USE OF PRECAST I-GIRDERS FOR ACCELERATED BRIDGE CONSTRUCTION IN HIGH SEISMIC REGIONS

Sri Sritharan[1]
Justin Vander Werff[2]
Rick Snyder[3]

Abstract

This paper summarizes the experimental investigation of an integral bridge pier system consisting of a concrete column, I-shaped precast concrete girders, and an inverted-tee concrete cap beam that facilitates accelerated bridge construction methods in seismic regions. The research focus included the behavior of the girder-to-cap connections and the overall system behavior. Two girder-to-cap connections—one that has already been implemented in practice and another that is proposed for future use—were studied in one large-scale test unit, which exhibited good seismic response with successful formation of plastic hinges in the column as intended. Both the as-built and the improved girder-to-cap connections performed well, with the improved connection showing more dependable response than the as-built connection.

---

1. Wilson Engineering Professor, Department of Civil, Construction and Environmental Engineering, Iowa State University, Ames, Iowa
2. Assistant Professor of Engineering, Dordt College, Sioux Center, Iowa; and Researcher, Iowa State University, Ames, Iowa
3. Laboratory Engineer, University of Minnesota, Minneapolis, Minnesota

Introduction

Precast concrete members in bridge systems are appealing because they lend themselves well to incorporating accelerated bridge construction (ABC) methods. In addition, integral column and cap beam systems for bridges utilizing precast concrete girders have several advantages over structures consisting of steel girders or cast-in-place concrete alternatives. However, the use of precast concrete girders for the design of earthquake-resistant bridges is limited, primarily because of the lack of research and design information regarding the connection between critical bridge components (Theimann 2009, Snyder 2010). The research detailed herein was conducted to investigate the seismic design of a bridge system utilizing a cast-in-place reinforced concrete column integrally connected to a cast-in-place concrete inverted-tee cap beam supporting I-shaped precast concrete girders. Of specific interest were (a) ensuring the girder-to-cap connection would have sufficient shear as well as positive and negative moment capacity to enable successful formation of a plastic hinge at the column top, thus exhibiting good overall system behavior; (b) experimental validation and documentation of the girder-to-cap connections; and (c) developing suitable design recommendations and specifications to promote and advance the use of such designs for ABC techniques. This paper focuses on the large-scale experimental validation tests conducted for the connection regions.

New bridge designs and bridge retrofits developed using the capacity design philosophy have proven to perform well compared to structures designed and built prior to advances made as a result of the Loma Prieta Earthquake in 1989 and the Northridge Earthquake in 1994 (Snyder 2010; Priestly, Seible, Uang 1994). Although a significant amount of research has been aggressively carried out, some structural details have yet to be investigated. One such area is the connection between the substructure and the superstructure when precast girders are implemented in the superstructure. Successful completion of analytical and experimental investigations of such connections will enhance the ability to build quality bridges at an accelerated and efficient pace in seismic regions. The improvements associated with using precast components are resulting in these methods becoming the preferred choice over traditional cast-in-place construction techniques (FHA 2009).

Inverted-tee Bent Cap

A particular connection detail that requires further investigation is the inverted-tee bent cap-to-girder connection. The inverted-tee bent cap system can be used for single or multi-column bent configurations and consists of a cap beam in the shape of an upside-down letter “T” that is placed on top of the columns. Precast girders, typically with dapped ends, are then placed with ease in the field on the ledge of the inverted-tee without requiring any falsework. Thus, this connection is well-suited for implementation in ABC methods. The bridge is made continuous for live load by casting a concrete diaphragm around the girders and cap followed by construction of the concrete deck over the length and width of the structure.

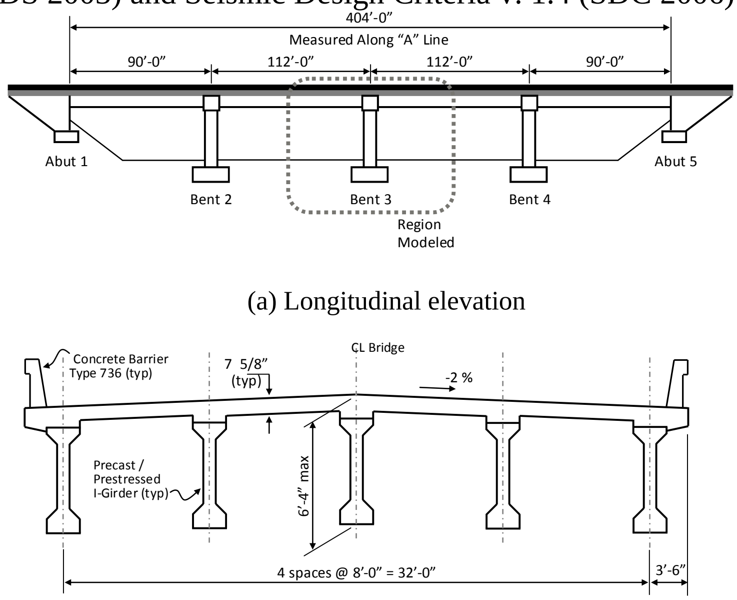
The inverted-tee connection detail has been used in a number of bridges throughout the state of California. When this detail had been implemented, the column was designed assuming a fixed connection at the base and a pin support at the column top adjacent to the cap (SDC 2006). Having a pin support at the column-to-cap connection is not efficient for seismic design, because it prevents the possibility of forming a plastic hinge at the top of the column, thereby increasing the foundation costs and making the precast option cost prohibitive. Although assumed as a pin connection in previous designs, analysis completed as a part of the project presented herein illustrated that inverted-tee connections, when properly designed, can be expected to behave more like fixed connections, with adequate resistance to both positive and negative moment at the girder-to-cap beam connection regions (Theimann 2009, Snyder 2010). The moment resistance of the previously assumed pin connection, along with its effect on the behavior of the remainder of the bridge, had not previously been investigated. Thus, the experimental investigation was conducted to quantify its behavior and possibly lead to design methods that can utilize the moment resistance of an inverted-tee connection for seismic design. Also of interest regarding the inverted-tee connection was the shear force transfer from the girder-to-cap beam, since such a transfer through the inverted-tee connection would be compromised if the girder-to-cap connection experienced deterioration. A further objective of this research was to develop and investigate means by which to improve this type of connection.

The inverted-tee bent cap system has a number of significant advantages over traditional cast-in-place systems. First, the inverted-tee bent caps allow for the use of precast girders. Shop construction results in higher-quality girders than would be produced in the field and allows for economic savings, being well-suited for ABC practices. The benefits of ABC methods have been well documented in recent years and include reduced field construction time and labor, reduced traffic control or divergence and hence reduced congestion, and reduced noise and air pollution (Billington, Barnes, Breen 1999; Caltrans 2008). In addition to ABC benefits, the inverted-tee system decreases the required depth of the superstructure compared to more traditional bent caps. This benefit is especially apparent when girders with dapped ends are utilized. Also, the inverted-tee system requires less supporting falsework than a method that utilizes splicing of the precast girders in the field, because falsework is only required for casting the inverted-tee bent cap itself. Hence, the girders can be placed directly on the bent cap without any direct support from falsework, which results in economic, time, and environmental savings.

Despite the aforementioned benefits, precast components are still not implemented frequently for bridges in areas of seismic activity. It is highly likely that the use of precast construction would become widely accepted in seismic areas if a design methodology were developed and proven to be reliable. The advantages of doing so would be numerable, as already discussed above. Successful improvement and testing of the specific connection between commonly used precast I-girders and an inverted-tee bent cap would likely to increase ABC of bridges in high seismic regions.

Prototype Bridge

The prototype bridge, shown in Figure 1, was selected for the experimental study. This bridge was designed in accordance to the AASHTO LRFD Bridge Design Specifications 3rd Edition with 2006 Interims and California Amendments (AASHTO 2003) as well as the Caltrans Bridge Design Aid (BDA 1995). In addition, Caltrans Bridge Design Specifications (BDS 2003) and Seismic Design Criteria v. 1.4 (SDC 2006) were also used in the design. Computer software packages WinRECOL (TRC/ Imbsen), Xtract (TRC/Imbsent), and Conspan (Bentley 2008) were used to aid in the design. The majority of the prototype design was completed by the structural design firm PBS&J with consideration to finite element work conducted as a part of this project. The outcomes of the finite element analysis as well as discussion and calculations for the design of the column, cap beam, girder dapped end and slab for the prototype have been documented in Thiemann (2009).



(a) Longitudinal elevation

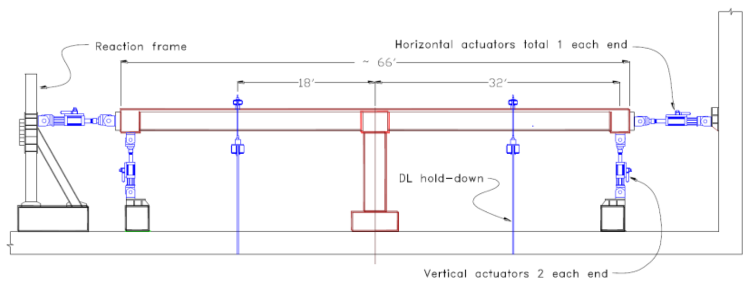
(b) Sectional elevation of superstructure

Fig. 1 Prototype Bridge (1 ft = 0.3048 m; 1 in. = 25.4 mm)

Experimental Unit and Test Plan

The test unit was developed based on a 50% dimensional scale of the center portion prototype structure, which represented a typical inverted-tee bridge. The specifics regarding the design of the test unit are outlined in Snyder et al. (2011). Since the behavior of the connection between the girders and the inverted-tee cap beam was the main focus of this study, only one column, with half of a span on each side, was constructed. Therefore, the test unit consisted of a single column with an inverted-tee cap beam and a superstructure of five I-girders overlaid with a deck on each side. In order to test both the "as-built connection" as well as an "improved connection" without building two test units, one side of the inverted-tee cap beam was constructed using the as-built details while the other was constructed using the improved connection details for the girder-to-cap region. The column was expected to develop a plastic hinge at the top, and thus this region was designed with adequate confinement. Since a majority of the negative moment contribution would be provided by the longitudinal reinforcement in the deck (Hastak, Mirmiran, Miller, Shah, Castrodale 2003), both connections were expected to provide comparable negative moment resistance. As a result, based on whether the superstructure of the test unit was subjected to a horizontal push or pull direction of loading, damage to either of the positive moment connection was expected to be adequately reflected in the overall response of the test unit. Test unit plan details are provided in Figure 2, and further information on the test unit has been documented in Snyder et al. (2011).

Figure 3 provides the details of the test unit's girder-to-cap connection, which was the primary focus of the experimental study. The connection shown in Figure 3 utilized the "as-built" details on the right side, replicating a connection detail that has already been used in practice, while the left side of the connection incorporated an "improved" detail. The enhancement of the improved detail was accomplished using a grouted, unstressed post-tensioning strand connecting each of the girders to the pier cap. In a new bridge design, this unstressed strand would run continuously through both girders on either side of the pier cap. However, since the right side of the pier cap, as shown, was intended to be the as-built condition, the unstressed strand was terminated at the right face of the pier cap.



(a) Schematic of Phase I configuration **(1 ft = 0.3048)**

(b) Photograph of Phase I configuration

Fig. 2 Inverted-tee Test Unit at 50% scale

As is typical for such details, hooked reinforcement was placed between the cap and diaphragm to establish a connection between the diaphragm and inverted-tee bent cap. Additionally, following another common technique, dowel bars were placed within the girders which extended into the diaphragm in order to further establish a connection between the embedded ends of the girders and the diaphragm.

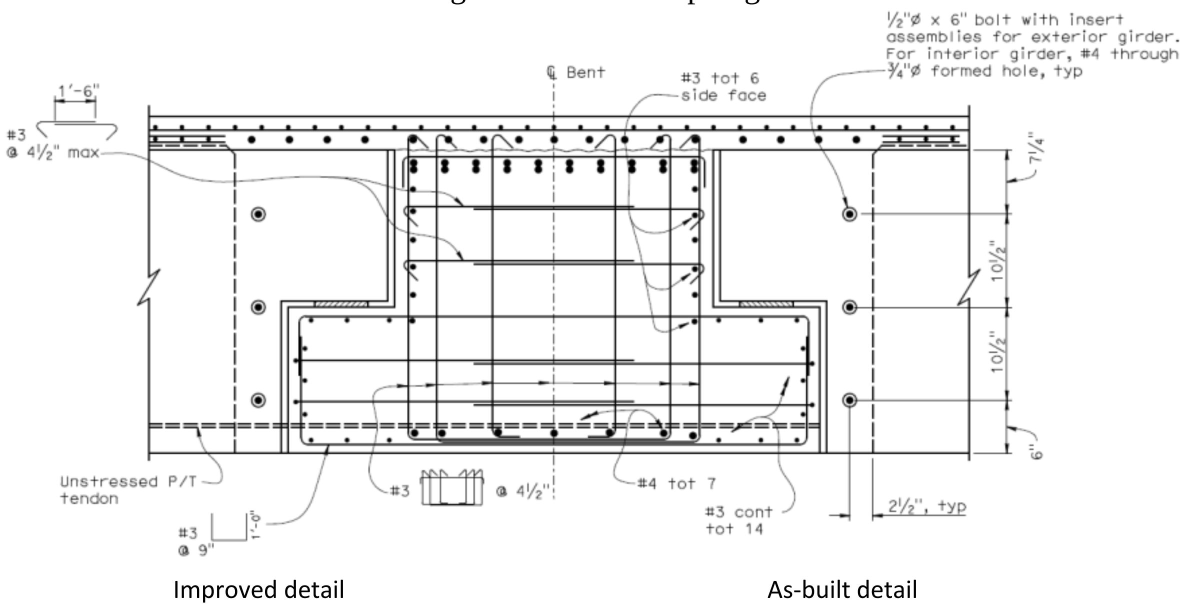


Fig. 3 Girder-to-cap Connection Detail for the Test Unit (1 ft = 0.3048 m; 1 in. = 25.4 mm)

The experimental investigation for this project was divided into two main phases. The first phase, referred to as Phase I, was primarily geared towards investigating the sufficiency of the cap-to-girder connection in having adequate capacity to develop the plastic hinge in the column under simulated gravity and horizontal seismic load. This phase of experimental work is the main focus of this paper. The second phase, designated Phase II, subjected the precast girders to cyclic vertical loads to fully exercise the cap-to-girder connections and to establish the ultimate capacity of the connections.

Phase I testing consisted of horizontal cyclic quasi-static loading of the superstructure. Using two horizontally mounted actuators on each end of the abutment, the superstructure was cyclically pushed and pulled through a series of increasing system displacement ductility levels, μ_Δ, until the specimen reached a maximum displacement ductility of 10. Phase I was intended to investigate the connection's ability to provide good system performance, including successful development of a plastic hinge at the column top just below the cap beam, a key component of such a bridge designed according to the capacity design philosophy for seismic loading.

Throughout Phase I testing, the gravity load effects on the test unit were simulated using two sets of vertical tie-downs and four actuators positioned in the vertical direction. The tie-downs were positioned appropriately to closely model the scaled shear and moment values at the girder-to-cap connection that would be experienced by the prototype structure.

Phase I Test Results

When subjected to a combined gravity and horizontal seismic load in a cyclic manner, Phase I loading of the test unit revealed excellent performance for both the as-built and improved connections as well as for the overall system. Plastic hinges were successfully developed at the top and bottom column ends. The test structure achieved a displacement ductility of 10, corresponding to 7 in. (178 mm) of total horizontal displacement, at which point the buckling of column longitudinal reinforcement and confinement failure in the plastic hinge regions were observed. Both the improved and as-built connections between the precast I-girders and the inverted-tee cap beam behaved as fixed connections and did not show significant signs of degradation. Visual observations revealed less-than-expected degradation of the positive as-built connection and almost no damage to the improved connection. Data analysis following the test confirmed a slight difference in the behavior of the as-built connection compared to the improved connection. Deck cracking that resulted from the Phase I test consisted almost exclusively of transverse cracks that extended across the entire width of the deck. The cracks were more tightly spaced near the cap beam, with spacing increasing further away. This extent of flexural cracking indicated that all of the girders were engaged in resisting the horizontal seismic load.

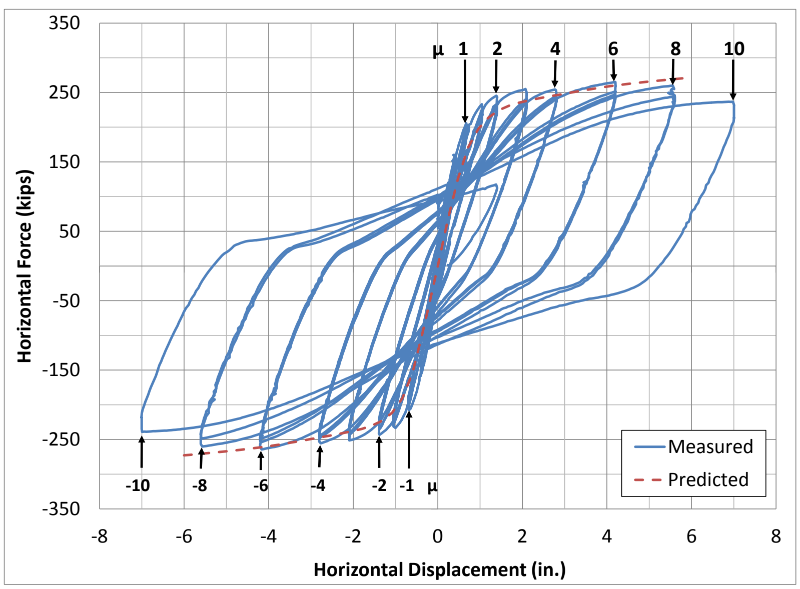


Fig. 4 Comparison of Measured vs. Predicted Force-displacement Response (1 in = 25.4 mm; 1 kips = 4.448 kN)

A comparison of critical data collected during the test to the predictions made prior to the test based on a SAP2000 grillage model analysis (Snyder 2010) showed generally good results. The horizontal force vs. lateral displacement of the superstructure is shown in Figure 4, which shows slight disagreement at small displacements as the grillage model used a effective cracked stiffness for both the column and superstructure sections, rather than the actual gross values for the crack-free stage of the test. However, the analytical and experimental results began to converge progressively with increasing lateral displacement as more of the structure began to soften due to the development of cracks and yielding of longitudinal reinforcement.

Furthermore, the force-displacement behavior in Figure 4 is seen to be quite similar in the two displacement directions, indicating that both the as-built and improved connections behaved similarly, in terms of their contribution to overall system behavior, when exposed to the horizontal seismic load testing in Phase I.

The test unit as a system behaved well, as the connections exhibited excellent seismic performance. The as-built girder-to-cap connections behaved as a fixed connection instead of a pinned connection, contrary to current assumptions (SDC 2006) regarding precast girder connections to an inverted-tee bent cap. This observation suggests that minimal measures would be required to the as-built bridges in order to ensure a satisfactory performance of the inverted-tee/I-girder bridges in the field. It was also established that a satisfactory agreement was achieved between the predicted response of the grillage model and the measured response of the test unit.

Phase II Test Results

Following the Phase I test, the loading setup for the test unit was reconfigured by removing the vertical tie-downs and horizontal actuators and reinstalling actuators in a vertical configuration closer to the mid-span of the girders on either side of the column, as shown in Figure 5. This configuration offered the opportunity to displace the girder ends vertically while retaining the fixed configuration of the column. Initial loading for Phase II consisted of using the vertical actuators to apply a hold-down force to the test unit simulating the moment at the girder-to-cap interface at the end of construction. Primary testing for Phase II followed, where the girder ends on both sides of the cap were simultaneously subjected to cyclic positive and negative displacements at gradually increasing magnitudes.

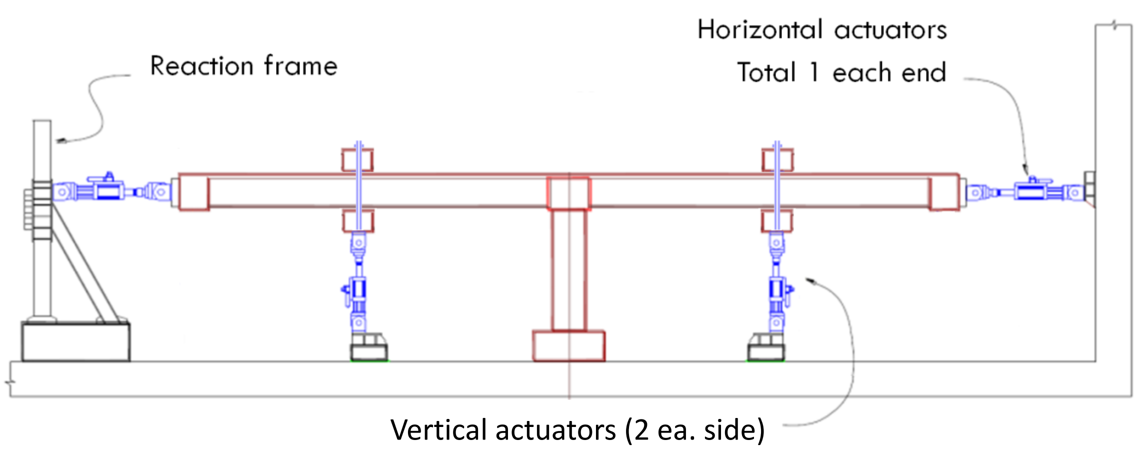


Fig. 5 Phase II Test Configuration

The goal of the Phase II test was to fully exercise the as-built and improved girder-to-cap connections in order to fully investigate their performance. The focus was placed on examining the ultimate moment capacity of each connection type as much as possible in order to determine the validity of the current design approach (SDC 2006), which assumes the as-built connection will eventually degrade to a pin condition under positive moments.

The test structure was subjected to a maximum positive (i.e., upward) displacement of 3 in. (76 mm) and a maximum negative (i.e., downward) displacement of 6 in. (152 mm). Both the positive and negative responses matched or exceeded expectations. In fact, the force vs. displacement plot indicated the structure still had additional negative moment capacity when the test was terminated, as a significant drop in strength was not recorded. Therefore, it is likely that a displacement greater than negative 6 in. (152 mm) could have been achieved. However, extensive and significant cracking was noticed in the deck at the end of the test, with the largest cracks corresponding to the stem of the inverted-T and the outer edge of the diaphragm. Since the cracks spanned the entire width of the structure, it was demonstrated that all of the girders were still actively engaged in resisting the applied moment.

Phase II was successful in exercising the as-built connections to their full capacity under positive moments, establishing the moment capacities, and ensuring a satisfactory shear transfer through the as-built girder-to-cap connection. The as-built connection was clearly observed to have a significant reserve capacity for both positive and negative moments, contrary to current design assumptions for this connection. Phase II subjected the connection to maximum negative and positive moment magnitudes that were approximately 4.9 and 1.4 times greater, respectively, than the demands imposed during Phase I.

The Phase II test did not, however, allow complete quantification of the improved connection performance. This limitation occurred progressively as the as-built connection began to fail and due to the damage to the column ends that was sustained during the Phase I test. The combination of the as-built connection degradation, and the column hinges that developed during Phase I testing produced a pinned-like mechanism, so larger vertical actuator displacements tended to only produce larger rotations in damaged regions, failing to significantly increase the moment demand in the improved connections. Although the pin-like behavior due to the as-built connection deterioration dominated the load-displacement response on both sides of the pier cap, careful reduction of the data revealed that the positive moment demand on the as-built side began decreasing while improved connections responding elastically throughout Phase II testing, providing clear indication that the improved connection exhibited better performance.

Conclusions

This experimental investigation was conducted to examine the performance of both an as-built and an improved girder-to-cap connection for precast concrete I-girders and an inverted-tee pier cap. The following conclusions have been as a result of this study:

- Contrary to current design assumptions, the as-built connection of the inverted-tee pier cap to the precast I-shaped girder behaved as a fully continuous connection instead of a pinned connection.
- The improved cap-to-girder connection performed as expected, ensuring fully continuous behavior under both positive and negative moments.
- Both the as-built and improved connections successfully transferred shear forces from the superstructure into the cap beam during both phases of testing.
- The inverted-tee pier cap detail can be used in an integral connection design to develop a plastic hinge in the top of the column. Thus, the inverted-tee pier cap is an excellent way to implement precast concrete girders in seismic regions and promote accelerated bridge construction in these regions.
- The improved cap-to-girder connection is sufficient to achieve the design goals intended in an integral connection. However, full quantification of the improved cap-to-girder connection was not achieved due to the degradation of the as-built connection. Further work is planned to complete this portion of the investigation.
- Since the as-built bridges are expected to have sufficient moment connections to act as fixed connections based on the details adopted, the columns are expected to develop plastic hinges at the top adjacent to the cap beam. In consideration of minimizing cost, only the column tops in these bridges are suggested for retrofitting with adequate confinement reinforcement so that this region can successfully develop a plastic hinge. It should be noted, however, that doing so will increase the column shear demand as well as other demands within the system, so these effects should be investigated prior to retrofitting to ensure satisfactory seismic performance of bridges in future earthquakes.
- The force vs. displacement predictions from a grillage model were observed to correlate well with the measured response of the test unit for both phases of testing. Thus, the grillage model is an adequate means of predicting the behavior of current and future inverted-tee bridge structures.

Acknowledgements

The research team thanks the following individuals for their support and assistance in the completion of the experimental study presented in this paper. Without their help and kindness, much of this research would not have been possible:

- Caltrans for sponsoring this research project and Mike Keever and Charly Sikorsky for their input, advice, and assistance with project management;
- Sami Megally of PBS&J for his expertise and guidance in the design and construction of the test unit; and
- Professor Jose Restrepo and the staff at UCSD and the Charles Lee Powell Laboratories for their assistance in the construction and testing of the test unit.

References

AASHTO LRFD Bridge Design Specifications, 3rd Edition, AASHTO, 2003.

Accelerated Bridge Construction Applications in California, A Lessons Learned Report, Version 1.1, Caltrans, 2008.

Billington, S.L., Barnes, R.W., Breen, J.E., "A Precast Segmental Substructure System for Standard Bridges," *PCI Journal,* V. 44, No. 4, July/August 1999, pp. 56-72.

Bridge Design Aids, Caltrans, 1995.

Bridge Design Specifications, Caltrans, 2003.

"Connection Details for Prefabricated Bridge Elements and Systems." Publication No. FHWA-IF-09-010., FHA, 2009.

Conspan, Bentley Systems, Inc., 2008.

Hastak, M., Mirmiran, Al, Miller, R., Shah, R., and Castrodale, R. "State of Practice for Positive Moment Connections in Prestressed Concrete Girders Made Continuous," *Journal of Bridge Engineering*, 2003, pp. 267-272.

Priestley, M.-N., Seible, F., and Uang, C., *The Northridge Earthquake of January 17, 1994,* The University of California, 1994.

Seismic Design Criteria, Version 1.4, Caltrans, 2006.

Snyder, Richard M., *Seismic Performance of an I-girder to Inverted-T Bent Cap Bridge Connection*, 2010.

Snyder Richard M., Vander Werff, J., Theimann, Zachary J., Sritharan, Sri, and Holombo, Jay *Seismic Performance of an I-Girder to Inverted-T Bent Cap Connection*, Final California Department of Transportation, Iowa State University, 2011.

Theimann, Zachary J., *3-D Finite Element Analysis of the Girder-to-Cap Beam Connection of an Inverted-tee Cap Beam Designed for Seismic Loadings,* Iowa State University, 2009.

WinRECOL, TRC/Imbsen Software Systems.

Xtract, TRC/Imbsen Software Systems.