*Proceedings of the*
*Annual Stability Conference*
*Structural Stability Research Council*
*San Antonio, Texas, March 21-24, 2017*

# Partial Depth Precast Concrete Deck Panels on Curved Bridges

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**Abstract**

A critical loading phase on steel I-girder bridges from a stiffness and strength perspective is the placement of the concrete bridge deck. At this stage, the non-composite steel girders resist the entire construction load. Traditionally, intermediate cross-frames or diaphragms are used to provide stability during construction; however alternative forms of bracing are of interest - especially those that increase construction speed and improve economy of the system. While partial-depth precast concrete deck panels (PCP) are often only used as stay-in-place formwork on straight bridges, they have significant in-plane stiffness and strength. With an adequate connection between the PCPs and the top flanges of the steel girders, the PCPs can provide significant contributions to the stability of both straight and curved bridges during construction. This paper documents results of an ongoing research study focused on the development of connection details for PCP bracing applications and the effectiveness of using the PCPs as braces during construction. The experimental portion of this study consists of full-scale laboratory tests on a 72 ft span twin girder system, monitoring the behavior of the system under simulated construction loads for straight and simulated curved girders. Buckling tests were carried out with and without intermediate bracing using a cross frame at midspan. The impact of the addition of the PCPs connected near the ends of the girders were evaluated with the laboratory tests. Additionally, parametric studies with a validated finite element analysis (FEA) of the system will be carried out to evaluate a wider array of systems.

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## 1. Introduction

The design of composite steel girder systems is often controlled by the lateral-torsional buckling of the girders prior to the curing of the concrete deck. At this phase, the girders must support the full construction load until the deck stiffens and can provide restraint to the top flange through composite action. For bridges with a curved geometry, significant torsional forces are induced from the self-weight of the system alone. Intermediate braces (usually cross-frames or diaphragms) are typically specified to reduce the unbraced length of the girders, control deformation, and resist the torsional loads during construction. Traditional cross-frames and diaphragm braces, however, are expensive to fabricate, can complicate the erection process, and cause fatigue issues during the service life of the bridge. These disadvantages have led to the investigation of partial depth precast concrete panels (PCPs) as a method of bracing the top flange during the construction phase.

In the state of Texas, PCPs are typically utilized in straight girder systems as formwork for the cast-in-place (CIP) deck while also acting as a structural deck element. There are several advantages to using a PCP formwork system, including an accelerated bridge construction time and an increase in durability from incorporating prestressing steel in the positive moment area of the deck (Merrill 2002). These panels are simply supported on the top flange of adjacent girder lines and rest on extruded polystyrene bedding strips, which can be cut to varying heights to accommodate differential elevations of the top flanges of adjacent girders. The unconnected configuration, however, does not allow for transfer of forces between the panels and the girders during the construction phase, thus not engaging the large in-plane stiffness and strength of the PCPs.

This paper discusses the results of an ongoing research study focused on the development of connection details for PCP bracing applications and the effectiveness of using PCPs as braces during construction. The experimental portion of this study consists of full-scale shear tests on PCPs, full-scale lateral tests on a twin steel I-girder system, and full-scale combined bending and torsion tests on a twin steel I-girder system. The PCP shear tests were conducted to aid in developing a simple and effective connection between the PCPs and the girder, as well as to empirically determine the in-plane stiffness and strength of the PCP/connection system. The full-scale I-girder tests were performed to investigate the performance of PCPs and their connection to a system that simulates the load experienced in a realistic construction situation. The results from the experimental tests are currently being used to validate the finite element models (FEM) that will be used for the parametric studies.

## 2. Background and Previous Work

### *2.1 Shear Diaphragm Bracing of Straight Girders*

The buckling capacity of an I-girder system can be substantially increased by connecting the compression flange of adjacent girders with shear diaphragms. The presence of a shear diaphragm restrains the warping deformation of the flanges (i.e., the in-plane bending of the flanges) that are connected to the diaphragm. From a structural perspective, it is advantageous to place the shear diaphragms at the location of maximum warping deformation (for example, at the end of the girders for a simply supported system). The solution for straight beams loaded with a uniform moment and braced by shear diaphragms was first solved by two independent studies published during the same time period (Errera and Apparao 1976; Nethercot and Trahair 1975). This solution has since been modified by Helwig and Frank (1999) so that it can be used for other loading conditions, resulting in the following equation:

$$M_{cr} = C_b^* M_g + mQd \tag{1}$$

where $M_{cr}$ = buckling capacity of the diaphragm-braced beam; $C_b^*$ = factor for moment gradient accounting for the effects of load height (Helwig et. al 1997; Galambos 1998), $M_g$ = buckling capacity of the girder without the shear diaphragm; $m$ = factor that accounts for the loading type; $Q$ = deck shear rigidity; and $d$ = depth of the girder. The deck shear rigidity is a function of $G'$ = diaphragm effective shear stiffness; and $s_d$ = the tributary width of deck bracing a single girder as shown in the following equation:

$$Q = G' s_d \tag{2}$$

As expected, the buckling capacity of a straight girder system connected with shear diaphragms grows as the effective shear stiffness of the diaphragm increases. In addition to stiffness, the capacity of the system also depends on the strength of the diaphragm and its connection to the girders since an effective brace must possess adequate stiffness and strength (Winter 1960). A cantilevered shear frame such as the one shown in Fig. 1 can be used to empirically determine the effective shear stiffness and ultimate strength of a shear diaphragm and its connection system. The effective shear modulus, $G'$, is determined as follows:

$$G' = \frac{PL}{fw\gamma} \tag{4}$$

where $P$ = lateral load on the test frame; $L$ = length of the test frame; $f$ = center to center spacing of the loading beams; $w$ = diaphragm width; and $\gamma$ = diaphragm shear strain.

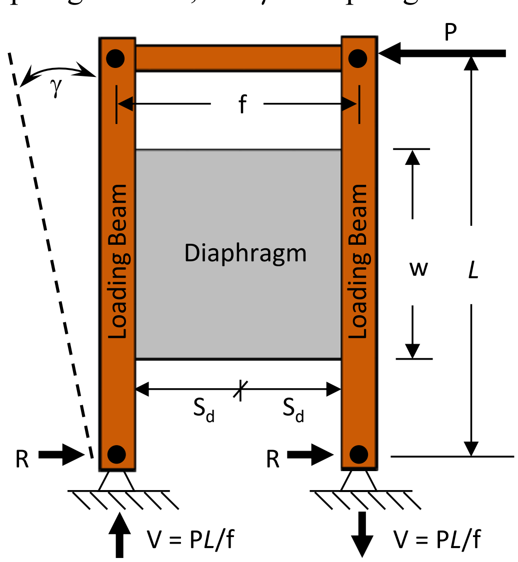


Figure 1: Shear Test Frame with Diaphragm

*2.2 Permanent Metal Deck Form Bracing*

Prior to the concept of using PCPs as shear diaphragms, a substantial amount of research was performed on the use of permanent metal deck forms (PMDF) as shear diaphragm bracing elements during the construction phase of a bridge (Helwig and Yura 2008a, 2008b). Developing a connection with adequate stiffness between the PMDFs and the girders proved challenging since the top flange of adjacent girders are often at varying elevations due to differential camber and/or a change in flange thicknesses. To maintain a constant deck thickness despite the variable girder elevations, support angles are welded to the edge of the girders' top flange that support the PMDFs as shown in Fig. 2a. Research by Currah (1993), however, showed that the eccentricity of the support angle significantly decreases the stiffness of the shear diaphragm system. To increase connection stiffness, Egilmez (2005) added stiffening angles that spanned between the adjacent girders (see Fig. 2b) which significantly stiffened the PMDF connection system.

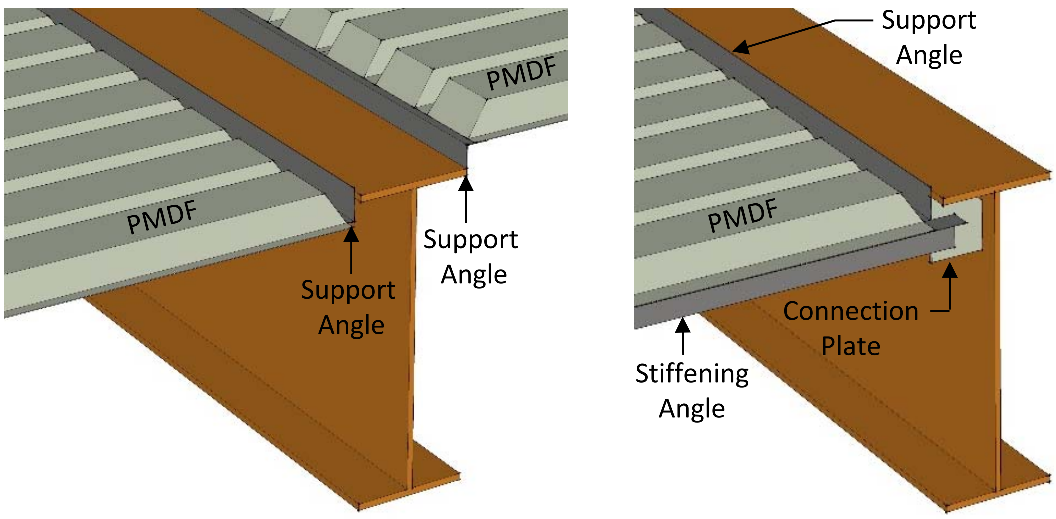


Figure 2: (a) Unstiffened PMDF Connection – Left (b) Stiffened PMDF Connection – Right

*2.2 Partial-Depth Precast Concrete Deck Panel Bracing*

Similar to PMDFs, one of the main challenges of using PCPs as a bracing element is developing a method of connecting PCPs to the girders that has adequate stiffness and strength. Furthermore, the connection must account for the variability in girder elevation and avoid complicating the precasting or construction processes. Prior to developing connection details, an understanding of the standard PCP fabrication and construction details was needed. According to the TxDOT specifications, PCPs have a standard thickness of 4" and must have a minimum 28 day strength of $f'c = 5000$ *psi*. The transverse panel reinforcement (perpendicular to the girder span) consists of 3/8" or 1/2" diameter (270ksi) prestressing strands typically spaced at 6" on center with a tension of 14.4 kips per strand. The longitudinal panel reinforcement (parallel to the girder span) can consist of unstressed prestressing strands, grade 60 reinforcing steel, or deformed welded wire reinforcement providing 0.22 square inches of steel per foot of panel width. The PCPs rest on extruded polystyrene bedding strips at the edge of the top flanges. According to the TxDOT standards, the maximum and minimum bedding strip heights are 4" and ½", respectively, which can accommodate a 3½" difference in elevation between adjacent girders. The PCPs must overlap the edge of the girder 1½" in addition to the width of the bedding strip to allow concrete to flow under the panel as the deck is placed. More specifics in regards to construction and fabrication details of PCPs can be found in the TxDOT standards (TxDOT 2006; TxDOT 2010).

## 3. Full-Scale PCP Shear Tests

### *3.1 Full-Scale Shear Test Frame*

To investigate the in-plane shear behavior of the PCPs with different connection details, a shear frame was fabricated at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin, as shown in Fig. 3. The shear frame consisted of six main parts, namely: two reaction blocks, two loading beams, one adjustable connecting strap, one hydraulic actuator, and four tie-down beams. To ensure accurate measurements, the members of the test frame were designed to have large axial, flexural, and torsional stiffness to minimize the elastic deformations of the frame during the tests. Furthermore, the frame was designed and detailed to minimize internal friction so that the measurements correctly reflected the strength and stiffness of the PCP/connection system. The shear frame used for this project resembles the one constructed by Currah (1993) that was used to investigate the in-plane stiffness and strength of PMDFs.

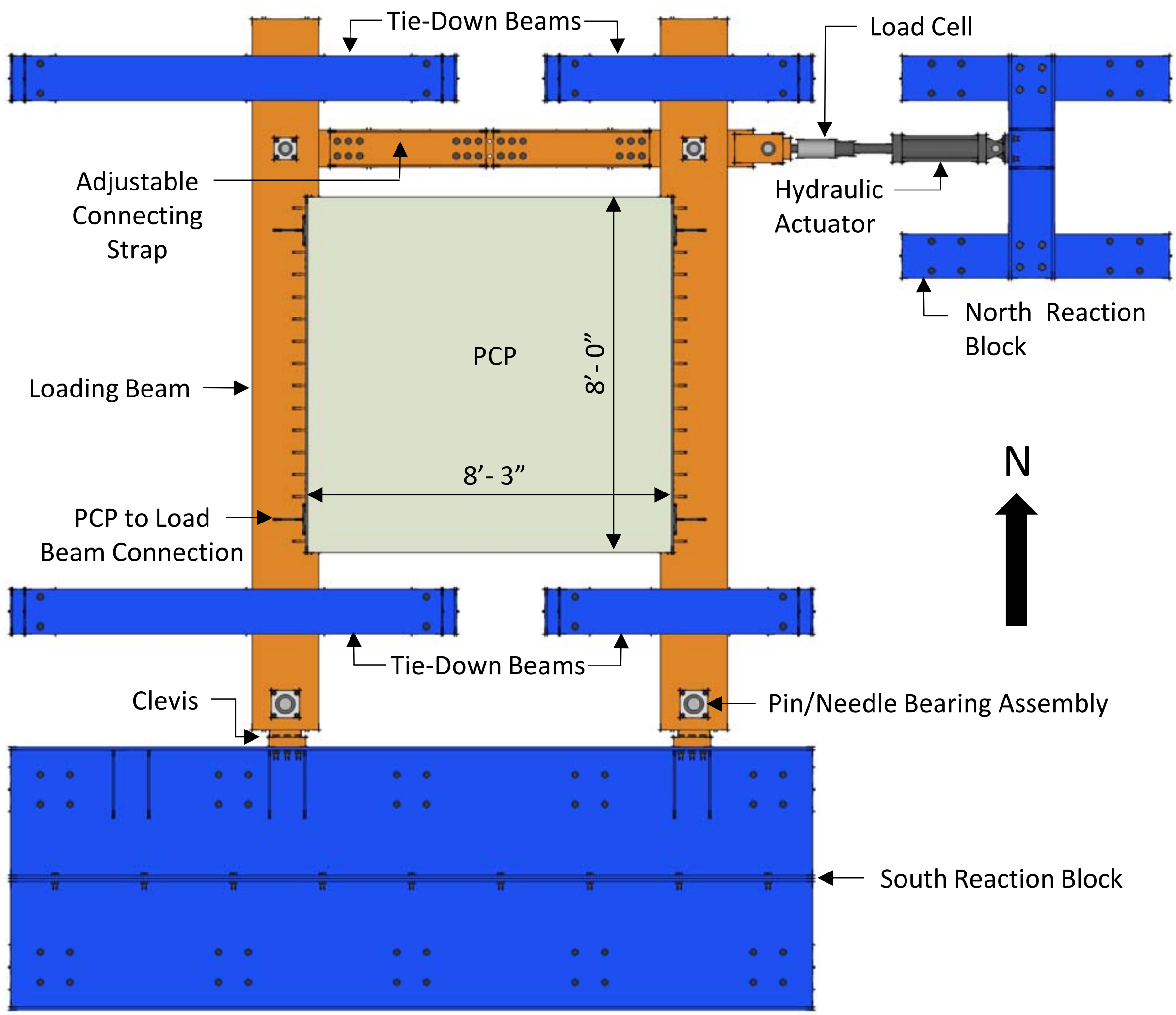


Figure 3: Shear Test Frame Plan View

Since the frame is a mechanism on its own (it can deflect laterally with insignificant load from the actuator), the PCP/connection system provides all of the lateral stiffness and strength to the system. As the load in the actuator increases, the loading beams remain parallel to each other while rotating at their base, inducing pure shear deformations on the connected PCP. From statics, the shear force on the PCP is equal to the axial force in the loading beams. Therefore, the relationship of shear force vs. shear strain and the effective shear modulus can easily be determined for the system.

*3.2 PCP to Girder Connections*

The research team sought the input from a local precaster and construction experts to help develop an adequate connection between the PCP and the girder without significantly complicating the construction or precasting process. After several preliminary tests, the connection shown in Fig. 4 was developed. The PCP is attached to the girders by a WT welded to the girder top flange and to an embed cast into the panel. The embed consists of a 2” wide flat bar extending the entire width of the PCP that rests above the prestressing strands. To transfer the load from the embed to the concrete, Nelson deformed bar anchors (D2L) were welded to the embed and cast into the PCP as shown in Fig. 5. Multiple WT sections accompanied by additional embed anchors can be utilized based on the load requirements for the system. A total of eight PCPs (8’-0” wide x 8’-3” long) were tested in which the following parameters varied: number of WTs, height of WTs, embed thickness, number of anchors, and anchor size. Table 1 shows a summary of the connection information for all eight PCPs. For consistency, all eight PCPs were cast from the same batch of concrete with the 28 day compression strength measured to be approximately *f’c = 8,600* psi.

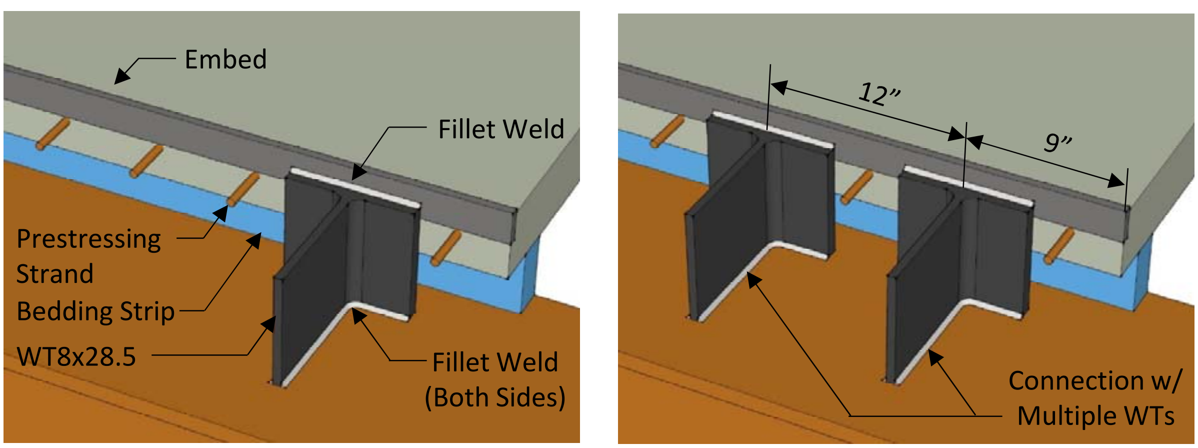


Figure 4: Detail of PCP Connection to Top Flange

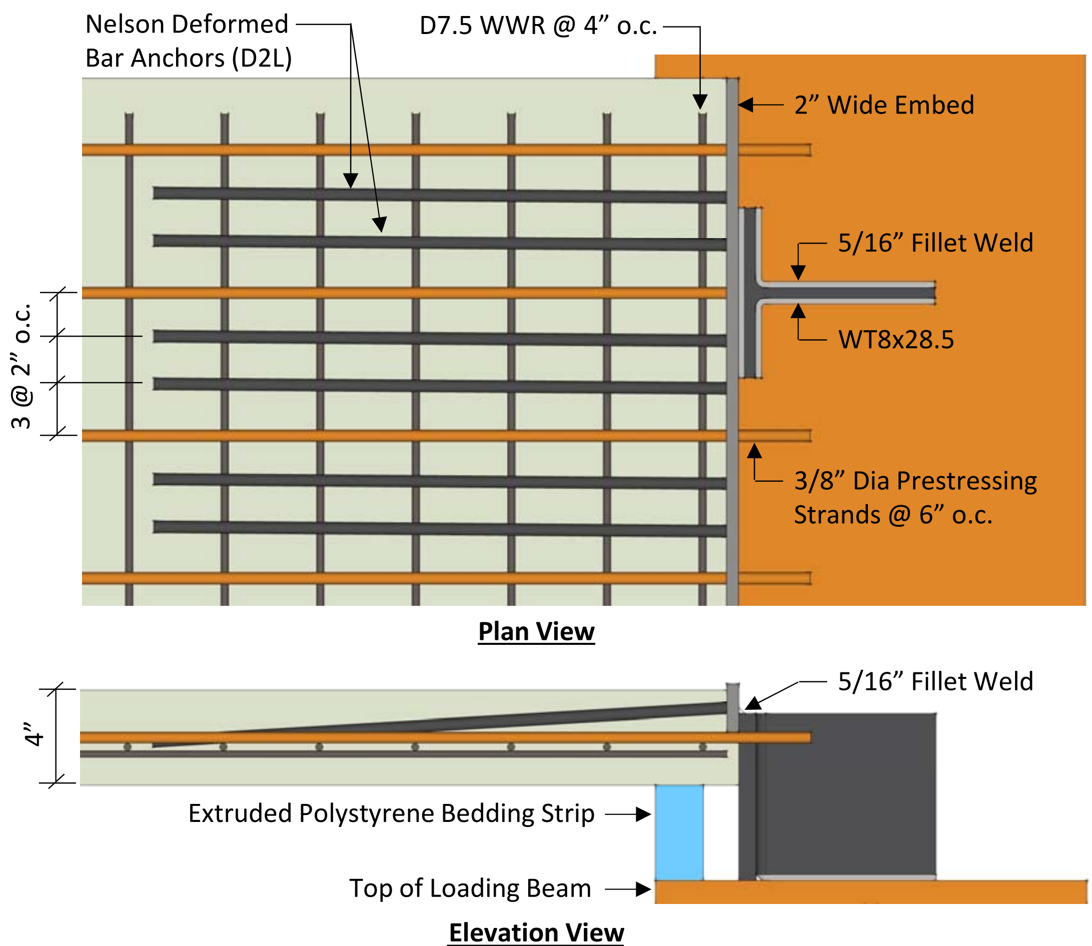


Figure 5: Plan and Elevation Views of the Embed-Anchor Detail

**Table 1: Summary of PCP Connection Details**

| Label | Anchors per Corner | Embed Size | WTs per Corner | Bedding Strip Height |
|---|---|---|---|---|
| A.1.MAX | (6) 1/2" Ø x 2'-0" Long | 2"x1/2" | (1) WT8x28.5 x 7" | 4" |
| A.1.MIN | (6) 1/2" Ø x 2'-0" Long | 2"x1/2" | (1) WT8x28.5 x 3.5" | 1/2" |
| B.1.MAX | (6) 5/8" Ø x 2'-6" Long | 2"x5/8" | (1) WT8x28.5 x 7" | 4" |
| B.1.MIN | (6) 5/8" Ø x 2'-6" Long | 2"x5/8" | (1) WT8x28.5 x 3.5" | 1/2" |
| C.2.MAX | (10) 1/2" Ø x 2'-0" Long | 2"x5/8" | (2) WT8x28.5 x 7" | 4" |
| C.2.MIN | (10) 1/2" Ø x 2'-0" Long | 2"x5/8" | (2) WT8x28.5 x 3.5" | 1/2" |
| D.2.MAX | (8) 5/8" Ø x 2'-6" Long | 2"x3/4" | (2) WT8x28.5 x 7" | 4" |
| D.2.MIN | (8) 5/8" Ø x 2'-6" Long | 2"x3/4" | (2) WT8x28.5 x 3.5" | 1/2" |

*3.3 Experimental Results*

Fig. 6 shows the experimental results for the shear tests on the eight PCPs. As expected, the stiffness of the system was inversely proportional to the height of the bedding strip (distance from the bottom of the PCP to top of the flange). Furthermore, an increase in ultimate load was seen with an increase in the number of WTs per corner. Also, no significant benefit was seen from using 5/8” Ø x 2’-6” long anchors vs. using 1/2” Ø x 2’-0” long anchors. All of the PCPs (except for B.1.MAX) failed via concrete breakout of the top face of the PCP at the location of the anchors (see Fig. 7). The B.1.MAX connection failed due to a weld rupture between the WT and the loading beams of the test frame.

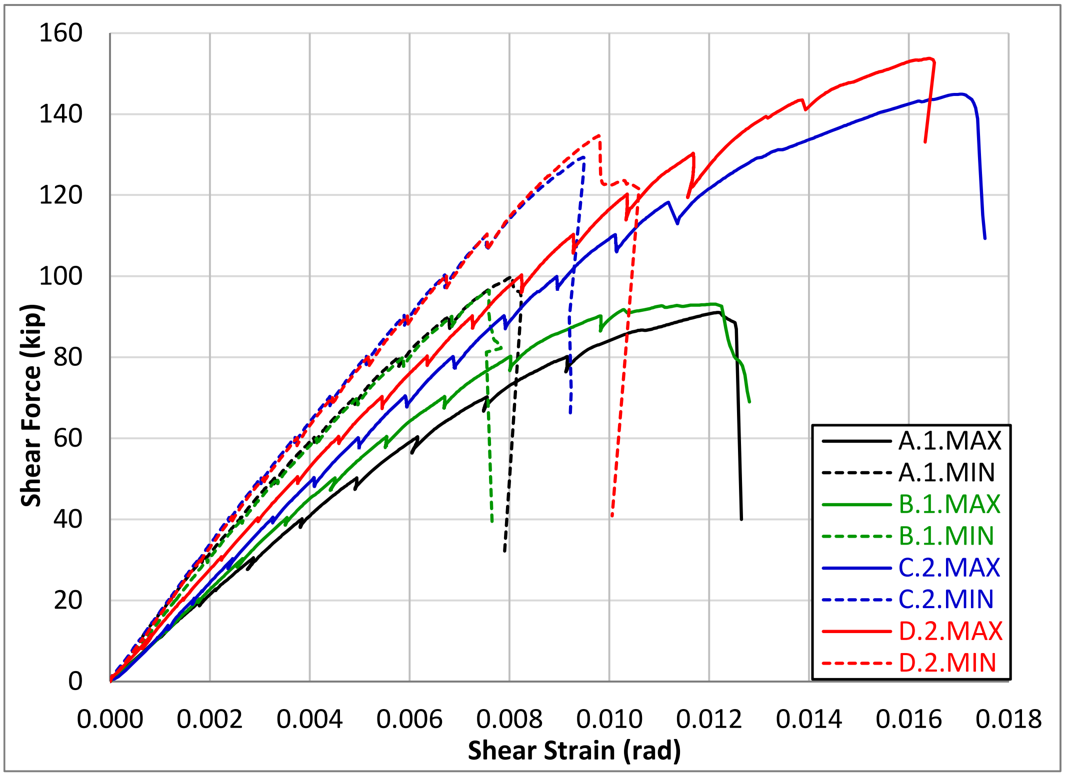

Figure 6: Empirical PCP Test Results

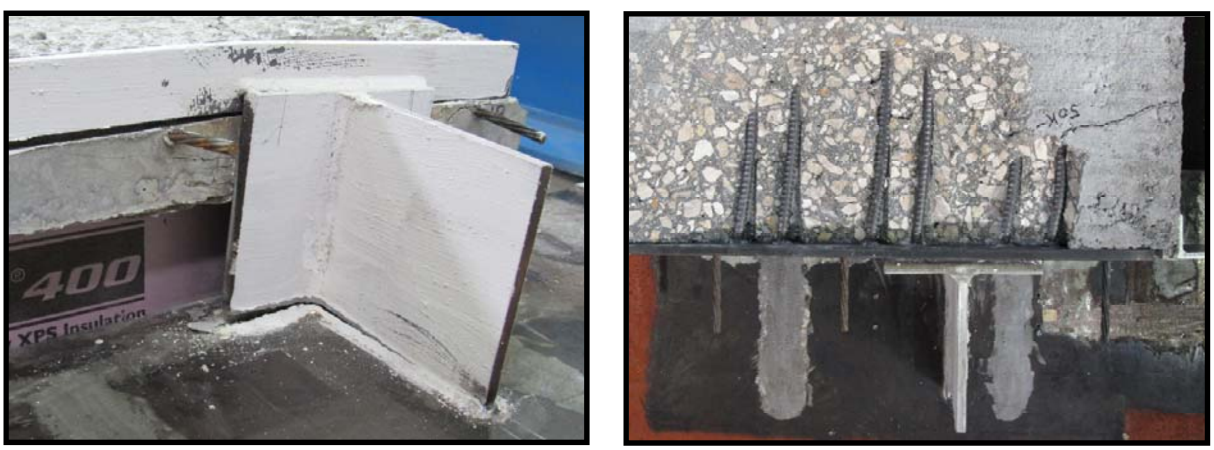
Figure 7: PCP Failure Mechanisms: (a) B.1.MAX – Left (b) All other PCPs - Right

## 4. Full Scale Twin Steel I-Girder Tests

*4.1 Twin Steel I-Girder Test Set-Up*

A full-scale twin I-girder assembly was fabricated to investigate the effectiveness of the PCPs as bracing elements in curved and straight girder systems. PCPs with connection detail A.1.MAX were used for the twin I-girder tests since this detail had the lowest stiffness and strength and would therefore provide conservative results for the other details. The configuration consisted of two straight, simply supported A992 steel W36x135 sections spaced at 8'-8" on center with a clear span of 72 ft (for an L/D ratio of 24). The system was designed to behave elastically so that multiple tests could be conducted with various PCP configurations and since inelasticity typically does not occur during the construction phase of a bridge. A removable cross-frame was designed so that the unbraced length of the girders could be halved and so that the influence of the attached PCPs on the forces in the cross-frame could be investigated. The girders were supported vertically by thrust bearing assemblies and laterally by jamb bolts at the edge of the flange to provide a torsional restraint at the end of the girders and to minimize the warping restraint of the top and bottom flanges.

*4.2 Full-Scale Lateral Tests on Twin Steel I-Girder Systems*

To test the system's lateral stiffness, three lateral load frames were placed on the west side of the structure and threaded rod assemblies were used to apply lateral force at the top flange of the girders. Forces were applied independently at midspan and at approximately the third points (20 ft from each end), creating multiple loading scenarios to validate finite element models. The PCPs were attached to the top flange near the supports at each end and the behavior of the system was observed for the cases without PCPs, with 2 PCPs, and with 4 PCPs. Fig. 8 shows a model of the lateral test assembly.

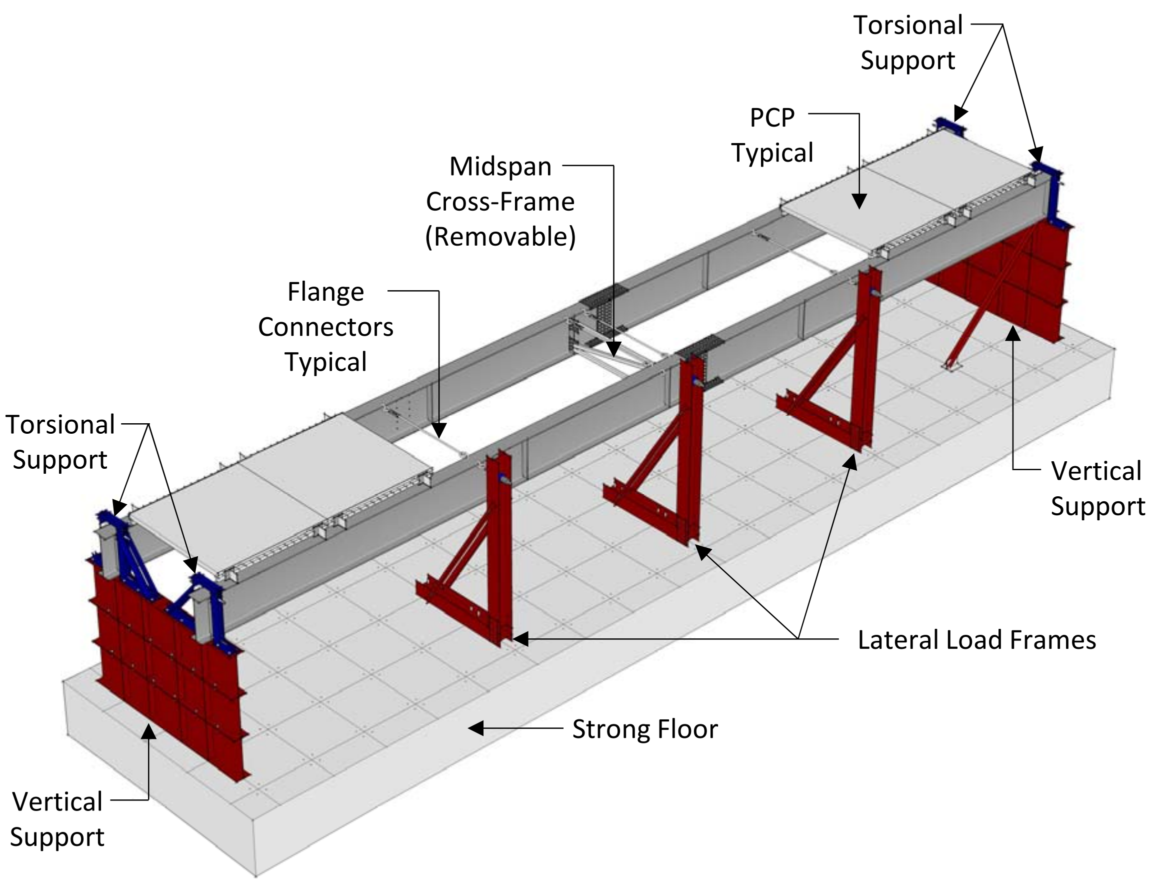


Figure 8: Twin I-Girder Lateral Test Set-Up

The results from the lateral loading test showed that connecting the PCPs to the top flange of the I-girders significantly reduced the lateral deflection (Fig. 9) and twist (Fig. 10) of the girders. For this case, the girders were loaded at the top flange at the approximate third points along the length without the centerline cross-frame installed. The addition of one PCP per end (2 PCPs total) reduced the maximum top flange lateral deflection by a factor of 6.8 (from 6.56 in to 0.97 in) while the addition of one more PCP per end (4 PCPs total) further reduced the top flange lateral deflection by a factor of 2.7 (from 0.97 in to 0.36 in).

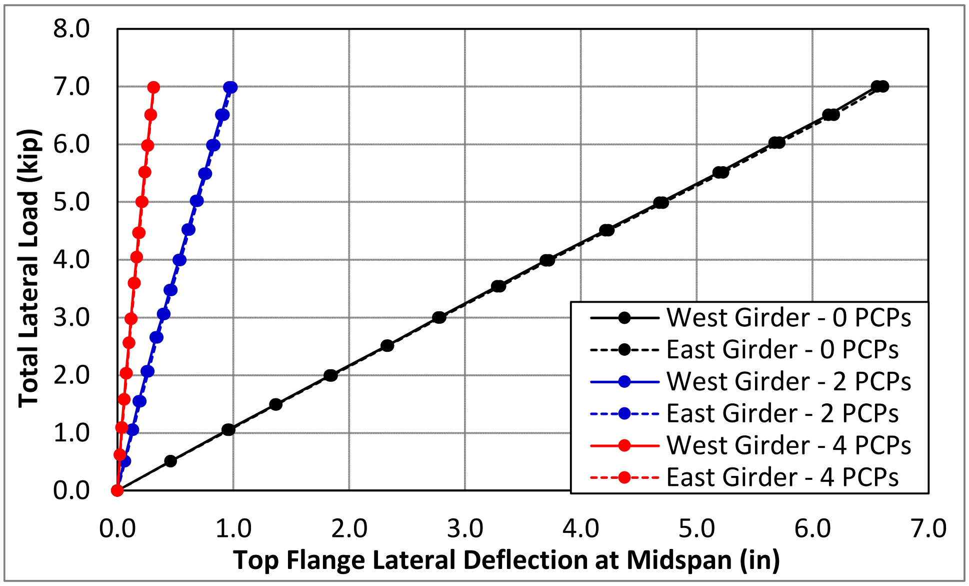

Figure 9: Midspan Deflection of Girders with Various Numbers of Attached PCPs

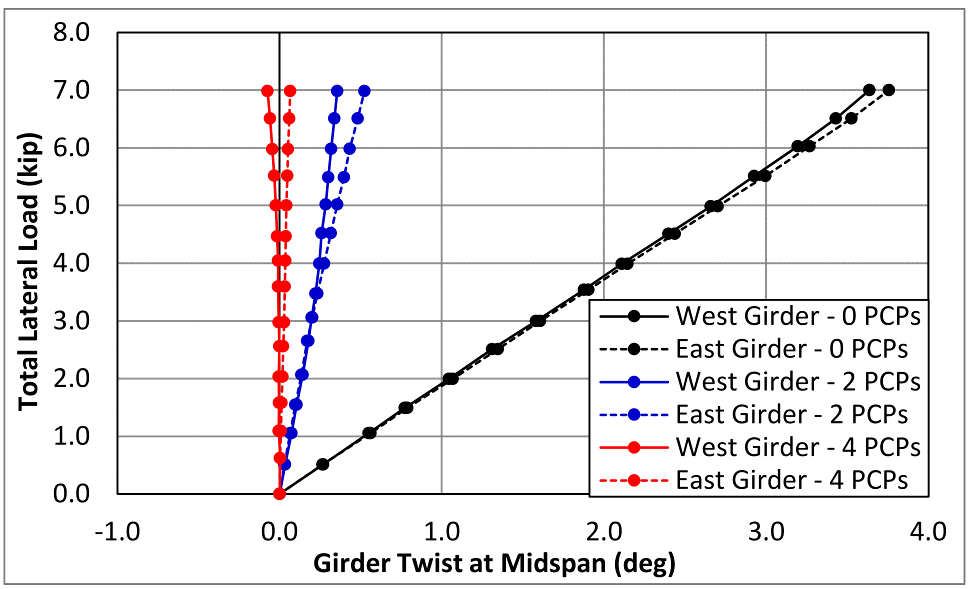

Figure 10: Midspan Twist of Girders with Various Numbers of Attached PCPs

Along with reducing deflections and twist, the addition of PCPs to the system reduced the centerline cross-frame forces when a cross-frame was included as seen in Fig. 11. The diagonal of the cross frame experienced a maximum load of 4.99 kips when no PCPs were installed on the system. Adding 2 PCPs reduced the diagonal force by a factor of 4.8 (from 4.99 kips to 1.03 kips) while adding two more PCPs (for 4 PCPs total) further reduced the diagonal force by a factor of 5.4 (from 1.03 kips to 0.19 kips).

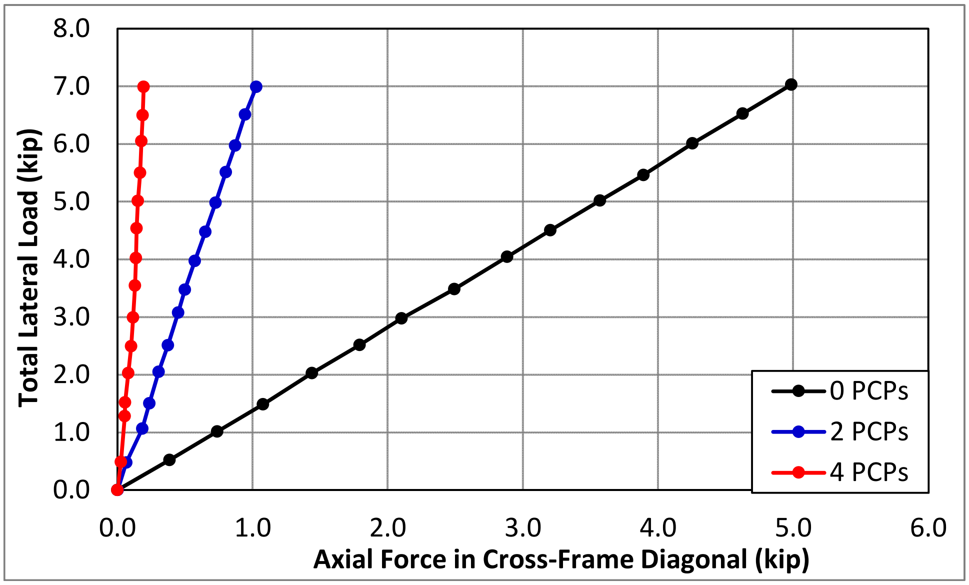


Figure 11: Forces in Cross-Frame Diagonal with Various Numbers of Attached PCPs

*4.3 Full-Scale Bending and Torsion Tests on Twin Steel I-Girder Systems*

The twin I-girder test set-up was fabricated so that various combinations of bending and torsion could be applied to a straight girder system and allow girders with multiple radii of curvature to be simulated with a single system (Fig. 12). The vertical load was applied by two gravity load simulators (GLS) attached to load beams by a clevis. The load beams applied the force to the system through knife edge and thrust bearing assemblies (to minimize the warping restraint of the loading beams). The GLS applied a vertical load to the beams without providing any lateral restraint to the system, shown in detail in Fig. 13. To induce a torsional load on the girders, the GLS and load beams were offset so that the knife edges were eccentric to the shear center of the girders. The tests were performed with varying numbers of PCPs attached to the ends of the system, at different load eccentricities, both with and without the centerline cross-frame installed.

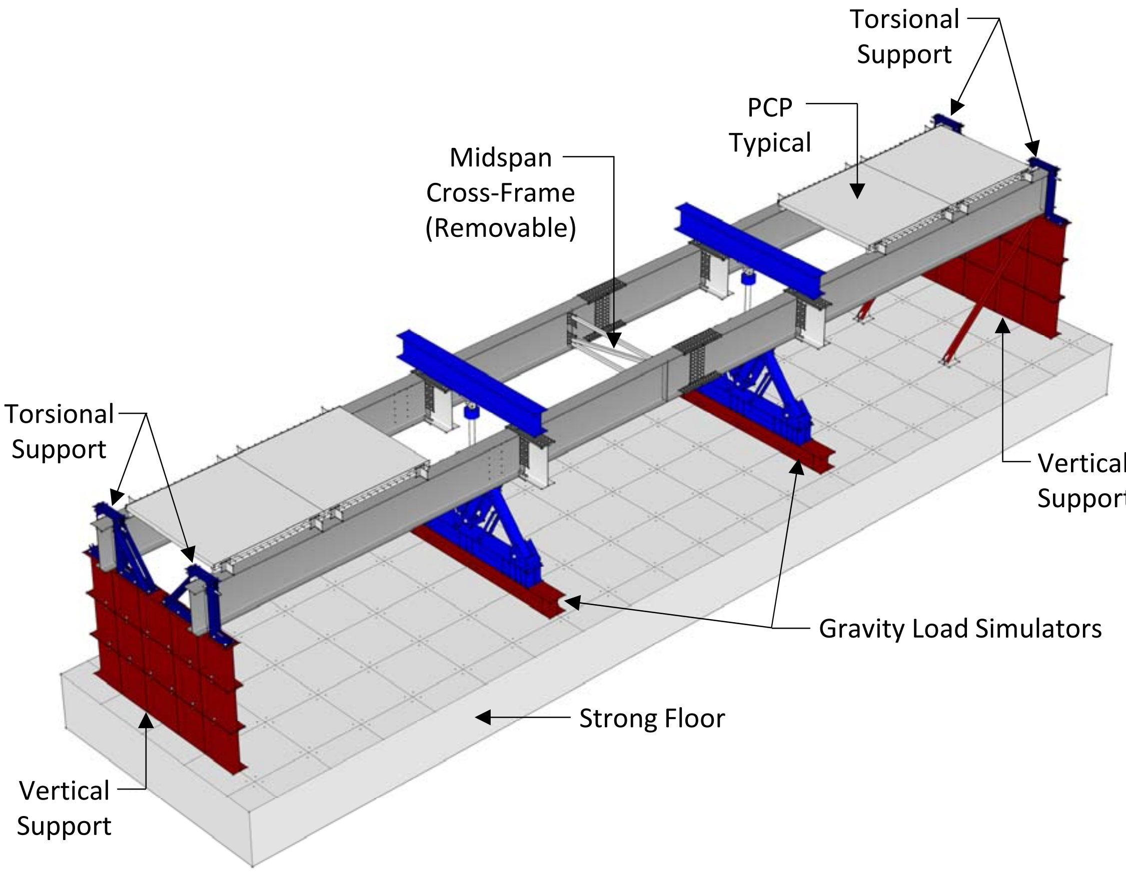


Figure 12: Twin I-Girder Combined Bending and Torsion Test Set-Up

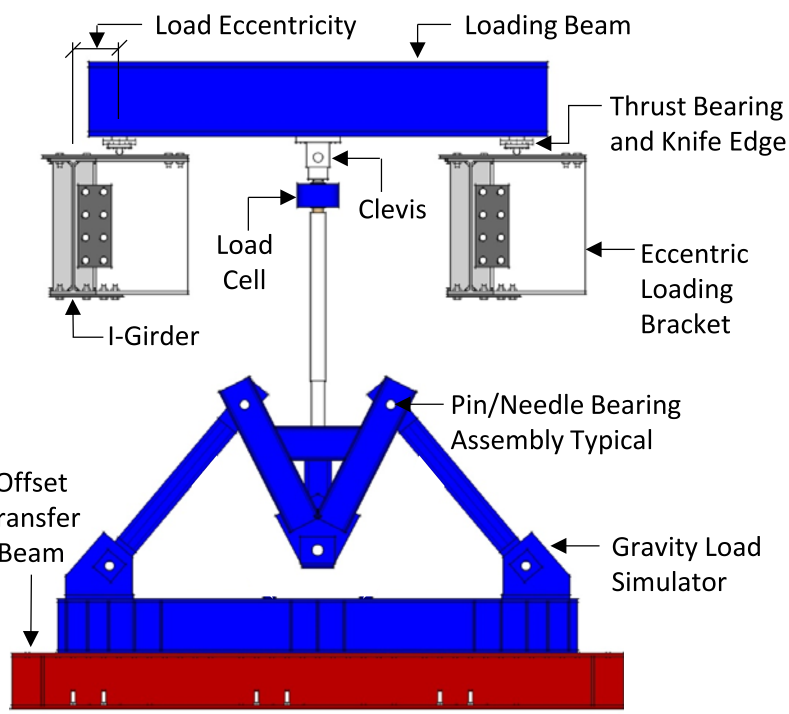


Figure 13: Twin I-Girder Vertical Loading System

The experimental data from the twin I-girder buckling tests showed that the PCPs had a significant impact on the lateral-torsional buckling behavior of both straight concentrically and eccentrically loaded systems. Fig. 14 shows the experimental results when the girders are loaded through the shear center and the intermediate cross-frame is not installed at midspan. Considering, for example, each case when the girders reach a rotation of two degrees, it is clear that the buckling capacity of the system greatly increased with the addition of PCPs. Without PCPs attached, a load of 13 kip in each GLS was required to reach two degrees of rotation in the girders. When a PCP was connected to each end of the system (2 PCPs total), a load of 40 kip was required to reach an average girder rotation of two degrees (increasing the system's capacity by a factor of 3.1). Adding another PCP to each end (4 total), 74 kip was required from each GLS to achieve an average girder rotation of two degrees (increasing the system's capacity by a factor of 1.8 from the two PCP case and by a factor of 5.7 from the case without PCPs).

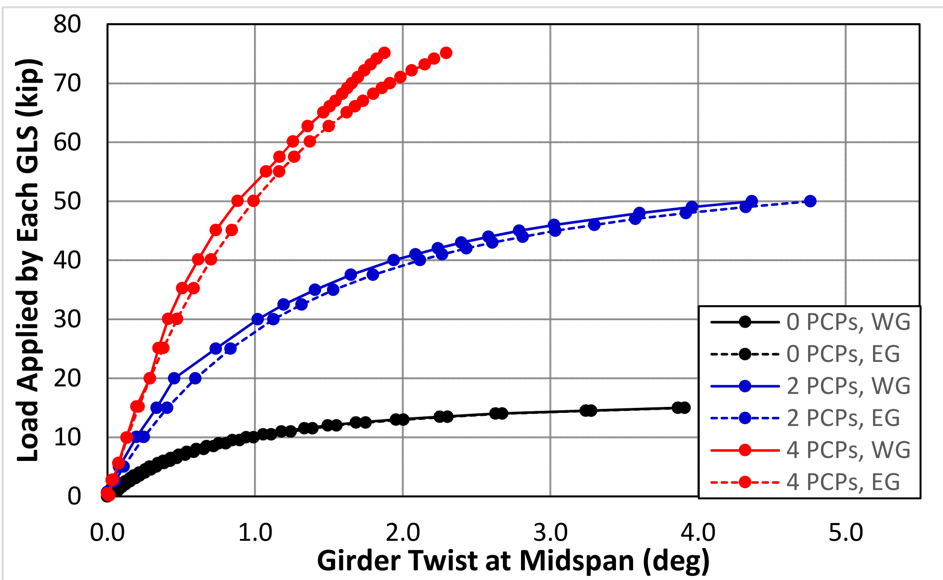

Figure 14: Midspan Girder Twist with Concentric Load, No Intermediate Cross-Frame and Varying PCPs

Similar results were observed when the load was applied on the same system with a 12-inch eccentricity as shown in Fig. 15. As before, the loads were applied at third points with no intermediate cross-frame. Considering again when the girders reach two degrees of rotation, the system capacity was significantly increased when PCPs were added. With no PCPs attached, the girders reached two degrees of rotation with 3.5 kip present in each GLS. When two PCPs were connected, a load of approximately 5.9 kip was required in each GLS to reach an average rotation of two degrees, (increasing the system's capacity by a factor of 1.7). When four PCPs were connected to the I-girder system, approximately 9.3 kip was required from each GLS to achieve an average rotation of two degrees, (increasing the system's capacity by a factor of 1.6 from the two PCP case and by a factor of 2.6 from the system without PCPs).

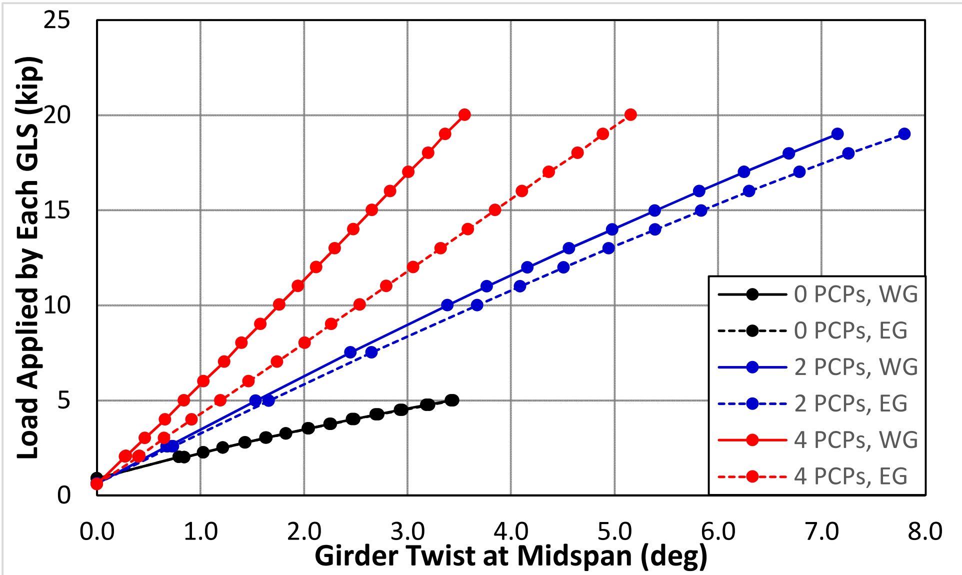

Figure 15: Midspan Girder Twist with 12" Eccentric Load, No Intermediate Cross-Frame, and Varying PCPs

The test results in Fig. 15 showed that the rotation of the west girder was less than the rotation of the east girder when PCPs were used in the system. This phenomenon was caused by the connection detail used to connect the PCPs to the top flange. The rotation direction for the west girder caused the WT to bear against the edge of the PCP, providing additional tipping restraint to the girder. The rotation direction for the east girder allowed the WT to pull away from the face of the PCP and thus less tipping restraint was provided for the east girder compared to the west girder. Fig. 16 shows the tipping restraint of the girders by the connection for each beam.

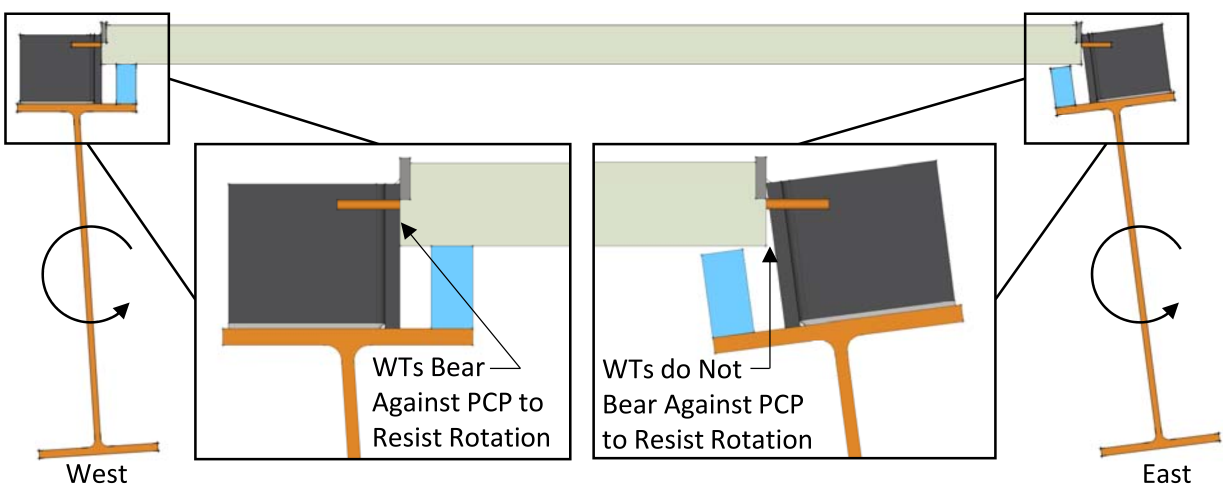

Figure 16: PCP to Top Flange Connection for the West and East Girders

## 5. Conclusions

This paper summarized the results of an ongoing research project focused on using PCPs as shear diaphragm bracing elements for curved girder systems. Researchers in this study developed a simple and effective means of attaching PCPs to the top flange of the girders. The results from the shear panel tests for eight variations of this connection were presented in this paper. The connection resulting in the least stiffness and strength was conservatively used in the twin I-girder tests. The lateral twin I-girder tests showed that using PCPs as bracing elements significantly reduced the deformation of the system and also reduced the cross-frame forces (when it was attached at midspan). The combined bending and torsion twin I-girder tests showed that connecting PCPs at the ends of the girders increased the stiffness and load carrying capacity of the system by a significant amount. The finite element models are currently being validated with the experimental results from both the lateral loading tests and the combined bending and torsion tests. Upon the completion of the FEM validation, parametric studies will be conducted to determine the bracing capabilities of PCPs on a wide range of curved bridge applications.


## Acknowledgments

The writers gratefully acknowledge the Texas Department of Transportation for providing funding for this project and for their assistance with this research. The authors also gratefully acknowledge Valley Prestress Products for providing the precast panels used in the experiments for their technical assistance and advice. The information and opinions presented in this paper of those of the authors, and do not necessarily reflect the views of the Texas Department of Transportation or Valley Prestress Products.


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