Accelerated Bridge Construction in High Seismic Regions by Use of Precast Concrete Girders

J. Vander Werff
Dordt College, Sioux Center, Iowa

S. Sritharan
Iowa State University, Ames, Iowa

R. Snyder
University of Minnesota, Minneapolis, Minnesota

SUMMARY:
This paper examines 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 to facilitate accelerated bridge construction (ABC) in high seismic regions. Using a half-scale test unit, the focus of the research included the behavior of the connections between the girders and cap beam as well as the overall system behavior. Connections in the experimental model included an as-built connection that has been implemented by the California Department of Transportation (Caltrans) and a similar but improved connection using unstressed prestressing strands. Both the as-built and improved connections exhibited satisfactory seismic response under horizontal loading, enabling successful system performance by the formation of plastic hinges in the column and system stability up to high levels of displacement ductility. However, when subjected to vertical reverse cyclic loads to fully exercise the girder-to-cap connections, the as-built connection was observed to deteriorate considerably while the improved connection continued to exhibit dependable behavior, even for high vertical displacements. Key experimental results, comparisons with analytical data, and recommendations for similar connections for precast members for ABC methods are included in this paper.

Keywords: seismic, precast, integral, bridges, connections

1. INTRODUCTION

Accelerated Bridge Construction (ABC) is increasingly the bridge construction method of choice, given the many benefits of decreased field construction time and the continuing advancements in industry that make the implementation of ABC methods feasible. Precast concrete members are appealing for ABC methods because of the minimal field time that is required for their installation. Precast concrete girders also have several advantages over structures consisting of steel girders or cast-in-place concrete alternatives for implementation of integral column and cap beam systems for bridges, and integral systems are often very desirable for high seismic regions because of their tendency to dissipate energy by the formation of plastic hinges. However, the implementation of precast concrete girders for the design of earthquake-resistant bridges has been limited. A primary reason for this lack of use is the absence of research and design information regarding the connections between critical bridge components (Theimann, 2009; Snyder, 2010). The research presented in this paper was directed at an investigation of the design of a bridge system for seismic regions implementing 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. Specific areas of interest included (1) good overall system behavior through verification of sufficient shear capacity as well as positive and negative moment capacity in the girder-to-cap connection to allow successful formation of a plastic hinge at the top of the column; (2) experimental quantification of the performance of the girder-to-cap connections with as-built and improved details; and

(3) suitable design recommendations and specifications to promote the use and advancement of similar designs in an effort to continue to provide increased opportunity and motivation to incorporate ABC methods in bridge construction. The primary focus in this paper is, however, given to the large-scale experimental tests conducted to validate the connection regions.

Seismic advances made since observations of the Loma Prieta Earthquake in 1989 and the Northridge Earthquake in 1994, namely new bridge designs and retrofits based on the capacity design philosophy (Snyder, 2010; Priestley, Seible, & Uang, 1994), have proven to provide structures and design approaches that are more suitable for resisting earthquake loads. A significant amount of research in these areas has been conducted in the last two decades with considerable success. However, even though acceptance of the capacity design philosophy is widespread, some structural details have still not been accordingly investigated. One such detail is the girder-to-cap connection when precast girders are utilized in the bridge superstructure. The advantages and improvements related to the use of precast components are already resulting in such designs becoming the preferred choice over traditional cast-in-place construction techniques (FHA, 2009). Analytical and experimental investigations of these connections, when successfully completed, will provide increased ability and impetus to utilize prefabricated components in building bridges using accelerated methods that are of high quality and are sufficient for seismic regions.

2. INVERTED-TEE BENT CAP

One connection detail for which further investigation would be useful is the connection between an inverted-tee bent cap and a precast girder due to its ability to speed up the construction of the superstructure. 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. 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 wellsuited 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.

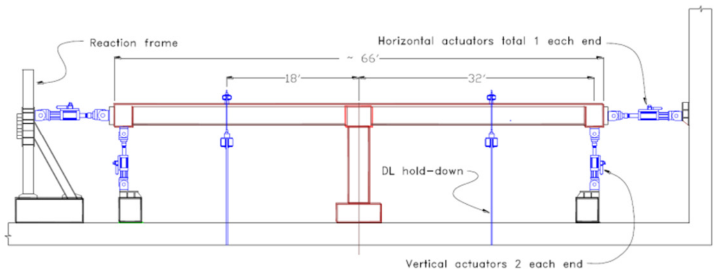
Despite the aforementioned benefits, precast components are still not implemented frequently for bridges in areas of seismic activity. It is highly likely that the use of precast construction would become widely accepted in seismic areas if a design methodology were developed and proven to be reliable and cost effective. 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.

3. PROTOTYPE BRIDGE

The prototype bridge selected for this study consisted of a four-span, five-girder bridge consisting of a reinforced concrete column, cast-in-place inverted-tee cap beam, and I-shaped precast concrete girders. The interior spans of the prototype bridge were 112 ft in length and the girders had 8 ft spacing and height of approximately 5.5 ft. 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 v. 5.0.2 (TRC/Imbsen), Xtract (TRC/Imbsen), 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). For further information on the prototype bridge, refer to Snyder (2010).

4. EXPERIMENTAL UNIT AND TEST PLAN

A half-scale test unit was developed for the center portion of the prototype structure, which represented a typical inverted-tee bridge. Specific information related to the design of the test unit is included 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 a half-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. The experimental plan consisted of testing both an “as-built connection” and an “improved connection,” described in more detail in the following paragraph. To accomplish both connections 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 connection region of the column was designed with adequate confinement, as it was expected, based on the analytical work, to develop a plastic hinge at the top. Past research has shown that a majority of the negative moment contribution would be provided by the longitudinal reinforcement in the deck (Hastak et al., 2003). Overall details of the test unit are provided in Figure 1, and further information on the test unit has been documented in Snyder et al. (2011).



(a) Schematic of Phase I configuration

(b) Photograph of Phase I configuration

Figure 1. Inverted-tee Experimental Test Unit

Figure 2 provides the details of the test unit's girder-to-cap connection. The connection shown in Figure 2 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 is expected to run continuously through girders along the entire bridge from one end of the abutment to the other. 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 in the test unit.

Hooked reinforcement was placed between the cap and diaphragm to establish a connection between the diaphragm and inverted-tee bent cap, as is typical for such configurations. Also, following another common technique, the girders contained transverse dowel bars that extended into the diaphragm in order to further establish a connection between the embedded ends of the girders and the diaphragm.

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 lateral seismic load. The second phase, designated Phase II, used cyclic vertical loads applied to the girders to fully exercise the cap-to-girder connections and to establish the ultimate capacity of the connections.

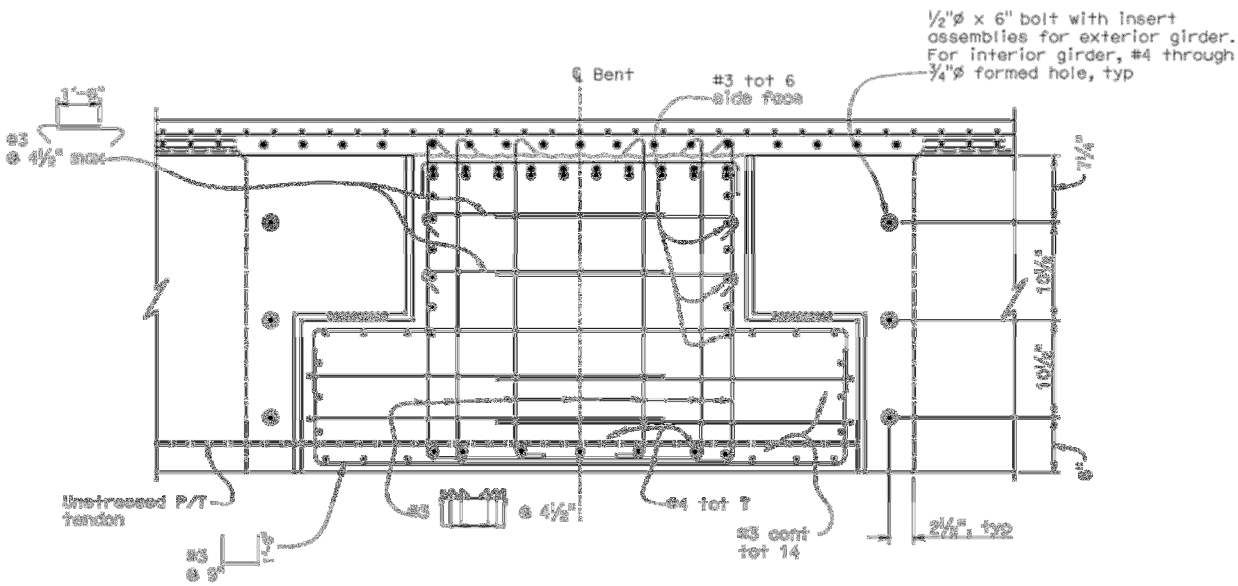


Figure 2. Girder-to-cap connection

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. The purpose of Phase I was to investigate the ability of the girder-to-cap connection to provide good system performance, specifically by providing sufficient strength for the development of a plastic hinge at the column top just below the cap beam. The development of such a hinge is a key component for a bridge designed according to the capacity design philosophy for seismic loading.

Two sets of vertical tie-downs and four actuators positioned in the vertical direction were used to simulate the gravity load effects on the test unit during Phase I testing. 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.

5. PHASE I TEST RESULTS

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 when subjected to combined gravity and seismic loads in a cyclic manner. Plastic hinges were successfully developed at the top and bottom ends of the column. 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.

Critical data collected during the test was compared to the predictions made prior to the test based on a SAP2000 grillage model analysis (Snyder, 2010). The comparison revealed generally good results. The horizontal force vs. lateral displacement of the superstructure is shown in Figure 3, which shows slight disagreement at small displacements as the grillage model used an 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. This progressive improvement in the comparison is likely an indicator that the system began to behave more like the analytical model as more of the structure softened due to the development of cracks and yielding of longitudinal reinforcement.

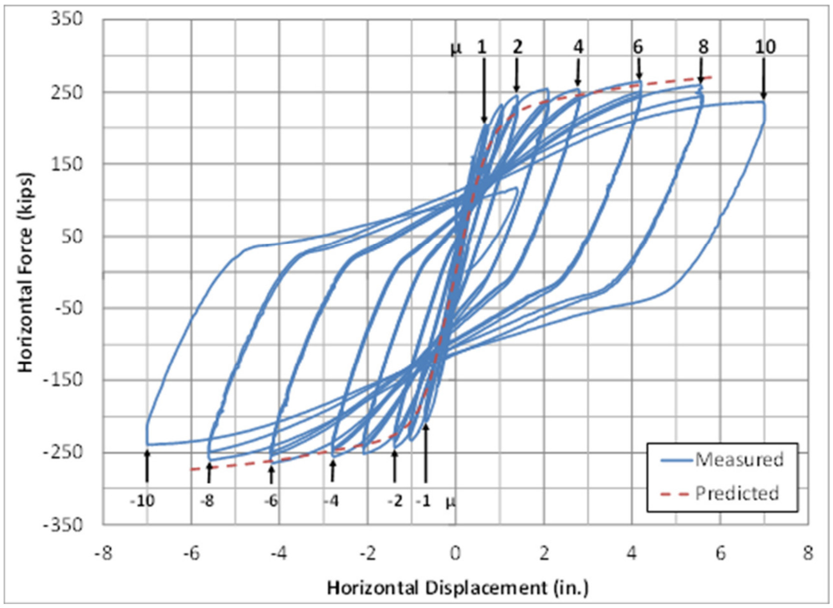


Figure 3. Load-displacement response

One more finding from Phase I that is worth mentioning is the similarity in force-displacement behavior in Figure 3 for the two displacement directions. This similarity indicates 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 design 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.

6. 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 4. This configuration was intended to allow the displacement of 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. Then, in the primary testing for Phase II, the girder ends on both sides of the cap were simultaneously subjected to cyclic positive and negative displacements at gradually increasing magnitudes.

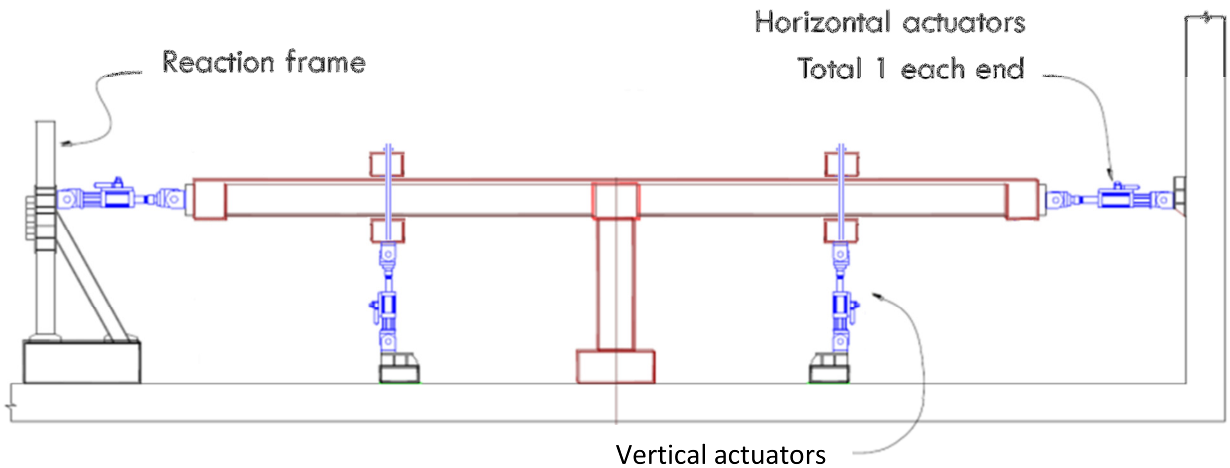


Figure 4. Phase II Test Configuration

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. Cracks spanning the entire width of the structure indicated 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.

7. LOAD DISTRIBUTION

One of the limitations to more widespread use of integral bridges in seismic regions is the lack of research and appropriate design recommendations regarding how lateral load is distributed among the girders in the superstructure. Current design recommendations allow very little, if any, distribution of lateral load to girders that are not immediately adjacent to the column. However, several studies in the last decade have revealed that significant portions of the lateral load from the column are transferred to intermediate and exterior girders (Holombo, Priestley, & Seible, 2000; Sritharan et al., 2005; Snyder et al., 2010). Similarly, the inverted-tee test specimen detailed in this paper revealed that sizeable portions of the lateral load were distributed to the intermediate and exterior girders. Figure 5 provides a graphical summary of the load distribution among the five girders in the test unit, on the basis of measured strain in the longitudinal deck reinforcement above each of the girders, with the data biased to isolate only the lateral load contribution. Figure 5a shows the lateral load distribution among the exterior, intermediate, and center girders for all of the peak conditions while Figure 5b shows the same data for the peaks of only the lower load conditions. Looking at these low-load distribution results, it is seen that significant lateral load distribution to all the girders was measured even at the lowest load levels. The exterior girders are seen, at the very first peak recorded, to individually carry 15 percent (or 30 percent when the two exterior girder contributions are combined) of the total lateral load. Although there is a bit of irregularity in the distribution for the next four peaks recorded, significant distribution is observed even through this irregular pattern, with the exterior girders never carrying less than 10 percent of the lateral load. More uniform distribution is documented for all of the higher peak conditions, implying that incorporation of a more realistic lateral load distribution would lead to more efficient seismic design of bridge superstructure.

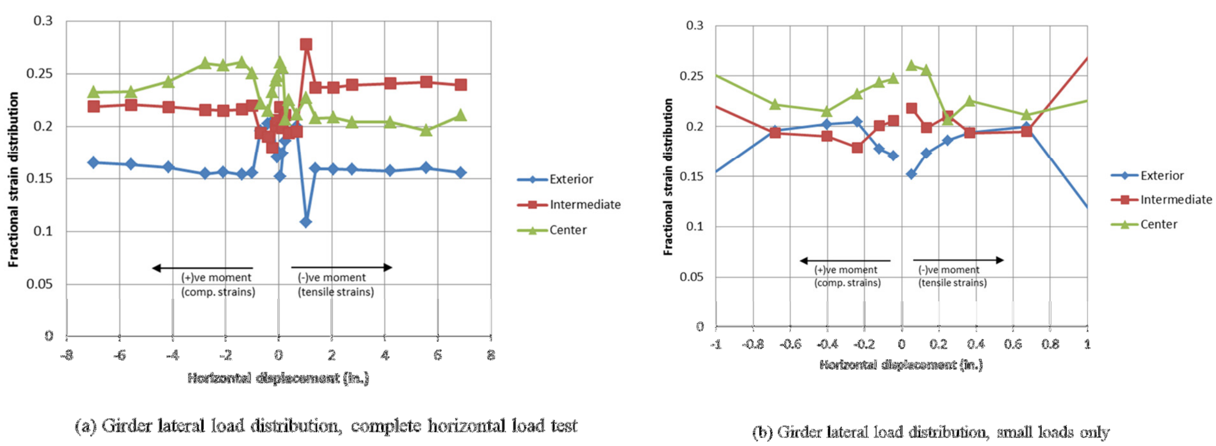


Figure 5. Experimental load distribution for test unit

8. CONCLUSIONS

This experimental investigation was conducted to examine and quantify the performance connections between precast concrete I-girders and an inverted-tee pier cap, including both an as-built connection that has been implemented and an improved variation of the as-built connection. This study has yielded the following conclusions:

- The as-built connection of the inverted-tee pier cap to the precast I-shaped girder did not behave as a pinned connection, as anticipated based on the current design assumption. Rather, it acted as a continuous connection with full moment sufficient positive and negative moment resistance between the girder-to-cap connection.
- The improved cap-to-girder connection performed as expected, exhibiting fully continuous behavior under both positive and negative moments.
- During both phases of testing, successful transfer of shear forces from the superstructure into the cap beam was observed for both the as-built and improved connections.
- Overall, the inverted-tee cap beam detail with the improved connection to girders 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 underway 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, and thus these effects should be investigated prior to retrofitting to ensure satisfactory overall 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.
- All girders (i.e., the center, intermediate and exterior) participated in resisting the seismic lateral load. This sizeable seismic resistance contribution from the intermediate and exterior girders does not match the current design recommendations; further consideration of the overall performance of the girder system in resisting lateral load may improve cost efficiency as well as alleviate congestion and unnecessary stiffness in the connection region of girders adjacent to the column.

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