FEASIBILITY STUDY

PID 115383

SUM-CR8-9.08 (High Level Bridge)
N Main Street over Cuyahoga River

May 3, 2024
Revised October 25, 2024

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Introduction

This study has been prepared by GPD Group for the Summit County Engineer's Office to determine the best structure alternative for the bridge carrying North Main Street over the Cuyahoga River in Akron and Cuyahoga Falls, Ohio. The existing deck truss bridge was opened in 1949 and is approaching the end of its 75-year design life. Gannet Fleming provided assistance with geotechnical recommendations as well as development of two structure alternatives. EMH&T provided environmental assistance.

The structure type study requirements set forth in the current version of the 2020 Ohio Department of Transportation (ODOT) Bridge Design Manual (BDM) served as the main guidelines for this report. Multiple other documents and publications provided additional guidance, including:

- American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 9th Edition, 2020
- ODOT Standard Bridge Drawings
- ODOT Manual of Bridge Inspection, 2014 with 2017 and 2021 Addendums
- ODOT Aesthetic Design Guidelines, 2018
- ODOT Location & Design Manuals, Volumes 1-3

All available information about the existing bridge and site were obtained and evaluated. Those documents and resources included:

Bridge:
- 1947 Existing Bridge Construction Plans (Wilbur Watson Associates Design & Consulting Engineers)
- 1950 Original Bridge Demolition Plans
- 1991 Major Rehabilitation Plans
- 2015 Coating Condition Assessment of the North Main Street High Level Bridge
- 2017 Repair and Strengthening Plans
- 2017 Life Cycle Cost Analysis Report
- 2018 (Most Recent) Bridge Load Rating Report
- 2022 Fracture Critical Plan
- 2022 (Most Recent) Fracture Critical Bridge Inspection Report
- 2023 (Most Recent) Routine Bridge Inspection Report

Site:
- Historical Akron Beacon Journal photographs obtained from the summitmemory.com website
- 2022 Gorge Dam Removal 50% Plans
- 2023 Gorge Dam Removal Sediment Removal Plans
- 2023 Gorge Dam Removal Sediment Disposal Area Plans
- 2023 Northside Interceptor Tunnel (NSIT) Plans
- 2024 Main Street North Hill 50% Plans

The existing bridge was evaluated to determine if extending the service life of the structure was possible and the estimated work/cost required for such a rehabilitation. Additionally, multiple new structure types and span arrangements were developed and assessed. The alternatives listed below are evaluated in the following sections.

- Alternative 1 - Rehabilitate Existing Bridge
- Alternative 2 - Welded Steel Plate Girder Bridge
  - Alternative 2A - Two-Span Welded Steel Plate Girder Bridge
  - Alternative 2B - Three-Span Welded Steel Plate Girder Bridge
  - Alternative 2C - Four-Span Welded Steel Plate Girder Bridge
- Alternative 3 - Open Spandrel Redundant Steel Plate Girder Arch Bridge
- Alternative 4 – Post Tensioned Segmental Concrete Box Girder Bridge


Photo 1: Existing Bridge Elevation Looking Southeast.

Existing Site and Existing Bridge

The High-Level Bridge (SFN 7730306) connects the cities of Akron and Cuyahoga Falls, which are the two largest cities in Summit County. It carries North Main Street over the Cuyahoga River within the Chuckery Area of the Cascade Valley Metro Park, managed by Summit County Metro Parks. At this location, the width of the valley is approximately 900′ rim-to-rim, and the valley floor sits approximately 200′-0″ below the surrounding landscape. The slope on each side of the river is steep, rocky, and heavily wooded. The Highbridge Trail is part of the existing network of trails throughout the valley that connects Cascade Valley Metro Park (to the west) to the Gorge Metro Park (to the east). It is a natural surface trail that exists below Span 1 of the existing bridge. There are apartments near the southwest quadrant, an assisted living facility near the northwest quadrant, and residential houses near the northeast and southeast quadrants of the bridge. There are high voltage overhead electric lines running east-west near the south end of the bridge. The only utility lines on the bridge are electric lines for overhead street lighting.

The original high-level bridge was a seven-span open spandrel reinforced concrete arch structure located approximately 200 feet to the west of the current/existing bridge. It began where North Howard Street currently dead ends and had a 26′-0″ wide roadway to accommodate one lane of traffic in each direction. It was constructed in 1914, but quickly became inadequate to carry the increasingly heavy loads and increasing traffic volume in the decades that followed its construction. The current bridge was built on the new alignment and opened in 1949 with double the vehicular capacity. The original bridge was demolished in 1950, but portions of the original abutments remain in place and are visible today.

The currently existing bridge is a three-span steel cantilever deck truss with reinforced concrete deck carrying two lanes and a sidewalk in each direction. The reinforced concrete abutments and piers are founded on spread footings. Span lengths are 210′-0″±, 480′-0″±, and 210′-0″± center-to-center of bearings. The end spans act as anchor spans with massive steel eye bar anchorage units embedded in the concrete abutment footings at each end. The middle span is 480′-0″± long consisting of a 120′-0″± suspended span between two 180′-0″± cantilevers. The bridge is on a tangent alignment with no skew. It was designed for S-20-40 Loading.

When it first opened, the bridge featured an open steel grid deck which gave it the nickname the singing bridge for the sound that it made when vehicles would drive over it. In 1991 a reinforced concrete deck replaced the open steel grid deck in the end spans, and a filled steel grid deck replaced the open steel grid deck in the center span. Various steel members were repaired and reinforced to accommodate the additional weight of the new deck. As a result of the redistributed dead loads, the tension forces in the end anchorage tie downs were greatly reduced. The new deck is 52′-0″± face-to-face of curbs and on each side of the road a 6′-0″± sidewalk sits between an 8′-0″ chain-link fence and guardrail with tubular steel backup. In 2017/2018 and in conjunction with the most recent load rating, various steel members were repaired and strengthened. The updated portions of the bridge were designed for HS20-44 and the Interstate Alternate Loading. Various existing bridge details (plan, elevation, and section) can be found on applicable plan sheets in **Appendix A** and **Appendix B**.

Design team personnel visited the site multiple times to observe and confirm existing site conditions. No detailed or cursory inspections of the existing bridge were performed.

Adjacent Projects

At the time of this study there are multiple other projects in various stages of design/construction in the vicinity of this bridge.

Gorge Dam Removal Project

Approximately 3,300 feet east (upstream) of the bridge, the existing 57′-0″ tall, 425′-0″ long Gorge Dam is scheduled to be demolished. Prior to the dam removal, approximately 865,200 cubic yards of sediment will be removed from the dam pool because sediment studies have determined that direct contact with the sediment may pose a human health risk, but the sediments are not considered hazardous waste. The sediment will be removed from the dam pool via mechanical dredging methods, screened, and pumped along the Highbridge Trail and under the existing bridge in 8″ diameter high density polyethylene (HDPE) pipes to designated disposal areas within the Cascade Valley Metro Park. The exact number and location of the pipes will be determined by the contractor. Once there, the sediment slurry will be mixed with cementitious material for workability/stability and stockpiled/contoured to match the characteristics of the surrounding area. Then it will be capped with natural soil and the site will be restored. The most recent schedule has the pipeline installed in mid-2024 and remaining through the end of 2025 when it should be removed. Pipeline construction, operation, and subsequent removal should not have any impact the existing or proposed bridges in this study. Jacobs Engineering designed the plans for the dredging phase of the project. At the time of this study, final plans have been submitted, and the project should go out to bid soon.

Once the sediment is removed, the dam will be demolished in stages so that a sudden drawdown of the dam pool does not negatively impact the upstream slope stability or the upstream infrastructure. The area encompassing the footprint of the dam and the dam pool will also be restored to a more natural condition. Impacts to river conditions at the bridge because of the dam removal (such as flow rate and water surface elevations) are expected to be negligible. The timeline for the dam removal phase is contingent on completion of the dredging phase but is expected to begin sometime in 2026 or 2027 and completed sometime in 2028 or 2029. GPD Group is leading the dam removal and dam pool restoration phases of the project. 50% plans were submitted in the Fall of 2022, and 90% plans are currently in development and should be submitted in mid-2024. A project site plan is shown in **Figure 1**.

Northside Interceptor Tunnel Project

The City of Akron is actively engaged in the construction of the Northside Interceptor Tunnel (NSIT) Project, a significant initiative aimed at enhancing the city's sewage management infrastructure. This tunnel is designed to efficiently collect and store combined sewers, contributing to the overall improvement of the city's wastewater system. The tunnel should be in operation by the end of 2026, and final restoration should be completed by mid-2027.

The 16′-6″ diameter tunnel will be bored through the bedrock along the south rim of the gorge. Where is crosses over North Main Street, the tunnel is approximately 100 feet south of the existing bridge limits, and the invert elevation is approximately 800, which is approximately 200 feet below existing grade. The construction of this tunnel should not have any impacts on the proposed bridge alternatives, and none of the proposed bridge alternatives should impact the tunnel.

As part of this project, the Rack 33 baffle drop structure will be built in the dead-end portion of North Howard Street. This structure will tie in several existing and proposed sewers to the new tunnel. It will not have any impact on the proposed bridge alternatives. However, grading down to existing will not be possible in the southwest corner of the proposed alternatives because it will bury portions of North Howard Street and the Rack 33 baffle drop structure, so a tall turn-back wingwall or retaining wall will be required.

As part of this endeavor, a crucial component involves the construction of an overflow outlet structure for the tunnel. Positioned approximately 1,200 feet west of the proposed bridge, this structure will serve as a vital element in managing sewer overflow effectively. Importantly, this construction will have no impacts on either the existing or the proposed bridges. A project site plan is shown in **Figure 2**.

City of Akron North Main Street Improvement Project

The City of Akron is currently in the initial design phases of implementing a road diet for North Main Street located immediately south of the bridge, a project aimed at enhancing the safety and functionality of this important roadway. Initially, the project proposal included the construction of a roundabout located just south of the existing bridge. However, recent developments have seen a shift in plans, with the current focus on implementing a road diet strategy instead. Under the revised plan, North Main Street is envisioned as a three-lane roadway, with dedicated bike lanes on either side. This approach not only aims to improve traffic flow but also prioritizes the safety and accessibility of cyclists, promoting alternative modes of transportation and fostering a more sustainable urban environment.

While the plans for the road diet are not finalized and still subject to potential adjustments, it is anticipated that the core concept will remain consistent. Construction is currently slated to begin in early to mid-2025 and last two construction seasons.

City of Cuyahoga Falls South State Road Corridor Study

The City of Cuyahoga Falls is embarking on a comprehensive study of the State Road corridor, focusing on the area north of the bridge. This initiative is still in its infant stages, with the primary objective being to establish a clear vision, identify actionable steps, and formulate recommendations for the future development of this critical roadway.

At this early juncture, citizen engagement plays a pivotal role in shaping the direction of the study. The city is actively seeking input from residents, property and business owners, and the traveling public to discern the most viable solutions that resonate with the community's desires and priorities. By soliciting feedback and insights from stakeholders, the study aims to ensure that the proposed strategies align closely with the collective aspirations and needs of the local population.

The study is scheduled to be completed and sent to the city in late November 2024. Given the early stage of the study, there is currently no predetermined direction towards which the city is leaning. However, the forthcoming completion of the study is anticipated to precede the construction of the proposed bridge.

Figure 1: Gorge Dam Removal Project Site Plan.

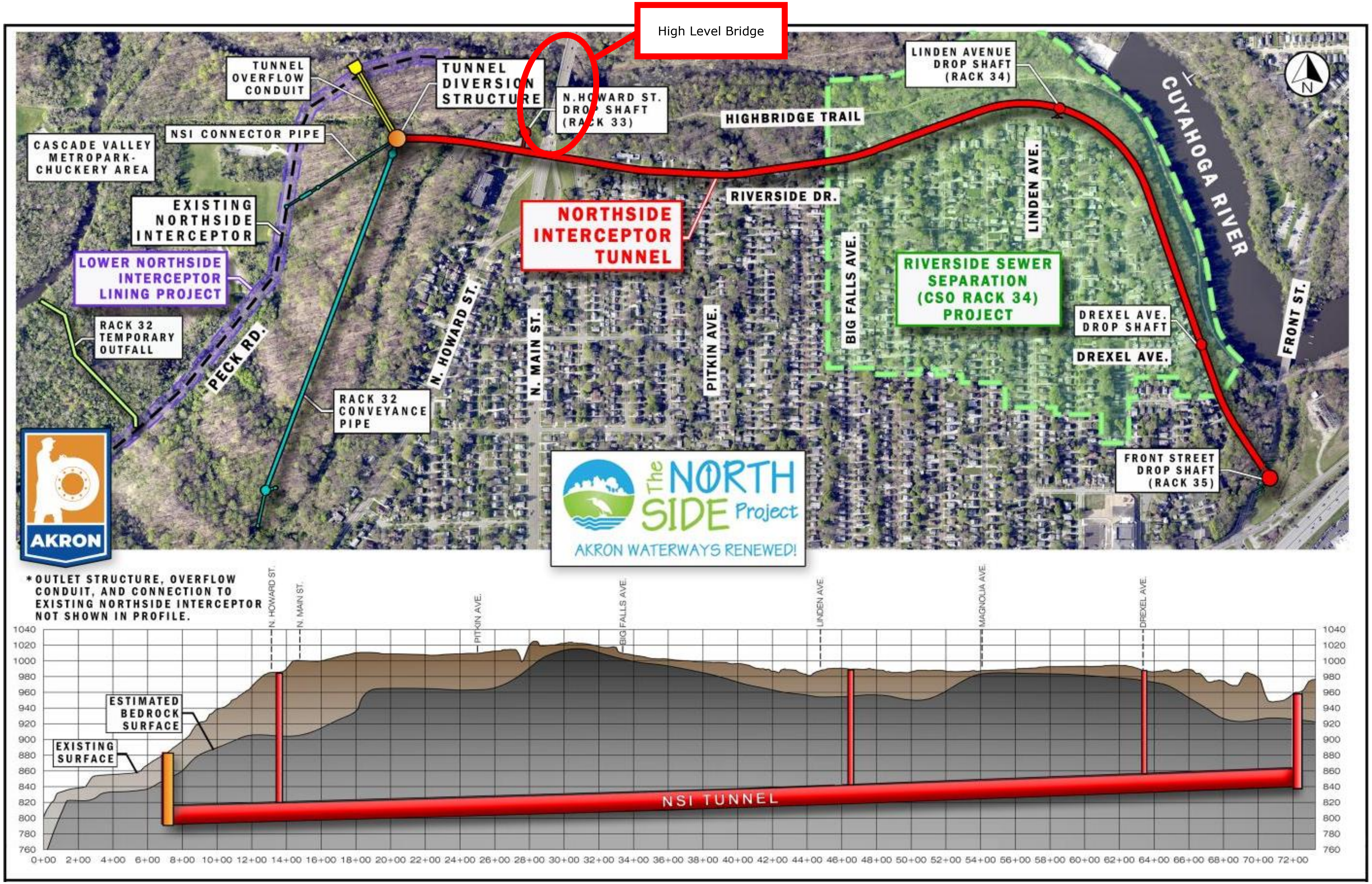

Figure 2: Northside Interceptor Tunnel Project Site Plan.

Roadway/Right of Way Considerations

Structure Location Alternatives:

Three alignment alternatives were assessed for the location of the new bridge, each presenting distinct considerations. The first option, relocating the bridge to the east, was deemed impractical due to significant obstacles. High-power transmission poles and lines situated on the southeast corner of the existing bridge, coupled with steeper terrain than the west side, longer valley width, and residential housing on the northeast corner, posed formidable challenges. The associated costs and impacts on the surrounding environment were deemed prohibitive, leading to the dismissal of this alternate.

Similarly, maintaining the bridge at its current location was explored but ultimately discounted due to its disruptive implications. While this option would have minimized impacts on the surrounding area, the necessity for complete closure of the roadway, a vital artery between Cuyahoga Falls and Akron, proved untenable. The logistical complexities of the ensuing lengthy detours and potential congestion at alternative crossing points, rendered this alternate impractical.

Construction of a new bridge in phases partially overlapping the footprint of the existing bridge was investigated, although ultimately ruled out. The concrete box girder alternative could not be built in phases, and the other alternatives faced different, insurmountable challenges. The existing cantilevered truss bridge type requires it to be demolished at one time, not in phases. In this scenario, half of the new bridge would be constructed adjacent to the existing bridge. All traffic would be shifted to the new bridge, which represents approximately half the lane capacity of the current bridge. The existing bridge would be demolished and removed prior to remobilization for the construction of the second half of the new bridge. The duplication of mobilization efforts proved costly in both time and budget. Due to additional MOT restrictions and inefficiencies in both time & budget, this alternative was ruled out.

The recommended course of action emerged with the proposal to relocate the bridge to the west, closer to its original position when Howard Street served as the connecting road. This option offers minimal disruption to surrounding properties, with ample right-of-way available on the south side of the bridge and limited impacts to lawns and driveways on the north side. Notably, the relocation is strategically offset to optimize cost-effectiveness in bridge construction, precluding the feasibility of phased construction overlapping with the existing structure's footprint while avoiding the original bridge foundation locations. Consequently, the decision was made to construct the new bridge in its entirety before shifting traffic and subsequently demolishing the old bridge.

In executing this plan, the roadway will be realigned to accommodate the proposed bridge, with demolition of the existing structure following the opening of the new bridge to traffic. Permanent right-of-way acquisitions are anticipated to be confined to the northwest quadrant of the project area and are anticipated to be the same for all proposed bridge alternatives since they will all occupy approximately the same footprint. Additional construction access areas are required to facilitate the construction process. The specifics of these access areas will vary depending on the equipment needed for bridge construction, reflecting the dynamic nature of the project's logistical requirements.

Proposed preliminary alignments shifted to the west and to the east are shown in **Figure 3**.

Bridge Width:

In the pursuit of determining the most suitable width for the bridge, multiple configurations were evaluated with a focus on ensuring that future maintenance can be conducted seamlessly, without necessitating detours. The chosen width for this study, in conjunction with the structure type, is pivotal in affording Summit County the flexibility to undertake comprehensive maintenance, including complete re-decking, without imposing constraints on traffic flow.

Figure 3: Proposed Preliminary Alignments Shifted to the West and to the East.

The initial width investigated was as per the request of the Summit County Engineer, encompassing four 12-foot lanes, two 7-foot bicycle lanes, and two 6-foot sidewalks, culminating in an overall width of 76’-4”. This configuration emerges as optimal for future maintenance activities, offering sufficient space for safe pedestrian access and accommodating traffic flow with minimal disruptions. Specifically, the provision of a 6-foot sidewalk, two 11-foot through lanes, and a 2-foot shy zone to a temporary barrier ensures efficient maintenance operations while preserving traffic functionality on the opposite side of the bridge. Positioning the bike lanes between the curbs and traveled lanes also provides additional buffer for drainage spread and reduces the need for scuppers.

Subsequently, an alternate (Alternate A) bridge width was explored, narrowing the structure to two 12-foot lanes, two 14-foot (12-foot and 2-foot shy zone) lanes, and a 15-foot wide shared-use path, resulting in an overall width of 69’-4”. However, despite its potential cost savings, this configuration was eliminated from consideration due to incongruence with the City of Akron's plan to provide dedicated bike lanes in each direction. The reliance on a shared-use path for cyclists during maintenance posed safety concerns and lacked consistency, thereby rendering it unsuitable for adoption.

Finally, a third alternate (Alternate B) bridge width was evaluated with the aim of cost minimization, proposing a narrower two-lane bridge with 7-foot bike lanes and 6-foot sidewalks. While this configuration offers space for pedestrian and cyclist accommodation, its reliance on a directional detour during future maintenance precluded its viability for consideration.

In conclusion, the selected bridge width, aligned with the Summit County Engineer's request, strikes a balance between accommodating diverse modes of transportation, facilitating future maintenance needs, and upholding safety standards. By prioritizing consistency and functionality, the chosen configuration ensures the seamless integration of the bridge into the surrounding transportation network, fostering connectivity and accessibility for all users. Figures depicting the three bridge widths discussed along with potential future maintenance of traffic layouts are shown in **Appendix B**. Additional bridge widths could be evaluated once future roadway configurations at each end of the bridge are finalized. Final width is to be selected at the final design stage.

Roadway Connection:

The City of Akron's plan to implement a road diet necessitates seamless integration between the existing four-lane bridge section and the proposed three-lane roadway configuration. To achieve this transition, the outside southbound lane on the bridge will be designated for exiting onto N Howard Street, effectively reducing the bridge's southbound capacity. Simultaneously, the northbound lane will be adjusted to align with the two northbound lanes on the bridge, maintaining consistency in traffic flow.

However, the road diet's implementation is contingent upon various factors, including the outcome of the City of Cuyahoga Falls Corridor Study. Given the uncertainty surrounding this study, it is anticipated that the road will retain its current four-lane configuration until further notice. The final determination regarding the connections at either end of the bridge will be made during the detailed design phase, taking into account the vetted plans for the connecting roads and any subsequent construction activities.

This approach ensures flexibility and adaptability in responding to evolving transportation needs and infrastructure developments, allowing for informed decision-making based on comprehensive analysis and community input. As the project progresses, careful consideration will be given to optimizing traffic flow, enhancing safety, and promoting connectivity between Akron and Cuyahoga Falls, ultimately facilitating efficient and sustainable urban mobility for residents and commuters alike. Figures depicting the Roadway Connection are shown in **Appendix B**.

Construction Haul Road:

Determining the most efficient route for construction materials to reach the site involves collaboration between the contractor and state and local authorities. In our assessment, we believe the least intrusive route to deliver the bridge beams will be to come southbound on State Road. The trucks can access State Road from SR 303 at the SR 8/SR 303 Interchange in Hudson and travel southbound through Cuyahoga Falls to the project site without the need to make turns through intersections. We believe traversing the local streets in Akron will be difficult and has the potential to create unwanted traffic issues. Access to the site for delivery of equipment and materials that will be used at river level will utilize Peck Road and the existing access drive along the river. Peck Road is a dedicated city street traversing Metro Parks Serving Summit County property, and the access drive is situated within the Metro Parks. The Highbridge Trail runs parallel to the access drive. Currently, Peck Road and the 16-foot-wide stabilized construction entrance access drive serve as a haul route for the Northside Interceptor Tunnel Project's construction. Upon completion, the construction access drive will be restored to a 10-foot-wide gravel access drive. For our project, the contractor will need to widen the access drive to accommodate construction equipment and any oversized materials, although beam delivery is not expected along this route. However, the crane used for beam placement must be able to navigate the drive. Based on Gannet Fleming’s geotechnical research, the existing slopes along the drive are nominally stable. However, changes to the geometry such as adding fill, removing material from the toe of slope, or intense rain events could cause failures. Shoring may be required regardless of the alternate utilized.

Two alternate haul road routes along the access drive were explored to assess feasibility. Both alternates utilize the same typical section consisting of a 20-foot-wide minimum aggregate road with 2-foot-wide shoulders and 3:1 slopes. This width will provide ample room for delivery of equipment and materials. The profiles will be as close to existing as possible to avoid large cuts or fills that could undermine the existing slopes. Truck turning movements through the site were established using the Civil 3D 2024 Vehicle Tracking module.

The first alternate investigated aims to minimize impacts on the Cuyahoga River, by navigating through the Northside Interceptor Tunnel Control Building site and considers the construction of the tunnel overflow outlet structure beneath the haul road. This alternate necessitates the construction of a temporary bridge over the structure, protecting it from being crushed during construction. After the haul road is no longer needed, the temporary bridge will be removed, and the area restored. Additionally, measures will be taken to protect monitoring equipment associated with the outlet structure and historical steps on the trail's south side.

The second alternate is similar to the first but diverges by extending the haul road around the Northside Interceptor Tunnel Control Building site and into the river near the overflow outlet structure instead of over it. Although there is a slight risk of occasional overtopping during a large storm event, this eliminates the need for a temporary bridge and provides greater separation between the construction activities and monitoring equipment and historical features. While backfilling and piping within the Cuyahoga River limits will be required, impacts will be minimal compared to those associated with constructing a pad at the proposed bridge site. Obtaining Flood Plain Development Permits for work in the Special Flood Hazard Area (SFHA) and/or Floodway would require coordination with the Flood Plain Managers for Akron and Cuyahoga Falls. Restoration efforts will be undertaken in this area as well. Considering costs and impacts, Alternate 2 emerges as the recommended choice for the haul road route.

By carefully evaluating these alternatives and prioritizing minimal environmental impact and efficient construction operations, we aim to ensure the successful execution of the project while preserving the integrity of the surrounding ecosystem and historical features. Figures depicting the Construction Haul Road Alternates are shown in **Appendix B**.

Maintenance of Traffic Considerations

The goal of maintenance of traffic is to minimize impacts to the traveling public while constructing the project in the most efficient way possible. The High-Level Bridge connects the cities of Akron and Cuyahoga Falls. Maintaining this connection is critical as there is no connection over the gorge within a 1-mile vicinity. Multiple maintenance of traffic options were considered for construction of the bridge. These options vary based on which alternative was selected. Potential future maintenance of traffic layouts are shown in **Appendix B**.

Structure Alternative 1 (Rehab) maintains the existing alignment during construction. Part-width phased construction was determined to be the best option for a future deck replacement considered with Alternative 1. It was assumed the North Main Street will be reduced to two 10-foot lanes, keeping a single lane open in both the northbound and southbound direction. Additionally, the Traffic Engineering Manual (TEM) requires a 2-foot clearance from the edge line to the toe of the portable barrier. Lastly, a minimum 1-foot clearance would be maintained from the back of the portable barrier to the sawcut. Utilizing these criteria, the minimum required width on the existing bridge for part-width, phased construction is 25-feet. The available width for part-width, phased construction is 26-feet allowing this to be a viable option.

The resulting 1-foot clearance between the back of the barrier and the saw-cut as discussed will require the portable barrier to be anchored to the new bridge deck and for that reason a two-way, one-lane signalized closure was considered. However, due to high traffic volumes, commercial and residential drives within the work zone, and the proximity of Highbridge Road and Grant Street near the work zone, this option was eliminated.

Structure Alternatives 2 - 4 construct the new bridge offline on a new alignment west of the existing bridge. A majority of the bridge can be constructed without disturbing traffic on North Main Street. When transitioning the realigned road back to the existing road, traffic on North Main Street will be reduced to a single lane northbound and southbound and shifted to the east side of the existing structure. Once the proposed southbound lanes are fully tied-in, the single lanes of traffic will be shifted to the new bridge while the northbound lanes are constructed. The existing bridge will be removed while traffic is fully operational on the new bridge.

When considering future maintenance of traffic, based on the discussion above, a minimum roadway width of 25-feet is necessary for part-width, phased construction. If the narrower width is built for Alternatives 2-4 for pedestrian and cyclist accommodation, a directional detour will be needed during future maintenance.

Geotechnical Considerations

In 1960 there were multiple borings advanced along the Highbridge Trail. One of them (B-20) exists near the southeast corner of the high level bridge. Additionally, in 1989 there were six borings advanced in the vicinity of the existing bridge as part of a subsurface investigation. Geotechnical information such as soil type and top of rock elevations was taken from these borings and used to develop preliminary recommendations for the substructure foundations of the proposed bridge alternatives. A preliminary slope stability analysis and shoring recommendations were also developed. Future project borings will need to be performed once a proposed bridge type is selected and the location of substructure units finalized. These will enable more accurate recommendations for footing elevation, foundation design, shoring requirements, etc.

The historic borings suggest a relatively horizontal stratigraphy with shallow competent bedrock. Where the surface elevation (EL) is below 910 feet, it is anticipated there is up to approximately ten feet of overburden soil above top of rock. For surface elevations above EL 910, the top of rock is assumed to be EL 910 with various overburden soil types between.

Because competent rock is so shallow, possible pier foundations include spread footings or drilled shafts. At this stage it is assumed that the foundation concrete strength will control when designing for bearing resistance and not the strength of bedrock. Spread footings would not require additional drilled shaft equipment mobilization but would need to be deeper to protect against potential scour and would also take up a much larger footprint to resist the loads from such a large structure. Drilled shafts are less susceptible to scour, better able to accommodate the high design loads, and require a smaller footprint (meaning the footing will not cut into the side slopes as much and require less excavation bracing). For these reasons, drilled shafts are the preferred option for the proposed pier foundations at this stage of planning.

Abutments could be supported by spread footings, H-piles, or drilled shafts. Like the piers, abutment spread footings would need to be exceptionally large to withstand the high design loads, and they would sit on the overburden soil, not bedrock. For these reasons, spread footings were not considered. H-piles could be driven to refusal, but because pier foundations are recommended to utilize drilled shafts, the abutments are also recommended to be use drilled shafts. This removes the need to mobilize multiple types of foundation equipment and is therefore expected to be more cost effective.

More information can be found in the Foundation Recommendation Memorandum prepared by Gannett Fleming and is included in **Appendix C**.

Environmental & Cultural Resources Considerations

As part of the planning and preliminary engineering effort, a secondary source review was conducted to identify any potential environmental or cultural resources "red flag" issues to be considered within the project area. The results of this evaluation were considered as bridge rehabilitation/replacement alternatives were evaluated. The red flag investigation was undertaken with the knowledge that a full-environmental National Environmental Policy Act (NEPA) review will be required should federal funding be acquired and the project be advanced for further consideration. Data was gathered through a combination secondary source literature reviews and information gathered through direct coordination with Summit County Metro Parks staff. Several environmental and cultural resources parameters were evaluated including surface waters and wetlands, parklands and recreational facilities, cultural resources, threatened and endangered species, air quality and noise, hazardous materials, environmental justice populations, and drinking water resources. The results of the red flag investigation are summarized in **Appendix D**.

Structure Design Considerations

This study investigated several alternatives to address deficiencies in the bridge carrying North Main Street over the Cuyahoga River. Rehabilitating the existing bridge was evaluated, and several full reconstruction projects were considered. Multiple structure types and span arrangements were considered for determination of the best/most economical structure replacement option. The alternatives include:

- Alternative 1 - Rehabilitate Existing Bridge
- Alternative 2 - Welded Steel Plate Girder Bridge
    - Alternative 2A - Two-Span Welded Steel Plate Girder Bridge
    - Alternative 2B - Three-Span Welded Steel Plate Girder Bridge
    - Alternative 2C - Four-Span Welded Steel Plate Girder Bridge
- Alternative 3 - Open Spandrel Redundant Steel Plate Girder Arch Bridge
- Alternative 4 – Post Tensioned Segmental Concrete Box Girder Bridge

Alternatives utilizing prestressed concrete I-beams were excluded from consideration due to the long span lengths which would have required extremely long and extremely heavy beams to be shipped to the site.

The proposed bridge options consisted of replacing the existing bridge with a new structure on a new alignment to the west of the existing bridge. A 6′-0″ sidewalk, 7′-0″ bike lane/buffer, and two 12′-0″ will occupy each side of the bridge for a total out-to-out deck width of 76′-4″. The transverse section was kept the same for all proposed bridge alternatives so that an apples-to-apples comparison can be made between them.

Temporary Highbridge Trail closures are required for construction activities regardless of the alternative selected. Street lights exist on the existing bridge and are proposed for all alternatives. Potential temporary utility impacts will be evaluated at the detailed design stage. No permanent utility relocations are anticipated for any alternative. Regardless of the alternative chosen, the greenspace at the south end of the bridge (east side and west side) can serve as an excellent staging areas for construction activities.

All alternatives keep the piers out of the main channel, so no river impacts such as scour, debris, and ice flow problems are anticipated. The extreme height of the bridge means that hydraulic opening is not an issue.

Obtaining Flood Plain Development Permits for temporary access fills (TAFs) and work in the Special Flood Hazard Area (SFHA) and/or Floodway will require coordination with the Akron and Cuyahoga Falls Flood Plain Managers.

Narrative of Bridge Alternatives

Alternative 1 - Rehabilitate Existing Bridge

General Information & Bridge History

Preliminary plans were not developed for Alternative 1. However, the existing structure block as well as the existing bridge in plan and elevation view all show up on the site plans for the other alternatives, which can be found in **Appendix A**. The existing bridge was opened to traffic in 1949 and has essentially reached the end of its design service life, typically considered to be 75 years. The Ohio Department of Transportation Historic Bridge Survey Report indicates that the existing structure is eligible for listing on the National Register of Historic Places (NRHP). Alternative 1 of this Feasibility Study therefore considers potential structure rehabilitation to extend the useful life of the structure. Consideration of structure adequacy including condition, geometry/safety and load-carrying capacity will be considered in determining whether it is feasible and prudent to rehabilitate the existing structure. Existing structure rehabilitation was incorporated into this study for direct comparison with complete replacement alternatives. Historic Considerations discussed below can be used for **Section 4f** coordination.

The following points briefly describe the general features of the existing structure and summarize the significant maintenance, rehabilitation and studies performed on the existing structure to date:

- The existing structure is 3-span steel cantilever deck truss with spans of 210′-0″±, 480′-0″±, and 210′-0″± for a total length of 900′-0″± center-to-center of abutment bearings. Each end span acts as anchor span and is provided with steel eye bar anchorages at abutments to prevent uplift. The 480′± middle span includes a central 120′± long suspended span between the two 180′± cantilevers extending from Piers 1 and 2. The bridge is on a tangent alignment with no skew. **The structure is considered FRACTURE CRITICAL having numerous non-redundant steel tension members throughout the two main support trusses as well as the floorbeams.**
- The structure was originally designed and constructed using an open steel grid deck system within the roadway on all spans to reduce dead loads. The exterior sidewalk surfaces were comprised of concrete filled steel grid deck. Original design loading was S-20-40.
- Substructures consist of reinforced concrete abutments and piers with spread footings founded on bedrock.
- A major rehabilitation in 1993 incorporated significant structural steel repairs including stringer replacement, miscellaneous rivet replacement to address deterioration or connection capacity, addition and replacement of truss member cover plates, lateral bracing repairs, caulking of pack rusted truss plates and members, addition of supplemental post-tensioned rods to strengthen truss tension members and floor beams, and complete painting of the structural steel. Substructure concrete patching repairs were also completed.
- As part the 1993 major rehabilitation, the open steel grid deck on spans 1 & 3 was replaced with 67′-7″± wide conventional reinforced concrete decks having 52′± wide roadway carrying two lanes in each direction, curbside twin steel tube/thrie beam vehicular railings and 7′-9″± wide sidewalks with 8′ tall chain link fence on each side. The open steel grid deck on span 2 was also replaced with a new 52′-0″± concrete filled steel grid deck, a 1/4″ epoxy wearing surface and curbside twin steel tube/thrie beam vehicular railings. Sidewalks on both sides were replaced with new concrete filled steel grid deck and 8′ tall chain link fence. The new concrete decks increased the dead load in most of the truss and floor beam members. Design loading for the 1993 rehabilitation was HS20-44 and the Alternate Military Loading.
- In 2015 a detailed Coating Condition Assessment was performed for the existing structural steel coating system. At the time of the 2015 assessment, the existing coating system was determined to be in fair condition with the coating system largely intact and a relatively small percentage of localized areas of corrosion or coating defects. Recommendations at that time included spot repairs for the majority of the bridge structure. At areas beneath the overlap of the grid decks in Span 2, the coating condition was worse and active corrosion of the grid deck and adjacent members was occurring. The amount of corrosion at the sidewalk fascia and underside of the deck adjacent to the fascia was relatively high. For those areas of Span 2, zone coating removal and replacement by abrasive blast cleaning was recommended. To date, the full extent of coating repairs recommended in the 2015 report have not been completed.
- In 2017 a comprehensive Life Cycle Cost Analysis was prepared for the Summit County Engineer to help establish an end-of-life plan for the bridge. The North Main Street Bridge was nearly 69 years old, and the last major rehabilitation of the bridge was completed in 1994 with those improvements having been in service for over 22 years as of 2017. Alternatives considered within the Life Cycle Cost Analyses included replacement of the structure at various points to maintain the structure in a serviceable condition. <u>Alternatives mentioned herein are based in the year 2017</u> and included: Alt. 1 maintain the bridge for 75 years through a second deck replacement, Alt. 2 replace the structure with a new bridge as soon as practical (assume to be in 10 years), and Alt. 3 replace the structure in 25 years with new bridge instead of replacing the deck a second time. The results of the life cycle cost analysis showed the net present value, including potential residual values, of all three alternatives to be very close with a spread from highest to lowest being within about 2%. Non-monetary benefits of Alternate 2 included sooner replacement of the fracture critical non-redundant structure providing lowered risk of sudden collapse, and economic benefits of reduced maintenance cost on a new bridge thereby providing financial means for better maintenance of other bridges within the county.
- In 2018 contracted maintenance repairs were performed on the structure which included repair of the wearing surface, repair of the epoxy overlay on Span 2, replacement of deteriorated rivets at various connections, repair and stiffening of bowed gusset plates at upper and lower truss chords, sealing of sidewalk cracks and other miscellaneous repairs.
- In 2018 a load rating report was prepared for the existing bridge which considered the previous repair and strengthening work completed on the bridge. In its 2018 condition, the bridge was determined to have an Inventory rating of HS13.7 controlled by the truss upper chord gusset plates. The floorbeams in Span 2 had an inventory rating of HS14.0. Other locations, including anchorage pins, lower chord gusset plates, floorbeams in Spans 1 & 3 and the deck in Span 2, also had Inventory ratings less than the design vehicle rating of HS20 (i.e. Inventory Rating Factor less than 1.00). **An HS13.7 Inventory rating indicates the structure can only carry 68.5% of the design vehicle loading at the normal design stress level**. Operating ratings pertaining to maximum permissible stress levels include values of 139% for Ohio Legal Loads, 123% for the 5C1 legal load <u>truck train</u>, 104% for the Special Hauling Vehicle Loading, and 110% for the Emergency Vehicle Loading. In general, past repairs and strengthening have provided just enough capacity to keep the bridge from requiring load posting signage for reduced live loading.
- It is unknown if any fatigue life studies have ever been undertaken for the existing structure and therefore the remaining fatigue life is unknown. When considering rehabilitation of the existing structure it may be prudent to also evaluate the fatigue life of the superstructure elements.

Current Physical Condition of the Bridge

The latest routine inspection of the bridge was conducted in September 2023. This routine inspection was performed from the deck surface, ground surface and the existing catwalk system, without the use of complex access techniques or equipment. See **Appendix H** for a full copy of this inspection report.

The current NBI condition Ratings for the bridge are as follows:

58 – Deck Summary 5 (fair condition)
  58.01 – Wearing Surface 5 (fair condition)
  58.02 – Joints 6 (satisfactory condition)
59 – Superstructure 5 (fair condition)
  59.01 – Paint & PCS 6 (satisfactory condition)
60 – Substructure 7 (good condition)
61 – Channel 9 (excellent condition)
  61.01 – Scour 7 (good condition)

67.01 General Appraisal 5 (fair condition)

Significant Findings of the 2023 inspection include, but are not limited to, the following:

- The east pedestrian fence posts exhibit heavy corrosion with multiple locations of 100% section loss (report photo 4). At some locations where heavy corrosion exists in the pedestrian fencing, steel plates were fastened to the horizontal pipes by county maintenance to stiffen the loose components.
- The longitudinal joint along the curb line in span 2 between the deck and sidewalk leaks and is corroding the grid deck along the joint (report photo 14). This problem persists throughout Span 2 along both sidewalk joints.
- The epoxy overlay in span 2 was repaired in 2018 but is failing. Sections of the epoxy overlay are deteriorated and there are locations where the overlay has delaminated and is able to be removed by hand (report photo 16).
- Approximately 10% of the wearing surface (including patches) in spans 1 and 3 is delaminated. Approximately 75% of the 2018 repair patches have cracked.
- Expansion joints have experienced a gradual closing over time, likely due to movement of the abutments towards the channel. Overall data shows that the joints have been steadily closing since first recorded in 1994. According to the 2023 inspection report, the joint openings still appear adequate for the normal range of temperature experienced by the bridge.
- Floorbeam vertical bearing stiffeners overtop of the truss upper chord exhibit painted over pack rust up to 1 inch thick bowing the stiffener angle legs outward. Additionally, painted over pack rust up to 3/4 inch thick exists between the bottom flange and fill plate at sway bracing connections.
- Main truss verticals members exhibit pack rust between built-up members and connection plates at isolated locations. Adjacent areas around existing pack rust require monitoring during in-depth inspections for distress.
- Main truss diagonal member retrofit post-tensioning rods for west U12L13 and U18L17 exhibit section loss reducing the diameter up to 30% at the lower panel point anchor (report photo 32).
- Main truss diagonal member east U2L1 outboard connection exhibits up to 1 1/4-inch-thick pack rust causing distortion of the fill plate up to 1 1/4 inches. The diagonal member requires monitoring for significant section loss where pack rust has developed.
- Minor to heavy pack rust up to 1 inch thick exists between built-up members throughout truss diagonals.
- Main truss diagonal member west U23L24 has up to 75% section loss to rivet heads.
- Main truss upper chord has typical pack rust existing between the bottom plate of the upper chord and bottom of the web distorting the bottom flange plate intermittently (report photo 33).
- Main truss lower chord west L15 interior web plates exhibit distortion up to 2 inches inward with adjacent heavy surface corrosion and failed paint due to pack rust. This condition exists at a few other isolated locations inside lower chord members. The cause of the pack rust could not be determined at the time of inspection; however, the distortion exists at sway bracing connection locations and possibly is from lateral forces at the connection. This specific condition was not previously identified, so it is difficult to determine if the area has deteriorated rapidly or slowly. The areas should be checked again for active corrosion, section loss, pack rust and distortion during in-depth inspections.
- Main truss west lower chord exhibits pack rust and active corrosion exists along the top edges of web plates at the panel points. This condition exists at multiple panel points due to past debris accumulations which retains moisture.
- Main truss lower chord heavy corrosion and some section loss exists mostly in the bottom flange plates of the members at the south end of span 1 and north end of span 3 near the abutments (report photo 35).
- Main truss gusset plate west L0 inboard gusset plate exhibits a 1-foot-long x 3-inch-wide x 3/16-inch-deep area of painted over section loss along the upper chord inside the panel point. Painted over section loss is common on the interiors and exteriors of gusset plates with the maximum amount of loss noted up to 1/4 inch deep (report photo 36).
- Main truss gusset plate east U29 outboard gusset plate was noted to be previously bowed and has been retrofitted with a bolted angle. Other isolated bows exist in gusset plates typically less than 3/16 inch. If bows of other gusset plates increase significantly, additional repairs may be required.
- Main truss gusset plate pack rust up to 3/4 inch thick with adjacent section loss in the gusset plates exists between the plate and angle members of diagonals inside the panel point prying the corners of the diagonal angles. The section loss and pack rust is not currently active.
- Eyebar tie down anchorages at each abutment exhibit active surface and laminating corrosion with less than 1/16-inch-deep section loss (report photo 40). The corrosion appears to be very gradual. Ultrasonic testing should be performed on the 6 pins at each abutment every 10 years. The last testing was performed in 2015. The anchorage eyebars and pits exhibit moisture which can accelerate the oxidation process of the eyebars (report photo 41).
- Paint/Coating System: Floorbeams, stringer ends, and truss members under the joints exhibit areas of corrosion, failing paint, and pack rust between components. Active corrosion with ineffective paint exists at isolated locations throughout.
- Both abutments show a gradual movement towards the channel. This condition is common for tall abutments where active soil pressure behind the abutment moves the abutment by either sliding or rotation over time. Reference is made to the related Expansion Joint finding listed above.
- Abutment rotation from the wingwalls exists at the southwest and northeast corners of the bridge. Measurements to acquire a baseline for each rotation were taken at the bases and tops of each wall. The separation between the wingwalls and abutment backwalls were:
  - The southwest wingwall - 1/2 inch measured at the base and 1 3/4 inches (previously 1 1/2 inches in 2022) measured at the top (report photo 48).
  - The northeast wingwall - 0 inches measured at the base and 1 1/4 inches measured at the top of the wall (previously 1 1/2 inches in 2022) (photo 49).
- Slope failures and washout areas exist at the southeast and northeast corners of the bridge (report photo 52). Additionally, the slope is washed out under panel points 26 and 27 near the forward abutment. Stone rip rap has been placed around the affected area; however, the stones no longer exist where the slope has washed out.

As mentioned above, <u>the existing structure is considered Fracture Critical</u> due to the presence of numerous non-redundant steel tension members existing within the structure. Since there are only two main superstructure members, i.e. the trusses, with each containing numerous non-redundant steel tension members, failure of one of these individual tension members could result in complete collapse of the structure. Additionally, the floorbeams, which are spaced at 30' center to center, are considered fracture critical due to their spacing being greater than 14'. Failure of an individual floorbeam has the potential to cause distortion induced stresses within the main trusses which could lead to collapse of the structure should these stresses affect one of the fracture critical truss members. A diagram showing the location of all Fracture Critical non-redundant steel tension members is included with the 2023 inspection report contained within Appendix H of this study document. It is also shown in Figure 4 on the following page. <u>As shown on the diagram approximately 50% of all main truss members are Fracture Critical, nearly 100% of the main truss gusset plates are Fracture Critical, and 100% of the floorbeams are Fracture Critical</u>.

Rehabilitation of Existing Bridge

To determine if it is feasible and prudent to consider rehabilitation as a viable alternative, a potential rehabilitation plan and related life-cycle cost estimate has been developed. The 2017 Life Cycle Cost Analysis (LCCA) Report for the North Main Street High Level Bridge has been used as a basis for various rehabilitation activities and their associated costs. For the purposes of this Structure Type Study and direct comparison with the overall life of other complete replacement alternatives, it is assumed the existing structure would have various rehabilitation projects completed in the future along with other regular maintenance activities performed by the County to keep the bridge in a serviceable condition for an additional 100 years. At that point, the entire existing structure would be replaced. The basic scope of work for the rehabilitation projects and the associated timeline is presented in the following table.

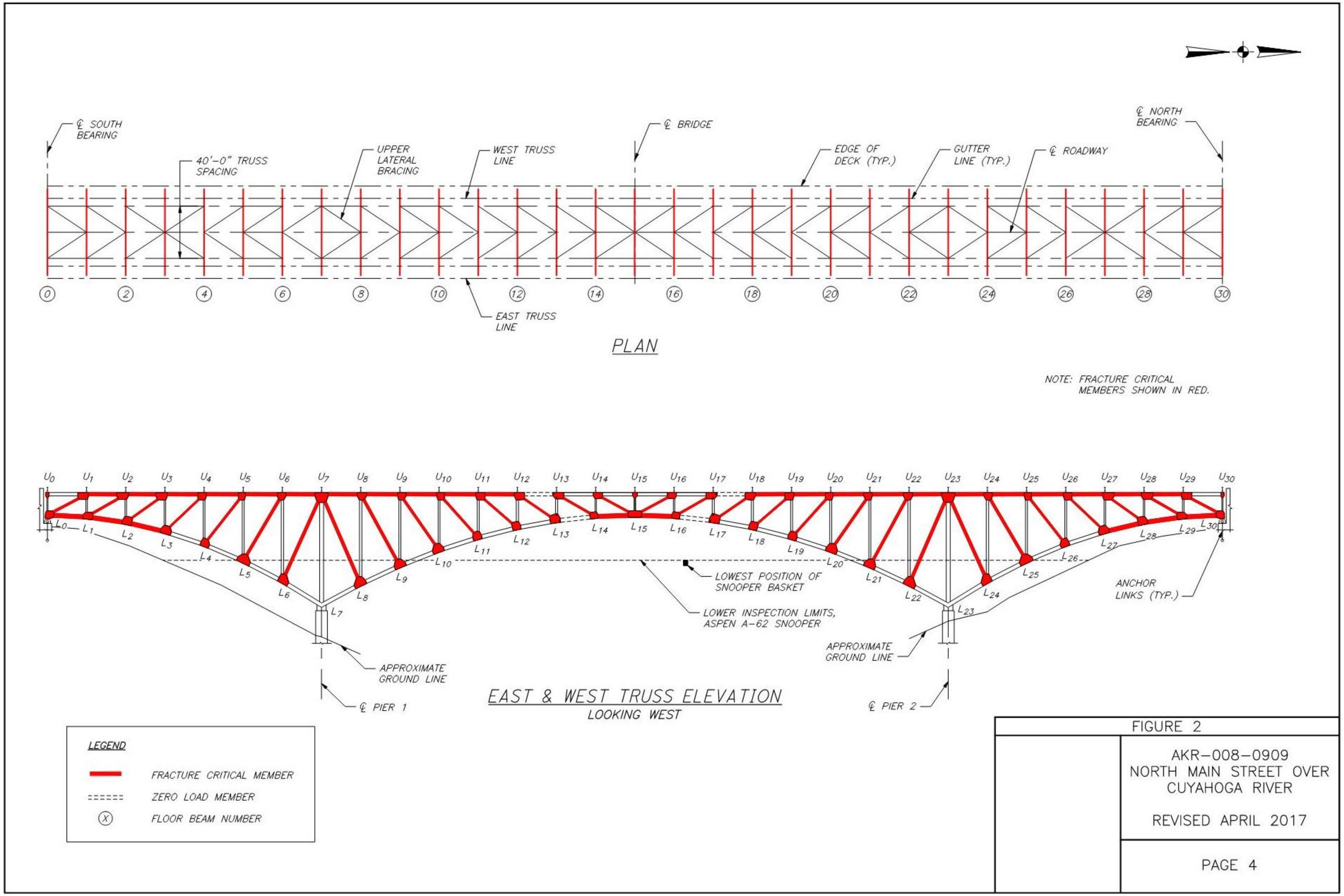

Figure 4: Diagram Showing Fracture Critical Members on the Existing Bridge (from the Fracture Critical Plan).

Year	Description of Work
0	Structure/Drainage/Erosion Repairs, Paint Steel, Replace Deck
20	Structure/Drainage/Erosion Repairs, Seal Deck & Walks, Repair Epoxy W.S. Span 2
35	Structure/Drainage/Erosion Repairs, Overlay Span 1 & 3, Replace Epoxy W.S. Span 2
50	Structure/Drainage/Erosion Repairs, Paint Steel, Replace Deck
70	Structure/Drainage/Erosion Repairs, Seal Deck & Walks, Repair Epoxy W.S. Span 2
85	Structure/Drainage/Erosion Repairs, Overlay Span 1 & 3, Replace Epoxy W.S. Span 2
100	Replace Structure

With respect to the above rehabilitation timeline and activities, GPD also performed a cursory analysis for replacement of the existing structure at Year 50. When accounting for the residual value of the replacement bridge at Year 100, there was little difference in total costs over 100 years. Thus, usage of a 100-year life cycle model for Alternative 1 is suitable for the purpose direct comparison with other complete replacement alternatives. Costs are presented later in this report.

Historical Considerations

What makes the bridge historic? The ODOT Historic Bridge Survey Report dated October 24, 2023 indicates the following:

> "*The 1948 continuous-cantilever design deck truss bridge is a later example of its type/design that is eligible from the prior inventory. The cantilever truss type/design developed in the U.S. during the 1880s and had emerged by the early 20th century as one of the dominant types for longer spans crossing deep or long rivers where it was difficult, if not impossible, to erect falsework. Truss designs used with cantilever trusses, e.g., Pratt or Warren, mirrored those of the period in which the bridge was built, as did the use of pinned or riveted connections. The great advantage of the cantilever is that it can be built outwards from the towers without falsework to block the channel. Suspended spans can be lifted into place between the cantilever arms. Span lengths of up to 500' are not uncommon, and in the longest examples can exceed 1,000'. The Ohio inventory includes 12 cantilever truss highway bridges dating from 1922 to 1960 (Phase 1A, 2008). The bridge is one of 11 remaining examples of the design used for long, major crossings of both deck and thru trusses. They date from 1922 through the interstate era. This is not the most significant example. The bridge has moderate significance.*"

Can bridge members be changed without adversely affecting historical significance? Since the 1994 major rehabilitation, certain structural members have already been altered, replaced or supplemented with other members to address deterioration or improve carrying capacity. The original bridge deck and pedestrian railings have also already been replaced. It appears that alteration of members can be done provided that the historically significant aspect of the bridge, being a cantilever deck truss with suspended span, remain intact. These features would be retained with any future rehabilitation project.

Can the **Condition**, **Load-Carrying Capacity** and **Geometry/Safety** be improved to an acceptable level to maintain functional and operational adequacy of the structure?

With respect to **Condition**, the existing structure's General Appraisal rating is a 5 (fair condition) based on the 2023 annual inspection. The rating of the Superstructure is controlling the General Appraisal in this case. It is generally assumed that future rehabilitation & repair projects described above combined with regular annual maintenance performed by the County would maintain the structure in an acceptable condition. However, three (3) of the Significant Findings listed with the 2023 Annual Inspection are concerning and warrant continued scrutiny which could result in more extensive and costly repair work. The referenced inspection findings include:

i. Expansion joints have experienced a gradual closing over time, likely due to movement of the abutments towards the channel. Overall data shows that the joints have been steadily closing since first recorded in 1994.

ii. Main truss lower chord west L15 interior web plates exhibit distortion up to 2 inches inward with adjacent heavy surface corrosion and failed paint due to pack rust. This condition exists at a few other isolated locations inside lower chord members. The cause of the pack rust could not be determined at the time of inspection; however, the distortion exists at sway bracing connection locations and possibly is from lateral forces at the connection. This specific condition was not previously identified, so it is difficult to determine if the area has deteriorated rapidly or slowly. The areas should be checked again for active corrosion, section loss, pack rust and distortion during in-depth inspections.

iii. Abutment rotation from the wingwalls exists at the southwest and northeast corners of the bridge. Measurements to acquire a baseline for each rotation were taken at the bases and tops of each wall. The separation between the wingwalls and abutment backwalls were:
   a. The southwest wingwall - 1/2 inch measured at the base and 1 3/4 inches (previously 1 1/2 inches in 2022) measured at the top (report photo 48).
   b. The northeast wingwall - 0 inches measured at the base and 1 1/4 inches measured at the top of the wall (previously 1 1/2 inches in 2022) (report photo 49).

With respect to **Load-Carrying Capacity**, it was noted above that multiple members within the structure have an Inventory Rating factor less than 1.000 with a minimum value of 0.685. The Operating Rating factors also vary but are above 1.000 with a minimum value of 1.040. The structure repairs referenced in the rehabilitation events listed above would maintain the structure in a serviceable condition but would not necessarily provide for improvements in the inventory rating factors to 1.000 or greater. It would be cost prohibitive to improve the structure to a minimum inventory rating factor of 1.000 as, among other things, this would require significant replacement of the floor system framing which would also require closure and detour of all traffic on the structure during construction. As such, a **Design Exception would be required for Design Loading Structural Capacity with any future rehabilitation project**.

With respect to **Geometry/Safety** of the structure, it was noted above that the structure is considered **Fracture Critical** due the non-redundant steel tension members which exist throughout the structure (50% of main truss members, nearly 100% of main truss gusset plates, and 100% of floorbeam members qualify). Future rehabilitation projects cannot change the Fracture Critical designation since the lack of redundancy cannot be improved. The risk of total collapse cause by failure of a non-redundant steel tension member within the structure could be reduced with structure maintenance and rehabilitation, but risk could never be eliminated.

The above-mentioned criteria of Condition, Load-Carrying Capacity and Geometry/Safety are used to measure the functional and operational adequacy of the rehabilitated structure. Since there are concerns related to each of these criteria and most notably the Safety criteria with the structure having 50% of the main truss members, nearly 100% of main truss gusset plates, and 100% of the floorbeam members being Fracture Critical before and after any future rehabilitation project, it is the conclusion of this section of the Feasibility Study that rehabilitation of the existing structure is not considered prudent.

Waterway Crossing Requirements

Rehabilitation of the existing structure would likely require installation of a temporary causeway or TAF spanning the Cuyahoga River to facilitate rehabilitation efforts.

Environmental & Cultural Resources Concerns

Rehabilitation of the existing structure would maintain the existing NRHP eligible historic bridge, per the ODOT Historic Bridge Inventory. However, while rehabilitation of the existing bridge structure would minimize the overall project footprint, impacts to environmental and cultural resources would still need to be considered. Environmental considerations associated with the Alternative 1 include temporary impacts to adjacent recreational Section 4(f)/6(f) resources (parks and trails), impacts to streams and wetlands, and potential impacts to threatened & endangered species resulting from necessary tree and vegetation removal. In addition to the historic bridge considerations, cultural resources concerns associated with Alternative 1 revolve primarily around potential impacts to the NRHP listed Chuckery Race due to its location under the existing bridge structure.

Alternative 2 - Welded Steel Plate Girder Bridge

Preliminary plans for Alternative 2B and Alternative 2C can be found in **Appendix A**. Per the scope of this study, a welded steel plate girder bridge type was evaluated. This bridge type has a composite reinforced concrete deck and has the advantage of being extremely common in Ohio and across the country. Primary materials are relatively cheap and readily available. Most contractors will have experience constructing this type of bridge, and there are multiple means & methods for setting girders, such as conventional erection sequencing and incremental launching. Although durability and longevity are main characteristics of multi-girder bridges, they are easily maintained using traditional repair procedures if required. The load path redundant aspect of this design generally allows multiple MOT options during future repair and rehabilitation projects. Although a typical multi-girder bridge may not be as elegant as the arch structure that exists today, the proposed bridge is designed with aesthetic features such as eye-pleasing span ratios, form liners, and other adornments to compliment the beauty of the site and surroundings.

To facilitate the proposed transverse section, the most economical girder layout was found to be six girders spaced at 13′-8″ center-to-center with 4′-0″ overhangs. The steel girders are comprised of ASTM A709 weathering steel. Grade 50W will be used where possible. However, hybrid sections consisting of a Grade 50W web and Grade 70W flanges will be used where greater moment resistance is required. The use of un-coated weathering steel eliminates the need for future maintenance painting of the structure. The reinforced concrete deck will be made composite with the steel girders and will be 10½″ thick. For structures of this magnitude, a thicker deck on fewer girders is generally more economical, as is the case with this structure.

Reinforced concrete will be used to construct wall abutments and tall hammerhead piers because of its strength, durability, and maintainability. Due to high design loads, the ability to provide lateral resistance, and shallow bedrock, all substructure units will be supported by drilled shaft foundations socketed into bedrock. Utilizing deep foundations at the abutments will likely eliminate whatever sliding/settlement/rotation issues that the existing bridge abutments are experiencing. Preliminary plan dimensions were estimated based on preliminary design, similar projects, and engineering judgement.

Multi-girder bridges are relatively low maintenance. Typical painting, sealing, expansion joint replacement, patching, overlays, etc. are all common repair items which can be performed by numerous contractors. Since the bridge is multiple spans with deep girders, inspection handrails will be provided for inspection access. However, catwalks could also be provided due to the height of the structure.

Generally, the preferred span arrangement is one that minimizes the number of substructure units (i.e. fewer piers with longer spans). However, due to the size of the bridge, the specific site conditions, and unique access constraints, several span configurations were considered as sub-alternatives. They include a 2-span hybrid welded steel plate girder bridge, a 3-span hybrid welded steel plate girder bridge with shorter deck length but taller abutments, and a 4-span hybrid welded steel plate girder bridge. Descriptions of specific features for each sub-alternative are described in the following sections.

Alternative 2A - Two-Span Welded Steel Plate Girder Bridge

General

A preliminary site plan was not developed for this alternative for reasons stated in the constructability section. However, the overall bridge length and the abutment type/sizes are the same as Alternative 2C, which can be found in **Appendix A**. Alternative 2A is a continuous welded two-span hybrid steel plate girder bridge with composite reinforced concrete deck on a reinforced concrete hammerhead pier and reinforced concrete abutments all founded on drilled shafts socketed into bedrock. Span lengths are 400′-0″ and 425′-0″ center-to-center of bearings.

Geometry

The rear (south) abutment is placed to the west and behind the existing rear abutment. By placing it farther up the slope, it minimizes the height of the breast wall and turnback wingwalls. Only a portion of the existing southwest wingwall would need to be removed to facilitate construction of the new abutment. Temporary sheeting is required longitudinally between the existing embankment and proposed southeast wingwall.

Like all other proposed bridge alternatives, the forward (north) abutment is placed to the west and approximately 70′-0″ in front of the existing forward abutment. By placing the abutment on the small plateau that occupies this spot, the northern span is shortened, and the overall bridge length is kept to a minimum. Temporary shoring is not required on this end, and the turnback wingwalls can be constructed in their entirety. A small temporary wire faced mechanically stabilized earth (TWFMSE) wall will occupy the gap between the end of the northeast wingwall and the existing northwest wingwall. Once the existing bridge is removed, then embankment can be placed to slope up the proposed wingwall to bury the TWFMSE wall and meet existing grade.

The single pier is placed on the south side of the river in the relatively flat area at the bottom of the south slope. This has a few major advantages, including not needing to cross the river and provide a TAF for pier construction and not needing to excavate into the steep slope and provide tall excavation bracing. However, TAF will be required later on for girder erection.

Due to the long spans, the steel plate girders are haunched and are hybrid sections, which provides the most economical superstructure section. Span ratios were kept as similar as possible with only a 6% difference.

Economics

Although it will have to carry much higher loads, this design only requires one pier, which will minimize the number of access areas (such as TAFs) and likely save time during construction, also leading to cost savings. As a tradeoff, the pier components and girders are required to be much larger, leading to increased costs. Additionally, the haunched girders makes them more complex (Level 5), which also increases costs.

Constructability

The substructure units for this design will be the easiest to construct. There is ample room around each abutment for construction, and only a small amount of temporary excavation bracing required at the rear abutment. The pier is located outside of the main channel and does not cut into the steep slopes. In contrast, the superstructure will be the most difficult plate girder alternative to construct. The large girder size, girder shape, and bridge height will require huge cranes to have access the valley floor, causing the largest disturbance to the surrounding area. Although pier construction will not require TAFs, TAFs will be required for crane access to set the Span 2 girders. Additionally, the haunched shape of the girders will rule out launching as an erection possibility. Lateral stability issues during erection would need mitigation due to the depth to the thickness ratio of the web. That would require installation of a lateral truss system at the top flange with shoring towers to stabilize the first two lines of girders. Shipping the haunched girder segments to the site is also a primary concern for this alternative. Since this is a two-span bridge, the negative moments over the pier are huge. The girders are bending over a single point and are not able to balance or spread out the moment over multiple intermediate supports. Preliminary depth estimates for a traditional haunched girder design come in at around 25′-6″ over the pier, which rule out shipping that section in one piece. For reference, a typical fabrication production limit is approximately 14′-0″ deep, and ideal shipping limits are approximately 11′-0″ deep. In order to get this alternative to work, the haunched section will need to be split up into multiple pieces for fabrication/shipping and combined with a longitudinal web splice. This would be a highly specialized design and is undesirable.

One option to force a two-span alternative to work is to switch from a haunched girder to a tub girder design. There would need to be at least three tub girder members in order to avoid the nonredundant steel tension member (formerly fracture critical) designation and all the additional requirements that come with that. Additionally, tub girders would be even more complex and costly to design/construct/inspect/maintain. For these reasons, switching to a tub girder design was ruled out.

Another option to force a two-span alternative to work is to switch from a haunched girder to a delta frame design. Two separate design examples of delta frame bridges are the I-90 Innerbelt Bridge in Cleveland, OH, Eastbound (**Figure 5**) and Westbound (**Figure 6**). Delta frames allow for shallower, continuous girders above and supported by the frame portion of the superstructure. However, these bridge types also require a lot of complex additional details. And with the additional detailing, comes additional cost for design/construction/inspection/maintenance. Additionally, a lot of tall temporary shoring would be required to stabilize the delta frames while the girders are erected. For these reasons, switching to a delta frame design was also ruled out.

All options to design a constructable two-span superstructure for this site (haunched girder, tub girder, delta frame) will introduce complex details that will be difficult to fabricate & construct and come at a significantly higher cost. As there are additional alternatives which are significantly easier to construct at a lower cost, Alternative 2A was all but ruled out as a feasible alternative for this site.

Figure 5: I-90 Innerbelt Bridge, Eastbound Girder Configuration

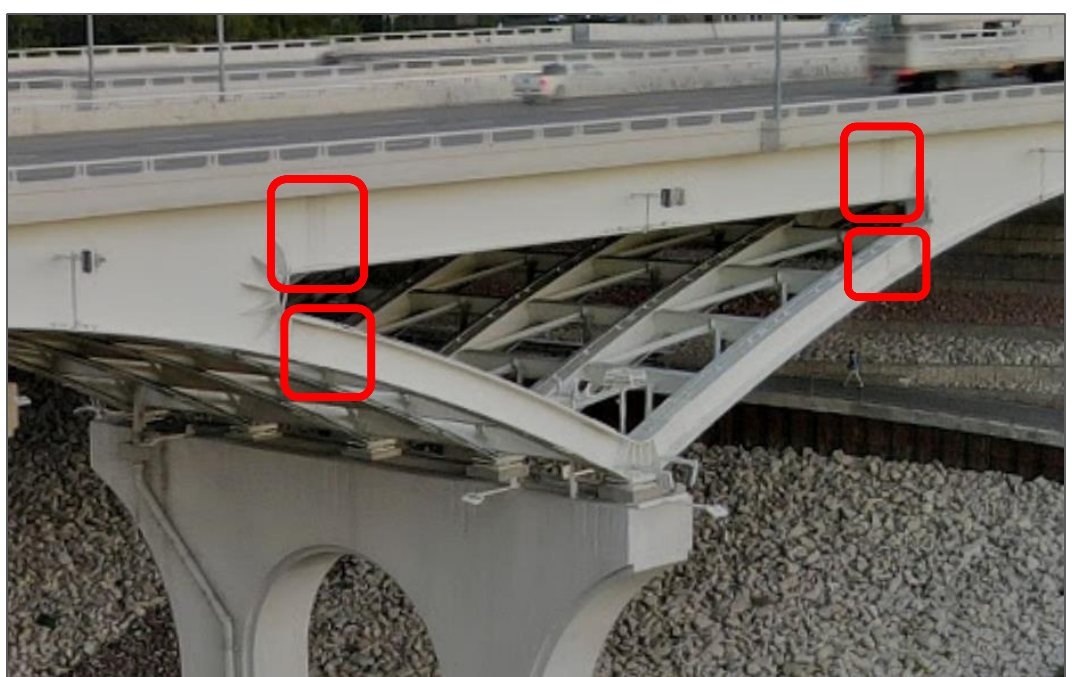

Figure 6: I-90 Innerbelt Bridge, Westbound Girder Configuration (Splice Plates Indicated)

Foundation Considerations

Since the pier is located outside the main channel (as shown in **Figure 7**), river impacts such as scour, debris, and ice flow problems should be minimal.

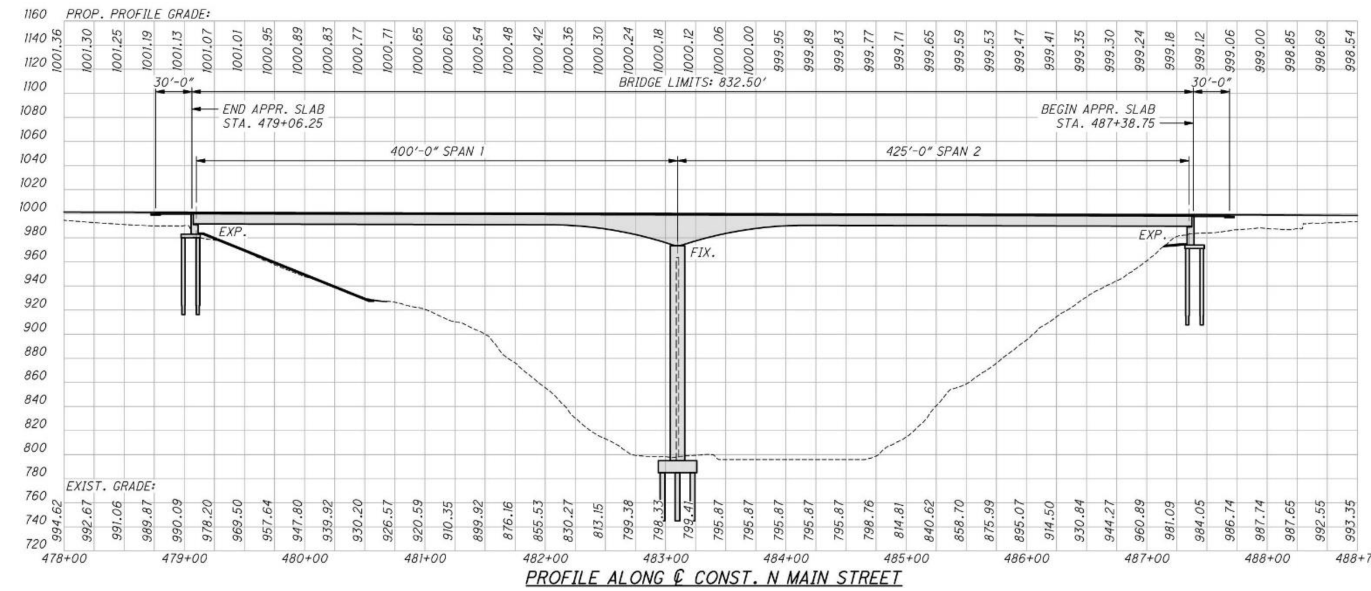

Figure 7: Two-Span Plate Girder Bridge Elevation View

Environmental & Cultural Resources Concerns

Construction of Alternative 2A expands the project footprint by shifting the existing alignment to the west. As such, the potential for additional environmental and cultural resources impacts increase. A shift to the west is more likely to increase both temporary and permanent impacts to recreational Section 4(f)/6(f) resources (parks and trails). Additionally, terrestrial habitat impacts in the form of additional tree clearing will occur. Although the footprint of this alternative maintains piers outside the limits of the Cuyahoga River, impacts to streams and wetlands may occur as a result of construction access requirements, site grading, and the need for a TAF causeway across the Cuyahoga River during construction. In addition to the removal of the existing NRHP eligible historic bridge, cultural resources concerns associated with Alternative 2A again revolve primarily around potential impacts to the NRHP listed Chuckery Race due to its location under the existing bridge structure.

Aesthetics

The nearly symmetrical design will create a sense of balance and equilibrium that is pleasing to the eye. Additionally, the parabolic girder shape can be mimicked with a parabolic hammer head pier design to give a harmonious appearance. Additional aesthetic features such as color, pattern, texture, landscaping, etc. can be considered to further blend the bridge in with the surroundings and provide a unified appearance. Sadly, the additional bracing and detailing required to obtain a constructable design section over the pier will likely take away from some of the elegance of this superstructure alternative.

Alternative 2B - Three-Span Welded Steel Plate Girder Bridge

General

Preliminary plans were developed for this alternative and can be found in **Appendix A**. Alternative 2B is a continuous welded three-span hybrid steel plate girder bridge with composite reinforced concrete deck on reinforced concrete hammerhead piers and reinforced concrete abutments all founded on drilled shafts socketed into bedrock. Span lengths are 250’-0”, 250’-0”, and 250’-0” center-to-center of bearings.

Geometry

The rear (south) abutment is placed to the west and approximately 60’-0” in front of the existing rear abutment. By placing it partly down the slope, it minimizes the length of Span 1, and the overall bridge length is kept as short as possible. One drawback of placing the abutment at this location is that it becomes very tall. For this fill situation, a full-height reinforced concrete abutment with counterforts is proposed to retain the embankment and then turn back to act as wingwalls. The existing southwest wingwall would need to be removed to facilitate construction of the new abutment. Temporary sheeting is required longitudinally between the existing embankment and proposed turnback wingwall. MSE walls are generally economical when in a fill situation and were studied at this location. However, the height required would likely put this abutment wall as one of the tallest in the state. Additionally, the large drilled shaft foundations would pose a large obstruction for the MSE wall soil reinforcement. In the end, the design challenges and concerns about future maintenance, inspection, and aesthetics all combined to sway the decision away from an MSE wall rear abutment to a counterfort rear abutment.

Like all other proposed bridge alternatives, the forward (north) abutment is placed to the west and approximately 70'-0" in front of the existing forward abutment. By placing the abutment on the small plateau that occupies this spot, the northern span is shortened, and the overall bridge length is kept to a minimum. Temporary shoring is not required on this end, and the turnback wingwalls can be constructed in their entirety. A small temporary wire faced mechanically stabilized earth (TWFMSE) wall will occupy the gap between the end of the northeast wingwall and the existing northwest wingwall. Once the existing bridge is removed, then embankment can be placed to slope up the proposed wingwall to bury the TWFMSE wall and meet existing grade.

The piers are placed on each side of the river near the toe of slope. This keeps the piers away from the river and allows for the maximum center span without building halfway up the steep slopes.

Ideal span ratios for a three span bridge are approximately 0.8 : 1 : 0.8. This ratio balances the positive and negative moments and leads to a more efficient design. The geometry of this site did not allow for this ratio without building piers halfway up the steep slopes (lengthening the middle span) or bringing the abutments in even farther down the slopes (shortening the end spans), so a 1:1:1 ratio was kept. This is not the most efficient girder design, but the use of hybrid steel sections is employed to reduce some inefficiencies.

Economics

With three equal spans, the loading on each substructure unit is similar, so the design sizes will be similar. This means similar size equipment can be used, saving on mobilization costs. Materials and labor costs will be more for two tall piers compared to a single pier. However, some associated costs such as mobilization will not increase as significantly from one to two pier design. Using counterforts for the tall rear abutment saves a lot of concrete, but it is more difficult to construct and thus would cost more. In contrast, the girders can be smaller, and they do not have to be haunched (They can be Level 4 – constant depth plate girders vs. Level 5 – haunched plate girders), so they will cost less on a per unit basis.

Constructability

Ease of construction for the forward abutment will essentially be the same as Alternative 2A. Construction for the rear abutment will be more involved since additional excavation, excavation bracing, structure removal will be required. On top of that, the work will be completed on a slope, making access more difficult. Pier construction will also require excavation bracing for construction of the footings at the toe of slopes. TAF will be required to construct the northern pier, but it can be left in place to facilitate the construction of the girders. A benefit of having smaller, constant depth girders over more piers (shorter spans) is that launching the girders could be considered as an alternative method of erection in lieu of large, tall cranes setting girder segments piece by piece. If launching is selected, the girders would not only have to have sufficient strength and serviceability in the final design, but they would also need to be designed for all potential forces during intermediate stages of the launching sequence.

Foundation Considerations

Since the piers are located far outside the main channel adjacent to the toe of slopes, river impacts such as scour, debris, and ice flow problems are minimal.

Environmental & Cultural Resources Concerns

Similar to Alternative 2A, construction of Alternative 2B expands the project footprint by shifting the existing alignment to the west. The difference in span configuration associated with this alternative would likely not result in additional environmental and cultural resources concerns. As such, the potential environmental and cultural resources impacts associated with Alternative 2B, both permanent and temporary, will likely be similar to the other alternatives proposed along a new alignment to the west.

Aesthetics

The bridge is a symmetrical design will create a sense of balance and equilibrium that is pleasing to the eye. Many highly visible bridges (highway overpasses) showcase a three-span design. Since this one is similar, it will provide a sense of strength, stability, and confidence that people depend on in their daily lives. Additional aesthetic features such as color, pattern, texture, landscaping, etc. can be considered to further blend the bridge in with the surroundings and provide a unified appearance. One downside to shifting the rear abutment north to make a smaller bridge footprint is that it becomes very tall and highly visible which will take away from the natural beauty of the surrounding area.

Alternative 2C - Four-Span Welded Steel Plate Girder Bridge

General

Preliminary plans were developed for this alternative and can be found in **Appendix A**. Alternative 2C is a continuous welded four-span hybrid steel plate girder bridge with composite reinforced concrete deck on reinforced concrete hammerhead piers and reinforced concrete abutments all founded on drilled shafts socketed into bedrock. Span lengths are 150'-0", 200'-0", 225'-0" and 250'-0" center-to-center of bearings.

Geometry

Both abutments are the same type and at the same location as Alternative 2A abutments. Both alternatives have the same advantages and disadvantages.

Pier 3 is placed at the toe of north slope to stay out of the river, to avoid construction on the steep slope, and to minimize the length of Span 4. Pier 2 is placed at the toe of the south slope to stay out of the river and to avoid construction on the steep slope. This makes the length of Span 3 90% of Span 4. Pier 1 was added just above the High Bridge Trail where there is a relatively flat area that can be used for access and staging. The length of Span 2 is approximately 89% of Span 3, and that leaves the length of Span 1 to be 75% the length of Span 2.

Overall, the spans become progressively larger from south to north. Span ratios are 0.6 : 0.8 : 0.9 : 1.0. Maximum positive and negative moments are relatively similar through Span 3 since the maximum span length is the same between both alternatives (250'-0"). The negative moment over Pier 3 and the positive moment in Span 4 are slightly larger, but the use of hybrid steel sections is employed to reduce some inefficiencies.

Economics

With smaller spans, the loading on each substructure unit is reduced, but the two tall piers will still need to be quite large to overcome their height alone. This alternative will have the largest substructure cost. On the flip side, these girders will be the smallest of the multi-girder alternatives, and without a haunch, they will also be Level 4 with a smaller unit cost.

Constructability

Constructability of the abutments will be the same as Alternative 2A since they are exactly the same. Constructability of the tall piers will be slightly easier than Alternative 2B since the pier at the toe of south slope is closer to the bottom of slope than it is in Alternative 2B. Pier 1 is considerably shorter than Pier 2 & Pier 3, has easy access from the High Bridge Trail, and should not require any excavation bracing. Smaller girders with more pier supports (shorter spans) will make stick building the girder segments the easiest of the three multi-girder alternatives. Shorter spans allows for smaller pieces to be shipped and handled during erection. There are also more spices made with one end cantilevered over a pier compared to other options where larger girders are spliced in the air while being supported from various cranes.

Like Alternative 2B, launching these girders is also a possibility. Since there is more room and since the spans become progressively larger from south to north, it is recommended that launching will start at the south and progress northward. This will allow a larger back/tail section at the time of pushing out over the largest span, Span 4. Once again, if launching is selected as the preferred method of construction, the girders would not only have to have sufficient strength and serviceability in the final design, but they would also need to be designed for all potential forces during intermediate stages of the launching sequence.

Foundation Considerations

These tall piers are also located outside the main channel and adjacent to the toe of slopes, so river impacts such as scour, debris, and ice flow problems are minimal.

Environmental & Cultural Resources Concerns

Similar to Alternatives 2A and 2B, construction of Alternative 2C expands the project footprint by shifting the existing alignment to the west. The difference in span configuration associated with this alternative would likely not result in additional environmental and cultural resources concerns. As such, the potential environmental and cultural resources impacts associated with Alternative 2C, both permanent and temporary, will likely be similar to the other alternatives proposed along a new alignment to the west. As designed, Pier 1 overlaps with the historical location of the Chuckery Race. If future investigation/coordination with the Summit County Metro Parks necessitates the avoidance/protection of any potential remaining structure in this area, then Pier 1 can be shifted to the north with minimal impact to the design and overall outcome of this alternative.

Aesthetics

Although this bridge design is not symmetrical, the spans becoming progressively larger in one direction will create a visual pattern for observers similar to the concept of perspective drawings in art where objects appear smaller and smaller the closer they are to the vanishing point. This alternative has the most piers with a consistent, repetitive shape, so it has the strongest sense of rhythm. The hammerhead piers could be designed parabolic in shape to mimic a tree canopy rising above the tree line to support the structure. Additional aesthetic features such as color, pattern, texture, landscaping, etc. can be considered to further blend the bridge in with the surroundings and provide a unified appearance.

Alternative 3 - Open Spandrel Redundant Steel Plate Girder Arch Bridge

General

Preliminary plans were developed for this alternative and can be found in **Appendix A**. Alternate 3 consists of a main open spandrel redundant steel plate girder arch span and two adjacent plate girder approach spans with a composite reinforced concrete deck. The proposed bridge will be constructed to the west of the existing bridge offline to maintain traffic during construction. The bridge will be founded on reinforced concrete abutments supported by drilled shafts socketed into rock and reinforced concrete skewbacks supporting the arch span. The span lengths are 219’-9”, 493’-6”, and 111’-0” center-to-center of bearings. The deck will consist of two sidewalks, two bike lanes, and 4 lanes for vehicular traffic with an out-to-out width of 76’-4”.

Geometry

The proposed bridge will have the rear (south) abutment placed approximately 26’-0” south from the existing rear abutment. The rear abutment will be founded on drilled shafts and its location was selected to minimize abutment retained soil height and to eliminate the use of MSE walls. The existing southwest wingwall will be removed during construction of the south abutment. Temporary sheeting will be placed longitudinally and will retain the existing embankment. Span 1 will consist of 6 plate girders as the superstructure elements. The use of plate girders was selected to increase the length of Span 1 to 219’-6” and minimize the arch span length. Span 1 will bear on the rear abutment and a reinforced concrete pier founded on the southern skewback.

Span 2 will be the open spandrel redundant steel plate girder arch consisting of 6 arched plate girders founded on skewbacks located approximately midway on each slope leading to the Cuyahoga River. The arch span will have a length of 493’-6”. The skewbacks will be founded directly on bedrock that is present as shown in the Geotechnical Report. The reinforced composite deck of Span 2 will be supported by rolled beams and rolled columns connecting to the spandrel arch plate girders.

Span 3 will consist of 6 plate girders as the superstructure elements with a composite reinforce concrete deck for a span length of 111’-0”. Span 3 will bear on a reinforced concrete pier founded on the northern skewback and the forward (north) abutment. The forward abutment will have a similar foundation to the rear abutment with a location approximately 92’-0” to the south of the existing bridge abutment, eliminating the need for MSE walls.

Economics

The complexity of an open spandrel redundant steel plate girder arch bridge brings increased cost of materials and construction. This alternate would also have a lower number of contractor bidders, reducing the number of competitive bids.

The steel construction will also have an increased maintenance cost due to periodic superstructure painting and upkeep over other traditional bridge types.

Maintainability

Similarly to the increased cost in economics of Alternate 3, the nature of the steel construction will require multiple iterations of superstructure painting and additional upkeep providing a higher life cycle cost making the maintainability of Alternative 3 will be the highest of the complete replacement alternatives stated in this study. The bridge will also require overlays and redecking throughout its life span.

Constructability

The constructability of the open spandrel redundant steel plate girder arch faces challenges due to the complex structure and limited number of these structures being built. The spans will also be built over the valley on a slope, increasing difficulty. Structures of this complex nature will have limited contractors that can perform the work making Alternate 3 the most complicated of the Alternates listed in this study.

Right-of-Way Constraints

The right-of-way constraints for this alternative will resemble those of Alternates 2A/2B/2C and 4.

Disruption to the Traveling Public

The construction of the Open Spandrel Redundant Steel Plate Girder Arch will occur offline and will not disrupt traffic flow on the existing bridge.

Waterway Crossing Requirements

The construction of the arch skewbacks will occur midway up the slopes leading to the Cuyahoga River and no waterway crossings will be required. A TAF may be required for steel erection.

Foundation Considerations

The forward and rear abutments are located at the top of the slopes and founded on drilled shafts socketed into rock. The configuration and location of the abutments will retain a minimum amount of soil, eliminating the need for tall abutment walls or MSE walls. The skewbacks will be constructed of reinforced concrete directly bearing on shallow rock located on the northern and southern slopes. The excavation for the skewback foundation could potentially interact with the existing bridge piers. Final site-specific geotechnical analysis would be required to fully determine this possibility (and needed excavation limits).

Environmental & Cultural Resources Concerns

Similar to the options considered under Alternative 2, construction of Alternative 3 also expands the project footprint by shifting the existing alignment to the west. The design differences associated with this structure type would likely not result in additional environmental and cultural resources concerns. As such, the potential environmental and cultural resources impacts associated with Alternative 3, both permanent and temporary, will likely be similar to the other alternatives proposed along a new alignment to the west.

Debris and Ice Flow Problems

The span configuration of this open spandrel redundant steel plate girder arch bridge allows for the skewback supports to be midway on the slope of each bank of the Cuyahoga River removing the risk of debris and ice flow problems.

Aesthetics

The spandrel arch bridge will provide an aesthetically pleasing structure that is symmetrically spanned over the Cuyahoga River with a gradual radius of the arch. The Highbridge Trail will pass under Span 1 and would allow for a scenic view of the unique spandrel arch span completely spanning the river and not encroaching on natural scenery of the valley. Unfortunately, the excessive amount of bracing required may take away some of the elegance and beauty of the structure. Additional aesthetic features such as color, pattern, texture, landscaping, etc. can be considered to further blend the bridge in with the surroundings and provide a unified appearance.

Alternative 4 – Post Tensioned Segmental Concrete Box Girder Bridge

General

Preliminary plans were developed for this alternative and can be found in **Appendix A**. Alternate 4 consists of a 2-span post tensioned segmental cast-in-place concrete box girder bridge that will be constructed to the west of the existing bridge offline to maintain traffic during construction. The bridge will be founded on reinforced concrete abutments supported on drilled shafts socketed into rock and a single reinforced concrete tapered pier founded on drilled shafts socketed into rock. The span lengths are 364’-1” and 457’-6” center-to-center of bearings. The deck will consist of two sidewalks, two bike lanes, and 4 lanes for vehicular traffic with an out-to-out width of 76’-4”.

Geometry

The Alternate 4 proposed bridge will have the rear (south) abutment placed approximately 26’-0” south from the existing rear abutment. The rear abutment will be founded on drilled shafts socketed into rock and its location was selected to minimize abutment retained soil height and to eliminate the use MSE walls. Span 1 will consist of a 364’-1” span continuous, three-web concrete box girder that utilizes a parabolic haunch between the piers and mid-span regions. The superstructure depth will increase as it spans out from the abutment to the pier. The three-web arrangement, which results in a single, two-cell box girder, is used due to the width of the top slab and is typical for a 76’-4” width structure.

A single pier will be located on the south bank of the Cuyahoga River with an asymmetrical span configuration between Span 1 and Span 2. The pier will be reinforced concrete with a solid column transitioning to a basic tapered twin wall system integral to the segmental box girder sections. The span will bear on the rear abutment and the pier. The pier will be supported on drilled shafts socketed into rock with proposed grading sloped towards the river.

Span 2 will have a length of 457’-6” and will have a similar segmental concrete box girder built up as Span 1. Span 2 will have a decreasing structural depth tapering from the pier to the forward abutment. The span will bear on the pier and the forward abutment.

Economics

The post tensioned segmental cast-in-place concrete box girder bridge will have an increased cost over traditional bridge types as it will have a unique construction with added complexities of post tensioning the segments with construction using a cast-in-place counterbalance method. Structures of this complex nature will have limited contractors that can perform the work making Alternative 4 one of the more complicated alternatives listed in this study. The maintenance of the structure is reduced compared to the other alternatives as laid out in Maintainability.

Maintainability

Maintenance of the concrete structure should not include anything except the periodic replacement of the Latex Modified Concrete (LMC) overlay, expansion joint maintenance, and possibly maintenance of the expansion bearings at the abutments. The industry standard use of plastic post-tensioning duct, coupled at the joint locations, as well as high quality grout means that the post-tensioning system should not require future supplementation or rehabilitation. Only a few bridges, nationwide, have ever had to have supplemental tendons added. Also, the post-tensioned concrete stress limits in the LRFD code are proven over many years of work throughout the world. The twin-walled piers are proportioned such that they are robust for this type of construction, as well.

Constructability

The structure will be erected using cast-in-place balanced cantilever method from the pier. Longitudinal cantilever post-tensioning tendons in the top slab carry the negative moments induced during cantilever erection and are sufficient to resist future negative bending. After two adjacent cantilevers are completed, they are joined, and longitudinal bottom slab continuity tendons are installed to resist positive bending. Similarly, near the abutments, falsework is used to support the ends of the bridges, and once the adjacent cantilever is complete and joined to the falsework section, bottom slab tendons are installed. Transverse post-tensioning in the top slab compresses the deck and resists transverse bending.

Right-of-Way Constraints

The right-of-way constraints for this alternative will resemble those of Alternates 2A/2B/2C and 3.

Disruption to the Traveling Public

The construction of the post tensioned segmental cast-in-place concrete box girder bridge will occur offline and will not disrupt traffic flow on the existing bridge.

Waterway Crossing Requirements

The construction of the pier will require work near the Cuyahoga River but will not require waterway crossings.

Foundation Considerations

The forward and rear abutments are located at the top of the slopes and founded on drilled shafts socketed into rock. A single tall pier is located outside the main channel and adjacent to the toe of slopes founded on drilled shafts socketed into shallow rock.

Environmental & Cultural Resources Concerns

Similar to the options considered under Alternative 2 and Alternative 3, construction of Alternative 4 also expands the project footprint by shifting the existing alignment to the west. The design differences associated with this structure type would likely not result in additional environmental and cultural resources concerns. As such, the potential environmental and cultural resources impacts associated with Alternative 4, both permanent and temporary, will likely be similar to the other alternatives proposed along a new alignment to the west.

Debris and Ice Flow Problems

The span configuration of this Post Tensioned Segmental Concrete Box Girder Bridge’s single pier is placed at the toe of slope of the south bank of the Cuyahoga River with a proposed graded slope into the river, reducing the risk of debris and ice flow problems.

Aesthetics

Although this bridge is not symmetrical, the single pier design reduces the impact to the surrounding scenic area. Additional aesthetic features such as color, pattern, texture, landscaping, etc. can be considered to further blend the bridge in with the surroundings and provide a unified appearance.

Existing Truss Demolition

Controlled demolition using explosives is the safest and most economical method to dismantle the existing truss. Prior to demolition, the existing deck will be removed, and temporary structure protection systems will be placed to shield the new bridge and exposed utilities from damage. Careful monitoring for excessive vibration and/or damage will be needed for the NSIT tunnel and other infrastructure in the vicinity. Once dropped, its members can be cut and hauled offsite using the access road prior to final site restoration.

The impact area associated with necessary demolition efforts will be evaluated as part of the future project ecological investigations and considered throughout the development of the freshwater mussel survey workplan. Efforts will be made to remove all construction demolition debris from the Cuyahoga River in accordance with all applicable waterway permit Special Provisions.

Cost Analysis

Initial construction cost estimates and future construction cost estimates were developed and compared for each alternative. The results are summarized in **Table 1** & **Table 2** and **Figure 8** & **Figure 9**. Initial cost estimate calculations are contained in **Appendix E** while future project cost estimate calculations are contained in **Appendix F**. Bridge inspection and maintenance cost estimate calculations are contained in **Appendix G**. All costs are for construction and/or Right-of-Way only. Engineering design and construction inspection costs were not included in this analysis.

Initial costs

General

For the bridge and roadway costs, a 30% contingency was included at this stage of the design to cover the cost of work not specifically itemized. Additionally, all bridge replacement costs were developed in 2024 dollars and inflated by 33% to 2031 dollars, which is based on a 2030 construction start date and a two-year construction duration. Existing bridge rehabilitation costs were based on the prior 2017 Life Cycle Cost Analysis report with adjustments for 30% contingency and inflation to 2031 dollars. A comparison of initial costs is shown in **Table 1** & **Figure 8**, and initial cost estimate calculations are contained in **Appendix E**.

Roadway

The roadway construction costs for major cost items were calculated based on the preliminary plans and calculations developed for this study. Roadway costs include roadway, erosion control, pavement/haul road, and incidental items. Allowances were estimated for other work such as drainage, lighting, traffic control, landscaping, maintenance of traffic, etc. For the new construction alternatives except Alternative 2B, the roadway costs are assumed to be the same. For Alternative 2B, an extra $162,000 in additional roadway costs were added since that alternative has approximately 75 feet more roadway to make up for the 75-foot shorter bridge length. Roadway and MOT costs associated with existing bridge rehabilitation were based on the prior 2017 Life Cycle Cost Analysis report.

Right-of-Way

Alternatives 2-4 will likely require some right-of-way takes, so an allowance of $2,500,000 was assumed and included in the initial costs. Rehabilitation of the existing bridge does not require right-of-way takes.

Bridge

Like the roadway costs, the bridge construction costs for major cost items were also calculated based on the preliminary plans and calculations developed for this study as well as engineering judgement and comparison between similar projects. Major bridge items for the new construction alternatives included things such as demolition of the existing bridge, excavation, drilled shafts, concrete, steel reinforcing, steel members, bearings, expansion joints, fence, etc. where applicable. Smaller cost items such as sealing, preformed expansion joint filler, drainage items, etc. were not included and assumed to be covered in the contingency. Existing bridge rehabilitation costs were based on the prior 2017 Life Cycle Cost Analysis report and engineering judgement. They included structure/drainage/erosion repairs, overlay of the existing deck, and a full repainting of the structural steel.

Analysis

As shown, the initial cost for rehabilitating the existing bridge is the cheapest with the cost ranging from approximately 54% to approximately 61% of a brand-new structure. The upfront cost to maintain an existing bridge and extend its useful lifespan will almost always be cheaper than the upfront cost of constructing a new bridge. However, you would be putting in over half the amount of money it would take to construct a new bridge just to maintain the use of an older and risk-prone structure.

The plate girder alternatives are the least costly of the new build alternatives since they are more traditional and common structure types with easier methods of construction. Amongst the plate girder alternatives, the 4-span alternative is the cheapest. This is primarily because the additional pier makes it the easiest superstructure to construct, and its rear abutment is much shorter & easier to access at the tops of the slopes. The 3-span alternative has a 75 feet shorter bridge deck and one less (relatively short) pier, but savings are not enough to overcome the increased cost of the tall rear abutment and more difficult steel erection.

The concrete box alternative follows the plate girder alternatives in initial cost. It has a single tall and massive pier that will need to support long cantilever spans during construction. It’s superstructure is also very specialized and time intensive to construct. Both of these components drive up the initial cost. The steel arch alternative is the most expensive alternative. It has the advantage of eliminating the tall piers altogether, which are costly. However, the initial construction of the superstructure will be much more difficult, requiring specialty contractors and a lot of temporary supports. This renders it the costliest overall.

Future Costs

General

All infrastructure, including bridges, require regular inspection & maintenance as well as occasional repair/rehabilitation projects to continue to perform their intended function as safely and efficiently as possible. Some structures require more future work than others, which is why it is important to take future costs into account when designing a structure for a specific site. For this study, future costs for the duration of each alternative’s assumed service life were developed. Like the initial costs, a 30% contingency was included at this stage of the design to cover the cost of work not specifically itemized.

The overall service life of each alternative is assumed to be the same for each structure (100 years). However, each alternative requires different amount of work throughout that life span, which will vary in cost. To account for the cost differences, the net present value (NPV) or present worth of each alternative is compared. The NPV is calculated based on an annual discount rate of 2.5%. This value is from the December 2023 Office of Management and Budget (OMB) Circular No. A-94, Appendix C, Real Discount Rate, which is the latest available information. Using this discount rate, all life cycle costs are converted to 2031-dollar values (midpoint of assumed construction) so that a fair comparison can be made between alternatives. A comparison of future costs is shown in **Table 2** & **Figure 9**. Future project cost estimate calculations are contained in **Appendix F**, and bridge inspection and maintenance cost estimate calculations are contained in **Appendix G**.

Initial Cost Comparison					
Alternative	**Description**	**Bridge Cost**	**Right-of-Way Costs**	**Roadway Costs**	**Total Initial Costs**
1	Rehabilitate Existing Bridge	$41,733,400	$0	$1,964,000	**$43,697,400**
2B	Three-Span Welded Steel Plate Girder Bridge	$61,626,700	$2,500,000	$8,057,700	**$72,184,400**
2C	Four-Span Welded Steel Plate Girder Bridge	$61,076,800	$2,500,000	$7,895,700	**$71,472,500**
3	Open Spandrel Redundant Steel Plate Girder Arch Bridge	$70,113,100	$2,500,000	$7,895,700	**$80,508,800**
4	Post Tensioned Segmental Concrete Box Girder Bridge	$69,069,900	$2,500,000	$7,895,700	**$79,465,600**

Note: Costs are based on values for construction beginning in 2030 with a 30% contingency applied.

Table 1: Initial Cost Comparison

Future Cost Comparison				
Alternative	**Description**	**Future Work Costs**	**Annual Insp. & Maint. Costs**	**Total Future Costs**
1	Rehabilitate Existing Bridge	$23,506,100	$5,652,800	**$29,158,900**
2B	Three-Span Welded Steel Plate Girder Bridge	$10,961,200	$1,794,600	**$12,755,800**
2C	Four-Span Welded Steel Plate Girder Bridge	$11,233,400	$1,794,600	**$13,028,000**
3	Open Spandrel Redundant Steel Plate Girder Arch Bridge	$17,328,000	$2,110,500	**$19,438,500**
4	Post Tensioned Segmental Concrete Box Girder Bridge	$10,633,400	$1,882,500	**$12,515,900**

Note: Costs are based on values for construction beginning in 2030 with a 30% contingency applied.

Table 2: Future Cost Comparison

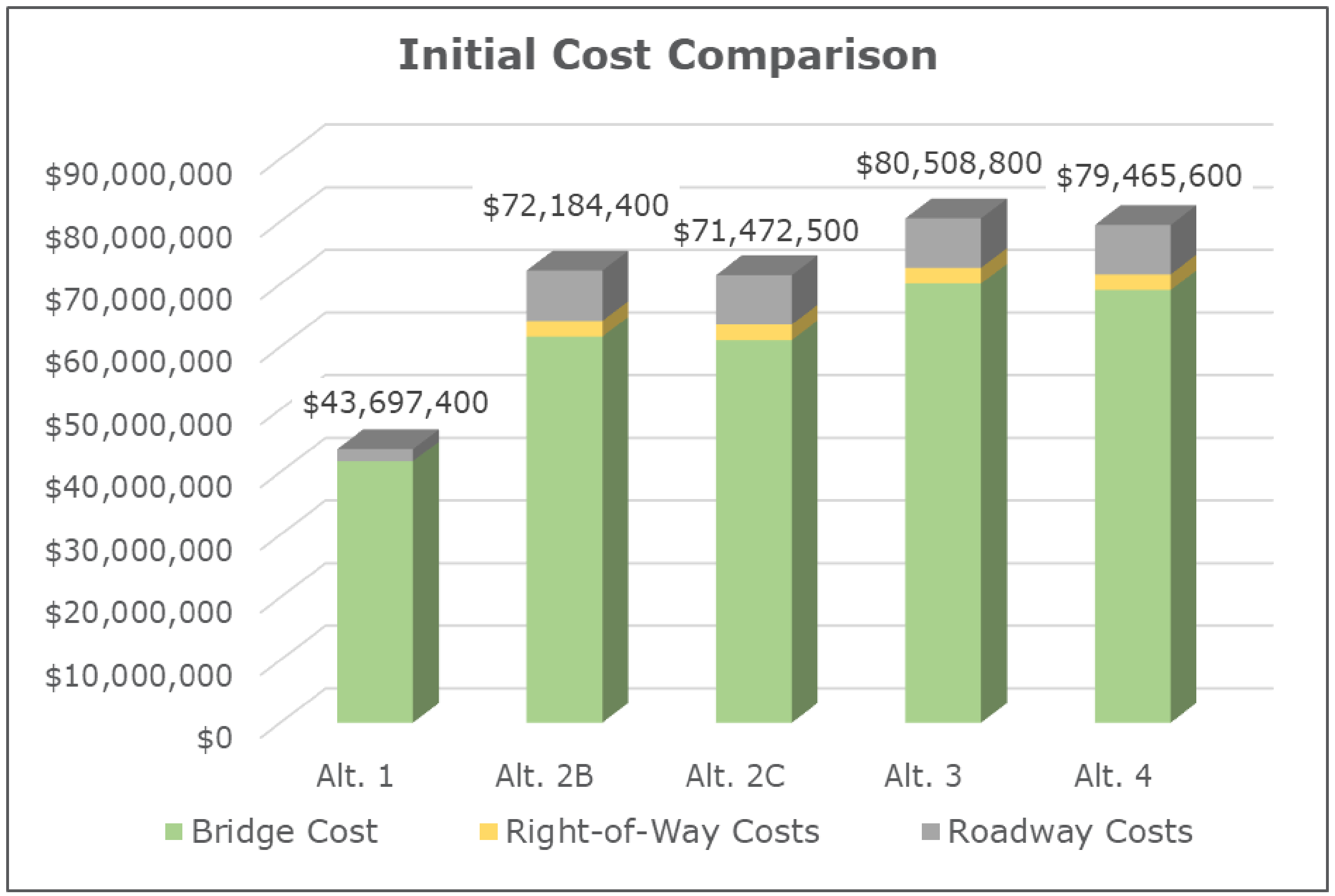

Figure 8: Initial Cost Comparison

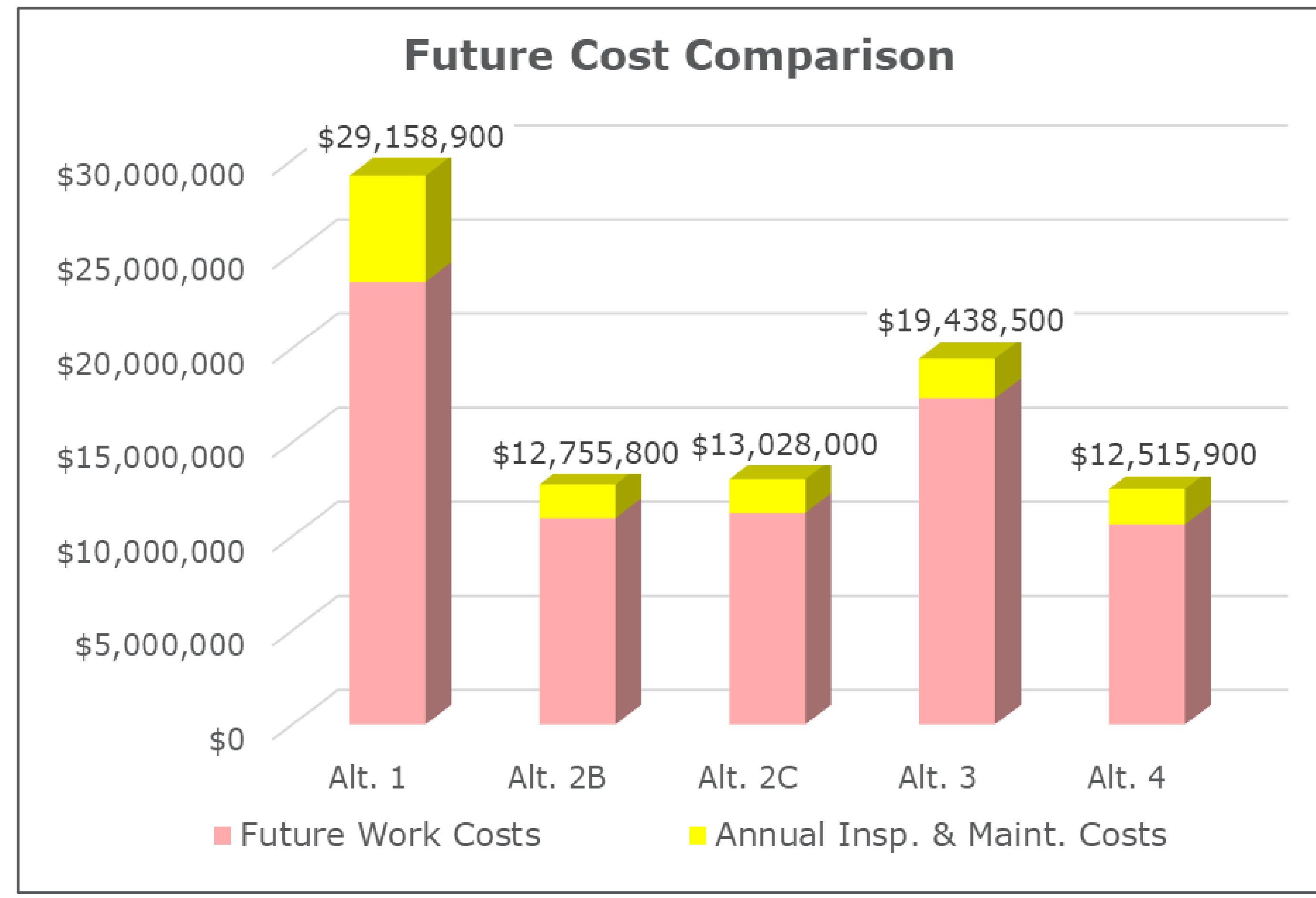

Figure 9: Future Cost Comparison

Future Rehabilitation & Repair Costs

For this study, it is assumed that either the existing bridge will undergo a major rehabilitation (Alternative 1) or a new bridge will be constructed (Alternatives 2-4) beginning in the year 2030 and opening in 2032 (year 0). For each alternative, it is assumed that the structures will last approximately 100 years before complete replacement is necessary. Bridge decks are assumed to have a 50-year service life (with proper maintenance), overlays are assumed to have a 15–20-year service life, and deck patching/re-sealing is assumed to have a 15-year service life. It is important to note that these are estimated service lives, and repair/rehabilitation/replacement timelines can be adjusted depending on actual levels of deterioration over time.

The following repair/rehabilitation events are assumed for Alternative 1:

- Year 2032 (Year 0) Structure repairs, replace deck, repaint (FYI; Not included in future work.)
- Year 2052 (Year 20) Structure repairs, seal deck
- Year 2067 (Year 35) Structure repairs, overlay deck
- Year 2082 (Year 50) Structure repairs, replace deck, repaint
- Year 2102 (Year 70) Structure repairs, seal deck
- Year 2117 (Year 85) Structure repairs, overlay deck
- Year 2132 (Year 100) Replace bridge

As previously mentioned, with respect to the above rehabilitation timeline and activities, GPD also performed a cursory analysis for replacement of the existing structure at Year 50. When accounting for the residual value of the replacement bridge at Year 100, there was little difference in total costs over 100 years. Thus, usage of a 100-year life cycle model for Alternative 1 is suitable for the purpose direct comparison with other complete replacement alternatives.

The following repair/rehabilitation events are assumed for Alternatives 2-4:

- Year 2032 (Year 0) Initial construction (FYI; Not included in future work.)
- Year 2052 (Year 20) Seal deck
- Year 2067 (Year 35) Deck overlay
- Year 2082 (Year 50) Replace deck, repaint
- Year 2102 (Year 70) Seal deck
- Year 2117 (Year 85) Deck overlay
- Year 2132 (Year 100) Replace bridge

Associated non-bridge related items such as roadway, MOT, incidental, etc. items were included in the future work project costs as deemed necessary.

Annual Inspection and Maintenance Costs

Annual inspection and maintenance costs were estimated for each alternative, as they also varied with each structure type. They were based on actual costs incurred by the county to inspect and maintain the existing bridge between 2006 and 2015. The costs were then projected to present values (Year 2024), and the totals were inflated to 2031 values for comparison and consistency with initial costs. Annual inspection and maintenance costs were assumed to be less for a new structure when compared to the existing structure. For the existing bridge inspections, a nonredundant steel tension member (NSTM) [formerly fracture critical (FC)] inspection was specified every other year beginning in 2026 while routine inspections will be performed on years that do not feature a FC inspection. Additionally, in-depth inspections and pin testing were specified every fourth year to coincide with every other FC inspection. For the new bridge alternatives, routine inspections were specified every year with in-depth inspections performed every 5 years, beginning in 2036. The values for bridge inspection costs were varied based on the complexity and number of bridge elements in each alternative. Routine maintenance costs include lubricating bearings & pins (for the existing bridge), pressure washing critical components, and expansion joint cleaning as well as the following on an as-needed basis: deck patching, railing repairs, adjacent road & guardrail repairs, security fence repairs, etc. Routine maintenance costs were assumed to be the same for all new bridge alternatives. Annual inspection and maintenance cost estimate calculations are contained in **Appendix G**.

Analysis

As expected, the cost to inspect and maintain a complex and aging structure is much greater than a new, simpler one and maintaining it over time. Although the initial costs are less, higher structure repair costs coupled with the higher inspection and annual maintenance costs puts Alternative 1 future costs substantially higher than future costs of the other alternatives.

Future repair/rehabilitation projects are primarily focused on the deck elements for the new bridge alternatives. With relatively similar deck areas, most of the future project costs are similar amongst the new bridge alternatives. The exception to this is Alternative 2B since it has a slightly smaller deck area, so its future work costs are slightly lower. Alternative 3 assumes painting the entire steel structure will occur with the deck replacement in Year 2082 (Year 50), which is costly and drives up the future work costs relative to the other alternatives. Additionally, the new bridge alternatives have the same annual maintenance costs and similar inspection costs.

Setting aside Alternative 1, the concrete box girder alternative is the cheapest new bridge option, the steel arch alternative is the most expensive, and the plate girder alternatives fall between the two.

Total Costs

General

Total costs, often called lifecycle costs, represent the total cost to construct and maintain each alternative for the duration of its service life. The lifecycle costs take into account the initial cost of construction, annual inspection & maintenance costs, and major future repair & rehabilitation costs. When the initial cost totals from **Table 1** & **Figure 8** are combined with the future cost totals from **Table 2** & **Figure 9**, the true total costs of bridge ownership are revealed. These totals are shown in **Table 3** & **Figure 10**.

As the data shows, maintaining a complex and aging structure over the course of a 100-year service life is the most cost-effective alternative, followed closely by constructing and maintaining a new plate girder structure within the same timeframe. Steel arch and concrete box structures will cost more.

Total Cost Comparison				
Alternative	**Description**	**Total Initial Costs**	**Total Future Costs**	**Total Costs**
1	Rehabilitate Existing Bridge	$43,697,400	$29,158,900	**$72,856,300**
2B	Three-Span Welded Steel Plate Girder Bridge	$72,184,400	$12,755,800	**$84,940,200**
2C	Four-Span Welded Steel Plate Girder Bridge	$71,472,500	$13,028,000	**$84,500,500**
3	Open Spandrel Redundant Steel Plate Girder Arch Bridge	$80,508,800	$19,438,500	**$99,947,300**
4	Post Tensioned Segmental Concrete Box Girder Bridge	$79,465,600	$12,515,900	**$91,981,500**

Note: Costs are based on values for construction beginning in 2030 with a 30% contingency applied.

Table 3: Total Cost Comparison

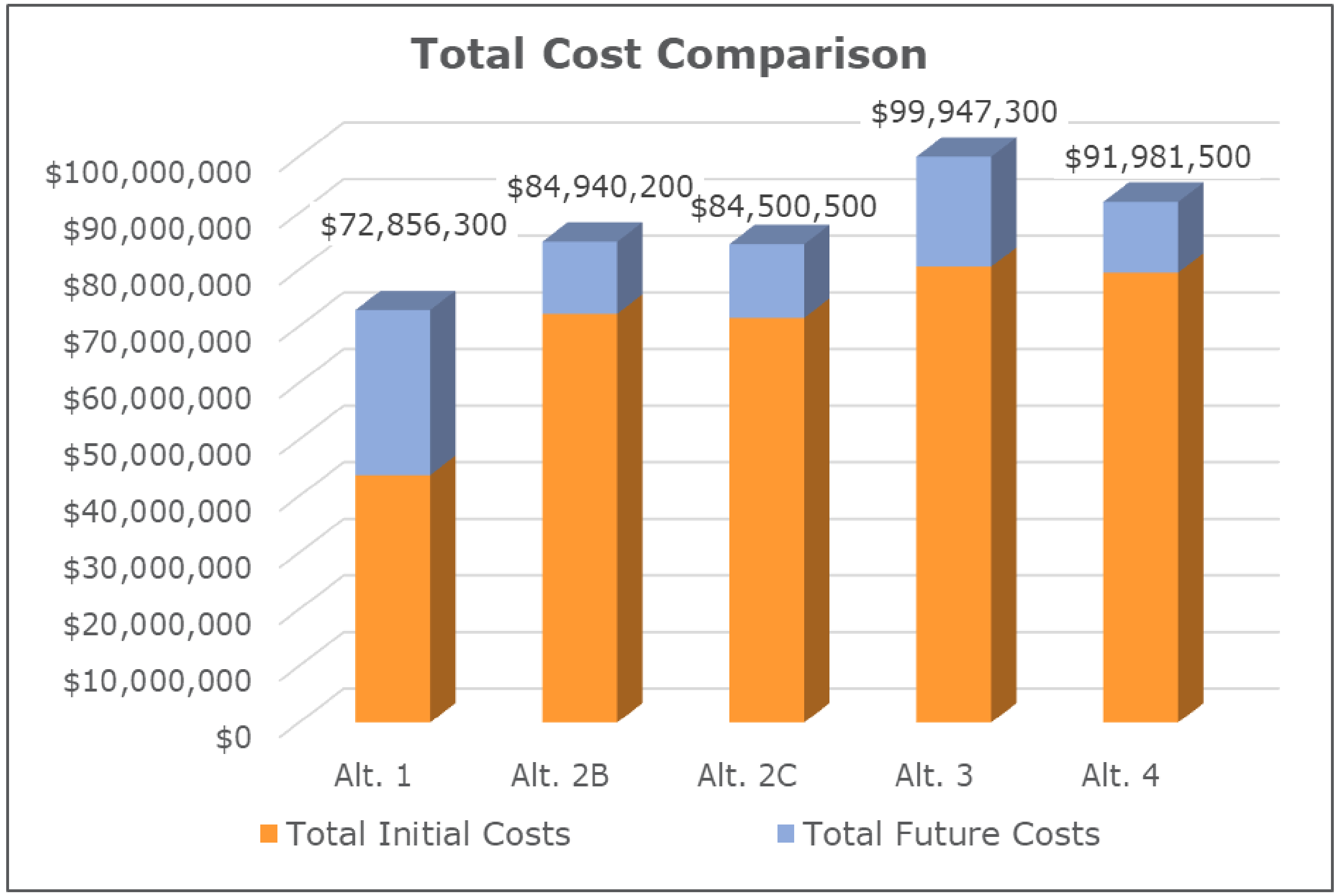

Figure 10: Total Cost Comparison

Conclusions and Recommendations

Alternative Matrix

A project of this magnitude has many factors to consider when attempting to select a preferred alternative, and some of those factors can be more important than others. An alternative comparison matrix is a decision-making tool used to help select the best option among alternatives. The matrix works by comparing advantages and disadvantages of each alternative and assigning points (base scores) to the categories for each alternative based on their rank. The base scores for the matrix range from 1 (worst/highest cost) to 10 (best/lowest cost) for each category. For a given category, each alternative score is then factored by a weighted multiplier to emphasize the advantages and disadvantages of each alternative relative to each other category. All the factored category scores are then summed to arrive at the matrix decision score for each alternative. The alternative with the maximum matrix decision score is chosen as the preferred alternative. When the sum of the multipliers is equal to ten, then a perfect score is equal to 100, as is the case here. The alternative matrix for this project is shown on the following page in **Figure 11**.

Initial Cost

Funding a project of this magnitude is very difficult with the recent and unprecedented inflation, so keeping the initial cost to a minimum is an important factor when selecting a preferred alternative. For this reason, the multiplier for the initial cost category was tied with the safety category for the highest at 2.0.

Alternative 1 has the lowest initial cost, so it was given a base score of 10. Alternative 2B and Alternative 2C had relatively similar initial costs, but there was a gap between Alternative 1 and Alternatives 2B/2C, so they were both given 8s. For Alternative 4, the balanced cantilever construction with form traveller system is an ideal application at this site. However, the cost savings would not be fully achievable because this project is a single, relatively short bridge with two spans, and the amount of repetition utilized by the form traveller is minimal. This makes the post tensioned segmental concrete alternative less cost competitive than the steel girder alternative, Alternative 2. Alternative 3 had relatively similar initial costs to Alternative 4, so they were both given base scores of 6.

Constructability and MOT

Ensuring that a project is constructable with minimal disturbance to the traveling public for the duration of construction is also an important factor to consider, so the Constructability and MOT category was given a relatively high multiplier of 1.8.

More detailed descriptions of the constructability and impact to traffic can be found in the previous report sections, but a generalized summary is provided below for ranking justification. Alternative 1 is the least amount of work and would receive a base score of 10 for constructability, but it is not built offline like the new build alternatives. It will require reduced traffic lanes for the overlay portion of the initial construction and could require a temporary full closure depending on the location of structure repairs that are required (initially, or in future rehabilitation projects). For this reason, several points were deducted to arrive at a base score of 7.

Alternative 2C is the easiest to construct since it is a common structure type with short abutments and the most piers, creating the shortest spans. This allows the smallest girders and multiple possible erection methods/ sequences. And like the other new build alternatives, it is built offline, so MOT impacts are minimal. It received a base score of 10. Alternative 2B is similar to 2C, but it receives a slightly lower base score of 9 because it requires a tall, counterfort rear abutment, and the longer spans will make steel erection slightly more difficult.

Alternative 4 received a base score of 8 since it will have extremely long and unequal cantilevered spans until reaching the abutments. Utilizing the cast in place balanced cantilever construction method is the most efficient way to construct the bridge over the valley where the access is limited, but this requires a specialty contractor (there is likely only one local to northeast Ohio that can perform this type of work). MOT impacts are as minimal as Alternatives 2B & 2C.

Alternative 3 will also require a specialty contractor and very tall temporary towers for support of the arch during initial construction. MOT impacts are also minimal for this offline build, and this alternative received a base score of 7.

Future Costs, Bridge Inspection, and Maintenance

Future work and associated costs are also critically important to consider during this phase of development, so this category was given a multiplier of 1.7.

Alternatives 2B, 2C, and 4 have the lowest future costs, which are almost identical. Alternative 2B and Alternative 2C were both given a base score of 10. Alternative 4's inspection of the box interior will require confined space training, so it was given a base score of 9. Alternative 3 was given a base score of 8, which was primarily cost driven. Alternative 1 has an extremely high future cost and requires non-redundant steel tension member (NSTM, formerly fracture critical) inspections every two years, so it's base score is 6.

Alternative	Description	Initial Cost		Constructability and MOT		Future Costs, Bridge Inspection, and Maintenance		Environmental		Safety		Aesthetics		Decision Score (Perfect Score = 100)	Decision Score Rank
		Factor Considered													
		Multiplier													
		2		1.8		1.7		1.5		2		1			
		Score													
		Base Score	w/ Multiplier	Base Score	w/ Multiplier	Base Score	w/ Multiplier	Base Score	w/ Multiplier	Base Score	w/ Multiplier	Base Score	w/ Multiplier		
1	**Rehabilitate Existing Bridge**	10	20.0	7	12.6	6	10.2	10	15.0	2	4.0	10	10.0	**71.8**	**5**
2B	**Three-Span Welded Steel Plate Girder Bridge**	8	16.0	9	16.2	10	17.0	8	12.0	10	20.0	8	8.0	**89.2**	**1**
2C	**Four-Span Welded Steel Plate Girder Bridge**	8	16.0	10	18.0	10	17.0	5	7.5	10	20.0	9	9.0	**87.5**	**2**
3	**Open Spandrel Redundant Steel Plate Girder Arch Bridge**	6	12.0	7	12.6	8	13.6	9	13.5	10	20.0	6	6.0	**77.7**	**4**
4	**Post Tensioned Segmental Concrete Box Girder Bridge**	6	12.0	8	14.4	9	15.3	8	12.0	10	20.0	7	7.0	**80.7**	**3**

Note: Base scores vary from 1 (worst/highest cost) to 10 (best/lowest cost).

Figure 11: Alternative Matrix

Environmental

Impact to the environment and surrounding residents is important to consider in alternative selection, so it received a multiplier of 1.5.

More detailed descriptions of potential environmental concerns associated with each alternative can be found in the previous report sections, but in general, salvaging the existing bridge eliminates a huge burden to the site, so it receives a base score of 10. All the new build alternatives not only impact the footprint of the new bridge, but also the access path down to the valley floor. For this reason, they all receive base scores of 8 except Alternative 3, which receives a base score of 9 since its piers are halfway up the slopes and far away from impacting the river, and Alternative 2C, which receives a base score of 5 since the southern pier will either disturb or need to be designed around the historic Chuckery Race.

Safety

Safety of the traveling public is of primary importance and is tied with initial cost at a 2.0 multiplier.

All new build alternatives receive a 10 base score while the existing bridge alternative receives a base score of 2. **No matter how many repairs and retrofits are performed, the fracture critical nature of the existing bridge will never be removed.** Additionally, there have been no known remaining fatigue life studies, it is approaching the end of its 75-year design life, it has a heavier deck than it was designed to carry, it has already had two major repair/retrofit projects performed on it. All of these factors combine to arrive at such a low base score.

Aesthetics

Aesthetics is inherently somewhat subjective, so it was given the lowest multiplier of 1.0.

The existing bridge shape is elegant and symmetrical across the gorge. Additionally, salvaging it rather than building a new bridge will eliminate disruption and tree clearing at the site, retaining the natural beauty of the area. For these reasons, Alternative 1 was given a base score of 10. While the plate girder bridges are of more typical construction and not as elegant as the arched-shape truss, they are familiar to passersby providing them with confidence in their strength. Alternative 2B has symmetrical span lengths, while Alternative 2C has progressively larger span lengths, which gives it a sense of pattern. Alternative 2C was given a slightly higher base score of 9 over Alternative 2B's 8 since 2B has the tall southern abutment which will stick out and detract from the surrounding beauty of the area. Alternative 4 was given a base score of 7 since it is nearly symmetrical and reduces the impact to the surrounding area with just one pier. Alternative 3 scored the lowest base score of the bunch with a 6. Although the center span arch shape most closely resembles the existing, the end plate girder spans are unsymmetrical and appear as an afterthought. Additionally, looking at the bridge at any angle other than head-on will reveal the exorbitant amount of bracing between the redundant arch lines and spandrel columns. This, and the fact that it is all weathering steel will detract from its overall appearance, hence the lower aesthetic ranking.

Decision Score

As shown, Alternative 2B received the highest decision score of 89.2 and ranks number 1 in the alternative matrix. This is followed closely by Alternative 2C at 87.5, which makes sense since they are similar structures. After that, there is a relatively larger gap before Alternative 4 (80.7) and Alternative 3 (77.7) respectively. Another relatively larger gap follows before arriving at Alternative 1's bottom ranking decision score of 71.8. Despite scoring the highest base score of 10 in three of the six categories, Alternative 1 still came last in the rankings. This highlights the importance of considering all factors when choosing a preferred alternative from a group. Some factors within an alternative matrix could be considered subjective. However, the decision score results in this study were spread out enough that any small multiplier or base score modifications are not likely to swing the decision score in any meaningful way as to change the overall alternative rankings.

Conclusion

Alternative 2B, the Three-Span Welded Steel Plate Girder Bridge, ranked the highest in the alternative matrix and is recommended as the preferred structure alternative for the bridge carrying North Main Street over the Cuyahoga River. This structure type stood out amongst the others since it was essentially tied for the lowest initial & total (lifecycle) cost out of the new bridge alternatives. It is also deemed as one of the most constructable new bridge alternatives with little impact to MOT and the third most aesthetic. Substructure locations were chosen to be easily accessible and out of the river so that they cause as little impact as possible to the surrounding site and adjacent historic Chuckery Race.

Appendix A

Structure Alternative Plans

Appendix B

Roadway Plans

Appendix C

Foundation Recommendation Memorandum

Appendix D

Environmental & Cultural Resources Red Flag Summary Report

Appendix E

Initial Cost Estimates

Appendix F

Future Cost Estimates

Appendix G

Bridge Inspection and Maintenance Cost Estimates

Appendix H

2023 Routine Bridge Inspection Report