CHALLENGING SKEW: HIGGINS ROAD STEEL I-GIRDER BRIDGE OVER I-90

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BIOGRAPHY

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SUMMARY

The reconstructed Higgins Road Bridges over I-90 are twin structures located approximately 30 miles northwest of Chicago. The structures span over a heavily traveled section of I-90 that carries 118,000 vehicles per day. Each bridge cross section consists of six equally spaced steel plate girders which support the 49’-3” wide cast-in-place concrete deck that is designed to carry three lanes of arterial traffic. Both bridges have two 280-foot spans with a total bridge length of 560 feet. The most unique feature of these bridges is that the median pier and abutments are skewed 70 degrees. The superstructure consists of 114 inch deep plate girders and includes full-depth diaphragms at the pier and abutments. The severe skew led to a complicated design that resulted in a unique framing arrangement and high lateral loads and displacements that must be resisted by the bearings and substructure.

In a bridge with such a severe skew, particular attention must be given to the fit condition that is specified on the design plans and used by fabricator. In the case of the Higgins Road bridges, the Steel Dead Load Fit (SDLF) was chosen, in which the girder webs are theoretically vertical after all of the steel is erected. The choice of SDLF was made to accommodate the limited windows for steel erection over the I-90 corridor. Additionally, because SDLF was chosen in lieu of Total Dead Load Fit, the design of the structure took into account the out-of-plane rotation caused by the concrete deck and composite loads.

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INTRODUCTION

The Illinois Tollway's 15-year $12 billion *Move Illinois* Program includes the 62-mile-long Jane Addams Memorial Tollway (I-90) Rebuilding and Widening Project. The project necessitates that the cross road bridges over I-90 be reconstructed with longer spans to accommodate the widened main line section. One such crossroad over I-90 is Higgins Road (Illinois Route 72) in Hoffman Estates. Within this segment, the project is providing four traffic lanes in each direction along I-90 with a widened median shoulder to accommodate future bus service. The reconstructed Higgins Road Bridges over I-90 are twin structures located approximately 30 miles northwest of Chicago. The structures span over a heavily traveled section of I-90 that carries 118,000 vehicles per day. Each bridge cross section consists of six equally spaced steel plate girders which support the 49'-3" wide cast-in-place concrete deck that is designed to carry three lanes of arterial traffic. Both bridges have two 280-foot spans with a total bridge length of 560 feet. The most unique feature of these bridges is that the median pier and abutments are skewed 70 degrees. The superstructure consists of 9'-6" deep plate girders and includes full-depth diaphragms at the pier and abutments. The severe skew leads to a complicated design that results in a unique framing arrangement and high lateral loads and displacements that must be resisted by the bearings and substructure.

EXISTING BRIDGES

The first of the Higgins Road dual bridges was constructed in 1957, and as traffic demands grew, the second parallel bridge was constructed in 1978. The designers of the first bridge had a unique approach to accommodate the skewed crossing abiding by the principle of avoiding the skew where practical. This was accomplished for the first structure with 5 simple spans and minimal structure skew for a total structure length of 474 feet. The superstructure consisted of five, 60" web built-up riveted plate girders with a 7½" thick reinforced concrete deck supported by counterfort wall type abutments and transverse girders. The abutments were squared off with respect to Higgins Road rather than parallel to I-90. The intermediate piers consisted of two sets of steel transverse girders and steel tie girders. The transverse girders provided four intermediate supports at a 20 degree skew with respect to Higgins Road but were considered fracture critical members. Each set of two parallel transverse girders was connected with two tie girders to form a square plan layout. The transverse girders were 96" web built-up riveted plate girders. Each set of pier girders were supported by concrete columns on pile supported footings. The unique substructure arrangement of this bridge led to many conflicts between the original pile foundations and the proposed bridge foundations.

The designers of the second original parallel bridge utilized 3 continuous spans but decided to avoid the fracture critical issues of the transverse support girders and designed the structure with a 70 degree skew. The superstructure consisted of six, 81" web plate girders with bottom lateral bracing between the fascia and first interior girders. Span lengths of this structure were 186 feet, 186 feet, and 131 feet. The superstructure was supported by a counterfort wall type abutment at the north end and a stub abutment at the south end. Both the median pier and south shoulder pier were multi-column type.

Figure 1: Existing Higgins Road Bridges

PROPOSED BRIDGE CONSIDERATIONS

The design and construction of the new Higgins Road Bridges first had to consider the proposed main line section. A 15'-2" median shoulder was required to accommodate a future bus lane. A 15-

foot wide outside shoulder was required along the I-90 corridor to accommodate the traffic control needs during construction. Additionally, a 5-foot wide snow storage area was needed in excess of the outside shoulder width. Due to the long span lengths that resulted from the main line section along the severe skew, closed wall type abutments were chosen to minimize the overall 560-foot bridge length. Alternatives to minimize the structure skew were considered but were constrained greatly by the inability to alter the orientation of the median pier and the dramatic increase in span lengths by squaring up the abutments. Reorienting the median pier would have required changes to the main line horizontal alignment since the median shoulder width could not be locally compromised. The resulting increased span lengths from squaring the abutments would also lead to significantly more bridge deck area and increased structure depths that would require a greater profile raise. A greater profile raise would lead to the need for retaining walls along the bridge approaches to stay within the Higgins Road right-of-way. Impacts to adjacent properties were especially sensitive and property acquisition was not an option. Lastly, construction of the new bridges had to carefully consider the traffic volumes along I-90 and the corridor traffic control scheme. The volume of I-90 traffic required that all three lanes in each direction be kept open except for off-peak hours. This constraint dictated locations of temporary shoring towers that would be required for steel erection.

Since it was not practical to avoid the severe skew, the design of the twin bridges would require a refined analysis employing a three-dimensional finite element model in an effort to obtain the design forces, deflections, and out-of-plane effects that occur in skewed bridges that cannot be accurately predicted by one-dimensional (line girder) or two-dimensional (grid) analysis methods. The proposed Higgins Road bridges over I-90 are twin continuous plate girder bridges with two spans, each 280 feet long. The decks are 49'-3" wide and are proportioned to accommodate three lanes of traffic. The superstructure consists of 6 plate girders with webs that are 9'–6" deep and 13/16" thick. Flange thicknesses vary from 1½" at the abutments to 3" at the pier. Cross-frames between the deep girders are X-type with both bottom and top chords. Intermediate cross-frames were placed normal to the girders and were placed in discontinuous lines near the abutment and median pier. Full-depth diaphragms are orientated along the skew at the abutments and normal to the girders in the vicinity of the median pier. The full-depth diaphragms at the median pier intersected each fixed bearing that resists lateral force.

The substructure consists of a multi-column pier supported on four rows of battered piles as well as stub abutments on battered piles located behind nearly 600-foot long soldier pile walls along the I-90 outside shoulders. The expansion joints are located at each abutment. With anticipated movement in both the transverse and longitudinal directions, swivel type modular deck joints are used. The twin bridges were designed in accordance with 2012 AASHTO LRFD Bridge Design Specifications.

SUPERSTRUCTURE DESIGN

BEHAVIOR OF SKEWED STRUCTURES

Skewed supports significantly complicate the behavior of steel I-girder bridges by introducing alternate load paths and causing greater interaction between the main girders and secondary framing members. The effects of the support skew are more pronounced in the case of the Higgins Road bridges because of the combination of severe skew and long spans. Thus the design must specifically consider the alternate load paths and component interactions, as well as the mitigation of issues associated with fit-up, detailing methods, and fatigue performance related to distortion induced fatigue.

For intermediate diaphragms that are perpendicular to the girders, the diaphragms connect adjacent girders at different locations along the span of each girder. As a result, the diaphragms connect adjacent girders at locations where the vertical displacement of the girders due to loading will be different. The deflecting girders try to force a racking distortion of the cross-frames, but the in-plane racking stiffness of the cross-frames is quite large, so instead the cross-frames rotate and force the girders to twist about the longitudinal axis of the bridge. These twisting deformations are different at different points along the span since they are a function of the vertical displacement of the girders. This twisting induces torsion in the girders. In addition, significant forces occur within the cross-frame itself as it resists the racking deformation the girders are trying to apply (Coletti et. al 2011).

For example, consider Figure 2, and the intermediate cross-frame line noted as XF A. At this location, Girder G1 will vertically displace more than Girder G6 since the location of XF A along G1 is closer to midspan than the connected location of G6. Thus, the bridge cross-section rotates out-of-plane, and forces are developed in the cross-frame members. However in span 2, at intermediate cross-frame line XF B, the bridge rotates out-of-plane in the opposite direction of the rotation experienced along cross-frame line XF A. One can think of this behavior as "twisting a washcloth" with the center being at the pier.

In addition, cross-frames in skewed bridges offer alternate load paths for vertical loads and — depending on the severity of the skew, the overall proportions of the bridge and the specific configuration of the framing plan — the effects of this "secondary" stiffness in the transverse direction can be quite severe, and is often referred to as "nuisance stiffness effects" (Coletti et. al 2011). In bridges with high width-to-span-length ratios and severe skew, the cross-frames which frame toward the obtuse corners of the bridge can provide a transverse load path with significant stiffness. These cross-frames thus attract significant loads, often forcing designers to increase the size of the cross-frames, which also tends to increase their stiffness, which then causes them to attract more load, and the process can continue on until maximum cross-frame size limits are reached.

FRAMING PLAN

The aforementioned skewed bridge behavior characteristics had a significant role in design decisions for the layout of the framing plan and other component designs for the Higgins Road bridges. As can be seen in Figure 2, the cross-frames in the vicinity of each abutment are arranged in a staggered pattern. This staggered pattern greatly reduces the ability for alternative load paths to develop in this area since intermediate cross-frame lines across the bridge do not frame directly into a support. When a cross-frame line frames directly into a support, significant member forces caused by racking can develop because the one end of the cross-frame line is a support and does not displace vertically. Therefore, the staggered cross-frame pattern relieves the structure from these alternative load paths by reducing the transverse stiffness of the bridge, forcing the load to "travel" to the support via the girders. However, staggering of the cross-frames does change how they interact with the girders in terms of induced torsional response in the girders, adding the complication of more lateral point loads applied to the girders and potentially increased local flange lateral bending effects. These lateral load points and local flange bending effects are economically considered in the design of the girders.

In the vicinity of the pier, intermediate cross-frames are selectively omitted to reduce nuisance stiffness effects. Omitting select cross-frames in the vicinity of an interior skewed support not only reduces the transverse stiffness and the cross-frame member forces, but severs an undesired load path and forces the load in the girders to travel along the girder to the support, and not through the cross-frame to the support. However, similar to the use of a staggered cross-frame pattern, when cross-frames are intentionally omitted, consideration must be given to the overall stiffness of the structure, lateral flange bending effects, and ensuring that the girders are sufficiently braced to prevent lateral torsional buckling

Two lines of full-depth diaphragms are employed at the center of the bridge at the Pier, as shown in Figure 2. The two diaphragm lines intersect the fixed bearings at girders G3 and G4. These two diaphragms resist significant forces caused by the out-of-plane rotation of each span. Severing the load path at this location would not appropriately relieve the forces caused by the twisting of each span. Thus, full-depth diaphragms are used since truss type cross-frames were insufficient. Additionally, the full-depth diaphragms intentionally attract more transverse load due to their stiffness, which helps to reduce the transverse loads in other X-type cross-frames near the pier.

Additionally, full-depth diaphragms are utilized at the abutment supports. The length of the diaphragms at this location is approximately 23.5 ft per bay. Due to this length, K-type cross-frames are not realistic due to the shallow angle of the potential diagonals, and the potentially long unbraced length and large slenderness value of the bottom strut. The full-depth diaphragms are connected to the girders via bent stiffener plates.

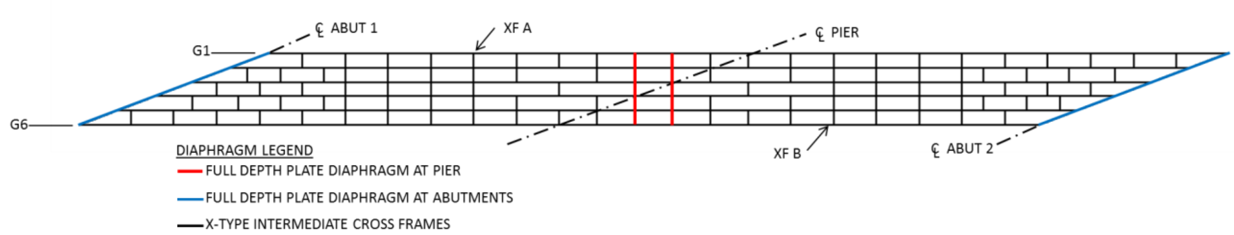

Figure 2: Framing Plan and Diaphragm Designation

The severe skew results in a conflict with the typical welded installation of the jacking stiffeners for future bearing replacement. The jacking stiffeners are located on the girder, forward of the stiffeners used to connect the end diaphragm, as shown in Figure 3. The conflict is associated with the ability to weld the jacking stiffener on both sides to the web after the end diaphragm stiffener connection plate is installed. Similarly, if the jacking stiffeners are installed first, there will not be access to the web on the one side of the diaphragm connection plate to make the web to plate weld. Thus, the design incorporates a bolted jacking stiffener, as shown in Figure 3, which can be installed in the field after the end diaphragm is placed. Additionally the end diaphragm flanges are coped so that they do not conflict with the jacking stiffener.

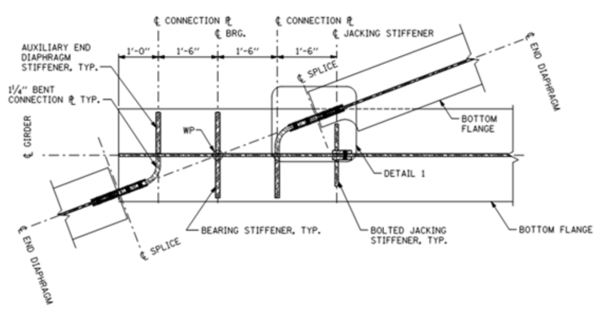

Figure 3: End Diaphragm Connection

It is important to note that it was most desirable to place the future jacking stiffeners on the girder and not on the end diaphragm. Locating these jacking stiffeners on the end diaphragms would result in much larger end connections of these end diaphragms, as the girder reaction would need to be transmitted through the connection during jacking operations. With long 280 ft spans, the dead load reactions are quite large. Given there is sufficient space on the abutment girder seat to locate a jack for future jacking operations (because of the severe skew), locating the jacking stiffener on the girder is much more reasonable and economical.

Most intermediate cross-frames utilize WT6x25 sections for all members of the X-type cross-frames. Near the pier, six cross-frames require larger members due the transverse forces resulting from the skewed bridge behavior. In these six special cases, WT members up to a WT13.5x73 are used.

3D FINITE ELEMENT ANALYSIS

With a severe skew of 70 degrees, this structure falls outside of the applicable range of any AASHTO live load distribution factor since all skew correction factor equations in AASHTO LRFD Section 4.6.2 have a range of applicability of 0° to 60°. Furthermore, per the FHWA Steel Bridge Design Handbook volume titled Structural Analysis, with a severe skew, the simplifications required for a 2D (grid) analysis may result in inaccurate results and a poor representation of the cross-frame/diaphragm forces, which are load-carrying members in skewed structures. Lastly, NCHRP Report 725 notes that 2D analyses may result in inaccurate results related to cross-frame/diaphragms forces and girder displacements for a bridge with the geometry of the Higgins Road bridges (White et. al 2012). Therefore, the final design of the steel superstructure employs several full three-dimensional (3D) models. The design used the LARSA finite element software package (see Figure 4).

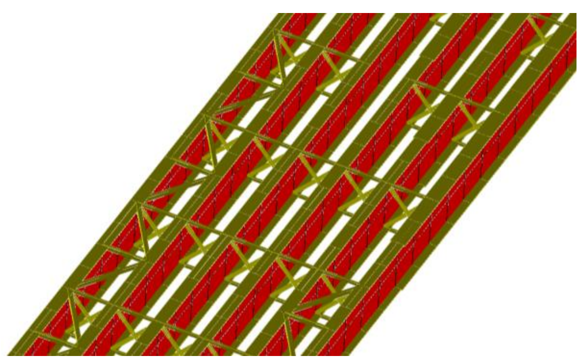

Figure 4: Non-Composite LARSA Model

MODELING ELEMENTS

The base, composite model of the structure is comprised of beam and shell elements. The girders were modeled with shell elements for the webs and beam elements for the flanges. The webs of the girders were comprised of two elements over their depth in order to both maintain a proper aspect ratio with the longitudinal joint spacing as well as to refine the mesh of the 9’-6” tall girder web.

The concrete deck was modeled with shell elements, which were divided into two elements between girders (oriented with the girders, not the skew). One shell element was used to represent the overhang portion of the deck. A rigid link, representing the shear studs, connected the top flange beam member to the deck shell element.

Cross-frames and lateral bracing members were defined as truss elements. Full-depth diaphragms were modeled similarly to the girders (beams as flanges and two shell elements over the depth of the web). Since the diaphragms are actually not as deep as the girders that they brace, but still connect to the girder top and bottom flange nodes in the model, a diaphragm with an equivalent moment of inertia was used in order to accurately obtain force effects.

Substructure support stiffnesses were determined and modeled as spring elements. Since the Higgins Road Bridge is a twin structure with identical superstructures, but differing pier heights, two different models with two different support stiffnesses were required. The structure with the shorter (stiffer) pier was used to determine force effects while the taller pier stiffness was used to determine maximum displacements.

In addition to the base, composite model, non-composite and long-term dead load models were created by removing the deck (and rigid links) or reducing the concrete deck modulus of elasticity, respectively. Also, short-term concrete models with certain portions of the deck removed were created to investigate deck pour sequences.

LOADING

Non-composite steel, non-composite concrete (deck and formwork), composite concrete (barriers – See Figure 5), and future wearing surface dead loads were applied to the appropriate models. Steel detail weight was applied as 6% of the structural steel weight.

In addition to standard dead loads, the deck pour sequence was analyzed by creating a geometric-nonlinear staged construction analysis that sequentially applied portions of the deck as non-composite loads, activated the shell stiffness and applied the next section. These results were used to check maximum non-composite loads in the structure.

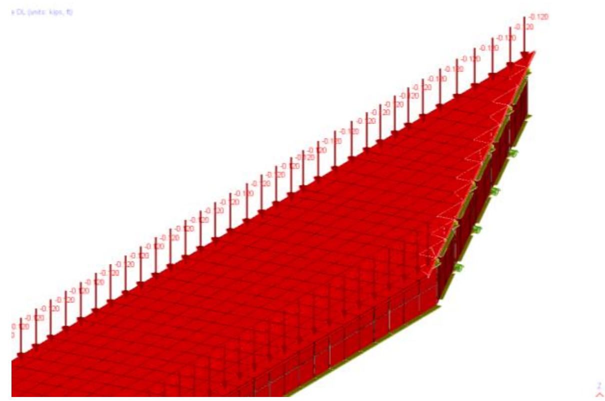

Figure 5: Composite LARSA Model With Barrier Load

Live load was applied to the composite model using a unit load influence surface and the LARSA software post-processor. The analysis placed the HL-93 (truck and tandem), fatigue truck, and deflection truck on the structure over the 46’ roadway width with the solver determining the controlling locations and maximum number of lanes (12’ width). Braking forces were also applied to the model and were combined with the vertical live load effects.

100 mph winds were applied as 50 psf pressure loads to the non-composite and completed, composite structure. Loads were applied at appropriate attack angles and then enveloped for design. Wind on live load and vertical wind loads were applied to the composite deck.

Thermal loads of ±80 ˚F were applied to the structure including all steel members and the concrete deck shell elements.

GIRDER DESIGN

The girders and all other elements were designed in accordance with the AASHTO LRFD Specifications with guidance from the Illinois Tollway Structure Design Manual.

Due to the span configuration, as well as the force effects due to skew, 9'-6" deep, constant depth girders were required to carry the design load (see Figure 6). The girder depth could not exceed this amount due to vertical clearance requirements for I-90 below.

In order to achieve economy with regard to fabrication and construction, all six girders used the same plate sizes and are symmetrical about the centerline of pier. Due to the skew effects, the design requirements at midspan did not vary greatly from the section adjacent to the abutment. Therefore, the same plate sizes are used for the entire positive moment region (218'). A field splice is used in this region to limit the member length to 120' for shipping and handling purposes. Plate widths were set in order to achieve a b/t ratio ≤ 24 (per AASHTO LRFD) for the most economical size to carry the induced stresses due to major and minor-axis moments.

For the stockier, negative moment region of the girder, which is 124' long, flange transitions are located 20' on either side of centerline of pier in order to economize the girder. For ease of fabrication, the flange width remains constant throughout the 124' length and the width was set in order to limit the maximum flange thickness to no more than 3" (for plate availability purposes).

The web thickness remains constant throughout the entire length of the girder and was sized large enough to eliminate the need for transverse stiffeners.

BEARING DESIGN

High load multi-rotational (HLMR) bearings were required for the Higgins Road Bridge due to the long span length and severe skew. Disc bearings were specified for the project. Pot bearings were considered, but it was determined that pot bearings were not a practical alternative for this particular bridge. Pot bearings can typically accommodate up to 0.05 radians of rotation. The design rotation at the abutments is in excess of 0.07 radians. The bearings were not designed for in-plane dead load rotations (about an axis transverse to the girder web) since the girders are cambered for dead load. However, out-of-plane dead load rotations (about an axis parallel to the girder web) are non-negligible and therefore are included in the design of the bearings. In the case of this bridge, the out-of-plane rotation due to dead load is a significant portion of the total design rotation. If pot bearings had been used, the sole plate would have needed to be beveled in both directions – in the longitudinal direction of the bridge to match the profile of the roadway and in the transverse direction to counteract the out-of-plane dead load rotations. However, a double-beveled sole plate was not considered practical for fabrication or installation. An additional complexity is the fact that every location is unique with respect to station and offset due to the skew. Two bridges with potentially 36 unique, double-beveled sole plates would be challenging and costly to fabricate and install properly.

The framing plan utilizes a combination of fixed bearings, guided expansion bearings, and non-guided expansion bearings. The arrangement is shown schematically in Figure 7. As is typical for curved and/or severely skewed steel bridges, careful consideration must be given to the bearing type and layout in the early stages of design in conjunction with determining the framing plan. The girder system and its resulting force and deformation effects are very sensitive to the support conditions. In particular, the fixed condition at the pier results in very significant horizontal force reactions at this support, which must be resisted by the pier and its foundation. The further the distance from the centerline of the bridge, the larger the magnitude of the horizontal reaction at the fixed bearing. Hence, the outermost bearings at the pier, at girder lines 1 and 6, were selected to be non-guided (free) expansion bearings. The horizontal reactions at the remaining 4 fixed bearings are still significant and were difficult to manage in terms of the pier design. In fact, the horizontal reactions at the G2 and G5 fixed bearings were similar in magnitude to the vertical reactions. Consideration was given to the option of providing just two fixed bearings at the pier, at girder lines 3 and 4. Although the maximum horizontal reaction at a single bearing decreased somewhat in this scenario, the option of having four fixed bearings was preferred for redundancy and geometry control. A schematic disc bearing design was provided in the plans along with the design loads, movements, and rotations.

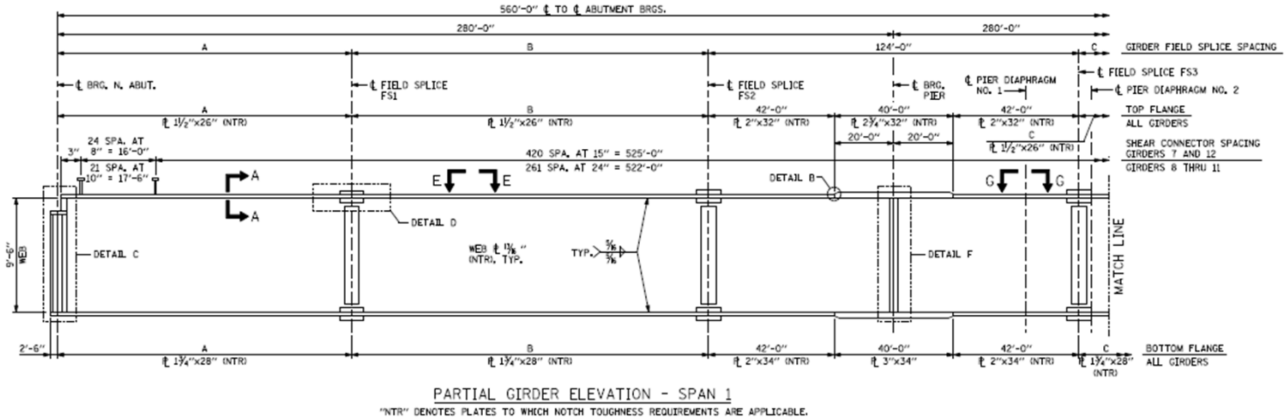

Figure 6: Girder Elevation

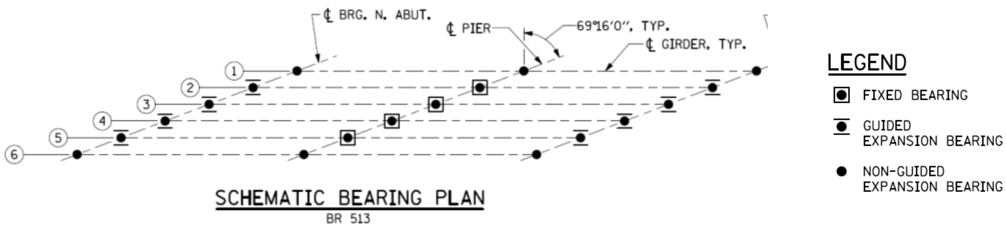

Figure 7: Schematic Bearing Plan

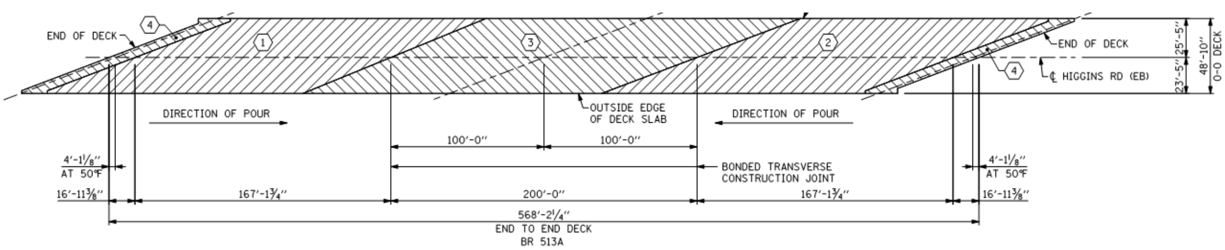

Figure 8: Deck Placement Sequence

DECK PLACEMENT ANALYSIS

A deck placement procedure is specified in the contract plans, as shown in Figure 8. The transverse construction joints are placed to be parallel with the skewed supports, and the end spans are to be placed before the concrete over the pier is placed. Placing the concrete deck in this sequence helps to reduce the potential for deck cracking in the negative moment region. Concrete is placed along the skew to the fullest extent possible, and any advance concrete placement is made with the use of a retarding agent. Placing the concrete along the skew results in similar loads and length of loads applied to all girders, therefore not exacerbating the differential deflection results related to the skewed supports.

The placement of the concrete deck is considered in the design of the girders for both checking the girder constructability limit state per AASHTO LRFD and for determining the girder camber. The 3D FE model is utilized to investigate the deck placement sequence. For this structure, the dead load deflection due to concrete varies significantly from the deflection assuming a single monolithic deck pour and the accumulated deflection due to the deck placement sequence. Therefore, the girder camber due to concrete dead load is based on the deck placement sequence and is noted as such in the contract drawings.

Additionally, the deck placement sequence was checked so that the maximum tensile stresses in the deck after each stage were less than 90% of the modulus of rupture of concrete. Using the 3D analysis model, the stress in the concrete deck is tracked for the three stages of the deck placement sequence. Verifying these deck stresses resulting from the deck placement sequence helps to reduce the potential for cracking of the concrete deck during deck placement.

CONCEPTUAL ERECTION SEQUENCE

The design of the Higgins Road bridges considered a conceptual erection sequence during the design phase of the bridges to ensure that the bridges could be erected. The conceptual erection sequence developed during design was provided in the contract plans (the first three stages of erection of one of the Higgins Road bridges is shown in Figure 9). The planned construction of the Higgins Road bridges had to be carefully coordinated with the I-90 corridor traffic staging plan, which was one of the main motivations for investigating a conceptual erection sequence. The locations available for temporary supports were limited, due to the traffic staging. The conceptual erection sequence verified potential location of temporary structures and their effect on the partially erected superstructure.

The conceptual erection sequence analysis considered the stability of long, temporarily unbraced, lengths of the various girder segments. The girder segments were checked for lateral torsional buckling considering only sufficient brace points. As such, any assumed hydraulic crane hold points were only considered to relieve the vertical dead load bending moments and were not considered as brace points for the buckling checks.

Top flange lateral bracing is utilized in several cross-frame bays near the abutments, in the exterior girder bays only. The top flange lateral bracing, consisting of WT members, is required while the bridge is being constructed. The top flange lateral bracing prevents excessive lateral movement due to wind at intermediate stages of steel erection and when the bridge is in the non-composite condition, prior to and during placement of the concrete deck. Additionally, in each span, it was assumed during design that the steel erection would begin with a twin girder system, as shown in Figure 10. The top flange lateral bracing adds torsional stiffness and increases global buckling strength of the initial twin girder systems during steel erection.

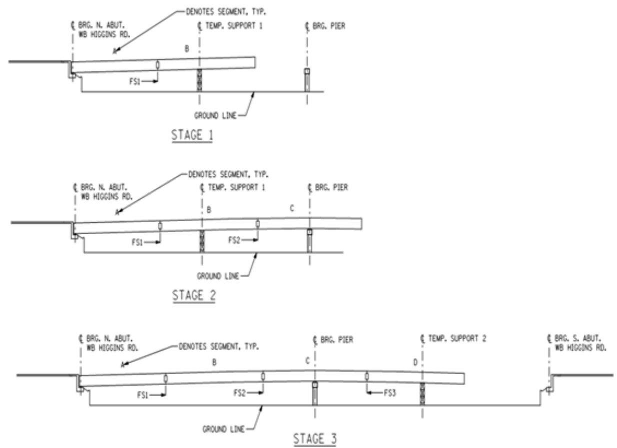

Figure 9: Conceptual Erection Sequence (first three stages

The issue of global lateral stability in multi-girder systems has been reported on by Yura et al. Including the lateral bracing during the design phase should be considered in long span plate girder design projects.

Lateral bracing is placed at the top flange level in order to reduce the amount of live load that it is subjected too in the final constructed condition. The lateral bracing participates in the system that resists applied load, and the farther away from the neutral axis the more force it will carry. In the final condition, with the deck in place, the neutral axis is closer to the top flange, and as such, the top flange level lateral bracing carries far less live load than it would had it been placed at the bottom flange level. Additionally, placing the bracing in the top flange level provides more of an increase in the global buckling strength than if it were located at the bottom flange level for twin girder systems during steel erection.

The top flange lateral bracing is connected directly to the girder top flanges. The wide girder flanges required by design allow for this connection. The holes caused by connecting directly to the flange were considered in the design of the flange. Connecting directly to the flange eliminates the need for gusset plates and eliminates additional connection eccentricity at the ends of the bracing members.

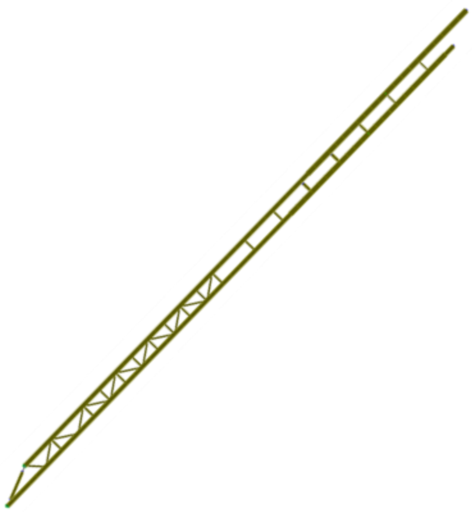

Figure 10: Conceptual Erection Sequence (first three stages

FABRICATION TO STEEL DEAD LOAD FIT

Severe structure skew leads to displacements and rotations that are significant and must be accounted for in the fabrication of the structural steel. At the acute corners in particular, the roll of the top flange outward is pronounced. These rotations if left unaddressed would lead to webs that would be significantly out of plumb once all dead load is applied. In general three detailing options exist: No Load Fit (NLF), Steel Dead Load Fit (SDLF) and Total Dead Load Fit (TDLF). NLF details the cross-frame/diaphragm members assuming that the girders are fully supported. This was not feasible due to the traffic underneath and the limited shoring tower locations. SDLF details the cross-frames so that the girder webs are theoretically plumb after all of the steel is erected but before the concrete deck is placed. TDLF details the cross-frames so that the girder webs are theoretically plumb after the steel is erected and after the concrete deck is placed. Detailing for SDLF was chosen because the out-of-plane rotation at the bearings once the bridge is completely erected can be accommodated in the design and because erection of the steel framing would be easier as compared to the use of TDLF detailing. TDLF requires more twisting of the girders in the field during steel erection than SDLF to accommodate the detailed cross-frames and diaphragms. Since the Higgins Road bridge girders are fairly stout in size due to the long span lengths, and time was limited for erection due to I-90 traffic maintenance, the design team selected SDLF detailing.

SDLF detailing for a bridge with the geometry of the Higgins Road bridges is an acceptable method of detailing in accordance with a recent white paper published by the National Steel Bridge Alliance (NSBA 2014). For more information on Steel Bridge Fit, the reader should consult this white paper.

The choice of SDLF detailing led to the need for blocking in the fabrication shop to match steel dead load deflections. In addition, the detailing of the holes in the girder connection plates for the cross-frame connections had to be carefully computed to result in webs that would be theoretically plumb at the steel dead load condition. The contract specified pre-assembly in shop to verify fit to ensure erection on site could be achieved. This requirement was necessary due to the limited windows available for steel erection and the associated maintenance of traffic procedures.

Figure 11: Photo from Shop Assembly

MODULAR EXPANSION JOINT

A swivel-type modular expansion joint was specified at each end of the deck. The governing effect is the racking, which is the movement along (parallel to) the modular joint. Because of the 70-degree skew of this bridge, the joint is nearly parallel to the longitudinal direction of the bridge, so large movements parallel to the joint should be expected. The main contributor to movement in the longitudinal direction of the bridge is thermal movement.

Because of the twisting effect of the bridge caused by the severe skew, movement in the transverse direction of the bridge at the ends of the bridge is more significant than bridges with less skew. In order to minimize the movement to which the joint is subjected, the modular joint was specified to be placed after all main deck pours are complete, just before the closure pour is made at the end of the deck. In other words, movement due to both the steel dead load and the concrete deck dead load were ignored in the movements specified for the joint since the modular joint is to be placed after the steel is erected and after the deck is poured. The movements specified for the modular joint are based on the results of the 3D finite element analysis of the superstructure and are primarily due to live load and thermal effects.

A sliding plate system was used to shield the parapet joint that was custom detailed to accommodate the design longitudinal and transverse displacements (see Figure 12).

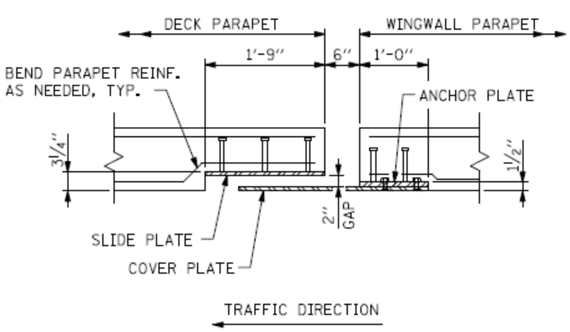

Figure 12: Plan View of Parapet Joint

SUBSTRUCTURE DESIGN

ABUTMENTS

Due to the severe skew, a jointless abutment type was not feasible. To minimize the structure span lengths, highwall abutments were considered. A combination of a stub abutment with expansion joints set back from a soldier pile wall was chosen. The soldier pile wall is orientated parallel to the I-90 freeway and set back from the edge of roadway to provide space for a full width shoulder plus an additional distance to account for snow storage. The soldier pile walls extends nearly 590 linear feet and averages 16 feet in height. This length of soldier pile wall is a result of the bridge width along the severe skew and the fact that this stretch of I-90 is in a cut section. This abutment arrangement minimized excavation to construct the abutments and minimized the abutment footprint by avoiding a large pile footing cap. This was especially important given the need to avoid the piles left in place from the original bridge. The wingwalls are orientated parallel to Higgins Road and extend 73 feet from the abutments in the obtuse corners. The stub abutments are set back approximately 7 feet from the soldier pile walls and are supported on two rows of HP 14x73 piles, with the front row on a 3:12 batter to resist the horizontal load.

The abutments were designed for the friction force developed in the expansion disc bearings as well as the transverse horizontal reactions at the guided bearings. Friction forces are determined by multiplying the coefficient of friction by the total dead load reactions on the bearing.

PIER

A multi-column, conventionally reinforced concrete pier was selected for the median support of this two-span structure. Various configurations were considered for the pier, and the final design consists of three pier segments: the first segment supports girders 1 and 2, the second segment supports girders 3 and 4, and the third segment supports girders 5 and 6. The cap consists of a rectangular section, 5'-6" wide and nominally 4'-6" tall. Eleven circular columns, 4'-6" diameter, are provided at the pier with one column under each girder line and an additional column in between each girder line as depicted in Figure 13.

Though the distance between the centerlines of the fascia girders is 41.67 ft measured perpendicular to the girders, the distance between the centerlines along the skew is nearly 120 ft. Thus the overall length of the pier at its base is 130 ft. The three pier segments share a continuous crashwall and pile cap. The crashwall extends 5 ft above the proposed top of roadway and has a minimum overall height of 6 ft. The pile cap is 4 ft deep and 17 ft wide and is supported by four rows of battered HP14x73 piles. Although the appearance of the pier from the outside may seem fairly ordinary and conventional, the design challenges and detailing of this pier were quite unique. Figure 14 shows a photo of one of the center piers just after the steel superstructure framing was erected

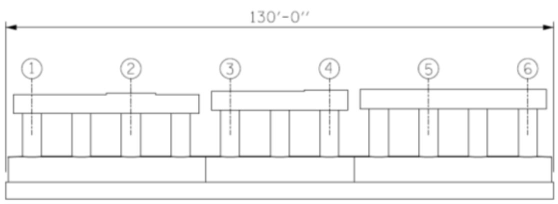

Figure 13: Elevation Sketch of Skewed Pier

Figure 14: Elevation Sketch of Skewed Pier

Determining the pier cap and column configuration was a function of the internal thermal force demands in combination with very high horizontal forces from the superstructure. Short and stiff piers are more sensitive to internal thermal forces than taller, more flexible piers. The IDOT Bridge Manual requires caps and crashwalls to have a joint at mid-length between bearings when the length exceeds 90 feet. In this case, a configuration with three cap segments measuring roughly 44 feet, 32 feet, and 44 feet was determined to be more practical than two equal segments. The segments need to be short enough to minimize the internal thermal forces but robust enough to handle the significant horizontal forces and torsion induced by the superstructure. Segmenting the pier also reduced the amount of torsion in the pier cap that had to be resisted. In a more conventional design, intermediate columns would not be necessary if columns are provided directly below each bearing; however, in this design they were required in order to resist the load demands.

The twisting effect of the superstructure caused by the severe skew is also apparent in the deflected shape of the pier as depicted in Figure 15. Because of the severe skew and long spans, the four fixed bearings at the pier, at the four interior girders G2 through G5, have very high horizontal forces due to thermal effects from the superstructure. The horizontal forces from the superstructure due to dead load and live load are also significant. Because of the high horizontal forces and in particular because the horizontal forces vary in magnitude from bearing to bearing, torsion in the cap is very high. Torsion and lateral bending (bending of the cap in the horizontal plane) were the main factors in the design of the pier. These were the driving factors in determining a feasible column layout.

Figure 15: Pier Deformation

Unlike a conventional pier cap, vertical bending moment in the cap was not the main concern. A typical section through the pier cap is shown in Figure 16. The cap contains 48 No. 10 longitudinal bars. The longitudinal bars were designed for torsion, and checked for other conditions. Whereas the side longitudinal bars along the vertical faces of the cap would be skin reinforcement in a more conventional pier, in this case they are main reinforcement resisting lateral bending moment in the pier cap in addition to satisfying torsion. Torsion was a major factor in the cap design as previously explained. Had the horizontal reactions been large in magnitude but equal and in the same direction at each bearing, there would have been essentially zero torsion in the pier cap.

One of the unique challenges of the pier design was the anchorage reinforcement at the fixed bearings. AASHTO refers the designer to ACI 318 Appendix D to check concrete anchor breakout strengths. A specialized approach was necessary, which included seismic-like detailing.

The reinforcement in the cap was carefully coordinated with the anchor bolts required at the fixed bearings. Sixteen anchor bolts are placed through an embedded plate flush with the top of the pier cap, prior to casting the pier cap. Twelve inner core anchor bolts extend down to bear on the top of the already cast column in a "table top" manner. In addition, four corner anchor bolts extend as deep as practical to avoid conflicts with the bottom longitudinal bars. Since the anchor bolts protrude above the top of the pier cap within the footprint of the bearing assembly, a "cheese" plate is provided over the anchor bolts and nuts in order to provide a smooth surface to receive the bearing. The base plate of the bearing is welded to the cheese plate, which is in turn welded to the embedded plate in the pier cap. Each element of the bearing-to-pier connection was checked for the governing lateral design forces. Because of the reinforcement congestion in the cap at the fixed bearing locations, a 3D parametric model of the reinforcement was created to ensure that bar clearances were sufficient and to aid in constructability checks.

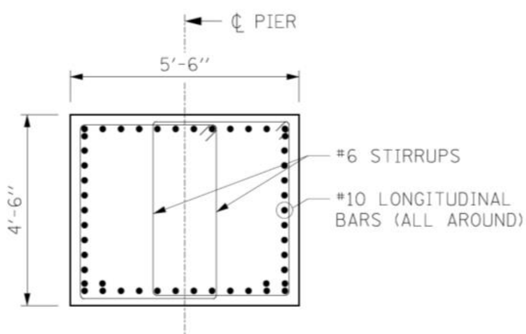

Figure 16: Pier Cap Reinforcement

Similar to the unique considerations within the pier, the design of the piles at the median pier also presented unique challenges due to the severe skew and corresponding high horizontal forces. The traditional formula for computing individual pile loads in a group of piles is based on the assumption that the pile cap acts as a rigid body. With a long pile cap subjected to equal (or nearly equal) and opposite horizontal forces at either end, it was recognized that the rigid body assumption was not valid. The pile cap was divided into segments for analysis purposes after considering its torsional resistance. These segments are capable of acting as rigid bodies, and pile loads were computed by traditional methods from the contributing forces within each segment. The design consists of four rows of piles battered at 3:12.

Figure 17: Table Top Bearing Plate

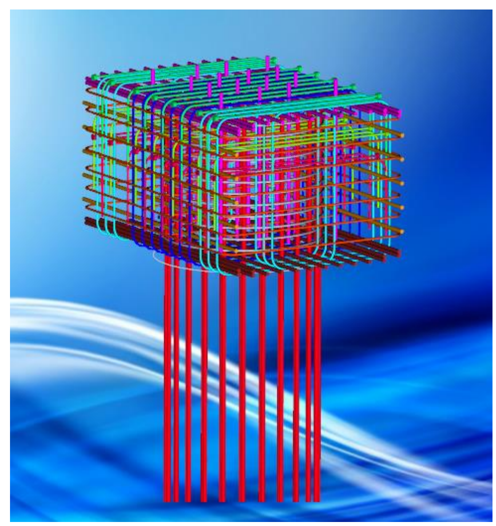

Figure 18: Parametric Design for Reinforcement Conflicts

SUMMARY

The Higgins Road Bridges presented many design and construction challenges that resulted from the severe skew and long spans. In addition, the design needed to consider avoiding the piles from the original bridges and the maintenance of traffic of a heavily traveled section of I-90 that carries 118,000 vehicles per day.

The significant support skew necessitated a refined 3D analysis to properly account for the force effects, displacements and rotations. A 2D grid analysis may result in inaccurate results and a poor representation of the cross-frame and diaphragm forces. An optimal framing plan can be developed with a refined analysis, and uplift conditions associated with severe skew can be avoided. Additionally, a more efficient design can be achieved in the vicinity of the skewed end supports with staggered cross-frames. With a staggered cross-frame arrangement, the undesirable transverse load path to the support is avoided and the girder can better transmit the load to the support. For structures with severe support skew and significant span length, high lateral forces can develop at the bearings. Resistance of these high lateral forces can lead to complicated and congested reinforcement detailing that in this project was addressed through 3D parametric design modeling. Severe structure skew leads to displacements and rotations that are significant and must be accounted for in the fabrication of the structural steel. In this project Steel Dead Load Fit was specified, which in turn defined the geometric relationship between the girders and cross-frames. This fit condition provided acceptable geometric control and reasonable ease of erection. A conceptual erection plan was considered during the design phase to ensure structural stability and minimize assembly issues during steel erection. In this project, with significant traffic considerations, a shop assembly to verify fit-up was specified in an effort to minimize issues in the field.

The first of the twin Higgins Road Bridges opened to traffic in the summer of 2015. The awarded bridge construction value was $22.7 million, which equates to approximately $390 per square foot.

ACKNOWLEDGEMENTS

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