Effect of Temporary Shoring Location on Horizontally Curved Steel I-Girder Bridges during Construction

M. Sharafbayani[1] and D. G. Linzell, P.E., F.ASCE[2]

Abstract: Temporary shoring supports are used in construction of horizontally curved bridges to help ensure that the final constructed geometry is maintained by mitigating excessive girder deformations. Limited guidance currently exists in available design specifications and guidelines with respect to optimal placement of shoring towers because the number and locations of these supports are often site specific. However, if preliminary information could be provided to bridge designers and constructors with respect to shoring tower placement as a function of global curved bridge parameters, such as number of spans and radius of curvature, the amount of time required to specifically locate and proportion the towers could be reduced. This research aimed to examine the effects of shoring tower positioning on curved bridge behavior at different stages of construction. Sequential analyses of multiple idealized double-span curved bridges with varying radii were conducted using nonlinear finite-element models and vertical deformations and rotations of the girders, and shoring tower reactions were compared for different shoring support locations and different erection sequences. On the basis of the results, optimal shoring locations were obtained for the curved girders at different construction stages. **DOI: 10.1061/(ASCE)BE.1943-5592.0000269.**

CE Database subject headings: Girder bridges; Steel bridges; Constructability; Curvature; Finite-element method.

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Introduction

Horizontally curved steel I-girder bridges can offer an economical solution for highway system crossings where roadway alignment and geometry require a smooth, curved transition across the bridge and limited space is available for interior piers. Attributable to their curved geometry, the centerline of the girder webs at sections away from the end supports in each span are not collinear with a cord between the supports. Resulting eccentricities induce torsional moments that, in turn, cause out-of-plane deformations and rotations in the girder cross sections. During certain stages of curved I-girder bridge construction, the girders are only partially braced, and this in conjunction with the curvature effects may cause excessive deformations and stresses under its own weight. As a result of these deformations, some unanticipated problems may arise during girder erection, such as large bearing deformations, section fit-up problems, and, in extreme cases, stability issues.

Many studies were completed in the last 50 years to investigate the behavior of horizontally curved I-girder bridges. One of the earliest research projects was conducted by the Federal Highway Administration (FHWA) in 1969. The project, entitled the Consortium of University Research Teams (CURT) Project, involved several full-scale laboratory tests and analytical studies to investigate the behavior of curved bridges before and after deck placement (Mozer and Culver 1970; Mozer et al. 1971, 1973; Brennan 1970, 1971, 1974; Brennan and Mandel 1979). A more recent large-scale research project was the Curved Steel Bridge Research Project (CSBRP) initiated by the FHWA in 1993. Early phases of this project involved full-scale experimental and supporting analytical studies of a single-span horizontally curved I-girder bridge that examined its behavior during construction (Zureick et al. 2000; Linzell et al. 2004). Later phases of this project examined the effects of erection sequence on the induced stresses and deformations (Chang 2006), provided improved design guidelines, and examined the capability of analysis tools to predict bridge response (Linzell et al. 2004; White and Grubb 2005). Since the initiation and completion of these projects, many researchers have conducted additional experimental and computational studies aimed at addressing some issues pertaining to curved I-girder construction behavior such as load distribution levels between curved girders (Hajjar and Boyer 1997; Galambos et al. 1996, 2000; Sennah et al. 2000), levels of geometric nonlinearity attributable to curvature effects on the girders (Pi et al. 2000; Bradford et al. 2001), and levels of girder uplift during erection (Alampalli and Morreale 2001). Other studies have been completed that investigated proper lifting techniques for curved sections (Schuh 2008; Farris 2008; Stith et al. 2009) and the effects of various erection procedures on response of the girders (Bell 2004; Chavel and Earls 2006a, b; Howell and Earls 2007; Nevling 2008; Linzell and Shura 2010).

Despite these efforts, current editions of the AASHTO LRFD Bridge Design Specification (AASHTO 2007) and the AASHTO LRFD Bridge Construction Specifications (AASHTO 2008) contain only a few articles that require considering curvature effects on girders response during fabrication, shipment, and erection. The limited information may not necessarily be enough to ensure safe and reliable construction schemes for all curved I-girder bridges. In addition, there are limited publications that examine

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shoring tower location along the girders at different stages of construction and the effects of their location on bridge construction behavior. Chavel and Earls (2006a) developed an analytical model to replicate the as-built erection scheme of Ford City Veterans Bridge, which included temporary shoring towers, and indicated that had the girders and cross frames been properly detailed, the temporary supports that were used could have limited deformations and stresses in the girders. A recent study examined the behavior of curved girders for a single shoring case where the girders were placed onto two permanent supports, representing either abutments or piers, and the temporary shoring support location was varied in the span (Stith et al. 2009). Optimal shoring locations for this condition were found to be close to the maximum vertical deformation points along the girders. The study did not address other support conditions or levels of girder continuity, and the effects of erection sequence on shoring support performance were not considered.

The main purpose of the study outlined herein is to develop preliminary guidelines for the use of temporary shoring for horizontally curved steel I-girder bridges with differing geometry and at different stages of construction. This was accomplished by placing shoring at various locations along the girders and examining its influence on bridge performance for different erection sequences. For this, the responses of multiple idealized curved bridges with varying radii of curvature are investigated using nonlinear three-dimensional computational finite-element models.

Studied Bridges

Development of idealized bridges used to complete the study began with statistical examination of a large set of actual curved bridge designs from Maryland, New York, and Pennsylvania. The statistical studies focused on concentrically curved bridges without skew and established statistically significant parameters for radius of curvature, span and girder numbers, span length, and girder and cross-frame spacings. These parameters were initially used by Linzell et al. (2010) to establish a set of 12 representative bridges with various radii of curvature and cross-frame spacings and a different number of spans and span lengths.

Findings from the initial studies, which investigated the influence of several geometric and environmental variables on the behavior of the horizontally curved bridges during construction, showed that radius of curvature was the geometric parameter that had the greatest impact on response for the representative bridges. Also from those studies, bridges with larger span lengths and cross-frame spacings were found to be more flexible during construction. These outcomes predominantly aided with selection of geometric parameters for the current study, which focused on five two-span, idealized bridges. These bridges were selected because they effectively represented the effects of shoring location on construction behavior for the larger bridge group.

Fig. 1 details general geometric parameters for the curved bridge models that were examined. All measurements in this figure are along the bridge centerline. Five different radii of curvature were used to represent a severely curved bridge ($R = 91.5$ m), a moderately curved bridge ($R = 305$ m), and three curved bridges between the two extreme cases ($R = 145$, 198, and 251.5 m). For all five bridges, the cross frames were positioned at 7.6-m intervals along the bridge centerline, which is a relatively large spacing. The four girders were spaced at 3.05 m radially to provide adequate width for the bridge deck to accommodate two lanes of traffic, shoulders, and barriers. The 68.6-m span length was chosen because it allowed for consideration of temporary supports at multiple locations along a span. In this study, the effects of shoring location on girder behavior were examined at construction stages that included erecting girders in the first constructed span. At these stages, the girders experienced larger deformations and rotations compared with future erection stages that included additional spans and increased continuity. Therefore, the representative bridges in this study were two-span structures and results for the effects of shoring placement in the first span reflected critical performance stages for other multispan bridges.

These bridges were designed following AASHTO LRFD specifications (AASHTO 2007). Controlling section property parameters included depth-to-span ratios (D/L) of approximately 25 (where D = web depth and L = arc length along the bridge centerline) and section aspect ratios (D/b_f) of approximately 5.5 (where b_f = flange width) along the entire girder length following AASHTO proportion limits. X-shaped cross frames were used. Bearing stiffeners were placed at the supports with transverse stiffeners being used at cross-frame connection points. Girder splices were positioned near dead load contra flexure locations in each span and, as a result, all girders were composed of three segments: from Abutment 1 to Splice 1, from Splice 1 to Splice 2, and from Splice 2 to Abutment 2 (Fig. 1).

Nonguided, translationally restrained bearings were used for all girders at the abutments and were incorporated into the models via pins along the girder bottom flanges. Tangentially guided rollers were provided at the girder bottom flanges at the piers by applying translational restraint in the vertical and radial directions. Finally, girder bottom flanges at the temporary shoring locations were restrained against vertical deformations only. The possibility of uplift during erection, which can occur in horizontally curved bridges during construction (Chavel and Earls 2006a), was evaluated by checking vertical support reactions at each analysis step for negative values. As discussed in the following section, shoring locations that were studied were selected so that no uplift occurred.

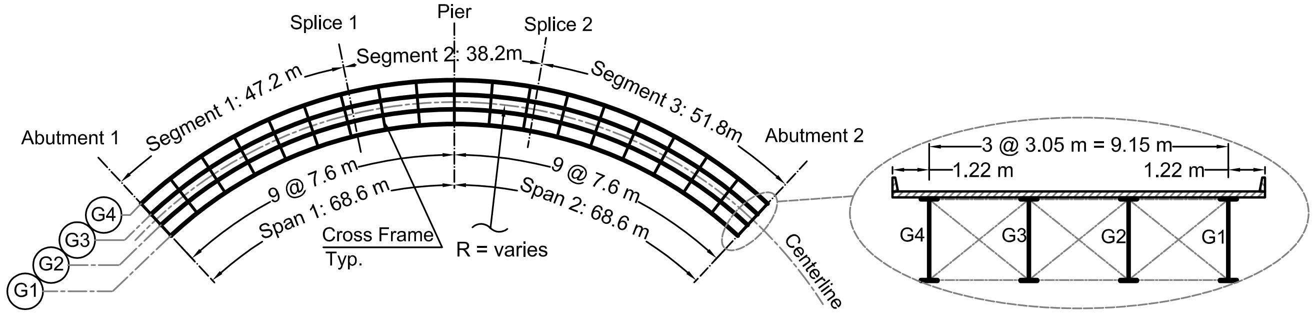


Fig. 1. Framing plan and cross section for bridge models with varying radii of curvature

Computational Models

Examination of shoring location effects on behavior was accomplished via numerical analyses using the ABAQUS finite-element program (ABAQUS Version 6.9). The models focused purely on the steel superstructure and consisted of ABAQUS S4R shell elements for the girder webs and ABAQUS B31 beam elements for the top and bottom flanges, bearing and transverse stiffeners, and cross-frame members. The shell elements had aspect ratios close to 1:1 based on validation and mesh sensitivity work by Nevling (2008). Fig. 2 is an isometric image of the ABAQUS models. Inasmuch as the models were designed to remain in the elastic range throughout all construction stages, a linear material model was assigned to all elements. Nonlinear geometric effects were, however, included in the analyses.

At each stage during curved bridge construction, the erected portion of the bridge was subjected to different loading and support conditions. Therefore, to effectively track bridge response at different stages of construction, sequential analyses are commonly employed and were used here.

For the current study, sequential analyses were performed by creating multiple ABAQUS analysis steps that mimicked each superstructure erection step. During each step, the program analyzed the structure having the appropriate structural components, loads, and boundary conditions. Two erection schemes were assumed for the sequential analyses. These procedures were based on actual, documented erection methods reported by Linzell and Shura (2010). The inner to outer girder (Method 1) and outer to inner girder (Method 2) erection sequences that were used are summarized in Table 1. As shown in the table, the first segments of the first two girders are erected as a pair, followed by single erection of the first segments of other girders. A similar approach was also assumed for the erection of the second and the third segments of the bridges. The girders segments in Table 1 are identified with two numbers. The first number indicates the girder number, and the second number is the segment number of the girder. For instance, the first segment for Girder G3 is named G31.

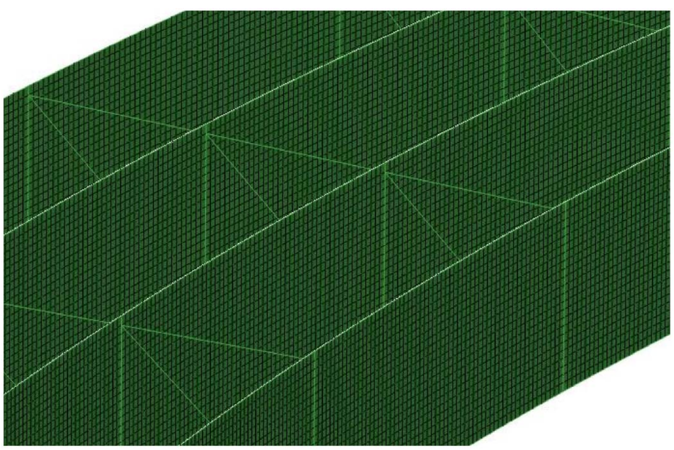


Fig. 2. ABAQUS finite-element model

Table 1. Sequential Analysis Erection Procedure

Stage	Method 1: Inner to outer girder erection	Method 2: Outer to inner girder erection
1	G11, G21, and cross frames	G41, G31, and cross frames
2	G31 and cross frames	G21 and cross frames
3	G41 and cross frames	G11 and cross frames
4	G12, G22, and cross frames	G42, G32, and cross frames
5	G32 and cross frames	G22 and cross frames
6	G42 and cross frames	G12 and cross frames
7	G13, G23, and cross frames	G43, G33, and cross frames
8	G33 and cross frames	G23 and cross frames
9	G43 and cross frames	G13 and cross frames

Temporary Support Locations

To examine the effects that temporary shoring placement had on curved girder behavior during erection, two shoring scenarios were considered. They included placing either a single shoring support or two shoring supports at different locations along the first span of the bridge models.

Single Shoring Support

To establish optimal locations for single shoring supports in the first span, the first erected girder segments were preliminarily examined by placing the shoring support at different locations in the ABAQUS models; starting from the end of the girder segment, Splice 1 (Fig. 1), and moving toward the first abutment. It was found from these preliminary analyses that placing the shoring tower at distances less than approximately 0.6 of the first segment length from the abutment resulted in girder uplift. Therefore, it could be stated that this location is the lower bound for possible tower locations in the first segment, where the upper bound exists at Splice 1.

On the basis of findings from the preliminary analyses, three shoring tower placement cases were considered for the bridge models. As shown in Fig. 3, each case assumed that towers were placed at different cross-frame locations in the first span, a placement approach that is common when shoring is required (Chavel and Earls 2006a). Models were modified at these locations to accommodate high concentrated forces via the addition of bearing stiffeners. Shoring 1 examined placement at the cross-frame immediately adjacent to Splice 1. Shoring 3 had towers positioned at 0.65 L in the first erected girder segments (close to the lower bound for possible tower locations), and Shoring 2 placed the towers between 1 and 3. Other intermediate shoring locations between Shoring 1 and 3 were also examined preliminarily, and findings from the three selected locations in Fig. 3 effectively bounded all influence that shoring support position had on behavior during construction.

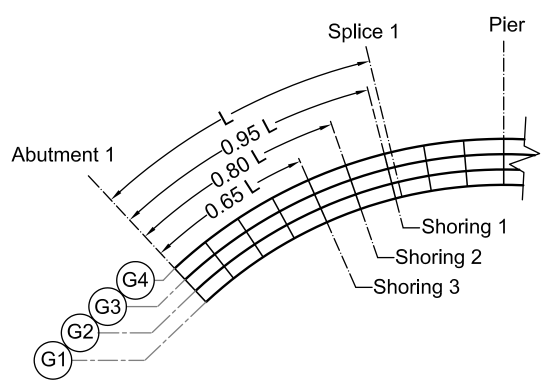


Fig. 3. Shoring tower locations, one shoring support

Two Shoring Supports

In this scenario, two shoring towers were placed under each girder in the first span. The first tower was positioned close to the abutment with the second tower close to the splice. These two towers will be referred to as Tower 1 and Tower 2, respectively, in the results for the two shoring support cases. Similar to the shoring cases with one tower, multiple locations were examined for two towers in the first span. As shown in Fig. 4, two different locations were considered for each tower in this scenario. Tower 2 was assumed to be either close to the end of the first girder segment, near Splice 1 (0.95 L) or at one cross-frame adjacent to its initial location (0.80 L). Tower 1 was located either at the midpoint of the first girder segment, between Abutment 1 and Splice 1 (0.50 L), or one cross frame adjacent to its initial position (0.30 L). The combination of these locations resulted in a total of four different shoring cases being examined, Shoring 4 to Shoring 7, as shown in Table 2.

Results and Discussion

The effects of the shoring placement on girder behavior are presented by comparing support reactions and girder deformations for the different shoring cases at different stages of construction using erection Method 1 in Table 1. As stated earlier, erection of the girders in the first span and the effects of different shoring locations along this span on behavior are discussed here and response quantities are presented for two general conditions: after completion of erection of the first segment (Stage 3) and after completion of erection of the second segment (Stage 6). Framing plans for these stages are shown in Figs. 5 and 6.

One Shoring Support: Erection Method 1

For this shoring scenario, at the completion of Stage 3 the girders are supported by the abutment and one shoring tower. Support reactions at the temporary shoring locations for each girder and the total vertical bridge reaction at the tower were studied for the

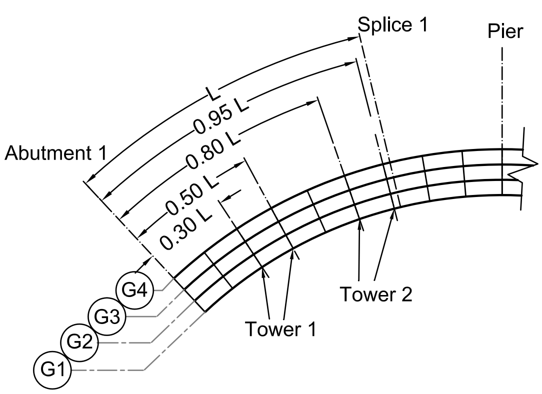


Fig. 4. Shoring tower locations, two shoring supports

Table 2. Two Shoring Support Cases

Shoring case	Location of Tower 1	Location of Tower 2
Shoring 4	0.50 L	0.95 L
Shoring 5	0.30 L	0.95 L
Shoring 6	0.50 L	0.80 L
Shoring 7	0.30 L	0.80 L

bridge models having varying radii of curvature. Figs. 7 and 8 show representative results for $R = 91.5$ and 305 m.

Results for Shorings 2 and 3 indicate that the towers experienced similar load levels for all girders for this shoring location

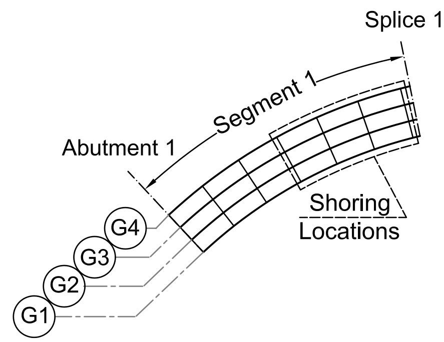


Fig. 5. Construction Stage 3

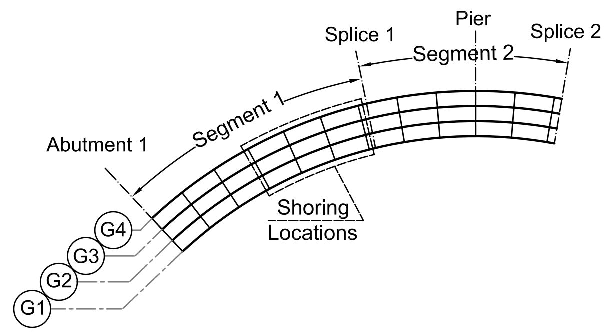
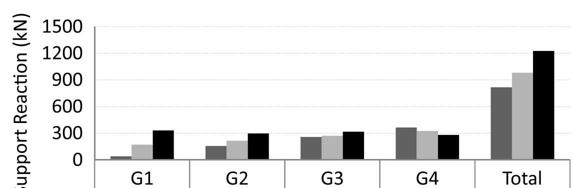
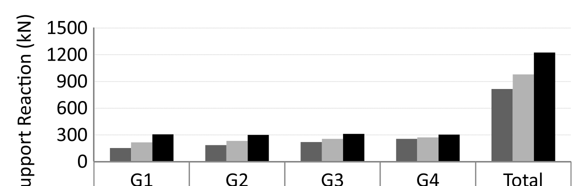


Fig. 6. Construction Stage 6



	G1	G2	G3	G4	Total
Shoring 1	40	155	257	364	817
Shoring 2	171	215	270	324	980
Shoring 3	331	297	317	280	1225

Fig. 7. Vertical support reactions at shoring tower, one shoring support, $R = 91.5$ m, Stage 3



	G1	G2	G3	G4	Total
Shoring 1	154	185	220	257	816
Shoring 2	217	234	256	272	979
Shoring 3	307	300	313	305	1224

Fig. 8. Vertical support reactions at shoring tower, one shoring support, $R = 305$ m, Stage 3

scheme, which produced a cantilevered segment over the shoring towers. As an example, the maximum difference between girder reactions for Shoring 3 in the severely curved bridge model is on the order of 15% of the maximum support reaction, which occurred at G1 as shown in Fig. 7. This difference becomes smaller for bridges with larger radii ($R > 198$ m, here). These results help clarify optimal shoring placement schemes and indicate that the Shoring 1 location, which placed the tower close to the end of the girder first segments, received lower loads than the other two options but experienced the largest curvature effects. However, when the girders were supported at the Shoring 2 and 3 locations, curvature effects were mitigated compared with Shoring 1.

Having web-plumb girders under self weight during erection is commonly desired by contractors to avoid problems when attempting to fit up girder segments. Research has indicated that, via the use of shoring towers, it is desired that a no-load condition be achieved during construction whereby girder deformations under their own self-weight or under their weight and additional superimposed dead load, such as that from the deck, be close to zero. The consequences of not achieving a no-load condition or inaccurately detailing superstructure components for the no-load situation have been reported in the literature (Chavel and Earls 2006b; Howell and Earls 2007). In the present study, the attainment of a web-plumb position and the no-load condition by the girders during construction was investigated via comparison of web rotations and girders vertical deformations for different shoring cases.

Figs. 9 and 10 show the web rotations in the first segments for the two fascia girders, G1 and G4, after Stage 3 for the most severely curved bridge in this study ($R = 91.5$ m). The reported rotation angles were obtained via the difference between radial deformations at the top and bottom flanges.

As shown in these figures, for each shoring case, maximum web rotation occurred halfway between the abutment and the shoring tower, as expected. Local valleys on the response curves represent rotational restraint provided by the cross frames, and these valleys are more pronounced for the exterior girder (G4), which has larger unbraced lengths along its arc. The figures indicate that relocating a shoring tower from Shoring 1 to Shoring 3 appreciably reduced the web rotation angles along the entire length. By placing the tower at Shoring 3, torsional moments that act on the cantilevered segment and on the simple span between the first abutment and shoring tower counterbalance each other. Similar behavior was observed for other radii of curvature. However, with increasing radii, differences between rotations for the various shoring cases become less pronounced. Figs. 11 and 12 show the web rotations for G4 in bridge models having larger radii of curvature ($R = 251.5$ and 305 m). As seen in these figures, the web rotations are generally small and relocating the shoring towers resulted in smaller changes in these rotations compared with Fig. 10.

Similar findings for the influence of different shoring locations on girder vertical deformations were obtained by comparing these results for the three shoring cases. Figs. 13 and 14 display vertical deformations, after Stage 3, at the bottom of G4 for bridge models with $R = 91.5$ and 305 m. G4 was selected because, as the girder having the largest arc length, it should experience the largest vertical deformations during construction. On the basis of these

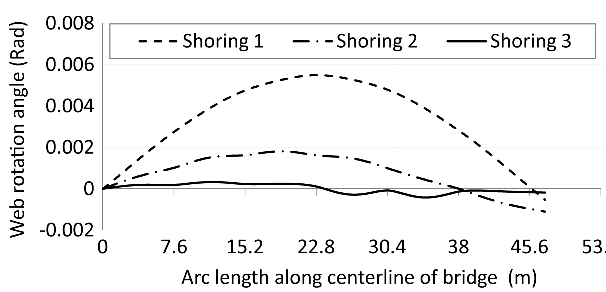


Fig. 9. G1 web rotations, one shoring support, $R = 91.5$ m, Stage 3

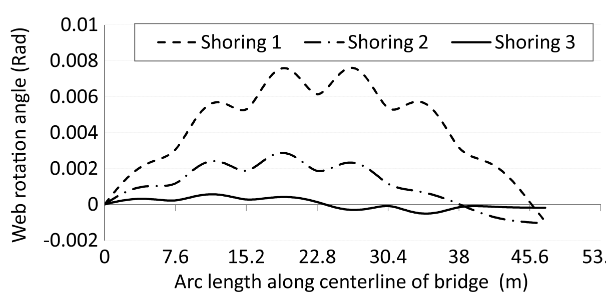


Fig. 10. G4 web rotations, one shoring support, $R = 91.5$ m, Stage 3

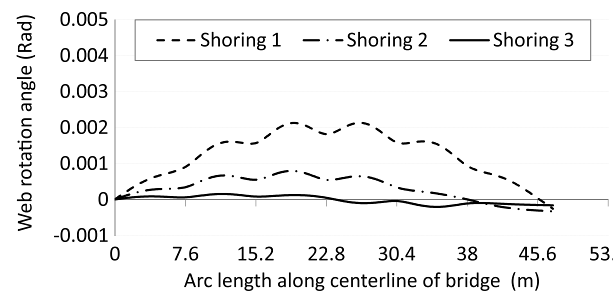


Fig. 11. G4 web rotations, one shoring support, $R = 251.5$ m, Stage 3

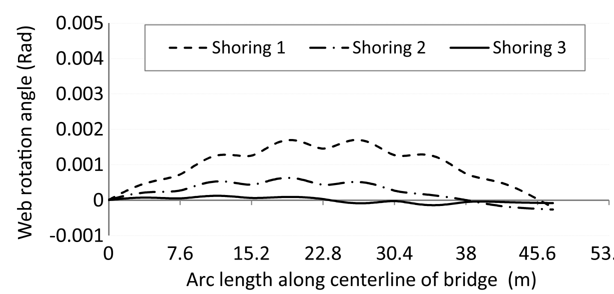


Fig. 12. G4 web rotations, one shoring support, $R = 305$ m, Stage 3

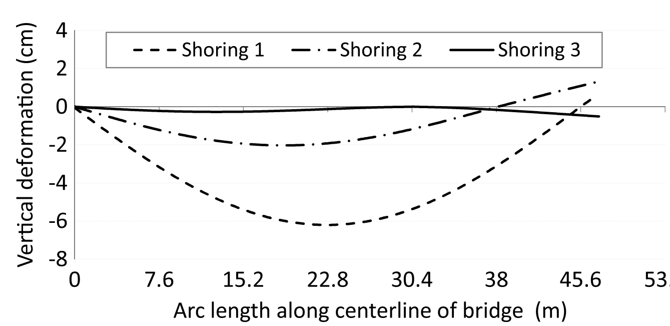


Fig. 13. G4 vertical deformations, one shoring support, $R = 91.5$ m, Stage 3

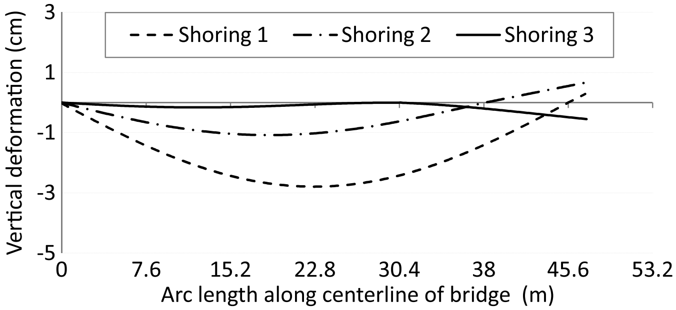
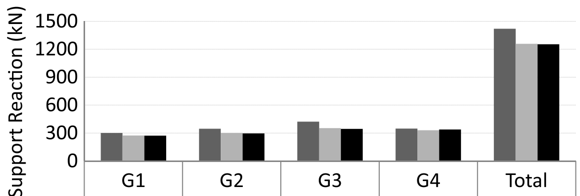


Fig. 14. G4 vertical deformations, one shoring support, $R = 305$ m, Stage 3



	G1	G2	G3	G4	Total
Shoring 1	301	347	423	349	1421
Shoring 2	274	302	353	331	1259
Shoring 3	273	296	345	339	1253

Fig. 15. Vertical support reactions at the shoring tower, one shoring support, $R = 91.5$ m, Stage 6

results, placing temporary support at the Shoring 3 location also resulted in smaller vertical deformations for the girders in all bridge models. Again, higher changes in vertical deformations occur for the more severely curved bridges ($R \leq 251.5$ m) when shoring is relocated along the span.

At the completion of Stage 6, Fig. 6 indicates that the girders are resting on two permanent supports and a single temporary shoring support in the first span. The common practice for shoring location for this support condition is placing the tower at the location of maximum vertical deflection in the span (Stith et al. 2009). The maximum deflection point for the bridge models falls between the locations for Shoring 2 and Shoring 3 (and slightly closer to Shoring 3) in the span. This was obtained from results for unshored girders after Stage 6. For this stage, again, reaction forces at the shoring tower were compared for the three shoring positions, with the addition of the second girder segment, which cantilevers over the pier, in place. In contrast to the results for Stage 3, no marked benefit was obtained from changing shoring location at this stage to attempt to mitigate the effects of curvature on girder reactions at the towers. This can be inferred from support reactions for the bridge model with the smallest radius of curvature ($R = 91.5$ m) in Fig. 15. The shoring tower experiences similar load levels at all girder locations for the different shoring cases. Smaller total support reactions at the towers for this stage occur for Shorings 2 and 3. Also, when results for both Stages 3 and 6 are considered, locating shoring towers at the Shorings 2 and 3 positions subjects them to smaller loads than Shoring 1.

Girder web rotations and vertical deformations were compared after Stage 6 to investigate the effects of shoring locations on achieving a web-plumb position and a no-load geometry. Figs. 16–18 show the web rotations for G4 in bridge models having radii of 91.5, 145 and 198 m, respectively. As expected, Shoring 3, which placed the towers near the maximum vertical deflection point

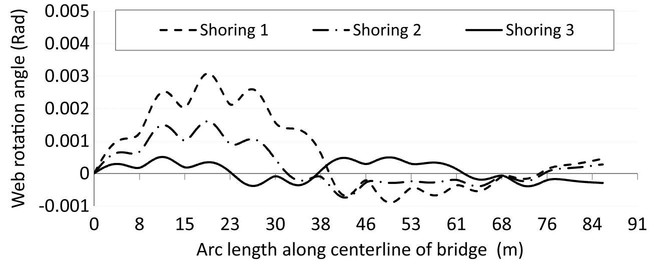


Fig. 16. G4 web rotations, one shoring support, $R = 91.5$ m, Stage 6

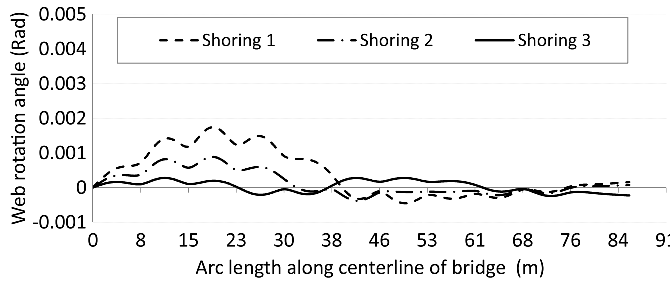


Fig. 17. G4 web rotations, one shoring support, $R = 145$ m, Stage 6

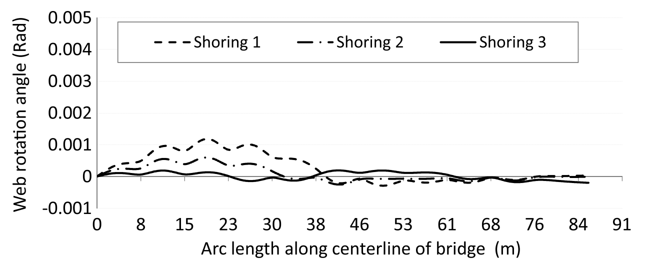


Fig. 18. G4 web rotations, one shoring support, $R = 198$ m, Stage 6

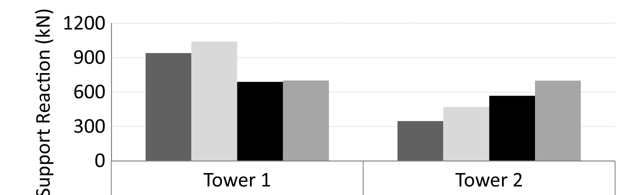
of the span, resulted in smaller girder rotations. The same effects were observed for the vertical deformations at this stage. Furthermore, it was understood from the girder responses at Stage 6 that finding the optimal location for a single shoring support at this stage was more critical for small radii of curvature ($R = 91.5$ and 145 m, in this study). Benefits from relocating shoring supports for this stage quickly diminished with increasing radii of curvature and this trend initiated at smaller radii of curvature than at the completion of Stage 3. Increased continuity provided by the three supports (abutment, tower, and pier) for the girders at Stage 6 was the main driver for these findings.

Two Shoring Supports: Erection Method 1

Similar to the one shoring support scenario, multiple locations for two shoring supports were examined in the first span of the bridge models as shown in Fig. 4. Again, the response quantities were studied at the completion of Stages 3 and 6 and erection Method 1 was used.

Reaction forces at the shoring tower locations were compared for the different shoring cases. At construction Stage 3, the girders are supported at Abutment 1, Tower 1, and Tower 2 (see Figs. 4 and 5) This support condition is similar to that for construction Stage 6, having one shoring support. Again, no considerable change in the effects of curvature on girder reactions was observed for the different shoring positions. However, when total reaction forces at the two shoring towers were compared, Shoring 7 resulted in a balanced load sharing between the two towers as shown in Fig. 19 ($R = 91.5$ m). As also indicated in the figure, large differences between reaction forces occurred for Shoring 4 and Shoring 5, which placed Tower 2 close to the end of the girder segments (see Fig. 9 and Table 2). Similar results were found for other radii of curvature. In addition, after construction Stage 6 the addition of the pier support generally reduced tower reaction forces and differences between them for both towers, as expected. The towers experienced their maximum reaction forces after Stage 3.

Web rotations and vertical deformations were studied for different shoring cases with two towers in the span, and only slight changes were observed between the different cases. Figs. 20 and 21 show web rotations and vertical deformations in G4 for $R = 91.5$ m. As indicated in these figures, Shoring 4 and Shoring 7 resulted in somewhat smaller deformations and rotations for the girders. For these two shoring schemes Tower 1 was placed at the maximum girder deflection point between the abutment and Tower 2 (see Fig. 4 and Table 2). These results are again similar to the findings for Stage 6 for the one shoring support case in which the girders were also supported at three locations. Moreover, changes in girders responses for different shoring positions with two towers were insignificant at Stage 6 where higher levels of continuity existed. In the bridges having radii of curvature larger than $R = 145$ m, no marked benefit was obtained from using two towers in the span compared with results for one shoring tower at the Shoring 3 location.



	Tower 1	Tower 2
Shoring 4	940	347
Shoring 5	1040	470
Shoring 6	688	567
Shoring 7	701	699

Fig. 19. Vertical support reactions at the shoring towers, two shoring supports, $R = 91.5$ m, Stage 3

Effect of Shoring Tower Removal

During construction of curved bridges having multiple spans, contractors might remove shoring towers from beneath girders in an erected span and use them for erection of girders in adjacent spans. This situation might occur when limited numbers of shoring towers are available. To study the effects of tower removal on the behavior of the girders in the analysis models, Stage 6 and Shoring 3 were considered. The shoring supports were sequentially eliminated from the girders, starting from the inner girder (G1) and proceeding to the exterior girder (G4). Resulting vertical deformations and web rotations in all girders were studied after each support removal step.

By removing the tower supports from the girders, unsupported girder weights counteracted the curvature effects in the superstructure. Figs. 22 and 23 demonstrate this phenomenon by showing vertical deformations in G1 and G4 for $R = 91.5$ m. As indicated in this figure, by removing the towers from G1 and G2 slight changes in the vertical deformation occurred for all girders. However, after removing the tower from G3, large deformations were observed. This was more pronounced for web rotations as shown in Figs. 24 and 25 for G4 and $R = 91.5$ and 305 m. In these figures, after removing the towers from G1 and G2, the curvature effects and the effects of the G1 and G2 self weights counterbalanced

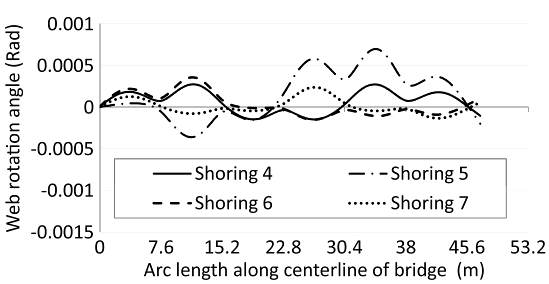


Fig. 20. G4 web rotations, two shoring supports, $R = 91.5$ m, Stage 3

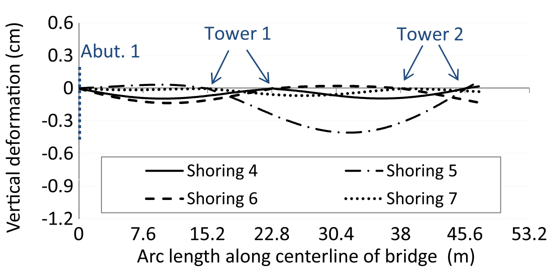


Fig. 21. G4 vertical deformations, two shoring supports, $R = 91.5$ m, Stage 3

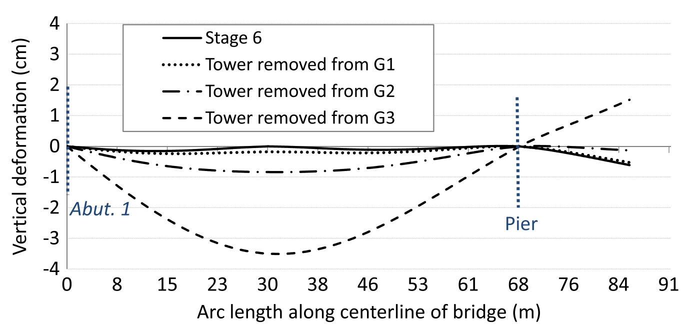


Fig. 22. G1 vertical deformations, $R = 91.5$ m, shoring removal

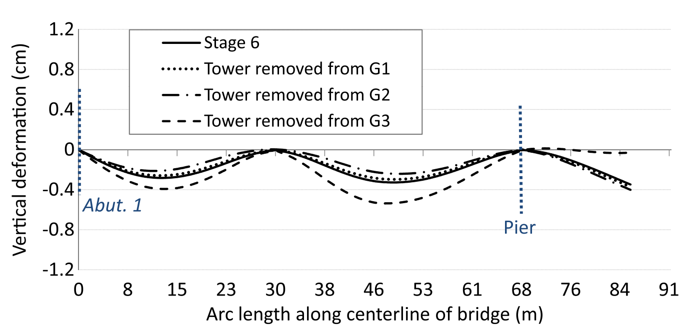


Fig. 23. G4 vertical deformations, $R = 91.5$ m, shoring removal

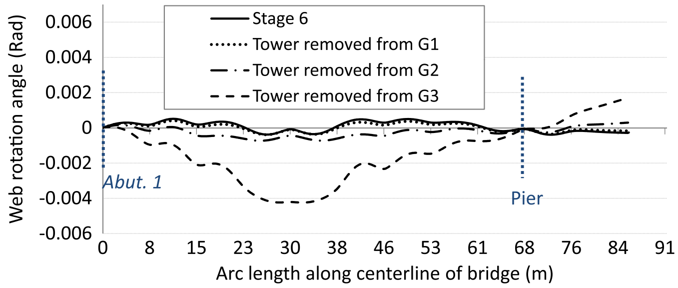


Fig. 24. G4 web rotations, $R = 91.5$ m, shoring removal

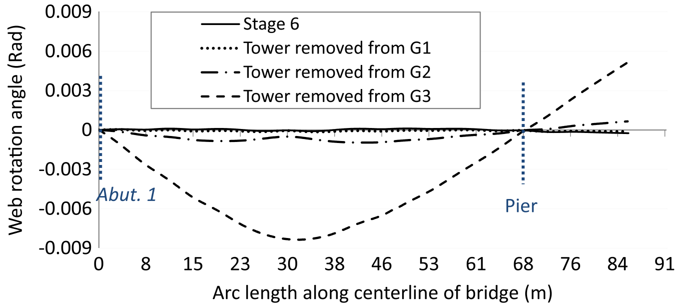


Fig. 25. G4 web rotations, $R = 305$ m, shoring removal

each other and small changes occurred in girder rotations. However, removing the shoring support from G3 resulted in torsion of the superstructure caused by the effects of the G1, G2, and G3 self weights that overwhelmed the curvature and produced a relatively large rotation in the girders. These web rotations were found to be greater for larger radii ($R > 198$ m) where curvature effects were smaller and the inner girders have larger arc lengths (and therefore larger self weights) relative to the exterior girders.

Finally, vertical deformations in G4 for all bridge models are shown in Fig. 26 after removal of the shoring tower from beneath G4 (no shoring in the span). As indicated in this figure, the girders then constitute a simple span that is cantilevered over the pier. On the basis of these results, very large deformations were experienced after removing the shoring towers from the first span for all bridges that were studied. For the bridge model with the largest radius ($R = 305$ m) at the end of G4, the girder moved upward by 10.1 cm when the G4 tower was removed, a change in deformation that was much larger than any experienced at this location in the presence of the shoring towers, even at nonoptimal locations. This situation is more critical for smaller radii of curvature. These

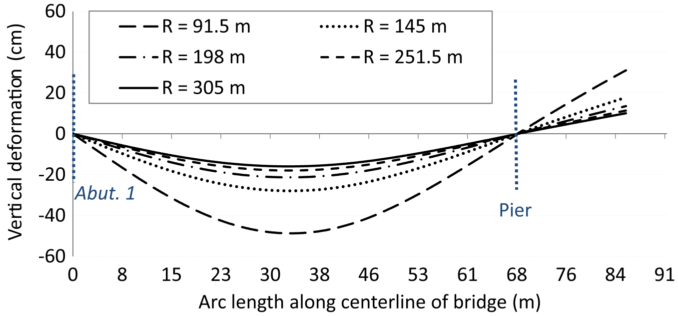


Fig. 26. G4 vertical deformations, no shoring

excessive girder deformations would more than likely be outside an acceptable range for many contractors to expediently complete erection of adjacent spans.

Finally, the effects of the shoring tower removal on cross-frame forces were also studied to understand if this action caused any detrimental effects on cross-frame forces between the girders. After removing towers from beneath G1 and G2, a significant increase was observed in the forces of the cross frames adjacent to the shoring location in the span. A large rise in cross-frame forces also occurred at the pier location when towers were removed from beneath G3 and G4. The order of increase in the cross-frame forces was almost the same for all radii of curvature that were studied. These forces were, however, below the design capacities for the cross-frame members.

Effect of Erection Method on the Curved Girder Response

The final component of this study examined the influence of order of girder placement on shoring tower effectiveness by repeating examinations in previous sections for erection Method 2 from Table 1. Results for the bridge model having $R = 91.5$ m were compared against Method 1 results for the Shoring 1 and Shoring 3 conditions.

Figs. 27 and 28 plot the web rotation angles for G4 after construction Stages 3 and 6 for both erection methods. These figures indicate that, irrespective of girder placement method, girder rotations are very small for all stages of construction for Shoring 3. For Shoring 1, which located the tower close to the girder splices at the first span, Method 2 resulted in smaller rotations and also deformations in the girder as presented in previous studies by Bell (2004) and Linzell and Shura (2010). However, when moving the tower to the optimal Shoring 3 location, the effects of erection scheme on girder behavior became quite small.

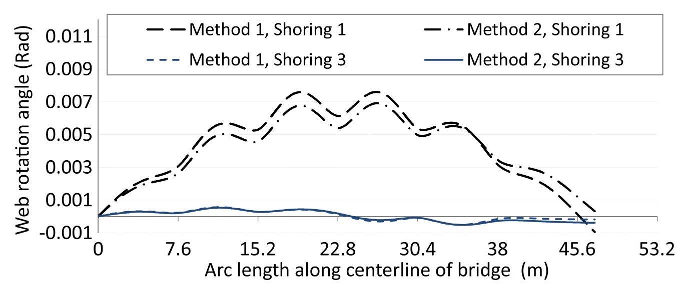


Fig. 27. G4 web rotations, $R = 91.5$ m, Stage 3

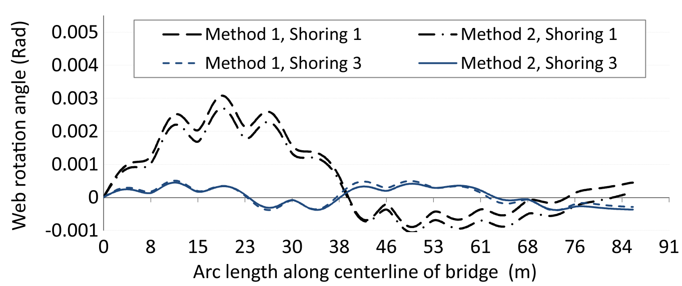


Fig. 28. G4 web rotations, $R = 91.5$ m, Stage 6

Conclusions

The effects of varying shoring support locations in the first erected span of five idealized curved bridges with varying radii were studied. The bridges were symmetrical two-span continuous structures with four girders and large cross-frame spacings. Sequential finite-element analyses were employed to mimic actual conditions encountered during different construction stages. Two shoring scenarios were considered: the first, having one shoring tower at different locations in the first span of the bridge models, and the second, having two towers at different locations in the first span of the bridge models. As a result, a total of seven shoring cases was examined to establish optimal shoring locations.

For the cases that considered one shoring tower in the span when the girders segments were supported at an abutment and a shoring tower (Stages 1 to 3 in Table 1), having the girders cantilevered over the shoring supports generally resulted in more constructible conditions by reducing curvature effects on girders support reactions at the shoring towers and by producing smaller deformations and rotations along the entire lengths of the girders. The optimal location for one shoring support in the first erected girder segment was found to be close to a distance of 0.65 of the segment arc length from the abutment. Changing the location of the shoring tower along the girders from the optimal location had more significant impact on the behavior of bridges with smaller radii of curvature ($R \leq 251.5$ m in this study).

For stages of construction where the girders were supported at abutment, pier, and a single shoring tower (Stages 4 to 6 in Table 1), the optimal location for one shoring tower was close to the maximum deflection point of the unshored girders in their fist span. Again this optimal location was more influential on the behavior of bridges with radii of curvature smaller than $R = 198$ m. These results substantiate findings obtained by Stith et al. (2009). They also extended that study by more realistically considering the effects of girder erection sequence on shoring performance for multiple bridge types and shoring conditions.

When two shoring towers were considered for the first span of the bridge models, the effect of various shoring placement schemes on bridge response during construction were noticeable for only very small radii of curvature ($R \leq 145$ m, here). In these bridges, for construction Stage 3 and Shoring Case 7 (see Table 2), slightly smaller deformations and rotations were produced in the girders compared with other shoring cases with two towers in the span. This support condition also resulted in balanced load levels between the two shoring towers for all bridges that were studied. In addition, for later stages of construction (Stages 4 to 6 in Table 1) no beneficial effect was observed from relocation of the shoring towers in the span when girder response was considered. Furthermore, comparing girder responses for placement of one shoring support and two shoring supports at their optimal locations in the span (Shoring 3 and Shoring 7) indicated that slightly smaller results were obtained for Shoring 7 in severely curved bridge ($R = 91.5$ m). These effects were insignificant for larger radii.

The effects of tower removal after completion of erection of the first span on girder response were also studied by sequentially removing towers from each girder, starting from the inner girder. For all radii of curvature, removing the towers from beneath the girders inside the centerline of the bridge (G1 and G2) resulted in no drastic changes in girder deformations. For radii of curvature larger than $R = 198$ m, removing the tower from beneath G3 resulted in large web rotations in all girders. Finally, removing the towers from beneath G4 created excessive deformations and rotations in the girders for all the bridge models that were studied. Attention should be paid to cross frames and also shoring supports that could experience significant rises in loads they experience when removing the shoring towers.

Finally, the effects of erection method on the response of girders in a severely curved bridge with different shorings were investigated. It was found that, when the shoring tower was placed at an optimal location, as described previously, nonlinear geometric effects became insignificant and similar girder behavior for the erection schemes resulted for the bridges that were studied.

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