KENTUCKY LAKE BRIDGE BASKET-HANDLE NETWORK TIED ARCH SUPERSTRUCTURE DESIGN

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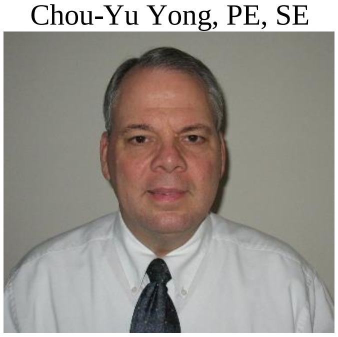

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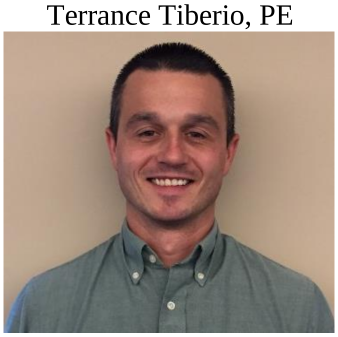

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BIOGRAPHY

Jason Stith is a Civil Engineer at Michael Baker International. He received his PhD from the University of Texas at Austin and an MSCE, MBA, and BSCE from the University of Kentucky. Dr. Stith was the lead designer of the tie and floorbeams for the new KY Lake Bridge

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SUMMARY

The Kentucky Transportation Cabinet (KYTC) is replacing the existing structurally deficient and functionally obsolete truss bridges across Kentucky Lake on US 68/KY 80 in western Kentucky. The replacement includes a four-lane roadway and a multi-use path.

Amidst many challenges surrounding the project including seismic design of flexible deep water foundations and an aggressive schedule, the project team coordinated with KYTC to design the main arch spans, an aesthetically pleasing yet durable and robust signature structure. The 550 foot basket-handle network tied arch span rises 110 feet from the springing line and inclines 15 degrees from the vertical. The steel parallelogram tie girder and floor system is hung from a system of inclined network hangers in a unique alternating single/double hanger arrangement. The integral floor system is comprised of composite stringers with moment connections into composite transverse floorbeams. The arch rib section is an open H-section that provides an economical design, and facilitates inspection but required rigorous consideration of local and global buckling stability. The entire arch span is mounted on seismic isolation bearings which protect the bridge from potentially damaging seismic motions.

INTRODUCTION

The Kentucky Transportation Cabinet (KYTC) is replacing the existing structurally deficient and functionally obsolete Eggner's Ferry truss bridge across Kentucky Lake on US 68/KY 80 in the Land Between the Lakes Recreation Area. The project located in Western Kentucky is shown on Figure 1.

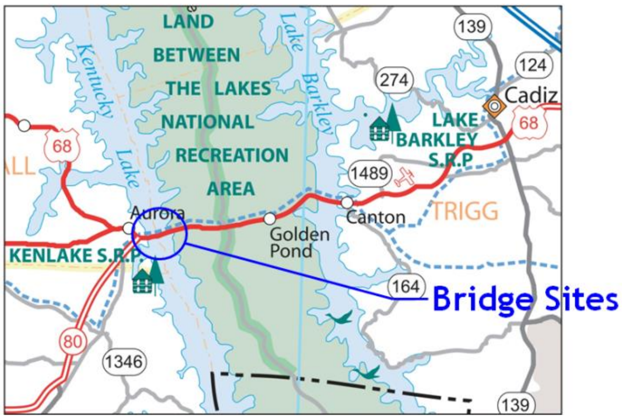

Figure 1: Kentucky Lake Site Map

This bridge represents a major investment in Kentucky's transportation infrastructure and is a key component of the overall US 68/KY 80 corridor improvement project along with the improvement to the maritime industry shipping channel by increasing the navigation span by over 50%. It is one of the longest bridges in the Commonwealth of Kentucky. The total bridge length for this project is 3,611 feet and includes the widening of 2,400 feet of existing causeway to accommodate the new alignment. The bridge type was selected following a robust public involvement process while incorporating fiscal and engineering practicalities. The replacement structure supports a four-lane roadway and a multi-use path. The signature structure chosen was a 550 feet basket-handle tied arch bridge over the navigation channel will be a landmark for the area. Long span steel plate girder approaches lead to the main arch span on a parallel alignment to the existing bridge.

Amidst many challenges surrounding the project, including the high seismic demands, deep water foundations using driven pipe piles, and an aggressive schedule, the Baker project team coordinated with KYTC to design the main span. The result was an aesthetically pleasing yet durable and robust signature structure for the Kentucky Lake crossing. The Kentucky Lake crossing contract was let in December of 2013 with construction to begin in spring of 2014. The construction is scheduled for completion in late 2016.

BRIDGE DESCRIPTION

Kentucky Lake is major navigable reservoir formed by the Kentucky Dam on the Tennessee River adjacent to the 170,000-acre Land Between the Lakes National Recreation Area. This route provides a vital access point to the recreation area and an important link in the region's transportation system. When completed, the new bridge will have total of 3,061 feet of approach spans and a 550 feet main span arch as illustrated in Figure 2.

The general public and a Citizens Advisory Committee (CAC) input have determined the main span bridge type. The arch is 94 feet wide measured from centerline of tie to centerline of tie with 15 degrees inclined arch rib supporting 75 feet and 4 inches wide deck. The new bridge carries four lanes of traffic and a multi-use path will replace the existing two lanes Eggner's Ferry Bridge built in early 1930s. Figure 3 shows the typical section comparison between the existing and the new bridge.

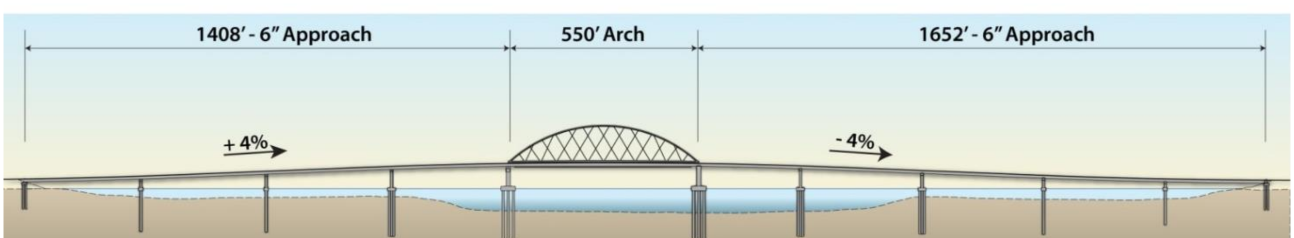

Figure 2: Schematic of KY Lake Bridge Crossing

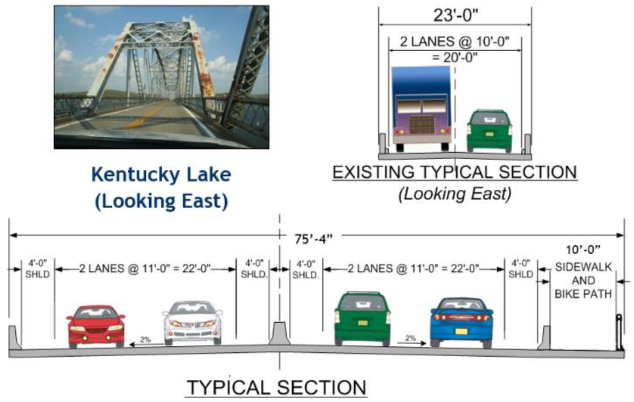

Figure 3: Kentucky Lake Bridge Typical Section

The new bridge improved navigational clearances. The main navigational channel will have at least 502 feet of horizontal clearance with a minimum vertical clearance of 63 feet to low steel from normal pool and 67 feet for a 200 feet wide sailing line in the middle of the span as shown in Figure 4. Combined with the additional rise of the arch for structural form, the new bridge will present a much larger scale than the existing Eggner's Ferry Bridge. It will also create a long distance visible land mark for barge captains identifying the designated navigational span that will mitigate potential confusion by waterway traffic.

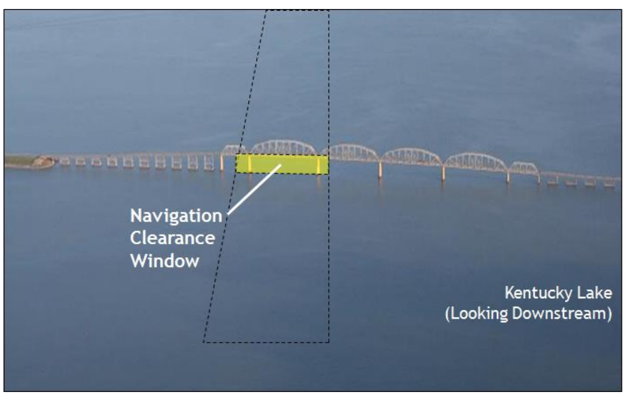

Figure 4: Eggner's Ferry Bridge with Navigation Window of the New Kentucky Lake Bridge Shown

A multi-use path will be provided on this bridge. Given the narrow driving lanes and the lack of shoulder, the existing roadway presents an extreme hazard to bicycle and pedestrian travel.

Contributing factors to the pleasing aesthetics of the bridge include the span-to-rise ratio of 5:1, inclined tie geometry, inclination for a basket-handle effect and the visual stability of the networked hanger configuration. Consistent with commitments made during type selection process, the arch exhibits an open look to fit the natural environment through elements such as the clean lines of the Vierendeel rib bracing system and the soaring height of the arch. In particular, the open pedestrian railing and custom combination vehicular barrier offer passenger and path users' views of the surrounding natural lake and forest areas from a breathtaking vantage point.

SUPERSTRUCTURE DESIGN FEATURES

The features of the bridge superstructure were chosen for efficiency, simplicity in design, ease of fabrication, facilitating construction, and reduce long-term maintenance.

Integral Floor System

The design team employed a twelve (12) panel floor system with floorbeams spaced at 45'-10". This permitted the use of rolled beam stringers that proved to be cost effective. The 10 feet spacing of the stringers minimized the deck structural thickness to 8". The integral floor system consists of eight rolled beam longitudinal stringers W33x118 supported by total of thirteen plate girder transverse floorbeams which frame into the tie girder. The longitudinal stringers are connected to the floorbeams with moment transfer connections through the use of a top tie plates bolted to the floorbeam top flange and supported by a bracket at the bottom of the stringer. Both the stringers and floorbeams are composite with the cast-in-place concrete deck. Figure 5 shows the bridge integral floor system. The use of an integral floor system offers structural advantages as well as long-term maintenance cost savings to the bridge over a more traditional stacked or "floating" floor system. The following list of advantages were identified by the design team.

1) The integral connection of the stringers to the floorbeam eliminates the need of stringer bearings that require future maintenance and replacement provisions.

2) The stringer to floorbeam connections provides lateral support to the floorbeam which reduces the unbraced length of the

floorbeam compression flange prior to composite action.

3) The stringers are connected to the floorbeam with the web perpendicular to the flange of floorbeam to facilitate erection and eliminate the need for beveled fill plates.

4) The integral floor system results in the top of stringers being flush with the top of floorbeam producing shallower structure depth and a lower roadway profile to meet the vertical navigational channel requirements.

5) The composite floorbeam improves the load distribution in the deck slab and reduces the moment demands on the floorbeams.

6) The floorbeam's capacity is increased by the composite action.

7) The composite system of stringer, floorbeam and deck provides additional structural redundancy to the floor system and improves the seismic performance.

The use of the integral floor system was not without its disadvantages and the design team identified the following challenges that had to be address.

1. Tension stresses that develop in the deck due to the floor tie elongation under concrete deck loads.

2. Tensile stresses in the deck that for as a result of creep and shrinkage of the concrete restrained by the integral floor system.

To reduce the possibilities of deck cracking due to the longitudinal tensile stress, a detailed deck placement sequence is developed for the arch span. The bridge slab placing starts with the panel pour from both ends toward the mid span of the bridge with block outs at the floorbeams. It is followed by the closure pour at the floorbeam. The stringers to floorbeam connections utilized oversized holes which were tightened following the deck placement. This means of deck placement allowed the majority of the floor system elongation to occur without a fully composite deck. To further increase the deck durability and provide for a more maintenance-free structure the design team adopted a multi-tiered approach to provide enhanced protection using stainless reinforcing steel in the top mat of the deck, and a latex modified concrete overlay. The 1 ½" latex modified overlay provides the advantage of further protection to the floor system and in addition provides a smooth riding surface that covers the closure pours on every other floorbeam.

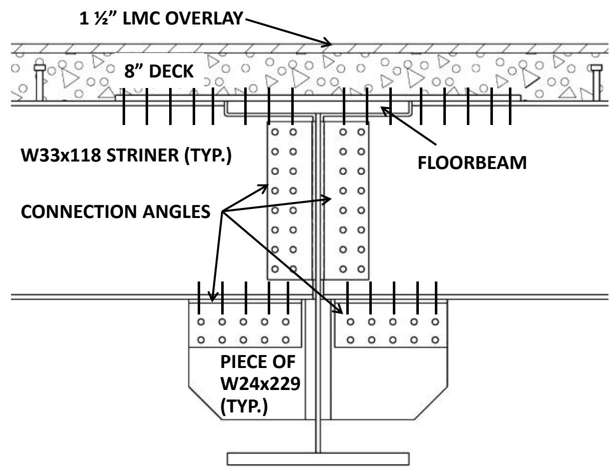

Figure 5: Integral Floor System

Knuckle

One of the most critical elements for an arch bridge is the knuckle which is located at the intersection of the tie girder, arch rib and end floorbeam. This intersection is a highly stressed area that is further complicated by the 15 degree slope inward of the arch rib. The knuckle plate is critical to maintaining geometric control of the arch, and it was decided to slope the webs of the tie girder so there would not be a bend or kink in the knuckle plate. The knuckle plate is fabricated out of one plate to provide continuity between the tie girder and arch rib as the flow of force from the rib into the tie occurs primarily through the knuckle plates without the need for a weld connection. This approach simplified the fabrication of this complicated region.

The arch rib connection is augmented with a bolted connection plate on top of the tie girder flange. The arch rib flanges and web are welded to an auxiliary horizontal connection plate and then the plate is bolted to the tie girder top flange. The top of the tie is stiffened by a plate diaphragm and all load from the tie transfer through the diaphragm in a mill to bear condition.

The tie girder and knuckle plate are extended beyond the permanent bearings in order to provide an area for jacking in future for bearing replacements. The jacking load applies significant shear flow to the bottom of the knuckle region which required a bent plate added to the exterior of the knuckle and attached to the sole plate to provide sufficient shear capacity. The end floor beam at the centerline of bearing also connects to the tie girder through the knuckle plate. Figure 6 illustrates the 3D isometric view of the knuckle.

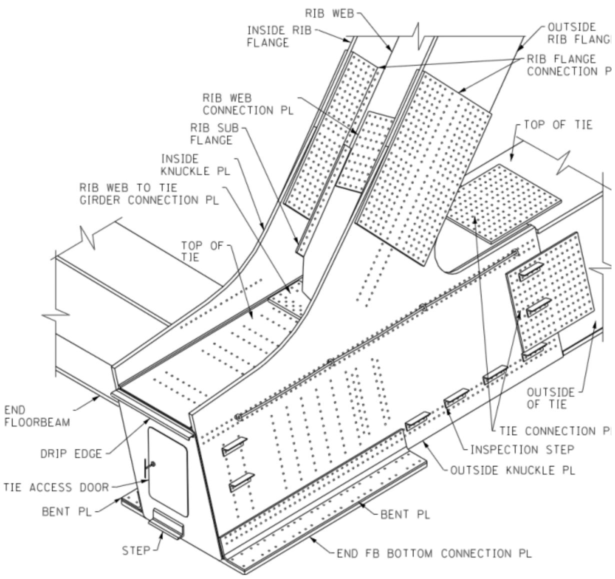

Figure 6: 3D Isometric View of the Knuckle

An exact solution to the boundary value problem is not mathematically feasible. The state of stress inside the various components of the knuckle cannot be determined accurately by the principles of elementary mechanics. Thus the best and most common solution is to discretize the area and utilize a finite element analysis to approximate the solution. A finite element model was developed using 3D Finite Element Program Midas Civil for stress analysis to determine the knuckle plate thickness and the internal diaphragms stress flow within the tie girder.

The detail knuckle region model which consisted of plate elements was merged with the beam element global bridge model. The limits of the detailed model were determined using engineering judgment and by applying St Venant's principle. St Venant's principle states that stress concentrations due to boundary conditions should be dispersed by a unit dimension away from the boundary condition or stated otherwise the boundary condition should be located at least one girder depth away from the area of interest. The boundaries of the model were set at locations sufficiently removed from areas of disturbed stress and deformations which are of the primary interest in the modeling. These locations were set to about the 28.5' along the arch rib, 24' along the tie girder and 9.2' along the end floorbeam. The challenge of transitioning from 3-D beam elements to built-up plate elements was overcome by utilizing a transition element called a "spider web" link. The spider web links are beam elements which are very stiff flexurally, but with a moment release or pinned at the plate element. Thus rotational continuity is preserved without applying in-plane bending of the plate elements. This type of transition element assumes the Euler beam theory assumption that plane sections remain plane. Additionally, the link elements are axial very soft to allow for the Poisson's effect in the plate elements without over-constraining the member and creating false stresses. The terms very stiff and very soft are deliberately used to indicate 2 or 3 orders of magnitude larger or smaller than the extreme stiffness value of the attached elements. Utilizing stiffnesses many orders of magnitude larger can result in numerical problems caused by computer number storage also known as an ill conditioned stiffness matrix and render the results of the entire model erroneous. Figure 7 is the screenshot of the knuckle model with "spider link" element.

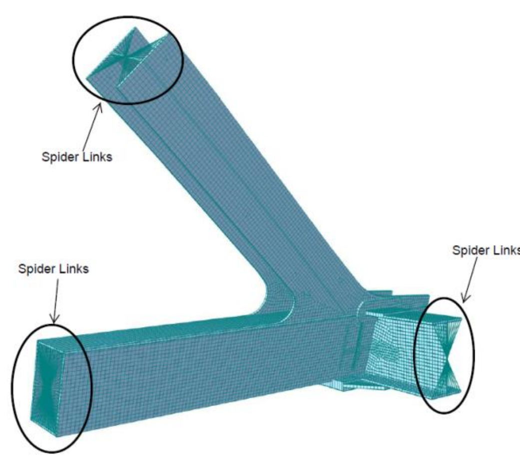

Figure 7: Finite Element Model of Knuckle Region with Spider Link Elements Shown

While global performance of the arch and tie girder proper were checked using the results of beam models employing the provisions of the AASHTO specifications[1] where applicable, the knuckle was designed by checking the effective stress of the knuckle and by imposing the Von-Mises yield criterion. The knuckle stress was comprised of factored strength load combinations which were enveloped in Midas. The results were then compared with the yield stress of the steel as follows:

$$\sigma_e \leq 0.95F_y \qquad (1)$$

The flow of forces in the knuckle originates from the loads applied by the tie, rib, and floor beam. The flow of force from the rib into the tie occurs primarily through the knuckle plates. Figure 8 is a screenshot of the vertical stress within the knuckle the red and yellow represent areas of higher stress and it can be seen that the knuckle plates are more highly stress with the stress flowing from the rib through the knuckle plates and to the bearing.

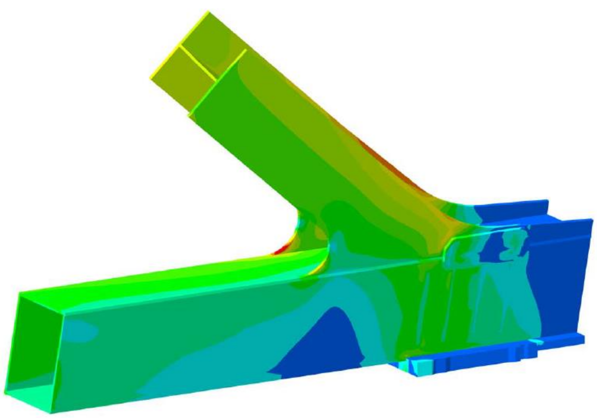

Figure 8: Knuckle Vertical Stress

In addition, the arch thrust is directly related to the decomposition of forces from the rib and since the rib transferred most of its load to the knuckle plates then the assumption was that thrust resulting in tension in the tie would originate in the knuckle plates and need to flow into the top and bottom flanges. Thus a proportional amount of tie girder tension was assumed to flow to the flanges as shear through the sub-web. The shear flow of the knuckle, arch ribs and tie girder were plotted to verify the hand calculation of horizontal shear shown in finite element model.

The design team performed a linearized eigenvalue buckling analysis on the merged model to determine the knuckle local buckling capacity. The critical local buckling zone was located near the tip of the knuckle-rib flange. The yielding of the plate governs over the plate local buckling and is consistent with the knuckle-rib design in hand calculation.

Tie Girder

The tie is a critical element in a tied arch bridge due to its role in resisting the thrust produced by the rib at the supports. This large tension force can be efficiently resisted by a properly designed steel tie member. The geometry of the tie is set to facilitate fabrication and erection of the entire bridge. As was mentioned previously the most geometrically complicated element in the bridge for fabrication is the knuckle region located at the intersection of the rib and the tie. By inclining the web of the tie to match the rib, the knuckle can be fabricated as a planar element from a single piece of steel. The design trade off in simplifying the knuckle connection is that the resulting tie is a parallelogram cross-section. The parallelogram is an unsymmetrical shape that experiences bi-axial bending. There are no standard AASHTO equations to determine the buckling capacity or interaction in equations to relate the axial and moment demand-to-capacity ratio.

One of the main design considerations and the controlling load case for the tie was the possibility of any one of the tie girder plates developing a crack that results in the complete loss a single plate's load carrying capability. The remaining three plates would then be designed to prevent failure of the structure in this extreme event. To prevent a crack from propagating to adjacent plates the tie girder was not welded together, but rather the plates are fabricated separately, a common practice for modern tied arches in the US. The plates were stitched together using sub-flange connections which are welded to the webs and bolted to the flanges. All other attachments to the tie were bolted rather than welded to prevent fatigue sensitive details from initiating cracks on the tie. Thus the tie was designed with internal redundancy.

To determine the capacity for the full plastic capacity of the tie the design team utilized a cross-sectional fiber analysis for determining longitudinal forces and combined this with the shear and torsional forces to produce the von Mises stress in the tie. This approach allow for the exact determination the of force distribution within the cross-section for each of the load cases. The fiber analysis allow the full plastic moment capacity of the tie to be calculated at any point including the extreme case of plate fracture.

The depth of the tie is set to a depth that facilitated the floor beam to tie connection. This connection utilizes connection plates on the top and bottom of the tie to connect to the top and bottom flange of the floor beam.

Arch Rib and Bracing

The use of H-section rib facilitates future inspection and maintenance on the arch ribs and hanger connections. The open surface of the H-section eliminates the need for access inside the rib and improves the quality and safety during inspection. In addition, the H-section offers fabrication advantages due to easier assembly and welding access as compared to a box section. This provides cost efficiencies and increases the quality of product. The bolted rib splices were designed to transfer 100% of the axial load due to past experience by the owner in which compression connections did not achieve full mill to bear. This eliminated the rigorous rib connection fabrication tolerances by adding a few extra rows of bolts to the connection. Figure 9 shows the typical open H-section Arch Rib.

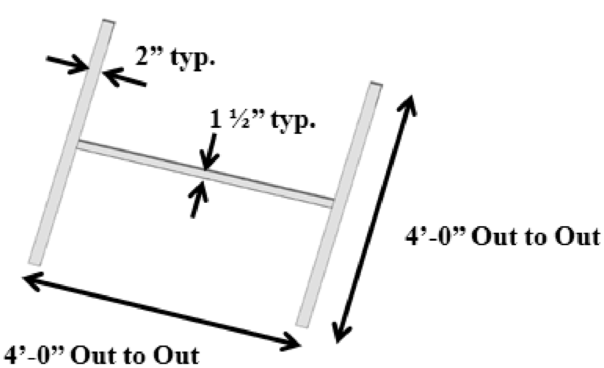

Figure 9: Typical Section of Arch Rib

Being a slender open section, torsional moments result in warping stresses which are additive to the flexural and axial stresses in the member. When subjected to torsion, an open section tends to warp out of its plane of bending. Where restraint of warping is provided, this restraint inherently builds warping stresses. There is a limitation of most finite element program's beam elements to take account for the warping stiffness since warping is not an independent degree of freedom. While a plate/shell representation of the rib can capture these effects, it is impractical for design purposes. Alternatively, when using beam elements, the torsional constant J may be altered to approximate for the additional stiffness attributable to the warping stiffness as recommended by NCHRP Report 725[1].

The magnitude of the second order effects, moment magnification, is included in the arch rib Strength and Service demand which shall be performed with an incremental geometric nonlinear analysis in accordance with A4.5.3.2.3[2]. The analysis is a single moment magnification and the both in-plane and out-of-plane moment magnification (δ_b and δ_s) are set to unity. This approach will also be used to determine the relative magnification applied to the internal stresses taken from the linear analysis for Extreme Event I –seismic and Extreme Event IV – hanger exchange/ hanger loss.

Baker team performed an elastic linearized buckling analysis, also referred to as linearized eigenvalue analysis on an undeformed structure with consideration of the full geometric stiffness matrix, including terms by solving the following equation adopted from W. McGuire, R.H. Gallagher and R.D. Ziemian (2000)[3].

$$[K_e + \lambda K_g]\{\Delta\} = \{0\} \qquad (2)$$

This analysis provides a theoretical solution for the buckling modes of a structure and the resulting buckling factor shall satisfy the following at the strength limit state:

$$\lambda > 2.5 \qquad (3)$$

This limitation is not codified but is selected using engineering judgment as a reasonable limit under factored loadings commensurate with the designer's experience.

An incremental, iterative geometric nonlinear analysis is carried out on imperfect geometry of a structure to compute second order effects on the internal forces. Initial imperfection is introduced to the structure as in accordance with AISC Section C2.2a (2011)[4], the magnitude of the initial displacements shall be the maximum amount considered in the design and the pattern of initial displacement shall be such that it provides the greatest destabilizing effect. For this analysis, the buckling load is calculated by means of a series of subsequent loadings of the factored AASHTO Strength load combinations that were incremented to the maximum loading until the analysis fails to converge. The buckling mode shape could be determined from the deformed shape from the last successfully iterated step of the analysis. It is found that the maximum stresses are larger than the 70 ksi yield stress of the steel arch rib. Thus, the design of the arch rib is controlled by strength, rather than global stability of the arch.

Lateral stability of the rib is provided by rib bracing. Again an open H-section is chosen to eliminate the need for internal access to a box. The web of the bracing is horizontal and the connection to the rib is made using extended end plate moment connections. Figure 10 is a connected to the rib. Large corner clips have been provided in the bracing web and stiffeners to aid in draining the area. The bracing gives a clean and open look as compared to X bracing which is a preferred by the client and public. The bracing was located as high as possible to deliver the desired visual effect while maintain the structural stability of the rib.

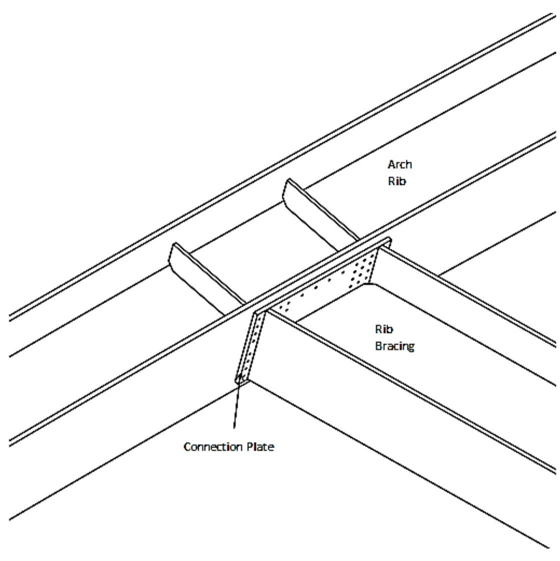

Figure 10: Isometric View of Rib Bracing Connection

Hanger System

The structural strand is chosen because of the client's preference to the inspect ability of the hanger system. Stay cable anchorages are typically located inside the tie and require a penetration of the tie girder that needs sealing and subjects the hanger to fretting corrosion at the hanger/tie interface. Inspection and maintenance is also more difficult with the anchorages inside the tie, and furthermore, the tie depth was found to be extremely tight to do strand jacking. Meanwhile, structural strand offers simplified constructability and much more accessibility during inspection and maintenance activities, and completely satisfies the client preference for a fully visible and accessible hanger system

The structural strand lower end connection is designed to be above the tie girder top flange which requires no penetrations that need sealing and reduced possibility of fretting corrosion. The upper end connection is designed to be hung from the H-section arch rib. The upper connection itself is made up of only bolted elements. The lower end anchorages are easily accessible from deck level for inspection and maintenance activities, and the upper end connections are fully accessible by vertical lift or by climbing. Figure 11 is a typical hanger to tie girder connection detail.

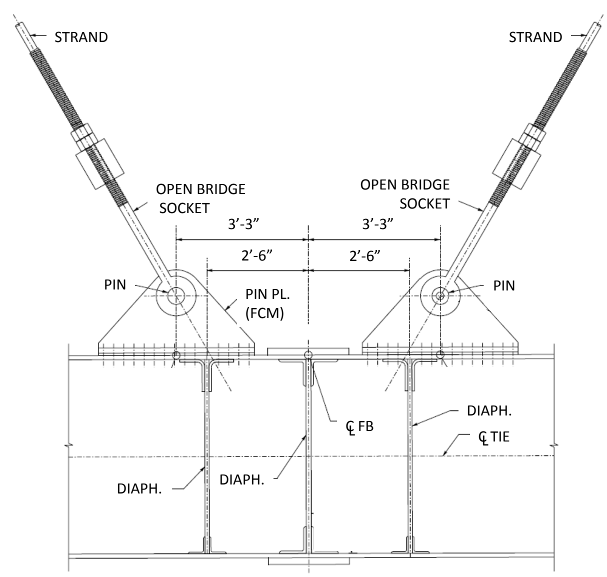

Figure 11: Hanger Connection at Tie Girder

Hanger corrosion protection is provided by hot-dipped galvanization of structural strand conforming to ASTM A586, Grade 1 Class A inner wires and Class C outer wires. Additional protection for the inner wires is provided by lubrication of the inner wires with corrosion inhibiting grease injected during manufacturing of the strand.

The network hanger configuration proved to perform better, resulting in lower live load moments in both the rib and tie and thus a more economic rib and tie design. The current configuration utilizes hangers with angles that vary approximate from 56° to 66°. The greatest economic benefit was the ability to utilize an H-section for the rib which would not be possible for the vertical hanger configuration. The in-plane bending moment is the only degree of freedom that changes significantly when a hanger is lost. The loss of either of these cables significantly increases the unbraced length of the rib. Due to the hanger's ability to brace the rib for in-plane buckling, which is critical in the H-section rib design, specific locations have two hangers to ensure buckling stability of the rib in the extreme event of a single hanger loss. The dual hangers system is those which geometrically provide stiffer bracing for the rib with regard to in-plane buckling. The interaction equations used in the design allowing for the simultaneous presence of axial load, biaxial bending and torsion are being used to assess the strength and stability of the rib and tie under this extreme event.

SEISMIC DESIGN

Kentucky Lake lies within the close proximity of the New Madrid Fault Line, approximately 80 miles away. Thus, seismic considerations factor heavily into the design of the bridge. KYTC is classifying the bridge as "Essential" and has adopted 1000-year return event earthquake for the bridge design. The seismic design followed a displacement-based design procedure including a pushover assessment of displacement capacity as outlined in the AASHTO Guide Specification for LRFD Seismic Bridge Design 2^{nd} Edition[5]. The bridge is designed to remain elastic during and after seismic event. Since the route will serve as an evacuation route and allow access to the area by first responders after a major earthquake, the bridge is designed to remain open and require only minor repairs after a significant seismic event. The type III earthquake resisting system (ERS) was chosen for this bridge which utilizes seismic isolation between the superstructure and the substructure to dissipate energy and lengths the fundamental period of the structure.

To increase competition and provide the most cost effective seismic design solution the two seismic isolation systems were utilized in the design, a friction type isolation bearing and a lead-core elastomeric isolation bearing. Both the superstructure and substructure seismic forces were bracketed with the most flexible isolation system and the stiffest isolation system so that either could be purchased by the contractor and perform within design parameters. The presences of the isolation bearings significantly reduced the seismic demands on the superstructure which resulted in only the weak axis bending of the end floorbeams significantly affected by the seismic demands. The effect on the substructure was equally beneficial as compared to ERS type I which would require plastic hinging of the pier columns. For the new Kentucky Lake Bridge the piers were very stiff and the pipe piles in deep water that supported the piers were relatively flexible. Thus producing a hinge in the column without first yielding the pipe piles would have been nearly impossible. Figure 12 is a photograph of a two seismic isolations bearings being proof tested prior to installation on the new Kentucky Lake Bridge.

Figure 12: Seismic Isolation Bearings Verification Test Used on KY Lake

Another seismic mitigation strategy was the use of viscous fluid dampers between the main span and the approaches. Installation of the viscous fluid dampers on the approach girders to the main span help the bridge to perform as a single unit and eliminates any potential out-of-phase seismic movement. To account for longitudinal and transverse large seismic movement, a modular joint and an articulated barrier that allow hinge movement is used at both ends of the arch span.

The majority of the seismic design was accomplished using a multi-modal response spectrum analysis. This method is relatively quick computationally and provides conservative results for member forces and displacements. At the end of the design phase a nonlinear time history analysis was performed to check specific critical member forces, compare displacement demands to pushover analysis, and ensure the modular joints were sized appropriately.

CONCLUSIONS

The commitment to fast track the project presents a challenge to the owner and design team. Baker design team took the approach to design for fabrication, construction, maintenance and inspection with major goal to minimize the project risk. Also, KYTC working together with Baker to market the project to contractors, fabricators and industry group to maximize the competition. This resulted in 8 contractor teams submitting very competitive bids with the eventual winner Johnson Bros. Inc., a Southland Company. The winning bid was $131 million or approximately 13% below the engineers estimate. These goals were met with challenges along the way and solved by the Baker design team with close support from KYTC. After the completion of the bridge, KY Lake Main Span will be the first network arch bridge in the state of Kentucky. A sister bridge to the Kentucky Lake Bridge was designed is currently under construction on eight miles east on Lake Barkley by PCL Constructors Inc. At the conclusion of the two projects they will form signature bookends to the US 68/KY 80 corridor through the Land Between the Lakes.

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Owner: Kentucky Transportation of Cabinet

Engineer of Record (Main Span)**:** Michael Baker International

Engineer of Record (Approach Spans): Palmer Engineering

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