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Erection behavior and grillage model accuracy for a large radius curved bridge

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ABSTRACT

The effects of construction procedures on the stresses and deformations in a large radius, horizontally curved, plate girder, bridge were examined along with the accuracy with which grillage models predicted the construction behavior. The examination included a study of the stresses and deformations during construction and a comparison of those quantities to the grillage model predictions. Results from the study indicated that, for the structure that was examined: (1) appreciable warping stresses were generated during girder erection; (2) the classical grillage model predictions were less accurate during girder erection while the "modified" model predictions were more accurate during deck placement; and (3) the predicted grillage model deflections were smaller for an exterior-to-interior girder erection procedure than an interior-to-exterior procedure.

1. Introduction

Most problems with horizontally curved, steel, plate girder bridges occur during construction, particularly during girder erection. This is caused by a lack of understanding of curved bridge behavior during construction by both designers and constructors. Because curved girders rely on adjacent elements for stability, both translations and rotations change throughout erection as members are added to the structural system. These deformations are difficult to predict using simplified numerical procedures, such as traditional line girder analyses that cannot effectively account for stiffness interaction between adjacent girders. If a simplified approach is used and inaccurate deformation predictions result, girder and cross-frame fit-up problems can occur.

Grubb et al. [1] stated that, in the past, the design of these bridges typically focused on the stability and strength of the completed structure during service and under ultimate load conditions, and ignored the construction. However, as curved steel bridges become shallower and longer and, consequently, less stiff, accurate prediction of movement during construction becomes even more important. Current AASHTO bridge design and construction specifications [2,3] require consideration of the effects of curvature during fabrication, shipment, erection, and deck placement, but provide little guidance for the designer or contractor for doing so.

To ensure that horizontally curved, steel, plate girder bridges are erected safely and economically it is important that more accurate and effective construction procedures, based upon examinations of the actual construction response, be developed. This can only be accomplished by advancing the understanding of the behavior of these structures during construction and coupling these advancements with continued modifications to current specifications and guidelines to aid designers and constructors.

Equally important to advancing the understanding of curved steel bridge construction behavior is the need to examine the effectiveness with which models that are more advanced than a line girder predict the construction response, especially deformations. It could be believed that three-dimensional (3D) finite element analyses (FEAs) would be the natural replacement given perceived increases in their accuracy and output. However, a desire to utilize a less sophisticated "middle-ground" model, one that requires reduced time to construct and interpret, still exists for practitioners when preliminary analysis results are needed or budgetary and time constraints preclude the development of a 3D FEA model. Grillage, also known as grid, models, which in their purest form are two-dimensional (2D) representations of 3D structures utilizing linear elements (either beam or frame elements) connected to one another in a form that represents the deck and the main elements supporting the deck, could be considered to be a "middle-ground" model due to reduced complexity related to their development and understanding. These types of model are still used extensively by practitioners and are permitted by the AASHTO LRFD Bridge Design Specifications for curved I-girder systems [2].

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The current study attempts to add to the knowledge base related to curved, steel, plate girder bridge behavior during construction by:

- Documenting and evaluating the actual construction procedures for a large radius, horizontally curved, steel, plate girder bridge and identifying the effects of these procedures on bridge behavior;
- Determining if various forms of two-dimensional grillage models can accurately predict the behavior of the examined bridge during construction. One form, used during steel erection, was constructed in similar fashion to models traditionally employed by practitioners (termed herein as "classical" grillage models) and the second form, used during deck placement, incorporated modifications to a traditional grillage to allow for modeling deck placement sequencing (termed "modified" grillage models);
- Establishing if equivalent straight structure analyses of the examined bridge, another modeling option that may be attractive to practitioners modeling curved bridges having large radii, using the model forms described above can accurately predict its construction response; and
- Examining the effects of girder erection sequencing on the final geometry.

It should be emphasized that the study summarized herein focuses on a single bridge and evaluates the accuracy of the analysis techniques and the effects of different erection sequences for that structure only. While results from this study cannot be expanded to all horizontally curved, steel, plate girder bridges, they do provide a contribution to the existing literature related to large radii, curved steel structure construction behavior and to studies that examine the grillage model accuracy when applied to curved bridges.

Research into the behavior of horizontally curved, steel, I-girder bridges has been extensively documented by the lead author [4], Hall et al. [5] and Zureick et al. [6]. Research that specifically focuses on the behavior of these structures during construction is more limited and, while also having been addressed elsewhere, will be briefly summarized herein.

Horizontally curved, steel, I-girder bridge research containing a component that examined construction behavior was first completed in association with the Consortium of University Research Teams (CURT) project by Brennan [7] and Brennan and Mandel [8]. Since that initial project, intermittent research and discussion related to construction behavior has occurred. Grubb et al. [1] outlined construction issues for steel curved I-girder bridges by describing the fabrication, erection, bracing and temporary shoring methods, and deck placement techniques. The Curved Steel Bridge Research Project (CSBRP), initiated in 1993, contained some components that addressed construction issues for curved I-girders. One set of laboratory tests that took place during initial assembly of the full scale CSBRP test frame was documented by Zureick et al. [9] and Linzell et al. [10–12]. Additional construction studies were performed in association with later phases of the CSBRP. These studies examined a specified erection sequence of the final test frame configuration, starting from having girders blocked to their design vertical camber on the ground and ending with placement of the deck, and its effects on induced stressed and deformations [13,14].

Hajjar and Boyer [15] and Galambos et al. [16,17] monitored a two-span, continuous, curved, steel, I-girder bridge during all phases of construction. A study that examines levels of moment distribution during the construction of curved I-girder bridge systems was completed by Sennah et al. [18]. Bradford et al. [19] studied single, simply supported curved I-girders subjected to top flange uniform or concentrated gravity loads using ABAQUS and a finite element code developed by Pi et al. [20] to investigate nonlinearity levels for curved girders.

Alampalli and Morreale [21] examined the uplift forces that occurred during the construction of a four-span, continuous, horizontally curved I-girder bridge with skewed supports to develop a corrective tie-down system for the structure. Chavel and Earls [22–24] studied horizontally curved bridge behavior during girder and deck placement and addressed fit-up difficulties in an actual I-girder structure during these stages by focusing on cross-frame detailing.

Bell [25] monitored the completion of girder erection of a large radius, curved, I-girder bridge after difficulties led to changes in the initial erection procedure. Analytical models of the superstructure were used to examine various erection procedures and their effects on the final girder geometry and stresses.

Chang [13] and Chang and White [14] used results from Chavel and Earls' work in conjunction with CSBRP test data to assist with the development of a computer program that intends to provide reliable construction response predictions for curved, I-girder, bridges. The program, GT-SABRE, utilizes what is termed a "beam-grillage" approach to predict response. The "beam-grillage" models utilize offset elements to model the main girder lines and the slab.

The behavior of curved I-girder bridges during construction was also recently studied by Madhavan [26]. A numerical study was completed that examined the effect of warping normal stresses due to curvature on the non-composite compression flange strength and stability. In addition, ongoing research related to the development of guidelines for analyzing and designing curved bridges for erection is being completed in affiliation with National Cooperative Research (NCHRP) Project 12-79, *Guidelines for Analytical Methods and Erection Engineering of Curved and Skewed Steel Deck-Girder Bridges* [27].

The brief summary presented above indicates that, while more emphasis has recently been placed on examining the curved steel bridge construction response, the complete body of published work is still somewhat small. In addition, there are very few published studies that examine the accuracy with which common numerical tools predict the actual curved structure response, regardless of the level of curvature. In addition to examining the construction response of an actual structure, the present work is attempting to examine the accuracy with which one commonly used analysis approach, that incorporating grillage models for the superstructure, predicts the response.

2. Structure description

The bridge that was examined is identified as Structure #207 and is located in central Pennsylvania. It is a two-span, continuous, composite, steel, plate girder bridge with a total length of 146.53 m (480′ 9″) supported on radially oriented piers and abutments. The superstructure utilizes five ASTM A709, Grade 50, singly symmetric, steel, plate girders that are 2.74 m (108″) deep and spaced 3.25 m (10′ 8″) center-to-center. The flange plate dimensions vary along each girder. Spans 1 and 2 measure 65.4 m (214′ 6″) and 81.5 m (266′ 3″) along the arc, respectively, and the radius of curvature to the center girder is 585.5 m (1920′ $10\frac{5}{16}$″). Transverse bracing is provided using radially oriented K-shaped cross-frames containing WT sections. For shipping purposes, each girder consists of five sections that are bolted using four field splices. Figs. 1 and 2 detail a framing plan and a typical section.

3. Girder erection and deck placement

The final erection procedure was documented in detail elsewhere [27]. A brief summary of the girder erection and deck placement sequence is provided herein.

A single girder erection procedure was used, working from the interior girder (G5) towards the exterior girder (G1). Span 1 was

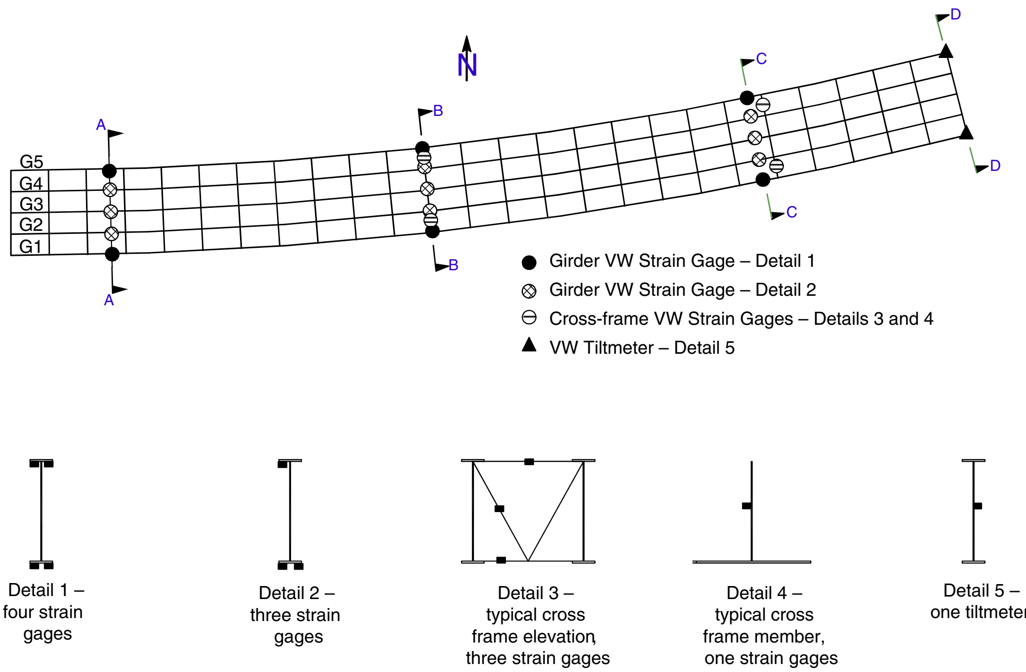

Fig. 1. Plan view, structure #207, detailing vibrating wire instrument locations [28].

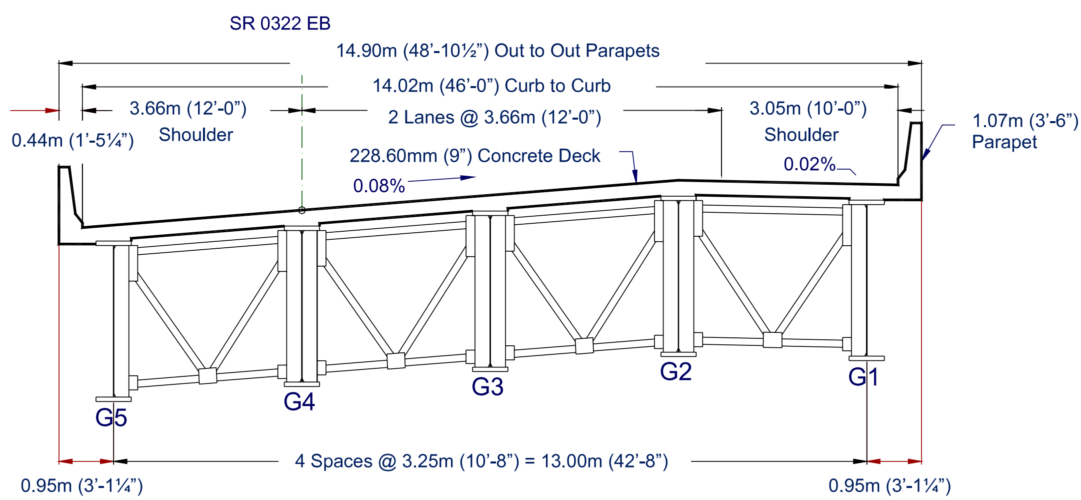

Fig. 2. Typical section, structure #207 [28].

erected along with a portion of Span 2 (to Field Splice 3) prior to any girder placement within Span 2. The contractor utilized temporary shoring towers under G4 and G5 and tie-downs at the abutments and piers. Three cranes were used to place the girders and a boom truck was used to place the cross-frames. Deadmen were affixed to G2 as detailed in Fig. 3 at the completion of erection to provide additional radial restraint. The deadmen remained in place until completion of the deck pour. Bolts were installed throughout the process but were not tightened until all girders and cross-frames were in place. Girder erection took ten days to complete. Table 1 summarizes the erection sequence and Fig. 4 gives a plan view of the crane and shoring tower locations relative to the completed superstructure. Note that the designations "A" through "E" in Table 1 refer to the five segments for each girder in their erected, alphabetized sequence from Abutment 1 to Abutment 2.

Girder erection largely proceeded without major errors or delays. Initially, the contractor attempted to tighten the bolted girder field splices by lifting a single girder splice at a time, which resulted in undesired movement of adjacent, tightened field splices. Therefore, all five girders were lifted at a given field splice simultaneously using the temporary shoring towers, and this problem was mitigated. As the tightening and aligning progressed, minor alignment issues (top of girder elevations) were observed and two splices needed to be reassembled.

Placement of the 229 mm (9″) reinforced concrete deck was conducted in four stages, as shown in Fig. 5. Deck reinforcement consisted of two epoxied rebar mats. The main reinforcing steel was oriented radially and consisted of #5 bar spaced 152 mm (6″) on center. The transverse reinforcement consisted of #5 bars spaced at 279 mm (11″) in positive moment regions, and #5 and #6 bar alternating at 127 mm (5″) in the negative moment region. To minimize deck cracking in the negative moment regions, concrete in the positive moment region of Span 1 was placed first, followed by the positive moment region of Span 2. The negative moment region over the pier was poured during the third stage with the blockouts at Abutments 1 and 2 poured during the final stage. A pump-truck was used to deliver concrete onto the bridge and a finishing machine and work bridges were used to place and finish

Table 1
Erection sequence summary.

Day	Event
1	Placement of G4 and G5 A,B, and C
2	Placement of cross-frames between G4 and G5 Span 1 and removal of temporary shoring towers
3	Placement of G3 A, B, and C and cross-frames
4	Placement of G2 A, B, and C and cross-frames
5	Placement of G1 A, B, and C and cross-frames
6	Cranes moved from Span 1 to Span 2 positions
7	Placement of G4 and G5 D and E, and cross-frames.
8	Removal of temporary supports in Span 2. Placement of G3 D and E and cross-frames.
9	Placement of G2 D and E and cross-frames
10	Placement of G1 D and E and cross-frames
11	Began tightening bolts at Field Splice 3, G1
12	Began tightening bolts at Field Splice 4, all girders
13	Continued tightening bolts at Field Splice 4 and proceeded to Field Splice 3
14	Checked field splice elevations.
15	Continued tightening bolts at Field Splice 3
16	Completed tightening bolts in Span 2. Began tightening bolts in Span 1, Field Splice 2.
17	Field Splice 4, G1 and Field Splice 3, G2 were "re-worked"
18–20	Completed tightening bolts in Span 1 and all cross-frames.

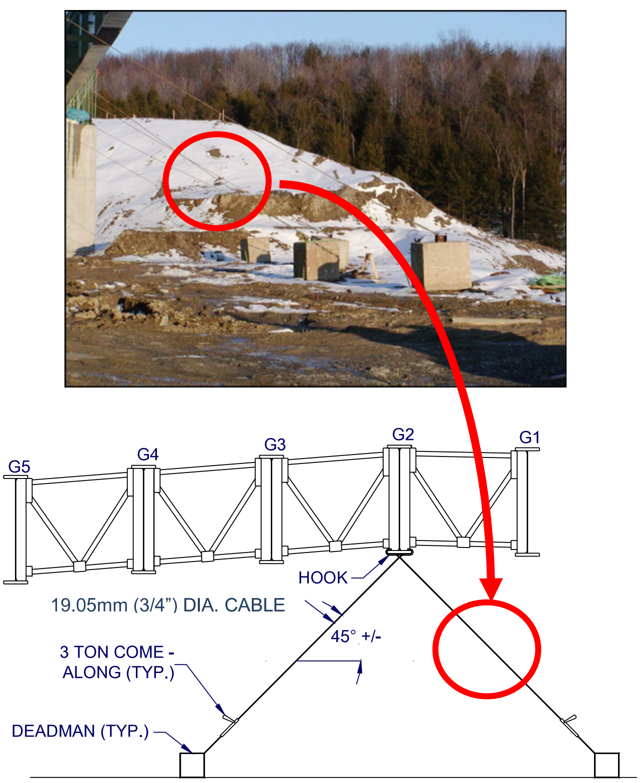

Fig. 3. Deadmen detail.

concrete to the desired elevation and cross-slope. Each consecutive placement step was performed within two days of the previous step. Parapets were added after the entire deck had achieved 28-day strength.

4. Field monitoring program

A total of 65 vibrating wire (VW) strain gages, four vibrating wire tiltmeters, and ten laser targets were used to monitor the bridge during construction. The majority of the instruments were placed on the exterior girders and on cross-frames identified as critical elements in the curved system from preliminary analyses [29]. Fig. 1 details the strain gage and tiltmeter locations.

Vibrating wire strain gages for Section C–C (Fig. 1) were installed on girders at the fabrication yard, and at the project site for other sections. Regardless of their location, the gages were spot welded to the girder top and bottom flanges 25 mm (1″) from their tips. The girders were blocked to their design camber with the webs vertical for fabrication yard installation and benchmark readings were taken using a portable readout box. The remaining top flange gages were installed while the girders were blocked at the project site. The bottom flange gages at Sections A–A and B–B (Fig. 1) were installed once the girders were in place along with the tiltmeters, located at the girder neutral axes, and the cross-frame gages, placed as close to the member shear center as possible. All strain readings were taken manually using the readout box until completion of girder erection, when a datalogger was installed. Datalogger readings were taken at half-hour intervals during placement of the deck steel and formwork and at five-minute intervals during concrete deck placement.

The global geometry of the bridge during construction was tracked using a laser measurement system [27]. The geometry was monitored after girder erection, deck steel placement and concrete deck placement.

5. Numerical program

Coupled with the examination of data generated during construction was a numerical program in which three models generated in SAP2000 were used for comparison against the field data. As stated earlier, the main intent of these studies was to assess the accuracy with which varying forms of grillage models, commonly used by practitioners for analyzing bridges, predicted the response of a curved structure during construction. For all three cases, the steel superstructure was modeled as a true grillage; however, in an attempt to more accurately represent the deck load distribution onto the steel superstructure, when the concrete deck was being placed it was modeled using shell elements. As a result, the final model could not be called a true grillage model as classically defined but will be referred to as a "modified" grillage model.

Three models were created, using both classical and "modified" grillages for certain stages, to model the entire construction process. The models are defined as follows:

- An analysis assuming the actual bridge geometry and documented construction procedures (Model A);
- An analysis assuming the actual bridge geometry and alternate construction procedures (Model B), and;
- An analysis assuming equivalent straight girder geometry and documented construction procedures (Model C).

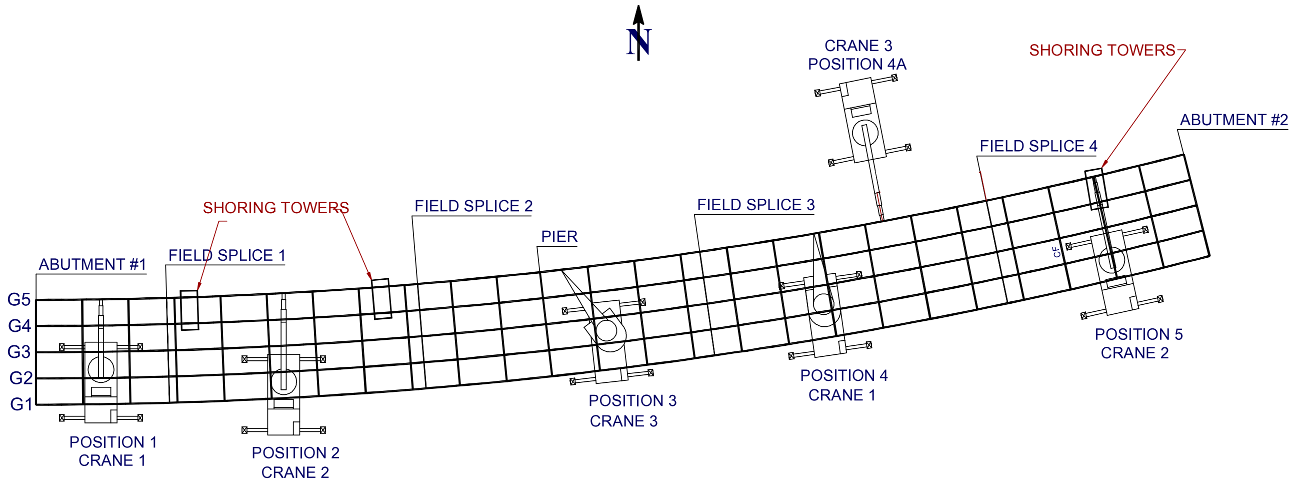

Fig. 4. Girder erection crane and shoring tower locations.

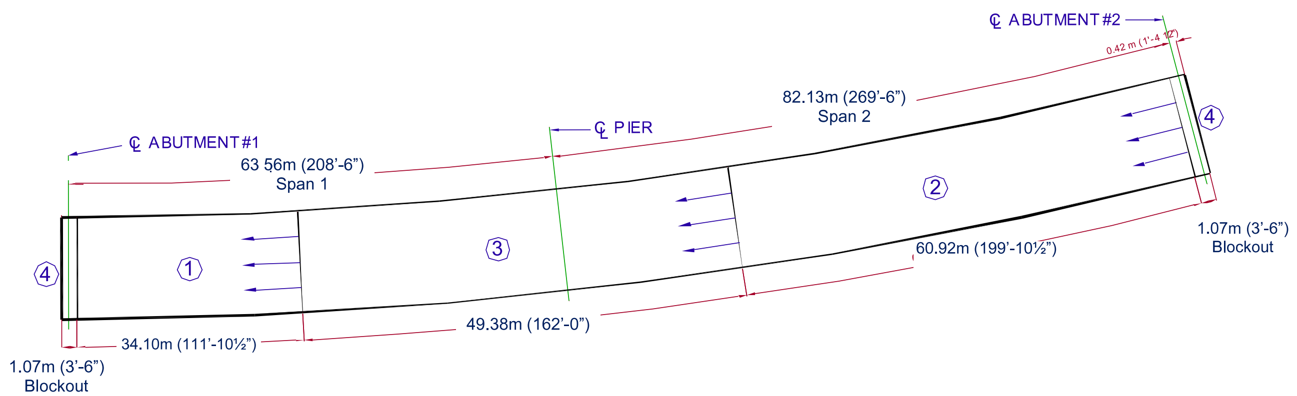

Fig. 5. Deck placement sequence.

Again, the main intent of the computational portion of this study was the examination of the accuracy with which grillage models employed by practitioners for many types of bridges, both during construction and under service loads, predicted the response of an actual structure.

All models were analyzed using geometrically nonlinear and staged construction capabilities available in SAP2000. Staged construction is modeled by calculating the deflections and internal forces after each stage and applying dead loads for additional stages to the previously deformed shape. Design support conditions at the pier and abutments were idealized and the SAP2000 self-weight function was used to apply all dead loads.

Nodes were placed at all cross-frame and flange transitions along the girder lines. The girders were modeled as straight SAP2000 frame members between these nodes, following common grillage modeling practice. All flange and web transitions were considered, as well as radial and tangential elevation changes. Intermediate stiffeners were not considered by the grillage model. The K-shaped cross-frames were modeled as straight frame members and assigned equivalent stiffness values, as discussed below. The geometric properties required for a typical cross-frame member included strong and weak axis moments of inertia, vertical and horizontal shear areas, torsional constants, and an equivalent area for self-weight calculations. The girder material response was assumed to be elastic.

Because no widely established procedure exists for estimating equivalent cross-frame properties for a grillage analysis, a basic procedure involving the analysis of a representative cross-frame was followed. A unit deflection was imposed at one end of the frame while the other end remained restrained against translation and rotation. Basic beam theory equations were used to estimate the equivalent strong axis moment of inertia for the frame. Since the vertical axis of a cross-frame coincided with the local strong (x) axis of the WT5 $\times$ 22.5 members, the equivalent frame weak axis moment of inertia was calculated by summing the member local strong axis moments of inertia. Similarly, the torsional constant for a representative cross-frame was determined by summing the WT member torsional constants. It was assumed that the diagonal cross-frame members predominantly resisted shear and, as a result, the frame vertical shear area was estimated using the diagonal member areas. The horizontal frame shear area was taken as the cross-sectional area of all plates lying in that plane (i.e. the areas of the WT stems).

No widely established approach exists for representing the concrete deck during construction with grillage models either, so for the "modified" grillage models utilized during deck placement the deck was modeled using shell elements with an average deck thickness and nominal material properties. Two types of shell element were used to model the deck. Wet concrete was modeled using a thick plate with pure plate characteristics and wet concrete density. The translational stiffness for these elements was reduced to zero to ensure non-composite behavior. Concrete that had started setting was modeled using both the thick plate elements and full shell elements having a density of zero. The concrete was assumed to remain elastic throughout construction, an assumption commonly made by practitioners when utilizing grillage models. At any location, these two element types could be superimposed to account for changing moduli and density during construction. The deck overhangs were modeled in a similar manner and included members representing overhang brackets when the concrete was

Table 2
Construction stages for Models A and C.

	Stage	Event
Girder erection	1	Addition of girders G4A-C, G5A-C, and cross-frames
	2	Addition of girders G3A-C and cross-frames
	3	Addition of girders G2A-C and cross-frames
	4	Addition of girders G1A-C and cross-frames
	5	Addition of girders G4D & E, G5D & E, and cross-frames
	6	Addition of girders G3D & E and cross-frames
	7	Addition of girders G2D & E and cross-frames
	8	Addition of girders G1D & E and cross-frames
Concrete deck and parapet placement	9	Addition of wet concrete over the positive moment region in Span 1 (thick plate elements)
	10	Addition of shell elements accounting for the stiffness of the deck over the positive moment region in Span 1 (full shell elements)
	11	Addition of wet concrete over the positive moment region in Span 2
	12	Addition of full shell elements over the positive moment region in Span 2
	13	Addition of wet concrete over the negative moment region
	14	Application of parapet dead load

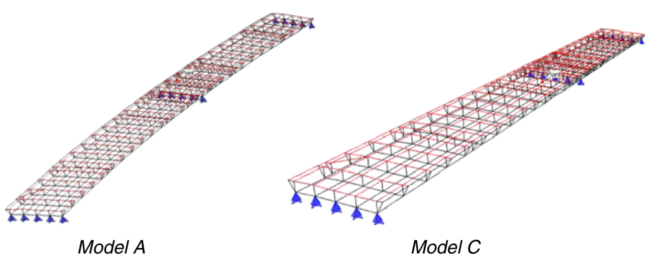

Fig. 6. Models A and C.

Table 3
Construction stages for Model B.

	Stage	Event
Girder erection	1	Addition of girders G1A-C, G2A-C, and cross-frames
	2	Addition of girders G3A-C and cross-frames
	3	Addition of girders G4A-C and cross-frames
	4	Addition of girders G5A-C and cross-frames
	5	Addition of girders G1D & E, G2D & E, and cross-frames
	6	Addition of girders G3D & E and cross-frames
	7	Addition of girders G4D & E and cross-frames
	8	Addition of girders G5D & E and cross-frames

considered wet. Composite action for set concrete was achieved by offsetting the shell elements above the girder frame members using stiff, weightless, rigid links. As was stated earlier, Model A (Fig. 6) examined the documented staged construction sequence. The analysis was completed using 14 stages, as listed in Table 2.

During each stage, corresponding frame and/or shell elements were added to the structure's deformed shape, and an analysis was performed for that stage. All relevant information (e.g. node deformations; frame and shell forces; support reactions) was retained from the previous stage. Relevant maximums and minimums for the sequence were also retained and used for modeling comparisons.

The construction procedure for Model B involved eight stages, as listed in Table 3. This model was developed to examine the effects of the erection procedure on the structure behavior, one of the objectives of the study. To examine the erection procedure effects, the results from this analysis were compared to those from Model A to determine which procedure resulted in the lowest final dead load deformations.

Model C (Fig. 6) considered the actual girder erection procedure (Table 2) but idealized the girders as straight chords between the substructure units, an approach that practitioners may consider when modeling curved structures with radii similar to the bridge studied here. The arc length for G3 was used for the length of all five girders in this analysis. The results were compared with the field data and the results from Model A.

Output was generated in the form of vertical and lateral bending moments and deformations. When necessary, moments were converted to vertical and lateral bending stresses at the flange tips for comparisons to the field-measured values. Basic bending equations were used to calculate the normal stresses at the gage locations using SAP2000 bending moments. V-load method [28,30] techniques were used to estimate the warping stresses. These techniques assume that the lateral bending moments used to calculate the warping normal stresses at the flange tips are found by representing the flange as a continuous beam supported at each cross-frame location. This multi-span beam is acted on by hypothetical uniform lateral loads represented by M_x/hR to estimate the warping stresses. The loads are converted into moments and divided by the flange's strong axis section modulus and, for rectangular flanges, Eq. (1) results:

$$\sigma_w = \frac{M_x d^2 \left(\frac{b_f}{2} - 1\right)}{hRt_f b_f^3}, \tag{1}$$

where

σ_w = warping normal stress;
M_x = vertical bending moment;
h = girder flange distance;
R = girder radius;
t_f = flange thickness;
b_f = flange width; and
d = cross-frame spacing.

The intent of selecting this equation matched the aforementioned theme of the numerical portion of the study, which involved ascertaining the effectiveness of approaches likely to be used by practitioners to determine important parameters when modeling curved steel structures using grillages.

6. Results

The presentation of the results and discussion are organized as follows: girder erection; deck placement; and erection sequencing. Each section includes an assessment of the actual bridge behavior and examines how accurately the numerical models predicted the response through a comparison between the predicted and measured values. All representative comparisons are presented for exterior and interior girders (G1 or G5) at either (a) a given instrumented cross-section (Fig. 1) or (b) cross-frame locations. Information is typically presented for the construction events outlined in Tables 2 and 3 with Event 8 being subdivided into Events 8(a) and 8(b). Event 8(a) represents girder erection prior to bolt tightening, while 8(b) represents girder erection after bolt tightening.

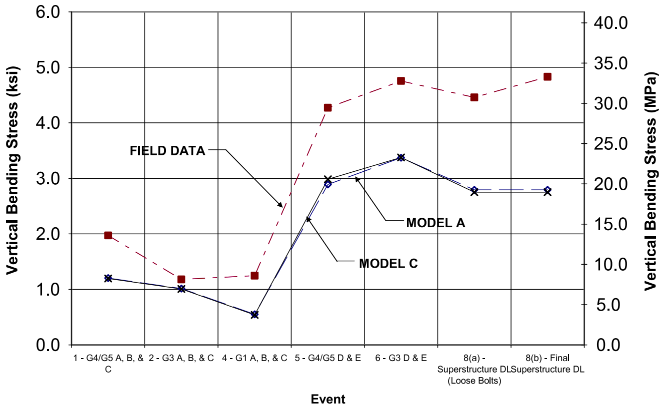

Fig. 7. G5, Section B–B top flange, vertical bending stress.

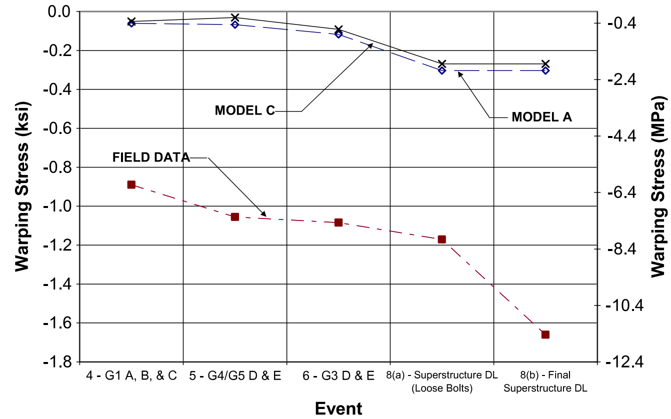

Fig. 8. G1, Section B–B bottom flange, North tip warping stress.

6.1. Girder erection

Comparisons during girder erection, which utilized classical grillage models of the superstructure, focused on the girder stresses developed at Sections B–B, and C–C (Fig. 1). Representative results for Section B–B vertical bending stresses are shown in Fig. 7, and the warping stresses are shown in Fig. 8. The vertical bending stress figures show the field data compared to Model A (includes curvature) and Model C (ignores curvature) stresses for the construction stages as detailed on the horizontal axis. The warping stress figures show the field data compared to numerical values from the SAP2000 models calculated using Eq. (1). The field-measured warping normal stresses were calculated from the measured strains by: (1) converting the measured flange strains to stresses using the modulus of elasticity for steel; (2) assuming a linear stress distribution across the flange width; (3) averaging the measured stresses at the flange tips to determine the vertical bending stress; and (4) subtracting this average stress from each tip stress to obtain the warping normal stress.

Fig. 7 shows that appreciable vertical bending stresses, which approached 68.9 MPa (10 ksi) at certain locations, were induced during girder erection. The field data also indicated that, depending upon the location, bolt tightening had a measurable effect on the vertical bending stress levels with stresses changing by upwards of 17.2 MPa (2.5 ksi). The SAP models predicted the trends well for the vertical bending stresses induced during girder erection. Differences of varying magnitude existed between the vertical bending stress predictions for Models A and C at certain sections while good agreement existed at other sections. In addition, the error magnitudes differed significantly from girder to girder and construction event to construction event. It was surmised that load sharing between girders was not completely accounted for by the models since the G1 stresses were generally overestimated while the G5 stresses were generally underestimated. One cause for these errors could be inadequate representation of the cross-frames' load-sharing capabilities between girders in a grillage model. While the approach adopted herein to establish equivalent cross-frame flexural, shear and torsional stiffnesses follows fundamentally sound techniques that would be likely to be adopted by a practitioner, the inherent interactions between various stiffness components in a cross-frame (e.g. axial and flexure, axial and shear) along with accurate representations of the connection conditions and subsequent load transfers between cross-frames and girders are always approximated when a grillage model is selected. While these simplifications may have minor consequences when a straight bridge is being modeled, they become considerably more important for curved structures where cross-frames are considered as one of the primary load-carrying components.

The warping stresses, calculated using Eq. (1) and compared as shown in Fig. 8, showed poorer model predictions of the field trends and magnitudes than was evident for the vertical bending stresses. The field-measured stresses approached 55.1 MPa (8 ksi) but generally were around 20.7 MPa (3 ksi), and they changed somewhat erratically between construction events. Lack of agreement between the field data and predicted models could be caused by a number of factors, including various effects not mimicked in the models during construction, such as bolt tightening and temperature change, and, as discussed earlier, the methods used to approximate the cross-frame properties in the models. In addition, the simplified approach followed when calculating the warping stresses using Eq. (1) could certainly have an influence. However, it should be reiterated that the focus of this study was to examine the effectiveness with which analysis approaches likely to be selected by practitioners predicted the actual response.

6.2. Deck placement

Comparisons during deck placement examined the actual structure response and the accuracy of the "modified" grillage models detailed previously. They focused on the girder stresses and girder vertical displacements.

Stress comparisons were performed for Sections A–A, B–B, and C–C (Fig. 1), and representative results are shown for Section C–C. The figures show the field data compared to Model A and C stresses for selected deck placement stages, starting with Stage 8(b). All values were stress changes relative to Stage 8(b), so that the model's effectiveness for predicting the deck placement response could be studied separately from girder erection.

The girder vertical bending stress comparisons during deck placement, as detailed in Fig. 9, again demonstrated good trend prediction for both Models A and C. The predictions for G1 and G5 at this section were generally non-conservative. However, the predictions for other girders at Section C–C and for all girders at Sections A–A and B–B were generally in good agreement or only slightly non-conservative when compared to the field data. For this structure, including the horizontal curvature effects appeared to have a minimal influence on the model accuracy for predicting the vertical bending stresses. The conservative nature of the model predictions when compared to the field data could be attributed to a number of items, including the boundary conditions selected for the girders and the method selected for representing the composite concrete deck as it was being placed and was setting using the "modified" grillages. More than likely the procedure used to replicate hardening of the deck, which involved varying the moduli in the deck shell elements, did not explicitly match how the concrete hardened in the field. Again, the emphasis of this portion of the study was to ascertain the effectiveness of the approaches for modeling curved structure construction response grillage models that practitioners may employ.

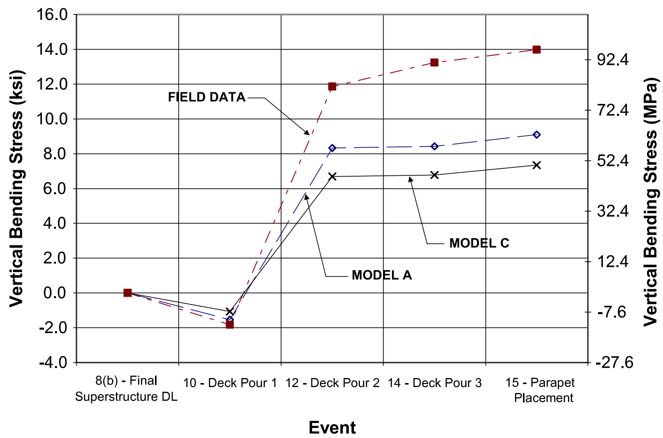

Fig. 9. G1, Section C–C bottom flange, vertical bending stress.

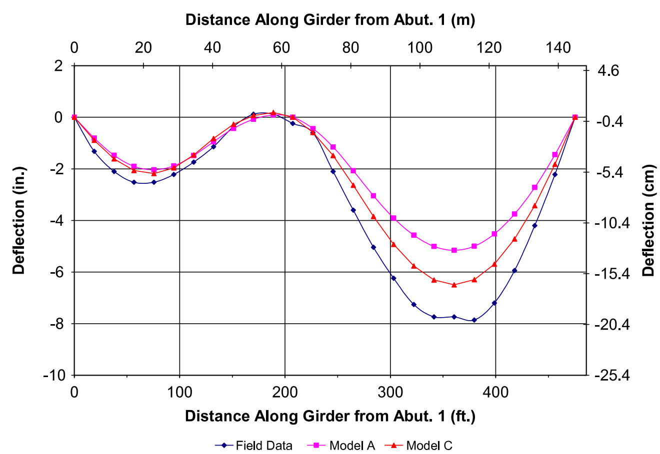

Fig. 10. G5 vertical displacements, completion of deck placement.

The girder warping normal stress magnitudes induced during deck placement were generally an order of magnitude less than the vertical bending stress at a given instrumented section. Therefore, comparisons between the field data and the model predictions are not presented. While having relatively small warping normal stresses induced during the deck pour is surprising, the use of deadmen coupled with the pour sequence, which allowed for composite action to be initiated for a completed deck section prior to initiating the next section pour, contributed to reducing the warping normal stresses.

Measurements of the vertical displacements, using the laser system, were taken at each cross-frame location and compared to the numerical predictions. Comparisons are shown at the completion of deck placement (Step 13) in Fig. 10. This figure shows that Models A and C under-predicted the measured vertical displacements at the completion of deck placement. The levels of non-conservatism varied between girders but differed from the field data by 28% on average and indicate that, as discussed previously, the modeling technique employed for the deck did not track the actual hardening effectively and resulted in model over-stiffening.

6.3. Erection sequence study

A final component of the numerical work was the completion of a small erection sequencing study using Models A and B. Model A mimicked the actual procedure, erecting single girders from interior to exterior radii (interior-to-exterior), while Model B erected the largest radius girder first and proceeded to the smallest radius girder (exterior-to-interior). The vertical displacements from Models A and B at the completion of girder erection were compared. Fig. 11 plots these displacements for G5.

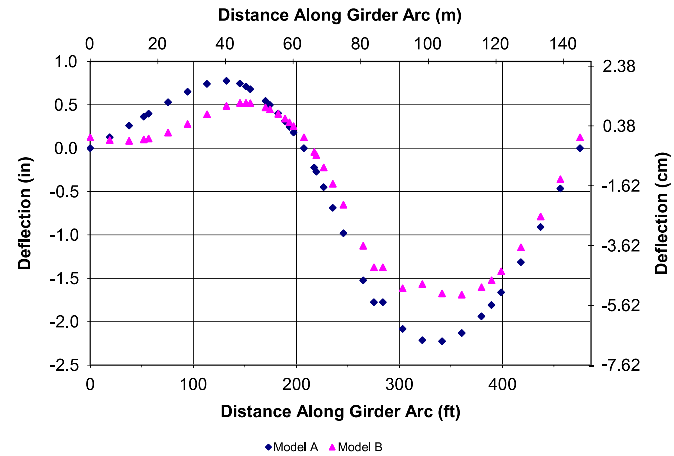

Fig. 11. G5 vertical displacements, Model A vs. Model B, completion of girder erection.

The figure indicates that the exterior-to-interior girder erection procedure (Model B) produced smaller vertical deflections for G1 and G5. These results are consistent for all five girders at the completion of girder erection and match those discussed by Bell [25].

7. Conclusions

Results from a coupled field monitoring and numerical study examining the effects of construction on the stresses and deformations in a single large radius, horizontally curved, steel, plate girder bridge were presented. Examinations of the stresses and deformations that developed during construction were performed separately for girder erection and deck placement, and were used to: (1) track the actual changes in measured quantities as construction progressed; and (2) examine the accuracy with which both classical and "modified" grillage models, constructed and examined using techniques followed by practitioners, predicted the measured behavior. An additional small numerical study that examined superstructure erection sequencing decision effects on structure deformations was also completed.

During girder erection, the results indicated that:

- Vertical bending stresses approaching 68.9 MPa (10 ksi) were induced into the girders. Bolt tightening had a measurable effect on the vertical bending stress levels, with stresses changing by upwards of 17.2 MPa (2.5 ksi).
- Grillage models predicted the vertical bending stress trends well, with actual magnitude accuracy levels varying between instrumented sections and construction events. While the predictions were generally good, it was noticed that load sharing between girders was not adequately accounted for in the grillage models, with the interior girder (G1) stresses being generally overestimated and the G5 stresses being generally underestimated. Causes for the inadequate load sharing were attributed to the generally accepted approximate techniques used to represent cross-frames in a grillage model that, while appropriate for straight bridges, may lead to errors similar to those found here when attempting to predict stresses in curved structures.
- The measured warping stresses varied between construction events. The maximum warping stress magnitudes were generally around 20.7 MPa (3 ksi); however, some locations had maximums approaching 55.1 MPa (8 ksi). The influence of bolt tightening was evident to varying degrees in the recorded field data.

- Grillage models did a poorer job predicting the measured warping stress trends and magnitudes than vertical stress trends and magnitudes. These errors were largely attributed to model construction techniques, which were based on standard approaches that practitioners would select when constructing a grillage model, and the method used to determine the warping stresses from the model output, which was, again, selected following a simplified approach that many practitioners may select.
- Including the effects of curvature appeared to have a minimal effect on the grillage model accuracy for the girder vertical bending stresses for the large radius bridge that was studied. Estimation of the girder warping stresses could not occur unless curvature was included in the model.

During deck placement, the results indicated that:

- The measured girder warping stresses were generally an order of magnitude less than the measured vertical bending stresses and were attributed to the selected deck pour procedure coupled with the restraint provided to the system by temporary construction supports.
- "Modified" grillage models, which included shell elements of varying thickness in an attempt to more accurately model deck placement, demonstrated good vertical bending stress magnitude and trend prediction.
- "Modified" grillage models gave unconservative predictions of the deformations at the completion of construction for the structure that was examined. Errors were attributed to the method used to mimic the hardening of the deck, which utilized a varying modulus. These results demonstrated the sensitivity that model results can have to variations in material properties.
- Again, including the effects of curvature appeared to have a minimal effect on the "modified" grillage model accuracy for predicting the vertical bending stresses. Curvature would have to be incorporated into the model to estimate the warping stresses.

The erection sequencing study indicated that:

- An exterior-to-interior girder erection procedure produced smaller vertical deflections when compared to the interior-to-exterior erection procedure used for the actual structure.

This study indicates that, for the bridge that was examined and the modeling techniques that were selected, the use of a grillage modeling technique coupled with other commonly applied simplifications to predict curved bridge construction response should be approached thoughtfully and with a thorough understanding of grillage model limitations.

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