

## **SEISMIC PERFORMANCE OF AN INVERTED-TEE BRIDGE TEST MODEL WITH PRECAST GIRDERS**

**Justin Vander Werff, PE**, Iowa State University, Ames, IA and Dordt College, Sioux Center, IA  
**Rick Snyder**, University of Minnesota, Minneapolis, MN  
**Sri Sritharan, PhD**, Iowa State University, Ames, IA  
**Jay Holombo, PhD, PE**, T. Y. Lin International, San Diego, CA

### **ABSTRACT**

*This paper summarizes an investigation on an integral bridge pier system consisting of a concrete column, I-shaped precast concrete girders, and an inverted-tee concrete cap beam that is suitable for applications in seismic regions. Using a half-scale, 58-ft long test unit, the primary focus of this research was to investigate the behavior of the connections between the concrete girders and the inverted-tee cap beam. However, the overall seismic behavior of the proposed system and the behavior of the connection between the concrete column and the inverted-tee cap beam were also important components of the investigation. Two different girder-to-cap connections were utilized in the test specimen. An as-built connection, as has been widely used by the California Department of Transportation (Caltrans) was designed and implemented on one side of the inverted-tee cap beam of the test unit. An improved connection detail was used on the other side of the cap beam. The test unit exhibited good seismic response with successful formation of plastic hinges in the column and system stability up to high levels of displacement ductility. Both the as-built and the improved connections performed successfully, allowing column plastic hinges to be fully developed. However, when subjected to vertical cyclic loads to exercise the girder-to-cap connection, the as-built connection deteriorated significantly while the improved connection continued to show dependable behavior even at high vertical displacements. Key experimental results, and their comparisons with analytical data are presented in this paper.*

**Keywords:** Seismic design, Earthquake, Integral, Bridges, Precast, Girder, Joints, Inverted tee

## **INTRODUCTION**

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. However, the use of precast concrete girders for the design of earthquake-resistant bridges in the United States is limited, due especially to the lack of research and design information regarding the connection of the cap beam and girders<sup>1,2</sup>. The research detailed herein was pursued to investigate the seismic design of a bridge system utilizing a reinforced concrete column integrally connected to a 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 and 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. This paper focuses on the large-scale 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 non-retrofitted structures designed and built prior to advances made as a result of the Loma Prieta Earthquake in 1989 and the Northridge Earthquake in 1994<sup>2,3</sup>. 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 the latter is designed with precast girders. Successful completion of such an investigation will enhance the ability to build quality bridges at an accelerated and efficient pace in seismic regions. Such improvements as are realized when using precast components are resulting in these methods becoming the preferred choice over traditional cast-in-place components<sup>4</sup>.

## **SELECTED CONCEPT: INVERTED-TEE BENT CAP**

One of the connection details 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, placed on top of the columns, in the shape of an upside-down letter “T”. 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 accelerated bridge construction (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<sup>2</sup>. Where the inverted-tee pier cap has 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<sup>5</sup>. Having a pin support at the column-to-cap connection is not desirable for seismic regions, 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, would be expected to behave more like fixed connections, with some moment capacity<sup>1,2</sup>. This moment capacity, and consequently its effect on the behavior of the remainder of the bridge, had not previously been investigated. Thus, experimental research of the connection was conducted to quantify its behavior and possibly lead to design methods that can utilize the moment capacity 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<sup>4,6,7</sup>. 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 girders with piers or pier and abutment, 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 benefits mentioned above, precast components are still not implemented frequently for bridges in areas of seismic activity. This infrequent use is mainly due to the lack of a proven design methodology and experimental validation confirming adequate performance of the connections for precast members. 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 lead to use in future bridges. Such a precast system, if shown to be viable, would lend itself well to ABC methods and likely increase the awareness and use of such precast systems.

## **PROTOTYPE BRIDGE**

The prototype bridge, shown in Figure 1, was designed in accordance to the AASHTO LRFD Bridge Design Specifications 3rd Edition with 2006 Interims and California Amendments<sup>8</sup> as well as the Caltrans Bridge Design Aid<sup>9</sup> for the design of the inverted-tee cap. In addition,

Caltrans Bridge Design Specifications (BDS)<sup>10</sup> and Seismic Design Criteria v. 1.4 (SDC)<sup>5</sup> were also used in the design. Computer software packages WinRECOL<sup>11</sup>, Xtract<sup>12</sup>, and Conspan<sup>13</sup> 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<sup>1</sup> conducted as a part of this project. Further discussion and calculations for the design of the column, cap beam, girder dapped end and slab for the prototype has been documented in Thiemann<sup>1</sup>.

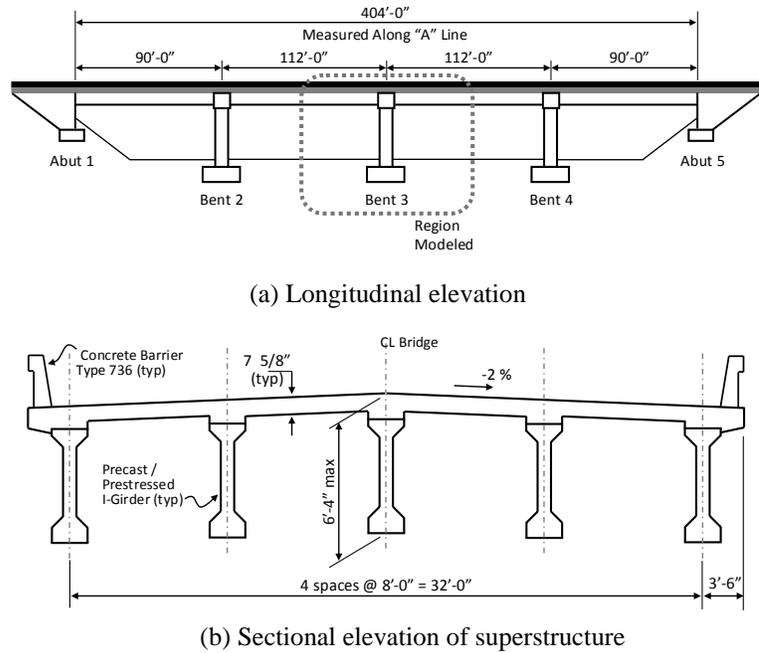


Fig. 1 Prototype bridge

## EXPERIMENTAL INVESTIGATION

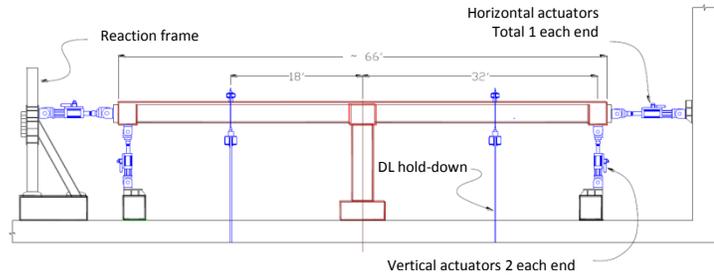
### TEST UNIT CONFIGURATION

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 Thiemann<sup>1</sup>. 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 the proposed “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. Since a majority of the negative moment contribution would be provided through the deck<sup>14</sup>, both connections would behave similarly when combined as such. As a result, based on whether the superstructure of the test unit was pushed or pulled horizontally, it was possible to isolate the effects of the behavior of only one of the connection types. Test unit plan details are provided in Figure 2, and further information on the test unit has been documented in Snyder<sup>2</sup>.

### TEST UNIT DETAILS

Figure 3 provides the details of the test unit’s girder-to-cap connection, which was the primary focus of the experimental study. The test unit was designed and built at a 50% scale

of the prototype bridge. 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. Figure 4 shows the ducts in which the unstressed strands would be located extended through the right side of the inverted tee.



(a) Schematic of Phase I configuration



(b) Photograph of Phase I configuration

Fig. 2 Inverted-tee experimental test unit

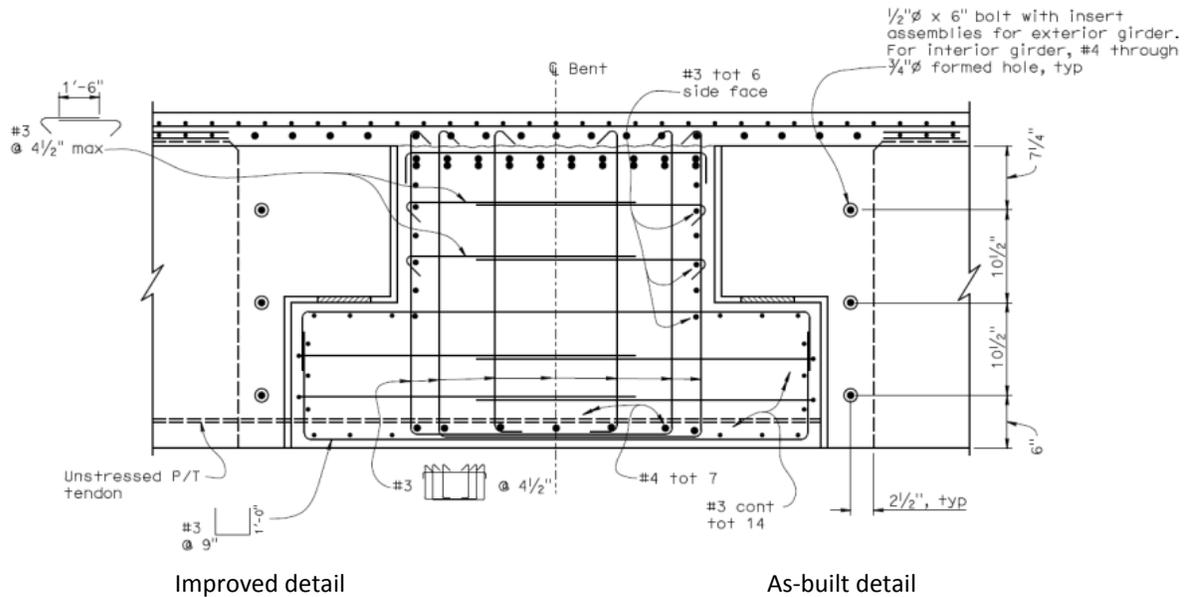


Fig. 3 Girder-to-cap connection detail for the test unit

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.

### PHASE I LOAD SEQUENCE

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 horizontal loading simulating seismic load. This phase of experimental work is the main focus of this paper. The second phase, designated Phase II, utilized vertical loads at the girder ends to fully exercise the cap-to-girder connections and to investigate the ultimate capacity of the connections.

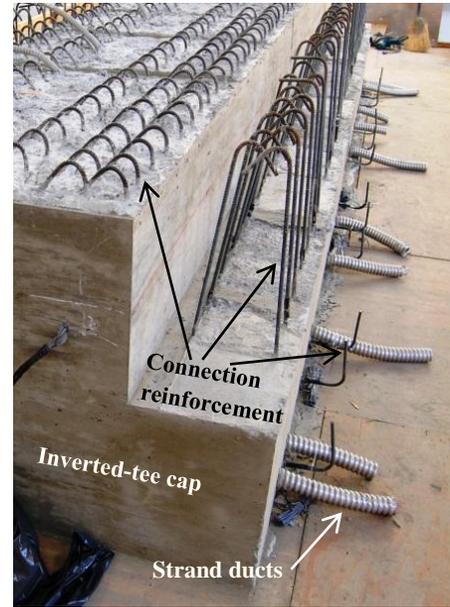


Fig. 4 Inverted-tee cap prior to girder placement

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,  $\mu_{\Delta}$ , 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 OBSERVATIONS

When subjected to a combined vertical gravity and simulated 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. 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.

### PHASE I FORCE-DISPLACEMENT RESPONSE

A comparison of critical data collected during the test to the predictions made prior to the test based on a SAP2000 grillage model analysis<sup>2</sup> showed generally good results. The horizontal force vs. lateral displacement of the superstructure is shown in Figure 5, which shows slight disagreement at small displacements as the grillage model used a cracked effective stiffness for both the column and superstructure sections, rather than the actual gross values for the crack-free stage of the test. However, the 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.

Further, the force-displacement behavior in Figure 5 is seen to be quite similar in both 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 Phase I's horizontal seismic loading.

### PHASE I SUPERSTRUCTURE RESPONSE

Figure 6 provides the deck reinforcement strain data for peak loads during the Phase I test. To provide a clear comparison, the strains from push direction loading on the improved side and pull direction loading on the as-built side have been combined in this figure, as these are the load directions that produced tension in the deck reinforcement.

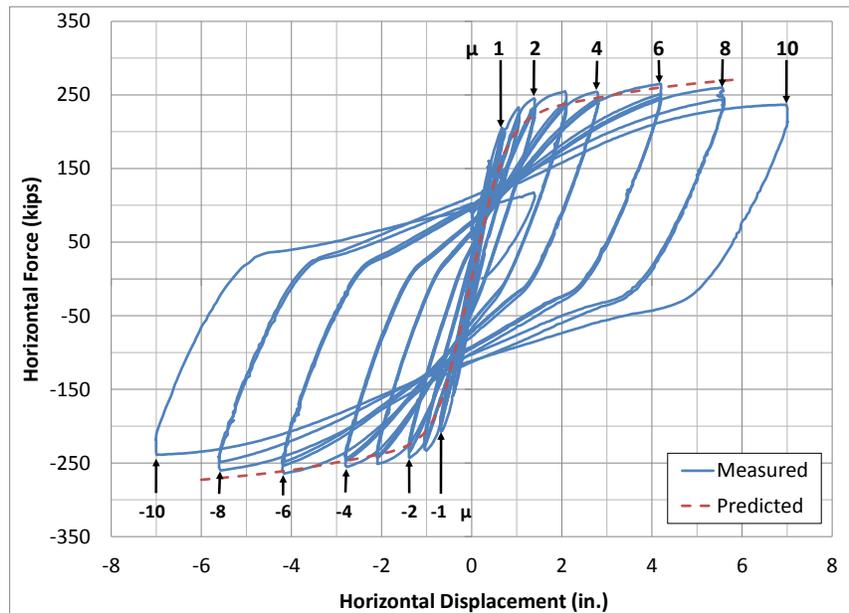


Fig. 5 Comparison of measured vs. predicted force-displacement response

Two key observations to note here are: (a) the load was seen to be distributed across the center, intermediate, and exterior girders already at 0.25  $F_y$  and also during the higher load cycles; and (b) a noticeable increase in strain magnitude was observed between the 1.0  $\mu_\Delta$  and 1.5  $\mu_\Delta$  load steps. This more abrupt change matches a similar behavior that was observed in dowels anchoring the girders to

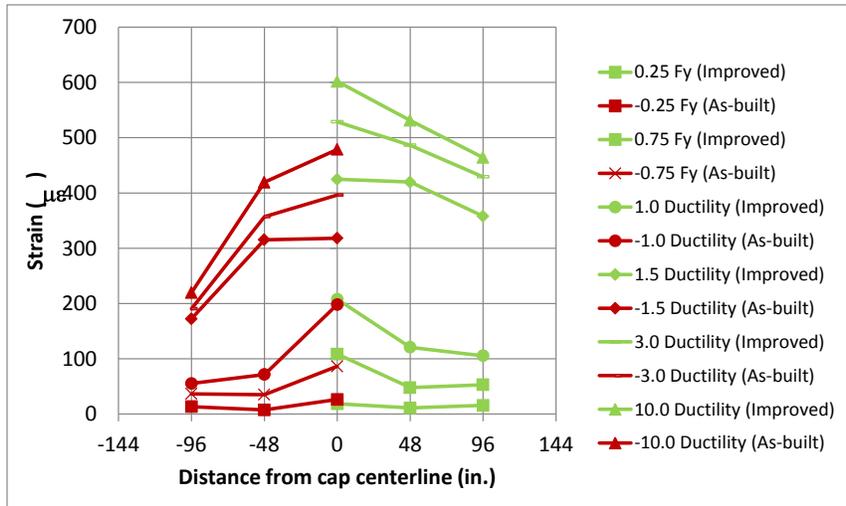


Fig. 6 Deck reinforcement transverse strain profiles for peak loads for improved side push direction and as-built side pull direction

the diaphragm in the connection region. Another observation to note here is that the strain demand was about 20% higher on the improved side than on the as-built side at high lateral displacements, confirming that a higher moment demand was generated on the improved connection in comparison to the as-built connection.

#### PHASE I TEST SYSTEM BEHAVIOR

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<sup>5</sup> 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. Furthermore, it was 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 CONFIGURATION AND LOADING

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 near the ends of the test unit girders, as shown in Figure 7. 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. 7 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<sup>5</sup>, which assumes the as-built connection will eventually degrade to a pin condition under positive moments.

#### PHASE II GENERAL OBSERVATIONS

The test structure was subjected to a maximum positive (i.e., upward) displacement of 3-in. and a maximum negative (i.e., downward) displacement of 6-in. 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. 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 required 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 those during Phase I.

The Phase II test did not, however, allow complete quantification of the improved connection performance. This limitation occurred due to the progressive failure of the as-built connection throughout the test along with the damage to the column ends that was sustained during the Phase I test. The combination of the as-built connection degradation, shown in Figure 8, and the column hinges that developed during Phase I testing produced a pinned-like mechanism, so larger displacements tended to only produce larger rotations and did not significantly increase the moment demand in the improved connection detail. Figure 9 provides a comparison of the vertical load demand in the girders on the improved and as-built sides. Although the pin-like behavior due to the as-built connection deterioration dominated the load-displacement response on both sides of the pier cap, this data reveals that the positive moment demand on the as-built side began decreasing while the demand on the improved side remained relatively constant for the higher displacement levels.

## 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 can be made 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.

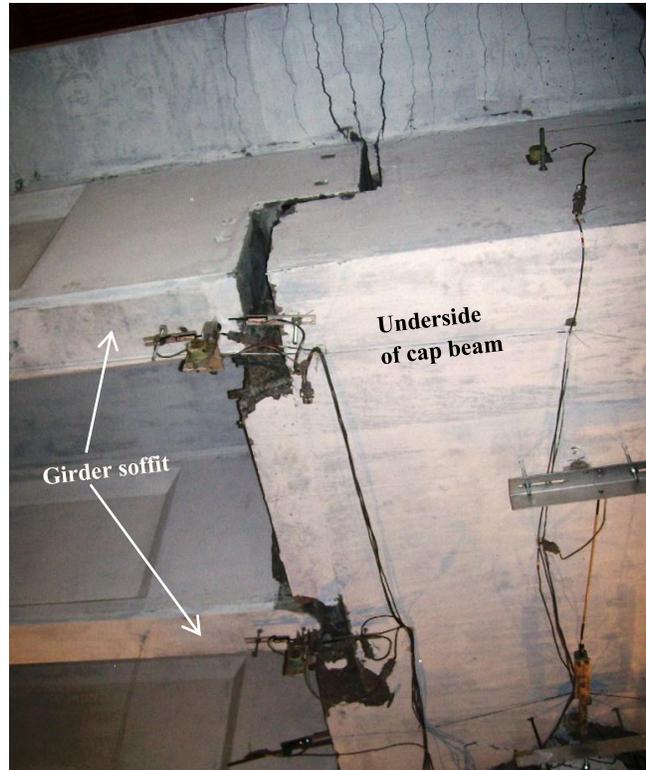


Fig. 8 Opening up of as-built connection under positive moment loading

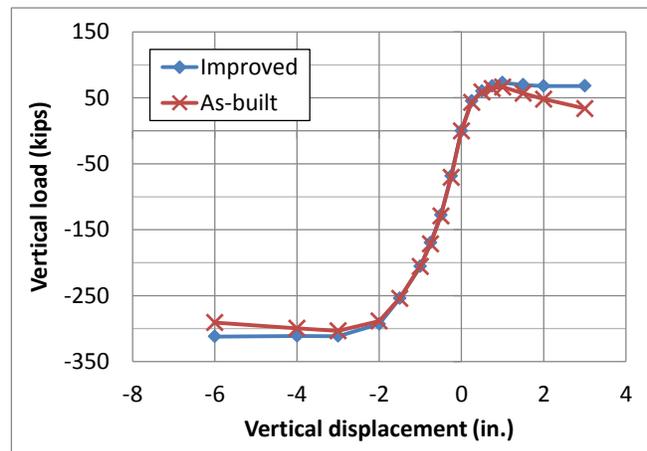


Fig. 9 Comparison of peak load-displacement behavior for Phase II test

- Both the as-built and improved connections successfully transferred shear forces from the superstructure into the cap beam.
- 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.
- Only the tops of the columns in as-built bridges require retrofitting in order to prevent premature column damage that would be prohibitive in successful plastic hinge formation. 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.
- 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.

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