EFFECTS OF INCONSISTENT DETAILING IN HORIZONTALLY CURVED STEEL I-GIRDER BRIDGES

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BIOGRAPHY

Brandon W. Chavel is a Structural/Bridge Designer with the Pittsburgh, Pennsylvania office of HDR Engineering, Inc. He joined HDR in 2002, and his professional experience includes the analysis and design of various steel and concrete bridge types and components, evaluating existing bridge structures, and bridge erection engineering. Additionally, he is a doctoral candidate at the University of Pittsburgh, Department of Civil and Environmental Engineering, where his research is focusing on construction and detailing practices used in horizontally curved steel I-girder bridges.

Dr. Christopher J. Earls joined the faculty of the University of Pittsburgh in 1999 and became the Department Chairman in 2004. In addition to his primary appointment as an Associate Professor of Civil Engineering, he also holds an appointment as a William Kepler Whiteford Faculty Fellow in the School of Engineering. Professor Earls' areas of research include the behavior steel structural members and systems, structural stability, nonlinear finite element modeling of solids and structures. He is active in numerous committees with the American Society of Civil Engineers and the Structural Stability Research Council. His research activities have been sponsored by organizations such as the Federal Highway Administration, the PennDOT, and the Army Research Labs.

SUMMARY

Bridge components are typically inconsistently detailed because the rotation of the girders and the subsequent lateral displacement of the top flange due to applied or dead loads are predicted to be in excess of what the owner may fell comfortable with. Within this context, it is then hoped that by intentionally detailing the diagonal cross-frame members to be either too short or too long, the girders may subsequently be forced to be in a vertically plumb condition under a given loading. However, this is not the case, and what will instead result is connection misalignments during the erection of the steel superstructure; or worse yet, in the case of large bridges, misfits of such a magnitude as to render the bridge un-constructible.

The current paper is intended to promote awareness of the issue of inconsistent detailing in horizontally curved steel I-girder bridges by studying two recently constructed horizontally curved steel I-girder bridges that were inconsistently detailed. The discussion highlights the fact that practice of inconsistent detailing can lead to very formidable and costly fit-up problems in the field; especially when girder sizes are large.

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INTRODUCTION

The construction of horizontally curved steel I-girder bridges is oftentimes more complex than construction of corresponding straight steel I-girder bridges of similar span. This is due to the fact that unlike a straight steel I-girder, a curved girder will rotate out-of-plane under gravity loading; including self-weight. Therefore, methods used in curved I-girder bridge construction must control not only the vertical displacement of the I-girders, but also the out-of-plane displacements, such that structural components, including the cross-frames, can be erected with limited difficulty. To prevent significant displacements during erection, temporary supports (falsework) and holding cranes can be utilized.

While it is that falsework may be employed during construction to affect a condition approximating a fully supported configuration, removal of the temporary support system must eventually occur. Upon removal of the falsework supports, the bridge superstructure will deflect and rotate out-of-plane by an amount that is related to girder span, cross-sectional properties, and radius of curvature. This may lead to a condition where the owner or design engineer becomes concerned that such deflections are excessive and thus some mitigation measure should be employed. One commonly used mitigation technique involves the intentional introduction of cross-frame misfits into the fabrication plan for the bridge system. These misfits are believed to be a viable vehicle by which objectionable girder web-out-of-plumbness might be controlled. The unfortunate truth is that this strategy rarely works out as planned as a result of the realities inherent in the uniqueness theorem of Kirchhoff in relation to elastic system response: girder webs may not be made to be plumb under two differing loading conditions in an elastic system. While some sort of acceptable geometric configuration may result from the application of inconsistencies in cross-frame fabrication in modestly proportioned bridges, such an approach will oftentimes be fraught with serious problems when deployed within the context of larger bridge systems.

Inconsistent detailing results in cross-frame member lengths that are incompatible with girders in the theoretical no-load position at time of erection. Therefore, the cross-frame members will have to be forced into place, twisting the girder webs out-of-plumb during erection. The direction of the out-of-plumb twist will be in the opposite direction of the natural out-of-plane rotation that will occur in the girders upon the application of dead load. Since the cross-frames are being forced into position, locked-in girder and cross-frame member stresses will develop, which are typically unaccounted for by the designer. This condition also results in construction difficulties because the girders must be twisted and pulled with cranes in the field to fit with the cross-frames, and this twisting can be resisted by stiff flanges or result in buckling of slender flanges.

Frequently the motivation for adopting an approach wherein inconsistent detailing of cross-frame members is embraced emanates from a concern regarding the perception of girder web out-of-plane rotations being excessive. Ironically, this perception comes about in the absence of hard evidence that points to the existence of such a problem. In fact, no such evidence can be found in the archival literature, and no guidance is given in the governing design specifications in use in the United States (2).

Through the consideration of two case studies, the present paper hopes to highlight potential pitfalls and shortcomings associated with the practice of inconsistent detailing within the context of larger bridge systems. One structure treated herein is the three-span continuous Ford City Veterans Bridge, which has a 322ft (98m) curved span with a radius of curvature of 511ft (160m). The other bridge is a curved and skewed simple span ramp structure, State Route 8002 Ramp A-1, with an approximate curved length of 150ft (46m) and centerline radius of 279ft (85m). The Ford City Veterans Bridge is located in Ford City, Pennsylvania, and the SR 8002 Ramp A-1 is located in King of Prussia, Pennsylvania.

NO-LOAD, STEEL DEAD LOAD, AND FINAL DEAD LOAD CONDITIONS

Within the current paper, a nomenclature is adopted that parallels that which has been noted to occur in practice. The no-load condition is defined as the theoretical condition in which the bridge is subjected to little or no deformations or stresses (i.e. the geometric configuration that the structure assumes once assembled but not acted upon by any external forces – including gravity). For horizontally curved I-girders, the no-load condition typically coincides with the girders webs being in a plumb position as a direct result of the design intent. The theoretical no-load condition can be practically approached during construction with the proper use of temporary supports and falsework (3). Similarly, in the fabrication shop, girders are blocked up at finite intervals along the girder length to achieve the desired camber and to simulate the no-load condition.

The steel dead load condition is defined as the theoretical state of the assembled steel structure under the action of steel self-weight gravitational forces (i.e. deck self-weight is assumed to not be present). The placement of steel dead load on a curved I-girder, initially in a web-plumb state, will cause the girder to immediately rotate to a girder web-out-of-plumb condition.

The final dead load condition is defined as the theoretical state of the assembled steel structure under the action of steel and concrete deck self-weight (i.e. full noncomposite dead load). The application of these dead loads will cause an initially web-plumb I-girder to rotate to a web-out-of-plumb position, and the rotation will be greater than the rotation due to steel dead load.

CONSTRUCTED BRIDGE DESCRIPTIONS

Ford City Veterans Bridge

The Ford City Veterans Bridge is a 1060ft (323m) three-span continuous steel I-girder bridge that spans the Allegheny River, shown in figure 1 (prior to deck placement). The bridge has 322ft (98m) end spans and a 417ft (127m) center span; the northernmost end span is curved. Figure 2 shows a plan view sketch of the curved section of the Ford City Veterans Bridge; also noted is the naming convention used throughout the remainder of this paper.

Figure 1. Ford City Veterans Bridge (prior to deck placement)

The Ford City Veterans Bridge consists of 44 individual, fabricated girder segments, utilizing 14ft (4.275m) deep I-girders spaced 13.5ft (4.1m) on center, aligned in four girder lines. The curved span has a mean radius of 511ft (160m), and ends just short of pier 1 by 29ft (8.7m). The concrete deck, out-to-out, dimension is 48.25ft (14.71m) and the deck thickness is 9.5in (240mm).

Cross-frames are typically spaced at equal radial intervals throughout the curved span (at the outside girder (G1), this cross-frame spacing is approximately 14.4ft (4.4m)). All of the cross-frames throughout the structure are of an "X" type geometry consisting of two diagonals and a top and bottom chord. The bridge is a longitudinal hybrid structure that employs HPS70W (HPS485W) steel at the negative moment regions over the piers and AASHTO-M270 Grade 50 (Grade 345) weathering steel throughout the remainder of the structure.

The girders are detailed to be in the web-plumb position at the no-load condition. However, the cross-frames were detailed to fit-up with girders in the web-plumb position at the steel dead load condition; an inconsistent combination obviously ensues. However, an error occurred in the detailing process, and the cross-frames were actually detailed to fit-up with girders in the concrete deck load position (i.e. concrete deck load applied only, which is not possible), nonetheless an inconsistent combination, by definition.

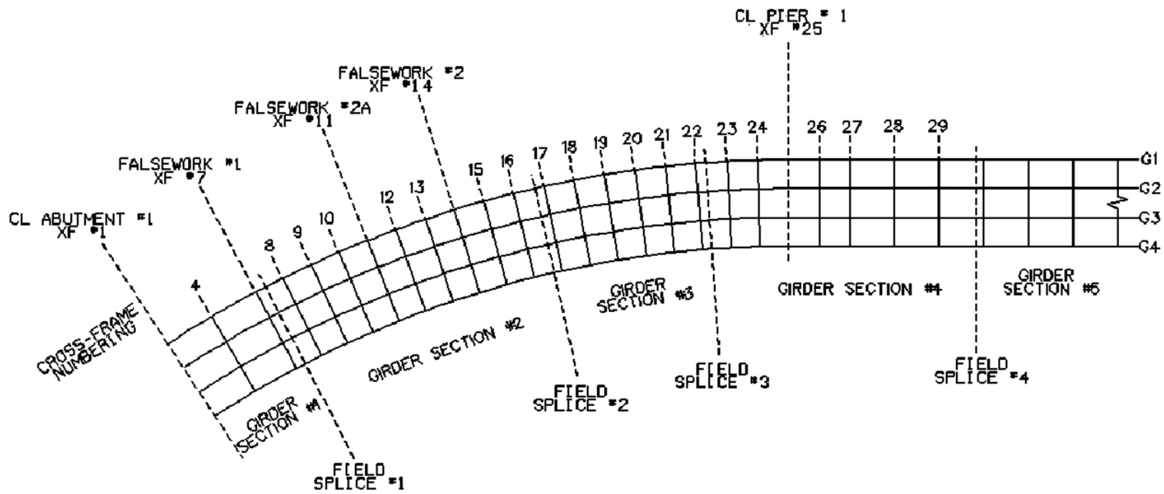

Figure 2. Framing Plan View of Curved Section of Ford City Veterans Bridge

As-built Bridge Erection Sequence – Ford City Veterans Bridge

The curved span erection of the Ford City Veterans Bridge consisted of the placement of 16 different girder field sections; each individually placed on account of size. Erection of each section of the curved span always began with the two interior girders, G2 and G3 individually. The construction staging began at the abutment and employed three falsework towers (Falsework #1, #2A, and #2 in figure 2) in between the abutment and the pier. Once the first two sections were completed, as shown in figure 3, the interior girders over the pier were placed as part of section 4. This erection sequence left a section of girders, section 3, which had to be "dropped" into place. It should be noted that there were no falsework bents under section 3, and that section 3 is at the mid-span of the curved section.

Figure 3. Ford City Veterans Bridge – Completed Steel Erection of Sections 1 and 2

In the field, during erection of this "drop-in" section, some difficulties occurred in placing pins that were part of the field-splices at both end of girder G3-3. Girder misalignments due to unpredicted lateral displacements at the top and bottom of the girder web led to difficulties in closing these field splices. Once girder G2-3 was placed next to G3-3, cross-frame connection misalignments, measuring approximately 1.5 inch (38mm), were observed. At this point in the construction, significant additional forces had to be applied to the structure in order to solve these girder and cross-frame misalignment problems. These same types of girder and cross-frame misalignment problems occurred when the fascia girders, G1 and G4, of section 3 were placed.

S.R. 8002 Ramp A-1

The State Route 8002 (SR 8002) Ramp A-1 bridge is a single span composite horizontally curved steel I-girder bridge (as shown in figure 4 after completion of steel erection). The northernmost abutment is radial (abutment 1), while the other abutment has a skew angle of 38.8 degrees; measured from the longitudinal axis of the structure. Figure 5 shows a sketch of the framing plan and the naming convention utilized in the current paper.

Figure 4. SR 8002 Ramp A-1

The centerline radius of the structure is approximately 279ft (85m), and the curved length of girder G1 is 141ft (43m) while the curved length of girder G6 is 162ft (49m). The concrete deck out-to-out dimension is 35.625ft (10.86m) and the deck thickness is 8in (200mm). The bridge consists of 6 girder lines equally spaced at 6.125ft (1.87m), with all girders having a constant web depth of 5.67ft (1.7m). Girders and cross-frames are fabricated form A709 Grade 50 steel.

In general, the cross-frame spacing varies throughout the structure, as shown in figure 5, but near the mid-span of the structure the radial spacing remains fairly constant at 10.417ft (3.175m). Intermediate cross-frames have an "X" type geometry consisting of two diagonals and a top and bottom chord.

The girders are detailed to be in the web-plumb position at the no-load condition. The intermediate cross-frames are detailed to fit-up with girders in the web-plumb position at the steel dead load condition, an inconsistent combination.

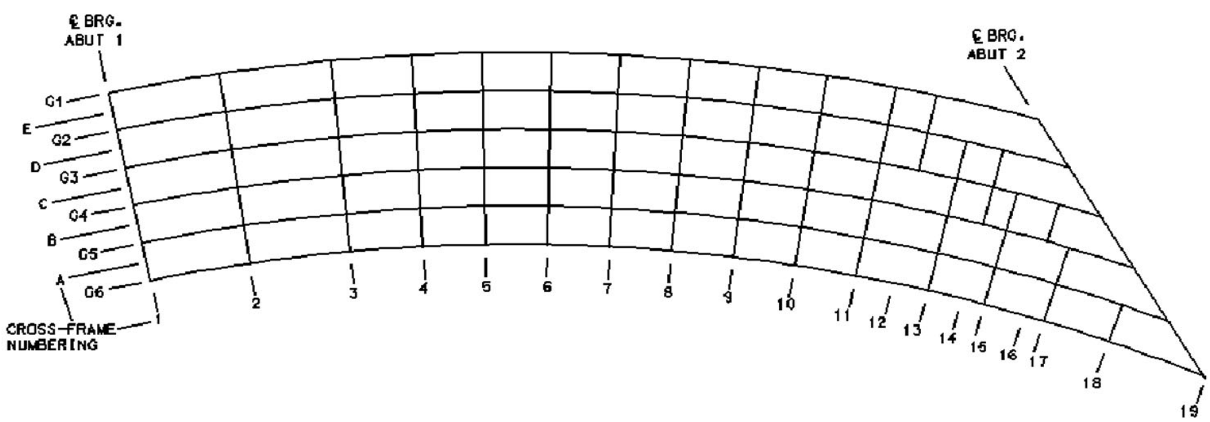

Figure 5. Framing Plan View of S.R. 8002 Ramp A-1

As-built Bridge Erection Sequence – S.R. 8002 Ramp A-1

In general, erection began with the outermost girder, G1, and progressed towards the interior of the curve; terminating with the innermost inside girder, G6. The field-splice for each girder was completed on the ground with the girder blocked to its correct no-load camber using cribbing. Each complete girder was subsequently picked with a crane and then placed on the abutments; temporary supports were not used for the erection of the structure. Rather, temporary tie-downs and a 6-ton come-along at each abutment for girder G1, were used during the steel superstructure erection. Additionally, two 130 ton lifting cranes, a 60 ton holding crane, and a boom truck were also employed during the steel erection of SR 8002 Ramp A-1.

The steel erection began with the girders G1 and G2 being placed. After placement, girder G1 was held with the 60 ton holding crane, while G2 was held in place with the two 130 ton lifting cranes. Figure 6 shows the use of the holding crane to hold G1 in place, as the two lifting cranes erect girder G2.

Figure 6. SR 8002 Ramp A-1 - Holding Crane Attached to G1 and Lifting Cranes Erecting G2

The boom truck was then used to erect cross-frames at the abutments and at four intermediate locations. At this time, the intended erection sequence guidelines stated that all of the cranes could be released. However, in the field it was realized that the girders would become unstable if this were attempted. Therefore, the holding crane had to be kept in place; remaining attached to girder G1. In fact, the holding crane was kept in place throughout the erection of the first four girders due to the relative instability in the sub-assemblage as construction progressed. This instability in the field was shown to be consistent with the results from the analytical study of the erection sequence (Chavel and Earls 2004). This result indicates the need for more robust analysis practice when evaluating alternate erection strategies for a given bridge system.

FINITE ELEMENT MODELING

Nonlinear finite element models of the Ford City Veterans Bridge and SR 8002 Ramp A-1 are created, using the commercial software package *ABAQUS* (1). These finite element models are subsequently used to study the behavior of the bridges during erection and illustrate the difference in cross-frame dimensions resulting from inconsistent detailing methods. The finite element techniques used in these studies were verified with results from a previous curved I-girder erection study (Chavel and Earls 2001). The nonlinear finite element models of the Ford City Veterans Bridge and SR 8002 Ramp A-1 are subsequently created employing these verified modeling techniques.

Experimental data from the Federal Highway Administration's (FHWA) Curved Steel Bridge Research Project (CSBRP) erection study ES1-4, as presented by Linzell (6), was used as the basis of the verification of the modeling techniques employed in the current study. Favorable agreement was shown between the results of the experimental study and the finite element results obtained using the modeling techniques adopted herein.

The finite element models of the Ford City Veterans Bridge and SR 8002 Ramp A-1 were subsequently based on the techniques vetted as part of the referenced verification study (3). While this cited reference provides a much more detailed treatment of the modeling strategies employed in the current work, a brief summary is useful to consider. In the present study, the *ABAQUS* S4R (4-noded, reduced integration, shear deformable) shell element is used to model the plate girder webs, flanges, transverse stiffeners, longitudinal stiffeners, and full-depth cross-frame connection plates. The cross-frame members are modeled with *ABAQUS* B31 beam elements (2-noded, 3-D linearly interpolated isoparametric beams). The cross-frame members are attached to the girders, at their specified workpoints, using the *ABAQUS* multipoint constraints (MPC) TIE and LINEAR commands. The cross-frames and girders used in every model for the current program of research are proportioned for the web-plumb position at the no-load condition. Further details regarding the modeling strategies employed and the erection sequencing studies for each bridge can be found in the related technical reports (3, 4).

STEEL DEAD LOAD DISPLACEMENTS

Upon application of gravity loading of any kind, complete horizontally curved steel I-girder superstructures will displace vertically and rotate out-of-plane and in the direction facing away from the structure's center of curvature. For the current research, the out-of-plane displacement is considered to be positive when the displacement is directed towards the outermost girder. In addition, the vertical displacement is considered to be negative when the girders displace downward. Furthermore, typical in curved steel I-girder structures, there will be a differential vertical displacement between adjacent girders. The girder on the outside of the curve will have a larger vertical displacement than all of the other girders, while the girder at the inside of the curve will have the smallest vertical displacement. However, the out-of-plane rotation, and out-of-plane displacement along the girder top flange, will be essentially the same from one girder to the next because the entire steel superstructure will rotate as one unit due to the presence of the required cross-framing in between the girders.

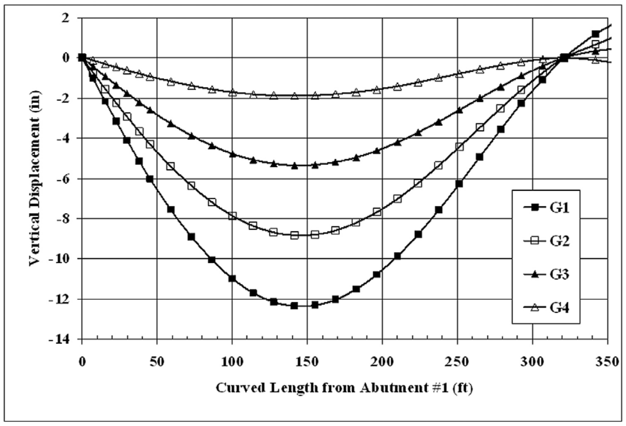

Figure 7. Ford City Veterans Bridge Steel Dead Load Vertical Displacements

Ford City Veterans Bridge Steel Dead Load Displacements

The entire steel superstructure of the Ford City Veterans Bridge is analyzed under the application of steel dead load. The steel dead load vertical displacement profile for the girders in the curved span of the Ford City Veterans Bridge is shown in figure 7. As is to be expected, the largest vertical displacement predicted by the finite element model occurs at the mid-span of the outer-most girder, G1. The vertical displacement at this location is predicted to be 12.4in (314mm), and the average differential displacement between the girders at the mid-span is 3.5in (90mm). Additionally, the top flanges of the girders displace in the out-of-plane direction by essentially the same amount from girder to girder under the action of steel dead load; the maximum displacement of 5.2in (131mm) occurs at the mid-span of the curved portion.

S.R. 8002 Ramp A-1 Steel Dead Load Displacements.

The vertical displacement caused by steel dead load, as predicted by the finite element model of S.R. 8002 Ramp A-1, is shown in figure 8. The largest maximum vertical displacement of 3.0in (80mm) occurs near the mid-span of the outer-most, girder G1. The maximum differential vertical displacement between adjacent girders is 0.41in (10mm). Also, as shown in figure 4, due the effects of the skewed end, the maximum vertical displacement of each girder occurs near the mid-span of that particular girder. Additionally, the maximum mid-span top flange displacement in the out-of-plane direction under the action of steel dead load is 0.44in (11mm).

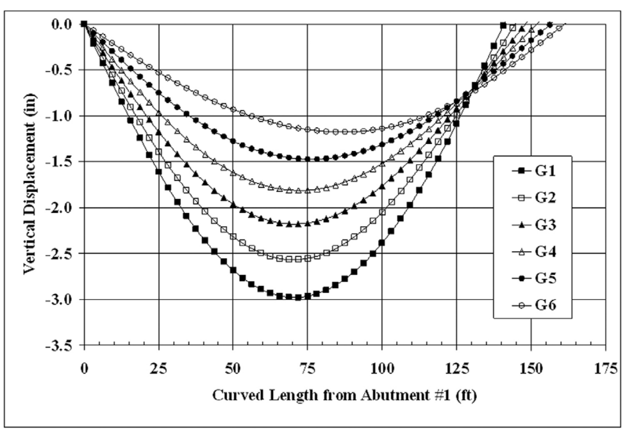

Figure 8. SR 8002 Ramp A-1 Steel Dead Load Vertical Displacements

In both of these bridge structures, intentional cross-frame misfits were introduced during fabrication as a means for ameliorating the predicted out-of-plane dead load deflections in the steel superstructure.

DETAILING OF GIRDERS AND CROSS-FRAME MEMBERS

Inconsistent detailing frequently arises when the rotation of the girders, and the subsequent lateral deflection of the top flange, due to dead loads, are deemed to be of such a magnitude as to constitute a problem for the owner (whether real or merely perceived). In cases such as these, the ability to have the girder webs vertically plumb at the steel dead load condition, or even the final dead load condition, is sought. In an effort to achieve a web vertical condition under dead load, it is often the case that the cross-frames are detailed for the web-plumb position at the steel dead load condition, while the girders remain conventionally detailed for the web-plumb position at the no-load condition; thus creating a detailing inconsistency since the cross-frames will not fit in between the girders. This, of course, results in the bolt holes in the cross-framing not matching up with the hones in connector plates on the girders. Nonetheless, it is then hoped that detailing the cross-frames in this manner will force the girder webs to be vertically plumb at the steel dead load condition. However, this detailing inconsistency will result in cross-frame members that will either be too long or too short to connect with girders detailed and fabricated for the web-plumb position at the no-load condition. Depending on the size of the bridge, this situation can be highly problematic as a result of the significant additional forces that must be introduced during bridge erection to correct this lack of fit between members.

There are three methods that could be used to detail girders and cross-frames in horizontally curved I-girder bridges: the girders and cross-frames could be detailed to be in the web-plumb position while under the no-load condition; the girders and cross-frames could be detailed to be in the web-plumb position under a prescribed load condition (steel dead load or final dead load); or the girders and cross-frames could be inconsistently detailed.

Detail Method #1 – Girders and Cross-Frames Detailed to Web-Plumb at No-Load

In this method, the girders and cross-frames are detailed so that the girder webs are vertically plumb at the no-load condition (a condition approachable in the field through the use of temporary supports and falsework). Figure 9 shows a general cross-sectional view of a curved I-girder bridge systems with girders and cross-frames detailed for the web-plumb position at the no-load condition (the dashed lines in the figure).

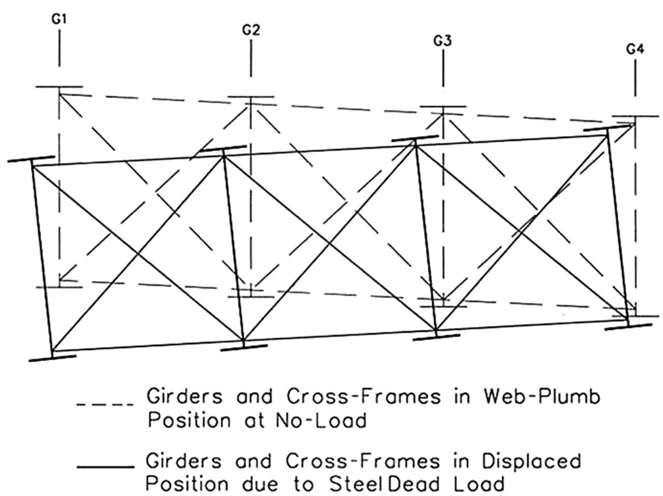

Figure 9. Girders and Cross-frames Detailed for Web-Plumb at the No-load Condition (Ford City Veterans Bridge Cross-section shown)

Upon the removal of the temporary supports in the field, after completion of steel erection, the structure will displace due to steel self-weight, and the girder webs will no longer remain plumb; the removal of the temporary supports results in the application of the steel dead load condition, and hence the structure is no longer in the no-load condition. Figure 9 shows a magnified view of the bridge cross section after application of steel dead load.

Theoretically, the structure will remain in an out-of-plumb position for the remainder of its service life (the girder rotations will subsequently increase with the placement of the concrete deck and live load). Currently, no guidelines exist as to what a maximum allowable girder rotation due to steel self-weight ought to be.

Detail Method #2 - Girders and Cross-Frames Detailed to Web-Plumb Position at Steel Dead Load (or Final Dead Load)

The girders and cross-frames can be detailed to a web-plumb condition under steel dead load, thus causing the girder webs to be out-of-plumb in the no-load condition. Upon application of steel dead load, the girders will rotate to a plumb position. Detailing the cross-frames and girders for web-plumbness under the final dead load condition is the same as detailing for steel dead load condition, except that the displacements due to the final noncomposite dead loads (steel plus concrete) are used. However, the current paper will mainly discuss detailing for the steel dead load condition since that is the case most focused on in current practice.

The first step in Detail Method #2 is to determine the displaced position of the bridge due to steel self-weight, (this can be obtained directly from the final displaced structure of Detail Method #1, as shown in figure 9). The girders are then geometrically rotated back to a web-plumb condition; as independent rigid bodies pivoting about the bottom flange-web junction. This rotation is basically equal from girder to girder because the entire cross-section rotates by the same amount under the action of steel self-weight due to the presence of cross-framing. With the girders in a vertically plumb position, new cross-frame member lengths can be determined, utilizing the same work-points on the girders as used for Detail Method #1. Therefore, since the girder rotation is reversed, the length of the cross-frame members is dependent on their vertical displacement of each girder, which is typically shown on the bridge design drawings.

The bridge cross-section is then rotated back by the same angle it displaced through under steel self-weight, as shown in figure 10. Basically, the vertical and lateral displacements are "reversed," such that the midpoint of each bottom flange is in the same location as they were for the no-load condition. This is the starting position of bridge erection for girders and cross-frames detailed to be web-plumb at steel dead load. Theoretically the girders will then rotate to a vertically plumb position, along the entire length of the girder, once steel dead load is applied, as shown in figure 11.

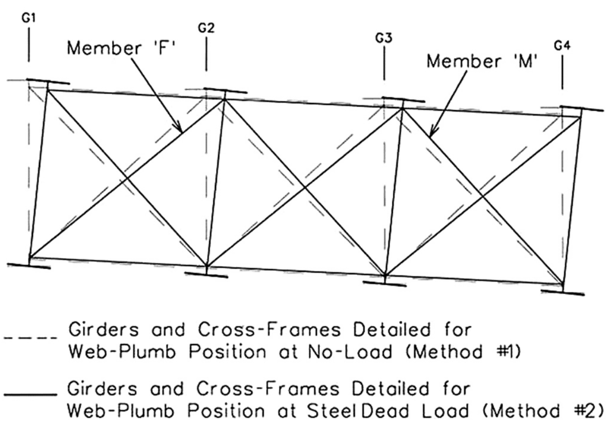

Figure 10. Starting Points of Bridge Erection for Both Detailing Methods (Ford City Veterans Bridge Cross-section shown)

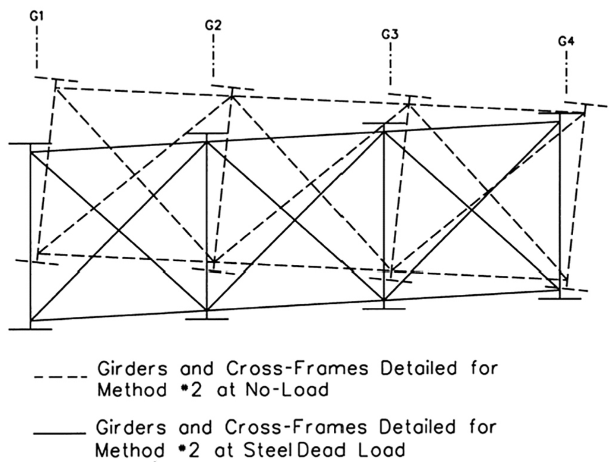

Figure 11. Final Position of Girders Detailed for Method #2 (Ford City Veterans Bridge Cross-section shown)

Since the girder rotation due to steel self-weight varies along the curved length, each cross-frame has to be detailed specifically for its longitudinal position. However, the girder rotation due to steel self-weight is the same for all of the girders in a given cross-section. In addition to specifically detailing each cross-frame for Detail Method #2, the girders must be given special attention as well. The girder rotation due to steel self-weight must be built into the webs in order to have a consistent combination of detailed girders and cross-frames. A consistent combination cannot be achieved in the field by simply tilting the girders inward; pre-twisting is required. Even though this pre-twisting of the girder webs can be fabricated, it rarely occurs in practice due to the associated difficulty and cost.

Detail Method #3 - Inconsistent Detailing

The typical detailing inconsistency occurs when girders are detailed for the web-plumb position at no-load (Detail Method #1), and cross-frames are detailed for the girder web-plumb position at steel dead load (Detail Method #2). The inconsistency associated with this detailing combination results in diagonal cross-frame members that are either too long, member 'F' in figure 10, or too short, member 'M' in figure 10. This can be seen by comparing the initial cross-frame positions for Detail Method #2 (Figure 10), with the initial position of the girders for Detail Method #1 (Figure 9). During bridge erection, the cross-frames detailed for web-plumb at steel dead load will have to be forced into girders detailed for the web-plumb position at no-load.

The difference in diagonal cross-frame member length resulting from inconsistent detailing varies along the curved length of the bridge, since the girder out-of-plane rotation and vertical deflection vary in a similar fashion. As shown in figure 12, the connection misalignment resulting from inconsistent detailing of the Ford City Veterans Bridge varies along the curved length of the steel superstructure, with the maximum misalignment occurring near the mid-span of the curved section. At the mid-span of the curved section, diagonal member 'F' is 2.4in (61mm) too long, and diagonal member 'M' is 2.4in (61mm) too short (cross-frame member designation, 'M' or 'F', is the same for Ford City Veterans Bridge as used in the cross-section of SR 8002 Ramp A-1, as shown in figure 10). The connection misalignment, along the length of the curve, resulting from the inconsistent detailing of S.R. 8002 Ramp A-1 is shown in figure 13. As with the Ford City Veterans Bridge, the maximum misalignment occurs at the mid-span of the horizontal curve, but in this case it is only approximately 0.21in (5.4mm).

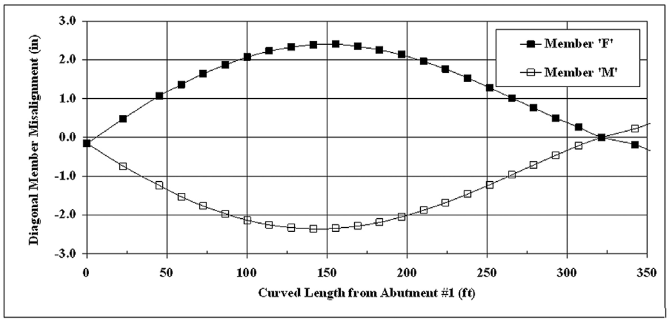

Figure 12. Ford City Veterans Bridge Cross-frame member misalignment for girders detailed per Method #1, and cross-frames detailed per Method #2

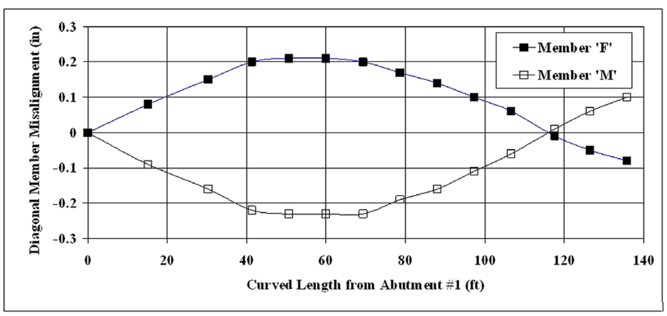

Figure 13. SR 8002 RampA-1 Cross-frame member misalignment for girders detailed per Method #1, and cross-frames detailed per Method #2

As shown in figures 12 and 13, there is a significant difference between the cross-frame misalignments associated with inconsistent detailing of each structure. In the case of the Ford City Veterans Bridge, the girders have a web depth of 14ft (4.275m), and the curved span length is 322ft (98m). Due to the size of the girders and span length, the steel dead load causes significant girder vertical displacement, 12.4in (314mm) at G1; as well as significant differential vertical displacement between girders, 3.5in (90mm) (see figure 12). In regard to SR 8002 Ramp A-1, the girders have a web depth of only 5.67ft (1.7m) and an average span length of 150ft (45.7m), therefore the steel dead load is not very large and hence the vertical displacements due to steel dead load are not very large (see figure 13). Furthermore, the maximum differential vertical deflection between adjacent girders is only 0.4in (10mm).

Therefore, it is evident that the driving force behind cross-frame misalignments due to inconsistent detailing are the girder vertical displacements and the differential displacements between girders caused by the horizontal curvature. In larger type bridges such as Ford City Veterans Bridge, the cross-frame connection misalignments caused by inconsistent detailing will be much greater than the cross-frame connection misalignments in a smaller type bridge such as SR 8002 Ramp A-1. In addition, the smaller girders are also much easier to force into different configurations at the time of construction using modest crane and "come-along" forces.

CONSEQUENCES OF INCONSISTENT DETAILING

Due to the fact that inconsistent detailing results in cross-frame members that are too long or too short to fit-up properly with the girders, bridge construction difficulties can arise. The girders and cross-frames in the bridge will have to be forced into place during steel erection because of the misalignments that inconsistent detailing creates. The structure will become increasingly difficult to erect as misalignments due to inconsistent detailing become larger. This is shown through a comparison of the construction difficulties observed during the erection of the Ford City Veterans Bridge and SR 8002 Ramp A-1.

Ford City Veterans Bridge

As discussed previously, in the case of the Ford City Veterans Bridge, an error occurred in the detailing process, and the cross-frames were incorrectly detailed fit girder webs made to be plumb under the application of concrete deck load only (a situation that is impossible to obtain due to the presence of steel dead load). For this detailing case, the diagonal cross-frame member misalignment is +/- 1.25in (32mm) at the mid-span of the curved section, instead of +/- 2.4in (61mm) due to detailing the cross-frames for steel dead load (the condition that would have occurred in the field were it not for the fortuitous mistake mentioned above). Nonetheless, the actual inconsistent detailing in the structure led to significant difficulties during construction where cross-frame misalignments of 1.5in (38mm) were observed in the field, as compared to 1.25in (32mm) misfits predicted via the finite element model.

In addition to the cross-frame connection fit-up problems, misalignments developed at the girder field-splice locations due to unanticipated lateral and torsional displacements resulting from the inconsistent detailing. Unanticipated lateral displacements effected the completion of field-splice #2 of the Ford City Veterans Bridge, with the erection of girder G3 section 3 (G3-3), which was a drop-in section between previously erected girders in section 2 and 4. Figure 14 shows the erection of girder G3-3, with field-splice #2 on the left side of the photo. Based on field observations, the completion of this field-splice was complicated due to unanticipated lateral displacements at the end of previously erected girder G3 section 2 (G3-2, refer to figure 2). Similar complications with the field-splices also developed during the erection of girder G2 section 3 (G2-3). The analytical investigation of the erection sequence, using a ***consistently detailed*** finite element model, showed that there should have been no significant displacements at the end of girder G3-2 at field-splice #2 (3). Therefore, the field-splice connection misalignments observed in the field can be directly attributed to the inconsistent detailing that occurred in the Ford City Veterans Bridge.

Figure 14. Ford City Veterans Bridge – Erection of Girder G3-3

Very large additional external forces had to be applied to the superstructure components in order to bring the cross-frames and girders into alignment. Forces were applied with the available cranes and jacking devices at the construction site. One 200 ton capacity crane, equipped with a 200ft boom and 88kip counterweight, and three 4-wheel-type cranes were available at the job site. However, a significant amount of additional construction time was required to bring the components of the curved section into alignment. For example, according to field records, field-splice #2 for girder G2 was not fully completed until six days after girder section G2-3 was erected. Since very large external forces were applied to the girders and cross-frames to affect fit-up, stresses were induced in the structure that could not have been accounted for by the bridge designer.

Additionally, it was observed in the field that the top flanges of the girders in the curved section were not at the predicted top of steel elevations, as shown in the bridge design drawings, upon completion of the steel erection. Near the mid-span of the curved section (curved length from abutment #1 is approximately 157ft (48m)), the field surveyed elevation is approximately 4in (100mm) higher than the predicted elevation using the finite element model, as shown in Figure 15 (the analytical results closely match the elevations called out in the design drawings). This discrepancy in top of steel elevations had to be made up by reducing the haunches at particular locations along the girders so as to achieve the required top-of-deck elevations called out in the contract documents. Consuming haunch depth to make deck elevations is obviously not a desirable outcome since haunch depths are counted on to ensure proper clearances for under deck systems passing

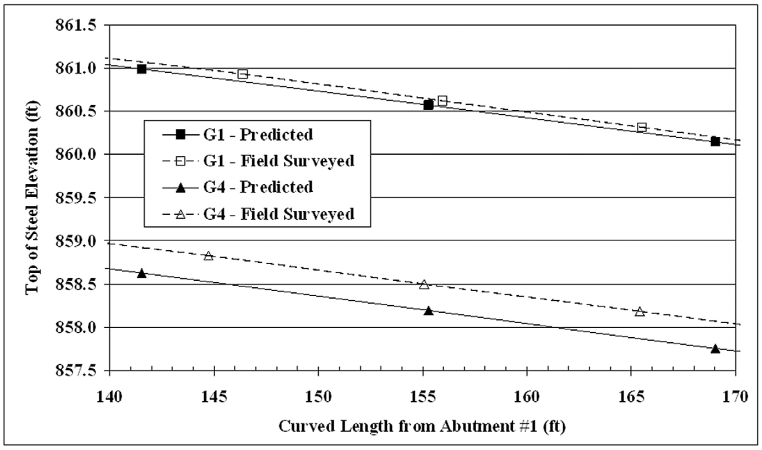

Figure 15. Field-Surveyed and Predicted Steel Elevations Near Span 1 Mid-span Prior to Deck Placement

between the steel superstructure and the pier caps. In addition, proper haunch depths are needed for desired functionality of pot bearings within the bridge system.

SR 8002 Ramp A-1

In the case of SR 8002 Ramp A-1, the misalignments due to inconsistent detailing are generally small, with a maximum cross-frame misalignment of 0.21in (5.4mm). It was noted that during steel erection of SR 8002 Ramp A-1, cross-frame misalignments related to the inconsistent detailing did not produce significant difficulties during steel erection. This can be attributed to the fact that the misalignments were indeed small, and the girders in the structure have a relatively small web depth and spacing, and therefore could be manipulated with much greater ease in the field.

OUT-OF-PLUMB GIRDER BEHAVIOR

Bridge components are typically inconsistently detailed because the rotation of the girders, and the subsequent lateral displacement of the top flange, due to applied or dead loads, are subjectively deemed excessive by the bridge engineer or owner. In many cases the bridge engineer or owner may believe that a girder with a web that is not plumb is not sufficient for service conditions; vis-à-vis in relation to required girder capacity and acceptable bending stresses. This assessment is made without any guidance from current design specifications (2) because no such guidance currently exists concerning the limits of web rotation. Beyond this, there is not even useful information in the literature concerning any limits on out-of-plane web rotation.

On-going research at the University of Pittsburgh is aimed at addressing the behavior of horizontally curved I-girders in an out-of-plumb position, through the use of experimentally verified nonlinear finite element modeling techniques. An evaluation of the structural capacity of an individual straight and an individual curved simple-span steel I-girders rotated out-of-plane has been carried-out, and comparisons to similar I-girders, that are not rotated out-of-plane, have been reported on (5). The straight I-girder investigated in the study has a span length of 12ft (3.7m) and a web depth of 17in (432mm), while the curved I-girder has a span length of 16.4ft (5.0m), a radius of curvature of 65.6ft (20m), and a web depth of 12in (307mm).

Initial results of the referenced study show that the ultimate load capacity of the beam does reduce as the out-of-plane rotation angle increases. However, this reduction is quite small, such that when the subject girders are rotated out-of-plane 2 degrees, the ultimate load capacity only reduces 0.8% for the straight girder, and 1.1% for the curved I-girder. In practice, curved steel I-girder bridges may experience an out-of-plane rotation of approximately 1 to 2 degrees (3, 4). However, it is observed that the flange tip stresses do increase much more significantly as the subject girders are rotated out-of-plane. For a web out-of-plane rotation of 2 degrees, the straight I-girder top flange tip stress increases by 8.0%, and the curved I-girder top flange tip stress increases by 3.0%. Therefore, in the case of design, it may be appropriate to consider the increased stress caused by the girder being in an out-of-plumb position. Conversely, it may not be necessary to consider the reduced structural capacity caused by the girder being in an out-of-plumb position.

Additional analytical investigations are currently being carried out that include a study of allowable girder web rotations for a single typical bridge size girder, as well as an investigation in regard to the allowable web rotation of a two-girder curved steel superstructure. This study will provide further data in regard to the capacity and flange stresses associated with individual girder, and girder systems, that are rotated out-of-plumb. If results are similar to those observed in the previous single girder investigations, more conclusive recommendations may be made in regard to the significance of an out-of-plumb girder and its effect of the design and detailing of the subject girder in a bridge structure. That is, is it necessary to inconsistently detail girders and cross-frames to limit the out-of-plane girder rotation due to dead loads; or can the girders both be detailed for the web-plumb position at the no-load condition, and then be allowed to rotate naturally to an out-of-plumb position after application of dead loads?

CONCLUSIONS

The objective of this research has been to promote awareness of the issue of inconsistent cross-frame and girder detailing. The concept of inconsistent detailing can be an extremely critical issue in relation to the successful erection of a horizontally curved steel I-girder bridge. Inconsistent detailing will result in cross-frame members that are of an incorrect length to fit-up with their girders. The magnitude of connection misfits due to inconsistent detailing is directly proportional to the differential displacement of adjacent girders under the action of steel dead load.

Consequently, in smaller type structures, such as SR 8002 Ramp A-1, where the dead load vertical and differential displacements between adjacent girders is not significant, cross-frame/girder misalignments due to inconsistent detailing may be overcome during bridge erection without causing significant problems. However, in larger type structures, such as the Ford City Veterans Bridge, where the dead load vertical and differential displacements between adjacent girders is quite large, cross-frame/girder misalignments due to inconsistent detailing will cause significant problems and delays during bridge erection.

Bridge engineers may wish to pay very close attention to the issue of inconsistent detailing when designing curved steel I-girder bridges in order to ensure construction difficulties and delays will not result from the practice. Furthermore, it may be necessary for the bridge engineer to investigate the locked-in stresses, and final girder positions that will result if inconsistent detailing is specified in larger curved steel I-girder bridge structures. Current specifications to do not address the issue of inconsistent detailing and allowable girder web rotation due to dead loads and thus indirectly help to motivate the problematic practice of inconsistent detailing.

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