Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Atlanta, Georgia, April 21-24, 2020

Experimental study on steel tub girders with modified cross-section details

Stalin Armijos Moya[1], Yang Wang[2], Todd Helwig[3], Michael Engelhardt[4],
Patricia Clayton[5], Eric Williamson[6]

Abstract
Steel box girder systems, consisting of steel tub girders with a cast in-place concrete deck, are a popular alternative for straight and horizontally curved bridges due to their high torsional stiffness and aesthetics. However, steel tub girders possess relatively low torsional stiffness during transport, erection and construction because of the thin-walled open section. Thus, they require extensive bracing during construction such as top flange lateral bracing and internal K-frames. This paper highlights results from a study related to improving the structural efficiency and economy of steel tub girders, and highlights results of experiments on steel tub girders with modified cross-sectional details. The experimental program is divided in two phases. The first part of the study consisted on testing three steel tub girders with different cross-section details subjected to bending, as well as combined bending and torsion to simulate construction loads in straight and horizontally curved girders, respectively. The impact of modified cross-section details was assessed by conducting multiple elastic-buckling tests on the specimens. For the second phase, a concrete deck was poured on top of the specimens with enough shear studs to guarantee full composite action. The impact of the modified cross-section details in the ultimate flexural strength of composite tub girders was assessed by testing the girders up-to failure under positive and negative moment demands. For the study, three steel tub girders were built. The first specimen was fabricated following current design practice, while the second and third girders were built with top flanges offset towards inside the tub and with flatter webs (1H:2.5V), respectively. The final goal of this study is to improve steel tub girders efficiency by providing better detailing without undermining their structural performance.

1. Introduction
Steel trapezoidal box girders have become a popular choice for straight and curved bridges. The steel girders, also referred to as "tub girders", are fabricated with a single bottom flange, two sloping webs and two top flanges. Besides the aesthetic smooth profile of the finished section, the trapezoidal box girder possesses numerous structural advantages in comparison to other type of girders. The large torsional stiffness of the close section makes box girders a convenient

[1] Staff Engineer, Pivot Engineers, <armijos@pivotengineers.com>
[2] Associate Engineer, Stress Engineering Services <yangwang.structure@utexas.edu>
3 Professor, The University of Texas at Austin <thelwig@mail.utexas.edu>
[4] Professor, The University of Texas at Austin <mde@mail.utexas.edu>
[5] Assistant Professor, The University of Texas at Austin <clayton@utexas.edu>
[6] Professor, The University of Texas at Austin <ewilliamson@mail.utexas.edu>

alternative for bridge applications where the geometry leads to large torsional moments, such as horizontally curved systems. However, steel tub girders are open sections during construction that generally require bracing which is usually provided in the form of a top flange lateral truss, and intermediate internal and external cross-frames (Fig. 1a)

Figure 1 – a) Bracing Systems in Twin Tub Girder during Construction, b) Shallow Tub Girder System Waco-Texas

In addition to horizontally curved bridges, steel tub girders have also been shown to be applicable for straight bridges with span lengths normally reserved for concrete girder systems. For instance, relatively shallow straight steel tub girders have been used in a bridge application by the Texas Department of Transportation in the Waco District (Fig. 1b). As consequence, in order to augment the viability of the tub girders in straight bridges, improved girder geometries and bracing details are studied. The details discussed in this paper are related to the cross-sectional geometry of the steel tub girders. Common geometrical practices for the tub girders consist of a 1H:4V web slope and the top flanges centered over the webs. A flatter web slope can lead to increased lateral coverage of a single girder and may eliminate a girder line, thereby improving economy. In addition, offsetting the top flanges towards the inside of the tub girder can provide increased efficiency with respect to connections to the bracing systems.

To study the impact of these cross-sectional modifications in the behavior of steel tub girders, three specimens were fabricated for the experimental study. The first part of the experimental program consisted on loading the steel tub girders in pure bending as well as in combined bending and torsion, to study the behavior of the steel girders under simulated construction loads. Subsequently, shear connectors were welded on top of the flanges and a concrete deck was poured on top of each steel tub girders. Each girder was loaded up to failure under positive and negative moment. The results obtained in both parts of the study are summarized herein.

2. Large Scale Steel Specimens

2.1 Description of Specimens

Three steel tub girders were designed and fabricated for the experimental study. First, the baseline girder (Tub 1) was fabricated with web slope of 1H:4V and with the top flanges centered over the webs. The second specimen also has a 1H:4V web slope with the top flanges offset towards the inside of the girders (Tub 2), while the last specimen was built with a web slope of approximately 1H:2.5V and top flanges centered over the web (Tub 3). The baseline girder was designed and fabricated according to current engineering practices for straight and curved tub girders. The other two specimens were sized by conducting preliminary finite element analyses so that the girders were able to reach global elastic lateral torsional buckling before any type of local buckling.

The focus of this study is on both straight and horizontally curved girders. Though the research team considered fabricating horizontally curved girders, laboratory space limitations as well as the limitation of being able to test a single girder curvature was not desirable. Instead, the research team focused on a setup that allowed eccentric loading that can simulate the torsion from the horizontal curvature of the girder. With the ability to offset the load to achieve a torque, girder geometries from straight to a simulated curvature of approximately 600 ft. were possible.

2.2 Tub Girder Geometries
The proportions of the girders were selected so that the girders would remain elastic during multiple bending and combined bending plus torsion tests. The clear span L of the simply supported specimens was selected to be 84 ft., while the girder depth D was defined as 3 ft. (L/D=28). A distance W equal to 5 ft. and 3 in. was selected as the separation of the top of the sloped webs (Fig. 2Figure 2). The resulting width-to-depth ratio (W/D) was 1.75, which is similar to values observed in current practice. The major difference between specimens is the thickness of the cross-section plates, the location of top flanges with respect to the webs, and the web slope of the webs. All the flanges and webs were fabricated with steel AASHTO M270 (ASTM A709), grade 50W.

The baseline steel tub girder (Tub 1) was sized with webs sloped to 1H:4V (Fig. 2a). The thickness of webs and flanges was set equal to 7/16 inches, what is considerably smaller than commonly utilized in current bridge practice (≥1 in). However, this thickness was deemed necessary to obtain the elastic-buckling response of the system based upon finite element studies. This base line tub girder was built with two 12-in wide top flanges which were centered to the center line of the sloped webs. The offset top flange girder (Tub 2) was built with two 13-in wide top flanges which were connected to the sloped webs at 1 inch from the edges, leaving 12 inches of unstiffened plate (Fig. 2b). Finite element analyses were performed to determine the top flange thickness for this second specimen to assure an elastic behavior of the girder during the tests. The top flange thickness was set equal to 9/16 inches. The bottom flange and sloped webs were sized with 7/16-inch thick plates. The flatter web girder (Tub 3), on the other hand, was fabricated with web slopes equal to approximately 1H: 2.5V (Fig. 2c), which exceeds the limits of AASHTO 2017. Similar to the Tub 1, the Tub 3 was built with top flanges centered over the webs, and webs and flanges were 7/16-inch thick.

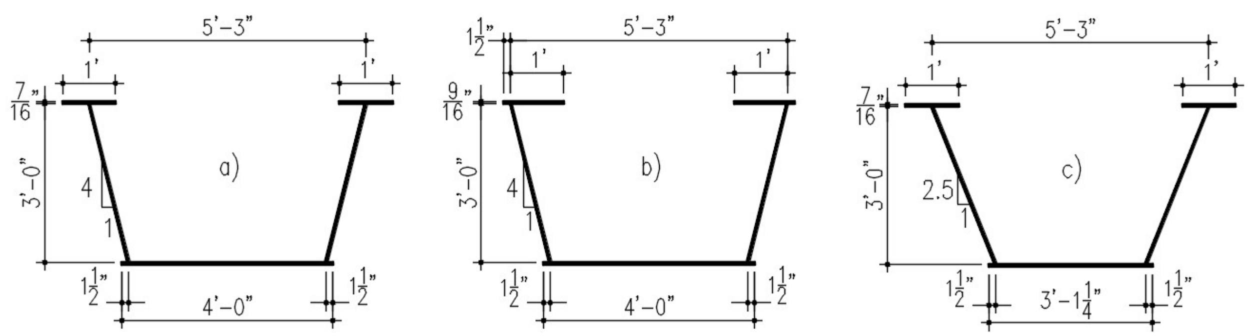

Figure 2. a) Baseline Girder (Tub 1), b) Offset Top Flange Girder (Tub 2), c) Flatter Web Girder (Tub 3)

2.3 Bracing Geometry
The spacing of the top lateral truss panel points was defined as 7 ft., generating 12 panels along the girder length (Fig. 3). Internal K-frames and top lateral bracing diagonals were connected to the girders through bolted connections so that they can be installed or removed as desired.

The single-diagonal type (SD-type) truss was used as the top lateral bracing system. This system consists of single diagonals and struts connected to the tub girder top flanges. The truss diagonals were WT5x22.5 members designed to be connected directly underneath the top flanges through three 3/4in. high strength bolts. The strut cross-section was sized as a 2-in diameter x-strong pipe (2.375 in. outside diameter and 0.218 in. wall thickness). The struts were connected to stiffeners welded to the webs of the tub girder through bolted connections made of 1/2 in. thick steel plates (ASTM A-36) and 7/8in. high strength bolts. The vertical eccentricity between the top flange and the centerline of the strut was 3.75in. which is an acceptable value (Helwig and Yura 2012). The diagonals and pipes were designed and fabricated with steel ASTM A705, Grade 50 and ASTM A53, Grade B, respectively. Three diagonals were installed at each end of the steel tub girders to simulate partial lateral bracing of the top flange.

One strut (which is part of the top lateral truss) and two diagonals formed the internal K-frames (cross-frames). The section of the strut was sized for the top lateral bracing system, and the same section has been adopted for the K-frame diagonals (2 in. x-strong) for facility during fabrication. The K-frame bracing elements were fabricated with ASTM A53 – Grade B steel. K-frames were installed every 2 panel points for this part of the study. In the panel points where internal K-frames were not provided, struts between the two top flanges were maintained at a 7 ft. spacing to control separation of the top flanges. Fig. 3 shows a plan view of the baseline tub girder, where the first two panel points denote a "strut-only" and K-frame condition.

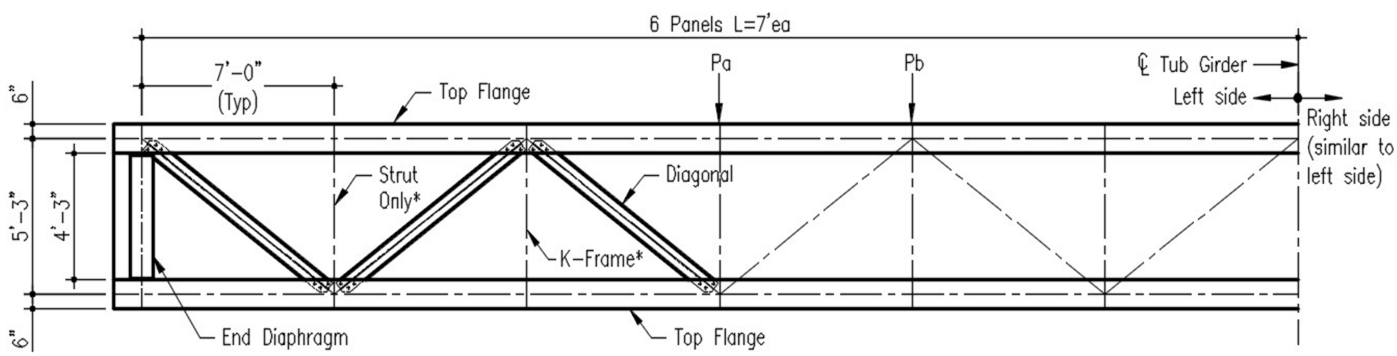

Figure 3 –Bracing Layout - Half of Baseline Steel Tub Girder Specimen - Plan View

3. Experimental Phase 1: Elastic Tests

3.1 Description of Test Setup
The test setup (Fig. 4a) consisted of two steel supports 84 ft. apart over which each specimen was tested as simply-supported straight girder under both pure positive bending and torsional loading conditions. Each steel support consists of three 12 ft. long W36x135 rolled beams stacked vertically so as to raise the elevation of the test girders above the loading system. The support located on the south side of the laboratory was supported laterally with two diagonal braces to stiffen the test setup and simulate "pinned conditions". The opposing support consisted

only of the stacked W36x135 sections and allowed some flexibility to simulate a “roller”. Two gravity load simulators (GLS), located at approximately quarter points (Fig. 4b), were used to apply either pure bending or bending with torsion. Each GLS is able to apply vertical loads up to 160 kips, and to keep the load vertical even if the ram moved laterally up to 6 inches. As result, the GLS provides minimal lateral restraint and essentially “simulates gravity load”.

Figure 4 – a) Test Setup with Unbraced Tub 1, b) Gravity Load Simulator (GLS)

3.2 Instrumentation and Initial Imperfections

Two 100-kip load cells were used to measure the loads applied with the two GLS. Horizontal and vertical deflections of the specimens were measured at the third points along the tub length (28 ft. and 56 ft.) and at mid-span (42 ft.). The deflections at the third points were obtained with four string potentiometers, while at mid-span they were collected with two infrared cameras that were able to monitor the signal from LED markers attached to the girder with relatively high accuracy (error of approximately 0.01 mm). Rotations were calculated from the measured deflections.

In order to obtain the bracing forces, stresses in the cross-section of the bracing members were calculated using collected data from conventional resistance-based foil strain gages. Six strain gauges were installed at mid-length on every top lateral truss diagonal (WT5x22.5). A linear regression method was used to calculate axial forces in the truss diagonals. Struts and diagonals of the K-frames were instrumented with strain gages at mid-length as well. A pair of gages were installed on opposite sides of the pipe to allow strains due to bending of the pipe to be separated from strains due to axial forces. Axial forces in these pipes were calculated by averaging the strains obtained with the opposite gauges.

Prior to testing, initial imperfections of each tub girder were measured. Two piano wires were extended between the test setup supports located 6 in. from both edges of the bottom flange. The taut wires served as reference point to measure lateral and vertical out-of-straightness of the tub girders. The baseline, top flange offset, and flatter web girders had an initial twist at midspan of 1.30, 1.60, and 2.30 degrees, respectively; and a maximum out-of-straightness on top flange of about L/1300 towards the east, L/750 towards the west, and L/500 towards the east, respectively.

3.3 Testing Procedure

Since the critical stages for both stability and lateral/torsional flexibility of steel tub girders generally occur during the construction phase, the stresses imposed over these sections are normally within the elastic range. Elastic-buckling tests were carried out by limiting the maximum loads applied to the specimen to keep stresses below 60% of nominal yield stress (30 ksi) to consider the impact of residual stresses and initial imperfections in the response, and to ensure that the girders remained elastic.

Two types of loading conditions were studied in this experimental phase: vertical positive bending, and combined bending with torsion to simulate construction demands on straight and horizontally curve girders, respectively. Two vertical loads were applied with gravity load simulators at approximately quarter points of the specimen (location denoted as "P_a" on Fig. 3). Henceforth, the load on each GLS will be referred to as load "P". The combined vertical bending and torsional demands were obtained by applying vertical eccentric loads at 8 in. and 16 in. from the shear center of the girders to simulate demands produced by curvature in tub girders with radii of curvature equal to 1260 and 630 ft., respectively. The eccentric loads were applied so that torsional demands towards the West of the girders were imposed.

3.4 Bracing Configuration

To evaluate the impact of cross-section modifications in the response of steel tub girders, the three specimens were tested with the same bracing configuration. Three top bracing diagonals were kept at each end, while internal K-frames were placed at every 2 panel points. The girders were subjected to concentric and eccentric (8" and 16" from shear center) vertical loads.

3.5 Experimental Results

The focus of the current study is to evaluate the behavior of steel tub girders with modified cross-sectional details. To assess the impact of the modified cross-sectional details, the torsional response and bracing forces of the girders are compared at the same loading conditions.

3.5.1 Impact of Cross-Sectional Details in Stiffness

To assess the impact of the proposed cross-sectional details in the stiffness of the tub girders, the cross-sectional twist (β) of the three specimens under the aforementioned demands was compared. In addition to the initial bracing configuration, the steel tub girders were tested without top lateral bracing to compare the torsional stiffness of the unbraced girders.

The GLSs were used to apply vertical concentric loads near quarter points to simulate the demands on straight tub girders. Fig. 5 shows the total vertical load applied (2P) versus the twist angle of the three specimens at midspan (β), when the specimens were tested with zero and three bracing diagonals at each end. The solid lines represent the response of the specimens when tested with three truss diagonals on each end, while the dashed lines correspond to the girders without top lateral bracing. The tub girders without top lateral truss presented a deformation curve that suggested the specimens were approaching the elastic lateral torsional buckling limit during the tests, which can be observed by the significant nonlinear response of the load versus deflection curves. However, the capacity to resist lateral torsional buckling (LTB) is significantly improved with the addition of truss diagonals at the ends of the girders. When comparing the specimens without top lateral bracing, Tub 2 presents the stiffer response, while Tub 3 is the most flexible girder. Without top lateral bracing, the torsional stiffness of the open sections depend on the width and thickness of the plates forming the steel tub girder cross-section. Tub 2 had thicker top flanges in comparison to the other two specimens, while Tub 3 was fabricated with a narrower bottom flange. Thus, the difference in torsional stiffness observed during the tests are consistent with what was expected. After adding partial top lateral bracing, the torsional stiffness of the three specimens improved significantly to the point that three specimens had similar response, as shown in Fig. 5 (overlapping of curves). At a total load of 30 kips, the twist angle in Tub 3 decreases from about 3.6 degrees to less than 0.01 degrees, while for Tub 1 and Tub 2 the twist goes from approximately 0.3 degrees to less than 0.01 degrees. Consequently, when truss diagonals were included on each end of the specimens to form a quasi-close section, the torsional response of the girders was very alike suggesting that the cross-sectional modifications on Tub 2 and Tub 3 had little impact on the response of the braced specimens.

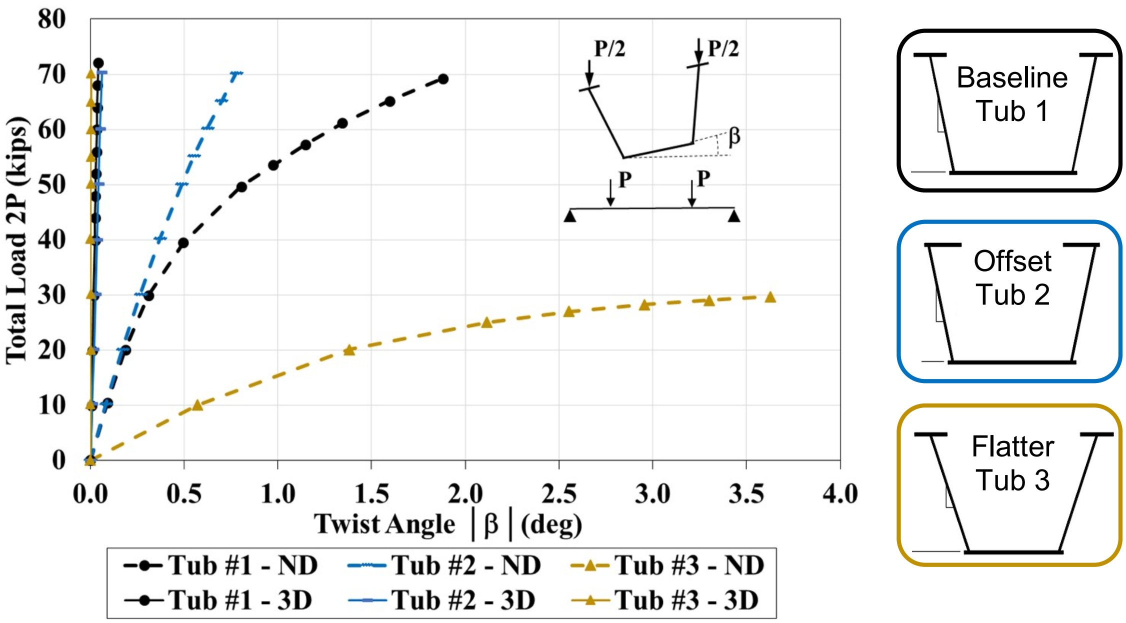

Figure 5 – Total Load vs Twist Angle at Midspan of Three Specimens with 0 and 3 Truss Diagonals (Concentric)

Besides applying concentric vertical loads, the gravity load simulators were used to apply eccentric vertical loads near the quarter points of the girders. Eccentric loads at 8 in. and 16 in. from the shear center of the section were applied to simulate the demands on horizontally curved bridges with radii of curvature of 1200 and 600 ft., respectively. Fig. 6 presents the total vertical load applied (2P) versus the twist angle of the three specimens at midspan (β) for the three steel tub girders with three and zero truss diagonals at each end, when vertical loads were applied at 8 inches from the shear center of the section. Similar to the concentric loading cases, the unbraced specimens showed very flexible response due to the low torsional stiffness of the open sections. When comparing the torsional response of the girders to the behavior observed in Fig. 5, the torsional stiffness of the unbraced girders subjected to eccentric loading is lower than the one observed in the concentric loading cases. This behavior suggests that when the torsional demands increase, such as increasing the radius of curvature of a horizontally curved girder, the torsional stiffness of the unbraced steel tub girders is reduced. When no top lateral bracing is added to the girders, the torsional stiffness still depends on the width and the thickness of the steel plates forming the trapezoidal cross-section. Thus, Tub 2 keeps showing the stiffer response due to thicker top flanges, while Tub 3 still presents the more flexible behavior. Additionally, the difference in torsional stiffness between the unbraced specimens in Fig. 6 is reduced when comparing to the plots of the unbraced girders from the concentric loading cases (dashed lines in Fig. 5). In fact, this difference in torsional stiffness keeps reducing when the torsional demands increase, such as increasing radius of curvature or increasing eccentricity of vertical loads (i.e. eccentricity=16 inches). In a similar fashion to the concentric loading cases, adding truss diagonals to brace the top flanges at the ends of the tub girders improved the torsional stiffness of the specimens significantly. As shown in Fig. 6, the torsional response of the three specimens with three truss diagonals on each end is very similar. Thus, the proposed cross-sectional details have minor impact on the torsional response of the girders when top lateral bracing is installed.

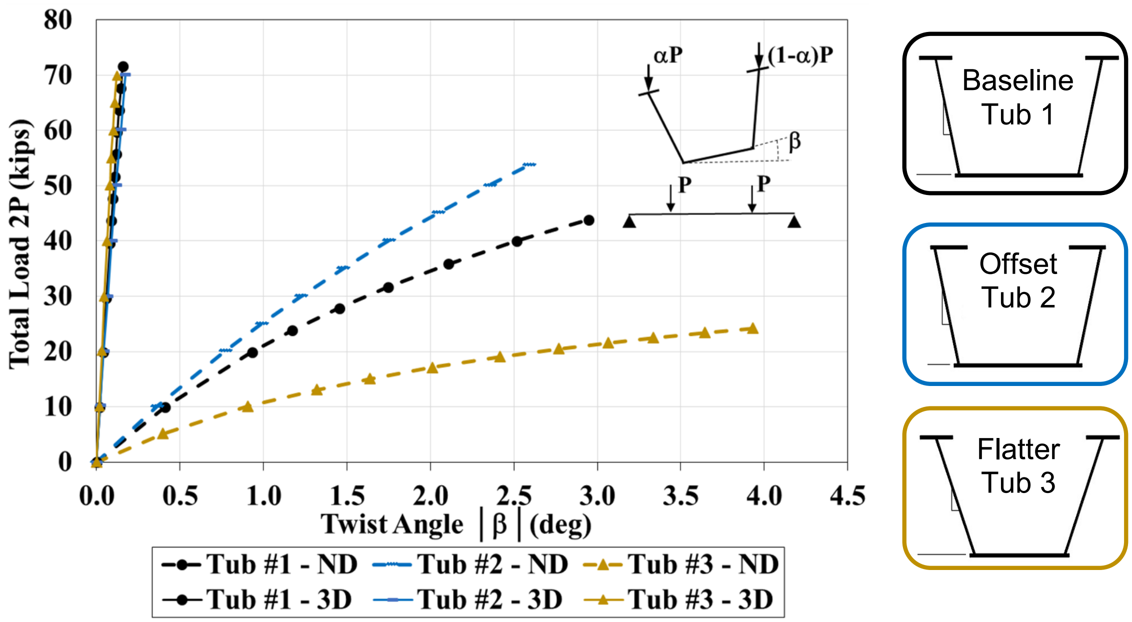

Figure 6 – Total Load vs Twist Angle at Midspan of Specimens with 0 and 3 Truss Diagonals (Eccentric e=8”)

3.5.2 Impact of Cross-Sectional Details in Top Lateral Bracing Forces

The results obtained when testing the specimens with three truss diagonals on each end subjected to vertical eccentric loads at 8 inches from the shear center are analyzed to evaluate the impact of cross-sectional details on the axial forces of the truss diagonals. To compare bracing forces of the specimens, a total load (2P) that represents construction loads was defined. Assuming 0.8 kip/ft. as a uniform construction load that represents the weight of a concrete deck, stay-in-place forms, and construction loads, a maximum moment of 706 k-ft. would be expected during construction. In order to produce the same maximum moment in the experimental specimens, a load of 35 kips on each gravity load simulator (P) is required. Thus, a total load (2P) of 70 kips is the load at which the bracing forces were compared.

Fig.7 presents the axial forces on the truss diagonals for the three steel tub girders when a vertical load of 35 kips was applied on each GLS (i.e. a total load of 70 kips) with an eccentricity equal to 8 inches. Each tub girder contains 12 panels, which are defined as the area between adjacent struts. Truss diagonals in panels 1, 2, and 3 were located in the North side of the girder, while diagonals in panels 10, 11, and 12, correspond to the South end of the girder. The higher axial forces were observed in Tub 2, relative to the other two specimens. Tub 2 was fabricated with thicker top flanges; thus, higher bending stresses were expected in these flanges. Due to compatibility of deformations between top flanges and top lateral diagonals, higher stresses in the top flanges would result in higher stresses in the truss diagonals, which consequently will produce higher axial forces in the diagonals. Additionally, increasing the slenderness of the top flanges would result in higher lateral bending stresses, which increases the axial forces in the truss diagonals as well. When comparing Tub 1 and Tub 3, the truss diagonal forces in Tub 3 are generally higher than the ones observed in Tub 1. The only difference between these two girders is the shallower webs due to the narrower bottom flange of Tub 3. When the angle of inclination of the webs is increased, higher lateral loads are developed in the top flanges since the lateral component of the shear force increases. Hence, even though the torsional response of the girders is almost not affected by the cross-sectional details under study as observed in Fig. 5 and Fig. 6, the axial forces in the top lateral bracing diagonals can show significant variation depending on the cross-sectional detail.

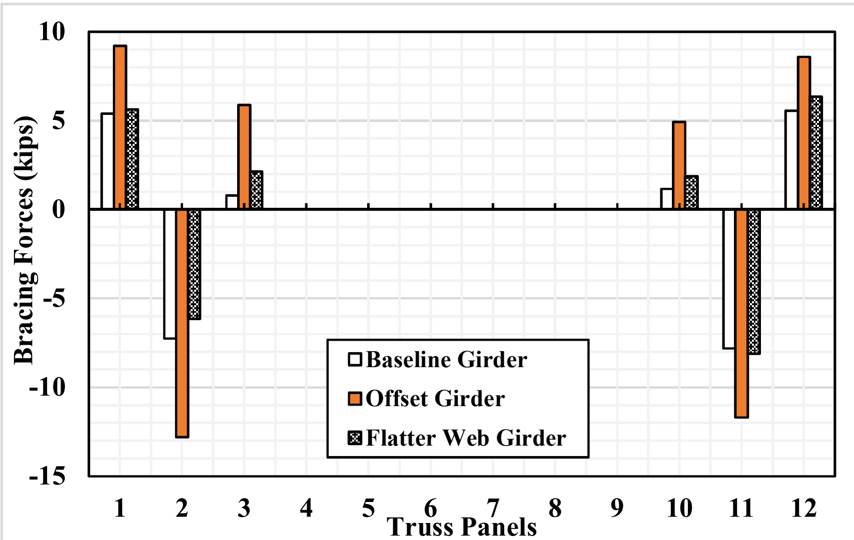

Figure 7 – Axial Forces on Top Lateral Diagonals for Three Specimens

3.5.3 Impact of Flange Offset in Plate Stiffness

Regarding the offset top flange girder (Tub 2), plastic buckling of one of the 13in-wide top flanges near midspan was observed (Fig. 8) after elastic LTB of the girder was observed during the tests. The girder with no top lateral bracing was loaded concentrically with the two GLSs. At the moment of the local buckling, the compressive stresses in the area of buckling were barely above 35 ksi (70% of yielding strength).

Clearly, the compressive stresses in the top flange due to its lateral bending were higher than the ones observed in the baseline tub girder (Tub 1). This increment of stresses might have been produced due to the high level of slenderness of the top flange and the absence of top lateral bracing to control local buckling. The change in the geometry of this girder is making the system more susceptible to local effects.

Figure 8 - Local Buckling of West Top Flange at Midspan - Offset Top Flange Girder

4. Experimental Phase 2: Ultimate Capacity Tests

4.1 Description of Composite Specimens

After concluding the elastic tests performed in the experimental phase 1, enough shear studs were welded along the top flanges in order to fabricate the composite specimens. Subsequently a concrete deck was poured on top of each girder. All three composite specimens were designed to behave as fully composite. The design compressive strength for the concrete and the nominal yield strength of the reinforcing bars was 4ksi and 60 ksi, respectively. The design geometry of the concrete deck was 9 feet 3 inches wide and 6.5 inches thick along the entire length of the specimens. The concrete deck was fabricated with two 2-foot-long overhangs, one on each side of the section. Fig. 9 shows the cross-section of the composite baseline girder (Tub 1), which is similar to the composite sections of the other two specimens.

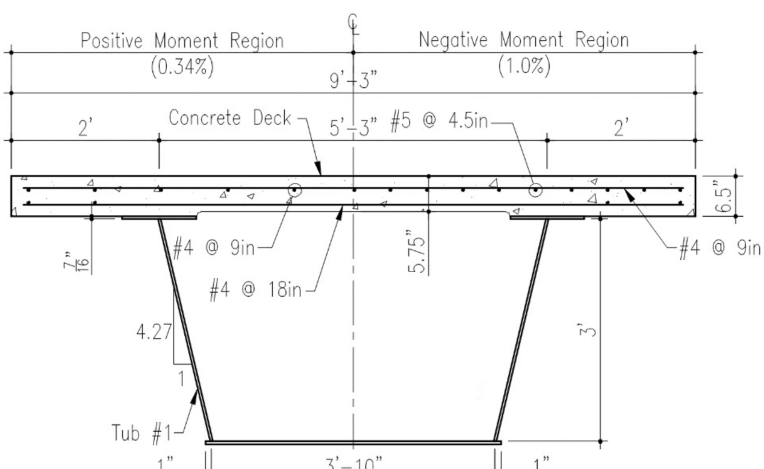

Figure 9 – Composite Baseline Girder (Tub 1) – Cross-Section

At first, the three steel tub girders described in Section 2 were designed so that they can behave elastically in a simple supported configuration during the experimental phase 1. Hence, the girders were not sized for negative bending. As result, the bottom flange of the specimens was reinforced to avoid local buckling in the region subjected to negative bending (over intermediate support) during the ultimate strength tests. Also, a solid steel diaphragm was added in the tub girders at the location of the intermediate support to withstand the reaction forces at that location.

4.2 Description of Test Setup

The setup for the experimental phase 2 comprised three supports and two loading frames (a third loading frames added to test Tub 2 and Tub 3) that were placed on and connected to the strong floor in the laboratory. The north and south supports permitted some rotational and lateral flexibility because of the web flexibility of the W36x135 sections that served as supports. The intermediate support was installed to function as a pin support by restraining translation and allowing rotation. Each composite specimen was resting over two tilt saddles on the north and south supports, while a larger single tilt saddle was placed over the intermediate support. Two 200-kip load cells were placed under the two tilt saddles on each end support, while a 1000-kip load cell supported a larger tilt saddle on the intermediate support. A 1000-kip hydraulic ram was mounted on each loading frame.

This test setup was fabricated and constructed to test the composite specimens in two different configurations. The first configuration allowed to test each girder as a two-span continuous girder, and subsequently, the specimens were able to be tested as a single span girder with simple supports in the second configuration. An elevation of the test setup for the continuous girder configuration is shown in Fig. 10. For the simply-supported girder configuration, the intermediate support was able to be removed after lifting the specimen with hydraulic jacks at the intermediate support location. Fig. 11 shows and elevation of the simply-supported girder setup configuration. A photo of the continuous girder test setup for Tub 3 is presented in Fig. 12.

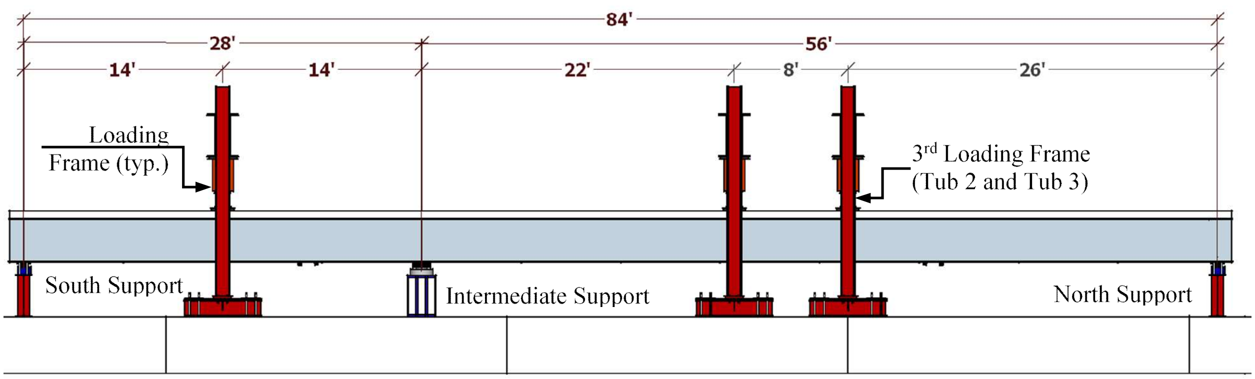

Figure 10 – Continuous Girder Configuration Test Setup - Elevation

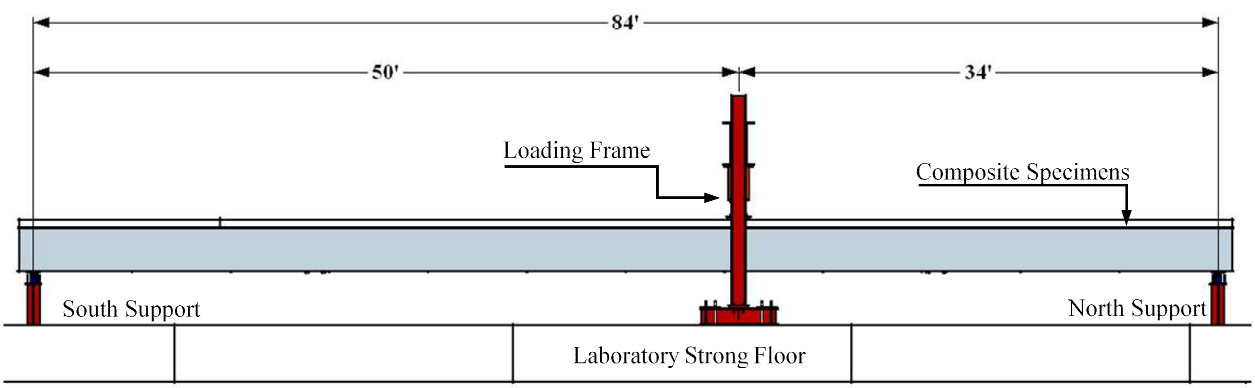

Figure 11 – Simply-Supported Girder Configuration Test Setup - Elevation

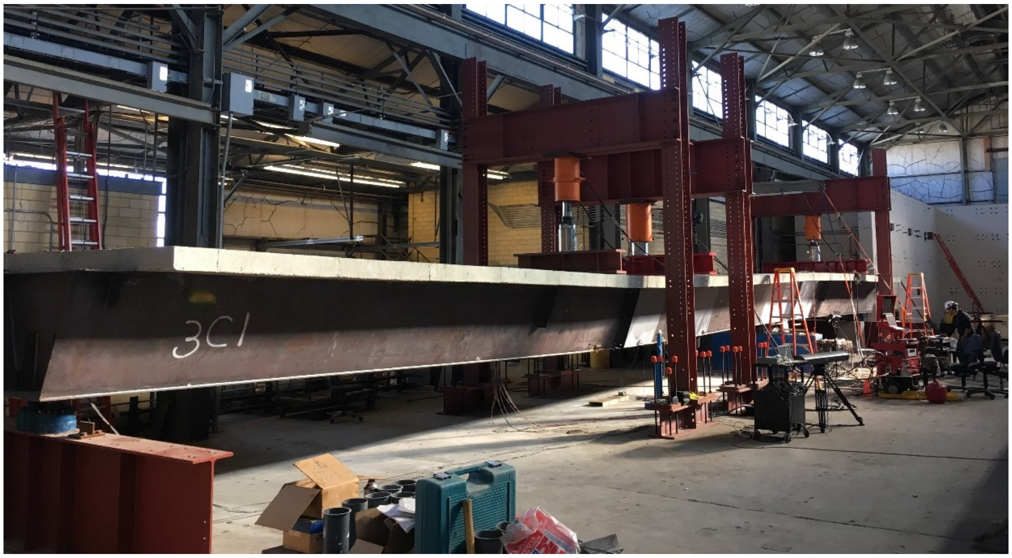

Figure 12 – Continuous Girder Configuration Test Setup – Tub 3

4.3 Instrumentation

The response of the specimens during the tests was monitored and recorded by instruments that measured reaction forces, longitudinal strains, vertical deflections and support movements. Two load cells with 1000 kips of capacity were calibrated to monitor loads applied at loading points. Additionally, four 200-kip and a 1000-kip load cells were used to monitor reaction forces at the supports. Vertical deflections of the girders were measured at the loading points using string potentiometers connected to opposite sides of bottom flange, while displacements in the three dimensions were captured with the infrared cameras and LED markers described in Section 3.2. Longitudinal strains were measured with linear foil gauges installed on the steel plates and concrete deck. An Agilent data acquisition system and a LabVIEW software were used to collect and record the experimental data from the instruments.

4.4 Testing Procedure

The purpose of the experimental phase 2 was to determine if the new cross-sectional details produced unexpected response of the composite girders at ultimate strength under negative and positive moment demands. To accomplish this, each specimen was tested as a continuous and as a simply supported system. The continuous girder configurations were monotonically loaded until significant yielding was observed in the cross-section over the intermediate support. The simply supported configurations were subjected to monotonic loading up to failure to determine the ultimate flexural strength of the composite box girders under positive moment. The application of the loads was manual by using a pneumatically driven hydraulic pump, while data was constantly recorded all along the tests at one-second intervals. Visual evaluation was carried out at increments of 25 kips in the total load applied in the elastic range, and every ½-inch increment in deflection beyond that.

4.5 Test Results

To assess the impact of the proposed cross-sectional details on the ultimate capacity of composite tub girders, the response of the girders in the ultimate strength tests is compared. First, the flexural response of the three specimens under negative moments is compared. Subsequently, the flexural behavior of the girders under positive moment demands is contrasted.

Based on the experimental data collected, the negative moment versus curvature response of the cross-section over the intermediate support was calculated for all the composite specimens (Fig. 13). Curvatures were calculated with the readings obtained from the foil strain gauges installed on the steel plates forming the tub girders located at the intermediate support. Moments were computed with the data collected from the load cells located at the loading points and supports. The negative curvatures are plotted on the X-axis, while the absolute value of the moments have been plotted in the Y-axis. The experimental plastic moment is the maximum moment measured during the tests. Additionally, cross-section analysis of each composite girder was carried out to estimate their plastic moment capacity, and absolute values are reported in Table 1.

During the experimental testing of Tub 1 as continuous girder, an unexpected buckling of the internal diaphragm at the intermediate support occurred due to a bad detail during the fabrication. As result, Tub 1 was not able to attain its plastic moment capacity and its final load was 70% of its estimated capacity. The other two specimens shown good agreement between the computed and experimental values, with a maximum difference of approximately 3%. Consequently, the proposed cross-sectional details did not produce any unexpected effect in the flexural ultimate strength of the composite tub girders under negative moment demands. The cross-sectional properties and ultimate capacity can be calculated using traditional section analysis when bending demands govern.

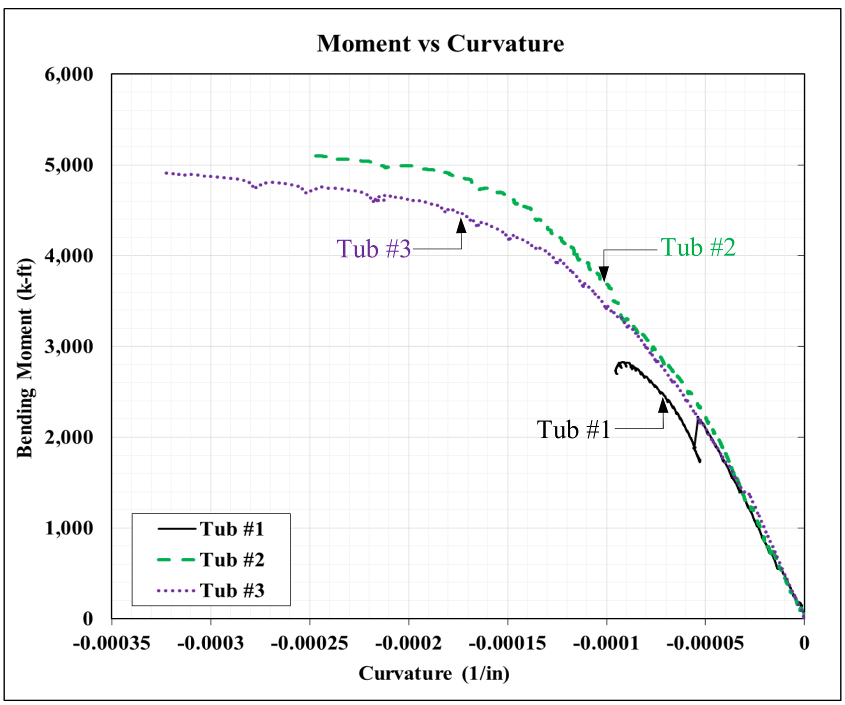

Figure 13 – Negative Moment vs Curvature – Composite Specimens

Table 1 – Computed vs Experimental Plastic Moment Capacity – Negative Moment

	Plastic Moment "Mp" (k-ft)		
	Calculated	Experimental	Diff (%)
Tub #1	4,080	2,826	-30.7
Tub #2	5,251	5,102	2.9
Tub #3	4,776	4,915	-2.8

Analogous to the negative moment region, the moment-curvature response of the cross-section of the three composite tub girders was computed at the location of maximum positive moment (load application point) for the simply-supported configuration tests. Fig. 14 shows the plots corresponding to the moment-curvatures obtained from the experimental data. Section analysis was performed to estimate the plastic moment capacity of the composite girders in positive moment, and the results are listed in Table 2. The computed and the experimental values show good agreement with a maximum difference of 2%. The moment-curvature curves for Tubs 1 and Tub 2 are very similar, both in the elastic and inelastic ranges of behavior, with the only difference that Tub 1 was not loaded until failure of the section. Hence, the thickness and offset of the top flanges of Tub 2 had minor effects in its ultimate strength. In addition, Tubs 1 and Tub 2 exhibited higher moment capacity and a more ductile response than Tub 3 due to the wider bottom flange in Tubs 1 and 2. Thus, the proposed cross-sectional details do not impact the ultimate positive moment capacity of composite tub girders.

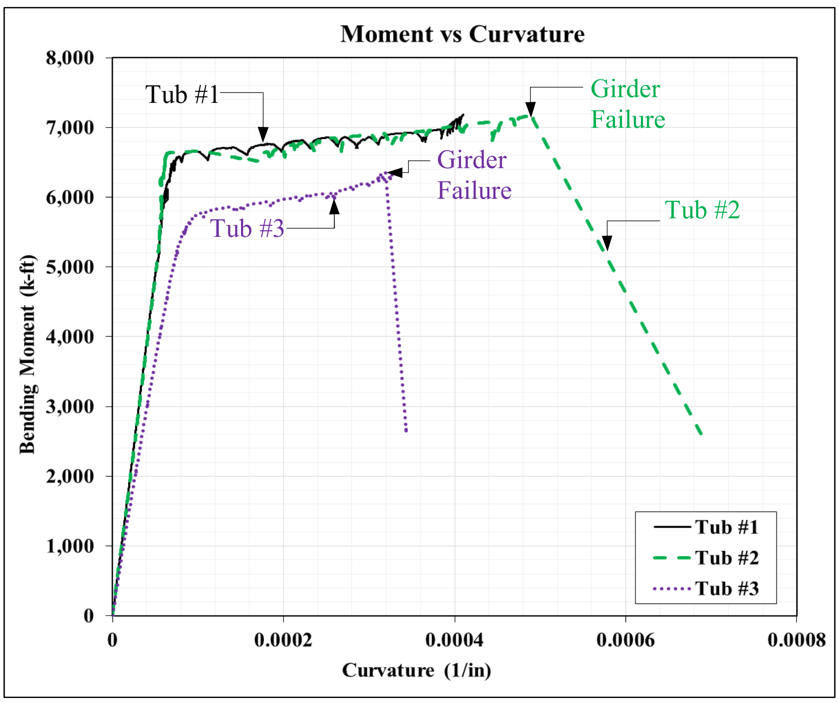

Figure 14 – Positive Moment vs Curvature – Composite Specimens

Table 2 – Computed vs Experimental Plastic Moment Capacity – Positive Moment

	Plastic Moment "Mp" (k-ft)		
	Calculated	Experimental	Diff (%)
Tub #1	7,147	7,184	0.5
Tub #2	7,055	7,175	-1.7
Tub #3	6,399	6,349	0.8

Tub 2 and Tub 3 were loaded up to failure of the cross-section under positive moment demands, and presented similar failure mode. Before failure, these two girders showed high ductility due to extensive plastic deformations. Subsequently, the concrete in the deck crushed, the top of the webs and top flanges buckled at the location of higher moment, and the girders suddenly unloaded, indicating ultimate failure of the girder. Fig. 15 shows the cross-section of Tub 2 that failed under positive moment. Fig. 16 shows a picture taken from inside of Tub 2 at the location of the failure where the offset top flanges buckled after failure of the concrete deck.

Figure 15 – Failed Section under Positive Moment Demands – Tub 2

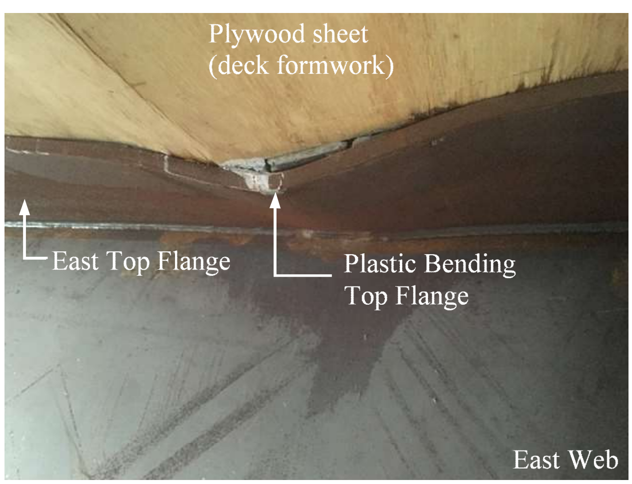

Figure 16 – Offset Top Flanges Plastic Bending – Tub 2

5. Conclusions

This paper summarizes the results of a 2-phase experimental study to evaluate the impact of modified cross-sectional details in the behavior of straight and horizontally curved tub girders. First, three steel tub girders were subjected to elastic-buckling tests under positive bending and torsional demands with the purpose of evaluating their behavior under simulated construction loads. Subsequently, a concrete deck was poured on top of each steel girder to create composite girders. Then, the new specimens were tested as continuous and simply-supported systems to assess the impact of the proposed section details in the ultimate capacity of the composite girders. The major findings are as follows:

- Experimental tests showed that top flange lateral bracing systems are more effective in the region near to the supports of straight girders where shear deformations are at the maximum.
- Changes in the size of the steel plates forming steel tub girders can affect the torsional stiffness of the open section. However, adequate levels of torsional stiffness can be achieved when adding appropriate amount of top lateral bracing along the tub girder depending on the demand imposed.
- No impact on the negative moment capacity of composite tub girders with offset top flanges and flatter webs was observed under negative moment.
- The flexural response of composite tub girders under positive moment demands was not affected by offsetting top flanges. Tub 1 and Tub 2 had very similar flexural response in the elastic and inelastic ranges of behavior.
- The positive moment capacity of Tub 3 was lower than that observed in the other two specimens due to a narrower bottom flange. The flatter webs had no significant impact in the ultimate positive moment capacity of the specimen.
- The composite steel tub girders exhibited ductile response.
- Similar failure modes were observed in the specimens with improved details as compared with the baseline specimen. Under positive bending, after significant yielding of the bottom flange and webs, the failure mode of the composite tub girders was produced by crushing of the concrete followed by buckling of the top flanges and the top of the webs
- For both negative and positive moment, conventional cross-section analysis accurately predicted measured values of strength.

6. Acknowledgement

The authors would like to extend gratitude to the Texas Department of Transportation for providing funding for this research study. Additionally, the authors are also appreciate of Hirschfeld Industries LP for assistance in fabricating the girders at a substantially reduced cost and for meeting the accelerated fabrication schedule necessary for the study to stay on schedule.

7. References

American Association of State Highway Transportation Officials (AASHTO) (2017). “AASHTO LRFD Bridge Design Specifications, 6th Ed.” American Association of State Highway and Transportation Officials, Washington, D.C.

Helwig, T. and J. Yura (2012). "Steel Bridge Design Handbook: Bracing System Design", U.S. Department of Transportation Federal Highway Administration. **13**.