16th World Conference on Earthquake Engineering, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 2646
Registration Code: S-Y1463367794

NEW CONNECTIONS FOR PRECAST GIRDERS TO CAP BEAMS IN CONCRETE BRIDGES

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Abstract

The use of precast girders has helped to establish Accelerated Bridge Construction (ABC) methods in the United States. However, this practice has not been extended to seismic regions due to lack of reliable connections between precast girders and cap beam. This paper characterizes the basic challenges associated with current design practice and introduces several options to establish reliable connections between precast girders and cap beams. Two mechanisms are used to establish positive moment continuity in these connections, namely shear friction and direct tension transfer. The first mechanism utilizes dowel bars running through girders while the second mechanism uses unstressed prestressed strands extended from girders into the cap beam.

All connection details have been found to provide significant improvements to an existing detail for precast girder and cap beam connections that has been used by the California Department of Transportation (Caltrans). Each new connections relies on deck reinforcement for tension continuity when subjected to negative moments. For positive moment tension transfer, all but one connection utilize both mechanisms to provide tension continuity for the connection when subjected to positive moments. In one connection, large diameter dowel bars are used and all tension force corresponding to the positive moments is resisted by a shear friction mechanism.

Using a large-scale experimental study, seismic performance of six different connection details between precast concrete I-shaped or bulb-tee girders and concrete cap beams was examined. The ability of the girder-to-cap connections to successfully resist positive and negative moments and the corresponding shear forces under combined gravity and seismic effects were the primary focus of this study. Additionally, the effect of vertical seismic acceleration on the connection behavior was also investigated. All connections exhibited excellent seismic performance, remaining elastic up to load levels well in excess of what would be required to develop a plastic hinge at the top of the column, including due consideration to vertical acceleration effects. All connections were subjected to large girder displacements beyond that required by seismic loading to fully quantify their performance. Experimental results from all connections and their comparisons with the as-built connection performance will be presented in this paper.

Keywords: precast, connection, design, seismic, girder, cap beam

1. Introduction

Accelerated bridge construction (ABC) using precast components offers multiple benefits for construction of new bridges and retrofitting of deficient bridges. These benefits include reduced onsite construction time, low lift-cycle costs and improved construction quality. Precast concrete girders are the most common precast components used in the bridge industry. They allow for the possibility of using cast-in-place (CIP) or precast cap beam and achieve continuity in multi-span bridges by use of wet or dry connections between the girder and cap beam. Concrete cap beams are conducive to the use of concrete columns integrally connected to the superstructure. Inverted-tee beat cap and CIP integral cap are two typical cap beam configurations. In addition to reducing the vertical clearance under the bridge, this option allows for formation of dependable plastic hinges at the column top adjacent to the cap beam [1].

Although ABC methods involving precast components for constructing bridges have been used increasingly in non-seismic regions, their applications are limited in moderate-to-high seismic regions due to inadequate seismic performance of precast structures [2]. Such failures were evident in the 1989 Loma Prieta [3] and 1994 Northridge [4] earthquakes. Current design practice, described in Seismic Design Criteria of the California Department of transportation (Caltrans), assumes that precast girder-to-cap connections may degrade to a pinned connection in severe seismic events, thus decreasing the appeal of using precast girders on a routine basis [5]. An additional limitation of the connection is related to vertical acceleration effects. The current Caltrans Seismic Design Criteria requires special reinforcement to protect against potential shear failure at the cap beam interface resulting from vertical acceleration [6], which makes the use of precast girders less competitive since the detail is not well-suited for incorporation of the additional reinforcement. To utilize the benefits of ABC methods in seismic regions, new connection details for precast concrete girders must be developed that are not only cost effective but also exhibit seismic performance that is equivalent to or better than those use in comparable cast-in-place bridges.

A 2010 system test [7] investigated the performance of the as-built connection detail that Caltrans has used for precast I-shaped girders along with an improved detail, which utilized extended unstressed girder strands across the connection interface to ensure positive moment continuity. Both connections incorporated short dowel bars through girder ends into a cast-in-place diaphragm in the connection region. The study revealed that the as-built detail may act as a fixed connection under service load and can potentially degrade to a pin connection during extreme seismic events. For demands corresponding to design-level earthquakes, the as-built connection was found to behave more like a fixed connection than a pinned connection. The improved connection with the extended girder strands produced improved performance in terms of strength and ductility.

This paper summarizes an investigation on the development of several different structurally-sufficient seismic connections between precast girders and cap beams to promote ABC in seismic regions. Specific areas of interest include: (1) description of new connections details that may be used between precast I-shaped or bulb-tee girders and precast inverted-tee or cast-in-place cap beams to ensure sufficient shear capacity as well as positive and negative moment resistance; (2) experimental quantification of the response of the proposed connections; and (3) suitable recommendations to promote the use and advancement of similar designs incorporating ABC methods in bridge construction.

2. Connection Details

The system test confirmed the potential of developing a moment resisting connection for precast girder to promote ABC in seismic regions. Precast girder-to-cap connections that can reliably resist positive and negative moments and the corresponding shear forces under combined gravity and seismic effects will result in successful implementations of ABC in seismic regions. Of the six different connections studied, two were designed to connect precast I-shaped girders with dapped ends to precast inverted-tee cap beams, namely Grouted Unstressed Strand Connection (GUSC) which duplicated the improved connection in the system test, and Looped Unstressed Strand Connection (LUSC). The remaining four connections are referred as Extended Strand

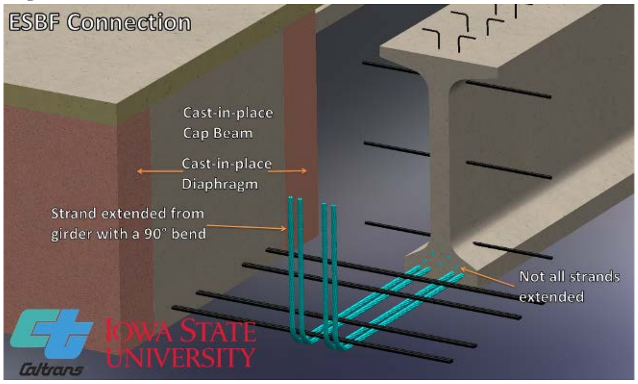
Bent with Free end connection (ESBF), Extended Strand with Splice and end Plate connection (ESSP), Extended Strand with a Mechanical Splice connection (ESMS), and Extended Strand with a Lap Splice connection (ESLS). The investigation of these connections targeted the use of precast bulb-tee girders and cast-in-place cap beam.

The GUSC and LUSC connections, shown in Fig. 1, included continuous deck reinforcement for negative moment tension continuity. In the GUSC connection, unstressed strands positioned in the ducts through the interface between the girder bottom flange and the lower region of the cap beam and then grouted in place to provide positive moment tension continuity. Dowel bars oriented transversely and passing through the girder web into the cast-in-place concrete diaphragm were also incorporated based on the existing Caltrans details to provide shear friction to increase positive moment resistance. Positive moment continuity in the LUSC detail was different from the GUSC detail in that positive moment continuity in LUSC connection was accomplished by enlarging and relocating the dowel bars to the lower portion of the girder and extending them through continuous looped unstressed strands that extended out from the cap beam ledge. Additional girder strands at the bottom of the girder were intended to provide confinement of the dowel bars passing through the girder by looping around the dowel bar block outs. The entire region was again encased by a cast-in-place concrete diaphragm, similar to the GUSC connection. A shear friction mechanism utilizing dowel bars was expected to be the primary path for positive moment tension load transfer.

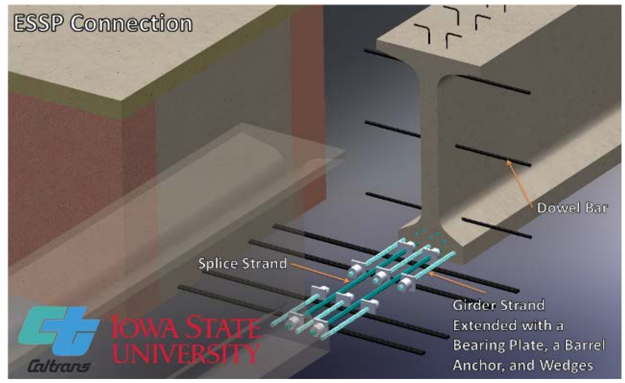
(a) GUSC Connection Detail (b) LUSC Connection Detail

Fig. 1 – Schematic of Two Connections used between Precast I-shaped Girders and Precast Inverted-tee Cap Beam

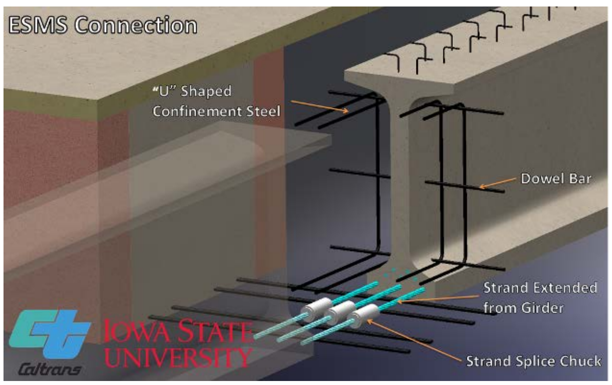
The ESBF, ESSP, ESMS, and ESLS connections, shown in Fig. 2, also relied on the continuous deck reinforcement that was placed over the interface between the precast bulb-tee girders and the cast-in-place cap beam to provide tension continuity needed to resist the negative moment. Positive moment resistance of these connections relied on direct tension transfer through unstressed girder strands that were extended from precast girders and embedded into the cap beam with several different anchoring details. Table 1 summarizes the anchorage details utilized in different connections. As for the ESMS and ESLS connections, the amount of extended girder strands was reduced to 80% of that those used in the ESBF and ESSP connections in order to optimize the strand design based on the expected moment demands and expected moment contributions of the shear-friction mechanism resulting from the use of dowel bars. Based on Caltrans' as-built details, dowel bars that were grouted through the web of girders and anchored into a cast-in-place concrete diaphragm surrounding the ends of girders were duplicated in all connection details proposed for precast bulb-tee girders.



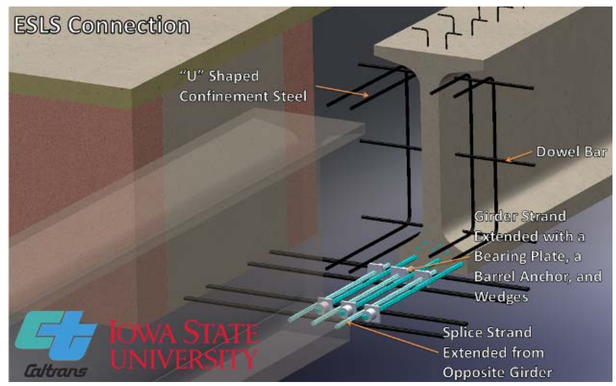
(a) ESBF Connection Detail



(b)ESSP Connection Detail



(c) ESMS Connection Detail



(d) ESLS Connection Detail

Fig. 2 – Schematic of Four Connections used between Precast Bulb-tee Girders and Cast-in-place Cap Beam

Table 1 – Strand Anchoring Details Utilized in Connections between Precast Bulb-tee Girders and Cast-in-place Cap Beam

Connection Detail	**Strand Anchoring Detail**
ESBF Connection	Free end with 90 degree bend
ESSP Connection	Strand chuck consisting of a bearing plate, a barrel anchor, and wedges; Splice strands lapping with unstressed girder strands extended from opposite girders
ESMS Connection	Mechanical splice chuck splicing unstressed girder strands extended from opposite girders
ESLS Connection	Strand chuck consisting of a bearing plate, a barrel anchor, and wedges; Extended girder strands overlapping with unstressed strands extended from opposite girder

3. Experimental Verification

3.1 Test units and loading protocol

To fully quantify and verify the connection details, one 50% scale test unit and two 40% scale test units were designed and constructed. Each unit consisted of a single column supported by a footing and a cap beam, along with two precast I-shaped or bulb-tee girders to examine the performance of two connection details, as shown in Fig. 3. Strands in the connection regions were easy to install due to their flexibility. Since prestressing strands have significantly higher strength than typical reinforcement, less reinforcement was required resulting in less congestion in the connection region. For the 50% scale test unit incorporating a precast inverted-tee cap beam (with GUSC and LUSC connections), the I-shaped girders were positioned on the cap beam and supported at the free end by temporary falsework. Unstressed strands and dowel bars were placed and grouted in placed as planned. The cast-in-place diaphragm was then poured simultaneously with the deck.

For the 40% scale test units consisting of the cast-in-place cap beams (ESBF, ESSP, ESMS, and ESLS connections), the precast bulb-tee girders were first placed on falsework. After the installation of extended girder strands and dowel bars according to each connection details, cap beam and diaphragm reinforcement cages were fabricated on the supporting formwork. The cap beam and deck concrete were then cast in one continuous pour.

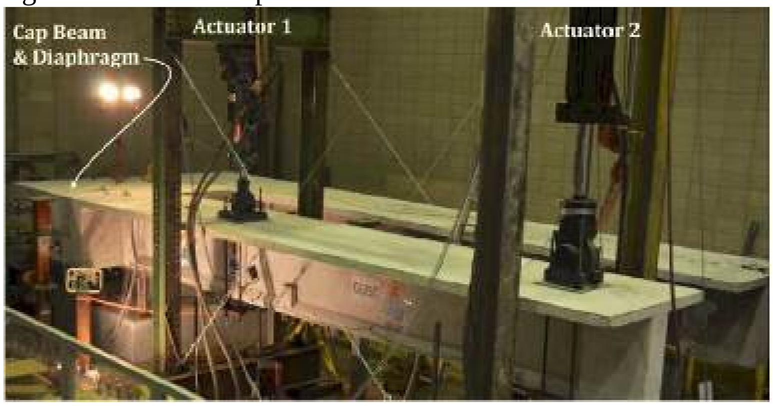
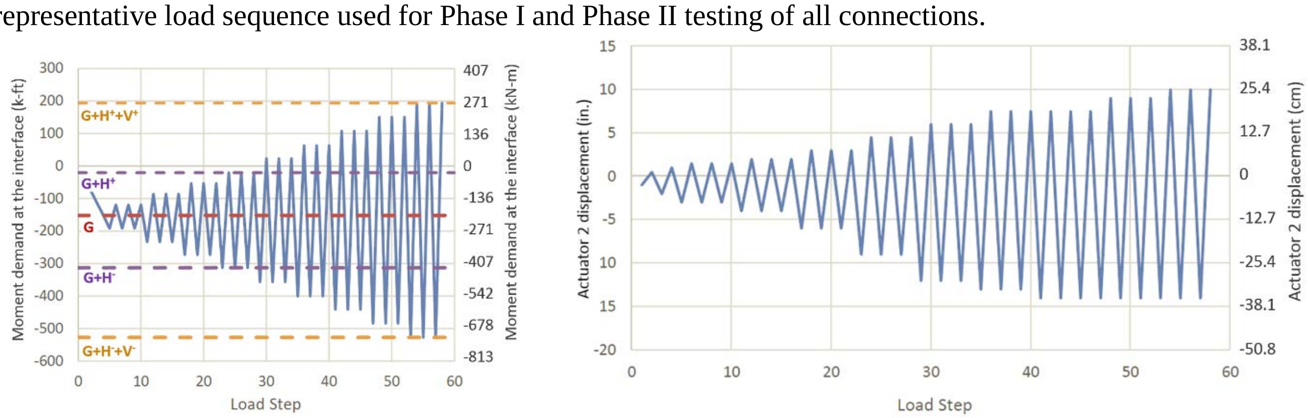


Fig. 3 – Photograph Showing the First Test Unit

To adequately verify the connection performance under combined gravity and seismic effects, a loading protocol was developed that would replicate the connection moment and shear forces for the gravity-only condition (Phase I, Part G), gravity-plus-horizontal-seismic condition (Phase I, Part G+H), and the gravity-plus-horizontal-and-vertical-seismic condition (Phase I, Part G+H+V). In addition, a large-displacement sequence (Phase II) was incorporated to fully exercise the connections beyond the expected moment demands to examine their ductility capacity. This portion of testing was controlled using the displacement of the actuator near the free end (i.e., Actuator 2). In the construction of an actual bridge with these connection details, the cast-in-place cap beam option produces a negative moment at the connection interface from the weight of the girder, deck, wearing surface, barriers, etc. when the supporting falsework is eventually removed. This creates a more favorable condition for the positive moment resistance than that would occur when precast cap beams are used since there is less gravity moment resulting from wearing surface and barriers only. Since the primary focus of this study was to quantify the positive moment capacity of each connection, the gravity-only condition replicated the moment and corresponding shear generated from wearing surface and barriers only, even though cast-in-place cap beams were used for the ESBF, ESSP, ESMS, and ESLS connections. The incorporation of two vertical actuators on a single girder, as seen in Fig. 3, produced the expected combinations of shear and moment in the connection region. For each loading step, a fully reversed cyclic sequence was incorporated. Fig. 4 shows representative load sequence used for Phase I and Phase II testing of all connections.



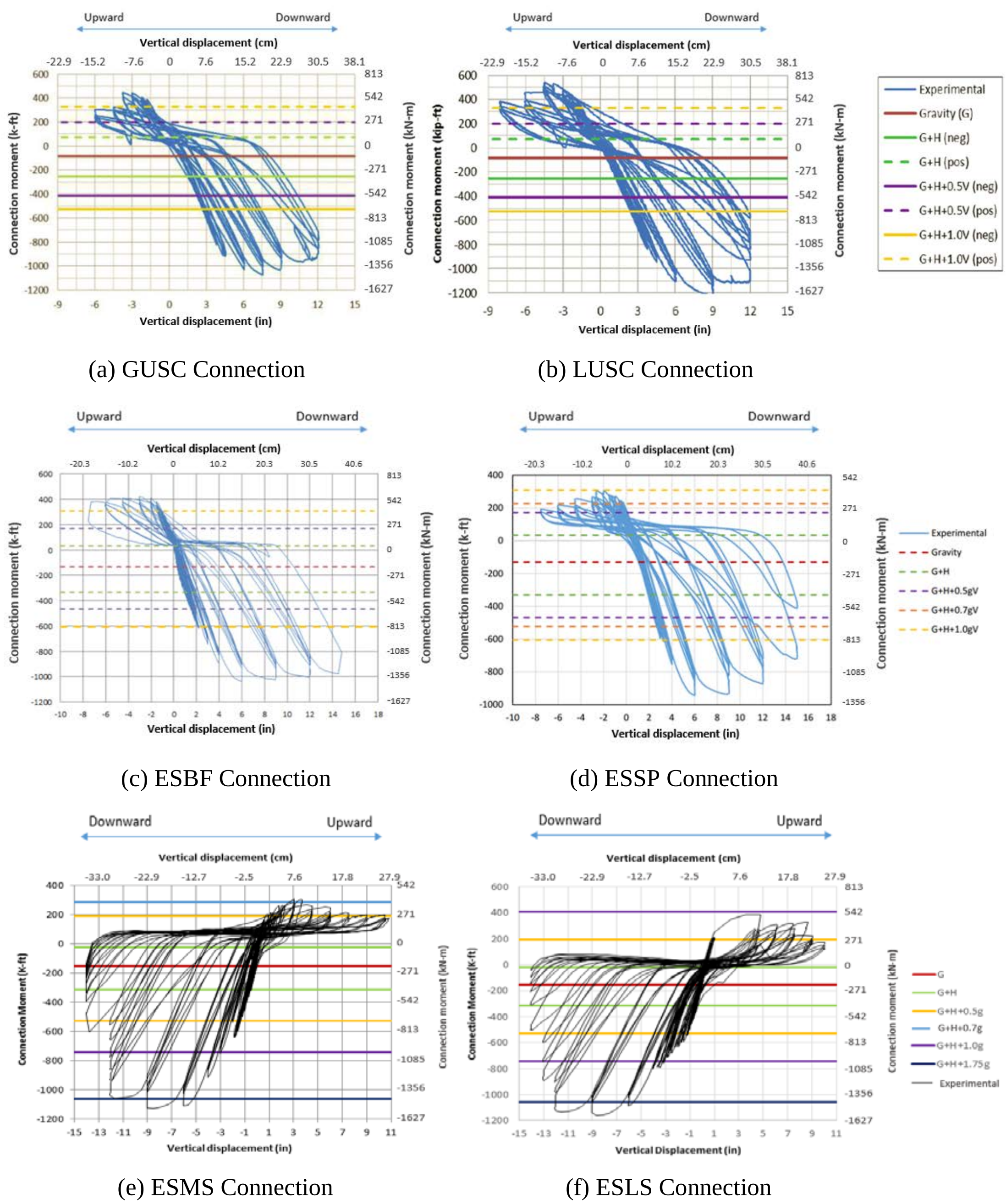
(a) Phase I Load Sequence (b) Phase II Load Sequence

Fig. 4 – Loading Protocol

3.2 Test results

All six connections performed well and successfully resisted the positive and negative moments and the corresponding shear forces under combined gravity and seismic load effects. This indicated that forming plastic hinges at the column top will be possible when the column is integrally connected to the cap beam and the precast girder to cap connection is designed using one of the six details described above. In contrast, the as-built connection performed adequately under negative moments but experienced failure when subjected to large positive moment rotations.

Fig. 5 shows the moment resisted by all connections as a function of vertical displacement at the Actuator 2 location for the entire tests. For each case, it is seen that the connection responded in an elastic manner for moment demand corresponding to that due to gravity, full horizontal seismic load, and at least 0.5g vertical acceleration effects. Furthermore, the figures confirm that all connections except ESMS provided moment resistance for vertical acceleration effects corresponding to 1.0g. In addition, the test results show that all connections exhibited considerable ductility under positive and negative moment directions during Phase II testing.



(a) GUSC Connection (b) LUSC Connection

(c) ESBF Connection (d) ESSP Connection

(e) ESMS Connection (f) ESLS Connection

Fig. 5 – Measured Connection Moment as a Function of Displacement of Actuator 2

The observed failure mechanisms of the connection for the negative and positive moment responses were unique. When the connections were subjected to negative moment rotations well beyond that was due to combined gravity, horizontal seismic effects, and 0.5g vertical acceleration, the deck reinforcement underwent strain hardening. The corresponding compression zone near the bottom region of the girder-to-cap interface caused spalling of cover concrete followed by mushrooming out of the strands locally (see Fig. 6(a)). As the strands were pushed out, further crushing and spalling of concrete took place especially in the region between the girder end and the first girder stirrup. As the negative moment rotation increased, strand mushrooming eventually caused the cap cover concrete to spall on the bottom surface, as shown in Fig. 6(b). As the cyclic testing continued, the core concrete in the girder-to-cap interface region began to crush and spall off, as shown in Fig. 6(c). The crushing and spalling eventually penetrated the connection interface, forming a void between the girder end and cap beam as cyclic of the load progressed, as shown in Fig. 6(d). Although these actions took place well pass the negative moment rotation corresponding to the design loads, the formation of the gap reduced the lever arm, causing the negative moment resistance to decrease gradually.

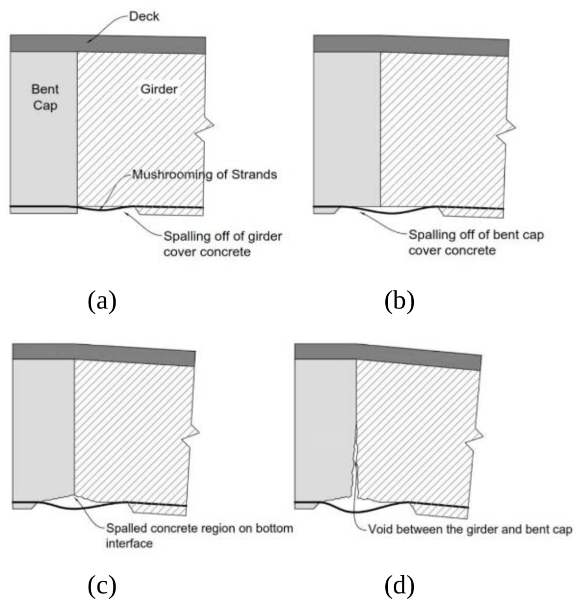
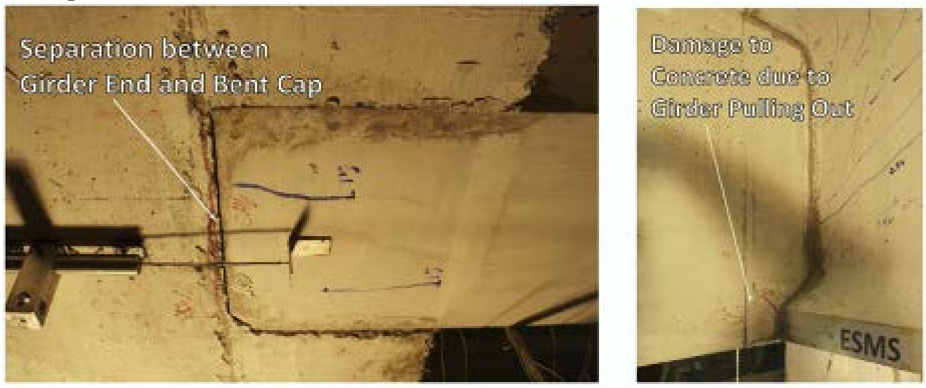


Fig. 6 – Illustrations showing Softening of Negative Moment Resistance

The positive moment failure in all connections except LUSC occurred due to combination of damage to concrete diaphragm from girder pull out and fracture of unstressed strands at the girder-to-cap interface region. In LUSC, large diameter dowel bars were used and all tension force required for the positive moment resistance was provided by a shear friction mechanism. The shear friction mechanism was developed between the precast girder and cast-in-place concrete diaphragm surrounding the girder end by utilizing dowel bars crossing girder-to-diaphragm interface. Three components, namely (1) cohesion; (2) dowel action; and (3) friction, constitute the shear friction mechanism. The cohesion is from shear transferred through the slip plane and results from aggregates from the diaphragm concrete bearing on the girder [8]. The dowel action provides additional resistance due to the stiffness of the reinforcing bars normal to their longitudinal axis. The friction component results from the "clamping" force developed at the interface due to tension in the dowel bars, thus allowing tension force resistance on the plane of the interface due to a friction mechanism. As the girder rotated upward in the connection region under positive moments, the embedded girder end would attempt to pull out from the diaphragm activating the shear friction mechanism. The first inelastic condition of the connections appeared to occur when the bottom of girders begins to separate from the cap beam as shown in Fig. 7(a), which initiates the degradation of cohesion. Friction resulting from "clamping" force and presumably dowel action maintain the shear friction mechanism until the damage to concrete on the front face of the diaphragm develops due to girder pullout (see Fig. 7(b)), ultimately reducing the dowel bars. At this point, the shear friction mechanism began to degrade, which is believed to be due to a reduction in the clamping force due to loss of bond between concrete and dowel bars. In addition, the unstressed girder strands extended from the girder and anchored into the cap beam with different anchoring details which provided direct tension transfer for the connections when subjected to large rotations. The eventual failure mechanism of the connections was due to fracture of unstressed strands at the girder-to-cap interface, as shown in Fig. 8, which confirmed that the anchorage details used in the connections were adequate. In LUSC, the failure was dictated by the shear-friction mechanism since no unstressed strands were provided across the connection interface.

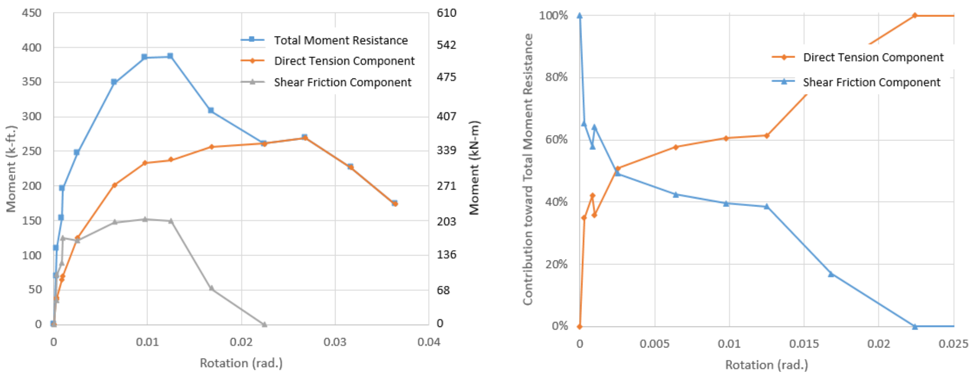


(a) (b)

Fig. 7 – Damage Resulting from Mobilizing the Shear Friction Mechanism

Fig. 8 – Fracture of Strands Providing Direction Tension Transfer

To illustrate the simultaneous contributions of two mechanisms in resisting the positive moment demand, Fig. 9 presents the positive moment contributions and percentages resisted by the shear friction mechanism and direct tension transfer as a function of the girder rotation. This figure was established based on the experimental data obtained from the ESLS connection test. The first significant change in the elastic response of the total moment resistance coincides with initiation of inelastic response of the shear friction mechanism. Thereafter, the increase in moment resistance is mainly due to the increased contribution of direct tension transfer through the extended girder strands. Meanwhile, the resistance of the shear friction mechanism remained constant until the damage to the diaphragm concrete occurred. The failure of the shear friction mechanism also triggered softening in the total moment resistance even though the direct tension transfer contribution continued to increase. The drop in this moment component eventually occurred due to the fracture of the strands.



(a) (b)

Fig. 9 – Moment Contributions and Percentages of Positive Moment Resistance in ESLS Connection

4. Guides for Selecting Connection Details

Since all six details investigated for the precast girder to cap beam connections have performed extremely well, this section provides guidance for selecting an appropriate concept and connection details for the use of precast girders in seismic regions. To facilitate with connection details beyond those tested, Table 2 to Table 5 list the advantages and challenges associated with the difference connection concepts and detailing options. Using this information, a designer can establish a suitable connection for precast girders in conjunction with cast-in-place or precast cap beams. More information appropriate for quantifying the reinforcement may be found in [9].

Table 2 – Choices for Positive Moment Resisting Mechanisms

Mechanism	Advantages	Challenges
Shear friction mechanism and Direct tension transfer through unstressed strands (the GUSC, ESSP, ESBF, ESMS, and ESLS connections)	Two complementary mechanisms contributing to the moment resistance. Unstressed strands provide a stable moment resistance at large rotations.	Unstressed strands need to be properly anchored in the bent cap.
Shear friction mechanism only (the LUSC connection)	Relatively less reinforcement is required in the connection region. Configuration allows for easy construction since strands are not placed through the girder-to-cap interface.	The mechanism of shear friction is considerably more complex due to more variables. Large diameter headed dowel bars are required. Additional strands are required within the precast girders. Only one test has been completed, and the connection reinforcement details are not optimized.

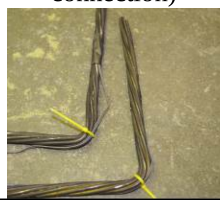
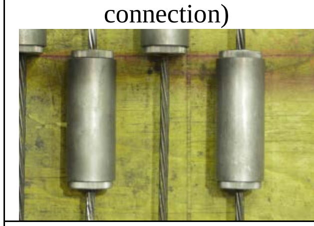
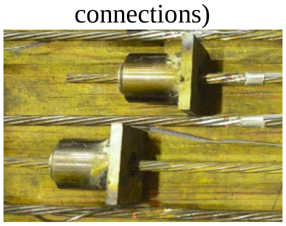
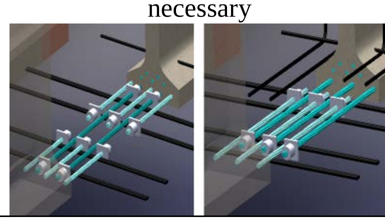
Table 3 – Bent Cap Options

Bent cap	Advantages	Challenges
Inverted-tee cap	The concept is suitable for precast bent cap option. No falsework is needed to support the precast girders before the continuity connection is cast. The ledges of the cap minimizes vertical shear slip between the girder and the cap.	Additional reinforcement and construction challenges may arise in the dapped end of girders and ledges of the inverted tee bent cap. The center of girder rotation is relatively difficult to determine due to the presence of ledges. A concrete diaphragm is required adjacent to the bent cap.
Rectangular cap	The design and constructions are relatively simple. Girders are embedded within the bent cap without needing a concrete diaphragm.	Falsework is required to support the precast girders. Girders may separate from the bridge deck if they are subjected to large inelastic rotations.

Table 4 – Options for Providing Unstressed Strands

Strand	Advantages	Challenges
Extend the prestressed strands from the precast girder (e.g., ESSP, ESBF, ESMS, and ESLS connections)	Relatively short embedment length to fully develop the strength of strand when embedded with anchorage devices.	Extended girder strands may be damaged during transportation. Care is needed to prevent the extended strands from unraveling when prestress is released. Use of precast bent cap may not be easy.
Grouted through a corrugated duct in the field (e.g., GUSC connection)	Precast bent caps can be easily implemented. No extension of strands is required. Strands are continuous through the cap and girder.	Corrugated ducts need to be placed in the precast girders and bent cap. A low construction tolerance is required to align the ducts during field assembly. The interface between the bent cap and girder require fiber-reinforced grout. Assurance of proper grouting of the duct may be difficult.

Table 5 – Options for Anchoring Strands within Cap Beam

Anchorage method	Advantages	Challenges
Free end with 90 degree bend (as used in ESBF connection) 	No anchorage device is required Easier to route the strands as they can be placed underneath the longitudinal reinforcement of the cap beam	Additional equipment may be required if strands are bent in field
Mechanical splice chuck (as used in ESMS connection) 	Short embedment length Installation is relative easy No significant congestion	Length of splice is longer than that used for the plate anchorage
Strand chuck consisting of a bearing plate, a barrel anchor, and wedges (as used in ESSP and ESLS connections) 	Relatively short embedment length is required to fully develop the strength of strand Extended strands from girders can be anchored by overlapping the strands or using splice strands. Test data, however, confirmed no overlapping is necessary 	Post-tensioning equipment is required to correctly install the strand chuck.

5. Conclusions

Given the lack of seismic connections between precast girders and concrete cap beams, an experimental investigation was undertaken. A total of six different connections were studied for precast I-shaped girders, precast bulb-tee girders, cast-in-place cap beams and a precast inverted-tee cap beam. The findings from this investigation have been presented in this paper. Based on these findings, the following conclusions can be drawn:

1. All six precast girder to cap beam connections performed satisfactorily when subjected to both positive and negative moments. The connections maintained elastic superstructure response at shear and moment demands expected due to gravity load, horizontal seismic loading and at least 0.5g vertical seismic acceleration effects. These observations confirmed the superiority of the new connections over the as-built details.
2. Based on the observed responses, the tested connection details can be used for precast I-shaped and bulb-tee girders in conjunction with precast or cast-in-place cap beam in seismic regions.
3. When combined with unstressed strands extended from girders for the purpose of establishing positive moment connection, the shear friction determines the initial stiffness of the connection and contributes to a large portion of the total positive moment resistance. The ultimate positive moment resistance of the connections is influenced by the number of extended girder strands.

4. Since the shear friction generated by the dowel bars in the diaphragm is a critical part of the positive moment transfer mechanism, the design of such connections should recognize the contribution of the two mechanisms.
5. All the strand anchoring details used in the tests were confirmed to provide sufficient anchorage to fully develop the strand strength.

6. Acknowledgment

The research team thanks Caltrans for sponsoring this research project and Dr. Charles Sikorsky for serving as the project manager for this research contract. The research team would also like to thank others who have also helped with this study for their support and assistance in the completion of the research presented in this paper.

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