EFFECTS OF STAY-IN-PLACE METAL DECK FORMS ON THE BEHAVIOR OF STEEL I-GIRDER BRIDGES DURING CONSTRUCTION

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BIOGRAPHY

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SUMMARY

In the construction of I-girder bridges, once the steel erection is completed, stay-in-place (SIP) metal deck forms often are attached at the top flange level. The SIP forms provide the surface necessary to place the concrete deck. Traditionally, in the design of steel bridges, the presence of the SIP forms is ignored in the structural analysis. However, under certain circumstances, the SIP forms can influence the displacements that occur in the noncomposite structure from the construction loads and the wet concrete. This paper focuses on identifying the conditions when the participation of the SIP forms in the structural response can be significant. Two bridges are considered that, due to stability-related characteristics are sensitive to the presence of the SIP forms under some scenarios. The studies provide insight to the aspects that need to be considered with regard to SIP forms and their influence on the control of the deflected geometry during construction. Recommendations to configure the bracing systems in I-girder bridges that lead to better overall performance of the steel structure during construction are provided.

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Introduction

Traditionally, in the engineering of steel bridges the use of stay-in-place (SIP) metal deck forms as a means to brace the girders during construction has been prohibited. In fact, in the AASHTO LRFD Bridge Design Specifications, Section 6.7.4.1 (1) it is stated that the use of SIP forms for these purposes is not allowed. On the other hand, in the steel building industry, metal deck forms have been considered as an effective method to provide stability bracing to floor beams. It is important, however, to recognize that there are differences between these applications. The methods to attach the forms to the steel members in bridges are different than used for buildings. In steel bridges, the forms are connected between the top flanges of adjacent girders (see **Figure 1**) with a detail that facilitates control of the surface profile prior to the deck placement. This detail's lack of stiffness, however, decreases the form's efficiency for bracing purposes, as discussed later. In steel buildings, the SIP form-flange attachment is done with puddle welds or self-drilling screws that do not interrupt the continuity of the forms over the flanges. Therefore, when the form flutes are perpendicular to the beams, the forms impede the top flange lateral displacement, stabilizing the beams during the slab casting.

Figure 1. SIP forms in a bridge prior deck placement

Another difference between bridge and building applications is the type of loadings during noncomposite stages. The dead loads and construction loads in these types of structures are significantly different. The strength requirements and therefore the bracing requirements in a bridge girder are often much larger than in floor beams in building construction. These reasons deter engineers from using SIP forms for stability bracing purposes in bridge construction.

The participation of the SIP metal deck forms on the structural behavior of steel bridges is a topic that has attracted the interest of researchers and practitioners. The need of systems that enhance the structural performance and reduce the construction costs has resulted in several research efforts to study the bracing properties of these traditionally non-structural components. The most relevant research conducted in this field is summarized in the following sections of this paper. In general, these studies show that, unless special details are used to attach the forms to the girder flanges, the SIP forms do not serve as an efficient means for girder bracing. On the other hand, researchers have recognized that even though the metal deck forms might not be used for bracing purposes, they are part of the structural system, and thus can affect the bridge deflections during construction.

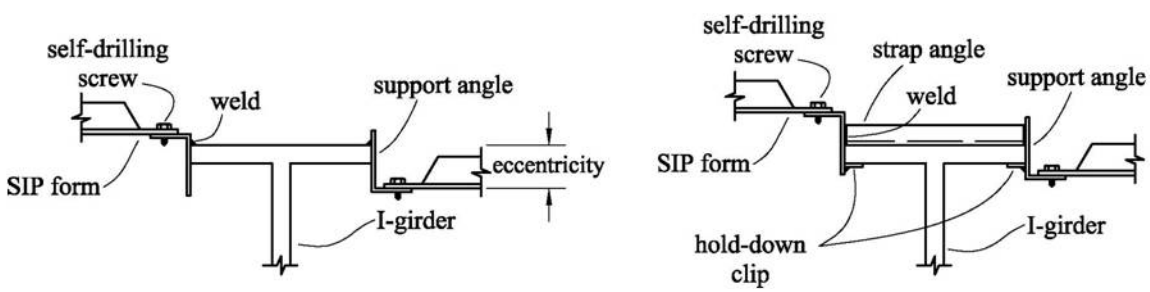
The studies presented in this paper highlight several circumstances when the participation of the SIP forms in the structural response can be significant. Various scenarios are studied in two bridges that, due to their stability-related characteristics, can be sensitive to the presence of the SIP forms. The studies provide an insight to the different aspects that need to be considered with regard to SIP forms and their influence on the deflected geometry during construction. In addition, recommendations to configure the bracing systems in I-girder bridges that lead to better overall performance of the steel structure during construction are provided.

Diaphragm Bracing of I-Girders

Construction Practices

In bridge construction, the practices used to connect SIP forms to the girder top flanges are intended to facilitate the surface control before the deck placement. **Figure 2** illustrates two of the most common details. In the first case, a light gage support angle is welded to the flange, and the forms are attached to the outstanding leg using self-screwing fasteners. When welding directly to the flange is not permitted, a strap angle is used to connect the support angles at each side, and hold-down clips are provided to tighten the support angles to the girder. The practices to attach the support and strap angles to the flanges vary from contractor to contractor, but the concepts shown in **Figure 2** are essentially kept the same.

The main advantage of these details is that by varying the level at which the support angle is placed relative to the flange, the positioning of the SIP forms can be adjusted to obtain an adequate surface profile. These details smooth out the differential deflections that may exist between girders at the end of the steel erection, and accommodate elevation differences due to different flange thicknesses, super-elevation, etc.



(a) Angle – flange weld permitted (b) Angle – flange weld prohibited

Figure 2. Details used to connect SIP forms to the top flange

Structural Properties of SIP Forms

In the last two decades, extensive experimental programs have been conducted to determine the structural properties of SIP forms used in bridge construction, and the feasibility of using them for bracing purposes. Most of these experiments have been conducted at the University of Texas at Austin and the University of Houston. The research in this field has significantly contributed to the understanding of the performance of SIP forms and the aspects that influence their structural properties.

The structural properties of SIP forms are normally characterized in terms of their shear stiffness. The resistance to shear racking is considered to be the largest source of stiffness in the panels. A test setup commonly implemented to determine the properties of SIP forms is depicted in **Figure 3**. In these tests, a panel is connected to a frame that is subjected to a lateral force, *P*. This force creates a shear racking of the panel, causing a tension along one of the diagonal directions. From the test, an effective shear modulus, $G^{'}$, is determined as

$$G^{'} = \frac{PL^2}{fw\Delta} \tag{1}$$

The variables in this equation are defined in **Figure 3**.

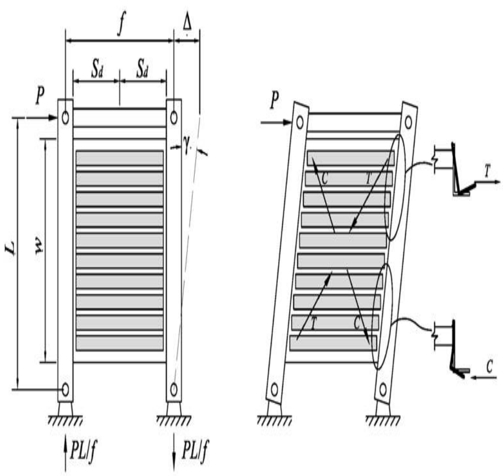


Figure 3. Shear test frame (adapted from References (2) and (3))

Several research efforts have been conducted to test SIP forms used in bridge construction. Reference (4) shows the tests in a number of panels of various gages and with various connection details to measure their shear stiffness properties. Similarly, References (5) and (6) document shear tests conducted on SIP forms that extended the experimental data available. These three references provide the same fundamental conclusion regarding the feasibility of using SIP forms for bracing purposes. The researchers concluded that the flexibility of the connection detail used to attach the forms to the top flange reduces drastically the effective stiffness properties of the panels. As depicted in **Figure 2** and **Figure 3**, there is an eccentricity between the weld and the panel level. In the region where the panel develops a tension field, the support angle pulls away from the flange, rotating about the weld essentially as a rigid body. In the region where the panel is subject to compression, the angle bends under the flange tip. Reference (4) reports that due to the eccentricity, the effective shear stiffness reduction can be larger than 80 % relative to the case where the weld and the panel level are collinear (i.e., no eccentricity).

Reference (3) documents the results of a different type of test conducted to determine the bracing properties of the SIP forms. As depicted in **Figure 4**, a twin-girder system subject to three load conditions was used to measure the stiffness properties of the panels. The advantage of this setup versus the typical shear frame test is that the SIP forms are tested under conditions that better represent their actual participation within the structural system. In the two-girder system shown, the deformations of the SIP panels vary along the length of the girders. From these tests, the researchers showed that the in-plane shear deformations in the SIP forms are larger near the supports, while near mid-span, the larger in-plane deformations are due to flexure. Hence, the stiffness properties determined from these tests are more representative of the physical behavior in bridge structures than the properties obtained from shear tests.

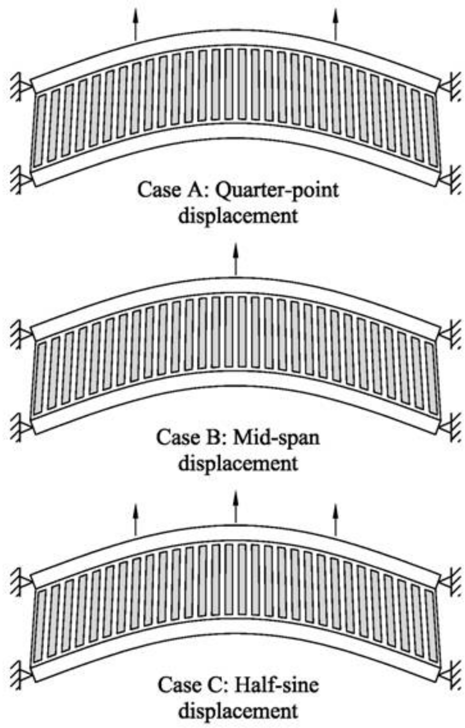


Figure 4. Lateral loading tests conducted in a twin-girder system (adapted from Reference (3))

Similar to previous studies, in References (3) and (7), the researchers concluded that the eccentricity of the support angle attachment severely decreases the bracing capabilities of the SIP forms. In addition to the support angle eccentricity, they determined that the effective shear stiffness of the panels depends on the following parameters:

- The thickness and dimensions of the support angle,
- The top flange thickness,
- The level of dead load that the SIP forms support, and
- The SIP form thickness.

In summary, the studies show that unless special details are provided to eliminate the incidence of these variables, especially the incidence of the support angle eccentricity, the SIP forms may not provide adequate bracing to stabilize the girders during deck placement. Also of importance, the stiffness contributions of the forms to the system are highly variable given that in a bridge, all the parameters described above vary along the structure's span. Therefore, it is difficult to predict accurately the contribution of the forms to the structural responses.

Contribution of SIP Forms to the Bridge Geometry Control

During recent years, studies have been conducted to determine the factors that influence the bridge deflections during construction, especially during deck placement. It is important to conduct an accurate assessment of girder deflections since a successful construction of the concrete deck largely depends on the control of the deformed geometry of the bridge. Some of the difficulties that contractors may face during construction, when the bridge displacements do not correspond to those anticipated by the analysis, include non-uniform deck thickness, undercutting or over-running of the deck thickness, and inadequate distance between the deck surface and the heads of shear studs.

One of the factors studied to ascertain its influence in girder deflections is the participation of the SIP forms. Field measurements and analytical studies have shown that, even though SIP forms may not be used for stability bracing purposes, they are part of the system, and their contributions, in some cases, should be considered in the analysis to properly capture girder deflections. In particular, the studies conducted at North Carolina State University in several research efforts support the conclusion that the SIP forms can influence the girder deflections and therefore, the overall geometry control of the structure. References (8) and (9) report the deflections measured during deck placement of nine straight and skewed I-girder bridges and one straight bridge with normal supports. According to their studies, the numerical simulations represent more accurately the field measurements when the SIP forms are included in the analyses. Further investigations conducted in two straight and skewed I-girder bridges show that the SIP forms can be a contributor to the bridge responses (10). Similar to References (8) and (9), Reference (10) concludes that the results of FE models developed to reproduce the field observations are more representative of the measured vertical deflections and girder layovers when the SIP forms are included in the analyses.

Practically speaking, it is very difficult to construct a model that accurately captures the stiffness contributions of the SIP forms in a given bridge. As reported in References (4) and (7), the effective stiffness of the panel-connection system can vary substantially with the parameters discussed previously. In a given bridge structure, the top flange thickness, the support angle eccentricity, and the detail to attach the support angle to the flange vary along the span. Since all these parameters have an influence on the effective stiffness of the SIP panels, it is not feasible to accurately represent their participation in a computer model. The only way this could be achieved would be for the engineer to take precise measurements of the as-built structure after the contractor completes the setting of the forms, which is obviously not practical. Therefore, it is difficult to validate the influence of SIP forms by comparing the results of computer models to field measurements.

Due to the limitations in representing the SIP form contributions in the structural analysis, it is desirable to determine when they have a significant impact on the response during construction. In the next section, two existing bridges that can be susceptible to the participation of the SIP forms are analyzed. The studies conducted on these structures serve to identify several attributes influencing the participation of the SIP forms on the structural behavior.

Influence of SIP Forms on the Structural Responses of I-Girder Bridges

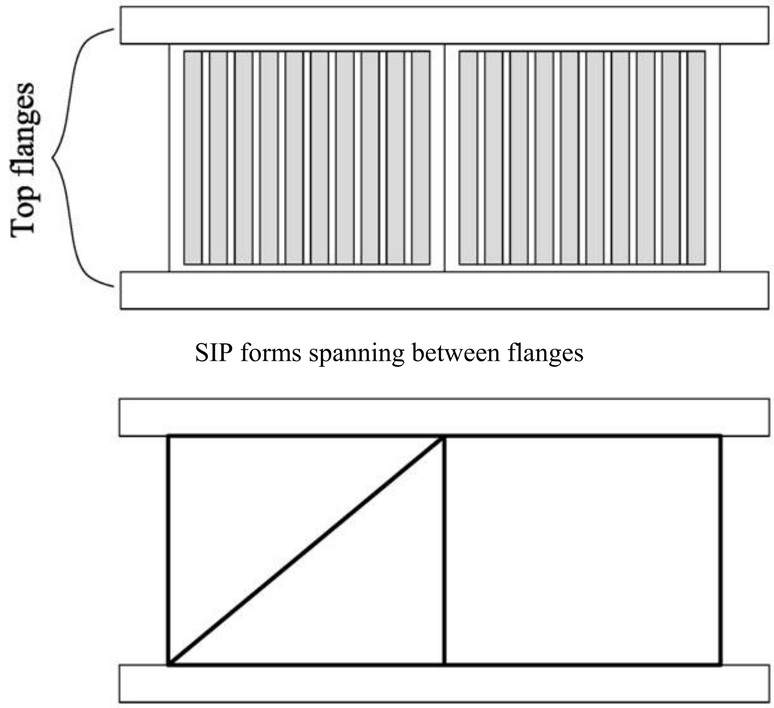
Modeling Techniques used in FEA Models to Represent SIP Forms

As discussed in the previous sections, the structural properties of the SIP forms are commonly represented in terms of their shear stiffness, calculated from Equation 1. Once the effective shear stiffness, G', is known, the diaphragm shear rigidity, Q, is calculated as

$$Q = G' S_d = G' \left(\frac{n_g - 1}{n_g} S \right) \qquad (2)$$

where S_d is the tributary width of diaphragm bracing a single girder, n_g is the number of girders connected to SIP forms (or diaphragms), and S is the panel width. The units of Q are [Force/rad], so it represents the force required to cause a rotation of one radian in the shear test frame. This concept is further illustrated in **Figure 3**. In the context of this figure, $Q = P/\gamma$.

One of the most common strategies to include the SIP forms in FEA is to model them as an equivalent truss. This technique has been implemented in various studies conducted to determine the structural properties of the SIP forms, leading to accurate predictions of laboratory tests (7) (11). As depicted in **Figure 5**, the SIP forms are modeled in the 3D FEA as a sequence of truss panels. In the FEA models, truss diagonals are provided for every two SIP panels.



SIP forms spanning between flanges

Representation of the SIP forms with equivalent truss panels

Figure 5. Equivalent truss concept implemented in FEA to represent the SIP forms

As shown in Reference (12), the in-plane shear stiffness of a truss panel with a single diagonal is equal to

$$\beta_b = \frac{AES^2 h_b^2}{2L_c^3 + S^3} \tag{3}$$

where A is the area of the struts and the diagonal (assuming they are the same), E is the steel modulus of elasticity, h_b is the SIP panel length, and L_c is the diagonal length. As shown in Reference (11), the equivalent shear rigidity is equal to

$$Q = \frac{\beta_b}{2h_b} \tag{4}$$

Equations 2, 3, and 4 are used to determine the area of the struts and the diagonals in the equivalent truss implemented in the 3D FE models. From these equations, the truss element area is

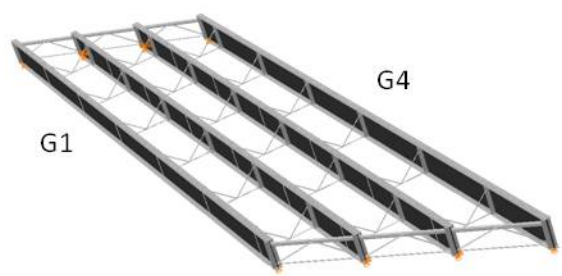
$$A = \frac{G' S_d \left(4L_c^3 + 2S^3\right)}{ES^2 h_b} \tag{5}$$

Except for E (which is equal to 29000 ksi for steel) and G', all the variables in Equation 5 are obtained from the geometry of the SIP forms and the bridge structure. The effective shear stiffness, G', must be determined based on experimental data from tests conducted in shear frames or from tests in twin-girder systems, such as the ones reported in Reference (7). The second method is the most reliable since it best represents the physical deformations experienced during deck placement. The challenge of determining a value of G' that can be used in analytical models is that it depends in several factors that practically cannot be controlled during the construction of a bridge, as discussed previously. For the studies conducted in this research, the values of G' are obtained from the lateral loading tests in a twin-girder system reported in References (3) and (7).

The SIP forms used in the construction of the bridges discussed in the next sections are made from ASTM A-653 material, and have a thickness of 0.0359 in. (20 ga). From the tests reported in Reference (3), G' for SIP forms of these characteristics is equal to 10.18 kip/in.-rad. It is important to emphasize that this value considers the influence of the connection detail flexibility in the effective stiffness of the panels. Reference (3) also provides a value of G' = 15.67 kip/in.-rad for a case where the effect of the support angle eccentricity is reduced by stiffening the connection. From these values, it is observed that the connection detail reduces the effective shear stiffness by 35%. In the next sections, the value of 10.18 kip/in.-rad is used as a reference to develop the truss models that represent the SIP forms in the 3D FEA of the case studies.

Case Study I: Simple Span Straight and Skewed I-Girder Bridge

The first structure considered for this study is a simply-supported four I-girder bridge with V-type cross-frames without top-chords. **Figure 6** shows the bridge in perspective and its key dimensions. The complete set of drawings with the dimensions of the structural components of this bridge is documented in Reference (13). This bridge is considered for the analyses conducted in this section because of the type of cross-frame bracing system utilized in the design. Reference (14) shows that V-type cross-frames without top-chords may not provide the required bracing to prevent excessive second-order amplifications of the girder responses. For this reason, this bridge is a good case to investigate the sensitivity of the structural behavior to the SIP form contributions and other modifications in the bracing system.



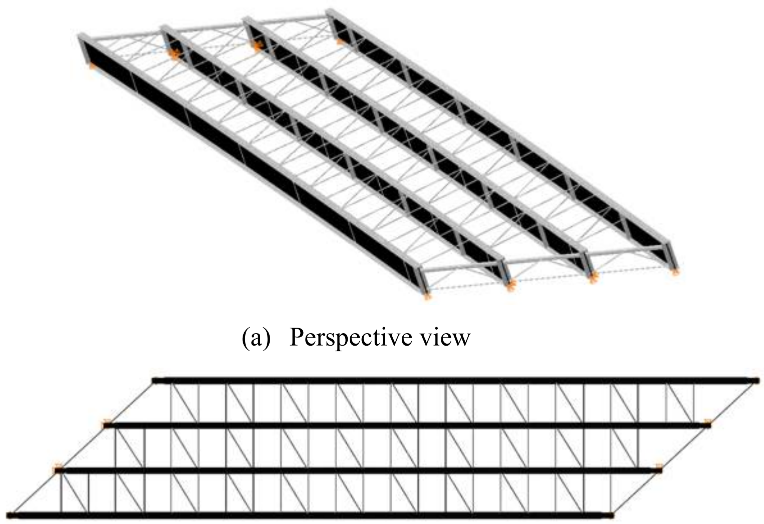
Span length L = 133 ft / Deck width w= 36.1 ft / Bearing line skew angles $\theta_1 = 46.2^o$ and $\theta_2 = 46.2^o$

Figure 6. Perspective view of Case Study I

Two approaches are considered in the analyses to model the SIP forms in the 3D FEA. The first approach follows the procedures discussed in the previous section to model the SIP forms as equivalent truss panels. For this purpose, it is necessary to determine the area of the elements in the truss, A, using Equation 5. As previously mentioned, G' = 10.18 kip/in.-rad. In this bridge, the distance between girders is 9.84 ft and the average top flange width is 14.17 in. Hence, S = 9.84·12-14.17=103.91 in. The panel length, h_b, is 6 ft (72 in.), so the diagonal length, L_c, is equal to 126.42 in. Finally, the tributary width of diaphragm bracing a single girder, S_d = [(4-1)/4]103.91 = 77.93 in. Substituting these values in Equation 5, the area of the elements in the equivalent truss panel is

$$A = \frac{10.18 \times 77.93\left(4 \times 126.42^3 + 2 \times 103.91^3\right)}{29000 \times 103.91^2 \times 72} = 0.36 \text{ in}^2$$

This area is relatively small, if the equivalent truss system is thought of as a top flange level bracing system. For example, the smallest hot-rolled angle available is an L2x2x1/8 and has an area of 0.491 in^2. **Figure 7** shows the perspective and the plan view of the bridge with the equivalent truss panels included. In this figure, the intermediate cross-frames are not shown to facilitate the visualization of the truss panels.



(a) Perspective view

(b) Plan view

Figure 7. Equivalent truss panels used to model the SIP forms in Case Study I

In the second approach, the SIP forms are modeled in the 3D FEA using membrane elements. A membrane is an element that has in plane strength, but does not have out-of-plane bending stiffness, which is the case for the SIP forms. The FEA program ABAQUS 6.10 (15) has a comprehensive library of triangular and quadrilateral membrane elements that can be used for the purpose of these studies. The element selected to simulate the SIP forms is the M3D4 membrane element.

There is an advantage of modeling the SIP forms with membrane elements rather than with equivalent truss panels. A membrane element can be defined to have orthotropic mechanical properties, which is the type of behavior exhibited by the SIP forms. In the SIP forms, the axial stiffness is different in two orthogonal directions. As shown in **Figure 8**, in the direction that is parallel to the flutes, the effective stiffness of the SIP form, k_{d1}, depends on the cross-section properties of the form and the flexibility of the connection between the top flange and the form. In the direction perpendicular to the flutes, the stiffness of the forms is negligible ($k_{d2} \approx 0$) since the forms deform as an accordion. Finally, the SIP forms have an effective in-plane shear stiffness, k_s, that depends on their resistance to shear racking as well as the connection detail, as discussed previously. In a membrane element, these three mechanical properties can be defined independently to best represent the SIP form stiffness contributions.

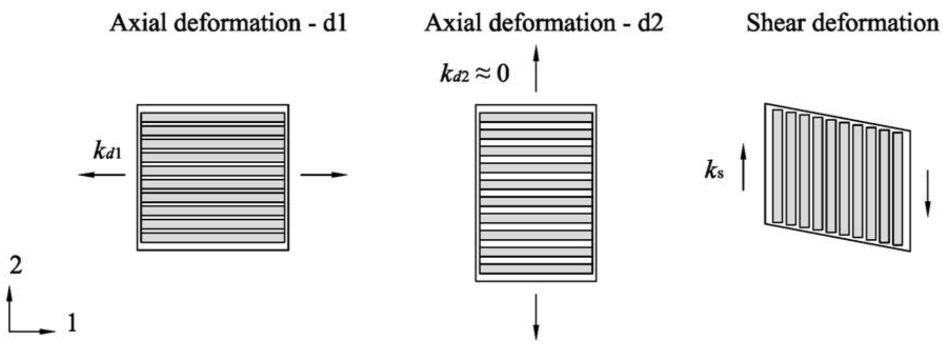
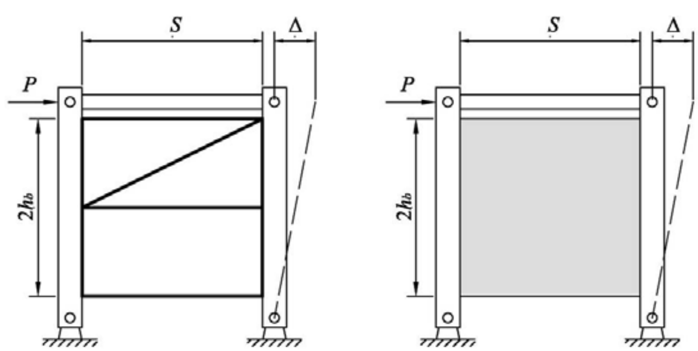


Figure 8. Deformation patterns in an SIP form

When equivalent truss panels are used to model the SIP panels, the area of the truss elements is calibrated to represent only their shear stiffness. In this model, the axial stiffness properties in the two orthogonal directions are not considered. Hence, the equivalent truss model may induce unintended, spurious sources of stiffness. In this study, it is desired to observe the influence of the SIP panels on the bridge deflections, so a more accurate representation of their contributions is necessary. Modeling the SIP panels with membranes not only eliminates possible sources of spurious stiffness, but also serves to compare and validate the results obtained from models that use equivalent truss systems to represent the forms.

To include the panels using membrane elements instead of truss panels, it is necessary to determine the thickness of the elements that captures the panel shear stiffnesses. For this purpose, an FE model of the equivalent truss panel with A = 0.36 in^2 is constructed to simulate the shear frame test depicted in Figure **9**. Then, the panel is subjected to the point load P that causes the lateral displacement Δ. A separate analysis is conducted using a membrane element instead of the equivalent truss panel, and the thickness of the element is calibrated to match the lateral displacement obtained from the previous tests. From this test, it is determined that the thickness of the membrane element needed to represent the shear stiffness of the equivalent truss panel is 0.004 in.

To represent the panels using membrane elements, in addition to the shear stiffness, it is required to compute the membrane (axial) stiffness. The stiffness of the panels in the direction perpendicular to the flutes, k_{d2}, is essentially zero. In the other direction, the panel stiffness is equal to $k_{d1} = E \cdot A_m / L$ = 29000·(0.004×72)/103.92 = 80.4 kip/in. This value represents the stiffness of the panel only. However, the flexibility of the support angle connection reduces this rigidity. As previously mentioned, the reduction in shear stiffness due to the connection detail is 35%. Assuming that the membrane stiffness is reduced by the same factor, k_{d1} = 80.4·(1-0.35) = 52.2 kip/in. This is a reasonable assumption since both shear and axial deformations of the panels cause bending of the support angle (see **Figure 3**).



Equivalent truss panel Membrane element

Figure 9. Tests to determine the thickness of the membrane elements used to represent the SIP forms

Analysis Results

In the studies conducted in Case Study I, seven types of analyses are conducted to observe the participation of the SIP forms on the system responses. Table 1 shows the different scenarios considered for the studies. The first model, M1, corresponds to the case where the SIP forms are not included in the analysis, and the cross-frames are modeled as V-type without a top-chord. In the second analysis, M2, the top-chords of the cross-frames are included in the analyses to observe the changes in the bridge behavior due to the presence of these components. Similar to the first case, in M2 the SIP forms are not considered in the analysis. Model M3 is one of two models that better represent the as-built conditions of the structure. In this analysis, the SIP forms are included in the model by representing them as equivalent truss panels. The area of the truss elements is determined previously in this section and is equal to 0.36 in^2. As in the physical bridge, the cross-frames are modeled without top-chords. The fourth model, M4, is the same as M3, but in this case, the SIP forms are modeled with equivalent trusses with an area two times larger than the one used in M3, i.e., A = 0.72 in^2. The intention of increasing the area of the truss elements is to determine the sensitivity of the structure to different stiffness contributions of the SIP forms. As mentioned previously, it is difficult to accurately capture the participation of the SIP forms given that their stiffness contributions depend on several highly variable factors. Therefore, it is desirable to investigate the sensitivity of the system to changes in the form stiffness.

Model M5 is the same as model M4 except that cross-frame top chords are included in the analysis. As in previous cases, it is desired to observe possible changes in the structural responses due to the inclusion of the top chords, but also considering the contributions of the SIP forms. In the sixth model, M6, the truss elements are modeled with elements that have ten times the area of the elements in model M3. The truss components have an area equal to 3.60 in^2, which is close to the area of L4x4x1/2 angles (A = 3.75 in^2). The truss system in the M6 model resembles a top flange level bracing system that is provided in some cases to increase the global buckling capacity of a bridge and also, to increase the resistance to lateral loads. Model M6 is an extreme scenario to observe the influence of structural members provided at the top flange level on the behavior of a bridge subject to gravity loads during construction. The last model, M7, is similar to M3 as a close representation of the bridge as-built condition. In model M7, the SIP forms are modeled with membrane elements, as described previously in this section. The advantage of using membrane elements rather than equivalent trusses is that with the membrane elements, the orthotropic properties of the SIP forms can be modeled. Since the use of membranes is a more refined method to represent the SIP forms, the results of model M7 are used to validate the predictions obtained from M3 and therefore the other truss panel models.

Table 1. Description of analyzed cases, Case Study I

Model	Top-Chord	SIP form model	Membrane thickness or truss element area
M1	-	-	-
M2	Y	-	-
M3	-	Equivalent truss	0.36 in^2
M4	-	Equivalent truss	0.72 in^2
M5	Y	Equivalent truss	0.72 in^2
M6	Y	Equivalent truss	3.60 in^2
M7	-	Membrane	0.004 in.

In all seven FE models, linear and nonlinear analyses are performed since it is important to observe the response amplifications due to second-order effects. The analyses are conducted at the total dead load level (TDL), which in this paper represents the weight of the steel structure, the wet concrete, and an additional 10 psf loading that represents the weight of the SIP forms plus other appurtenances used to facilitate the construction.

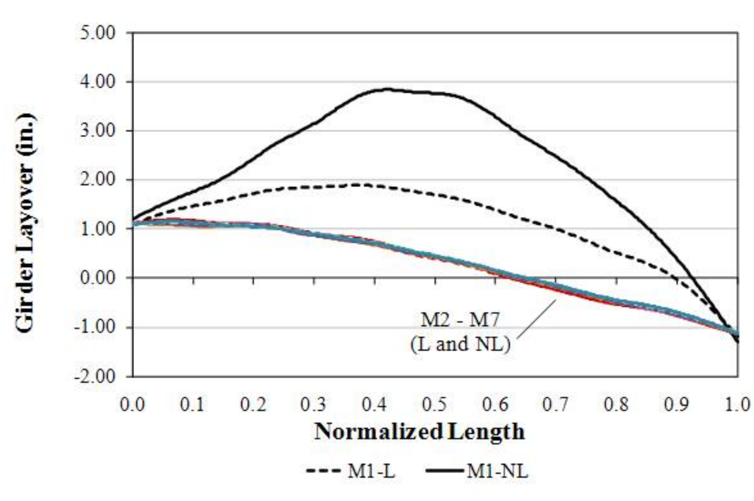
Figure 10 shows the layovers and the vertical displacements of the fascia girder G1. As shown in the plot of the girder layovers, in M1 the amplifications in the response due to second-order effects are important. At the mid-span, the layover predicted by the linear analysis is equal to 1.70 in. while the nonlinear analysis predicts a 3.77 in. layover. This shows that if the system is analyzed with model M1 (no cross-frame top chords, no SIP form participation), it exhibits significant nonlinear behavior. In addition, the results obtained from this model show that the cross-frames are not bracing the girders adequately at the connection points. This affects the lateral-torsional buckling strength of girder segments. In fact, a separate nonlinear analysis conducted to determine the ultimate capacity of the structure shows that the collapse load is 1.05 of the total dead load (TDL) for this model, which is less than the 1.5 TDL design load required by the STRENGTH IV load combination.

The layovers for the fascia girder G1 obtained from the other six models are essentially the same from both linear and nonlinear analyses, as depicted in **Figure 10**(a). There are several conclusions that can be drawn from these results. First, the results of model M2 show that when the top chord is included in the intermediate cross-frames, the system is dominated by first-order actions. This analysis shows that when the girders are braced properly with V-type cross-frames that include top chords, the layovers are reduced drastically and the second-order effects are negligible.

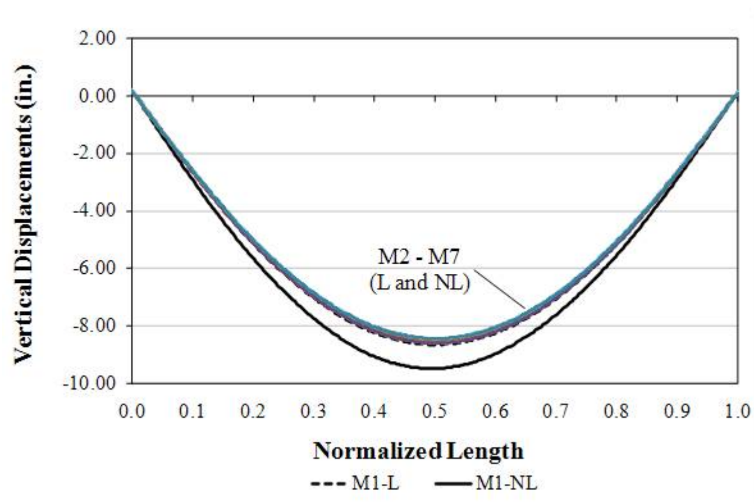
The results of model M3 show that in this bridge, the participation of the SIP forms in the girder layovers (and in the control of the bridge deformed geometry) is significant. The SIP forms are preventing the large rotations of the girder that are observed in the results of model M1-NL. In addition, a comparison of the responses of models M3 and M4 show that the system is insensitive to variations of the SIP form stiffness contributions since doubling the area of the equivalent truss element does not result in significant changes in the girder layovers.

Similar to the previous analyses, the influence of second-order effects on the girder layovers of model M5 are negligible. Additionally, a comparison of the M4 and M5 model results shows that when the top-chords are included in the cross-frames, the contribution of the SIP forms to the layover is irrelevant. This behavior demonstrates that when the in-plane stiffness of the cross-frames is large enough to brace the girders and minimize the local second-order effects that may occur, the SIP forms do not need to be included in the structural analysis. As shown in Reference (14), except for the case of V-type cross-frames without top chords, all the cross-frame configurations commonly used in the design of steel girder bridges (including X-type cross-frames without top chords) are essentially rigid, compared to the lateral-torsional stiffness of the girders. Therefore, excluding the case of bridges connected with V-type cross-frames without top chords, the participation of the SIP forms may be negligible.

The results of model M6 show that even when the bridge is analyzed with an equivalent truss that resembles a top flange level bracing system, the layovers in girder G1 remain essentially the same. As in the previous cases, this highlights the relevance of the cross-frame top chords on the system behavior. Finally, model M7 shows the results obtained with a different representation of the SIP forms. The response obtained from model M7 show that the contributions of the in-plane axial stiffness of the SIP forms (k_{d1} and k_{d2} in **Figure 8**) to the layovers of girder G1 are irrelevant, since the same predictions are obtained as in model M3.



(a) Girder layovers

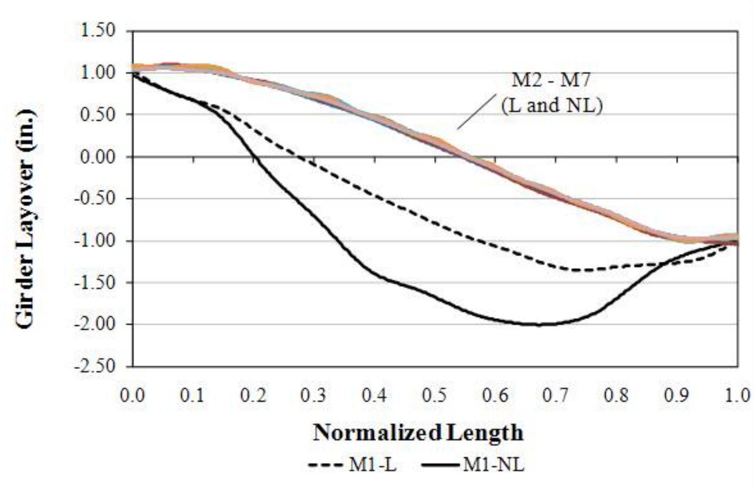


(b) Vertical displacements

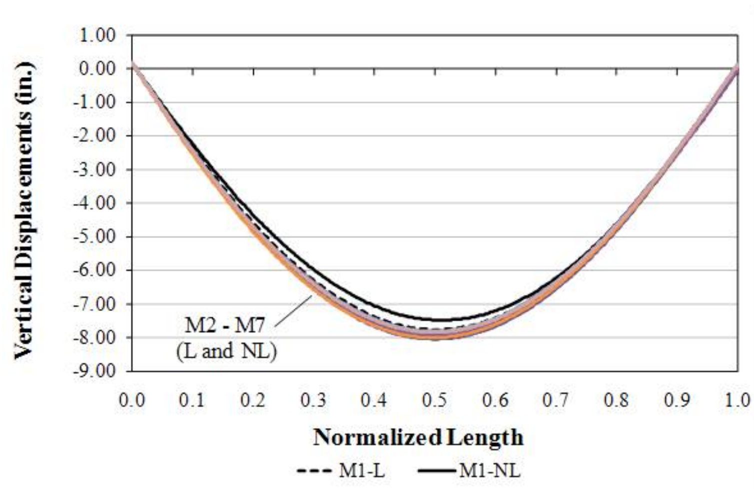
Figure 10. Displacement responses in the fascia girder G1 at the TDL level

The vertical displacement predictions obtained for girder G1 and shown in **Figure 10**(b), follow the same trend as the girder layovers. The second-order effects are less noticeable on the vertical displacements compared to the results for the girder layovers. However, it is evident that except for the results of model M1, the other six models yield essentially the same predictions.

Figure 11 shows the layovers and vertical displacement predictions obtained for the interior girder G2. Similar to girder G1, except for M1 the analyses show that the responses are dominated by first-order actions, and the contributions of SIP forms are insignificant when the system is braced with V-type cross-frames that include top chords.



(a) Girder layovers

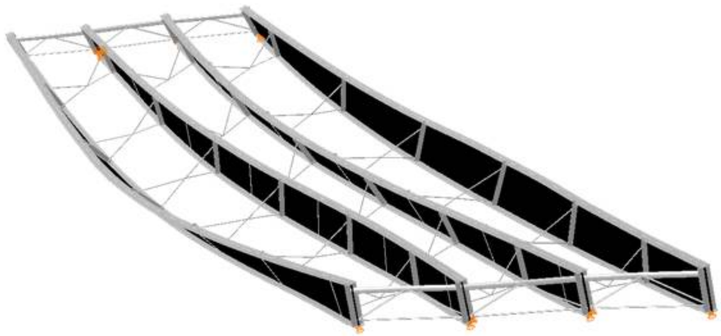


(b) Vertical displacements

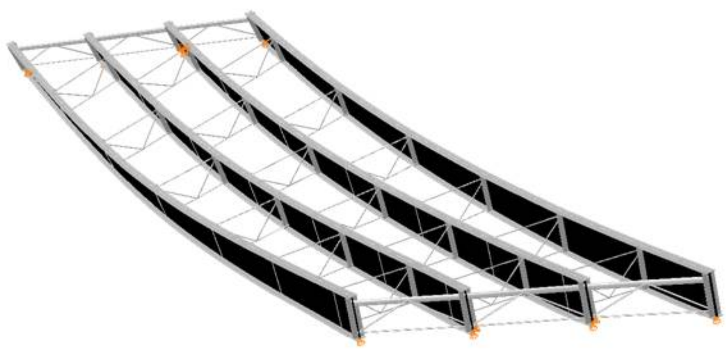
Figure 11. Displacement responses in the first interior girder G2 at the TDL level

To further illustrate the importance of the top chord and the SIP forms on the control of the deformed geometry of the case study bridge under the gravity loads, **Figure 12** depicts the structure in its deflected position. The figure shows the perspective of the structure obtained from models M1, M2, and M3, with the deflections scaled by a factor of ten to facilitate the visualization. As shown in **Figure 12**(a), the cross-frame diagonals rotate about their intersection with the bottom chord. Hence, the in-plane stiffness of the intermediate cross-frames depends mostly on the flexural stiffness of the bottom chord rather than in the axial stiffness of the chords and diagonals. Due to the poor bracing properties of this type of cross-frame, the girders experience large second-order amplification, which results in excessive girder layovers. As shown in the figure, deflections that correspond to the characteristics of lateral-torsional buckling occur along the entire length of the girder. When the top chords are included, the girders layovers are reduced and the girders deflect more uniformly, as depicted in **Figure 12**(b). In this case (M2), the cross-frames are stiffer than in the first case (M1) and brace the girders such that the bridge system behaves as a unit. Also, contrary to the first case, the lateral-torsional buckling strength is determined by the strength of girder segments with a length equal to the distance between cross-frames.

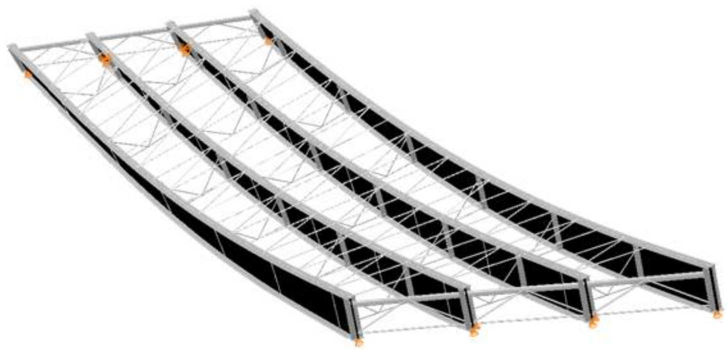
Finally, **Figure 12**(c) shows that the SIP forms participate reducing the effects of not including the top chords.



(a) Model M1 (V-type cross-frames without top chords, SIP forms not included)



(b) Model M2 (V-type cross-frames with top chords, SIP forms not included)

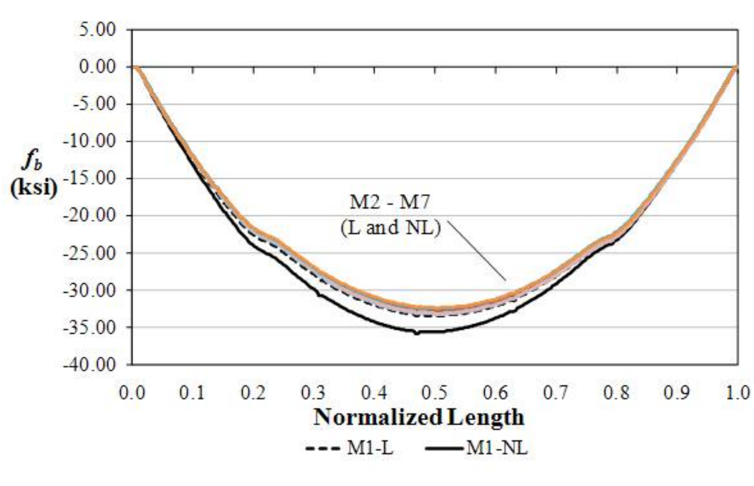


(c) Model M3 (V-type cross-frames without top chords, SIP forms included)

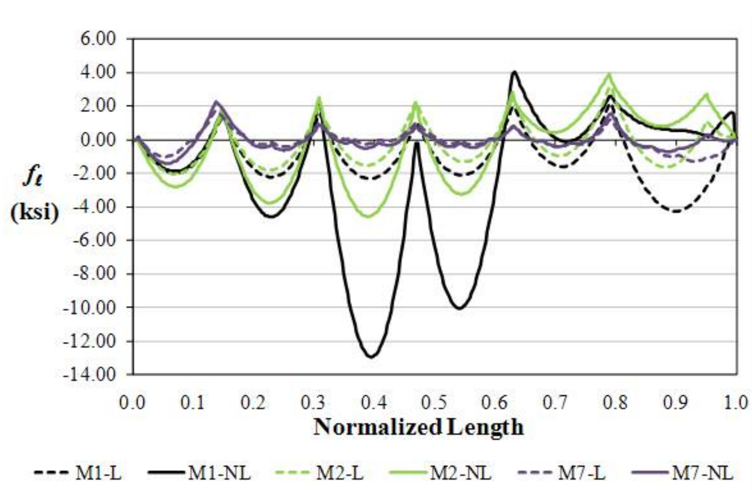
Figure 12. Deflected geometry of Case Study I at the TDL level (Scale factor = 10x)

In addition to the girder displacements, it is important to investigate the stress responses obtained from the different models. **Figure 13** shows the major-axis bending, f_b, and flange lateral bending stresses, f_ℓ, in girder G1. In **Figure 13**(b), only the predictions from models M1, M2, and M7 are included to facilitate the visualization of results. The predictions captured by these three models are sufficient to investigate the influence of the SIP forms and the cross-frame top chord in the structural behavior.

The results in **Figure 13**(a) show that the major-axis bending stresses are insensitive to the variations included in the models. The stress predictions obtained from model M1-NL are slightly larger than in M1-L, showing that the amplifications due to second-order effects in f_b are minor. In the case of f_ℓ, the differences are more evident. The solution for M1-NL shows that the levels of f_ℓ are large in the third and fourth girder segments. The amplifications associated with nonlinear behavior in this response are considerable. However, when top chords are provided, flange lateral bending stresses are substantially smaller. Model M7, which closely represents the as-built condition, shows that the SIP forms participate in reducing the f_ℓ levels, especially the "local" lateral bending within the girder unbraced lengths.



(a) Major-axis bending stresses



(b) Flange lateral bending stresses

Figure 13. Stress responses in the fascia girder G1 at the TDL level

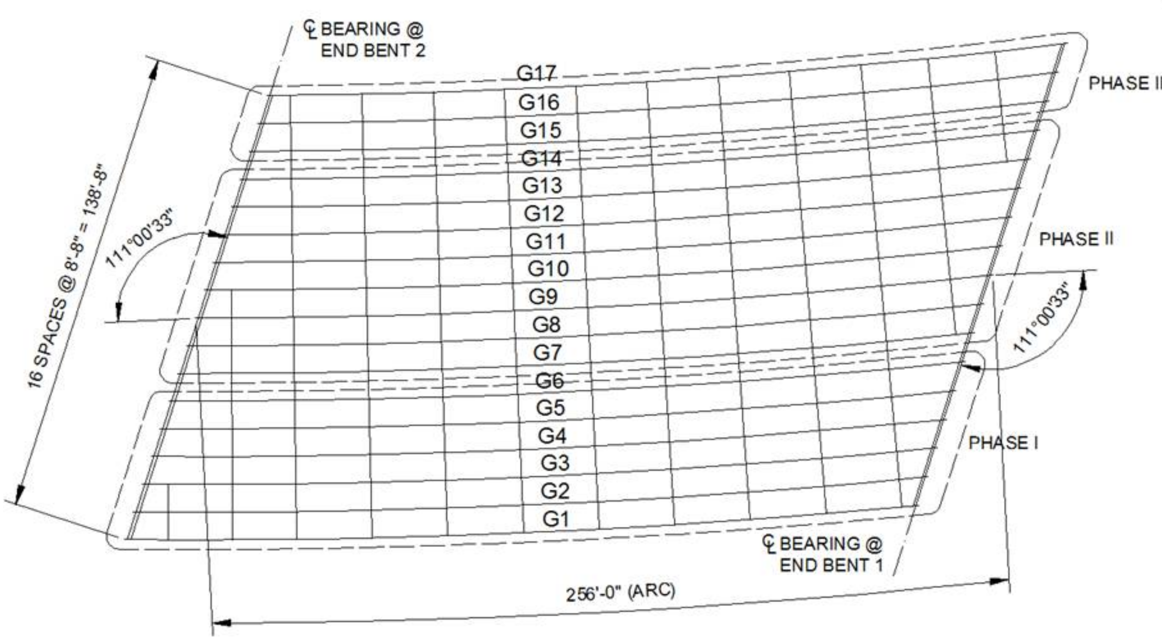
The results obtained from the different analyses conducted on this bridge show that under certain circumstances the SIP forms influence the system performance. The displacement responses demonstrate that the control of the deformed geometry during construction is sensitive to the presence of the SIP forms only if the cross-frame top chords are not included in the bridge. When the top chords are included in the models, the impact of considering the SIP forms in the analyses is negligible. In addition, the results show that the system is insensitive to fluctuations in the SIP form stiffness when the in-plane stiffness of the cross-frames is sufficient to brace the girders properly. In conclusion, the studies conducted on this bridge suggest that the use of V-type cross-frames without top chords may lead to difficulties in predicting the behavior of the structure during construction, due to the difficulties in defining the true stiffness contributions from the SIP forms. When the cross-frames are stiff enough to connect the girders and make them work as a unit, the participation of the SIP forms may be insignificant.

Case Study II: Simple Span Curved and Skewed I-Girder Bridge

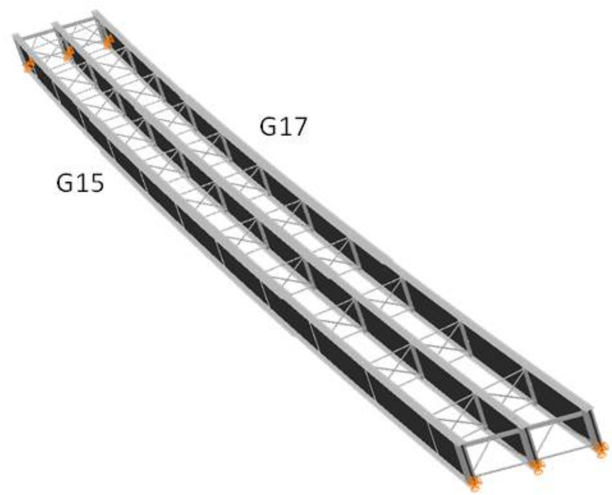
Another type of bridge where the participation of the SIP forms may be considerable is structures susceptible to large response amplifications due to global second-order effects. This is the case for I-girder bridge units with large span-to-width ratios. These structures can exhibit important global stability problems (17) rather than individual unbraced length response amplifications, as experienced by the bridge presented in the first case study.

The bridge discussed in this section is an existing structure with these characteristics. It is a three-girder unit with a span of 256 ft that experienced large global second-order amplifications during its construction. The unit was the third phase of a construction project erected next to an I-girder system consisting of 14 girders that had been constructed previously, in Phases I and II. Figure **14**(a) shows the plan view of the 17 girder bridge, and Figure **14**(b) shows a perspective view of the FE model of the three-girder unit studied in this section. The basic information that describes the geometry of this bridge unit is included in Figure **14**. Reference (13) contains a complete set of drawings that define the bridge dimensions.

During the physical deck placement on Phase III of the above bridge, it was observed that the three-girder unit was deflecting more than expected. By the time that two-thirds of the slab had been cast, there was a substantial difference between the top-of-slab levels between Phases II and III. At this point, the concrete casting was halted since the three-girder unit was potentially near a condition of collapse.



(a) Plan view of the 17 I-girder bridge

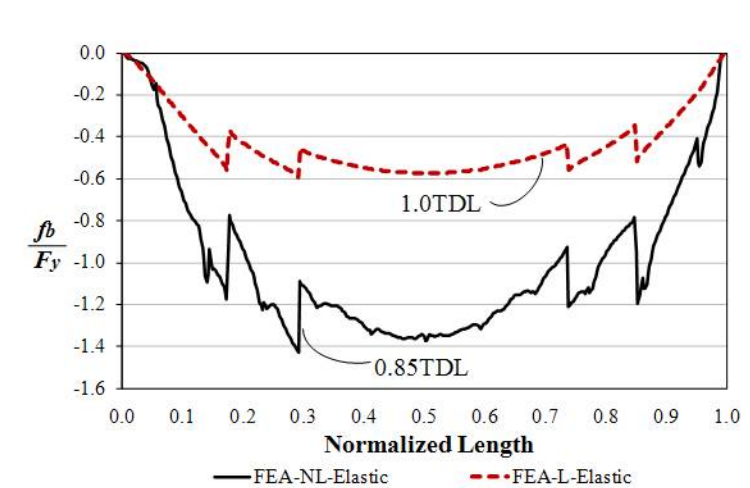


Span length L = 256 ft / Horizontal radius of curvature R = 2269 ft / Deck width w = 26.6 ft / Skew of the bearing lines, $\theta_1 = -24.71^o$ and $\theta_2 = -18.36^o$

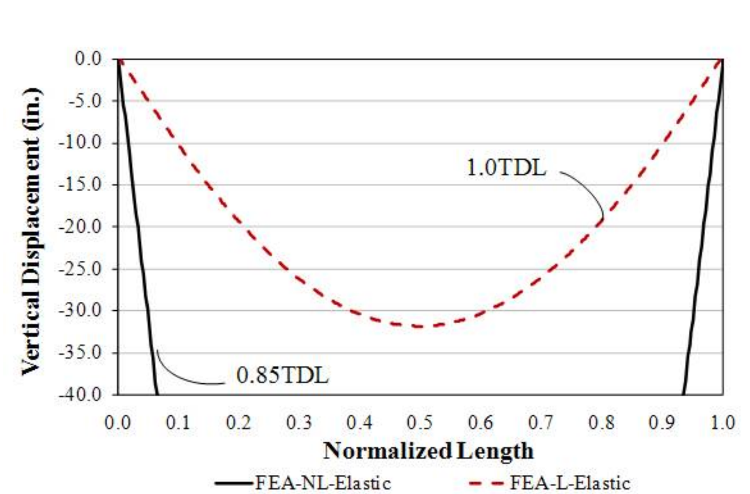
(b) Three-girder unit (Phase III)

Figure 14. Plan view if the 17 I-girder bridge and the three girder unit

The relevance of the global second-order effects in this structure is depicted in **Figure 15**, which shows the responses in girder G15 obtained from elastic linear and geometrically nonlinear FEA. In these analyses the participation of the SIP forms is not included. **Figure 15**(a) shows the results for the top flange major-axis bending stress, f_b, normalized with respect to the yield strength of the steel (F_y = 70 ksi). **Figure 15**(b) show the vertical displacements, with the vertical axis truncated at 40.0 inches to facilitate the comparisons. The maximum deflection at mid-span predicted by the nonlinear FEA is 216 inches. As shown in these plots, the response amplifications in both f_b and the vertical displacements due to geometric nonlinearity are substantial. The results obtained from the nonlinear analysis are shown at 0.85TDL since this is the limit load predicted by the elastic nonlinear FEA.



(a) Major-axis bending stresses



(b) Vertical displacements

Figure 15. Responses predicted in girder G15 of Case Study II by elastic linear and nonlinear FEA

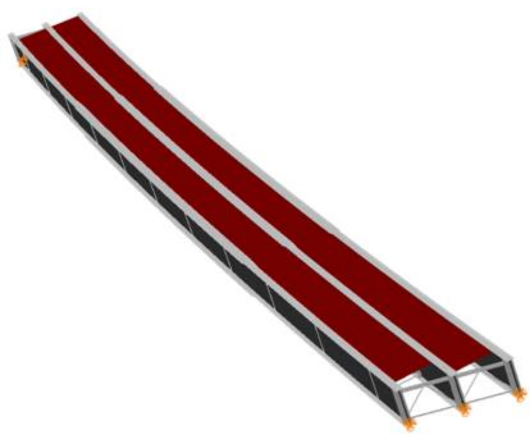
Similar to the analyses conducted in Case Study I, the influence of the SIP forms on the structural response of the three-girder unit is studied. It is intended to observe the participation of the forms on the behavior of a long-and-narrow bridge under construction. For this purpose, the SIP forms are included in the 3D FEA models as membrane elements, following the techniques discussed previously.

To represent the SIP forms with membrane elements it is necessary to determine the membrane dimensions that represent the mechanical properties of the forms. As in the previous case study, the membrane thickness is obtained by comparing its shear stiffness to the stiffness of an equivalent truss panel. For this purpose, the area of the truss elements, A, is calculated using Equation 5. The SIP panels in this structure are the same as those used for Case Study I, so the same shear stiffness G' = 10.18 kip/in.-rad and panel width, h_b = 6 ft (72 in.) is used. In this structure, the girder spacing is 8.67 ft, and the average top flange thickness is 24.0 in. Hence, the panel width, S, is equal to 8.67–24.0/12 = 6.67 ft (80.04 in.). The tributary width of diaphragm bracing is S_d = [(3-1)/3]6.67 = 4.45 ft (53.36 in.). Finally, with h_b = 6 ft and S = 6.67 ft, the diagonal length, L_c, is equal to 8.97 ft (107.66 in.). Substituting these values in Equation 5, the area of the elements in the equivalent truss panel is

$$A = \frac{10.18 \times 53.36\left(4 \times 107.66^3 + 2 \times 80.04^3\right)}{29000 \times 80.04^2 \times 72} = 0.24 \text{ in}^2$$

Similar to the procedure described in previous sections, the thickness of the membrane elements used to represent the SIP forms is determined by simulation of the shear frame test. From this analysis, it is determined that a membrane element with a thickness of 0.002 in. is needed to obtain the same shear stiffness of a truss panel with A = 0.24 in^2. In addition, the membrane stiffness in the direction parallel to the flutes is equal to $k_{d1} = E \cdot A_m / L = 29000 \cdot (0.002 \times 72)/80.04 = 52.2$ kip/in. Considering the stiffness reduction due to the connection detail, the in-plane stiffness of the membrane parallel to the direction of the flutes is $k_{d1} = 52.2 \cdot (1-0.35) = 33.9$ kip/in. As previously stated, in the other orthogonal direction, the stiffness of the panels is negligible, so $k_{d2} = 0$ kip/in. **Figure 16** shows the models with and without the SIP forms.

(a) Model without the SIP forms



(b) SIP forms modeled with membrane elements

Figure 16. Perspective view of the two models used in the analyses of Case Study II

In addition to the elastic models described above, this bridge is analyzed using full nonlinear FEA procedures. Besides including geometric nonlinear effects, a full nonlinear FEA captures the plastic deformations in the system, considering the effects of the residual stresses that result from the fabrication of the I-girders. Due to these characteristics, this analysis is the closest representation of the structural performance of the physical bridge. The intent of conducting a full nonlinear FEA is to determine the influence of the SIP forms on the bridge deflections as well as the system limit load.

As the load level is increased up to the limit load, buckling of the SIP forms and fastener shear or tear out can occur at the connections between the panels and the top flange. These aspects are not included in the model with the SIP forms. However, the comparison between the analyses conducted without SIP forms and modeling them with membrane elements bound the structural response of the bridge. If these aspects have an effect on the structure's behavior, this has to be bounded between the responses predicted with the models described previously.

Figure 17 shows the fraction of the TDL versus the lateral displacement of girder G15 at mid-span for the linear and nonlinear elastic analyses, as well, as for the full nonlinear FEAs. In the plot, the dotted horizontal line corresponds to the fraction of the TDL where the theoretical elastic structure reaches its load limit, i.e., 0.85TDL. The dot-dash line represents the fraction of TDL that corresponds to the steel structure's self-weight or steel dead load (SDL), i.e., 0.33TDL. As shown in this figure, the SIP forms provide only a minor contribution to the response. In general, at a given load level, the SIP forms result in a slight reduction in the lateral deflections of the top flange. Regarding the limit load, the results from the full nonlinear FEA show that when the forms are included in the analysis the collapse load increases slightly from 0.70TDL to 0.72TDL.

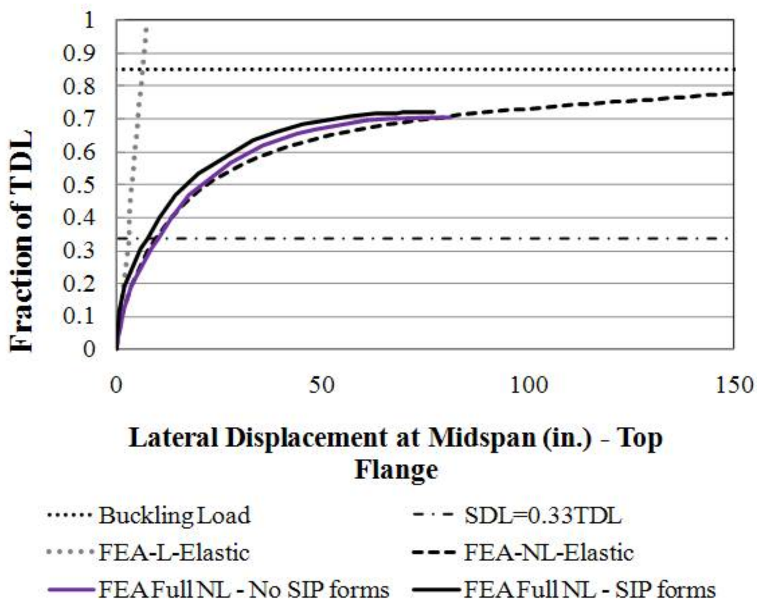


Figure 17. TDL fraction vs. flange lateral displacement of girder G15

Another response that is of interest in this study is the girder vertical displacements. **Figure 18** shows the vertical displacements at the mid-span of girder G15 plotted versus the fraction of the TDL. Similar to the previous plot, the difference between the responses predicted by the full nonlinear models with and without the modeling of the SIP forms is minor.

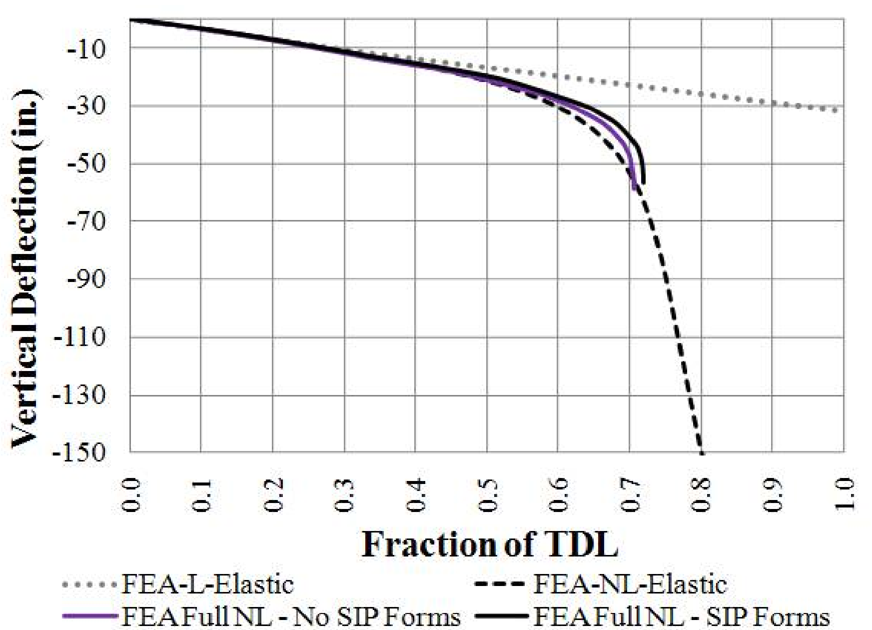


Figure 18. Vertical deflection at mid-span of girder G15 in Bridge EISCS4

The results obtained from the above analyses show that the SIP forms have a small effect on the structural responses. In this case study, the SIP forms do not significantly reduce the displacement amplifications associated with global geometric nonlinear behavior. From these results, it is apparent that SIP forms may not participate significantly on the control of the deformed geometry of narrow bridges that are susceptible to global second-order effects. Finally, the analyses conducted in this long-and-narrow structure show that the SIP panels do not increase the limit load considerably. As shown for this bridge the collapse load only increased from 0.70TDL to 0.72TDL when the SIP forms are included in the model.

Summary

This paper presents studies conducted to determine the influence of SIP forms on the behavior of steel I-girder bridges during deck placement. The two bridges discussed in this paper were selected for the studies because they are susceptible to large response amplifications due to second-order effects.

In Case Study I, the nonlinear behavior is due to the poor stability bracing provided by intermediate V-type cross-frames without top chords. The analyses of this bridge show that when the in-plane stiffness of the cross-frames is not sufficient to connect the girders and make them work as a unit, the SIP forms can have a significant influence on the structure deflections. However, when the girders are connected at sufficient locations with cross-frames that have sufficient in-plane stiffness (by use of a top chord), the influence of the SIP forms may be negligible. In the context of analysis and design of I-girder bridges, this means that the SIP forms may not need to be included in the analyses if the intermediate cross-frames are stiff enough to maintain their geometry and behave essentially as rigid bodies during deck placement. Fortunately, except for V-type cross-frame without top chords, all the cross-frame configurations commonly used in the design of I-girder bridges (including X-type cross-frames without to chords) satisfy this requirement.

In Case Study II, the second-order amplifications observed on the system responses are due to different reasons than in Case Study I. Long-and-narrow structures can exhibit nonlinear behavior due to global stability effects rather than the individual unbraced length problems observed in bridges that are poorly braced. However, in bridges susceptible to global nonlinear behavior, the participation of the SIP forms may not be as significant as in bridges with response amplifications due to local second-order effects. The results of Case Study II show that the forms do not have an important influence on the girder deflections or the system strength.

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