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Rapid Post-Earth-Quake Safety Evaluation of a Suspension Bridge using Fragility Curves and Strong Motion Data

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Abstract

This paper, *Rapid Post-Earthquake Safety Evaluation of a Suspension Bridge Using Fragility Curves and Recorded Strong-Motion Data"* is part of the Data Interpretation Project of the California Strong Motion Instrumentation Program (CSMIP) in the Department of Conservation's (DOC) California Geological Survey. In existing Structural Health Monitoring (SHM) systems, sensor data collection capacity has far outpaced the ability to analyze the data and is therefore not truly empowering engineers to make critical decisions. This project was proposed to address this shortcoming, by accelerating the application of the strong- motion data in reducing risk due to the strong earthquake shaking which occurs in California.

The application of the procedure undertaken in this study is to provide for the selected New Carquinez Bridge the ability to assess the damage immediately following an earthquake using the ground motion parameters of the earthquake event and fragility curves developed for the bridge so that a decision can be made on the continued use or closure of the bridge.

Immediately after any earthquake, Caltrans (California Department of Transportation) has to make decisions about the post-earthquake conditions of bridges. The decision-making process will be based on the earthquake intensity, location of a bridge, instrument data, the understanding of the performance of the bridge in the subject earthquake, and other factors related to risk and consequences.

Keywords: strong motion records, suspension bridge, earthquake response, post-earthquake bridge closure, fragility of bridge components, pushover analysis, time history analysis, scenario earthquakes

1. Introduction

Most of the critical bridges in California that are in high seismic zones have been instrumented. These instrument bridges are monitored in real time and this stored data can be used as part of a decision-making process. The foundation or free field ground motions near the bridge and some of the structural performance data can be obtained immediately after an earthquake. However, this limited instrument data doesn't provide adequate information about the conditions of all critical components of bridges immediately after an event. Therefore, additional understanding of the bridge performance and fragility functions should be developed for each of these critical bridges to assist the post-earthquake decision making process. This new process rapid post-earthquake decision making process for critical bridges. Immediately after any earthquake, Caltrans must make decisions about the post-earthquake conditions of bridges. The decision- m a k i n g process will be based on the earthquake intensity, location of a bridge, instrument data, the understanding of the performance of the bridge in the subject
earthquake, and other factors related to risk and consequences. An overview of the program is shown in Figure 1 [1].

The application of the procedure described in this paper is to provide for the selected New Carquinez Bridge, as shown in Figure 2, the ability to assess the damage immediately following an earthquake using the ground motion parameters of the earthquake event and fragility curves developed for the bridge so that a decision can be made on the continued use or closure of the bridge.

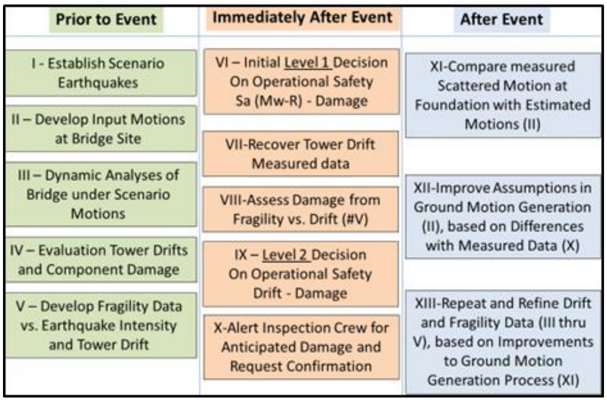

Figure 1. Program Overview

2. Description of the New Carquinez Bridge

The Alfred Zampa Memorial Bridge, also known as the New Carquinez Bridge (NCB), connects the cities of Vallejo, CA and Crockett, CA that sit on opposite sides of the Carquinez Straight [2].

Figure 2. Aerial View looking towards Sacramento of the New Carquinez Bridge

The main span is 2,388 ft long and is bounded by a south span (towards Oakland) of 482 ft. and a north span (towards Sacramento) of 594 ft. as shown in 2 . The superstructure is supported by a total of 158 hangers. The principal components of this suspension bridge include reinforced concrete towers supported on large-diameter concrete pile foundations, parallel-wire cables, gravity anchorages, and a closed orthotropic steel box deck system. The main concrete towers are approximately 400 ft. tall, and are tied together with a strut below the deck and an upper strut between the cable saddles. The lower strut supports the deck vertically using two rocker links and transversely through a shear key. The scope of the current study is to evaluate seismic performance of Tower T3 as "the critical component" of the bridge. As shown in the photograph this is the tower towards Sacramento. The procedure that is presented in this paper can be equally applied to any other component of the bridge.

The bridge site, at approximately twenty miles northeast of San Francisco, is in an active seismic zone. Seismic hazard assessments have shown that the site could be subject to strong ground motions originating on the San Andreas Fault, the Hayward Fault, the Concord- Green Valley, the Napa Valley, and the Franklin Faults. However, studies have shown that the Hayward fault, Concord-Green Valley fault system, and the Napa Valley seismic zones are the dominant sources of seismic hazard for the bridge's frequency range.

The seismic design of the New Carquinez Bridge considers both the Safety Evaluation Earthquake (SEE) and the lower level Functional Evaluation Earthquake (FEE). Caltrans performance requirements for these events are higher than the minimum level required for all transportation structures but below that required for an Important Bridge. As much as possible, the Important Bridge criteria are to be met for the Safety Evaluation Earthquake (SEE) corresponding to a maximum credible event which has a mean return period in the range of approximately 1,000 to 2,000 years. In this earthquake, the bridge can be subject to primarily "minor" damage with some "repairable" damage to piles, pile caps and anchorage blocks and remain open.

1. Methodology

The proposed procedure recognizes three stages for data processing and decision making, namely: "**Prior to an Event**", "**Immediately after an Event**", and "**Post Event**" as shown in Figures 1 and 3 . Data processing related to "Immediately after an Event", and "Post Event" requires recovering measured data from CSMIP sensors after an event and possible enhancement of the modeling procedure and SSI approach.

Generally, the objectives for the proposed three stages are:

Prior to an Event - evaluate and compile structural response and fragility data in a "Bridge Seismic Report" for a range of probable ground motions at the site.

Immediately after Event – based on the event location and magnitude, reference the "Bridge Seismic Report" to arrive at a "Level 1" safety/serviceability assessment to facilitate the continue service/emergency use only/full closure decision. Following the event and using the ground motions data and instrumentation data when available (CSMIP), reevaluate the structural response to arrive at a more accurate "Level 2" serviceability decision.

Post Event – compare instrumentation and calculated structural response to refine assumptions and parameters used in the analysis to achieve an improved process for the next event.

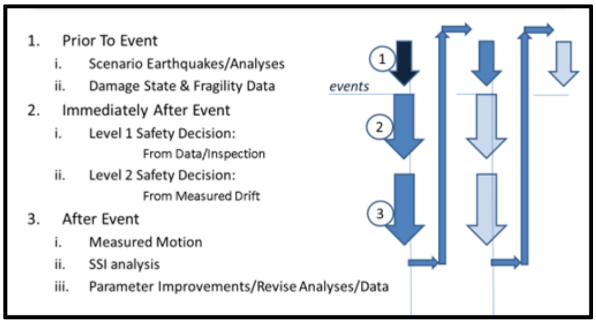

Figure3 Procedure

In developing a system to improve the current Caltrans rapid post-earthquake decision making process for critical bridges. Immediately after any earthquake, Caltrans must make decisions about the post-earthquake conditions of bridges. The decision- m a k i n g process will be based on the earthquake intensity, location of a bridge, instrument data, the understanding of the performance of the bridge in the subject earthquake, and other factors related to risk and consequences. In the process of developing this procedure the following tasks were undertaken:

1. **Scenario Earthquakes:** Develop 26 sets of earthquake scenario events based on the location of the bridge and the active faults in the vicinity of

the bridge site.

2. **Drift-Strain Curves:** Using pushover analysis, obtain drift-strain (concrete and reinforcement) curves. For this study Tower T3, located at the North side of the bridge, was selected as a critical component of the bridge.
3. **(Mw-R-Sa)-Drift data:** Using 26 nonlinear time-history analyses of the detailed model of the bridge, obtain 26 samples of data between drift and the earthquake characteristics. In this study Moment magnitude (Mw), distance to the fault (R) and spectral acceleration (Sa) are used to characterize the ground motion.
4. **Fragility Analysis:** Using the (Mw-R-Sa)-Drift data, and the fragility curve database, obtain fragility curves for Tower T3.
5. **Bridge Seismic Report:** using the fragility curves, prepare a "Bridge Seismic Report".

As shown in Figure 3, prior to an event, several automated procedures were completed and compiled in a "Bridge Seismic Assessment" report, as a reference document for Caltrans decision making, after an event. The steps include the following:

1. Establish Scenario Earthquakes

To develop fragility functions, a set of pre-earthquake scenario events must be selected based on the location of the bridge and the active faults in the vicinity of the bridge site. For the purpose of this project, 26 sets of scenario ground motions were generated based on different magnitude earthquakes on regional faults. These motions ranged from low fault activity and spectral acceleration, through Design Spectra, and spectral acceleration values both less than and greater than design levels prescribed for the site. The characteristics of each motion were identified by moment magnitude (Mw), distance to the fault (R), and spectral acceleration (Sa).

2. Develop Input Ground Motions at the Bridge Site

Using the available site-specific ground motion, generation tools and design spectra, the SSI analytical model customized for the Carquinez site was used to bring the scenario earthquakes to the site and to generate scattered motions.

3. Dynamic Analyses of Bridge under Scenario Ground Motions Demand

The existing detailed Finite Element model of the New Carquinez Bridge [3, 6], developed by SCS, was used in the demand analyses subjected to the scenario ground motions. Drift values of the critical components of the bridge were related to the motion characteristics (Mw, R, Sa). For each critical component, a primary response parameter should be identified. In this project, the proposed approach and scope-of-work is based on the use of Tower 3 drift as the primary response parameter to reflect the damage state of Critical Tower Components, as an example of the process. This methodology can be applied to different primary response parameters to reflect damage status of other critical components

4. Pushover Analysis Capacity

A Finite Element model of Tower 3 was used to perform pushover analysis. Values of drift and strain (concrete and reinforcement) were extracted and correlated with those from demand analysis.

5. Evaluation of Tower Drift and Component Damage (relationship between demand and capacity)

Governing tower drifts as the primary response parameters were documented vs. motion characteristics (Mw, R, and Sa), and finally a series of relationships between the ground motion characteristics (Mw, R, and Sa), Tower Drift, and strain values (damage) of the critical tower were generated.

6. Develop Fragility Data versus Earthquake Intensity and Tower Drift

Based on the analyses, the following response parameters were related to the scenario earthquake intensity, fault, and distance to site as shown in Figure 4:

1. Relation between damage states (DS) and strain (Fragility),
2. Relation between strain and drift (pushover analysis)
3. Using (1) and (2): Relation between damage state (DS) and drift (Fragility),
4. Relation between (Mw, R, Sa) and drift (26 time-history analyses

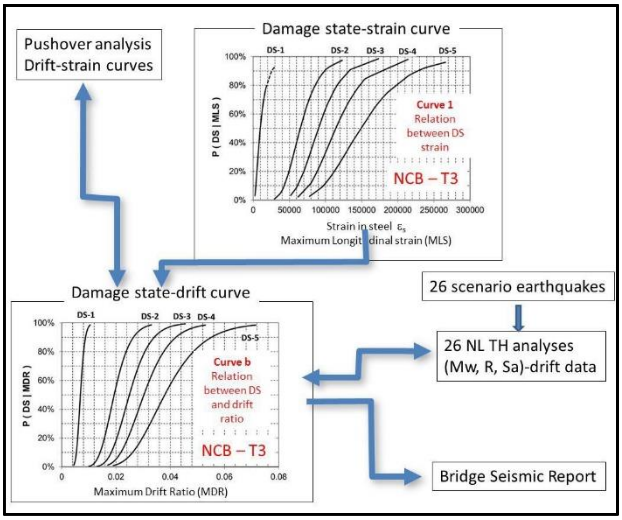

Figure 4 Relationship between Pushover, Time-History and Fragility

4 Seismic Hazard

Results from the seismic hazard memorandum for the Carquinez Bridge [indicate that the ranges of controlling magnitude and distance of earthquakes are from moment magnitudes of 5.8 to 7.9 at distances of 1 to 42 km. Therefore, the recorded seed motions were selected based on these ranges in magnitude and distance [5].

4.1 Local Seismic Design Hazard

For this study, the values of peak design rock accelerations for the New Carquinez Bridge are based on the original bridge design.

4.2 Determination of Dynamic Characteristics for the Conditional Mean Spectra (CMS)

Using the results from the dynamic response analysis conducted on the detailed FE model of the New Carquinez Bridge [13], the dynamic characteristics were readily available to determine the periods, mode shapes, and participation factors that were the major contributors to the dynamic response of Tower 3 in the longitudinal direction. Although there are other modes with larger participation factors in the longitudinal direction, their contribution to the longitudinal participation is very small. As shown in Table 4-2, the largest mass participation factors in the longitudinal direction, which result in the longitudinal oscillation of Tower 3, are at modes 11, 12, 13 and 19 (Figure 4-1). Therefore it can be concluded that the modes having periods ranging from 2.18 to 2.64 seconds were the primary contributors to the longitudinal response of Tower 3. A target period of 2.4 seconds, within the range, was selected as the target period for the Conditional Mean Spectra (CMS). Using well-established procedures to develop CMS [14, 17], it is believed that the CMS would provide more realistic ground motions than the deterministic 84th percentile ground motions from the Ground Motion Prediction Equations (GMPEs).

4.3 Development of the Scenario Earthquakes

The development of the 26 sets of ground motions (each set with two horizontal components and one vertical component) follows standard practices for:

4.3.1 Determining moment magnitude (Mw) and site-to-source distance (R) of earthquake scenarios and site conditions,

4.3.2 Computing horizontal and vertical design spectra,

4.3.3 Selecting seed motions, and

4.3.4 Spectrally matching selected seed motions.

This section presents the details of the procedures used in the ground motion development for 26 different scenario earthquakes [5]. Among these 26 scenario earthquakes, 15 of them are designated to have velocity pulses in order to consider directivity effects from near-fault motions. The percentage of scenario earthquakes with velocity pulses is about 60%, consistent with the fraction of ground motions with velocity pulses used for nonlinear time-history analysis in current state of practice.

5.0 Structural Analysis

A detailed finite element model of the New Carquinez Bridge was developed based on the marked up as-built drawings using the ADINA FE program [3]. All structural components of the new Carquinez Bridge were explicitly modeled. A cross-section of the steel box girders and the bulkhead details are included in the model. The key structural components that were included in the global FE model. Suspension bridges belong to a category of structures that are highly nonlinear in geometry and therefore, during the construction simulation and for their seismic evaluation, large displacement capability was included in the analysis. Geometry iteration was used for the construction sequence of the NCB FE detailed model [3].

5.1 Validation of the Detailed FE Model

SCS used the detailed FE model of the NCB in two major projects in the past. As shown in .and Table 5-2, measured axial forces in the hangers and the measured frequencies match well with those estimated by the detailed FE model of the NCB.

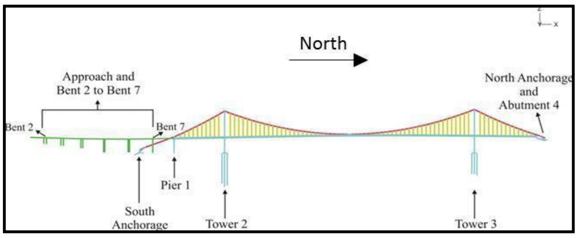

Figure 5 Elevation View of Analytical Model

5.2 Pushover Analysis of Tower 3 (Capacity Calculation) - Drift-Strain Curves

The stand-alone FE model of Tower T3 was developed with a fixed base. The pushover profile is proportional to the first longitudinal mode of tower, which was obtained from the global model. The main reason to perform pushover analysis is to obtain drift-strain curves (capacity), which will be used as an input to the fragility analysis.

5.3 Force-Displacement Curves from Pushover Analysis

The pushover analysis of Tower T3 was performed using the first longitudinal mode of the tower. The inflection point location varies as the push forces increase. The values of strain in confined concrete and steel are also shown in this figure. The steel and concrete strain values along with the location of the point of inflection are summarized in Table 5-3. The steel and concrete strain limits, based on the PS&E design criteria [2] are 0.012, and 0.06 for concrete and steel, respectively. The steel strain reached its limit, before the concrete, and at about a 6-ft displacement at the top of the tower. The maximum relative top-to-bottom displacement of tower T3 from the PS&E analysis is 1.45-ft [88].

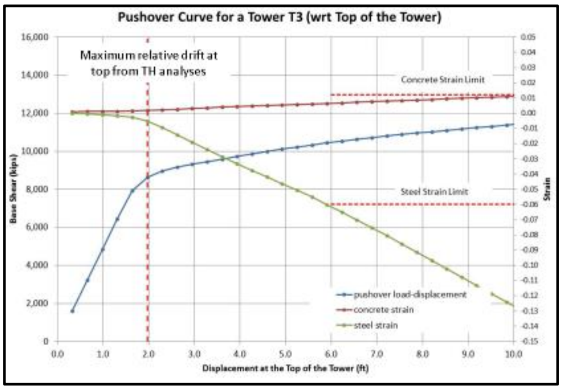

Figure 6 Force-Displacement Relationships of Tower T3 – Total Base Shear versus Displacement

6. Fragility Analysis

This Section presents a summary of the work on the development of fragility relationships for the Carquinez Bridge Tower 3 subjected to earthquakes in the longitudinal direction of the bridge. The purpose of the curves is to provide a probabilistic estimate of damage states as a function of the maximum drift ratio and spectral acceleration (Sa), the moment magnitude (Mw), and the distance to the site (D), expressed as Sa(Mw, D).

The objective of this study was to develop fragility curves for the Carquinez Bridge Tower 3 (T3) using experimental database obtained at the University of Nevada, Reno (UNR), and analytical ADINA response data. More than 100 shake table test data from studies of over 20 reinforced concrete (RC) bridge column models conducted at the University of Nevada, Reno (UNR) was used. The test columns were designed based on recent or current seismic design provisions used at Caltrans .

6.1 Definition of Damage States

Six apparent damage states (DS) were developed for RC columns in cooperation with Caltrans engineers involved in the reconnaissance investigations [20]. These damage states were correlated with different seismic response parameters. The apparent damage states and the corresponding maximum longitudinal bar strains (MLS) were used respectively as limit states and the response parameter. The damage states are defined as:

- DS-1: Flexural cracks;
- DS-2: Minor spalling and possible shear cracks;
- DS-3: Extensive cracks and spalling;
- DS-4: Visible lateral and/or longitudinal reinforcing bars;
- DS-5: Compressive failure of the concrete core edge (imminent failure); and
- DS-6: Failure.

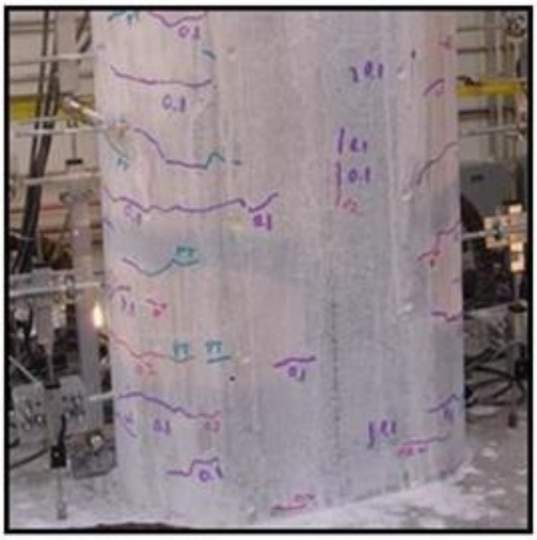

Figure 7 DS-1 Flexural Cracking

As Shown in Figures 7, 8 and 9 the progressive damage states from flexural cracking to imminent failure illustrate the degree of damage for three damage states. Lognormal cumulative distribution function was used to correlate damage states to response parameters. The correlation between the first 5 damage states and MLS is presented in Figure 10.

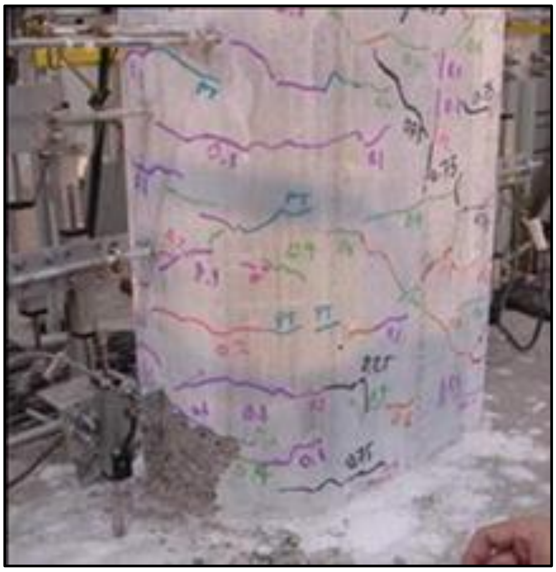

Figure 8 DS-3 Extensive Cracking and Spalling

Figure 9 DS-5 Imminent Failure

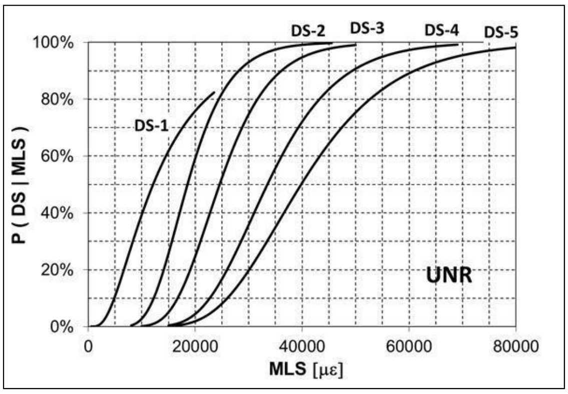

Figure 10 Fragility Curves

7 Conclusions

The proposed procedure recognizes three stages for data processing and decision making, namely: “**Prior to an Event**”, “**Immediately after an Event**”, and “**Post Event**”. Generally, the objectives for the proposed three stages are above in Section 3 Methology.

7.1 Prior to an Event

In this study, the prototype for a procedure was successfully developed to assess the damage immediately following an earthquake using the ground motion parameters of the earthquake event and fragility curves developed for the bridge so that a decision can be made on the continued use or closure of the bridge. This procedure, in this prototype, was implemented the west tower (T3) of the New Carquinez Bridge. For the 26 scenario ground motions the damage was observed for the longitudinal direction using the MDR and the fragility curves. The following steps were taken to successfully evaluate the damage state for the tower in the longitudinal direction. The scope of the current study is to evaluate seismic performance of Tower T3 in the longitudinal direction as “a critical component” of the bridge. The procedure that is present this report can be equally applied to all other key components of the bridge, to produce system-wide fragility information, and base the bridge serviceability decision on the response of the governing key component.

7.2 Immediately after an Event

Data processing related to “Immediately after an Event”, requires recovering measured data from CSMIP sensors after an event and possible enhancement of the modeling procedure and SSI approach.

7.3 Post Event

Data processing related to “Post Event” requires recovering measured data from CSMIP sensors after an even. Comparison of predicted and measured response of the bridge components, can then be used to calibrate analytical model, as well as improve ther key assumptions, such as geotechnical factors affecting Soil-Structure Interaction results In the “Post Event” phase, and after each event, this comparison of predicted and measured response can result in self-improving cycle for the analysis models and geotechnical assumptions, resulting in enhanced predictions for the subsequent events and pre-event data set in Bridge Seismic Report.

8. References

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