CHALLENGES OF DESIGNING A HIGHLY SKEWED TWO-SPAN CONTINUOUS STEEL GIRDER BRIDGE

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BIOGRAPHIES

Peter Mahally is a Senior Structural Engineer at Greenman-Pedersen, Inc. (GPI) and was the Structures Task Manager for the subject project. Pete has 20 years of experience in structural design and plan presentation of highway and railroad bridges. He is a licensed Professional Engineer in the state of Pennsylvania and received both his Masters degree and Bachelors degree in Civil Engineering from The Pennsylvania State University.

Brian McFadden is a Design Engineer at GPI. Brian has 7 years of experience in the structural design and plan presentation of highway bridges. He is a licensed Professional Engineer in the state of New Jersey and received his Bachelors degree from Rutgers University. Brian was the structural design engineer for the subject project and worked on the superstructure and substructure design as well as the 3D modeling of the structure.

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SUMMARY

Severely skewed bridges create some unique design concerns as compared to similar structures with little to no skew. Many of the issues are due to the behavior of the structure, which experiences differential deflections between adjacent girders at any section cut along the length of the structure. Some of the unique design challenges include selecting a contiguous or staggered cross frame layout, complex detailing of end diaphragm and intermediate cross frame connection plates, refined analysis required for the design of girders and cross frames, and the use of High Load Multi-Rotational Bearings (HLMR) to accommodate out of plane rotations and high lateral forces.

This paper attempts to raise and address some of the challenges faced during the design process. It discusses current code requirements and guidelines offered by various sources to aid the designer in analyzing a highly skewed, two-span continuous, steel multi-girder bridge.

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Bridge Background

As part of an $85 million shoulder widening project along the Garden State Parkway (GSP), the New Jersey Turnpike Authority (NJTA) is replacing the existing northbound and southbound bridges carrying Parkway traffic over US Route 9/NJ Route 166 in Toms River Township, New Jersey. Geometric improvements to the highway cross section require widening the out to out width of each existing structure by approximately 20 feet to provide standard lane and shoulder widths. The proposed bridges will replace the existing thru-girder, floor beam superstructures (Figure 1) that were deemed functionally obsolete and which were not able to be widened and rehabilitated.

Figure 1. Southbound Garden State Parkway bridge over US Route 9/NJ Route 166.

Similar to the existing structures, the proposed structures will have a structure skew angle of approximately 70 degrees, measured from the centerline of bearings of each substructure (abutments and pier) to a line perpendicular to the centerline of girders. The severe skew angle is a result of the substructures placed parallel to the horizontal alignment of US Route 9/NJ Route 166 that was being maintained, with no future plans to widen or re-align the roadway. The layout resulted in two sister structures with identical span arrangements and cross section widths.

In an effort to eliminate the severe skew angle, a 3-span continuous (135'-170'-135') superstructure supported by outrigger piers (transverse box girders) and stub abutments was investigated. This option eliminated the severe skew angle which in turn reduced the lengths of substructures, deck joints and end diaphragms.

Although this option eliminated many of the negative impacts associated with the severe skew angle, the transverse box girders are considered to be non-redundant Fracture Critical Members (FCM) which must satisfy more stringent fabrication and testing procedures and would increase future inspection costs. In addition, the transverse box girders would control the minimum vertical clearance over US Route 9/NJ Route 166 and the northbound and southbound GSP vertical profiles would have to be raised.

Based on a preliminary cost study, the increased superstructure length, FCM box girders, and additional retaining walls made the 3-span continuous non-skewed structure more expensive and therefore, it was eliminated from consideration.

The final alternative advanced into design consisted of two-span continuous structures with equal spans of 135 feet. Each cross-section consists of nine girders with a spacing of 7'-4" measured normal to the girder webs with an 8 ½" composite High Performance Concrete deck slab. All girders have a constant web depth of 56 inches. The out-to-out width of each bridge is approximately 64 feet and contains three 12-foot lanes and a 12-foot left and right shoulder.

Since the bridges are identical in all geometric aspects, except minor differences in the vertical profiles, only one of the structures was required to be analyzed during the final design phase.

Introduction

Severely skewed, straight I-girder superstructures behave quite differently than their non-skewed counterparts. In all superstructures, the girders deflect under their self-weight and applied loadings. This deflection varies along the length of each girder with zero deflection at the supports and gradually increasing to the maximum deflection at or near midspan, depending upon the span arrangement. In the case of a non-skewed straight superstructure, the deflections across any section taken normal to the bridge due to girder self-weight and the deck slab weight are roughly the same, assuming relatively equal girder section properties and spacing. This is due to the fact that at a given section, each girder point is located the same distance away from the support. By contrast, on a skewed superstructure the deflection of each girder at a section is not the same since the girders are longitudinally offset from each other due to the skewed supports. Therefore, differential deflections will exist between adjacent girders across any section of a skewed bridge, which will affect the design of the superstructure components and bearings.

For skew angles greater than 20 degrees, The American Association of State Highway Transportation Officials (AASHTO) LRFD Design Specifications (1) states that intermediate cross frames are to be positioned normal to the girder webs. This is done to eliminate the use of bent connection plates which can produce more flexibility within the cross frame, counteracting the intent of this member to provide stability to the adjacent girders. Skewed cross frames also require longer members and hence larger sections may be required to provide adequate axial capacities. For steel fabricators and erectors, cross frames set normal to the girders are easier to fabricate and quicker to erect and install compared to cross frames positioned along the skew.

The AASHTO LRFD Design Specifications (1) also provides guidance on the orientation of cross frames at skewed interior supports, stating that elimination of skewed cross frames along the support may be considered if intermediate cross frames normal to the girders are placed at every bearing which can resist lateral forces such as guided or fixed bearings. Placing cross frames oriented normal to the girder webs at bearing locations will provide adequate bracing of the bottom flanges in addition to eliminating lateral bending moments in the bottom flange that would occur if the cross frames were offset from the bearing.

As stated earlier, there will be differences in the deflection values of adjacent girders at a section across a skewed superstructure. Since it is standard practice to place cross frames normal to the girder webs for skewed supports greater than 20 degrees, each end of the cross frame will be connected to locations on adjacent girders which do not deflect equally. In the case of this structure, the maximum differential deflection caused by the non-composite dead load (girder self-weight and deck slab) was found to be located at the acute corner between the fascia girder and adjacent first interior girder at the first intermediate cross frame connecting the two girders. Since the cross frames are rigid in the transverse direction as compared to the flexible longitudinal girders, when dead loads are applied the cross frames will partially restrict the girders from deflecting and induce a twist or rotation in the girders.

Depending on how the cross frames are detailed to be connected to the girders at the time of steel erection will dictate the amount and direction of twisting at installation of the cross frames and further twisting due to application of dead loads. Structural detailers for the steel fabricators will develop the cross frame connection locations with respect to the top of girders by referring to the girder camber values provided on the design plans. AASHTO and National Steel Bridge Alliance (NSBA) Steel Bridge Collaboration (4) discusses three methods used in detailing girders and cross frames for skewed and curved bridges:

The No-Load Fit (NLF) method of detailing the cross frame members to the girders does not consider girder self-weight deflection to occur during erection and the girders will be plumb when the cross frames are installed. It may be necessary to install temporary shoring towers under the girders to achieve this condition. Once the temporary supports are removed and other dead loads (permanent deck forms, deck slab, etc.) are applied, the girders will rotate out of plumb due to the differential deflection of adjacent girders.

The Steel Dead Load Fit (SDLF) method of detailing the superstructure members such that the girders are allowed to deflect under self-weight and the girder webs will be vertical when the cross frames are installed. The girders are expected to rotate out of plumb when the other dead loads are applied and this rotation is dependent on the magnitude of the support skew angle.

Since this structure has a support skew angle of approximately 70 degrees, there was concern that if the SDLF condition was proposed, significant out of plane rotations would occur after the deck pour and girder webs would not be plumb under the final, permanent condition. In order to compensate for this the contract plans and contract special provisions directed the Contractor to fabricate the cross frames using the Full Dead Load Fit (FDLF) method as discussed in AASHTO/NSBA (4). This method is also widely known as the Total Dead Load Fit (TDLF) method and details the cross frame members so the girder webs will actually rotate out of plumb once the cross frames are installed. Therefore, when the deck slab is placed the girders webs will rotate to, or near the plumb condition.

In order to detail as per the TDLF method the structural detailer will detail the cross frames (lengths and connection locations) based on the final dead load condition of the steel framing. The detailer will first review the camber ordinates of all girders to gain an understanding of the dead load cambers. Then the detailer will determine the elevation difference between the tops of adjacent girders after all dead load has been applied to the framing. Only the final camber due to vertical profile and deck cross slope will be present in the girders. The detailer will detail the cross frame members and location of connections to the adjacent girders based on this method.

In the TDLF method, all of the dead loads are not on the structure when the cross frames are installed and the girders will not be in their final deflected position. Connecting cross frames that are detailed to fit girders when full dead load deflection takes place will cause the girders to twist and rotate out of plumb. This twisting effect of girders is commonly referred to as web layover.

The reader is alerted to refer to AASHTO/NSBA G12.1 (2003) (4) and AASHTO/NSBA G13.1 (2011) (2) for additional discussions and graphical illustrations of the various fit methods.

AASHTO directs the designer to *"clearly indicate an intended erection position of the girders and the condition under which that position is to be theoretically achieved"*. Based on the authors' experience and research it has been noted that some agencies and owners provide guidance on when to use the SDLF and TDLF detailing methods. Typically, for straight bridges with no skew or minor skew (less than 20 degrees), both SDLF and TDLF methods are practical. The girders will be erected plumb and minor variation from plumb will occur when the deck slab is placed. However, for skew angles greater than 20 degrees the amount of twist that will occur due to the deck slab loading could be significant and therefore, erecting the girders with a web layover by use of the TDLF method is specified by some agencies.

Even though the girders are rather flexible and susceptible to twisting, the erector will need to apply an auxiliary force in the form of a come-along or other means to displace the girders as required to align the bolt holes in connection plates. Force fitting the girders to the cross frames will induce stresses within the cross frames, but the majority of the stresses will be released upon application of the deck slab since the girders will untwist and girder webs will approach the plumb condition. It should be noted that AASHTO/NSBA (2) states that the connections between cross frames and girders must be tightened before the deck concrete is poured. Otherwise the cross frames will not be able to maintain the twisted shape of the girders upon erection as well as force the girders to rotate back to the plumb position when the deck is poured.

It is the opinion of some designers and erectors that cross frame members need not be designed for forces developed by the deck slab placement when TLDF detailing is used. The theory is that the amount of force induced in a cross frame upon force fitting the connection to adjacent girders is approximately equal and opposite to the force generated in the cross frame upon application of the deck slab dead load. However, this theory hinges upon the accuracy of the camber table and does not account for any fabrication tolerances. In the case of this structure, the results of the two-dimensional (2D) and three-dimensional (3D) models indicated that the cross frame forces resulting from the deck slab pour were substantial and were therefore considered when designing the cross frame members and their connections to the girders. While this approach may be overly conservative, it avoids the need to determine the percentage of dead load stress to be considered as locked-in.

At the time of design there was no specific code or guideline which recommended a procedure to determine the amount of locked-in force sustained by the cross frames after the application of all dead load, although approximate methods for calculating such stresses were available.

Framing Models

During the course of the final design phase three different models were created to analyze the bridge. The models generated included a line girder analysis using MDX Software (6), 2D Plate and Eccentric Beam (PEB) model using MDX Software, and a 3D Finite Element Model using CsiBridge Software (5). The 2D plate and eccentric beam model was chosen because it provides slightly more refined results as compared to a typical 2D grid analysis. The 3D finite element model is largely considered the most refined type of analysis and it was developed in order to analyze more global effects on the structure as well as to model the relationship between the superstructure and substructure elements.

The structure skew angle is outside the range of applicability for determining distribution factors as set forth by the AASHTO LRFD Bridge Design Specifications (1). Therefore, use of traditional line girder analysis was restricted to only very basic girder sizing runs during preliminary design.

The majority of the superstructure design was completed using the 2D model while the 3D model was used more as a benchmark to verify and compare results obtained through the 2D model. As previously noted, the 3D model was also used to analyze global effects (such as thermal effects, seismic analysis, slab pour sequences, substructure movement, etc.) that could not be adequately captured with the 2D model.

Initially during final design, the fascia girder section was set to match the interior girder, but it was observed that the maximum non-composite dead load deflection of the fascia girder was 4.17 inches. In addition, the maximum girder end twist at the acute corner was 1.38 degrees. In an effort to reduce these values the fascia girders were stiffened by making the flanges roughly 30% larger and making the web roughly 10% thicker than those of interior girders so as to increase the fascia girder section and reduce the differential deflection between the fascia girder and first interior girder leading to slightly smaller girder end twist (out-of-plumb rotation).

Girder forces, reactions, vertical deflections, and girder end rotations computed by MDX (PEB) were within reason of what was predicted by the 3D model. However, cross-frame forces based on 3D modeling were found to be larger than the MDX (PEB) values. Also, the 3D model provided additional data such as displacement and rotation about all three axes along the entire length of girders, thermal behavior, deflection and rotation under different phases of deck pour. The ability to view the deformed shapes and stress/load contours under different load cases proved very beneficial and assisted the authors in better understanding the behavior of a severely skewed bridge and being able to design accordingly.

It is important to note that an accurate 2D or 3D model is paramount in determining vertical displacements, from which the camber table and cross frame detailing will be generated. Designers are faced with several key issues when generating models for severely skewed bridges which include; stage of cross frame fit up (no load fit, steel dead load fit, or total dead load fit), girder rotations due to fit up, whether to include locked in forces due to fit up, and at what stage cross frames should be considered effective in the model.

Subsequent to the original design of the bridges discussed in this paper, NCHRP Report 725 (8) was published. A comprehensive review and interpretation of the NCHRP Report is not within the scope of this paper, however, the final section of this paper provides information on the report findings based on our cursory review, as well as a brief discussion about its implications for this structure. The NCHRP Report 725 offers significant guidance on modeling techniques and and should be referred to during future designs of severely skewed bridges.

Cross Frame Layout

The cross frame layout was developed based on taking the following into consideration:

- Provide intermediate cross frames normal to girder webs.
- Eliminate the application of skewed cross frames along the pier.
- Provide a uniform cross frame spacing along the length of the structure.
- Provide intermediate cross frames in contiguous lines as much as practical.

In the absence of the skewed cross frames at the pier, a line of intermediate cross frames placed normal to the girder webs was located at every pier bearing location. Using this approach defined the cross frame layout for the majority of each span since the spacing of cross frames followed the longitudinal offset between each girder at the pier. This pattern resulted in a cross frame spacing of approximately 19'-8". A contiguous line of cross frames placed across the full width of the structure was provided where the cross frames intersected the pier bearings of interior girders G3 through G7. Because of the severe skew angle and distance measured normal between the fascia girders, the cross frame lines which intersected the pier bearings for G1, G2, G8 and G9 could not extend the full bridge width. These cross frame lines terminated when placement normal to a girder web could no longer be achieved (Figure 2).

Girder splice locations also need to be considered when developing a cross frame layout. The GSP bridges are two-span continuous structures with a total structure length of 270 feet and girder splices were required to transport the girders to the site. Each girder line was separated into three segments which resulted in two splice locations per girder. Ideally, the girder splices are located at, or near the dead load points of contraflexure. In the case of this severely skewed bridge, the location of the contraflexure points varied between the girders. As stated earlier, since skewed cross frames along the pier were not used, the cross frame layout originated by placing lines of cross frames normal to the girder webs at the pier bearings. Therefore, the location of the girder splices had to account for these lines of cross frames whose locations could not be altered (Figure 3).

To provide for economy during fabrication, it was imperative that some consistency in the splice locations was held so multiple uniform girder segments could be fabricated. Taking this into consideration, the interior girder splice locations were offset 34 feet into each span from the centerline of pier, measured along each interior girder. These locations could not be held for the exterior girders due to conflicts with cross frame locations. In order to provide some uniformity in the splice layout for the exterior girders on each side of the bridge, the fascia girder splice locations and the adjacent lines of contiguous cross frames were adjusted to eliminate conflicts. The resulting girder splice layout provided a minimum one foot clear distance from the edge of the flange splice plate to the nearest adjacent cross frame connection plate.

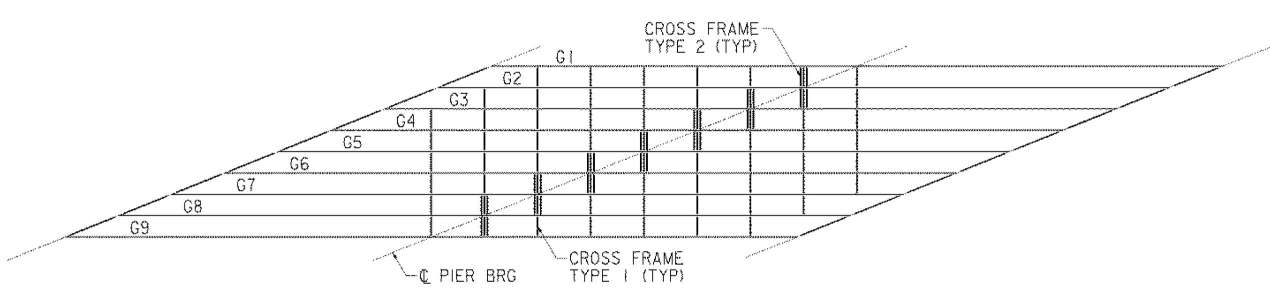

Figure 2. Cross frames placed in contiguous lines at pier bearings.

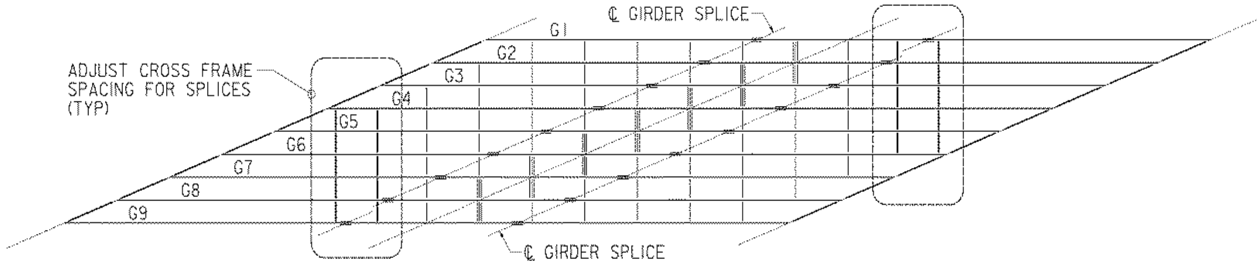

Figure 3. Cross frame placement with respect to girder splice locations.

When evaluating the cross frame layout for a skewed structure, the designer should consider the location of the cross frames nearest to the abutment supports. At these locations the differential deflections between adjacent girders tends to be large since one end of the cross frame is connected to a girder which is at or near a vertical support and therefore will have a small deflection. In these instances, high forces in the cross frames could be observed and there is the potential difficulty during erection in making the connections at each side of the cross frame.

The final cross frame layout did provide cross frames as close as 3'-0" from the abutment

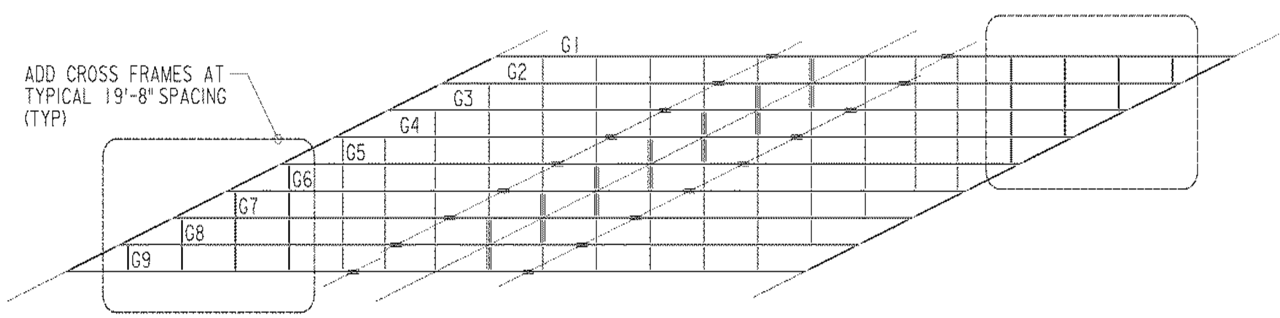

Figure 4. Cross frame placement continued to end of spans.

bearings. This distance was a result of attempting to continue the cross frame spacing of approximately 19'-8" that was controlled by the longitudinal spacing of the girder bearings at the pier and also avoid using a staggered cross frame layout in these areas (Figure 4). Design results showed that the forces in these cross frame members were not significantly larger than at other locations. However, placing cross frames so close to the abutment bearings does increase the potential for fit-up difficulties during erection.

During design, a staggered cross frame layout was evaluated. When a staggered cross frame layout is used out-of-plane bending on the girder webs and lateral flange bending must be taken into consideration in design due to the lack of bracing on the other side of the web. Results of the 2D analysis generally produced cross frame forces in the range of 25% lower than the non-staggered layout. This is expected since the staggered layout decreases the transverse stiffness of the bridge (1).

The staggered cross frame layout produced non-uniform spacing along the length of the structure and more cross frames were required using this layout. These drawbacks essentially led to the placement of cross frames in contiguous lines except at locations adjacent to abutment supports as discussed above. Figure 5 shows the completed framing plan.

Cross Frame Type

The cross frame type used at the intermediate locations was an X-brace which is commonly used throughout the state of New Jersey. The geometry of the cross frame which is dependent on the girder spacing and girder depth produced an angle

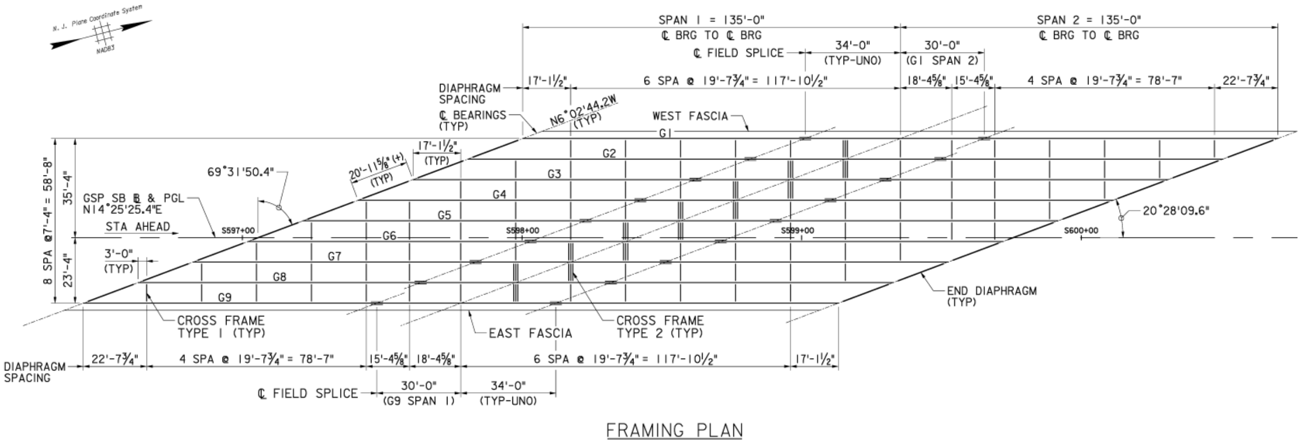

Figure 5. Framing plan of proposed southbound bridge.

between the diagonals and bottom chord of approximately 30 degrees which is considered to be the lower limit of recommended use for this cross frame type. Adding a top chord segment to this cross frame resulted in an angle of approximately 20 degrees between the diagonals and bottom chord and therefore a design with a top chord was not pursued. A K-frame type diaphragm with horizontal top chord was also evaluated. However, the 2D analysis results produced high forces in the K-frame elements and this diaphragm type was no longer pursued.

Results of both the 2D and 3D models showed that the maximum forces were near the columns. After a few iterations of cross frame member sizes it was determined that the cross frames with higher capacities would only be used where required. This was done since using a more rigid cross frame throughout simply increased the transverse stiffness of the bridge and hence higher cross frame forces were observed. Figure 6 shows the two cross frame sections used on the structure. The Type 1 cross frames consisted of one angle for the bottom chord which was adequate for the majority of locations. The Type 2 cross frames with a double angle for the bottom chord was used at isolated locations near the pier columns.

Thermal Analysis

During initial design a multi-column pier bent was evaluated for the fixed pier. The pier consisted of 3'-6" diameter columns spaced at 21 feet and a 4-foot wide by 5-foot deep pier cap. A pier cap length of 175 feet was required based upon the severe substructure skew angle. To reduce the internal thermal forces caused by expansion and contraction of the pier cap, an open joint was located near mid length of the cap which resulted in two isolated pier cap segments approximately 95 feet and 75 feet in length. The vertical reactions

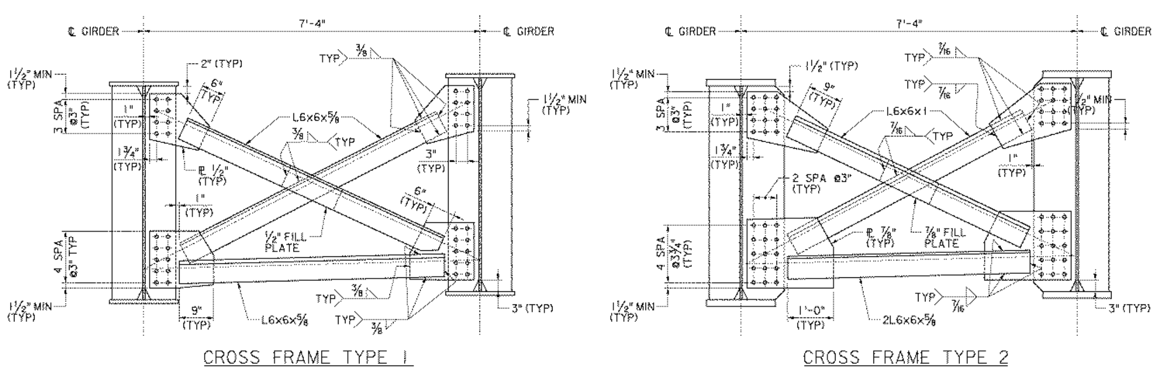

Figure 6. Intermediate cross frame types.

used during the initial design of the pier were taken from the 2D model since the 3D model was not developed at this point.

Once the 3D model of the superstructure was developed the pier cap and column elements were added. The purpose of including the pier in the 3D model was so both a thermal and seismic analysis of the structure could be performed.

In a typical non-skewed two-span continuous bridge consisting of equal spans with a fixed pier, the lateral forces developed at the pier bearings as a result of applying a change in temperature to the superstructure elements are small in magnitude. This is attributed to the fact that when a temperature change occurs, the abutment expansion bearings will generally displace the same amount, but in opposite directions. Therefore, the friction or shear force developed by the longitudinal translation of the bearings at each abutment will be equal, but in the opposite direction.

When the thermal analysis was performed using the 3D model it was evident that the skewed pier cap played a significant role in restraining the thermal movements of the superstructure as large lateral forces at the fixed and guided pier bearings were recorded. In addition, the cross frame forces caused by the thermal effects were larger than could be designed for. Bearing configurations utilizing different combinations of fixed, guided, and unguided bearings at both the abutments and the pier were evaluated in the 3D model in an attempt to reduce the lateral forces at the pier bearings. However, only moderate reductions were observed. Although an open joint was provided in the pier cap to reduce the thermal effects of the pier bent, excessively high moments were observed in the caps and columns as a result of the pier cap expansion and contraction over multiple columns.

The 3D model was then revised by completely removing the pier cap and placing a column directly beneath each girder bearing (Figure 7). This revision produced significantly lower lateral forces on the pier bearings than the model which included the pier cap. The absence of the pier cap allows greater flexibility of the skewed superstructure since the columns do not provide much rigidity or restraint from translational movements. The columns will displace and as a result the lateral forces at the pier bearings will decrease.

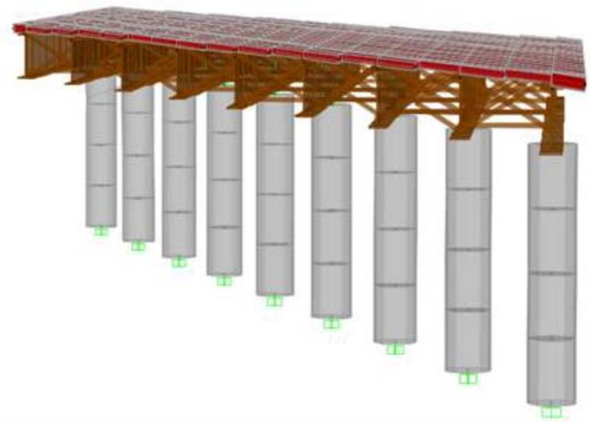

Figure 7. 3D model elevation view of pier and cross section of framing.

Although eliminating the pier cap was beneficial to the design and layout of the bearings it affected the design of the columns. Removal of the pier cap results in a non-redundant substructure as the columns are considered as independent elements. Also, the effective length factor in the transverse direction of the columns increased. Taking these factors into consideration the diameter of the columns was increased from 3'-6" to 4'-0".

As part of the pier column design the New Jersey Turnpike Authority Structures Design Manual (7) instructs the designer to investigate a 200 kip vehicular impact force applied four feet above the roadway surface. This loading is classified as an AASHTO Extreme Event II load case (1).

In a multi-column bent the presence of the pier cap which is integral with the columns will distribute a portion of this impact force to the adjacent columns. Since the pier cap was removed from this structure there was no lateral distribution and hence the columns were treated as pure cantilevers in flexure. Based on the analysis it was determined that the 4 foot diameter columns had adequate capacity to resist and transmit the impact force to the foundation.

In addition to satisfying the vehicular design requirement, the NJTA requested that the design of the cross frames and connections be of adequate strength to transfer load and support any girder in the superstructure should its supporting column be removed from service due to damage from this impact force during an Extreme Event II Limit State. The 3D model was used to investigate this case and the results indicated that girder deflections and cross frame forces increased as expected, but the superstructure framing was capable of supporting any girder without failure. It was determined that both the Type I and Type II cross frames and connections shown in Figure 6 were adequate for this load case given the structure is severely skewed and rather stout cross frame members and connections were required based on the Strength and Service Limit States designs. The magnitude of the stresses in the deck elements over the columns directly adjacent to the column considered to be ineffective indicated that some localized deck cracking could be expected.

By observation one can see that the cross frame layout chosen for this structure provides an alternate load path to the pier columns for the case where a column has been lost. A framing layout consisting of staggered cross frames with skewed pier diaphragms would have a different load path near the pier and the cross frame forces and deck stresses could have been significantly greater than what was observed.

Bearing Selection

The bearing orientation plan (Figure 8) was developed based on our findings from the 3D models. High Load Multi-Rotational (HLMR) bearings were selected since they have high rotational and lateral movement capacities to accommodate the racking movements and out of plane rotations associated with this severely skewed structure.

NJTA does not require the design engineer to perform design and provide detailed drawings of the HLMR bearings. Rather, a bearing orientation layout, summary of design vertical and lateral reactions, as well as material and design specifications are provided in the contract documents. It is the responsibility of the Contractor's bearing manufacturer to select the type of HLMR bearing (pot bearing, disc bearing) that will be used. The bearing manufacturer must submit the bearing design and detailed shop drawings during the post-design phase for the design engineer's review and acceptance.

As a result of the thermal 3D analysis, unguided bearings were assigned to the outer three bearings at each abutment. The thermal analysis showed that displacements of the inner three girders were predominantly in the longitudinal direction. Therefore the bearings for these girders were guided along the centerline of girder to allow longitudinal movements and resist lateral movements. At the pier columns, unguided bearings were used at the exterior and first interior girders on each side of the bridge. Guided bearings that restrain longitudinal movement were used at the second interior girders and fixed bearings were used under the center three interior girders.

As per AASHTO (1), the construction sequence should be considered when determining the bearing rotations. The results of the 3D model

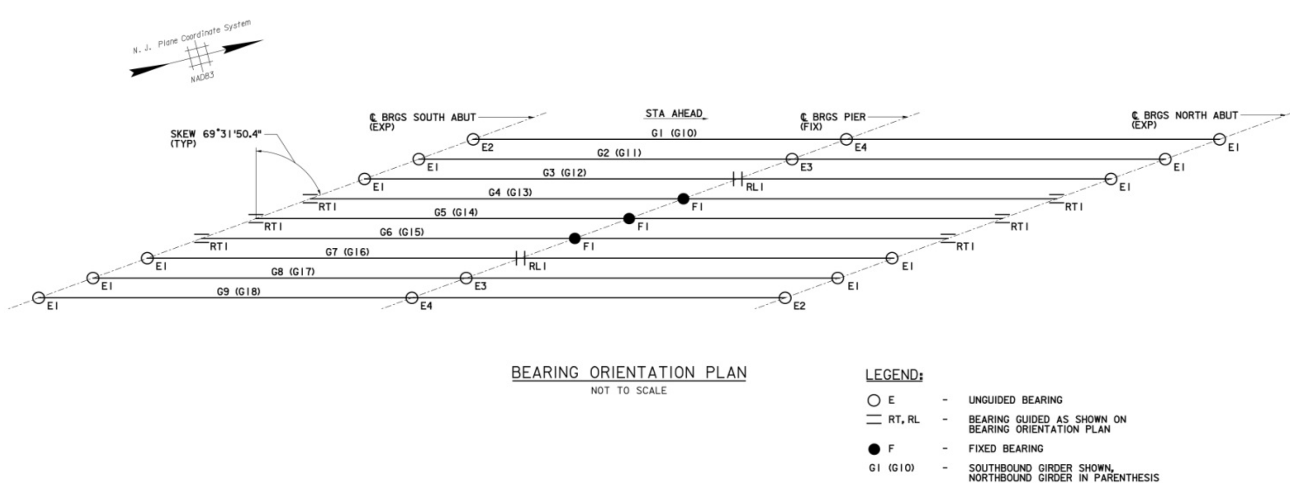

Figure 8. Bearing orientation plan.

were reviewed to determine the non-composite girder out of plane rotations at the abutment and pier bearings. Since the cross frames are to be detailed per the TDLF method, high girder out-of-plane rotations are expected at the abutment bearings when the cross frames are initially connected to the girders. A note was placed on the contract plans which provided the approximate rotations and directed the bearing manufacturer to investigate this temporary condition during the bearing design.

Once the deck slab pour is completed, the girders will rotate back toward the plumb condition and as a result, the out of plane rotations will decrease from the initial erected condition. This is true because the TDLF method was used. The Design Engineer should be aware that the detailing condition of the cross frames (NLF, SDLF, TDLF) could affect the total rotation at the bearing.

The bearing fixity and orientation plan as shown on the contact drawings was a result of an iterative process, which involved multiple rounds of 3D thermal and seismic analyses simulating different bearing fixity and orientation combinations. During this process the decision to remove the pier cap as discussed in the Thermal Analysis section reduced the lateral forces exerted on both the superstructure and substructure elements. The final bearing layout was consistent with the recommendations stated in AASHTO/NSBA (3) in regards to limiting the fixed and guided bearings to the center girders of the structure which experience negligible transverse thermal movements.

Deck Pour Sequence

For a typical two-span continuous bridge in the state of New Jersey, the deck pour sequence is usually completed by pouring the deck slab in the positive moment regions first. After the positive moment regions have attained a minimum concrete compressive strength the negative moment region over the pier is then poured. The positive moment pour in each span may be performed simultaneously, or with a waiting period between pours, as some agencies may have specific requirements that prohibit simultaneous pours. In the case of this structure, the positive moment region of each span contains roughly 190 cubic yards of concrete and 5,800 square feet of deck area. Due to the severe skew of the substructure supports the set up required prior to each pour will take considerably longer than a typical bridge with the same concrete quantity and deck area. During the design phase, our research indicated that in most instances Contractors would elect to use one crew and pour the positive moment region of each span separately, since not a significant amount of concrete volume needed to be placed. This would require a separate setup of the screed machine in each span.

Several deck pour sequences were modeled using the 3D model to check deck stresses, girder rotations and displacements, and bearing reactions at various stages of the deck pour sequence to gain an understanding of the effects caused by the skew. The results showed that deck placement in the positive moment region of span 2 could introduce tensile stresses in the previously poured region in span 1. Based upon this observation, we elected to include in the contract plans a waiting period between the positive moment pours in spans 1 and 2 so the concrete in span 1 has adequate time to cure and gain the necessary strength to minimize or eliminate cracking. In an effort to control cracking, additional longitudinal deck reinforcement in the negative moment region was extended further into the positive moment regions. Additionally, the waiting period is beneficial in the sense that the concrete pour in span 1 will provide some composite action and stability to the girder segments located within the pour limits.

The results of the 3D analysis also indicated that starting the span 1 pour within the span and moving towards the abutment would produce uplift at the opposite abutment obtuse fascia girder bearing. Uplift was not observed during the span 2 positive moment region pour since the positive moment pour in span 1 provided enough dead load to counteract any uplift forces. As a result of this analysis, we noted on the contract plans that the deck pour in span 1 was to initiate at the abutment and proceed into the span.

NCHRP Report 725 – Compliance Checks

As discussed previously, NCHRP Report 725 (8) was published after the design of this structure and we have since performed a cursory review of the report contents as it relates to skewed structures. In general, our analysis and detailing approach is consistent with the report. Below are a few specific issues of note.

NCHRP Report 725 provides guidance for the girder web out-of-plumb tolerance after all dead loads are placed on the structure. The tolerance should be within D/96, where D is the girder web depth. Since this structure has a 56-inch girder web depth, the tolerance is 0.58” or 0.6 degrees from plumb. Based on our analysis, the web layover at the abutment bearings is approximately 1.2 degrees from the application of deck slab dead load. By using the TDLF method of detailing, the girders will be rotated out of plumb during erection and rotate back toward plumb after the deck slab dead load is placed.

NCHRP Report 725 defines the severity of a straight, skewed bridge by using the Skew Index, I_s, as follows:

$$I_s = w_g(\tan \Theta)/L_s$$

Where w_g is the width of the bridge measured between fascia girders, Θ is the skew angle measured from a line perpendicular to the tangent of the bridge centerline, and L_s is the span length. For this structure, the I_s=1.20. As per the report, for an $I_s > 0.65$, cross frame forces and flange lateral bending stresses computed based on a 1D or 2D analysis are considered unreliable, and hence a 3D analysis is recommended. We performed both a 2D and 3D analysis of the structure and found that the magnitude of cross frame forces were fairly similar. However, the 3D analysis was beneficial in our study of the lateral forces resisted by the bearings as well as obtaining girder rotations for the different dead load conditions. In addition, for $I_s > 0.35$, NCHRP Report 725 recommends using the TDLF method for small to moderate span lengths which is consistent with the method used for this structure.

NCHRP Report 725 recommends that in general, the first intermediate cross frame should be located at an offset distance along the girder of 1.5D from the abutment bearing location, where D is the girder web depth. The intent is to limit the magnitude of the cross frame forces in these areas, which have been known in the past to be excessively high. Also, it is often difficult to make the cross frame connections to the girders at these locations and the Contractor may need to use excessive force during erection.

These potential fit-up issues were echoed by fabricators who were contacted during the final design phase. In the case of our structure, there are cross frames as close as three feet from the adjacent abutment bearing, which produces a ratio of 0.65D. Results of the 2D and 3D analyses did not indicate any significant increase in cross frame member forces when compared to other locations along the structure. However, provisions were included in the plans to provide some additional flexibility in the framing system at these locations to aid in fit-up. These provisions included using oversized holes in the connection plates at cross frames located three feet from the abutment bearings as well as all abutment end diaphragms. In addition, the connections of these select few intermediate cross frames were specified to be finger tight prior to the deck pour and then fully tightened after the deck pour sequence was completed.

Subsequent to the design of this structure, NCHRP Report 725 was published and does not recommend the use of oversized or slotted holes in the cross frame connection plates because this can decrease the bracing characteristics of the cross frame. However, due to the rigidity of our framing plan near the abutments (full depth end diaphragms and cross frames placed three feet from the abutment bearings), it was thought that providing oversized holes will allow the necessary flexibility during erection while not compromising the functionality of the bracing.

Conclusion

At the time of this writing, the subject structure had not been erected and therefore, additional information on the steel erection, deck pour sequence, and performance could not be discussed.

In general, this paper attempts to detail some specific issues faced by the authors during their design, but also to offer some general guidance that will direct other designers to specific code requirements and industry guidelines for severely skewed bridges that have been established to date. Awareness of the codes and guidelines in conjunction with the use of advanced modeling techniques will prepare the designer for some of the obstacles that he or she will face when designing a severely skewed bridge.

In conclusion, designers are faced with many challenges while designing highly skewed continuous steel girder bridges; including framing plan and cross frame layout, differential deflections between adjacent girders, web layover, thermal analysis, bearing orientation, bearing uplift, deck pour sequence, and more. The challenges and subsequent decisions made during design have a ripple effect across the structure that is compounded by the severity of skew. It is vital that the above issues are evaluated and the decisions reflected in the bid documents to minimize or eliminate fabrication and erection issues during the construction phase.

Fortunately, the recent release of NCHRP Report 725 has provided designers with an important resource that compiled a generous amount of information and outlined the typical challenges of designing highly skewed bridges. In addition, NSBA's continuing efforts to address proper fit conditions is a valuable resource for designers who face the challenge of designing severely skewed bridges.

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