LATERAL BRACING OF STEEL GIRDERS by PERMANENT METAL DECK FORMS

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SUMMARY

Permanent metal deck forms (PMDF) [sometimes referred to as Stay-in-Place (SIP) forms] are often used in bridge construction to support the wet concrete deck. Although the formwork has good potential for stability bracing, the current connection-method drastically reduces the stiffness and strength of the forms as a bracing element. The bridge industry typically supports the forms on angles that allow the contractor to adjust the form elevation to account for changes in the flange thickness as well as differential camber between adjacent girders. Although the support angles are beneficial for constructability issues, the eccentric connections drastically reduce the bracing effectiveness of the PMDF. The Texas Department of Transportation (TxDOT) funded a study to improve the understanding of the bracing provided by PMDF and also to develop improved PMDF connection details. The stiffness of the forms was dramatically improved using relatively simple modifications that incorporate “stiffening angles” that span between adjacent girders. The addition of the stiffening angles significantly improves the stiffness and strength of the forms. TxDOT is currently implementing the new PMDF connection details on three bridges in Houston. The utilization of the PMDF as a bracing element will enable the elimination of a significant number of intermediate cross-frames from the implementation bridges. Construction of the bridges is scheduled to begin during the Fall of 2005.

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INTRODUCTION

Conventional bracing systems for steel bridges consist of cross-frames or diaphragms spaced along the length of the bridge. These bracing systems are relatively expensive components on the bridge due to the amount of fabrication that is required. In addition, routine maintenance such as inspections and painting around cross-frame locations are often more complicated than in other parts of the bridge. Therefore, minimizing the number of intermediate brace points that are required along the length of the bridge is of interest. One possible source for stability bracing is the permanent metal deck forms (PMDF) that are frequently used to support the concrete bridge deck during construction. While the term PMDF will be used throughout this paper, many engineers also refer to the formwork as Stay-in-Place (SIP) metal forms. Although PMDF systems are frequently relied upon for lateral bracing in the building industry, the forms are generally not considered for bracing in the bridge industry. Although the shapes of the forms in the building and bridge industry are different, the primary difference that affects the bracing behavior is the method of connection between the forms and the girders. In the building industry, the forms are typically continuous over the tops of the girders and are fastened directly to the girder flanges by the shear studs or other mechanical fasteners.

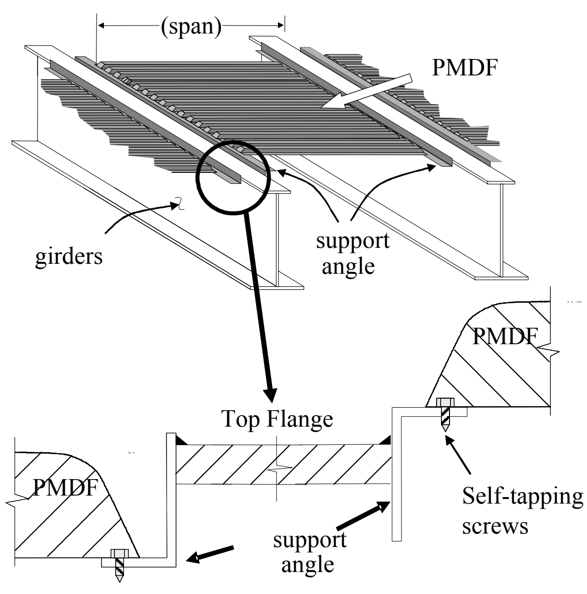

Figure 1: PMDF support angle.

In the bridge industry, the forms are often supported on a cold formed angle as shown in Figure. 1. The deck is typically fastened to the angle using self-tapping TEK screws. The angle allows the contractor to adjust the elevation of the form to account for changes in flange thickness along the length or to adjust for differential camber along the bridge length. The ability to adjust the form elevation improves the constructability of the bridge since the contractor is more likely to achieve a uniform deck thickness along the length and width of the bridge. Although the adjustable support angle connection provides convenience with respect to constructability issues, the eccentricity produced by this connection can substantially reduce the stiffness and strength of the deck form system as a bracing element. Research conducted at the University of Houston has shown that the bracing behavior can be substantially improved with relatively simple modifications to the PMDF connections [1, 2, 3].

This paper provides an overview of a research study at the University of Houston that was focused on the bracing behavior of PMDF systems commonly used in the bridge industry. The study has entered the implementation stage and the research recommendations are being incorporated into the design of two bridges in Houston. The research consisted of both experimental and computational studies. Experimental tests were performed on PMDF systems with and without modified connection details. The experimental investigations consisted of tests on the PMDF systems in a shear frame and also tests on a twin girder system with PMDF for bracing. The shear frame tests were conducted to measure the shear stiffness and strength of the PMDF systems with a variety of connection details [2]. The lateral load tests on the twin girder systems provided measurements of the lateral stiffness of the girder system with PMDF bracing with deformations that are consistent with the shapes of the buckled girders [3]. These tests were used to develop a finite element analytical (FEA) model of the PMDF systems so that parametrical analyses could be carried out. Twin girder buckling tests were then carried out to measure the buckling behavior of the PMDF-braced systems [1]. These tests provided further confirmation of the FEA model.

Following this introductory section, an overview of the background information on shear diaphragm bracing will be provided. A summary of the experimental and computation studies in this investigation will then be provided along with an overview of the implementation project. A summary of the study will then be provided.

BACKGROUND AND PREVIOUS WORK

Shear Stiffness of PMDF Systems

Adequate stability bracing must satisfy both stiffness and strength requirements. From a stiffness perspective, the material property of interest in PMDF for bracing purposes is the shear rigidity, Q, which represents the shear force along the length of the beam per unit shear strain. The following expression relates the shear rigidity to the effective shear stiffness of the PMDF system

$$Q = G' s_d \quad (kip/rad) \tag{1}$$

where; G' (kip/in-rad) = effective modulus of shear rigidity

s_d (in) = tributary width of diaphragm bracing a single girder

The effective shear modulus, G', can be determined experimentally by utilizing a testing frame as shown in Figure. 2.

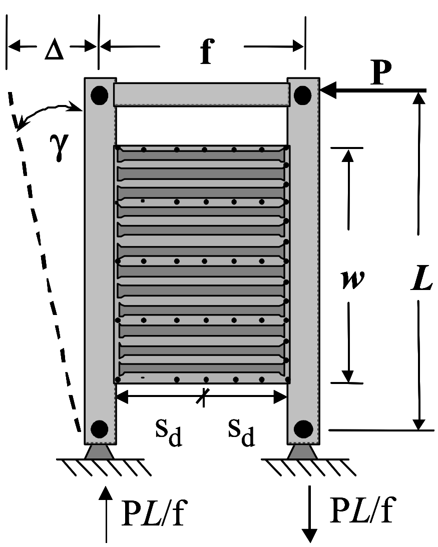

Figure 2: Shear frame for measuring PMDF stiffness and strength.

Parameters shown in Figure. 2 include: γ = shear strain; Δ = lateral deflection at the end of the testing frame; P = lateral load; L = length of the testing frame; f = spacing between test frame support beams and w = PMDF specimen panel width. In the building industry, the Steel Deck Institute (SDI) [4] provides equations and design tables that can be utilized to evaluate the stiffness of various building PMDF systems. Currah [5] found that the SDI expressions showed reasonable agreement with laboratory test results on bridge decking provided the terms for warping deformation in the corrugations were neglected.

Perhaps the most significant body of work on shear diaphragm bracing was conducted at Cornell University during the 1960's.

The work on beam bracing was summarized by Errera and Apparao [6], who presented the following expression for the buckling capacity of a beam braced by a shear diaphragm:

$$M_{cr} = M_g + 0.5Qd \tag{2}$$

where; M_{cr} = buckling capacity of the diaphragm-braced beam
M_g = buckling capacity of the girder with no shear diaphragm
Q = shear rigidity of the diaphragm
d = beam depth

In a separate study at approximately the same time, Nethercot and Trahair [7] essentially published the same expression given in Equation 2. The expression shown in Equation 2 is applicable for a beam subjected to uniform moment loading.

The effects of transverse loads and moment gradients were considered by Helwig and Frank [8] who showed that the brace stiffness requirements for shear diaphragms are a function of the type of loading as well as the load position and proposed the following solution for evaluating the buckling capacity of a diaphragm-braced beam:

$$Mcr = C_b^* M_g + mQd \tag{3}$$

where; M_{cr} = buckling capacity of the diaphragm-braced beam
C_b^* = factor for moment gradient that includes effects of load height (if applicable) [9, 10]
M_g = buckling capacity of the girder with no shear diaphragm
m = factor that depends on the type of loading
Q = deck shear rigidity
d = depth of the girder

Helwig and Frank recommended m-values of 1.0 for uniform moment, 0.625 for transverse loads applied at midheight, and 0.375 for transverse loads applied at the top flange. These values were the result of studies on slender-web plate girders. Helwig and Yura [11] showed that larger m-values can be used for girders with stockier webs such as those encountered with rolled sections. The expression shown in Eq. 3 was developed based upon eigenvalue buckling solutions and therefore represents the behavior of the diaphragm-bracing as a function of the “ideal stiffness” requirements, which are representative of the bracing requirements for perfectly straight members. To determine the behavior of girders with imperfections, a larger stiffness is required. Helwig and Yura [11] recommended that a stiffness of 4 times the ideal stiffness be provided to control deformations and brace forces.

Shear Strength of PMDF Systems

The ultimate strength of the shear diaphragm system can be determined directly from the test frame from the value of the frame reaction PL/f at failure of the PMDF system. Although there have been a number of previous studies on the bracing behavior of shear diaphragm systems, much of the early work focused on the stiffness requirements. Helwig and Yura [11] considered the strength requirements for diaphragm braced beams that are connected along two sides such as the case when the metal sheeting spans between adjacent girders. The strength requirements of the diaphragm bracing are given by the following expression:

$$M'_{br} = 0.001 \frac{M_u L}{d^2} \tag{4}$$

where; M_u = maximum factored beam moment
L = span
d = beam depth

The strength requirement, M'_{br}, represents a warping moment in the plane of the top flange per unit length. Therefore to resolve this force into a strength requirement for the metal sheeting and the fasteners, assumptions must be made about the distribution of the fastener forces in the sheeting. Work is still being conducted on the distribution of the fastener forces in the present study.

Equations 3 and 4 were developed for the strength requirements for diaphragms connected/supported along two sides. Work is still underway in the present study to develop the stiffness and strength requirements for diaphragms connected along 4 sides.

LABORATORY TESTS

Shear Diaphragm Tests and Connection Details

As mentioned in the introduction, the laboratory tests were divided into three phases: (1) shear frame tests, (2) lateral load tests on a twin girder system, and (3) twin girder buckling tests. The purpose of the shear frame tests was to determine the stiffness and strength behavior of PMDF system used in the bridge industry while also improving the connection details. The shear frame that was used in the tests was very similar to the frame depicted in Figure 2. The frame was anchored at one end and a hydraulic actuator was used to displace the frame at the other end. The force in the actuator was monitored with a load cell, while frame deformations were monitored using linear potentiometers. Figure 1 showed the typical connection details that are often utilized in the bridge industry. While the cold formed angle provides advantages to the contractor for construction related issues, the eccentricity introduced by the angle dramatically reduces the brace stiffness of the PMDF system. The effects of the eccentric angle can be seen in Figure 3, which shows a close up of the angle connection at failure. PMDF systems with the conventional connection details often failed in the connection region with the support angle rotating about the connection to the flange. The typical connection between the support angle and the flange consists of 2 in. long fillet welds space 12 in. on center. In all of the tests with eccentric connections, the maximum eccentricity was used. With the conventional L3x2 10 gage angle, this resulted in an eccentricity of approximately 2.875 inches.

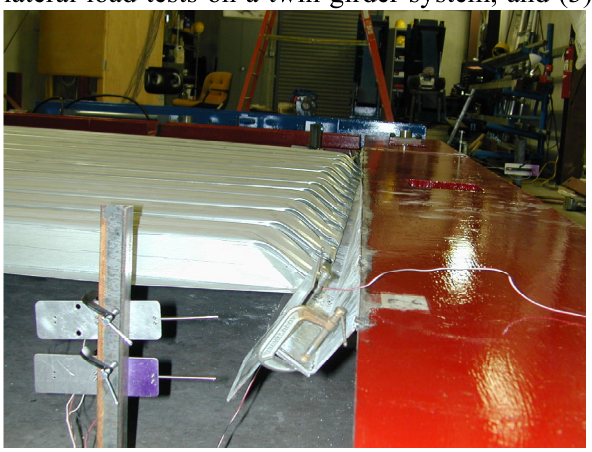

Figure 3: Rotation of support angle using conventional connection detail.

A goal of this research was to improve the connection detail while also retaining the support angle that provides the contractor the ability to adjust the form elevation. Therefore to improve the stiffness, the modified connection needed to control the support angle deformation shown in Figure 3. The resulting connection detail consisted of incorporating "stiffening angles" that were spaced along the girder lengths. The stiffening angles were positioned to coincide with the seam between two adjacent sheets so that the PMDF could be screwed to the stiffening angle.

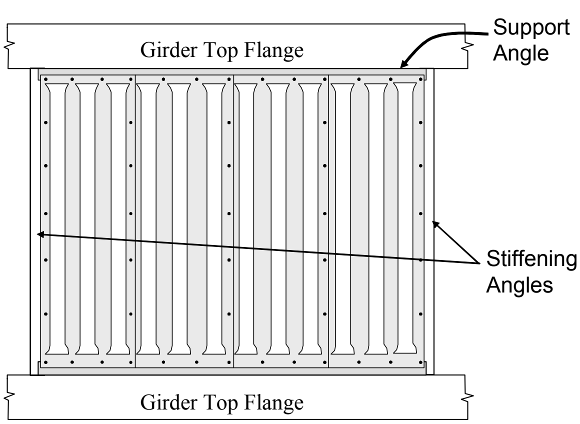

Figure 4: Plan view of stiffening angle layout.

Figure 4 shows a plan view of the stiffening angles. Stiffening angle spacing of 8 ft., 12 ft., and 16 ft. were considered in the shear tests. The stiffening angles

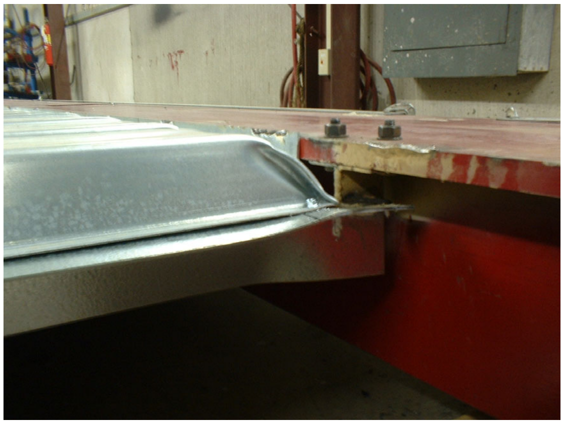

Figure 5: Stiffening angle detail at failure.

performed very well at controlling the deformation of the support angles as shown in Figure 5, which shows the deformation at failure. A "T-stub" fabricated from pieces of the support angle were bolted to the flange so that the support angle had the same eccentricity as the support angle. Figure 6 shows an alternate connection detail for the stiffening angle with a connection plate, which is being used in the implementation bridges. The failure in the PMDF system shown in Figure 5 actually consisted of buckling of the stiffening angle. PMDF gages of 18, 20, and 22 were considered in the shear frame tests. The benefits of the stiffening angle on the stiffness and strength of the PMDF system can be seen in Figure 7, which shows a graph of the shear stress versus the shear strain for 3 different connection details for the 22 gage deck. The connection details consist of Case A – No stiffening angle with maximum eccentricity; Case B – No stiffening angle with zero eccentricity; and Case C – Stiffening Angle with maximum eccentricity. The stiffness and strength of the Case C with stiffening angles is dramatically higher than the Case A with the conventional connection and maximum eccentricity. The stiffness and strength of the Case C system is actually higher than a conventional PMDF connection detail with zero eccentricity.

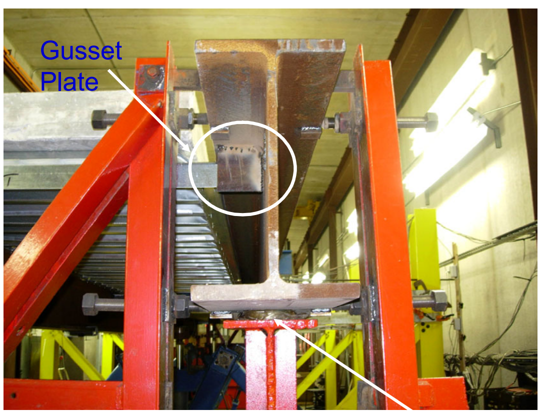

Figure 6: Alternate Stiffening Angle Detail.

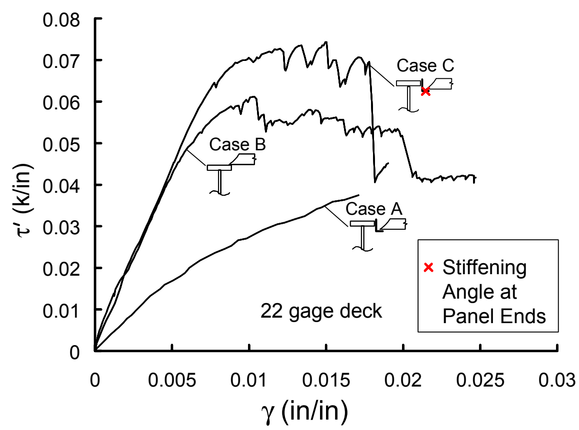

Figure 7: Effective shear Stress versus shear strain.

Twin Girder Tests

The twin girder tests were divided into two phases, the lateral load tests and the buckling tests. The lateral load tests were utilized to measure the stiffness of the PMDF braced systems with deformations consistent with the buckled shape of the girders. Figure 8 shows the twin girder system with the PMDF for bracing. The lateral displacements were applied using threaded rods connected to the top flange of the girders and anchored to loading frames. The threaded rods were positioned at the quarter points and midspan of the girder system. An adjustable turn buckle was included in the threaded rods so that lateral deformations could be applied to the girder flanges. A load cell mounted on the rods at the load frame points was used to measure the force in the threaded rods. A measure of the stiffness was obtained by dividing the force in the rod by the corresponding lateral displacement. Results from the lateral load tests were used to calibrate the finite element analytical (FEA) model. The twin girder buckling tests provided information on the strength and behavior of diaphragm braced beams as well as confirming the accuracy of the FEA model. The loads in the buckling tests were applied using gravity load simulators positioned at the third points of the beams. The gravity load simulators were connected to loading beams that applied loads to the top flanges of the beams through knife-edges. The two loading beams can be seen in Figure 8. In addition to the load applied through the gravity load

Figure 8: Twin girder system with PMDF bracing.

simulators, concrete blocks were used to simulate some self-weight from the concrete that causes friction between the PMDF and the support angle.

Tests were conducted with three different girder sizes. The first series of tests was conducted on two W30x90 girders with a reduced top flange (reduced form 11 inches to 6 inches) to make a singly-symmetric section. The girders were designed to buckle elastically so that several tests could be conducted with a variety of PMDF systems. The variables that were tested with the W30x90 section consisted of the PMDF gage (20 ga., 18 ga., and 16 ga.) as well as the size, gage, and geometry of the support angles and the stiffening angles. The girders were simply supported with a span of 48 feet. The second section that was used consisted of a W18x119 section, which was the same cross-section as the girders in the implementation stage. The PMDF system that was tested matched the details that are to be used in the implementation bridge. Work is still underway on the final section to be tested, which consists of a W18x71 section.

As mentioned above, a variety of PMDF systems were considered on the W30x90 section with the reduced top flange. Figure 9 shows a graph of the applied moment versus the total midspan twist for several of the tests. The midspan twist has been normalized by the initial imperfection, θ_o. Most bracing provisions are based upon an initial twist of L/500d, where L is the spacing between points of zero twist and d is the depth of the section. Wang and Helwig [12] demonstrated the shape and distribution of the initial imperfection that is often critical in most beam bracing problems. The imperfection in the lab tests was similar to the L/500d imperfection and was achieved by offsetting the point of load application relative to the webline for the girders. The horizontal line labeled Eigenvalue in Figure 9 represents the buckling capacity for the beams with no PMDF bracing and a cross-frame spacing of 25 ft., which historically was the maximum spacing between cross-frames that was often used. This spacing limit has been removed from recent AASHTO Specifications. The PMDF test results that are shown are for 16, 18, and 20 gage PMDF systems. The curve labeled 20ga-unst represents the results for a 20-gage deck with no stiffening angles, which matches the conventional bracing details. The other curves had stiffening angles spaced at 16 ft. The tests with the stiffening angles carried substantially more load and had much smaller deformations than the system without stiffening angles. For the systems with the stiffening angles, the stress at the points when the curves began becoming nonlinear were at approximately 30 ksi. The yield moment M_y=745 k-ft in the figures is for a yield stress of 50 ksi. The PMDF systems provided a substantial amount of bracing to the twin girders. In addition the actual bracing in most bridge systems would have been substantially larger than in the tests. Most bridges will have more than two girders, which will significantly increase the PMDF bracing. For example if three girders are used, the girders requirement bracing go up by 50% while the amount of PMDF bracing increases by 100%. In addition, the eccentricity of the support angles was the maximum value of 2.875 in. all along the girder length. In most bridges the eccentricity will vary along the length.

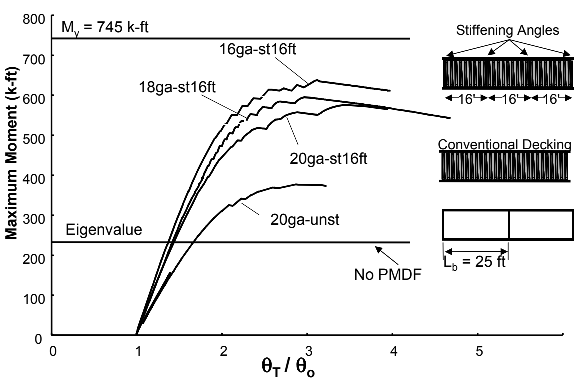

Figure 9: Test results for twin girder system with PMDF bracing.

Laboratory tests were also conducted on the W18x119 sections that will be used in the implementation bridges. The span of the girders in the implementation bridges are 50 feet, with spacing of approximately 60 inches. Stiffening angles are to be spaced at 16 feet along the girder length. No intermediate (between supports) cross-frames are being utilized on the girders. The laboratory tests were conducted on a twin girder system with the same PMDF forms that will be used in the implementation project. Figure 10 shows a graph of the applied moment versus the midspan twist in the laboratory test. The horizontal line in the graph represents the construction stress based upon the girder self-weight, the weight of the wet concrete and a construction live load of 50 lb/ft^2. The single panel of PMDF between the twin girders provided enough bracing to carry over twice the design load. As outlined above, the actual bracing in the field will be substantially more due to the additional PMDF panels with more bridge girders as well as the fact that the actual support angle eccentricity will be smaller than tested in the lab.

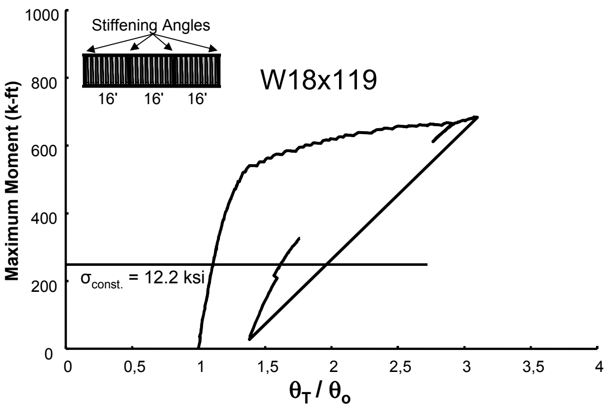

Figure 10: Test results for W18x119 girders with PMDF for bracing.

The Texas Department of Transportation (TxDOT) is currently implementing the research recommendations in the design of two bridges in Houston on the IH-610 loop. The two bridges consist of 10 simple spans of approximately 50 feet. There are 35 girders across the width of the bridge and utilizing the PMDF for bracing has resulted in the elimination of numerous cross-frames on the bridge. Had cross-frames been based upon the traditional 25 ft max spacing 340 intermediate cross-frames would have been required. TxDOT engineers estimate that two intermediate cross-frame lines would most likely have been utilized, which would have resulted in 680 intermediate cross-frames. Therefore, depending on the bracing scenario that was utilized, between 340 and 680 cross-frames have been eliminated. The estimated savings in fabrication costs range between \$160000 and \$320000 depending on whether a 25 or 16.7 ft. unbraced length would have been used. In addition to the savings in fabrication, there should be a significant savings in the construction and maintenance costs on the bridges. Since cranes are often required to lift cross-frames into place, the elimination of the cross-frames should significantly increase the speed of construction since these cranes are not tied up erecting the intermediate braces. Additional time savings during the construction should also be realized since potential delays due to problems fitting up the cross-frame should be reduced. The girders should also be easier to inspect and paint during routine maintenance since cross-frame and diaphragm regions often require close review and additional attention.

SUMMARY

Results have been presented in this paper on the stability bracing behavior of PMDF in steel bridge applications. The addition of the stiffening angles to the PMDF systems resulted in substantial improvements in the bracing behavior. The stiffening angles improved both the stiffness and the strength of the PMDF systems relative to conventional connection details. Utilizing the recommended connection modification has improved the potential of the PMDF to be considered as a bracing element in steel bridge construction. TxDOT is currently employing the recommended details in the design of two bridges in Houston. The resulting bracing system has significantly reduced the number of cross-frames that are required on the bridge. Construction on the implementation bridges is scheduled for the Fall of 2005. The bridges will be monitored during construction so as to document the performance of the PMDF system as a bracing element.

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