# Erection Procedure Effects on Deformations and Stresses in a Large-Radius, Horizontally Curved, I-Girder Bridge

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**Abstract:** Special attention is required in the construction of horizontally curved steel I-girder bridges due to coupled effects of primary bending and torsional forces. Misguided steel erection procedures can lead to undesired stresses, deflections, and rotations in these types of bridges, resulting in a structure with misaligned geometry and in an unknown state of stress. Further complicating the issue, little guidance related to curved bridge behavior during construction is provided by current design codes, leaving contractors and designers uncertain as to the most appropriate steps to take to achieve an efficient, safe structure. A horizontally curved, six-span steel I-girder bridge located in central Pennsylvania that experienced severe geometric misalignments and fit-up complications during steel erection was studied to investigate curved girder behavior during construction. The structure was monitored during corrective procedures intended to realign it with the design geometry, and field data used to calibrate a three-dimensional computer model generated via SAP2000. The techniques and assumptions proven in the calibration process were used to create a numerical model of a three-span continuous portion of the bridge, which was the subject of several analyses exploring the effects erection sequencing, implementation of upper lateral bracing, and use of temporary supports had on the final deformed shape of the curved superstructure. Findings indicated that using paired girder erection produced smaller radial and vertical deformations than single girder techniques for this structure, and that the use of lateral bracing between the fascia and adjacent interior girders and the placement of temporary shoring towers at span quarter points are both effective means of further reducing levels of deflection.

**DOI:** 10.1061/(ASCE)1084-0702(2007)12:4(467)

**CE Database subject headings:** Bridge construction; Curved beams; Plates; Bridges, girder; Deformation.

## Introduction

The use of horizontally curved steel girder bridges has been on the rise over the past 50 years due to environmental demands and alignment restrictions. Despite performing well in service, they are inherently more susceptible to instability during construction than their straight girder counterparts, and are prone to erection complications due to their distinct behavioral tendencies and three-dimensional stress interactions. The specific sequence of erection, the number of girders erected during a construction stage and a host of other factors can have a significant effect on bridge behavior or constructability. Without adequate bracing and shoring, torsion, warping and second-order deformations can lead to structural deficiencies ranging from misalignment of members to premature yielding of the flange tips. To complicate the issue, no specifications currently exist that provide quantitative criteria addressing curved bridge construction, and with current designs resulting in structures with longer span lengths and tighter radii, it is likely that construction problems will continue to occur without guidelines being provided to designers and erectors. Therefore, any research that attempts to provide such guidance to practitioners and contractors regarding how curved bridges can be designed and erected in a fashion that reduces excessive deflections and mitigates costly delays is of importance. Past published efforts, which contain extensive references relevant to this topic that are not discussed herein, have largely been qualitative in nature, or focused on a specific behavioral tendency of a single girder with well defined boundary conditions. Several studies involved field monitoring techniques to investigate curved girder behavior (Galambos et al. 1996; Chavel and Earls 2002; Pi et al. 2000), while others focused on analytical studies to explore response to a certain phenomenon, (Huang 1996; Bradford et al. 2001; Schelling et al. 1989; Davidson et al. 1996; Sennah et al.), while still others conducted laboratory tests (Linzell, 1999, 2000, 2001; Zureick et al. 1994, 2000). All of these provide valuable information to better understand the behavior of curved girder bridges, but there is so much more that needs to be done. A need for studies related to the systemic response of girders interacting within a complex, dynamic system of varying stiffness, configuration, and load exists so that, in the future, more quantitative criteria can be developed.

This project stemmed from complications that arose during construction of a horizontally curved steel I-girder bridge in central Pennsylvania. The structure experienced undesirable deformations during erection, which had to be remedied before construction could continue. Field data were acquired as procedures were executed to realign the superstructure and served as a means of calibration for computer models created to investigate the effects differing erection methods had on the response of the framing system. Specifically, these numerical studies were used to

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Note. Discussion open until December 1, 2007. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on May 31, 2005; approved on July 6, 2006. This paper is part of the ***Journal of Bridge Engineering***, Vol. 12, No. 4, July 1, 2007. ©ASCE, ISSN 1084-0702/2007/4-467–476/$25.00.

**Table 1.** Girder Plate Dimension Ranges

| | Top FLG plate | | Web plate | | Bottom FLG plate | |
|---|---|---|---|---|---|---|
| Girder | Width [mm (in.)] | Thickness [mm (in.)] | Depth [mm (in.)] | Thickness [mm (in.)] | Width [mm (in.)] | Thickness [mm (in.)] |
| G1 | 508–889 (20–35) | 38.1–1,01.3 (1.5–4) | 3,200 (126) | 20.6 (0.8125) | 660.4–1,092.2 (26–43) | 44.5–76.2 (1.75–3) |
| G2 | 431.8–863.6 (17–34) | 25.4–76.2 (1–3) | 3,200 (126) | 20.6 (0.8125) | 457.2–1,016 (18–40) | 38.1–76.2 (1.5–3) |
| G3 | 406.4–711.2 (16–28) | 25.4–76.2 (1–3) | 3,200 (126) | 20.6 (0.8125) | 457.2–863.6 (18–34) | 31.8–76.2 (1.25–3) |
| G4 | 406.4–711.2 (16–28) | 25.4–76.2 (1–3) | 3,200 (126) | 20.6 (0.8125) | 457.2–863.6 (18–34) | 31.8–76.2 (1.25–3) |
| G5 | 431.8–863.6 (17–34) | 25.4–76.2 (1–3) | 3,200 (126) | 20.6 (0.8125) | 558.8–1,016 (22–40) | 31.8–76.2 (1.25–3) |

examine the ramifications erection sequences, implementation of temporary support towers, and the use of lateral bracing had on the behavior and final geometry of the curved superstructure.

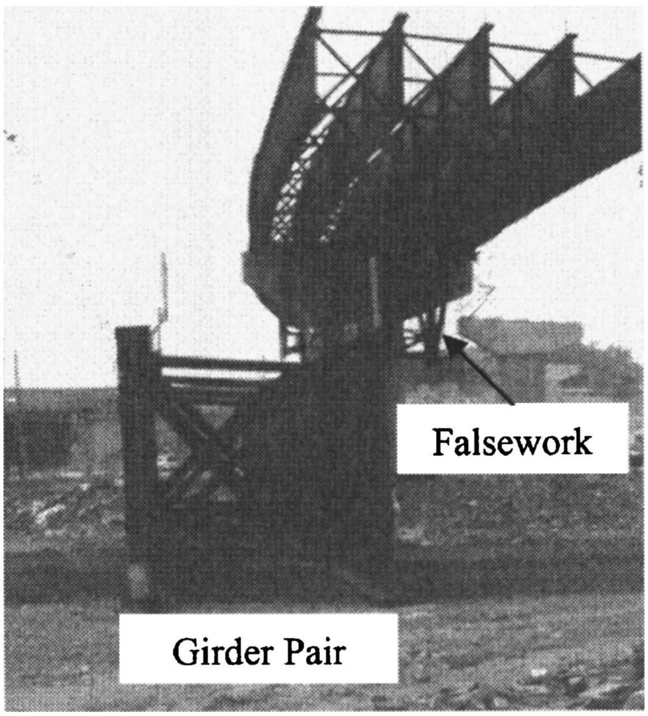


**Fig. 1.** Girder framing and preassembled girder pair, Structure No. 7

## Structure Description

The focus of this research was Structure No. 7A, which is one of two side-by-side horizontally curved, composite, steel I-girder bridges constructed at an interchange in central Pennsylvania. It is a six-span structure whose cross section consists of five singly symmetric plate girders spaced at 2.97 m (9.75 ft) with radii varying from 585.3 to 597.2 m (1,920–1,959 ft). The girders are composed of stiffened web plates with a constant depth and thickness, and flange plates of varying dimensions. Exact web plate sizes and flange plate dimension ranges are provided in Table 1. Girders are braced radially using cross frames made up of WT sections, and no lateral bracing system was included in the original construction plans. A representative photo of the superstructure is shown in Fig. 1.

The complete structure consists of two three-span continuous units spanning a total distance of 530.1 m (1,739 ft) along the roadway's construction line. The eastern of these, which is the focus of this manuscript, is composed of spans designated Spans 4, 5, and 6. Relevant geometric information for these spans is found in Fig. 2.

## Erection Procedure

A construction plan was prepared by the original contractor that called for erection of girders in pairs, with the single girder line having the highest radius of curvature being placed after the other four girders lines (two pairs) had been erected in a given span.

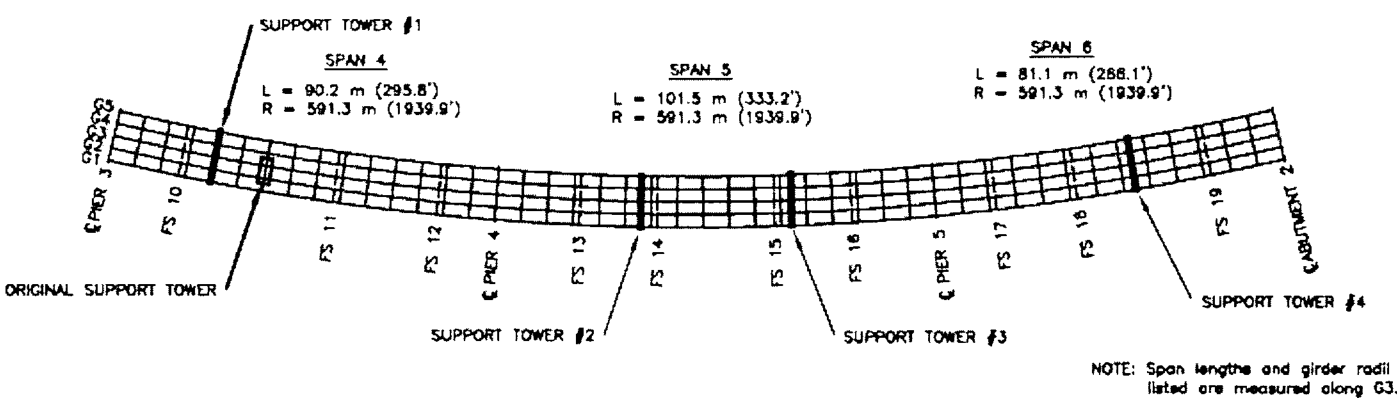


**Fig. 2.** Framing plan of Spans 4–6 detailing girder numbers, substructure units and field splice, and support tower location

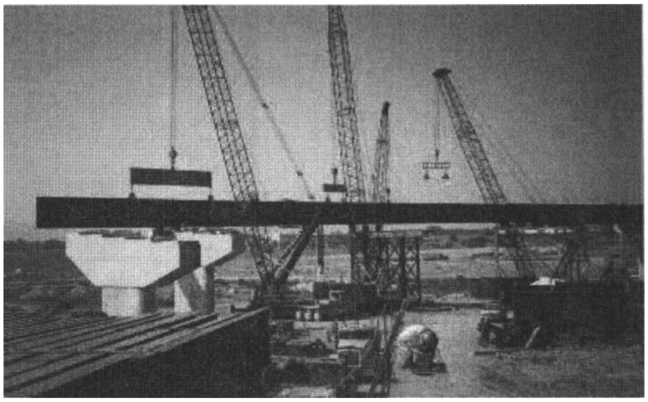

**Fig. 3.** G1 erection

Each girder pair was preassembled on the ground with cross-frames fully bolted and snug tightened, and the units were then raised incrementally from splice to splice.

Erection was initiated with placement of Span 4 steel, beginning at Pier 4 and working toward Pier 3 (Fig. 2). Prior to the erection phase, falsework (Fig. 1) was mounted to Pier 4 to support hydraulic jacks that stabilized and adjusted the system during construction to achieve the design "no-load" geometry prior to deck placement. Originally, construction was to be accomplished using four cranes and no temporary shoring towers would be employed.

Halfway through erection of Span 4, twisting of G2 and G3 caused undesired deformations and made it impossible to continue without revising construction measures and/or adding bracing. The proposed solution included implementing a single support tower in Span 4, which was placed approximately 36 m (118.11 ft) east of Pier 3 ("original support tower" in Fig. 2) and would allow cranes to be released to resume erection. Complications continued to occur during the erection procedure, and upon completion of Span 4 and a cantilevered portion of Span 5, the superstructure was surveyed and shown to be severely out of lateral alignment from the intended geometry, with horizontal misalignments of 274 mm (10.8 in.) at field splice No. 11 in Span 4 and 351 mm (13.8 in.) at field splice No. 14.

A second contractor was recruited to realign the superstructure and complete the erection of the remaining spans. Once the design geometry of the erected portion of the bridge was attained, upper lateral bracing was inserted between the fascia and first interior girders, and steel placement continued using a revised plan that called for the erection of single girder lines rather than girder pairs, placing the girder line with the largest radius of curvature first and working inward (Fig. 3). Furthermore, the revised scheme required the implementation of temporary support towers in all spans.

## Field Monitoring Program

A limited field monitoring program was instituted during construction of Structure 7A. Collected data were used in conjunction with field survey information to improve the accuracy of numerical models created to examine erection procedure effects. Data were collected during two phases of steel erection: (1) realignment of the previously erected portion in Span 4 and a portion of Span 5; and (2) completion of steel erection in Spans 5 and 6. Field survey data tracking deformations at various stages throughout both processes were collected by a third party.

Instrumentation locations were established using results from grillage models developed in SAP2000. It was intended that selected locations provided consistent readings as support conditions changed during the anticipated corrective procedure and the continuation of erection. Therefore, demountable strain transducers were placed near the flange tips of G1, G2, and G5 approximately 45.72 m (150 ft) east of Pier 3 to monitor positive vertical and lateral bending experienced during the corrective procedure, and on G1 and G5 over Pier 4 to monitor negative bending. Transducers were also mounted near cross frame member neutral axes between G1 and G2 at these positions to pick up load sharing between the girders. A total of 16 demountable strain transducers were used to measure system response during the realignment of the superstructure. Twelve vibrating wire strain gauges were used to monitor steel erection in Spans 5 and 6 and were placed on the flange tips of G3 and G5 over Pier 5, and on the tips of G1 and G5 approximately 50.29 m (165 ft) east of Pier 5 in Span 6. Instrument locations are detailed in Fig. 4.

A total of 22 data scans were taken during the realignment of the partially erected structure, with each scan correlating to a specific event prior to, during, and after corrective measures were taken. A detailed summary of these readings and their corresponding events is found elsewhere (Bell 2004).

Vibrating wire strain gauge data were collected with the aid of a portable readout box. Baseline readings were obtained immediately following gage installation onto the girder flanges, which occurred while girder segments were stored in a web-plumb position at the bridge site. Additional readings were recorded at set instances during the remaining erection process.

## Numerical Program

A series of three-dimensional models of Structure No. 7A were generated using SAP2000 to explore the effects steel erection practices had on the behavior of the superstructure. SAP2000 was selected due to its common use as a bridge analysis tool by practitioners and to its recent incorporation of modules that would theoretically permit "erection" of Structure No. 7A numerically.

Quadrilateral shell elements were used to define girder flange and web plates with frame elements representing stiffeners, cross-frame, and bracing members. Shell elements terminated at all stiffener, plate transition, field splice, and support locations, as well as at some intermediate locations along the girder web to maintain acceptable aspect ratios. All elements were defined using nominal geometric and material properties, and analyses incorporated the limited higher-order deformation capabilities available in SAP2000.

Two numerical model types were developed. The first contained approximately 40,000 degrees of freedom (Fig. 5) to model the corrective procedure and was used for numerical accuracy verification and calibration. The second utilized the same modeling techniques as the calibrated model and investigated various erection sequences for the three-span section of the structure that was studied (Spans 4–6), and contained approximately 85,000 degrees of freedom.

### *Calibration*

Top flange radial and vertical displacements served as the primary basis for model accuracy verification since stations, offsets, and elevations were known at girder splices both prior to and after the

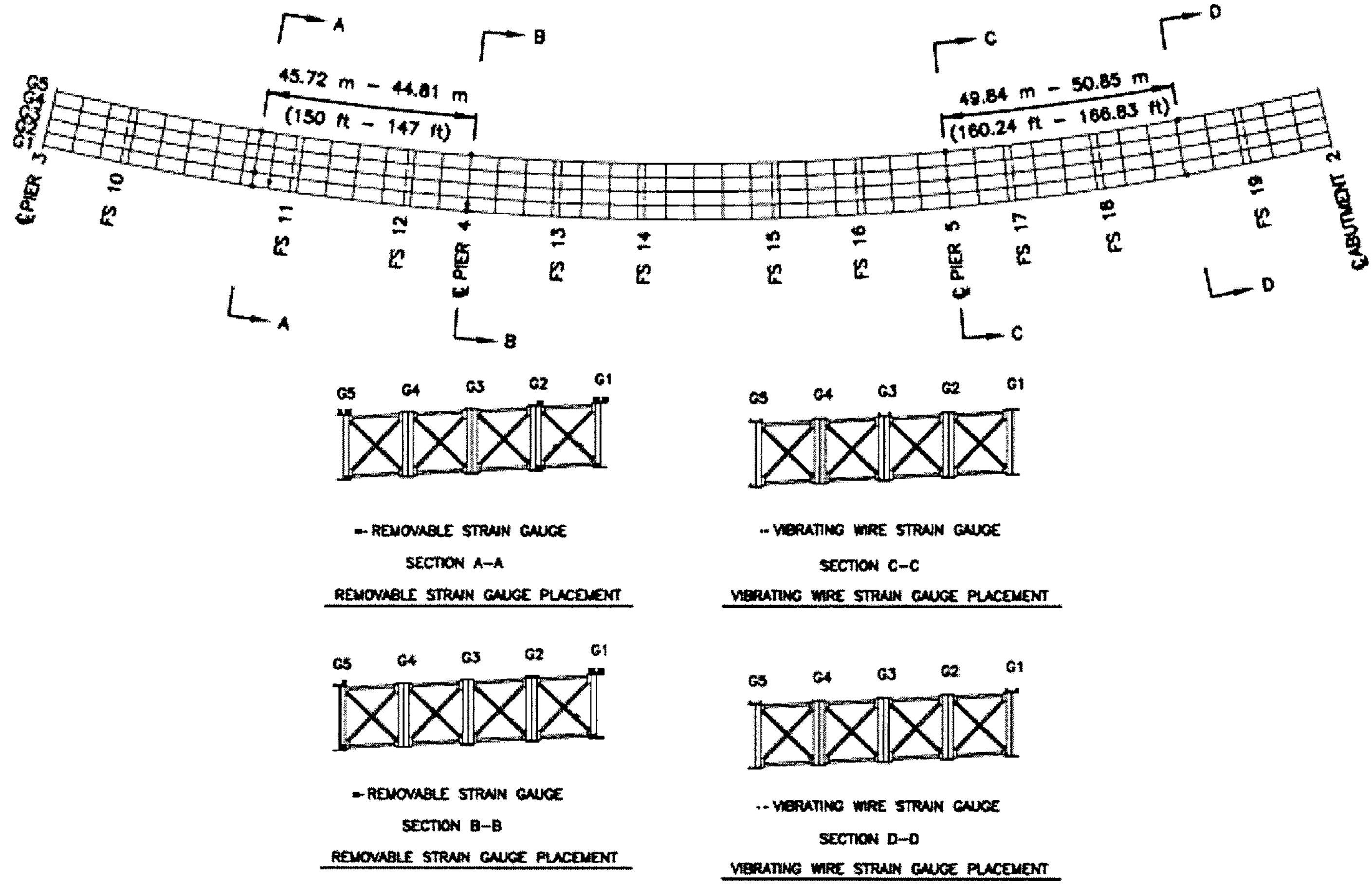


**Fig. 4.** Instrumentation details

corrective procedure. Curve fitting techniques were employed and used these points of known orientation to develop geometrics along the entire length of the girders, aiding in model development and verification processes. The model was created to match the misaligned geometry and subjected to a series of joint displacements that reconstructed the final elevations and offsets obtained via the corrective procedure. This was accomplished by imposing deformations at field splice locations equal to the difference between the misaligned condition and the corrected geometry as reported in surveyor information. Boundary conditions at some locations were modified to improve the agreement between measured and predicted values with the final model containing modified support conditions at Piers 4 and 5. This was necessary to account for construction tolerances that resulted in a restraint behaving differently than what was assumed in the numerical model. Final comparisons between measured and predicted deformations at the completion of the corrective procedure are shown in Figs. 6 and 7 for G5, the girder with the largest observed differences. On average, calculated values were within 20% of measured radial and vertical values, with the largest differences being 0.4 in. (10.2 mm) vertically and 2.4 in. (61.0 mm) radially. These percentage differences are higher than would normally be acceptable for model calibration, however, they were deemed acceptable for this study due to the relative coarseness of the field data used for calibration and to the small magnitudes being measured for certain deformation quantities. In addition,

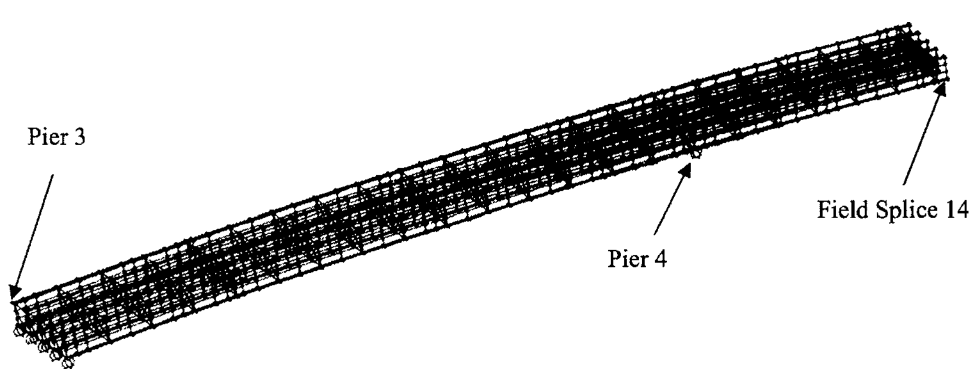


**Fig. 5.** Calibration model

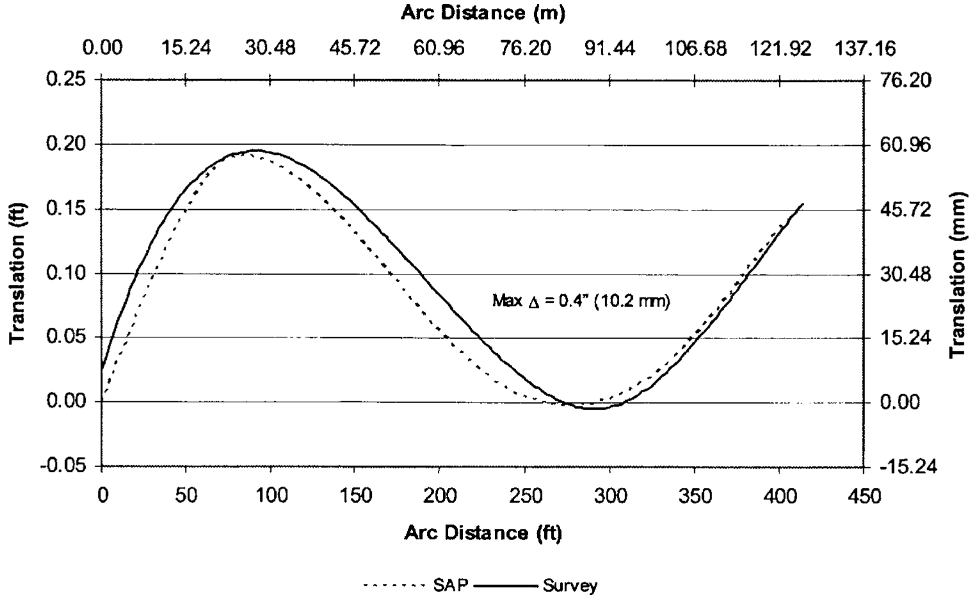


**Fig. 6.** G5 vertical displacements, corrective procedure completion

SAP2000 could impose a limited set of boundary conditions, either: (1) completely constraining certain degrees of freedom; (2) imposing prescribed displacements; or (3) imposing linear springs. While linear spring conditions may have been desirable for certain boundary conditions to improve agreement, not enough field data were available to accurately establish their stiffness.

Stresses calculated from the strain transducers were used as a secondary means of numerical model verification. Agreement was generally good, except in the negative moment region over the pier where stresses reported from SAP 2000 were significantly lower than those obtained in the field. These differences were attributed to localized influences from the bearings on field data and on modeling the bearing stiffeners using frame members, which ignored any projected width of, and subsequent restraint provided by, the stiffener to the girder flange. On average, stresses predicted by SAP 2000 were within 10% of field data when gauges at Pier 4 were excluded from the comparisons.

### Erection Studies

Erection studies were initiated with the creation of a model of Spans 4, 5, and 6 using the techniques and assumptions proven through the calibration process. This model numerically "erected" the steel superstructure following a series of prescribed erection schemes to study girder response. Deformations induced by each procedure were documented to assist with identifying beneficial procedures for constructing Structure No. 7A.

Evaluation of numerical model accuracy continued as the erection studies were being completed. The continuation of calibration into the erection study phase was instituted for two reasons: (1) vibrating wire strain gauge data were available for girder seg-

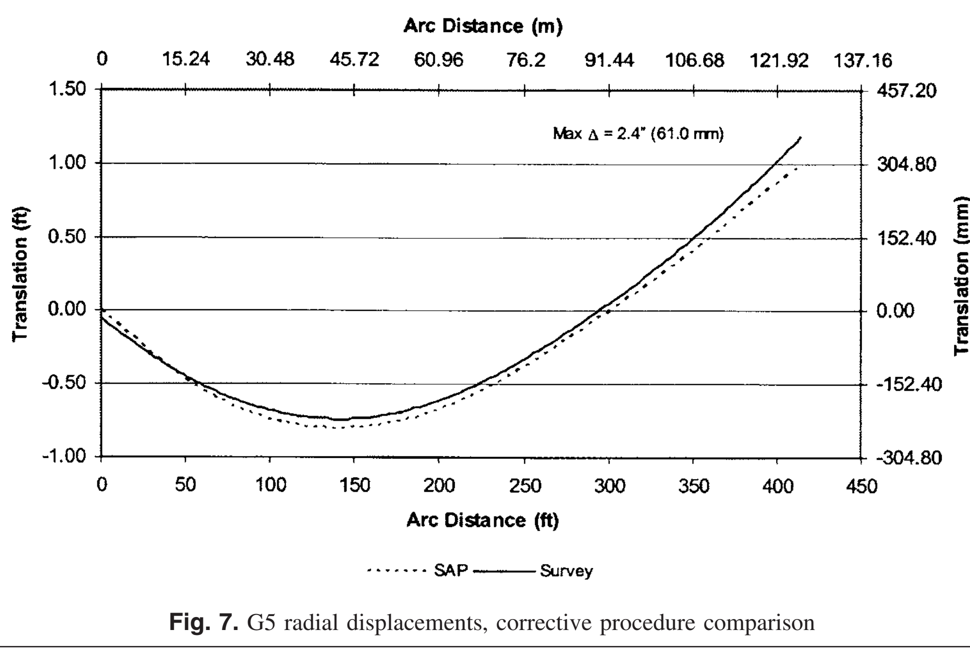


**Fig. 7.** G5 radial displacements, corrective procedure comparison

ments over Pier 5 and in Span 6; and (2) SAP2000 sequential analysis capabilities accounted for additive stiffness and weight contributed by "erecting" structural elements, but did not connect those elements to the deformed shape of the structure. Rather, the added field sections were placed at their undeformed locations, providing a rigid offset to previously constructed and deformed elements of the bridge. These discontinuities, located at each field splice, necessitated a revision to the method serving as the fundamental means of field section placement in the models. To minimize effects the rigid offsets had on the presentation and evaluation of numerical results, it was decided to examine the influence that various erection procedures had if entire girder lines, or pairs of girder lines, were erected prior to initiating the placement of any portion of another girder line or girder pair. This translated into placement of an entire girder line or girder pair piece by piece for the entirety of Spans 4, 5, and 6 before any section of the adjacent girder line, or pair of girder lines, was "erected." While this approach may not be practical for all structures, comparing the effects of using various placement techniques following this sequence did provide insight into the influence of select parameters on system response. Comparisons between recorded stresses and those produced numerically were of the same order of magnitude, so this procedure was deemed acceptable.

The erection studies examined the influence of single girder erection, erection of girders in pairs, use of temporary supports, and the inclusion of lateral bracing on the response of the 7A superstructure. Single and paired girder erection practices examined placing either (1) all girders individually; or (2) two girder pairs and a single girder into Spans 4–6. Both construction from inner-to-outer and outer-to-inner girder radii were examined for the single and paired girder studies, and comparisons between deformation resulting from the two techniques (single versus pair) and two placement orders (inner to outer and outer to inner) were made. No temporary supports or lateral bracing were included in any of these analyses.

Examination of the influence of temporary supports and the inclusion of lateral bracing on construction response both focused on a girder pair erection sequence method that erected Spans 4–6 from inner-to-outer radii of curvature. For the temporary support study, vertical translational restraint was imposed at the quarter points of each of the three spans for all five girder lines and the resulting influence on deformations was examined. The lateral bracing study utilized upper lateral bracing placed between both fascia and first interior girders in all three spans. Modeled bracing members mimicked the 19 mm (3/4 in.) diameter steel cables used by the second contractor to stabilize the system prior to completing erection.

## Results

As previously mentioned, erection studies were compared to assess which method produced the smallest deformations, with an emphasis on radial and vertical components. Comparisons were made at the midpoint of the top flange and focused on the fascia girders (G1 and G5).

### *Single Girder Erection*

For the inner-to-outer single girder erection sequence, the inner girder (G5) generally experienced larger overall deformations, as summarized in Table 2. The table does indicate slightly higher upward vertical deflections in some spans for G1, which can be attributed to its larger arcspan and increased dead load, but generally G5 experienced larger cumulative deformations. Being the first girder erected, there were no other elements that could aid in stabilization and load sharing, and G5 alone had to resist all lateral and primary forces. However, subsequent girder lines added experienced smaller deformations due to the resistance provided by a system connected by crossframes consisting of the current piece being placed and previously erected elements, and the influence of this system response on section deformation was clearly
evident.

Similar results were obtained for the outer-to-inner single girder erection sequence, with G1 experiencing more overall deflection than G5 at completion of erection of Spans 4–6 (Table 3).

Explanations for outer girder deformations being larger than those for the inner girder are similar to those for the previous method that was examined. The outer girder lacked restraint from external sources other than holding cranes, and although G5 had the tightest radius, it benefited from load sharing with the other four girders.

Comparisons between the two single-girder erection procedures were used to establish a preferred single-girder erection method for Structure No. 7A, and was accomplished by comparing deformations of the first and last girder lines placed for each method. The writers felt this type of comparison was necessary so fair examination of the influence of the procedures on superstruc-

**Table 2.** Maximum Fascia Girder Deflections, Inner-to-Outer Single Girder Procedure

| Inner-to-outer single girder erection | | | | |
|---|---|---|---|---|
| | (a) G1 (outer) | | | |
| | Radial | | Vertical | |
| Span | (in.) | (mm) | (in.) | (mm) |
| 4 | 1.0 | 25.9 | −6.6 | −167.9 |
| 5 | 0.1 | 3.8 | 0.9 | 23.9 |
| 6 | 0.6 | 14.2 | −4.3 | −110.2 |
| | (b) G5 (inner) | | | |
| 4 | 11.2 | 285.5 | −6.1 | −153.9 |
| 5 | 3.2 | 80.3 | −1.9 | −49.3 |
| 6 | 3.7 | 93.0 | −3.1 | −78.7 |

**Table 3.** Maximum Fascia Girder Deflections, Outer-to-Inner Single Girder Procedure

| Outer-to-inner single girder erection | | | | |
|---|---|---|---|---|
| | (a) Girder 1 (outside grd) | | | |
| | Radial | | Vertical | |
| Span | (in.) | (mm) | (in.) | (mm) |
| 4 | 4.6 | 117.1 | −11.5 | −290.8 |
| 5 | 3.3 | 83.8 | 2.6 | 65.8 |
| 6 | 2.4 | 61.0 | −5.8 | −148.3 |
| | (b) Girder 5 (inside grd) | | | |
| 4 | 0.3 | 8.9 | −3.8 | −97.3 |
| 5 | −0.4 | −9.4 | −1.7 | −43.7 |
| 6 | −0.2 | −5.3 | −2.6 | −66.8 |

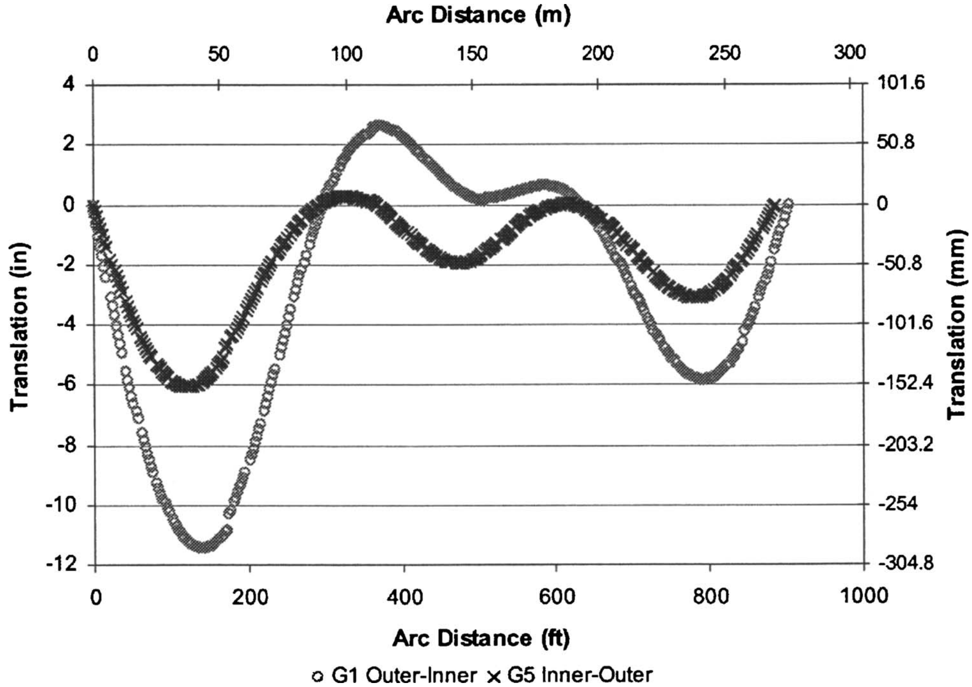


**Fig. 8.** Vertical deformations, completion of single girder erection, first girder line

ture response could be made by looking at movements for girder lines placed at similar instances in the erection procedure. Fig. 8 is a representative plot that compares vertical displacements for G1, the first girder erected for the outer-to-inner approach, and for G5, the first girder erected for the inner-to-outer approach. Fig. 9 repeats the process for the last girders erected (G5 for outer to inner, G1 for inner to outer). Gaps observed in certain plots resulted from the previously outlined procedure by which SAP2000 introduced additional elements to the numerical construction sequence in the original, undeformed state.

Through examination of these plots, similar plots for radial deformations and data produced by the numerical models, it was ascertained that the inner-to-outer erection procedure tended to result in larger cumulative displacements than the outer-to-inner single girder approach. Though the outer girder has a larger arc-span, it was proportioned as a stiffer member and was better able to accommodate deformations, especially radial deformations, when it was erected first. It was observed that, initially, vertical deformations were greater for the outer-to-inner procedure but radial deformations were consistently of smaller magnitude than

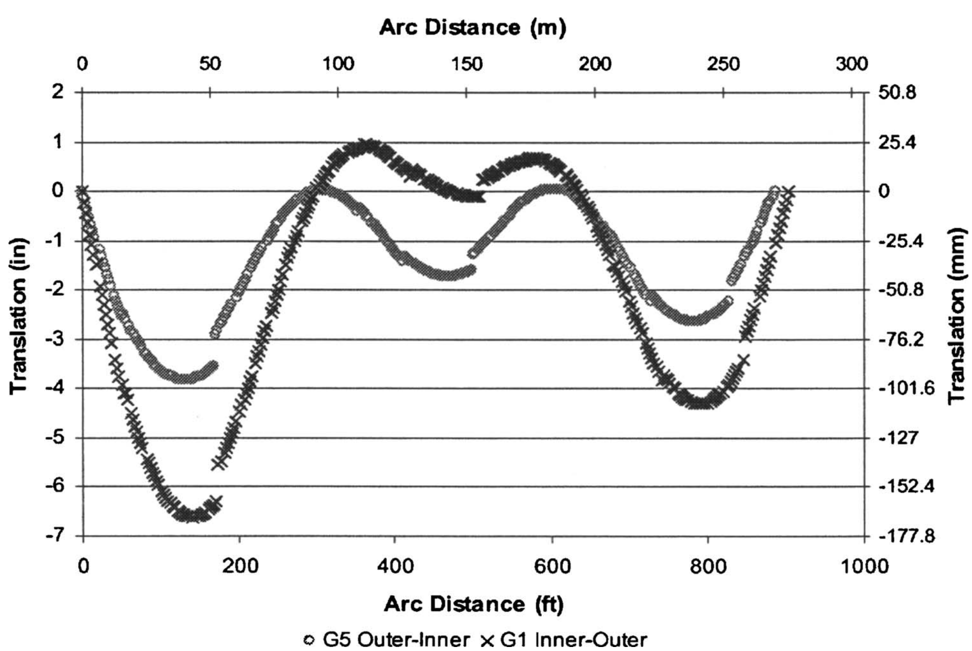


**Fig. 9.** Vertical deformations, completion of single girder erection, last girder line

**Table 4.** Maximum Fascia Girder Deflections, Inner-to-Outer Paired Girder Procedure

| Inner-to-outer girder pair erection | | | | |
|---|---|---|---|---|
| | (a) Girder 1 (outside grd) | | | |
| | Radial | | Vertical | |
| Span | (in.) | (mm) | (in.) | (mm) |
| 4 | 0.9 | 23.6 | −3.3 | −83 |
| 5 | −0.2 | −5.1 | −1.8 | −45.0 |
| 6 | 0.4 | 9.7 | −2.2 | −56.1 |
| | (b) Girder 5 (inside grd) | | | |
| 4 | 0.9 | 21.6 | −6.4 | −162.6 |
| 5 | 0.2 | 4.1 | 0.9 | 23.4 |
| 6 | 0.5 | 11.7 | −4.0 | −100.8 |

those for the inner-to-outer method. This can be explained by the way in which each individual girder line relates to the behavior of the entire superstructure system. For the outer-to-inner procedure, as subsequent girder lines are erected, lighter and less stiff elements are being added to an increasingly stiff system. Hence, overall deformations are less than adding heavier sections to a system with lower stiffness, as is the case for the inner-to-outer erection procedure. Furthermore, the placement of additional girder lines to the inside of a girder line helps counteract the tendency of the outer girder to rotate away from the center of curvature, thereby reducing overall deformations from a procedure where girders are added away from the center of curvature.

### Paired Girder Erection

As had occurred for the single girder erection sequence comparisons, erecting girder pairs from outer-to-inner and from inner-to-outer radii were numerically studied. It was assumed that girder pairs would be preassembled on the ground with all of the cross-frames in place and fully tightened prior to erection.

For the paired girder erection methods, largest deformations were experienced by the pair erected first, as shown in Table 4 for the inner-to-outer sequence and Table 5 for the outer-to-inner girder sequence. This demonstrated that when paired girder erection is used for this structure, element dead load has a large influence on deformations.

**Table 5.** Maximum Fascia Girder Deflections, Outer-to-Inner Paired Girder Procedure

| Outer-to-inner girder pair erection | | | | |
|---|---|---|---|---|
| | (a) Girder 1 (outside grd) | | | |
| | Radial | | Vertical | |
| Span | (in.) | (mm) | (in.) | (mm) |
| 4 | 0.8 | 20.6 | −6.5 | −163.8 |
| 5 | −0.6 | −15.2 | 1.2 | 30.0 |
| 6 | 0.2 | 4.6 | −3.7 | −94.5 |
| | (b) Girder 5 (inside grd) | | | |
| 4 | 0.4 | 9.7 | −3.6 | −92.2 |
| 5 | −0.3 | −7.9 | −2.1 | −52.1 |
| 6 | 0.1 | 3.8 | −1.8 | −46.7 |

Deformations for the first and last girder lines placed for each paired girder erection procedure were compared in similar fashion to the single girder sequences to help identify a preferred paired erection sequence.

It was observed that the outer-to-inner paired erection procedure tended to result in slightly larger vertical and moderately larger radial displacements than the inner-to-outer approach. These findings are opposite to those for the single girder erection procedure. Increased stiffness associated with paired erection in combination with decreased weight and arcspan of the inner girder pair resulted in smaller final deformations when it was erected first.

### Single versus Paired Girder Erection

When the preferred sequences identified from the single and paired girder studies were compared to see which provided the smallest deformations, it was apparent that, as expected, paired girder erection produced smaller overall radial and vertical deformations. The increased stiffness and redundancy provided by erecting pairs of girders over single girder lines was clearly evident.

### Implementation of Lateral Bracing

Lateral bracing was added to the inner-to-outer girder pair sequence, which tended to produce the lowest deformations of the techniques that were previously studied, to examine the effect on deformations during erection. It was understood that the addition of lateral bracing would reduce deformations, but it was of interest to determine the degree to which they would be reduced. As expected, the addition of the lateral bracing reduced all deformations at the completion of erection, as shown in the representative plot presented in Fig. 10.

### Implementation of Temporary Support Towers

Temporary support towers were independently added to the inner-to-outer girder pair analysis to quantify effects of shortening the effective span lengths of girder lines during erection. Towers were modeled as joints supporting vertical translation only, and were placed at the quarter points of Spans 4, 5, and 6, which approximates the common practice of placing shoring towers near field splices at points of inflection. Again, as expected and shown for vertical deformations in Fig. 11, the addition of support towers reduced deformations when compared to the original, unshored, paired girder inner-to-outer erection procedure. The same trend was evident for radial translations as well.

### Lateral Bracing versus Shoring Towers

The use of temporary towers at span quarter points generally controlled deflections more than the use of lateral bracing along the length of the exterior bays, though both methods proved to substantially limit movements over the base condition. This information is of interest for cases where bridges are being erected in locations where temporary towers cannot be practically placed.

## Conclusions

The goal of this research was to investigate various erection procedures and their effects on the final geometry of a large radius,

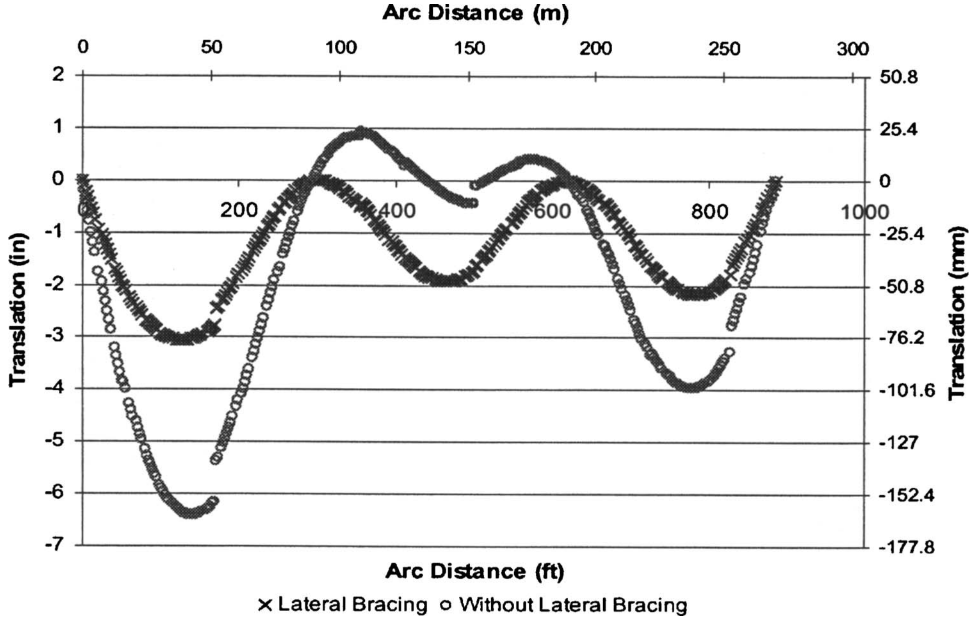


**Fig. 10.** Vertical deformation comparison, first girder pair line, inner-to-outer girder pair procedures with and without lateral bracing

horizontally curved, steel I-girder bridge. Specifically, single girder erection, the erection of girders in pairs, the effects of upper lateral bracing, and the implementation of support towers during construction of Structure 7A were explored. Specific research findings are as follows:

1. Erecting single girder lines initiating with placement of the outer girder (largest radius) and culminating with placement of the inner girder line (smallest radius) resulted in smaller overall final radial and vertical deformations than those obtained when erecting the inner girder line first;
2. Structure 7A deformations were further reduced when girder lines were erected in a paired configuration, with the procedure that placed the two girder lines with the smallest radii first, resulting in slightly lower deformations than the sequence that placed the outer girder pair first; and
3. The addition of temporary shoring towers in each span to the girder pair erection scheme produced lower deformations than procedures lacking support members, which generally compared favorably to the level of deformation reduction provided by the addition of upper lateral bracing.

This study indicates that, as expected, taking steps to stiffen the erected system will reduce final overall deformations, which are assumed to result in lower induced and locked-in stresses, and improved fit-up between superstructure elements. It was demonstrated that erecting girders in pairs provided measurable improvement in deformation control over procedures that utilized

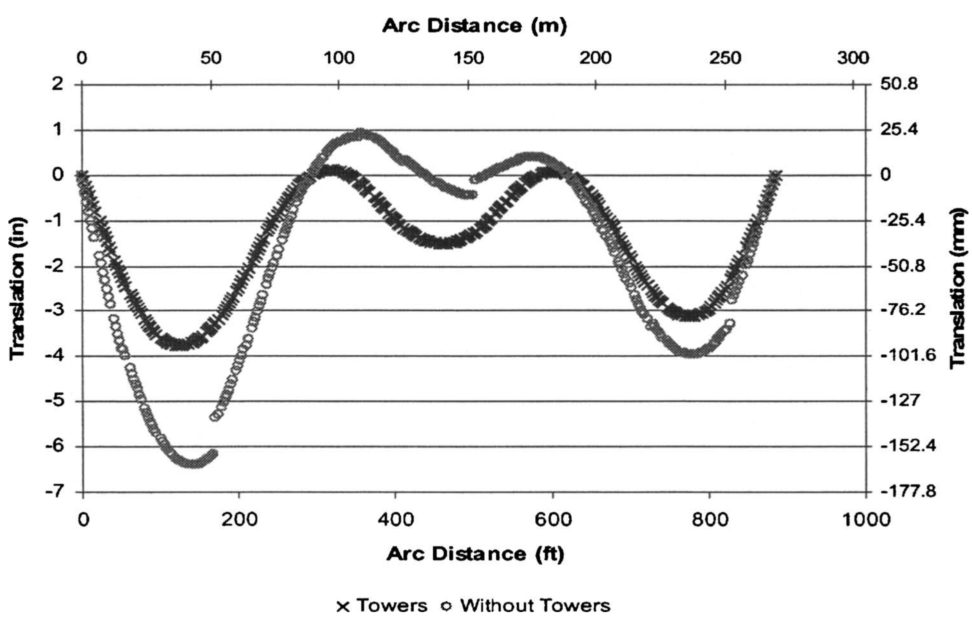


**Fig. 11.** Vertical deformation comparison, first-erected girder pair, inner-to-outer girder pair with and without support towers

single girder erection and that the implementation of upper lateral bracing or use of temporary support towers are effective means to further reduce deformations experienced during construction.

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