May 31,
2017

City of Cedar Rapids

8th Avenue Bridge Validation Study

Introduction

As part of the proposed Cedar River Flood Control System (FCS) for the City of Cedar Rapids, Iowa (City), the City is investigating options for replacement of the 8th Avenue Bridge over the Cedar River. The purpose of the bridge replacement is to provide an alternative lifeline facility (to the Interstate 380 bridges) that would be owned and operated by the City to maintain vehicular access to the downtown area of Cedar Rapids in the event of a major flood. As part of the FCS, floodwalls and earthen levees would also be built on both sides of the river for flood protection, with the profile of the new 8th Avenue Bridge to be placed above the floodwalls.

Through various polls, panels and public input surveys, the City, along with Shoemaker and Haaland Professional Engineers has identified a preference for either an arch or a cable-supported extradosed structure for the new 8th Avenue Bridge. HDR was retained as a bridge specialist to help define a final bridge type selection for the City. The purpose of this engineering study is to validate the feasibility of these two structure types and determine an initial preferred structure type and material based on historical cost data for these types of bridges as well as other contributing factors.

The existing 8th Avenue Bridge over the Cedar River was originally constructed in 1938 and is 659-foot long steel girder bridge. The two west approach spans and the single east approach span originally consisted of six lines of rolled steel beams and the main unit over the river consisted of six lines of riveted plate girders. In 1969, the bridge was widened from its original 55′-2″ width to 66′-6″ with the addition of new steel edge beams. In 1986, a major deck rehabilitation project was performed. Both abutments and Pier No. 1 are supported on timber piles, while Piers 2-7 are supported on steel H-piles.

The vulnerabilities of the existing bridge were demonstrated in the historic flooding of 2008. Although the top of the bridge deck was not inundated in 2008, the approach roadways were overtopped, rendering the 8th Avenue Bridge useless as a lifeline structure into downtown. Additionally, the bridge was closed due to flooding in September 2016. The planned FCS, which will incorporate floodwalls throughout the downtown area of Cedar Rapids, highlights the need for a reliable alternative route into the downtown area in addition to Interstate 380. The addition of the floodwalls, while providing protection to the downtown area, will narrow the floodway and raise the flood profile. This adds priority to the primary purpose of replacing and raising the 8th Avenue Bridge — public safety during a flood event, which includes flood response, erecting the FCS, and providing access to hospitals.

Site Constraints

The area surrounding the proposed 8th Avenue Bridge has several constraints that will affect design. On the east side of the river, the U.S. Federal Courthouse sits on the north side of 8th Avenue SE. The courthouse was completed in 2011 and includes access points west of 2nd Street SE and on 8th Avenue SE, which would be accommodated via a private tunnel; both access points need to be maintained. South of 8th Avenue SE is a parking lot known as Lot 44, which is under consideration for redevelopment. The proposed levee will be constructed through this Lot 44, which has now been acquired as an FCS asset.

A railroad spur crosses east and west through Lot 44 and connects to a bridge over the river leading to Ingredion, a manufacturer of agricultural and food products situated on the west side of the river and south of 8th Avenue SW. The Cedar River Flood Control System Master Plan includes a concrete floodwall extending from 8th Avenue SW on the riverward side of Ingredion's property. North of 8th Avenue SW is Sunner Memorial Park alongside other underutilized grounds. This open-space park and adjacent grounds were intended for, but never used for festivals and other events. The levee through this park, though still under evaluation, would terminate at the bridge abutment and a stormwater pump station will be situated adjacent to Sunner Memorial Park. The City is currently evaluating the pump station and opportunities for it to be a multi-purpose facility to compliment the park functions. To the northwest of the park is the Cedar Rapids Police Department headquarters.

Geometric Constraints of Proposed Bridge

A Flood Risk Reduction System Hydraulic Analysis was prepared for the City by Hanson Professional Services, Inc. in October, 2015 in order to establish the limits and heights of flood walls required to provide protection to the downtown area for the design flood event. At a cross section taken immediately adjacent to the proposed 8th Avenue Bridge, the water surface elevation at the design flood event is estimated to be Elevation 730.80. This report states that the design event is a flood with a 0.20% annual probability of occurrence and a freeboard of 4′-1″ is required above this elevation. As such, the proposed top of floodwall at both the east and west sides of the river has been established at Elevation 735.90. The roadway profile at the proposed bridge location utilizes a crest vertical curve over the Cedar River and the bridge structure depth and roadway profile must be designed such that the underside of the bridge clears the floodwall at both abutments and maintains freeboard requirements across the river.

The new 8th Avenue Bridge will provide four 12-foot lanes, 4-foot shoulders, concrete barriers, an 8-foot sidewalk on the north side, and a 14-foot shared use path (SUP) on the south side for a total minimum bridge width of approximately 81-feet.

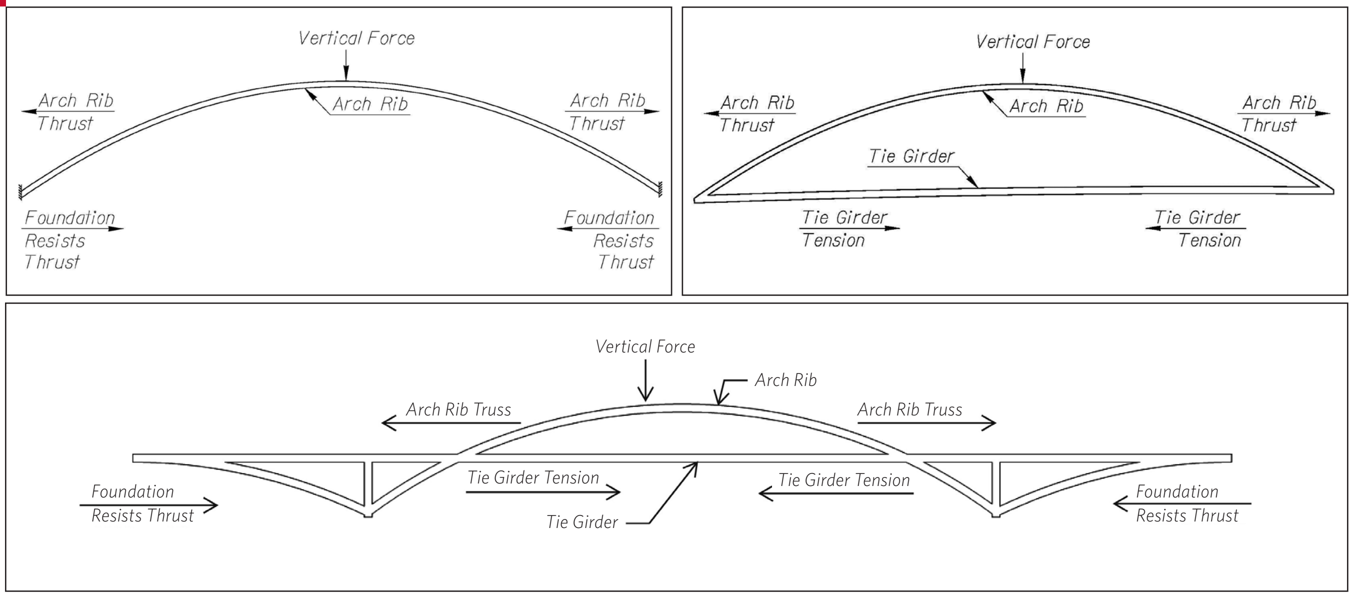


Figure 1: Comparison of true arch (upper left), tied arch span (upper right), and hybrid arch (lower center).

Arch Span Alternative

There are two general categories of arch spans – identified as "true arch" and "tied arch" spans. Both of these structures function by transferring the applied gravity loads, including the self weight of the structure and any live load from vehicles and/or pedestrians, to the arch rib through the hangers. As vertical loading is applied, the tension in the hangers tends to flatten the arch rib, causing the tips of the rib to expand longitudinally, or thrust, until they are restrained in some fashion. In a true arch, the thrust is resisted by a rigid abutment, while in a tied arch, the thrust is resisted by tension in the tie girders that connect the opposite ends of the arch rib. In a part-thru hybrid arch span, the thrust of the arch rib is resisted by a combination of the tie girders and abutment foundations. **Figure 1** illustrates the basic functions of true arch, tied arch and hybrid arch spans.

In a tied arch span, the arch rib thrust is not resisted by the foundation, which provides a significant advantage over a true arch at locations where the following conditions exist:

- the arch span may be supported on tall abutment or wall elements,
- the subsurface material is incapable of resisting this horizontal thrust,
- the arch must be constructed off-alignment or off-site and moved into its final position later.

These advantages match very closely with the constraints and objectives of the 8th Avenue Bridge site, and thus will direct the focus on a tied arch span configuration for purposes of this study.

A specific type of tied arch bridge, called a network tied arch, was originally developed by Norwegian engineer Per Tveit in the 1950s. This bridge type was constructed at multiple locations across Europe, but was relatively unknown elsewhere in the world. However, over the past 10 to 15 years, the network arch bridge has gained widespread acceptance in the US as a reliable and efficient structure and one which has been constructed for spans up to 880 feet long.

The defining characteristic of a network tied arch is that the hangers are inclined as much as 45 degrees from vertical in the plane of the arch and many of the hangers cross at least two others along their length. Functionally, the network hangers cause the arch to act like a very efficient truss, resulting in nearly pure compression forces in the arch rib, thus reducing bending moments and shear forces to very small values. Likewise, the tie member carries nearly pure tension forces. Therefore, the cross-sectional area of the arch rib and tie members can be very slender when compared to more traditional tied arch spans with vertical hangers. In addition to the savings in materials available with this slender cross-section, a network tied arch is highly attractive. This is a very positive attribute, especially in a highly populated area such as downtown Cedar Rapids in which the bridge will be viewed by thousands of people each day.

Arch Span Description

The proposed 8th Avenue Bridge will provide four 12-foot traffic lanes with 4-foot shoulders and concrete barriers. In addition an 8-foot sidewalk will be provided on the north side with a 14-foot wide shared use path (SUP) on the south side. Depending on the arch span configuration chosen, the overall structure width may vary slightly to provide adequate vertical clearance over the traffic lanes and shoulders.

Figure 2: Rendering of Iowa City Park Road Bridge - Iowa City, IA

A significant advantage of the tied arch span is the ability to completely clear span the Cedar River without the need for pier construction in the water. For this location, the proposed flood walls along the east and west riverbank were considered as the preferred location for the arch span abutments. This approach allows the elimination of any piers in the river (or on the "wet side" of the floodwall) and uses the bridge abutments as a functional part of the floodwall system. However, this will require the abutments to be very robust in order to resist lateral earth pressure along with the vertical loads from the arch span.

An alternative to be considered is the use of a shorter arch span supported on piers located in relatively shallow water near the edge of river along with two relatively short, conventional girder spans which pass above the top of the floodwall and connect the conventional abutments to the ends of the arch span. If considered further, additional study of this alternative could occur during the preliminary design phase of the project.

A part-through hybrid arch span could also be considered for this location. As the name implies, this bridge utilizes a pair of tie girders at the deck level along with the abutment foundations to resist the thrust of the arch ribs. Since a portion of the arch is located beneath the deck level, it would slightly reduce the hydraulic opening beneath the bridge, which may be a consideration at the proposed project site. A bridge using this configuration is currently being constructed in Iowa City and when complete, will carry Park Road over the Iowa River near the new Hancher Auditorium. The project is scheduled for completion in 2018. A rendering of this bridge is shown in **Figure 2.**

In evaluating the tied arch alternative, the design team considered a number of feasible options. For purposes of this feasibility study, a detailed comparison of recent arch bridges of similar span length and width was used to assess the most reasonable and cost-effective structure.

Arch Rib Configuration

Most network arch bridges utilize two arch ribs that are installed in a vertical position. This is the generally recognized form of an arch bridge when viewed by the public. Basket handle arch bridges, in which the arch ribs are inclined inward toward each other approximately 10-15 degrees, offer a unique and distinctive appearance. The state of Iowa has a few examples of basket handle arch bridges for visual comparison, including three pedestrian bridges over I-235 in Des Moines and the soon-to-be constructed I-74 twin arch spans over the Mississippi River in the Quad Cities. A basket handle configuration is used to reduce the quantity of lateral bracing material and instead, use each arch rib to partially brace the other against buckling and out of plane forces such as wind. In order to use a basket handle arch configuration, the tie girders are widened and the floor beams are lengthened slightly in order to ensure adequate vertical clearance above the roadway. A slight decrease in the angle of inclination could accomplish the same objective.

The arch rib members themselves can consist of steel box sections, H-shaped sections or even tubular sections. Most arch ribs are constructed of steel box sections, but a growing number of H-shaped ribs have been constructed recently as shown in **Table 1.** The H-shaped ribs are configured with the web of the section horizontal and the two flanges arranged in a vertical alignment. The H-shaped ribs offer the advantage of simple fabrication which is quite similar to that used for conventional curved steel I-girder bridges. In addition, since the section is open on the bottom, the connection of the hanger plates is greatly simplified. However, due to the open configuration of this section, the overall torsional stiffness is greatly reduced, necessitating a larger amount of lateral bracing between the ribs. Further investigation of the economic advantages of the H-shaped arch rib should be included in the preliminary design phase of the project.

Tubular steel arch ribs are very attractive and offer a sleek, modern appearance for the bridge. There are only a few examples of tubular rib arch bridges in the US and they are used for much shorter spans than necessary for the proposed site. The rib members required for a bridge in the 650-foot span range could not be achieved with readily-available pipe sections and would instead be constructed from steel plate sections which would be bent and welded into a tubular shape. Internal diaphragms needed to connect the hangers would be installed throughout the length of the tubular section. In addition, a tubular rib does not allow the use of more typical bolted splice connections in the field. Therefore, consideration of a tubular arch rib has been excluded from further consideration in this study.

It is also possible to construct the arch rib members from reinforced or post-tensioned concrete, but given the length of this span, that material will add considerable weight to the structure and require expensive forming and falsework, making concrete too expensive for further consideration at this stage of the project.

Braced or Unbraced Arch Ribs

In a tied arch bridge, the rib functions as a heavily loaded compression member. As such, the arch rib is capable of carrying vertical loads up until the point where the rib begins to distort out of plane, a condition known as buckling. Until very recently, arch bridges have been designed using some form of lateral bracing between the arch ribs to stiffen the arch ribs in the out-of-plane direction and greatly increase the buckling capacity of the rib. A well-known example of an unbraced arch is located in St. Louis as shown in **Figure 3.**

Figure 3: Gateway Arch - St. Louis, MO

Recent advances in structural analysis software have given designers the ability to numerically evaluate the buckling capacity of an arch span very accurately. In the analysis model, the bridge is loaded with an increasingly heavy uniform load until buckling occurs. The design of the arch span is performed such that the buckling capacity of the arch is several times higher than the greatest load ever expected on the bridge.

In fact, a network arch span is especially well suited to being constructed without lateral bracing because the diagonal hanger network stabilizes the arch rib against buckling and provides a restoring force to resist any out-of-plane movement of the arch rib. In order to eliminate lateral bracing, the ribs would take the form of a trapezoidal box section to provide the necessary stiffness and resistance to buckling.

The fabrication of trapezoidal arch ribs is not especially complicated for shorter bridges because the shape remains constant for the entire length of the member. However, for spans of this length, it is structurally efficient and visually striking to utilize an arch rib that is tapered from end to end – larger at the base of the arch and more slender at the crown. Fabrication of a tapered, trapezoidal arch rib section is possible, but the costs can quickly become quite expensive.

A part-through hybrid arch span does not include lateral bracing for the arch ribs. This is possible because the portion of the arch rib that projects above the top of the arch is much shorter than it would be in a fully-through arch span.

The use of unbraced arch ribs for the 8th Avenue Bridge was briefly considered as part of this study. However, the proposed span length for this location would make this bridge the longest arch span in the US to be constructed without bracing, exceeding the length of the Hastings Bridge in Minnesota by nearly 100 ft.

Steel Tie Girder vs. Post-tensioned Concrete Tie Girder

Historically, tie girder members have consisted of steel box sections made up of four plates bolted together using steel angles at the corners of the box. However, in recent years, the use of a post-tensioned concrete tie girder has been used to provide a corrosion resistant system that is highly redundant to overheight vehicle strikes or damage from floating debris. For purposes of this study, a steel tie girder is shown, but both alternatives could be further considered in preliminary design.

Ideally, the use of a symmetrical cross section is preferred for complex bridges in order to maintain a uniform dead load distribution to the primary load carrying members – whether arches or the extradosed system is used. The 14-foot width of the shared use path would likely require the use of outrigger type supports or a transversely post-tensioned deck – or perhaps both.

The combination of the structural efficiency and aesthetically pleasing appearance of this structure type, has resulted in a number of network tied arch bridges being constructed in the US over the past several years. **Figures 4 and 5** depict two attractive examples of network arch bridges.

Figure 4: Blennerhassett Bridge, WV

Figure 5: Hastings Bridge, MN

Figure 6: Offsite steel erection - Broadway Bridge, AR

Figure 7: Float-in Construction - Hastings Bridge, MN

Lateral Bracing of Arch Ribs

Lateral bracing of the arch ribs can take one of three forms: X-bracing, K-bracing or vierendeel bracing and each system has their advantages. Traditionally, X-bracing and K-bracing members are used and these members are designed to only carry axial loads. These members are relatively lightweight and the choice of bracing style is usually governed by the spacing between the arch ribs, with wider bridges showing a preference toward K-bracing members. For the 8th Avenue Bridge, either style could be acceptable and economical.

Vierendeel bracing, sometimes called ladder bracing, uses a reduced number of bracing members aligned perpendicular to the arch rib. These bracing members typical consist of a relatively stiff structural tube sections which transfer applied loads as axial, moment and shear forces. The reduced number of members is attractive, especially when used in conjunction with basket handle arch ribs. The bracing members simply become shorter near the crown of the arch, but maintain the same visual appearance.

Proposed Arch Cross Sections

Two potential tied arch alternatives are presented in the appendix – one alternative with vertical arch ribs and a system of X-bracing members between the arch ribs, and a second alternative with basket handle arch ribs and vierendeel bracing.

Arch Constructability

Construction of a network arch span is normally performed using either float-in construction or construction in place using falsework.

Float-in construction includes the full offsite assembly of the arch “skeleton” consisting of the structural steel elements including the arch ribs, tie girders, floor beams, stringers and lateral bracing, on falsework at a nearby location **(see Figure 6).** Once the steel skeleton has been floated into place, the deck concrete is cast using either stay-in-place or removable forms **(see Figure 7).** A number of recent tied arch bridges in this span range have been floated into position. These include the following:

- Hastings Bridge – Minnesota (545-foot span)
- Lake Champlain Bridge – New York (402-foot span)
- Broadway Bridge – Little Rock, Arkansas (two 440-foot spans)

However, the 8th Avenue Bridge site has a number of constraints that would likely preclude the use of float-in construction. First, the normal flow depth of the Cedar River may not be sufficient to facilitate float-in construction. Secondly, due to the presence of the CRANDIC railroad bridge just downstream and the 3rd Avenue bridge a few blocks upstream, any float-in construction would need to be performed between these locations. Typically, contractors prefer to construct a bridge downstream of the final location to provide better control when moving the bridge into place against the river current.

Given the disadvantages of a float-in construction, the other alternative for arch span erection is construction in place, which would require the use of either temporary tieback towers at each abutment or falsework towers in the river to support the tie girders and ribs until the arch skeleton is erected. **Figures 8 and 9** show examples of construction of an arch span on falsework. Two or more falsework towers would be used to support each plane of the arch. In this method, erection of the tie girder would be performed in a cantilever fashion from each abutment with falsework towers, or tiebacks towers could be added as shown in **Figure 10.** Due to the non-navigable status of the Cedar River, this method has some advantages because falsework towers will not impede navigation traffic. A number of recent tied arch bridges in this span range have been erected in place. These include the following:

Figure 8: Falsework Construction - Blennerhassett Bridge, WV

Figure 9: Falsework Construction - Whittier Bridge, MA

Figure 10: Tieback Tower Steel Erection - Amelia Earhart Bridge, KS

- Blennerhassett Bridge – West Virginia (878-foot span) - falsework
- Whittier Bridge – New York (420-foot span) – falsework
- Amelia Earhart Bridge – Kansas (525-foot span) – tieback towers

Arch Inspection and Maintenance

Routine maintenance inspection of the tied arch span will likely be conducted using a combination of methods. A manlift operating from the bridge deck could be used to inspect the arch rib, upper lateral bracing and hanger connections and an underdeck "snooper" inspection unit could be used from the side of the bridge with the 8-foot sidewalk to inspect a portion of the floor system of the bridge. The maximum height of the arch span is within the reach of typical manlift equipment.

The 14-foot width of the shared use path, combined with the space required for cables or hangers will make the underdeck inspection of the bridge from the other side of the roadway very difficult. The largest currently available snooper trucks (Aspen A-75) have an overhang reach capacity of 12'-6" so the snooper will need to operate from the shared use path in the current bridge configuration. Snooper trucks are quite heavy and the final design of the bridge should consider the weight of the snooper truck operating from the shared use path. The arch rib and tie girder members are large enough to permit access for internal inspection in the future, but lateral bracing members would likely be closed sections that do not permit internal access.

Another option for inspection of below-deck members is to incorporate features into the design to facilitate inspection access. These could include below deck inspection catwalks, safety cables or grab bars or even the incorporation of a movable inspection platform.

The most common corrosion protection system for steel bridges of this type is a three part paint system which includes a zinc-rich primer along with intermediate and protective urethane or polyurethane top coats. These paint systems have an expected life of 35 to 40 years. Good maintenance practices, such as limiting salting of bridge decks and annual washing of steel members, will help to prolong the paint system.

Weathering steel, has a specific chemistry that creates a rust-preventative patina over the first few years of the bridge's life, does not require painting. However, this material is not recommended for use with the enclosed spaces of the arch alternative because the material must be subjected to cycles of wet and dry environments in order for the steel to form the surface patina.

Alternative coatings which are gaining popularity for major bridges include galvanizing and metallizing. Both systems add a protective zinc coating by either dipping or spraying the steel members followed by a protective top coat that reduces oxidation of the thin zinc layer. Costs for galvanizing or metallizing are up to 50% higher than traditional three-part paint systems, but offer substantially longer coating lives. The protective top coat layer will last 30-35 years before the need for recoating, but the zinc layer can last far longer – perhaps as much as 75 years before a full recoating is required.

The key to programming maintenance painting is to perform recoating operations before extensive repairs are needed due to corrosion loss. Although maintenance painting can be costly, if properly programmed, it is considerably more economical compared with total bridge replacement.

Deck Construction and Service Life – Arch Span Alternative

Deck construction and corrosion protection is one of the most vital parts of a bridge and must be given serious consideration. The potential to remove and replace the original deck in the future, if possible, is a critical aspect of the design. For the tied arch alternative, it would be possible to replace the deck concrete, but it would be advisable to divert traffic and fully close the bridge during the deck replacement.

Although a staged deck replacement of an arch bridge could theoretically be feasible, this approach would be highly unusual for a bridge of this kind. In order for the bridge to perform as intended, the dead loads from the concrete deck, overlay, sidewalk/SUP, barriers and any future wearing surface are anticipated to be applied symmetrically in both the longitudinal and transverse directions. For example, during the original construction of an arch span, the deck concrete is placed starting at the center of the span and working outward in both directions. For short and narrow spans, this concrete placement can be performed in continuous operations, but for larger bridges, this would typically be done in a series of alternating placements that balance the load symmetrically about midspan.

As an illustration of the relative magnitude of the deck and associated loads, for a network tied arch bridge currently under design for another client, the dead load of the deck, sidewalk, overlay and future wearing surface contribute approximately 57% of the total superstructure dead load for the entire span. Therefore, removal of the deck from one half of an arch bridge width at a time for re-decking will dramatically unbalance the overall load of the bridge. In this condition, the deck dead load is being shifted to one side of the structure, along with the vehicular and pedestrian live loads. This arrangement has the potential to overload hangers on one side, while creating nearly "slack" conditions on the other side of the same span. In addition, the substructure and foundations of the bridge will be subjected to much higher loads than they would normally be expected to support due to the eccentricity effects. In the case of an arch span, the eccentric loading may also create a unique buckling condition for the arch ribs on opposing sides of the bridge. Although the structure can theoretically be designed for each of these loading conditions, the cost of additional materials and analysis to resist these eccentric loads will be very costly.

Furthermore, replacement of the concrete deck for an arch bridge would not be anticipated for 50 or more years after the bridge is opened to traffic. For signature structures like the 8th Avenue Bridge, bridge owners typically specify some form of a bridge deck overlay as part of the original construction. The Iowa DOT has traditionally used a low slump concrete or latex modified concrete overlay on their decks for major river crossings. These systems provide a degree of impermeability and allow the overlay to be milled off and replaced in the future so that the structural bridge deck can have a longer service life. Recently the use of Polyester Polymer Concrete overlays has seen a growing acceptance as a means to extend the service life of bridges up to 75 or even 100 years.

For purposes of this initial screening of arch alternatives, we would recommend considering deck replacement using a full closure only or avoid deck replacement by extending the deck's service life using the strategies discussed above.

Opinion of Construction Cost – Arch Alternative

At this stage of project development, construction cost estimates are typically developed on a square foot basis using comparisons to bridges of similar type and span length. As such, no effort has been made as part of this study to estimate material quantities or associated unit prices. In addition, these estimated costs are based on 2018 construction dollars and do not include future inflation or variations in material and labor costs.

There are a number of comparable network tied arch spans that have been constructed over the past several years from which to draw a cost comparison. The cost data for these alternatives is incomplete and much of the data was drawn from published reports of the complete bridge cost. For purposes of this study, we have attempted to isolate the arch span cost from the remainder of the project cost.

Using these similar bridges as a point of reference, we estimate a unit cost of $750 to $800 per square foot for the tied arch span. Using an overall deck area of 56,930 square feet, we estimate the overall construction cost of the arch span is estimated to range from approximately $43M – $46M. This opinion of construction cost is based on historical unit costs ($/sf) of similar tied arch bridges escalated using 3% inflation to an assumed construction year of 2018 and an allowance of 20% for contingency of unknown items of work.

A summary of recent network tied arch spans, sorted by span length, and their associated construction costs is shown in **Table 1** on page 08. Note that the arch span width is measured from center-center of tie girder unless the bridge has a cantilever sidewalk. In that case, the out-out width includes the sidewalk.

The estimated construction cost of the proposed bridge could potentially be reduced through a number of measures:

- Reduce width of shoulders
- Reduce width of shared use path or balance width to each side of span
- Reduce arch span length with conventional approach spans on one or both end with piers located near the river edge

Table 1: Recent Tied Arch Bridges in the United States

Bridge Name & Location	Arch Span Dimensions Length (ft)	Arch Span Dimensions *Width (ft)	Arch Span Type	Bid Year	Arch Span Unit Cost ($/sf)	Arch Span Unit Cost (2018 dollars) ($/sf)
Blennerhassett (WV)	878	107	Braced, vertical box ribs	2005	575	863
Kentucky Lake (KY)	550	94	Basket handle, H-shaped ribs	2014	690	773
Lake Barkley (KY)	550	94	Basket handle, H-shaped ribs	2015	685	754
Hastings (MN)	545	104	Unbraced, trapezoidal ribs	2010	590	743
Earhart (KS)	525	78	Braced vertical box ribs	2009	540	702
Lowry Avenue (MN)	450	91+ (varies)	Basket handle box ribs	2007	700	980
Broadway (AR)	440	88	Braced, vertical H ribs	2014	505	566
Lake Champlain (NY)	396	43	Basket handle, H-shaped ribs	2010	710	895
Sauvie (OR)	360	66	Braced vertical ribs, sunburst hangers	2006	485	703

*Denotes out-out width including cantilever sidewalk(s)

Arch Span Hybrid Alternatives

In order to reduce the construction cost of the arch span alternative and provide a more direct comparison to the extradosed span alternative, the study team also considered a three-span alternative for the tied arch structure type which includes two conventional approach spans with piers located near the edge of the river and a shorter tied arch. A sketch of a feasible alternative is presented in the appendix. For purposes of this study, we have assumed a tied arch span of 400-feet with two approach spans of 125-feet in length.

In this alternative, the approach spans would likely consist of standard, concrete bulb-tee beams. For this span length, an Iowa BTD section would be adequate, with a beam depth of 4’-6”. It would be reasonable to match the depth of the arch span tie girder with the depth of the approach span beams to provide a uniform and aesthetic appearance of the overall bridge.

An opinion of construction cost for this arch alternative would be much closer to that presented for the extradosed alternative. A reasonable unit cost for a bulb tee span in this range is approximately $150 per square foot. In addition, the unit cost of a shorter tied arch span would be reduced to approximately $650 per square foot including a 20 percent contingency. Assuming the same bridge deck area as previously used, the weighted unit price for the total bridge would be on the order of $475 per square foot. In this case, the estimated cost of the bridge would be $27M-29M.

Another hybrid arch alternative is a system similar to what is currently being constructed in Iowa City for Park Road over the Iowa River. This 3-span continuous concrete hybrid arch bridge utilizes concrete arch ribs and tie girders. In the center span, a through arch is utilized and the end approach spans utilize deck arch spans. The Iowa City Park Road Bridge was bid in April 2016 at a cost of approximately $302 per square foot. Although there are few recent comparable bridges of thus type to compare unit costs, utilizing the $302 per square foot would yield an estimated bridge cost of approximately $17.2M.

Table 2: Extradosed Bridges in the United States

Project	Main Span (ft)	Cross-Section	Bid Year	Status
Pearl Harbor Memorial Bridge, CT	515	Multi-Cell CIP Concrete Box	2009	Open to Traffic
Brazos River Bridge, TX	250	Steel Box Edge Girders	2011	Open to Traffic
St. Croix River Bridge, MN	600	Multi-Cell Precast Concrete Box	2013	Under Construction (Fall 2017 Completion)

Figure 11: Pearl Harbor Memorial Bridge, CT

Figure 12: Brazos River Bridge, Waco, TX

Figure 13: Driver's Perspective of St. Croix Bridge, MN

Cable Supported Extradosed Span Alternative

Extradosed bridges are relatively new types of bridges in the US that provide a cross between prestressed girder bridges and cable-stayed bridges. Generally speaking, the extradosed bridge has the appearance of a cable-stayed bridge with shorter towers; however, the bridge behaves structurally closer to a prestressed girder bridge with external prestressing. The cables from the shorter towers in an extradosed bridge intersect with the girder at a lower angle. For this reason, in an extradosed bridge, tension forces in the cables act more to compress the bridge girder longitudinally, rather than support it vertically; thus, the cables act as prestressing cables for a concrete girder. Even though the cables act more as prestressing members than stay cables, extradosed bridges with concrete and steel superstructures have been built in the United States. **Table 2** lists the previous extradosed bridges in the US.

This unique layout allows extradosed bridges to utilize a thinner profile than a girder bridge of a comparable span, but thicker than that of a conventional cable-stayed bridge. Extradosed bridges provide an economical means for spans in the 300 to 800 ft. range while also offering new aesthetic opportunities relative to cantilever constructed girder bridges and cable-stayed bridges. An extradosed structure has the ability to provide longer spans at a constant depth since it behaves structurally much like an externally prestressed girder bridge, combining the aesthetic appeal of a cable-stayed design with the stability and performance of a traditional girder bridge.

Photos of these extradosed bridge examples are presented in **Figures 11-13.**

The process of developing extradosed concepts for the 8th Avenue Bridge consisted of evaluating structural schemes, span layouts and cross-sections that have successfully been used on other projects and that are applicable to this study. Even though extradosed bridges are relatively new, the construction technology and techniques used to build them have a successful track record with cable-stayed bridge construction in the United States.

Some of the characteristics of extradosed bridges include:

- Shorter tower than cable-stayed bridge
- Shallower girder than a girder bridge, but deeper than a cable-stayed bridge
- Flatter cable angles than a cable-stayed bridge, and only over a portion of the span
- Cables sized to prestress the deck
- Low fatigue ranges for cables
- Uniform size range for cables

For the 8th Avenue Bridge, the relative comparison used among the extradosed layout alternatives considered in this study was generally based on the following considerations:

- Geometric compliance with the floodwalls, geometric compatibility at the tie-ins, location of existing piers and freeboard.
- Ability to function in the expected bridge configuration and ability to resist expected structural demands.
- Constructability and construction cost.
- Develop concepts that do not preclude concrete or steel from further consideration.
- Avoid conflicts with existing 8th Avenue Bridge foundations.

Structure Description

As previously noted, the new 8th Avenue Bridge will provide four 12-foot lanes, 4-foot shoulders, concrete barriers, an 8-foot sidewalk on the north side of the bridge and a 14-foot SUP on the south side for a minimum bridge width of approximately 81-feet. To accommodate this width, it was important to evaluate structural schemes that are not only considered feasible but also offer advantages in terms of least risk and probable lowest cost.

The proposed extradosed bridge is a 3-span continuous unit supported by abutments at or near the floodwalls and two piers in the river. Each river pier consists of two pylons (one on each side of the roadway) used for anchoring the cables and a transverse beam between the pylons to support the superstructure. The superstructure can be supported on high load multi-rotational bearing at the pylons and abutments or built integrally at the pylons. Bearings are sometimes used to avoid additional structural demands from creep, shrinkage and thermal loading; however, the decision to support the superstructure on bearings or build it monolithically with the pylons should be studied further during preliminary design as it could have a significant impact on the construction schedule and construction cost.

As noted in **Table 2** on page 08, both concrete and steel superstructures have successfully been used on cable-supported bridge with similar proportions to the 8th Avenue Bridge so it is recommended that both materials be advanced to the preliminary design phase. Also as previously noted, a number of strategies are commonly being used for both steel and concrete superstructures to extend the service life of today's bridges. These include the use of high performance coatings on steel members; use of high performance steel, concrete and bridge overlays; and the use of stainless steel reinforcing steel.

Span Layout

The extradosed option consists of a 3-span continuous unit with a span arrangement of 180 ft.-290 ft.-180 ft. as illustrated in the appendix. This span arrangement has the following benefits:

- allows for a continuous and balanced structure with no uplift at the abutments thereby avoiding additional maintenance issues with complex tie down systems
- abutments are located at or near the floodwall thus reducing the overall bridge length required
- shorter spans reduce the required overall structural depth
- minimizes the number of piers in the river / avoids conflicts with existing bridge's foundations

Vertical concrete pylons and cable stays act in two parallel single vertical planes. The corresponding pylon height for this span layout is anticipated to be 40 to 50 ft. above deck level.

Development of Cross-Section

The development of cross-sections for cable supported bridges is driven by several factors including:

- total bridge width
- span length
- construction material
- stay cable arrangement

Historically for either concrete or steel cable-supported superstructures, the deck has been suspended by a central plane of stays along the center median or by multiple planes of stays supporting the deck at or near the edges of the superstructure. If the superstructure is supported by a central plane of cables, a box girder type cross-section is likely required. As the bridge width increases, the cross-section needs to be stiff enough to transfer the majority of the dead and live load transversely to the centrally located supports. The trapezoidal box girder is a closed section that is torsionally stiff and capable of efficiently resisting the torsional loads applied by wind and live load on the bridge.

In the United States, the widest central plane, cast-in-place bridge deck is approximately 57.5 ft. while the widest central plane precast bridge deck is approximately 90 ft. in width. As noted above, both of these bridges required a box girder superstructure.

The 8th Avenue Bridge requires a minimum width of 81 ft. to accommodate the four travel lanes, and the desired sidewalk and SUP; however, this width does not include any allowance in width for the proper anchorage of the cables along the deck so it is important to investigate if the required bridge width is conducive to a central and/or dual plane configuration.

When making this assessment, it is important to consider the following:

1. Precast superstructure segments required for a central plane of stays are economically feasible when there is a high volume of precasting. The 8th Avenue Bridge does not have enough quantity to economically utilize precast segmental construction.

2. Cast-in-place cantilever construction with form travelers offers the benefit of reducing construction activities on the river and the ability to cast-in-place a variety of cross-sections. Several cable stayed bridges in the United States have been cast-in-place with longitudinal edge girders and transverse beams. This structural configuration also lends itself to structural steel.

3. The widest centrally supported cast-in-place bridge deck is approximately 57.5 ft. The 8th Avenue Bridge would be much wider.

In consideration of the above and as illustrated in the appendix, the cross-section developed for the extradosed option consists of two edge girders each supported by a plane of cable stays with a shallow transverse crossbeam hidden within the depth of the deck section to provide transverse frame action.

This cross-section does not preclude steel or concrete and leverages construction technologies that have been used for numerous cable supported structures in the United States. The edge girder depth near the pylons is expected to be in the range of 10 to 12 ft. and at midspan in the range of 7 to 9 ft. The sidewalk and SUP are supported by cantilever wings outboard of the planes of stays.

Constructability

While extradosed bridges are relatively new to the United States, the construction techniques used to build them have a proven track record on other cable-supported structures.

Foundation and Substructure Construction

The foundations for the extradosed bridge can be built utilizing conventional methods whether waterline or submerged footings are used. During preliminary and final design, the type and size of the foundations would be checked for not only the permanent but also for the temporary loads generated during construction. The temporary loads are driven by the means and methods assumed for construction during design. The portions of the pylons below the superstructure including the transverse struts and pier tables will be supported off of the footings by shoring towers and falsework.

Pylon Construction (above superstructure)

The main span pylons can be constructed using either conventionalformwork, as shown in **Figure 14** for the Brazos River Bridge, or using a climbing formwork system that allows a repeatable cycle, similar to the construction cycle used on the Pearl Harbor Memorial Bridge and shown in **Figure 15.**

Figure 14: Brazos River Bridge Pylon Construction

Figure 15: Pearl Harbor Memorial Bridge Pylon Construction

Figure 16: Pearl Harbor Memorial Bridge Superstructure Construction

Superstructure

Upon completion of the tower and abutments, the superstructure construction can begin. The assumed means and methods will depend on the superstructure material selected during preliminary design. For the cast-in-place cantilever with form travelers alternative, it is likely that two form travelers will be erected at each pier table to facilitate balanced cantilever construction of the extradosed segments as shown in **Figure 16.**

With this technique, the bridge would be cast-in-place in balanced cantilever fashion working from the pylon towards the abutment and mid-span simultaneously. Once cantilever construction from one pier is complete, the form travelers can be lowered and reused for cantilever construction from the other pier. The superstructure construction cycle would be very similar to the cycle used to construct the Pearl Harbor Memorial Bridge.

Once both cantilevers are complete, the superstructure section adjacent to the abutments will likely be constructed on falsework and will be completed prior to the final closure pours. The final closure pours between cantilevers will be completed using strongbacks to maintain geometry between the opposing cantilevered bridge decks.

For the steel alternative, it is likely that the contractor will use a temporary trestle, barge mounted cranes or a combination of both to set the structural steel box girders prior to deck and cable placement. On the Brazos River Bridge, the contractor used barge mounted cranes to erect the structural steel, refer to **Figures 17 and 18** below.

The exact construction sequence for a steel alternate, including assumed construction loads, will need to be studied during the preliminary and final design phases of the project.

Extradosed Inspection and Maintenance

Routine maintenance inspection of extradosed bridges would likely be conducted using a combination of conventional equipment such as manlift operating from the bridge deck to inspect the pylons and cables and an underdeck "snooper" inspection unit to inspect the underdeck of the bridge. The maximum height of the pylons is within the reach of typical manlift equipment.

Similar to the arch option, the 14-foot width of the shared use path, combined with the space required for cables will make the underdeck inspection of the bridge from the roadway very difficult. Although generally available only from east coast suppliers, the largest currently available snooper trucks (Aspen A-75) have an overhang reach capacity of 12'-6" so the snooper would need to operate from the shared use path in the current bridge configuration. Snooper trucks are quite heavy and the final design of the bridge should consider the weight of the snooper truck operating from the shared use path. Alternatively to the snooper truck, the underside of the bridge can also be inspected utilizing a barge mounted manlift.

Figure 17: Construction of the Brazos River Bridge Foundations with Barge Mounted Cranes

Figure 18: Construction of the Brazos River Bridge Superstructure with Barge Mounted Cranes

Deck Construction and Service Life – Extradosed Alternative

Compared to an arch bridge, extradosed bridges pose a different challenge when considering full re-decking or re-decking in stages. Similar to cable-stayed bridges, the deck of an extradosed bridge is an integral part of the structural system and is highly compressed through the combined action of the horizontal component of the stay cable forces and the extensive use of longitudinal post-tensioned in the deck itself. As the bridge is built and cables are stressed, longitudinal compression is added to the deck; thus this longitudinal compressive force is built into the deck. In addition to longitudinal post-tensioning, the deck will also have transverse post-tensioning to decrease the slab thickness and to increase its durability. If portions of the deck were to be removed, it would be necessary to slacken the necessary stay cables, longitudinal and transverse post-tensioning, remove the portion of deck, replace the portion of deck and re-stress the stay cables and post-tensioning. For this reason, it would be extremely challenging and costly to fully replace the deck on a cable-supported extradosed bridge. As noted previously for the arch bridge type, even if a feasible scheme could be developed for deck replacement, replacing the deck of an extradosed bridge during partial width closures of the bridge deck will place highly eccentric loads on the tower and bridge foundations and could result in a slack condition in the stay cables on the other side of the bridge.

In lieu of deck replacement, bridge owners have opted to utilize construction methods to provide extended life to the bridge and deck. Post-tensioning places the deck in a permanent compressive state to close cracks and provide protection from intrusion of roadway salts and chlorides. Also, the use of high performance materials such as concrete, overlays and stainless steel reinforcing steel have been used to extend the deck service life and avoid deck replacements.

For purposes of this initial screening of alternatives, we recommend excluding deck replacement as a consideration for an extradosed alternative. Instead, emphasis should be placed on extending the bridge service life using the strategies discussed above.

Opinion of Construction Cost – Extradosed Alternative

Evaluation of bridge construction costs is vital to the project success in assuring the funding and construction of the project. Since limited structural design has been performed at this stage, this opinion of construction cost estimate for the bridge is based on historical unit costs ($/sf) of similar extradosed bridge projects escalated using 3% inflation to an assumed construction year of 2018. An allowance of 20% for contingency of unknown items of work has also been included.

Due to the preliminary nature of the study, the comparison between the extradosed and tied arch alternatives should be considered relative rather than absolute and is intended to screen structure types and select that structure that will be studied further in the preliminary and final design phases of the project. As previously noted and as illustrated in **Table 3** below, there are three (3) vehicular extradosed bridges in the United States. The Pearl Harbor Memorial Bridge in Connecticut and the St. Croix River Bridge in Minnesota consist of wide multi-cell concrete boxes built in cantilever each with dual planes of stays anchored along the outside edge of the superstructure. The Brazos River Bridge in Texas consists of a narrower deck supported by two steel box edge girders with dual plane of stays also anchored along the outside edges of the superstructure.

Both the Pearl Harbor Memorial Bridge and St. Croix Bridge have much longer spans so it would be reasonable to expect the unit cost of the 8th Avenue Bridge to be between the unit cost of the Brazos River Bridge (250 ft. main span) and the Pearl Harbor Memorial Bridge (515 ft. main span). For purposes of this study, a range of unit costs of $450-$500/sf (including 20% contingency) is recommended. Based on a deck area of 56,930 square feet, the corresponding estimate of construction cost would range from $26M-$28M.

Table 3: Extradosed Bridge Costs in the United States

Project	Side Span (ft)	Main Span (ft)	Cross-Section	Bid Year	Unit Costs ($/sf)	Unit Costs Excalated to 2018 ($/sf)
Pearl Harbor Memorial Bridge, CT	249	515	Multi-Cell CIP Concrete Box	2009	$550	$718
Brazos River Bridge, TX	185	250	Steel Box Edge Girders	2011	$240	$295
St. Croix River Bridge, MN	340	600	Multi-Cell Precast Concrete Box	2013	$621	$720

Table 4: Qualitative Comparison of Study Alternatives

Characteristics	Extradosed	Arch	Arch Hybrid
Speed of construction	🟡	🟢	🟢
Unique visual appearance	🟢	🟡	🟡
Offsite construction and float-in	🔴	🟢	🟢
Constructability	🟢	🟡	🟡
Potential service life	🟢	🟢	🟢
Estimated construction cost	🟢	🟡	🟢
Inspection and Maintenance	🟢	🟡	🟡
Estimated Cost (incl. 20% contingency)	$26M - $28M	$43M - $46M	$27M - $29M

Comparison Of Alternatives

As a means of comparing the arch and extradosed bridge alternatives, the **Table 4** above presents the relative advantages and disadvantages of the two bridge alternatives that were studied. This comparison represents the qualitative evaluation based on review of recent example bridges. For this comparison, the relative merits of the alternatives indicated by colored circles with green being the most preferred, yellow being less preferred and red being the least preferred.

Conclusions

Both the arch and extradosed bridge alternatives offer an iconic appearance for the downtown Cedar Rapids area. Although network tied arch spans are not universally common in the US, these bridges are becoming more widely used in recent years for spans in this range. Since a 650-foot arch spanning the entire river may be a somewhat visually overwhelming for the location, other arch alternatives that utilize either deck arch or prestressed concrete beam approach spans to shorten the main arch span would help balance the visual impact of the large span and significantly reduce the construction cost.

The extradosed bridge is quite unique and at present there are only three such bridges that have been constructed in the US. The unique appearance provides the opportunity to really make a signature statement for downtown Cedar Rapids. The extradosed bridge alternative also provides an advantage from a cost standpoint and, once construction of river foundations is completed, has the advantage of minimizing impacts in the river due to its cantilever construction methodology. For these reasons, the extradosed bridge type is recommended for further consideration.

Further Recommendations

During this validation study, a number of potential considerations have been identified that should be more thoroughly investigated during preliminary and final design. These recommendations include the following:

- The recommended concepts included in this study are for the purpose of this feasibility report and for feasibility-level cost estimation. It is expected that upon preliminary and final design, these suggested methods will be refined and modified to reflect the final design requirements.
- Further consideration of a shorter tied arch span utilizing approach spans of conventional girders on each side of the river or part thru deck arch spans could make the tied arch alternative more economical.
- For the extradosed alternative, further study of monolithic connections vs. bearings at the towers is warranted.
- For the extradosed superstructure, a more detailed investigation of the merits of a steel vs. cast-in-place concrete superstructure is warranted.
- If a tied arch alternative is advanced, systems using bolted steel vs. post-tensioned concrete for the arch tie members should be studied.
- Development of an assumed construction sequence would provide added validation to the construction cost estimate.
- Prepare a quantities based cost estimate and detailed construction schedule.
- Consider a project-specific corrosion protection plan that provides guidance for the selection of materials and coatings.

- Consider high-performance concrete with low permeability characteristics to resist chloride intrusion.
- For the extradosed alternative, consider a deck post-tensioned in two directions to limit crack widths.
- For the extradosed superstructure and the arch tie element, consider high-strength concrete (except for mass concrete sections) to provide greater durability and longer life.
- For both alternatives, consider the use of a low-slump concrete or latex modified concrete overlay over the structural deck to provide added protection to the deck from the intrusion of chloride ions and to provide a sacrificial surface that can be milled and replaced without having to remove the structural deck.
- Consider the use of stainless steel reinforcing steel in the bridge deck for extended service life.
- A reduce shared use path width would provide a reduction in construction costs and easier access for inspection and maintenance.
- Consider replacing the 14-foot shared use path with an 8-foot sidewalk and provide 6-foot bicycle lanes in lieu of 4-foot shoulders adjacent to the outside lanes of traffic to improve inspection access. Using City standard 6-foot bicycle lanes and a 2-foot center painted median would maintain the same overall bridge width and provide flexibility for traffic lane impacts during bridge maintenance activities. (See alternate bridge cross section showing bicycle lanes in **Figure 19.)**

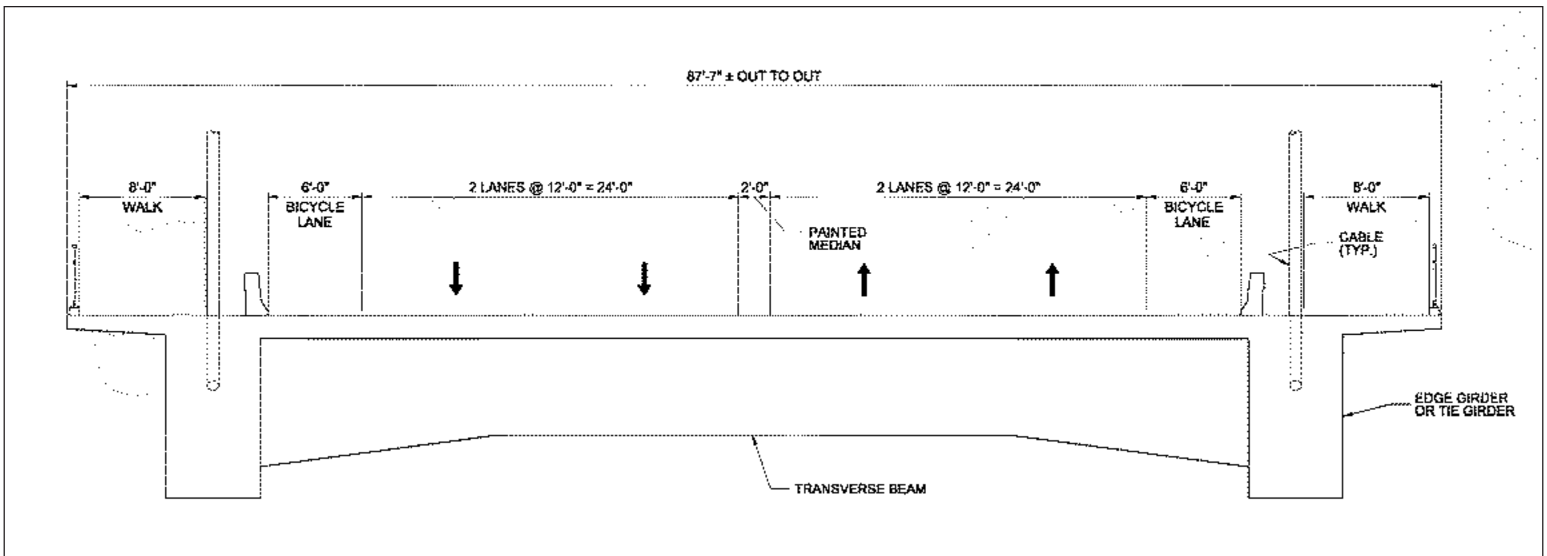
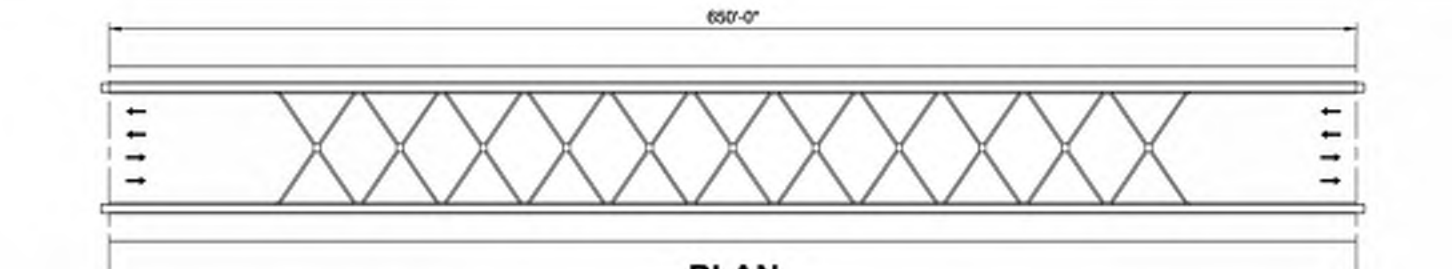


Figure 19: Alternate Cross Section Showing Bicycle Lanes

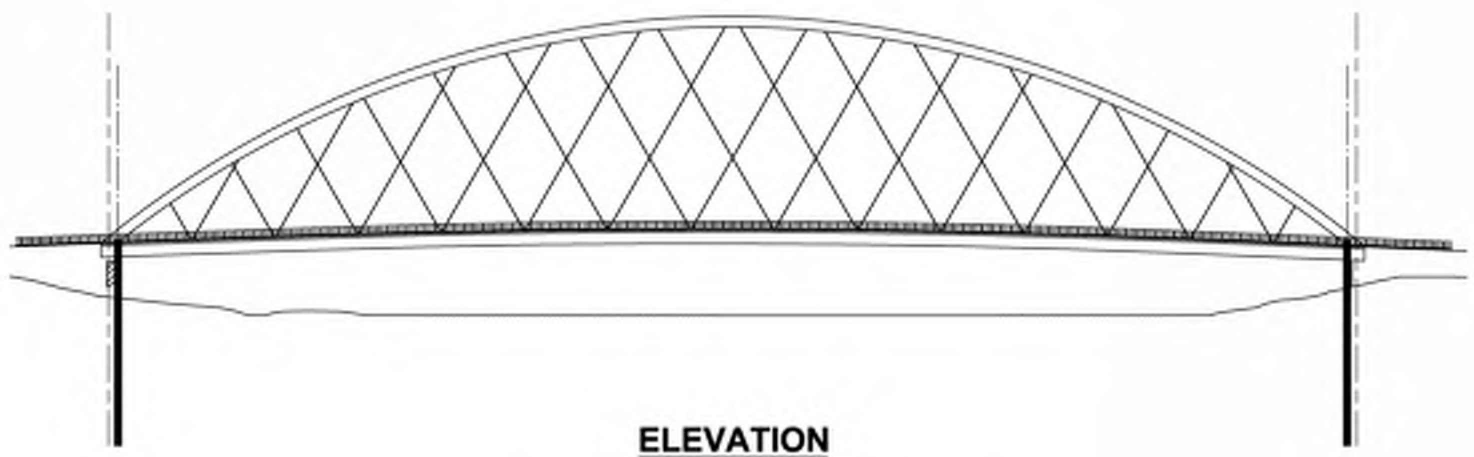
Appendix

Bridge Layouts

- Arch Span Alternative – Plan View, Elevation View and Typical Section - 650-foot Span
    - Vertical Arch Rib Option
    - Basket Handle Rib Option
- Arch Span Alternative – Plan View, Elevation View and Typical Section – 400-foot Vertical Rib Arch Span with Single Span Prestressed Concrete Beam Approaches
- Extradosed Alternative – Plan View, Elevation View and Typical Section



PLAN



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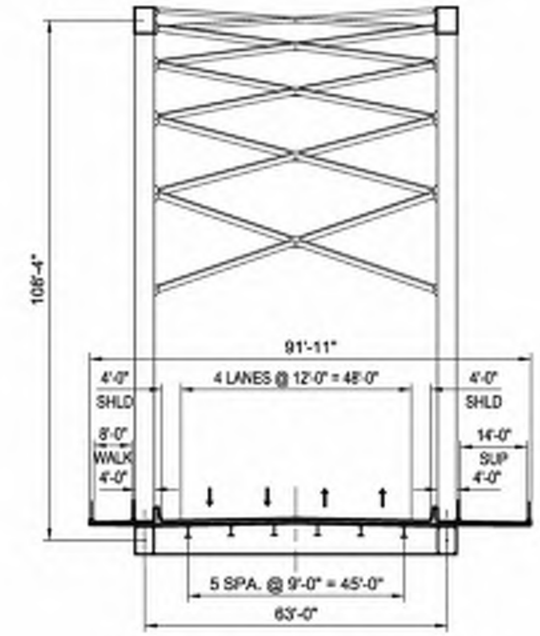

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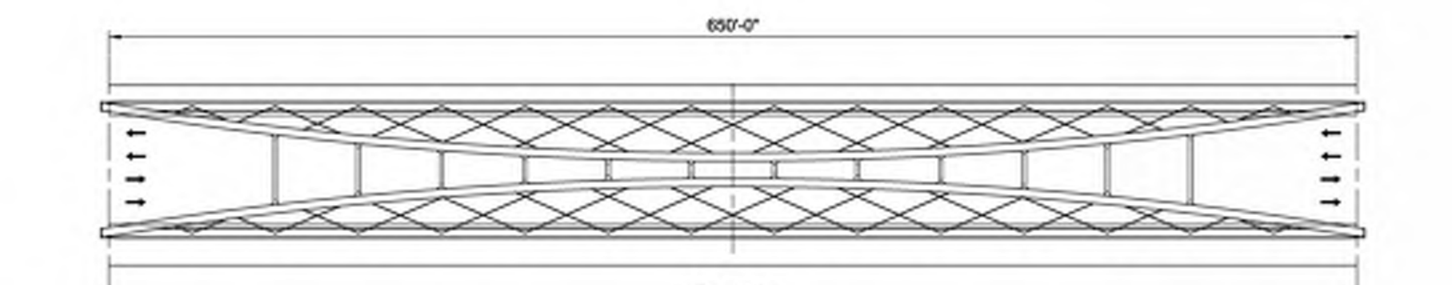

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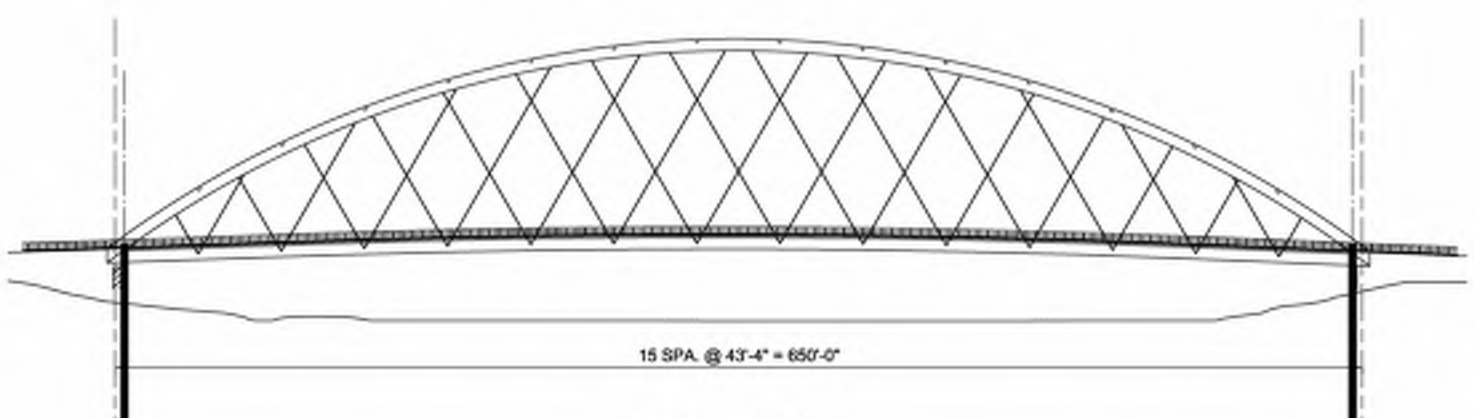

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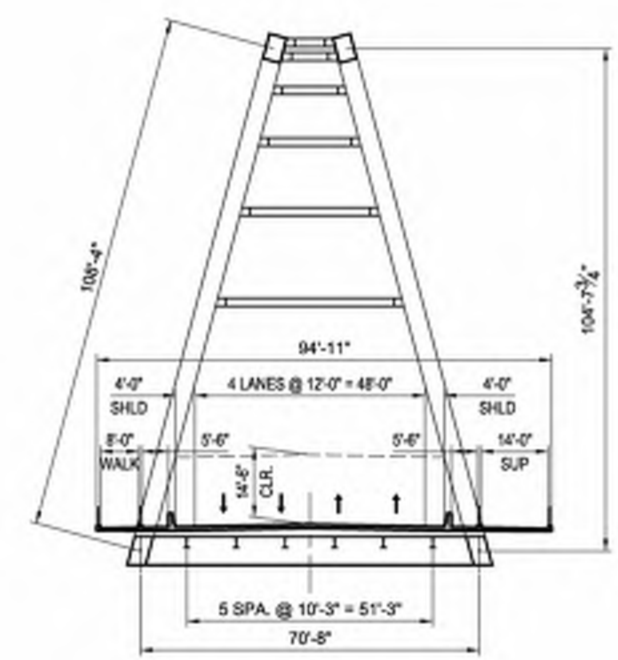

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APPROVED BY: ML	
DATE: 2017-04-13	
SCALE: NTS	

8TH AVENUE BRIDGE OVER THE CEDAR RIVER
BRIDGE VALIDATION STUDY
TIED ARCH BRIDGE ALTERNATIVE: 650 FOOT ARCH SPAN
GENERAL PLAN AND ELEVATION

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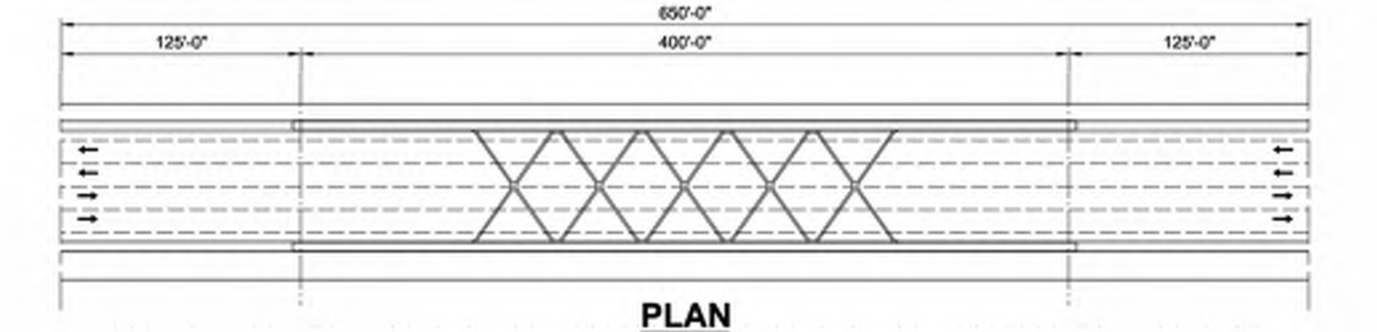

PLAN

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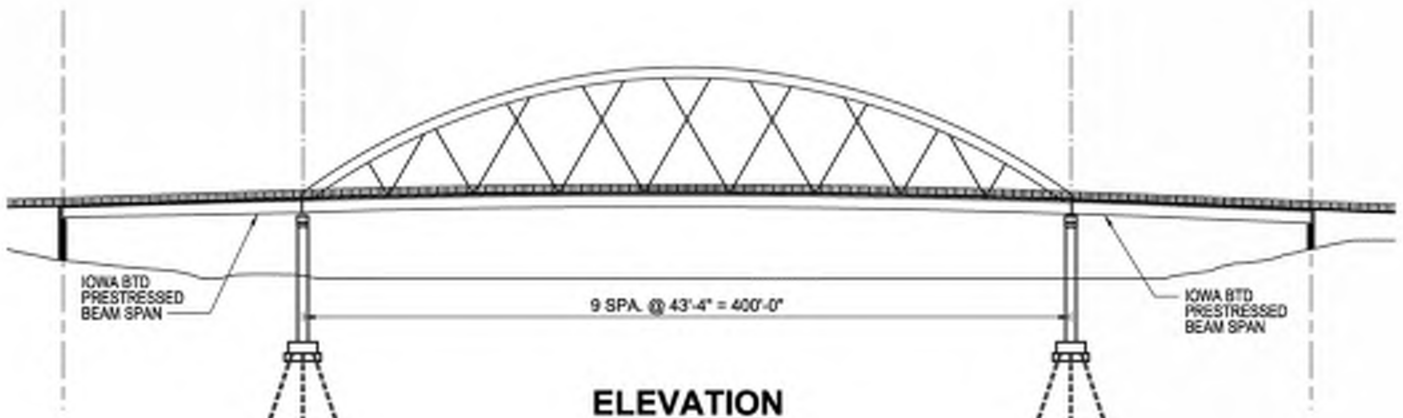

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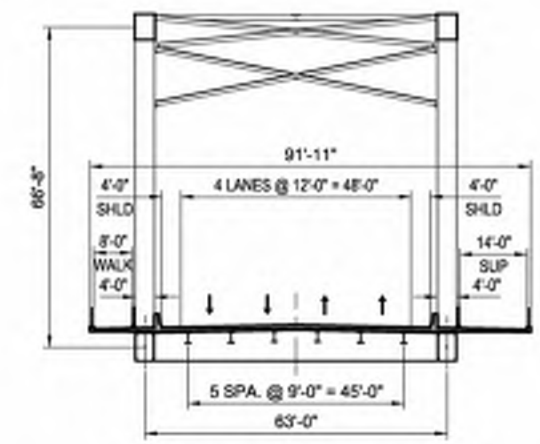

TRANSVERSE SECTION

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APPROVED BY: ML	
DATE: 2017-04-17	
SCALE: NTS	

8TH AVENUE BRIDGE OVER THE CEDAR RIVER
BRIDGE VALIDATION STUDY
TIED ARCH BRIDGE ALTERNATIVE: 400 FOOT ARCH SPAN
GENERAL PLAN AND ELEVATION

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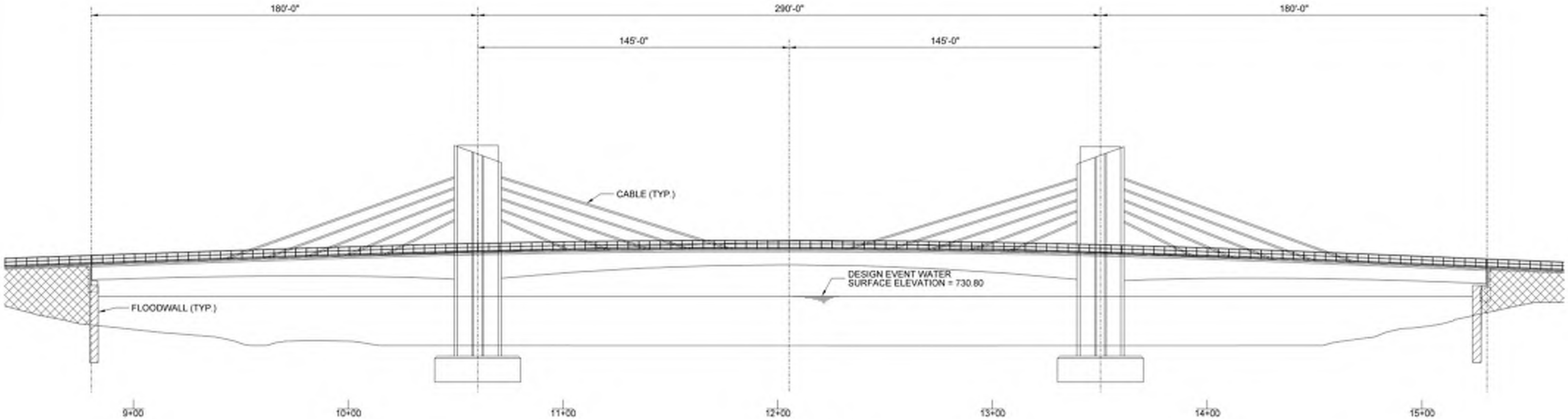
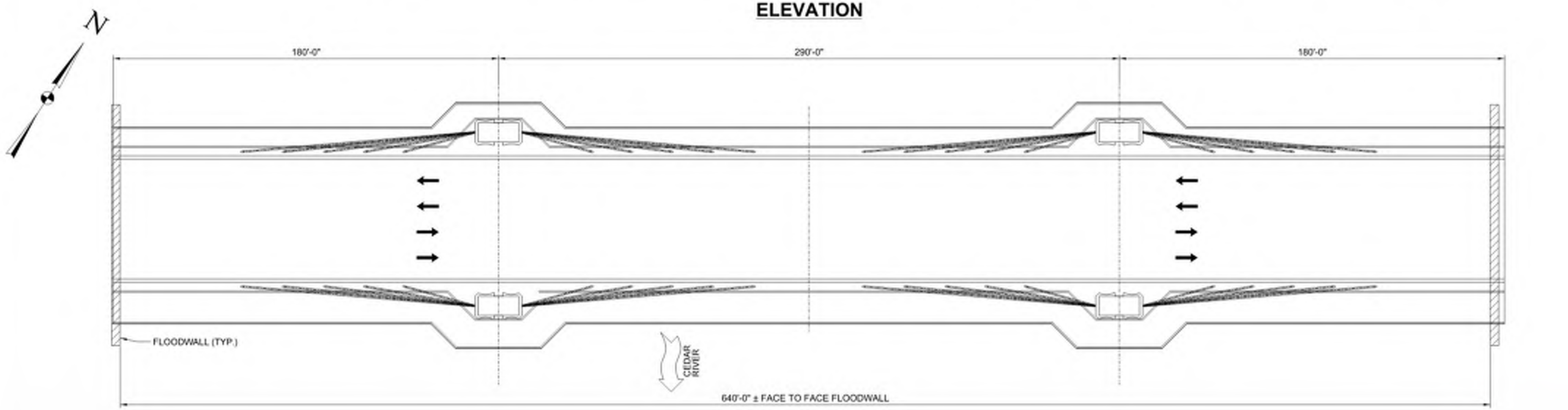


ELEVATION



PLAN

DRAWING NO.: EXTRADOSED GPE
DRAWN BY: JBS
APPROVED BY: MFC
DATE: 2017-04-13
SCALE: NTS

8TH AVENUE BRIDGE OVER THE CEDAR RIVER
BRIDGE VALIDATION STUDY
EXTRADOSED BRIDGE ALTERNATIVE
GENERAL PLAN AND ELEVATION

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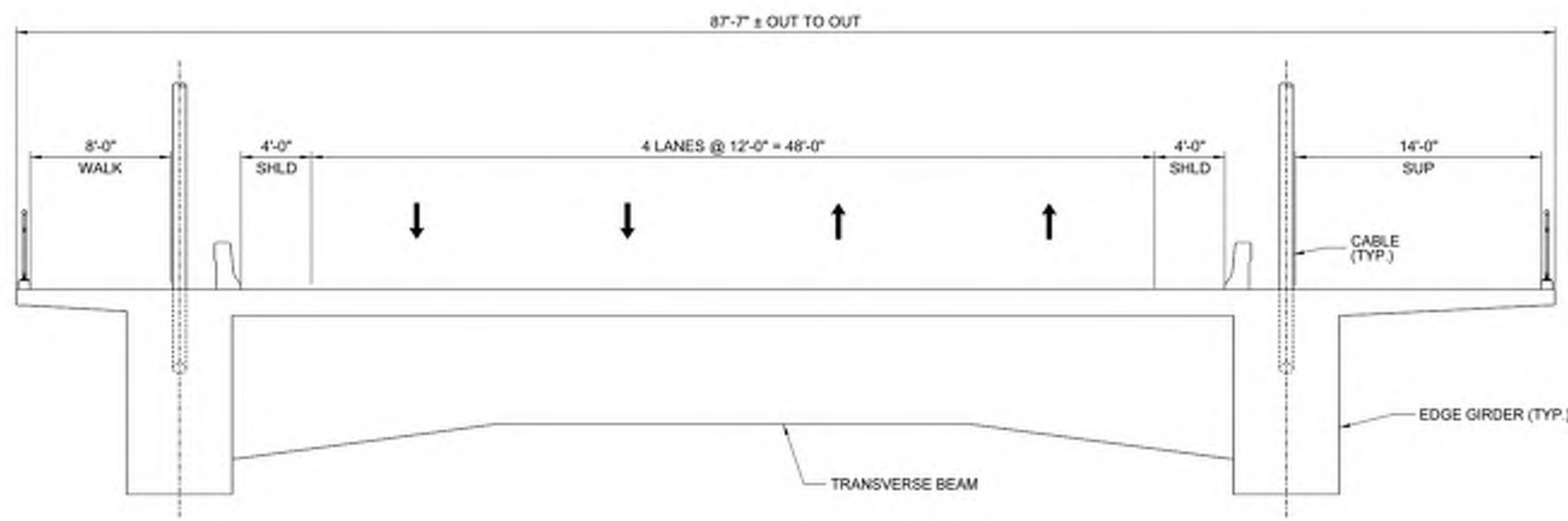

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8TH AVENUE BRIDGE OVER THE CEDAR RIVER
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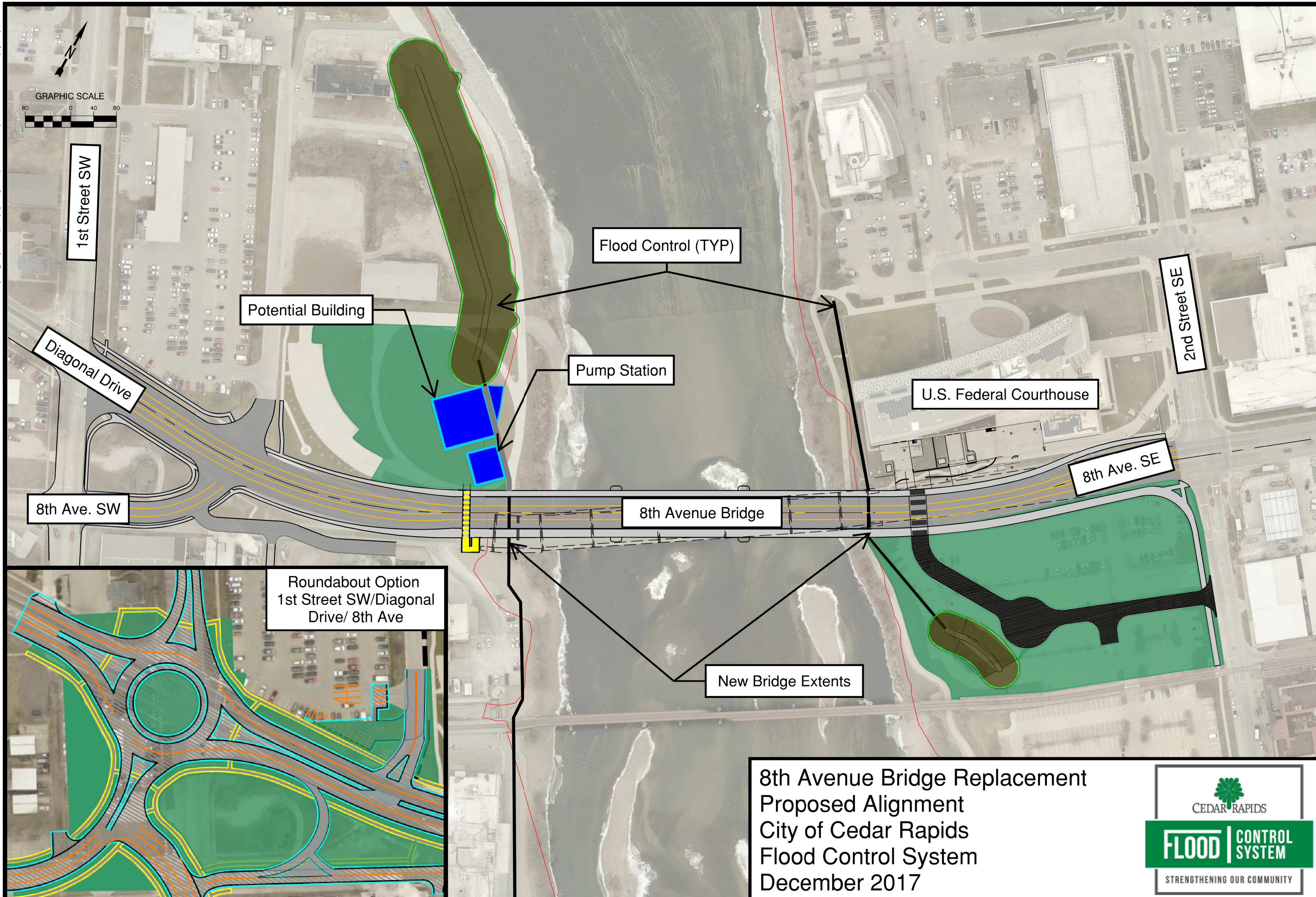
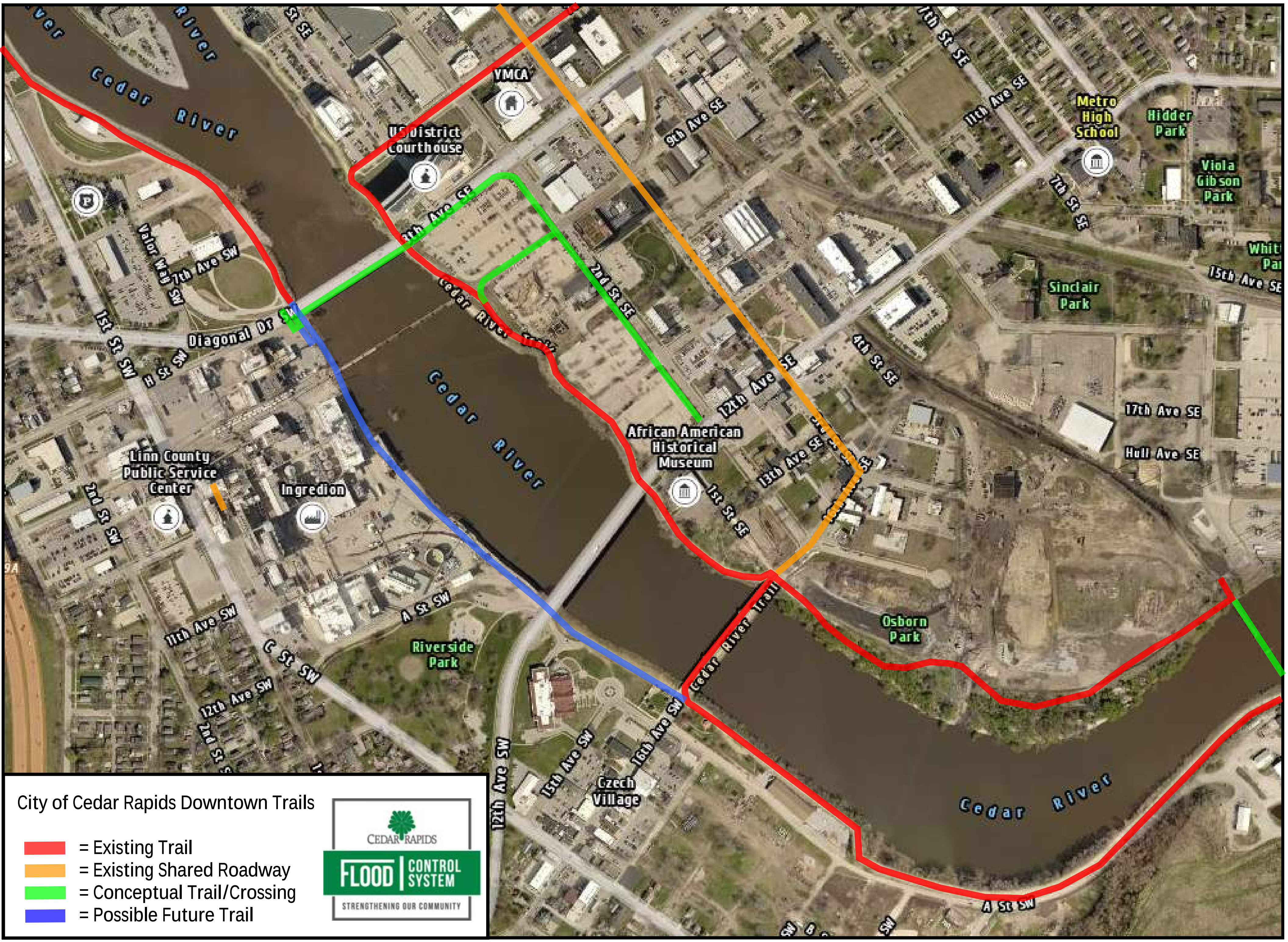

N
GRAPHIC SCALE
80 0 40 80
1st Street SW
Diagonal Drive
Potential Building
Flood Control (TYP)
Pump Station
U.S. Federal Courthouse
2nd Street SE
8th Ave. SE
8th Ave. SW
8th Avenue Bridge
Roundabout Option
1st Street SW/Diagonal
Drive/ 8th Ave
New Bridge Extents
8th Avenue Bridge Replacement
Proposed Alignment
City of Cedar Rapids
Flood Control System
December 2017
CEDAR RAPIDS
FLOOD CONTROL SYSTEM
STRENGTHENING OUR COMMUNITY


City of Cedar Rapids Downtown Trails
= Existing Trail
= Existing Shared Roadway
= Conceptual Trail/Crossing
= Possible Future Trail
CEDAR RAPIDS
FLOOD CONTROL SYSTEM
STRENGTHENING OUR COMMUNITY
Cedar River
YMCA
US District Courthouse
Metro High School
Hidden Park
Viola Gibson Park
Sinclair Park
African American Historical Museum
Linn County Public Service Center
Ingredion
Riverside Park
Osborn Park
Czech Village
Cedar River Trail
Diagonal Dr SW
7th Ave SW
8th Ave SE
9th Ave SE
11th Ave SE
12th Ave SE
13th Ave SE
15th Ave SE
17th Ave SE
Hull Ave SE
11th Ave SW
12th Ave SW
13th Ave SW
16th Ave SW
12th Ave SW
1st St SW
2nd St SW
H St SW
A St SW
C St SW
1st St SE
2nd St SE
4th St SE
7th St SE
Valor Way SW