US-70 Over Lake Texoma (Franklin D. Roosevelt Memorial Bridge) Section 4(f) Alternatives Analysis & Report - Revision 2

Oklahoma Department of Transportation
JP 33873(07)
Bryan and Marshall Counties

Prepared by:

GARVER

6100 S. Yale Avenue
Suite 1300
Tulsa, OK 74136

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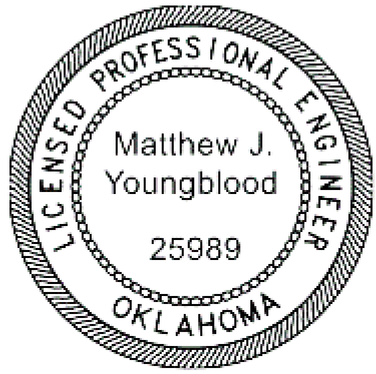
Garver Project No.: 20T03060

Engineer's Certification

I hereby certify that this Section 4(f) Analysis and Report for the bridge on US-70 Over Lake Texoma (Franklin D. Roosevelt Memorial Bridge) was prepared by Garver under my direct supervision for the Oklahoma Department of Transportation and reviewed by a licensed Professional Engineer in the State of Oklahoma.



Matthew Youngblood, PE
State of Oklahoma PE License 25989

Garver
Oklahoma Registration No. F-4193

Executive Summary

The Oklahoma Department of Transportation (ODOT) tasked Garver with studying the historic Franklin D. Roosevelt Memorial Bridge (Roosevelt Bridge) over Lake Texoma in Bryan and Marshall Counties. The purpose of the project is to provide a safe crossing along US-70 over Lake Texoma that accommodates current and future traffic demand; the need for this project is to correct safety shortcomings resulting from a structurally deficient bridge that has sub-standard roadway width and insufficient vertical clearance. In accordance with the Department of Transportation Act of 1966, a Section 4(f) alternatives analysis is required to show that there is no feasible and prudent alternative to reuse the existing bridge prior to replacement. This report outlines the existing roadway, traffic, safety, and bridge conditions of the approximately 4-mile project corridor (US-70 from State Park Road, west of Lake Texoma to Willow Springs Road, east of Lake Texoma); and summarizes the alternatives analysis as required by Section 4(f).

Historic Significance of the Existing Bridge

The Roosevelt Bridge is an 87-span bridge constructed in 1942 as part of the U.S. Army Corps of Engineers (USACE) construction of a hydroelectric dam over the Red River, creating Lake Texoma. The bridge includes a 250-foot-long Warren through-truss central span with a polygonal top chord. The existing bridge has been determined eligible under Criterion C and Criterion A for inclusion in the National Register of Historic Places (NRHP). The bridge is eligible under Criterion C as the only surviving example of a vehicular Warren through-truss span with a polygonal top chord on Oklahoma’s highway system, and the bridge is also eligible under Criterion A for its association with a major USACE dam project. Concurrence from the Oklahoma State Historic Preservation Officer (SHPO) on the historic significance of the bridge was received in May 2021.

General Existing Roadway, Traffic, and Safety Conditions

The existing roadway varies in width across the project corridor. The first segment of the corridor extending from State Park Road to the western bridge abutment is approximately 0.3 miles and transitions from four 12-foot lanes and 10-foot shoulders with a 16-foot two-way left turn lane (TWLTL) at the intersection of US-70 and State Park Road to two 12-foot lanes with 10-foot shoulders at the approach of the Roosevelt Bridge. The second segment of the corridor extends the full bridge length (approximately 5,000 feet). The typical section on the bridge is two 12-foot lanes with no shoulders. The vertical clearance at the truss span is 14’-9” which is less than the 16’-9” recommended clearance. The third segment of the corridor, extending from the eastern bridge abutment to Willow Springs Road, is approximately 2.7 miles. See Figure 4 showing the project corridor and segments. The roadway section transitions from the bridge to include 8-foot shoulders and guardrail on both sides along an elevated causeway. After the causeway, the roadway transitions from a two-lane section with shoulders to a five-lane section with a TWLTL and shoulders at the intersection of Willow Springs Road with the same dimensions as the west end of the project.

Garver performed a traffic analysis for the project corridor and found that the Roosevelt Bridge will not adequately accommodate the anticipated future traffic volumes. The current configuration of the bridge (two 12-foot lanes with no shoulders) is narrow and provides no opportunity for passing or safe refuge for vehicles. Furthermore, a safety analysis along the corridor revealed that the fatal crash rate was almost 2.5 times higher than the statewide crash rate. See Appendix K for Traffic Analysis Memo.

General Existing Bridge Conditions

The existing 4,943-foot-long bridge is composed of 86 approach spans (concrete deck on steel floor beams and girders) and one truss span (250’ steel Warren through-truss) all of which are supported on a combination of concrete substructure elements including two-column bents, four-column tower frames, two-column webbed piers, a conventional abutment, and a tower frame abutment. The concrete deck has multiple large spalls throughout and areas where the deck lifts off of the steel floor beams due to pack rust. All joints have lost their seals allowing water to flow onto the steel beams and girders supporting the deck. Many of the steel floor beams in the approach spans have corrosion and section loss resulting in member capacity reduction. Numerous bearings have sheared bolts and shifted bearing plates. The metal bridge rail has numerous connections that are sheared, missing, or other failed connections. The rail has also been impacted multiple times by vehicles resulting in misalignment and damaged posts throughout. The steel truss members have minor corrosion throughout resulting in no appreciable to minor member capacity reduction. Due to the low vertical clearance, the bracing in the portal frames of the truss has impact damage from vehicular collisions. The concrete substructure elements have minor spalls and cracks throughout, that does not result in appreciable member capacity reduction.

The latest routine bridge inspection report gives the existing deck and superstructure NBI ratings as "5 = Fair"; however, this is based on the emergency repairs conducted in 2021 to avoid load posting the bridge. An investigation would be required of these repairs to understand the anticipated service life of these repair measures and the bridge. See Appendix M and Appendix N for Existing Bridge Data and Inspection reports respectively.

Alternatives Analysis

Garver evaluated seven total alternatives for the purposes of Section 4(f), all of which preserve the historic integrity of the existing bridge to various degrees. Of the seven alternatives investigated, five are considered "No-Use" alternatives (alternatives with impacts not severe enough that the protected features or attributes of the Section 4(f) property are substantially impaired), and two are considered "Use" alternatives (alternatives with impacts so severe that the protected features or attributes of the Section 4(f) property are substantially impaired). Below is a general description of each alternative evaluated. See Section 4.0 of the report for a detailed explanation of Alternatives.

"No-Use" Alternatives

- Alternative 1: Do Nothing. This alternative leaves the existing bridge in its existing configuration (2 lanes; no shoulders) without any rehabilitations to the existing bridge.
- Alternative 2A: Rehabilitate (No Widening). This alternative rehabilitates the existing bridge structure; however, the existing typical section (2 lanes; no shoulders) is not widened.
- Alternative 3A: One-Way Pair (No Widening). This alternative rehabilitates the existing bridge structure and converts traffic to two lanes in one direction on the existing bridge and adds an adjacent new sister bridge with two lanes in the other direction improving the traffic capacity; however, the existing bridge is not widened to provide shoulders.
- Alternative 4: Pedestrian/Bicycle Only. This alternative converts the existing bridge into a pedestrian/bicycle shared use path and utilizes a proposed adjacent bridge with 4 total lanes and shoulders for improved vehicular traffic capacity.
- Alternative 5: Monument Only. This alternative converts the existing bridge into a monument with no vehicular or pedestrian traffic allowed and utilizes a proposed adjacent bridge with 4 total lanes and shoulders for improved vehicular traffic capacity.

"Use" Alternatives

- Alternative 2B: Rehabilitate (With Widening). This alternative keeps the existing bridge intact with retrofits and modifications, and it improves traffic capacity by widening the roadway width to accommodate 4 lanes of traffic with shoulders.

- Alternative 3B: One-Way Pair (With Widening). This alternative is identical to Alternative 3A; however, the existing bridge is widened to provide shoulders.

The structural capacity of the existing bridge elements was evaluated for each alternative using the latest AASHTO LRFD Bridge Design Specifications. Accounting for the specified loading conditions as well as the member capacity reductions due to existing conditions, multiple rehabilitation measures (in the superstructure and substructure) are required for all alternatives except Alternative 1. The "Use" alternatives require more rehabilitation than the "No-Use" alternatives.

Of the alternatives studied, Alternatives 4, 5, 2B, and 3B were found to meet the purpose and need of the project; Alternative 3A partially meets the purpose and need of the project; and Alternatives 1 and 2A do not meet the purpose and need of the project. Figure 1 provides a brief summary of the alternatives analysis; Appendix C provides a more detailed summary of the alternatives. Life cycle cost estimates for all alternatives can be found in Appendix E.

Alternative Analysis					Meets Project Purpose & Need	Operational & Safety Risk	Social, Economic, & Environmental Risk	Community Disruption	Construction & Future Cost **	Life Cycle Cost ***
Alternatives		Existing Bridge Section Width	Total Traffic Lanes *	No Use or Use						
ALT 1	Do Nothing	No Change 2 Lanes No Shoulders	2	No Use	No	High	High	High	Low	Very High
ALT 2 (Opt. A)	Rehab (No Widen)	No Change 2 Lanes No Shoulders	2	No Use	No	High	High	High	Low	High
ALT 3 (Opt. A)	One-Way Pair Rehab (No Widen)	No Change 2 Lanes No Shoulders	4	No Use	Partially	Moderate	Low	Low	High	Moderate
ALT 4	Pedestrian/ Bicycle	No Change (Shared Use Path)	4	No Use	Yes	Low	Low	Low	High	Moderate
ALT 5	Monument	No Change (Not Used)	4	No Use	Yes	Low	Low	Low	Moderate	Moderate
ALT 2 (Opt. B)	Rehab (Widen)	Widened 4 Lanes With Shoulders	4	Use	Yes	Low	High	High	High	High
ALT 3 (Opt. B)	One-Way Pair Rehab (Widen)	Widened 2 Lanes With Shoulders	4	Use	Yes	Low	Moderate	Low	High	Moderate

** "Total Lanes" accounts for an additional vehicular bridge where applicable.*

*** Accounts for rehabs, proposed construction, future inspections and future maintenance*

**** Accounts for Construction & Future Cost as well as User Costs*

Figure 1: Alternatives Analysis Summary

JP No. 33873(04), US-70 over Lake Texoma (Roosevelt Bridge), Project Summary Matrix (Alt 1 - 5)															
Alternative Name and Description		**Existing Bridge Section Width**	**Total Traffic Lanes (1)**	**No Use or Use**	**Life Cycle Costs (Millions) (2)**	**Right-of-Way Cost (Millions)**	**Utility Relocation Cost (3)**	**Wetlands (ac)**	**Streams (ac)**	**Johnson Creek PUA (ac)**	**Texoma State Park (ac)**	**USACE Property (ac)**	**Tribal Land (ac)**	**Hazardous Materials Site**	**Archeological Site 34BR11**
Alternative 1	Do Nothing	No Change 2 Lanes No Shoulders	2	No Use	$1,891.60	$0.00	$ 0.00	0	0	0	0	0	0	N	N
Alternative 2 (Option A)	Rehabilitation (No Widening)	No Change 2 Lanes No Shoulders	2	No Use	$265.70	$0.00	$ 0.00	0	0	0	0	0	0	N	Y
Alternative 3 (Option A)	One-Way Pair Rehab (No Widening)	No Change 2 Lanes No Shoulders	4	No Use	$205.60	$1.11	$ -	0.17	0.03	0.60	0	35	0	N	Y
Alternative 4	Pedestrian / Bicycle	No Change (Shared Use Path)	4	No Use	$247.50	$1.43	$ -	0.28	0.03	0.62	0	45	0	Y	Y
Alternative 5	Monument	No Change (Not Used)	4	No Use	$218.40	$1.32	$ -	0.65	0.06	0.72	0	41	0	N	Y
Alternative 2 (Option B)	Rehabilitation (Widening)	Widened 4 Lanes With Shoulders	4	Use	$437.70	$1.31	$ -	0.67	0.05	0.77	0.82	42	0.38	N	Y
Alternative 3 (Option B)	One-Way Pair Rehabilitation (Widening)	Widened 2 Lanes With Shoulders	4	Use	$253.50	$1.11	$ -	0.17	0.03	0.60	0	35	0	N	Y

(1) "Total Lanes" accounts for an additional vehicular bridge where applicable
(2) Costs include construction, future maintenance and repairs, and user costs.
(3) Utility relocation costing information not provided at the time of the report submittal. Information is to be provided at a later date.

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1.0 Introduction

1.1 Scope

The intent of this Section 4(f) Alternative Analysis report is to provide documentation of the Franklin D. Roosevelt Memorial Bridge over Lake Texoma (Roosevelt Bridge) existing conditions, state a clear purpose and need for the project given the findings of the existing conditions, and evaluate the avoidance alternatives stipulated by the Federal Highway Administration (FHWA) Programmatic Section 4(f) Evaluation for the Use of Historic Bridges.

1.2 Location

The existing Roosevelt Bridge is located on US-70 between Kingston (to the west) and Durant (to the east) in Marshall and Bryan Counties. The bridge carries westbound and eastbound traffic across Lake Texoma with a main truss span crossing over the old channel of the Washita River. The bridge was built in 1942 and has been determined eligible for inclusion in the National Register of Historic Places (NRHP). An existing causeway extends from the east end of the bridge back to the east bank of the lake. Natural recreation areas in the vicinity of the bridge include, but are not limited to, Lake Texoma State Park, Johnson Creek Campgrounds, and Willow Springs Public Use Area. The Chickasaw Nation also owns land on the west side of the lake. There is a United States Army Corps of Engineers (USACE)-owned air strip approximately 2000 feet to the west of the bridge. Figure 2 shows the general location of the project, and Figure 3 shows the approximate limits of the existing bridge and causeway.

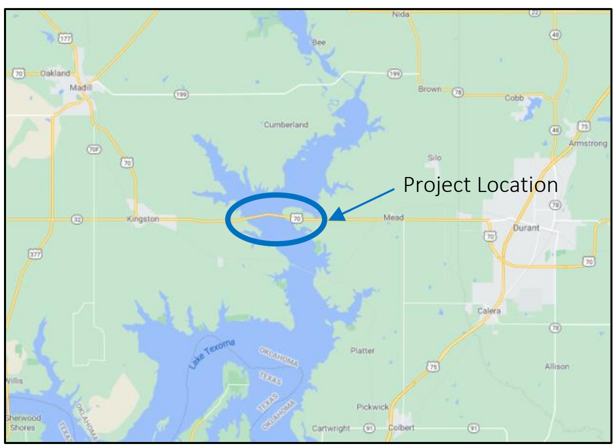


Figure 2: Project Vicinity Map

Figure 3: Bridge Location Map

1.3 Stakeholder Meeting

The Roosevelt Bridge project involves land owned and/or managed by several state and federal agencies, including officials with jurisdiction in the Section 4(f) process. Through an initial ownership search, ODOT Identified the following stakeholders:

- U.S. Army Corps of Engineers, Tulsa District and Lake Texoma Office
- Chickasaw Nation
- Oklahoma Tourism and Recreation Department (State Parks)
- Oklahoma State Historic Preservation Officer (SHPO)
- Oklahoma Archaeological Survey (OAS)/State Archaeologist

In order to inform these agencies of the proposed project and obtain input on the Section 4(f) alternatives, ODOT held a Stakeholder Meeting on August 9, 2021. Thirty-three (33) people attended the meeting, which was held via video conference call. Attendees included representatives from ODOT, the Chickasaw Nation, USACE, Oklahoma State Parks, SHPO, OAS, and the consultant team.

The meeting included a presentation describing the purpose and need for the project, the Section 4(f) statute and the alternatives, the Section 4(f) resources within the project area, and the Section 6(f) process and resources within the project area. The presentation then described the specific resources of interest to each agency and the decisions and/or permits that the project will require from each agency. The presentation concluded with an anticipated timeline for the project.
Questions and comments from the Stakeholder meeting included:

- Clarification of State Parks lease for Lake Texoma State Park – lands are held by USACE, Chickasaw Nation, and privately. State Parks has a lease for lands south of US-70. Concessions include the marina and gas station. Lake Texoma Association owns a small parcel.
- USACE will need additional information to make decisions about Section 4(f) use of USACE property and future permitting.
- Question whether the new bridge will include bicycle/pedestrian facilities.

The Stakeholders were asked to provide additional feedback within 30 days. The USACE responded with information about the Section 4(f) status of their lands east of the bridge. No comments on the Section 4(f) alternatives were received. Stakeholder Meeting minutes are provided in Appendix L.

1.4 Key Terms

The following is a summary of some key terms used throughout this report.

Criterion A

Evaluation criteria under NRHP for a property's significance that is associated with events that have made a significant contribution to the broad patterns of American history.

Criterion C

Evaluation criteria under NRHP for a property's significance that embodies the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction.

Level of Service (LOS)

A qualitative measure used to describe the traffic operating conditions of a roadway based on factors such as speed, travel time, maneuverability, delay, and safety.

Functionally Obsolete

A bridge inadequate to properly accommodate traffic can be due to inadequate clearances, either horizontal or vertical, approach roadway alignment, structural condition, or waterway adequacy. Any posted or narrow bridge would also be included in this category.

Structurally Deficient

Bridges are considered structurally deficient if they have been restricted to light vehicles, closed to traffic, or require rehabilitation. Structurally deficient means that based on field inspection findings there are elements of the bridge that need to be monitored and/or repaired. The fact that a bridge is "structurally deficient" does not imply that it is likely to collapse or that it is unsafe. It means the bridge must be monitored, inspected, and maintained.

Use Alternative

An alternative with impacts so severe that the protected features or attributes of the Section 4(f) property are substantially impaired.

No-Use Alternative

An alternative with impacts not severe enough that the protected features or attributes of the Section 4(f) property are substantially impaired.

De minimis Impact

A de minimis impact is one that, after considering avoidance, minimization, mitigation and enhancement measures, results in no adverse effect to the activities, features, or attributes qualifying a Section 4(f) property for protection under Section 4(f).

Performance Ratio

Performance Ratios describe the level of structural load demand to structural capacity of a component. Performance Ratios below 1.0 indicate the component has sufficient capacity to meet current design criteria. Performance Ratios above 1.0 indicate the component does not have sufficient capacity to meet current design criteria. A Performance Ratio does not directly reflect the capability of the bridge to carry legal loads.

2.0 Existing Conditions

2.1 Roadway

2.1.1 Project Corridor Limits

The roadway corridor studied for this Section 4(f) Alternative Analysis report extends approximately 0.3 miles to the west of the bridge and approximately 2.7 miles to the east of the bridge where the five-lane sections at the two boundary intersections taper into a two-lane undivided facility on the bridge and causeway. The approach roadway geometric elements of US-70 were evaluated based on the design criteria provided by the American Association of State Highway and Transportation Officials (AASHTO), see Appendix A for applicable tables. The roadway through this area is classified as a Principal Arterial, according to ODOT maps (see Appendix A). A Principal Arterial roadway is one that provides a high-speed, high-volume network for travel between key locations. The corridor has been broken up into three segments:

- Segment 1: Western Approach to the Bridge (green line)
- Segment 2: The Bridge (blue line)
- Segment 3: Eastern Approach to the Bridge (orange line)

Figure 4 provides the Project Limits, defines the segments, and shows the defining features within each segment including posted speed limits, lighting, passing opportunities, shoulder width, approach to the single truss span, and grade information.

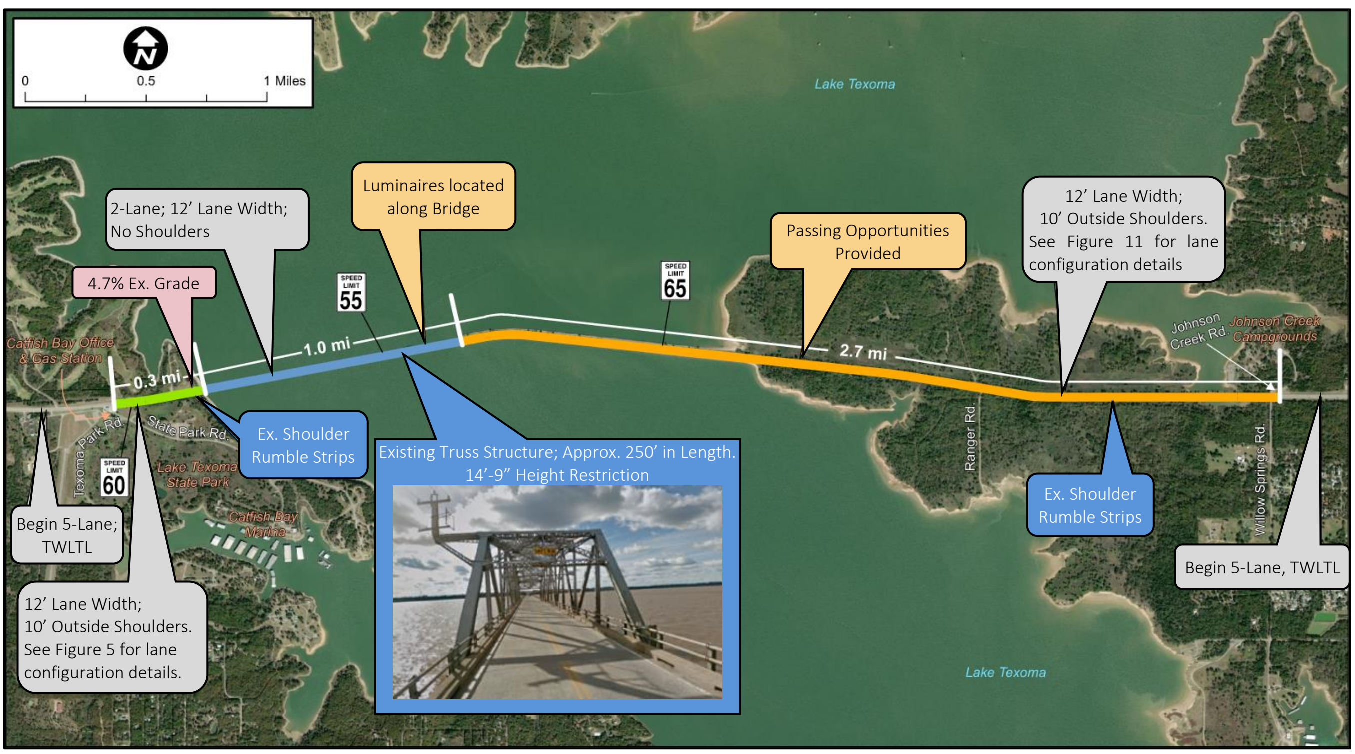


Figure 4: Project Limits Map & Segment Details

2.1.1.1 Segment 1: Western Approach

Segment 1 stretches 0.3 miles from the intersection of US-70 and State Park Road (the entrance to Lake Texoma State Park) to the beginning of the Roosevelt Bridge. The roadway transitions from 4-12' lanes and 10' shoulders with a 16' two-way left turn lane (TWLTL) at the intersection of US-70 and State Park Road to 2-12' lanes with 10' shoulders at the approach of the Roosevelt Bridge (See Figure 5). Approximately one mile west of the intersection at State Park Road, US-70 transitions from a two-lane route to a five-lane section with two lanes in each direction and a center TWLTL. At the State Park Road intersection, the outer eastbound lane terminates as a right turn lane with additional pavement on the departure side that serves as a de facto acceleration lane for right turning traffic from State Park Road. On the westbound approach to the State Park Road intersection, a second through lane develops just beyond the western terminus of the Roosevelt Bridge and the center TWLTL develops approximately 250' in advance of the intersection. Rumble strips along the 10' shoulders are provided on each side of the highway.

Segment 1 is adequate for only 60 mph based on the vertical alignment. This vertical curve does not meet current Full Reconstruction (4R) criteria, but does meet the reduced criteria used for Resurfacing, Restoration, and Rehabilitation (3R) projects. The alternatives discussed later in this report pertain to the 3R criteria, therefore, the existing vertical roadway alignment on this approach is adequate under these conditions. The existing horizontal curve west of the bridge is adequate for 65 MPH but does not meet the 15(V) length of curve design criteria. The curve has a superelevation of 3.2% and a length of 527.31 feet.

Figure 5: Segment 1 - Western Approach to Roosevelt Bridge Lane Transitions

Figure 6: Western Approach Roadway Grade (Looking East)

Figure 7: Roadway Intersection at State Park Rd.

2.1.1.2 Segment 2: The Bridge

Segment 2 includes the Roosevelt Bridge section, stretching 4,943 feet across Lake Texoma. The Roosevelt Bridge carries a two-lane highway with 12’ lanes, no shoulders, and barrier rails on either side (29 feet overall width). The grade of the bridge is mostly flat. The main span of the bridge is a 250’ long truss with vertical clearance of 14’-9”. Overhead electric runs along the south edge of the bridge with luminaires mounted on the bridge.


Figure 8: Segment 2 - Roosevelt Bridge (Looking East)


Figure 9: Bridge Typical Roadway Section

Figure 10: Bridge Section at Truss Span

2.1.1.3 Segment 3: Eastern Approach

East of the bridge, US-70 remains a 2-lane roadway with 8' shoulders and guardrail on both sides over a causeway before transitioning to a 5-lane roadway with a TWLTL at the intersection of Willow Springs Road/Johnson Creek Road as depicted in Figure 11. The vertical curve near this intersection does not meet current 4R criteria, but does meet the reduced criteria used for 3R projects. Throughout Segment 3, the roadway grade along the causeway is relatively flat. The causeway has 3 horizontal curves two of which are 2-degree curves with 6.3% superelevation. The remaining horizontal curve is a 1-degree curve with 3.2% superelevation. The existing horizontal curves are adequate for 65 MPH but do not meet the 15(V) length of curve design criteria.

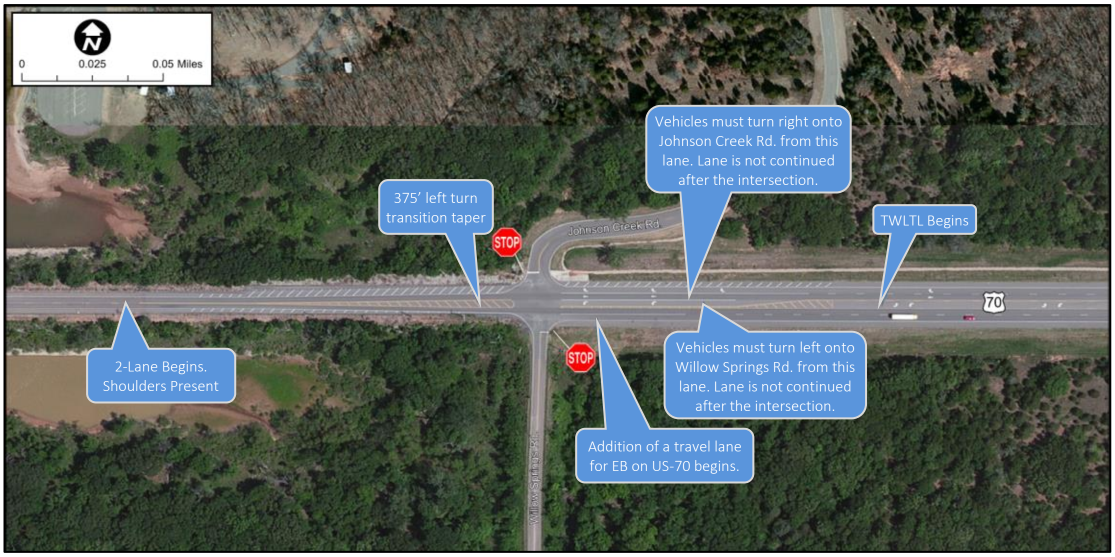


Figure 11: Segment 3 – Eastern Approach to the Bridge at Johnson Creek Road Lane Transitions


Figure 12: Horizontal Curve on Causeway (Looking East)


Figure 13: Roadway Section Transition (Looking East)

2.1.2 Traffic

A traffic study to determine what improvements will be needed along US-70 over Lake Texoma and at the adjacent intersections of US-70 at State Park Road and at Willow Springs Road/Johnson Creek Road was conducted. Traffic counts conducted in May 2021 indicate the Roosevelt Bridge carries approximately 8,500 vehicles per day with 10% trucks. Safety, signalization, and geometry were evaluated in the development of recommendations. Additional information regarding the traffic analysis is provided in Appendix K.

Two sets of future traffic projections were developed – "background growth only" and "with development" volumes. For "background growth only", traffic volumes were projected to the year 2050 using a 1.5% annual growth rate based on historical data trends from ODOT count stations. Using the 1.5% growth rate, US-70 will carry approximately 13,200 and 11,400 vehicles per day east and west of the study area, respectively, by 2050 based on background growth only. For "with development" projection, an expansive proposed development (PointeVista) was considered. PointeVista is planned west of the Roosevelt Bridge near the intersection of US-70 and State Park Road consisting of hotels, retail, tourist activities, and restaurants. The additional demand brought forth by this development would significantly increase traffic volumes on US-70 within the study area. Projected 2050 traffic volumes inclusive of the development were estimated at approximately 28,200 and 26,700 vehicles per day east and west of the bridge, respectively. Therefore, if at full build-out, the expected 2050 volume on the Roosevelt Bridge would double.

2.1.2.1 Safety

To evaluate the performance of US-70 in terms of safety, crash data was collected from ODOT's Safe-T Database for a five-year period from 2015 to 2019. Over the five-year period, a total of 52 collisions occurred within the project limits with 16 crashes (31%) occurring on or near the western approach to the bridge, and 18 crashes (35%) classified as intersection-related. The most common crash types included 12 rear-ends, 11 angle-turning, nine fixed-object, eight sideswipe-opposite direction, and six head-on collisions. Four fatal crashes occurred on the route, along with two incapacitating injuries and nine non-incapacitating injuries.

The corridor crash rate (78 crashes per 100 million vehicle miles traveled (MVMT)) was comparable to the statewide crash rate (76 per 100 MVMT). However, the fatal crash rate for the corridor was almost 2.5 times larger at 6.0 per 100 MVMT than the statewide fatal crash rate at 2.6 per 100 MVMT.

Intersection-related collisions accounted for over one-third of the total collisions experienced within the study area – which can be attributed to limited sight distance and high travel speeds along US-70. The nine fixed-object collisions involve elements located closely alongside the roadway. Guardrails or barrier rails accounted for six of the collisions. One collision was with a tree, another with a traffic sign, and one with a curb.

2.1.2.2 Geometry

The level of service (LOS) along US-70 was evaluated for the existing two-lane conditions as well as a proposed widened four-lane section (two thru lanes per direction) for 2021, 2050 (background growth only), and 2050 (with Development) traffic volumes. This analysis showed that drivers along US-70 will experience a reduction in the LOS on the bridge to LOS E that would bottleneck US-70 and a LOS E and LOS F at the two intersections by 2050 if the PointeVista property is built out and no improvements are made. With the potential widening of the bridge for the Build scenario, mainline LOS results for the bridge improve to LOS A and LOS B creating a free flow for drivers along US-70.

The widening of the bridge was also analyzed in terms of safety benefits, utilizing *Highway Safety Software*. The analysis indicated that a widened bridge with additional elements (such as providing a median, lighting, or wider shoulders) would result in a reduction of 57 to 64 fewer total bridge collisions over the 29-year time period (using 2050 background growth only volumes) – which includes an anticipated reduction of 7 to 10 fatal or injury collisions.

2.1.2.3 Traffic Recommendations

The results of the traffic study demonstrate that the Roosevelt Bridge will not be able to adequately accommodate the proposed 2050 (with Development) volumes as a two-lane facility, as the segment LOS worsens on the bridge to LOS E conditions. Compiled LOS results for the bridge analysis for Build and No-Build conditions are shown in Table 1 below for each design year scenario.

Table 1: Bridge Level of Service Results

Scenario	Level of Service (LOS) Results	
	No-Build Condition	*Build Condition*
2021	C	A
2050 (background growth only)	D	A
2050 (with Development)	E	B

The current configuration of the bridge (2-12' lanes with no shoulders) is narrow and provides no opportunity for passing or safe refuge for vehicles. A widening of the roadway width would also improve the intersection LOS for each of the study intersections. Widening the route from one-lane to two-lanes in each direction will provide additional passing opportunities and a safer route for the projected traffic volumes along US-70.

2.1.3 Detours

For the limits of this project, the length of roadway is approximately 4 miles; however, for the purposes of determining a detour distance, it is assumed the majority of the traffic is travelling to and from Kingston and Durant (shown in green in Figure 14). In the event a detour is needed, whether for proposed construction or future maintenance, the only feasible route is to the north. The detour length is 39.1 miles (shown in red in Figure 14). Delays to the traveling public will be significant if detours are required resulting in substantial user costs.

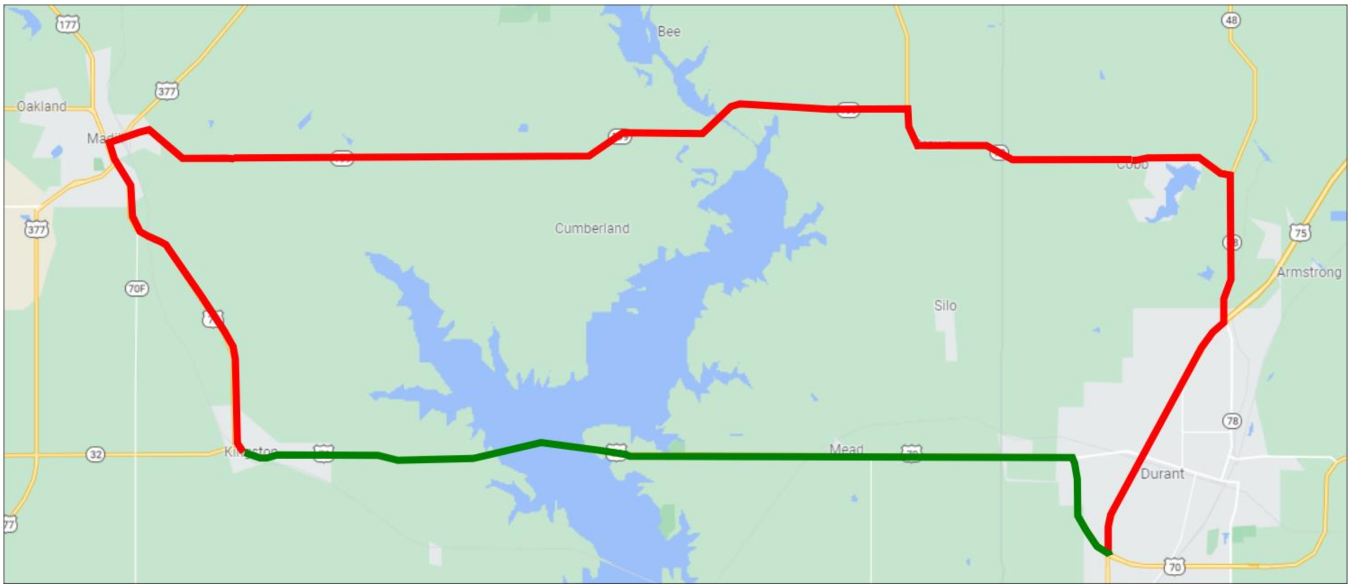


Figure 14: Detour Route

2.2 Bridge

The existing 4,943-foot-long bridge is composed of 86 approach spans (concrete deck on steel floor beams and steel built-up girders) and 1 truss span (250' steel Warren through-truss) all of which are supported on a combination of concrete substructure elements including two-column bents, four-column tower frames, two-column webbed piers, a conventional abutment, and a tower frame abutment. The orientation of the span and support line numbering is from east to west (matching the orientation from the As-Built documents).

2.2.1 Historic & Structural Significance

The Roosevelt Bridge is an 87-span bridge constructed in 1942. The bridge includes a Warren through-truss central span with polygonal top chord. It was determined eligible for inclusion in the National Register of Historic Places (NRHP) as part of the 2007 update of the "Spans of Time" Oklahoma Historic Bridges study.

In February 2021, Cox McLain Environmental Consulting (CMEC) completed an assessment of the boundaries and historic significance of the bridge as it relates to the NRHP eligibility criteria as defined in 36 C.F.R. Part 60 and as applied in National Register Bulletin 15 (Attachment C). Concurrence from the Oklahoma State Historic Preservation Officer (SHPO) on the historic significance of the bridge was received in May 2021. The boundary of the NRHP-eligible property is proposed as the main Warren polygonal through-truss span, the 86 approach spans, all connected cantilevered power lines, and the steel pipe railing to its greatest extent. The earthen causeway to the bridge was constructed prior to the bridge and is not considered part of the historic property.

Figure 15: Roosevelt Bridge Truss Span

2.2.1.1 Criterion C

The bridge is eligible under Criterion C as the only surviving example of a vehicular Warren through-truss span on Oklahoma's highway system with a polygonal top chord. The character-defining features for this bridge type are the diagonal members forming a "W" with triangles, the vertical members, the inclined end posts, and the "curved-shaped" top chord, as well as the struts and bracing of the portal features of the through-truss. The loss of these character-defining features would impair the bridge's ability to convey its significance under Criterion C.

Character-defining features unique to this bridge, unrelated to the bridge's subtype, are the concrete piers and bents comprising the substructure of the single main span and 86 approach spans, steel pipe railings, as well as the cantilevered powerlines on the truss and the 10 cantilevered electrical poles attached to the approach spans (2 east of the main span, 8 to its west). The bridge functioning as a truss is also character-defining. The loss of these character-defining features unrelated to the bridge's subtype would not inhibit the conveyance of significance under Criterion C.

Figure 16: Roosevelt Bridge Early Construction

2.2.1.2 Criterion A

The bridge is also eligible under Criterion A for its association with a major U.S. Army Corps of Engineers dam project. The bridge has associative significance with the important trend of water impoundment and the creation of dams and lakes across Oklahoma. In 1944 the U.S. Army Corps of Engineers constructed the hydroelectric dam over the Red River, backing up the Washita River, a tributary, and creating Lake Texoma. Prior to creation of the lake, the Roosevelt Bridge was constructed as a vehicular structure linking the east and west sides of the lake on US-70 and providing primary public access in the region. The Roosevelt Bridge began construction on dry land before the reservoir filling began in 1944. The bridge opened for traffic on June 21, 1945. It is significant as an example of a vehicular bridge created in direct response to the completion of a major water impoundment project, the Denison Dam. The main character-defining feature under Criterion A is the bridge's use as a vehicular crossing over a waterway. To retain Criterion A significance, the bridge must continue to provide a vehicular crossing as a major thoroughfare. Loss of material integrity would not automatically impact the bridge's Criterion A significance if it continues to serve its historical function.

2.2.2 Inspection Report & Sufficiency

The Roosevelt Bridge is currently inspected on an alternating 24-month Fracture Critical (FC) and 24-month Other Special (OS) inspection cycle, meaning that in odd years, a full FC inspection is performed; and in even years, an OS inspection is performed. Both inspection types require access to primary components in all spans with the FC inspection requiring arm's length access to all FC components/regions including tension zones of the girders and truss span tension/reversal members. The latest FC inspection was performed in July of 2019, and the latest OS inspection report was performed in July 2020. A general summary of the findings in the inspection reports and structural condition of the bridge is provided in Section 2.2.4.

The following is a summary of the condition and appraisal ratings for the bridge. Bold and Italics ratings indicate the bridge being either functionally obsolete or structurally deficient as discussed in Sections 2.2.3 and 2.2.4.

- Item 36a – Bridge Rail: 0 = Substandard
- Item 58 – Deck: 5 = Fair
- Item 59 – Superstructure: 5 = Fair
- Item 60 – Substructure: 6 = Satisfactory
- Item 67 – Structural Condition: 5 = Above Min Tolerable
- ***Item 68 – Deck Geometry: 2 = Intolerable – Replace***
- Item 71 – Waterway Adequacy: 5 = Above Tolerable
- Item 72 – Approach Roadway Alignment: 8 = Equal Desirable Criteria

The sufficiency rating formula is a method of evaluating a bridge's sufficiency to remain in service, based on a combination of several factors including fields that describe its structural evaluation, functional obsolescence, and its essentiality to the public. The result of the formula is a percentage in which 100 percent represents an entirely sufficient bridge and zero percent represents an entirely insufficient or deficient bridge. The current sufficiency rating for this structure is 42.3.

2.2.3 Functionality

Functionality is defined as the ability to provide the user with a product at its full designed purpose. It is related to the geometric components of the bridge structure such as lane widths, shoulder access, vertical clearance and sight distances. In order to be considered functionally obsolete, the bridge structure must meet at least one of the following criteria set from the Non-Regulatory Supplement 23 650D(9a) of the Federal-aid Policy Guide:

1. An appraisal rating of 3 or less for
    a. Item 68 – Deck Geometry; or
    b. Item 69 – Underclearances; or
    c. Item 72 – Approach Roadway Alignment; or
2. An appraisal rating of 3
    a. Item 67 – Structural Condition; or
    b. Item 71 – Waterway Adequacy

The existing bridge has a substandard vertical clearance of 14'-9" (less than the standard vertical clearance of 16'-9"). This vertical clearance is at the top portal bracing of the truss span. The existing clear roadway width is 24'-0" (less than the standard 40'-0" clear roadway for a 2-lane facility). These two conditions result in the Roosevelt Bridge being categorized as functionally obsolete (Appraisal Rating for NBI "Item 68 – Deck Geometry" is "2=Intolerable – Replace").

2.2.4 Structural Condition

A bridge is considered structurally deficient if significant load carrying elements of the structure are found to be in poor condition due to deterioration and/or damage. In order to be considered structurally deficient, the bridge structure must meet at least one of the following criteria set from the Non-Regulatory Supplement 23 650D(9a) of the Federal-aid Policy Guide:

A condition rating of 4 (Poor Condition) or less for

a. Item 58 – Deck; or
b. Item 59 – Superstructure; or
c. Item 60 – Substructure; or
d. Item 62 – Culverts and Retaining Walls; or

The existing bridge is "at-risk" of being classified as structurally deficient. Currently, the superstructure and deck condition ratings are 5; however this is based on the emergency repairs conducted in 2021 to avoid load posting the bridge. An investigation would be required of these repairs to understand the anticipated service life of these repair measures and the bridge.

The following sections summarize the components of the structure and their respective conditions as observed in the inspection reports. The structural evaluation of the bridge components is discussed in Section 4.0 of this report and accounts for the physical deterioration as outlined in the inspection reports. The bridge has been grouped into three sections:

- Superstructure (Approach Spans)
- Superstructure (Truss Span)
- Substructure

2.2.4.1 Superstructure Condition – Approach Spans (Spans 1-16 & Spans 18-87)

The approach span superstructure has 86 spans. Each span is composed of a concrete deck supported on transverse, steel W-shape floor beams. The floor beams are connected to primary built-up, riveted, steel I-girders (two-girder system). The primary girders are braced at each floor beam with cross frames or struts. Metal bridge rails are connected to the steel floor beams and concrete deck curb. Figure 17 depicts the typical section of the approach spans. Span lengths vary between either 60'-1" (63 spans) or 34'-0" (23 spans).

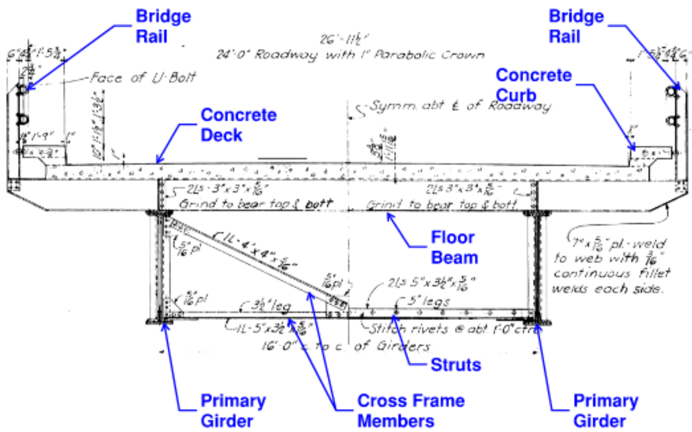


Figure 17: Approach Span Typical Section

The following section provides a description of the different approach span superstructure components as depicted in Figure 17 along with the component's existing physical condition as of the latest inspection report referenced in this report. In Section 4.0 of this report, member capacities account for the section loss (if applicable) outlined in the existing conditions presented below. Sample photos from inspection reports have been provided for each of the components as well.

Concrete Deck

- *Description*: The concrete deck provides a 24' roadway with 1'-6" wide curbs. The deck thickness varies from 7 3/8" to 8 3/8".
- *Existing Physical Condition*: The deck generally has cracking, spalling with exposed rebar, failed asphalt patching, abrasion, wear, and lifting off floor beams throughout. The deficiencies cause serviceability concerns for rideability and safety concerns due to loose concrete at the bridge soffit above a navigable waterway.

Figure 18: Inspection Photo: Deck Soffit Spall

Figure 19: Inspection Photo: Deck Spall

Bridge Rail

- *Description*: The metal bridge rail is composed of steel W-shape posts anchored to the concrete curb and each floor beam. The horizontal rails are 3 ½" STD. steel pipes attached to the posts with U-bolts.
- *Existing Physical Condition*: The metal bridge railing has been impacted multiple times by vehicles and exhibits misalignment throughout. Past experience has shown that this rail does not adequately re-direct vehicles. Approximately 33% of connection locations are compromised due to sheared, missing, and/or failed connections resulting in complete loss of support capacity. The deficiencies present a safety hazard for the traveling public due to the inadequacy of the rail to protect drivers.

Figure 20: Inspection Photo: Failed Rail Post

Figure 21: Inspection Photo: Sheared Rail Bolt

Floor Beams

- *Description*: The floor beams are W16x45 steel beams supporting the deck transversely. They are spaced at 8'-7" max center-to-center and cantilever past the primary steel girders 6'-3 ¾".
- *Existing Physical Condition*: The intermediate floor beams exhibit section loss of varying degrees. In general, the section loss is about 1/8" in the flanges. The end floor beams have more substantial section loss with up to 3/8" in the web and 9/16" in the flanges. Recent and ongoing blasting/painting operations have resulted in moresection loss than prior inspections revealed. Many of the floor beam exhibit knife edging in the top flanges and in some locations, complete section loss.

Figure 22: Inspection Photo: Floor Beam Section Loss & Knife Edging in Top Flange

Figure 23: Inspection Photo: Floor Beam Section Loss & Corrosion in Web

Primary Girders

- *Description*: The two-girder system primary girders are built-up, rivetted "I" shapes that use web plates and double angle flanges with cover plates. They are approximately 54" deep and spaced 16'-0" center to center. Double angles are used for bearing and intermediate web stiffeners.
- *Existing Physical Condition*: In general, the existing girders are in good condition with some minor areas of deterioration as outlined below:
  - The top flanges have plug welds which are considered a Fatigue Category E'. This category is suggestive of potential fatigue concerns. The beam ends have isolated 1/4" max deep (1/8" average) pitting and surface corrosion. Multiple beams have cracked welds in utility support attachments in compression zones (top flange) and do not significantly affect capacity.
  - The bottom flanges have plug welds which are akin to a Fatigue Category E' but is not specifically defined in current fatigue criteria. This category is suggestive of potential fatigue concerns. The beam ends have isolated 1/4" max deep (1/8" average) pitting and surface corrosion.
  - The web at the beam ends has isolated 1/4" max (1/8" average) deep pitting and surface corrosion at the ends adjacent to bearings; however, the loss is typically beyond the bearing and does not significantly impact the beam shear capacity.

Figure 24: Inspection Photo: Primary Girder Plug Welds

Figure 25: Inspection Photo: Primary Girder Pitting

Cross Bracing

- *Description*: The cross bracing is made up of various single angle cross braces and double angle struts.
- *Existing Physical Condition*: There is pack rust with section loss (including through holes) with adjacent knife edging at many end panels adjacent to piers.

Figure 26: Inspection Photo: Bracing Corrosion

Utility Tower Frame

- *Description*: Utility towers carrying overhead electric run the full length of the bridge. These towers are framed to the superstructure at the midspan of 34' spans. A combination of channels and angles form the frame that cantilevers off of the primary girders to support the tower. The tower, made up of a network of single angle members is connected to the superstructure frame with plates.
- *Existing Physical Condition*: The frames exhibit cracks in the connected bracket welds. The floor beam members of the frame have similar corrosion and deterioration as the W shape floor beams

Figure 27: Inspection Photo: Floor Beam Section Loss & Corrosion

Figure 28: Inspection Photo: Cracks in Connected Bracket Welds

Joints & Bearings

- *Description*: Deck joints are either expansion joints or fixed joints with corresponding bearing types.
- *Existing Physical Condition*: The deck and soffit at the expansion joints have cracking and spalling throughout. The abutments do not have sufficient expansion capability due to inward rotation/translation. The superstructure supported by bents and towers do not have the required expansion capability throughout. Joint filler has failed at all joints allowing free flow of water. The expansion and fixed bearings have multiple bolts that have sheared bolts resulting in 100% capacity reduction. The expansion bearings have multiple misalignments outlined below:
  - Support No. 16/Forward Span/North girder – Shifted to east ½"
  - Support No. 31/Forward Span/South girder – Shifted to east 3/8"
  - Support No. 54/Forward Span/North girder – Shifted to east ½"
  - Support No. 66/Forward Span/South girder – Shifted to east ½"

Figure 29: Inspection Photo: Shifted Rocker Bearing

Figure 30: Inspection Photo: Sheared Bolts at Bearing

2.2.4.2 Superstructure Condition – Truss Span (Span 17)

The truss span is a single 250' long Warren through-truss span. The truss span is composed of a concrete deck supported on stringers and floor beams that connect to the truss main members. The metal rail extends from the approach spans onto the truss span and connects to the truss main members. Lateral wind bracing and sway bracing connect the left and right trusses together.

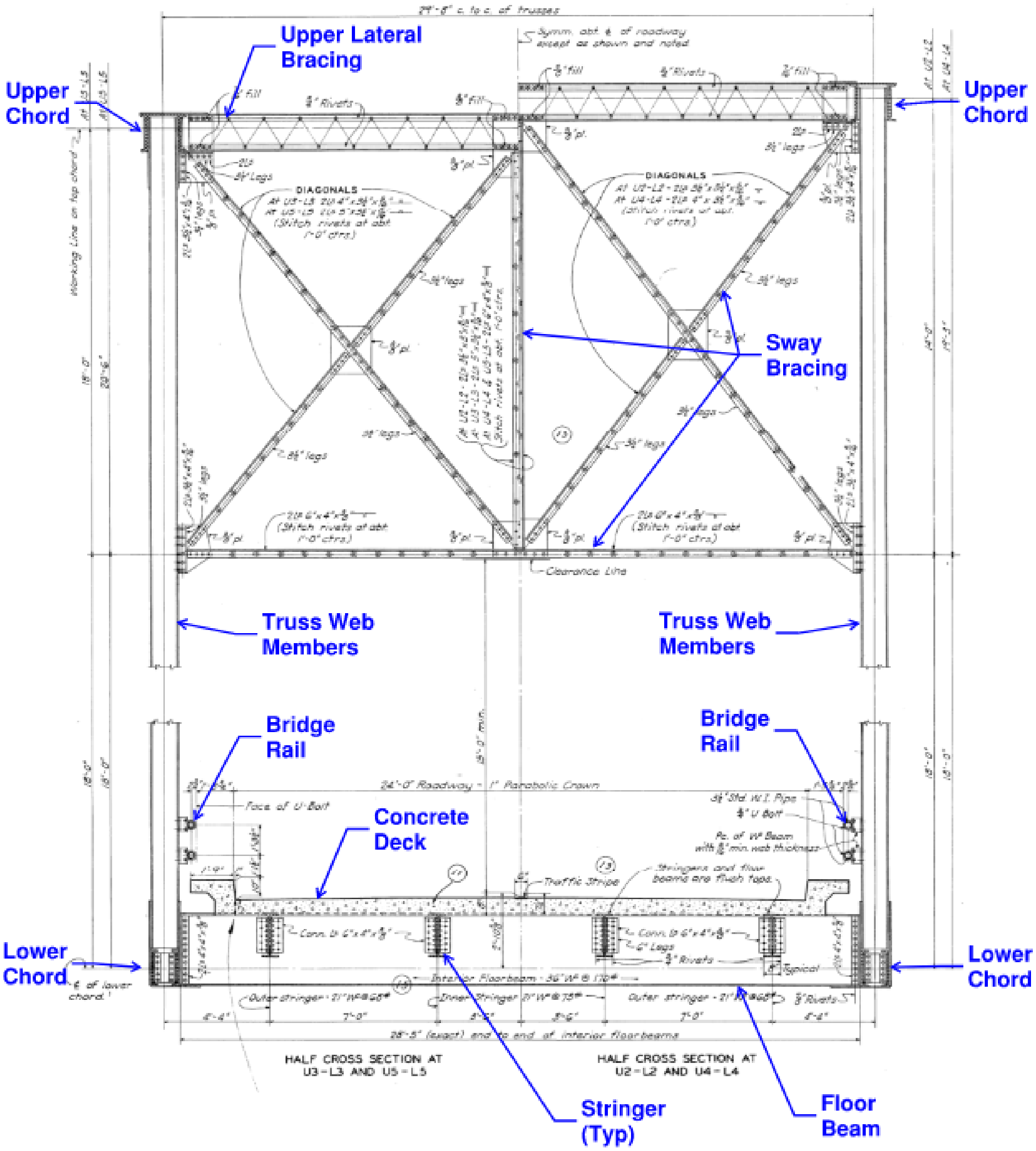


Figure 31: Truss Span Typical Section

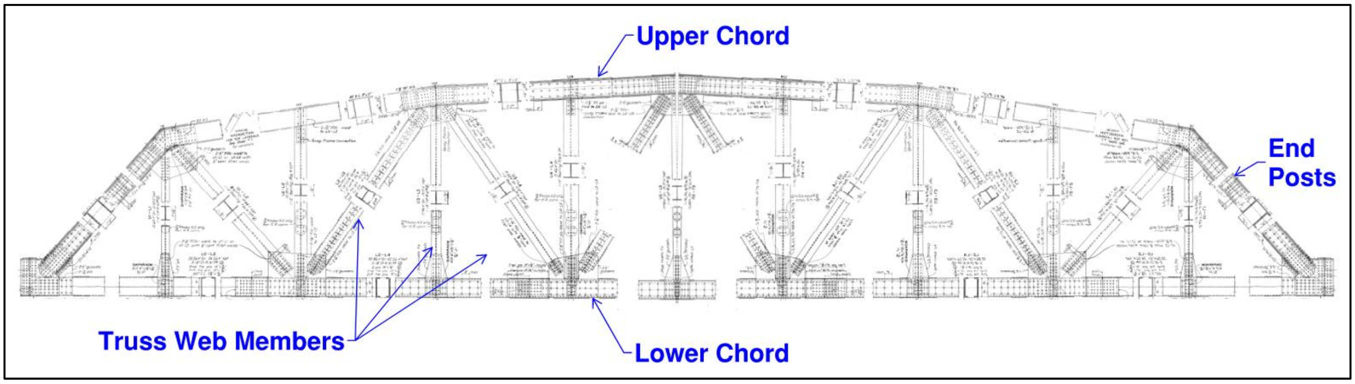


Figure 32: Truss Elevation

The following section provides a description of the different truss span superstructure components as depicted in Figure 31 along with the component's existing physical condition as of the latest inspection report referenced in this report. In Section 4.0 of this report, member capacities account for the section loss (if applicable) outlined in the existing conditions presented below.

<u>Concrete Deck</u>

- *Description*: See approach span description.
- *Existing Physical Condition*: See approach span condition.

<u>Upper Chord</u>

- *Description*: Upper chord truss members consist of double-channel members with a riveted cover plate linking the top flanges and riveted lacing connecting the bottom flanges
- *Existing Physical Condition*: The upper chord members exhibit minor surface corrosion, producing in no appreciable reduction in member capacity.

<u>Lower Chord</u>

- *Description*: Lower chord truss members consist of double-channel members with riveted batten plates connecting the top and bottom flanges.
- *Existing Physical Condition*: The lower chord members exhibit minor corrosion resulting in about a 5% member capacity reduction. A 2½" crack is present in one of the lower chord batten plates. Because the batten plate is attached to the low chord with a riveted connection, this crack is unable to propagate into the main members.

Figure 33: Inspection Photo: Crack in Fill Plate Adjacent to Vertical Inside Upper Panel Point

Truss Web Members (Vertical & Diagonals)

- *Description*: Vertical members consist of I-shaped rolled members; diagonal members consist of I-shaped rolled members or built-up members consisting of and I-shape riveted to double channels.
- *Existing Physical Condition*: Some verticals are misaligned due to traffic impact to a maximum magnitude of ¼" longitudinal & ¾" transverse. Due to these members being tension or zero-force members, the capacity is not affected. Some cracks have been observed in fillet welds between zero-force verticals and fill plates, but due to the members being zero-force, no capacity reduction is needed.

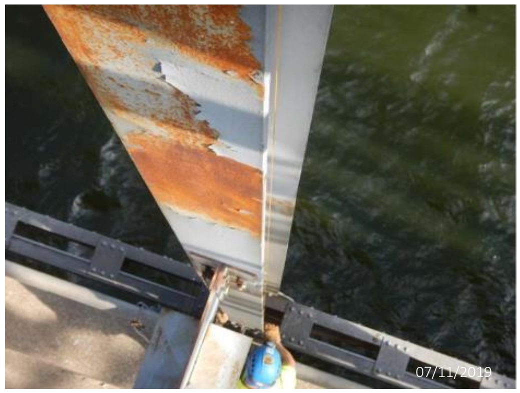


Figure 34: Inspection Photo: Truss Vertical Member Misalignment

End Posts

- *Description*: End posts consist of double-channel members with a riveted cover plate linking the top flanges and riveted lacing connecting the bottom flanges.
- *Existing Physical Condition*: The end posts exhibit minor corrosion resulting in about a 5% member capacity reduction.

Lower Lateral Bracing

- *Description*: Lower lateral bracing diagonals consist of double angle members placed in an X-configuration in each bay between floor beams.
- *Existing Physical Condition*: The lower lateral diagonals exhibit minor surface corrosion, resulting in no appreciable reduction in member capacity.

Upper Lateral Bracing

- *Description*: Upper chord lateral brace diagonals consist of double angle members placed in an "X"-configuration in each bay at the top of the truss.
- *Existing Physical Condition*: The upper lateral diagonals exhibit minor surface corrosion, resulting in no appreciable reduction in member capacity.

Sway Bracing

- *Description*: The lower strut of the sway bracing was previously damaged by traffic impact, and all sway bracing was replaced at a higher elevation to increase vertical clearance.
- *Existing Physical Condition*: The upper (original) portion of the sway braces exhibit minor surface corrosion, resulting in no appreciable reduction in member capacity.

Figure 35: Inspection Photo: Relocated Sway Bracing Connection

End Portal Frames

- *Description*: The end portal frames brace the ends of the truss and are composed of built-up struts and X-frame cross bracing (double angles).
- *Existing Physical Condition*: The portal bracing has been damaged by vehicular impact. The bottom inboard angle of the west portal exhibits several local kinks due to vehicle collision damage near the roadway centerline.

Figure 36: Inspection Photo: Damaged Portal Bracing

Gusset Plates

- *Description*: Connecting plates for all truss members.
- *Existing Physical Condition*: Plates are bowed out of plane up to 1/8” due to pack rust. Minor corrosion is visible along the horizontal shear plane of the lower chord gussets (at the top of the chord member) resulting in approximately 1/16” reduction in plate section.

Stringers

- *Description*: Stringers run parallel with the deck and span between floor beams (approximately 25'). Outer stringers are W21x68 steel beams and inner stringers are W21x78 steel beams.
- *Existing Physical Condition*: Some stringer copes are overcut up to 1/8", with accompanying cracks up to 3/8" long; however, no significant section loss is evident.

Floor Beams

- *Description*: Floor beams run perpendicular to the deck and tie to the truss web members. End floor beams are W36x160 steel beams and interior floor beams are W36x170 steel beams
- *Existing Physical Condition*: The floor beams exhibit corrosion in the top flange resulting in approximately 5% member capacity reduction.

Rivet Connections

- *Description*: Rivetted connections between floor beams and stringers.
- *Existing Physical Condition*: No significant section loss is evident at the rivet connections; however, pack rust between the connection angles and the floor beam web will need to be addressed if any rehabilitation of the existing structure is performed.

Truss Bearings

- *Description*: Steel expansion rocker and fixed bolster bearings.
- *Existing Physical Condition*: No significant section loss is evident.

2.2.4.3 Substructure Condition

The Roosevelt Bridge is supported by a series of reinforced concrete bents, towers, truss piers, and abutments. There are 88 total support lines consisting of 40 bents (two-column frames), 23 tower frames (four-column system), 2 truss piers (two-column piers with web walls), and 2 abutments (one abutment is on a spread footing, the other is a four-column system with wingwalls). Of the 88 support line locations, the as-built construction plan set combines these support lines into 29 uniquely detailed groups consisting of 1-story, 2-story, 3-story, and 4-story structures. These constructed groupings were condensed into 18 modeling groups (See Table 2). Figure 37 through Figure 40 provide isometric views of each of the substructure support types.

Table 2: Substructure Support Line Groups

Type	Model Designation	Included Support Line Nos.	Height (ft)
Bents	B59	83	59
	B67	82, 5	67
	B83	79, 78	83
	B87	75, 74, 38	87
	B90	63, 62, 59, 58, 39, 35, 30, 27, 26, 23, 22, 19, 16, 13, 9, 8	90
	B93	67, 66, 51, 50, 47, 46, 43, 34, 31, 12	93
	B94	71, 70	94
	B109	55, 54, 42	109
Towers	T31	87, 86	31
	T55	85, 84, 4, 3	55
	T76	81, 80	76
	T87	77, 76, 37, 36, 21, 20	87
	T93	73, 72, 69, 68, 65, 64, 61, 60, 53, 52, 49, 48, 45, 44, 33, 32, 29, 28, 25, 24, 15, 14, 11, 10, 7, 6	93
	T95	57, 56	95
	T100	41, 40	100
Piers	P103	18, 17	103
Abutments	A1	2, 1	51
	A88	88	-

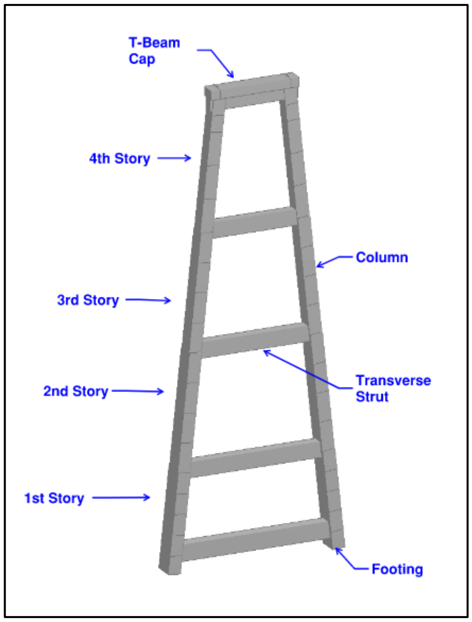


Figure 37: Typical Bent Model

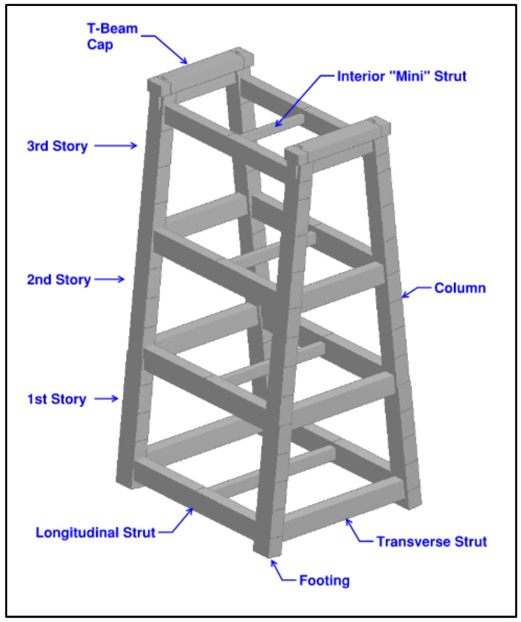


Figure 38: Typical Tower Frame Model

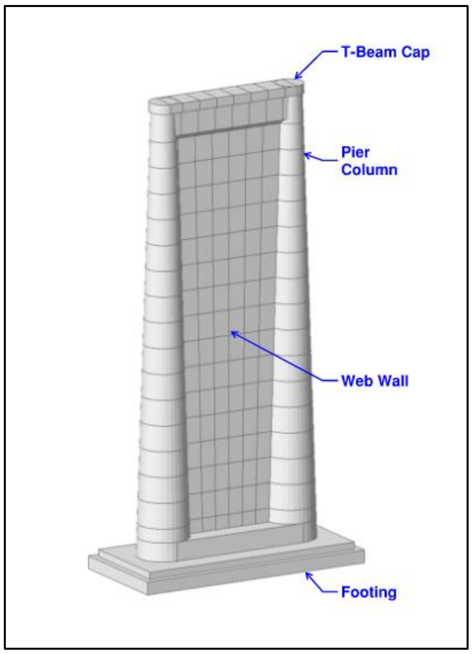


Figure 39: Typical Truss Pier Model

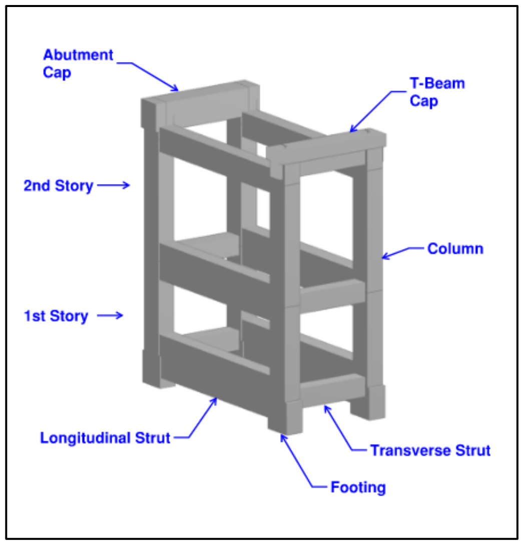


Figure 40: Abutment A1 Tower Frame Model

The following section provides a description of the different substructure support lines along with the component's existing physical condition as of the latest inspection report referenced in this report.

Bents

- *Description*: The bents are two-column frames with tapering columns tied together with a T-beam cap at the top and transverse struts throughout. The columns are supported on multi-pile (timber and concrete) footings.
- *Existing Physical Condition*: The grout pads at the bearing supports have cracking and spalling resulting in undermining of the anchor plates (range: 1/8" to 4"). Steel shim bars have been stitch welded to the majority of

most heavily undermined grout pads. No observed concrete deterioration results in appreciable member capacity reduction.

Towers

- *Description*: The towers are four-column frames with tapering columns tied together with a T-beam cap at the top and transverse and longitudinal struts throughout. The columns are supported on multi-pile (timber and concrete) footings.
- *Existing Physical Condition*: The grout pads at the bearing supports have cracking and spalling resulting in undermining of the anchor plates (range: 1/8" to 4"). Steel shim bars have been stitch welded to the majority of most heavily undermined grout pads. No observed concrete deterioration results in appreciable member capacity reduction.

Piers

- *Description*: The piers are composed of two circular columns framed together with a full-height web wall. The columns are supported on multi-pile footings.
- *Existing Physical Condition*: Moderate abrasion at normal water line and a few isolated spalls. No observed concrete deterioration results in appreciable member capacity reduction.

Abutments

- *Description*: The west abutment (No. 88) is a conventional spread-footing abutment; the east abutment (Nos. 1 & 2) is a tower frame abutment similar to the bent tower frames.
- *Existing Physical Condition*: Both abutments have rotation/translation inwards with the backwall touching the deck. The grout pads at the bearing supports have cracking and spalling resulting in the undermining of the anchor plates (Range: 5/8" to 7"). Steel shim bars have been stitch welded to the majority of most heavily undermined grout pads. The wingwalls have exposed back faces due to erosion. No observed concrete deterioration results in appreciable member capacity reduction.

2.2.5 Load Rating

Load rating of the existing bridge components were not included in the scope of work of this project. The current load ratings have been provided by the Department and are included in Appendix O.

2.3 Environment & Community

The Roosevelt Bridge crosses Lake Texoma in the Eastern Cross Timbers region of southern Oklahoma, which is characterized by rolling hills, cuestas, long narrow ridges, and a few strongly dissected areas. Natural vegetation includes cross timbers (oak species and black hickory) and tall grass prairie (bluestem, switchgrass, and Indiangrass). Landcover is mostly grassland, rangeland, and woodland. Woodland distribution is increasing and consists of scrubby oak forests, oak savannas, and riparian forests. Within the immediate project area, land use is developed on the west side of Lake Texoma with a gas station, golf course, and Lake Texoma State Park. The east side of the bridge consists of undeveloped recreational and wildlife management lands administered by the USACE. The Johnson Creek Public Use Area (campground) is on the very eastern end of the project area.

Lake Texoma is a popular recreational destination attracting over 6 million visitors per year. Amenities such as Lake Texoma State Park offer campsites, hiking trails, and marina facilities. Much of the land surrounding the lake is owned by the USACE, who manages the lake's various functions. Other lands are owned by the Chickasaw Nation and private entities including developments such as golf courses and housing. The town of Kingston is approximately 4 miles to the west, and the city of Durant is approximately 9 miles to the east. Much of the land on US-70 between these two population centers (outside of the immediate Lake Texoma area) consists of commercial and industrial property adjacent to the highway, and small residential areas set further back.

Environmental constraints within the study area include waters and wetlands associated with Lake Texoma and its shoreline, two previously documented archeological sites (one of which is currently under water), and petroleum storage tanks at the Catfish Marina gas station. Lake Texoma State Park occupies much of the land on the south side of US-70 west of the lake, with the land within the park boundary owned by the USACE, Chickasaw Nation, and private entities. There also an airfield associated with the park. There is no residential development directly within the study area. Recreational vehicle and boating use is high and is reflected in the types of vehicles using US-70.

3.0 Project Purpose & Need

The purpose of this project is to provide a safe crossing along US-70 over Lake Texoma that accommodates current and future traffic demand.

The need for this project is to correct an at-risk of being structurally deficient bridge that has a sub-standard roadway width and insufficient vertical clearance at the truss span. Specifically, the following areas need to be addressed:

- Address the structural capacity of the existing bridge which does not meet the latest AASHTO loading conditions.
- Address the bridge rail which does not meet MASH TL-4 rating.
- Address the low vertical clearance at the truss span which does not meet today's design standards.
- Address the narrow roadway and shoulder widths which do not meet today's design standards for the anticipated travel demand.

4.0 Alternative Analysis

The FHWA Programmatic Section 4(f) Evaluation for the Use of Historic Bridges (approved July 5, 1983) requires that the following avoidance alternatives be fully evaluated and shown to be not feasible and prudent:

- Do Nothing
- Improve (rehabilitate and/or widen) the existing structure without a "use" under Section 4(f)
- Build a new structure at a new location without a "use" of the existing structure

In the case of historic bridges, a "use" is defined as an adverse effect to the structure's significance and/or integrity, as defined through the Section 106 (National Historic Preservation Act) process.

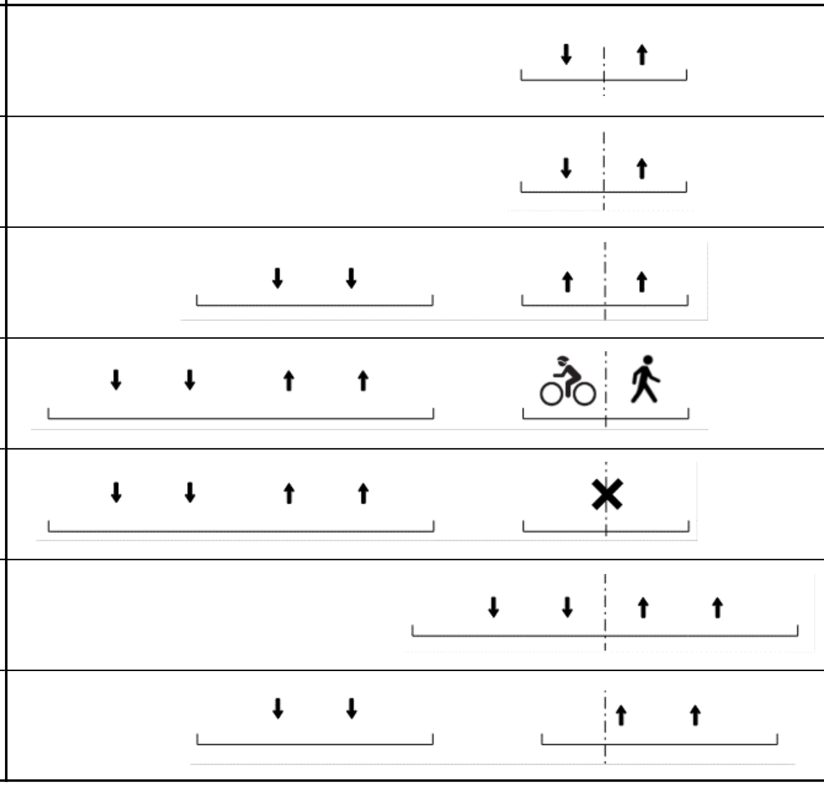
For this study, Garver evaluated seven alternatives as outlined below (Table 3) and summarized in a matrix in Section 5.0. Those listed as "No Use" alternatives are presumed to meet the criteria of the Section 4(f) Programmatic Evaluation as avoidance alternatives. The two "Use" alternatives would likely have an adverse effect on the existing structure. Alternative 6 Replacement will be considered only if the avoidance alternatives are not feasible and prudent. A feasible and prudent alternative avoids using the Section 4(f) property and does not cause other severe problems of a magnitude that substantially outweigh the importance of protecting the Section 4(f) property.

Table 3: Alternatives

"No-Use"		"Use"	
Alternative 1	Do Nothing	Alternative 2B	Rehabilitate Existing Bridge (With Widening)
Alternative 2A	Rehabilitate Existing Bridge (No Widening)	Alternative 3B	One-Way Pair & Rehabilitate Existing Bridge (With Widening)
Alternative 3A	One-Way Pair & Rehabilitate Existing Bridge (No Widening)	Alternative 6	NOT IN THIS REPORT: Replacement
Alternative 4	Pedestrian/Bicycle Only		
Alternative 5	Monument Only		

See Figure 41 for a summary with traffic diagrams

Alternative Analysis					Typical Section Traffic Diagram
Alternatives		Existing Bridge Section Width	Total Traffic Lanes Accomodated	No Use or Use	
ALT 1	Do Nothing	No Change 2 Lanes No Shoulders	2	No Use	
ALT 2 (Option A)	Rehab (No Widen)	No Change 2 Lanes No Shoulders	2	No Use	
ALT 3 (Option A)	One-Way Pair Rehab (No Widen)	No Change 2 Lanes No Shoulders	4	No Use	
ALT 4	Pedestrian/ Bicycle	No Change (Shared Use Path)	4	No Use	
ALT 5	Monument	No Change (Not Used)	4	No Use	
ALT 2 (Option B)	Rehab (Widen)	Widened 4 Lanes With Shoulders	4	Use	
ALT 3 (Option B)	One-Way Pair Rehab (Widen)	Widened 2 Lanes With Shoulders	4	Use	



Note: "Total Lanes Accomodated" accounts for an additional vehicular bridge where applicable.

Figure 41: Alternative Diagrams

The regulations in 23 CFR 774.17 set out factors to consider in determining whether an avoidance alternative is feasible and prudent:

- An alternative is not feasible if it cannot be built as a matter of sound engineering judgment.
- An alternative is not prudent if:
    - It compromises the project to a degree that it is unreasonable to proceed with the project in light of its stated purpose and need;
    - It results in unacceptable safety or operational problems;
    - After reasonable mitigation, it still causes:
        - Severe social, economic, or environmental impacts;
        - Severe disruption to established communities;
        - Severe disproportionate impacts to minority or low-income populations; or
        - Severe impacts to environmental resources protected under other Federal statutes;
    - It results in additional construction, maintenance, or operational cost of an extraordinary magnitude;
    - It causes other unique problems or unusual factors; or
    - It involves multiple factors listed above, that while individually minor, cumulatively cause unique problems or impacts of extraordinary magnitude.

Examples of unique problems include unacceptable social, economic, or environmental impacts; serious community disruption; unacceptable safety or geometric problems; or excessive construction costs. An accumulation of these problems (as opposed to a single factor) may be a sufficient reason to use a Section 4(f) resource, but only if the problems are truly unique. Excessive cost alone will not necessarily prevent an alternative from being considered prudent.

For each alternative the following considerations were evaluated:

- *Purpose & Need*: how the alternative meets or does not meet the project purpose and need
- *Existing Structural Capacity*: the existing structure's capacity (accounting for observed section losses) to support the loading conditions specific to each alternative
- *Anticipated Rehabilitation & Character Defining Feature Modifications*: strengthening/modification of the existing bridge members required to meet current design standards and what character defining features may be modified
- *Environmental & Community Impacts:* disruption to the environment and/or local community
- *Estimated Life Cycle Cost*:
    - Construction Cost
        - Rehabilitation of existing components
        - Construction of proposed bridge or proposed components for the existing bridge
    - Future Cost
        - Inspections
        - Maintenance (deck replacement and painting of steel members)
        - Repairs (necessary repairs to keep the bridge from being load posted)
    - User/Indirect Cost
        - Costs associated with vehicular collisions (safety)
        - Driver costs associated with Detours (for proposed construction and/or future maintenance)

The life cycle costs presented in this report are high level estimates and are calculated as present value (See Appendix E for interest and inflation assumptions). There are many factors that can change costs especially future conditions. Specific factors that were not accounted for in the cost analysis for this report include but are not limited to: costs associated with traffic level of service, costs associated with load posting or full closure of the existing bridge, costs associated with impacts to local businesses.

The roadway widths presented with each alternative are approximate at this time and shown for conceptual evaluation. Further evaluation of safety features such as shoulder widths, medians, median barriers, and bridge lighting may increase the overall roadway width and cost. More detailed information regarding traffic and safety analysis can be found in the Traffic Analysis Memo (Appendix K).

The criteria and guidelines used for evaluation of each alternative include:

- Structural Analysis: conforms to the current
    - *AASHTO LRFD Bridge Design Specifications*
    - *AASHTO Manual for Bridge Evaluation*
    - *AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges*
- Historic Bridge Rehabilitation: conforms to current
    - *AASHTO Guidelines for Historic Bridge Rehabilitation and Replacement*
    - Specific project guidelines provided by ODOT.
- Performance Ratios (PR) describe the level of load demand to structural capacity of a component
    - Large PR (Greater than 1.0) = Component fails required capacity
    - Small PR (Less than or equal to 1.0) = Component meets required capacity
    - PR Tables in this report are summarized for the existing structural components without impacts of rehabilitation. Any rehabilitation will require all PRs for each component to be less than 1.0.
    - PRs reported do not directly reflect the capability of the bridge to carry legal loading in its current state; they are presented only to compare the load capacity of the bridge to the capacity of a new structure designed to current specifications.
- FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges

4.1 No-Use Alternatives

All of the "No-Use" alternatives generally keep the existing bridge in its current configuration. However, varying levels of member replacement or retrofitting are required depending on the loading conditions of the specific alternative. The majority of retrofits that are required apply to the existing bridge deck, metal rail, approach span floor beams, approach span primary girders, and the substructure. The truss span will only require minor retrofits and repairs under all the "No-Use" alternatives.

4.1.1 Alternative 1: Do Nothing

Alternative 1 will leave the existing bridge with its current alignment, traffic direction, geometric configuration, and structural condition. This alternative assumes there will be no rehabilitation to the existing structure to bring the structural components up to current design standards, the truss will not be modified to provide sufficient vertical clearance, and no proposed adjacent bridge will be provided to increase traffic capacity. This alternative assumes that in 30 years the bridge will have to be load posted and permanently closed 30 years after load posting. These time frames were chosen as reasonable assumptions for the repaired and original elements of the bridge, based on current minimum service life expectations for newly constructed bridges of 75-years to match the design life of the structure. Although the repaired elements will only be 80% of this minimum service life, the other non-replaceable parts will exceed 110 years of service in 30 years, and 140 years of service in 60 years. The existing structure should not be considered to meet the current design standards based on the year of design and construction, and the service life and reliability of the structure would be reduced accordingly.

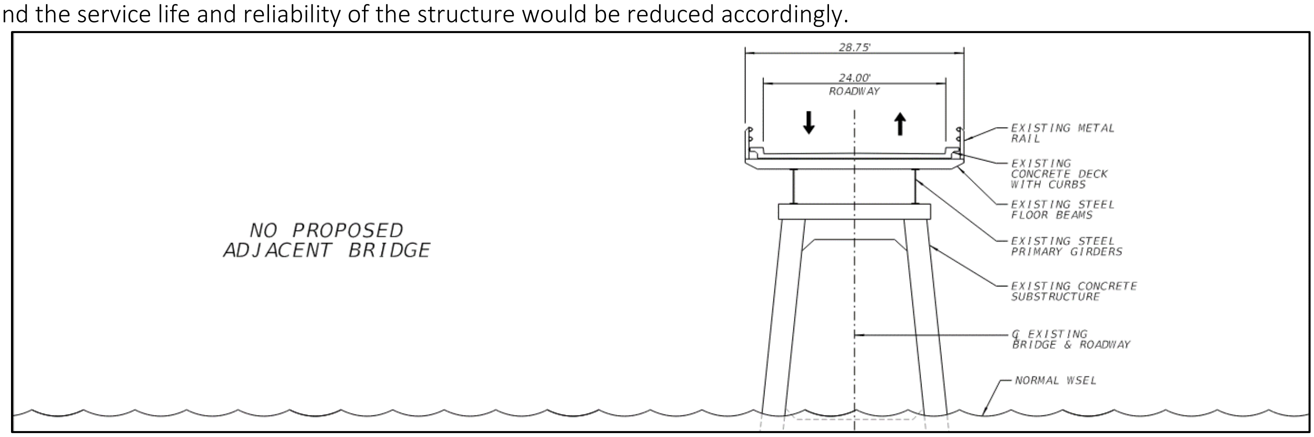


Figure 42: Alternative 1 Typical Section (Approach Spans)

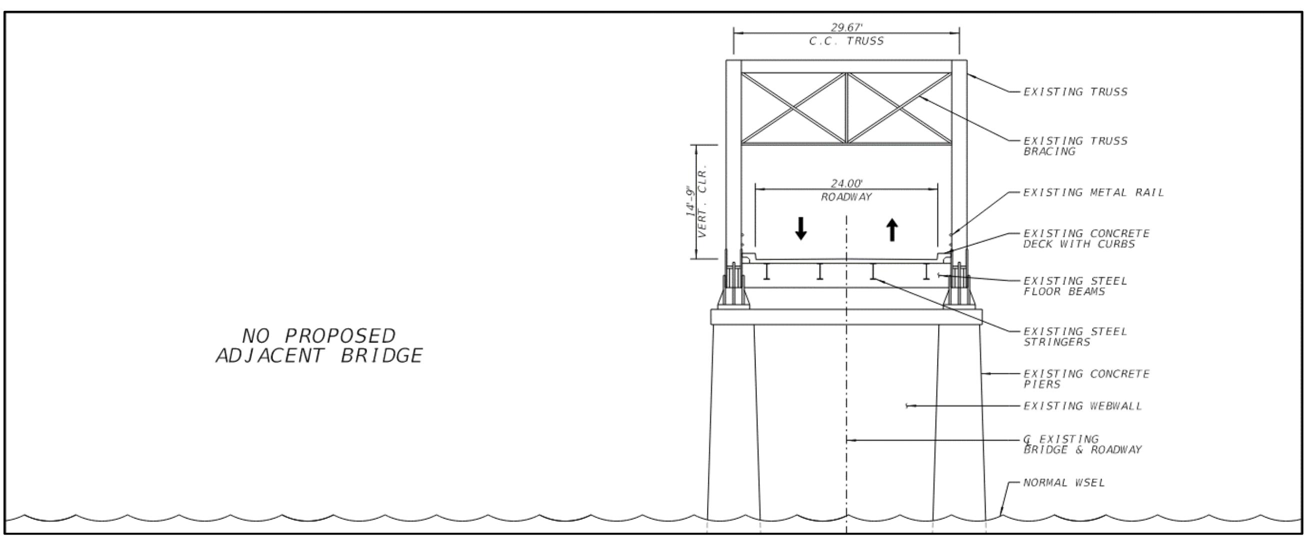


Figure 43: Alternative 1 Typical Section (Truss Span)

4.1.1.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. Due to the substandard roadway width, substandard vertical clearance, and structural deficiencies of the existing bridge, this alternative does not provide a safe crossing; and therefore, does not meet the purpose of the project. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative does not provide any correction to the deficiencies of the existing bridge and roadway; and therefore, does not meet the need of the project.

4.1.1.2 Existing Structural Capacity

The Roosevelt Bridge was designed in the early 1940's, and as such, it was designed to accommodate loadings that were anticipated to be seen during the life of the structure. However, the AASHTO bridge design specifications have evolved since the design of this structure including the weight of vehicular design loadings and the application of wind loadings, braking loadings, and impact loadings. As discussed during the scoping of the project, in order to make appropriate comparisons, the analysis of this structure was performed in accordance with the current *AASHTO LRFD Bridge Design Specifications*, and Alternative 1 serves as a baseline to compare all re-use and replacement alternatives. The design live load associated with this alternative is HL-93 (See Figure 44 for comparison between HL-93 and original design live load) and the "Oklahoma Overload Vehicle" (ODOT Permit Load). After analyzing the existing structure (including any section loss or other observed deterioration), components of the superstructure and the substructure were found to have capacities that did not meet the current design requirements.

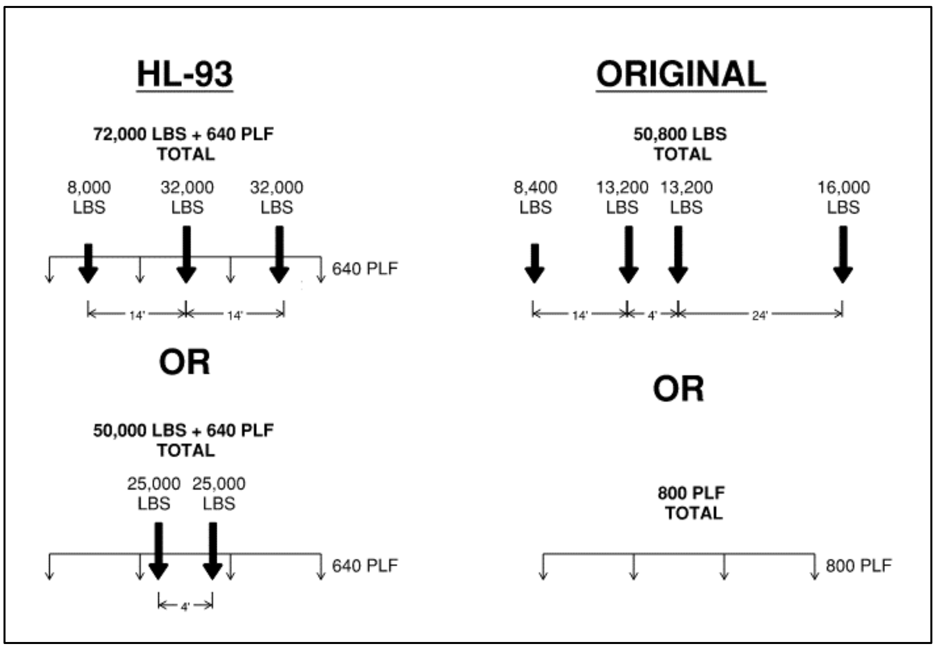


Figure 44: HL-93 vs. Original Live Load

Superstructure – Approach Spans

A summary of controlling performance ratios (PRs) for the approach span superstructure is provided in Table 4. Table 5 indicates the extent of the superstructure deficiencies across the entire bridge. Of note are the significant capacity deficiencies seen in the existing deck, bridge rail, and floor beams. These deficiencies can be attributed to the increase in live load demand as well as deterioration of members compared to when the bridge was originally designed. The current HL-93 loading produces load demand effects roughly twice that of the live load the bridge was originally designed for. The bridge rail also fails under the current MASH criteria TL-4 impact force of 54 kips.

Table 4: Alternative 1 – Superstructure (Approach Spans) Performance Ratio Summary

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	3.53	4.11	-	-	-
Bearings	-	-	-	-	-
Bridge Rail	18.73	-	-	-	4.20
Floor Beams	2.14	1.63	1.67	1.33	1.61
Utility Tower Frame	1.02	1.06	1.07	0.45	0.86
Wind Bracing	-	-	0.14	-	0.10
Primary Girders (G-60)*	1.06	-	0.87	1.00	1.28
Primary Girders (G-34)**	1.14	-	0.79	1.03	1.23

** 60'-1" Spans*
*** 34-0" Spans*

Table 5: Alternative 1 – Superstructure (Approach Spans) Deficiency Summary

Component	Percent Deficient
Concrete Deck	100%
Bridge Rail	100%
Floor Beams	100%
Utility Tower Frame	100%
Wind Bracing	0%
Primary Girders	100%

Superstructure – Truss Span

A summary of controlling PRs for the truss span superstructure is provided in Table 6. Table 7 indicates the extent of the deficiencies. Similar to the approach spans, the elements with the most deficient capacities are the deck, floor beams, and stringers. The truss members, however, perform adequately even under the latest live load conditions.

Table 6: Alternative 1 – Superstructure (Truss Span) Performance Ratio Summary

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	1.00	1.15	-	-	-
Floor Beams	1.15	-	0.82	0.48	-
Stringers	1.30	-	0.72	0.42	-
Truss Members	-	-	-	-	0.98
Truss Bracing	-	-	-	-	0.55

Table 7: Alternative 1 – Superstructure (Truss Span) Deficiency Summary

Component	Percent Deficient
Concrete Deck	100%
Floor Beams	100%
Stringers	100%
Truss Members	0%
Truss Bracing	0%

Substructure

A summary of controlling PRs for the substructure is provided in Table 8. Table 9 indicates the extent of the substructure deficiencies (Strength Only) across the entire bridge. As evident in the PR summary table, there are bents and towers that fail flexural and shear strength requirements; however, a majority of the substructure elements have sufficient reserve capacity to withstand the current AASHTO loading criteria.

The deficiencies with the bents are only found in the column members, and these capacity failures can be attributed to the AASHTO Strength III Load Combination. This load combination is controlled by wind forces regardless of the magnitude of live load. The options to address these deficiencies would include designing for a lower wind load, distributing the lateral and longitudinal loads to new substructure elements retrofitted to the existing substructure, or performing underwater jacketing to strengthen the existing columns.

The deficiencies with the towers are primarily found in the longitudinal and transverse struts that allow the tower bents to act as a three-dimensional frame.

Figure 45 through Figure 50 depict the specific members of the bent/tower models with PRs greater than 1 (purple indicates members with PRs less than 1.10, and orange indicates members with PRs greater than 1.10). Additionally, a summary of the controlling failure modes and AASHTO criteria for the members of each substructure model is provided in Appendix G.

Table 8: Alternative 1 – Substructure Performance Ratio Summary

Type	Model Designation	Occurs	Axial Strength		Flexural Strength	Shear & Torsion Strength
			Tension	Compression		
Bents	B59	1	-	0.27	1.09	0.84
	B67	2	-	0.28	1.32	0.97
	B83	2	-	0.27	0.94	0.90
	B87	3	-	0.27	0.98	0.91
	B90	16	-	0.27	0.98	0.95
	B93	10	-	0.26	0.90	0.98
	B94	2	-	0.26	0.85	0.98
	B109	3	-	0.27	1.13	0.95
Towers	T31	1	-	0.25	0.86	1.06
	T55	2	0.00	0.33	1.01	1.27
	T76	1	0.01	0.32	0.67	0.99
	T87	3	0.01	0.34	0.73	1.00
	T93	13	0.00	0.35	0.80	0.99
	T95	1	0.00	0.29	0.79	1.00
	T100	1	0.01	0.29	0.83	1.09
Piers	P103	2	-	0.75	1.00	0.89
Abutments	A1	1	0.01	0.32	0.67	0.39
	A88	1	-	-	-	-

Table 9: Alternative 1 – Substructure Deficiency Summary

Component	Percent Deficient
Bents	15%
Towers	18%
Piers	0%
Abutments	0%

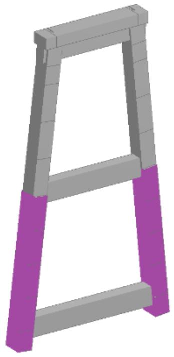


Figure 45: B59 Failing Members

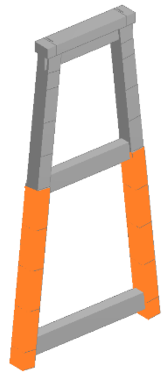


Figure 46: B67 Failing Members

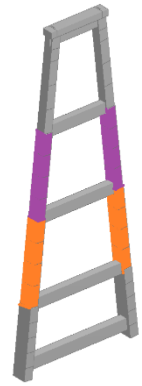


Figure 47: B109 Failing Members

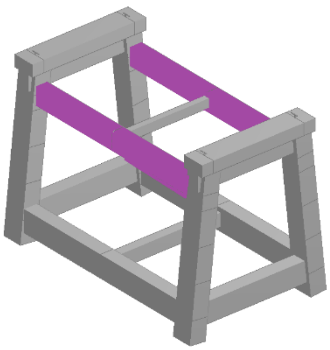


Figure 48: T31 Failing Members

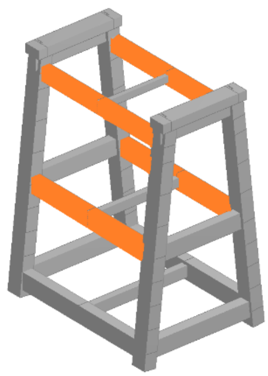


Figure 49: T55 Failing Members

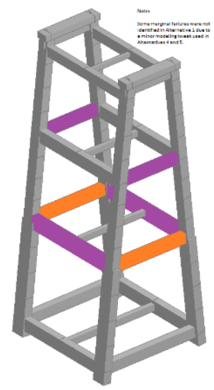


Figure 50: T100 Failing Members

4.1.1.3 Anticipated Rehabilitation & Character Defining Feature Modifications

This alternative does not provide any rehabilitation of the existing structure; however, due to the past vehicular impacts to the truss end portal frames and bridge rail, it is recommended that those members be repaired immediately.

No modifications to the character-defining features of the truss span will be made with this alternative.

4.1.1.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts

Economic impacts of Alternative 1 would include increased delays, future load posting and eventual full closure that would affect freight travel times and reliability. Load posting of the bridge is likely due to the bridge being at risk of becoming structurally deficient. In this event, heavy truck traffic would have to be re-routed to a detour of up to 40 miles (See Section 2.1.3 for discussion on the detour route), increasing travel times and operating costs. The substandard vertical clearance would continue to be a risk for trucks, and as traffic volumes increase, the possibility of trucks impacting the truss end portal frames will increase leading to more short-term closures to repair the bridge.

This alternative would have little to no immediate environmental impact, as no work would be performed on the existing bridge or highway. Impacts to noise and air quality could occur as traffic increases and congestion worsens in the future. Alternative 1 is not anticipated to have an adverse effect on the historic Roosevelt Bridge.

Community Impacts

This alternative would have future impacts to the community. The current roadway width does not provide sufficient traffic capacity or safety for this corridor and as tourism and development expands in this region, traffic demands will increase and elevate the importance of this corridor for the traveling public. If the roadway capacity does not change, traffic delays will increase, and a higher frequency of vehicular collisions is more likely. Collisions on the narrow bridge make access for emergency vehicles difficult and often require closures to through traffic. Load postings and eventual closure of the bridge would have impacts to the surrounding communities, as lengthy detours with associated user costs would be required.

4.1.1.5 Estimated Life Cycle Cost

The future costs associated with this alternative include periodic inspections, and standard maintenance (re-decking, joint repairs and painting steel) for up to 60 years. The user costs associated with this alternative assume the bridge is load posted in 30 years, and 30 years after that, the bridge is permanently closed to traffic. These costs are mostly controlled by the required detour (See Section 2.1.3 for discussion on detours). Additional user costs include the safety costs associated with an increased number of vehicular crashes but are only a fraction of the costs due to load posting and closing the bridge. The user costs do not include impacts due to level of service because of the limitations of the traffic modeling; however, delays as a result of substandard roadway width and traffic demand increasing will certainly have an effect on user costs.

Life Cycle Cost Summary

- Construction Cost: $0
- Future Cost: $7.1 Million
- User Cost: $1.88 Billion
- Total Life Cycle Cost: $1.89 Billion

See Section 5.0 for a cost comparison for all alternatives (Figure 79 and Figure 80)

4.1.2 Alternative 2A: Rehab Existing Bridge (No Widening)

Alternative 2A will leave the existing bridge with its current alignment, traffic capacity, and general geometric configuration, but will rehabilitate the superstructure and substructure members to meet current AASHTO design loads. However, no proposed adjacent bridge will be provided to increase traffic capacity. Safety improvements with this alternative is the replacement of the existing bridge rail. The TxDOT Type T2P rail was selected as a rail replacement option due to its metal post-and-beam features that are similar to the existing rail. This alternative also provides a retrofit to the truss span to increase the vertical clearance to the truss.

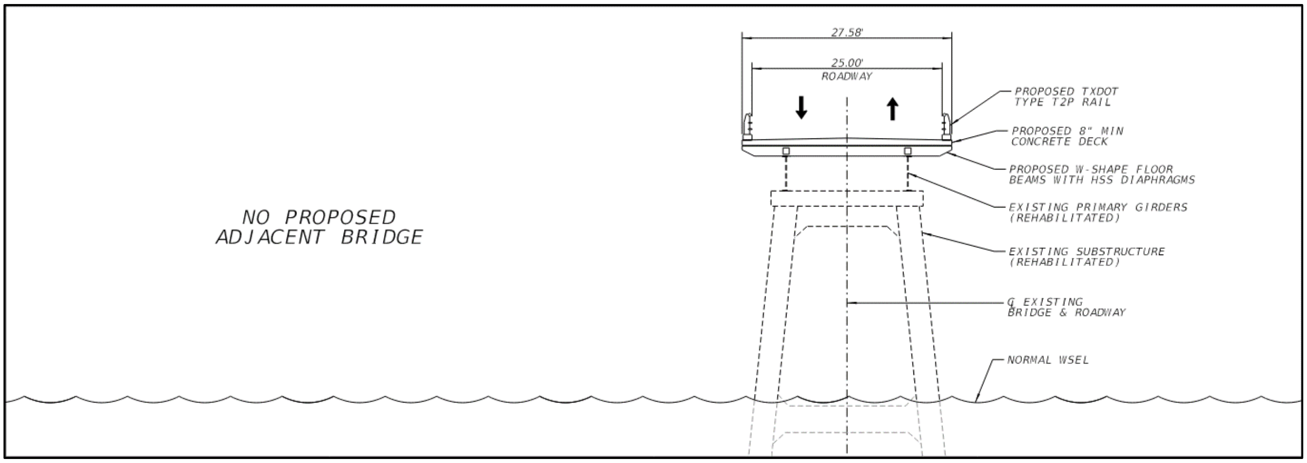


Figure 51: Alternative 2A Typical Section (Approach Spans – Not Showing Substructure Retrofits)

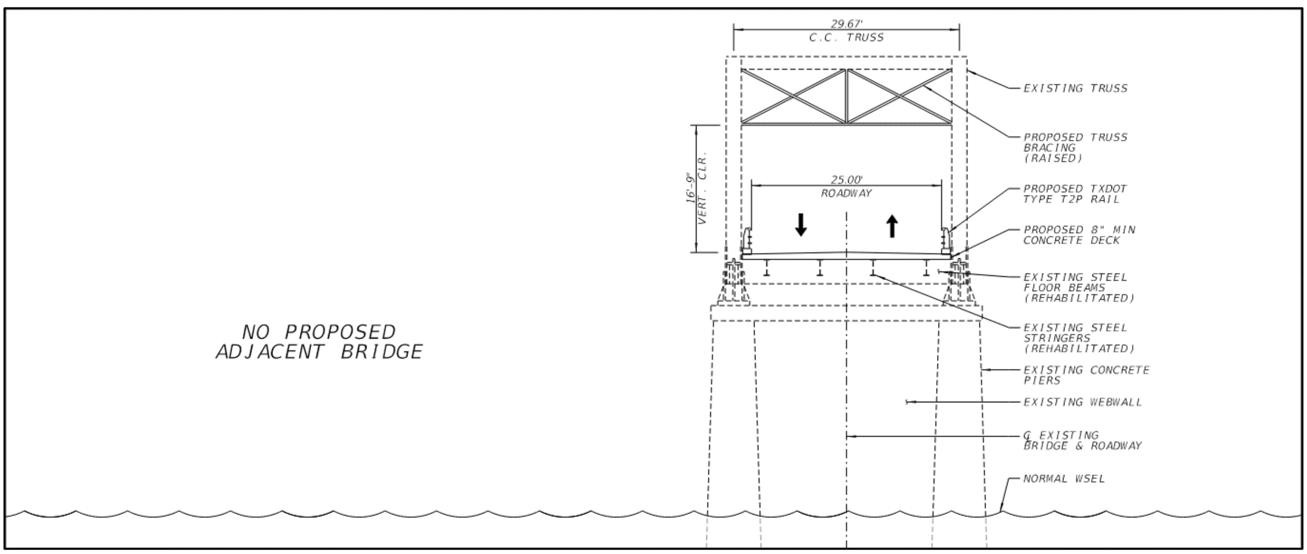


Figure 52: Alternative 2A Typical Section (Truss Span)

4.1.2.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. Due to the substandard roadway width of the existing bridge, this alternative does not provide the necessary safety to meet the purpose of the project. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative provides corrections to the structural deficiencies and vertical clearance improving some of the safety concerns; however, the substandard roadway width is not corrected. The corrections in this alternative only partially meet the need of the project.

4.1.2.2 Existing Structural Capacity

Superstructure & Substructure

As presented in Alternative 1, the existing structural capacity of both the superstructure and substructure elements do not meet current AASHTO design loading specifications, and rehabilitation will be required for this alternative. Refer to Section 4.1.1 for PR summaries for the superstructure and substructure.

4.1.2.3 Anticipated Rehabilitation & Character Defining Feature Modifications

To bring the bridge components' structural capacities up to the latest design requirements, multiple rehabilitations including replacement of some components and retrofits to others will be required. Appendix D provides details for each of the rehabilitations that will be required. The goal of the rehabilitations is to keep as much of the existing structure as possible to minimize impacts to the historic integrity of the bridge. Where structural components must be replaced, components that are as similar to the existing ones will be used. The bridge rail will be the primary structural component that differs the most from the existing bridge. A 3D rendering of the proposed bridge rail is shown in Figure 54. This rail concept is only one option; if this alternative was selected, other options such as a concrete F-shape parapet can be further studied.

Figure 53: Existing Metal Bridge Rail

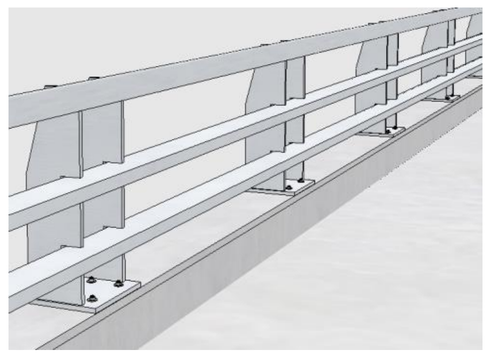


Figure 54: Rendering of Proposed T2P Bridge Rail

The following is a summary of the anticipated rehabilitation work.

Approach Span Replacement & Retrofit Components

- Replace the existing concrete deck with a new 8" minimum reinforced concrete deck with new joints.
- Replace the existing metal bridge rail with a MASH TL-4 rated barrier.
- Replace the existing steel W16x45 floor beams with new steel W16x67 beams and add HSS steel tubes as diaphragms to help provide rotational resistance for the new bridge rail.
- Replace all existing steel bearings on the primary girders with elastomeric bearing pads.
- Retrofit the existing steel primary girders with additional steel cover plates and stiffeners.

Truss Span Replacement & Retrofit Components

- Replace the existing concrete deck with a new 8" minimum reinforced concrete deck with new joints.
- Replace the existing metal bridge rail with a MASH TL-4 rated barrier.
- Replace the portal frames and sway bracing to increase vertical clearance.
- Retrofit existing stringers and floor beams with shear studs to make the beams composite with the new deck.

Substructure Retrofit Components

- Bents (Redistribute load to new substructure elements)
    - Add additional drilled shafts on either side of the existing columns.
    - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
- Towers (Redistribute load to new substructure elements)
    - Add additional drilled shafts on either side of the existing columns.
    - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
    - Add at-water-level longitudinal struts between the drilled shafts.

If this alternative is selected, another option that can be investigated for rehabilitating the tower struts is retrofitting additional struts and substructure cross frame members to help redistribute the load from the existing struts.

Modifications to the character-defining features of the truss span will be made with this alternative; specifically, the lateral sway bracing members will require replacement to increase the vertical clearance. All other modifications to the truss span members apply to the deck, rail, floor beams, and stringers, all of which would not be considered character-defining features under Criterion C. Reference Appendix D for a color-coded diagram of the level of modification for all truss members.

4.1.2.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts

Economic impacts of Alternative 2A are similar to Alternative 1, except for the potential for load posting/closure which would be eliminated under Alternative 2A. The inadequate roadway capacity of the bridge would continue to negatively affect travel times and freight reliability, as congestion would worsen and the potential for collisions and/or bridge strikes would continue to increase. Freight would also be affected by a temporary closure during construction, which could last approximately 2 years. Not only will a detour be needed for rehabilitating the existing bridge in the near-term, detours for future maintenance (deck replacement) will also be required, adding more periodic disruptions. During closures freight vehicles would be subject to a detour of up to 40 miles, which would have impacts on travel times and operating costs.

Environmental impacts of Alternative 2A are anticipated. While much of the bridge would be reconstructed, work would occur primarily within the existing footprint of the structure. Additional drilled shafts could cause impacts to archeological Site 34BR11, which is mapped as submerged beneath Lake Texoma at the east end of the Roosevelt Bridge. Impacts to the other archeological sites, USACE lands, and Lake Texoma State Park are not anticipated under Alternative 2A. Similar to Alternative 1, impacts to noise and air quality could occur under Alternative 2A as traffic increases and congestion worsens in the future.

Alternative 2A would involve rehabilitation of certain elements of the bridge. The truss span would be retrofitted to provide adequate vertical clearance (16’-9”). The existing cross bracing would be replaced with a similar steel structure to maintain the visual appearance of the truss. The existing bridge railing will be replaced with a crash-tested, historically consistent design. Many other major components of the bridge’s substructure and superstructure not related to the bridge’s significance will be replaced or retrofitted. Alternative 2A is not anticipated to have an adverse effect on the historic Roosevelt bridge.

Community Impacts

This alternative is similar to Alternative 1 with the impacts to the community regarding safety and delays; however, load posting the bridge would not be a potential for this alternative. See Section 4.1.1.4 for more discussion on these impacts. Community traffic would also be affected by the construction detour. This alternative, however, addresses the structural deficiencies of the bridge, provides adequate vertical clearance, and improves the safety with a new bridge rail.

4.1.2.5 Estimated Life Cycle Cost

The construction cost includes all the retrofit and replacement work required to rehabilitate the existing bridge. The future costs associated with this alternative include periodic inspections and maintenance (deck replacement, joint repairs, deck repairs, and painting steel members). The user costs associated with this alternative are mostly composed of the construction detour for the rehabilitation work and future major detours required for deck replacements (See Section 2.1.3 for discussion on detours). These user costs are contributed to the near-term and long-term length of detour required and no adjacent bridge or wider bridge to temporarily detour traffic to. Additional user costs include the safety costs with vehicular crashes. The user costs do not include impacts due to level of service because of the limitations of the traffic modeling; however, delays as a result of substandard roadway width and traffic demand increasing will have an effect on user costs.

Life Cycle Cost Summary

- Construction Cost: $55.9 Million
- Future Cost: $6.4 Million
- User Cost: $203.4 Million
- Total Life Cycle Cost: $265.7 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

4.1.3 Alternative 3A: One-Way Pair & Rehab Existing Bridge (No Widening)

Alternative 3A is the same as Alternative 2A with the following difference: a one-way pair is created by constructing a new "sister" bridge adjacent to the existing bridge allowing for two lanes of traffic in one direction on the existing bridge (no shoulders) and two lanes of traffic in the other direction on the proposed bridge (with shoulders).

The proposed "sister" bridge will be built at an offset alignment from the existing bridge and provide the required roadway width (40') for two 12-foot lanes of traffic in one direction with 8-foot shoulders. The superstructure will either be steel plate girders, prestressed concrete girders, or post-tensioned spliced girders. The substructure concept shown provides columns and T-beam caps that mimic the shape and look of the existing bridge columns and caps. The proposed columns will be supported on straight drilled shafts below the normal water surface elevation. Span arrangements for the proposed bridge will not necessarily match the existing spans; they will be optimized for efficiency and cost-effectiveness. The long span of the proposed bridge may implement a signature structure such as a tied arch as discussed further in the Signature Bridge Type Study (Garver, 2021).

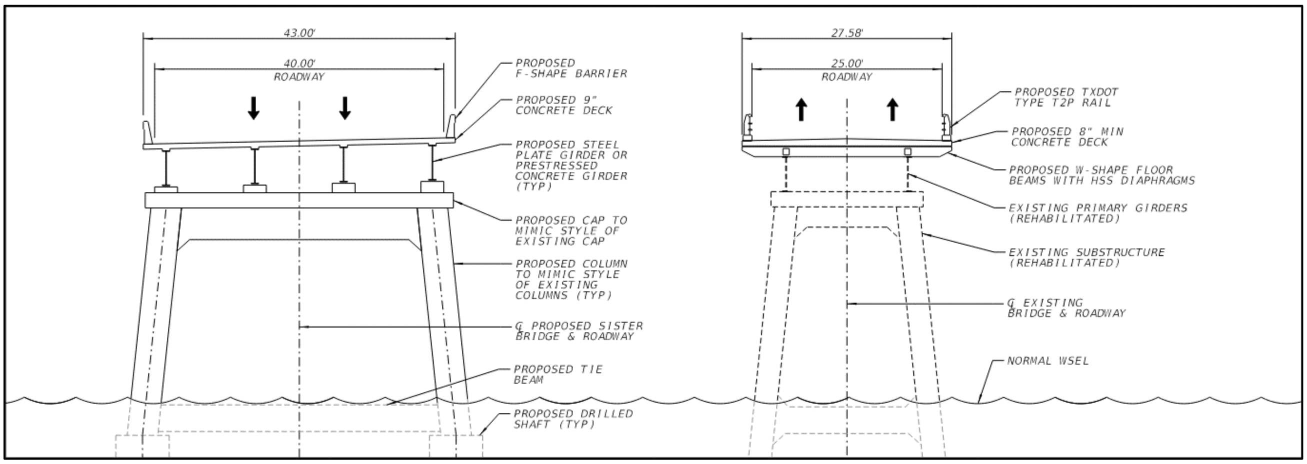


Figure 55: Alternative 3A Typical Section (Approach Spans – Not Showing Substructure Retrofits)

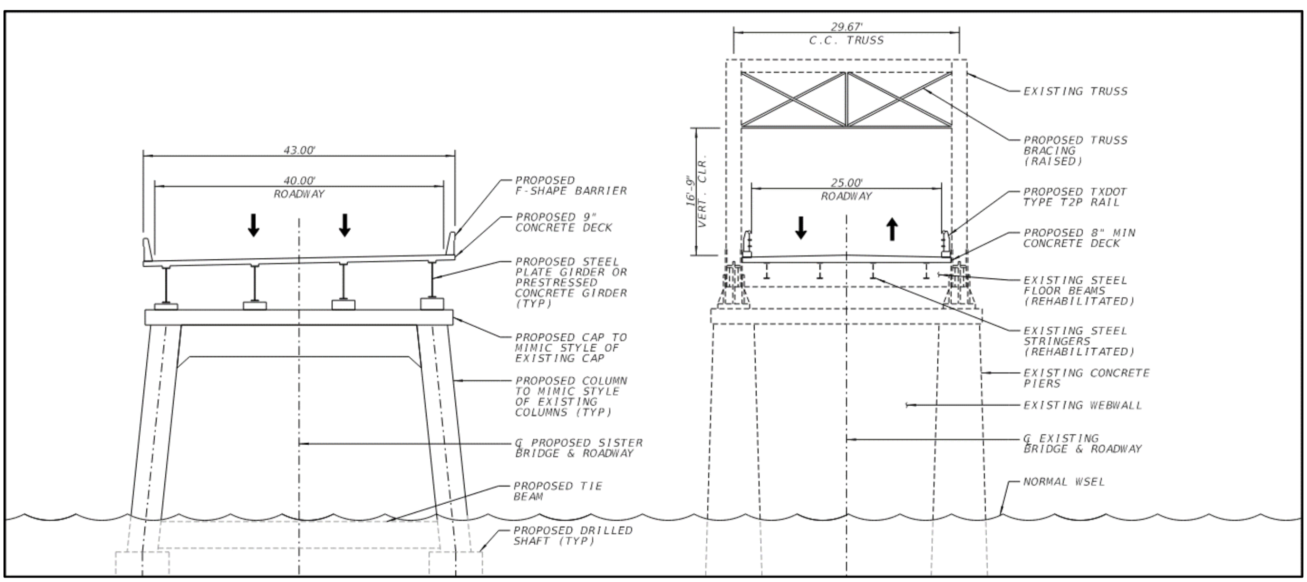


Figure 56: Alternative 3A Typical Section (Truss Span)

4.1.3.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. This alternative provides a crossing that meets current and future traffic demands by providing four total lanes of traffic; however safety is not fully improved on the existing bridge due to the lack of shoulders. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative provides corrections to the existing bridge structural deficiencies improving some of the safety concerns; however, the existing roadway width does not provide the recommended shoulder width for two lanes of traffic in one direction. This alternative does not fully meet the need of the project due to the existing roadway width, but with the addition of an adjacent sister bridge, the alternative is closer to meeting the need of the project than Alternative 2A.

4.1.3.2 Existing Structural Capacity

See Alternative 1 (Section 4.1.1) and Alternative 2A (Section 4.1.2) for discussion on existing structural capacity.

4.1.3.3 Anticipated Rehabilitation & Character Defining Feature Modifications

See Alternative 2A (Section 4.1.2) for discussion on anticipated rehabilitation to the existing structure. The same rehabilitation will apply for Alternative 3A and the same character-defining features of the truss span will be modified.

4.1.3.4 Environmental & Community Impacts

<u>Social, Economic, and Environmental Impacts</u>

Economic impacts of Alternative 3A would be reduced over Alternatives 1 and 2A as the capacity of the one-way pair would accommodate anticipated future traffic volumes. However, safety concerns would remain on the existing bridge as shoulders would not be provided and the width would remain below today’s design standards. Construction impacts would be reduced, as the new bridge would carry traffic during the rehabilitation of the existing bridge and extensive detours would not be required.

Environmental impacts of Alternative 3A would be increased over Alternatives 1 and 2A due to the construction of a new two-lane bridge. For the purposes of this report, the new bridge is assumed to be constructed on a north offset to the existing bridge. Construction of the new bridge could impact archeological site 34BR11 which is mapped as submerged beneath Lake Texoma. Because Alternative 3A would include a widened causeway east of the bridge, there would be fill placed in Lake Texoma, which could require compensatory storage. Impacts to USACE lands would occur due to the widened/new causeway, including potential minor impacts to Johnson Creek Public Use Area. These impacts would be confined to the portion of Johnson Creek located adjacent to US-70.

On the west side of the bridge, Alternative 3A would impact the future PointeVista development on the north side of US-70. The Chickasaw Nation trust property on the south side of US-70 would not be affected. Given that the new bridge would carry 2 lanes of traffic in addition to the 2 lanes provided by the existing bridge, future impacts to air quality due to congestion would not be anticipated. Noise would be anticipated to increase with increased traffic and the additional roadway; however, there are few noise-sensitive land uses within the study area. A noise study would be required if this alternative is selected.

Alternative 3A would involve the same rehabilitation work on the existing bridge as Alternative 2A.

<u>Community Impacts</u>

This alternative improves safety and congestion for the community by providing two lanes of traffic in each direction that accommodates the traffic demands for this corridor. This option has a lack of shoulders on the existing bridge; however, improved safety is still achieved by providing a physical separation between the eastbound and the westbound traffic as the two directions are carried on their own structures; this eliminates head-on collisions within the bridge segment. A full detour is not required for this alternative because a new twin structure is constructed offset from the existing structure allowing it to be used as a temporary route to shift the traffic on to while the existing bridge is rehabilitated. The twin structure also provides future short-term traffic shifts for any inspection or other maintenance work that would be required on the existing bridge which eliminates the need to fully detour the corridor.

4.1.3.5 Estimated Life Cycle Cost

The construction cost includes all the retrofit and replacement work required to rehabilitate the existing bridge (same as Alternative 2) in addition to the construction of the proposed sister bridge which makes up the majority of the overall cost. The future costs associated with this alternative include periodic inspections and maintenance (deck replacement, joint repairs, deck repairs, and painting steel members). The user costs associated with this alternative are mostly composed of the construction detour for the rehabilitation work and future detours required for deck replacements. These user costs are lower than Alternative 1 or 2 because the two separate structures allow for a more cost-effective short-term traffic shift detour onto an adjacent bridge while construction or maintenance is performed on the other bridge. Additional user costs include the safety costs with vehicular crashes; however, these costs are minimized due to having both directions of traffic physically separated along the bridge segment.

<u>Life Cycle Cost Summary</u>

- Construction Cost: $171.3 Million
- Future Cost: $11.7 Million
- User Cost: $22.6 Million
- Total Life Cycle Cost: $205.6 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

4.1.4 Alternative 4: Pedestrian/Bicycle Only

Alternative 4 will leave the existing bridge with its current alignment and general geometric configuration, but the existing bridge will be converted (and rehabilitated as necessary) to a pedestrian/bicycle shared use path only. In order to provide sufficient rail height appropriate for pedestrians, a new supplemental pedestrian rail will be provided at the current face of deck curb so that the existing rail can remain on the outside. The supplemental rail will also provide the required maximum gap between elements required for pedestrian rails and restrict the overall usable width of deck to 24’ keeping emergency/maintenance vehicles within more structurally beneficial limits of the floor beam cantilevers.

A new vehicular bridge will be constructed at an offset alignment from the existing bridge and provide the required roadway width (68’) for two 12-foot lanes of traffic in each direction with 10-foot shoulders. The new vehicular bridge alternatives are not included in this report. The superstructure will either be steel plate girders, prestressed concrete girders, or post-tensioned spliced girders. The substructure concept shown provides columns and T-beam caps that mimic the shape and look of the existing bridge columns and caps. The proposed columns will be supported on straight drilled shafts below the normal water surface elevation. Span arrangements for the proposed bridge will not necessarily match the existing spans; they will be optimized for efficiency and cost-effectiveness. The long span of the proposed bridge may implement a signature structure such as a tied arch as discussed further in the Signature Bridge Type Study (Garver, 2021).

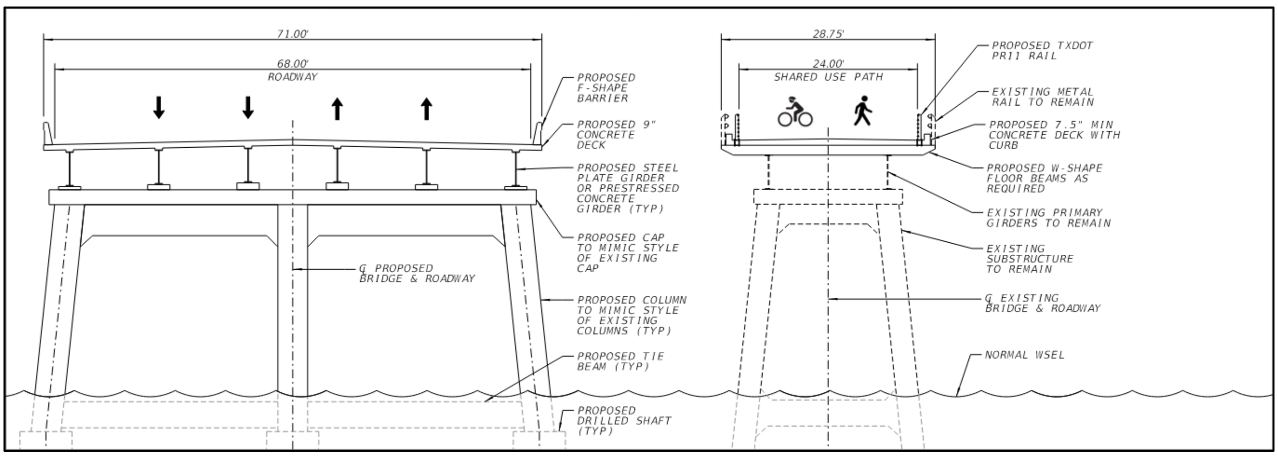


Figure 57: Alternative 4 Typical Section (Approach Spans – Not Showing Substructure Retrofits)

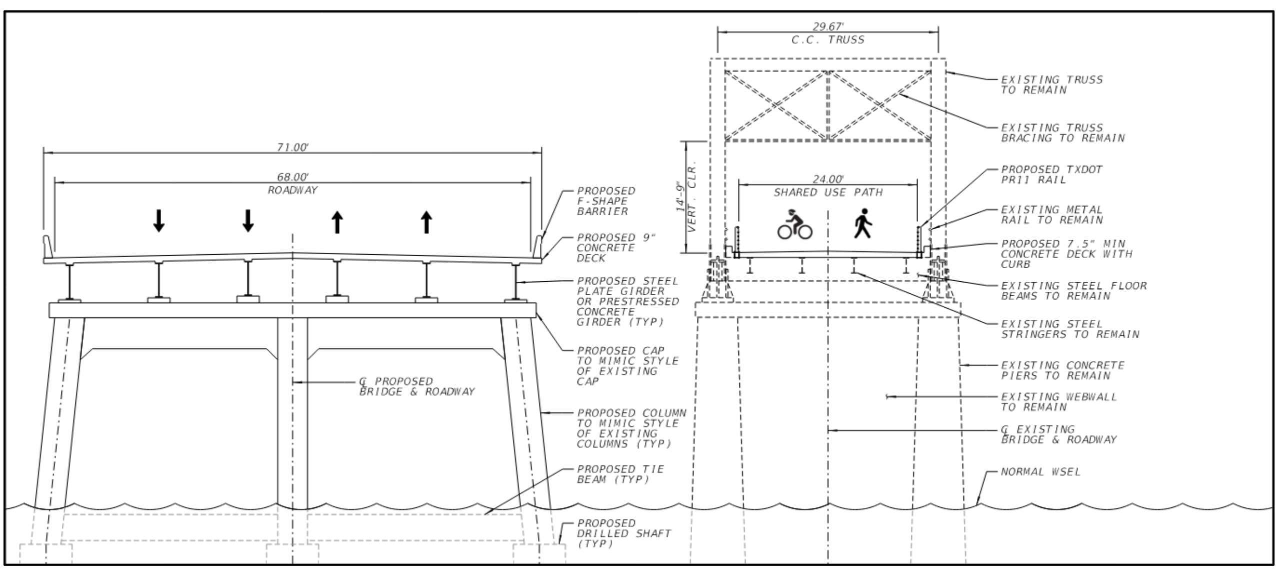


Figure 58: Alternative 4 Typical Section (Truss Span)

4.1.4.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. This alternative provides a crossing that is both safe and meets current and future traffic demands by providing four total lanes of traffic. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative provides corrections to the existing bridge structural deficiencies and shifts traffic completely off the existing bridge, precluding the need to widen the bridge or increase vertical clearance at the truss. This alternative fully meets the need of the project and provides an added benefit of a dedicated pedestrian/bicycle shared use path on the existing bridge.

4.1.4.2 Existing Structural Capacity

The existing structure was analyzed for this alternative using the latest version of *AASHTO LRFD Bridge Design Specifications* and *AASHTO LRFD Design Specification for Design of Pedestrian Bridges*. The design live load associated with this alternative is either a pedestrian area load of 90 psf or an H10 (maintenance/emergency) vehicle, whichever produces the maximum force effects. After analyzing the existing structure with the lighter live load (including any section loss or other observed deterioration), multiple deficiencies in existing structural components were still observed.

Superstructure – Approach Spans

A summary of controlling PRs is provided in Table 10. The primary structural deficiencies for Alternative 4 in the existing approach span superstructure include: the concrete deck, steel floor beams, and the bridge rail (geometric criteria failure). These deficiencies are not as drastic as Alternative 1; however, they must still be addressed to meet the need of the project.

The Floor Beam PRs shown in the table represent the floor beams with the most section loss and are not representative of the majority of floor beams. A number of the end floor beams have section loss in the flanges and web as identified in the inspection report; however, this section loss results in only a small percentage of the floor beams not providing the required capacity to support the live load in this alternative. With the lighter pedestrian load, most of the existing floor beams, even with varying levels of section loss, provide sufficient capacity to support the live load in this alternative. Table 11 indicates the extent of the deficiencies. It should be noted that all of the primary girders have sufficient capacity for supporting the pedestrian live load cases and do not require rehabilitation for this alternative.

Table 10: Alternative 4 – Superstructure (Approach Spans) Performance Ratio Summary (Without Rehab)

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	1.57	1.43	-	-	-
Bearings	-	-	-	-	-
Bridge Rail	0.09	-	-	0.09	0.46
Floor Beams	1.12	1.02	0.56	1.09	0.54
Utility Tower Frame	-	-	-	-	-
Wind Bracing	-	-	0.14	-	0.10
Primary Girders (G-60)	0.55	-	0.62	0.65	0.59
Primary Girders (G-34)	0.52	-	0.49	0.54	0.52

Table 11: Alternative 4 – Superstructure (Approach Spans) Deficiency Summary

Component	Percent Deficient
Concrete Deck	100%
Bridge Rail*	33%
Floor Beams	3%
Utility Tower Frame	0%
Wind Bracing	0%
Primary Girders	0%

**Percent shown corresponds to the number of damaged connections observed in the inspection report*

The existing bridge rail PRs are less than 1.0; however, the existing bridge rail in its current configuration does not meet geometric criteria including the minimum height requirement for a pedestrian rail (42”) and the maximum opening size between elements (4”); the rail is only 31” above the top of the existing curb and has openings between elements greater than 4” (See Figure 60). Furthermore, 33% of the rail connections to the floor beams are severely damaged with some locations missing a connection altogether (See Figure 59).

Figure 59: Missing Rail Connection

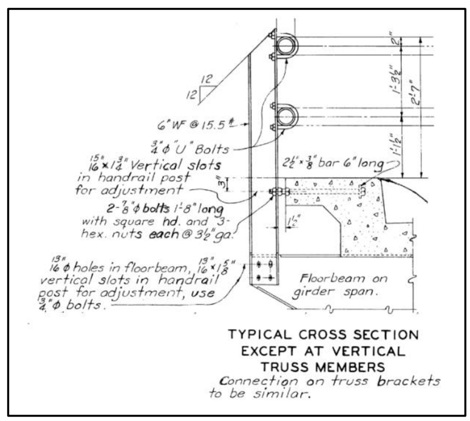


Figure 60: Existing Rail Detail

Superstructure – Truss Span

For Alternative 4, all steel truss members and floor beam framing have sufficient capacity to resist the loading conditions in this alternative. The only component of the truss that will need retrofit or replacement is the low chord of the west portal frame that is damaged from a vehicular impact. A summary of controlling PRs is provided in Table 12.

Table 12: Alternative 4 – Superstructure (Truss Span) Performance Ratio Summary (Without Rehab)

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	0.47	0.58	-	-	-
Floor Beams	0.60	-	0.42	0.24	-
Stringers	0.46	-	0.25	0.15	-
Truss Members	-	-	-	-	0.81
Truss Bracing	-	-	-	-	0.55

Substructure

The substructure capacity under Alternative 4 is similar to the other alternatives. Because the controlling LRFD load combination is Strength III (wind with no live load), the lighter pedestrian live load does not provide a benefit. A summary of controlling PRs is provided in Table 13. See Table 2 for Substructure support line summary and corresponding model designations.

Table 13: Alternative 4 – Substructure Performance Ratio Summary (Without Rehab)

Type	Model Designation	Occurs	Axial Strength		Flexural Strength	Shear & Torsion Strength
			Tension	Compression		
Bents	B59	1	-	0.22	1.03	0.80
	B67	2	-	0.23	1.27	0.95
	B83	2	-	0.21	0.91	0.87
	B87	3	-	0.22	0.95	0.89
	B90	16	-	0.21	0.95	0.93
	B93	10	-	0.21	0.88	0.97
	B94	2	-	0.21	0.84	0.97
	B109	3	-	0.21	1.11	0.93
Towers	T31	1	-	0.18	0.46	0.56
	T55	2	0.00	0.25	0.83	1.02
	T76	1	0.01	0.27	0.62	0.97
	T87	3	0.01	0.29	0.73	1.00
	T93	13	0.00	0.30	0.80	0.99
	T95	1	0.00	0.25	0.79	1.00
	T100	1	0.00	0.26	0.89	1.20
Piers	P103	2	-	0.71	0.89	0.63
Abutments	A1	1	0.02	0.18	0.62	0.32
	A88	1	-	-	-	-

Table 14: Alternative 4 – Substructure Deficiency Summary

Component	Percent Deficient
Bents	15%
Towers	14%
Piers	0%
Abutments	0%

Figure 61 through Figure 65 depict the specific members of the bent/tower models with PRs greater than 1 (purple indicates members with PRs less than 1.10, and orange indicates members with PRs greater than 1.10). Additionally, a summary of the controlling failure modes and AASHTO criteria for the members of each substructure model is provided in Appendix G.

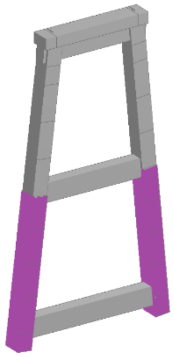


Figure 61: B59 Failing Members

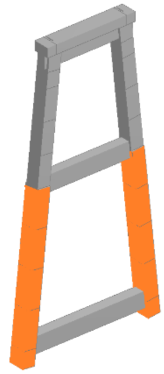


Figure 62: B67 Failing Members

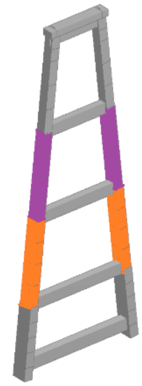


Figure 63: B109 Failing Members

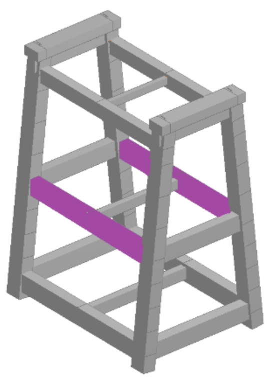


Figure 64: T55 Failing Members

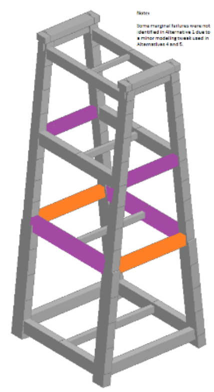


Figure 65: T100 Failing Members

4.1.4.3 Anticipated Rehabilitation & Character Defining Feature Modifications

To bring the bridge components' structural capacities up to the latest design requirements, multiple rehabilitations including replacement of some components and retrofits to others will be required. Appendix D provides details for each of the rehabilitations that will be required. The goal of the rehabilitations is to keep as much of the existing structure as possible to minimize impacts to the historic integrity of the bridge. Where structural components must be replaced, components that are identical or similar to the existing ones will be used. The following is a summary of the anticipated rehabilitation work.

A lightweight concrete deck replacement was considered; however, it had little benefit for the existing superstructure since the normal weight concrete deck option resulted in satisfactory results for the floor beams and primary girders. Furthermore, it was found that the lighter dead load reactions on the substructure adversely affected the column capacities. The reason for this non-intuitive outcome is the nature of the combined flexural and axial interaction behavior of a column. The controlling flexural capacity of the column reduced as the axial load demand was also reduced.

Approach Span Replacement & Retrofit Components

- Replace the existing concrete deck with a new varying deck thickness (7.5" minimum) reinforced concrete deck.
- Replace the existing steel W16x45 floor beams with new steel W16x45 beams as needed.
- Supplement the existing metal bridge rail with a new pedestrian rail anchored to the proposed deck (See Figure 66)
- Re-attach existing bridge rail to existing and proposed floor beams and proposed concrete deck.

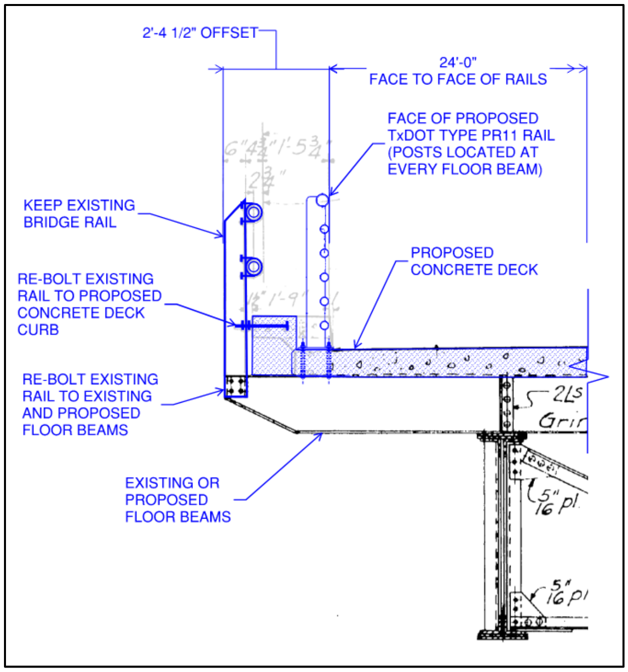


Figure 66: Alternative 4 Bridge Rail Rehabilitation Concept

Truss Span Replacement & Retrofit Components

- Replace damaged low chord of the west portal frame of the truss with new steel angles.
- Replace the existing concrete deck with a new varying deck thickness (7.5" minimum) reinforced concrete deck to maintain proposed deck unity across the entire bridge.
- Supplement the existing bridge rail with a new pedestrian rail anchored to the proposed deck.

Substructure Retrofit Components

- Bents (Redistribute load to new substructure elements)
  - Add additional drilled shafts on either side of the existing columns.
  - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
- Towers (Redistribute load to new substructure elements)
  - Add additional drilled shafts on either side of the existing columns.
  - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
  - Add at-water-level longitudinal struts between the drilled shafts.

No modifications to the character-defining features of the truss span will be made with this alternative. The only modifications to the truss span members apply to the deck and rail, both of which would not be considered character-defining features under Criterion C. Reference Appendix D for a color-coded diagram of the level of modification for all truss members.

4.1.4.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts

Economic benefits would be anticipated under Alternative 4 since a new 4-lane bridge with sufficient width and capacity would be constructed to carry all traffic. This alternative would also increase multimodal transportation opportunities through the provision of bicycle/pedestrian facilities.

Environmental impacts of Alternative 4 would be higher than Alternative 3A due to the new 4-lane bridge (vs. 2-lane bridge under Alternative 3A) constructed adjacent to the existing structure. For the purposes of this report, the new bridge and US-

70 roadway are assumed to be constructed on a north offset to the existing bridge. Alternative 4 would require an offset causeway east of the bridge and would require more fill placed in Lake Texoma than Alternative 3A. Construction of the new bridge could impact archeological site 34BR11, which is mapped as submerged beneath Lake Texoma. Because Alternative 4 would include a widened or new causeway east of the bridge, there could also be potential impacts to Site 34BR25, located on the north side of US-70 on the east side of the lake. Because the new bridge would be 4 lanes wide, impacts to these archeological resources would potentially be greater under Alternative 4 than Alternative 3A. Similarly, impacts to USACE lands and Johnson Creek Public Use Area would be higher under Alternative 4 than Alternative 3A. Alternative 4 would affect 45 acres of USACE property, 0.62 acres of Johnson Creek Public Use Area, 0.28 acres of wetland, and 0.03 acres of stream.

On the west side of the bridge, Alternative 4 would have higher impacts to the PointeVista development than Alternative 3A. Impacts to the Chickasaw Nation trust land are not anticipated. Alternative 4 could impact the gas station property at US-70 and State Park Road, although retaining walls could be included to avoid these impacts. Given that a new bridge would be constructed to accommodate four lanes of traffic, future impacts to air quality due to congestion would not be anticipated. Noise would be anticipated to increase with increased traffic and the additional roadway; however, there are few noise-sensitive land uses within the study area. A noise study would be required if this alternative is selected.

Alternative 4 would involve rehabilitation of certain elements of the historic bridge. The truss span would remain in its existing condition except for repairs to low chord of the west portal frame. The existing bridge railing will remain in place and a new pedestrian rail will be constructed to the inside. Some other components of the bridge's substructure and superstructure not related to the bridge's significance will be replaced or retrofitted. Alternative 4 is not anticipated to have an adverse effect on the historic Roosevelt bridge.

Community Impacts

This alternative reduces disruption to the community and increases multimodal opportunity through the addition of a dedicated pedestrian/bicycle shared use path that utilizes the existing historic bridge. Because a new structure is constructed offset from the existing structure, no short-term detours will be required. Furthermore, because the existing structure would no longer carry vehicular traffic, there would be no need for a long-term closure or load posting due to insufficient structural capacity.

4.1.4.5 Estimated Life Cycle Cost

The construction cost includes all the retrofit and replacement work required to rehabilitate the existing bridge in addition to the construction of the proposed sister bridge and roadway/causeway work which both make up the majority of the overall cost. The future costs associated with this alternative include periodic inspections and maintenance of the proposed vehicular bridge (deck replacement, joint repairs, deck repairs, and painting steel members). The user costs associated with this alternative are mostly composed of future minor detours required for deck replacements and maintenance. Because there is no adjacent vehicular bridge, deck replacements will have to be phased to keep from detouring the corridor. Additional user costs include the safety costs associated with vehicular crashes. These costs are the highest among the alternatives due to adding more lanes and the way in which the crashes are modeled. The most effective way to lower the user costs associated with crashes is to add a median barrier which would increase the bridge width.

Life Cycle Cost Summary

- Construction Cost: $187.2 Million
- Future Cost: $13.2 Million
- User Cost: $47.1 Million
- Total Life Cycle Cost: $247.5 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

4.1.5 Alternative 5: Monument Only

Alternative 5 will leave the existing bridge with its current alignment, geometric configuration, and general structural condition (with minimal rehabilitation as required) and convert it to a monument with no vehicular or pedestrian traffic. A new vehicular bridge will be constructed adjacent to the existing bridge to provide two lanes of traffic in each direction with shoulders. The proposed bridge in this alternative will be identical to the proposed bridge in Alternative 4.

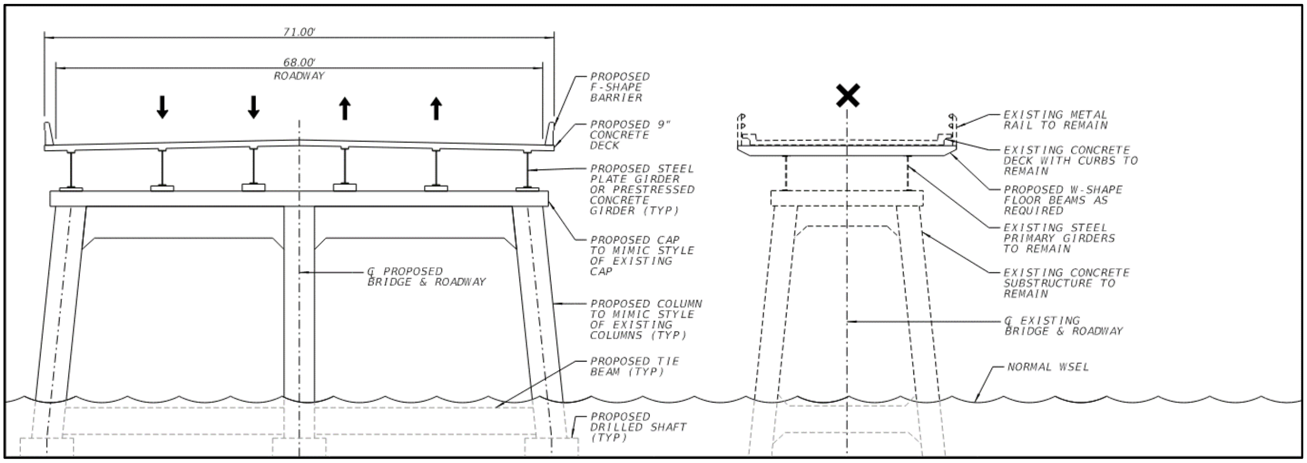


Figure 67: Alternative 5 Typical Section (Approach Spans – Not Showing Substructure Retrofits)

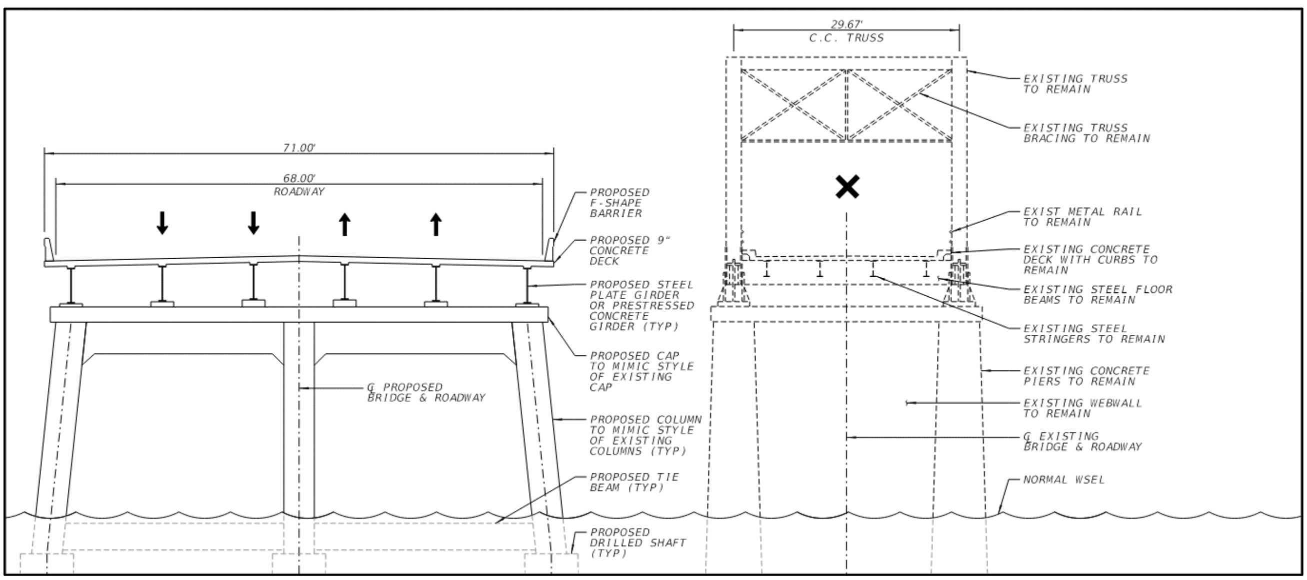


Figure 68: Alternative 5 Typical Section (Truss Span)

4.1.5.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. This alternative provides a crossing that is both safe and meets current and future traffic demands by providing four total lanes of traffic. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative provides corrections to the existing bridge structural deficiencies and shifts traffic completely off the existing bridge precluding the need to widen the bridge or increase vertical clearance at the truss. This alternative fully meets the need of the project.

4.1.5.2 Existing Structural Capacity

The existing structure was analyzed for this alternative using the latest version of *AASHTO LRFD Bridge Design Specifications*. There is no design live load associated with this alternative. After analyzing the existing structure (including any section loss or other observed deterioration) with no live load, multiple deficiencies in existing structural components were still observed; however, they were less than those found in Alternative 4.

Superstructure – Approach Spans

A summary of controlling PRs is provided in Table 15. The majority of the superstructure components for the approach spans are sufficient to support dead load only. The only members that do not have sufficient capacity are a number of the floor beams that have significant deterioration. Table 16 indicates the extent of these deficiencies.

The existing bridge rails for this alternative are not needed to protect pedestrians or vehicles, and therefore technically have "sufficient capacity". However, due to damaged connections, it is recommended that these elements be repaired so that further deterioration does not lead to members falling from the bridge onto lake traffic below. A similar repair condition exists where the existing concrete deck spalls should be repaired.

Table 15: Alternative 5 – Superstructure (Approach Spans) Performance Ratio Summary (Without Rehab)

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	0.12	0.24	-	-	-
Bearings	-	-	-	-	-
Bridge Rail	-	-	-	-	-
Floor Beams	1.00	0.64	0.20	1.00	0.19
Utility Tower Frame	-	-	-	-	-
Wind Bracing	-	-	0.14	-	0.10
Primary Girders (G-60)	0.56	-	0.40	0.66	0.59
Primary Girders (G-34)	0.53	-	0.30	0.54	0.58

Table 16: Alternative 5 – Superstructure (Approach Spans) Deficiency Summary

Component	Percent Deficient
Concrete Deck	0%
Bridge Rail*	33%
Floor Beams	2%
Utility Tower Frame	0%
Wind Bracing	0%
Primary Girders	0%

**Percent shown corresponds to the number of damaged connections observed in the inspection report*

Superstructure – Truss Span

A summary of controlling PRs is provided in Table 17. All of the truss span structural components are sufficient to support dead load only.

Table 17: Alternative 5 – Superstructure (Truss Span) Performance Ratio Summary (Without Rehab)

Superstructure Element	Flexural Capacity		Rivet Capacity	Shear Capacity	Axial Capacity
	Positive Moment	Negative Moment			
Concrete Deck	0.17	0.24	-	-	-
Floor Beams	0.32	-	0.22	0.13	-
Stringers	0.24	-	0.12	0.07	-
Truss Members	-	-	-	-	0.65
Truss Bracing	-	-	-	-	0.55

Substructure

A summary of controlling PRs is provided in Table 18. Table 19 indicates the extent of the deficiencies. The same members as Alternative 4 are failing as well.

Table 18: Alternative 5 – Substructure Performance Ratio Summary (Without Rehab)

Type	Model Designation	Occurs	Axial Strength		Flexural Strength	Shear & Torsion Strength
			Tension	Compression		
Bents	B59	1	-	0.19	1.04	0.80
	B67	2	-	0.20	1.28	0.95
	B83	2	-	0.20	0.91	0.87
	B87	3	-	0.20	0.95	0.89
	B90	16	-	0.20	0.95	0.93
	B93	10	-	0.20	0.89	0.97
	B94	2	-	0.20	0.84	0.97
	B109	3	-	0.21	1.11	0.93
Towers	T31	1	-	0.15	0.46	0.56
	T55	2	0.00	0.24	0.72	1.01
	T76	1	0.01	0.27	0.62	0.97
	T87	3	0.01	0.28	0.73	1.00
	T93	13	0.00	0.29	0.80	0.99
	T95	1	0.00	0.25	0.79	1.00
	T100	1	0.00	0.25	0.89	1.20
Piers	P103	2	-	0.67	0.79	0.60
Abutments	A1	1	0.01	0.15	0.62	0.32
	A88	1	-	-	-	-

Table 19: Alternative 5 – Substructure Deficiency Summary

Component	Percent Deficient
Bents	15%
Towers	14%
Piers	0%
Abutments	0%

Figure 69 through Figure 73 depict the specific members of the bent/tower models with PRs greater than 1 (purple indicates members with PRs less than 1.10, and orange indicates members with PRs greater than 1.10). Additionally, a summary of the controlling failure modes and AASHTO criteria for the members of each substructure model is provided in Appendix G.

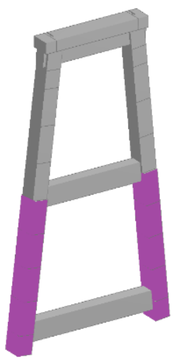


Figure 69: B59 Failing Members

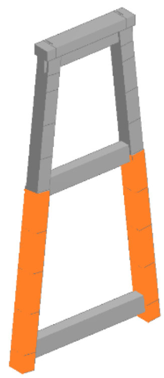


Figure 70: B67 Failing Members

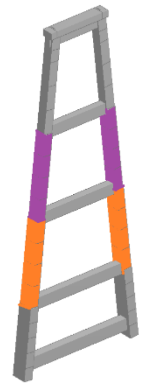


Figure 71: B109 Failing Members

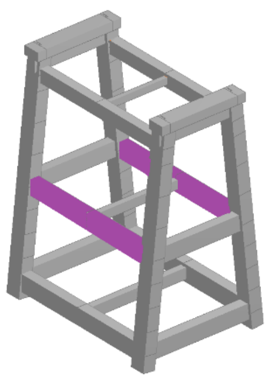


Figure 72: T55 Failing Members

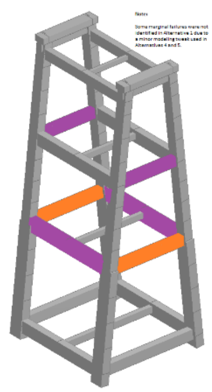


Figure 73: T100 Failing Members

4.1.5.3 Anticipated Rehabilitation & Character Defining Feature Modifications

To bring the bridge components' structural capacities up to the latest design requirements, multiple rehabilitations including replacement of some components and retrofits to others will be required. Appendix D provides details for each of the rehabilitations that will be required. The goal of the rehabilitations is to keep as much of the existing structure as possible to minimize impacts to the historic integrity of the bridge. Where structural components must be replaced, components that are identical or similar to the existing ones will be used. The following is a summary of the anticipated rehabilitation work.

Approach Span Replacement & Retrofit/Repair Components

- Replace the existing deficient steel W16x45 floor beams with new steel W16x45 beams as needed.
- Re-attach existing bridge rail to existing and proposed floor beams and existing concrete deck.
- Patch spalls in concrete deck soffit to prevent loose concrete from falling on lake traffic

Substructure Retrofit Components

- Bents (Redistribute load to new substructure elements)
  - Add additional drilled shafts on either side of the existing columns.
  - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
- Towers (Redistribute load to new substructure elements)
  - Add additional drilled shafts on either side of the existing columns.
  - Add diagonal and horizontal struts to tie the existing substructure to the proposed drilled shafts.
  - Add at-water-level longitudinal struts between the drilled shafts.

No modifications to the character-defining features of the truss span will be made with this alternative.

4.1.5.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts
Economic benefits of Alternative 5 are the same as Alternative 4.

Environmental impacts of Alternative 5 would be higher than Alternative 4. Construction of the new bridge could impact archeological site 34BR11 which is mapped as submerged beneath Lake Texoma. Alternative 5 would require a wider lake causeway that would involve a similar amount of fill in Lake Texoma as Alternative 4. However, because Alternative 5 does not require maintenance of the existing causeway for bicycles and pedestrians, it utilizes more of the existing causeway and highway easement on the east side of the lake for the roadway. Impacts to USACE property are therefore reduced. Impacts to Site 34BR25 on the east side of the lake are not anticipated. Alternative 4 would affect 41 acres of USACE property, 0.72 acres of Johnson Creek Public Use Area, 0.65 acres of wetland, and 0.06 acres of stream. Impacts on the west side of the lake would be the same as Alternative 4, including possible impacts to the gas station at US-70 and State Park Road. Retaining walls could be used to avoid this impact.

Alternative 5 would involve rehabilitation of certain elements of the bridge. The truss span would remain in its existing condition. The existing bridge railing will remain in place and be re-attached to the deck. Some other components of the bridge's substructure and superstructure not related to the bridge's significance will be replaced or retrofitted. Alternative 5 is not anticipated to have an adverse effect on the historic Roosevelt bridge.

Community Impacts
This alternative provides the same benefits to the community as Alternative 4 except the added benefit of a dedicated pedestrian/bicycle shared use path is not provided.

4.1.5.5 Estimated Life Cycle Cost

The life cycle costs for this alternative are similar to Alternative 4. The construction cost is less due to a smaller extent of rehabilitation work and less roadway/causeway construction required.

Life Cycle Cost Summary

- Construction Cost: $158.1 Million
- Future Cost: $13.2 Million
- User Cost: $47.1 Million
- Total Life Cycle Cost: $218.4 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

4.2 Use Alternatives

The two “Use” alternatives studied for this Section 4(f) report require modification to the existing structure (both the approach spans and truss span). These alternatives would likely have an adverse effect on the existing structure.

4.2.1 Alternative 2B: Rehab Existing Bridge (With Widening)

Alternative 2B will leave the existing bridge with its current alignment but widen the bridge to provide two lanes of traffic in each direction with a new roadway width of 64’ (including shoulders) and rehabilitate and repurpose the superstructure and substructure members to meet current AASHTO design loads. This option includes changes to the superstructure and substructure. The bridge deck is replaced, new F-shape concrete parapet bridge rails are used, existing floor beams are removed, additional steel plate girders are added, the existing cap is replaced, and new columns and foundations are provided to support the widening. The truss requires replacing the floor system and deck, replacing the bracing, and retrofitting the existing truss members and connections.

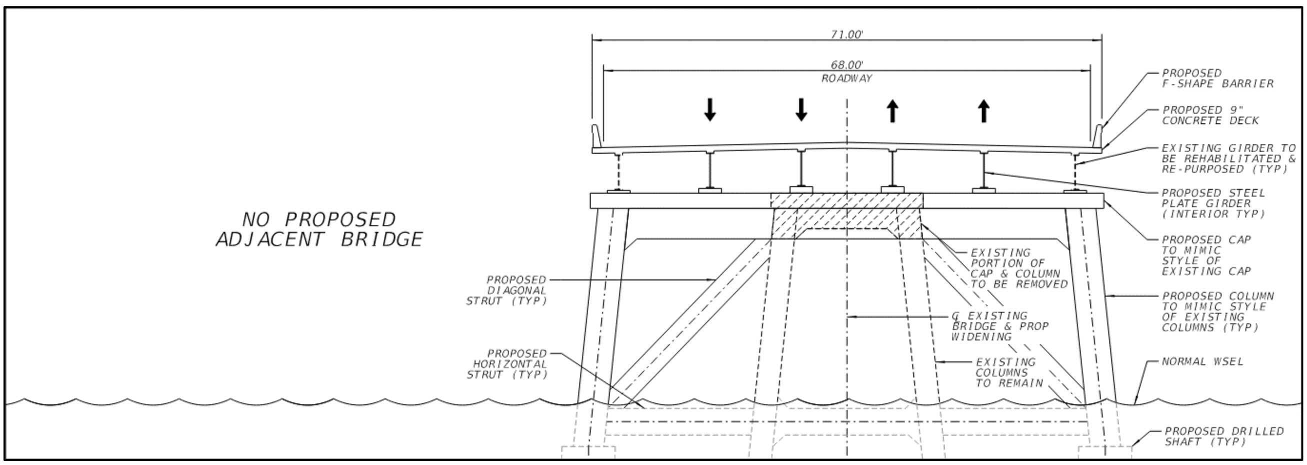


Figure 74: Alternative 2B Typical Section (Approach Spans)

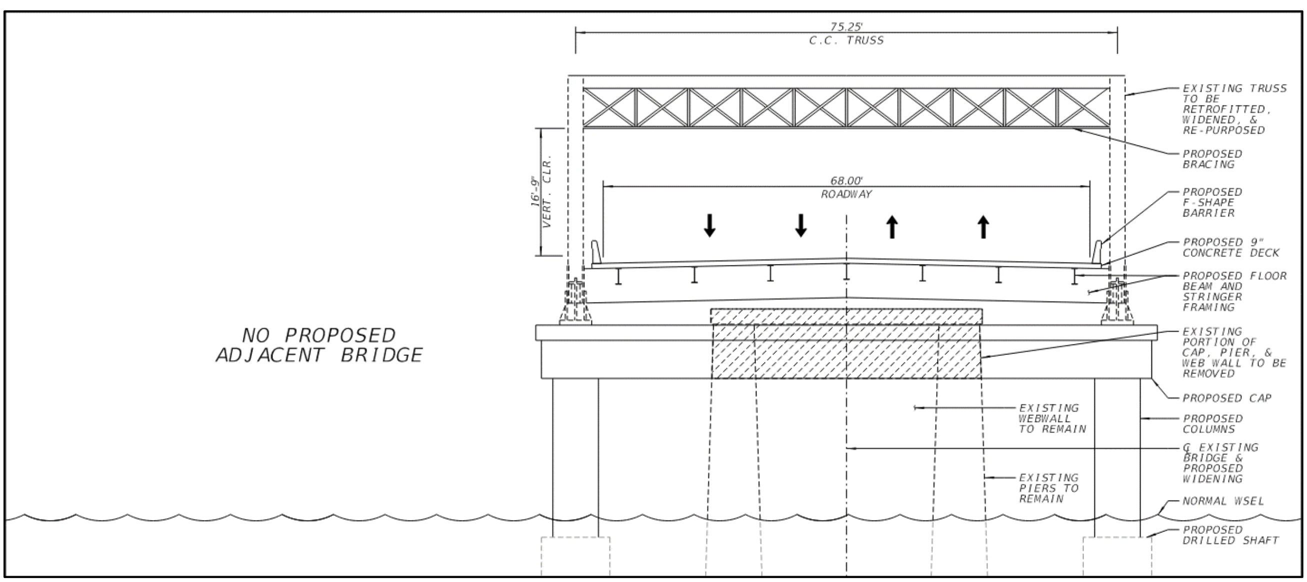


Figure 75: Alternative 2B Typical Section (Truss Spans)

4.2.1.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative meets both the purpose and need of the project by providing sufficient roadway width and correcting all of the deficiencies.

4.2.1.2 Existing Structural Capacity

As presented in Alternative 1, the existing structural capacity of both the superstructure and substructure elements is not sufficient to take the current AASHTO design loading nor is it sufficient to support the widening required. Refer to Section 4.1.1 for PR summaries.

4.2.1.3 Anticipated Rehabilitation & Character Defining Feature Modifications

Because the existing bridge is substantially widened in this alternative, rehabilitation of existing components is required. The following is a summary of the anticipated rehabilitation and additional new construction.

Approach Span Replacement/Removal & Retrofit Components

- Replace the existing concrete deck with a new 9” minimum reinforced concrete deck with new joints.
- Replace the existing bridge rail with a MASH TL-4 rated barrier (42” tall concrete F-shape parapet).
- Remove all existing floor beams.
- Replace all existing steel bearings on the primary girders with elastomeric bearing pads.
- Re-purpose existing primary girders and make them composite with the new deck by adding shear studs to the top flanges.
- Provide four additional steel plate girders in addition to the two existing primary girders.

Truss Span Replacement & Retrofit Components

- Replace the existing concrete deck with a new 9” minimum reinforced concrete deck with new joints.
- Replace the existing traffic rail with a MASH TL-4 rated barrier (42” tall concrete F-shape parapet).
- Replace all floor beams and stringers.
- Replace all bracing elements.
- Replace all gusset plates and rivetted connections with high-strength bolted connections
- Re-purpose existing truss members and strengthen with additional members

Substructure Replacement & Retrofit Components

- Replace the existing T-beam caps
- Provide two additional columns with straight drilled shafts at bents and towers
- Tie proposed columns to existing substructure with diagonal and horizonal struts

A second option that can be investigated for the truss is converting the truss to a purely “aesthetic” feature (i.e. a faux truss) that does not support vehicular traffic. This option would allow a steel plate girder system to be used throughout the entire bridge length and would provide some cost savings; however, the truss would no longer provide structural function as it currently does specifically for supporting vehicular traffic. If the original span length is maintained, the superstructure depth would have to be increased to obtain enough capacity for the navigational span length plus the weight of the truss. The additional superstructure depth can be achieved by either raising the profile grade, modifying the existing substructure, providing a beam with dapped ends, or a combination of these options. The raise in profile grade would require the approach spans to be modified for grade continuity, and substructure modifications or dapped beam ends would lower the low beam elevation. The superstructure depth could possibly be maintained by providing additional substructure supports to achieve shorter span lengths.

The typical application of faux trusses for ODOT projects are common for trusses that span up to 100'. Spans up to 100' can have minimal impact to the grade and low beam elevations; however, this application spans 250' and it was decided that the associated impacts would not warrant analyzing a separate alternative.

Modifications to the character-defining features of the truss span will be made with this alternative; specifically, the lateral sway bracing members will require replacement and the main truss members will have to be strengthened or replaced to support the additional weight of bridge due to widening. If the faux truss option is used, the modifications would be less to the main truss members. All other modifications to the truss span members apply to the deck, rail, floor beams, and stringers, all of which would not be considered character-defining features under Criterion C. Reference Appendix D for a color-coded diagram of the level of modification for all truss members.

4.2.1.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts
Economic impacts of Alternative 2B would involve the same construction detour as discussed under Alternative 2A. Estimated at approximately 2.5 years, this detour would require freight vehicles to use a detour of up to 39.1 miles in length and would negatively affect travel times and operating costs. Ultimately, Alternative 2B would provide a 4-lane bridge with sufficient capacity to accommodate future traffic demand, with a typical section that meets today's design standards.

Environmental impacts of Alternative 2B would be higher than Alternative 2A since it involves widening of the existing bridge. Widening could impact archeological site 34BR11. Impacts of Alternative 2B would be similar to Alternative 5 east of the bridge because both alternatives involve symmetrical widening of the existing causeway. Alternative 2B would affect 42 acres of USACE property, 0.77 acres of Johnson Creek Public Use Area, 0.67 acres of wetland, and 0.05 acres of stream. On the west side of the lake, Alternative 2B would impact the PointeVista development. Alternative 2B could also potentially impact 0.38 acres of the Chickasaw Nation trust property; however, retaining walls could be used to avoid this impact.

Alternative 2B would require reconstruction of the existing bridge. Some elements of the bridge's substructure and superstructure not related to the bridge's significance would have to be strengthened and/or replaced to accommodate the wider typical section. On the truss, all bracing elements would require replacement. Existing truss members would be repurposed as feasible but would be strengthened with additional members. An option to replacement of the truss would be to widen and replace the truss as an "aesthetic" (i.e., non load-bearing) element. This would require less alteration of the truss span. Regardless, Alternative 2B is anticipated to have an adverse effect on the historic Roosevelt Bridge.

Community Impacts
This alternative provides additional roadway capacity and safety for the corridor which will help minimize delays for the traveling public. Protection against vehicular crashes is also improved if a median barrier is added. This alternative, however, has an impact to the community during construction. Similar to Alternative 2A, due to the rehabilitation and widening work required, it is estimated the construction could last 2.5 years or more. During this time, a full detour route for the corridor would be required which would impact through traffic as well as Lake Texoma access. Detours for future maintenance (deck replacements) would not be required if the deck replacements are phased similar to Alternatives 4 and 5.

4.2.1.5 Estimated Life Cycle Cost

The life cycle costs for this alternative are the highest of all the alternatives. Significant rehabilitation and new construction are required to keep the alignment of the existing roadway unchanged. Future costs are high due to the width of deck replacement and the number of steel elements requiring painting. The user costs are also higher due to the construction detour required.

Life Cycle Cost Summary
- Construction Cost: $233.8 Million
- Future Cost: $15.6 Million
- User Cost: $188.2 Million
- Total Life Cycle Cost: $437.7 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

4.2.2 Alternative 3B: One-Way Pair & Rehab Existing Bridge (With Widening)

Alternative 3B will shift the existing bridge's alignment, widen the bridge to provide two lanes of traffic in one direction with a new roadway width of 38' (including shoulders), rehabilitate and repurpose the superstructure and substructure members to meet current AASHTO design loads, and provide a "sister" bridge adjacent to the existing making the two a one-way pair. This option includes changes to the superstructure and substructure. The bridge deck is replaced, new F-shape concrete parapet bridge rails are used, existing floor beams are removed, additional steel plate girders are added, the existing cap is replaced, and a new column and foundation element are provided to support the widening. The truss requires a complete rebuild. The proposed "sister" bridge will be identical to the sister bridge described in Alternative 3A.

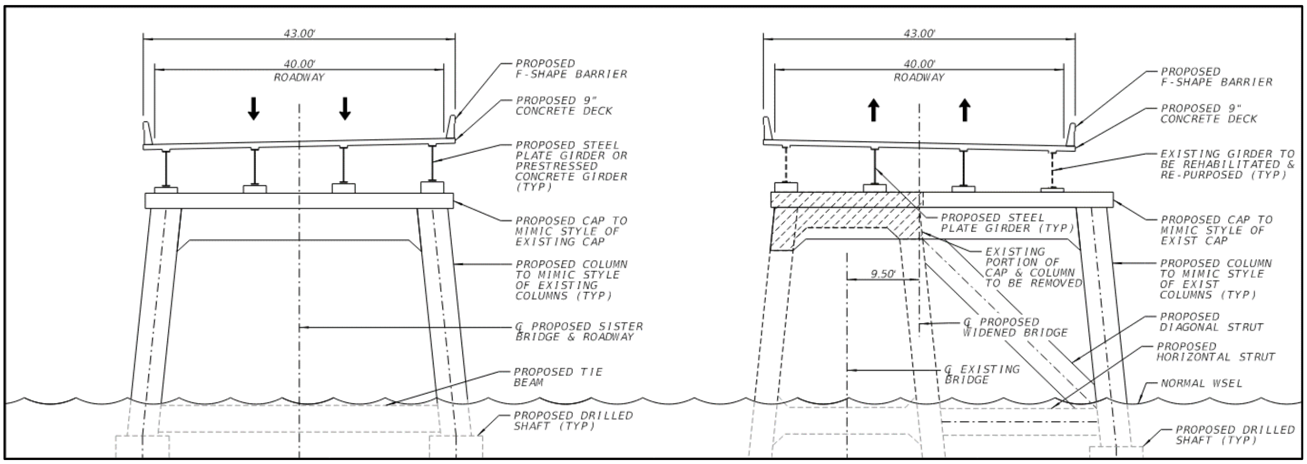


Figure 76: Alternative 3B Typical Section (Approach Spans)

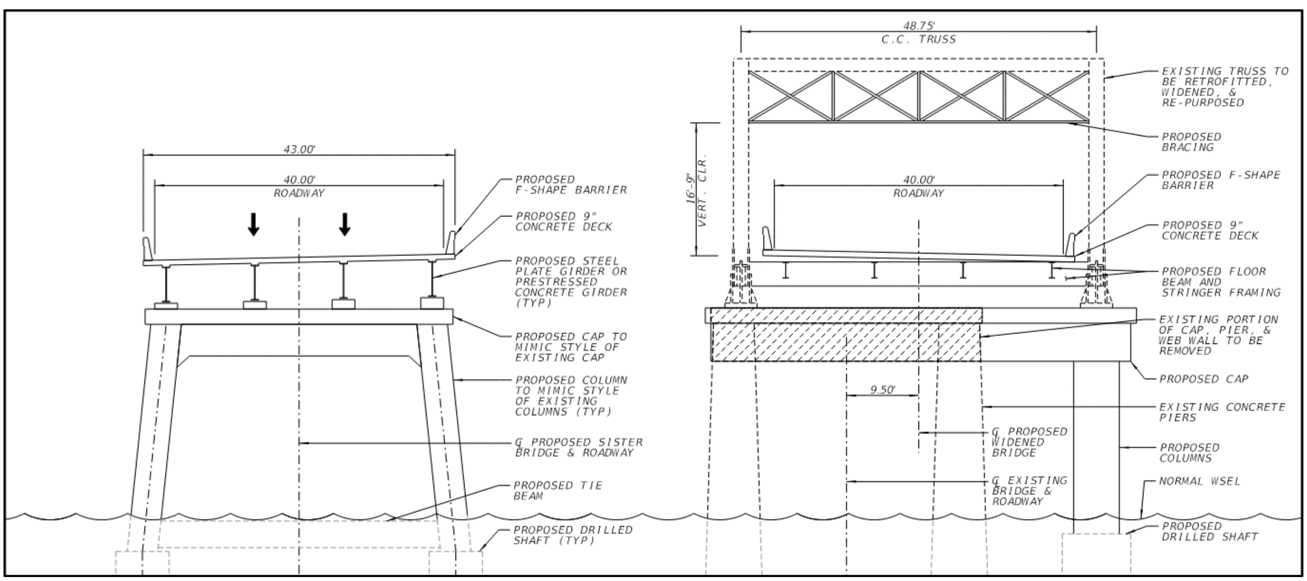


Figure 77: Alternative 3B Typical Section (Truss Spans)

4.2.2.1 Purpose & Need

To meet the purpose of the project, the alternative must provide a safe crossing that accommodates current and future traffic demands. To meet the need of the project, the alternative must correct the deficiencies of the existing bridge and roadway outlined in Section 3.0. This alternative meets both the purpose and need of the project by providing sufficient roadway width and correcting all of the deficiencies.

4.2.2.2 Existing Structural Capacity

As presented in Alternative 1, the existing structural capacity of both the superstructure and substructure elements is not sufficient to take the current AASHTO design loading nor is it sufficient to support the widening required. Refer to Section 4.1.1 for PR summaries.

4.2.2.3 Anticipated Rehabilitation & Character Defining Feature Modifications

Because the existing bridge is widened in this alternative, rehabilitation of existing components is required. The following is a summary of the anticipated rehabilitation and additional new construction.

Approach Span Replacement/Removal & Retrofit Components

- Replace the existing concrete deck with a new 9" minimum reinforced concrete deck with new joints.
- Replace the existing bridge rail with a MASH TL-4 rated barrier (42" tall concrete F-shape parapet).
- Remove all existing floor beams.
- Replace all existing steel bearings on the primary girders with elastomeric bearing pads.
- Re-purpose existing primary girders and make them composite with the new deck by adding shear studs to the top flanges.
- Provide two additional steel plate girders in addition to the two existing primary girders.

Truss Span Replacement & Retrofit Components

- Replace the existing concrete deck with a new 9" minimum reinforced concrete deck with new joints.
- Replace the existing traffic rail with a MASH TL-4 rated barrier (42" tall concrete F-shape parapet).
- Replace all floor beams and stringers.
- Replace all bracing elements.
- Replace all gusset plates and rivetted connections with high-strength bolted connections
- Re-purpose existing truss members and strengthen with additional members

Substructure Replacement & Retrofit Components

- Replace the existing T-beam caps
- Provide additional columns at bents and towers
- Tie proposed columns to existing substructure with diagonal and horizonal struts

A second option that can be investigated for the truss is converting the truss to a purely "aesthetic" feature (i.e., a faux truss) that does not support vehicular traffic. This option would allow the steel plate girder system to be used throughout the entire bridge length and would provide some cost savings; however, the truss would no longer provide structural function as it currently does. If the original span length is maintained, the superstructure depth would have to be increased to obtain enough capacity for the navigational span length plus the weight of the truss. The additional superstructure depth can be achieved by either raising the profile grade, modifying the existing substructure, providing a beam with dapped ends, or a combination of these options. The raise in profile grade would require the approach spans to be modified for grade continuity, and substructure modifications or dapped beam ends would lower the low beam elevation. The superstructure depth could possibly be maintained by providing additional substructure supports to achieve shorter span lengths.

The typical application of faux trusses for ODOT projects are commonly for trusses that span up to 100'. Spans up to 100' can have minimal impact to the grade and low beam elevations; however, this application spans 250' and it was decided that the associated impacts would not warrant analyzing a separate alternative.

Modifications to the character-defining features of the truss span will be made with this alternative; specifically, the lateral sway bracing members will require replacement and the main truss members will have to be strengthened or replaced to support the additional weight of bridge due to widening. If the faux truss option is used, the modifications would be less to the main truss members. All other modifications to the truss span members apply to the deck, rail, floor beams, and stringers, all of which would not be considered character-defining features under Criterion C. Reference Appendix D for a color-coded diagram of the level of modification for all truss members.

4.2.2.4 Environmental & Community Impacts

Social, Economic, and Environmental Impacts

Economic benefits would be anticipated under Alternative 3B since both 2-lane bridges would have sufficient width and together would provide sufficient capacity to carry all traffic.

Environmental impacts of Alternative 3B would be slightly higher than Alternative 3A. Impacts to the archeological site within the lake (34BR11) could be higher than Alternative 3A due to widening of the existing structure. Alternative 3B would affect 35 acres of USACE property, 0.60 acres of Johnson Creek Public Use Area, 0.17 acres of wetland, and 0.03 acres of stream.

Similar to Alternative 2B, Alternative 3B would require rehabilitation of the existing bridge. Many of the bridge's elements, including the truss span, would have to be strengthened and/or replaced to accommodate the wider typical section. This would include replacement of the deck, railing, and floor beams. On the truss, all bracing elements would require replacement. Existing truss members would be repurposed as feasible but would be strengthened with additional members. An option to replacement of the truss would be to widen and replace the truss as an "aesthetic" (i.e., non load-bearing) element. This would require less alteration of the truss span. Regardless, Alternative 3B is anticipated to have an adverse effect on the historic Roosevelt Bridge.

Community Impacts

The impacts to the community are almost identical to Alternative 3A. The added benefit of this alternative is the additional width of bridge to accommodate shoulders on the existing bridge. This adds extra safety and a refuge for accidents on the bridge or vehicles that may break down.

4.2.2.5 Estimated Life Cycle Cost

The majority of the costs for this alternative are in the construction of the sister bridge (similar to Alternative 3A). However, the rehabilitation required to widen the existing bridge puts the total construction cost closer to Alternative 2B. The future and user costs are similar to Alternative 3A.

Life Cycle Cost Summary

- Construction Cost: $216.9 Million
- Future Cost: $14.0 Million
- User Cost: $22.6 Million
- Total Life Cycle Cost: $253.5 Million

See Section 5.0 for a cost comparison for all of the alternatives (Figure 79 and Figure 80)

5.0 Summary of Alternatives

A total of seven alternatives, five categorized as "No-Use" and two categorized as "Use", were studied for this Section 4(f) report. Figure 78 provides a summary matrix of all the alternatives studied (See Appendix C for a larger version of this matrix as well as supplemental analysis matrices).

Alternative Analysis						Meets Project Purpose & Need	Involves Substantial Operational or Safety Problems	Results in Unacceptable & Severe Adverse Social, Economic or Other Environmental Impacts	Causes Substantial Community Disruption	Additional Construction & User Costs of a Substantial Magnitude	Life Cycle Costs (Millions)**
Alternatives		Existing Bridge Section Width	Total Traffic Lanes *	No Use or Use							
ALT 1	Do Nothing	No Change 2 Lanes No Shoulders	2	No Use		No	Yes. Substandard roadway width, structural deficiences and significant deterioration (superstructure & substructure)	Yes. Potential load posting and/or closure would significantly increase travel times and increase freight and operating costs.	Yes. Insufficient traffic capacity would cause major delays for recreational and local traffic	Very High	$1,891.6
ALT 2 (Opt. A)	Rehab (No Widen)	No Change 2 Lanes No Shoulders	2	No Use		No	Partially. Substandard Roadway Width (need two more lanes).	Yes. Lack of sufficient capacity will increase travel times for people and freight and increase the potential for collisions. Construction detour would result in significant impacts to freight travel times and operating costs.	Yes. Insufficient traffic capacity would cause major delays for recreational and local traffic. Detour would be needed during construction.	High	$265.7
ALT 3 (Opt. A)	One-Way Pair Rehab (No Widen)	No Change 2 Lanes No Shoulders	4	No Use		Partially	Partially. Substandard Roadway Width (need shoulders on existing)	Safety concerns remain on narrow existing bridge (no shoulders). Some impacts to USACE lands, Johnson Creek, and potentially two archeological sites are likely due to new bridge construction. Fill will be required in Lake Texoma.	No	Moderate	$205.6
ALT 4	Pedestrian/ Bicycle	No Change (Shared Use Path)	4	No Use		Yes	No	Greater impacts to USACE lands, Johnson Creek, and potentially three archeological sites are likely due to new bridge and causeway construction. More fill will be required in Lake Texoma.	No	Moderate	$247.5
ALT 5	Monument	No Change (Not Used)	4	No Use		Yes	No	Impacts to USACE lands, Johnson Creek and potentially two archeological sites are likely due to new bridge construction. Fill will be required in Lake Texoma. Impacts will be higher than Alt. 3A but lower than Alt 4.	No	Moderate	$218.4
ALT 2 (Opt. B)	Rehab (Widen)	Widened 4 Lanes With Shoulders	4	Use		Yes	No	Construction detour would result in significant impacts to freight travel times and operating costs. Some impacts to USACE lands, Johnson Creek, and potentially two archeological sites are likely due to new bridge construction. Some fill will be required in Lake Texoma. Environmental impacts greater than Alternative 2A.	Yes. Detour needed during construction	High	$437.7
ALT 3 (Opt. B)	One-Way Pair Rehab (Widen)	Widened 2 Lanes With Shoulders	4	Use		Yes	No.	Some impacts to USACE lands, Johnson Creek, and potentially two archeological sites are likely due to new bridge construction. Fill will be required in Lake Texoma. Impacts similar to but greater than Alternative 3A.	No	Moderate	$253.5

** "Total Lanes" accounts for an additional vehicular bridge where applicable.*

*** Costs include construction, future maintenance and repairs, and user costs.*

Figure 78: Alternatives Matrix

Figure 79 and Figure 80 provide graphs of life cycle cost estimates for each of the alternatives. The life cycle costs include proposed construction, future costs (shown in present value), and user/indirect costs. Appendix E provides a detailed breakdown of life cycle costs including the assumptions made. Alternatives 2A, and 2B all have high user costs due to the assumption that conventional construction methods are used for rehabiliation work and future deck replacements and that the only detour available is what is shown in Section 2.1.3. Methods that consider accelerated bridge constuction, more durable decks, or alternative routes could bring these costs down and are discussed in Section 0. Alternative 1 has very high user costs due to the assumption that the bridge is load-posted in 30 years, and subseqently closed to all traffic 30 years after that.

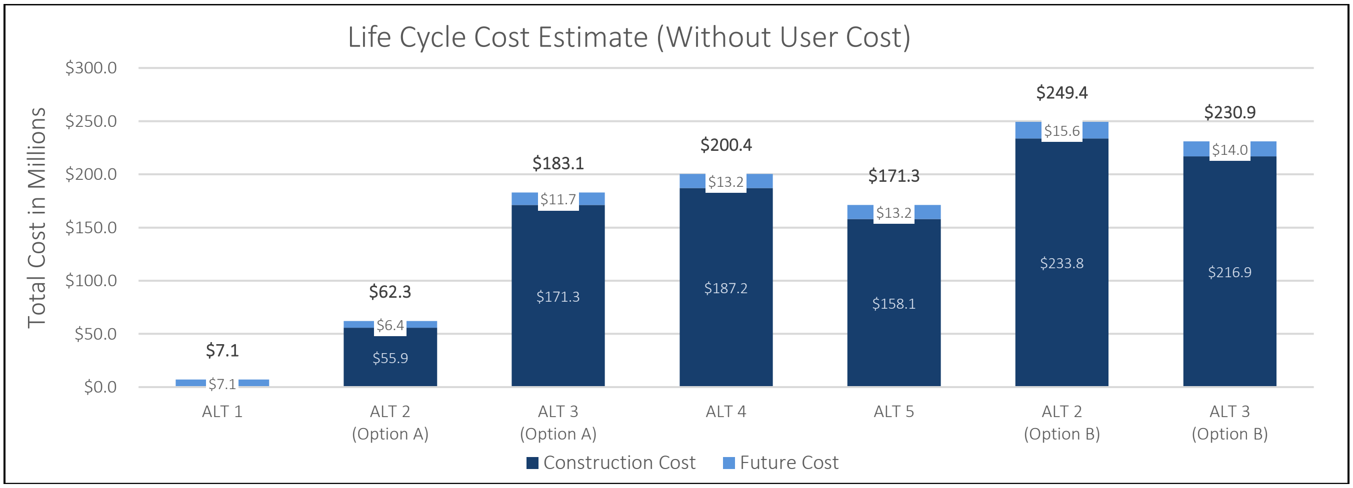


Figure 79: Alternatives Life Cycle Cost (Construction & Maintenance Only)

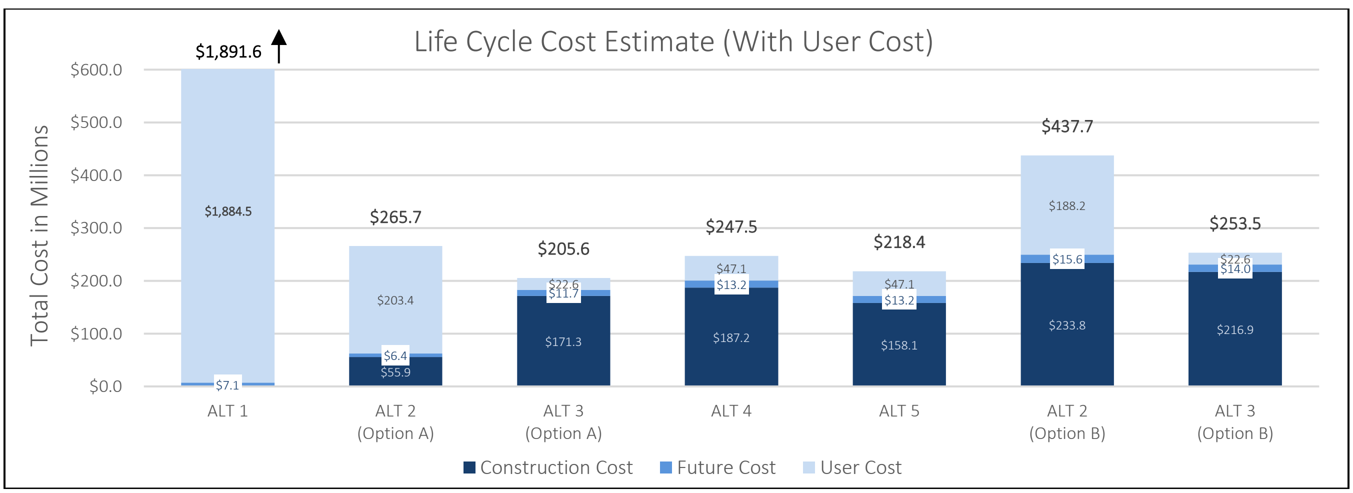


Figure 80: Alternatives Life Cycle Cost Estimate (Including User Cost)

6.0 Conclusions

Of the seven alternatives investigated for this Section 4(f) report, four alternatives fully meet the purpose and need of the project (Alternatives 4, 5, 2B, and 3B), one alternative partially meets the purpose and need of the project (Alternative 3A), and two alternatives do not meet the purpose and need of the project (Alternatives 1 and 2A).

Alternatives 1 and 2A not only do not meet the purpose and need of the project, but they have higher user costs associated with them due to construction/future maintenance detours that would be required or load-posting/closure of the bridge as discussed for Alternative 1.

Alternative 3A is the least expensive alternative when all the life cycle costs are considered; however, it only partially meets the purpose and need of the project due to the existing bridge not being widened to accommodate the required shoulders for safety.

Alternatives 2B and 3B are considered "Use" alternatives that will likely have an adverse effect to the Roosevelt Bridge's significance and/or integrity, as defined through the Section 106 (National Historic Preservation Act) process. Both of these options have the highest construction costs of all the alternatives. Alternative 2B also has the highest user costs of all the alternatives.

Alternatives 4 and 5 fully meet the purpose and need of the project and would not require the use of the historic bridge. Alternative 5 is the second least costly alternative when all the life cycle costs are considered; however, Alternative 4 provides a unique benefit of pedestrian/bicycle access on the existing bridge that none of the other alternatives provide.

The alternatives that require either near-term or future detours (Alternatives 1, 2A, and 2B) have impacts to the traveling public. To minimize the impacts of detours, alternative concepts can be investigated. Accelerated bridge construction is an option to minimize the closure duration of the bridge. Components like the superstructure can be fabricated and assembled on shore while traffic remains on the bridge. Then the bridge could be shut down for a shorter period of time so that the existing components needing to be replaced could be demolished and then have the new superstructure spans floated and lifted into place. Another option to minimize the need for future detours would be to design the bridge decks for a 100-year service life. The use of Glass Fiber Reinforced Polymer (GFRP), galvanized, or stainless-steel bars in lieu of epoxy coated bars in conjunction with high performance concrete (HPC) or ultra high performance concrete (UHPC) could achieve better durability of the deck so that it would not have to be replaced periodically; and thus avoiding future detours for deck replacements. These options to minimize or eliminate detours will increase construction cost and complexity; however, they will save on future maintenance and user costs.

In order to present the results of the analysis in a clear format, a Project Evaluation Matrix was created and presented below in Table 20.

INSERT

Table 20: Project Summary Matrix

7.0 Works Cited

AASHTO. (2018). *The Manual for Bridge Evaluation 3rd Ed.*
AASHTO. (2020). *LRFD Bridge Design Specifications 9th Ed.*
Archives, N. (2021). *Code of Federal Regulations.*
Garver. (2021). *US-70 Over Lake Texoma (Franklin D. Roosevelt Memorial Bridge) Signature Bridge Type Study.*