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MUS-CR32-0.00
Muskingum County, Ohio

MUS-CR32-0.00
OVER THE MUSKINGUM RIVER

STRUCTURE TYPE STUDY

REVISION 3

Prepared for:

MUSKINGUM COUNTY
MCEO
ENGINEER'S OFFICE

Muskingum County Engineer's Office

155 Rehl Road, Zanesville, OH 43701

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Revision 3 Submittal Date: 1/4/2017

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APPENDIX

I. Executive Summary

E.L. Robinson Engineering of Ohio Co. (ELR) has prepared this **revised** MUS-CR32-0.00 Structure Type Study (STS) for the Muskingum County Engineer's Office (MCEO). Comments, dated September 19, 2016 from the MCEO, have been addressed in this revised report and a disposition of these comments has been included in Appendix F. This STS, referred to as Revision 3, contains updated hydraulic information. See Appendix "G". MECO has requested that this revised Structure Type Study Report place additional emphasis on the maintenance items that are related to the new proposed structure. This report is one component of the Preliminary Engineering Process for this project. A Project Feasibility Study dated February 2016 was completed for the purpose of choosing a Preferred Alignment for this bridge replacement project. Roadway Stage 1 plans were submitted on September 2, 2016. Following the approval of this Structure Type Study, ELR will submit Stage 1 Bridge plans, the Hydraulic Report for the Muskingum River and the Structure Foundation Exploration Report. The purpose of the MUS-CR32-0.00 project is to replace the existing CR 32 structure over the Muskingum River with a new wider structure and related realignment of the approach roadway.

The development of this report and the attached appendices are based on Path 3 of the ODOT Project Development Process (PDP). Path 3 projects involve marginally complex projects with moderate roadway and structure work. Path 3 projects also can involve some utility relocation and right of way acquisition.

Several ELR staff members visited the project site on May 25, 2016, to assess the field conditions and gain knowledge of the area surrounding the proposed bridge. Special attention was given to observing the slopes of the river banks for the purpose of understanding how the proposed abutments should best be located for determining the length of the proposed bridge. Existing plans have been obtained and alignment information from these plans has been added to the project files. This study has been developed in accordance with the latest ODOT design manuals and specifications. This report outlines various design considerations, evaluates preliminary options for the replacement of the structure, and provides supporting information for the evaluation of the proposed bridge types.

This Structure Type Study has been developed under direction from the Muskingum County Engineer utilizing the preferred Alignment Alternative "C" as defined in the January 2016 Feasibility Study. This Structure Type Study has evaluated a four-span steel girder bridge, a five-span steel girder bridge, and also a seven-span concrete beam bridge to determine the most economical and/or the preferred structure design. Designs with five and six girder/beam lines were studied. Additionally, several corrosion protection systems were evaluated for the steel girder alternatives. While initial costs vary, the present value life cycle cost of weathering steel, galvanized steel and concrete beams is similar. The preferred structure is a 7-span, 6-beam, prestressed concrete structure on wall-type piers with joints placed at each end of the bridge.

II. Introduction

PROPOSED PROJECT

This project involves the replacement of the CR 32 Bridge over the Muskingum River with a new wider structure. This structure connects the Village of Philo, OH with Duncan Falls, OH as shown in Figure 1: Project Location Map. This bridge is located near Philo Lock & Dam #9 and bridges the operational lock chamber. CR32 is classified as a rural major collector and has a design speed of 35 mph. The existing MUS-CR32-0.00 structure provides one 13'-0" lane in each direction without shoulders. A sidewalk for pedestrians is located on the downstream side of the existing bridge.

It is anticipated that the existing bridge located just south of the Philo Bridge will be replaced using a culvert rather than a bridge.

As part of the previously completed Feasibility Study, the ELR team analyzed potential alignment alternatives. It was determined that Alternative "C" is the Preferred Alignment. Alternative "C" is located just downstream of the existing structure. Stage 1 roadway plans have been completed. Stage 1 bridge plans will be developed shortly after the selection of the preferred bridge alternative. Immediately after the Stage 1 details have been approved, the detail design plans for the new bridge will begin to be prepared.

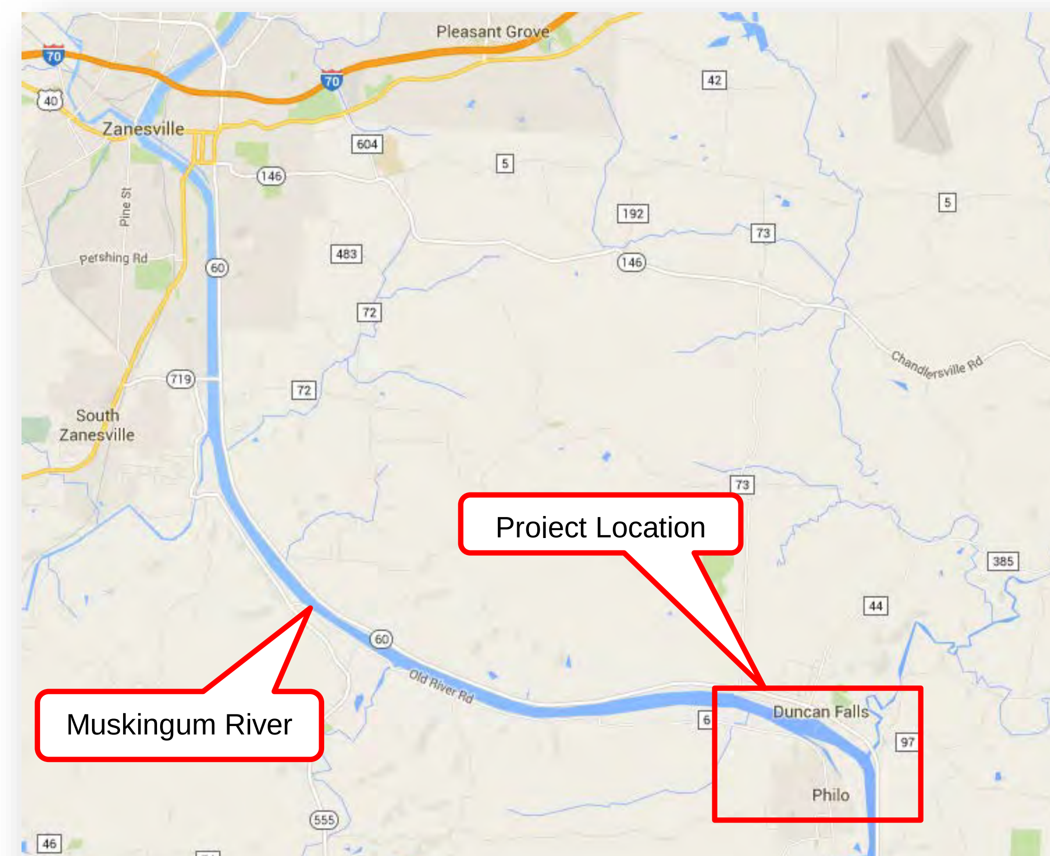


Figure 1: Project Location Map

Figure 2: Structure Type Study Location

PROJECT BACKGROUND

The Philo/ Duncan Falls Bridge over the Muskingum River serves as a vital link between the communities of the Village of Philo on the south side of the river and Duncan Falls on the north side. The two communities make up the rural Franklin Local School District which serves over 2,000 students in five different buildings. The building locations are divided by the river with the Duncan Falls Elementary and the High School on the north side of the river, the Junior High School, Roseville Elementary, and Franklin Local Community School on the south side of the river.

As a result of the school district being divided by the Muskingum River, the bridges crossing the river are vital links for the school district. Currently, two county bridges (one located at Philo/Duncan Falls and the second to the south at Gaysport) are the main crossings of the river in the area. When the bridge located at Philo/Duncan Falls is closed, the resulting effect is a ten mile detour for school buses which adds fuel and labor costs. Two detour routes for the Philo/Duncan Falls Bridge exist, both of these routes result in a five mile trip on County Road 6 (Old River Road), which is a curvy two lane road that closely follows the river. The route to the south requires crossing the Gaysport Bridge which is limited to one lane traffic with traffic signals due to failing exterior floor stringers. The route to the north requires the crossing of a county bridge limited to an 11 ton load limit for passenger cars and trucks. Both routes pose a risk for inexperienced high school age drivers and school buses.

The impact to the local business community is substantial. The 10 mile detour affects food delivery, groceries, banking, gas stations, and convenience items all of which are limited to crossing of the existing bridge sidewalk. Some services are available on both sides of the river; however, banking and gas stations are only available on the Duncan Falls side. Business from commuter traffic is impacted due to the northern detour route passing the larger community of South Zanesville resulting in a 10 mile trip for services to South Zanesville in lieu of a 20 mile trip to Duncan Falls and back.

Emergency services are always a concern for two communities that rely on support from each other for fire and emergency medical services. When the bridge is closed, response time is delayed due to the detour. Both communities serve aging residents in a rural setting with a long travel time to hospitals and any added delay can be the difference between life and death.

The bridge has been in its current location since circa 1875 and has kept the two communities connected for over 141 years. The replacement of this bridge is necessary to maintain commerce, school traffic, and a physical link between the communities.

The current bridge was closed at the end of 2015 for emergency repairs of failing structural members (one floor beam and two stringers had failed). In all, eighteen of the forty five 36-inch deep floor beams exhibit holes in their webs close to the connections to the truss. Based on the observed rapid deterioration of the bridge, we anticipate annual closures of 6-8 weeks in length for repairs and hold reservations whether the bridge can remain open through construction of a new crossing, which is scheduled to begin in 2019.

The existing Philo (MUS-CR32-0.00) Bridge (SFN 6054129), built in 1953, is a 5 span, steel truss structure with a total length of 828 feet and a bridge deck roadway width of 26 feet. A detailed visual inspection conducted on March 10, 2015, by the Muskingum County Engineer's Office concluded that the existing bridge over the Muskingum River is Structurally Deficient; therefore, it meets the criteria for replacement based upon the Federal Highway Bridge Inventory & Appraisal System. The detailed visual inspection also concluded the existing bridge requires posting for load-carrying capacity restrictions.

Currently, the existing Philo Bridge has an overall General Appraisal and Operation Status Rating of 2P. The "2" rating indicates that the bridge is in critical condition while the "P" rating means that the structure is currently posted for a load-carrying restriction of 90% of the Ohio legal load limit. Based on the March 10, 2015 inspection, the existing superstructure and substructure are in serious and poor condition, respectively.

The bridge has been assigned a sufficiency rating of 20 Structurally Deficient (SD). A rating of less than 50 with a SD classification qualifies the structure for replacement with the use of federal funding. Structures with this low of a sufficiency rating are not considered candidates for rehabilitation, and thus, warrant replacement.

The load rating for the existing Philo Bridge is 90% of the Ohio legal load limit. The bridge was not constructed to current design standards. The existing bridge cannot carry the legal loads that similar bridges are being designed and constructed for today. After a recent inspection of the Philo-Duncan Falls Bridge over the Muskingum River, the Engineer's Office has discovered advanced deterioration at several connections of structural members on the bridge. Effective immediately, the bridge was placed under a weight limit of 15 tons. Vehicles over 15 tons are not permitted to cross the bridge at this time.

TRAFFIC

The existing crossing carries approximately 4,500 vehicles per day and is the primary connection between the Village of Philo and Duncan Falls. A new bridge which is capable of carrying legal loads will likely see an increase in traffic usage. With the nearest alternative Muskingum River crossing more than five miles south of the existing structure, substantial time and costs are associated with any closures for repairs to the existing structure.

III.Design Criteria

Existing Structure Data

Spans: 41'-8" – 154'-5" – 157'-6" – 3 spans at 157'-0"
Superstructure Type: Steel Truss
Abutment Type: Stub (Rear), Wall (Forward)
Pier Type: Wall
Roadway: 26' curb to curb
Alignment: Tangent
Skew: None
Loading: S-20-46
Approach Slabs: 15' long
Wearing Surface: 3" thick Asphaltic Concrete Overlay
Date Constructed: 1953

Proposed Structure Data

Spans: 4, 5, or 7 spans totaling 850' in length
Superstructure Type: Steel Girders or Prestressed Concrete I Beams
Abutment Type: Stub
Pier Type: Wall-Type Pier
Roadway: 36' face of barrier to face of barrier, with a 5' sidewalk
Alignment: Tangent
Skew: None
Loading: HL-93
Approach Slabs: 25' long
Wearing Surface: 1" thick Monolithic Concrete

Bridge Design Criteria

Minimum Vertical Clearance: 28' above normal pool elevation of 661.90'
Minimum Horizontal Clearance: 76' from edge of water to face of pier
The Estimated Scour Depth at the piers is 7 feet

Roadway Design Criteria

Required minimum Shoulder width is 4 feet; Shoulder width provided is 6 feet
Legal Speed: 35 mph
Design Speed: 35 mph
Design ADT: Certified Traffic has been requested
Roadway Classification: Rural Major Collector

Proposed Alignment

The January 2016 Feasibility Study describes four alignment options for replacement of the structure, including utilizing the existing alignment and three relocated alignment alternatives. Each alignment alternative was evaluated in terms of maintenance of traffic, roadway, drainage, structures, geotechnical, right of way, utility, and environmental concerns. The preferred alignment was determined to be Alternative C. Alignment Alternative C enables traffic to be maintained utilizing the existing structure while the proposed structure is constructed. Alternative C – Bridge Street is the replacement of the structure on a new alignment crossing the Muskingum River 96' downstream of the existing structure utilizing a tie in on Bridge Street just north of SR 60. This structure type study focuses on potential steel and concrete structure alternatives that are located on Alignment Alternative C.

Maintenance of Traffic

The proposed preferred Alternate C alignment will allow the existing CR 32 Bridge to remain open so that traffic can be maintained during the construction of the new structure.

The new construction will temporarily cut off access to a dead-ended portion of Water St on the north side of the river and to a private access road for a power transmission site on the south side of the river. Maintenance of traffic issues are covered in the Feasibility Study and do not affect the recommended structure type.

Proposed Bridge Transverse Section

The proposed transverse section consists of two 12'-0" lanes with 6'-0" shoulders on each side. A 5'-0" wide sidewalk is included on the downstream (right) side of the bridge. Traffic barriers will be located on both sides of the shoulders and a pedestrian barrier will be located on the right side of the bridge adjacent to the sidewalk. One of the traffic barriers is located between the sidewalk and the roadway traffic.

Traffic and pedestrian barriers can be constructed using concrete at the base which will require storm drainage to be accounted for in the design of the superstructure drainage system. Alternatively, steel railing could be provided in place of concrete barriers. This would allow for over-the-side deck drainage. This alternative will have a higher initial cost.

Profile Grade

The proposed profile grade was analyzed to determine the best drainage pattern on the bridge. Because of debris build-up and the possibility of water ponding on the bridge, a sag vertical curve is not desired on the structure. To meet this criterion, the majority of the bridge drains to the north off the bridge. The low point of the sag vertical curve is located on the roadway. The deck discharge flowing off of the end of the bridge will be directed to catch basins.

Minimum Vertical and Horizontal Navigational Channel Clearances

The Muskingum River is designated as a navigable waterway within the specific reach of the river that includes the location of the Philo Bridge. Preliminary coordination with The United States Coast Guard has been ongoing, including several email and telephone discussions. To facilitate the discussions, ELR prepared and sent numerous navigational clearance related exhibits to:

Rodney Wurgler
U.S. Coast Guard
Bridge Management Specialist
St. Louis, MO

and

Eric Washburn,
USCG Bridge Administrator,
Western Rivers STL.

On July 27, 2016, Mr. Washburn offered the following **Preliminary Navigational Channel Clearances** which are subject to a final approval that becomes available at the conclusion of the overall evaluation process conducted by the U.S. Coast Guard.

(1) Minimum Vertical Navigational Channel Clearance: 28 feet above the normal pool elevation defined to be at 661.90' and located within the navigational channel.

(2) Minimum Horizontal Navigational Channel Clearance: 76 feet of clearance provided from the water's edge to the face of the first descending pier in the water.

(3) Consider the following for the existing left descending pier (the round swing span pier): Need to discuss the disposition of this pier with the Army Corps to decide if it should remain or be removed. Note that DNR plans to rehab the lower guide wall and replace the upper wall (one now missing); therefore it could make sense to leave the existing pier in place.

(4) Mr. Washburn has reached out to the Muskingum River Waterway Association for their comments. The chair of the Association doesn't see any issues with the above proposed clearances, but has sent the proposed clearances out for comments. Muskingum County can use 28 feet (vertical) and 76' (horizontal) as preliminary design clearances. The USCG Bridge Administrator cannot provide final approval of the clearances until he gets to the conclusion of public notice stage. By contacting boaters now, it will help to avoid issues down the road.

(5) Mr. Washburn will be at the project site on the morning of September 19, 2016, to review the area surrounding the site of the proposed bridge project. Mr. Washburn will want to review environmental documents as well as discuss the process for approving pier and superstructure construction and demolition.

(6) Maintenance of marine traffic through the existing lock structure during construction needs to be discussed and addressed.

Right-of-Way

The proposed roadway alignment alternative will require right-of-way increases. This report studies six structure alternatives along the chosen roadway alignment. All six structure alternatives share the same right of way requirements.

Pedestrian Sidewalk

The existing structure over the Muskingum River includes a sidewalk. The proposed structure will also include a 5 feet wide sidewalk in order to permit pedestrian access between the Village of Philo and Duncan Falls. Due to the relatively long bridge crossing, a permanent traffic barrier is recommended to separate the vehicles from the pedestrians.

Environmental

Environmental studies relating to the replacement of the MUS-CR 32-0.00 structure over the Muskingum River have been performed. To date, environmental and ecological literature reviews identifying and describing existing features in the project study area have been done, and a Phase I and Phase II Environmental Site Assessment Screening Report has been completed as a part of the Feasibility Study.

While various environmental issues affect the proposed alignment, there are no issues that will affect the selection of a preferred structure type.

Construction

Construction of a new structure, including approach roadway work, is expected to require two construction seasons regardless of the structure type selected. The Muskingum River at this project site is relatively shallow and therefore a causeway could be utilized to provide access for equipment and the delivery of material to the site of the proposed structure during construction. If a causeway is constructed at the south end of the bridge, it would block the opening of the lock and would impede marine traffic. The contractor will be required to coordinate the temporary closure intervals of the operation of the lock with ODNR.

Existing Bridge Demolition

The existing bridge is expected to be appropriately maintained to ensure that the bridge will be functional during the construction of the proposed bridge. Demolition of the existing bridge will commence once the proposed replacement bridge is operational. The causeway constructed to build the proposed bridge could be placed between the proposed and existing bridges to facilitate construction of the new bridge and also the removal of the existing bridge.

Scour

The existing river bed generally consists of shale bedrock. There is some historical evidence of long term degradation of the river bed. This channel streambed degradation process is referred to as scour when the channel bottom erodes around bridge foundations. In order to ensure that the stability of the pier foundations for the proposed bridge is resistant to any potential lowering of the river bed by scour action, the proposed pier walls will be supported by drilled shafts socketed into bedrock at a depth well below the predicted scour. The drilled shafts for the piers will be designed to include an assumption at this time that 7 feet of scour could occur at each of the drilled shafts used to support the piers.

Deck Surface Drainage

Three deck surface drainage options were evaluated for the design of the deck surface run-off occurring on the proposed structure. A detailed discussion of the drainage options is presented in Appendix E. Providing steel posts and steel railing on the bridge will allow the bridge to drain over the side of the deck and directly into the Muskingum River. Providing concrete barriers will contain the drainage to the deck where it will be funneled into catch basins at the ends of the bridge. The structure type study has been completed assuming that the roadway drainage will be contained to the deck using concrete barriers and that the sidewalk will be sloped inward so that drainage will flow toward the median barrier. Drainage from both the roadway and sidewalk will be collected in catch basins at the ends of the bridge.

Utilities

To date, few utility responses have been received. The residential nature of the project area increases the potential that design and construction conflicts with the existing locations of electric, cable, telephone, water and gas facilities. The construction of the new bridge will require the removal of an existing unused electric tower near the southeast corner of the existing bridge. Additional utility coordination will be required to determine the exact impacts caused by the proposed improvements.

Summary of Geologic and Geotechnical Concerns

Subsurface investigations will be completed in the next phase of the project. Existing geological and geotechnical data was obtained from a search of generalized geological references available from ODNR and available geotechnical data from ODOT records. The search of ODOT records resulted in the original subsurface investigation records from 1963 for the construction of SR 60 on its current alignment through Duncan Falls, along with another subsurface investigation in 1968 for a reported landslide along Main Street in Duncan Falls. A subsurface investigation for the SR 60 Bridge over Salt Creek was also found. A summary of the findings is given below.

The project area is located in the Muskingum-Pittsburgh Plateau physiographic region. The area is a dissected plateau with moderately high to high relief (300’ to 600’). This area was not glaciated. The soil in the area consists of glacial outwash deposits, fluvial and alluvial deposits in the flood plains and stream valleys with colluvium deposits on the hillsides. The ground surface within the Muskingum River flood plain is generally at elevation 660’-700’, while the hills to the south of the river extend to elevation 920’.

Based on the ODNR Bedrock Geology and Topography maps of the area, the underlying bedrock consists of the Allegheny and Pottsville Groups, which include shale siltstone, sandstone, limestone, and some coal. The hills to the southwest of the project area may also include rocks from the Conemaugh Group, which include shale siltstone, claystone, sandstone, limestone, and coal. When subject to weathering, the claystone in the Conemaugh Group can weaken and cause landslides. The top of bedrock in the area is generally between elevations 650 to 700, and increasing in elevation away from the Muskingum River. The depth to bedrock is anticipated to be 20’ to 80’ in the floodplain and decreasing in depth along the hillside slopes away from the river. The project is not in an area where karst would normally be encountered. There are two abandoned underground coal mines in the area south of the river, but unfortunately there are no mine maps for those particular mines.

The subsurface investigation from 1963 for the construction of SR 60 on its current alignment indicated that the soil north of the river generally consists of five to ten feet of fine-grained soil (such as silty clay, silt and clay, and silt) overlying sand and gravel. The subsurface investigation from 1968 includes six boring logs for two reported landslides. Unfortunately, only the boring logs are available. There are no reports or other documentation that describe the landslides further. The boring logs do not record any conditions that are typically associated with landslides (e.g. soft clay or water) but do show loose cinders and sand in some borings and open voids in the rock. Depending on the selected alignment for the project, these conditions may need to be investigated further.

Although it is east of the project area, the subsurface conditions encountered by the 1983 investigation for the bridge carrying SR 60 over Salt Creek should be similar to the soil and rock conditions at the bridge over the Muskingum River. These borings encountered medium stiff to stiff clay and silt to a depth of about 30’, underlain by medium dense to dense sand and gravel. One boring encountered shale bedrock at a depth of 50’, while the other boring extended to 60’ without encountering bedrock.

Foundation recommendations are to be finalized after the soil investigation is complete. The piers are expected to be supported on drilled shafts socketed into bedrock. The abutments are expected to be supported on either drilled shafts socketed into bedrock or H-Piles driven to bedrock. Shale bedrock is located at approximately elevation 655. At this time the roadway subsurface investigation has been completed and the borings for the bridge abutments have been obtained. The borings for the pier foundations will be obtained when the river water surface elevation raises up high enough for a barge to maneuver in the river.

Hydraulic Analysis

A hydraulic analysis was completed for the preferred structure alternative which utilizes 7 spans and 6 prestressed concrete beams. For the complete Hydraulic Analysis and Report, refer to Appendix G.

We have evaluated the water surface elevation for both the design and check year discharges at the cross-section immediately upstream of the proposed and existing bridge (river station 361706.7) using the statistically determined discharges. By reviewing the results of the hydraulic analyses, it is seen that there is a slight decrease in the water surface elevation for the design year discharge and a slight increase in the water surface elevation for the check year discharge. However, there is no change in the Federal Emergency Management Agency (FEMA) Base Flood Elevation using the Flood Insurance Study (FIS) discharge. The increase in the water surface elevation is 0.01 foot for the statistically determined 25-year discharge. The decrease in the water surface elevation is 0.01 foot for the statistically determined 100-year discharge.

When comparing the FEMA Base Flood Elevation calculated using the FIS discharge for the proposed and existing conditions at the FIS named cross sections, it is seen that there is either no increase or a 0.01 foot decrease in the Base Flood Elevation at the named cross sections. This is an indication that the proposed work within the statutory floodway meets the FEMA requirement for no increase in the base flood elevation.

The proposed replacement structure results in a slight decrease in the water surface elevations for the 25-year discharge and a slight increase in the 100-year water surface elevation. Section 1006.3 of the Location & Design Manual requires that the proposed water surface elevation match the existing to the maximum extent practicable and maintain a free water surface for the design year event.

The use of a steel beam bridge could potentially reduce the number of piers and consequently the design and check water surface elevations. However, this structure type study report concludes that the steel beam bridge was determined to be economically impractical. Therefore, giving consideration to the outcome of this report and the minimal increase in the design and check year water surface elevations, it is believed that the preferred structure meets the maximum extent practicable provision of Section 1006.3 of the Location & Design Manual.

Section 1006.3 requires that the impact associated with an increase in the check year water surface be assessed. The proposed 7-span replacement structure results in a maximum increase in the check year discharge of 0.01 feet and a maximum increase in the water surface top width of less than one foot. These impacts can be considered to be *de minimis* impacts. Therefore, the proposed replacement structure does not result in a significant flood hazard when compared to the existing structure.

Bridge Lighting

At this time, we understand that MCEO's preference is to provide bridge overhead deck lighting at each end of the bridge. Additionally, lighting provided in the lower portion of the concrete traffic and pedestrian barriers will satisfy a request from the public. This lighting will provide sidewalk and roadway illumination at night time and will help illuminate the roadway during frequent fog events. Consider protecting this lighting with ballistic grade glass that will make the glass less susceptible to vandalism.

IV. Structure Considerations

Proposed Structure

Each of the structure alternatives that were studied will utilize a bridge width that is 45'-2" out to out of bridge deck. This bridge width provides two 12' lanes, two 6' shoulders, and a 5' sidewalk. The proposed shoulders on the structure will be wider than the standard required 4 feet shoulders in order to accommodate the desire to have extra bridge width for maintenance of traffic, to accommodate the deck surface drainage, and for parking a snooper truck during inspection of the bridge. A 1'-4" wide barrier is to be located between the sidewalk and the shoulder, a steel pedestrian railing is located on the outside of the sidewalk, and a 1'-2.5" traffic barrier is provided on the left edge of the bridge deck. In order to provide a reasonable view of the river for the traveling public, the edge-of-deck traffic barrier will include steel tubes similar to Section A-A of ODOT's BR-2-15 standard drawing. The traffic barrier located between the sidewalk and the roadway shoulder will be tapered down at each end of the bridge. The proposed bridge transverse section is shown in Appendix C. Two superstructure stringer types have been considered for the proposed structure:

1. Prestressed concrete I-beams
2. Painted steel, galvanized/metalized steel and weathering steel girders (straight or haunched)

Prestressed Concrete I-Beams

The most economical span length for prestressed concrete I-beams is generally in the range of 120 feet to 140 feet (prestressed concrete I-beams can be designed as long as 180'). Concrete beam shapes conforming to the details provided in ODOT Standard Drawings PSID-1-13 will be considered. Wide flange shapes shown in PSID-1-13 provide a good solution for 130' long spans. The bridge length being considered is 850 feet long, which will require the following spans lengths;

- Seven Spans 70' – 130' – 130' – 130' – 130' – 130' – 130'

The short 70' span required near the lock structure may result in uplift at the rear abutment. This will be investigated further if this span arrangement is chosen and can likely be accommodated for by the weight of diaphragms. When compared with steel girders, concrete beams will provide a stiffer superstructure resulting in less live load deflection. Approved ODOT concrete beam shapes were considered for the analyzed span arrangement. Emphasis was placed on utilizing concrete release strengths (f'_{ci}) of 5,000 psi and final strengths (f'_c) of 7,000 psi, as recommended by the ODOT Bridge Design Manual. However, due to the relatively large spans considered, taller prestressed concrete beam shapes with 4'-1" wide top flanges, concrete release strengths up to 7,000 psi and final concrete strengths up to 9,000 psi were also considered. All alternatives studied have been used on previous ODOT projects where relatively long span lengths needed to be achieved. During final design, all recommended beam shapes and strand patterns will need to be approved by ODOT and confirmed as constructible by regional suppliers.

On July 24, 2016, ELR submitted a request to Prestressed Services Incorporated (PSI) to dispatch a scout to investigate the potential delivery routes for shipping prestressed concrete I-beams from the fabrication plant to the site of the proposed new Philo Bridge. On, August 2, 2016, PSI informed ELR that they recommend that the prestressed concrete I-beams to be shipped to the Philo Bridge site not exceed 140 feet in length.

Steel Girders

Constant depth structural steel plate girders have been considered. Haunched girders can also be considered because of their desirable aesthetic appearance and the ability to utilize a lower profile grade when meeting the required minimum vertical clearance. Painted steel can be considered if a specific color is preferred for the structural steel members. We understand that painted steel is generally not desired and is not economically competitive. Galvanized steel is becoming a very popular preferred design because of the documented low maintenance costs and very desirable life cycle cost features. In the past, weathering steel has been a common choice for this type of structure. For this proposed 850-foot-long bridge, a structural steel bridge could consist of;

- Four Spans 185' – 240' – 240' – 185'
- Five Spans 164' – 174' – 174' – 174' – 164'

Bridge Design Features

Section 205.2 of the ODOT Bridge Design Manual states that when 4 or more spans are required for a structure, the designer should perform a cost analyses study to determine the number of spans that will result in the most economical bridge. This study is referred to as a substructure and superstructure cost optimization study. A minimum span of 76' is required for the navigable waterway opening.

The bridge abutments are expected to be relatively tall stub abutments founded on drilled shafts or piles driven to bedrock. Spill-through slopes graded at approximately 2:1 will be provided in front of the abutments and the location of the abutments will be such that the slopes will not encroach on the area bound by the ordinary high-water elevation. The piers will be wall-type piers supported on four drilled shifts, assumed to be five feet in diameter and socketed into bedrock.

Bridging the Lock Channel (Minimum Navigational Channel Width)

The bridge structure must provide enough horizontal clearance in the region of the lock in order to allow river traffic to enter the lock. In order to do this, three options exist:

1. A stub type abutment could be installed set back from the edge of water and the first pier can be placed at the water's edge providing a relatively short span over the lock channel.

2. A full height abutment could be placed at the water's edge. This also would allow for a relatively short span over the lock channel.

3. A stub type abutment could be installed set back from the bank of the river. This would require a long first span in order to bridge both the river bank and the channel. Concrete beams are not viable for this option.

Abutment Design

One abutment option would be to provide a stationary (fixed) beam seat and backwall. An expansion joint would then be installed between the superstructure and the backwall to allow for the thermal expansion and contraction of the superstructure. This abutment option has been used traditionally in bridge construction for relatively long bridges. However, in situ performance of existing structures has shown that there exists potential for the expansion joints to leak and for the water to corrode the ends of the beams/girders. The recent trend in bridge construction is to use an integral or semi-integral abutment design where feasible. Typically a semi-integral or integral design is only permissible on bridges below 300' to 400' in length because of the amount of expansion which results in a substantial soil pressure that is developed on the back of the semi-integral or integral diaphragms.

The ODOT Bridge Design Manual recommends a maximum total bridge length of 400 feet for use with integral or semi-integral type abutments. This structure length of 850 feet exceeds this requirement. Therefore, standard integral or semi-integral abutment designs are not recommended unless special details are developed.

The magnitude of thermal expansion on bridges longer than 400 feet will likely cause significant soil pressure on the backside of a semi-integral diaphragm as the superstructure expands and pushes the diaphragm into the soil. One solution to this is to place a layered geotextile fabric wall behind a semi-integral diaphragm. The wall would be placed several inches behind the diaphragm providing a gap between the diaphragm and embankment soil. This gap will allow the bridge to expand and contract without engaging the soil behind the diaphragm thereby removing soil pressure from the back of the diaphragm. This reinforced soil detail would allow a semi-integral design to be used for an 850 feet long structure. Expansion joints would still need to be provided to accommodate the thermal expansion of the superstructure. By providing a semi-integral abutment, the potentially leaky expansion joints could be moved away from the beam/girder ends to the ends of the approach slab. This would allow the advantages of the semi-integral design to be realized on this structure.

On the south end of the bridge, the end of the approach slab falls within the limits of the location of the access roads for the lock control house to the west and for the AEP site to the east. If a design is chosen where the expansion joint is installed at the end of the approach slab, the layout of the site is such that the expansion joint would be placed in the middle of this access road.

Typically a bridge with a semi-integral design is considered to be restrained by the embankment soil behind the semi-integral diaphragm and none of the piers are fixed. With the layered geotextile fabric wall behind the semi-integral diaphragm, the bridge may no longer be fully restrained. Therefore, if this design is provided, ELR will investigate whether at least one of the piers needs to be fixed in order to properly restrain the superstructure.

Pier Type

For structures over waterways, in addition to supporting the design roadway loadings and dead load of the superstructure, the piers will be designed to withstand the impact force from water, ice, debris and the effects of scour. The height, width, and drilled shaft size for each pier is affected by the span arrangement and beam depth; these factors have been considered in the cost estimates.

A wall-type pier is a common design often used to support bridges spanning over major waterways. This pier type provides only one location for debris to build up in the river whereas a cap-and-column pier with three columns will have three potential locations for debris accumulation. Additionally, a wall-type pier more closely matches the visual appearance of the current bridge.

Deck Joints and Bearings

The superstructure expansion and contraction movements could be accommodated at the abutments. When using a 5-inch strip seal, ODOT's design guidelines limit the thermal expansion length of the contributing portion of the structure to 427 feet for 0° to 15° skew structures. The proposed structure length of approximately 850 feet places this bridge near the allowable thermal expansion limits for a strip seal.

Providing an even number of spans and fixing the middle pier at the center of the bridge would provide thermal expansion lengths of approximately 425 feet. Expansion movements produced by 425 feet of either steel or concrete stringers could be accommodated with strip seals. However, this is near the design limit of what can be handled by strip seals. Depending on the final pier arrangement, modular expansion joints or finger joints may need to be provided. This will be investigated during detail design of the structure.

It is anticipated that elastomeric bearings with internal steel laminates can be used at the piers. ODOT recommends that the height of elastomeric bearings be limited to a total of 5 inches. In order to accommodate the movement at the abutments, elastomeric bearings would need to be taller than 5 inches. Therefore, elastomeric bearings modified to include a PTFE sliding surface, disc bearings or other bearing types are recommended at the abutments due to the magnitude of thermal expansion movement at these locations.

For an 850 feet long bridge, the types of deck joints and the types of bearings (fixed or expansion) have an effect on each other and influence how movements and loads are transferred to the substructure units. These cause and effect relationships will be further studied during detail design to determine the most cost effective combination of joints and bearings.

An investment in bearings that accommodate large movements and transmit minimal longitudinal loads can influence the design and cost of the substructure units. A study will be performed for the various bearing and deck joint options and the resulting substructure design demands. Minimizing longitudinal forces and the associated design moments imposed on the substructure units will lessen the reinforcing steel requirements and drilled shaft embedment into bedrock.

Corrosion Protection – Steel Girders

Three primary options exist for protection against corrosion of the steel girders:

- Painted Steel
- Galvanized Steel
- Weathering Steel (MCEO does not want weathering steel because of the moisture in the air from the nearby water flowing over the dam)

Painted steel is a common corrosion protection method used in the State of Ohio. The main drawback of a paint protection system is that it will need to be sand blasted and repainted periodically in order to continually protect the superstructure structural steel from corrosion. Sand blasting and repainting is complicated because of containment requirements and also is expensive because of the fact that the bridge traverses a relatively large waterway making the beams hard to reach for painting

Hot-dip galvanizing provides an excellent long-lasting corrosion protection system which is anticipated to be relatively maintenance free throughout the life of the galvanized coating (approximately 100 years). Galvanized steel is often used on smaller bridges where beam/girder units can easily be hot-dipped. ELR contacted Kevin Irving with AZZ Galvanizing and Tom Langill with the American Galvanizers Association to discuss the design requirements and the feasibility of hot-dip galvanizing a structure of this size. Based on information from Mr. Irving and Dr. Langill, the girder sections would need to be a maximum of 55 feet long. This is a relatively short section when compared to the span lengths of 185 feet or 240 feet. Using relatively short 55 feet girder sections will require the girders to be fabricated in approximately twice as many sections as would typically be designed. This will increase the number of field splices and the cost of construction.

The hot-dip galvanizing process will subject the girders to rapid changes in temperature. Girder components with different thicknesses will thermally expand and contract at different rates. A typical plate girder design pairs a relatively thin web with a relatively thick flange. This design has the potential to warp during the heating and cooling processes due to the varying rate of expansion and contraction of the girder components. In order to reduce the risk of warping, it would be prudent to provide thicker girder webs. However, this still does not eliminate the risk of warping. There are not any published guidelines to comply with which would reduce the risk of warping. If a galvanizing bridge is pursued, it would be prudent to hot-dip a test girder section prior to bidding the whole bridge in order to determine whether warping will occur. The logistics for galvanizing a test girder will need to be determined. The additional cost of galvanizing this structure is estimated to be $0.20/lb to galvanize the steel and an additional $0.20/lb in construction costs associated with smaller girder sections and additional field splices. These costs do not include any contingencies for the risk of warping.

Weathering steel can provide an excellent long-lasting corrosion protection system when used in the appropriate atmospheric circumstances. Typically, weathering steel performs well on bridges which are not exposed to a highly corrosive environment and which are not continuously exposed to long term moisture conditions. The proposed structure is elevated relatively high above the Muskingum River. But due to the dam nearby, the bridge is frequently encased in fog. If the girders were to remain relatively dry, the steel will form a stable rust-like appearance when exposed to the elements for a prolonged period of time. The beams may require painting in the future if the moisture from the frequent fog conditions causes extended damp conditions on the surface of the steel which thereby leads to unanticipated deterioration. The deck will protect the girders from deicing salts. Leaky deck joints can allow salt laden roadway runoff to corrode the girder ends. Modern semi-integral abutment construction eliminates this potential point of leakage by removing the joints at the ends of girders. Weathering steel girders combined with a semi-integral abutment design could make weathering steel a viable alternative for the MUS-CR32-0.00 structure. Of the three corrosion protection systems (painting, galvanizing, weathering steel patina) , weathering steel will require the smallest initial investment. For cost analysis purposes, it is reasonable to plan for the weathering steel girders to be painted after 50 years due to the potential for fog to cause deterioration. Special pier protection details are recommended to be applied during construction to avoid the potential for rust staining of the proposed new piers.

Deterioration Prevention – Concrete Beams

Concrete is generally a corrosion resistant material when compared with structural steel. However, deterioration can still occur, particularly in the fascia beams which have the greatest exposure to the elements. This deterioration will reduce the lifespan of the concrete beams. In order to ensure that the estimated 75 year lifespan is achieved, the fascia beams should be re-sealed with a silane/siloxane sealer every fifteen years. We understand that MCEO prefers to avoid having to perform ongoing maintenance efforts.

Stay-in-Place Deck Formwork

Stay-in-place (SIP) forms will speed up construction of this bridge by eliminating the time required to remove the deck formwork. However, their additional weight will require stronger beams in order to support the additional loading. Based on preliminary analysis, SIP forms will require a 2%-3% increase in beam/girder strength in order to support the weight of the SIP forms and the additional concrete. This translates to an increase of approximately 4% in the cost of beams/girders.

Maintenance Considerations

For some structural elements, keeping up with preventative maintenance is necessary in order to achieve the full design lifespan. It can be the case with budget-constrained agencies that preventative maintenance, such as painting or concrete sealing, is ignored in favor of more urgent maintenance issues, such as failing roadways or structural elements. This results in robbing from the future in order to satisfy current budgetary needs. This comes in the form of reducing structure lifespans and causing more costly repair work down the line. Some consideration should be given to alternatives which will require minimal preventative maintenance. These alternatives (such as galvanized steel) will have a greater likelihood of achieving their full design lifespan when constructed in Muskingum County.

The bridge deck may need to be replaced in approximately 40 years. It will be beneficial if traffic can be maintained while the deck is being replaced. Modern part-width construction techniques allow for the deck to be replaced in a part-width phase construction manner, while maintaining traffic on a portion of the structure. Implementation of future part-width construction is affected by current design choices. Part-width construction can only be performed if a sufficient number of beams are provided. This report analyzes alternatives with five and six beam lines. Providing a greater number of beams will provide more flexibility for future part-width construction.

Aesthetics

This structure will impose a prominent visual impact on the river setting and the overall environment surrounding this CR 32 bridge crossing site as well as Philo Lock & Dam #9. The following list of aesthetic design features has been provided for consideration:

- For the steel option, a slight haunch could be provided. (we understand that a haunch is not preferred By MCEO)
- For the steel option, the fascia beams could be painted.
- Wall-type piers, supported on a single row of drilled shafts, tapered to match existing piers could be provided, although cap and column piers cost less to construct. (MCEO prefers the wall-type design)
- Texturing of concrete pier and abutment surfaces would provide a desirable overall appearance.
- Pedestrian lighting could be provided on the concrete traffic barrier adjacent to the sidewalk.
- The appearance of the roadway barriers could be enhanced by using form liners to provide architectural recesses rather than a smooth continuous surface. The current bridge transverse section width assumes aesthetic treatment will be applied to all barriers.

Concrete Alternative Design Considerations

The following information was used to assist in the design of the preliminary girder sections:

- Resources: Prestressed Concrete Institute (PCI) Design Handbook, ODOT Bridge Design Manual, ODOT's Standard Drawings for Prestressed Concrete I-beams, review of current bridge types in ODOT's inventory and compilation of pertinent design projects.
- Girder spacing: Based on the LRFD design code, a balanced deck overhang is approximately 30% of the interior girder spacing. In order to optimize and stretch the span length limits of the concrete I-beams, girder spacing that use 5 and 6 beam lines were studied.
- Preliminary Design: LEAP Conspan was the software used to perform the beam design. In order to provide a reasonable span length over the existing lock channel, a stub abutment and a 70' first span were provided on the south bank of the river. This configuration allows the lock channel to be spanned using a 130' concrete span. The overall length of 850 feet was determined based on providing a stub abutment on the south and north sides of the river.

Steel Alternative Design Considerations

The following is background information used to determine the preliminary steel girder design sections:

- Resources: National Steel Bridge Alliance (NSBA) Steel Bridge Design Handbook Chapter 8, ODOT Bridge Design Manual (BDM), USS steel plate girder design charts & technical bulletins and compilation of past pertinent design projects.
- Girder Spacing: Based on the LRFD design code a balanced deck overhang is approximately 30% of the interior girder spacing. With the Muskingum County Engineer's preferences in mind two different girder spacings were evaluated for cost. Five (5) girders with 4 spaces at 9'-6" with 3'-7" overhangs and six (6) girders with 5 spaces at 7'-8" with 3'-5" overhangs were considered. Both alternatives will accommodate future part-width deck replacement and the designs provide the desired structural redundancy. Consideration was given to a four (4) girder alternative. However, this would provide less redundancy, require the profile grade be increased, and future part-width deck replacement would be less feasible.
- Span Length and Galvanized Steel: Using galvanized steel will require that girder sections be limited to approximately 50 ft in length. If galvanized steel is utilized, shorter spans are more desirable as they will result in fewer mid-span girder splices. The five-span steel alternatives are preferred for a galvanized steel bridge.
- Galvanized Steel Considerations: Preliminary design of galvanized girders uses a ¾" thick web in order to reduce the risk of girder warping during the hot-dipping process. This is thicker than what is required to meet the structural design requirements for the web.

V. Summary of Studied Design Alternatives

A total of six (6) structure design alternatives are presented. Two (2) alternatives consist of pre-stressed concrete I-beams and four (4) alternatives consist of steel plate girders. Each plate girder alternative was analyzed using a painted, galvanized, or weathering steel corrosion protection system. For a site plan of each design alternative, see Appendix A.

All presented alternatives share the following design criterion:

- 850' Total structure length
- At least 76' of horizontal navigational clearance is provided near the lock structure between the edge of water and the face of the pier
- 45'-2" Bridge deck width with two 12'-0" traffic lanes, two 6'-0" shoulders and a 5'-0" sidewalk.
- Individual piers consist of a wall-type pier with a footing supported on drilled shafts
- Stub type abutments supported on drilled shafts with expansion joints provided at the ends of the deck.
- Elastomeric bearings with one fixed pier at or near the center of the bridge

The primary differences between each of the presented design alternatives are detailed in Table 1.

	Structure Alternative Summary					
	Concrete Alternatives		Steel Alternatives			
Alternative Name	C1	C2	S1	S2	S3	S4
# of Spans	7	7	5	5	4	4
# of Beam Lines	5 Spaced @ 9'-6"	6 Spaced @ 7'-8"	5 Spaced @ 9'-6"	6 Spaced @ 7'-8"	5 Spaced @ 9'-6"	6 Spaced @ 7'-8"
# of Piers	6	6	4	4	3	3
Span Arrangement	70', 6 @ 130'	70', 6 @ 130'	164', 3 @ 174', 164'	164', 3 @ 174', 164'	185', 240', 240', 185'	185', 240', 240', 185'
Beam/Girder Type/Size	WF72-49 (72") Wide Flange Prestressed	WF66-49 (66") Wide Flange Prestressed	Plate Girder with 66" Web	Plate Girder with 66" Web	Plate Girder with 84" Web	Plate Girder with 84" Web

Table 1 - Alternative Summary

Construction Cost Comparisons

Preliminary construction cost estimates were prepared for the six concrete and steel beam alternatives. Costs are presented for each steel beam alternative using either weathering steel, galvanized steel, or painted steel. Cost Estimates were reviewed and confirmed by Ron Bauer with ODOT's Office of Estimating and also by the Kokosing Construction Company. Tables reflecting the unit costs for each material and the quantities associated with each alternative are provided in Appendices C and D. A summary of the cost for each alternative is shown in Table 2 and Table 3. The bridge structure located south of the main bridge is to be replaced by a culvert pending the completion of the environmental study. This replacement cost is estimated to be $350,000 and is not included in the totals shown in Table 2 and Table 3.

	Concrete Beam Alternatives	
	Alt. C1 7 Span 5 Beams	Alt. C2 7 Span 6 Beams
Total Structure Cost	$12,152,897	$12,419,486

Table 2 - Concrete Beam Alternatives Cost Summary

		Steel Girder Alternatives			
		Alt. S1 5 Span 5 Girders	Alt. S2 5 Span 6 Girders	Alt. S3 4 Span 5 Girders	Alt. S4 4 Span 6 Girders
Total Structure Cost	Painted	$13,302,249	$13,603,326	$14,219,090	$14,758,266
	Galvanized	$13,525,307	$14,094,855	$14,624,899	$15,123,327
	Weathering Steel	$12,357,807	$12,473,605	$13,094,899	$13,413,327

Table 3 - Steel Girder Alternatives Cost Summary

Providing either painted or galvanized steel comes at a cost premium relative to providing weathering steel. Providing a haunched steel girder will increase fabrication costs but reduce the total quantity of steel. These costs will typically offset each other and a straight steel girder alternative can be provided at similar cost to a haunched girder alternative. All cost estimates are produced by using a straight girder.

Life Cycle Cost Analysis

A structure with a lower up-front cost which requires significant recurring maintenance may prove to be less economical than a structure with a higher up-front cost and little to no recurring maintenance. Additionally, a structure with high up-front costs and a long service life may prove to be more economical than a structure with lower up-front costs and a shorter service life.

Potential maintenance items include:

- Deck Replacement: 40 years
- Sealing of concrete: 15 years
- Painting of Girders
    - (Painted Steel Alternative): 30 years
    - (Weathering Steel Alternative): 50 years

A number of factors affect the lifespan of the structure including the effect of corrosive elements in the environment, the type of corrosion protection system used, and the as-built thickness of corrosion protection systems. A review of available information indicates the following lifespans for each beam/girder type with the associated maintenance work for this bridge site:

- Prestressed Concrete I-Beams: 75 years; with facia girder sealing every 15 years
- Weathering Steel Girders: 75 years; with painting at 50 years
- Galvanized Steel Girders: 100 years; with little to no maintenance

The value of a structure whose lifespan extends far into the future is hard to accurately quantify. Predicting the operational and loading needs of a structure 50 years into the future is at-best a guess. For example, a structure may take 100 years to deteriorate beyond repair, but it may need to be replaced or substantially modified after only 50 years' time due to a change in the demands on the structure. Many existing structures built more than 50 years ago, including the current CR32 structure, are functionally obsolete due to changes in traffic demands. A structure whose predicted lifespan is 100 years may be no more valuable than a structure whose predicted lifespan is 50 or 75 years.

A life cycle cost analysis was performed based on a 75 year time horizon for three different beam/girder material designs including their associated maintenance items and lifespans. The total Life Cycle Cost is equal to the sum of the Present Worth of all anticipated maintenance work over the service life of the structure. Recurring maintenance items which apply to all three alternatives, such as deck replacement, would not affect the final results and have been ignored. The least expensive 6-beam line alternatives were chosen and analyzed for each superstructure beam/girder type:

- Alt. C2 - Prestressed Concrete I-Beams
- Alt. S2 - Weathering Steel Girders
- Alt. S2 - Galvanized Steel Girders

Results of the analyses are presented in Table 4. The complete analyses mathematics are presented in Appendix C.

Total Structure Life Cycle Costs (Present Worth)	
Prestressed Concrete	$12,533,236
Weathering Steel	$12,896,793
Galvanized Steel	$13,235,967

Table 4 - Life Cycle Cost Analyses Results Summary

At the end of the 75 year time horizon, the galvanized structure is estimated to have 25 years of remaining service life. This value was included in the analysis. Results from the life cycle cost analysis indicate that a prestressed concrete beam superstructure will be the most economical design.

VI. Conclusions and Recommendations

This report provides preliminary design information for concrete and steel superstructure alternatives and recommended substructure design details for a variety of span arrangements. The information in this Structure Type Study will be the basis used to choose the preferred span arrangement and girder type. A final hydraulic analysis has been performed to verify that the preferred span arrangement does meet the project hydraulic requirements for the proposed roadway alignment. A foundation investigation plan has already been established for this project and some of the borings have been completed.

It is assumed that for this stage of the design, a margin of accuracy in the cost estimate is in the 5% range. Both five (5) and six (6) beam/girder line alternatives were studied for each span arrangement. Analyses indicate that providing five beams will be slightly more economical for each span arrangement and beam/girder type. However, a six beam/girder line alternative is preferred because it will provide more flexibility for future phased construction.

Span arrangements consisting of four (4) and five (5) spans were studied for the steel girder superstructure alternatives. Results from the cost analyses indicate that providing (5) spans will be a more economical solution. The increased cost of providing an additional pier for the 5 span arrangement is more than offset by the savings in steel girder costs.

Life cycle costs and initial costs need to be considered. Both the initial costs and the life cycle costs favor prestressed concrete beams.

	Design Alternative		
Consideration	**Concrete Beam Alternative C2**	**Weathering Steel Alternative S2**	**Galvanized Steel Alternative S2**
Initial Cost	$ 12,419,486	$ 12,473,605	$ 14,094,855
Life Cycle Cost*	$ 12,533,236	$ 12,896,793	$ 13,235,967
Maintenance Needs	Minimal, painting of surfaces	Painting @ 50 years due to fog prone area	None
Ease of Repairs	Difficult	Moderate	Moderate
Matches Current Bridge	No	Somewhat	Yes
Aesthetics	Acceptable	Undesirable	Preferred
Life Expectancy	75 years	50-100 years	100 years

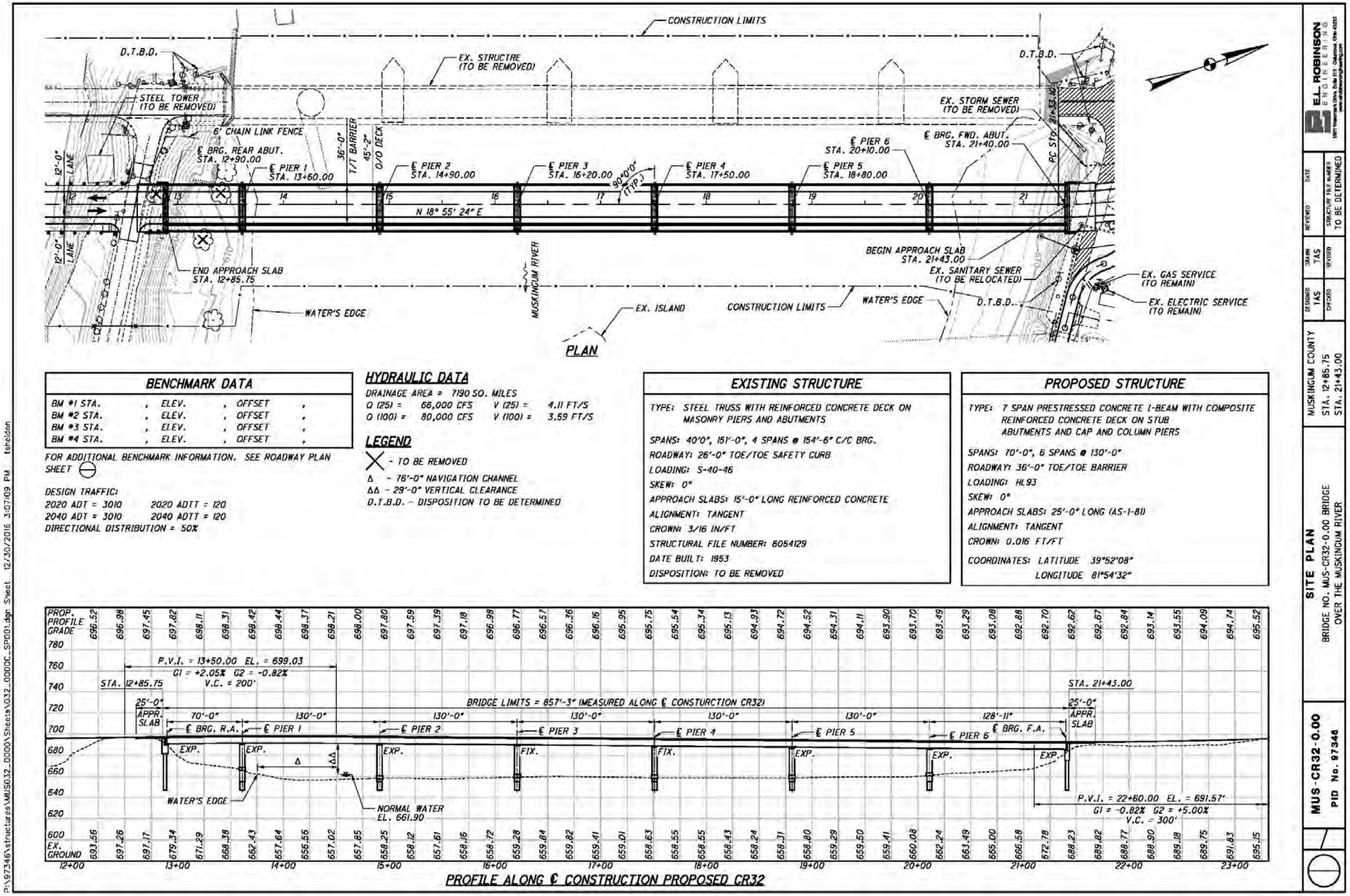
*Based on 75 year analysis

Table 5 - Design and Cost Considerations

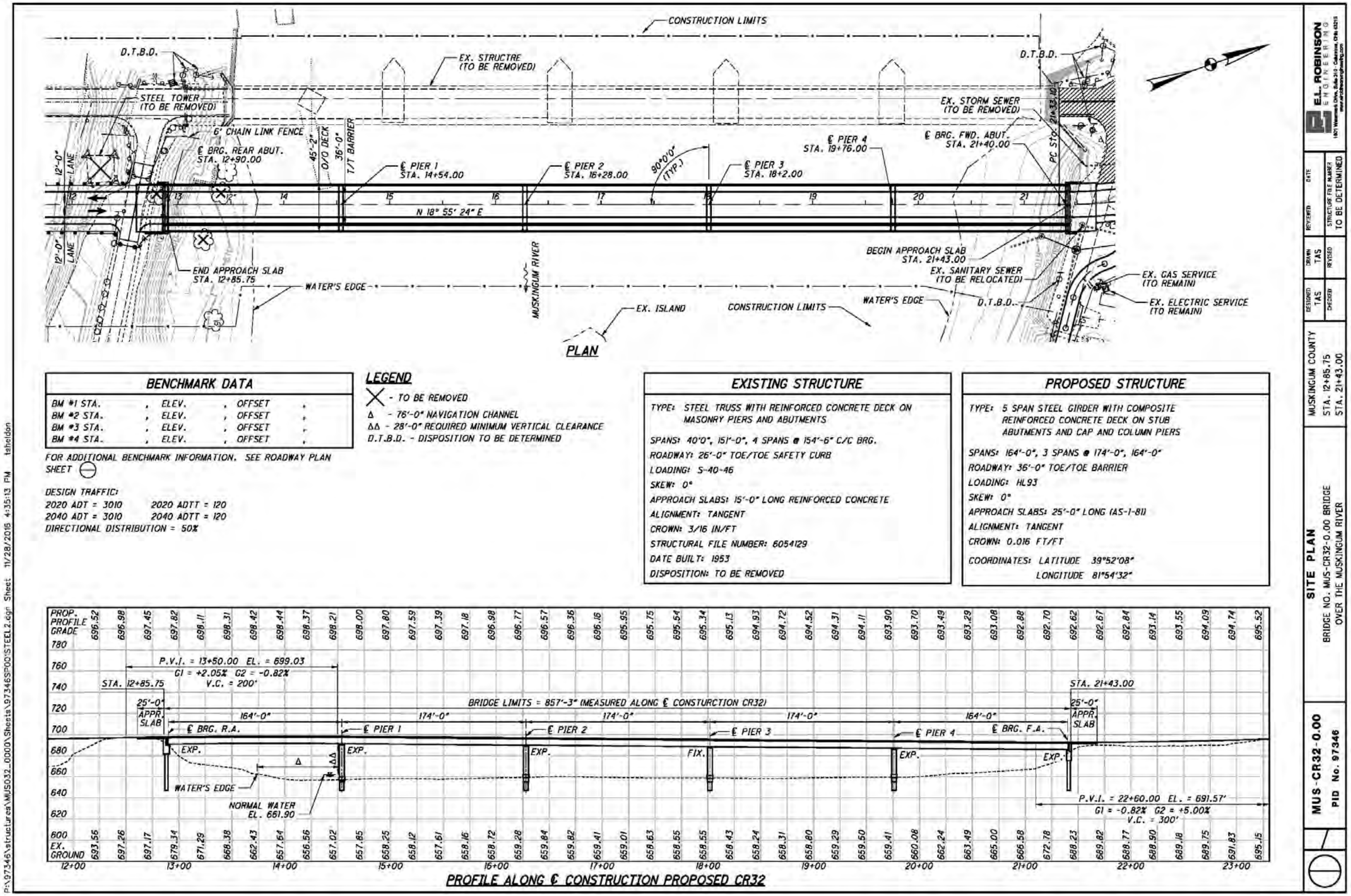
Based on all design, construction, and life cycle considerations, the final recommendation is to construct a 7 span, 6 beam prestressed concrete I-beam superstructure on 6 wall-type piers with joints placed at each end of the bridge.

APPENDICES

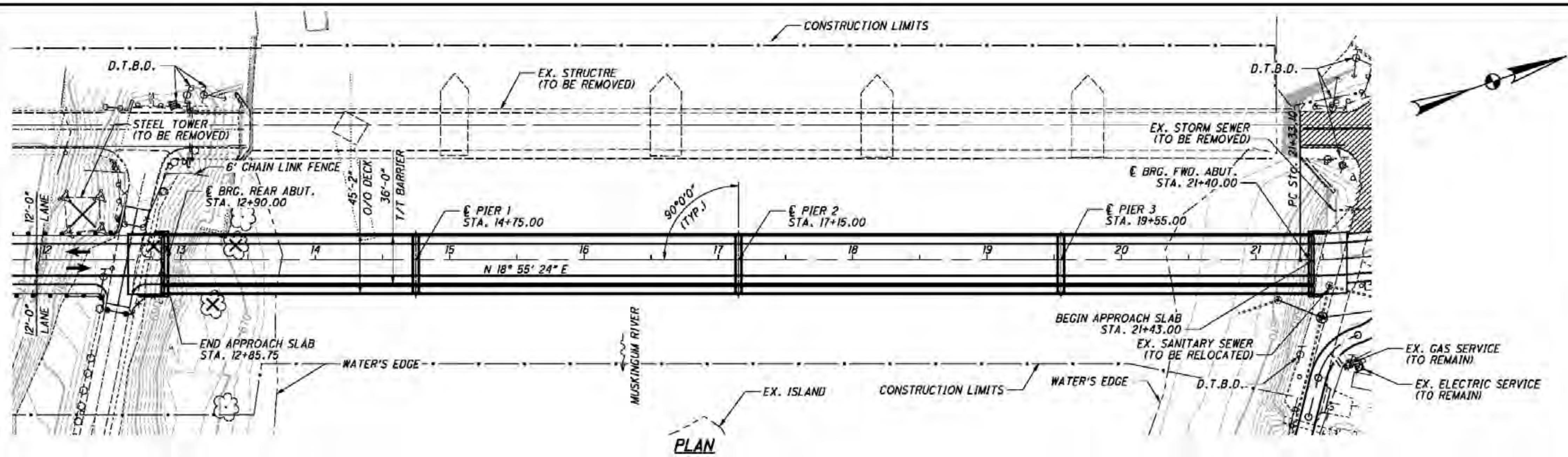
Appendix A Site Plans for All Structure Alternatives



SITE PLAN FOR ALTERNATIVES C1 & C2



SITE PLAN FOR ALTERNATIVES S1 & S2



BENCHMARK DATA

BM #1 STA. ,	ELEV. ,	OFFSET ,
BM #2 STA. ,	ELEV. ,	OFFSET ,
BM #3 STA. ,	ELEV. ,	OFFSET ,
BM #4 STA. ,	ELEV. ,	OFFSET ,

FOR ADDITIONAL BENCHMARK INFORMATION, SEE ROADWAY PLAN SHEET

DESIGN TRAFFIC:
2020 ADT = 3010 2020 ADTT = 120
2040 ADT = 3010 2040 ADTT = 120
DIRECTIONAL DISTRIBUTION = 50%

LEGEND

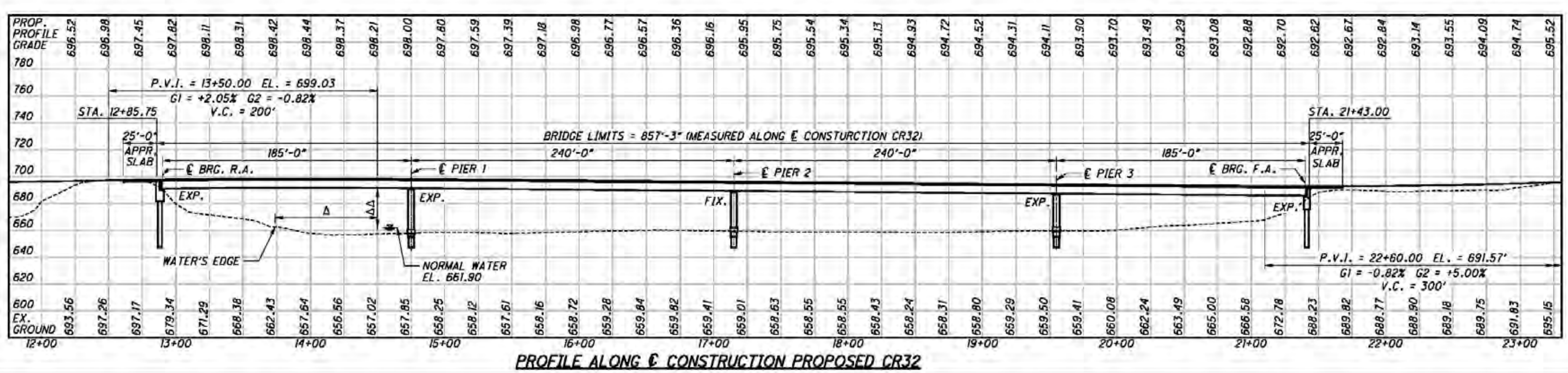
X - TO BE REMOVED
Δ - 76'-0" NAVIGATION CHANNEL
ΔΔ - 28'-0" REQUIRED MINIMUM VERTICAL CLEARANCE
D.T.B.D. - DISPOSITION TO BE DETERMINED

EXISTING STRUCTURE

TYPE: STEEL TRUSS WITH REINFORCED CONCRETE DECK ON MASONRY PIERS AND ABUTMENTS
SPANS: 40'0", 151'-0", 4 SPANS @ 154'-6" C/C BRG.
ROADWAY: 26'-0" TOE/TOE SAFETY CURB
LOADING: S-40-46
SKEW: 0°
APPROACH SLABS: 15'-0" LONG REINFORCED CONCRETE
ALIGNMENT: TANGENT
CROWN: 3/16 IN/FT
STRUCTURAL FILE NUMBER: 6054129
DATE BUILT: 1953
DISPOSITION: TO BE REMOVED

PROPOSED STRUCTURE

TYPE: 4-SPAN STEEL GIRDER WITH COMPOSITE REINFORCED CONCRETE DECK ON STUB ABUTMENTS AND CAP AND COLUMN PIERS
SPANS: 185'-0", 240'-0", 240'-0", 185'-0"
ROADWAY: 36'-0" TOE/TOE BARRIER
LOADING: HL93
SKEW: 0°
APPROACH SLABS: 25'-0" LONG (AS-1-81)
ALIGNMENT: TANGENT
CROWN: 0.016 FT/FT
COORDINATES: LATITUDE 39°52'08"
LONGITUDE 81°54'32"



E.L. ROBINSON ENGINEERING

MUSKINGUM COUNTY STA. 12+85.75 STA. 21+43.00

SITE PLAN BRIDGE NO. MUS-CR32-0.00 BRIDGE OVER THE MUSKINGUM RIVER

MUS-CR32-0.00 PID No. 97346

DESIGNED TAS DRAWN TAS

STRUCTURE FILE NUMBER TO BE DETERMINED

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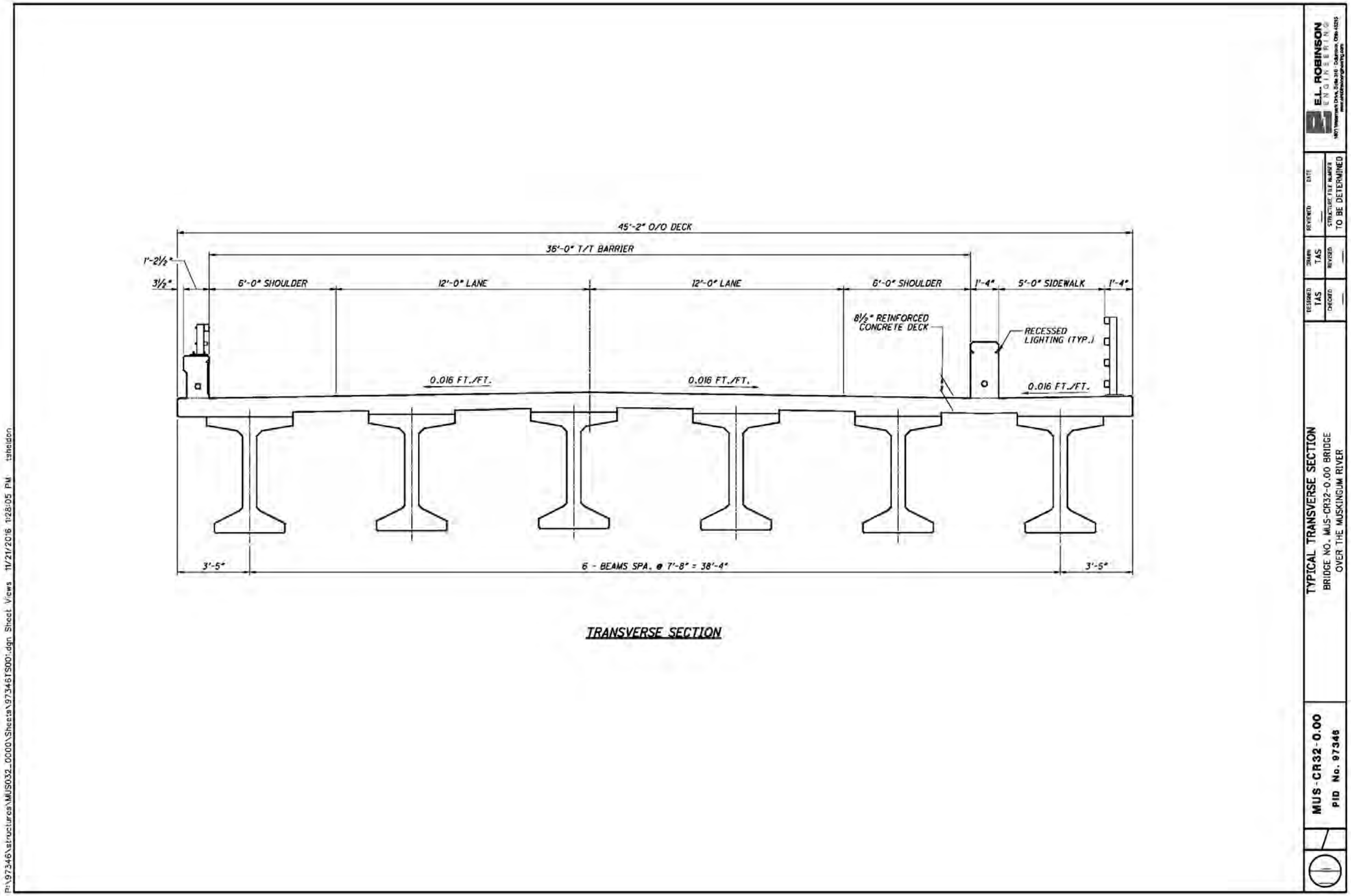
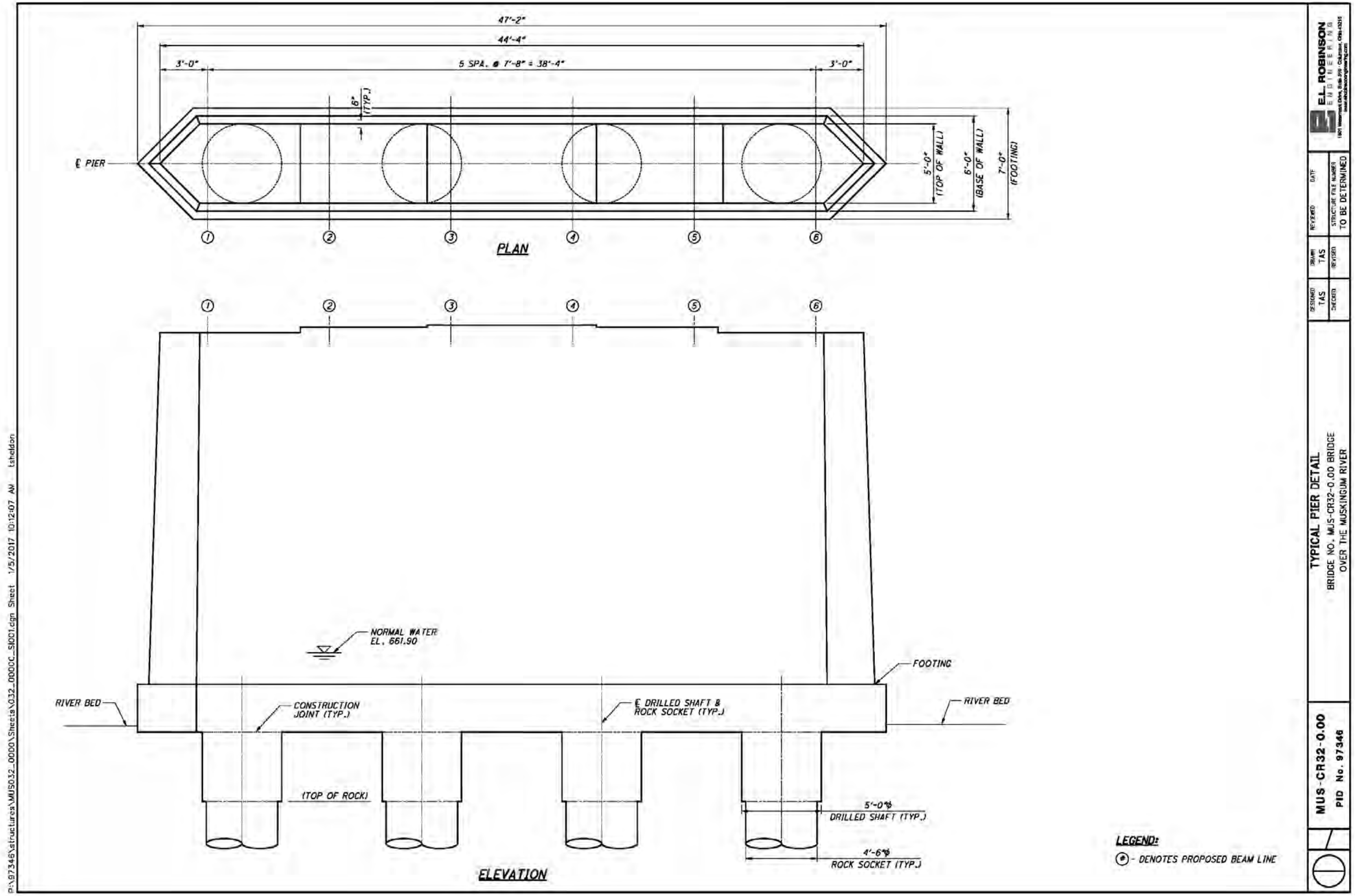
SITE PLAN FOR ALTERNATIVES S3 & S4

Appendix B Transverse Section, Pier and Abutment Details


45'-2" O/O DECK
36'-0" T/T BARRIER
1'-2½"
3½"
6'-0" SHOULDER
12'-0" LANE
12'-0" LANE
6'-0" SHOULDER
1'-4"
5'-0" SIDEWALK
1'-4"
8½" REINFORCED CONCRETE DECK
RECESSED LIGHTING (TYP.)
0.016 FT./FT.
0.016 FT./FT.
0.016 FT./FT.
3'-5"
6 - BEAMS SPA. @ 7'-8" = 38'-4"
3'-5"
TRANSVERSE SECTION
E.L. ROBINSON ENGINEERING
DESIGNED TAS
DRAWN TAS
TO BE DETERMINED
TYPICAL TRANSVERSE SECTION
BRIDGE NO. MUS-CR32-0.00 BRIDGE
OVER THE MUSKINGUM RIVER
MUS-CR32-0.00
PID No. 97346



47'-2"
44'-4"
3'-0"
5 SPA. @ 7'-8" = 38'-4"
3'-0"
6" (TYP.)
℄ PIER
5'-0" (TOP OF WALL)
6'-0" (BASE OF WALL)
7'-0" (FOOTING)
PLAN
NORMAL WATER EL. 661.90
FOOTING
RIVER BED
CONSTRUCTION JOINT (TYP.)
℄ DRILLED SHAFT & ROCK SOCKET (TYP.)
RIVER BED
(TOP OF ROCK)
5'-0"⌀ DRILLED SHAFT (TYP.)
4'-6"⌀ ROCK SOCKET (TYP.)
ELEVATION
LEGEND:
① - DENOTES PROPOSED BEAM LINE
E.L. ROBINSON ENGINEERING
DESIGNED TAS
DRAWN TAS
STRUCTURE FILE NUMBER TO BE DETERMINED
TYPICAL PIER DETAIL
BRIDGE NO. MUS-CR32-0.00 BRIDGE OVER THE MUSKINGUM RIVER
MUS-CR32-0.00
PID No. 97346
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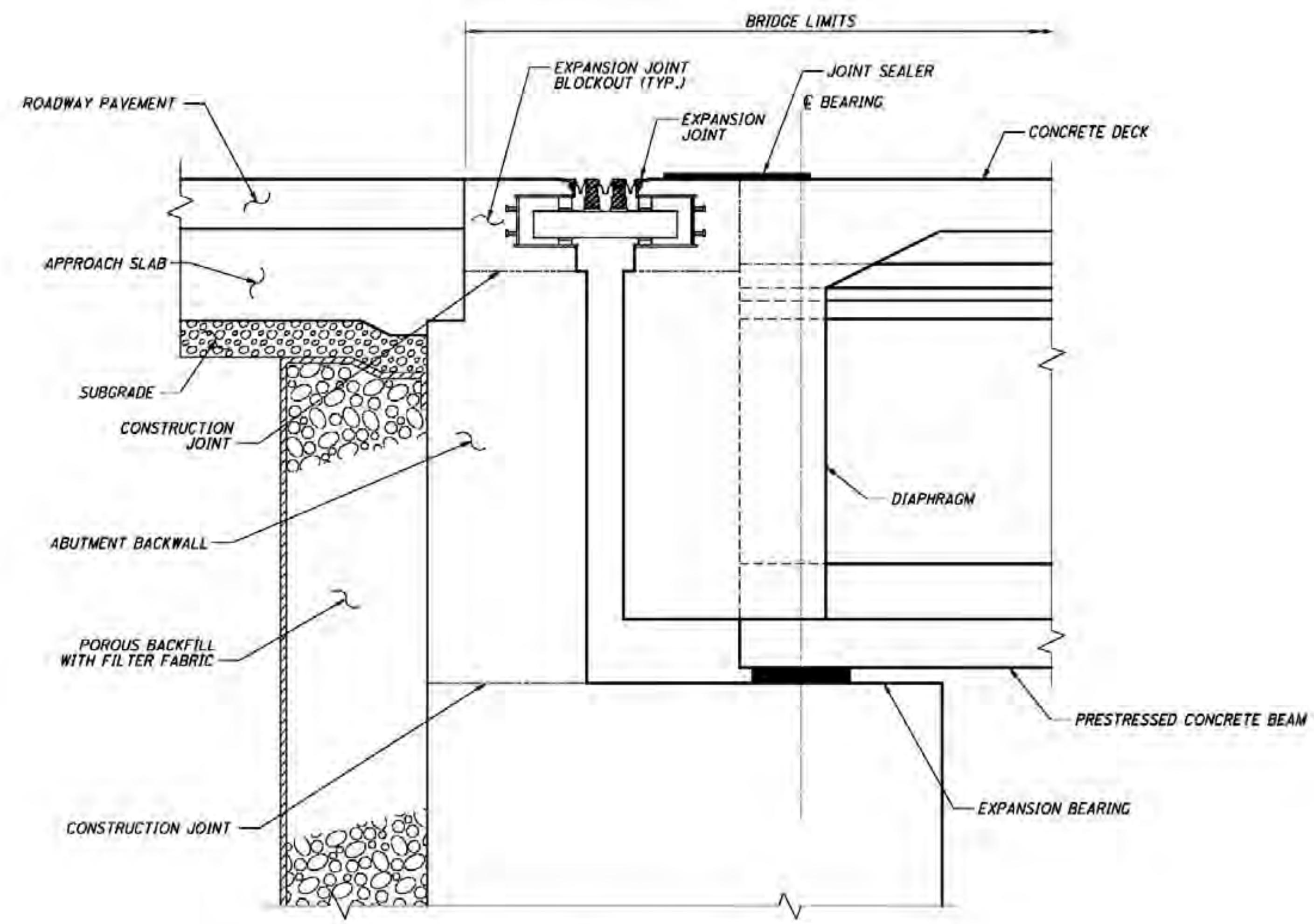

TYPICAL ABUTMENT SECTION

E.L. ROBINSON ENGINEERING

DESIGNED TAS | DRAWN TAS | REVIEWED | DATE | CHECKED | REVISED | STRUCTURE FILE NUMBER TO BE DETERMINED

TYPICAL ABUTMENT DETAILS
BRIDGE NO. MUS-CR32-0.00 BRIDGE
OVER THE MUSKINGUM RIVER

MUS-CR32-0.00
PID No. 97346

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Appendix C Preliminary Cost Estimates and Life Cycle Cost Analyses

Itemized Unit Price List					Concrete Beam Alternatives				Steel Girder Alternatives							
					Alt. C1 - 7 Span - 5 Beam		Alt. C2 - 7 Span - 6 Beam		Alt. S1 - 5 Span - 5 Girder		Alt. S2 - 5 Span - 6 Girder		Alt. S3 - 4 Span - 5 Girder		Alt. S4 - 4 Span - 6 Girder	
Item	Extension	Description	Unit	Unit Price	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost
202		STRUCTURE REMOVED, OVER 20 FOOT SPAN	SQ FT	$85	29,700	$2,524,500	29700	$2,524,500	29,700	$2,524,500	29,700	$2,524,500	29,700	$2,524,500	29,700	$2,524,500
503		COFFERDAMS AND EXCAVATION BRACING (BASIC CAUSEWAY ACROSS RIVER)	LUMP	$950,000	1	$950,000	1	$950,000	1	$950,000	1	$950,000	1	$950,000	1	$950,000
503		COFFERDAMS AND EXCAVATION BRACING (CAUSEWAY FOR PIER WORK)	EACH PIER	$40,000	6	$240,000	6	$240,000	5	$200,000	5	$200,000	4	$160,000	4	$160,000
503		UNCLASSIFIED EXCAVATION (PER SUBSTRUCTURE UNIT)	EACH	$10,000	8	$80,000	8	$80,000	6	$60,000	6	$60,000	5	$50,000	5	$50,000
509	10000	EPOXY COATED REINFORCING STEEL	POUND	$1.05	577,990	$606,889	581,916	$611,012	479,611	$503,591	474,931	$498,677	459,193	$482,153	454,993	$477,743
511		CLASS QC1 CONCRETE WITH QC/QA, PIER	CU YD	$850	1,446	$1,229,431	1,473	$1,251,680	970	$824,565	970	$824,565	688	$585,049	688	$585,049
511	43512	CLASS QC1 CONCRETE WITH QC/QA, ABUTMENT INCLUDING FOOTING	CU YD	$600	475	$284,828	475	$284,827	471	$282,356	471	$282,356	446	$267,534	446	$267,534
511	34446	CLASS QC2 CONCRETE WITH QC/QA, BRIDGE DECK	CU YD	$650	1,206	$783,900	1,206	$783,900	1,104	$717,600	1,086	$705,900	1,112	$722,800	1,097	$713,050
513	10300	STRUCTURAL STEEL MEMBERS, LEVEL 5 (WEATHERING OR PAINTED STEEL)	LB	$1.50					1,670,000	$2,505,000	1,770,000	$2,655,000	2,300,000	$3,450,000	2,470,000	$3,705,000
513	10300	STRUCTURAL STEEL MEMBERS, LEVEL 5 (GALVANIZED STEEL)	LB	$1.90					1,810,000	$3,439,000	2,080,000	$3,952,000	2,460,000	$4,674,000	2,670,000	$5,073,000
513	20000	WELDED STUD SHEAR CONNECTORS	EACH	$3.50					10,218	$35,763	12,262	$42,916	10,218	$35,763	12,262	$42,916
514	00060	FIELD PAINTING STRUCTURAL STEEL, INTERMEDIATE COAT	SF	$5.00					75,555	$377,777	90,378	$451,888	89,935	$449,677	107,595	$537,976
514	00066	FIELD PAINTING STRUCTURAL STEEL, FINISH COAT	SF	$5.00					75,555	$377,777	90,378	$451,888	89,935	$449,677	107,595	$537,976
515		DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM MEMBERS, LEVEL 3, TYPE WF66-49 (70')	EACH	$23,100			6	$138,600								
515		DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM MEMBERS, LEVEL 3, TYPE WF66-49 (130')	EACH	$42,900			36	$1,544,400								
515		DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM MEMBERS, LEVEL 3, TYPE WF72-49 (70')	EACH	$25,200	5	$126,000										
515		DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM MEMBERS, LEVEL 3, TYPE WF72-49 (130')	EACH	$46,800	30	$1,404,000										
515	20000	INTERMEDIATE DIAPHRAMS	EACH	$1,300	108	$140,400	135	$175,500								
516	10500	STRUCTURAL EXPANSION JOINT INCLUDING ELASTOMERIC COMPRESSION SEAL	FT	$550	92	$50,600	92	$50,600	92	$50,600	92	$50,600	92	$50,600	92	$50,600
516		ELASTOMERIC BEARING WITH INTERNAL LAMINATES AND LOAD PLATE (NEOPRENE)	EACH	$1,350	40	$54,000	48	$64,800	30	$40,500	36	$48,600	25	$33,750	30	$40,500
517	73200	RAILING (DEFLECTOR PARAPET TYPE)	FT	$150	882	$132,300	882	$132,300	882	$132,300	882	$132,300	882	$132,300	882	$132,300
517		RAILING (PEDESTRIAN)	FT	$240	882	$211,680	882	$211,680	882	$211,680	882	$211,680	882	$211,680	882	$211,680
517	75120	RAILING (CONCRETE PARAPET WITH TWIN STEEL TUBE RAILING)	FT	$175	882	$154,350	882	$154,350	882	$154,350	882	$154,350	882	$154,350	882	$154,350
518	21200	POROUS BACKFILL WITH GEOTEXTILE FABRIC	CU YD	$90	186	$16,740	186	$16,740	186	$16,740	186	$16,740	186	$16,740	186	$16,740
524	94704	DRILLED SHAFTS, 36" DIAMETER, INTO BEDROCK	FT	$600	240	$144,000	240	$144,000	240	$144,000	240	$144,000	240	$144,000	240	$144,000
524	94802	DRILLED SHAFTS, 42" DIAMETER, ABOVE BEDROCK	FT	$500	440	$220,000	440	$220,000	440	$220,000	440	$220,000	440	$220,000	440	$220,000
524	94908	DRILLED SHAFTS, 54" DIAMETER, INTO BEDROCK	FT	$1,000	192	$192,000	192	$192,000	176	$176,000	120	$120,000	168	$168,000	168	$168,000
524	94914	DRILLED SHAFTS, 60" DIAMETER, ABOVE BEDROCK	FT	$1,000	120	$120,000	108	$108,000	80	$80,000	80	$80,000	60	$60,000	60	$60,000
524	94918	DRILLED SHAFTS, 60" DIAMETER, INTO BEDROCK	FT	$1,400												
524	94930	DRILLED SHAFTS, 66" DIAMETER, ABOVE BEDROCK	FT	$1,300												
524	94934	DRILLED SHAFTS, 66" DIAMETER, INTO BEDROCK	FT	$1,900												
524	94946	DRILLED SHAFTS, 72" DIAMETER, ABOVE BEDROCK	FT	$1,600												
526	25000	REINFORCED CONCRETE APPROACH SLABS (T=15")	SQ YD	$225	252	$56,700	252	$56,700	252	$56,700	252	$56,700	252	$56,700	252	$56,700

Alt. C1		Alt. C2		Alt. S1		Alt. S2		Alt. S3		Alt. S4	
Sub Total =	$9,722,317	Sub Total =	$9,935,589	*Sub Total =	$10,820,246	*Sub Total =	$11,275,884	*Sub Total =	$11,699,919	*Sub Total =	$12,098,662
Contingency	25%	Contingency	25%	Contingency	25%	Contingency	25%	Contingency	25%	Contingency	25%
Superstructure	$3,386,468	Superstructure	$3,585,368	*Superstructure	$5,083,185	*Superstructure	$5,594,824	*Superstructure	$6,342,171	*Superstructure	$6,740,914
Substructure	$2,804,649	Substructure	$2,818,588	Substructure	$2,205,861	Substructure	$2,149,861	Substructure	$1,826,548	Substructure	$1,826,548
General	$3,531,200	General	$3,531,200	General	$3,531,200	General	$3,531,200	General	$3,531,200	General	$3,531,200
Total =	**$12,152,897**	**Total =**	**$12,419,486**	***Total =**	**$13,525,307**	***Total =**	**$14,094,855**	***Total =**	**$14,624,899**	***Total =**	**$15,123,327**

*Note: Totals are presented for girders fabricated from galvanized steel with no painting

Life Cycle Cost Analyses
(75 year time horzion)

	Concrete Beam Alternative				Weathering Steel Anternative				Galvanized Steel Alternative			
	Structure Alternative	C2			Structure Alternative	S2			Structure Alternative	S2		
	Est. Service Life	75	years		Est. Service Life	75	years		Est. Service Life	100	years	
WORK PERFORMED	YEAR	EST. COST	PWF	PWF x COST	YEAR	EST. COST	PWF	PWF x COST	YEAR	EST. COST	PWF	PWF x COST
New Bridge Construction	0	$12,419,486	1.000	$12,419,486	0	$12,473,605	1	$12,473,605	0	$14,094,855	1	$14,094,855
Re-Sealing of Facia Beams	15	$64,458	0.754	$48,601								
Re-Sealing of Facia Beams	30	$64,458	0.569	$36,677								
Re-Sealing of Facia Beams	45	$64,458	0.429	$27,653								
Paint Structural Steel					50	$1,084,531	0.390203	$423,188				
Re-Sealing of Facia Beams	60	$64,458	0.323	$20,820								
Value of Remaining Service Life @	**75**	$0	0.244	$0	**75**	$0	0.244	$0	**75**	$3,523,714	0.244	$858,889
		Maintenance Total =		$133,750		Maintenance Total =		$423,188		Maintenance Total =		$0
		Life Cycle Cost of Structure =		**$12,553,236**		Life Cycle Cost of Structure =		**$12,896,793**		Life Cycle Cost of Structure =		**$13,235,967**

$PWF = 1/(1+d)^n$

n = life cycle year in which work is performed

d = real discount rate* = 1.9%

*Note: The real discount rate is obtained from data in the Office of Management and Budget Circular A-94 Appendix C, dated 2/12/2016

Source: https://www.whitehouse.gov/sites/default/files/omb/memoranda/2016/m-16-05_0.pdf

Appendix D Estimated Quantities

Barrier Quantities

850' bridge length	
Traffic/Pedestrian Median Barrier	
Length of Individual Barriers	882 ft
Quantity	1
Total Length of Traffic/Pedestrian Barrier	**882 ft**
Edge-of-Deck Barriers	
Length of Individual Barriers	882 ft
Quantity	2
Total Length of Barriers With Twin Steel Tubes	**1764 ft**

Deck Quantities

850' bridge length						
Alternative	**C1**	**C2**	**S1**	**S2**	**S3**	**S4**
# Beams	**5 Beams**	**6 Beams**	**5 Girders**	**6 Girders**	**5 Girders**	**6 Girders**
Beam Type	**Concrete**	**Concrete**	**Steel**	**Steel**	**Steel**	**Steel**
Width	45.17 ft	45.17 ft	45.17 ft	45.17 ft	45.17 ft	45.17 ft
Thickness	8.50 in	8.50 in	8.75 in	8.50 in	8.75 in	8.50 in
Length	854 ft	854 ft	854 ft	854 ft	854 ft	854 ft
Haunch Thickness	3 in	3 in	2 in	2 in	2 in	2 in
Haunch Width	49 in	49 in	28 in	28 in	32 in	32 in
# beam lines	6	6	5	6	5	6
Volume =	**1206 cu yd**	**1206 cu yd**	**1104 cu yd**	**1086 cu yd**	**1112 cu yd**	**1097 cu yd**
Rebar Density	260 lb/cy	260 lb/cy	260 lb/cy	260 lb/cy	280 lb/cy	280 lb/cy
Total Rebar =	**313,560 lbs**	**313,560 lbs**	**287,040 lbs**	**282,360 lbs**	**311,360 lbs**	**307,160 lbs**

Approach Slabs

Width	45.17 ft
Length	25 ft
Area =	**126 sq yd**

Concrete Beam Totals

Number of Intermediate Diaphragms per Structure							
Alternative	First Span Length	Number of Diaphragms in First Span	Main Span Length	Number of Diaphragms per Main Span	Number of Main Spans	Number of Beam Lines	Total Number of Diaphragms
C1	70	3	130	4	6	5	108 each
C2	70	3	130	4	6	6	135 each

	C1 - Beam Costs		C2 - Beam Costs	
	130 ft span	70 ft span	130 ft span	70 ft span
Beam	ODOT WF72-49	ODOT WF72-49	ODOT WF66-49	ODOT WF66-49
Beam Linear Cost	$360 /LF	$360 /LF	$330 /LF	$330 /LF
Cost Per Beam	$46,800	$25,200	$42,900	$23,100
# of Spans	6	1	6	1
# of Beams Per Span	5	5	6	6
Total # of Beams	30	5	36	6
Beam Costs	$1,404,000	$126,000	$1,544,400	$138,600
Total Beam Cost	$1,530,000		$1,683,000	

Bearing Pads

Alternative	C1	C2	S1	S2	S3	S4
Number of Spans	7	7	5	5	4	4
Number of Beam	5	6	5	6	5	6
Number of Bearing	40	48	30	36	25	30

Weathering Steel Beam Alternatives Analyzed

Unit Weight of Steel = 490 lbs/cf   Contingency = 110%

Alternative	# Girders	Total Number of Spans	End Span (ft)	Middle Spans (ft)	Web						Girder Flange 1						Girder Flange 2					
					Depth (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)
S1	5	5	164.00	174.00	66.00	0.5	850.00	194.79	11	9350	18.00	1.75	268.00	117.25	4.71	1262	24.00	3.00	140.00	140.00	6.42	898
S2	6	5	164.00	174.00	66.00	0.5	850.00	194.79	11	9350	18.00	1.25	268.00	83.75	4.63	1240	24.00	2.50	140.00	116.67	6.33	887
S3	5	4	185.00	240.00	84.00	0.625	850.00	309.90	14	11900	16.00	1.25	510.00	141.67	4.10	2093	26.00	3.25	340.00	399.03	6.94	2359
S4	6	4	185.00	240.00	84.00	0.625	850.00	309.90	14	11900	16.00	1.00	510.00	113.33	4.06	2072	26.00	2.75	340.00	337.64	6.85	2330

Alternative	Girder Flange 3						Girder Flange 4						Girder Flange 5						Total Steel Volume (cu ft)	Total Steel Weight (lbs)	Total Paintable Surface Area (SF)
	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)			
S1	18.00	0.750	208.00	39.00	4.54	945	24.00	2.50	120.00	100.00	6.33	760	18.00	1.00	114.00	28.50	4.58	523	3098	1,670,000	75,555
S2	18.00	0.750	208.00	39.00	4.54	945	24.00	2.25	120.00	90.00	6.29	755	18.00	0.75	114.00	21.38	4.54	518	3274	1,770,000	90,378
S3																			4253	2,300,000	89,935
S4																			4565	2,470,000	107,595

Galvanized Steel Beam Alternatives Analyzed

Unit Weight of Steel = 490 lbs/cf   Contingency = 110%

Alternative	# Girders	Total Number of Spans	End Span (ft)	Middle Spans (ft)	Web						Girder Flange 1						Girder Flange 2					
					Depth (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)
S1	5	5	164.00	174.00	66.00	0.75	850.00	292.19	11	9350	18.00	1.50	268.00	100.50	4.63	1240	24.00	2.75	140.00	128.33	6.33	887
S2	6	5	164.00	174.00	66.00	0.75	850.00	292.19	11	9350	18.00	1.25	268.00	83.75	4.58	1228	24.00	2.50	140.00	116.67	6.29	881
S3	5	4	185.00	240.00	84.00	0.75	850.00	371.88	14	11900	16.00	1.25	510.00	141.67	4.08	2083	26.00	3.25	340.00	399.03	6.92	2352
S4	6	4	185.00	240.00	84.00	0.75	850.00	371.88	14	11900	16.00	1.00	510.00	113.33	4.04	2061	26.00	2.75	340.00	337.64	6.83	2323

Alternative	Girder Flange 3						Girder Flange 4						Girder Flange 5						Total Steel Volume (cu ft)	Total Steel Weight (lbs)
	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)	Width (in)	Thickness (in)	Total Length (ft)	Volume (cu ft)	Surface Area (SF/ft)	Surface Area (SF)		
S1	18.00	0.750	208.00	39.00	4.50	936	24.00	2.25	120.00	90.00	6.25	750	18.00	0.75	114.00	21.38	4.50	513	3357	1,810,000
S2	18.00	0.750	208.00	39.00	4.50	936	24.00	2.25	120.00	90.00	6.25	750	18.00	0.75	114.00	21.38	4.50	513	3858	2,080,000
S3																			4563	2,460,000
S4																			4937	2,670,000

Pier Quantities (Per Pier)

Alternative	C1	C2	S1	S2	S3	S4
Spans	**7 span**	**7 span**	**5 span**	**5 span**	**4 span**	**4 span**
# Beams/Girders	**5 Beams**	**6 Beams**	**5 Girders**	**6 Girders**	**5 Girders**	**6 Girders**
Average Wall Thickness	66.00 in	66.00 in	66.00 in	66.00 in	66.00 in	66.00 in
Average Wall Height	24.00 ft	24.50 ft	24.17 ft	24.17 ft	22.67 ft	22.67 ft
Average Wall Length	42.83 ft	42.83 ft	42.83 ft	42.83 ft	42.83 ft	42.83 ft
Wall Volume	**209.4 CY**	**213.8 CY**	**210.9 CY**	**210.9 CY**	**197.8 CY**	**197.8 CY**
Footing Thickness	3. ft	3. ft	3. ft	3. ft	3. ft	3. ft
Footing Width	6.5 ft	6.5 ft	6.5 ft	6.5 ft	6.5 ft	6.5 ft
Footing Length	43.83 ft	43.83 ft	43.83 ft	43.83 ft	43.83 ft	43.83 ft
Footing Volume	**31.7 CY**	**31.7 CY**	**31.7 CY**	**31.7 CY**	**31.7 CY**	**31.7 CY**
Total Concrete Volume	**241.1 CY**	**245.4 CY**	**242.5 CY**	**242.5 CY**	**229.4 CY**	**229.4 CY**
Reber Density	150 lb/cy	150 lb/cy	150 lb/cy	150 lb/cy	150 lb/cy	150 lb/cy
Total Rebar	**36,160 lbs**	**36,814 lbs**	**36,378 lbs**	**36,378 lbs**	**34,415 lbs**	**34,415 lbs**
Number of Columns/D.S.	4	4	4	4	4	4
Drilled Shaft Diameter	60.00 in	60.00 in	60.00 in	60.00 in	60.00 in	60.00 in
Average Drilled Shaft Length	5 ft	5 ft	5 ft	5 ft	5 ft	5 ft
Total Drilled Shaft Length	20 ft	20 ft	20 ft	20 ft	20 ft	20 ft
Bedrock Socket Diameter	54.00 in	54.00 in	54.00 in	54.00 in	54.00 in	54.00 in
Bedrock Socket Length	8.00 ft	8.00 ft	11.00 ft	11.00 ft	14.00 ft	14.00 ft
Total Bedrock Socket Length	32.00 ft	32.00 ft	44.00 ft	30.00 ft	56.00 ft	56.00 ft
Cofferdams	Yes	Yes	Yes	Yes	Yes	Yes

Shear Connector Quantities

Shear Connectors				
Alternative	S1	S2	S3	S4
Bridge Length	851.5 ft	851.5 ft	851.5 ft	851.5 ft
Number of Beam Lines	5	6	5	6
Estimated Spacing	1.25 ft	1.25 ft	1.25 ft	1.25 ft
Number of Connectors per Row	3	3	3	3
Total =	**10,218 each**	**12,262 each**	**10,218 each**	**12,262 each**

Abutment Quantities (each abutment)

Alternative	C1	C2	S1	S2	S3	S4
Spans	**7 span**	**7 span**	**5 span**	**5 span**	**4 span**	**4 span**
Beam Depth	**72 in**	**66 in**	**69 in**	**69 in**	**87 in**	**87 in**
Footing						
Width	5 ft	5 ft	5 ft	5 ft	5 ft	5 ft
Height	3 ft	3 ft	3 ft	3 ft	3 ft	3 ft
Length	75 ft	75 ft	75 ft	75 ft	75 ft	75 ft
Volume	42 cu yd	42 cu yd	42 cu yd	42 cu yd	42 cu yd	42 cu yd
Breastwall						
Width	7.00 ft	7.00 ft	7.00 ft	7.00 ft	7.00 ft	7.00 ft
Height	13.00 ft	13.00 ft	12.75 ft	12.75 ft	11.25 ft	11.25 ft
Length	46 ft	46 ft	46 ft	46 ft	46 ft	46 ft
Volume	155 cu yd	155 cu yd	152 cu yd	152 cu yd	134 cu yd	134 cu yd
Backwall						
Width	2.17 ft	2.17 ft	2.17 ft	2.17 ft	2.17 ft	2.17 ft
Height	7.00 ft	7.00 ft	7.25 ft	7.25 ft	8.75 ft	8.75 ft
Length	46 ft	46 ft	46 ft	46 ft	46 ft	46 ft
Volume	26 cu yd	26 cu yd	27 cu yd	27 cu yd	32 cu yd	32 cu yd
Wingwall						
Width	1.50 ft	1.50 ft	1.50 ft	1.50 ft	1.50 ft	1.50 ft
Height	13.33 ft	13.33 ft	13.33 ft	13.33 ft	13.33 ft	13.33 ft
Length (One WW)	10 ft	10 ft	10 ft	10 ft	10 ft	10 ft
Volume (Both WW)	15 cu yd	15 cu yd	15 cu yd	15 cu yd	15 cu yd	15 cu yd
Concrete Volume (Per Abutment)	**237 cu yd**	**237 cu yd**	**235 cu yd**	**235 cu yd**	**223 cu yd**	**223 cu yd**
Rebar Density	100 lb/cy	100 lb/cy	100 lb/cy	100 lb/cy	100 lb/cy	100 lb/cy
Total Rebar	**23,736 lbs**	**23,736 lbs**	**23,530 lbs**	**23,530 lbs**	**22,295 lbs**	**22,295 lbs**
Drilled Shafts/Rock Sockets						
# of Shafts	10 each	10 each	10 each	10 each	10 each	10 each
Drilled Shaft Length	22 ft	22 ft	22 ft	22 ft	22 ft	22 ft
Total Drilled Shaft Length	220 ft	220 ft	220 ft	220 ft	220 ft	220 ft
Rock Socket Length	12 ft	12 ft	12 ft	12 ft	12 ft	12 ft
Total Rock Socket Length	120 ft	120 ft	120 ft	120 ft	120 ft	120 ft
Porous Backfill						
Backfill Length	63 ft	63 ft	63 ft	63 ft	63 ft	63 ft
Backfill Height	20 ft	20 ft	20 ft	20 ft	20 ft	20 ft
Backfill Width	2 ft	2 ft	2 ft	2 ft	2 ft	2 ft
Backfill Volume	**93 cu yd**	**93 cu yd**	**93 cu yd**	**93 cu yd**	**93 cu yd**	**93 cu yd**

Appendix E Deck Surface Drainage Evaluation

Drainage Options

Three deck drainage options are presented for handling the rainfall discharge that accumulates on the proposed bridge deck and sidewalk surface area:

- full deck over-the-side drainage
- no over-the-side drainage
- sidewalk surface area only over-the-side drainage.

The over-the-side drainage solutions will consist of drainage water freely flowing over the sides of the deck and into the river below. The other solutions will consist of containing drainage water on the deck and funneling the water into a collection system consisting of catch basins located on the roadway at the ends of the bridge.

Option 1: Over-the-Side Drainage, Full Deck

Full deck over-the-side drainage can be accomplished by providing steel traffic and pedestrian railing in all three railing locations and by providing a sidewalk which is not elevated relative to the roadway. With this option, the deck will be partially cleaned during rain events and this option will also allow the deck to be cleaned with a water truck that includes a pressurized hose.

During rain events, water from the roadway will flow off the traveled lanes and onto the sidewalk area. Pedestrians using the sidewalk will have to walk through this runoff. The roadway runoff will include oils and other deposits from vehicular traffic along with trash that has been deposited on the bridge.

The combination of the height of the structure, the depth of the beams/girders, and the relative openness of the surrounding river area will allow a moderate amount of wind to blow the water from the deck over-the-side drainage onto the outside superstructure beams/girders. This drainage water will include vehicular oils as well as saltwater. Past experience has shown that the water from over the side drainage will tend to deteriorate the edge of the deck and also deteriorate the outside beams/girders.

The choice to use over-the-side drainage will limit the type of beams/girders which can be used for this structure. Weathering steel will likely be the least expensive option for a steel girder superstructure. However, if over-the-side drainage is provided, ELR does not recommend the use of weathering steel. The deck drainage will likely cause the girders to corrode at an accelerated rate.

The ODOT Bridge Design Manual 2007 Section 304.3.1 recommends the use of concrete traffic barriers when the finished deck surface is 25 feet or more above the water surface. The finished deck surface for the MUS-CR32-0.00 Bridge will be more than 25 feet above the water surface. However, ELR has contacted Tim Keller with ODOT, and he has agreed to waive the Section 304.2.1 recommendation for this structure. Therefore, it will be acceptable (although not preferred) by ODOT to provide steel railings on the bridge deck.

ELR and GPD contacted Michael Joseph and Joni Lung with the Ohio EPA to determine whether there are restrictions preventing the use over-the-side drainage for bridges over the Muskingum River. Neither Michael nor Joni are aware of any restrictions preventing the use of over-the-side drainage on the MUS-CR2-0.00 Bridge.

When assessing the long term performance of bridges with over-the-side drainage, it is worth considering the experience of engineers who are tasked with the long term maintenance of bridges. ELR discussed the topic of over-the-side drainage with Tim Keller (ODOT), David Flood (ODOT) and Mark Sherman (Franklin County Engineer's Office) and found the they strongly prefer that over-the-side drainage be avoided on relatively large bridges because of the potential for long term deterioration of the edge of the deck and the outside beams/girders.

Option 2: No Over-the-Side Drainage

This option can be accomplished by providing barriers with a concrete base which will contain and prevent the rainfall runoff from flowing over the sides of the deck. Barriers at the edge of the deck will have an upper portion made of steel tubing to ensure that the pedestrians and vehicle occupants will have a good view of the surroundings and the scenic river. Runoff will flow primarily toward the north end of the bridge and be funneled into catch basins placed at or near the end of the bridge. The sidewalk will be sloped toward the traffic/pedestrian barrier and drainage will be collected using a catch basin off of the bridge which is located under the traffic/pedestrian barrier. Cleaning of the bridge, if necessary, would be accomplished by using a pressurized hose or a street sweeper truck.

Containing the drainage to the bridge deck and funneling the drainage into a collection system will prevent water, laden with vehicular deposits and road salt, from flowing over the edge of the deck and potentially blowing onto the bridge beams/girders. This will reduce the potential for deterioration of the main beams/girders. Additionally, it will prevent roadway deposits and trash from flowing directly into the river.

Option 3: Over-the-Side Drainage, Sidewalk Surface Area Only

With this option, the roadway drainage would be contained to the bridge and collected at the end of the bridge using a catch basin. The sidewalk would be allowed to drain over the side of the deck and directly into the river. This can be accomplished by providing concrete barriers on both sides of the roadway while installing a steel railing at the edge of the deck. Sidewalk drainage will consist of a relatively low volume of over-the-side drainage. The more highly contaminated deck drainage will be contained on the bridge deck and subsequently collected in basins located on the roadway.

	Pros	Cons
Option 1 Over-the-Side Drainage, Full Deck, Steel Railing	• Deck is somewhat self-cleaning. Water will not be contained and therefore will spread into the shoulder and/or sidewalk area during rainfall events and ultimately will flow over the side of the deck.	• Deck drainage runoff may increase beam/girder and deck edge deterioration, thereby reducing the structure lifespan. • Greater initial costs. • Debris and litter from pedestrians and cars enters the river flow directly into the river.
Option 2 No Over-the-Side Drainage, Concrete Barriers	• Lowest initial cost. • Bridge deck and sidewalk surface area drainage will not be allowed to blow onto the outside girders. • Less anticipated overall maintenance • A potential longer structure lifespan due to reduced beam/girder deterioration.	• Bridge cleaning may require greater effort • Larger catch basins and pipes may be required, thereby increasing the total project cost.
Option 3 Over-the-Side Drainage, Sidewalk Only, Concrete Barriers adjacent to the roadway and a steel rail on the outside of the sidewalk	• Sidewalk is somewhat self-cleaning • Bridge deck traveled lane area drainage will not be allowed to blow onto the outside girders.	• Bridge cleaning may require greater effort. • Larger catch basins and pipes may be required, thereby increasing the total project cost • Sidewalk drainage may increase the deterioration of the outside beams/girders.

Table 1: Drainage Option Benefit Matrix

Drainage Options Cost Summary

The steel railing required for over-the-side drainage is more costly than the concrete barriers used to contain drainage on the surface of the deck. The over-the-side drainage options may increase the deterioration of the outside beams/girders corrosion reducing the lifespan of the structure. The life-cycle cost benefit of the over-the-side drainage is a reduction in deck cleaning costs. A summary of the initial costs incurred with each drainage option is shown in Table 2. The costs include a 25% contingency. The railing/barrier cost estimates were reviewed and confirmed by the Kokosing Construction Company.

	Cost	Cost Savings
Option 1: Over-the-Side Drainage, Full Deck, Steel Railing	**$750,000**	$0
Option 2: No Over-the-Side Drainage, Concrete Barriers	**$500,000**	$250,000
Option 3: Over-the-Side Drainage, Sidewalk Only, Concrete Barriers adjacent to the roadway and a steel rail on the outside of the sidewalk	**$540,000**	$210,000

Table 2: Drainage Option Cost Summary

Summary and Recommendations

Providing full deck over-the-side drainage (steel railing) requires an additional initial investment of $250,000 when compared with no over-the-side drainage (concrete barrier). There is a perception among practicing engineers that over-the-side drainage could lead to an increase in the deterioration of the outside of the deck and to the outside beams/girders that will be exposed to deck runoff. Allowing trash and deck runoff to directly enter the Muskingum River could be considered undesirable. Therefore, we recommend Option 2 which contains the roadway drainage to the deck surface and funnels the rainfall discharge into catch basins at the ends of the bridge. The recommended option has the lowest initial cost and the potential to increase the lifespan of the bridge. Representative photos showing the various railing alternatives are shown in photos 1 through 5.

Photo 1: Representative Steel Traffic Railing (Option 1)

Photo 3: Representative Concrete Traffic Barrier (Option 2)

Photo 2: Representative Steel Sidewalk Railing (Options 1 & 3)

Photo 4: Representative Concrete Sidewalk Barrier (Option 2) Note sidewalk will not be elevated

Photo 5: Representative Concrete Traffic/Sidewalk Barrier (Options 2 & 3)

Appendix F Disposition of Comments

Date: 9/28/2016
Project: MUS-CR32-0.00 (PID 97346)
ELR Project Manager: Rick Engel, P.E.

Structure Type Study - Disposition of Comments

The initial Structure Type Study is revised by ELR to incorporate comments received by Sandie Mapel of the Muskingum County Engineer's Office (Comments were given to ELR verbally and also in a marked set of ELR's report). Additionally, ELR received comments from Bill McEleney of the National Steel Bridge Alliance. A disposition of comments and ELR responses is provided below.

Comments by: Sandie Mapel
Agency: MCEO
Date of Comments: 9/19/2016

Comment #S1
A new bridge that allows legal loads will likely see additional traffic volumes.

ELR Response
Concur. Our report now reflects the concern for an increase in the number of legal loads.

Comment #S2
The MCEO preference is to use wall-type piers because of the maintenance related to debris collecting on pier columns.

ELR Response
Concur. Our report and cost estimates are now based on providing wall-type piers.

Comment #S3
Even with over the side drainage, a sag curve located on the bridge deck can create an ice and water trap if future deck replacements are not properly installed or debris builds up.

ELR Response
Concur. Our report now reflects this.

Comment #S4
Provide lighting with ballistic grade protection in the traffic/pedestrian barrier as well as in the outside traffic barrier to illuminate the roadway and sidewalk.

ELR Response
Concur. Our report now contains this lighting preference.

Comment #S5
Bridges requiring continuing maintenance are a challenge for the county because of the ongoing need to find funding for maintenance operations. The county's preference is for a structure with minimal required continuing maintenance.

ELR Response
Concur. Our report now reflects this MCEO concern.

Comment #S6
Alternatives providing 6 beam/girder lines are preferred over alternatives providing 5 beam/girder lines. Providing 6 beam/girder lines will simplify future part-width construction.

ELR Response
Concur. Life-cycle costing analysis now shows the costs of 6 beam/girder line alternatives.

Comment #S7
Paint will be expensive on this structure because the structural elements will be hard to reach.

ELR Response
Concur.

Comment #S8
MCEO prefers Alternative S2, five spans with six lines of galvanized beams.

ELR Response
Concur.

Commenter: Bill McEleney
Group: National Steel Bridge Alliance
Date of Comments: 9/20/2016

Comment #B1
I'm having trouble reconciling the cost numbers in the spreadsheet in Appendix C. When checking the superstructure and substructure categories, I can't get the category totals to match the sum of the individual line items. Example, summing all the Superstructure items in orange for Alt S1 (not including the 3 coatings items); I don't match the total shown below. Same situation for the Substructure items in blue. However, the general items in Green do match. And, the Subtotal at the bottom matches the total of the 3 category totals. Perhaps the cost in Item 509, rebar, has somehow been distributed into the Superstructure and Substructure subtotal? I'm not sure this affects the validity of the cost analysis, I just found it confusing

ELR Response
Correct. The rebar is distributed to the superstructure and substructure as required. This was done because rebar is typically shown under one pay item. The cost numbers are correct.

Comment #B2
Regarding the life cycle cost analysis – at the end of the service life the steel bridge will have value as recyclable material. Currently mills are paying in the order of $0.10/lb for scrap so the analysis should be adjusted to show (1,670,000 x 0.10 = $167,000) remaining value for both the weathering and galvanized steel alternates – narrowing the life cycle cost difference to 1% for the weathering steel option and 2% for the galvanized option.

ELR Response
The concrete may have some recycling value in the future as well. The beams could potentially be broken up and used as riprap on future projects at this site. These salvage values will likely offset each other and can be ignored in the cost analysis.

Additionally, the $167,000 salvage cost for steel needs to be converted to present dollars for use in the comparison. Salvaging the weathering steel 75 years into the future would only provide a present value savings of $41,000. Salvaging the galvanized steel 100 years into the future would only provide a present value savings of $25,000.

Comment #B3
Further regarding life cycle cost analysis, the report states that future deck replacement costs 'would not affect the final results and have been ignored'. This may be incorrect as experience shows that removing an old deck from WF72-49 type precast beams is more difficult than removing an old deck from a steel girder and often causes damage to the flanges of the precast beam. While this is difficult to quantify for the purpose of a life cycle cost analysis, it is a future risk that must be recognized.

ELR Response
Many older bridges do not have shear studs which would make deck removal relatively easy. This bridge will have shear studs which will complicate the deck removal. It seems that deck removal will be somewhat complicated for both the steel and concrete alternatives. I'm not sure either alternative sees a true advantage here. This is why this cost was ignored.

Comment #B4
Do the unit costs in the cost analysis consider erection/construction costs? If so, has consideration been given to the fact that significantly heavier cranes will be necessary to erect the prestressed beams - up to 80 tons each for 130' span vs less than 20 tons for the likely steel girder segment?

ELR Response
Crane costs may be higher for prestressed beams. However, beam/girder placement will be much more complicated for the steel girders, especially for the galvanized alternative which will have several splices in each span. The prices we provided are meant to reflect the contractor bid prices which include installation.

Comment #B5
Finally, for what's it's worth; weathering steel plate girders similar to this have bid in the Midwest (IL, OH) in the past 2 weeks for ± $0.90 / lb delivered to the job site. Adding $0.25-0.30 / lb for erection brings steel first cost close to that of prestress and steel life cycle cost lower than prestress for both the weathering steel and galvanized options.

ELR Response
Based on contact with ODOT and investigating costs they have recently paid for steel, $1.15-$1.20/lb seems low for a bid price. The prices we've shown reflect our best estimate of the average contractor's bid price. We are unsure whether the numbers above reflect the bid price paid by the state or agency. Galvanized steel requires an additional premium to dip as well as an additional handling and construction costs (extra splices, more pieces, shipping to the galvanizer, etc.). We have included all handling efforts in the cost as well. ELR received advice from Franklin County, an ODOT senior bridge estimator and also from the Kokosing Construction Company.

Commenter: ODOT District 5
Date of Comments: 10/14/2016

Comment #O1
Page 7 – We find the Cap and Column is acceptable with a straight alignment. But we are ok with the Wall type decision and understand the concern for debris collection. On page 6 last paragraph you still show Cap and Column as the pier of choice that should be corrected.

ELR Response:
Concur

Comment #O2
Page 8 – On page 8 in the second column under Deterioration Prevention you state that ODOT recommends that the fascia beams be resealed every 5 years. ODOT does not recommend a 5 year cycle or provide any resealing cycle for I beams. A 15 to 20 year cycle would be more realistic. Please revise.

ELR Response
The 5-year facia beam resealing schedule was taken from the preventative maintenance recommendations in ODOT's On-line Bridge Maintenance Manual. Our report has been updated to reflect a 15 year resealing schedule.

Comment #BO3
Page 11 – With the concerns of weathering steel that are expressed in the Structure Type Study, we believe that painting of the beams in the future should be part of any life cycle cost. We suggest one painting of whole structure at 50 years as a minimum. With over the side drainage the outside fascia beam should be painted initially and periodically on normal painting cycle of 30 years.

ELR Response
The structure type study has been updated to account for this recommended painting of the weathering steel.

Comment #O4
Page 12 – ODOT D-05 has the opinion that the Design should be a Prestressed I beam based on the Structure Type Study. The Galvanized option is not competitive with the Prestressed I beam and we do not feel that the Life Cycle cost between the two products are enough to justify the Galvanized option. Please make appropriate changes and show the Prestressed I beam as the preferred alternative.

ELR Response
The prestressed I beam is now shown as the preferred alternative.

Comment #O5
Page 12 – 6 beams in lieu of 5 beams. 5 beams would allow half width construction but 6 would be better for future traffic to be maintained during construction. We are ok with 6 beams for future Maintenance of Traffic considerations.

ELR Response
Concur.

Comment #O6
Page 12 - If the MCEO would prefer Galvanizing Steel versus the recommended Prestressed I beam, the MCEO may have the Designer prepare two sets of plans at the cost of the MCEO. We could prepare an alternate bid and if the Galvanized Steel bid is cheaper then it will be awarded and even if it is not the low bid, an arrangement could be made to have the MCEO pay for any additional cost to provide the Galvanized Beam over the Prestressed I beam option.

ELR Response
Concur

Appendix G Hydrology & Hydraulic Report

I. Project Overview

The Muskingum County Engineer's Office intends to construct a replacement structure for the existing County Route 32 Bridge over the Muskingum River. A new alignment will be utilized to allow continued use of the old structure during construction of the replacement structure. This structure connects the Village of Philo, OH with Duncan Falls, OH as shown in Figure 1 . This bridge is located near Philo Lock & Dam #9 and bridges the operational lock.

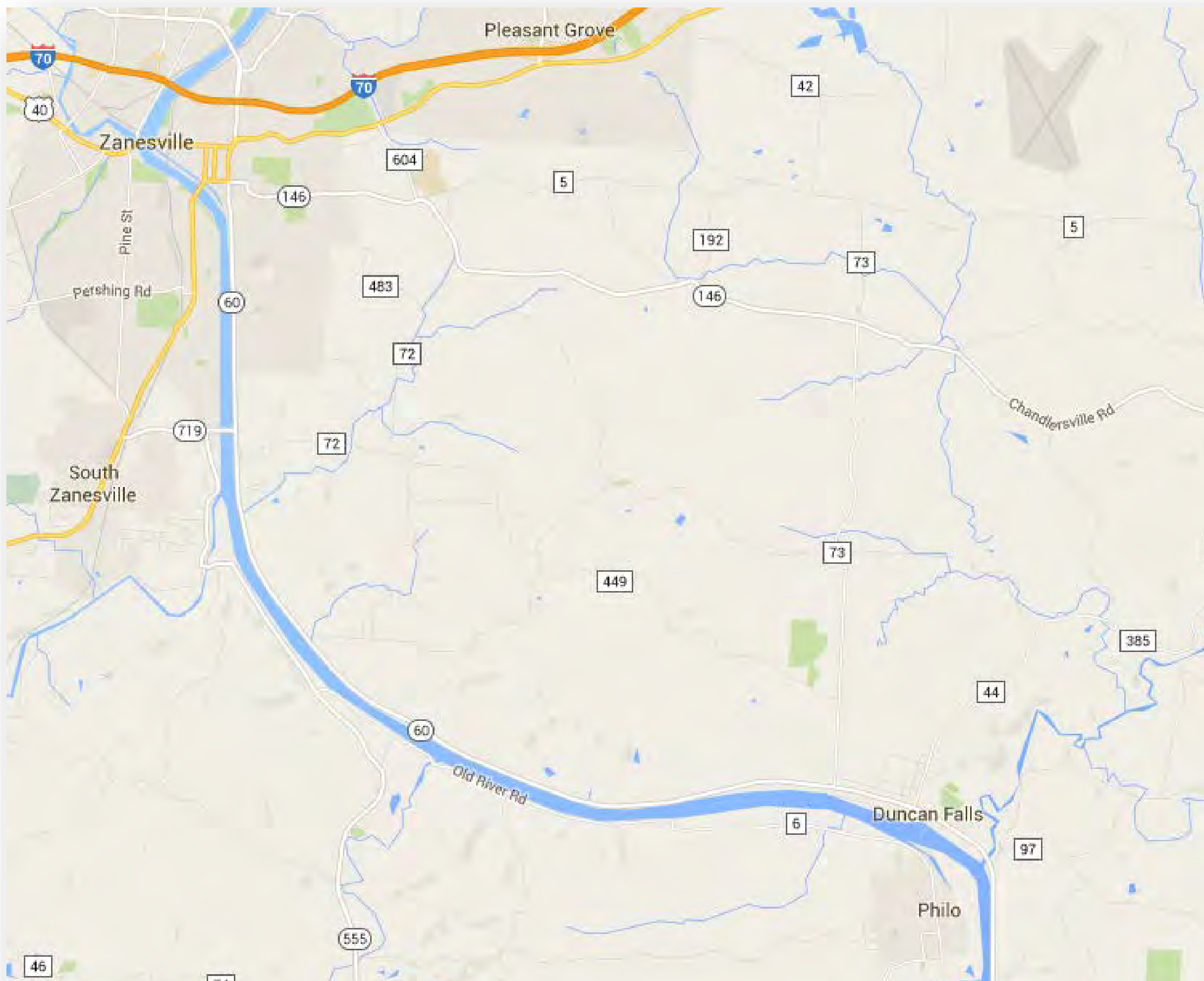

Figure 1 Project Location Map

Pictures of the existing structure are provided in Figure 2, Figure 3, and Figure 4.

Figure 2 Existing Philo Bridge (MUS-CR32-0.00)

Figure 3 Existing Philo Bridge Looking Downstream

Figure 4 Existing Philo Bridge Looking Upstream

The Muskingum River is part of a Federal Emergency Management Agency (FEMA) Special Flood Hazard Area. The existing and proposed bridge replacement are both located within a FEMA Zone AE. This indicates that a detailed study has been performed, and a Base Flood Elevation (BFE) has been established. The effective date of the FEMA Flood Insurance Study (FIS) is July 6, 2010. Figure 5 is a portion of the FEMA Flood Insurance Rate Map (FIRM) at the project location.

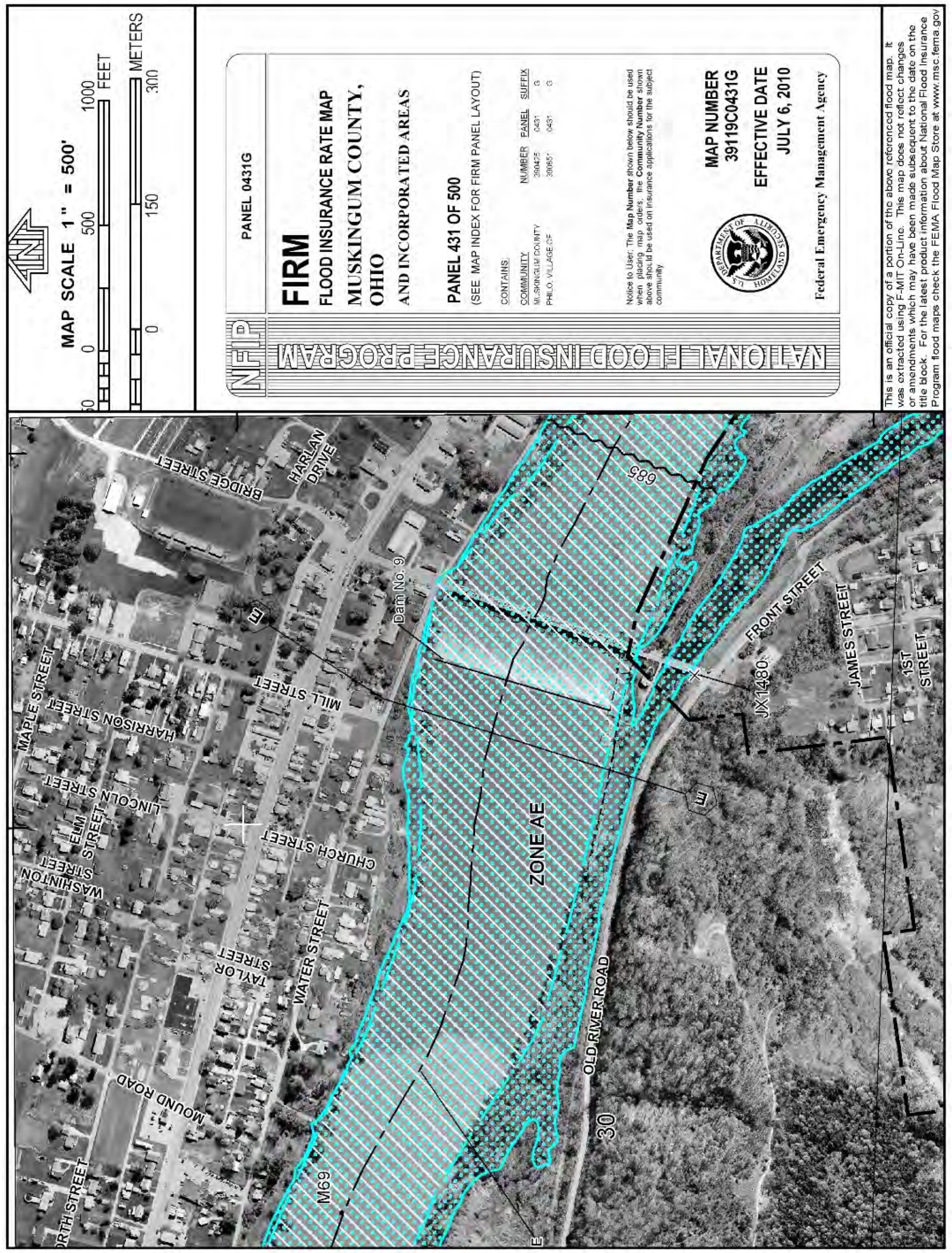

Figure 5 FEMA Flood Insurance Rate Map

Original hydraulic data used in the preparation of the FIS and in the calculation of the BFE were requested from the FEMA Engineering Library. The only available data consisted of a HEC-2 output report file. Contact was also made with Ohio Department of Natural Resources, as well as the Engineering Firm listed on the HEC-2 output report file. Unfortunately, it was not possible to obtain the original HEC-2 model for the Muskingum River.

E.L. Robinson Engineering is scoped with the hydraulic analysis of the both the existing and proposed structure. The following is a documentation of findings related to the hydraulic analysis performed.

II. Hydraulic Analysis

Structure Description

The existing structure is a 6-span steel truss bridge built in 1953. The total structure length is 825 feet. The low chord elevation of the existing structure is 694.79.

The proposed structure is a 7-span prestressed concrete I-beam bridge with an approximate length of 850 feet. The low chord elevation for the proposed structure is 685.43. The low chord elevation for the proposed structure was established to reduce right-of-way impacts along the proposed roadway alignment while maintaining the required navigational clearance established during communication with the United States Coast Guard. This elevation is higher than the existing FEMA FIS reported 100-year water surface elevation of 684.5. The site plan for existing bridge can be found in Figure 6 and Figure 7. The site plan for the proposed bridge can be found in Figure 8.

General Information

The existing bridge is located at approximate river station 211546 and the proposed bridge is located at approximate river station 361586. The direction of river flow is from north to south.

Design Year Recurrence Interval

A 25-year storm will be used as the design frequency based on Section 1004.2 Design Year Frequency of the Ohio Department of Transportation (ODOT) Location and Design Manual, Volume 2 for a design year ADT in excess of 2000.

Contributing Drainage Area

The contributing drainage area at the site (7190 square miles) was determined from the U.S.G.S. Ohio StreamStats application.

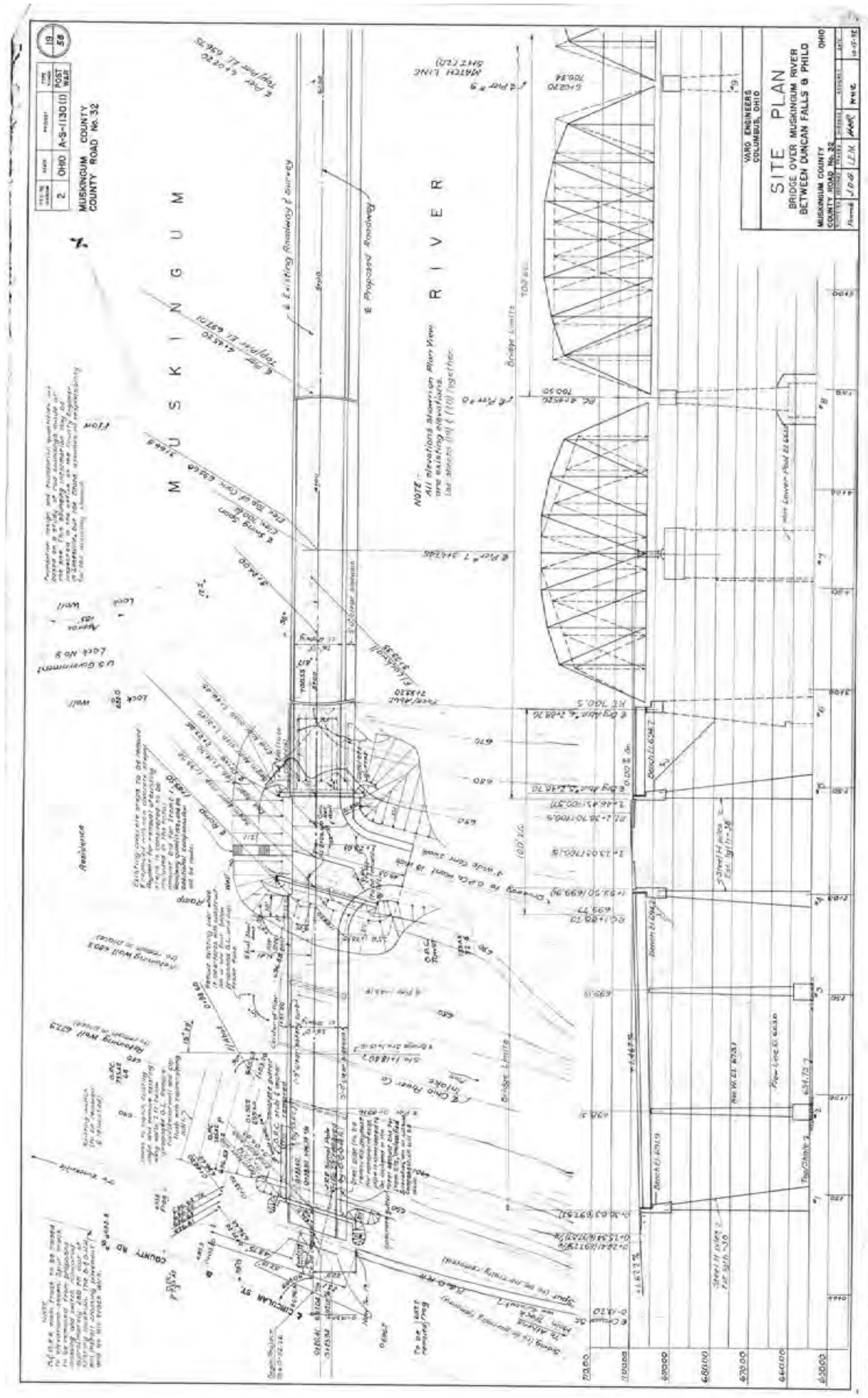

Figure 6 Site Plan for Existing Structure (Sheet 1)

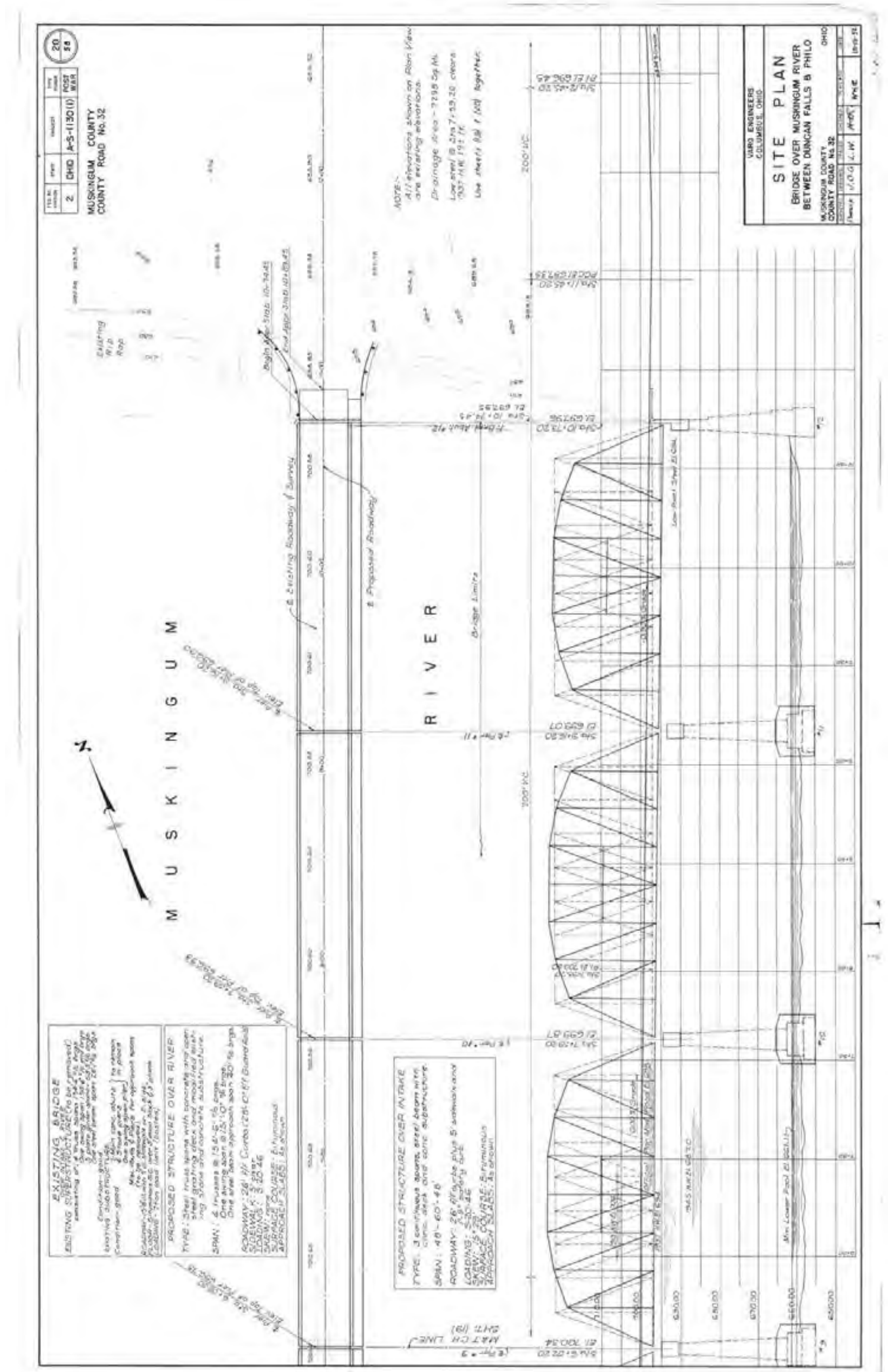

Figure 7 Site Plan for Existing Structure (Sheet 2)

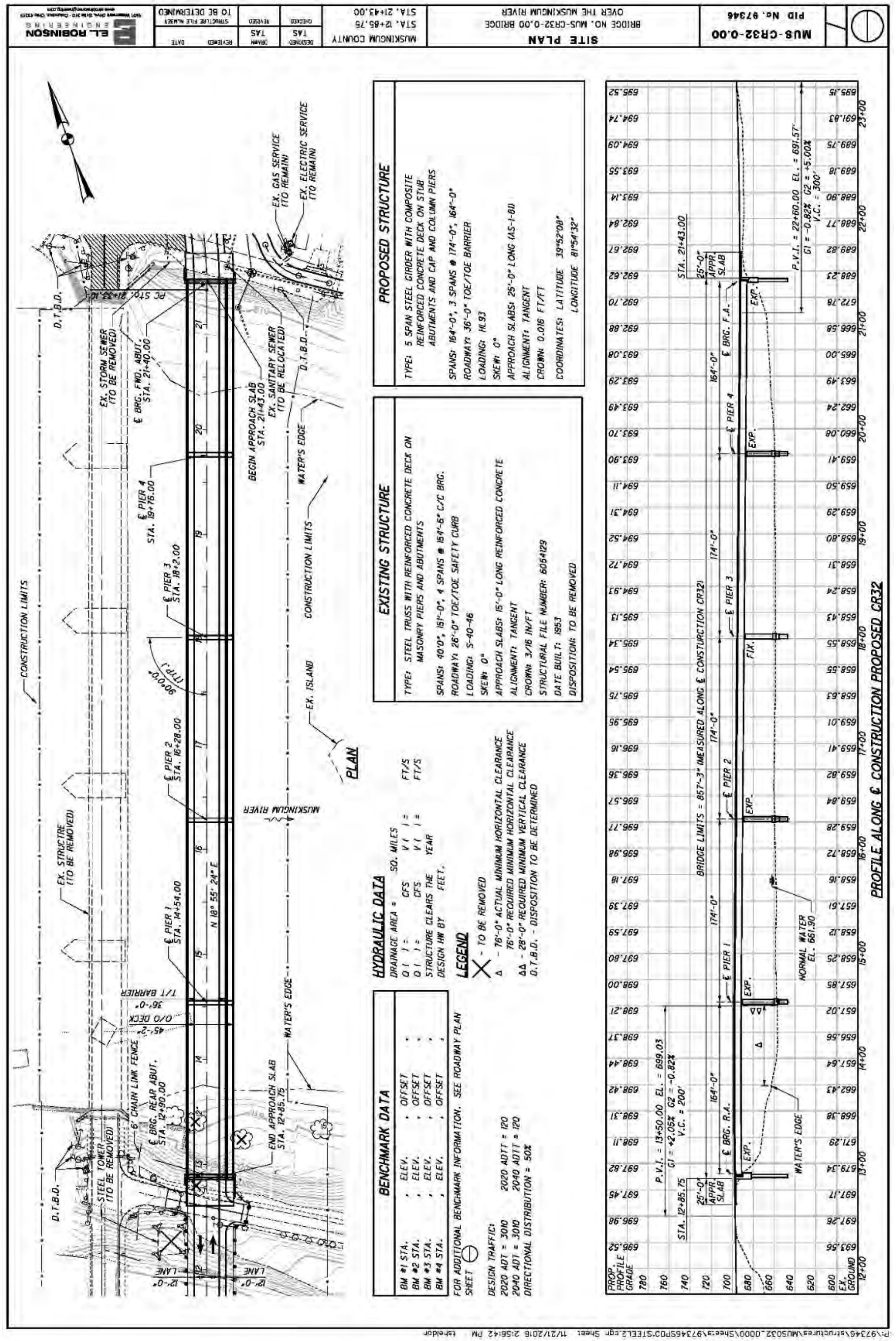

Figure 8 Site Plan for Proposed Replacement Structure

Design and Check Year Discharges

The peak discharges were calculated using the U.S.G.S. Ohio StreamStats application:

Q_{25} = 115,000 cfs Q_{100} = 143,000 cfs

Because the site is part of a Special Flood Hazard Area, the discharges reported in the FIS were used for the hydraulic analysis of the proposed and existing structures for conformance with FEMA floodplain development requirements. The 100-year discharge reported in the FIS is 77,700 cfs at the downstream end of the hydraulic study limits and 74,000 cfs in the upper reach of the study.

It is unusual for USGS methodologies to produce discharge results significantly greater than FEMA reported discharges. The reason for the substantial difference at this site is because the USGS method is for unregulated streams, whereas the Muskingum River is regulated.

There is an existing river gage along the Muskingum River at McConnelsville, just downstream of the hydraulic study limits. The contributing drainage area at this site is 7422 square miles.

A statistical analysis of the yearly peak flow discharges of the Muskingum River at the McConnelsville gage was conducted beginning with 1943. This year was selected to begin data interpretation after the majority of the stream regulation facilities had been constructed while ensuring a large enough data set to be considered statistically valid.

Discharges were determined using both a Log-Normal distribution and a Log-Pearson Type III distribution using a generalized skew.

The results are as follows:

Table 1 Calculated Discharges at McConnelsville.

Return Period	Exceedance Probability	Log-Normal Discharge (cfs)	Log-Pearson Discharge (cfs)
2	0.5	39922	39533
5	0.2	51140	50972
10	0.1	58209	58543
25	0.04	66825	68152
50	0.02	73059	75363
100	0.01	79161	82632

Plots of the predicted and actual values for the Log-Normal and Log-Pearson Type III distributions are provided as Figure 9 and Figure 10, respectively.

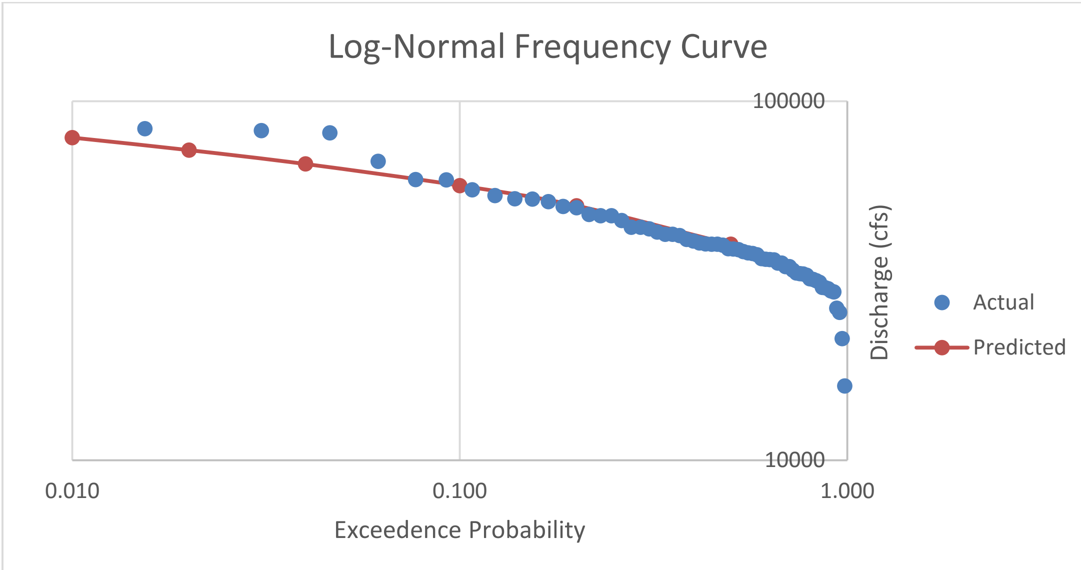

Figure 9 Log-Normal Frequency Curve

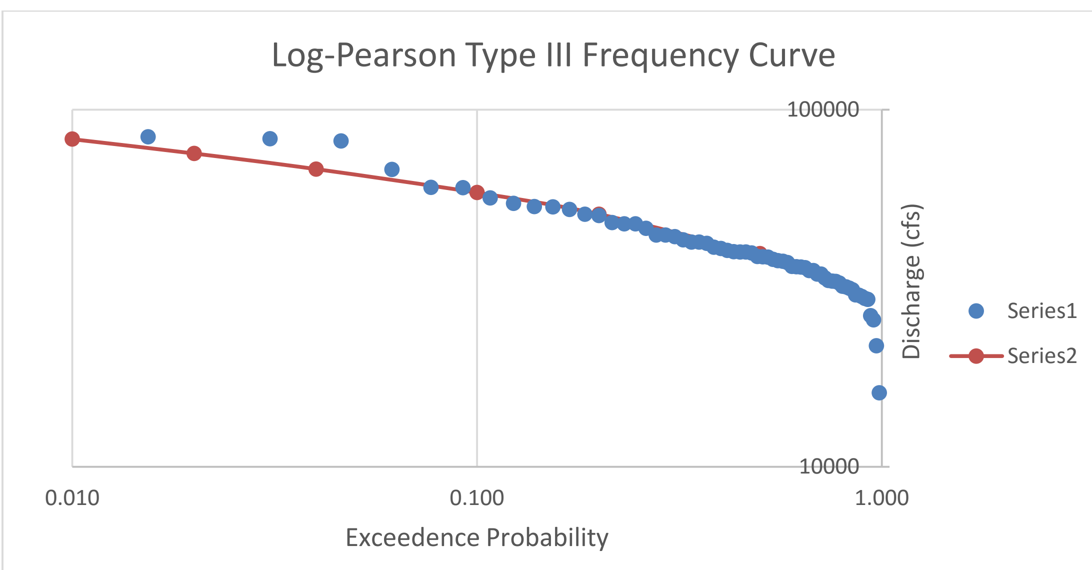

Figure 10 Log-Pearson Type III Frequency Curve

The sum of squares was then calculated using both methods. The Log-Pearson Type II method has a lessor sum of squares indicating a better fit with the actual data. This is as expected since the Log-Pearson method is known to accurately fit peak stream flow data and is the current method recommended by USGS.

Finally, the McConnelsville data was adjusted for the Philo Bridge site by the ratio of each contributing drainage area. The Philo Bridge drainage area is 7190 sq.mi. and the McConnelsville drainage area is 7422 sq. mi. The ratio is 0.968 The final recommended discharges for use at the Philo Bridge site are provided in Table 2 .

Table 2 Calculated Discharges at Philo Bridge.

Return Period	Exceedance Probability	Log-Pearson Results (cfs)
2	0.5	38300
5	0.2	49300
10	0.1	56700
25	0.04	66000
50	0.02	73000
100	0.01	80000

The FEMA FIS reported 100-year discharge at the Philo Bridge site is 74,000 cfs . The difference between the two of less than 10% tends to confirm the methodology utilized in this analysis.

Hydraulic Modeling

The original HEC-2 input file was not available via the FEMA Engineering Library. Therefore, a direct conversion of the HEC-2 data to HEC-RAS Version 5.0.1 was not possible.

Field survey was obtained at the FIS named cross-sections as well as just upstream and downstream of the proposed and existing bridge locations. Using this information, a HEC-RAS Duplicate Effective Model was established. The Duplicate Effective Model covers the Muskingum River from river station 352176 to river station 373484.9. These station limits correspond to the FUS named cross-sections D and F, respectively.

Consistent with FEMA regulations, the Base Flood Elevation (BFE) of the Duplicate Effective Model cannot vary from the published BFE by more than 0.1 foot at any FIS cross-section. The results showed that the calculated 100-year water surface elevation greatly exceeds the published BFE. Thus, the Duplicate Effective Model is not sufficiently accurate for reuse.

An investigation of the FIS stream profiles versus the field survey data shows that the stream bed elevation is significantly higher than when the FIS was developed. This change in streambed elevation is resulting in a higher water surface elevation.

When the Duplicate Effective model cannot be created with the required accuracy, it is necessary to develop a corrected effective model. The Corrected Effective model is identified as “Corrected Effective”. This is the basis for comparison of the proposed and existing water surface elevations.

A Proposed Conditions model was also created. For this model, the existing bridge and associated cross-sections were removed from the model. Then the proposed bridge was added to the model. The Proposed Conditions model is identified as "Proposed".

Because of the limited number of cross sections, HEC-RAS provides informational messages indicating excessive head loss between river sections. In an attempt to stabilize and improve the model, interpolated cross-sections were added to the model. The results were not significantly better than the model without interpolated cross-sections. The use of interpolated cross sections was abandoned.

The final model developed includes the 27' diameter swing pier. This was modeled in HEC-RAS as a pier in the corrected effective (existing bridge) model, but a blocked obstruction in the existing conditions (proposed bridge) model. This is because a pier can only be entered as part of a bridge unit and the existing bridge is being removed. This model is identified as "Proposed w Pier".

The results of the HEC-RAS computations are presented in Table 3. The tabulated water surface elevations are at river station 361706.7. This is the first coincidental cross-section for the Corrected Effective and Proposed Conditions models. The section is located immediately upstream of the existing bridge. It is generally not recommended that water surface elevation be reported within the potential contraction zone of the bridge. In this case the proximity of upstream dam requires the comparison at this location. The tabulated velocities are those at the cross-section immediately downstream of the respective bridge. Table 2 provides a comparison of the water surface elevation at the named FEMA cross-sections. Full HEC-RAS output can be found in Figure 5, Figure 6, and Figure 7. The HEC-RAS data files are also included.

Table 3 Hydraulic Comparison of Existing and Proposed Bridge

Hydraulic Comparison of Existing and Proposed Bridge (361706.7)						
	25-Year Discharge 66,000 CFS		**100-Year Discharge 80,000 CFS**		**FEMA 100-Year Discharge 74,000 CFS**	
Plan	**Water Surface Elevation**	**Velocity (ft/s)**	**Water Surface Elevation**	**Velocity (ft/s)**	**Water Surface Elevation**	**Velocity (ft/s)**
Existing Bridge (Corrected Effective Model)	685.61	3.98	688.42	3.59	685.89	3.57
Proposed Bridge (Proposed Condition Model)	685.60	3.97	688.43	3.59	685.89	3.56
Proposed Bridge with Swing Pier (Proposed Condition Model w/pier)	685.60	4.11	688.42	3.59	685.88	3.69

Table 4 Water Surface Elevation at FEMA Named Cross Sections

Plan	**Cross Section E (362281.5)**	**Cross Section F (373484.9)**
Existing Bridge (Corrected Effective Model)	686.86	691.36
Proposed Bridge (Proposed Condition Model)	686.85	691.35
Proposed Bridge with Swing Pier (Proposed Condition Model w/pier)	686.85	691.36

Table 5 HEC-RAS Output for Corrected Effective Model (Existing Bridge)

HEC-RAS Plan: Corrected Effective River: 346 Reach: MUSKRIVER — Reload Data

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)
MUSKRIVER	373484.9	FEMA 100-YR	74000.00	659.20	691.36		691.77	5.13	15659.52	848.54
MUSKRIVER	373484.9	Q25	66000.00	659.20	690.38		690.73	4.73	14862.32	764.07
MUSKRIVER	373484.9	Q100	80000.00	659.20	693.31		693.72	5.16	17601.69	1097.71
MUSKRIVER	364261.9	FEMA 100-YR	74000.00	659.47	687.55		688.02	5.58	14227.55	685.70
MUSKRIVER	364261.9	Q25	66000.00	659.47	687.07		687.47	5.08	13903.60	679.31
MUSKRIVER	364261.9	Q100	80000.00	659.47	689.91		690.36	5.50	15951.12	779.43
MUSKRIVER	364211.9	FEMA 100-YR	74000.00	659.20	687.55		687.99	5.34	14312.53	707.61
MUSKRIVER	364211.9	Q25	66000.00	659.20	687.07		687.44	4.86	13978.64	695.33
MUSKRIVER	364211.9	Q100	80000.00	659.20	689.91		690.33	5.24	16131.17	846.04
MUSKRIVER	363693.0	FEMA 100-YR	74000.00	659.50	687.32		687.74	5.24	15457.91	928.25
MUSKRIVER	363693.0	Q25	66000.00	659.50	686.88		687.23	4.77	15046.66	926.94
MUSKRIVER	363693.0	Q100	80000.00	659.50	689.73		690.12	5.08	17705.00	936.82
MUSKRIVER	362281.5	FEMA 100-YR	74000.00	661.15	686.86		687.14	4.27	17810.91	938.74
MUSKRIVER	362281.5	Q25	66000.00	661.15	686.49		686.72	3.88	17463.25	925.62
MUSKRIVER	362281.5	Q100	80000.00	661.15	689.37		689.63	4.12	20190.74	957.83
MUSKRIVER	362048.0	FEMA 100-YR	74000.00	658.90	686.84	667.90	687.06	3.75	19774.30	889.38
MUSKRIVER	362048.0	Q25	66000.00	658.90	686.47	667.41	686.65	3.39	19483.56	886.84
MUSKRIVER	362048.0	Q100	80000.00	658.90	689.35	668.25	689.56	3.69	21763.43	914.17
MUSKRIVER	361975		Inl Struct							
MUSKRIVER	361706.7	FEMA 100-YR	74000.00	656.76	685.89	665.75	686.09	3.58	20720.19	900.13
MUSKRIVER	361706.7	Q25	66000.00	656.76	685.61	665.27	685.77	3.23	20495.13	897.38
MUSKRIVER	361706.7	Q100	80000.00	656.76	688.42	666.11	688.61	3.53	22736.11	928.35
MUSKRIVER	361679.5		Bridge							
MUSKRIVER	361651.7	FEMA 100-YR	74000.00	657.12	685.85		686.05	3.57	20762.52	927.00
MUSKRIVER	361651.7	Q25	66000.00	657.12	685.58		685.74	3.22	20536.21	925.37
MUSKRIVER	361651.7	Q100	80000.00	657.12	688.38		688.57	3.51	22840.50	943.96
MUSKRIVER	361474.7	FEMA 100-YR	74000.00	654.90	685.81		686.01	3.66	20563.52	972.07
MUSKRIVER	361474.7	Q25	66000.00	654.90	685.54		685.71	3.30	20334.06	970.46
MUSKRIVER	361474.7	Q100	80000.00	654.90	688.35		688.54	3.59	22817.92	1142.06
MUSKRIVER	352176.0	FEMA 100-YR	77700.00	653.88	682.50	665.50	683.03	5.94	14460.96	885.86
MUSKRIVER	352176.0	Q25	66000.00	653.88	683.13	664.45	683.49	4.89	15012.05	921.04
MUSKRIVER	352176.0	Q100	80000.00	653.88	685.89	665.70	686.29	5.23	17551.79	1147.34

Table 6 HEC-RAS Output for Proposed Model

HEC-RAS Plan: Proposed River: 346 Reach: MUSKRIVER — Reload Data

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)
MUSKRIVER	373484.9	FEMA 100-YR	74000.00	659.20	691.35		691.75	5.12	15266.38	848.11
MUSKRIVER	373484.9	Q25	66000.00	659.20	690.38		690.72	4.74	14669.14	763.71
MUSKRIVER	373484.9	Q100	80000.00	659.20	693.32		693.72	5.16	16476.46	1098.03
MUSKRIVER	364261.9	FEMA 100-YR	74000.00	659.47	687.54		688.02	5.58	14224.03	685.62
MUSKRIVER	364261.9	Q25	66000.00	659.47	687.07		687.46	5.08	13899.79	679.25
MUSKRIVER	364261.9	Q100	80000.00	659.47	689.91		690.37	5.50	15958.45	779.69
MUSKRIVER	364211.9	FEMA 100-YR	74000.00	659.20	687.54		687.98	5.34	14308.82	707.47
MUSKRIVER	364211.9	Q25	66000.00	659.20	687.07		687.43	4.86	13974.74	695.05
MUSKRIVER	364211.9	Q100	80000.00	659.20	689.92		690.34	5.24	16139.13	849.09
MUSKRIVER	363693.0	FEMA 100-YR	74000.00	659.50	687.32		687.74	5.24	15452.81	928.23
MUSKRIVER	363693.0	Q25	66000.00	659.50	686.87		687.22	4.77	15041.28	926.92
MUSKRIVER	363693.0	Q100	80000.00	659.50	689.74		690.13	5.08	17714.09	936.87
MUSKRIVER	362281.5	FEMA 100-YR	74000.00	661.15	686.85		687.14	4.27	17805.29	938.67
MUSKRIVER	362281.5	Q25	66000.00	661.15	686.48		686.71	3.88	17457.49	925.59
MUSKRIVER	362281.5	Q100	80000.00	661.15	689.38		689.64	4.11	20200.51	957.90
MUSKRIVER	362048.0	FEMA 100-YR	74000.00	658.90	686.83	667.90	687.05	3.75	19769.53	889.34
MUSKRIVER	362048.0	Q25	66000.00	658.90	686.46	667.41	686.64	3.39	19478.64	886.80
MUSKRIVER	362048.0	Q100	80000.00	658.90	689.36	668.25	689.57	3.69	21771.56	914.28
MUSKRIVER	361975		Inl Struct							
MUSKRIVER	361706.7	FEMA 100-YR	74000.00	656.76	685.89		686.09	3.58	20715.41	900.07
MUSKRIVER	361706.7	Q25	66000.00	656.76	685.60		685.76	3.23	20490.30	897.32
MUSKRIVER	361706.7	Q100	80000.00	656.76	688.43		688.62	3.53	22744.11	928.47
MUSKRIVER	361651.7	FEMA 100-YR	74000.00	657.12	685.88	666.34	686.08	3.56	20782.71	927.15
MUSKRIVER	361651.7	Q25	66000.00	657.12	685.59	665.86	685.76	3.21	20551.71	925.48
MUSKRIVER	361651.7	Q100	80000.00	657.12	688.42	666.73	688.61	3.51	22872.36	944.19
MUSKRIVER	361586		Bridge							
MUSKRIVER	361474.7	FEMA 100-YR	74000.00	654.90	685.81		686.01	3.66	20563.52	972.07
MUSKRIVER	361474.7	Q25	66000.00	654.90	685.54		685.71	3.30	20334.06	970.46
MUSKRIVER	361474.7	Q100	80000.00	654.90	688.35		688.54	3.59	22817.92	1142.06
MUSKRIVER	352176.0	FEMA 100-YR	77700.00	653.88	682.50	665.50	683.03	5.94	14460.96	885.86
MUSKRIVER	352176.0	Q25	66000.00	653.88	683.13	664.45	683.49	4.89	15012.05	921.04
MUSKRIVER	352176.0	Q100	80000.00	653.88	685.89	665.70	686.29	5.23	17551.79	1147.34

Table 7 HEC-RAS Output for Proposed Model with Swing Pier

HEC-RAS Plan: Prop w Pier River: 346 Reach: MUSKRIVER Reload Data

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)
MUSKRIVER	373484.9	FEMA 100-YR	74000.00	659.20	691.36		691.76	5.13	15654.18	848.24
MUSKRIVER	373484.9	Q25	66000.00	659.20	690.37		690.72	4.73	14855.57	762.31
MUSKRIVER	373484.9	Q100	80000.00	659.20	693.31		693.72	5.16	17601.15	1097.68
MUSKRIVER	364261.9	FEMA 100-YR	74000.00	659.47	687.54		688.01	5.59	14220.27	685.53
MUSKRIVER	364261.9	Q25	66000.00	659.47	687.06		687.45	5.08	13894.15	679.17
MUSKRIVER	364261.9	Q100	80000.00	659.47	689.90		690.36	5.50	15950.55	779.41
MUSKRIVER	364211.9	FEMA 100-YR	74000.00	659.20	687.54		687.98	5.34	14304.98	707.33
MUSKRIVER	364211.9	Q25	66000.00	659.20	687.06		687.42	4.87	13968.93	694.63
MUSKRIVER	364211.9	Q100	80000.00	659.20	689.91		690.33	5.24	16130.54	845.80
MUSKRIVER	363693.0	FEMA 100-YR	74000.00	659.50	687.31		687.73	5.24	15447.54	928.21
MUSKRIVER	363693.0	Q25	66000.00	659.50	686.87		687.21	4.77	15033.25	926.89
MUSKRIVER	363693.0	Q100	80000.00	659.50	689.73		690.12	5.08	17704.38	936.82
MUSKRIVER	362281.5	FEMA 100-YR	74000.00	661.15	686.85		687.13	4.27	17799.56	938.60
MUSKRIVER	362281.5	Q25	66000.00	661.15	686.47		686.70	3.88	17448.91	925.53
MUSKRIVER	362281.5	Q100	80000.00	661.15	689.37		689.63	4.12	20190.05	957.83
MUSKRIVER	362048.0	FEMA 100-YR	74000.00	658.90	686.83	667.90	687.05	3.75	19764.73	889.30
MUSKRIVER	362048.0	Q25	66000.00	658.90	686.46	667.41	686.63	3.39	19471.29	886.73
MUSKRIVER	362048.0	Q100	80000.00	658.90	689.35	668.25	689.56	3.69	21762.89	914.17
MUSKRIVER	361975		Inl Struct							
MUSKRIVER	361706.7	FEMA 100-YR	74000.00	656.76	685.88		686.09	3.71	20013.41	872.98
MUSKRIVER	361706.7	Q25	66000.00	656.76	685.60		685.77	3.34	19797.44	870.25
MUSKRIVER	361706.7	Q100	80000.00	656.76	688.42		688.63	3.65	21974.89	901.38
MUSKRIVER	361651.7	FEMA 100-YR	74000.00	657.12	685.87	666.54	686.08	3.69	20064.35	900.08
MUSKRIVER	361651.7	Q25	66000.00	657.12	685.59	666.02	685.76	3.33	19842.73	898.42
MUSKRIVER	361651.7	Q100	80000.00	657.12	688.41	666.90	688.62	3.63	22086.98	917.13
MUSKRIVER	361586		Bridge							
MUSKRIVER	361474.7	FEMA 100-YR	74000.00	654.90	685.81		686.01	3.66	20563.52	972.07
MUSKRIVER	361474.7	Q25	66000.00	654.90	685.54		685.71	3.30	20334.06	970.46
MUSKRIVER	361474.7	Q100	80000.00	654.90	688.35		688.54	3.59	22817.92	1142.06
MUSKRIVER	352176.0	FEMA 100-YR	77700.00	653.88	682.50	665.50	683.03	5.94	14460.96	885.86
MUSKRIVER	352176.0	Q25	66000.00	653.88	683.13	664.45	683.49	4.89	15012.05	921.04
MUSKRIVER	352176.0	Q100	80000.00	653.88	685.89	665.70	686.29	5.23	17551.79	1147.34

III. Conclusions and Recommendations

We have evaluated the water surface elevation for both the design and check year discharges at the cross-section immediately upstream of the proposed and existing bridge (river station 361706.7) using the statistically determined discharges. By reviewing the results of the hydraulic analyses provided in Table 3 it is seen that there is a slight decrease in the water surface elevation for the design year discharge and a slight increase in the water surface elevation for the check year discharge. However, there is no change in the FEMA Base Flood Elevation using the FIS discharge. The increase in the water surface elevation is 0.01 foot for the statistically determined 25-year discharge. The decrease in the water surface elevation is 0.01 foot for the statistically determined 100-year discharge.

When comparing the FEMA Base Flood Elevation calculated using the FIS discharge for the proposed and existing conditions at the FIS named cross sections in Table 4 it is seen that there is either no increase or a 0.01 foot decrease in the Base Flood Elevation at the named cross sections. This is an indication that the proposed work within the statutory floodway meets the FEMA requirement for no increase in the base flood elevation.

The proposed replacement structure results in a slight decrease in the water surface elevations for the 25-year discharge and a slight increase in the 100-year water surface elevation. Section 1006.3 of the Location & Design Manual requires that the proposed water surface elevation match the existing to the maximum extent practicable and maintain a free water surface for the design year event.

The use of a steel beam bridge could potentially reduce the number of piers and consequently the design and check water surface elevations. However, as part of the Structure Type Study performed for this project, the steel beam bridge was determined to be economically impractical. Therefore, giving consideration to the outcome of the Structure Type Study and the minimal increase in the check year water surface elevations, it is believed that the proposed structure meets the maximum extent practicable provision of Section 1006.3 of the Location & Design Manual.

Flood Hazard Evaluation

Section 1006.3 requires that the impact associated with an increase in the check year water surface be assessed. The proposed 7-span replacement structure results in a maximum increase in the check year discharge of 0.01 foot and a maximum increase in the water surface top width of less than one foot. Additionally, the increase does not impact any occupied structures. These impacts can be considered to be *de minimis impacts.* Therefore, the proposed replacement structure does not result in a significant flood hazard when compared to the existing structure.