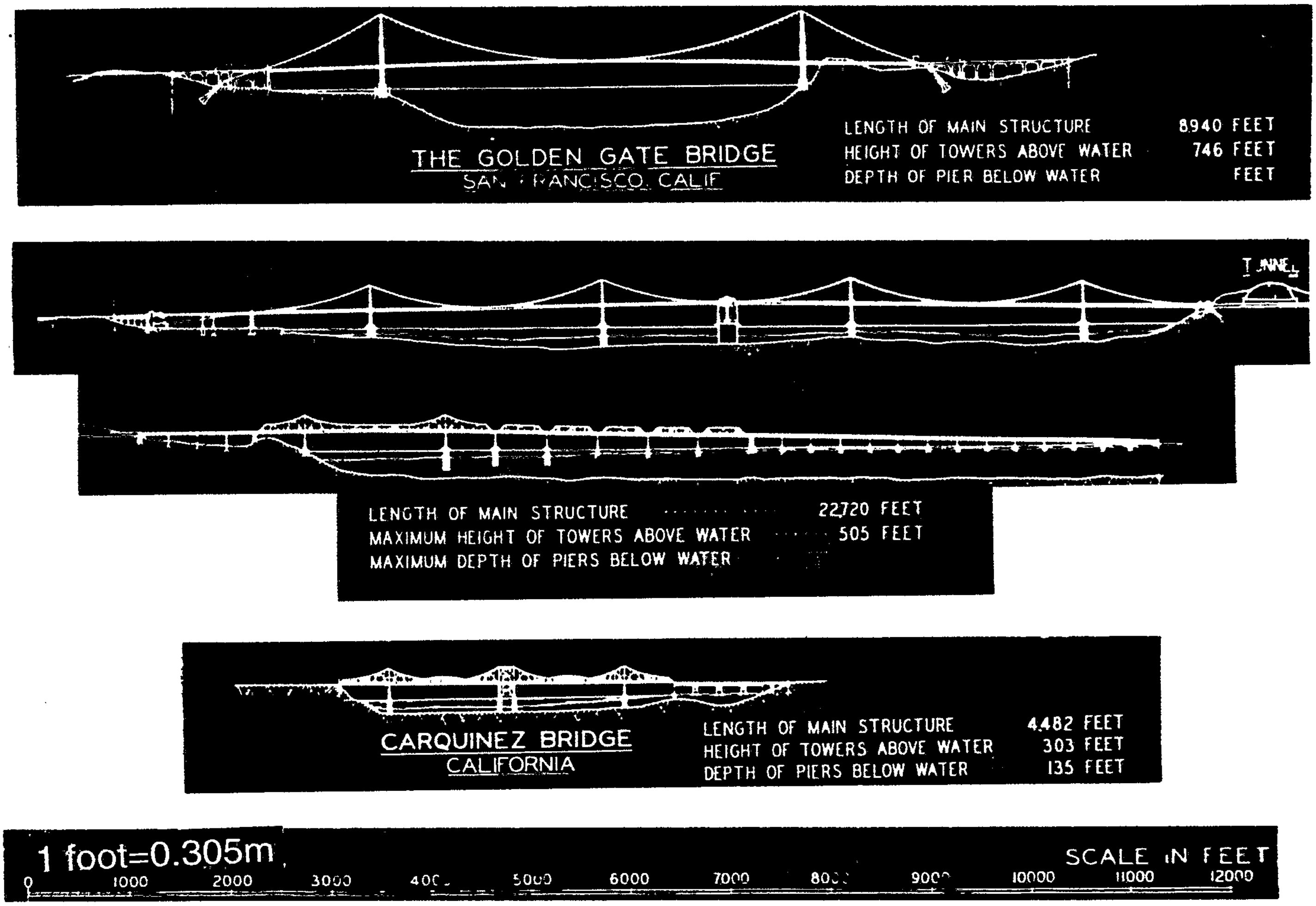

Fig. 1. Views of the Golden Gate bridge, the San Francisco-Oakland Bay bridge and the Carquinez bridge

The relatively short duration closure of the San Francisco Oakland Bay bridge due to Loma Prieta earthquake damage and its consequences acted as a signal that the existing stock of major long span steel bridges could be vulnerable to seismic damage during major earthquakes. In the aftermath of the Loma Prieta earthquake, the California Department of Transportation started a major effort to evaluate bridges in California in order to identify seismic vulnerabilities, to develop appropriate seismic retrofit and to implement the retrofit plans.

Seismic behavior of the long span steel bridges is very complex. The complexity is due to:

1. Site condition varies significantly from bridge to bridge and in some cases even from pier to pier. The suspension bridges are supported on the bedrock. However, most truss spans in the San Francisco Bay area are supported on the pile driven into soil with depth of 30 to 200 meters.

2. A variety of reinforced concrete multi-cell caissons have been used. The deepest caisson reaches a depth of about 80 meters. Hollow box, solid box and bell shaped hollow foundations have been used. Piles consist of steel H-shapes, reinforced concrete and Douglas Fir timber piles.

3. Each long span bridge has its unique structural configuration and articulations. A variety of structural systems such as suspension span, cantilever span, simply supported trusses, plate and box girders are used.

4. Light weight and regular weight concrete as well as steel orthotropic decks have been used.

5. The original design of each bridge was based on a unique specification for that bridge. As a result margin of safety varies considerably among the long span bridges. Some bridges such as San Francisco Oakland Bay bridge is designed to carry heavy military live loads. As a result, the bridge has considerable margin of safety against the dead load that can be used to resist seismic forces. Others, such as Richmond San-Rafael bridge is designed with variable factor of safety for each family of members. This unique approach to manipulating the load factors at member level has resulted in cost efficiency during original construction, but has left smaller margin of safety for seismic resistance.

6. A variety of member types such as laced, battened, perforated, built-up and rolled shapes are used.

7. The long span bridges in the San Francisco Bay area have been built during 1920's through 1960's. Depending on the time of construction, a variety of connector types such as rivets, bolts and welds are used.

8. The steel used in these long span bridges varies from wrought iron to a variety of steels spanning 70 years of steel mill technology and production.

9. Long span bridges usually have hundreds if not thousands of members and connections. It is expected that many of these members will experience inelasticity during major earthquakes. Therefore, the analytical model of these bridges should include material as well as kinematic non-linearities.

10. Long span bridges have a large number of relatively closely spaced modes of vibration with mass participation distributed over these modes. As a result, response spectra analysis does not provide reliable results. Time history analyses with multiple support excitation becomes necessary to capture realistic seismic response.

11. Prior to the Loma Prieta earthquake, there has been almost no test data on cyclic behavior of components of the existing steel bridges particularly bridges built using rivets.

Considering the above issues, currently seismic studies and retrofit of a long span bridge requires a comprehensive and multi-disciplinary plan. In the aftermath of the Loma Prieta earthquake, the California Department of Transportation (Caltrans) has developed a comprehensive plan to study long span bridges in California and to develop seismic retrofit as needed.

To conduct the seismic vulnerability studies and to design seismic retrofit an engineering team has been formed for each major long span bridge. The teams are made of bridge engineers from governmental transportation agencies (Caltrans) and private sector as well as researchers from the academia. The author has been a member of a number of these teams to participate in development of performance and seismic design criteria. In addition, as a researcher, a number of experimental research programs has been conducted by the author and his research associates at the Department of Civil and Environmental Engineering of the University of California, Berkeley on steel bridges. In the following, due to limitations of space, the highlights of these activities related to seismic evaluation and retrofit design are presented. More information can be found in publications listed as references.

METHODOLOGY USED IN SEISMIC RETROFIT OF LONG SPAN BRIDGES

Currently, seismic retrofit of long span complex bridges is an interactive process conducted with heavy participation of not only bridge engineers but also the owner(Caltrans) and the research community. Due to complexity of long span bridges and tens of millions of dollars needed for seismic retrofit of each long span bridge, the economical aspects of design of seismic retrofit becomes very important.

The methodology discussed here is currently applied to seismic retrofit design of a number of bridges in California. For each bridge, depending on the specifics of the bridge and its needs, the methodology has to be refined and appropriately adapted.

The main steps to develop seismic retrofit for a long span bridge are:

1. Formulate the performance criteria and establish expected performance of the bridge during major and moderate earthquakes.

2. Conduct a seismic vulnerability studies to establish seismic hazard level and the cost to reduce the hazard.

3. Based on the results of seismic vulnerability studies, refine Step 1 above and establish level of expected performance and tolerable damage during major and moderate earthquakes.

4. Conduct comprehensive analyses of seismic behavior of the existing long span bridge to develop a number of retrofit strategies and the associated cost for each strategy.

5. Select the final strategy and conduct detailed analyses and design of retrofit.

6. Conduct analyses of the bridge with final retrofit strategy added.

7. Prepare construction drawings and documents.

The following sections discuss he above steps.

SEISMIC PERFORMANCE CRITERIA

Seismic Performance Criteria defines the expected performance of the bridge. The criteria usually is established by the owner based on the bridge-priority ratings (Gates and Maroney, 1990). Bridge priority ratings are established by the owner by considering such factors as site condition, condition of the bridge, consequences of damage on safety of the bridge users, importance of the bridge in emergency response after the quake, role of the bridge in local and state transportation, impact of the loss of the bridge on well being and quality of life of bridge users and others, cost of replacement, cost of repairing damage, availability of parallel detour routes and future plans for construction of new bridges in the area.

SEISMIC VULNERABILITY STUDIES

The objective of seismic vulnerability studies is primarily to assess seismic condition of the bridge in order to establish seismic safety hazard, to develop preliminary retrofit strategies and to estimate cost of retrofit. The procedures used in conducting seismic condition assessment of the East Bay Crossing of the San Francisco Oakland Bay bridge (Astaneh-Asl et al., 1993) and (Astaneh-Asl, 1992) were as follows. The procedures have been used in seismic vulnerability studies of a number of bridges in California.

Seismological Aspects

By considering location and character of the faults in the area, a series of Safety Evaluation Earthquakes are established. The earthquakes are defined in the form of three components of time history of rock accelerations. The earthquakes are artificially generated. However, the spectra that forms the backbone of the artificial generation is adjusted such that in addition to representing the character of the fault, it is appropriate for long span bridges. The 1994 Northridge and the 1995 Kobe earthquakes clearly indicated the importance of vertical component of the quake as well as the damage that can be caused by high velocity near field quakes (Astaneh-Asl et al. , 1994) and (Astaneh-Asl and Kanada, 1995). These issues are considered in generating ground motions. The ground motions generated at the location of the fault rupture bedrock are carried over to the bedrock under the bridge and established at the location of a number of important piers.

Geotechnical Aspects

For bridges that are supported on soil instead of bedrock, the bedrock motions are applied to the bottom of soil layer. By conducting dynamic analyses of the soil columns under a number of important piers, the free field motions at the location of these piers are established. In addition, by conducting soil-pile-structure interaction analyses the ground motions felt by the substructure of the bridge are established.

Structural Analysis Aspects

To conduct seismic vulnerability studies, in many cases, it is impractical and in some cases impossible to conduct a full-fledged inelastic time-history dynamic analyses. This is due to the fact that data on inelastic cyclic behavior of components of many existing long span bridges is very limited and mostly non-existent. In addition, due to large number of inelastic elements, conducting a time-history analyses with multiple-support excitations becomes very time consuming and in many cases non-convergent. In some cases, to verify validity of elastic analyses and to understand actual inelastic behavior, in parallel to elastic 3-dimensional analyses, inelastic two dimensional analyses can also be conducted (Astaneh-Asl, et al., 1993).

The dynamic analyses discussed above provide the seismic demand side of the basic equation of evaluation and design. The equation of evaluation and design is in general form of Equations (1) and (2) for inelastic or elastic analyses respectively.

$$\left(\frac{D_D}{\mu}\right) \leq 1.0 \tag{1}$$

$$\left(\frac{F_D}{F_c}\right) \leq R \tag{2}$$

where, D_D is displacement demand established by inelastic analyses, μ is displacement capacity (ductility) established by tests or in the absence of tests by conducting finite element analyses, F_D is force demand established by elastic analyses, F_c is force capacity (strength) established by tests or governing codes and R is the response modification factor. Establishing R is discussed in more details in (Astaneh-Asl, 1996a). By conducting analyses to establish demands and by using above equations, seismic vulnerabilities of a long span bridge are established. Based on vulnerabilities, a number of retrofit strategies that have the potential of mitigating the vulnerabilities are developed. With this, the seismic vulnerability studies are completed.

SEISMIC RETROFIT DESIGN

The methodology summarized here has evolved from the work of many engineers from the California Department of Transportation, private sector companies and research projects (Astaneh-Asl et al., 1993) and is currently being implemented in a number of long span bridges in California. In this approach, after seismic *vulnerability studies* are completed a multi-disciplinary team of bridge engineers, seismologists, geotechnical engineers, material engineers, researchers and other professionals is formed for each bridge. The team then performs the following tasks to design seismic retrofit.

A. Refine Ground Motions

If necessary, the ground motions used in the vulnerability studies are refined and modified to incorporate any new data that might have surfaced. By drilling bore holes and conducting in-situ and laboratory tests, more reliable data on dynamic character of soil is collected. Such data is used to refine ground motions.

B. Refine Information on the As-Built Condition of the Bridge

By conducting material tests such as steel tension tests and concrete core tests, the actual material properties are established and used in design of seismic retrofit. In addition, by detailed site inspections and survey the condition of the existing bridge is better established and verified.

C. Conduct 3-D Elastic Dynamic Time History Analyses to Finalize Retrofit Strategies

Due to complexity of long span bridges and the large number of members involved, currently conducting inelastic analyses is very time consuming. In some cases, due to significant inelasticity in the bridge, the inelastic model does not converge to obtain meaningful results. Therefore, to develop retrofit strategies, in most cases, only elastic analyses are conducted and the elastic response is modified to represent the more realistic response by using Response Modification Factors. Discussion of Response Modification Factor based seismic design is provided in (Astaneh-Asl, 1996a).

D. Conduct Dynamic Time History Analyses of Partially Inelastic Bridge

The main goals of these analyses is to study the performance of retrofitted bridge and to obtain a better understanding of level of damage that can occur in the retrofitted bridge. The partially inelastic model of the bridge is a three-dimensional model with a relatively small number of critical members modeled as inelastic while the rest of the structure is modeled as elastic. In truss bridges, such as Carquinez Bridge in Fig. 1, usually the towers are modeled inelastically whereas the super-structure truss is modeled elastic. Of course the expansion joints, foundations supports and energy dissipating devices are always modeled as nonlinear. To conduct a meaningful inelastic analyses, the actual cyclic behavior of members and their connections should be modeled properly. However, since a large number of members are involved, the inelastic hysteresis models should be simple yet representative of strength, stiffness, cyclic ductility and most importantly the energy dissipation behavior. An example of such models developed for truss members of towers of the Carquinez bridge is shown in Fig. 2. (Astaneh-Asl, 1996b).

Normally, by completing Part D above, seismic retrofit design of a long span bridge is completed. However, due to importance of long span bridges, the actual inelastic behavior of retrofitted bridge needs to be established. This is due to the fact that, even after retrofit, a considerable number of members of the super-structure will become inelastic during a major earthquake. Therefore, the question to be answered is whether the retrofitted bridge can experience full or partial collapse? To answer this very important question one has to conduct a realistic dynamic analysis of an inelastic model of the bridge. However, as discussed earlier, conducting dynamic time-history analyses of a long span bridge, particularly a truss bridge with many members, in many cases is very difficult if not impossible. To overcome such difficulties and to conduct a meaningful inelastic analysis of the retrofitted long span bridge, the concept of "Mega-truss' is developed by the author and proposed in the next section.

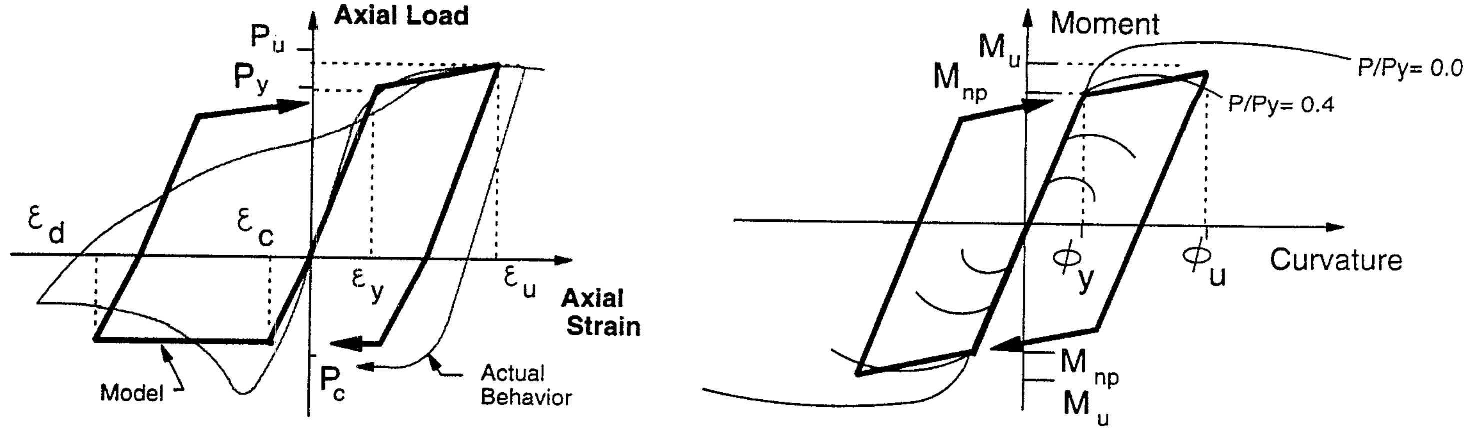

Fig. 2. Axial load- axial strain and moment-curvature models for truss members of Carquinez bridge.

E. Conduct Dynamic Time History Analyses of Fully Inelastic Bridge Modeled as a "Mega-Truss"

This step has been proposed by the author recently and at this writing has not yet been applied to long span bridges. However, it shows good potential for enabling bridge engineers to easily conduct a full-fledged inelastic analysis and capture the major portion of the inelastic behavior without modeling the entire bridge as inelastic.

By studying configuration of a main trusses in a long-span bridge, often it is possible to divide members into two categories of Global Members and Local Members. Global members are those that are necessary to maintain global stability of the truss. Local members are those that either brace the global members or transfer local gravity load to the global members. Figure 3 shows the cantilever span of the East Bay Crossing of the San Francisco Oakland Bay bridge. The global and local members of this truss are identified by thick and thin lines respectively

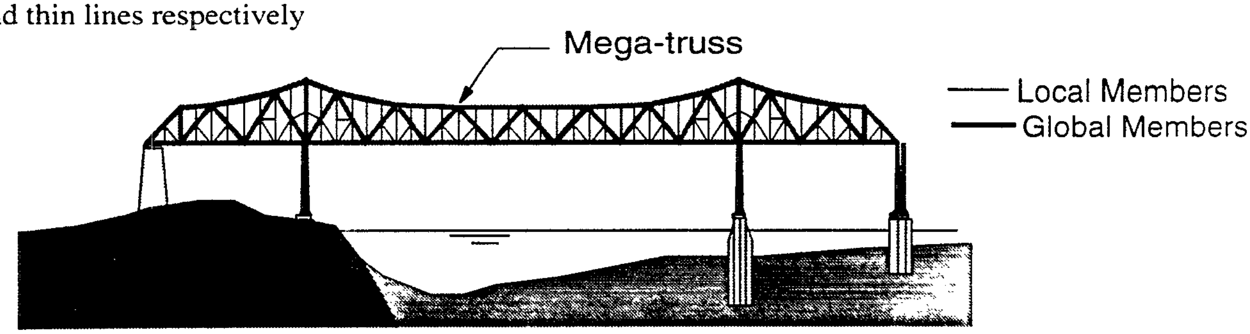

Fig. 3. "Global" and "Local" of the cantilever span of the San Francisco Oakland Bay bridge

The local members usually have only gravity load. Because of their relatively small stiffness and discontinuity at the truss joints, very small amount of seismic load is carried by these local members. As a result, their effect on global seismic response of the bridge is usually very minimal. Therefore, to obtain a structure with small number of members, one can ignore the local members in conducting three-dimensional inelastic analyses of a long span bridge. A test of identifying a member as local is that if by eliminating the member only local collapse occur then the member is a local member. Also, by studying the results of elastic analyses of entire bridge one can identify local members by studying how much seismic load is transmitted by the truss members? If seismic load is small, then the member is a good candidate to be categorized as local member.

By eliminating the "local" members and by only including "global" members of the truss a Mega-truss is formed. It should be realized that a Mega-truss is not a "super-element" that frequently is used in dynamic analyses to reduce degrees of freedom in large systems. In case of super-element, an entire portion of a truss is replaced by a fictitious element that is calibrated to represent the dynamic characteristics of the original structure. Whereas in a Mega -truss, there is no fictitious element. All members of the Mega-truss are the same members as in the original truss. The only difference between the Mega-truss and the original truss is that the Mega-truss does not have the local members.

In forming a Mega-truss, after local members are eliminated, two adjustments have to be made. One is to distribute the mass the was in the local members to their supporting global members. Second adjustment is to change effective length factor of the Mega truss members to have their buckling capacity correctly represented in the inelastic model.

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