Buckling Behavior of Steel Bridge I-Girders Braced by Permanent Metal Deck Forms

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Abstract: Permanent metal deck forms (PMDFs) are often used in the bridge industry to support wet concrete and other loads during construction. Although metal formwork in the building industry is routinely relied on for stability bracing, the forms are not permitted for bracing in the bridge industry, despite the large in-plane stiffness. The forms in bridge applications are typically supported on cold-formed angles, which allow the contractor to adjust the form elevation to account for changes in flange thickness and differential camber between adjacent girders. Although the support angles are beneficial toward the constructability of the bridge, they lead to eccentric connections that substantially reduce the in-plane stiffness of the PMDF systems, which is one of the reasons the forms are not relied on for bracing in bridge applications. This paper documents the results of an investigation focused on improving the bracing potential of bridge deck forms. Modifications to the connection details were developed to improve the stiffness and strength of the forming system. Research included buckling tests on a 15-m (50-ft) long, twin-girder system with PMDFs for bracing. In addition, twin-girder tests were also used to validate computer models of the bracing systems that were used for parametric finite-element analytical studies. The buckling test results demonstrated that modified connection details make PMDF systems a viable bracing alternative in steel bridges, which can significantly reduce the number of cross-frames or diaphragms required for stability bracing of steel bridge I-girders during construction. **DOI: 10.1061/(ASCE)BE.1943-5592.0000276.**

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Introduction

Lateral torsional buckling is a failure mode that often controls the design of steel I-girders in steel bridge construction. The critical stage for buckling usually occurs during placement of the concrete bridge deck, when the noncomposite steel girder supports the entire load. Conventional steel bridge systems use bracing in the form of intermediate cross-frames or plate diaphragms that reduce the unbraced length to increase the buckling capacity of the girders. These bracing systems are relatively expensive components of bridges because of the amount of fabrication required. In addition, routine maintenance, such as inspections and painting around cross-frame locations, is often more complicated than in other parts of the bridge. Therefore, minimizing the number of intermediate brace points along the length of the bridge is of interest. One possible source for stability bracing during concrete placement is the permanent metal deck forms (PMDFs), which are frequently used to support the concrete bridge deck during construction. PMDFs are also referred to as stay-in-place forms or metal decking. Although PMDF systems are frequently relied on for stability bracing in the building industry, the forms are generally not considered for bracing in the bridge industry.

The forming systems used in the building industry differ from those in the bridge industry in both shape as well as method of connection. While the forming sheets in the bridge industry are usually stiffer than comparable forms in the building industry, the overall forming system in bridges is typically more flexible because of the differences in the connection method. Forms in the building industry are typically continuous over the tops of the girders and are fastened directly to the girder flanges by shear studs or other mechanical fasteners. In the bridge industry, the individual form panels span between the flanges of adjacent girders and are supported on cold-formed angles, as shown in Fig. 1. The angles allow the contractor to adjust the form elevation to account for variations in flange thickness or differential camber between adjacent girders. Although the support angles provide the ability to adjust the form elevation, they lead to eccentric connections that substantially reduce the in-plane stiffness of the PMDF systems as a bracing element.

The purpose of this study is to increase the understanding of the bracing behavior of PMDF systems, while also developing improved connection details for the forms in bridge applications. The investigation included both experimental and computational studies. The experimental program consisted of tests on PMDF systems in a shear frame and also tests on a 15-m (50-ft), twin-girder system with PMDFs for bracing. The shear tests (Egilmez et al. 2007) were used to develop modified connection details to enhance the bracing behavior of the forming system. In addition to the shear tests, large-scale tests were conducted on systems with both conventional and modified connection details between the formwork and the top flange of the twin-girder system. The large-scale tests consisted of both lateral load tests and buckling tests on the twin-girder systems with the PMDFs for bracing. The lateral

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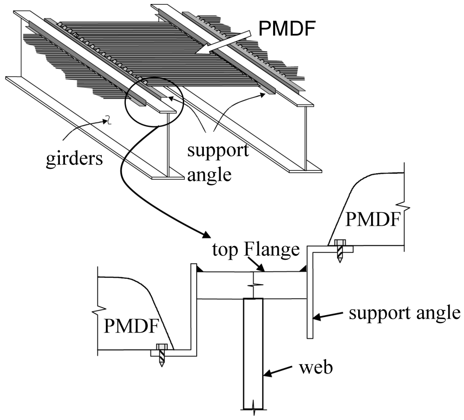

Fig. 1. Eccentric connection utilized in the bridge industry

displacement tests provided measurements of the lateral stiffness of the twin-girder system braced by PMDFs with deformations consistent with the shapes of the buckled girders. These tests were also used to develop a finite-element analytical (FEA) model of the PMDF systems; therefore, the parametrical analyses could be carried out to measure the buckling behavior of the PMDF braced systems. Complete results from shear diaphragm and lateral displacement tests are documented in Helwig et al. (2005) and Egilmez et al. (2009).

This paper presents results from the twin-girder buckling tests as well as comparisons of results from the FEA model. Buckling tests were performed on the twin-girder system to improve the understanding of the buckling behavior of steel I-girders braced with PMDFs. The PMDFs act as a shear diaphragm that can reduce the number of cross-frames required for stability bracing on steel I-girder bridges.

The following section provides pertinent background information on shear diaphragm bracing for beams, existing and proposed connection details for PMDF bracing, and information on modeling techniques for diaphragm-braced beams. An overview of the test setup is then provided followed by a presentation of results from the twin-girder buckling tests. The test data are compared with results from the finite-element analyses to validate the FEA model used in parametric studies to develop design methodologies (presented in a future publication. The final section provides a summary of observations and conclusions.

Background

There are a number of methods for providing stability bracing to beams; however, the majority of beam bracing systems can be categorized as either lateral or torsional bracing. Conventional bracing systems for bridge girders consist of cross-frames or diaphragms that restrain girder twist and therefore fit into the category of torsional bracing. In the building industry, metal deck forms are routinely relied on for stability bracing. The forms are typically considered as a shear diaphragm, which resist the lateral deformation of the top flange in positive moment regions by restraining the warping deformation along the length of the beams.

Although the current AASHTO standard specifications (AASHTO 2002) do not allow PMDFs as a bracing source for steel bridge I-girders, previous studies in the building industry have demonstrated that metal deck forms can significantly increase the buckling capacity of girders, provided that the decks are properly attached to girder top flanges (Errera and Apparao 1976; Nethercot and Trahair 1975; Helwig and Frank 1999). These investigations resulted in approximate equations for calculating the buckling capacity of girders braced by a shear diaphragm on the top flange and subjected to different loading conditions. Helwig and Yura (2008a, 2008b) conducted parametric finite-element analyses on girders to develop stiffness and strength requirements for shear diaphragm bracing. The stiffness requirements for stability bracing are often reported as a function of the ideal brace stiffness, which is the stiffness required for perfectly straight members to support a certain load. To control deformations and brace forces, the actual stiffness required is higher than the ideal stiffness.

Although most work on shear diaphragm bracing has primarily targeted forming systems used in the building industry, there have been investigations focused on the systems used in bridges. Most of this work was conducted at the University of Texas at Austin during the 1990s (Currah 1993; Soderberg 1994; Helwig and Frank 1999). These included laboratory tests and analytical studies and demonstrated that the bracing provided by the bridge deck forms were significant, as long as the support angle rotation was controlled. The research in this study included three stages of testing, along with parametric finite-element analyses. The following section provides a summary of some of the work in this study and is important for understanding the results from the buckling tests and subsequent FEA modeling.

Shear Panel and Lateral Stiffness Tests

The primary experiments consisted of full-scale buckling tests; however, several preliminary tests were necessary prior to the buckling experiments to obtain measurements of the bracing properties of the deck. These were determined from shear panel tests and lateral load tests on the twin-girder systems with PMDFs for bracing. The shear panel tests played an important role in the development of connection details to improve the PMDF bracing ability and also provided the fundamental properties for shear stiffness and strength for bracing applications. The lateral load tests (Egilmez et al. 2009) provided a direct indictor of the stiffness of the metal deck forms fastened to the girders and was important for ensuring that a good model of the forming systems was achieved. The shear panel tests and lateral load tests are discussed subsequently. A summary of the finite-element model is also provided, containing complete comparisons between lateral displacement experiments and the FEA solution.

Shear Diaphragm Tests

Shear tests on PMDF systems were conducted in the shear test frame similar to that depicted in Fig. 2. The frame consisted of two relatively rigid beams linked together at the ends. An actuator was used to displace the end of the frame and subject the test panel to shearing deformations. A discussion of the testing frame fabrication and geometry along with the testing procedure is provided in Egilmez et al. (2007). The tests were conducted to measure the stiffness of PMDF systems with existing connection details, as well as to develop connection modifications that improve the stiffness of eccentrically connected metal deck forms. The failure of the deck panel for cases with the eccentric connection typically involves a severe deformation of the support angle, as shown in Fig. 3. The flexibility of the eccentric connection has a significant impact on the bracing behavior of the PMDF system, because bracing systems typically follow the behavior of springs in series. The small stiffness of the connection usually dominates the stiffness of the PMDF system, as indicated in the following expression:

J. Bridge Eng., 2012, 17(4): 624-633

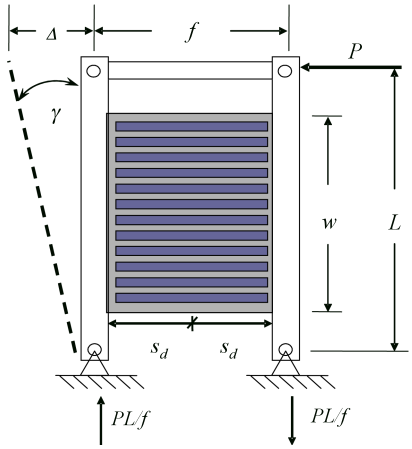

Fig. 2. Shear test frame with the PMDF specimen

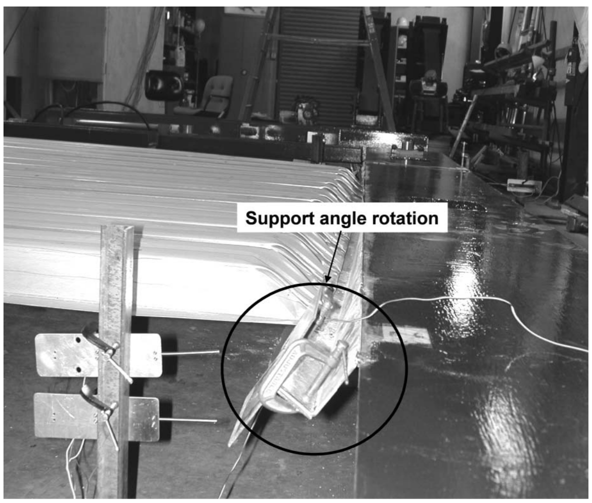

Fig. 3. Failure of eccentric support angle [Reprinted with permission from Helwig et al. (2005)]

$$\frac{1}{\beta_{\mathrm{sys}}} = \frac{1}{\beta_{\mathrm{deck}}} + \frac{1}{\beta_{\mathrm{con}}} \tag{1}$$

where the inverse of the total system stiffness (β_{sys}) is equal to the sum of the inverses of the component stiffnesses (β_{deck} = stiffness of the deck form and β_{con} = stiffness of the connection). The system stiffness is smaller than the smallest component (β_{deck} or β_{con}).

A number of details were tested to improve the connection stiffness; however, one proved both practical and effective. The proposed modification involves a transverse stiffening angle that spans between adjacent girder flanges, positioned to coincide with a PMDF sidelap joint; therefore, the deck was fastened directly to the angle, as shown in Fig. 4. In order to have the same eccentricity for the stiffening and support angle, the ends of the stiffening angle were welded to the webs of fabricated *T*-stubs, which were bolted to the underside of the top flange plate, as shown in Fig. 4. Rolled WT-shape sections could also be used, which would be conservative because the angle material used to fabricate the *T*-stubs for the tests are more flexible than typical WT sections. Tests were conducted with conventional eccentric connection details as well as stiffened connections. Hereafter, the conventional support angle connection detail currently used in bridge practice is referenced as the unstiffened detail and the new connection with added stiffening angles is called the stiffened detail. The stiffening angle substantially increases the stiffness of the PMDF system.

Twin-Girder Lateral Displacement Tests

While the shear panel tests provide valuable information related to the PMDFs shear stiffness and connection behavior, to ensure proper modeling of the deck system additional tests were necessary on the entire PMDF system prior to conducting full-scale buckling tests. As a result, the validated FEA models allowed for detailed parametric studies on the bracing behavior of the PMDF systems and consideration of a wider range of variables than was practical for the experimental tests. However, it was critical to accurately model the PMDF system as connected to the girders. In actual bracing applications, the shear strain that the PMDF system is subjected to varies along the girder length, and this behavior is not captured in the shear panel tests conducted in a relatively rigid frame. Lateral displacement tests were conducted to measure the stiffness of the system with shear strain distributions consistent with the girder buckling deformations. Because the shear strain varies along the girder length, the lateral load tests on the twin-girder system with PMDFs for bracing provided a good indication of the system stiffness for validating the accuracy of the FEA model. Therefore, prior to conducting buckling tests on the twin-girder system with PMDF bracing, the lateral load tests were conducted on the system.

The twin-girder system with PMDFs for bracing is shown in Fig. 5. The test setup consisted of two 15-m (50-ft) long beams. Three different beam sizes were used and will be subsequently discussed. The identical system was used for the lateral displacement and buckling tests. One of the primary sources of the loading for which the PMDFs provide bracing is the weight of the wet concrete during construction. The wet concrete weight acting through the forms leads to friction forces that develop between adjacent PMDF sheets along the sidelap joints, as well as between the metal

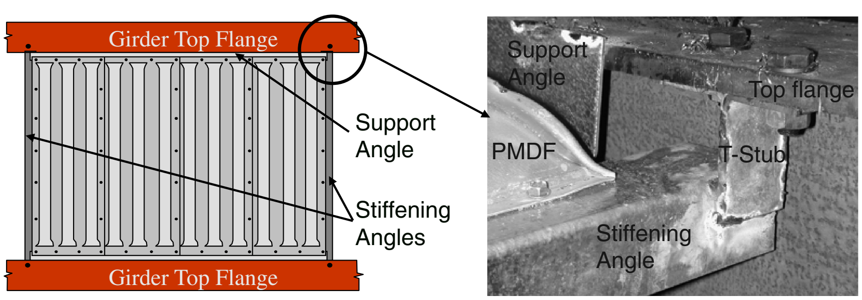

Fig. 4. Stiffening angle to control support angle deformation [Reprinted with permission from Helwig et al. (2005)]

J. Bridge Eng., 2012, 17(4): 624-633

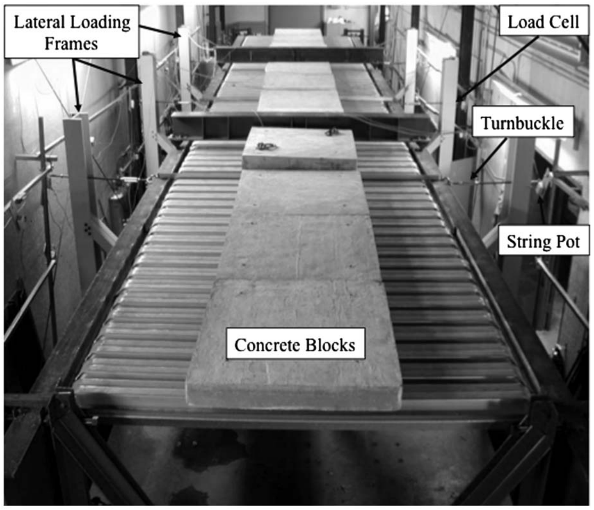

Fig. 5. Twin-girder test setup with PMDF bracing

sheeting and the support angles. Although it is not practical to apply all the loading through the metal deck form in the laboratory, simulating some of the friction is important in determining a good measure of the bracing potential of the forms. Therefore, some of the loading to the girders was applied using $1,220 \times 1,220 \times 150$-mm ($4 \times 4 \times 0.5$ ft) concrete blocks (~450 kg [1,000 lb] each), as shown in Fig. 5. The concrete blocks were utilized in both lateral displacement and buckling tests. Lateral displacement test results revealed that the lateral stiffness of the PMDF systems increased approximately 8–10% with the addition of the superimposed dead weight (Helwig et al. 2005).

The twin girders were simply supported with lateral movement prevented at the top and bottom flange at the supports. The lateral stiffness tests were conducted by applying a lateral displacement to the top flange of the girders near the quarter points and midspan of the beams. The lateral displacement was applied by adjusting a turnbuckle connected between a reaction column and the top flange, as shown in Fig. 5. A load cell was used to monitor the force in the turnbuckle; therefore, the lateral stiffness could be determined by dividing the turnbuckle force by the resulting flange displacement that was measured with a string potentiometer.

Finite-Element Analytical Model

The three-dimensional, finite-element program ANSYS (2002) was used to perform parametric studies. A shear diaphragm truss panel model similar to that developed by Helwig and Yura (2008a and 2008b) was used. Fig. 6 shows an illustration of the truss panel model. The truss panels were built up from two-node truss elements connected to the centerline of the girder top flange. The effective shear modulus of both the unstiffened and stiffened PMDF systems was determined from the laboratory lateral displacement test results. A discussion of the effective shear modulus is provided in Egilmez et al. (2007). The area of the shear diaphragm truss panels used in the finite-element model was adjusted so that the FEA model provided the same lateral stiffness as measured in the laboratory tests. These calculated areas of the truss panels were then used in an FEA model of a shear test frame (similar to that shown in Fig. 2) to calculate the effective shear stiffness values for the PMDF systems.

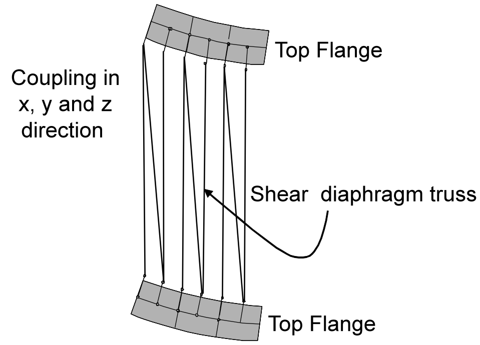

Fig. 6. Truss elements used to simulate shear diaphragms

Twin-Girder Buckling Test Program

Test Setup

The test setup outlined in the discussion of the lateral displacement tests was also used to conduct the buckling tests. The twin-girder setup before the PMDF was installed and the components of the loading system are shown in Fig. 7. The girders are labeled as south and north girders, which is the nomenclature used throughout the paper. Elastic lateral load tests and buckling tests were first conducted on the bare steel girders; therefore, a measure of the increase in the stiffness and buckling capacity could be determined when the PMDF was added. As shown in Fig. 5, concrete blocks were also added to the PMDF system to simulate the friction present between the forms and support angles.

In addition to the concrete blocks, vertical loading was applied using two gravity-load simulators positioned near the third points of the beam. The gravity-load simulators consist of a mechanism with a hydraulic actuator that allows vertical loads to be applied while minimizing lateral restraint (Yarimci et al. 1967).

The system was designed to remain elastic; therefore, several tests could be conducted on the same system. It was, therefore, important to limit the amount of lateral deformation to avoid inelasticity in the system. During the buckling tests, excessive lateral deformation was avoided by using the load frames from the lateral displacement tests as lateral stops to limit the amount of displacement that could occur. The gravity-load simulators and the lateral load frames are indicated in Fig. 7. Two 90-t (200-kip) capacity hydraulic actuators mounted in the gravity-load simulators were

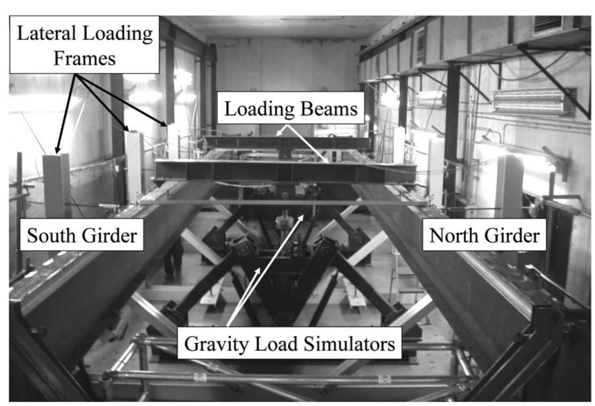

Fig. 7. Buckling test setup

J. Bridge Eng., 2012, 17(4): 624-633

used to apply vertical load to both girders through the loading beams, as shown in Fig. 7. The loading beams applied point loads to the top flange of the test girders. Knife-edges were bolted to the ends of the loading beams to eliminate tipping restraint to top flanges of the test girders. Loads applied through the gravity-load simulators were monitored by load cells attached to two hydraulic actuators.

The gravity-load simulators were positioned at the third points along the girder length; therefore, the constant moment region could be achieved across midspan of the girders during buckling tests. The distribution of the moment along the full girder length provided a reasonable simulation of the moment diagram that results from a uniformly distributed top flange load, which is the typical loading from wet concrete for bridge girders during construction.

During the buckling tests, lateral displacements were measured at the girder quarter-points and midspan by means of 10-cm (4-in.) linear spring potentiometers placed at the top and bottom of each web. In addition, the displacements at the top and bottom of the web provided a measure of the twist of the cross section. String potentiometers were used to measure the vertical displacement of the bottom flange of the girders at midspan and quarter-spans. Strain gauges were applied to the girder top flange tips at midspan to monitor the maximum flange stresses and avoid inelasticity in the girders.

Tests were conducted with three different girder sizes produced from ASTM A992, Grade 345 MPa (Grade 50) material. The girders were simply supported with a span of 15 m (50 ft). Lateral movement and twist of the girders were prevented at the supports; however, the girder flanges were free to warp. The first series of tests was conducted on two W762 × 134 (US: W30 × 90) girders with a reduced top flange (reduced from 264.2 [11 in.] to 158.8 mm [6.25 in.]) to create a singly symmetric section similar to one that might be used in composite steel girder bridges. The second and third set of girders that were used consisted of two W457 × 177 (US: W18 × 119) and two W457 × 106 (US: W18 × 71) sections, respectively. These girder sizes were selected to match girders in a bridge utilizing PMDF as bracing as an implementation of recommendations from this research investigation. Two bridges in Houston, Texas with W457 × 177 (US: W18 × 119) girders, were designed with PMDFs as construction bracing by the Texas Department of Transportation (TxDOT). Field studies were conducted on the two bridges and will be discussed in a future publication. Although three different sections were used in the tests, because of space limitations the results focus on the modified W762 × 134 (US: W30 × 90) section. Test results from the W457 × 177 (W18 × 119) girders, which were used in an implementation project in Houston, Texas, and W457 × 106 girders will be presented in a future publication, along with the results from the implementation project.

Test Parameters

Four basic parameters of the PMDF specimens were investigated in the twin-girder buckling tests: deck length, deck metal thickness, support angle size, and stiffening angle spacing. The deck panel specimens were factory closed, 76 × 203 (76-mm [3-in.] depth, 203-mm [8-in.] pitch) bridge deck forms with a 610-mm (2-ft) cover width. The values of the deck metal thickness were 0.91, 1.22, and 1.63 mm (0.036, 0.048, and 0.060 in.). The forms were installed with maximum eccentricity (70 mm [2.75 in.]) to simulate the most extreme conditions that might be found in practice. Deck spans of 2,718 mm (107 in.) (for the W762 [US: W30] girders)) and 1,270 mm (50 in.) (for the W457 [US: W18] girders) were considered. Although the 1,270-mm (50 in.) span is relatively small, this span was selected to match the forming system that was used in the implementation project.

To match support details used in practice, the metal deck spans were 50 mm (2 in.) less than the clear span between the edges of the flanges of the adjacent girders. The forms were supported on cold-formed L76 × 51 × 3.3 or L76 × 76 × 3.3 (US: L3 × 2 × 0.128 or L3 × 3 × 0.128) galvanized angles. The L76 × 51 × 3.3 (L3 × 2 × 0.128) angles are typical of those used in bridge construction; however, shear panel tests demonstrated that better behavior was achieved using L76 × 76 × 3.3 (L3 × 3 × 0.128) angles because of larger edge distances for the fasteners. Edge fasteners are the connectors between the deck forms and the support angles, while sidelap fasteners are connectors between overlapping plies of adjacent deck form sheets. The fasteners that were used consisted of #14 self-drilling TEKS screws that were 190 mm (0.75 in.) long with a 6 mm (0.25 in.) diameter. At sidelap locations, the screws were fastened at a maximum center-to-center spacing of 450 mm (18-in.) as required by the TxDOT PMDF standards (TxDOT 2004). The forms were fastened to the support angles in every trough. The 76 mm (3 in.) long vertical leg of the support angles were welded to the top flanges with 50-mm (2-in.) fillet welds at 305-mm (12-in.) intervals, which matches typical practice. The welds at the ends of the support angles were 75-mm (3-in.) long. The support angles were always welded to the top flanges with the maximum eccentricity expected in practice (~70 mm [2.75 in.]).

The stiffening angles were the same size of the support angles used in each test. Four different spacings between stiffening angles were tested in the twin-girder system. The first stiffening angle was always positioned 160 mm (6.3 in.) away from the supports. The spacing between the stiffening angles were 2.4, 2.9, 4.9, and 7.3 m (8, 10, 16, and 24 ft).

A total of 17 PMDF systems were tested. Thirteen of the tests were conducted with the W762 × 134 (W30 × 90) girders, which is the focus of this paper. The thickness of the PMDF material consisted of five tests with 0.91 mm (0.036 in.) thick sheeting, three tests with 1.22 mm (0.048 in.) thick sheeting, and five tests with 1.63 mm (0.060 in.) thick sheeting.

Buckling Test Results of W762 × 134 (US-W30 × 90) Twin-Girder System

Results from the laboratory buckling tests of the W762 × 134 (US-W30 × 90) twin-girder systems with PMDF bracing are presented. Following a description of the testing procedure, the test results are outlined along with comparisons with the FEA predictions. More than 35 buckling tests were performed. Fifteen of these buckling tests were ultimate tests, where loading was applied until failure occurred. In the remaining tests loading was kept well within the elastic range of the deck and support angles in order to study the effects of different parameters. The results presented primarily focus on the ultimate buckling tests.

An important aspect of conducting the buckling tests was to subject the girders braced with PMDFs to the worst conditions likely expected in the field. The PMDFs in the laboratory tests had a constant eccentricity of 70 mm (2.75 in.) along the entire girder length, which is conservative because in actual practice, the support angle eccentricity on a given bridge will usually vary from no eccentricity in some regions and larger eccentricities in others. The bracing behavior in practice will be substantially better, because the bracing stiffness increases significantly with smaller connection eccentricities. In addition, a twin girder is the worst bracing situation. By adding girders, the bracing per girder increases. For example,

J. Bridge Eng., 2012, 17(4): 624-633

a three-girder system has twice as much decking for bracing with only 50% more girders than a comparable twin-girder system.

A difficult feature of the buckling tests was to simulate the critical initial imperfection condition expected for the girders. The global imperfection in the girders consisted of the out-of-straightness of the flanges along the girder length. Wang and Helwig (2005) demonstrated that the worst case imperfection for the bracing of beams consists of a twist and lateral sweep of the beam, where the bottom flange is straight and the top flange is displaced laterally. The magnitude of the lateral sweep, which is usually used in the bracing provisions in design specifications, such as the AISC specifications (AISC 2001), consists of an amount equal to $L_b/500$, where L_b is the spacing between points of zero twist (cross-frame locations).

Because the magnitude of the brace forces is directly dependent on the initial imperfection, it was desired to test the girders with the worst possible imperfection that might be encountered in the field. Because the actual imperfections in the beams did not match the critical shape of the imperfection as outlined previously, the decision was made to offset the load to achieve a similar effect of a beam with critical imperfection. The finite-element models were used to determine the necessary load offset to achieve this effect. The process began by measuring the actual initial imperfections of the girders before each buckling test using a taut wire to measure the lateral sweep of each flange. Two FEA models of the girder system were then created: one with the measured imperfection and one with the critical imperfection. Large displacement analyses were conducted and the FEA results with the actual imperfections were compared with results from a model with the critical shape imperfection. The load point was then offset in the model with the actual imperfection until similar brace forces and deformations were achieved between the two models.

Three phases of tests were conducted for each deck system. In Phases 1 and 2, loading was kept within the elastic range of the system. The third phase involved ultimate loading on the system, whereas the first two phases were conducted to determine a load offset that would mimic the critical $L_b/500$ imperfection. In the Phase 1 and 2 tests, the loading was limited; therefore, the applied girder stresses were less than 185 MPa (26.5 ksi), as a result the combined effect of the residual and applied stresses were well within the elastic range. Testing in the elastic range facilitated using the laboratory setup to study the behavior of multiple deck systems with different values of the stiffening angle spacing. In Phase 1 (no load offset) tests, the knife-edges that were bolted to the loading beams were aligned with the centerlines of the webs of the girders; therefore, the imperfection consisted of only out-of-straightness. In Phase 2 tests, the knife-edges were placed with an eccentricity of 13 mm (0.5 in.) with respect to the web centerlines (load offset in the same direction on both girders). The load offset was applied in the direction that the twin-girder system tended to displace in Phase 1 (no load offset) buckling. The offset was then adjusted to match FEA behavior with the critical shape imperfection.

Fig. 8 shows the midspan moment versus the twist data for the Phase 2 test of a 0.91-mm (0.036-in.) deck system with stiffening angles spaced at 4.9 m (16 ft), along with the companion FEA results. The test results and FEA solutions demonstrate how the necessary load offset in the experiments was determined. Two FEA models were prepared. The first model possessed the actual imperfection of the girders, along with a specified load offset (first analyses were done with a 13 mm [0.5 in.] load offset). The second model possessed the critical $L_b/500$ initial imperfection from Wang and Helwig (2005) with no load offset. The FEA results for the critical shape imperfection are presented for only one girder, because the behavior of both girders was quite similar. Phase 2 tests with the 13-mm (0.5-in.) load offset on each girder showed that the south girder was stiffer than the north girder, which was expected because the initial imperfection measurements showed that the south girder was straighter than the north girder. Comparisons of the laboratory tests and FEA solutions showed that the 13-mm (0.5-in.) load offset was sufficient for the north girder to mimic the $L_b/500$ initial imperfections; however, the load offset for the south girder had to be increased to mimic the $L_b/500$ imperfection. Several FEA analyses were conducted with the first model by keeping the 13-mm (0.5-in.) load offset on the north girder and increasing the load offset in the south girder. The 23-mm (0.9-in.) load offset on the south girder reasonably mimicked the impact of $L_b/500$ initial imperfections along with the 13-mm imperfection on the North girder.

Fig. 9 shows a comparison of midspan moment versus twist data for the Phase 3 ultimate test of the same deck system along with the FEA results. The respective 23 mm (0.9 in.) and 13 mm (0.5 in.) load offsets on the south and north girders; the response of the two

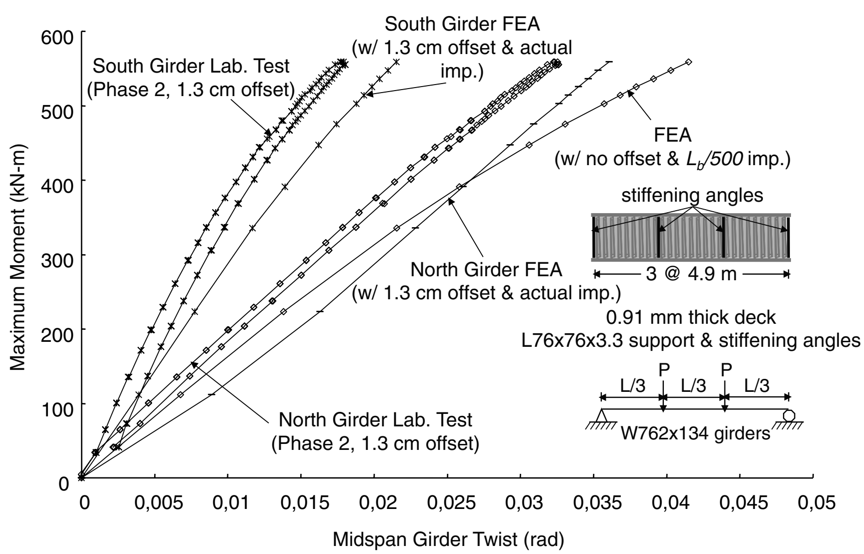

Fig. 8. Comparison of Phase 2 test and FEA results to simulate $L_b/500$ imperfection

J. Bridge Eng., 2012, 17(4): 624-633

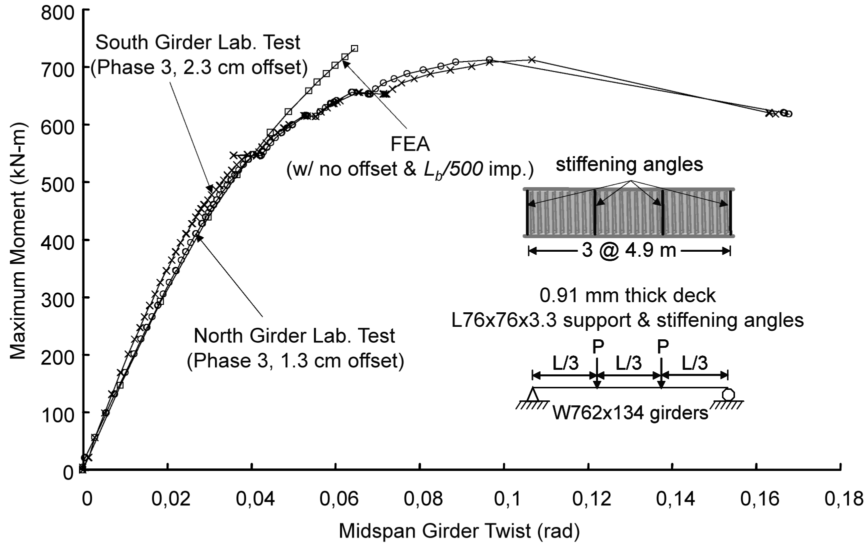

Fig. 9. Comparison of Phase 3 test and FEA results

girders was very similar. The FEA results for girders with the $L_b/500$ initial imperfection and no load offset compare well with the laboratory results up to moment levels of about 590 kN m (437 kft). Above this moment level, inelastic behavior was initiated in the deck system; consequently, the elastic FEA model used was not expected to match laboratory results beyond the elastic limit. In practice, the PMDF bracing will be designed to remain elastic and therefore, the FEA model provides good estimates of the behavior applicable to practice.

Readings during the laboratory tests were based on predetermined load increments. While the system was in the elastic range, load and displacement readings were recorded at every 13-kN (3-kip) (point load at third points) increment, which corresponds to moment increments of 65 kN m (50 kft). When the system began exhibiting inelastic behavior (from the load deformation curve) the load increments were decreased to 2.2 kN (500 lbs). It should be noted that the nonlinear behavior in the curves in Fig. 9 was because of inelasticity in the PMDF around the fastener locations (the girders were still elastic). There was a tendency for the deformations to continue creeping after the load increment was applied. At each 2.2-kN (500-lbs) load increment in the inelastic range, the load was kept constant for 10 min to observe the rate of change of the top flange lateral displacements. If the magnitude of the lateral displacements stabilized under constant load during the 10-min period, another 2.2 kN (500 lbs) was applied to the system. This procedure was continued until failure occurred. In every test, failure occurred at a deck to support angle fastener because bearing or shearing of the deck material around the fastener at approximately 3.65 m (144 in.) away from the girder ends (quarter points along the girder length). This area is the approximate location with maximum shear deformations. Therefore, at this location the deformation profile of the PMDF system is dominated by shearing deformations. Near midspan, the shearing deformations are very small and the PMDF mainly just displaces laterally with the top beam flanges.

Fig. 10 shows a graph of the top and bottom flange lateral displacements of the north girder during the ultimate test. The graph shows that there was very little displacement in the top flange until the deck system started behaving inelastically, whereas the bottom flange had significant lateral deformation as soon as the load was applied to the system. This response shows that the center of twist

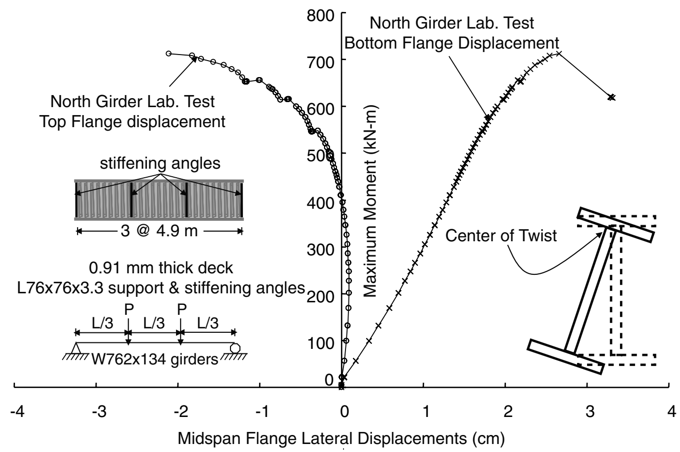

Fig. 10. Midspan moment versus top and bottom flange displacements

of the cross section is very close to the top flange and that the twisting deformation is primarily caused by lateral movement of the tension flange. Establishing a slenderness limit on the tension flange would control the magnitude of the lateral deformation. In longer bridges, the bottom flange's slenderness would be controlled by the spacing between the cross-frames. Larger unbraced lengths can be achieved with the PMDF bracing; however, cross-frames will be required to stabilize the girders until the PMDF is installed and also to control excessive lateral deformations in the bottom flange during concrete deck placement. Establishing the suitable bottom flange slenderness to control deformations is part of the criteria in the design methodology, which will be addressed in a future publication.

To assess the amount of bracing the PMDF provides, the buckling behavior of the girder system was compared with the behavior that would be expected from girders braced using conventional cross-frame systems. Historically, the AASHTO standard specifications (AASHTO 2002) limited the cross-frame spacing for steel bridges to 7.5 m (25 ft). Although this limitation was removed in the AASHTO LRFD specifications (AASHTO 2010), many bridge

J. Bridge Eng., 2012, 17(4): 624-633

designers still utilize a maximum cross-frame spacing of close to 7.5 m (25 ft) in steel girder bridges. A graph of the midspan moment versus the midspan girder twist for the north girder during the ultimate load test is shown in Fig. 11. The girder twist was normalized by the initial twist imperfection. Results are shown for PMDF deck thickness values of 0.91, 1.22, and 1.63 mm (0.036, 0.048, and 0.060 in.). The support and stiffening angles were L76 × 76 × 3.3 (L3 × 3 10 ga) angles and a stiffening angle spacing of 4.9 m (16 ft). Results are also shown for the 0.91 mm (0.036 in.) thick conventional deck system (without stiffening angles) with L76 × 51 × 3.3 (L3 × 2 10 ga) support angles. The measured results were very similar for both girders and therefore only the results for the north girder are shown for clarity. In all tests, the load was offset, as previously discussed, to mimic an initial critical imperfection of $L_b/500$. Fig. 11 also contains two lines that depict the eigenvalue buckling analysis results for the twin-girder system with no decking. The lower eigenvalue result is for a system with an unbraced length of 15 m (50 ft). The higher eigenvalue result is for a 15 m (50 ft) long system with an intermediate cross-frame, reducing the unbraced length of the girders to 7.5 m (25 ft). A buckling test of the girders was performed for the system with an unbraced length of 15 m (50 ft), and results from this test are also shown in Fig. 11. The buckling curve of the girder-only system with an unbraced length of 15 m (50 ft) approaches the eigenvalue, as shown in Fig. 11.

The traditional philosophy for the design of bracing members is to achieve a desired load level while limiting the deformation of the system. Many bracing provisions for the stiffness are based on a requirement to limit the amount of twist at the design load to a value equal to the initial imperfection (corresponds to a total twist/initial twist value of 2.0). An adequate bracing system must possess sufficient strength and stiffness to support the design loads and control deformations (Winter 1960). For Grade 345 MPa (Grade 50) steel during construction, a maximum bending stress level of 200 MPa (29 ksi) would likely be a reasonable limit for steel I-girder bridges. For the test girders, this corresponds to a moment level of 610 kN·m (450 kft). Fig. 11 shows that all of the stiffened systems (stiffening angles at 4.9 m [16 ft] and L76 × 76 × 3.3 [L3 × 3 10 ga] support and stiffening angles) provide good control of deformations up to a stress level of approximately 200 MPa (29 ksi). At this moment level, the value of the total twist/initial twist was below 2.0 for all of the stiffened girders. However, the test of the girder system with conventional (unstiffened) deck systems was stopped at approximately 500 kN·m (370 kft) of moment when the PMDF experienced large inelastic deformations around the fasteners. Fig. 11 shows that all of the stiffened system with L76 × 76 × 3.3 (L3 × 3 10 ga) support-stiffening angles carried six to seven times more moment than the girders alone with an unbraced length of 15 m (50 ft). If conventional methods of bracing were utilized for the 15 m (50 ft) long girder, the designer would have put a cross-frame at midspan, reducing the unbraced length to 7.5 m (25 ft). The stiffened PMDF systems carried more than twice the buckling moment capacity of the girder with a midspan cross-frame and no PMDF bracing at a total twist/initial twist value of 2.0.

Fig. 12 shows a comparison of the buckling behavior of deck systems with different values of the stiffening angle spacing. The midspan moment versus twist data of three 1.63 mm (16 ga) thick deck systems are compared together with one 0.91 mm (20 ga) thick deck system. All of the girders had load offsets as described previously to enable the girders to mimic an initial critical imperfection of $L_b/500$. L76 × 76 × 3.3 (L3 × 3 10 ga) stiffening angles were utilized in all tests. In all cases, stiffening angles were positioned at the supports, and the number and spacing of intermediate stiffening angles were varied. Considering the 1.63-mm (16-ga) deck, either one or two intermediate stiffener angles were considered. For the case with two intermediate stiffening angles, the end panel sizes of 3 (10 ft) and 4.9 m (16 ft) were considered. The 3 m (10 ft) was selected, because the maximum shearing deformations in the buckling tests generally occurred around 3–3.4 m (10–11 ft) away from the supports. In addition, most of the support angles are 3 m long, and this spacing allows the stiffening angle to be placed between consecutive support angles. There was no remarkable difference in the behavior of the systems with 3-m or 4.9-m panel sizes.

Using only a single stiffener angle at midspan produced a 7.3-m (24-ft) panel and resulted in a decrease in the stiffness of the system relative to the 3- and 4.9-m spacing; however, the buckling strength was approximately the same. Reducing the thickness of the deck material from 1.63 to 0.91 mm resulted in the same initial stiffness; however, the strength was significantly reduced.

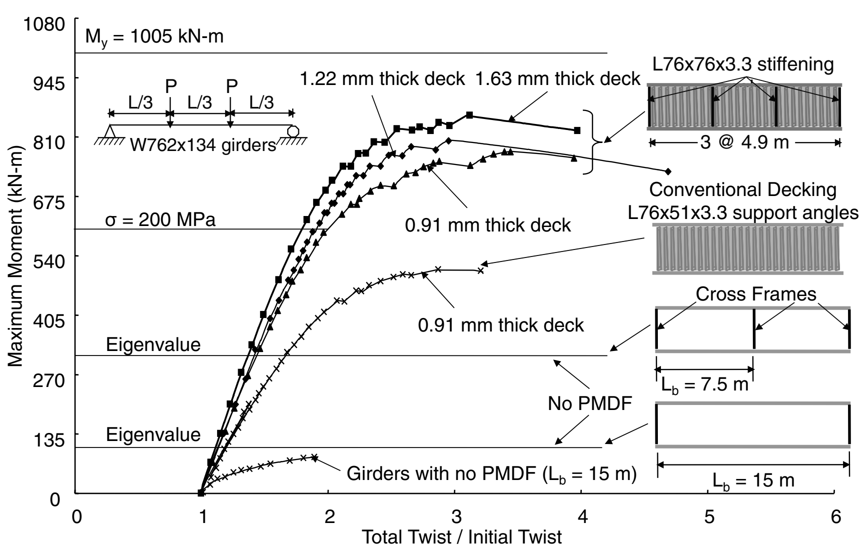

Fig. 11. Comparison of maximum moment versus total twist/initial twist behavior for deck systems with different deck thicknesses

J. Bridge Eng., 2012, 17(4): 624-633

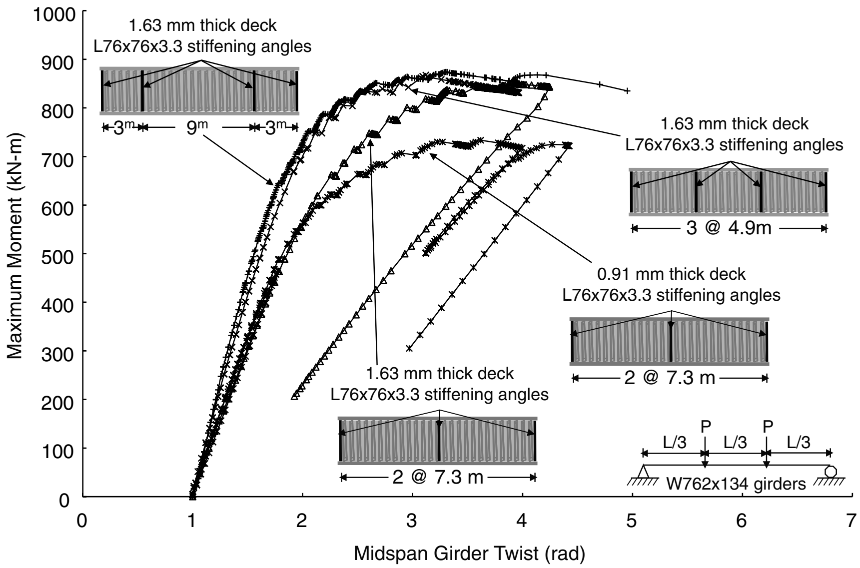

Fig. 12. Comparison of maximum moment versus total twist/initial twist behavior for deck systems with different stiffening angle spacings

Although not shown in the figure, deck systems with 2.4-m (8-ft) stiffening angle spacing were also tested; however, they were not tested to failure. In the elastic range, the deck systems with stiffeners spaced at 2.4 m (8 ft) were approximately 5% stiffer than the 4.9-m (16-ft) spaced deck systems.

As noted earlier, the twin-girder tests are actually a worst case scenario with regard to the amount of PMDF bracing that would likely be present in practice. With twin girders, a single PMDF deck system braces two girders. If the number of girders is increased by 50% to three girders, the amount of bracing doubles because there would be two PMDF deck systems bracing the three girders.

Summary and Conclusions

Current AASHTO provisions do not allow PMDF systems to be relied on for stability bracing. Laboratory tests and FEA analyses demonstrated that the PMDF systems can provide significant bracing to steel bridge girders, which would reduce the number of cross-frames required for stability during construction. This paper outlined an improved connection detail that can be used for PMDF systems in the bridge industry; therefore, reliable bracing might be achieved from the forming system. The modified detail consists of incorporating stiffening angles that span between adjacent bridge girders. A 15 m (50 ft) long, twin-girder test setup was constructed to perform buckling tests on the girder system braced by both stiffened and unstiffened PMDFs. In addition to providing valuable information on the buckling behavior of PMDF braced steel bridge I-girders, buckling tests enabled the verification of the accuracy of the FEA model that was used in the parametrical phase (presented in a future publication).

A difficult aspect of laboratory tests on stability bracing systems is measuring the brace strength, which is a function of the magnitude of the initial imperfection. In evaluating the brace strength, it is desirable to have imperfections, which are similar to the critical shape and produce the largest expected brace forces. However, the shapes and magnitudes of the imperfections in the test specimens are nearly always significantly different than the critical shapes. An iterative approach was used in these studies, where the applied load was offset by comparing measured deformations from tests in the elastic region with FEA solutions. This process resulted in the ability to measure the brace strength of the PMDF system with an equivalent imperfection that included an initial out-of-straightness of the beams combined with an offset in the applied loading.

Buckling test results revealed that the stiffened deck systems provided much better control of the deformations compared with the unstiffened conventional systems. They also significantly increased the moment carrying capacity of the girder systems. Comparing the buckling test results of 15 m (50 ft) long stiffened systems with systems braced by conventional bracing methods (cross-frames or diaphragms spaced at 7.5 m [25 ft]) revealed that the stiffened systems significantly improved the buckling behavior of the girders and provided better control of girder deformations. The modified connection detail with the stiffening angles make the PMDF systems a viable bracing alternative in steel bridges, which can significantly reduce the number of cross-frames and diaphragms required for stability bracing of steel bridge I-girders during construction.

Based on the results, it is recommended that the conventional L76 × 51 × 3.3 support angles be replaced by L76 × 76 × 3.3 angles, and that the L76 × 76 × 3.3 size also be used for the stiffening angles. Utilizing L76 × 76 × 3.3 angles not only increases the strength of the deck to support angle fastener connections, but also improves the effective shear stiffness of the deck system, as was also observed in the lateral displacement tests (Helwig et al. 2005). Use of the larger angle size will also facilitate constructability by making it easier to meet fastener edge distance requirements. In ongoing work on this project, parametrical studies are being conducted to develop design expressions for diaphragm-braced, bridge I-girders that reflect the behavior of PMDFs with stiffening angles.

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J. Bridge Eng., 2012, 17(4): 624-633

References

AASHTO. (2002). *Standard specifications for highway bridges*, 17th Ed., AASHTO, Washington, DC.

AASHTO. (2010). *Load and resistance factor design (LRFD) bridge design specifications*, 5th Ed., AASHTO, Washington, DC.

AISC. (2001). *Manual of steel construction load and resistance factor design*, 3rd Ed., LRFD, Chicago.

ANSYS. (2002). *Finite element program user's manual, version 8.0*, ANSYS, Houston, PA.

Currah, R. M. (1993). "Shear strength and shear stiffness of permanent steel bridge deck forms." M. S. thesis, Univ. of Texas at Austin, Austin, TX.

Egilmez, O., Helwig, T., Jetann, C., and Lowery, R. (2007). "Stiffness and strength of metal bridge deck forms." *J. Bridge Eng.*, 12(4), 429–437.

Egilmez, O., Herman, R., and Helwig, T. (2009). "Lateral stiffness of steel bridge I-girders braced by metal deck forms." *J. Bridge Eng.*, 14(1), 17–25.

Errera, S., and Apparao, T. (1976). "Design of I-shaped beams with diaphragm bracing." *J. Struct. Div.*, 102(4), 769–781.

Helwig, T. A., Egilmez, O. O., and Jetann, C. A. (2005). "Lateral bracing of bridge girders by permanent metal deck forms." *Texas Dept. of Transportation Research Rep. 05/4145-1*, Texas Dept. of Transportation Research Report, Houston.

Helwig, T. A., and Frank, K. H. (1999). "Stiffness requirements for diaphragm bracing of beams." *J. Struct. Eng.*, 125(11), 1249–1256.

Helwig, T. A., and Yura, J. A. (2008a). "Shear diaphragm bracing of beams. I: Stiffness requirements." *J. Struct. Eng.*, 134(3), 348–356.

Helwig, T. A., and Yura, J. A. (2008b). "Shear diaphragm bracing of beams. II: Design requirements." *J. Struct. Eng.*, 134(3), 357–363.

Nethercot, D., and Trahair, N. (1975). "Design of diaphragm-braced I-beams." *J. Struct. Div.*, 101(10), 2045–2061.

Soderberg, E. (1994). "Strength and stiffness of stay-in-place metal deck form systems." M.S. thesis, Univ. of Texas at Austin, Austin, TX.

Texas Department of Transportation (TxDOT). (2004). *Permanent metal deck form standards*, TxDOT, Austin, TX.

Wang, L., and Helwig, T. A. (2005). "Critical imperfections for beam bracing systems." *J. Struct. Eng.*, 131(6), 933–940.

Winter, G. (1960). "Lateral bracing of columns and beams." *Trans. Am. Soc. Civ. Eng.*, 125(1), 807–826.

Yarimci, E., Yura, J. A., and Lu, L. (1967). "Techniques for testing structures permitted to sway." *Experimental Mechanics J. Soc. Exp. Stress Analysis*, 7(8), 321–331.

J. Bridge Eng., 2012, 17(4): 624-633