KINKED STRAIGHT GIRDERS FORMING HORIZONTALLY CURVED ALIGNMENTS ON NORTHEAST ANTHONY HENDAY DRIVE

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BIOGRAPHY

Myles Lewis is an Intermediate Bridge Engineer with the Calgary, Alberta office of Stantec Consulting Ltd. Myles has experience with detailed design of pre-stressed and post-tensioned concrete members, cable supported structures, truss structures, and structural steel girder structures. With use of these components applied primarily on highway bridges, pedestrian bridges, and limited experience in rail bridges.

SUMMARY

The extension of the Anthony Henday Drive from Manning Freeway to Whitmud Drive forms the northeast link completing Edmonton’s free-flow ring road. The project consists of nine interchanges with forty seven bridge structures. Of the four horizontally curved multi span steel girder structures on the northeast link of the ring road all of them utilize kinked straight girders.

This paper showcases two multi span kinked straight girder bridges; a 315m long five span bridge with a radius of 340m and a 415m long six span bridge with a 347m radius. Design challenges and considerations encountered during the design and construction are presented. Through the course of the project, benefits and short comings of kinked girders became apparent and are discussed and compared to more traditional horizontally curved girders.

The continuous kinked girder system proved to be an efficient solution facilitating a successful project.

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Figure 1: Bride 23.3 North End (Pier 2 to Abutment 1) [Courtesy of Jad Kfouri, Flatiron Construction Corp.]

Introduction

The purpose of this paper is twofold. First, to showcase two bridges utilizing kinked girders forming horizontally curved alignments. Second, to compare the as built design to a hypothetical curved girder solution that is more consistent with a conventional curved bridge. For frame of reference project particulars, design choices, and results are presented within. It is not the intent of the author to suggest trends to be extrapolated; rather the intent is to share the experiences and context of a notable highlight with industry colleges. As always, it is the duty of the design engineer to determine the most appropriate solutions for the project criteria and local conditions.

Project Overview

The extension of the Anthony Henday Drive from Manning Freeway to Whitmud Drive forms the northeast link completing Edmonton's free-flow ring road. The existing components of the ring road service an estimated average annual daily traffic of 50,000 vehicles. Scheduled for completion in the fall of 2016, Anthony Henday Drive will be the first completed free-flowing ring road in Canada.

Figure 2: Edmonton Ring Road

The project, awarded as a public private partnership (P3) to Capital City Link Group, consists of nine interchanges with forty seven bridge structures. Ten of these structures support curved horizontal alignments. Of these curved structures three are single span bridges constructed with straight girders; three are two span bridges constructed with discontinuous straight NU girders kinked over the pier; and the remaining four are multi span bridges constructed with continuous kinked straight steel plate I-girders.

The two structures being showcased are 23.3 and 25.5. Both are constructed with continuous straight plate I-girders arranged with a series of kinks forming segmentally curved girder lines. The girders are supported by bearings on conventional abutments and hammer head pier / straddle substructures. The structures are shown on plan in figure 3.

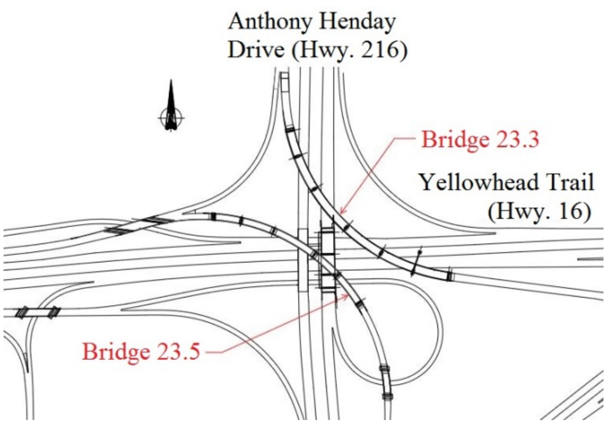

Figure 3: Anthony Henday Drive / Yellowhead Trail Interchange Configuration

Structure 23.3 is a 415m long six span (48m to 92m) flyover with a radius of 347m. The bridge serves to connect southbound Anthony Henday Drive to eastbound Yellowhead Trail with two lanes of traffic on a 14.85m wide deck.

Structure 23.5 is a 315m long five span (48m to 86m) flyover with a radius of 340m. The 11.75m wide deck supports one lane connecting northbound Anthony Henday Drive to westbound Yellowhead Trail.

Project Team for Bridges 23.3 & 23.5:

- Owner: Alberta Ministry of Transportation
- Contractor: Flatiron-Dragados-Aecon-Lafarge (FDAL) Joint Venture
- Owner's Engineer: ISL Engineering and Land Services Ltd.
- Design Engineer: Stantec Consulting Ltd.
- Independent Check Engineer: Dillon Consulting Limited
- Fabricator: Rapid-Span/Structal Joint Venture (Structal-Bridges fabricated 23.3, Rapid-Span Structures Ltd. fabricated 23.5)
- Erection Engineer: All-Span Engineering & Construction Ltd.

Design Methodology

The project specific technical requirements, which dictate the design criteria, is captured by the "Schedule 18" document (1) created by Alberta Ministry of Transportation. Schedule 18 dictates that the bridge design is required to meet the Canadian Highway Bridge Design Code (CHBDC) CAN/CSA-S6-06 (2) in addition to unique and superseding requirements. However, since Schedule 18 and the CHBDC are silent on the specific treatment of continuous kinked straight girders, AASHTO was used as a reference point.

The bridge behavior is largely dependent on the overall geometry. Since the kinked or chorded girders are aligned in segmental chords to approximate a curved girder it is expected that the continuous kinked straight girders will behave similar to curved girder structures. This is consistent with AASHTO LRFD Bridge Design Specifications (4) commentary:

> Note that kinked (chorded) girders exhibit the same actions as curved girders, except that the effect of the noncollinearity of the flanges is concentrated at the kinks. Continuous kinked (chorded) girders should be treated as horizontally curved girders with respect to these Specifications.

As noted above, the presence of the kinks causes concentrated effects that require consideration unique to kinked girders. Further fundamental differences between kinked straight girders and curved girders are noted and discussed in the subsequent sections. Contrary to AASHTO, the resulting methodology used on the project was similar but not identical to curved girders.

Note that 23.3 and 23.5 bridges utilize continuous kinked straight girders, not to be confused with discontinuous chorded girders that act more akin to a skewed bridge with a series of simple supported spans. The effect of continuity increases girder design efficiency, eliminates intermediate joints, increases durability, and inherently changes the behavior of the system.

Flange Forces

Flange forces observed for kinked girder structure are inherently similar to curved girders. Both girder systems are subjected to lateral thrust from noncollinear flange forces. The curvature in a curved girder generates a distributed out of plane flange load, compared to a concentrated point load from a kinked girder that is shown in figure 4.

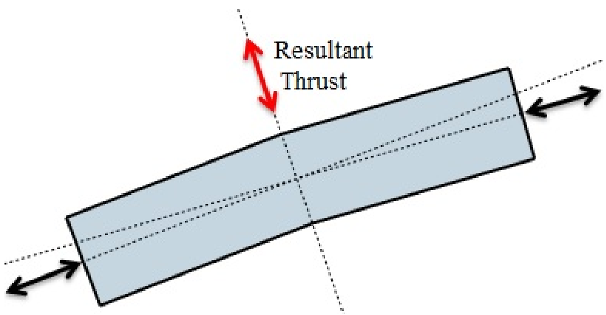

Figure 4: Flange Forces (Plan View)

The resulting flange lateral bending in a kinked girder is predominantly concentrated at the kink locations between cross frames, which is illustrated in figure 6 (case 1). Conversely, flange lateral bending in a curved girder is encountered throughout and is directly proportional to the girder primary bending. Thus, all else being equal, lateral bending forces are highest in governing positive and negative vertical moment regions.

Kinked girder major axis bending is comparable to simplified V-load method of analysis, which is typically applied for curved girders. Thus, in regards to vertical moment demands the girder systems are similar. In both systems, bridge cross section torsion is generated from an eccentric load path formed by the curvature. The bridge torsion is resolved by vertical force couples on the girder lines resulting in additional bending loads on the exterior girders.

Straight flanges are not affected by the tendency of a curved girder to bow under compression or straighten under tension. The capacity of the kinked girders thus resolves to straight girder equations. At the point of the kinks these effects as well as second-order stability need to be considered. Although, with significantly reduced unbraced lengths the second-order bending can be negligible.

Regardless of the girder system, peak stresses resulting from vertical bending and flange lateral bending are of prime concern and checked appropriately.

Web Forces

During the course of design it was noted that for curved webs CHBDC did not allow the use of tension field action in determining shear capacity. However, the updated 2014 CHBDC (3) allows tension field action reflecting research that shows vertically stiffened webs in curved girders have shear resistance similar to straight girders. Thus, with transverse stiffeners, kinked girders no longer have an edge over curved girders in shear capacity.

Tension field action was neglected at the discrete bends in the web alignment. However, the shear demand at the kink locations was sufficiently small for this to be a non-issue.

Level of Modeling

Of course one of the early decisions made was in regards to how the analysis of the structures would be performed. With justified interest in the kinked locations a 3D model was implemented. The first models created were 3D plate eccentric beam models employing a frame element to represent the I-girder section. While this model adequately captured the vertical moments, validated against simplified methods, it was not without shortcomings. Flange lateral bending was difficult to determine with this model type, although, were initially approximated based on the flange axial forces and misalignment (figure 4). Additionally, the non-composite deflections were not adhering to expected results.

Well documented in NCHRP report 725 (5), modelling software commonly neglects the contribution of flange lateral bending, aka girder warping, contributing to torsional rigidity. In light of this, a revised model employed frame elements for the flanges and a shell element for the web as depicted in figure 5. It is noted that the deflected shape was smoother, less disjointed, at the kink locations than the former model and reported over 30% less deflections for non-composite loads. The difference in composite load deflections were less pronounced, which is attributed to the torsional stiffness of the deck.

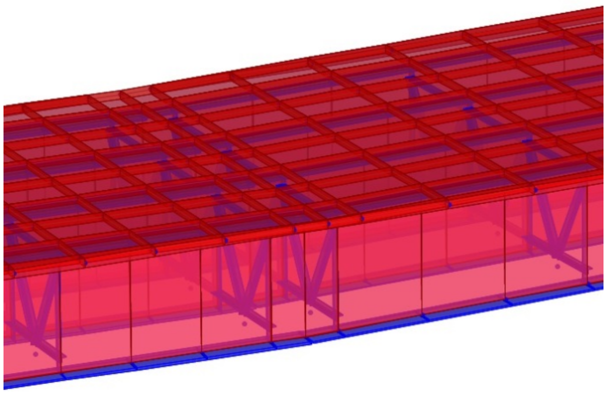

Figure 5: 3D Finite Element Analysis Model

Overall the difference in deflections between the model types was not as staggering as those reported in NCHRP report 725 for a curved girder bridge example. This is suspected to be the result of the kinked geometry forming pseudo hinges not far from natural inflection points. Essentially, straight balanced cantilevers over the piers support simple span straight segments.

Since 3D FEA models were employed to capture the kinked effects, an argument can be made that the modeling efforts are more extensive. This is certainly true if one were not predisposed to the same level of modelling regardless of the girder system.

Primary Tension Members

In structures with horizontally curved alignments, the cross frames may form an essential load path to transfer bridge torsion between girder lines. It is well known and documented within CHBDC and AASHTO that all cross frames within curved girder bridges are primary members. However, the distinction of primary members for cross frames within kinked girder bridges is less well known. Complicating matters, the definition and treatment of primary members is inherently different between the codes. Thus, it would be over conservative to designate all of the cross frames as primary members as per AASHTO commentary and then proceed to treat these members as CHBDC would. This is because CHBDC requires fracture-toughness requirements for primary tension members to meet Charpy V-notch testing. Conversely, AASHTO leaves the requirement for Charpy V-notch testing of primary cross frames subjected to tension at the discretion of the owner.

With this in mind, the members are categorized as primary tension members and secondary members as defined by CHBDC to be consistent with the project design criteria. A primary tension member is a member, or portion thereof, subjected to tensile stress; where the member performs an essential role in load transfer and, on becoming ineffective, will substantially reduce the load-carrying capacity of the system. A secondary member is defined as a component that does not have an essential role in load transfer and whose removal from the structure would result in at most a minor redistribution of load effects to adjacent components.

The fabricator, with coordinated input from the owner’s engineer, proposed that only the cross frames in the immediate vicinity of the kinks were required to be considered primary tension members subjected to mandatory Charpy V-notch testing. The remaining cross frames were considered as secondary bracing members, similar to straight girder bridges. Consensus was made to ratify this methodology for Northeast Anthony Henday Drive kinked girder bridges. However, not all engineers may agree with this approach; as made apparent by an individual resistance during independent design review. The methodology rationale is that the increase in flange lateral bending from cross frame failure, at a kink location, is an order of magnitude more than from failure of an adjacent cross frame. Figure 6 illustrates the concept for a kink located at approximately 376m; where case 1 represents the design configuration; case 2 represents the removal or failure of an intermediate cross frame located at 385m; and case 3 represents a kink cross frame removal at 377m.

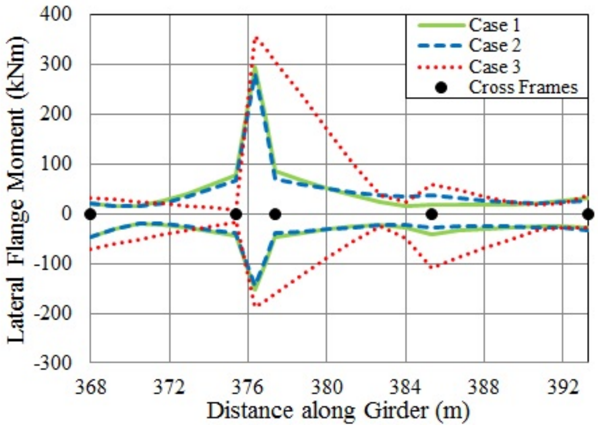

Figure 6: Flange Lateral Bending Under Various Cross Frame Failure Scenarios

As demonstrated the failure of a non-kink cross frame induces a minor change in the primary load path, which is consistent with the CHBDC definition of a secondary member. Conversely, failure of a kink cross frame significantly increases the flange lateral bending and in turn reduces the load-carrying capacity of the structure. This is one of the most substantial differences in methodology between kinked straight girders and the CHBDC approach to curved girders that require Charpy V-notch testing for all cross frames.

Alternately, all cross frames are primary members as defined by AASHTO. However, Charpy V-notch testing is not required for cross frames located away from girder kinks as the consequence of failure is minimal.

Adhering to AASHTO requirements the bottom flange lateral bracing, utilized to control wind deflections during construction, would be required to meet fracture-toughness requirements. However, Schedule 18 and CHBDC have no such stipulation. The lateral bracing is not considered a primary member because the lateral bracing is only strictly required during construction. After the deck has cured the bracing could be removed without repercussion. To facilitate the desired secondary nature of the lateral bracing, the girders were designed for service conditions without the added stiffness of the lateral bracing.

Regardless of the girder system utilized, plate I-girders are welded primary tension members, and are thus required by Schedule 18 to be CSA G40.21M-Grade 350AT CAT 3 or ASTM A709 Grade 345WT Type B with a Charpy value of 27 J @ -30°C. The kink cross frames are primary tension members, comprised of bolted back to back angles that must satisfy CSA G40.21M Grade 350AT, Category 2 or ASTM A709 Grade 345W Type B, with Charpy V-notch toughness of 27J at -20°C. All the remaining cross frames and lateral bracing are secondary members, comprised of bolted back to back angles that comply with G40.21M-350A.

Fracture Critical Members

If the girders were fracture critical members, it is possible that the fracture critical requirement may extend to the primary tension member cross frames. However, since the cross frames are typically spaced frequently, there is inherent redundancy by virtue of the quantity of cross frames on a curved girder bridge. Conversely, for kinked girder bridges the effect of curvature is concentrated at the kink locations; thus the number of cross frames engaged is reduced resulting in reduced or eliminated the redundancy. As a result, kinked girders have an increased likelihood of progressive collapse initiating from cross frame tension failure. However, this is only applicable where failure of one girder leads to the collapse of the structure or failure of a single cross frame member would simultaneously fail two girders.

With Schedule 18 mandated minimum of four girder lines, well above the established two girder line limit for fracture critical girders, consideration of fracture critical members was not required for the cross frames nor the girders for this project.

Geometry Selection

Notwithstanding the locked in road geometry, there remains several key geometry choices that affect the bridge behaviour and design. The subsequent sections discuss the evolution of the internal geometry choices for bridge 23.3 and 23.5 that are independent of the road alignments.

Girder Geometry

As required by Schedule 18 structures 23.3 and 23.5 employ four continuous girder lines each. The resulting girder spacing is 3.85m and 3.15m respectively.

The decision to utilize kinked straight girders, instead of curved girders, for bridges with horizontally curved alignments was the result of early discussions with the contractor and fabricator team. Curved girders were contemplated but ultimately ruled out in favor of kinked straight girders because of increased simplicity, fabrication productivity, and decreased cost. Furthermore, the early discussions served to increase design efficiency with established section size criteria.

Flange widths were limited to multiples of 300mm to ensure efficient material use, ripping multiple flanges form a rolled plate. Flange thickness increments were limited to 5mm in an effort to standardize the plate used for all plate girders on the project. The resulting flange sizes varied from 600mm x 30mm to 900mm x 75mm over high demand regions near the piers.

The maximum feasible web depth was limited by the fabricators capacity at 3.7m. Deeper webs would have required a longitudinal web splice that is cost prohibitive. Additionally, clearance requirements from the underpassing roadways limit the maximum girder depth possible near the flyover entrance and exit. As a result, variable depth girders were required to allow for suitable clearance at the end spans, while meeting a reasonable depth to span ratio for the longer middle spans. Linear depth transitions were used for simplicity and ease of fabrication. The resulting web depths for 23.3 varied from 2.22m to 3.52m, allowing for the camber to be cut from the limiting 3.7m plate. Web thickness varied between sections of 23.3 from 20mm to 25mm reducing the number of transverse stiffeners required. The depth variance on bridge 23.5 was less pronounced at 2.4m to 3.0m. With less incentive to vary the web thickness, bridge 23.5 utilized a constant 18mm web.

Kink Locations

The number of the discrete kinks inversely impacts the order of magnitude of the lateral bending in the flanges and the amount of variance in the deck overhang. Due consideration may include the desired field segment length and field splice locations. With the bridge alignment radii and spans involved, two kinks within each intermediate span and one kink at each end span proved to be reasonable for girder design. The resulting kink obtuse angles were in the range of 173.2 to 175.1 degrees. Thus, the flanges are subjected to lateral bending from 6.8 to 4.9 degree difference in flange axial force alignment.

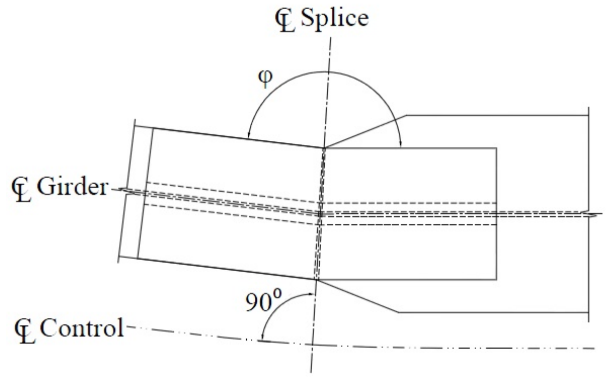

Figure 7: Girder Kink (Top View)

The kink locations were chosen to coincide with a reasonable field segment length for erection. The resulting deck overhang was determined and found to be constructible. With a nominal deck overhang at the kink locations of 1650mm, the overhang on structure 23.3 varied from 1062mm to 2258mm. Similarly, the overhang varies on structure 23.5 from 569mm to 1747mm with a nominal overhang of 1150mm at the kink locations.

Field Splice Locations

Traditionally, the primary consideration for the field splice locations is the location of low demand regions, such as inflection points, and appropriate field section lengths for girder transportation and erection. With implementation of kinked girders, the location of the kinks also requires consideration. The field splices may be located at the kinks or far enough away to not interfere with the kinks.

The kink locations were chosen to produce kinked field splices, eliminating competing efficiencies of placement in low demand regions. This also serves to eliminate a kinked welded splice and reduces the total number of shop splice welds required. Bolted splice plates at the field splices further serve to increase the total steel area of the flanges at kink locations.

The maximum field segment was 41m for both structures. The average field segment length was 37m and 35m for bridges 23.3 and 23.5 respectively.

Cross Frame Locations & Geometry

One distinct advantage of having the flange noncollinearity concentrated at the kinks is the ability to locally strengthen the flanges for these effects. To increase flange capacity, while simultaneously reducing the flange lateral bending demand, an additional cross frame is introduced at the kink locations. With a cross frame located on both sides of the kink the flange unbraced length is greatly reduced. Furthermore, the resultant lateral thrust from the misaligned flange forces has a decreased eccentricity to the supporting cross frames. Figure 5 and 8 illustrate the proximity of the cross frames to the kinked field splice.

Figure 8: Field Splice [Courtesy of AECOM]

Away from the kinks the cross frame members were evenly spaced, such that no spacing exceeded 8m to meet Schedule 18 project requirements.

Cross frames were comprised of back to back angles arranged as K-frames with top chords. Back to back angles serve to increase redundancy and eliminate eccentric connections. The sizes of the angles were limited across the project to facilitate large orders for each size. Angle sizes varied from L102x102x13 to L152x152x16. Note that mass ordering angles has the advantage of permitting orders of CSA G40.21-350A steel. Angles that pass the fracture-toughness requirements by Charpy V-notch testing can be treated as notch tough steel, CSA G40.21-350AT, for use on the primary tension member cross frames.

Lateral Bracing

Bottom flange lateral bracing, consisting of back to back L203x203x16 angles, resist wind deflection during girder erection prior to deck placement. The lateral bracing is located between the interior girders and is not continuous. Rather, the bracing is only located at the girder segments over the intermediate substructure supports. The angles are bolted to gusset plates that are welded to the cross frame stiffeners and the girder web.

Articulation

The substructure supports are aligned radially, aka non-skewed relative to the deck curvature. As a product of the kinked girder alignment, the girders have a slight skew of 3.3 degrees at the abutment ends. The girders are approximately perpendicular to the radial substructure at the intermediate supports.

Each structure is tangentially restrained at the centermost pier support with fixed pot bearings at the interior girders and radially guided pot bearings at the exterior girders. The remaining exterior girder bearings are free in both directions, eliminating radial thermal induced forces. Articulation is provided by guided pot bearings at the interior girders allowing translation parallel to the flange.

As described in G13.1 Guidelines for Steel Girder Bridge Analysis (6), the unrestrained anticipated thermal movement is not aligned with a curved girder axis. The unrestrained temperature movement has both radial and tangential components.

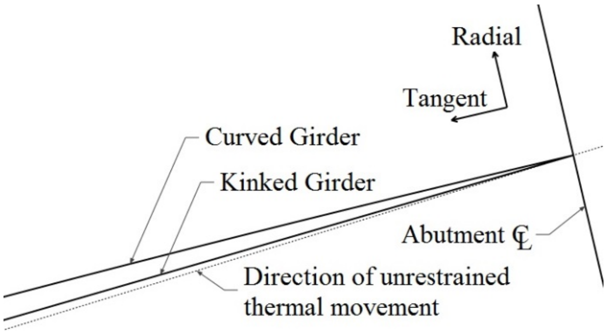

Figure 9: Unrestrained Thermal Movements

Similar to curved girders, although less pronounced, the unrestrained thermal movement is not aligned with the axis of the kinked girders as shown in figure 9. In due consideration to bearing orientation, the bearings must be designed for the forces they attract. With guided bearings aligned to the unrestrained thermal movement of curved structures, the bearings will attract lateral loads from girder major axis bending. Conversely, with the bearings aligned to be parallel to the girder flange, the bearings are instead subjected to temperature loads. Lateral loads acting on restrained bearings are unavoidable, as the bearing restraint is applied to the bottom of the girder and the unrestrained thermal movement is not aligned with the girder axis. At best these loads are mitigated.

To simplify the bearing orientation, the bearing guided direction was chosen to match the girder flange. For the bridge geometry involved, the thermal loads attracted by the bearings proved to be negligible compared to other load effects, such as wind and seismic loading.

Construction Considerations

As is standard practice, the design requires due consideration to the construction conditions and the desired finished product. The subsequent sections discuss the erection plan anticipated and encountered during the course of construction.

Erection Procedure

Despite traffic constraints with Anthony Henday Drive and Yellowhead Trail underpassing the bridges, there was sufficient clearance for shoring towers. Standard shored construction was the accepted erection procedure for design purposes. However, during the course of construction in a P3 project, it is not uncommon for the contractor to request design changes to facilitate the construction schedule in light of unforeseen considerations.

During the course of 23.3 girder erection, a revision was requested by the Joint Venture to accelerate the schedule and reduce the frequency of road closures/detours required by crane operations. The contemplated change to the construction sequence was to install precast deck panels after the three northern spans were completed. Simultaneous erection of the girders and partial depth precast panels would ensue thereafter. To accommodate the panel placement, the bearings must be grouted in stages during girder erection to prevent masonry plate failure. As part of the request, shoring towers were to be removed from completed spans prior to continued girder erection.

To be expected this significantly changes the locked in, non-composite loads experienced by the girders. However, with enough completed lead spans in place, continuity action is not dissimilar from that of the completed structure. It was determined that the construction stresses and resulting locked in forces were manageable with one to two unloaded lead spans.

Mitigating stresses is only one part of the problem. Ensuring fit during erection proved to be more onerous. Excessive deflections, complicating girder installation, precluded the removal of key shoring towers. Figure 10 demonstrates the issue encountered with the requested construction sequence. Figure 1 shows the shoring towers in question, located under the 92m long span.

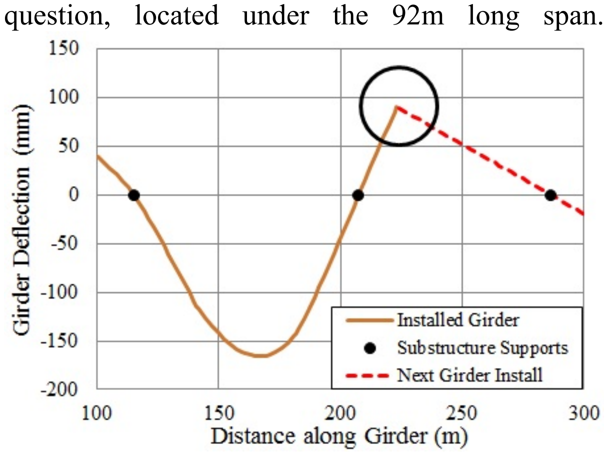

Figure 10: Erection Misfit

Furthermore, the owner's engineer was concerned with additional locked in bearing rotations produced by grouting the bearings in the proposed stages. Girder erection was completed prior to resolution, rendering the request a moot point. However, with advanced consideration this could have been a potential erection scheme.

The girder erection for bridge 23.3 took place from April 27th to July 7th, 2015. Starting at the tangentially fixed support, Pier 3, the girders were erected individually onto shoring towers as required. The heaviest girder segment weighed over 62 metric tonnes. Installation of cross frames and lateral bracing was completed between all four girder segments before advancing to the next span. Figure 11 illustrates the completed cross frames to be connected to the next girder segment. Girder erection proceeded to the north abutment before continuing from Pier 3 to the south abutment.

Similarly, bridge 23.5 began girder erection at the tangentially fixed support, Pier 2, and proceeded to the east abutment. The girder erection then continued from Pier 2 to the west abutment. Figure 12 and 13 capture the erection sequence from Pier 2 to Abutment 1. Figure 14 demonstrates the proximity to the underpassing Yellowhead Trail and the Anthony Henday Drive NU girder bridges. Girder erection was completed from September 17th to October 30th, 2015.

Figure 11: Bride 23.3 North End (Pier 2 to Pier 1) [Courtesy of AECOM]

Figure 12: Bride 23.5 East End (Pier 1 to Pier 2) [Courtesy of AECOM]

Figure 13: Bride 23.5 East End (Abutment 1 to Pier 2) [Courtesy of AECOM]

Figure 14: Bride 23.5 Mid-span (Pier 2 from Pier 3) [Courtesy of AECOM]

Fit Conditions

As a result of the bridge curvature, differential deflections between girders on bridge 23.3 are found to be up to 47mm from total dead loads. Detailing the cross frames to connect to the girders under a no load fit condition (NLF) yields a girder layover of 43mm or 1.2% of the 3.6m deep girder exceeding tolerance requirements.

To minimize a lateral bending component from an out-of-plumb girder, our preference is to employ cross frames detailed to fit under total dead load conditions (TDLF) unless it is particularly problematic to do so. With the differential deflection occurring over a 92m long span, the installation forces required to achieve fit under TDLF condition were not expected to be unmanageable. This is facilitated by straight girders that are more torsionally flexible without a curved web, allowing the girders to be more readily twisted to achieve fit.

With a TDLF connection specified, the fit up was performed without incident for both structures. As expected no layover is observed at the radial supports during the course of construction. Upon completion of the deck, the girders were inspected for layover and found to comply with the specified plumb webs for total dead loads. Barrier and asphalt loads, while required for inclusion in the total dead load fit conditions, are not expected to add noticeable girder layover.

A curved girder system has more difficulty obtaining a total dead load fit condition. Instead a steel dead load fit (SDLF) would likely have been employed, as recommended by NCHRP report 725, to obtain a manageable layover.

Girder System Comparison

The commentary herein explores the observed non-technical differences between a curved girder bridge and a kinked straight girder bridge. Table 1 highlights advantages and disadvantages of kinked straight girders vs. curved girders.

Table 1: Girder System Comparison

Consideration	Ideal System	Advantage
Design Simplicity	Curved	Cost
Optimization	Kinked	Cost
Girder Shipping	Kinked	Cost
Girder Fabrication	Kinked	Cost & Schedule
Girder Erection	Kinked	Cost & Schedule
Deck Soffit Forms	Curved	Cost & Schedule
Aesthetics	Curved	Quality
Versatility	Curved	NA

Design Simplicity

The basic design principles for kinked and curved girders are identical. However, the design of curved girders is slightly less demanding. This is in part because of the increased detailed modeling required to capture the effects of the kinks and also because the examples and guides are generally catered to curved girders. Determination of primary tension members exemplifies the increased intricacy of kinked straight girders.

Furthermore, the advantage of a curved girder system is the simplicity in the girder layout. Greater effort is required to determine kink locations and associated girder geometry of a kinked girder system.

Optimization

Provided that the kinked locations are strategically placed, the flange lateral bending from girder noncollinearity is negligible at typically governing locations. The result is optimized sections in maximum positive and negative vertical bending regions. This advantage is offset in part by a higher demand at the kink locations from the concentrated lateral bending. However, with the kinks placed within low demand regions the overall kinked girder system can be optimal. Furthermore, the reduction of primary tension member cross frames that require Charpy V-notch testing is significant.

Cost

With local market and project specific conditions highly variable, it is reasonable to deduce that the most economic girder system, kinked or curved, will likewise vary. NCHRP report 424 (7) suggests that a decreased deck forming cost will generally more than offset the increase in curved girder fabrication and erection. However, our experience, including but not limited to this project, suggests the opposite which is more in line with G13.1 (6) recommendations.

It should be noted that despite the fact that the contractor both financed the project and selected kinked girders, it is not necessarily indicative of cost or other benefits of the girder system itself. Factors such as familiarity may also contribute to the decision process. Familiarity creates inherent efficiencies and reduces risk leading to reduced costs. This efficiency is exclusive to the contractor's experience and is not a direct product of the girder system.

Schedule

Curved girders, while more time consuming to fabricate and erect, expedite deck formwork by using a constant deck overhang. Conversely, kinked straight girders are faster to fabricate and erect but produce variable deck overhangs that are less efficient to construct. Depending on the capability of the contractor, one system may be advantageous over the other. Most local contractors are highly accustomed to variable deck overhangs and generally less so with curved girders. As such, it is intuitive that kinked girders locally prevail for being cited as more schedule efficient.

Performance

No discernable difference in the performance of the kinked straight girders over curved girders is expected. Both structure types are subjected to similar service and fatigue stress ranges considered during the course of design.

Aesthetics

Curved girders are generally understood to be more aesthetically pleasing. The discrete kinks of kinked girders are pronounced when viewed from directly under the bridge. However, the visual difference between curved girders and segmentally chorded girders is subtle, potentially unnoticeable, from a typical viewing distance. Figure 15 & 16 compare a kinked girder to a curved girder structure. Unfortunately, at the time of writing this paper, November 2015, neither bridge 23.3 nor 23.5 are completed for a finished product comparison.

Figure 15: Kinked Girder Bridge Example (Westbound Anthony Henday Drive to Southbound Queen Elizabeth II Highway, Edmonton, Alberta)

Figure 16: Curved Girder Bridge Example (Southbound Deerfoot Trail to Eastbound Stoney Trail NE, Calgary, Alberta)

Limitations

A chorded girder layout is greater impacted by overall geometry. If the curvature is small, requiring numerous kinked locations to manage deck overhang or cross frame forces, than the fabrication and erection costs escalate potentially making kinked girders cost prohibitive. However, for the geometry encountered on Northeast Anthony Henday Drive, kinks were only necessary at standard field splice distances making for a preferable alternative to curved girders.

Conclusion

Despite increasing trends of curved girders, their predecessor, kinked girders should not be overlooked. Continuous kinked straight girders used to form horizontally curved alignments are demonstrated to be a viable alternative to more traditional curved girders. Each system hosts a unique set of advantages and disadvantages. Kinked girders, serving to simplify fabrication and girder erection, have proven to be cost effective and preferred under certain conditions.

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