Title
Finite Element Analysis of the Meloland Road Overpass and Alternate Model Configurations /

Permalink
https://escholarship.org/uc/item/6415k4d0

Author
Chrupalo, Travis Roman

Publication Date
2014

Peer reviewed|Thesis/dissertation
The Thesis of Travis Roman Chrupalo is approved and it is acceptable in quality and form for publication on microfilm and electronically:

Chair

University of California, San Diego

2014
# Table of Contents

Signature Page ........................................................................................................................................ iii

Table of Contents .................................................................................................................................... iv

List of Figures .......................................................................................................................................... vi

List of Tables ........................................................................................................................................... x

Acknowledgements .................................................................................................................................... xi

Abstract of the Thesis ............................................................................................................................ xii

Chapter 1. Introduction ............................................................................................................................ 1

1.1 Scope of Investigations ...................................................................................................................... 1

1.2 Methodology ..................................................................................................................................... 1

1.3 Thesis Layout ................................................................................................................................... 2

Chapter 2. Description of the Meloland Road Overpass ........................................................................ 4

Chapter 3. Literature Survey .................................................................................................................. 6

Chapter 4. Input Ground Motions ........................................................................................................ 8

4.1 Calexico, CA, April 4, 2010 ............................................................................................................. 8

4.2 Calexico, CA, December 30, 2009 .................................................................................................. 11

4.3 Imperial Valley, CA, October 1979 ............................................................................................... 13

4.4 Caltrans Site Specific Record ......................................................................................................... 16

Chapter 5. Model Configurations and Model Progression .................................................................. 21

5.1 Seat Type Abutments ....................................................................................................................... 21

5.2 Column Transverse Reinforcement ............................................................................................... 25

5.3 Steel Jacket Retrofit ....................................................................................................................... 26

Chapter 6. Benchmark Model Description and Calibration .................................................................. 28
List of Figures

Figure 1 – MRO Sensor Locations ................................................................................. 5
Figure 2 – Calexico 4/4/10 Acceleration Response Spectrum ......................................... 9
Figure 3 – Calexico 4/4/10 Transverse Ground Acceleration Time History ...................... 9
Figure 4 – Calexico 4/4/10 Longitudinal Ground Acceleration Time History ................. 10
Figure 5 – Calexico 4/4/10 Vertical Ground Acceleration Time History ....................... 10
Figure 6 – Calexico 12/30/09 Acceleration Response Spectrum ..................................... 11
Figure 7 – Calexico 12/30/09 Transverse Ground Acceleration Time History ............... 12
Figure 8 – Calexico 12/30/09 Longitudinal Ground Acceleration Time History ............. 12
Figure 9 – Calexico 12/30/09 Vertical Ground Acceleration Time History .................... 13
Figure 10 – Imperial Valley 10/15/79 Acceleration Response Spectrum ....................... 14
Figure 11 – Imperial Valley 10/15/79 Transverse Ground Acc. Time History ............. 15
Figure 12 – Imperial Valley 10/15/79 Longitudinal Ground Acc. Time History ............ 15
Figure 13 – Imperial Valley 10/15/79 Vertical Ground Acc. Time History .................... 16
Figure 14 – Calculated Spectra for MRO (dap3.dot.ca.gov/ARS_Online/) ....................... 17
Figure 15 – Caltrans 1979 Imperial Valley Matched Spectra (USGS # 952) .................. 18
Figure 17 – Imperial Valley Fault Parallel Ground Accelerations ................................. 19
Figure 18 – Imperial Valley Fault Vertical Ground Acceleration ................................. 19
Figure 19 – SDC 1.7 Specification of Backwall Width (W_{bw}) ...................................... 22
Figure 20 – Schematic of Longitudinal Abutment Model (MS Bridge Manual) ............. 23
Figure 21 – Representative Longitudinal Abutment Response (MS Bridge Manual) .... 23
Figure 22 – Schematic of Transverse Abutment Model (MS Bridge Manual) .............. 24
Figure 23 – Representative Transverse Abutment Response (MS Bridge Manual) ....... 24
Figure 78 – Longitudinal Pushover Analysis – Abutment Left Force-Disp. ............... 83
Figure 80 – Transverse Pushover Analysis – Abutment Force-Disp. ......................... 84
# List of Tables

Table 1 – Model Configurations ........................................................................................................... 2
Table 2 – Transverse Reinforcing Models ............................................................................................. 26
Table 3 – Modal Analysis (Case 1 and Model a) .................................................................................. 31
Table 5 – Benchmark vs. Measured Displacement Response .............................................................. 42
Table 6 – Benchmark Model vs. Measured Acceleration Response ..................................................... 44
Table 7 – Abutment Displacement Demand, Wingwall Parameter ...................................................... 47
Table 8 – Maximum Transverse Column Displacement, Wingwall Parameter ................................. 49
Table 9 – Damage Limit State (DLS) Curvatures .................................................................................. 53
Table 10 – Displacement Ductility Demands ......................................................................................... 56
Table 11 – Curvature Ductility Demands, Long. Direction, Reinforcing Parameter ......................... 58
Table 12 – Column End Moment Demands, Steel Jacket Parameter .................................................. 63
Table 13 – Column End Shear Demands, Steel Jacket Parameter .......................................................... 63
Table 14 – Longitudinal Abutment Disp. Ductility, Steel Jacket Parameter ........................................ 66
Table 15 – Transverse Abutment Disp. Ductility, Steel Jacket Parameter .......................................... 67
Table 16 – Max. Trans. Response Profile Comparisons, Steel Jacket Parameter ............................ 69
Table 17 – Max. Long. Response Profile Comparisons, Steel Jacket Parameter ............................ 69
Acknowledgements

I greatly appreciate all involved in assisting me throughout the process of constructing this report. I truly appreciate the help I received from Professor Elgamal, Professor Mackie, and Dr. Lu throughout each phase of the project, allowing for its completion, and for that I am grateful. A thank you is also extended to Caltrans for providing the opportunity to help work in this new bridge modeling effort.
ABSTRACT OF THE THESIS

Finite Element Analysis of the Meloland Road Overpass and Alternate Model Configurations

by

Travis Roman Chrupalo

Masters of Science in Structural Engineering

University of California, San Diego, 2014

Professor Ahmed Elgamal, Chair

The primary purpose of this project is to explore the nonlinear seismic response of an ordinary two span bridge using advanced nonlinear computational techniques. For that purpose, MSBridge, a new nonlinear finite element graphical user interface which conducts calculations using the platform OpenSees is employed. The Meloland Road Overpass is used as the benchmark structure for a parametric study of alterations in salient modeling details. The investigated bridge systems are subjected to three component earthquake accelerations scaled to fit an appropriate Caltrans site-specific spectrum.
The examined model configurations address the influence of the transverse reinforcing ratio, and the general effect of implementation of a steel jacket for the column. The various model response characteristics are compared and discussed. Throughout the conducted MSBridge analyses, every modified model responded in a logical fashion with respect to the specified alteration. Therefore, MSBridge proves to be a useful tool when conducting ordinary bridge model analyses. Overall, the conducted numerical studies provide valuable insights into the mechanisms of resistance supplied by the bridge column and its abutments.
Chapter 1. Introduction

1.1 Scope of Investigations

This report will explore a number of model configurations for a single bent, two span highway bridge structure. The graphical user interface MSBridge for the open source software, OpenSees, is the primary employed analytical tool. MSBridge supports modal analysis, pushover analysis, and time-history analysis of both linear and nonlinear bridge systems.

The Meloland Road Overpass (MRO) was selected as the model structure due to availability of seismic records, as well as the wealth of existing literature about it. A benchmark numerical model of this structure is calibrated by previously recorded onsite earthquake records. Acceleration and displacement responses are compared to recorded responses from the structure.

1.2 Methodology

A computer model of the MRO structure is generated for analysis. Modifications to the original model configurations and properties are analyzed using a fully nonlinear time history analysis approach. Table 1 lists the investigated model configurations and system properties.
Table 1 – Model Configurations

<table>
<thead>
<tr>
<th>Models Analyzed</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benchmark</td>
<td>Elastic 6-dof spring abutment model, used in matching recorded motions.</td>
</tr>
<tr>
<td>Seat Type Abutment</td>
<td>Implemented a seat type abutment to replace the elastic 6-dof spring model.</td>
</tr>
<tr>
<td></td>
<td>Tested 2 lengths of the wingwall dimension.</td>
</tr>
<tr>
<td>Transverse Reinforcing Ratio</td>
<td>Tested various layouts of transverse reinforcing steel in the column.</td>
</tr>
<tr>
<td>Steel Jacket Implementation</td>
<td>Modeled two thicknesses of steel jackets attached to the central column.</td>
</tr>
</tbody>
</table>

An acceleration spectrum was generated using the Caltrans Acceleration Response Spectrum (ARS) online tool, and the PEER Database was used to select earthquake records. These records were then matched to fit the acceleration spectrum for the structure location. Using these motions, results of the nonlinear responses of the MRO are presented and discussed.

1.3 Thesis Layout

Chapter 2 of this report goes into detail about the structural system of the Meloland Road Overpass and general details about the site. A layout of the acceleration sensors is shown.
Chapter 3 consists of a brief discussion about earlier findings relates to the MRO structure. Some general background regarding the conducted studies is provided.

Chapter 4 presents the set of ground motions employed for analysis. This chapter will address three of the selected motions that were recorded at the site of the MRO, as well as one record that was spectrally matched to the Caltrans ARS site 5% in 50 year event.

Chapter 5 introduces the model configurations that are analyzed in this report. The models include a benchmark model, set to represent the current structure, as well as a number of modified configurations with seat type abutments.

Chapter 6 presents the results of the benchmark model. These results will include comparisons to the measured responses of the MRO structure and other general response characteristics.

Chapter 7 comprises the results of the modified model configurations. The results are displayed such that each tested parameter is in a section of this chapter. These sections will include the seat type abutment models with a wingwall variation, column transverse reinforcing ratio variation, and finally implementations of a column steel jacket.

Finally, in Chapter 8, the major findings of the report are reviewed and discussed. Possible future studies are suggested.
Chapter 2. Description of the Meloland Road Overpass

Constructed in 1971, the Meloland Road Overpass (Werner et al. 1987, Levine and Scott 1989) is located above Interstate 8 roughly 0.3 miles from the Imperial Valley fault rupture site. Figure 1 displays locations of all of the sensors installed on and around the MRO, as well as the general layout and dimensions of the bridge. The MRO is comprised of two 104 ft spans supported by a single column bent. The bridge deck is a 5.5 ft prestressed reinforced concrete box girder, supported by a 5 ft diameter column of about 21 ft in height. The column foundation consists of 25 timber piles which are connected to a 15 ft square concrete cap. The deck is supported at the ends by integral monolithic abutments. Each abutment is normal to the longitudinal direction, representing a straight (non-skewed) bridge configuration. Each abutment is supported on a row of 7 timber piles along the transverse direction. Lastly, the abutment backwall has a 13 ft height.

The transverse reinforcing ratio $\rho_s$ is defined by Caltrans SDC 1.7 as

$$\rho_s = \frac{(4 \cdot A_b)}{(D' \cdot s)}$$

where $A_b$ is the area of the transverse reinforcing, $D'$ is the diameter of the confined column cross section, and $s$ is the spacing of the transverse reinforcing. The constructed MRO contains a transverse reinforcing ratio of 0.43% as calculated by the Caltrans SDC 1.7. This is important to state as the transverse reinforcing ratio will be a parameter to be further investigated in this report.
Figure 1 – MRO Sensor Locations
(http://www.strongmotioncenter.org/NCESMD/photos/CGS/lllayouts/ll01336.pdf)
Chapter 3. Literature Survey

The MRO is a well-documented structure in the field of seismic structural analysis. Many research papers are available for comparison and evaluation of the work completed in this report. Numerous research papers deal with system identification of this structure, each using different shaking events for modal identification procedures (e.g., Werner et al. 1987, Levine and Scott 1989).

One highly referenced paper is titled Seismic Response Evaluation of Meloland Road Overpass Using 1979 Imperial Valley Earthquake Records by Werner et al. (1987). This paper deals with a methodology of modal identification from the acceleration recordings onsite during the 1979 Imperial Valley Earthquake. Two cases for system identification are discussed, which differ in the selected input acceleration channels used for evaluation. Case 1 deals with a situation in which records at the deck ends and column foundation constitute the acceleration input channels. This case represents deck and column structure movement only, excluding translational movement of the integral abutments. Case 2 uses records of the column foundation and approach ramp locations for acceleration input channels, and therefore results in a system identification of the entire superstructure as well as movement of the abutments.

A publication by Levine and Scott (1989) examines the dynamic response of a simple bridge-foundation computer model. The paper was written based on the 1979 Imperial Valley earthquake, and the system identification conducted by Werner et al. (1987). The model is constructed to be representative of Case 1 from the Werner et al.
(1987) publication. The analysis goes about calculating equivalent abutment rotational spring constants (Appendix A). On this basis, results are compared to the system identification outcome. Transverse mode shape frequency was predicted with an error of -10%, while the vertical symmetric mode frequency was within an error of +8%.

As mentioned previously, the constructed MRO column has a transverse reinforcing ratio of 0.43%. Current typical values range from 1.0-1.5%. Therefore, increasing this ratio by implementing a steel jacket would make for a worthwhile scenario to investigate. Steel jackets are a popular option for retrofit of current bridges built before 1971, an era when columns were built with relatively small amounts of confining steel (low confining steel ratios). The principal intention of the steel jacket is to provide added confinement to the concrete column, which in turn generates a significant increase in the ultimate compressive strain in the concrete (Chai 1996). This increase in the ultimate compressive strain causes an increase in the ductility capacity of the column (Chai 1996). An additional impact that the jacket introduces is the increase in lateral stiffness of the column, which may translate into an increase in the seismic design force level for the structure (Chai 1996).
Chapter 4. Input Ground Motions

The seismic motions discussed in the following sections resulted from three large events that the MRO has experienced since built in 1971 (e.g., Werner et al. 1987, Levine and Scott 1989). In addition to using the onsite recorded motions, a probabilistic response spectrum using a 975 year return period (typical for California Bridges) has been generated. A near fault ground motion recorded in California has been selected from the PEER database to match this spectrum.

4.1 Calexico, CA, April 4, 2010

The April 4, 2010 Calexico Earthquake epicenter was located roughly 33 miles SSE of Calexico CA, at 32.26 °N, 115.29 °W (strongmotioncenter.org). In relation to the MRO, the epicenter was located approximately 38.3 miles South-Southeast. The earthquake was believed to have occurred on the Laguna Salada Fault in Baja California.

The earthquake registered a 7.2 on the Moment magnitude scale, while reaching an intensity of IX on the Modified Mercalli Scale. The event had a focal depth of 6.2 miles. A peak ground acceleration of 0.588 g was measured for this event (strongmotioncenter.org). The measured maximum horizontal ground acceleration at the location of the MRO was 0.213 g, while the maximum horizontal structural acceleration was 0.474 g. Figure 2 below displays the acceleration response spectrum for each direction, and Figure 3–Figure 5 display the acceleration time history records.
Figure 2 – Calexico 4/4/10 Acceleration Response Spectrum

Figure 3 – Calexico 4/4/10 Transverse Ground Acceleration Time History
Figure 4 – Calexico 4/4/10 Longitudinal Ground Acceleration Time History

Figure 5 – Calexico 4/4/10 Vertical Ground Acceleration Time History
4.2 Calexico, CA, December 30, 2009

The December 30, 2009 Calexico Earthquake epicenter was located 23 miles SE of Calexico CA, at 32.47 °N, 115.20 °W (strongmotioncenter.org). In relation to the MRO, the epicenter was located approximately 25.4 miles South-Southeast.

The earthquake registered a 5.8 on the Moment magnitude scale, while reaching an intensity of VII on the Modified Mercalli Scale. The event had a focal depth of 3.7 miles (strongmotioncenter.org). A peak ground acceleration of 0.352 g was measured for the event. The measured maximum horizontal ground acceleration at the location of the MRO was 0.169 g, while the maximum horizontal structural acceleration 0.509 g. In comparison to the Calexico 4/4/2010 event this motion has a lower PGA at the MRO site, while producing a larger maximum structural response. Figure 6 below displays the acceleration response spectrum for each direction, and Figure 7– Figure 9 display the acceleration time history records.

![Calexico 12/30/09 Acceleration Response Spectrum](image)

*Figure 6 – Calexico 12/30/09 Acceleration Response Spectrum*
Figure 7 – Calexico 12/30/09 Transverse Ground Acceleration Time History

Figure 8 – Calexico 12/30/09 Longitudinal Ground Acceleration Time History
4.3 Imperial Valley, CA, October 1979

The October 15, 1979 Imperial Valley Earthquake epicenter was located south of the California – Mexico border a few miles East of Mexicali, located at 32.64 °N, 115.33 °W. Ruptures along parts of the Imperial Fault, Rico Fault, and the Brawley Fault Zone triggered the earthquake. In addition, the Superstation Hills Fault and San Andreas Fault indicated some movement (Brady et al. 1980). In relation to the MRO, the epicenter was located approximately 9 miles southeast, with a much closer fault distance of 0.31 miles.

The earthquake registered a 6.6 on the Moment magnitude scale and an intensity of IX on the Modified Mercalli Intensity Scale. The event had a focal depth of 7.45 miles (Brady et al. 1980). Figure 10 below displays the acceleration response spectrum
for each direction, and Figure 11– Figure 13 display the acceleration time history records.

The measured peak ground acceleration for the event was 1.74 g, and the measured maximum horizontal ground acceleration at the location of the MRO was 0.380 g. The maximum horizontal structural acceleration was 0.482 g. The MRO sustained no visible structural damage due to this the shaking event (Levine and Scott, 1989).
Figure 11 – Imperial Valley 10/15/79 Transverse Ground Acceleration Time History

Figure 12 – Imperial Valley 10/15/79 Longitudinal Ground Acceleration Time History
4.4 Caltrans Site Specific Record

A portion of the conducted studies are based on a spectrally matched earthquake record as a 5% in 50 year Caltrans ARS specified motion. Using this input motion, potential nonlinear behavior of the bridge is investigated.

The Caltrans ARS online tool was used to generate acceleration spectra at the location of the MRO. The tool uses location, as well as the $V_{S30}$ (representative site soil shear wave velocity of the upper 30 m), and calculates a response spectrum using probabilities based on nearby faults. Location of the MRO is very close to many faults, < 1 km from the Imperial Fault and < 20 km from many others. Figure 14 below depicts the outcome of this online tool.
The PEER Database was used to find scaled records that fit the calculated spectra for the MRO site. The specified conditions for the final search included < 10 km from a fault rupture, a strike slip fault type, and a Vs30 in the range of 150 - 300 m/s. A different 1979 Imperial Valley Earthquake record was selected to be spectrally matched to the ARS, taken from El Centro Array # 5 (USGS Station # 952). Each of the ground horizontal components were fit to the full Caltrans ARS, while the vertical components were fit to a 2/3 factor of the spectrum, which is typical. Figure 15 below displays the Imperial Valley matched spectrums vs. the Caltrans ARS spectra.

A time domain matching algorithm was used to match the PEER Database Imperial Valley record to the Caltrans site spectra. RSPMATCH was used to complete this process where the record was altered to match at 100 points per frequency decade between 0.1 and 100 Hz. Each matched point is within a 10% buffer range from the target spectrum at each frequency point. The final acceleration was modified such that the ending displacement and velocity have not been altered from the original record. As
such, the scaled and original time history records are shown below in Figure 16 - Figure 18. The horizontal records clearly show the large pulse associated with near fault seismic motions.

**Figure 15 – Caltrans 1979 Imperial Valley Matched Spectra (USGS # 952)**

**Figure 16 – Imperial Valley Fault Normal Ground Accelerations**
It is typical for Caltrans to use the following earthquake record arrangements. The ground accelerations above are to be analyzed where in one case the primary transverse response characteristics will be from the use of 100% of the transverse (fault normal) direction and 30% of the longitudinal (fault parallel) direction along with 100% of the vertical direction. The second case will analyze the primary longitudinal response characteristics and will be from the use of 30% of the transverse (fault normal) direction and 100% of the longitudinal (fault parallel) direction along with 100% of the vertical direction.
For simplicity, each of the results in Chapter 7 uses this scaled earthquake, and the entirety of the transverse results come from case one above where 100% of the transverse ground motion is used. Similarly, the entirety of the longitudinal results comes from the second case above where 100% of the longitudinal ground motion is used.
Chapter 5. Model Configurations and Model Progression

The following model configurations are analyzed using the Caltrans site specified spectrally matched earthquake records. Each of the different model configurations will investigate the alterations (Table 1) to the response of the structure as carried out from a nonlinear time history analysis. The model configurations are listed below, and have specifics as to the main parameter for study.

5.1 Seat Type Abutments

The MRO is a monolithic structure with integral abutments (Table 1). This configuration will replace the integral abutments with the common current California design, that being the seat type abutment with bearing pads and shear keys. When implementing this abutment type, the wingwall parameter is a large factor in dictating its transverse response. This portion of the study will use 1/3 and 1/2 factors of the backwall length to model the wingwall length (MSBridge Manual 2013). The study of different size wingwalls will address the resistance effects of this bridge component on overall system response.

When performing the analysis using the seat type abutments, current SDC Caltrans models are used. This models include shear key as well as bearing pads (the box girder contains 2 webs as well as the 2 end sections, and so 4 bearing pads are used for this setup). The sand SDC 2010 abutment model has been implemented in MSBridge, and is selected for this analysis. The backwall for the model is set to be 27’
with a backwall height of 5’. As such, models with 9’ and 13’ wingwall sections have been selected for analysis as typical values ranging from 1/3 - 1/2 of the backwall length (Figure 19). All other shear key and bearing pad specifications can be found in Appendix A.

![Figure 19 - SDC 1.7 Specification of Backwall Width (W_{bw})](image)

This seat type configuration will be used in the parametric studies of this report (Table 1), as this is a standard design in California. Figure 20 - Figure 24 display representative schematics of the abutment models along with representative illustrations of force displacement relationships for the transverse, longitudinal, and vertical directions.
Figure 20 – Schematic of Longitudinal Abutment Model (MS Bridge Manual)

Figure 21 shows the overall general response of the left and right abutment model in the longitudinal direction. The particular values in this plot are representative only and do not correspond to any particular simulation in this report. Figure 22 depicts the model configuration in the transverse direction. In Figure 23, a representative overall response is shown of the total combination of the set of transverse nonlinear springs (Figure 22).
Figure 22 – Schematic of Transverse Abutment Model (MS Bridge Manual)

Figure 23 – Representative Transverse Abutment Model Response (MS Bridge Manual)
5.2 Column Transverse Reinforcement

The MRO was designed and constructed to have a transverse reinforcing ratio of 0.43%, which by current standards is low. This study will look at the effect of increasing the transverse reinforcing ratio to various levels, and observing the changes in response via the conducted time history analyses. Currently, MSBridge is not configured to model moment-curvature softening (failure). Consequently, monitoring the computed strains levels is important for the interpretation of the results (in terms of performance based on the attained level of drift). Table 2 below lists the investigated transverse reinforcement scenarios. A moment curvature analysis will be conducted to assess column sections curvature ductility demands based on the resulting nonlinear time history responses.
Table 2 – Transverse Reinforcing Models

<table>
<thead>
<tr>
<th>Transverse Steel Reinforcing Ratio</th>
<th>Factor</th>
<th>Rebar Size</th>
<th>Spacing (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.43%</td>
<td>1x</td>
<td>#5</td>
<td>5</td>
</tr>
<tr>
<td>0.78%</td>
<td>2x</td>
<td>#6</td>
<td>4</td>
</tr>
<tr>
<td>1.60%</td>
<td>4x</td>
<td>#8</td>
<td>3.5</td>
</tr>
<tr>
<td>2.03%</td>
<td>5x</td>
<td>#9</td>
<td>3.5</td>
</tr>
</tbody>
</table>

With the alteration of this parameter (and use of the OpenSees uniaxial fiber elements for the column), minor changes would be expected in and post-peak strength and strain response. In essence, the employed column model will not exhibit a post-peak softening effect. For this reason, a damage limit state curvature will be estimated for each model (Table 2). This damage limit-state curvature will be compared to the computed maximum curvature and the results will be discussed accordingly.

### 5.3 Steel Jacket Retrofit

Steel jackets are a popular retrofit option for reinforced concrete columns in order to increase the capacity and improve performance under lateral loading conditions. Two common steel jacket thicknesses, 1/4 and 3/8 inch will be studied and contrasted. The usual approach to implementation of a steel jacket is to do so throughout the entire height of the column involved, so as not to create any plastic hinge zones with increased plasticity around the end of the jacket. The implementation of the steel jacket in MSBridge involves an additional steel fiber section added to the existing concrete column fiber section. It also includes the confining effect through calculations of uniaxial material properties, increasing the strength and ductility of the confined
concrete core. The current modeling of the jacket specifies a fixed-fixed condition at both ends of the column.
6.1 Benchmark Model Assumptions

The MSBridge benchmark model is composed of beam-column elements to represent the deck and the column. Below is a short list of the main assumptions used in this bridge model construction (Figure 25):

- Fixed base at column foundation
- Elastic 6-dof spring abutment model
- Elastic deck elements
  - 20 elements per deck section (104’ span)
- Nonlinear displacement-based column elements
  - 100 elements along column height (24.1’ height)

Many publications (e.g., Levine & Scott 1989, Mosquera et al. 2009) were consulted during the creation of this benchmark model. The modeling parameters used in these papers, and reported research findings were employed in defining this benchmark model (to be in agreement with earlier studies). The list of these parameters along with further assumptions are presented in Appendix A. The benchmark model of this report aims to furnish a realistic basis for the models that are studied in the second part of this report.
The notion of modeling the MRO integral abutments with linear 6 degree of freedom springs will be investigated herein under weak to mid-level shaking events. The use of elastic springs is a reasonable assumption since each of the previously recorded strong motions showed no visual damage to the structure (Levine and Scott 1988). Therefore, this assumption is based on elastic response of both the integral concrete abutment, as well as the soil interacting with the abutment.

The assumption of using these linear elastic translational and rotational springs will appear practical or impractical based upon the level of the shaking event. Two of the three events used resulted in a reasonable response for the elastic springs. However, one of the events (strongest) was not well represented.

A very important part of creating an accurate model is selection of an appropriate damping level to represent energy dissipation in the system. In this report, a Rayleigh damping model is used with values for the mass and stiffnesses proportions which come from a report by Mosquera et al. (2009). Figure 26 below shows the damping curve for the system, with the listed frequencies referenced to the benchmark system.
6.2 Benchmark Model Development

Upon employing the constants and parameters mentioned above, the first model (Model a) was created using a pinned end connection with three elastic springs with rotational stiffness. A simple finite element bridge model (Levine and Scott, 1989) was constructed for a modal analysis comparison to the system identification conducted by Werner et al. (1987). In the paper by Levine and Scott (1988), simple calculations were used to determine the rotational stiffnesses for these abutment springs. Model a (and the Levine and Scott model) was created to compare the superstructure results to the work done by Werner et al. (1987).

In the publication by Werner et al. (1987), two separate system identification cases are conducted. Case 1 deals with similar pinned end scenario, where the dynamic characteristics are of a subsystem which contains the deck members and center single
column pier. This system uses channels 1, 2, 3, 6, 13, and 19 as input for the system identification (Figure 1). The listed channels are located at the bridge deck end / abutment connection, and the column foundation. Table 3 shows the system identification findings from the publications of Werner et al. (1987) and the modal analysis results of Levine and Scott (1989). These results are compared to Model a (Appendix A) in Table 3 below.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Symmetric</td>
<td>4.74</td>
<td>5.12</td>
<td>5.53</td>
</tr>
<tr>
<td>Transverse</td>
<td>3.72</td>
<td>3.34</td>
<td>4.07</td>
</tr>
</tbody>
</table>

As such, model a results in a reasonable match for the two frequencies shown. Possibilities for the discrepancies include the effect of a fixed vs. flexible foundation at base of column, column height, and other modeling parameters used.

The benchmark model was created directly from model a, by introducing three additional elastic translational springs at each abutment location (Appendix A). The transverse and vertical direction springs were modified such that the fundamental transverse and vertical modes would be similar to Case 2 from Werner et al. (1987) as discussed in earlier sections. The benchmark model is now set to match the fundamental transverse / vertical frequencies. However, in terms of higher mode shapes, there is no data to directly connect the model to other observed frequencies. It is noted that the vertical spring stiffness was also selected to produce a closer match in the time history studies as discussed further in Chapter 6. Table 4 shows the benchmark model tuned to Case 2.
Table 4 – Modal Analysis (Case 2 and Benchmark Model)

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Werner (1987)</th>
<th>Benchmark Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Symmetric</td>
<td>4.56</td>
<td>5.23</td>
<td></td>
</tr>
<tr>
<td>Transverse</td>
<td>2.47</td>
<td>2.53</td>
<td></td>
</tr>
</tbody>
</table>

These two modes (Table 4) represent little about the longitudinal response of the bridge abutment system. No data was presented in Werner et al. (1987) regarding the longitudinal bridge system identification. In order to model the longitudinal spring stiffness, the model responses were compared to measured longitudinal acceleration and displacement time history records, (Chapter 6) and the spring stiffness were tuned to result in a close match.

The height of the column to use in analysis is a very important parameter that greatly influences the frequency as well as the time history response. MSBridge does not support the addition of column extension elements to account for the deck box girder vertical extent (thickness). Therefore, this must be handled by the user as an additional column height. In a PEER (2008) report titled “Guidelines for Nonlinear Analysis of Bridge Structures in California”, the process describes using an extended length of a “rigid link” from the bottom of the superstructure girder box, to the geometrical centroid (Figure 27).
In MSBridge there is no feature which allows for the use of a rigid link element as described above, so the column has been extended to the geometric center of the deck. For the purpose of this report, this has been considered a good approximation when analyzing the results at this location.

6.2 Benchmark Model Time-History Results

MSBridge supports a single 3D input ground motion profile. The three site recorded input ground motions listed in Section 4 have been used in a time-history analysis in MSBridge. The input channels for each ground motion are:

- Transverse – Channel 24
- Longitudinal – Channel 15
- Vertical – Channel 14

The above three channels have been selected as the best-matched set of four possible sets of input motions. These four sets are located at the abutments end, as well as the base of the column and the free field location, with the free field location being the best match in terms of general time history response comparisons.
When observing results from MSBridge it is difficult to directly relate the time history results to the recorded acceleration records. Figure 28 displays the major difference when relating the results from MSBridge (Acceleration, Displacement) and the sensor recordings (Channel 7). This is a difference of 2.4’ from the top of the deck (true) to the top of the column (MSBridge column length which is to center of gravity of deck). The blue (single) lines represent the location of the elements representations used in MSBridge. This discrepancy has been accepted because the primary reason for comparison are not to attempt to exactly replicate the bridge response, as much as to generate a reasonable representative model of the bridge.

Figure 28 – MRO Sensor and MSBridge Response Locations
The following figures display the benchmark model acceleration time history response at the location shown in Figure 28. This response is shown with the recorded acceleration response of Channel 7.

For conciseness, comparisons of the Calexico 4/4/10 event responses are shown below, with the remaining events plots shown in Appendix B. The figures below represent the records described in the input ground motions section. Each of the records below are shown twice, once to show the majority of the time duration of the record, as well as a closer view to better compare the measured response and the MSBridge model response.

Figure 29 and Figure 30 display the transverse acceleration time history response at the deck center for the Calexico 4/4/10 event. Overall, the measured and modeled acceleration time histories display similar response with discrepancies, such as missed peaks, possibly associated with higher frequency response.
The displacement data records are transformed from their published state, as the original records are absolute displacements. These recorded motions were converted to a relative displacement (by subtracting the displacement response at the base of the column from the displacement response at the column top). After this process, significant low frequency response was still apparent, so the data was then filtered to
include frequencies between 0.8 and 13 Hz. This range fittingly omitted the low and high frequencies that were interfering with the actual response in this frequency range. Figure 31 below displays an example of the relative displacement response at both filtered and unfiltered states.

Figure 31 – Example of Data Filtering

Figure 32 - Figure 37 display the displacement time history results from the MSBridge benchmark model, in comparison to the recorded data from for the three test earthquake records. Figure 32 and Figure 33 display the transverse displacement time history response at the deck center for the Calexico 4/4/10 event. The MSBridge model displacement response time history shows a very similar response to the filtered Channel 7 response. Similar to the acceleration time history comparison, the largest discrepancies appear when the model does not appear to occur in the higher frequency
components. Figure 34 and Figure 35 display the transverse displacement time history response at the south abutment (Figure 1) for the Calexico 4/4/10 event. When comparing the abutment transverse displacement time histories, larger dissimilarities begin to appear. The model still appears to track the primary frequency content.

Figure 32 – Column Transverse Disp. Time-History (Calexico 4/4/2010)

Figure 33 – Column Transverse Disp. Time-History (Calexico 4/4/2010)
Figure 34 – Abutment Transverse Disp. Time-History (Calexico 4/4/2010)

Figure 35 – Abutment Transverse Disp. Time-History (Calexico 4/4/2010)

Figure 36 and Figure 37 display the longitudinal displacement time history response at the south abutment (Figure 1) for the Calexico 4/4/10 event. The abutment response in the longitudinal direction encounters many of the same errors listed for the transverse abutment. It is very important to note that the measured response is located at the edge of the abutment, as shown in the sensor layout (Figure 1). It is very clear that
the Imperial Valley record produces the largest level of column nonlinearity, as well as the largest bending moment. The results also show that the Calexico responses maintained a linear response during much of the event.

Figure 36 – Abutment Longitudinal Disp. Time-History (Calexico 4/4/2010)

Figure 37 – Abutment Longitudinal Disp. Time-History (Calexico 4/4/2010)

Figure 38 shows the transverse moment-curvature response located at the base of the column, for each of the input ground motions used.
Figure 39 shows the maximum transverse relative displacement profile (drift ratio) of the central column. Each of these profiles displays the highest curvature near the column base. Table 5 compares the maximum displacement response of the benchmark model to measured data.
Table 5 – Benchmark vs. Measured Displacement Response

<table>
<thead>
<tr>
<th>Event</th>
<th>Transverse Displacement (in)</th>
<th>Measured</th>
<th>Benchmark</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial Valley 1979</td>
<td></td>
<td>1.53</td>
<td>1.17</td>
<td>-24%</td>
</tr>
<tr>
<td>Calexico 4/4/10</td>
<td></td>
<td>0.72</td>
<td>0.60</td>
<td>-17%</td>
</tr>
<tr>
<td>Calexico 12/30/09</td>
<td></td>
<td>0.83</td>
<td>0.76</td>
<td>-8%</td>
</tr>
</tbody>
</table>

When relating the measured maximum displacement responses to the model maximum displacement, the benchmark model is under-predicting the peak displacements by up to around 20%. This could be from a variety of reasons including the model fixed column base versus a base which is allowed to rotate with a rotational
stiffness. This difference at the maximum response quantities is accepted with the reasoning that a small amount of nonlinear action may be also occurring at the abutment, allowing for a larger global displacement of the deck center. These nonlinear actions are not captured with the linear translational springs currently being used at the abutments.

Figure 40 below displays the maximum computed transverse absolute acceleration column profiles for each of the events. The two Calexico events more closely match the measured maximum results for acceleration (Table 6). However, the computed Imperial Valley event produces a significantly larger maximum acceleration, which may be an indicator that the elastic abutment springs are no longer a good representation when an event of this size is modeled.
Table 6 – Benchmark Model vs. Measured Acceleration Response

<table>
<thead>
<tr>
<th>Event</th>
<th>Transverse Acceleration (g)</th>
<th>Benchmark Model</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial Valley 1979</td>
<td>0.49</td>
<td>0.66</td>
<td>33%</td>
</tr>
<tr>
<td>Calexico 4/4/10</td>
<td>0.41</td>
<td>0.39</td>
<td>-5%</td>
</tr>
<tr>
<td>Calexico 12/30/09</td>
<td>0.51</td>
<td>0.47</td>
<td>-8%</td>
</tr>
</tbody>
</table>

Figure 40 – Transverse Maximum Acceleration Profile
Chapter 7. Parametric Study and Analysis Results

A number of configurations were studied using pushover analysis, as well as time history simulation using the Caltrans specified scaled input motion (see Chapter 4.4). Primary results of interest include column moment-curvature and shear force response, transverse force displacement response of the abutments, and acceleration/displacement profiles of the column. These parameters will help assess the ductility performance considerations of the bridge column.

The notion of relative displacement and member ductility is a very important subject in the design and analysis of structures. Caltrans has set specific standards relating to this concept in more recent design and analysis manuals (e.g., SDC 1.7). In this manual, displacement ductility as a measure of the imposed post-elastic deformation on a member is defined as (SDC 1.7):

\[
\mu_D = \frac{\Delta_D}{\Delta_Y(i)}
\]

where: \(\Delta_D\) = The estimated global frame displacement demand.

\(\Delta_Y(i)\) = The yield displacement of the subsystem from its initial position to the formation of a plastic hinge (i)

SDC 1.7 lists target displacement ductility demands for various bridge configurations, based on the bent design. The target displacement ductility demand (SDC 1.7) for a bridge such as the MRO (a single column bridge with a fixed foundation...
configuration), is $\mu_D \leq 4$. Being able to accurately estimate the demand and capacity of a system's ductility allows for contrasting and determining if a built structure is able to perform as specified.

### 7.1 Seat Type Abutments

This section presents a comparison between two bridge models with different abutment wingwall widths. The wingwall width has primary influence on the transverse stiffness and strength of the abutment system. Figure 41 and Figure 42 display the right (north) abutment force displacement time history results when analyzed with the spectrally matched earthquake motion. Table 7 displays maximum displacement at the abutment for the configurations studied.

![Abutment Force Displacement Curve](image)

**Figure 41 – Abutment Transverse Force-Disp., Wingwall Parameter**
When comparing the 9’ wingwall to the 13’ wingwall in the transverse direction (Figure 41), the 13’ wingwall has a larger initial stiffness, as well as a greater ultimate yield force. The overall abutment displacement demand in the transverse direction is lower for the 13’ case (the difference is nearly one inch) of total displacement at the abutment.
Figure 43 and Figure 44 display the corresponding displacement and acceleration column profiles in the transverse direction, respectively. The column profile responses are as expected, with the larger wingwall model experiencing less relative displacement and a larger total acceleration at the deck location. Table 8 displays the maximum displacement at the column top for the configurations studied.
The 13’ wingwall model was selected for all further studies below. Displacement-based pushover analyses were conducted to identify important yield points for the structural response for each direction (longitudinal and transverse). These yield displacements will help in the interpretation of ductility demands under the conducted time history analysis.

In the following section, a yield displacement for the bridge deck center is determined, by looking at the column and abutment yield points. The result from
pushover analysis is shown in Figure 45 and Figure 46. Appendix C displays figures with the response of the column and abutments, where yield points correspond to those shown in Figure 45 and 46 occurring at the same pushover load step (e.g., the yield curvature in Figure 76 corresponds to the yield force in Figure 45 both being at the same pushover load step). It can be also noted that column yield point displacement in Figure 45 is somewhat greater than the corresponding abutment displacement in Figure 77 and column yield transverse displacement in Figure 46 is greater than the abutment yield displacement of Figure 80, that being a consequence of the overall bridge pushover behavior (each corresponding to the same pushover load step). In both cases the, first yield of the structure is at a center deck displacement of about 1.3 in.

![Longitudinal Load Displacement at Deck Center/Column Top](image)

**Figure 45 – Longitudinal Pushover Analysis – Load Displacement**
7.2 Column Transverse Reinforcing

The column concrete core is a very significant part of any structure and it is important to ensure that it delivers adequate strength and ductility to withstand loading demands. A large influence on these parameters is the columns transverse reinforcing ratio.

It is very important to state that the current version of MSBridge beta does not include access to post-peak moment-curvature softening mechanisms. As such, the only model of failure for the column is longitudinal reinforcing fracture. Figure 47 below displays the compression stress strain relationships for the various confined concrete cores with different simulated transverse reinforcing ratios. These values were calculated using Mander’s Model for concrete core properties. The vertical lines represent what will be considered herein as a damage limit state. This damage limit state
is when concrete core crushing will begin to occur and thus result in significant softening thereafter.

![Figure 47 – Stress Strain Analysis – OpenSees](image)

A moment curvature analysis was conducted (Figure 48) to determine the ductility capacity of the column at various levels of transverse reinforcement (after loading the column with the actual bridge gravity dead load). The vertical lines denote the curvature (Table 9) at which the damage state previously discussed is reached (i.e., they correspond to the outer confined concrete fibers reaching the yield strain levels specified in Figure 47).

In Figure 48, it may be seen that the adopted moment-curvature response is quite similar for all cases, with a yield curvature of approximately 0.0001 rad/in or 1.0E-04
rad/in. As such, in the following sections, curvature ductility demands will be referenced to this value. In addition, the 1.3 inch yield displacement of Figures 45 and 46 will be used to estimate displacement ductility demands for all cases as well.

![Figure 48 – Moment Curvature Analysis – OpenSees](image)

<table>
<thead>
<tr>
<th>Reinforcing Ratio</th>
<th>DLS (rad/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.43%</td>
<td>0.00053</td>
</tr>
<tr>
<td>0.78%</td>
<td>0.00083</td>
</tr>
<tr>
<td>1.60%</td>
<td>0.00155</td>
</tr>
<tr>
<td>2.03%</td>
<td>0.00193</td>
</tr>
</tbody>
</table>

A nonlinear time history analysis was conducted for each case using the spectrally matched earthquake record. For this section it is important to note again that MSBridge generates moment curvature response as depicted in Figure 48, so the results
will not display any softening, and judgments related to ductility will be based on logic described above (Table 9).

From the conducted time history analysis, Figure 49 and Figure 50 display the shear force displacement time history response at the column mid-height. As expected, it is seen that the additional confining steel does not greatly affect the overall shear force displacement response (since the moment curvature response of all cases is quite similar as shown in Figure 48).

Based on the above discussions, Table 10 shows the maximum displacements, and the corresponding displacement ductility demands. In this Table, maximum displacement is the peak column top displacement relative to its base. Ductility is calculated by dividing these values by the 1.3 in displacement at first yield as depicted in the pushover results of Figures 45 and 46. Overall, little change is seen in the displacement and ductility demand due to the change in transverse reinforcing ratio (2.59” versus 2.42” for the currently constructed MRO column with 0.43% reinforcing, and the case with nearly 5 times that at 2.03% reinforcing).
Figure 49 – Longitudinal Shear Force-Disp., Reinforcing Parameter

Figure 50 – Transverse Shear Force-Displacement, Reinforcing Parameter
Table 10 – Displacement Ductility Demands

<table>
<thead>
<tr>
<th>Transverse Reinforcing Ratio</th>
<th>Displacement in inches</th>
<th>Longitudinal Ductility Demand</th>
<th>Transverse Ductility Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.43%</td>
<td>2.59</td>
<td>1.99</td>
<td>2.18</td>
</tr>
<tr>
<td>0.78%</td>
<td>2.54</td>
<td>1.95</td>
<td>2.15</td>
</tr>
<tr>
<td>1.60%</td>
<td>2.45</td>
<td>1.89</td>
<td>2.10</td>
</tr>
<tr>
<td>2.03%</td>
<td>2.42</td>
<td>1.86</td>
<td>2.08</td>
</tr>
</tbody>
</table>

As discussed previously, a $\mu_D$ is targeted to be less than 4. In each case above, the largest ductility demanded is 2, within the target range. Even though each of these satisfy the global displacement ductility, local ductility demands must be also assessed.

Generally, the importance of transverse reinforcing ratio is relates to the large increase in the ultimate compressive strain capacity of the concrete core. There are also minor increases in overall concrete strength, and therefore column stiffness. However, these are less vital to the overall maximum ductility capacity that the different column transverse reinforcement cases would exhibit.

Based on the conducted time history analysis, the longitudinal moment curvature relationships can be seen in Figure 51 - Figure 52. Maximum curvature taken from Figure 52 is shown in Table 11. Ductility demand is obtained by dividing these curvature values by the 0.0001 rad/in, taken from Figure 48. The column labeled % of damage limit state (DLS) represents the curvature as a percentage of the damage limit state for each transverse reinforcing ratio as shown in Table 9. As the results in Table 11 include the longitudinal direction only, it is noted that the square root of sum of
squares (srss) combination (to add the contribution from the transverse direction) added on the order of 10% to the maximum curvature totals.

Figure 51 – Longitudinal Moment Curvature at Column Base
Table 11 – Curvature Ductility Demands, Long. Direction, Reinforcing Parameter

<table>
<thead>
<tr>
<th>Transverse Reinforcing Ratio</th>
<th>Max. Curvature (rad/in)</th>
<th>Ductility Demand</th>
<th>% of DLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.43%</td>
<td>0.000481</td>
<td>4.8</td>
<td>91%</td>
</tr>
<tr>
<td>0.78%</td>
<td>0.000471</td>
<td>4.7</td>
<td>57%</td>
</tr>
<tr>
<td>1.60%</td>
<td>0.000465</td>
<td>4.7</td>
<td>30%</td>
</tr>
<tr>
<td>2.03%</td>
<td>0.000467</td>
<td>4.7</td>
<td>24%</td>
</tr>
</tbody>
</table>

In Table 11, it is seen that in the first case, the column base in the longitudinal direction reaches 90% of the curvature set for the damage limit state. The transverse direction could add as much as 10% to the curvature total as mentioned above, bringing the case very close to the damage limit state under the Caltrans specified 5% in 50 year event. When the transverse reinforcing ratio is doubled, the curvature demand as a
percentage of DLS is reduced to 57%, displaying a large increase in the safety factors associated with a brittle failure of the column concrete core. Details about the post-earthquake bridge inspection from the 1979 Imperial Valley event displayed no significant column damage (Werner et al. 1987). In this event, the maximum relative displacement the MRO column top has undergone was nearly 1.5” in the transverse direction (Table 5). Still, the investigated Caltrans 5% in 50 year event might result in larger displacements than the MRO has experienced. An event of this size could result in maximum curvature and therefore concrete core stability concerns if the column top experiences displacements in excess of 2” as shown in Table 10. To further explore this issue, a model with the original monolithic abutment construction is needed.

7.3 Steel Jacket Retrofit

This section deals with modeling of a steel jacket applied to the central column with a specified thickness. The jacket is assumed to have the same steel properties as specified within the column, and is applied throughout the entire height of the column. MSBridge assumes a seamless bond between the jacket and the concrete column, and does not account for any bond slip. In addition, it is assumed to be fixed at the foundation level and at the deck connection.

Figure 53 displays the column moment curvature relationships in presence of a steel jacket. Generally, the response shows an increase in moment capacity, along with a very similar yielding behavior. Ductility of the two steel jacket retrofit columns will have increased as well. However, it would be expected that significant permanent
damage would likely occur in the abutments, much before the column ductility capacity is reached.

Figure 53 – Moment Curvature Analysis – From Pushover Analysis

Figure 54 and Figure 55 display the shear force-displacement curves for the center of the column height for the original case, as well as each case with the steel jackets implemented. In these plots it is very clear that the addition of the steel jacket brings a great amount of linearity to the column shear force displacement at the column mid height. The models with the steel jacket seem to remain nearly linear throughout the entirety of the transverse primary event.
Figure 54 – Longitudinal Shear Force-Disp., Steel Jacket Parameter

Figure 55 – Transverse Shear Force-Disp., Steel Jacket Parameter
The plots in Figure 56 display the maximum moment profiles for each case. These plots show a large difference between the jacketed and unjacketed column. The moment is much larger throughout the entire column profile with the steel jacket, as the jacket has greatly increased the bending stiffness and moment capacity of the column. Maximum moments at the base and top of the column are listed in Table 12.

Consequently, the increase in moment demand from the application of a jacket is of importance. This fixed base steel jacket condition demands nearly twice the moment capacity from the foundation than with the current column. The top of the column also sees a large increase in moment demand for the bent cap connection. The plots in Figure 57 display the maximum shear profiles for each case.
Table 12 – Column End Moment Demands, Steel Jacket Parameter

<table>
<thead>
<tr>
<th>Jacket Thickness</th>
<th>Column Top</th>
<th>Column Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Jacket</td>
<td>9844</td>
<td>10320</td>
</tr>
<tr>
<td>1/4”</td>
<td>15002</td>
<td>19034</td>
</tr>
<tr>
<td>3/8”</td>
<td>15753</td>
<td>20439</td>
</tr>
</tbody>
</table>

Figure 57 – Column Shear Profiles, Steel Jacket Parameter

Table 13 – Column End Shear Demands, Steel Jacket Parameter

<table>
<thead>
<tr>
<th>Jacket Thickness</th>
<th>Column Top</th>
<th>Column Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Jacket</td>
<td>799</td>
<td>815</td>
</tr>
<tr>
<td>1/4”</td>
<td>1424</td>
<td>1348</td>
</tr>
<tr>
<td>3/8”</td>
<td>1454</td>
<td>1444</td>
</tr>
</tbody>
</table>

These maximum section shear forces are not exactly constant throughout the entire profile because they are looking only at the primary direction of loading for each
of the transverse and longitudinal directions. The addition of the steel jacket introduces a larger shear demand. An increase occurs of nearly 1.5 times at the column deck connection, and 1.2 times at the column foundation connection.

The moment curvature response from the sections located at the top and the base of the column are shown below in Figure 58 - Figure 60. These plots display the steel jacket retrofit columns preforming nearly elastically at the column top and base. Therefore, the curvature ductility demands that are required are very small, and nowhere near the ultimate ductility capacity of the column with the steel jacket retrofit.

![Figure 58 – Transverse Moment Curvature, Base Section, Steel Jacket Parameter](image)
Figure 59 – Longitudinal Moment Curvature, Base Section, Steel Jacket Parameter

Figure 60 – Longitudinal Moment Curvature, Top Section, Steel Jacket Parameter
Figure 61 and Figure 62 show the force displacement time history response of the abutments of the models with the steel jacket parameter. Table 14 and Table 15 display maximum values from these force displacement figures, as well as associated ductility demands. These ductility demands were obtained by dividing maximum displacements by the estimated abutment yield displacements in the respective direction (Abutment yield displacements can be found in Figure 77, Figure 78, and Figure 80).

![Abutment Force Displacement Curve](image)

**Figure 61 – Longitudinal Abutment Force Disp., Steel Jacket Parameter**

<table>
<thead>
<tr>
<th>Steel Jacket</th>
<th>Max. Displacement (in)</th>
<th>Ductility Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>2.60</td>
<td>1.67</td>
</tr>
<tr>
<td>1/4”</td>
<td>1.79</td>
<td>1.15</td>
</tr>
<tr>
<td>3/8”</td>
<td>1.62</td>
<td>1.04</td>
</tr>
</tbody>
</table>

**Table 14 – Longitudinal Abutment Disp. Ductility, Steel Jacket Parameter**
Figure 62 – Transverse Abutment Force Disp., Steel Jacket Parameter

Table 15 – Transverse Abutment Disp. Ductility, Steel Jacket Parameter

<table>
<thead>
<tr>
<th>Steel Jacket</th>
<th>Max. Displacement (in)</th>
<th>Ductility Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>2.28</td>
<td>4.08</td>
</tr>
<tr>
<td>1/4”</td>
<td>1.77</td>
<td>3.16</td>
</tr>
<tr>
<td>3/8”</td>
<td>1.67</td>
<td>2.98</td>
</tr>
</tbody>
</table>

The longitudinal abutment force displacement curve (Figure 61) is nearly representing an unyielding backwall in the cases that include a steel jacket. The transverse abutment force displacement curve (Figure 62) also shows a significant decrease in the overall local abutment ductility demand when implementing the steel jacket parameter.
The plots in Figure 63 and Figure 64 display the central columns maximum acceleration and displacement profiles, respectively, for both the primary transverse and longitudinal direction events.

In the column profile plots it is seen that in both the transverse and longitudinal primary directions the implementation of the steel jacket greatly affects the maximum column response profiles. The overall maximum relative displacement response is reduced in each case, while the overall maximum total acceleration response is increased. Table 16 and Table 17 compare these maximum values.
Table 16 – Max. Trans. Response Profile Comparisons, Steel Jacket Parameter

<table>
<thead>
<tr>
<th>Steel Jacket Parameter</th>
<th>Max. Disp. (in.)</th>
<th>% Difference</th>
<th>Max. Acc. (g)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Jacket</td>
<td>2.18</td>
<td>-</td>
<td>1.30</td>
<td>-</td>
</tr>
<tr>
<td>1/4&quot; Jacket</td>
<td>1.50</td>
<td>-31%</td>
<td>1.53</td>
<td>17%</td>
</tr>
<tr>
<td>3/8&quot; Jacket</td>
<td>1.39</td>
<td>-36%</td>
<td>1.53</td>
<td>17%</td>
</tr>
</tbody>
</table>

Table 17 – Max. Long. Response Profile Comparisons, Steel Jacket Parameter

<table>
<thead>
<tr>
<th>Steel Jacket Parameter</th>
<th>Max. Disp. (in.)</th>
<th>% Difference</th>
<th>Max. Acc. (g)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Jacket</td>
<td>2.59</td>
<td>-</td>
<td>1.28</td>
<td>-</td>
</tr>
<tr>
<td>1/4&quot; Jacket</td>
<td>1.79</td>
<td>-31%</td>
<td>1.56</td>
<td>21%</td>
</tr>
<tr>
<td>3/8&quot; Jacket</td>
<td>1.62</td>
<td>-37%</td>
<td>1.56</td>
<td>21%</td>
</tr>
</tbody>
</table>

In Table 16 and Table 17, it is seen that the maximum relative displacement at the column deck connection is reduced by nearly 31% for the 1/4 inch jacket implementation with a 36% reduction for the 3/8 inch case. The total acceleration
comparisons display an increase by roughly 17% in transverse response direction, and 21% in the longitudinal case.

When implementing the steel jacket the overall reduction in relative displacement at the column top is substantial, this reduction in maximum displacement is also seen in the longitudinal and transverse abutment response. This reduction is much more important to the overall system response than the drawback from the increased accelerations of the deck.

The implementation of a well bonded 1/4 inch steel jacket throughout the entire length of the column would reduce the overall required curvature during an extreme ground motion event, and significantly increase the total column curvature ductility.
Chapter 8. Conclusion

In summary, the overall nonlinear analysis and modeling of the MRO Bridge and its various model parameter modifications performed in an expected fashion. The performance of the uniaxial beam-column finite elements for modeling the bridge was satisfactory for the purposes of this study. One shortcoming of the software is the lack of ability to implement an ultimate concrete core strain limit as a mode of column failure. However important response characteristic was monitored in a different way. Specifically damage limit states (Table 9) were estimated and contrasted with demands from the conducted time history analysis.

8.1 Benchmark

The developed benchmark model performed well when compared to the displacement and acceleration records from the MRO event responses. This bridge model performed well under the weaker shaking events that contained no relatively large pulses of load. When such large pulses occurred, the true systems monolithic abutment and the surrounding soil may have behaved with nonlinear principals that were not captured by the elastic springs.

8.2 Seat Type Abutment

The implementation of the seat type abutment in contrast to the integral abutment allowed for inspections of the structural response properties at much larger levels of shaking. The effect of the different modeled lengths for the wingwalls at a
constructed abutment exhibited an overall increased displacement ductility demand with the minimum wingwall dimension tested, along with larger system accelerations and transverse abutment displacement ductility.

**8.3 Transverse Reinforcing Ratio**

The constructed MRO column contains a relatively small amount of transverse reinforcing, resulting in a ratio of 0.43%. This parameter was studied at various possible layouts of spiral reinforcement to increase the amount of transverse reinforcing and contrast the response characteristics between the trial models. An important issue to discuss in this section is the lack of a column concrete strain failure mechanism. The implementation of additional transverse reinforcing introduces a large increase in the ultimate column core compression strain, therefore the ultimate column curvature ductility. This was not incorporated in the models used throughout this report. However, this mechanism was assessed based on estimates of damage limit states.

**8.4 Steel Jacket Retrofit**

The implementation of a steel jacket retrofit introduced a great amount of linearity to the entire system, not just the column relative displacements or curvatures. The abutments saw pronounced decreases in maximum displacements and ductility demands from the implementation. A few drawbacks include increased maximum accelerations throughout the column and at the location of the deck, and a large increase in shear and moment demand at both the column foundation and deck connections. Overall, the steel jacket proved to accommodate the large Caltrans basis ground motions
and the resulting nonlinearities were much reduced compared to the unjacketed case. For bridges with a low column transverse reinforcement (similar to the MRO), a steel jacket implementation would greatly decrease the maximum curvatures and increase the curvature ductility capacity.
Appendix A: Model Constants and Parameters

Benchmark Model:

Elastic Abutment Spring Stiffnesses:
• Longitudinal = 750 k/in
• Transverse = 250 k/in
• Vertical = 3000 k/in

Elastic Abutment Rotational Spring Stiffnesses: (Levine and Scott, 1989)
• Rotation about Longitudinal Axis = 1000000 k-in/rad
• Rotation about Transverse Axis = 20000000 k-in/rad
• Rotation about Vertical Axis = 2600000 k-in/rad

Concrete Column Properties: (Mosquera et al., 2009)
• Compressive Strength $f'_c = 3250$ psi
• Young's Modulus $E_c = 3705$ ksi

Elastic Deck Properties: (Bridge Details)
• Cross Sectional Area = 45.82 ft$^2$
• $I_{\text{Horizontal}} = 222.03$ ft$^4$
• $I_{\text{Vertical}} = 3897.6$ ft$^4$
• Weight per unit length = 7.027 k/ft
• Young’s Modulus = 3705 ksi
• Shear Modulus = 1667.94 ksi

Damping Ratios: (Mosquera et al., 2009)
• Mass Matrix * 0.9134
• Stiffness Matrix * 0.0022

Column Fiber Section: (Bridge Details)
• Force-Based Beam-Column
• Diameter = 60 in
• Longitudinal Bar Size = # 18
• 18 Bars Evenly spaced with 3” of cover
• Transverse Bar Size = # 5
• Transverse Spiral w/ 5” Pitch
• Steel Model = Steel02 (opensees manual)
• Concrete Model = Concrete02 (opensees manual)
OpenSees time integration Parameters

- Krylov Newton
- Gamma – 0.50
- Beta – 0.25
- Timestep – 0.005 sec
- Convergence Tolerance – $10^{-6}$

**Seat Type Model:**

Items from above still apply, modifications only to the abutment model.

SDC 2010 sand model used

**Bearing Pad Specifications:**

- 4 Pads per Abutment
- Bearing Height = 2 in
- Pad Length = 20 in
- Shear Modulus = 0.15 ksi
- Young’s Modulus = 5 ksi
- Vertical Yield Strength = 2.25 ksi
- Yield Strain = 150 %
- 1 in Longitudinal Gap (to reach abutment)

**Shear Key Specifications:**

- 2 keys per Abutment
- Initial Elastic Stiffness 17000 kip/in
- Ultimate capacity (as ratio of dead load) 0.3
  - This comes directly from the SDC 2010 version
- No Transverse Gap
Appendix B: Benchmark Time History Results

Figure 65 – Column Transverse Acc. Time History (Calexico 12/30/09)

Figure 66 – Column Transverse Acc. Time History (Calexico 12/30/09)
Figure 67 – Column Transverse Disp. Time History (Calexico 12/30/09)

Figure 68 – Column Transverse Disp. Time History (Calexico 12/30/09)
Figure 69 - Abutment Transverse Disp. Time History (Calexico 12/30/09)

Figure 70 – Abutment Transverse Disp. Time History (Calexico 12/30/09)
Figure 71 – Abutment Longitudinal Disp. Time History (Calexico 12/30/09)

Figure 72 - Abutment Longitudinal Disp. Time History (Calexico 12/30/09)
Figure 73 – Column Transverse Acc. Time History (Imperial Valley 1979)

Figure 74 – Column Transverse Disp. Time History (Imperial Valley 1979)
Figure 75 – Abutment Transverse Disp. Time History (IV 1979)
Appendix C: Seat Type Abutment Result Figures

Figure 76 – Longitudinal Pushover Analysis - Column Moment Curvature

Figure 77 – Longitudinal Pushover Analysis – Abutment Right Force-Disp.
Figure 78 – Longitudinal Pushover Analysis – Abutment Left Force-Disp.

Figure 79 – Transverse Pushover Analysis – Column Moment Curvature
Figure 80 – Transverse Pushover Analysis – Abutment Force-Disp.
References


Caltrans (1968). As-Built Drawings, Meloland Road Overcrossing. California Department of Transportation, Sacramento, CA.

Caltrans (2013). Seismic Design Criteria (V1.7). California Department of Transpiration, Sacramento, CA


