Extending Application of Simple for Dead and Continuous for Live Load Steel Bridge System to ABC Applications in Seismic Regions- Phase I- Numerical Study

PROGRESS REPORT

Period ending March 31, 2016

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April 2016

ABC-UTC
Research Progress Report
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Table of Contents

Template for Preparing Proposal - ABC-UTC

RESEARCH PROPOSAL DESCRIPTION

A. PROJECT TITLE: Extending Application of Simple for Dead and Continuous for Live Load Steel Bridge System to ABC Applications in Seismic Regions- Phase I- Numerical Study

B. PROPOSAL ABSTRACT

The steel bridge system referred to as Simple for Dead load and Continuous for Live load (SDCL) has gained popularity in non-seismic areas of the country. Accordingly, it results in many advantages including enhanced service life and lower inspection and maintenance costs as compared to conventional steel systems. To-date, no research studies have been carried out to evaluate the behavior of the SDCL steel bridge system in high seismic areas.

The main objective of this research is to extend the application of SDCL to high seismic areas. This project is Phase I of an envisioned effort that will culminate in the development of a set of details and associated design provisions to develop a version of the SDCL steel bridge system, suitable for high seismic application.

C. DESCRIPTION OF RESEARCH PROJECT

Recently a new steel bridge system, referred to as “Simple for Dead Load and Continuous for Live Load (SDCL)” has gained popularity in non-seismic areas of the country. Research conducted over the past 10 years (Azizinamini et al.) has resulted in development of complete design and detailing provisions for this new system for non-seismic applications. Results of this work are now summarized and five journal papers area accepted to be published in 2014 in AISC Engineering Journal.

In the SDCL bridge system there are no bolts or expansion joints in this system, resulting in higher service life, in addition to many other advantages. This system is also best suited for accelerating the construction process. It is ideal for building individual spans off site, transporting them to the final location and then joining the spans over the middle piers to create continuity for live load. Another version of the system which utilizes adjacent beam technology can significantly reduce the onsite construction activities.

C.1. PROBLEM STATEMENT

To-date no research studies have been conducted to extend the applicability of this system to seismic regions, either for conventional or ABC. The main objective of this proposed concept as a research area is to use the available research data generated over the past 10 years and conduct combination experimental and numerical work to develop details and design provisions for extending the application of the SDCL bridge system to highly seismic areas. This project, as Phase I, will concentrate on developing

the suitable details, using numerical analysis and later in phase II, verify the recommendation using experimental testing. The focus of the study will be on using this system in conjunction with accelerated bridge construction philosophy.

C.1.a. Brief Overview of Seismic Design Requirements

Steel bridges are lighter (about 40%) than concrete alternates and, at first glance, are expected to provide better performance in a major earthquake. However, observations from a past earthquake (Astanh et al. 1994) demonstrate that use of wrong details or systems could result in steel bridges sustaining major damage. In the case of the 1995 Hyogoken-Nanbu earthquake in Kobe, Japan (Bruneau et al. 1996; Chung 1996; Shinozuka et al. 1995; Azizinamini and Ghosh 1997) steel bridges suffered damages to superstructure elements (inadequate cross-frame detailing leading to lateral bending of the girder webs near girder ends) resulting in major retrofit activities and closing of major highways, such as the Hanshine Expressway, for more than a year. The Kobe experience demonstrated that even “minor damage” to steel bridges in seismic events can result in types of damage that could be very difficult to repair. In the case of Kobe earthquake, many steel bridges sustained “minor damage” at the end of the girders seating over pier, because of cross frames being connected to web (Azizinamini and Ghosh 1996). Nevertheless these “minor damages” demanded complete replacement of the bridge.

Among the lessons to be learned is that critical elements of the bridge that are difficult to inspect and repair must be protected from any level of damage and remain elastic during the entire seismic excitation.

Seismic input is largely unknown, therefore the design philosophy for buildings and bridges is to work on behavior of the structure under known conditions. Specifically, the design objective is to predefine the damage locations and design them accordingly by providing adequate levels of ductility. In the case of bridges, the preferred damage locations are at the ends of pier columns (formation of plastic hinges). In the direction of traffic, it is preferred to put columns in double curvature as shown in Figure 1. This allows larger portions of the pier column (two plastic hinges vs. one for single curvature) to participate in energy dissipation.

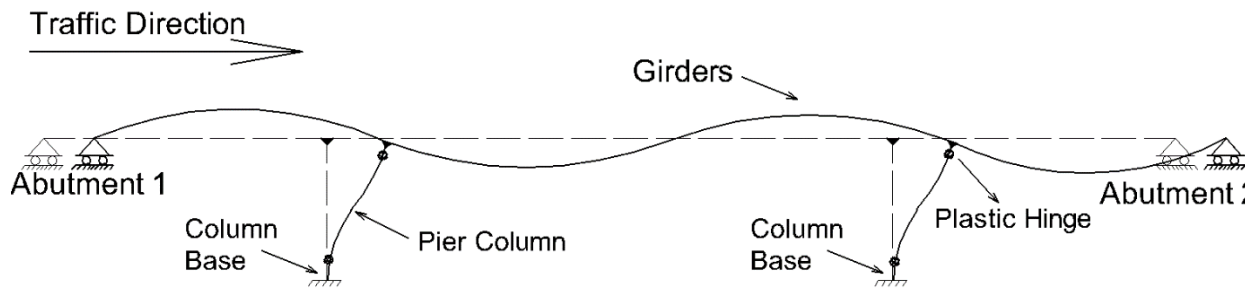

Figure 1. Deflected shape of a three span bridge under longitudinal (along traffic) direction

In the transverse direction, pier columns usually are designed to act in single curvature, as shown in Figure 2.

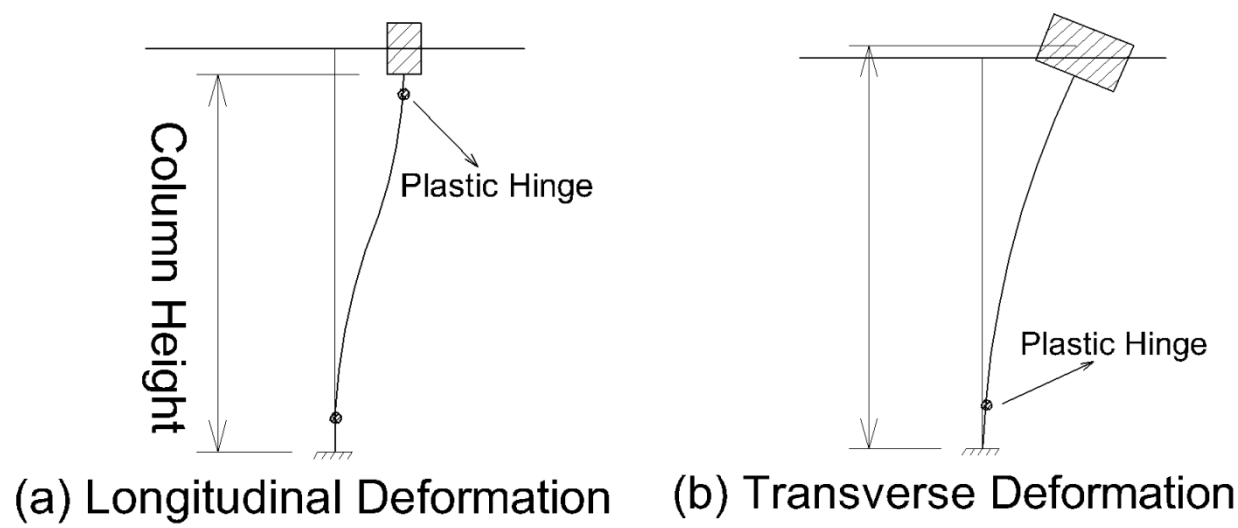

Figure 2. Deflected shape of pier column in longitudinal and transverse directions

Under longitudinal excitation, plastic hinges are located near the top and bottom of the columns, while under transverse excitation; the plastic hinge is located near the bottom of the pier column.

One of the most comprehensive design provisions for design of bridges for seismic events is given by Caltrans Seismic Design Criteria (Caltrans, Seismic Design Criteria, version 1.4, Aug 2008).

The main design feature in seismic design of bridges is to keep the superstructure elements completely elastic during an entire seismic event. These elements are called protected elements. The inelasticity is then forced to take place at predefined locations within the substructure. The predefined damage locations are the weak links or fuses that control the level of forces to be transmitted to superstructure elements. This design approach is referred to as the capacity design approach and is used for designing bridges in seismic regions.

In the capacity design approach, protected elements are designed for the largest possible force effects they might experience, considering over-strength that may exist because of higher actual material strength than that specified in design. The capacities of the bridge elements in the desired damage location (plastic hinge locations) are controlled through design. The plastic hinge regions are also detailed so that they can provide the desired capacities, while deforming inelasticly during a seismic event (ductility through adequate detailing).

C.1.b. Brief Description of the Simple for Dead and Continuous for Live Load Steel Bridge System

Continuous steel bridges are usually constructed so that the system provides continuity for non-composite dead loads in addition to superimposed dead load and live loads. Figure 3 shows a conventional two-span continuous steel bridge girder. For a large number of bridges, the construction sequence consists of first placing a middle segment over the interior support and then connecting the two end pieces using a bolted or welded field splice.

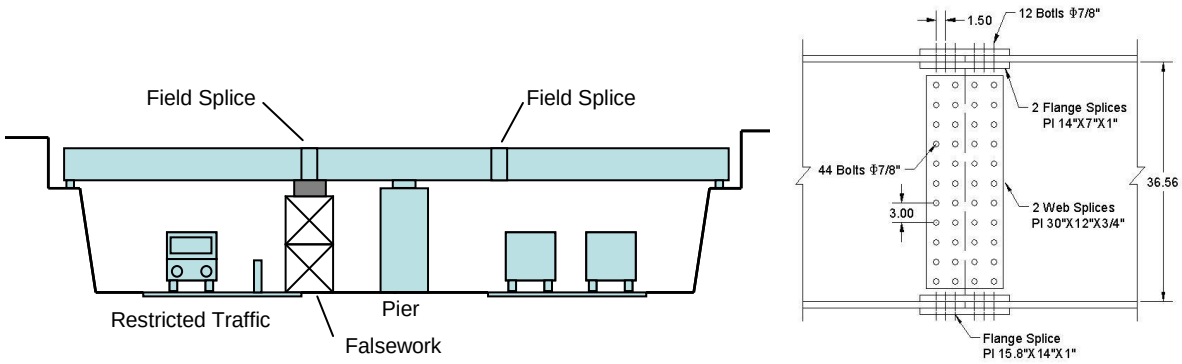

Figure 3. Conventional Two Span Continuous Bridge Girder and Typical Splice Detail

In Simple for Dead Load and Continuous for Live Load system, the girders are spliced over the pier. Girders are placed spanning directly from abutment to pier within each span. The individual spans are simply supported when the deck is cast. Once the deck is in place, reinforcing steel cast into the deck provides continuity of the tensile forces for live load and superimposed dead loads (weight of barrier and future wearing surfaces) only. The compressive component is transferred through direct bearing of the bottom flanges. An example of this detail is shown in Figure 4.

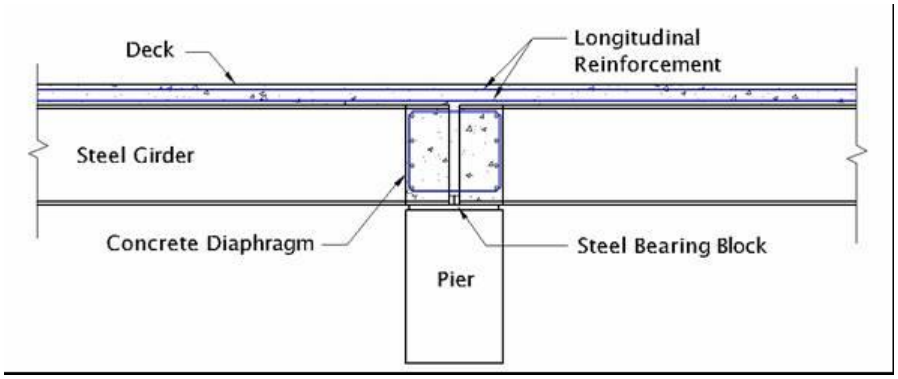

Figure 4. Simple for Dead and Continuous for Live Detail

The main challenge is to use the right detail over the pier to connect the girder ends. Bottom flanges of steel embedded in the concrete diaphragm transmit large stresses to the concrete and, if not detailed correctly, can crush the concrete. Figure 5 shows results of tests on three different details that could be used to connect girder ends over the pier, in non-seismic regions.

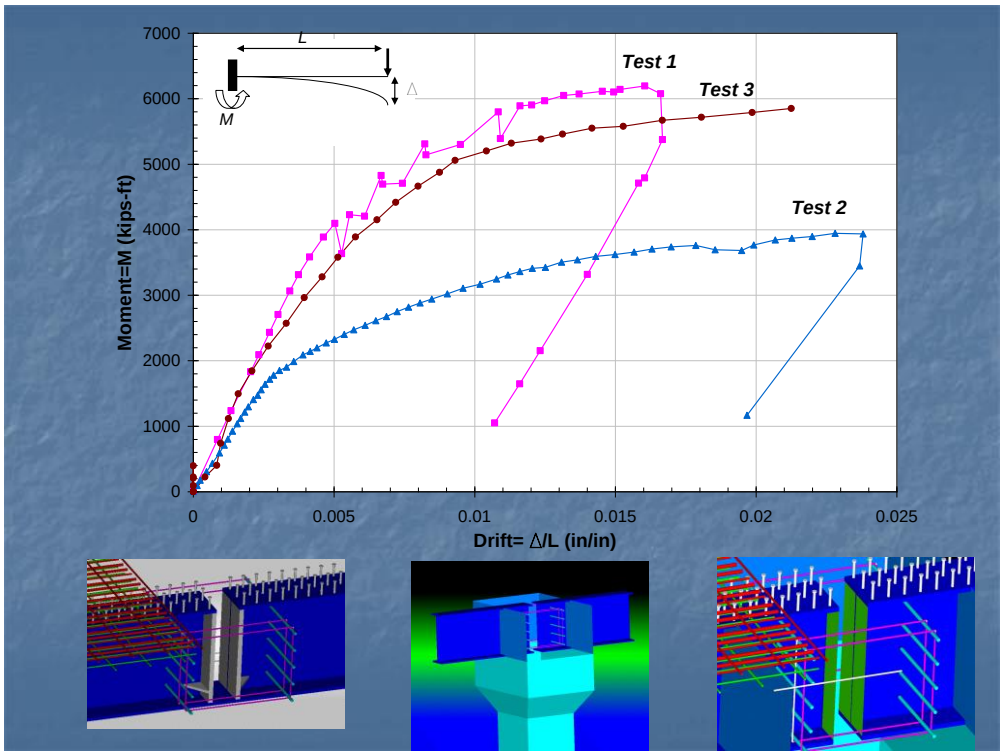

Figure 5. Comparing Performances of Various Details

The main lesson learned is that the bottom flanges of the steel girders embedded in the concrete diaphragm should be connected somehow. This will allow transfer of compressive force (generated by live loads) from one girder to another girder, without passing through the concrete diaphragm. In Figure 5, all three test specimens were

identical, except the details used at the ends of the girders, embedded in the concrete diaphragm. In test No. 2, the ends of the girders did not incorporate any detail and were simply embedded in the concrete diaphragm. As noted, the capacity of the specimen No. 2 is significantly less than the other two specimens, where ends of the girders incorporated details that allowed a smooth path for transferring the compressive force from one girder to another. In seismic design there is another challenge that needs to be addressed. In non-seismic applications the forces to be transmitted from the bottom flange of one girder to another girder, in the concrete diaphragm, is predominantly compressive, and the detail to be used is not required to handle tension force. Figure 6 shows a detail that is suitable for non-seismic application and used in several bridges in service. However, in seismic application, the types of details shown in Figure 6 will not work. In seismic areas, the detail at the ends of the girder in the concrete diaphragm region needs to be able to resist cyclic loading, which will involve both tension and compression, whereas in non-seismic application it only needs to resist compressive-type force. Further, in seismic applications the entire concrete diaphragm region, including the girders ends and details needs to remain completely elastic (protected elements) during the entire seismic event. An additional requirement is that the pre-defined damage areas (plastic hinge locations) must be forced to be at some distance away from the concrete diaphragm region, allowing repair and inspection after a major seismic event.

Figure 6. Detail used in Non-Seismic regions

C.1.c. Simple for Dead and Continuous for Live Load System Using Adjacent Beam Technology

The adjacent box concept utilizes prefabricated units consisting of an individual steel box girder topped by a portion of deck slab, as shown in Figure 7. These units are prefabricated and then shipped to the job site. The portion of the deck shown in Figure 7 is cast at the fabrication shop or temporary staging location. Once onsite, the individual units are set into place on the supports adjacent to one another, shown in Figure 8. A longitudinal deck closure strip between the individual units is then cast, thereby joining

them together. At the same time, the turndown over the interior support is cast. The interior turndown connects the spans together and provides continuity between the spans for subsequent loading (live load).

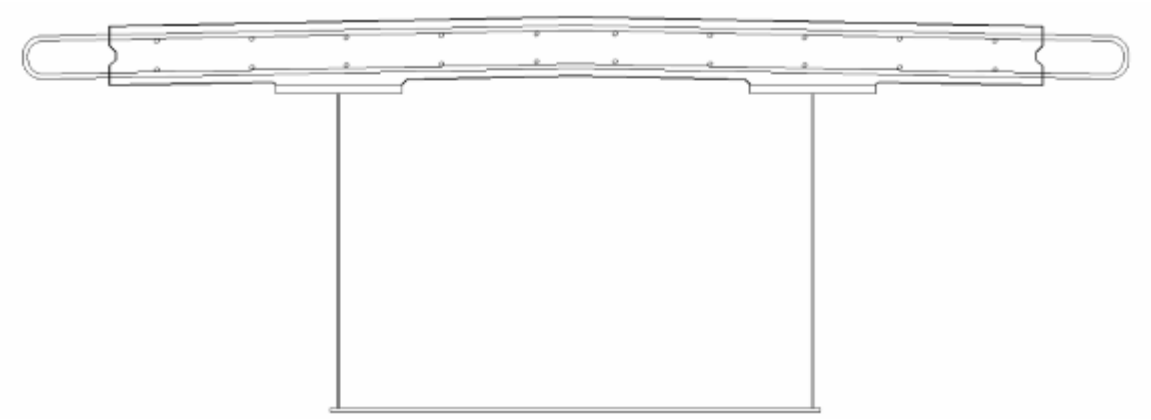

Figure 7. Single (Interior) Box Girder and Deck Unit

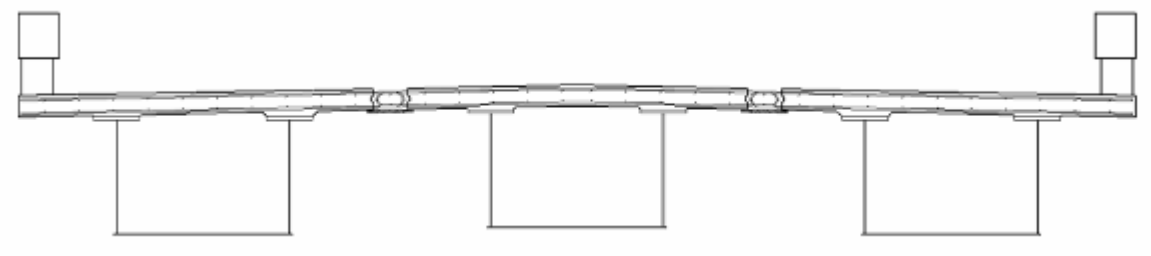

Figure 8. Three Adjacent Box Girder Units

The individual units (steel box and pre-top deck) are relatively lightweight; typically less than 100 tons (Each ton is 2000 pounds) for spans of 140 feet. Once the pre-top girders are placed side-by-side, they are then connected through casting closure pour regions.

Figure 9 below shows the closure region detail. One option is to use headed bars as shown in Figure 10 in closure pour regions.

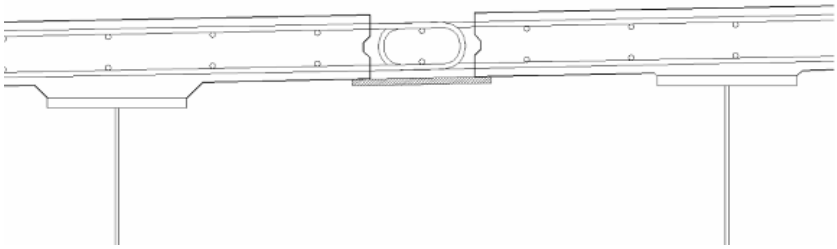

Figure 9. Closure Region Detail

Figure 10. Headed Bar Detail

The Figure 11 shows a rendering of the closure splice region over the interior support. The hooked bar ends from each span anchor the longitudinal reinforcement into the turndown and provide continuity over the support. A similar detail is shown in Figure 12.

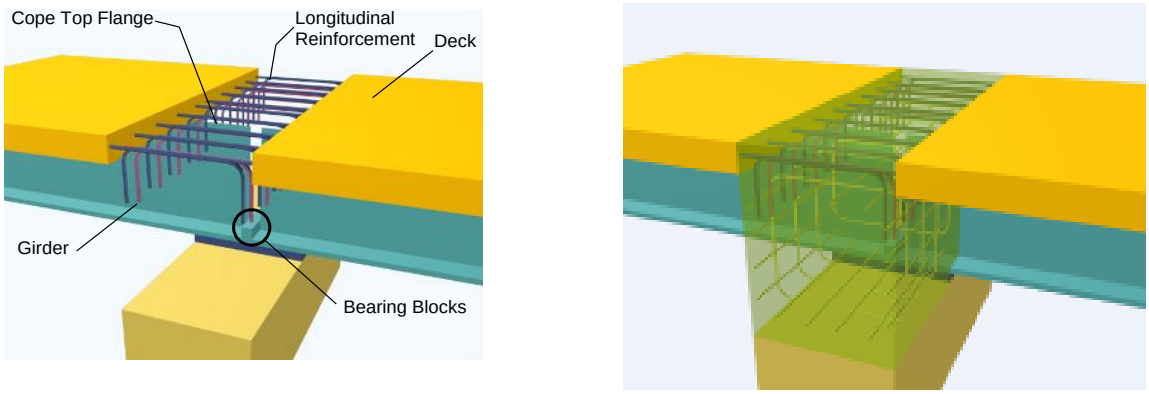

Figure 11. Interior Support Continuity Detail

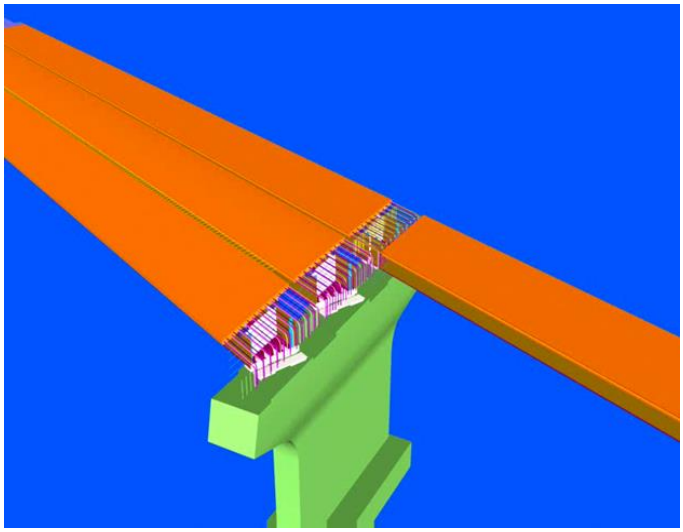

Figure 12. Interior Support Detail Showing Multiple Girders

C.2. CONTRIBUTION TO EXPANDING USE OF ABC IN PRACTICE

Development of the necessary details and design provisions for extending the application of the simple for dead and continuous for live load steel bridge system to highly seismic areas will provide highly seismic areas with an excellent steel bridge alternative that does not have expansion joints and potentially produces long service life, while meeting seismic design requirements.

C.3. RESEARCH APPROACH AND METHODS

An extensive amount of research data for the proposed system in non-seismic regions exists. However, there are specific requirements for seismic applications that need to be addressed. For seismic applications, the detail over the pier should have the following characteristics:

1. It should be adequate to resist reversal and cyclic loads (be designed for tension and compression)

2. The plastic hinge locations should be at some distance away from the girder ends and column ends near bearings

3. Design should follow capacity design philosophy and the detail used to connect the girders over the pier should remain elastic during entire seismic event.

4. The detail connecting the girders over the pier should be monolithic with pier and form a frame action.

C.4. DESCRIPTION OF TASKS TO BE COMPLETED IN RESEARCH PROJECT

To achieve the objectives, the following tasks will be carried out.

Task 1: Develop ideas on types of details that could be used to join the girder ends over the pier and develop a frame action. During a seismic event, the column moment becomes a torsional force for the concrete diaphragm, as shown in Figure 13.

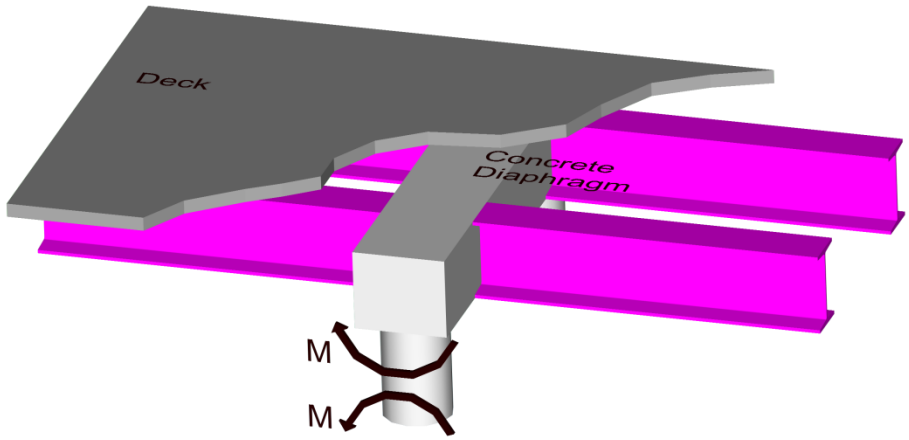

Figure 13. Transfer column moment to concrete diaphragm

The concrete diaphragm will be the protected element and must remain elastic. Therefore, the concrete diaphragm must be designed for maximum credible torsion and remain elastic. Several approaches can be used to ensure that the concrete diaphragm remains elastic under maximum credible torsion. Post-tensioning the concrete diaphragm is one possibility (Patt, Seible, and Uang). Further, the plastic hinge at the column end must form at some distance away from column ends. The detail to be selected will be such that it will create a frame action between superstructure and substructure (integral pier cap). The integral pier cap detail is the preferred choice in the West Coast because of aesthetics considerations.

In the case of seismic application it was determined that the bridge girders over the middle support could very well be subjected to positive moment. i.e. bottom flanges will be subjected to tension and try to pull out of the concrete diaphragm. Figure 14 shows an example of moment envelop diagram that was developed by conducting nonlinear dynamin time history analysis.

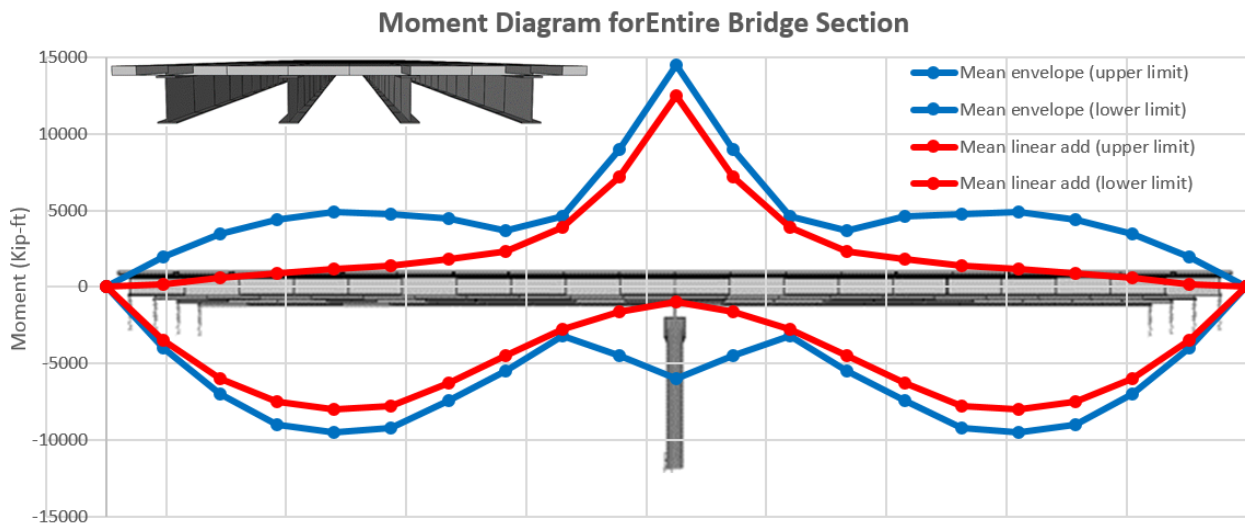

Figure 14 Top: Mean envelope and mean linear add of moment plots for the entire bridge section for all selected ground motions, (Model No. 17).

It was observed that the detail used in non-seismic application (Figure 1) is not suitable for use in high seismic areas. The modified details is shown in Figure 15. In this detail, the connection is similar to non-seismic detail, except that it uses high strength bolts to positively connect the girder ends. It was also observed that amount of dowel bars in the concrete diaphragm needs to be increased, so that concrete diaphragm remains elastic during entire seismic events.

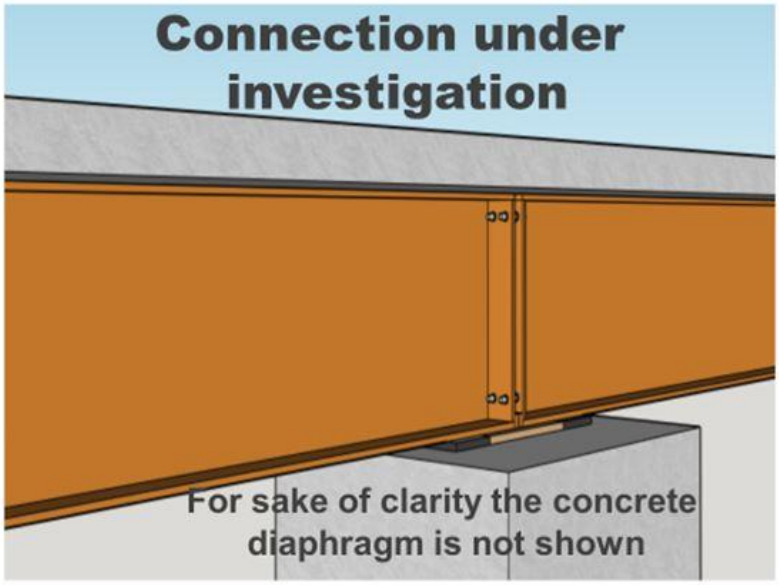

Figure 15 Envisioned connection

The results obtained from the bolted connection indicated that a SDCL connection was feasible for use in seismic areas. The next step was to revise the connection detail in an attempt to simplify and improve the connection. The new proposed connection shown in Figure 16. In this detail, shear studs are attached to bottom flange, which are encircled by horizontal reinforcement ties to transfer the tensile force that may develop during a seismic event.

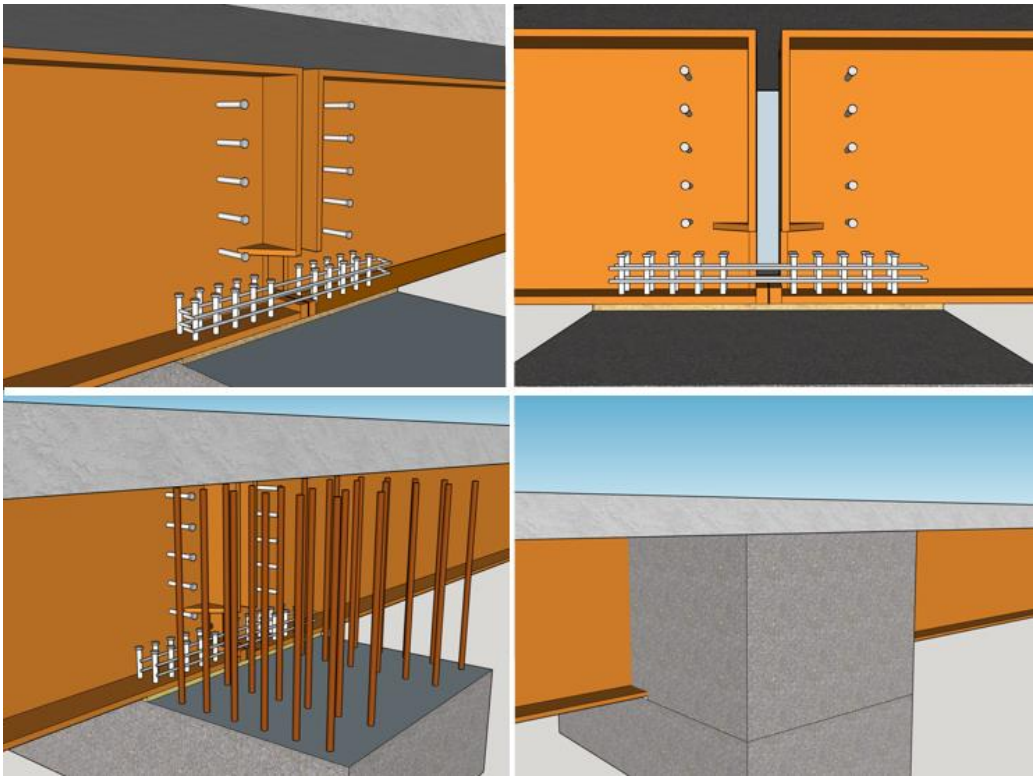

Figure 16 Revised seismic detail of SDCL bridge connection over the pier.

Task 2: Modify the existing detailed non-linear finite element models used by the P.I. to study the non-seismic details to determine the force transfer mechanisms of the details identified under Task 1. Figure 17 shows the finite element model which used to investigate the new detail.

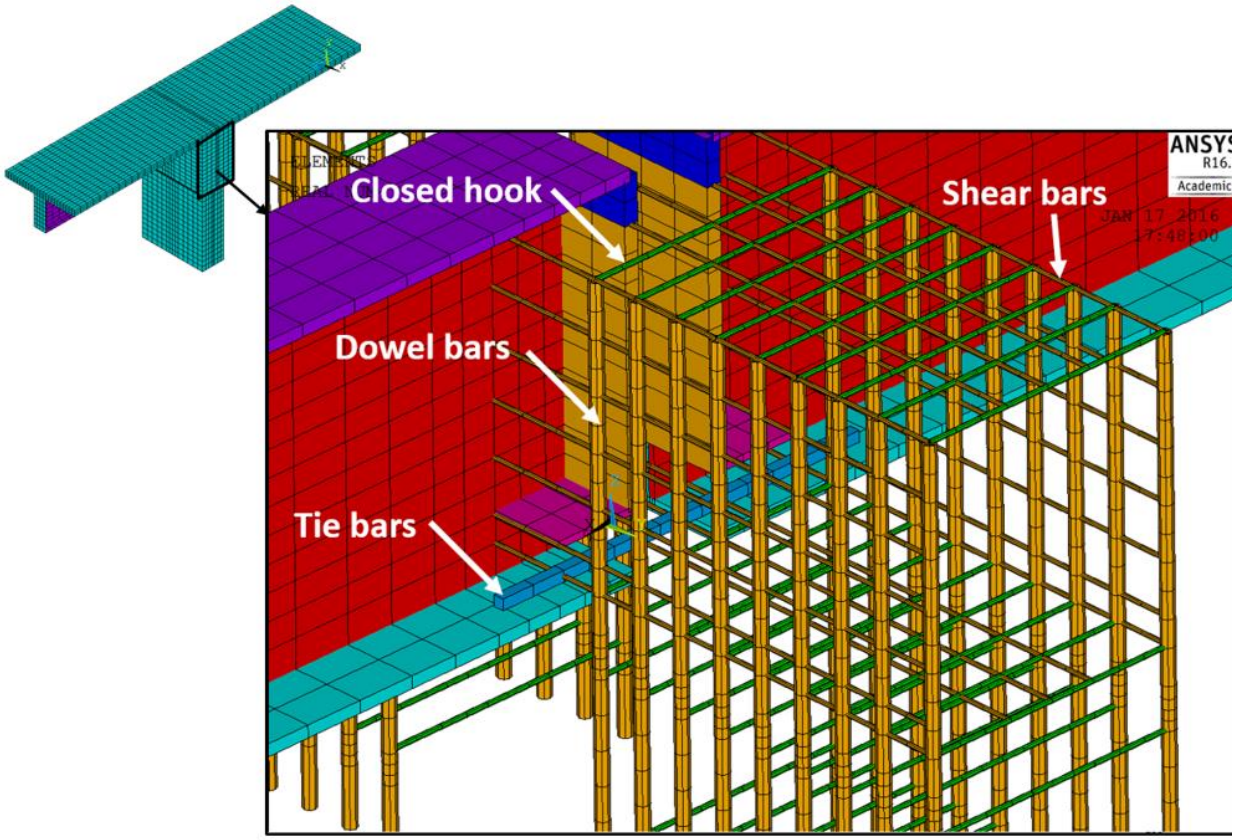

Figure 17. FEM models used to investigate force transfer mechanism for seismic details

Task 3: Using the non-linear FE model, conduct a parametric study to comprehend, both behavior of the proposed connection detail(s) during seismic event at the macro and micro levels. It is expected that, given the past research experiences with SDCL system, project, using entirely numerical study could result in fairly good understanding of the proposed detail(s). Nevertheless these recommendations will need to be verified using experimental tests, which will be the focus of Phase II study.

Using the model shown under task 2, series of nonlinear analysis were carried out to comprehend:

a) Force transfer mechanism for the envisioned connection detail for seismic areas.

b) Numerically identify, as best as possible, the modes of failure.

One of the challenging point in achieving the two objectives listed above was the type of loading that the model should be subjected to. Series of prototype bridges were designed based on Caltrans recommended design provisions and were subjected to various ground motions. However, one could question such approach to establish the demand side, as outcome may change based on the details of the prototypes and

ground motions used. Therefore it was decided to subject the model to three different types of loading as shown in Figure 18.

It should also be mentioned that the model consists of portion of girders on either side of the concrete diaphragm, equal to distance from middle support to point of inflection. The three types of loading considered are

- Pushing down the two ends of the girders.
- Pushing up the two ends of the girders.
- Pushing up one end and pushing down the other end.
- Applying axial force to one end.

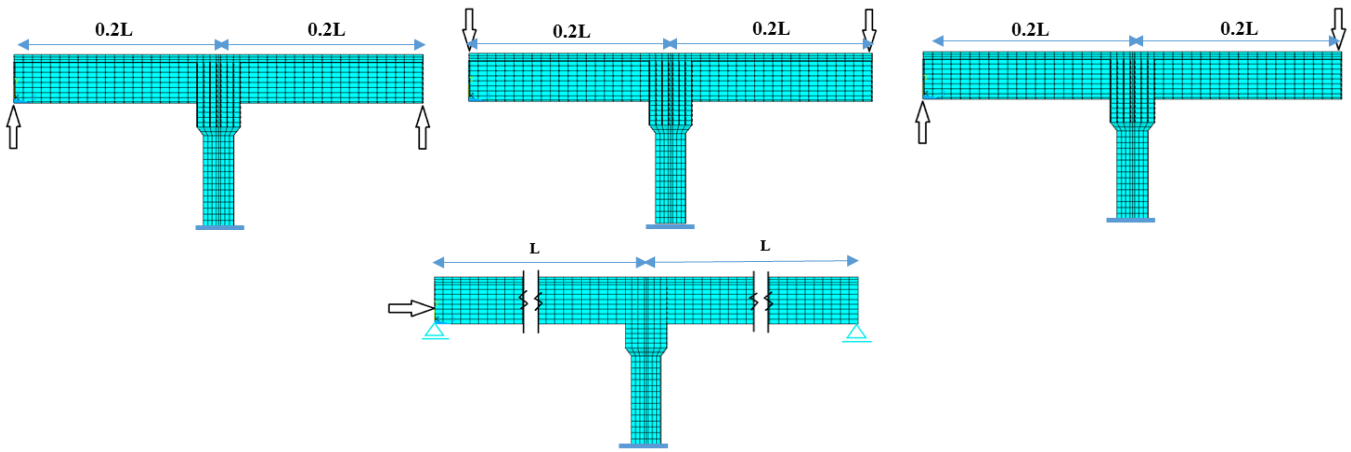

Figure 18 Finite-element model and boundary condition.

Preliminary analyses performed on the proposed details indicated that the two most influential parameters are the amount horizontal tie steel at the bottom flange, and the amount of the dowel bars between the pier cap and diaphragm. A parametric study was conducted to evaluate the behavior of the connection with varying amounts of reinforcement.

Figure 19 shows the results of the analyses when the connection is loaded by push up forces only. The area of dowel bars reported in the figure is the total area of steel at each face of the diaphragm. The area of tie bars is the total area of tie bars at the bottom flange. Also shown in the figure is a line parallel to the initial stiffness offset to a drift value of 0.2%. The value of the moment when the individual curves cross the offset line is the reported maximum capacity used in subsequent discussions.

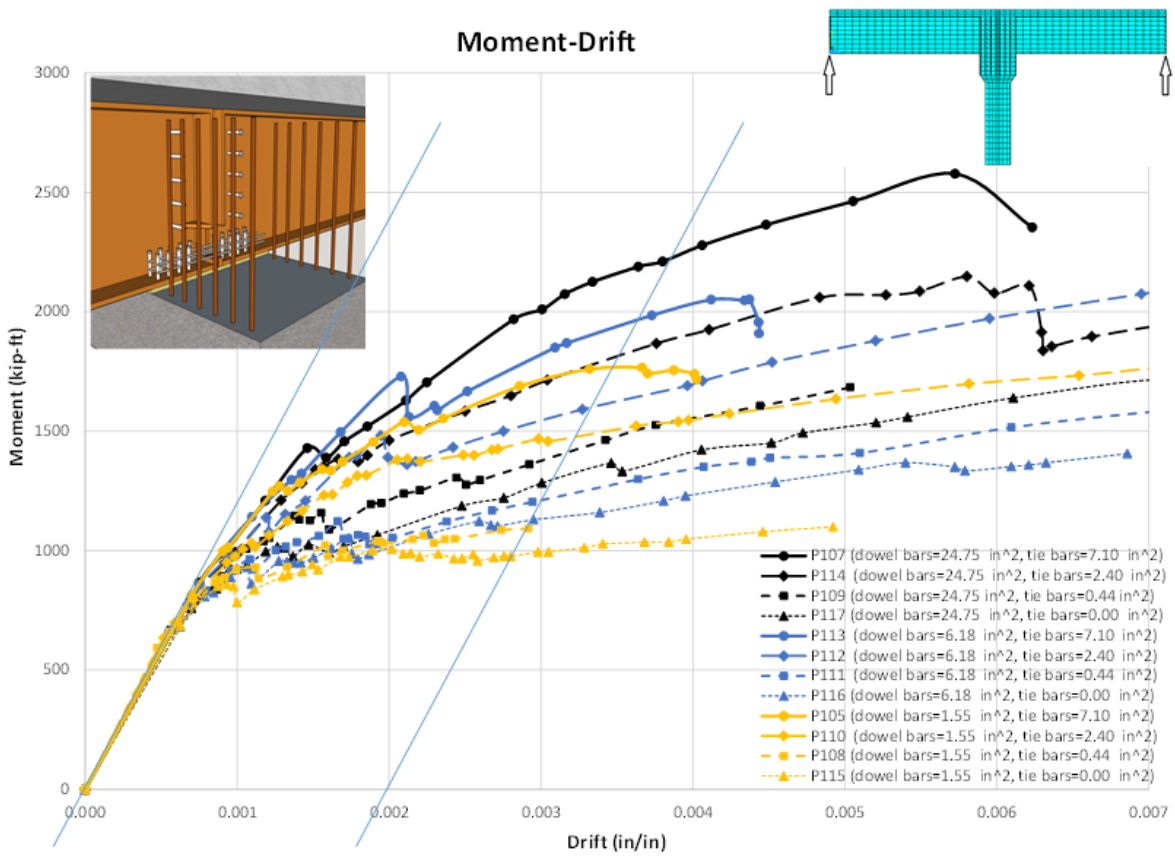

Figure 19 Moment Drift Results for New Detail under Push-Up Loading.

Figure 20 reports the maximum load observed at a drift offset of 0.2% under push-up forces. There are several notable observations that can be made about this data. First, the value of the upward resistance is not heavily influenced by the amount of steel in the dowel bars. Second, the presence of the tie steel increases the capacity of the system. Finally, the inclusion of tie steel is a more efficient means of increasing the capacity versus increasing the dowel steel.

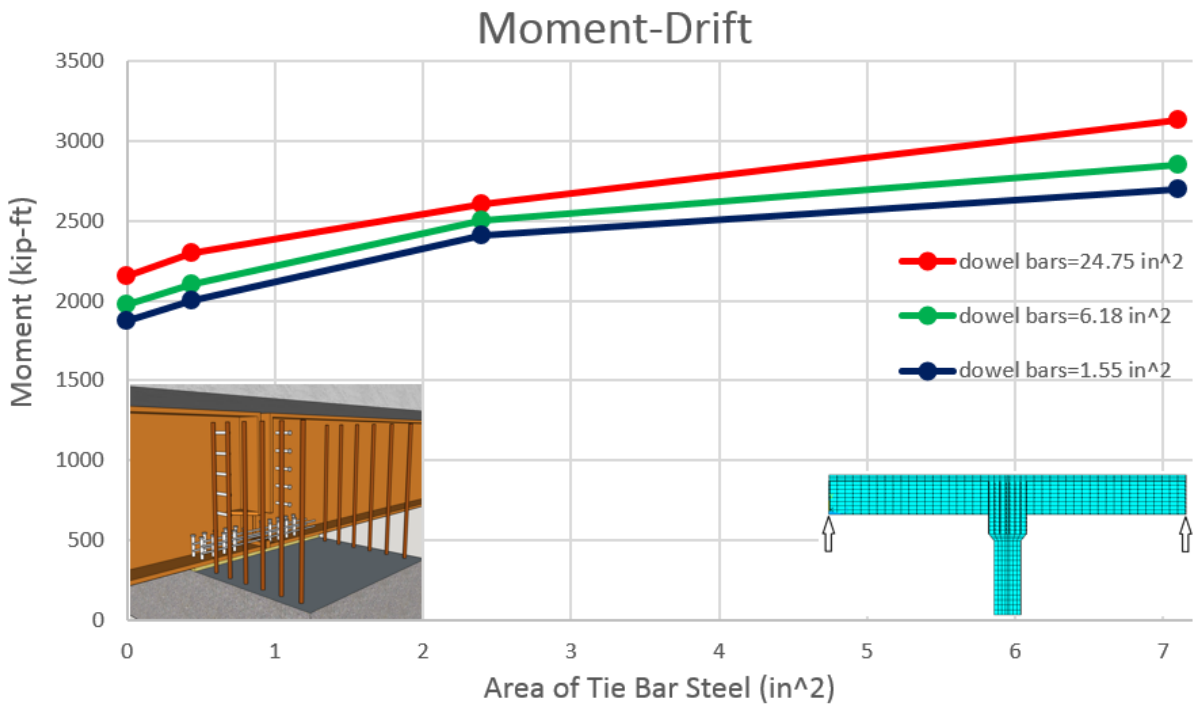

Figure 20 Observed capacity at 0.2% drift offset under push-up forces.

A second series of analyses was conducted for the revised detail to evaluate the performance of the system under the inverse loading pattern. According to Moment-Drift plots in Figure 21, Increasing the size of the dowel bars increases the moment capacity of the system while the tie bars have no affect on the capcaity. Results from the preliminary bolted detail had indicated the dowel bars are the weak link of the system under inverse loading.

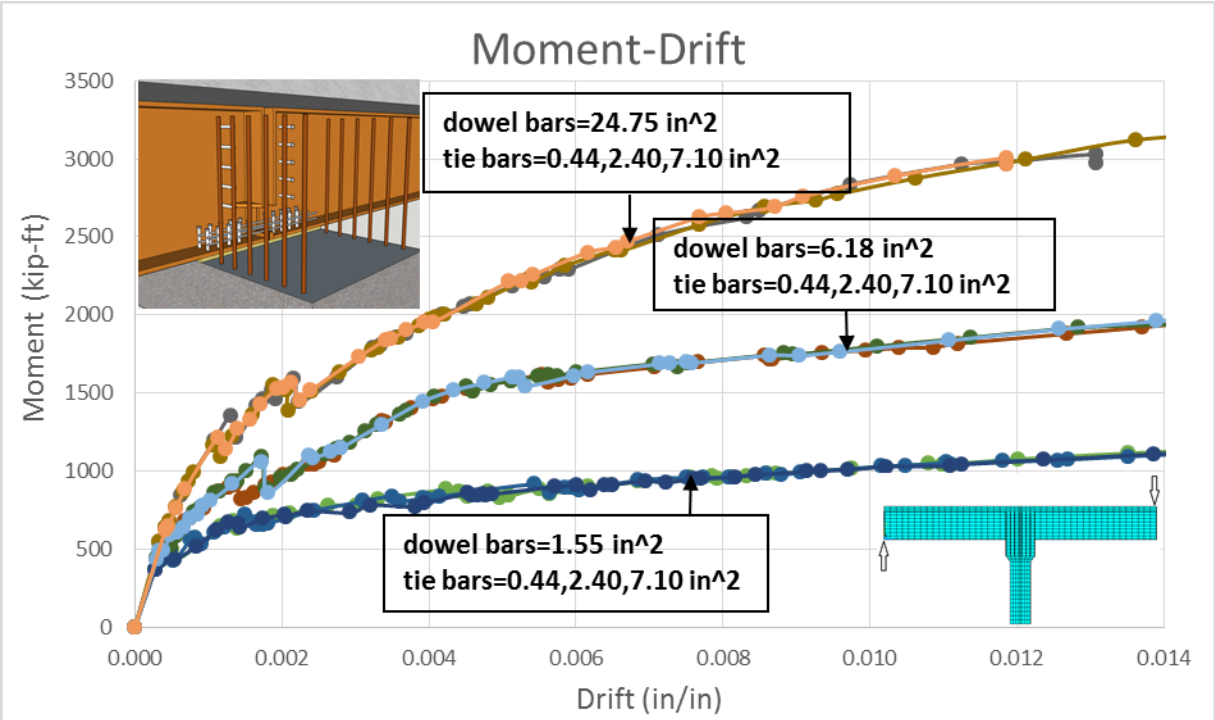

Figure 21 Effect of dowel bars and tie bars in the moment capacity under inverse loading.

Figure 22 demonstrate the moment capacity of the system according to different drift offsets for different dowel bars volume ratio. The results indicates the role of dowel bars are more significant at last step of analysis (larger drift offset) while dowel bars come into account and getting fully nonlinear specially those are located at the vicinity of steel girders. In addition, plots in the Figure 22 shows approximatly there is a linear relation between the area of dowen bar steel and moment capacity of the system.

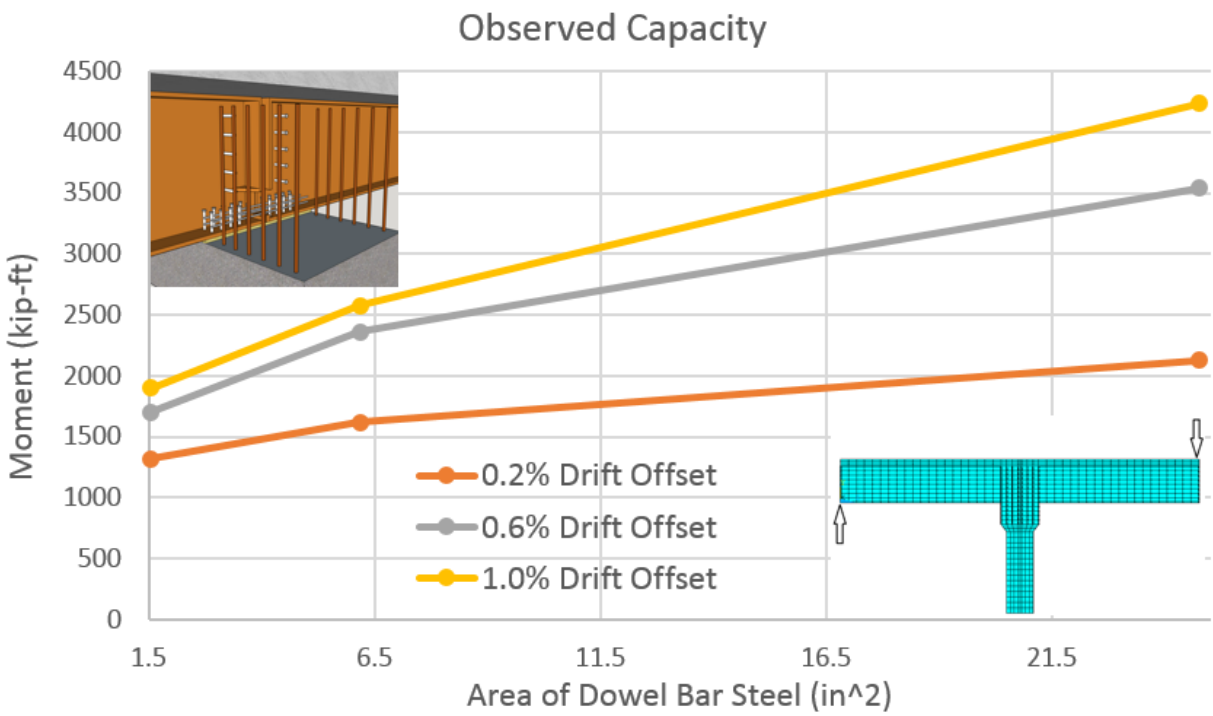

Figure 22 Observed Capacity (0.2%, 0.6%, 1.0% Drift Offset) under inverse loading.

Task 4: Develop an experimental testing program, capable of verifying the project recommendation in Phase II of the study. For this purpose, a column with girders at two side considered to construct in the structure lab of FIU. An schematic view of the component test was shown in Figure *C-23* . According to CALTRANS capacity protected members such as footings, bent cap and superstructure should be designed to remain essentially elastic during seismic events when the column reaches its over strength moment capacity. The moment curvature plot and over strength moment capacity of column illustrated in Figure *C-24*. Therefore, the designed super structure should stay elastic once the column reaches its over strength moment capacity.

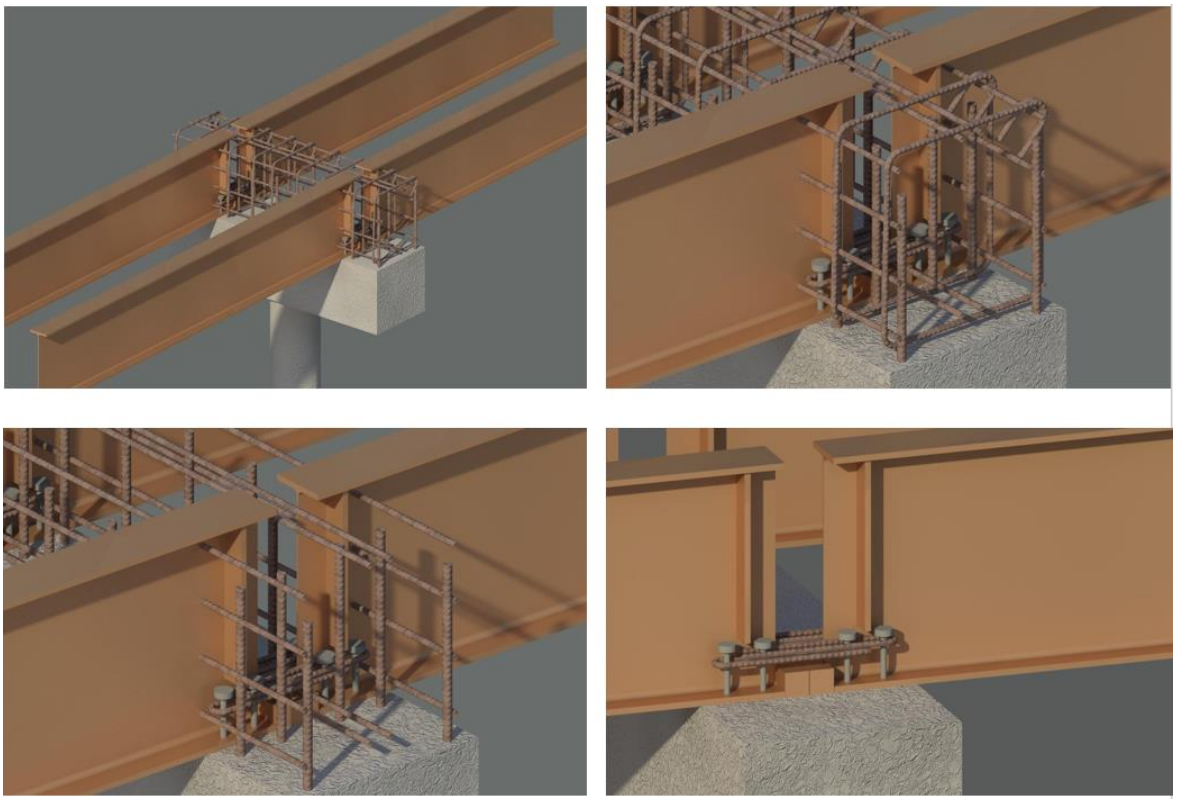

Figure C-23 Schematic view of component test

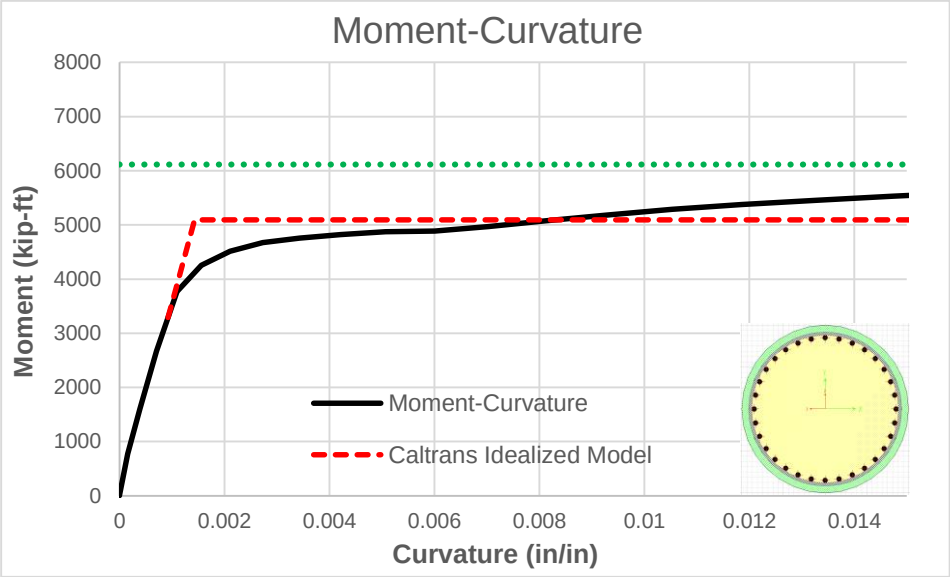

Figure C-24 Moment-curvature for column of component test.

Task 5: A final report will be prepared meeting the RITA requirements for UTC funded projects. The content of the report will provide directions for incorporating future

systems and projects into the database with guidelines to assist categorization consistent with the methodology developed by the project.

D. LITERATURE CITIED

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