BRIDGE ON ROUTE 3 OVER RAPPAHANNOCK RIVER (ROBERT O. NORRIS JR. BRIDGE)

Middlesex and Lancaster Counties, Virginia

CONCEPT STUDY FOR SUPERSTRUCTURE REPLACEMENT

Final Report
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Table of Contents

List of Figures

List of Tables

Executive Summary

The Robert O. Norris Jr. Bridge carries Route 3 over the Rappahannock River between Middlesex and Lancaster counties. As the structure enters its seventh decade of service, the Virginia Department of Transportation has initiated planning for future requirements to address physical and functional deficiencies in the structure. As a part of the planning process, the Department commissioned AECOM to study concept alternatives for replacing the bridge superstructure as a bridge rehabilitation alternative. The superstructure alternatives were contrasted with several total bridge replacement alternatives.

This report summarizes the development of concept alternatives for superstructure replacement and presents the results of a comprehensive evaluation of these alternatives. The scope of this study includes development and evaluation of potential alternative concepts for replacement of the bridge superstructure, based on several criteria and considerations as outlined in Section 2, and considering several structure types as reviewed in Section 3.

The Route 3 Bridge, also known regionally as the Norris Bridge, was constructed in 1957 and carries an average daily traffic of 8,208 vehicles per weekday and 6,326 vehicles per weekend day. The Norris Bridge is 9,985-feet long with a bridge deck width of 23-feet curb-to-curb and 26-feet out-to-out. Its channel span provides 110-feet vertical and 300-feet horizontal clearance for marine navigation.

Preliminary evaluation of the existing approach and channel span piers indicates that the existing piers may be reused with some strengthening and modifications. The condition of the existing piers in the beam and girder spans is unfavorable for supporting a replacement superstructure, so this study also considers the complete replacement of these piers.

The overhead electric utility currently supported by the existing bridge will require temporary relocation during construction of any superstructure replacement. Consideration of supporting the electrical line through under-deck conduits is included in all alternative concepts. Impacts to natural and cultural resources will require coordination with regulatory agencies. The environmental impacts of each superstructure replacement alternative are considered reasonably similar for comparison purposes. Impacts to navigation clearance over the river will require coordination with the U.S. Coast Guard.

Due to the lack of an acceptable detour route, impacts to traffic during construction presents a significant challenge to the project. The scale of a superstructure replacement project and lack of a functional detour prompted consideration of rapid replacement construction methods, of which several alternatives are evaluated as outlined in Section 4. This evaluation concluded that the preferred construction method for rapid replacement includes construction of the new superstructure on temporary foundations located on an alignment offset immediately adjacent to the existing. Once the bridge superstructure construction is complete on the offset alignment, traffic may be moved to the new deck by use of temporary diversion ramps at each end of the bridge. This enables an extended schedule for deconstruction of the old bridge and modification of the existing piers before slide in.

The Department's previous project to replace the superstructure of the U.S. Route 17 Bridge over York River (known as the George P. Coleman Memorial Bridge) in 1996 provides some perspective for rapid replacement options. The Coleman Bridge is a swing span bridge adjacent to the Yorktown National Park, site of the final battle of the American Revolution. With the substructure in good condition, replacement of the Coleman Bridge superstructure with another swing span configuration was chosen to minimize impacts on this adjacent historic resource and maintain access for naval and commercial marine traffic. The project scope was prepared to allow two 12-day road closures. The contractor eventually elected to float out sections of the old bridge and float in sections of the new bridge on construction barges. The new Coleman Bridge spans are configured with the same configuration of joints and span interfaces as the old spans. This configuration permitted effective reuse of the complex barge support towers for both deconstruction of the original bridge and construction of the new superstructure.

In contrast to the Coleman Bridge, the Norris Bridge has no movable spans and it is approximately three times longer. The vehicular traffic on the Norris Bridge is much lower, there is no naval or significant commercial marine traffic on the Rappahannock River, and there are no sensitive historical resources nearby the project site. The Norris Bridge includes spans of varying configuration and elevation, with pinned hangers in most spans, which results in a less efficient construction sequence and precludes cost-effective reuse of the complex barge support towers required for float in operations.

Among a variety of superstructure replacement concepts initially considered, seven superstructure replacement alternatives were developed and evaluated. These alternatives are described in detail in Section 5 of this report. Five of these alternatives were developed to provide a desirable structure width, and two alternatives were developed to provide the minimum structure width required by VDOT Structure and Bridge geometric criteria, in order to minimize project cost. The two most feasible superstructure replacement alternatives are summarized below, followed by a table which summarizes the conceptual cost estimate data for each alternative.

Alternative D1 provides for rapid replacement of the superstructure using the construction methods noted above, to reduce the duration of road closure to a few weeks and minimize the impacts to users. The curb-to-curb width is established as 30 feet in all spans. The beam and girder spans are replaced with prestressed concrete girders on new substructure. The approach and channel spans are replaced with continuous steel deck truss spans, which are fracture critical. This alternative assumes that the navigation channel vertical clearance may be reduced to 75 feet, which requires U.S. Coast Guard approval.

The costs summarized in the table below indicate that the use of rapid replacement construction methods at the Norris Bridge increase the construction costs by a significant proportion. This is exacerbated by the unfavorable subsurface conditions and the high cost of the temporary foundation construction. This cost increase is proportional to the cost premium experienced for the construction methods used for the rapid replacement of the Coleman Bridge.

Alternative F provides replacement of the superstructure using more conventional construction methods, to minimize project cost. Conventional construction requires that the bridge be closed to traffic for the duration of construction of approximately 4 years. The curb-to-curb width is 30 feet in all spans. The beam and girder spans are replaced with prestressed concrete girders on new substructure. The superstructure of the approach and channel spans is replaced with continuous steel plate girders in all spans except the navigation span, where a networked steel tied arch with a post-tensioned concrete tie is employed. Modifications to the existing piers are more extensive for this alternative but the superstructure is not fracture critical. This alternative assumes that the navigation channel vertical clearance may be reduced to 75 feet, which requires U.S. Coast Guard approval. At the time of this report, USCG coordination is ongoing, but if the vertical clearance cannot be reduced, the cost of Alternative F will increase by approximately $2M.

Given the high priority to minimize impacts to traffic during construction, and the high cost of completing a superstructure replacement project with rapid replacement construction methods, it is evident that complete replacement of the bridge on a new alignment should also be evaluated for comparison with the superstructure replacement alternatives. For comparative purposes, this report develops and evaluates several total bridge replacement alternatives. The most cost-effective complete replacement alternative is summarized below.

Alternative 7A provides for construction of a new bridge on a new alignment, approximately 100 to 200-feet upstream from the existing bridge, with a curb-to-curb width of 32-feet. The superstructure type consists of prestressed concrete girders and steel plate girders supporting a concrete deck. This alternative assumes that the navigation channel vertical clearance may be reduced to 75-feet, which requires U.S. Coast Guard approval. At the time of this report, USCG coordination is ongoing, but if the vertical clearance cannot be reduced, the cost of Alternative 7A will increase by approximately $2M. By constructing on a new alignment, the impacts to traffic during construction would be minimal compared with other alternatives.

Conceptual Cost Estimates for Alternatives

Component	**Superstructure Replacement Alternative D1**	**Superstructure Replacement Alternative F**	**Total Replacement Alternative 7A**
Bridge Superstructure	$71	$108	$53
Bridge Substructure	$19	$27	$98
Mobilization & Demolition	$15	$17	$21
Temporary Works for Rapid Replacement	$148	-	-
Contingency	$51	$30	$26
Project Development & Administration	$46	$54	$60
Total Cost (present day $)	**$349M**	**$237M**	**$258M**
Fracture Critical Structure	Yes	No	No
Road Closed to Traffic	15-days	4-years	Not required

In conclusion**, Alternative 7A** for complete bridge replacement on a new upstream alignment results in a longer service life with less maintenance costs than the alternatives that reuse significant portions of the existing substructure with a new replacement superstructure. This alternative is also considered to offer the most optimal balance of costs and user impacts during construction.

1 Introduction

1.1 Scope and Purpose

The Robert O. Norris Jr. Bridge carries Route 3 over the Rappahannock River between Middlesex and Lancaster counties (see Figure 1-1). As the structure enters its seventh decade of service, the Virginia Department of Transportation has initiated planning for future requirements to address physical and functional deficiencies in the structure. As a part of the planning process, the Department commissioned AECOM to study concept alternatives for replacing the bridge superstructure as a bridge rehabilitation alternative. The superstructure alternatives were contrasted with several total bridge replacement alternatives.

The scope of this study includes assessment of criteria and considerations affecting the project, development of potential alternative concepts for superstructure replacement, and evaluation of alternatives based upon these criteria. The preliminary structure type alternatives are developed based on specific geometric requirements, marine navigation clearance, and future service needs. The existing roadway section is functionally obsolete and it is desirable to bring it as close as possible to modern standards. The construction means and methods are investigated to identify the challenges and associated risks with construction. The construction evaluation also considers the minimization of traffic impacts.

The purpose of this report is to summarize the development of concept alternatives for superstructure replacement and present the results of a comprehensive evaluation of the factors affecting feasibility and the costs associated with replacing the superstructure. This Concept Study Report fulfills the requirements of Task Letter of Agreement No. 5 under VDOT Contract No. 43283.

1.2 Overview of Existing Structure

The Route 3 Bridge, also known regionally as the Norris Bridge, was constructed in 1957 and carries vehicular traffic over the Rappahannock River between Middlesex County and Lancaster County (see Figure 1-1). Based on 2017 data, the roadway carries an average daily traffic of 8,208 vehicles per weekday and 6,326 vehicles per weekend day. The Norris Bridge is 9,985-feet long between abutments. The existing bridge deck is 23-feet curb-to-curb and 26-feet out-to-out, with two 11-foot lanes. The Norris Bridge spans a marine navigation channel 110-feet vertical and 300-feet wide. Dominion Virginia Power operates a transmission line that is attached to the east side of the bridge. See Appendix A for selected sheets from the original bridge plans.

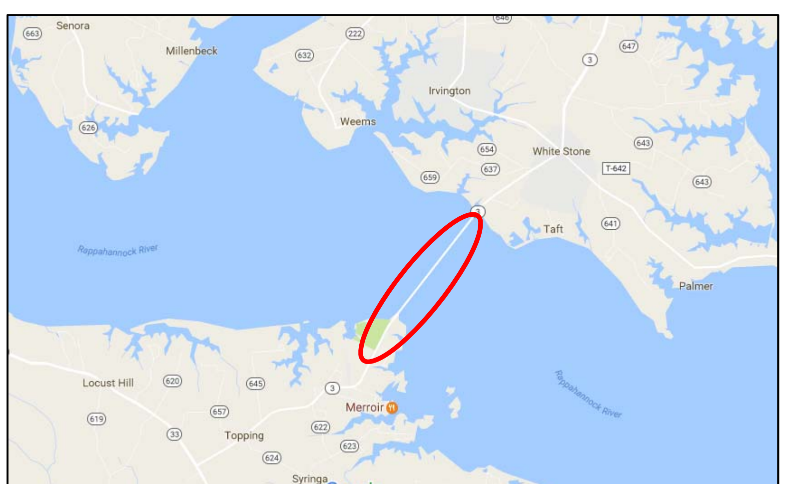


Figure 1-1: Location Map

The bridge consists of 44 spans of mixed superstructure types. The bridge begins with steel multi-beam sections ranging from 70 to 90-feet in length for five spans at the southern end and 12 spans at the northern end. The beam spans are supported on conventionally reinforced precast concrete pile bents.

The beam section units at each end of the bridge transition to steel two-girder sections. The southern section consists of three spans and the northern section is nine spans, all of which are 125-feet long. The girder spans are supported by reinforced concrete column piers founded on timber piles.

The ends of the girder section units are supported by the approach span deck truss units on each side of the channel spans. The southern section consists of seven spans and the northern section is five spans, varying in length from 351 to 468-feet in length. The three-span unit over the navigation channel transitions the deck truss section to a continuous through truss in the navigable channel span. The channel span is 648-feet long. Each of the truss piers consist of reinforced concrete column piers that have a web wall that extends from the bottom of pier to a limit just above the waterline. The piers are founded on sunken caissons.

1.3 Design Objectives

To remain consistent with the scope and purpose of this study, the superstructure replacement alternatives are developed in consideration of the following objectives:

- Completely replace all superstructure members,
- Maximize shoulder width on the replacement superstructure,
- Maximize reuse of the existing substructure elements with repairs and modifications as needed,
- Minimize construction of new foundation elements,
- Minimize duration of road closure.
- Minimize project costs.

In an effort to extend the service life, the development of superstructure replacement alternatives will also consider the following goals:

- *Elimination of Pin and Hanger Connections* – These non-redundant structural details are susceptible to corrosion and introduce additional expansion joints into the deck system. All superstructure replacement alternatives avoid using these connections.
- *Jointless Deck Construction* – All of the steel truss design alternatives, composite, continuous-span construction is proposed to minimize the number of deck expansion joints on the structure as well as eliminate the deflection joints typical of earlier truss construction. Lock-up devices (viscous dampers) can provide a means to more uniformly distribute the dynamic loadings along the longitudinal axis of the bridge that are associated with wind, traffic, and potential seismic events with respect to the response of the substructure units.
- *Deck System* – Standard deck construction details are utilized to the greatest extent possible. This includes the use of low permeability, low shrinkage deck concrete and stainless steel deck reinforcing.
- *Painting* – A duplex galvanized and epoxy paint system mitigates life-cycle painting costs for the structural steel members in this local salt-water environment. A zinc metallized coating is used where larger girder elements are utilized.
- *Bearings* – Low-maintenance, “off-the-shelf” High Load Multi-Rotational low friction bearings are proposed. The configurations of these bearings are consistent with the Department’s standard details.

2 Project Constraints and Criteria

Considerations for superstructure replacement are related to multiple constraints associated with the feasible structural configurations, the desirable roadway section, marine navigation, limiting bridge closures, and future bridge serviceability. The primary goal of this study is to consider preservation of the substructure while replacing the existing superstructure in as timely a fashion as possible given the extraordinarily burdensome detour or alternative crossing options associated with this bridge site. The constraints that affect development of feasible superstructure replacement alternatives are discussed below.

2.1 Project Constraints

2.1.1 Overhead Utility Impacts

In addition to serving as a vital transportation link between the Northern Neck and Middle Peninsula, the Norris Bridge carries a single circuit 115 kV Virginia Electric and Power Company electric line. The transmission line crosses the Rappahannock River supported by seven wooden frame structures extending from each bank before attaching to the Norris Bridge truss spans. In these spans, the line is mounted on the bridge with 14 davit arm style structures as shown in Figure 2-1. Given its high voltage, the line must be de-energized for the majority of bridge maintenance activities.

Figure 2-1: Overhead Electric Utility

The service lines are part of the Eastern Interconnection transmission grid that connects with all other systems in the U.S. and Canada between the Rocky Mountains and the Atlantic coast, where each system is dependent on each other. While there is redundancy within the power grid, its integrity is affected when this line is de-energized. During regular bridge maintenance, the power company can re-energize the line when there are issues with the other redundant lines in the power grid.

The same level of service is expected to be maintained during construction and on the rehabilitated structure. In order to maintain line service during construction, it should be temporarily relocated off of the structure. The lines need to be installed within insulated conduits under the deck of the rehabilitated structure to minimize the frequency of de-energizing during bridge maintenance operations.

In February 2016, the Virginia Electric and Power Company submitted an application to the Virginia State Corporation Commission (SCC) to rebuild their facilities at the Norris Bridge.

The proposed relocation places the line approximately 100-feet east of the bridge, to be supported on new towers independent of the bridge. The SCC evaluation of this proposal is on-going. Therefore, for the purpose of this study, it is assumed that the line will be supported on the rehabilitated structure. The conceptual cost estimates for the various alternatives include the cost of providing conduits as a part of the rehabilitated structure. The cost of temporary relocation and maintenance of the electrical service is assumed to be borne by Dominion Power.

2.1.2 Existing Superstructure Configuration

Developing alternatives for replacement of the superstructure of a long bridge includes consideration of opportunities to divide the work into separate manageable stages. There are several elements of the existing Norris Bridge configuration that complicate the sequential replacement of the superstructure. The bridge is comprised of seven different structure units from south to north: multi-beam, two-girder, deck truss, through truss, deck truss, two-girder, and multi-beam. Both between and within each structure unit are distinctive features that complicate the replacement of the bridge superstructure.

The beam unit is rigidly connected to the two-girder unit at the juncture between the structure units. The two-girder unit is rigidly connected to the deck truss unit at the juncture between these units (see Figure 2-2). The dependency between structure units necessitates that the beam unit be removed prior to the removal of the girder unit or else a temporary support is required to support the shallower members. The same situation is true with the two-girder and deck truss connection.

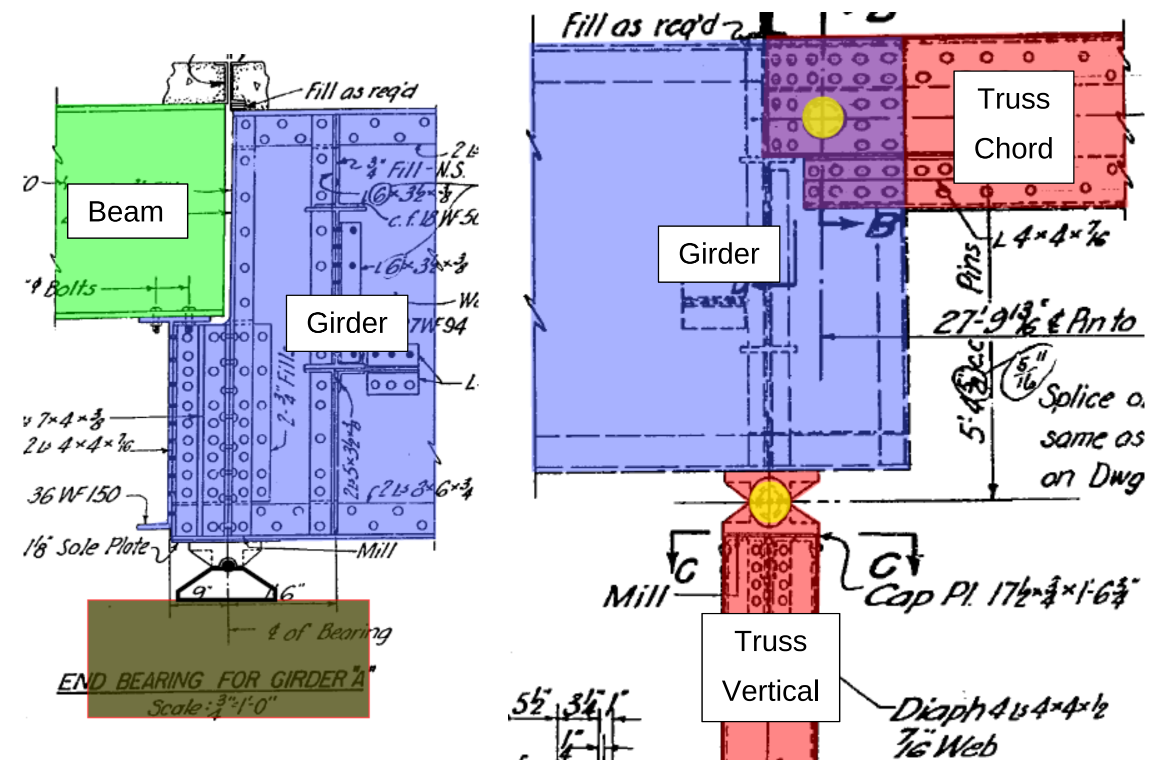


Figure 2-2: Girder Span End Conditions

Additionally, the original structural analysis was simplified by utilizing pin and hanger assemblies in the beam and girder spans and pins in the deck and through truss spans. These hinges are all located away from the bents and piers. This structure configuration consists of an anchor span set on two piers and a short section cantilevered out toward the next pier. The other span(s) attaches underneath the anchor span cantilever and extends to the next pier or past it if there are additional suspended spans. The Norris Bridge, shown in Figure 2-3, exhibits up to four suspended spans extending out from a single anchor span (Spans

9-13). If conventional deconstruction techniques are implemented, Span 9 cannot be removed until Span 10 is removed. Span 10 cannot be removed until Span 11 is removed and this continues until Span 13. The specified demolition arrangement requires extended bridge closures because the contractor cannot work ahead outside of this pattern. This configuration is found in all of the different structure type units within the Norris Bridge.

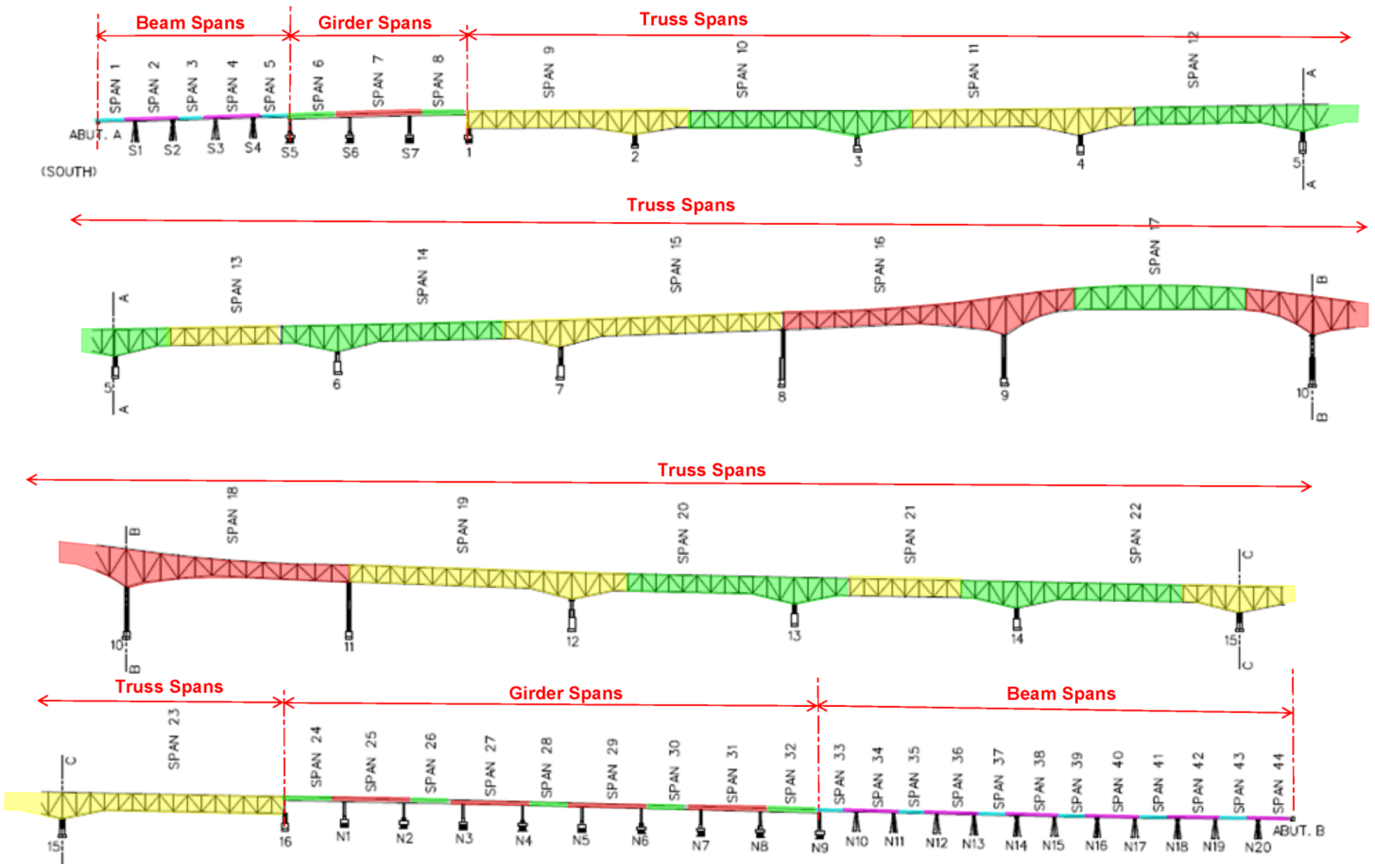


Figure 2-3: Existing Structure Configuration

The feasible superstructure replacement types are also governed by the structure depth over the piers in the approach and channel span truss units. The approximately 55-foot depth at this location restricts the structure type options that can be implemented without substantial pier modifications to raise the top of pier cap elevation. The profile grade of the bridge can be lowered to make additional structure type options feasible. However, this will reduce the navigation channel vertical clearance and require U.S. Coast Guard approval.

2.1.3 Existing Substructure Configuration

In order to mitigate requirements for retrofit of the existing substructure and foundations, the new superstructure requires similar geometry and structural response as the existing structure. This requirement is associated with the location of bearings, as well as application of similar loads from the superstructure. Notably, the existing pier columns have very little primary and confinement reinforcement. In most cases, the primary column reinforcement is less than 0.1 percent of the gross column area, and the confinement reinforcement consists of #5 bars spaced at 12-inches. Additional detail is provided in Section 3. To mitigate the effects of column bending, the proposed structure options limit outboard placement of bearing pad locations. This provides a structure with limited width and without any modifications to the existing columns' heights.

An additional weakness of the existing pier configuration is the lack of reinforcement in the pier caps of the two-column bents (Piers 6 through 13), which is limited to somewhat nominal reinforcement, making these sections susceptible to overstress associated with lateral loads and frame action. Piers 1 through 5 and 14 through 16 of these spans are configured with two individual columns and for any structure option utilizing more than two support lines (as in the existing trusses), additional cap construction and pier retrofit is required.

2.1.4 Existing Substructure Condition

As a prerequisite to any proposed superstructure replacement, an investigation of the adequacy of the existing substructure and foundations is mandated. Before investing in a superstructure replacement, it is prudent to confirm that the existing substructure that will remain can provide a similar life span as the new construction. For purposes of this study, consideration of the substructure units' condition is based on current inspection reports, and no material sampling or testing was performed.

The beam spans are supported on concrete pile bents. The majority of the piles exhibit heavy deterioration, including reinforcement section loss. Approximately one third of the piles have been jacketed, but several require re-jacketing as shown in Figure 2-4. The two-girder spans are supported by multi-column bents founded on timber piles. These piers are heavily deteriorated with areas of 100 percent section loss to the reinforcement. The advanced deterioration in the beam and girder span substructure units and the required on-going maintenance makes them bad candidates for use in a long-term solution.

Figure 2-4: Existing Pile Jacket Deterioration

The approach and channel span units are supported on two-column piers with a partial height pier wall and some with pier caps, all founded on sunken caissons. These bents have isolated moderately sized spalls with exposed reinforcement but are generally in good condition. The structural adequacy of the approach and channel span piers is discussed further in Section 3.2.

2.1.5 Subsurface Characteristics

At the Norris Bridge, water depths range up to 60-feet below MLW (see Figure 2-5). The boring logs from the original plans, included in Appendix A, indicate unfavorable soil conditions throughout the width of the river at the bridge location. Soft river bottom sediments of very low shear strength extend to depths as great as 100-feet below water. While the superstructure replacement concepts are intended to minimize the need for new foundations, all temporary works necessary for construction will require temporary foundation installation. The combination of water depth and poor soil quality will make the cost of all permanent or temporary foundations a major contributor to the overall project cost.

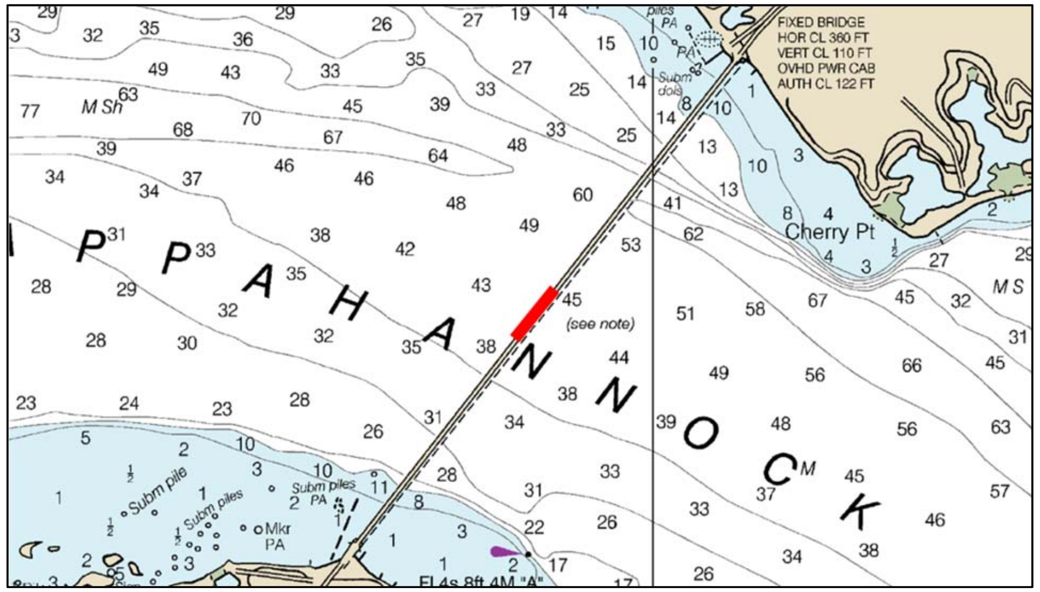


Figure 2-5: NOAA Nautical Chart (navigation span shown in red)

2.1.6 Pier Vessel Protection System

The navigation channel span of the existing Norris Bridge is located at the approximate center of the bridge length, and slightly south of the midpoint between the banks of the Rappahannock River. This span carries the designation Span 17 and is supported at either end by Pier 9 and Pier 10. The existing bridge includes no form of pier vessel protection. The proposed bridge rehabilitation presents the opportunity to evaluate feasibility and effectiveness of installing of pier protection measures.

The AASHTO *Guide Specification for Vessel Collision Design of Highway Bridges* presents guidance related to the design of new bridges and for the evaluation of existing bridges for vessel collision. The provisions of this Guide are intended to prevent collapse of the superstructure by considering the water characteristics, the size and type of vessel fleet navigating the channel, vessel speed, structure response, the risk of collision, and the operational classification of the bridge.

The Rappahannock River channel depth is approximately 40-feet at the navigation channel of the Norris Bridge, where the channel width with full vertical clearance is 300-feet, and the distance between centerlines of piers is 648-feet. The yearly mean channel current of the Rappahannock River has not been determined due to the absence of flow data. Vessels generally transit the bridge through the center of the channel and do so one at a time. The bridge crosses the channel at a small skew of 15 degrees, as there is a slight curve in the river at the crossing.

In the preparation of this report, the size and type of vessel fleet navigating the Rappahannock River in the vicinity of the Norris Bridge was researched. This research includes interviews with craft operators and marina facilities, as well as review of published data from the USACE Waterborne Commerce Statistics Center. The marine traffic of significance to this study corresponds to the Jumbo Open Hopper barge identified in the AASHTO Guide. See Section 2.2.3 on Marine Traffic Impacts for additional information. For risk assessment purposes, the vessels for consideration are classified as inland waterway barges with tow boats and no ship-type vessels identified. The distribution of vessel size using the channel is small and the marine traffic density is considered low.

The Operational Classification of the Norris Bridge is Typical, due to the relatively low traffic volumes, as well as the presence of emergency responders and health services on both sides of the bridge.

Initial review indicates that the channel piers and one pier adjacent to the channel piers in each direction (Piers 8 to 11) will likely be within the primary zone of vessel collision risk, as outlined in the AASHTO Guide for the dimensions of the vessels noted. The vessel design criteria for the existing piers, if any, are likely less than that prescribed by the AASHTO Guide. However, preliminary assessment of the pier type, proportions, and foundations, indicates that the existing piers within the primary zone of risk would have substantial capacity to resist collision forces from the vessels commonly using the waterway.

In consideration of the factors noted above, installation of a fender system at the Norris Bridge to reduce the risk of vessel collision is likely to be associated with an unfavorable comparison of the costs and reduction in risk. For the purpose of this report, vessel collision risk is not a differentiating factor among superstructure replacement alternatives. If superstructure replacement is determined to be the most feasible scope of rehabilitation, a final assessment of vessel collision can be performed based on the AASHTO Guide using Method I or Method II. The superstructure replacement alternatives costs do not include a pier vessel protection system.

2.1.7 Fire Protection System

In 2011, the Commonwealth Transportation Board adopted the National Fire Protection Association (NFPA) 502: Standard for Road Tunnels, Bridges, and Other Limited Access Highways as the fire and life safety standard for the design, construction, and operation of its roadway bridge and tunnel structures. The objective of these provisions is to maintain life safety, mitigate structural damage and prevent progressive structural collapse, and minimize economic impact.

The scope of a project to completely replace the superstructure of the Norris Bridge requires evaluation of the life safety provisions in accordance with the requirements of NFPA 502. This evaluation includes consideration of such elements as signage, emergency communication or closed circuit television systems, traffic control devices, water supply standpipe system, and portable fire extinguishers.

Specific design concerns must be addressed in order to design the fire and life safety system for longevity and maintenance. System components require access for inspection, periodic testing, and maintenance. Where exposed to marine environments with the potential for freezing temperatures in the winter, bridge standpipes are typically designed as dry manual standpipes. Provisions for drainage after use must also be incorporated. For a bridge of this length, flexible couplings are required to accommodate thermal expansion, which may occur at a different rate than the bridge structure, depending on the structure details.

The length of the Norris Bridge, at nearly 10,000-feet, is well in excess of the maximum bridge length of 1,000-feet for waiver of the NFPA 502 provisions. The standard does allow for waiver of these provisions by the Authority Having Jurisdiction (AHJ), which is the Chief Engineer within VDOT. Such a determination is based on engineering evaluation, conducted in coordination with local emergency first responders, with specific consideration of probability and size of potential fire events, local emergency response resources, system effectiveness and reliability, and cost/benefit evaluation.

For the purposes of this study, it is assumed that consideration of the site-specific factors at the Norris Bridge will substantiate the decision to waive the installation of extensive fire and life safety systems in conjunction with a superstructure replacement project. The superstructure replacement alternatives costs do not include a fire protection system.

2.2 Evaluation Criteria

2.2.1 Replacement Superstructure Width

The existing bridge roadway section shown in Figure 2-6 provides a 23'-0" roadway width that allows for no shoulders and two 11'-6" wide lanes. This limited lane and shoulder width categorizes the bridge as functionally obsolete because current design standards require both a wider lane and shoulder. This study investigates ways to make the geometry adequate for current standards.

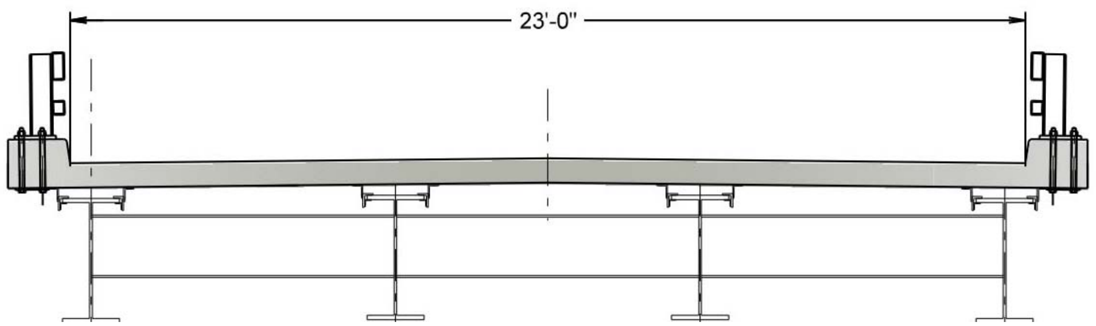


Figure 2-6: Existing Bridge Roadway Width (Beam Span Shown)

The project scope seeks to accomplish bridge widening as part of the superstructure replacement, to the extent feasible without excessive widening of the substructure units. It is assumed that no separate pedestrian or bicycle facilities are required. The project team referenced several standards, as shown in Table 2-1, for consideration in the study. At a minimum, the roadway is slightly upgraded from 11'-6" lanes to 12'-0" lanes, but there still will not be a shoulder provided. The applicable minimum shoulder width per VDOT geometric criteria is 3'-0". Considering the bridge length, it is desirable to provide enough shoulder width to allow for disabled vehicles to pull into the shoulder and not impede traffic. Based on discussions with the project team, it is desirable that the superstructure replacement alternatives include widening that provides 8'-0" shoulders, for a curb-to-curb width of 40'-0"; however consideration is also given to using the minimum required width, in order to minimize project cost.

Table 2-1: Superstructure Replacement Bridge Width Options

Shoulder Width (ft.)	Roadway Width (ft.)	Description/Specification
0	24	Similar to Existing
3	30	VDOT Superstructure Replacement Stds
4	32	AASHTO Green Book
10	44	VDOT New Bridge Standards
8	40	Project Team Desirable

2.2.2 Vehicular Traffic Impacts

The Norris Bridge carries an average daily traffic of 8,208 vehicles per weekday and 6,326 vehicles per weekend day, based on 2017 data. Limited detailed traffic count information suggests that the traffic usage is slightly heavier during the summer, and that the daily variation is steady throughout the daylight hours. The bridge is a significant element of infrastructure for commuters and vacationers and an important part of the local economy.

The only feasible detour route for these vehicles in the event of a bridge closure is through Tappahannock utilizing Routes 17, 360, and 3. This detour route is approximately 80 miles in length, between the ends of the Norris Bridge, with a driving time of approximately 90 minutes.

Vehicular traffic impacts during construction of a superstructure replacement are anticipated to include both single lane closures and complete road closures. Lane closures are commonly utilized for bridge inspections or maintenance activities, and are known to result in lower level of service across the bridge, with backups historically less than ½-mile in length.

In contrast, complete closure of the bridge will result in significant impacts to the traveling public. In the development of superstructure replacement alternatives, the goal of minimizing the need for bridge closure is of high priority.

Several strategies are considered to minimize or mitigate the impacts to traffic include, as noted below. Strategies to minimize the impacts seek to reduce the number and duration of bridge closures. Mitigation strategies are aimed at reducing the severity of the impacts to traffic.

Staging of construction is an approach commonly used on bridge replacement projects to maintain at least one lane of traffic at all times during construction. The fracture critical configuration of the majority of the existing Norris Bridge does not facilitate the partial demolition typically needed for staged construction on the existing alignment. In order to make staged construction feasible, some widening is required off the alignment of the existing bridge, which is not compatible with the purpose and scope of this study. The use of staged construction sequences are not considered further.

Other means to minimize the number and duration of closure involve the selection of the new structure type and the construction methods, such as rapid installation of prefabricated elements. This consideration is reviewed in detail in Section 4 of this report.

Mitigation strategies may include public communication and outreach to inform the traveling public of the expected schedule of closure. This facilitates planning and may encourage users to make alternate plans during the closure period. The timing of the closures may also reduce the impacts, with consideration of local school schedules or traditional vacation seasons. More substantial mitigation measures include alternate transportation facilities. These facilities may include such components as supplemental bus routes, park and ride facilities, or ride share programs. A temporary bridge crossing of the Rappahannock River is considered impractical due to the width of the river, the depth of the channel, and the potential impacts to marine traffic.

One final mitigation strategy considered as a part of this study involves the operation of a temporary ferry service at the bridge site. The existing bridge was originally constructed to replace a ferry that formerly operated between Greys Point and White Stone. The vessel that operated on that route, known as the Virginia, is still in service at the Department's Jamestown-Scotland Ferry facility (see Figure 2-7). Preliminary feasibility analysis indicates that a temporary ferry service may be able to serve 10 to 20 percent of the vehicles using the bridge today. Alternatively, the ferry can be provided for the use of passengers rather than vehicles, although this introduces other challenges for the handling of arriving and departing passengers at each terminal. Even if a vessel is borrowed from one of the Department's other ferry facilities, the cost of this temporary service includes crew personnel, channel modifications, pier and mooring dolphin installation, terminal facilities, utilities, and security. The superstructure replacement alternatives costs presented in this report do not include costs for any of the noted traffic impact mitigation strategies.

Figure 2-7: VDOT Ferry "Virginia"

Further traffic data relating to the origin and destination of the bridge users is required for a more thorough evaluation of the potential mitigation strategies. Section 4 of this report reviews the potential number and duration of bridge closures required for the proposed rapid superstructure replacement alternatives.

2.2.3 Marine Traffic Impacts

As a part of the on-going coordination efforts with the United States Coast Guard (USCG), a draft Navigation Impact Report (NIR) has been prepared. The NIR documents the research conducted regarding the channel characteristics, how it is used, and how it is affected by the project.

The Rappahannock River is a navigable waterway and, as shown in Figure 2-8, the main channel span provides a fixed 110-foot vertical clearance and a 300-foot horizontal clearance. For the purpose of the replacement study, the concept alternatives will provide approximately equal horizontal navigation clearance. The vertical clearance will also be maintained through the alternatives presented with the exception of Alternative D, which reduces vertical clearance to 75-feet. Such a reduction in clearance may be acceptable considering the vessel fleet navigating the waterway, as discussed in the following paragraphs.

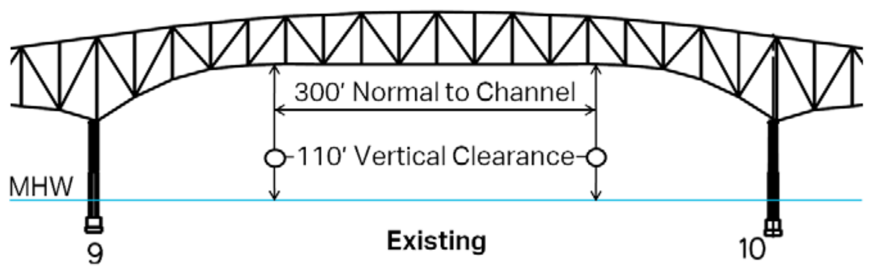


Figure 2-8: USCG Navigation Channel

There is significant recreational activity occurring under the Norris Bridge. There are 18 marina locations on the Rappahannock River reaching from its mouth all the way to Thomas J. Downing Bridge on Route 360, 37.5 miles upstream. These marinas harbor an estimated 900 recreational craft regularly used on the river. The maximum height of these recreational craft is governed by the sailboats at 60-feet above the waterline. The estimated maximum length of the vessels is approximately 100-feet and is governed by yachts. Given the vertical and horizontal clearances of the proposed bridge, the ability of any recreational vessel to navigate the waterway will not be affected.

The Rappahannock River also supports a moderate amount of commercial traffic. While there are no major commercial ports on the river, commerce and fishing vessels regularly transit the channel. The fishing operations consist primarily of oyster fishing while the commerce vessels carry domestic farming products such as animal feed and corn. According to Perdue Agribusiness, they ship farming products through the waterway on a few hundred barges per year. These barges run on a seasonal schedule, navigating the channel between June and February. The largest of these barges are 195-feet by 35-feet. In addition, an Army Corps of Engineers (USACE) study completed in 2014 shows 686 total trips taken by commerce vessels under the Norris Bridge annually. No commerce vessel greater than 300-feet in overall length and 54-feet in overall breadth currently transits the waterway. The vessels governing these dimensions are barges pushed by tugboats which require 52-feet of vertical clearance. It is assumed that no commercial vessels navigating the channel today or in the future will require larger vertical or horizontal clearances. Given the existing vertical and horizontal clearances of the proposed bridge, the ability of any commercial or pleasure vessel to navigate the waterway will not be affected if vertical clearance is reduced to 75-feet.

2.2.4 Environmental Impacts

AECOM completed a preliminary desktop evaluation of potential environmental issues in the vicinity of the Norris Bridge study area, including tidal and non-tidal wetlands, resource protection areas, special habitats (submerged aquatic vegetation and oyster and shellfish beds), floodplains, protected species, hazardous materials and cultural resources. Two noteworthy natural resources will likely be impacted by the superstructure replacement.

The existing bridge site is mostly underlain by public oyster grounds, as shown in Figure 2-9. If impacts to the public oyster grounds cannot be avoided, coordination with the Virginia Marine Resources Commission (VMRC) regarding minimization and compensation is necessary. Additionally, because there are six species of anadromous fish in the Rappahannock River, the Virginia Department of Game and Inland Fisheries recommends a Time of Year Restriction from February 15th to June 30th for instream work.

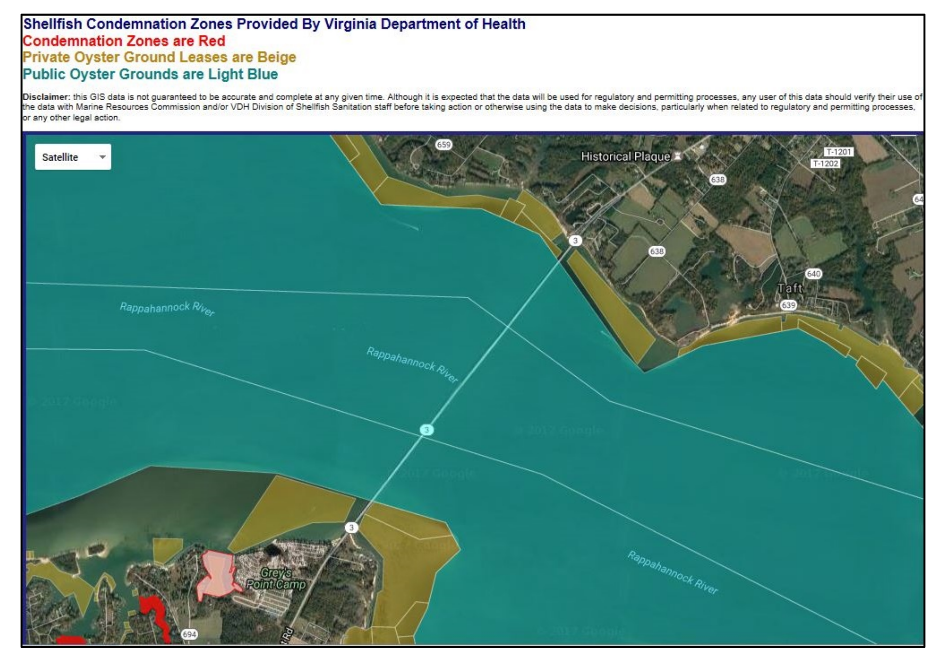


Figure 2-9: VMRC Oyster Map

The identified natural and cultural resources generally require documentation of avoidance, minimization, and potential compensation for unavoidable impacts. However, provided the design team sufficiently describes the purpose, need, and justification for the bridge crossing alternatives evaluated, this preliminary evaluation did not identify any major critical resources that cannot be managed through early and frequent regulatory communications and standard permitting processes. Therefore, pre-permitting coordination is suggested prior to submittal of the Joint Permit Application to expedite permitting with the U.S. Army Corps of Engineers, the Virginia Department of Environmental Quality, the VMRC, and local wetland boards. Avoidance, minimization and compensation for unavoidable impacts should be documented throughout the design alternative evaluation process.

3 Superstructure Types Evaluated

3.1 Approach and Channel Span Criteria

The existing truss spans are configured of three continuous-span cantilevered units approximately 7,090-feet in total length. The specific unit lengths follow, consisting of:

- 7 spans (469-foot maximum) with a total length of 3,161-feet (South Approach Unit)
- 3 spans (648-foot maximum) with a total length of 1,587-feet (Channel Span Unit)
- 5 spans (469-foot maximum) with a total length of 2,342-feet (North Approach Unit)

The approach units are deck trusses haunched at the intermediate supports. The channel unit is comprised of a combination deck truss that transitions to a through configuration in the main channel span. Each truss unit carries a 23'-0" roadway, 1'-6" curbs, and bridge rails. Typically, the trusses are configured with Warren bracing, suspended-span hangers at the intermediate hinges, and built-up riveted sections.

The replacement superstructure must be geometrically and structurally compatible to the existing substructure. By mimicking the location of the bearings and the superstructure loads, the need to retrofit the existing substructure and foundations is diminished.

The proposed roadway sections initially considered for the preliminary superstructure replacement alternatives are:

- Two 12-foot lanes with 10-inch barriers and 25'-8" deck width,
- Two 12-foot lanes with two 8-foot shoulders and 10-inch barriers and 41'-8" deck width,
- A combination of the two above sections with a widened (41'-8" wide deck) in the approach span units and a narrow (25'-8" wide deck) in the channel span unit.

The existing structure provides a marine navigation channel under the main span. The vertical and horizontal clearances are 110-feet and 300-feet (normal to channel) respectively. The channel is skewed 15 degrees with respect to a line normal to the bridge longitudinal centerline.

To accommodate the rehabilitation, the allowed periods of bridge closure are extremely limited given the extraordinary detour and traffic requirements particular to this site. Bridge erection methods are developed to mitigate the need for extended bridge closures and are discussed in detail in Section 4.

3.2 Approach and Channel Span Replacement Alternatives

Several structure configurations are initially identified as suitable for the span and roadway section requirements.

- Continuous-Span Steel Truss
- Steel Delta Girder
- Steel Box Girders with Rocker Bents
- Steel I-Girders with Modified Piers
- Continuous-span Steel Arches
- Concrete Girders

Each of these alternatives also represents a structure that meets the needs associated with a long, low-maintenance service life. However, given the limitations associated with modifying the piers to accommodate the shallower girder sections and the extensive closure time required, the steel I-girders are removed from preliminary consideration. Similarly, the concrete girders represented additional weight that the piers cannot accommodate. The piers can be modified with rocker bents, but the significant construction time to install these elements requires an intolerable closure length. Continuous-span steel arches will apply differential thrust components to the piers that will require reconstruction of each pier. Thus, to best emulate the structural behavior and geometry required to mitigate modification of the piers and have the least detrimental effect upon the pier loading and time of construction, two alternative structural configurations are initially considered for further study for replacement of the existing truss spans: **Continuous-Span Steel Truss** and **Steel Delta Girder.** These two alternatives meet the requirements for a low-maintenance service life; however, depending upon the constraints placed upon pier construction and/or the required marine navigation clearance, some compromises may be realized in the allowable roadway section width.

3.2.1 Continuous-Span Steel Truss

The configuration chosen for this option is a structure with Warren bracing and no intermediate vertical truss members. The deck truss spans are also configured with the supporting floor system and truss top chord members in the same plane such that the deck can be constructed to be composite with the supporting superstructure. This type of structure has been demonstrated to be significantly more efficient with respect to required structural steel weight. Recent applications include the Lehigh River Bridge, (see Figure 3-1) in Bethlehem, Pennsylvania and the Route 60 crossing of the Tennessee River in Paducah, Kentucky (see Figure 3-2). Using this configuration, there is a savings of approximately 20 to 25 percent of steel (even for a widened section) with respect to the original structure. Other major items associated with this efficiency are the elimination of sway bracing and many of the portal frames within the truss framework.

Figure 3-1: Lehigh River Bridge

Figure 3-2: U.S. 60 Tennessee River Bridge

Additional advantages of this particular configuration are:

- Jointless deck construction throughout each truss unit,
- "Modular" / repetitive detailing of truss joints for simplified fabrication,
- Bearing locations compatible with existing piers.

Disadvantages of this construction include:

- Modification of Piers 8-11 is required to allow widening of a thru-section in the channel span unit,
- Supplemental sections for load redistribution are required if a redundant design is specified (some truss members are considered non-redundant).

Notably, for this option, if the requirement for the marine navigation channel vertical clearance is reduced to 75-feet or less (allowing local marine traffic to continue unimpeded), a widened deck truss section may also be utilized in the channel span unit.

3.2.2 Steel Delta Girder

This option allows for the economy associated with a conventional steel girder bridge. The slant legs allow the girders to bear at the same elevations on the piers as did the deck trusses. Therefore, no pier height modifications are needed and lateral loads are applied through the bearings at the existing elevations as well. We anticipate using three girder lines, with the outer girders bearing at the same locations as the existing deck trusses, and the center girders supported on new bearing seats installed along the centerline axis. Having three girder lines provides a desirable combination of fabrication economy, erection stability, and load path redundancy. The use of this delta-configuration has been demonstrated on recent major projects, including the Cleveland, Ohio I-90 Innerbelt Project (see Figure 3-3). As with any conventional girder design, jointless deck construction is used for each structural unit.

Notable disadvantages of this configuration are associated with the longer channel span where the structure will need to transition to a more robust configuration given the length of the channel span. As a way to continue structural redundancy through this span, a networked steel tied-arch with a post-tensioned concrete tie and end elements are considered. A similar type of structure transition between a delta-girder and a suspended arch span has been demonstrated in the recently constructed Lake Champlain Bridge between Vermont and New York (see Figure 3-4).

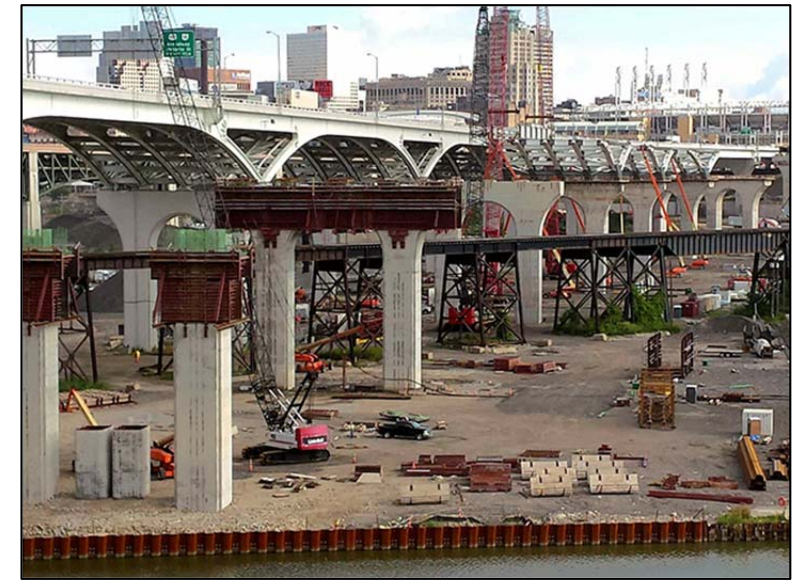


Figure 3-3: I-90 Innerbelt Bridge

Figure 3-4: Lake Champlain Bridge

Additional pier retrofitting is required to accommodate the middle girder of the three girder design. As with the truss option, to allow for a widened section in the channel span, modification of the adjacent piers is required.

3.3 Evaluation of Existing Approach and Channel Span Piers

The approach and channel spans are supported on Piers 1 through 16. For this study, loads are compiled for a superstructure widening option and applied to a simple *CSiBridge* model of Pier 7. Pier 7 governs the pier analysis because of its height and relative span lengths.

The twin octagonal columns are individually modeled with body constraints to the pier section below and connected across the top with a representative cross beam. *CSiBridge* developed the various load combinations for Strength I, III, IV, and V with any redundant combinations removed, resulting in ten Strength combinations. Service load combinations do not need checking because the section reinforcing percentages are approximately 0.08 percent and will not meet serviceability requirements. Rehabilitation by carbon fiber wrap is proposed for all of the pier columns to provide additional structural confinement, as well as corrosion protection.

Results from each different section (pier base above the caisson, section between the base and the twin octagonal columns, the base of the octagonal columns, top of the octagonal columns, and the ends of the cross beam) are placed into *SPcolumn*. All column sections passed the Strength combinations. The cross beam is severely under-reinforced for cross loadings (Strength III, V). Additionally, the analysis showed that the cross beam does not behave as a structural beam but as a strut. The section can be easily retrofitted with additional drilled and bonded bars into the octagonal columns and wider concrete section. The required substructure retrofits for each superstructure replacement alternative are summarized and depicted in Drawing 5-11 of Appendix B.

The caisson foundation is checked using *FB-MultiPier*. The soil strata are simplified to four layers based on the original soil boring logs. Parameters for *FB-MultiPier* are supplied to develop the correct input for the model. The caisson model is simplified to be a hollow section to approximate the existing caisson conditions. The caisson lateral response results indicate that the caisson is adequate.

3.4 Beam and Girder Span Replacement Options

The existing beam span unit superstructure consists of four steel beams in a continuous cantilever configuration, with pin and hanger connections located within the spans. Existing span lengths range from 70 to 90-feet. The total length of beam span units is approximately 400-feet at the south approach and 1,000-feet at the north approach.

The existing girder span unit superstructure consists of two steel edge girders supporting a floor beam and stringer floor system. The spans are nominally 125-feet in length and are configured in a continuous cantilever arrangement with pin and hanger connections located within the spans. The total length of girder span units is approximately 375-feet at the south approach and 1,125-feet at the north approach.

The substructure under the existing beam span units consists of concrete caps supported by precast concrete piles. Approximately 44 percent of the piles have been repaired by installation of pile jackets over the course of several previous repair projects, with some of the piles repaired multiple times. The condition of the piles, as noted in recent inspection reports, indicates that further repairs are required in the near future. Reuse of these pile bents to support a new superstructure will require substantial rehabilitation to provide the appropriate durability and service life. As a part of this concept study, complete replacement of these substructure units will be considered.

The substructure under the existing girder span units consists of concrete two-column piers with struts at the top and above high tide. These piers are supported on timber piles. Pier height from beam seat to footing ranges from 34 to 70-feet, with as much as 18-feet of the column height below water. Recent inspections indicate that the piers are in fair condition, noting localized areas of cracking, delamination, and exposed reinforcing steel with isolated areas of section loss. Reuse of these pile bents to support a new superstructure will require localized repairs of the pier elements.

See Section 2.1.2 for additional information regarding the superstructure configuration in these units. See Section 3.3 for further discussion regarding the serviceability and structural capacity of the existing piers.

Based on the configuration and condition of the existing beam and girders span units, as noted above, the following options are considered for superstructure replacement:

- Option 1 - Match existing deck width and re-use all existing substructure elements,
- Option 2 - Widen deck width and re-use all existing substructure elements,
- Option 3 - Widen deck width and replace beam/girder substructure units.

Various construction alternatives for these options are discussed in Section 4.

3.4.1 Option 1 – Similar to Existing

This option is most closely aligned to the initial objective of the superstructure replacement concept study by replacing the superstructure and reusing the existing substructure without significant widening. The proposed deck width is 24-feet curb-to-curb, similar to the existing deck width. This option serves as a baseline for comparison to other options.

Superstructure types are similar to the existing structure with beam superstructure on the existing pile bents and a girder superstructure on the existing two-column concrete piers (see Figures 3-5 and 3-6). Beam and girder elements are continuous at pier locations. This proposed configuration incorporates deck joints at the ends of the units, at the transitions of superstructure types (beam spans to girder spans), and at intermediate piers as required.

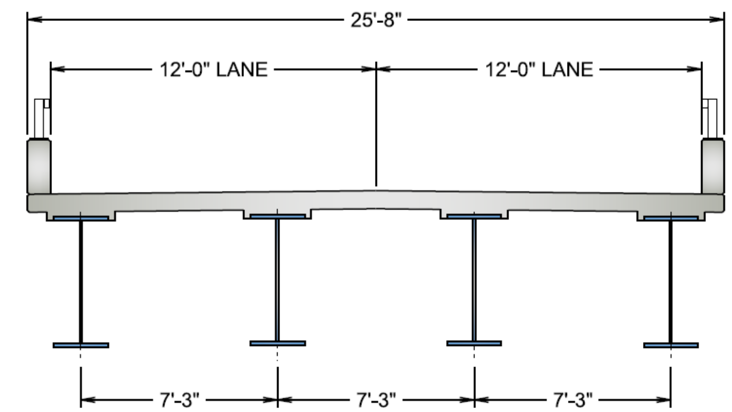


Figure 3-5: Beam Span Section – Option 1

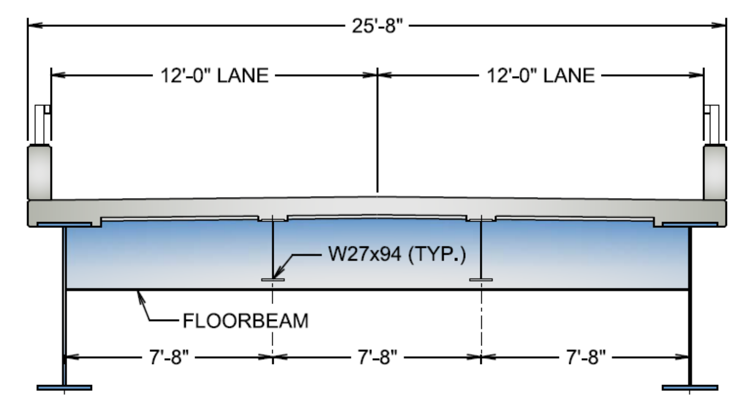


Figure 3-6: Girder Span Section – Option 1

This option results in the least amount of substructure modification to accommodate the new superstructure, though strengthening and repairs are required for certain elements of the existing substructure. This option likely results in the lowest overall cost. The proposed structure type incorporates fracture critical elements and is not well suited to construction in multiple stages and significant closures are required

3.4.2 Option 2 – 40-foot Roadway on Steel Beams and Repaired Piers

This option is similar to Option 1 in replacing the superstructure and reusing the existing substructure, except that Option 2 incorporates the desirable widening discussed in Section 2.2, using a proposed deck width of 40-feet curb-to-curb.

To minimize modifications to the existing piers, consideration is given to a two-girder system with cantilevered floor beams to accommodate the widened deck section throughout both the beam and girder span units (see Figure 3-7). Preliminary evaluations determined that a beam section (see Figure 3-8) is more cost effective and eliminates the fracture critical details associated with the girder configuration. Steel beam elements are selected for the control over structure depth, in order to minimize impacts to seat elevations and changes to profile grade. Steel girders also more easily facilitate the special consideration required for the transition of these spans to the adjacent longer approach spans. This proposed configuration incorporates deck joints at the ends of the units in both the south and north approaches and at two intermediate piers in the north approach.

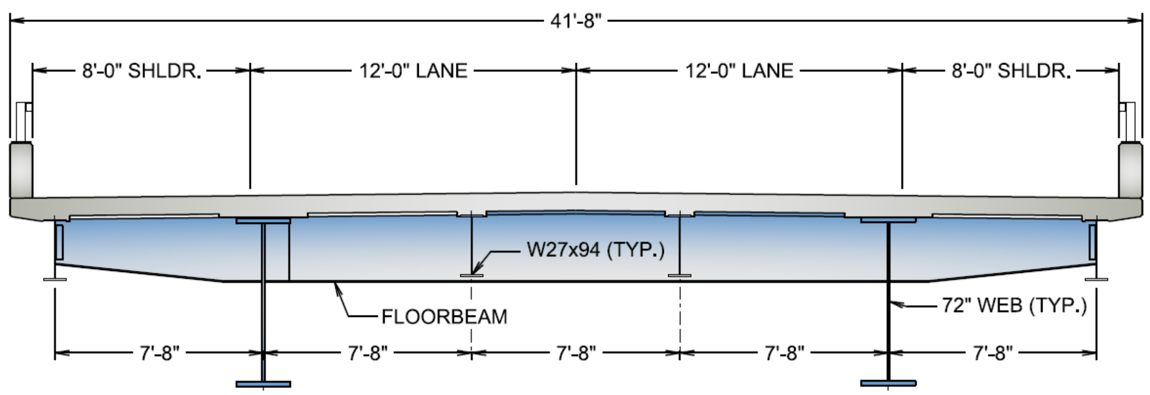


Figure 3-7: Two-Girder Configuration – Option 2

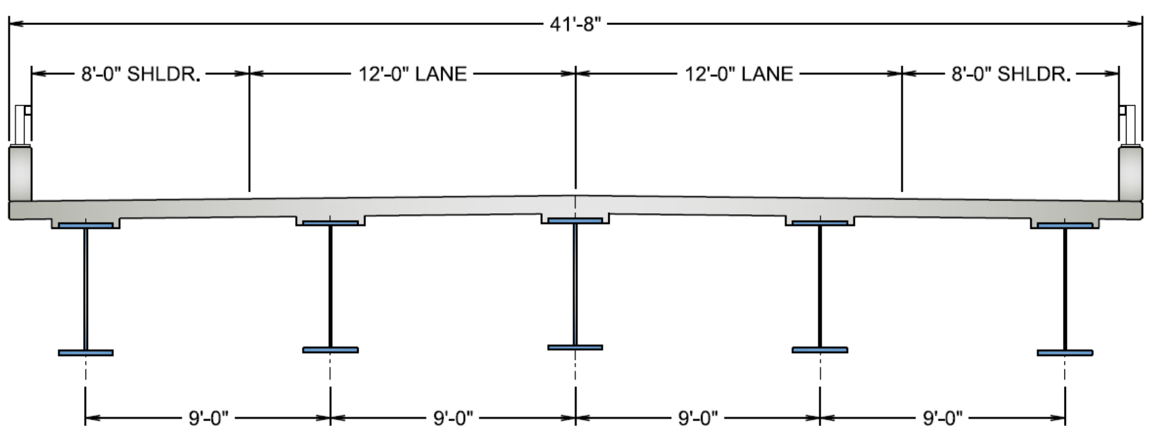


Figure 3-8: Multi-Beam Configuration – Option 2

The multi-beam option provides the desired widening while minimizing substructure modifications, though some new foundation construction is required to accommodate superstructure widening. Strengthening and repairs are required for certain elements of the existing substructure. The proposed structure type does not incorporate fracture critical elements. Construction of this alternative in stages is complicated by the necessary substructure modifications. After construction, the multi-beam option would allow for phased redecking.

3.4.3 Option 3 – 40-foot Roadway on Prestressed Concrete Girders and New Piers

This option is similar to Option 2 in replacing the superstructure, including the desirable widening discussed in Section 2.2, using a proposed deck width of 40-feet curb-to-curb.

As described for Option 2, this proposed widening of the superstructure results in the need for significant modifications to the substructure to accommodate the new superstructure elements while reusing the existing substructure. Based on the extent of new substructure construction, as well as the condition of the existing substructure noted in earlier, complete replacement of the pile bent foundations is considered for this option. This consideration is extended to replacement of the girder span piers as well, to allow for the use of a uniform superstructure throughout the existing beam and girder units. For the benefit of long-term durability and reduced future maintenance costs associated with the pier elements, this option is recommended to include construction of new piers within the limits of the existing beam and girder units. Figure 3-9 shows a potential span arrangement in the south approach that utilizes a nominal maximum span length of 130-feet, with new pier locations selected to avoid the existing pier locations. The span replacement concept for the north approach is similar, though the length of the unit requires additional deck expansion joints.

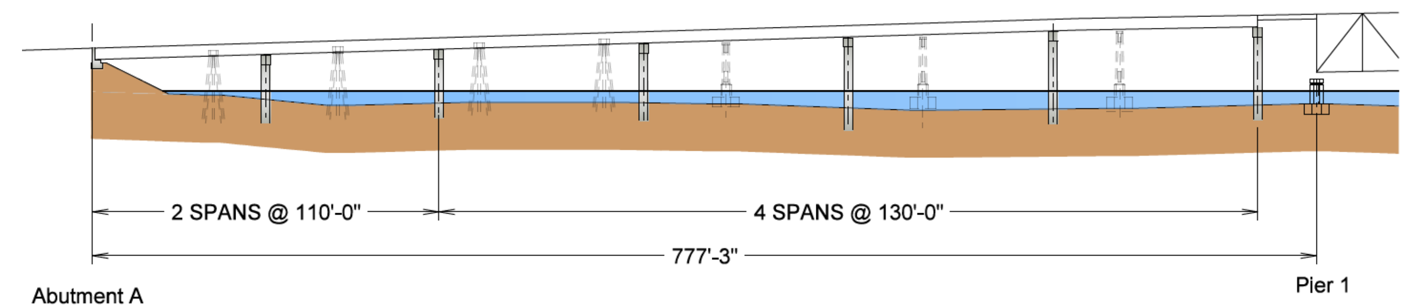


Figure 3-9: Potential New Span Arrangement (south approach shown)

Preliminary evaluation of this option indicates that a multi-beam type of superstructure is the most cost effective. Both concrete and steel girder elements are considered in the development of this option, and concrete girders are recommended for both cost effectiveness and durability. See Figure 3-10 for the cross section for this option. This proposed configuration incorporates deck joints and Virginia Pier Caps at the ends of the units, as well as at two intermediate piers in the north approach. Special consideration is required for the transition of these spans to the adjacent longer approach spans.

The new pier adjacent to Pier 1 is provided so that a 35-foot intermediate span can be constructed between the beam and girder span units and approach span units. The prestressed concrete girders in the beam and girder span units require a pier cap to support them near Pier 1. Regardless of the approach span unit superstructure type that is selected, there is a large difference between its required top of pier elevation and the beam seat elevation for the prestressed girders. The intermediate span will be a continuation of the approach span unit floor system members. This allows for a cleaner transition between the structure types and eliminates the need for a pier that can accommodate dramatically different beam seat elevations.

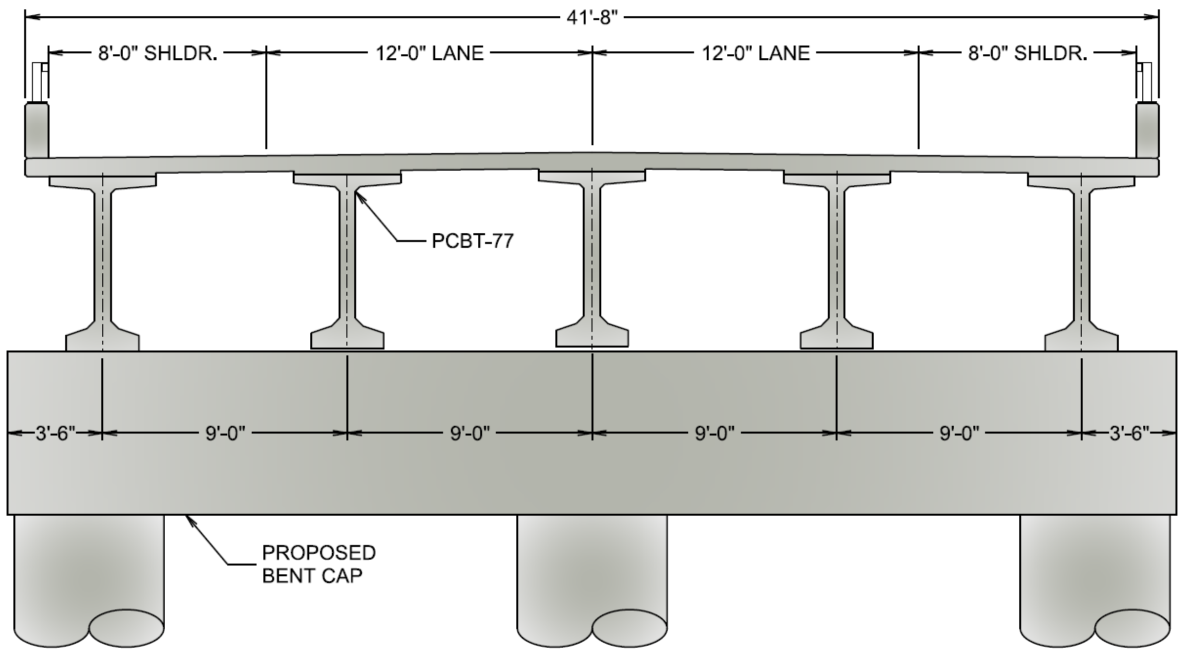


Figure 3-10: Beam and Girder Span Section – Option 3

The substructure for this option consists of a concrete cap supported by driven concrete cylinder piles. This option provides a more durable structure, with the lowest long-term maintenance cost when compared to reusing the existing substructure. Construction of this option in stages is considered most feasible.

4 Construction Methods

This section summarizes various construction methods evaluated for rapid replacement of the entire superstructure on the existing horizontal alignment, and utilizing the existing piers to the maximum extent possible. As noted earlier, the lengthy detour available to bridge users means that any closure of the bridge for construction activities will result in significant impacts to the traveling public. For this reason, constructability and the bridge closure duration are important considerations in evaluating all superstructure replacement alternatives. For all the methods described here, it is assumed that the existing overhead electric utility is temporarily relocated onto temporary works off of the structure during construction. In order to minimize the duration of bridge closure, the construction method should:

- Utilize components prefabricated off-line before installation in the field,
- Quickly remove the existing superstructure once traffic outage is triggered or deconstruct the existing superstructure in place without outage after shifting traffic to a temporarily offset alignment,
- Place new components quickly during traffic outage period.

To achieve these goals, the construction methods considered are:

- Construction Method 1: Float Out Existing Spans / Float In New Spans
- Construction Method 2: Deconstruct Existing Spans In Place / Slide In New Spans
- Construction Method 3: Slide Out Existing Spans / Slide in New Spans

The table below summarizes each construction methods' features, which are presented in detail in the sections that follow. For simplicity, these methods are described in relation to construction of a truss-type structure. The principles apply similarly to any of the superstructure types noted in this report.

Table 4-1: Summary of Construction Methods

	Method 1	Method 2	Method 3
Existing Bridge Removal	Float out during traffic outage	Deconstruct in place after moving traffic to new bridge on temporary offset alignment	Slide out during traffic outage
New Bridge Placement	Float in during traffic outage	Slide in during traffic outage	Slide in during traffic outage
Traffic Temporarily Shifted to Offset Alignment	No	Yes	No

4.1 Construction Method 1: Float Out Existing Spans / Float In New Spans

4.1.1 Features

In this method, segments of the new superstructure are constructed off site on temporary foundations. Bridge segments may be constructed elevated above water to allow barges to float underneath for pickup. Alternatively, bridge segments may be constructed elevated above land to allow heavy transporters to access underneath for rollout onto barges. For safe floating stability with an elevated bridge span aboard, two barges are needed for support of each segment, with the length of each barge oriented perpendicular to the length of the span (see Figure 4-1).

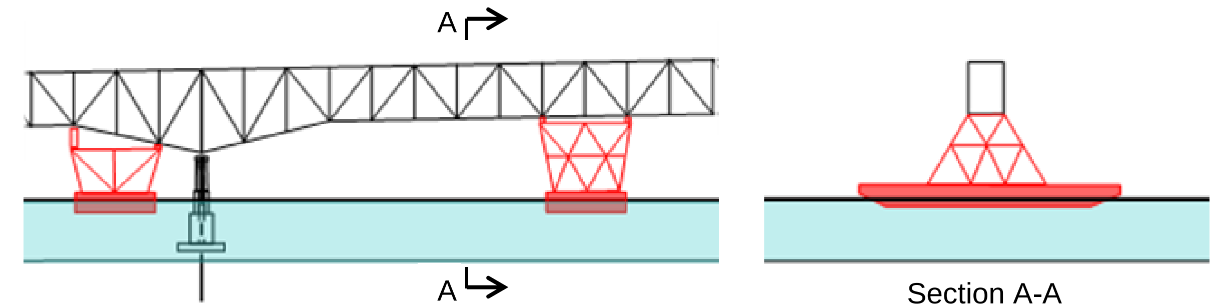


Figure 4-1: Barge Configuration for Float Out

The set of barges are fitted with support towers configured to the geometry of the bridge span above and to the geometry of the barge hull below, with due regard for how loads are safely transferred to the internal structural framing within the hull. Each set of barges and support towers can only move one segment at a time. Smaller spans at lower elevation (as for the beam and girder spans of the Norris Bridge) can each be handled by a single barge.

This method is commonly used for steel truss assemblies prefabricated without concrete deck, but is more complex and hence uncommon for shipping steel truss assemblies prefabricated complete with concrete deck, which is necessary to minimize the traffic outage.

4.1.2 Relevant Previous Project: George P. Coleman Memorial Bridge

This construction method was previously utilized successfully in the construction of the bridge that carries U.S. Route 17 over York River (known as the George P. Coleman Memorial Bridge), connecting Yorktown and Gloucester Point, Virginia. This project was constructed by Tidewater Construction Corporation (TCC; now a division of Skanska).

Originally a two lane bridge, it was replaced with a new structure accommodating four lanes on modified existing foundations. The rehabilitated bridge consists of six steel truss spans with an overall length of 2,540-feet. The longest of these spans was 559-feet, and the heaviest segment was 4,128-tons. The truss span part of the bridge is symmetric, with the three southern spans being essentially mirror images of the three northern spans. Two of the spans are designed as swing sections, allowing the bridge to open for tall ships (see Figure 4-2).

Figure 4-2: Coleman Bridge Construction: Float In of Swing Span

In the construction contract, VDOT specified that the truss spans be out of service for only two 12-day periods, while the old spans were replaced. TCC elected to reduce the shutdown period to a single 9-day window (five business days plus two weekends) for replacement of the entire truss bridge, with the goal of earning an early-completion bonus.

TCC constructed the new spans on temporary piers over water in a fabrication yard 30 miles from the old bridge (see Figure 4-3). Each completed span, fully finished with concrete deck and other appurtenances installed, was lifted from its fabrication yard piers by towers mounted on two barges under the span. The buoyancy of the barges, controlled by the level of ballast water inside them, lifted the span (see Figures 4-2, 4-4 and 4-5. The barges transported each bridge section to the permanent site, where each span was lowered onto the piers by adding ballast to the barges.

Figure 4-3: Coleman Bridge: Construction of Bridge in Fabrication Yard

According to VDOT's contract requirements, the existing structure was demolished only after the new bridge was successfully put into service. Hence, the existing bridge sections were similarly lifted from its site, transported to the fabrication yard, and lowered on the temporary piers by another set of barges outfitted to the existing bridge's geometry.

Taking advantage of the bridge's symmetry, three sets of barges were created for float in of new bridge, with each set used twice; similarly, three sets of barges created for float out of old bridge, with each set used twice.

Figure 4-4: Coleman Bridge: Anchor span and Swing Span Afloat Awaiting Installation

Figure 4-5: Coleman Bridge: Float In of Anchor Span

4.1.3 Challenges

A float out/float in scheme presents a number of challenges for successful implementation. Barge structures have limited capacity to receive high concentrated loads such as bridge reactions. Using two barges for support of a single bridge segment, relative rolling and pitching actions of barges in response to hydrodynamic loads can induce undesirable stresses in the bridge assembly carried. Also, the bridge segment is supported at locations distant from final bearings, so the transport condition can govern capacity of certain bridge members. Typical concerns are:

- Members which experience only tension in the service condition can experience compression in the transport condition and vice versa,
- With deck cast before transport and made composite with stringers/floor beams/top chords, large changes in dead load top chord stress caused by the transport condition can cause high stresses in floor beam and stringer connections,
- High variation in bottom chord stress causes axial deformations during pickup; the pickup shoes need to be detailed with expansion capability to accommodate this effect.

Finally, this type of construction is weather sensitive, since it requires an extended period of time afloat for travel from fabrication yard to bridge site. This results in a high risk to maintaining schedule due to unexpected adverse weather.

4.1.4 Applicability to Norris Bridge

The geometry of the existing Norris Bridge trusses requires configuring barges on opposite sides of a pier to avoid excessive overhang, as shown in Figure 4-1. The existing bridge will need modifications to sustain the redistribution of stress induced by the floating condition. Some members will need to be strengthened. The lower chord panel points which will newly serve as supports will need strengthening with supplemental gusset plates, as shown in Figure 4-6. All these modifications will need to be detailed in a manner which allows them to be safely performed while the bridge remains in service.

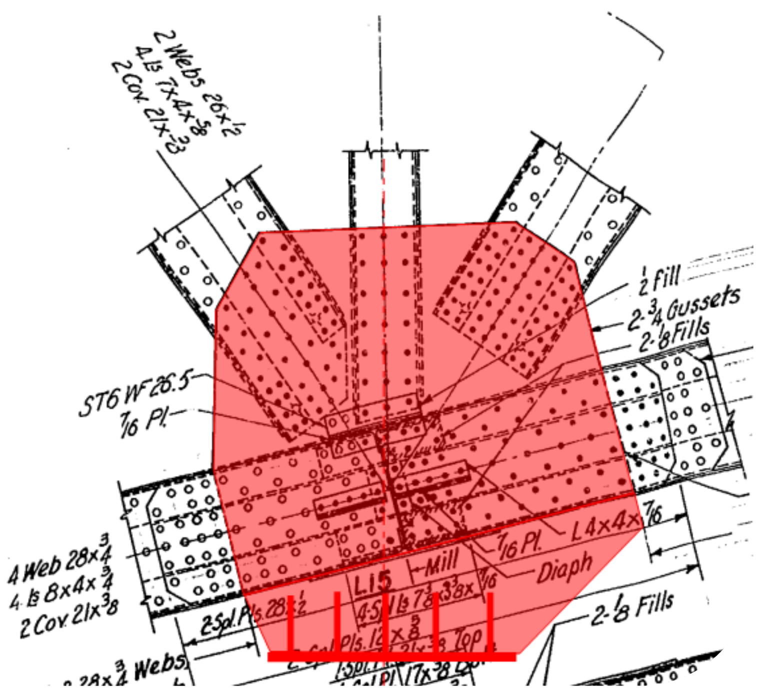


Figure 4-6: Gusset Plate Reinforcement at Pickup Points

In comparison to the Coleman Bridge, the Norris Bridge is much longer, with 15 truss spans of varying configuration and elevation, plus many girder and beam spans on approaches. For the truss spans, eight different barge sets are needed for float in and another eight sets for float out. For float out, many existing bridge segments rely on adjacent segments for support via pin and hanger connections, as shown in Figure 2-3. Hence, removal requires sequential rather than overlapping scheduling, elongating the required traffic outage.

The new Coleman bridge spans are configured with the same configuration of joints and span interfaces as the old spans. No complicated connections need to be made in the field between the swing spans and the adjacent spans. Connections between the anchor and suspended spans were done quickly, by insertion of pins. In contrast, for the Norris Bridge, it is much preferred that the new superstructure be changed to a continuous configuration over numerous spans, with fewer expansion joints than the existing bridge and no pinned hanger connections. Splicing of members of a delivered span to the mating members of the adjacent span is a time-consuming process which will increase the traffic outage time needed for installation of the new bridge segments. Splicing includes both steel superstructure members and the concrete deck and parapets.

Although a float out/float in scheme can be utilized for the beam and girder spans on the Norris Bridge approaches, it is disadvantageous for several reasons. Some spans have insufficient water depth to allow barge access. With the extensive use of pin and hanger details in the existing spans, float out sequencing is complicated by the dependencies of existing spans on each other. As for the truss spans, it is desired that the new superstructure be changed to a continuous configuration over numerous spans, with much fewer expansion joints than in the existing and no pinned hanger connections. Splicing members of a delivered span to the mating members of the adjacent span is a time-consuming process which will increase the outage time needed for installation of the new bridge segments.

For superstructure replacement alternatives that consider replacement of the substructure within the beam and girder units, construction of new foundations will also be required. Although some preparation of the new foundations can be done while the existing bridge is in service, much would remain to be done once full access is attained after superstructure float out. This would further increase the required traffic outage.

4.2 Construction Method 2: Deconstruct Existing Spans in Place / Slide in New Spans

4.2.1 Features

In this method, the new bridge is built on temporary foundations, on an alignment immediately adjacent to the existing. Based on what is most efficient for their operations, the contractor can choose to stick-build the new bridge on site or to fabricate large assemblies off-site and float them to the site on barges. For the slide in operations, continuous perpendicular tracks are provided, extending from the temporary position to final alignment.

Multiple spans can be slid into place simultaneously by use of central computer control of the lateral jacking mechanisms. Hence, continuity connections between spans can be made prior to the move. Once the bridge construction is complete on the offset alignment, traffic may be moved to the new bridge by use of temporary diversion ramps. This enables an elongated existing bridge deconstruction schedule and existing pier modifications before slide in.

Once the modified piers are ready to receive the new bridge, the slide in may be performed during a single traffic outage. The temporary works supporting the bridge, while carrying traffic on the offset alignment, is designed for live load and wind load per AASHTO requirements. This design includes secondary members to resist lateral loading. The slide girders do not need to sustain bridge live loading, and can be designed for a reduced wind loading. The temporary alignment reduces weather risks because traffic can continue until there are optimal conditions.

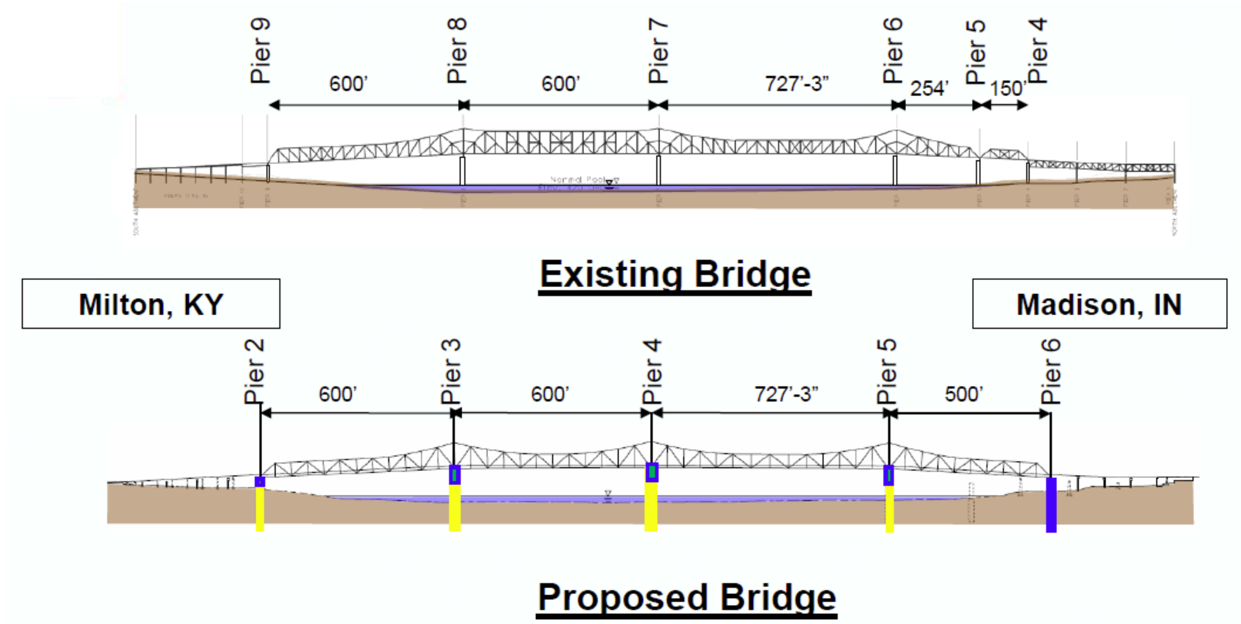
4.2.2 Relevant Previous Project: Milton-Madison Bridge

This construction method was previously utilized successfully in the construction of the bridge that carries U.S. Route 421 over the Ohio River connecting Milton, Kentucky with Madison, Indiana. The available detour for road closures was 26-miles upstream or 32-miles downstream. This project was constructed by Walsh Construction Company.

This structure has an overall truss length of 2,427-feet, including a main span of 727-feet (see Figure 4-7). The original thru-truss bridge was 20-feet in width, and the replacement structure widened to 40-feet of roadway width with an additional 5-foot cantilevered sidewalk.

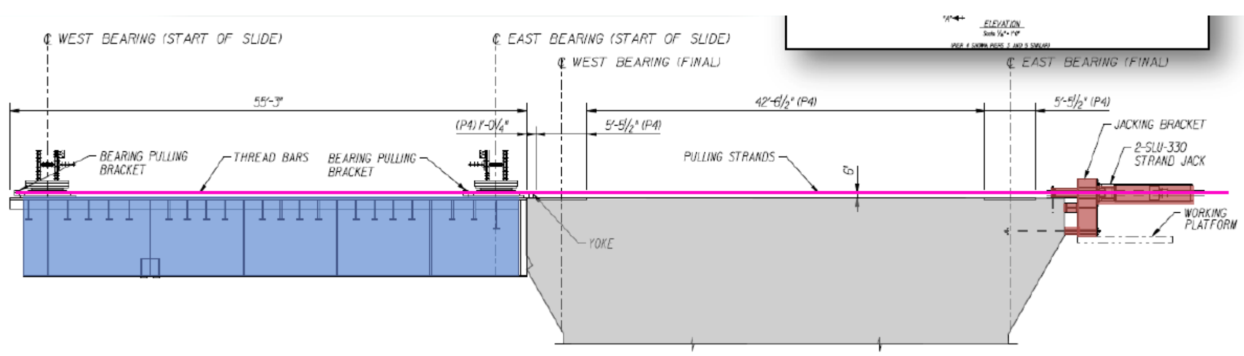
The new bridge was constructed on temporary supports, which were braced against existing piers for lateral stability. Truss span steelwork was preassembled and floated in. They were lifted off barges onto the temporary foundations using strand jacks. The deck was added while the bridge was on temporary foundations. Although the new bridge was built on an offset alignment, it was built on its permanent bearings. The bearings were modified to include a lower slip plane utilized for sliding the bridge into position.

Once the new bridge was completed in the temporary alignment, traffic was diverted on to it at each end of the bridge, allowing demolition of the old bridge and widening of existing pier caps, while maintaining traffic. The entire new bridge (four spans, weighing more than 16,000-tons) was slid into final position in a single 55-foot lateral move, by use of coordinated strand jacks (see Figure 4-8). This operation required a ten-day traffic closure.



(http://transportation.ky.gov/SASHTO/Milton%20Madison%20Bridge%20Project.pdf)

Figure 4-7: Milton-Madison Bridge: Existing and Proposed Bridge Configurations



(http://transportation.ky.gov/SASHTO/Milton%20Madison%20Bridge%20Project.pdf)

Figure 4-8: Milton Madison Bridge: Slide Configuration

4.2.3 Challenges

A deconstruct/slide in scheme presents a number of challenges to successful implementation. This method is better suited to the Milton-Madison project for a number of reasons.

The piers of the Milton-Madison Bridge are configured as solid walls, which can sustain the load of the bridge span being slid across the top of the piers. The existing Norris Bridge piers only provide vertical support at columns located beneath the existing bearings. The cross member between columns (where present) is not sufficient to sustain the heavy vertical loads of sliding operations. The existing Norris Bridge piers also have little reserve capacity for lateral load and cannot be relied upon to provide lateral restraint for the temporary supports, which results in the piers resisting storm winds on both the new and old bridges simultaneously.

The Norris Bridge has less favorable subsurface conditions than at Milton-Madison. The piles for the temporary works at the Norris Bridge must be very long to attain the desired capacity. Due to their length, they will experience significant vertical deflection as the load is shifted during a slide operation. This complicates maintaining elevation of a slide girder in alignment with the existing piers as the heavy load is shifted. The caissons supporting the existing piers have large footprints due to the need to limit bearing pressure. These footprints limit the space available for placing temporary piles around each pier. The challenges of tight space constraints and the required pile batter for lateral capacity (see Drawing 4-1) will complicate construction of the temporary works. The scheme will also require permits to build temporary foundations in river.

4.2.4 Applicability to Norris Bridge

Due to the limitations of the existing foundations, the temporary works envisioned are configured completely independent of the existing foundations. A possible scheme for the truss spans is presented in Drawings 4-1 to 4-3 in Appendix B. It includes the following features:

- Temporary supports offset one panel point from the existing piers,
- Piles are battered as needed to sustain lateral and longitudinal load,
- The new truss is outfitted with reinforced pickup points in line with temporary works,
- Pedestals are provided to support temporary bridge bearings,
- Heavy steel girders are provided between pedestals for the slide,
- For the slide, the bridge is jacked off its temporary bearings. After the slide, it is jacked down to the permanent bearings on top of the existing piers. The jack strokes compensate for any settlement caused by pile shortening during the move.

The truss spans are moved in three segments, divided based on the anticipated locations of the new bridge's expansion joints:

- North Approach Spans (Spans 19 to 23; 2,342-feet long)
- Channel Spans (Spans 16 to 18; 1,587-feet long)
- South Approach Spans (Spans 9 to 15; 3,161-feet long)

For the beam and girder spans of the Norris Bridge approaches, some superstructure replacement alternatives include complete replacement of the substructure under these units (see Section 3.4). In order to temporarily relocate approach and channel span traffic to the offset alignment, it will also be necessary to temporarily relocate beam and girder span traffic to that alignment. Two options can be considered for this:

- Construction Method 2A: Similar to the truss spans, build the beam and girder spans on temporary foundations on an offset alignment. With traffic shifted, deconstruct the existing approach spans, and construct the new piers. Once the piers are ready, slide the approach spans into position during the same traffic outage used for the truss span slide. With additional crews provided, the approach span slides can have a schedule which overlaps that of the truss slides, to avoid increasing the traffic outage duration. See Drawing 4-4 in Appendix B.
- Construction Method 2B: Construct temporary beam and girder spans using modular leased bridge elements (such as those supplied by Mabey or Acrow) on temporary foundations in line with the temporary offset alignment of the truss spans. With traffic shifted, deconstruct the existing beam

and girder spans, and conventionally construct the new spans on new foundations along the permanent alignment. Open new spans to traffic once truss spans are moved in line with them. See Drawing 4-5 in Appendix B. This method is preferred due to its simplicity. A comparison of construction costs detailed in Appendix C concludes that Construction Method 2B is also more cost effective than Construction Method 2A.

Temporary transition roadways are needed to be constructed onshore north and south of the bridge to divert Route 3 traffic onto the offset alignment. These are not needed for the other methods considered.

4.3 Construction Method 3: Slide out Existing Spans / Slide in New Spans

4.3.1 Features

This method is similar to Construction Method 2, with the new bridge built on temporary foundations along an alignment offset immediately adjacent to the existing. Based on what is most efficient for their operations, the contractor can choose to stick-build the new bridge on site or to fabricate large assemblies off-site and float them to the site on barges. For this method, temporary foundations are also being built on the opposite side of the existing bridge.

Once the construction of the new bridge superstructure is complete, the existing bridge is slid out to an offset alignment on the opposite side of the existing, and the new bridge is slid into the existing alignment. As in Construction Method 2, multiple spans can be slid simultaneously with central computer control of lateral jacking mechanisms. Unlike Construction Method 2, traffic can only utilize the new spans once they are in permanent position atop the existing piers. Hence, the temporary supports do not need to sustain live load and full storm wind load.

For the slide out, continuous tracks are provided from present alignment to temporary position for the slide out. Depending on the configuration of the new bridge, these tracks may or may not line up with those needed for the slide in. If they do not, additional temporary works are needed to provide overlapping slide out/slide in tracks. For the slide in of the new structure to the final position, continuous tracks are provided, extending from the temporary position to final alignment.

Unlike Construction Method 2, the traffic outage need not be one continuous period. The old bridge can be slid out in segments of limited length, to be replaced with corresponding segments of the new bridge during multiple consecutive outages. Between outages, traffic is carried on existing alignment by combination of slid-in new segments and remaining old segments. This approach reduces the complexity and risk of the sliding operations, but extends the duration of the road closure. It may be feasible to reuse some temporary works components, but Construction Method 3 will likely require more temporary works than Construction Method 2. Old bridge segments can be kept intact until replacement segments of new bridge are successfully in service on final alignment.

4.3.2 Relevant Previous Project: Milton-Madison Bridge

Although the Milton-Madison Bridge is an example of Construction Method 2, the approach used is easily adapted to Construction Method 3. See description of Milton-Madison bridge construction under Construction Method 2.

4.3.3 Challenges

The challenges of adapting Construction Method 3 to the Norris Bridge include those discussed for Construction Method 2. In addition, Construction Method 3 will require modifying members of the existing bridge for pickup by the sliding system. Similar to those described for Construction Method 1, modifications need to be made to the existing bridge while in service.

4.3.4 Applicability to Norris Bridge

Considerations are similar to those described for Construction Method 2, except that the slide out operations require modifications to the existing bridge, similar to those described for Construction Method 1.

For the beam and girder spans on the Norris Bridge approaches, the options proposed for Construction Method 2 do not apply since they rely on traffic having been temporarily shifted to an offset alignment, a phase which does not occur in Construction Method 3.

4.4 Comparison of Construction Methods

Evaluation of the three construction methods noted above requires consideration of several different factors, as follows:

<u>In-Place Modifications to Existing Bridge</u>:

- Construction Method 1: Modifications required for pick up points, and likely strengthening for other members since support configuration during move differs greatly from existing configuration.
- Construction Method 2: None required, since there is no requirement to maintain existing bridge integrity when deconstructing bridge.
- Construction Method 3: Modifications required for pick up points. The need for strengthening other members is less likely than in Construction Method 1, since support configuration during move differs less from existing configuration.

<u>Temporary Works Needed in Field</u>:

- Construction Method 1: With the weight carried by barge buoyancy, minimal temporary works are needed in the field. Avoids the need to deal with the poor soil conditions and vertical deflections.
- Construction Method 2: Major works needed, including temporary foundations sized for full live loading.
- Construction Method 3: In addition to the slide in foundations and slide girders needed for Construction Method 2, also need slide out foundations and slide girders. However, unlike Construction Method 2, temporary foundations do not need to sustain bridge live loading, and slide girders may be able to be reused in other locations in later phases of the work. In addition, unlike Construction Method 2, no onshore transition roadways are needed north and south of the bridge to divert Route 3 traffic onto an offset alignment.

<u>Temporary Works in Fabrication Yard</u>:

- Construction Method 1: Major works needed, including barge assemblies with span support towers. Need elevated foundations for fabricating bridge segments and for receiving existing segments floated out from bridge site.
- Construction Method 2: As needed for conventional construction, subject to contractor's discretion.
- Construction Method 3: As needed for conventional construction, subject to contractor's discretion.

<u>Sensitivity to Weather</u>:

- Construction Method 1: High sensitivity to rough weather. The barge assemblies with bridge spans on board are designed for limited wind and wave criteria. If these are exceeded, there is a risk of damage to the bridge span carried. If excessive winds or waves materialize, lead time is needed to tow the barge assemblies to more sheltered waters or back to the fabrication yard. During installation of a bridge span upon float in, the instant of set-down of the span on the bearings is particularly sensitive to heaving action of the barges in response to wave activity; vertical impact caused by a premature set-down can cause damage to bridge components. Quiet waters are crucial during this stage of installation.

- Construction Method 2: Low sensitivity, since bridge is on a solid foundation at all times. Slide operations can be postponed on short notice if excessive winds materialize when move is scheduled.
- Construction Method 3: Low sensitivity, same as Construction Method 2.

Reversibility:

This concerns maintaining existing bridge segments in operational condition until new bridge is fully assembled, installed and in service on permanent alignment. Intent is to provide a quick workaround to restore traffic should mishap cause damage to a segment of the new bridge during its installation.

- Construction Method 1: Can keep removed old bridge segments intact for a high level of reversibility. However, the removal process may still cause damage to certain components, requiring repair before segment can be reused. This method has a higher need for reversibility than the other methods, since it is more sensitive to mishaps due to movements caused by sudden rough weather or high seas.
- Construction Method 2: Keeps old bridge operational until new bridge is fully assembled and operational on temporary offset alignment. However, the old bridge is irreversibly deconstructed before the new bridge is slid into the permanent alignment, making its components unavailable for reuse to address a casualty occurring during slide in operations. Then again, a casualty is unlikely to occur during a slide in operation due to the high level of control the method allows, and its insensitivity to weather and high seas.
- Construction Method 3: Provides highest level of reversibility, but also very little need for it.

Accommodation of Change in Superstructure Configuration:

- Construction Method 1: Does not require the new superstructure to be configured similar to the existing, since the float in equipment can be configured different than the float out equipment.
- Construction Method 2: Does not require the new superstructure to be configured similar to the existing, since the existing is not slid out.
- Construction Method 3: Although new superstructure need not be configured similar to the existing, the temporary works needed is more economical if the configurations are similar. If the panel points of the new spans line up with the existing, much of the slide in works can also be utilized for slide out. If they don't line up, separate works could be required, with the associated costs.

Length of Required Traffic Outage

- Construction Method 1: Assume replacement is done in three outage periods. Each outage is estimated to have a 17-day duration, based on the following assumptions:
    - 1-day each to remove and float out an average of five spans/outage = 5-days
    - 1-day each to float in and install an average of five spans. Assume the float in operation has separate crews and equipment from the float out operation, allowing some overlap. Assume float in lags 2-days behind float out, to allow time for preparation of exposed foundations and installation of new bearings. Total addition to critical path = 2-days
    - 3-days each to establish continuity connections between mating components of adjacent spans over an average of four supports. Assume this operation lags behind the float in operation by 2 spans. Total addition to critical path = 12-day duration – 3-day overlap = 9-days
    - 1-day allocation for potential rough weather delay during float out/float in operations.
    - Assume establishing connections between adjacent in-service segments of bridge is performed concurrently with the above, with no net effect on outage duration.
    - Assume beam and girder spans replaced in a manner which does not increase the outage.
- Construction Method 2: Assume move of truss spans is done in three segments. The outage is estimated to have a 15-day duration, based on the following assumptions for each move:
    - 2-days for slide in
    - 2-days for establishing connections between adjacent segments of bridge.
    - 1-day for repositioning crews and equipment between moves
    - Assume beam and girder spans replaced in a manner which does not increase the outage.
- Construction Method 3: Assume replacement is done in three outage periods. Each outage is estimated to have an 8-day duration, based on the following assumptions:
    - 2-days for slide out
    - 2-days for preparation of exposed foundations and installation of new bearings
    - 2-days for slide in
    - 2-days for establishing connections between adjacent in-service segments of bridge.
    - Assume beam and girder spans replaced in a manner which does not increase the outage.

4.5 Staging of Construction

Staging of construction is an approach commonly used on bridge replacement projects wherein the bridge is constructed in portions and at least one lane of traffic is maintained at all times during construction. Temporary signals are often employed to control two-way use of a single lane. The fracture critical configuration of the majority of the existing Norris Bridge does not facilitate partial demolition typically needed for such staging of construction on the existing alignment. In order to make such staging of construction feasible in the spans which do not have a fracture critical configuration, additional widening is required off the alignment of the existing bridge, which is not compatible with the purpose and scope of this study. The use of staged construction sequences is not considered further as a part of this study.

4.6 Conclusions Regarding Construction Method

The lengthy detour available to bridge users means that any closure of the bridge for construction activities will result in significant impacts to the traveling public. For this reason, construction method and the duration of bridge closure required are important considerations in evaluating all superstructure replacement alternatives. In order to minimize the duration of bridge closure during superstructure replacement, the construction method should provide means to:

- Utilize components prefabricated off-line before installation in the field,
- Quickly remove the existing superstructure once traffic outage is triggered or deconstruct the existing superstructure in place without outage after shifting traffic to a temporarily offset alignment,
- Place new components quickly during traffic outage period.

To achieve these goals, three construction methods are considered:

- Construction Method 1: Float Out Existing Spans / Float In New Spans
- Construction Method 2: Deconstruct Existing Spans In Place / Slide In New Spans
- Construction Method 3: Slide Out Existing Spans / Slide in New Spans

Table 4-2 presents a summary of the methods for comparison. Based on the comparison, Construction Method 2 is preferred. This construction method includes construction of the new superstructure on temporary foundations located on an alignment offset immediately adjacent to the existing. Once the bridge superstructure construction is complete on the offset alignment, traffic may be moved to the new deck by use of temporary diversion ramps at each end of the bridge. This enables an extended schedule for deconstruction of the existing bridge and modification of the existing piers before slide in. Once the modified piers are ready to receive the new superstructure, the slide in may be performed during a single traffic outage. Multiple spans can be slid into place simultaneously by use of central computer control of the lateral jacking mechanisms. Hence, continuity connections between spans can be made prior to the move. Construction Method 2 is preferred for the following reasons:

- Construction Method 2 requires no modifications to the existing trusses for demolition, as the other methods do.
- Construction Method 2 requires no off-site fabrication facilities, as Construction Method 1 does.
- Construction Method 2 is not sensitive to the risks associated with weather events.
- Construction Method 2 facilitates changes to the bridge vertical profile more conveniently and cost effectively than Construction Method 3.
- Construction Method 2 offers a shorter overall road closure than the other methods. Due to the particular details of the Norris Bridge construction, Construction Method 1 requires an intolerably lengthy traffic outage.
- Although Construction Method 2 does not offer the level of reversibility theoretically offered by the other methods, there is little need for this feature since the risk of failure is low in a slide in scheme. That is why the Milton-Madison Bridge slide in was completed without reversibility.
- Construction Method 2 is anticipated to be substantially less expensive than Construction Method 3. Although Construction Method 3 allows reuse of some temporary works components (e.g., the slide girders) in later phases, this does not result in much economy. The cost of the additional foundations which Construction Method 3 needs to facilitate slide out, the cost of modifying the existing bridge for slide out, and the cost of implementing the slide out operation, more than offset any cost saving advantages of Construction Method 3, which include reuse of selected members, design without live load, and lack of transition roadways.

Due to the unfavorable soil conditions at the site, the bulk of the costs for all methods are attributable to the required foundation elements. Previous experience suggests that Construction Method 1 does not offer cost savings over Construction Method 2.

A conceptual cost estimate is prepared for the temporary works associated with Construction Method 2, including both Construction Method 2A and Construction Method 2B for replacement of the beam and girder spans. These estimates indicate that Construction Method 2B is more cost effective. These estimates are based on the configuration shown in Drawings 4-1, 4-2, 4-3 and 4-5 in Appendix B. This cost information is presented in Section 5 and detailed in Appendix C.

Table 4-2: Comparison of Construction Methods

	Construction Method 1	Construction Method 2	Construction Method 3
Existing Bridge Removal	Float Out	Deconstruct in Place	Slide Out
New Bridge Placement	Float In	Slide In	Slide In
Traffic temporarily shifted to Offset Alignment	No	Yes	No
In-Place Modifications to Existing Bridge	Strengthen pick-up points Strengthen bridge members	None required	Strengthen pick-up points
Temporary Works Needed in Field	Minimal	Slide in foundations Components designed for live load and full wind load Components can't be used multiple times	Slide in foundations Slide out foundations Components designed for 25-year wind load and no traffic live load Some components may be used multiple times
Temporary Works in Fabrication Yard	Barge systems with support towers Foundations for construction of new bridge segments Foundations for set down of existing bridge segments	Fabricator's means and methods	Fabricator's means and methods
Sensitivity to Weather	High	Low	Low
Reversibility	Existing bridge spans can be kept intact but separated until new bridge operational on permanent alignment	Existing bridge spans kept intact until new bridge operational on offset alignment; not intact at slide in to permanent alignment	Existing bridge spans can be kept intact until new bridge operational on permanent alignment
Accommodation of Change in Superstructure Configuration	Flexible	Flexible	Increases cost of temporary works
Length of Required Traffic Outage	Removal must be performed during outage Can only remove/install one span at a time **Estimated Outage:** **3 stages x 17-days each = 51-days**	Removal performed before outage **Estimated outage:** **1 stage of 15-days = 15-days**	Removal must be performed during outage **Estimated Outage:** **3 stages x 8-days each = 24-days**

Key:	Dark shading	= Disadvantage	Light shading	= Neutral	No shading	= Advantage

5 Superstructure Replacement Alternatives

Each of the proposed superstructure replacement alternatives has been developed to consider a range of feasible solutions to achieve the design objectives noted in Section 1. These alternatives represent a combination of the various options reviewed in earlier sections, including bridge width, structure type, navigation clearances, and substructure modifications. The alternatives are also developed with due consideration of the required construction methods – especially with respect to duration of bridge closure given the extraordinary inconvenience imposed upon the traveling public by the extreme distance associated with even the shortest detour route. The following alternative designs are considered based upon the aforementioned criteria for development. A summary follows in Table 5-3.

5.1 Superstructure Replacement Alternative A

This superstructure replacement alternative very closely replicates the existing condition with respect to both structure type and roadway width, and serves as a baseline for comparison to other alternatives. The proposed deck width is 24-feet curb-to-curb, for the entire bridge length. Schematic details are provided in Drawings 5-1 and 5-2.

The approach spans are comprised of two units (seven and five spans) of continuous-span composite deck trusses and the channel unit consists of a continuous three-span through truss section in the main span that transitions to deck units in the flanking spans. The existing bearing locations are utilized and overall, the applied forces to the substructure are similar to or less than the existing condition. Pier modifications in these units are limited to encapsulation of the existing pier caps for lateral load carrying capacity and fiber wrapping of the existing pier columns to provide additional strength to these sections in accordance with current AASHTO requirements. Minor additional work is required to prepare the bearing areas for the new structure (see Drawing 5-11).

In the beam and girder units, the superstructure types are similar to the existing structure, with multi-beam superstructure on the existing pile bents and a two-girder superstructure on the existing two-column concrete piers. Beam and girder elements are continuous at pier locations. Substructure repairs to address deterioration are required, but major strengthening is not anticipated. This proposed configuration incorporates deck joints at the ends of the units, at the transitions of superstructure types (beam spans to girder spans), and at intermediate piers as required.

Conventional truss and deck construction methods are utilized to assemble the superstructure. Access to each pier unit to accomplish the required substructure modifications are the primary cost elements associated with the work on the main piers. Construction Method 2 is easily adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

This alternative results in the least amount of substructure modification to accommodate the new superstructure. The proposed structure type incorporates fracture critical elements.

5.2 Superstructure Replacement Alternative B

This superstructure replacement alternative is similar to Alternative A, except that this alternative partially incorporates the desirable widening discussed in Section 2.2. The proposed bridge width is 40-feet curb-to-curb except for the channel span unit, where the width is 24-feet curb-to-curb. This approach is considered to avoid the expense and complexity of widening the piers supporting the thru-truss of the channel span. Schematic details are provided in Drawings 5-3 and 5-4.

The approach span units are configured as described in Alternative A, with the exception that the floor beams will extend cantilevered beyond the limits of the trusses in order to provide support to the wider deck width. The existing bearing locations are utilized and overall, the applied forces to the substructure are similar to or less than the existing condition. Pier modifications in these units are limited to encapsulation of the existing pier caps for lateral load carrying capacity and fiber wrapping of the existing pier columns to provide additional strength to these sections in accordance with current AASHTO requirements. Minor additional work is required to prepare the bearing areas for the new structure (see Drawing 5-11).

In the beam and girder units, the recommended superstructure configuration is based on Construction Method 2B described in Section 3, which includes replacement of the existing pile bents and piers within these units. The superstructure type includes precast Bulb T girders with a concrete deck. The replacement spans in each unit are based on a nominal span length of 125-feet, with new pier locations selected to avoid the existing pier locations.

Conventional truss and deck construction methods are utilized to assemble the superstructure. Construction Method 2 is easily adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

This alternative is advantageous because of the minor substructure modifications required in the approach and channel span units, since the existing bearing locations are utilized and the applied forces to the substructure are similar to the existing condition. New substructure is constructed in the beam and girder span units. The proposed structure type incorporates fracture critical elements. The transition of the curb-to-curb width in the navigation channel unit will require specific evaluation and detailing.

5.3 Superstructure Replacement Alternative C

This superstructure replacement alternative is similar to Alternatives A and B, except that the structure width incorporates the desirable widening discussed in Section 2.2 for the full length of the bridge, including the navigation channel span. The proposed bridge width is 40-feet curb-to-curb. Schematic details are provided in Drawings 5-5 and 5-6. The implications of modifying the channel span piers to accommodate a widened through truss are considered.

In the approach and channel span units; this alternative requires major reconstruction of Piers 8, 9, 10, and 11. Because of the scope of the pier reconstruction required to add 8-feet of width to each side of the structure, demolition and reconstruction of the existing column and cap elements down to the "Top of Wall" Elevation (7.83-feet) is considered. In addition, the remaining 12 piers only require the minor substructure modifications required for Alternatives A and B, since again, the existing bearing locations are utilized, and the applied forces to the substructure are similar to the existing condition. Minor additional work is required to prepare the bearing areas for the new structure (see Drawing 5-11).

In the beam and girder units, the recommended superstructure configuration is the same as for Alternative B, including replacement of the existing pile bents and piers, to support a precast girder superstructure with a concrete deck. The replacement spans in each unit are based on a nominal span length of 125-feet, with new pier locations selected to avoid the existing pier locations.

Conventional truss and deck construction methods are utilized to assemble the superstructure. Construction Method 2 is easily adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

This alternative is advantageous because it provides the desirable structure width for the entire length of the structure. This alternative requires an increased degree of substructure modifications when compared with the previous two alternatives. The proposed structure type also incorporates fracture critical elements.

5.4 Superstructure Replacement Alternative D

This superstructure replacement alternative is similar to Alternative C, except for the structure configuration in the navigation channel. The goal of this alternative is to eliminate the thru-truss over the navigation channel and avoid the pier reconstruction associated with Alternative C, but at the cost of reduced vertical clearance. Schematic details are provided in Drawings 5-7 and 5-8.

Currently, this vertical clearance is 110-feet above mean water level. For this alternative, this clearance is reduced to approximately 75-feet to accommodate the necessary structure depth for the 648-foot channel span. Notably, the river has very little commercial marine traffic and is primarily used for recreational sailing. This reduction in vertical clearance is judged to be reasonable based on the review of marine traffic data presented earlier. However, approval of the U.S. Coast Guard is required prior to selection of this alternative. There is precedent for this proposed vertical clearance criteria on a recent project, such as the United States Naval Academy Bridge over the Severn River in Annapolis, Maryland, where a 75-foot vertical clearance was approved as adequate.

In the approach and channel span units, a deck truss superstructure type is utilized for the entire length of the unit. The proposed reduction in vertical clearance will accommodate the required structure depth. The existing bearing locations for the trusses are utilized, with floor beams cantilevered beyond the beyond the limits of the trusses in order to provide support to the wider deck width. The applied forces to the substructure are similar to the existing condition. Pier modifications are limited to encapsulation of the existing pier caps for lateral load carrying capacity and fiber wrapping of the existing pier columns to provide additional strength to these sections in accordance with current AASHTO requirements. Minor additional work is required to prepare the bearing areas for the new structure (see Drawing 5-11).

In the beam and girder units, the recommended superstructure configuration is the same as for Alternative B and C, including replacement of the existing pile bents and piers, to support a precast girder superstructure with a concrete deck. The replacement spans in each unit are based on a nominal span length of 125-feet, with new pier locations selected to avoid the existing pier locations.

Conventional truss and deck construction methods are utilized to assemble the superstructure. Construction Method 2 is easily adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

Like Alternatives A and B, this configuration is advantageous because it provides the desirable widening without the thru-truss configuration. Minor substructure modifications required since the existing bearing locations are utilized, and the applied forces to the substructure are similar to the existing condition. The proposed structure type also incorporates fracture critical elements.

5.5 Superstructure Replacement Alternative E

This superstructure replacement alternative represents the use of a different structure configuration to accomplish the desirable widening. The proposed bridge width is 40-feet curb-to-curb throughout the entire length of the structure. Schematic details are provided in Drawings 5-9 and 5-10.

In the approach and channel span units, a three-girder system which transitions to a through tied arch over the navigation channel, as described in Section 3. This alternative is considered to realize structural redundancy and to simplify construction issues associated with girder stability. The ramifications of modifying the channel span supports to accommodate widening are considered. The steel tied-arch section of the main span also utilizes a post-tensioned concrete tie-girder and end elements to attain a higher level of redundancy.

This alternative requires major reconstruction of the four piers supporting the channel span unit (Piers 8-11). Because of the scope of the pier reconstruction required to add eight-feet of width to each side of the structure, demolition and reconstruction of the existing column and cap elements down to the “Top of Wall” Elevation (7.83-feet) is considered. In addition, the remaining 12 piers also require construction of adequate caps and center column elements to accommodate the center third girder. Minor additional work is required to prepare the bearing areas for the new structure. Application of lateral loading to the substructure units is similar to the existing bridge, given the geometric similarities between the existing truss and delta configurations (see Drawing 5-11).

Non-conventional girder construction methods are required to assemble the superstructure given the lateral stability / size of the unassembled delta field sections at the piers. Also, it is likely that the tied-arch (without the deck elements) is assembled on land and “floated in” as a single unit adding additional complication to the construction process. Construction Method 2 is adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

In the beam and girder units, the recommended superstructure configuration is the same as for Alternative B and C, including replacement of the existing pile bents and piers, to support a precast girder superstructure with a concrete deck. The replacement spans in each unit are based on a nominal span length of 125-feet, with new pier locations selected to avoid the existing pier locations.

Additional details are provided in Drawings 5-9 and 5-10.

5.6 Superstructure Replacement Alternative Costs

The proposed superstructure replacement alternatives presented in this section represent a combination of the various options reviewed in earlier sections, including bridge width, structure type, navigation clearances, and substructure modifications. Table 5-3 summarizes the primary characteristics of each superstructure replacement alternative. Each option is feasible and constructible. The impacts to natural resources are considered reasonably similar for comparison of alternatives. Table 5-1 presents a summary of the estimated cost of each alternative. Detailed cost estimate data is included in Appendix C.

The estimated costs presented here include construction cost, temporary works, right-of-way, engineering and development, and contingency. These costs are for comparative purposes and are presented in present-day dollars with no inflation. For development of these estimates, it is assumed that each alternative is constructed by erection of the new bridge on temporary alignment, deconstruction of the existing bridge with traffic detoured to the temporary alignment, and sliding of the structure into final position on the existing piers. The cost of these temporary works is a significant factor in the overall superstructure replacement project cost.

Table 5-1: Superstructure Replacement Alternative Cost Summary (values expressed in millions)

Component	Alt. A	Alt. B	Alt. C	Alt. D	Alt. E
Superstructure	$61	$84	$92	$90	$118
Substructure	$5	$20	$22	$19	$19
Mobilization & Demo.	$14	$16	$16	$16	$17
Temporary Works for Rapid Replacement	$148	$148	$148	$148	$148
Contingency	$46	$53	$56	$55	$61
Project Dev. & Admin.	$38	$51	$55	$53	$63
Total Alternative Cost	$312	$371	$389	$381	$426

A review of the cost data indicates that Alternative A, which does not provide the desired widening or replacement of beam and girder span substructure, shows the least overall cost. Alternative D is the least expensive alternative among those that do provide both the desired widening and the beam and girder span substructure replacement.

In order to evaluate the minimum feasible project costs for superstructure replacement, two supplemental alternatives were developed to consider a new set of criteria. Rather than provide the desirable roadway width, the minimum roadway width permissible by VDOT standards for a superstructure replacement project is considered. The cross section consists of two 12-foot lanes with 3-foot shoulders for a 30-foot roadway width. Consideration is also given to allow conventional construction methods for the superstructure replacement. Conventional methods include allowing a full closure of the bridge while the superstructure is deconstructed and rebuilt in place.

Alternative D1

Alternative D1 is similar to Alternative D except that the roadway width is reduced to 30-feet for the entire bridge length. The goal of this alternative is to measure what cost savings are associated with providing the minimum required roadway width. Alternative D1 retains the rapid replacement construction methods that provide a small closure window. Schematic details are provided in Drawings 5-12 and 5-13.

The superstructure in the approach and channel span units is a deck truss superstructure type similar to Alternative D, however, the reduced roadway width eliminates the need to cantilever the floor beams. The substructure modifications for the approach and channel units are similar to Alternative D.

The beam and girder unit superstructure and the substructure modifications are similar to Alternative D. Despite the reduction in superstructure width and loads, there is minimal cost savings realized because the substructure costs are driven by the significant pile depth necessary to resist lateral loads.

Conventional truss and deck construction methods are utilized to assemble the superstructure. Construction Method 2 is easily adapted to this alternative, using Construction Method 2B for the replacement of the beam and girder spans.

Alternative F

Superstructure replacement Alternative F represents the implementation of both revised criteria discussed above. The roadway width for this alternative is also 30-feet curb-to-curb. The goal of this alternative is to remove the closure time limitations and permit an extended bridge closure. Schematic details are provided in Drawings 5-14 and 5-15.

The superstructure in the approach and channel span units is a three-girder system that transitions to a through tied arch over the navigation channel similar to Alternative E. Unlike Alternative E, the three-girder system for Alternative F is steel plate girders. Because the bridge closure restrictions are removed, a steel plate girder option is now available for consideration, as discussed in Section 3.2. Additionally, VDOT previous project experience dictates that steel plate girders outperform steel delta girders. The substructure modifications for the approach and channel units are similar to Alternative E.

The beam and girder unit superstructure and the substructure modifications are similar to Alternative E. Despite the reduction in superstructure width and loads, there is minimal cost savings realized because the substructure costs are driven by the significant pile depth necessary to resist lateral loads.

Conventional multi-girder construction methods are utilized to assemble the superstructure. The entire structure will be closed for the deconstruction of the existing superstructure, modification and reconstruction of the substructure, and construction of the new superstructure. The closure time required for Alternative F is estimated to be approximately 4-years long.

Table 5-2 presents a summary of the estimated cost for the revised superstructure replacement alternatives that provide the minimum required roadway width. Detailed cost estimate data is included in Appendix C. For development of these estimates, Alternative D1 assumptions are the same as for Alternatives A through E. It is assumed that Alternative F is constructed by traditional methods consisting of deconstruction of the existing bridge and construction of the superstructure on the existing piers.

Table 5-2: Superstructure Replacement Supplemental Alternative Cost Summary (values expressed in millions)

Component	Alt. D1	Alt. F
Superstructure	$71	$108
Substructure	$19	$27
Mobilization & Demo.	$15	$17
Temporary Works for Rapid Replacement	$148	$0
Contingency	$51	$30
Project Dev. & Admin.	$46	$54
Total Alternative Cost	$349	$237

Comparing the cost data between Alternatives D and D1 indicates that reducing the deck width saves approximately $30M, but total cost of Alternative D1 is still cost prohibitive. Alternative F is the least expensive among all of the superstructure replacement alternatives. The savings from using conventional construction methods make Alternative F a cost-viable alternative, but it requires a lengthy full bridge closure window.

Table 5-3: Summary of Superstructure Replacement Alternatives

	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E	Alternative D1	Alternative F
Beam and Girder Span Road Width	24 ft (match existing)	40 ft	40 ft	40 ft	40 ft	30 ft	30 ft
Beam & Girder Span Units Structure Type	Steel beams made cont. (match existing)	Concrete girders made cont.	Concrete girders made cont.	Concrete girders made cont.	Concrete girders made cont.	Concrete girders made cont.	Concrete girders made cont.
Beam & Girder Span Units Substructure	Rehab & reuse	Replace	Replace	Replace	Replace	Replace	Replace
App Span Road Width	24 ft (match existing)	40 ft	40 ft	40 ft	40 ft	30 ft	30 ft
App Span Units Structure Type	Cont. steel deck truss (with no pinned hangers)	Cont. steel deck truss with cantilever floor beam brackets	Cont. steel deck truss with cantilever floor beam brackets	Cont. steel deck truss with cantilever floor beam brackets	3 Cont. steel delta girders with cantilever floor beam brackets	Cont. steel deck truss	3 Cont. steel plate girders
App Span Units Substructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure
Channel Span Road Width	24 ft (match existing)	24 ft (match existing)	40 ft	40 ft	40 ft	30 ft	30 ft
Channel Span Units Structure Type	3-span cont. steel deck truss transitioning to thru truss center span	3-span cont. steel deck truss transitioning to thru truss center span	Steel deck & thru truss, with desired deck width	Steel deck truss with cantilever floor beam brackets	Steel delta girders with tied arch over channel	Cont. steel deck truss	3 Cont. steel plate girders with tied arch over channel
Channel Span Units Substructure	Rehab and modify existing for new superstructure	Rehab and modify existing for new superstructure	Reconstruct channel pier caps and columns	Rehab and modify existing for new superstructure	Reconstruct channel pier caps and columns	Rehab and modify existing for new superstructure	Reconstruct channel pier caps and columns
Vertical Channel Clearance	Match existing	Match existing	Match existing	Reduced from existing	Match existing	Reduced from existing	Reduced from existing
Horizontal Channel Clearance	Match existing	Match existing	Match existing	Match existing	Match existing	Match existing	Match existing
Construction Method	Construct on temporary alignment, slide superstructure in place during closure	Construct on temporary alignment, slide superstructure in place during closure	Construct on temporary alignment, slide superstructure in place during closure	Construct on temporary alignment, slide superstructure in place during closure	Construct on temporary alignment, slide superstructure in place during closure	Close bridge and construct on existing alignment	Close bridge and construct on existing alignment
Advantages	Least cost of all superstructure replacement alternatives	Avoids pier reconstruction in channel span units	Provides desired widening for full length of bridge	Provides desired widening for full length of bridge, least cost of desired widening alternatives	Provides desired widening for full length of bridge. Non-fracture critical structure	Provides minimum widening required for less cost than the desirable widening	Provides minimum widening required for less cost than the desirable widening. Non-fracture critical structure.
Disadvantages	Not the desired widening	Not the desired widening for full bridge length	Requires pier reconstruction in the channel span units	Requires reduction in vertical clearance	Greatest cost of all alternatives	Requires reduction in vertical clearance and not the desired widening	Requires reduction in vertical clearance and not the desired widening

6 Complete Bridge Replacement

6.1 Selection of Structure Types

Given the high priority to minimize impacts to traffic during construction, and the high cost of completing a superstructure replacement project with rapid replacement construction methods, it is evident that complete replacement of the bridge on a new alignment should also be evaluated for comparison with the superstructure replacement alternatives. This section summarizes the results of a conceptual study to determine the most viable and least cost bridge replacement structure type considering site specific constraints, construction, low maintenance materials and durable design details.

For purposes of this evaluation, it is assumed that the alignment of the new crossing would be located upstream of the existing bridge, to avoid the existing overhead electric utility infrastructure on the downstream side of the existing bridge. Determination of the new alignment location would seek to minimize the acquisition of new right-of-way and to provide sufficient clearance to the existing bridge to facilitate construction and minimize traffic impacts during construction. This conceptual evaluation of bridge concepts is not sensitive to the exact alignment location.

6.1.1 Span Optimization

Span optimization curves are generally developed to determine the efficiency of various bridge alternatives. Increasing span length (fewer piers) decreases substructure costs and increases superstructure costs. These costs are added for various span arrangements to determine the least cost span length. This type of span optimization curve is extremely beneficial for a project like the Route 3 Bridge. Past experience with bridges having similar length and height over water has yielded optimal span lengths of about 225. Figure 6-1 represents a typical span optimization curve expected for this type of crossing.

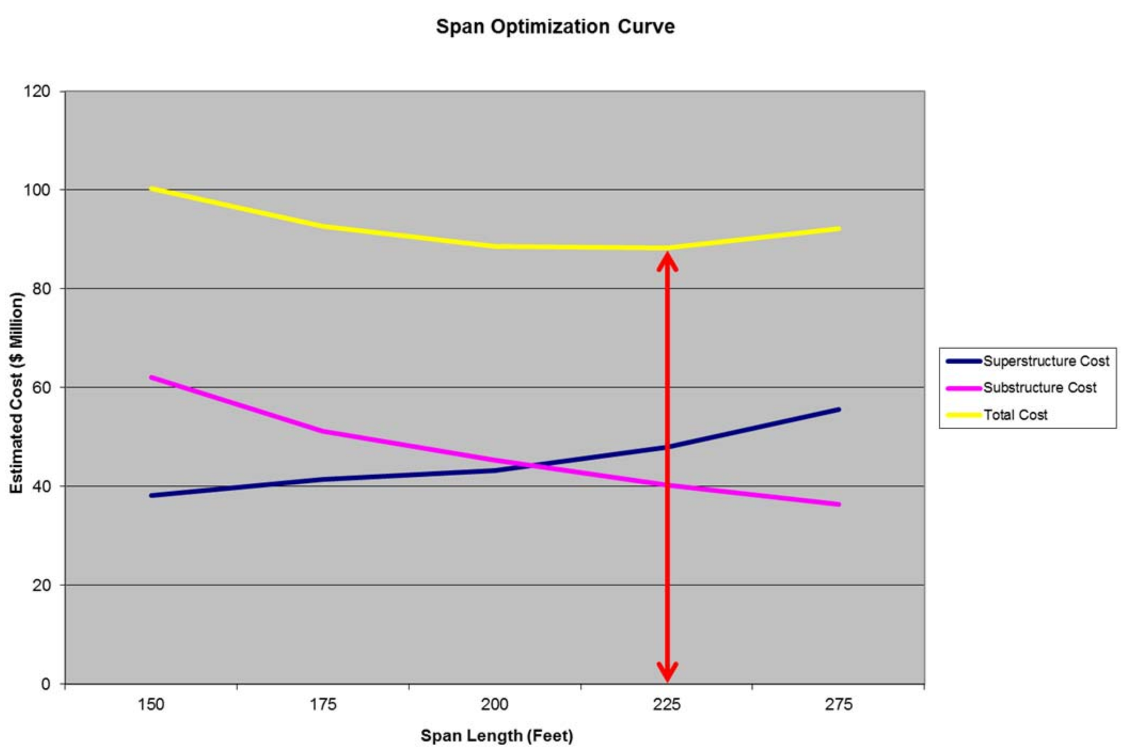


Figure 6-1: Typical Span Optimization Curve

The span optimization curve illustrated in Figure 6-1 is predicated on deep foundations on the order of 100 to 120-feet deep. This depth represents relatively good subsurface conditions that provide the necessary axial bearing and lateral support capacities required. Under these conditions, the increased cost of the substructure associated with shorter span lengths usually outweighs the savings gained from the less expensive superstructure using shorter spans.

Based on this past experience, six bridge alternative types are initially evaluated including:

- Replacement Option 1 – Prestressed Concrete Bulb T Girders Spans (140-feet and 185-feet) and Steel Channel Spans
- Replacement Option 2 – Steel Plate Girder Spans (250-feet typ and 400-feet channel span)
- Replacement Option 3 – Precast Segmental Box Girder, Balanced Cantilever Spans (250-feet typ and 400-feet channel)
- Replacement Option 4 – Precast Segmental Box Girder, Balanced Cantilever Spans (400-feet typ)
- Replacement Option 5 – Steel Plate Girder Spans (400-feet typ)
- Replacement Option 6 – Extradosed Cable Stay (500-feet typ) and Prestressed Concrete Bulb T Girders Spans (140-feet)

The expectation is that Replacement Options 1, 2 and 3 are the most cost effective. Figure 6-2 represents the span optimization curve developed for these six options based on foundation information described in Section 6.3. The major difference in the span optimization curves illustrated in Figures 6-1 and 6-2 is that the Norris Bridge requires deep foundations on the order of 180 to 220-feet long from water surface level (i.e. twice as long as normally expected). The high costs associated with these deep foundations results in the span optimization curve never converging. Rather than the substructure costs decreasing with increased span lengths, the substructure costs continue to increase.

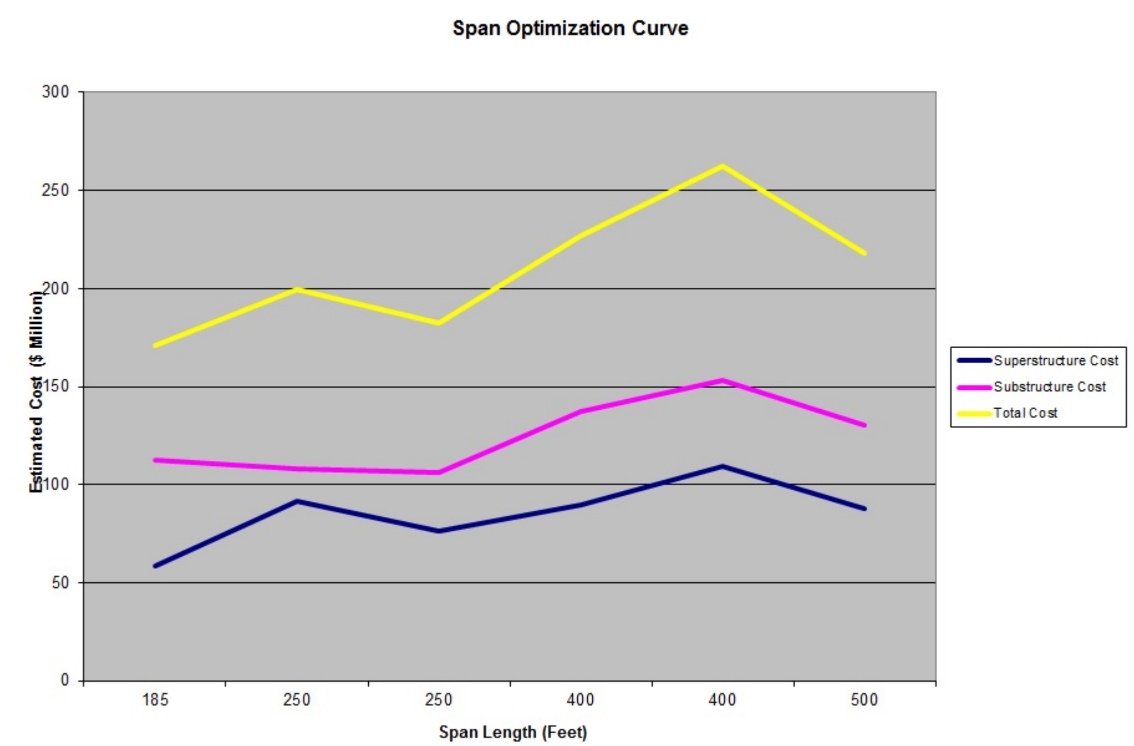


Figure 6-2: Route 3 Bridge Span Optimization Curve

As a result of the lack of convergence, two additional bridge alternatives are considered.

- Replacement Option 7 – Prestressed Concrete Bulb T Girder Spans (150-feet) and Steel Channel Spans
- Replacement Option 8 – Precast Segmental Box Girder, Span-By-Span (150-feet typ and 400-feet channel span)

These additional two alternatives attempted to use smaller spans with lower superstructure costs and lighter dead loads to reduce foundation costs. Note that prestressed concrete spliced girders, similar to what was used in West Point, VA for the Route 33 bridges, are not considered. This type of structural system is economical for span lengths up to 320-feet. Since the main span requirement is 400-feet, this option is ruled out. Furthermore, this bridge type is not considered for the 185-foot long approach spans because 96-inch deep precast concrete square girders have proven to be more economical on previous projects.

Each of the eight bridge alternatives are described in detail in Section 6.2. Based on the results of the span optimization curve, it is clear that foundation costs are driving the bridge type selection and smaller span lengths yield lower overall costs. Conceptual costs are presented in Section 6.4.

6.2 Total Bridge Replacement Conceptual Alternatives

The feasibility study discussed in the following sections of this report considers eight viable superstructure types that meet the site specific requirements for the Route 3 Bridge. The conceptual alternatives are based on our past experience with similar projects.

6.2.1 Replacement Option 1 – Prestressed Concrete Bulb T Girder Spans (140-feet and 185-feet) and Steel Channel Spans

Total bridge length for this option is 10,280-feet, consisting of a combination of 140-foot and 185-foot long prestressed concrete Bulb T girder spans leading up to the channel. There are six 140-foot long pile bent spans followed by nineteen 185-foot long spans on the south approach. Similarly, there are fifteen 140-foot long pile bent spans followed by fifteen 185-foot long spans on the north approach. The 140-foot spans are comprised of 77-inch deep Bulb T girders spaced at 8’-3” center to center spacing. The 185-foot spans are comprised of PCEF XC-95-60 Bulb T (modified VDOT Bulb T) girders spaced at 8’-3” center to center spacing. The expansion joint units have been set between 420-feet to 740-feet for a total of seventeen expansion joints. The channel expansion joint unit is comprised of 325-feet - 400-feet - 325-feet long spans consisting of constant depth (140-inch webs) steel plate girders spaced at 11’-9” center to center spacing. The 400-foot long center span provides the minimum navigational envelope of 110-feet vertically by 300-feet horizontally.

Piers heights range from 14 to 110-feet tall above mean high water elevation. Foundations consist of 66-inch diameter concrete cylinder piles up to 170-feet long for the 140-foot spans and 72-inch concrete filled steel pipe piles for the 185-feet spans and channel spans. Water line footings are used in order to eliminate the need for expensive coffer dams. All piers are assumed to be designed for vessel impact using a probability based approach to determine the likelihood of impact and associated design forces (i.e. reduced impact forces).

The steel spans are treated with a duplex system (metalizing and painting) for corrosion protection in this aggressive environment. In accordance with VDOT’s IIM-S&B-81.7 requirements, Class III corrosion resistant reinforcing steel is used in the superstructure including the deck, barriers, and diaphragms. Class I corrosion resistant reinforcing steel is used for all substructures in tidal waters, prestressed concrete girders stirrups, and other reinforcement extending into concrete deck slab.

Refer to Drawings 6-1 through 6-4 in Appendix B for details.

Figure 6-3 is an example of a bridge designed and built using prestressed concrete Bulb T girder approach spans in combination with steel plate girder channel spans. The Foley Beach Express Bridge over the Intracoastal Waterway is 1,860-feet long and is comprised of 135-foot long 78-inch deep Bulb T girders and a three span steel girder channel unit with a 300-foot long main span. Additionally, Bayshore Concrete Products (BCP-Skanska) precast yard located in Cape Charles Virginia recently precast and delivered 185-foot long, 96-inch deep Bulb T girders for a bridge along the Garden State Parkway in New Jersey.

Figure 6-3: Foley Beach Express over Intracoastal Waterway, AL

6.2.2 Replacement Option 2 – Steel Plate Girder Spans (250-feet typ and 400-feet channel span)

Total bridge length for this option is 10,250-feet consisting of 250-foot long steel plate girder spans leading up to the channel. The expansion joint units have been set at 1,150-feet to 1,400-feet minimizing the number of expansion joints to ten. The interior to end span length ratio is set to 1.25 (250:200) based on experience designing optimal steel cross sections (i.e. reduced structural steel weight). The 250-foot spans are comprised of constant depth (84-inch webs) steel plate girders spaced at 11’-9” center to center spacing. The channel unit is the same as for Replacement Option 1.

Piers heights range from 14 to 110-feet tall above mean high water elevation. Foundations consist of 72-inch concrete filled steel pipe piles and water line footings.

Similar to Replacement Option 1, the steel spans are treated with a duplex system (metalizing and painting) and corrosion resistant reinforcing steels (Class I and III) are used.

Refer to Drawings 6-5 and 6-6 in Appendix B for details.

Figure 6-4 is an example of a long river bridge designed and built using steel plate girder spans. The Driscoll Bridge over the Raritan River is 4,379-feet long with a maximum span length of 260-feet. AECOM designed both steel plate girder and precast concrete segmental box girder alternates for bid. The steel alternate was low bid. Construction was completed in 2006.

Figure 6-4: Driscoll Bridge over the Raritan River, NJ

6.2.3 Replacement Option 3 – Precast Segmental Box Girder, Balanced Cantilever (250-feet typ and 400-feet channel)

Replacement Option 3 is considered a viable alternative because of the large size of the project. The size of a segmental project and cost are directly proportional. The initial setup costs stays the same (e.g. gantry, trusses, casting yard set up, etc.), but as the project increases in size the cost per segment (or cost of concrete) decreases. Economy of scale can be realized for this option, which makes it very competitive with the other options.

The span arrangement for this option is the same as for Replacement Option 2. Instead of using steel plate girders for the 250-foot long spans, precast segmental box girders built by the balanced cantilever method of construction are considered. The 250-foot long spans consist of a single box girder that is 12-feet deep. The expansion joint units have been set at 1,150-feet to 1,400-feet minimizing the number of expansion joints to ten. The spans are assumed to be built with an overhead erection gantry.

The 325-feet-400-feet-325-feet channel unit utilizes variable depth box girders; 12-feet at mid-span to 22-feet deep at the channel piers. These spans are assumed to be built in cantilever by erecting segments from barges.

Piers heights range from 14 to 105-feet tall above mean high water elevation. Precast box column sections are utilized. The 10-foot long sections are epoxied together and post-tensioned with a combination of bars and strands. Foundations consist of 72-inch concrete filled steel pipe piles and water line footings.

In accordance with VDOT's IIM-S&B-81.7 requirements, corrosion resistant reinforcing steel, Class III stainless, is used in the superstructure including in the box girders and barriers. Corrosion resistant reinforcing steel, Class I stainless, is used for all substructures in tidal waters. A deck overlay is not considered necessary because high performance low permeability concrete in combination with stainless reinforcing steel and post-tensioning is considered sufficient protection from corrosion, but an overlay is required by VDOT's IIM-S&B-91.

Refer to drawings 6-7 through 6-10 in Appendix B for details.

Figure 6-5 is an example of a bridge designed and built using precast segmental balanced cantilever construction. Designed by AECOM, the Roosevelt Bridge over the St. Lucie River is 4,565-feet long and is comprised of 260-foot long precast segmental box girder spans. Water line footings are used to reduce costs.

Figure 6-5: Roosevelt Bridge over the St. Lucie River, FL

6.2.4 Replacement Option 4 – Precast Segmental Box Girder, Balanced Cantilever Spans (400-feet typ)

This option with longer spans is considered in order to gain more data points on the cost optimization curve. The 325-feet - 400-feet - 325-feet channel unit with variable depth box girders, 12-feet at mid-span to 22-feet deep at the channel piers, is used for the entire length of the structure.

6.2.5 Replacement Option 5 – Steel Plate Girder Spans (400-feet typ)

Similar to Replacement Option 4 this option considered longer steel plate girder spans for the entire length of bridge based on the 140-inch constant depth web used for the channel unit in Replacement Options 1 and 2.

6.2.6 Replacement Option 6 – Extradosed Cable Stay (500-feet typ) and Prestressed Concrete Bulb T Girders (140-feet)

Extradosed cable stayed bridges are a structure type that have been used extensively in Japan and are beginning to become popular in the United States for span lengths in the range of 400 to 800-feet. AECOM has designed the only two extradosed cable stay bridges in the United States. These bridges filled a necessary gap between cost efficient span lengths for conventional superstructure types (e.g. steel plate girders and segmental concrete box girders) and cable stayed bridges. Steel plate girder and segmental concrete box girder bridges are cost effective for span lengths ranging from 180 to 400-feet. Conventional cable stayed bridges are cost effective for span lengths ranging from 800 to 1,200-feet. So the extradosed bridge type is more cost efficient for span lengths that fall between these span lengths (i.e. 400 to 800-feet).

The extradosed system is actually a hybrid technology that combines the structural aspects of a segmental concrete box girder bridge with cable stayed technology. The behavior of an extradosed bridge is similar to a conventional cable stayed bridge, with the exception that the deck is stiffer relative to the cable system and therefore carries a greater proportion of the load. The stress range in the stays under live loading is less than that in a conventional cable stayed bridge. The live load stresses approach what are typically seen in post-tensioned segmental concrete box girder bridges. Thus, cable fatigue is less of a concern with extradosed bridges.

The optimum girder height of an extradosed bridge is approximately 1/35 of the main span length, versus 1/18 of the span length for the section depth at the pier of a concrete girder bridge. Thus, an extradosed bridge will have less superstructure depth and is lighter than a concrete girder bridge. However, different than a girder bridge, stay towers must be constructed above the roadway for an extradosed bridge and cable stays installed. Required tower heights are approximately 1/10 of the main span length.

Replacement Option 6 considers 500-foot long extradosed cable stay spans and a 12-foot constant depth precast segmental box girder with a single plane of cable stays down the center. The extradosed cable stay spans are 7,340-feet long broken into five expansion joint units. There are six 140-foot long pile bent spans on the south approach and fifteen 140-foot long pile bent spans on the north approach. The 140-foot spans are comprised of 77-inch deep Bulb T girders spaced at 8'-3" center to center spacing.

Figure 6-6 is an example of an extradosed cable stayed bridge. Designed by IBT, the 2,890-feet long main bridge crossing the Hooghly River consists of seven 360-foot long spans. Expansion joints are located at mid span to simplify balanced cantilever construction. Steel beams placed inside the box girder provide moment resisting capacity for concrete creep and shrinkage redistribution effects and live loads.

Figure 6-6: Second Vivekananda Bridge, Kolkata, India

6.2.7 Replacement Option 7 – Prestressed Concrete Bulb T Girder Spans (150-feet) and Steel Channel Spans

Based on the costs associated with Replacement Options 1 through 6, the 150-foot long spans are evaluated for the entire length of bridge with the exception of the channel spans. The 150-foot spans are comprised of 85-inch deep Bulb T girders spaced at 8'-3" center to center spacing. The overall bridge length is 10,250-feet with 9,150-feet making up the 150-foot long Bulb T spans and 1,050-feet making up the steel channel spans.

6.2.8 Replacement Option 8 – Precast Segmental Box Girder, Span-By-Span (150-feet typ and 400-feet channel span)

Instead of using 85-inch deep Bulb T girders (Replacement Option 7), precast segmental box girders built by the span-by-span method of construction are considered. The 150-foot long spans consist of a single box girder that is 9-feet deep. The expansion joint units have been set to minimize the number of expansion joints to fifteen (15). The spans are assumed to be built with an underslung erection truss as depicted in Figure 6-7. Construction is simple, proceeds quickly, and is very economical.

Figure 6-7: Precast Segmental Box Girder Erection Truss

The 325-feet - 400-feet - 325-feet channel unit utilizes variable depth box girders, 12-feet at mid span to 22-feet deep at the channel piers. These spans are assumed to be built in cantilever by erecting segments from barges. Figure 6-8 is an example of a bridge designed and built using precast segmental balanced cantilever and span-by-span construction methods. Designed by Figg Engineers, the Victory Bridge over the Raritan River is 3,971-feet long and is comprised of 150-foot long precast segmental box girder spans for the approaches. The approach spans were built using the span-by-span method of construction. The channel spans consist of 330-feet - 440-feet - 330-feet precast segmental box girders built by the balanced cantilever method of construction.

Figure 6-8: Victory Bridge over the Raritan River, NJ

6.3 Foundation Considerations

The greatest potential to reduce construction costs is in the foundations. Therefore, a detailed investigation to determine viable foundation alternatives is necessary during the evaluation and selection of potential bridge replacement options for the Route 3 Bridge over the Rappahannock River. Based on the existing boring information, assumed soil parameters, and computed foundation loads, it is evident that large diameter, deep pile foundations are the most cost-effective solution. Based on a detailed review of the soil resistance characteristics and structural demand requirements, the following pile types and sizes are appropriate for the bridge replacement alternative study:

- 66-inch Diameter Concrete Cylinder Piles (6-inch wall thickness; pre/post-tensioned with SS strand)
- 72-inch Diameter Steel Pipe Piles (1 to 1 ¼-inch wall thickness; concrete filled in upper portion of pile)

In general, concrete cylinder piles are considered to be the least cost alternative in comparison to steel pipe piles. However, from a construction standpoint, there are practical limitations to the length and weight of cylinder piles that can be adequately handled and ultimately secured during driving. Based on our experience, contractors prefer steel pipe piles over concrete cylinder piles when the length of a cylinder pile exceeds 170-feet and/or its weight is greater than 100 tons. The size limitation is driven by the capability of large offshore cranes (i.e. 4100w barge mounted ringer crane) to handle and install piles, as well as the cost and complexity required for pile templates and driving leads needed during construction. For example, Figure 6-9 shows a photo of the pile driving operation for the Chesapeake Bay Bridge Tunnel project (circa 1973) which utilized 66-inch diameter concrete cylinder piles that were approximately 200-feet long. This photo illustrates the size and scale of the cranes, templates and leads required to set and drive these massive piles. In addition, Figure 6-10 shows a photo of the pile driving operation currently underway for the construction of the Bonner Bridge over Oregon Inlet, North Carolina. The foundation for the Bonner Bridge includes 54-inch diameter cylinder piles that are approximately 130-feet long. The contractor (PCL) noted that the size of the piling for this project pushed the practical limits for pile construction.

Figure 6-9: 66-inch Dia. Concrete Cylinder Pile Construction (Chesapeake Bay Bridge Tunnel Project)

Figure 6-10: 54-inch Dia. Concrete Cylinder Pile Construction (Bonner Bridge Replacement Project)

Currently, Dominion Power has already purchased 66-inch diameter concrete cylinder piles at 160-feet long from BCP-Skanska for the relocation of the overhead transmission lines currently attached to the Route 3 Bridge. They have also had BCP precast the waterline footing caps. It is our understanding that the cylinder piles used stainless steel reinforcing.

Therefore, concrete cylinder piles are only considered at proposed pier locations for the replacement bridge where the estimated pile length and corresponding weight are less than 170-feet and 100 tons, respectively. This is typically the case when shorter span configurations and low level multi-column bents are selected along the bridge approach spans.

For foundations greater than 170-feet, concrete filled steel pipe piles are recommended. Steel pile lengths range from 170 to 230-feet with wall thicknesses of 1.5-inches. Both Skanska and PCL recommended using steel piles for these lengths. PCL has also successfully driven 30-inch pressurized concrete screw piles that are 240-feet long on the Lake Underhill Project in Orlando, which required two splices (three pieces spliced together). These piles can be further investigated if total replacement becomes the preferred solution.

Figure 6-11 shows a photo of the pile driving operation for the Woodrow Wilson Bridge which utilized 72-inch diameter steel piles that were approximately 210-feet long.

Figure 6-11: Woodrow Wilson Bridge over the Potomac River, MD (72-inch Dia. Steel Pipe Piles)

Due to the anticipated cost implications for foundation construction, a preliminary foundation design is performed for each bridge replacement option. The design included a general review of the existing boring information, determination of "idealized" subsurface soil parameters, computations to determine strength load combinations for the piers, structural design review of the piles, and foundation stability evaluation.

As shown in Figure 6-12, nominal axial resistance charts are developed for various concrete cylinder and steel pipe pile sizes in order to estimate the required embedment depths and quantities for the pile foundations. The axial resistance of the soil is estimated by the Nordlund method using the computer program APILE. In order to minimize pile penetration depths and overall pile lengths, it is assumed that a pile test program (static load test and dynamic testing) is implemented prior to construction. Therefore, a phi factor of 0.80 is used to determine the factored nominal axial resistance of the piles.

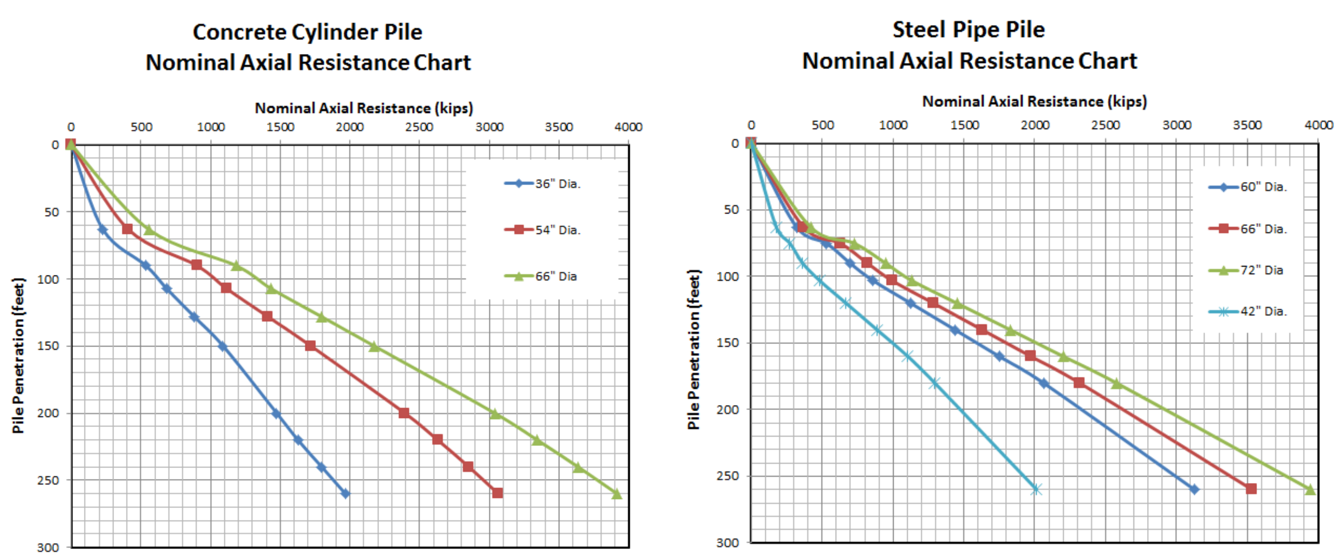


Figure 6-12: Idealized Nominal Axial Resistance Charts for Concrete Cylinder and Steel Pipe Piles

To ensure that the piles can be driven to the capacities required, computer program GRLWEAP is also used to evaluate and confirm the drivability (hammer energy and driving stresses). Nonlinear soil structure interaction models are developed using the computer program FB-MultiPier to evaluate the structural design of the piles and determine minimum pile penetration depths for lateral stability.

6.4 Conceptual Bridge Replacement Cost Comparisons

Conceptual quantities are developed for the eight options. Many items are based on past experience and historical values. For example the steel channel spans (325-feet - 400-feet - 325-feet) are based on 100 lb/sf of structural steel based on similar projects that ranged between 90 to 110 lb/sf. Similarly for the precast segmental box girder, balanced cantilever longitudinal post-tensioning is estimated to be 7.5 lb/sf based on previous experience.

Historical unit prices based on bid tabulations are used to develop construction costs. Mobilization is estimated separately, based on a percentage of the combined superstructure and substructure costs. A five percent value is utilized based on historical costs. A contingency of 15 percent is utilized for the cost estimates for each alternative. Based on the conceptual nature of the study, the contingency value is deemed warranted.

In summary, Table 6-1 provides a cost comparison for the eight options considered.

Table 6-1: Bridge Replacement Option Cost Comparisons ***(values expressed in millions)***

Option No.	Bridge Alternative	Structure Cost	Cost per SF	Superstructure Cost	Substructure Cost	Cost Ratio
7	Prestressed Concrete Bulb T Girder Spans (150') & Steel Plate Girder Channel Spans	168.4	397	59.6	108.8	1.00
1	Prestressed Concrete Bulb T Girder Spans (140' & 185') & Steel Channel Spans	170.8	399	58.4	112.4	1.01
8	Precast Segmental Box Girder, Span-By-Span (150' typ & 400' Channel Span)	178.0	419	63.1	114.9	1.06
3	Precast Segmental Box Girder, Balanced Cantilever Spans (250' typ and 400' Channel Span)	182.2	430	76.1	106.1	1.08
2	Steel Plate Girder Spans (250' typ and 400' Channel span)	199.3	470	91.4	107.9	1.18
6	Extradosed Cable Stay (500' typ) & Precast Concrete Bulb T Girder Spans (140')	218.0	512.9	87.5	130.5	1.29
4	Precast Segmental Box Girder, Balanced Cantilever Spans (400' typ)	226.9	534	89.4	137.5	1.35
5	Steel Plate Girder Spans (400' typ)	262.2	617	109.3	152.9	1.56

A review of the cost data indicates that the total replacement cost for the top five options ranges from $168.4 to $199.3 million. The remaining three options are considered less cost effective and can be ruled out.

Similar to the superstructure replacement alternatives, the cost reduction associated with the minimum required roadway width is further considered for evaluation by development of a supplemental option. Rather than provide the desirable roadway width, the minimum roadway width permissible by VDOT specifications is considered. Because Option 7 is the least cost indicated above, this supplemental alternative is similar but with the reduced roadway width.

Option 7A

Option 7A is similar to Option 7, where the cross section consists of two 12-foot lanes and two 4-foot shoulders for a 32-foot wide roadway. The span arrangement and superstructure girder types are the same as Option 7. See Drawings 6-11 and 6-12 for details. Some small cost savings are realized in the substructure due to the reduced roadway width, but minimal cost savings is realized because the substructure costs are driven by the significant pile depth necessary to resist lateral loads.

Table 6-2 presents a summary of the estimated cost for the supplemental full bridge replacement alternative that provides the minimum required roadway width. Detailed cost estimate data is included in Appendix C.

Table 6-2: Bridge Replacement Supplemental Option Cost Summary (values expressed in millions)

Component	Opt. 7	Opt. 7A
Superstructure	$60	$54
Substructure	$109	$98
Mobilization & Demo.	$22	$21
Contingency	$29	$26
Project Dev. & Admin.	$67	$61
Total Alternative Cost	$285	$258

Comparing the cost data between Options 7 and 7A indicates that reducing the deck width saves approximately $30M.

6.5 Conclusions Regarding Bridge Replacement

Based on the results of this preliminary evaluation, complete replacement of the Route 3 Bridge is a cost effective means to address the long term bridge rehabilitation needs. In addition, the complete replacement concept offers less impact to traffic during construction, when compared to the superstructure replacement alternatives. Several different structure configurations were considered and found to be of similar costs, so that further evaluation and design development is recommended to refine the bridge replacement recommendation. This further design development will also include more specific consideration of the location of the new crossing alignment and the associated right-of-way impacts. The objectives at this next stage of development are to present all pertinent information for review to VDOT so key decisions can be made prior to beginning the final design or proceeding with design build procurement.

7 Conclusions

The scope of this study included development and evaluation of potential alternative concepts for superstructure replacement, based on several criteria and considerations. The most significant of these criteria included the following:

- Completely replace all superstructure members,
- Maximize shoulder width on the replacement superstructure,
- Maximize reuse of the existing substructure elements with repairs and modifications as needed,
- Minimize construction of new foundation elements,
- Minimize duration of road closure,
- Minimize project costs.

Among a collection of superstructure replacement concepts considered, seven superstructure replacement alternatives are developed for detailed evaluation. Each of the alternatives is presented as a feasible and constructible means to completely replace the bridge superstructure. The other objectives are achieved to varying degrees among the alternatives. The impacts to natural resources are considered reasonably similar for comparison of alternatives. Alternative A proposes to reconstruct the bridge to a similar configuration as the existing bridge, and is included as a baseline for comparison with other alternatives.

This study includes evaluation of several alternatives for maximizing the shoulder width in the replacement superstructure. In order to provide a safety improvement, the study team established the criteria that any alternative which incorporates widening of the superstructure will provide for two 12-foot lanes and two 3-foot shoulders at a minimum, but 8-foot shoulders are desirable. Alternatives C, D, and E provide the desired widening for the entire length of bridge. Alternative B provides the desired widening for all but the channel span unit. Alternatives D1 and F provide the minimum roadway width required by VDOT for superstructure replacement projects. By comparison of the cost data presented in Table 7-1, the cost premium to provide the desired widening is approximately $30M.

The suitability of the existing substructure for reuse is critical to a bridge superstructure replacement project. Preliminary structural evaluation of the existing approach and channel span piers for the purposes of this study indicates that the existing piers may be reused with some strengthening and modifications. For the purpose of the study, it has been presumed that the material condition of the substructure to remain in service is adequate for the remaining service life. Alternatives C, E and F require the most extensive modifications, with channel span piers widened to support a wider superstructure. The condition of the existing beam and girder span piers is unfavorable for the proposed widened replacement superstructure. Alternatives B, C, D, E, D1 and F all include complete replacement of the piers under those spans.

Based on overall cost and the advantages afforded, Alternative D1 is the most cost effective alternative. This alternative requires reductions in the vertical navigation clearance, which has not yet been approved by the U.S. Coast Guard.

Due to the lack of an acceptable detour route, minimizing the impacts to traffic during construction presents the greatest challenge to the project cost and complexity. The scale of a superstructure replacement project and lack of adequate detour requires consideration of a rapid replacement construction method, of which several alternatives this study evaluates.

The Department's previous project to replace the superstructure of the U.S. Route 17 Bridge over York River (known as the George P. Coleman Memorial Bridge) in 1996 provides some perspective for rapid replacement schemes. The Coleman Bridge is a swing span bridge adjacent to the Yorktown National Park. With the substructure in good condition, replacement of the Coleman Bridge superstructure with another swing span configuration was chosen to minimize impacts on this adjacent asset and maintain access for naval and commercial marine traffic. The project scope was prepared to allow two 12-day road closures. The contractor eventually elected to float out sections of the old bridge and float in sections of the new bridge on construction barges.

Constructed in 1996, the Coleman Bridge superstructure replacement cost was approximately $73M, about $367 per square foot. At that time, a fixed a cable stay or variable depth box girder total replacement cost was approximately $110 per square foot, about one third the cost of the movable span superstructure replacement. Even a total replacement with new swing spans on a new alignment would have cost approximately $250 per square foot, suggesting that the innovative float in and float out construction sequence cost approximately 150 percent of a more conventional solution.

The costs summarized in Table 7-1 indicate that the use of rapid replacement construction methods increases the construction costs by a significant proportion. This is largely due to the unfavorable subsurface conditions and the high cost of the temporary foundation construction. In contrast to the Coleman Bridge, the Norris Bridge has no movable spans and it is approximately three times longer. The vehicular traffic volume on the Norris Bridge is much lower, there is no naval or significant commercial marine traffic on the Rappahannock River, and there seems to be no sensitive historical resources nearby the project site.

Table 7-1: Superstructure Replacement Alternative Cost Summary **(values expressed in millions)**

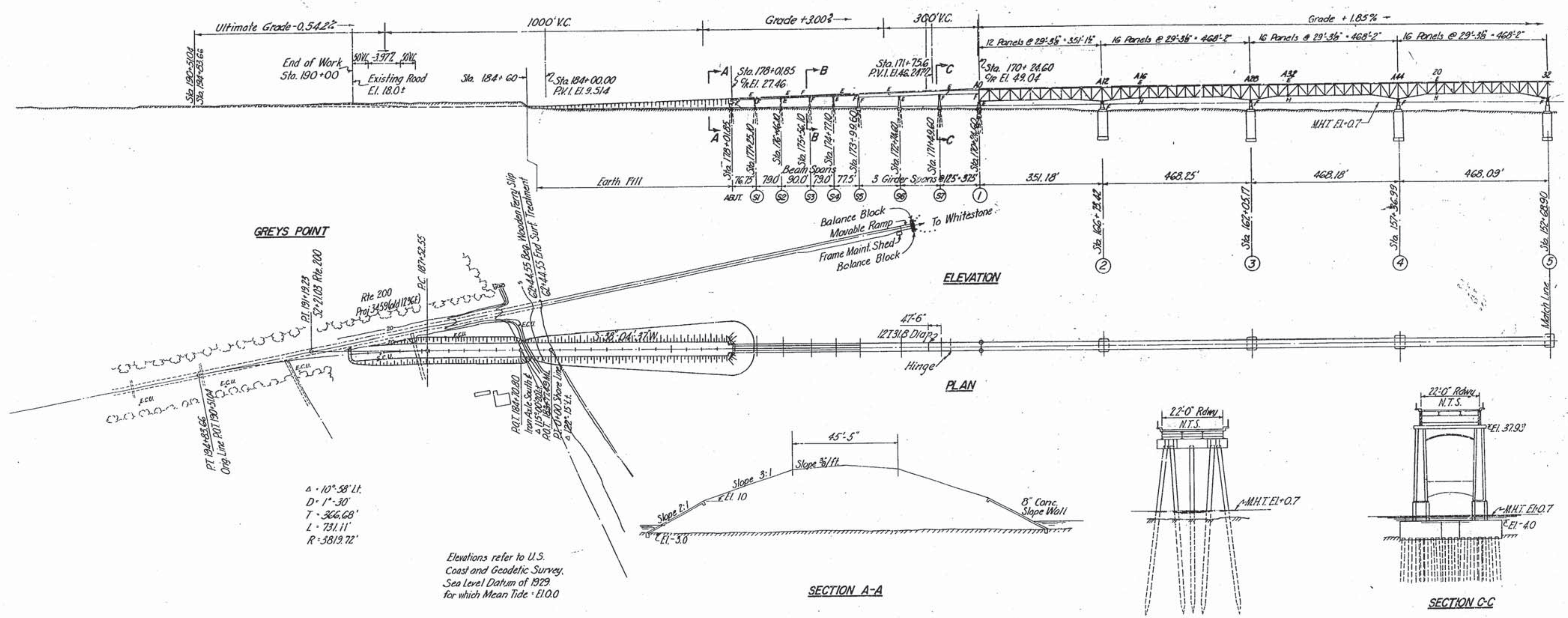
Component	Alt. A	Alt. B	Alt. C	Alt. D	Alt. E	Alt D1	Alt. F	Alt 7A
Superstructure	$61	$84	$92	$90	$118	$71	$108	$53
Substructure	$5	$20	$22	$19	$19	$19	$27	$98
Mobilization & Demo.	$14	$16	$16	$16	$17	$15	$17	$21
Temporary Works for Rapid Replacement	$148	$148	$148	$148	$148	$148	--	--
Contingency	$46	$53	$56	$55	$61	$51	$30	$26
Project Dev. & Admin.	$38	$51	$55	$53	$63	$46	$54	$60
Total Alternative Cost	$312	$371	$389	$381	$426	$349	$237	$258

Given the high priority to minimize impacts to traffic during construction, and the high cost of completing a superstructure replacement project with rapid replacement construction methods, it is evident that complete replacement of the bridge on a new alignment should also be evaluated for comparison with the superstructure replacement alternatives. For comparative purposes, several bridge replacement alternatives are developed and evaluated. As shown in Table 7-1, the most attractive of the complete replacement alternatives is estimated to be of comparable cost to the superstructure replacement alternatives, while offering better service life with less maintenance costs and minimal impacts to bridge users.

In conclusion**, Alternative 7A** for complete bridge replacement on a new upstream alignment results in a longer service life with less maintenance costs than the alternatives that reuse significant portions of the existing substructure with a new replacement superstructure. This alternative is also considered to offer the most optimal balance of costs and user impacts during construction.

Appendix A – Existing Bridge Plans

CONTRACT NO.	ROUTE NO.	PROJECT NO.	SHEET NO.	TOTAL SHEETS
2	200	3459-01	2	45
1	200	3459-01	2	16



NOTE: Dimensions marked N.T.S. are not to scale.

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

ROUTE 200 - PROJECT 3459

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

GENERAL PLAN & ELEVATION-A

150' 100' 50' 0 150'
SCALE IN FEET

RECOMMENDED: H. J. Engel
FOR MODJESKI & MASTERS

CONTRACT NOS. 1 & 2

Sheet 28 of 92
DRAWING NO. 2

MODJESKI & MASTERS, ENGINEERS

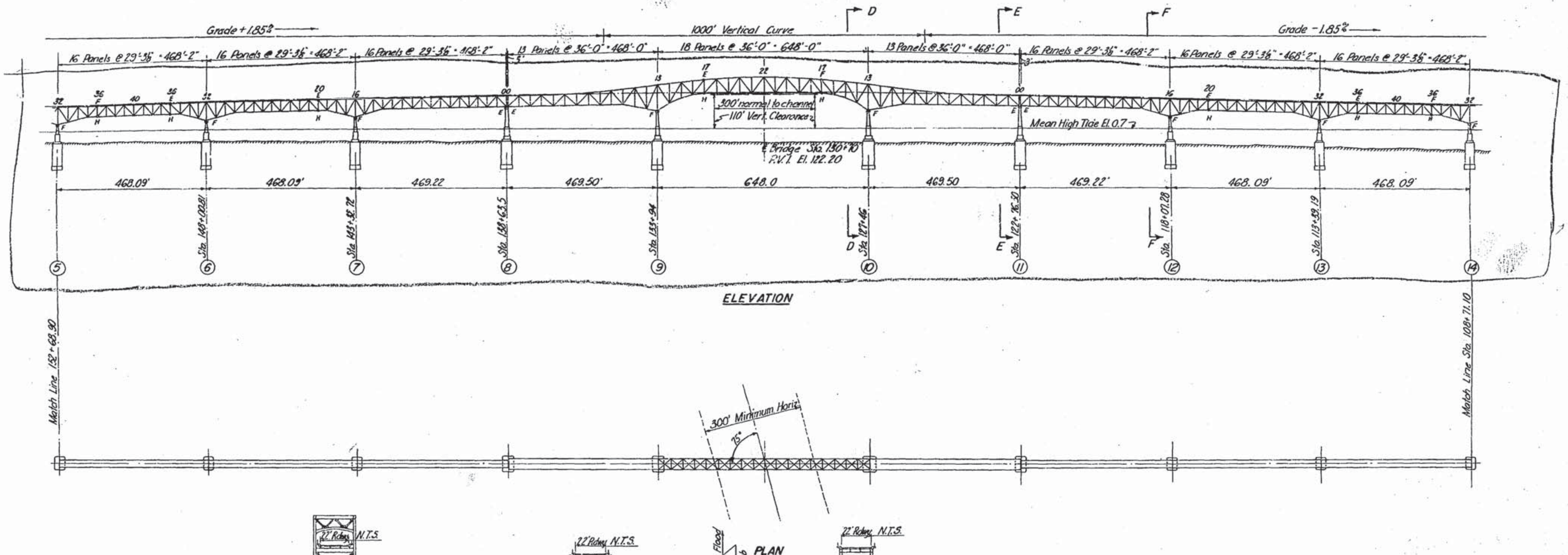
REV. JUNE, 1953 JANUARY, 1952

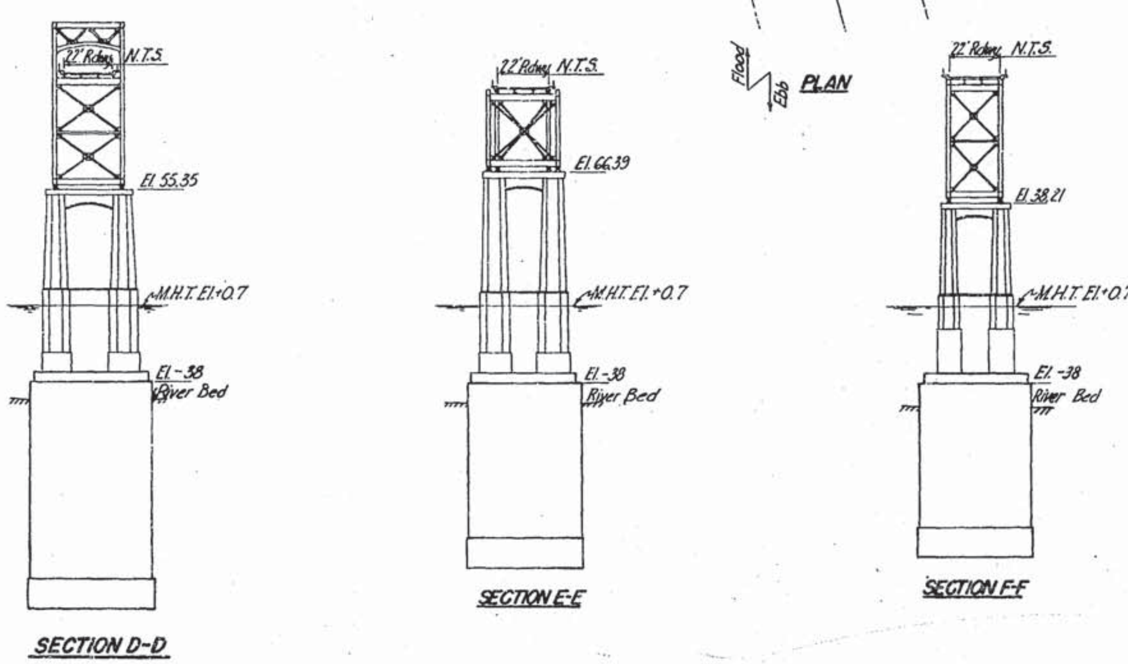
CXIX-25-B & C

DR. M.W.S. TR. M.W.S. CK. D.H.E.

119-25 028X

CONTRACT NO.	ROUTE NO.	PROJECT NO.	SHEET NO.	TOTAL SHEETS
2	200	3459-C/3		45
1	200	3459-C/3		16





NOTE:
Elevations refer to U.S. Coast and Geodetic Survey, Sea Level Datum of 1929 for which Mean Tide = El. 0.0
Dimensions marked N.T.S. are not to scale.

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

ROUTE 200 - PROJECT 3459

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

GENERAL PLAN & ELEVATION-B

SCALE IN FEET

RECOMMENDED: H. J. Engel
FOR MODJESKI & MASTERS

CONTRACT NOS. 1 & 2
MODJESKI & MASTERS, ENGINEERS

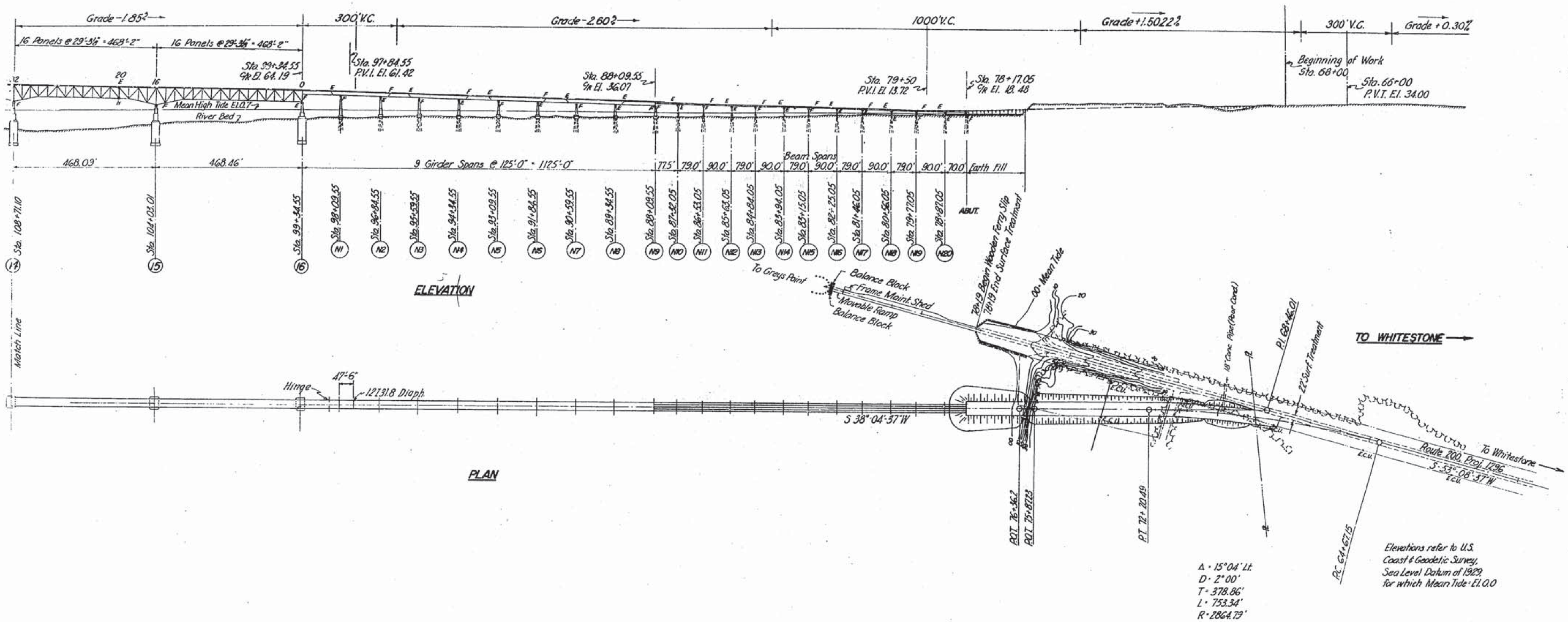
Sheet 27 of 92
DRAWING NO. 3

REV. JUNE, 1953 JANUARY, 1952

CXIX-25-B&C

DR. M.W.S. TR. M.W.S. CK. D.H.E.

CONTRACT NO.	ROUTE NO.	PROJECT NO.	SHEET NO.	TOTAL SHEETS
2	200	3459-01	4	45
1	200	3459-01	4	16



COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

ROUTE 200 - PROJECT 3459

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

GENERAL PLAN & ELEVATION-C

150' 100' 50' 0 150'
SCALE IN FEET

RECOMMENDED: H. J. Engel
FOR MODJESKI & MASTERS

CONTRACT NOS. 1 & 2

Sheet 30 of 99
DRAWING NO. 4

MODJESKI & MASTERS, ENGINEERS

REV. JUNE, 1953 JANUARY, 1952

CXIX-25-B & C

DR. M.W.S. TR. M.W.S. CK. D.H.E.

19-250-030 X

BORING NO. B2 – PIER 2
Sta. 166+72.42

Casing 3½" I.D., Spoon 3" O.D., 300# Hammer, 18" Drop samp.-24" cas.

Classification	Stratum Elev.	Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
Soft gray wet silt and shells	-12.5	15.5	1	Push	0	
		20.5	2	Push	0	
Soft gray silt; trace of shells	-24.0	25.5	3	Push	0	
		30.5	4	Push	0	
		35.5	5	Push	32	
		40.5	6	Push	11	72% 97.5
		45.5	7	Push	27	
		50.5	8	Push	12	
		55.5	9	Push	25	
Medium gray silt; trace of vegetation	-58.0	60.5	10	Push	34	
		65.5	11	Push	30	
Medium gray silt; traces of vegetation and fine sand lenses	-69.0	70.5	12	2	25	
		75.5	13	Push	30	
		80.5	14	4	37	
Medium light gray silt with fine lenses of sand	-83.0	85.5	15	Push	36	
		90.5	16	2	39	
		95.5	17	3	36	82.2% 95.7
		100.5	18	4	109	
		105.5	19	4	48	
Fine medium gray sand; trace of silt and pea gravel; some organ. mtl.	-109.0	110.5	20	6	49	
		115.5	21	32	31	
Coarse white sand; pea gravel-¼" max.	-119.0	120.5	22	25	38	
		125.5	23	21	68	
Fine medium green sand	-128.0	130.5	24	19	112	
Med. to coarse brown sand; lumps of bl. wh. silt	-133.0	135.5	25	24	189	
Medium to coarse white sand	-138.0	140.5	25	15	123	
Stiff green clay; fine sand; trace of shells	-143.0	145.5	27_{T1}	12	231	30.0% 121.2 (25%Sd)
Fine green clayey sand	-148.0	150.5	28	18	109	
Very fine green silty sand	-153.0	155.5	29_{T2}	13	69	25.75% 122.[illegible] (86%Sd)
	-155.5					

Sample Data-Boring B2

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
27_{T1}	3.16	3.24
29_{T2}	0.493	10.96

BORING NO. B3 – PIER 3
Sta. 162+05.17

Casing 3½" I.D., Spoon 3" O.D., 300# Hammer, 18" Drop samp.-24" cas.

Classification	Stratum Elev.	Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
Soft gray silt; river mud with some organ. mtl.	-27.5	34.5	1	Push	0	
		39.5	2	Push	0	
		44.5	3	Push	0	
		49.5	4	Push	0	
		54.5	5	Push	0	
		59.5	6	Push	3	
		64.5	7	Push	4	
		69.5	8	Push	8	
		74.5	9	Push	12	
Gray stiff sand with some organic material	-77.5	79.5	10	Push	14	74% 98.0
		84.5	11	Push	16	
		89.5	12	Push	17	
		94.5	13	Push	17	
Brown and gray silt; layers of fine gray sand	-97.5	99.5	14	2	24	
		104.5	15	10	38	34.4% 117.0
Brown and gray silty sand	-108.5	110.5	16	10	45	
		115.5	17	8	47	
Green silty sand with gray silt; organ. mtl.	-118.5	120.5	18	9	53	
		125.5	19	9	62	
		130.5	20	10	57	
Stiff green clay; some sand	-132.5	135.5	21_{T1}	13	78	32.8% 114.5 (24.9%Sd)
		140.5	22	12	91	
Very fine green clayey sand pkts. of silt, sand and shells	-143.5	145.5	23_{T2}	13	101	36.61% 112.5 (33%Sd)
Fine green silty sand	-148.5	150.5	24	9	118	
Very fine green clayey sand with pockets of sand; many small shells	-153.5	155.5	25_{T3}	10	130	25.3% 121.2 (73.5%Sd)
		160.5	26	13	142	
		165.5	27	12	154	
Stiff green clay; trace of shells	-168.5	170.0	28	13	162	
	-170.5					

Sample Data-Boring B3

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
21_{T1}	1.29	3.14
23_{T2}	—	0.488
25_{T3}	.147	.743

BORING NO. B4 – PIER 4
Sta. 157+36.99

Casing 3½" I.D., Spoon 3" O.D., 300# Hammer, 18" Drop samp.-24" cas.

Ref. Elev. -10

Classification	Stratum Elev.	Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
Soft gray silt	-31.2	35.5	1	Push	0	
		40.5	2	Push	0	
		45.5	3	Push	0	
		50.5	4	Push	0	
		55.5	5	Push	0	
		60.5	6	Push	0	
		65.5	7	Push	0	
		70.5	8	Push	0	103.1% 90.5
		75.5	9	Push	0	
		80.5	10	Push	13	
Soft gray silt; decayed peat; veg.	-84.0	85.5	11	Push	10	
Soft gray silt	-87.0	90.5	12	Push	24	
Very fine gray-brown sandy silt; decay. veg.	-94.0	95.5	13	11	34	
Soft brown silt; peat; traces of fine gray sand	-99.0	100.5	14	4	43	
Very fine green silty sand	-104.0	105.5	15	8	35	24.8% 126.0
		110.5	16	7	45	
		115.5	17	5	37	
		120.5	18	6	53	
		125.5	19	6	51	
		130.5	20	11	58	
Medium coarse light gray sand	-135.0	135.5	21	15	64	
Green silty clay	-139.0	140.5	22_{T1}	10	221	30.6% 118.3 (23.5%Sd)
Very fine green sandy silt; pockets of clay	-144.0	145.5	23	8	126	
		150.5	24_{T2}	11	101	24.7% 124.5 (77%Sd)
		155.5	25	10	79	
Stiff green clay; traces of fine shells	-159.0	160.5	26_{T3}	12	73	26.2% 122.3 (37.5%Sd)
		165.5	27	19	82	
		170.5	28	29	101	
	-171.0					

BORING NO. B5 – PIER 5
Sta. 152+68.90

Casing 3½" I.D., Spoon 3" O.D., 300# Hammer, 18" Drop samp.-24" cas.

Classification	Stratum Elev.	Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
Soft gray silt	-32.4	38.5	1	Push	0	
		43.5	2	Push	0	
		48.5	3	Push	0	
		53.5	4	Push	0	
		58.5	5	Push	0	
		63.5	6	Push	0	67.4% 100.0
		68.5	7	Push	3	
		73.5	8	Push	4	
		78.5	9	Push	6	
		83.5	10	Push	13	
Gray sand and silt; silty sand with decay. organic vegetation	-85.4	88.5	11	2	28	
		93.5	12	3	35	
		98.5	13	1	32	
Very fine green silty sand	-101.4	104.5	14	12	30	
		110.5	15	11	30	26.7% 124.5
Very fine blue-green sand; much silt	-113.4	115.5	16	13	40	
		120.5	17	12	53	
Fine green silty sand	-123.4	125.5	18	13	56	
		130.5	19	17	73	
Medium to coarse white sand	-134.4	135.5	20	Jetted	149	
		140.5	21	19	204	
Stiff green sandy clay; trace of shells	-142.4	145.5	22_{T1}	13	154	26.2% 124.6 (56%Sd)
Very fine green silty sand	-148.4	150.5	23	23	182	
Green clayey fine sand with shells and pockets of sand	-153.4	155.5	24_{T2}	17	154	25.9% 124.0 (72.4%Sd)
		161.5	25	28	194	
Stiff green clay; small shells; sand	-164.4	166.9	26_{T3}	18	140	29.8% 122.7 (49.0%Sd)
	-167.4					

BORING NO. B6 – PIER 6
Sta. 148+00.81

Casing 3½" I.D., Spoon 3" O.D., 300# Hammer, 18" Drop samp.-24" cas.

Classification	Stratum Elev.	Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
Soft gray silt	-34.6	40.5	1	Push	0	
		45.5	2	Push	0	
		50.5	3	Push	0	
		55.5	4	Push	0	
		60.5	5	Push	0	
		65.5	6	Push	0	
		70.5	7	Push	0	
		75.5	8	Push	6	
		80.5	9	Push	7	
Soft gray silt with very fine lenses of sand	-83.0	85.5	10	Push	5	
		90.5	11	2	8	57.3% 104.0
		95.5	12	Push	13	
		100.5	13	Push	13	
		105.5	14	Push	10	
		110.5	15	Push	12	
		115.5	16	Push	20	
		120.5	17	1	29	
Very fine gray silty sand	-123.6	125.5	18	6	42	28.3% 123.0
		130.5	19	23	36	
Fine to medium coarse sand; gravel lenses	-132.0	135.5	20A 20B	17	51	
Green silty clay	-13[illegible].5	140.5	21_{T1}	13	264	31.7% 120.2 (34%Sd)
Very fine green silty sand; traces of fine shells	-143.0	145.5	22	8	70	
		150.5	23_{T2}	11	54	
		155.5	24	10	75	
Stiff green clay; trace of small shells	-158.0	160.5	25_{T3}	12	102	30.4% 118.4 (23.5%Sd)
Stiff green clay	-164.0	165.5	26	33	90	
Fine green silty sand; large shells	-168.0	170.5	27	30	111	
		174.5	28	32	173	
	-175.0					

CONSOLIDATION IN PERCENT & COEFFICIENT OF CONSOLIDATION (SQ. CM/SEC) × 10^{-3}

Hole	Sample	Pressure, Tons per sq. ft. 0	0.5	1	2	4	8	2	0.5	
B2	27_{T1}	0	2.51%	4.09%	6.24%	9.08%	13.47%	11.43%	8.88%	← % Consol.
			2.25	1.92	3.75	3.87	1.98			← Coef. Consol. $\times 10^{-3}$
	29_{T2}	0	2.56%	3.31%	4.36%	5.55%	7.12%	6.44%	6.04%	
			1.35	8.20	5.49	8.13	1.27			
B3	21_{T1}	0	0.87%	2.22%	4.21%	7.17%	11.32%	8.79%	5.93%	
			1.01	.493	.493	.414	.375			
	23_{T2}	0	7.21%	10.37%	14.85%	20.26%	25.71%	23.85%	21.27%	
			.354	—	.148	.168	.156			
	25_{T3}	0	3.31%	4.44%	5.67%	7.16%	8.95%	8.26%	7.86%	
			1.146	—	2.87	2.43				
B4	22_{T1}	0	5.02%	7.46%	9.54%	12.93%	18.17%	14.52%	11.19%	
			0.79	0.62	0.63	0.82	0.66			
	24_{T2}	0	4.10%	4.84%	6.09%	7.54%	9.70%	8.98%	8.34%	
			0.53	0.71	1.18	3.49	1.63			
	26_{T3}	0	3.21%	4.57%	6.58%	8.53%	11.54%	10.67%	8.53%	
			1.29	1.53	1.59	2.20	2.26			
B5	22_{T1}	0	2.70%	5.08%	9.62%	16.98%	25.8[illegible]%	21.91%	19.31%	
			3.87	2.72	1.48	1.72	1.5			
	24_{T2}	0	2.62%	3.68%	5.06%	6.71%	8.76%	8.09%	7.60%	
			4.62	3.53	5.50	2.99	6.82			
	26_{T3}	0	3.35%	5.01%	7.50%	10.97%	15.99%	12.27%	9.41%	
			.90	.83	.43	.57	.50			
B6	21_{T1}	0	2.61%	4.52%	7.27%	11.53%	17.55%	13.81%	10.44%	
			1.27	.55	.82	.30	.34			
	23_{T2}	0								
	25_{T3}	0	4.08%	6.12%	9.00%	13.03%	18.63%	15.12%	11.70%	
			.24	.27	.29	.31	.31			

Sample Data-Boring B4

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
22_{T1}	1.63	2.68
24_{T2}	0.77	4.9% under 1.64
26_{T3}	2.23	3.09

Sample Data-Boring B5

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
22_{T1}	0.76	1.30
24_{T2}	0.30	0.60
26_{T3}	4.71	—

Sample Data-Boring B6

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
21_{T1}	1.21	6.69
23_{T2}		
25_{T3}	1.52	8.07

NOTES

Borings were made in April, 1953 by Warren George of New York City. Boring work was supervised by Virginia Department of Highways. Soil tests suggested by Modjeski and Masters were made by the University of Virginia.

Percent of sand is shown beneath Moisture Percentage & Weight of Sample, where Tube Samples designated T1, T2 etc. were taken, thus: 30.0% 121.2 (25% Sd.)

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

SUPPLEMENTARY BORINGS B2 TO B6

10' 8' 6' 4' 2' 0 10'
SCALE IN FEET

MODJESKI & MASTERS ENGINEERS | OCTOBER, 1953 | SUPPLEMENTARY DRAWING NO. 8

Sheet 81 of 92
CXIX-25-D

DR. Tam | TR. | CK. H.J.L.

Contract I

19-25-081X

BORING NO. B7-PIER 7
Sta. 143+32.72

Casing $3\frac{1}{2}$" I.D.
Spoon $3\frac{1}{2}$" O.D.
300# Hammer
18" Drop samp.-24"cas.

Classification	Stratum Elev.
Soft gray silt; some green sand in pockets	-37.5 to -95.5
Medium gray silt with many sand lenses	-95.5 to -106.5
Fine gray very silty sand with pockets of silt	-106.5 to -122.5
Fine brown and gray silt; traces of organ. mtl.	-122.5 to -126.5
Medium greenish-gray silty sand with silt	-126.5 to -131.5
Medium coarse white running sand	-131.5 to -139.5
Very fine gray-green silty sand with some shells	-139.5 to -147.5
Stiff green clay; some sand pockets and fine shells	-147.5 to -163.5
Stiff green clayey sand with lots of shells	-163.5 to -176.5

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs./cu.ft.)
47.5	1	Push	0	
47.5	2	Push	0	
52.5	3	Push	0	
57.5	4	Push	0	
62.5	5	Push	0	
67.5	6	Push	5	104.8% 90.2
72.5	7	Push	15	
77.5	8	Push	6	
82.5	9	Push	7	
87.5	10	Push	12	
92.5	11	Push	22	
97.5	12	1	18	
103.5	13	1	28	
108.5	14	6	42	
114.5	15	7	46	
119.5	16	5	38	30.5% 121.0
124.5	17	6	59	
130.5	18A 18B	14	140	
135.5	19	Jetted	336	
140.5	20_{T1}	5	170	27.88% 122.0 (84%Sd)
145.5	21	9	103	
150.5	22_{T2}	12	121	30.06% (16%Sd)
155.5	23	9	120	
160.5	24_{T3}	15	12[illegible]	39.4% 115.7 (10.8%Sd)
165.5	25	15	131	
170.5	26	17	174	
176.0	27	18	162	

BORING NO. B8-PIER 8
Sta. 138+63.50

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24"cas.

Classification	Stratum Elev.
Soft gray silt	-40.1 to -97.0
Fine gray sand with layers of silt	-97.0 to -108.0
Very fine gray silty sand	-108.0 to -113.0
Soft brown organic silt; veg.; peat	-113.0 to -118.0
Very fine gray silty sand	-118.0 to -128.0
Coarse white sand; gravel - $\frac{3}{4}$" max.	-128.0 to -130.7
Very fine green clayey sandy silt	-130.7 to -144.0
Very fine green silty sand; shells	-144.0 to -158.0
Very fine clayey silty sand; some shells	-158.0 to -164.0
Stiff green clay; some fine sand; pkts. of sand	-164.0 to -173.0
Fine green silty sand; large shells	-173.0 to -175.0

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs./cu.ft.)
45.5	1	Push	0	
50.5	2	Push	0	
55.5	3	Push	0	
60.5	4	Push	0	
65.5	5	Push	0	
70.5	6	Push	0	
75.5	7	Push	0	
80.5	8	Push	0	
85.5	9	Push	6	50.2% 103.0
90.5	10	Push	0	
95.5	11	Push	16	
100.5	12	2	60	
105.5	13	10	41	
110.5	14	5	26	36.0% 116.5
115.5	15	4	27	
120.5	16	9	29	
125.5	17	12	88	
130.5	18A 18B	15	80	
135.5	19_{T1}	7	182	26.1% 120.7 (61%Sd)
140.5	20	8	162	
145.5	21_{T2}	15	186	24.0% 122.9 (84%Sd)
150.5	22	13	98	
155.5	23_{T3}	10	125	23.7% 124.8 (56.1%Sd)
160.5	24	9	128	
165.5	25_{T4}	17	65	36.44% 115.85 (10%Sd)
170.5	26_{T5}	15	57	30% 116.9 (29%Sd)
174.5	27	32	45	

BORING NO. B9-PIER 9
Sta. 133+94.00

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24"cas.

Ref. Elev. -20

Classification	Stratum Elev.
Soft gray silt	-42.0 to -91.0
Fine to medium gray silty sand	-91.0 to -113.0
Brown and gray soft silt; organ. mtl.	-113.0 to -117.0
Coarse green sand; small gravel	-117.0 to -121.0
Fine gray silty sand; streaks of br. bl. grn. color	-121.0 to -12[illegible]
Green silty clay with some sand	-12[illegible] to -138.0
Soft fine green silty sand; some shells	-138.0 to -148.0
Stiff green clay with variable amts. of sand	-148.0 to -173.0
Soft green silty sand with many shells	-173.0 to -180.0

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs./cu.ft.)
47.5	1	Push	0	
52.5	2	Push	0	
57.5	3	Push	0	
62.5	4	Push	0	
67.5	5	Push	0	45.8% 108.6
72.5	6	Push	0	
78.5	7	Push	4	
83.5	8	Push	4	
88.5	9	Push	7	
93.5	10	6	14	
98.5	11	7	18	
103.5	12	9	19	
108.5	13	11	21	29.6% 121.4
113.5	14	3	28	
118.5	15	21	36	
124.5	16	17	40	
130.5	17_{T1}	11	45	35.9% 115.1 (38%Sd)
135.5	18	11	87	
140.5	19_{T2}	7	72	27.1% 124.9 (76%Sd)
145.5	20	9	75	
150.5	21_{T3}	12	94	28.2% 121.4 (38.6%Sd)
155.5	22	12	101	
160.5	23	13	130	
165.5	24	13	121	
170.5	25	12	141	
175.5	26	13	142	
179.5	27	14	179	

BORING NO. B10-PIER 10
Sta. 127+46.00

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24"cas.

Classification	Stratum Elev.
Soft gray silt	-45.8 to -90.8
Spongy gray and brown silt with variable amts. of organic mtl.	-90.8 to -118.3
Fine to medium gray sand; trace of silt	-118.3 to -125.5
Fine gray sandy silt; possibly clay	-125.5 to -130.0
Stiff green clay with variable amts. of shells	-130.0 to -137.8
Green sandy clay with few shells	-137.8 to -140.8
Green very silty very fine sand; large shells	-140.8 to -147.[illegible]
Green spongy sandy silt with shells	-147.[illegible] to -152.[illegible]
Stiff green clay with variable amounts of sand and shells	-152.[illegible] to -169.8
Fine greenish-gray sand; many shells; trace of silt	-169.8 to -179.3

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs./cu.ft.)
50.5	1	Push	0	
55.5	2	Push	0	
60.5	3	Push	0	
65.5	4	Push	0	55.4% 105.2
70.5	5	Push	0	
75.5	6	Push	0	
80.5	7	Push	0	
85.5	8	Push	10	
90.5	9	Push	14	60.9% 102.6
95.5	10	1	8	
100.5	11	2	18	
105.5	12	2	14	
110.5	13	3	30	
115.5	14	4	46	
120.5	15	20	43	
125.5	16A 16B	5	45	
130.5	17_{T1}	13	5[illegible]	40.2% (21%Sd)
135.5	18	10	40	
140.5	19_{T2}	12	90	29.6% 120.6 (31%Sd)
145.5	20	10	69	
150.5	21_{T3}	6	71	26.0% 124.7 (64%Sd)
155.5	22	13	68	
160.5	23	12	92	
165.5	24	14	95	
170.5	25	24	124	
175.5	26	27	130	
179.5	27	32	172	

BORING NO. B11-PIER 11
Sta. 122+76.50

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24"cas.

Classification	Stratum Elev.
Very soft gray silt	-48.4 to -90.0
Fine to very fine gray silty sand	-90.0 to -101.0
Soft brown-gray silt; traces of organ. veg.	-101.0 to -112.0
Soft silty clayey sand; bl.-grn. color	-112.0 to -116.0
Fine grayish-green sand	-116.0 to -123.0
Very fine green silty sand; some clay	-123.0 to -128.0
Very fine green silty clay	-128.0 to -144.0
Very fine green silty sand	-144.0 to -149.0
Very fine spongy green silty sand; shells	-149.0 to -153.0
Very fine green sandy silt; shells	-153.0 to -159.0
Stiff green clay	-159.0 to -169.0
Fine-medium green silty sand; shells	-169.0 to -180.4

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs./cu.ft.)
53.5	1	Push	0	
58.5	2	Push	0	
63.5	3	Push	0	
68.5	4	Push	0	
73.5	5	Push	0	
78.5	6	Push	0	
83.5	7	Push	0	
88.5	8	Push	8	
93.5	9	30	15	
98.5	10	26	26	
103.5	11	4	21	
108.5	12	4	35	
113.5	13	5	40	27.6% 124.0
118.5	14	8	49	
124.5	15	17	57	
130.5	16_{T1}	18	[illegible]	31.3% 118.7 (50%Sd)
135.5	17	9	183	
140.5	18_{T2}	9	173	29.8% 121.8
145.5	19	14	[illegible]	
150.[illegible]	20_{T3}	9	175	26.3% 121.8 (71.3%Sd)
155.5	21	14	[illegible]	
160.5	22_{T4}	14	305	33.5% 117.8 (33%Sd)
165.[illegible]	[illegible]	15	321	
170.5	24	[illegible]	305	
175.5	25	[illegible]	32[illegible]	
179.[illegible]	[illegible]	21	[illegible]	

CONSOLIDATION IN FERCENT & COEFFICIENT OF CONSOLIDATION (SQ. CM/SEC.) x10[-3]

Upper line of each sample: % Consol.; lower line: Coef. Consol. $\times 10^{-3}$

Hole	Sample	0	0.5	1	2	4	8	2	0.5
		Pressure, Tons per sq.ft.							
B7	20_{T1}	0	1.61%	2.62%	3.97%	5.56%	7.46%	6.75%	6.09%
			9.16	4.46	9.34	13.74	7.40	—	—
	22_{T2}				No Consolidation Test				
					"	"		"	
	24_{T3}	0	0.87%	1.65%	3.30%	6.46%	9.97%	7.71%	5.30%
			1.64	1.07	.44	.34	.41		
B8	19_{T1}	0	7.12%	9.02%	11.60%	14.90%	20.89%	17.17%	14.20%
			1.05	.91	1.19	.63	.89		
	21_{T2}	0	2.14%	2.74%	3.69%	4.87%	6.18%	5.82%	5.55%
		Sample too sandy to obtain Coeff. of Consol.)							
	23_{T3}	0	2.55%	3.62%	5.02%	6.84%	9.03%	8.10%	7.44%
			3.36	4.32	5.41	6.10	9.56		
	25_{T4}	0	2.0%	3.71%	6.31%	10.12%	15.24%	10.92%	7.21%
			0.47	.31	.31	.38	.28		
	26_{T5}	0	2.49%	4.05%	6.29%	9.33%	13.30%	11.28%	8.61%
			0.74	.33	.31	.37	.35		
B9	17_{T1}	0	0.68%	1.56%	3.37%	6.28%	11.00%	8.53%	5.24%
			—	.78	.41	.34	.22		
	19_{T2}	0	2.70%	3.89%	5.65%	8.11%	12.00%	9.44%	8.37%
		0	19.8	4.54	3.27	4.05	9.88		
	21_{T3}	0	1.35%	2.85%	5.19%	8.67%	13.80%	9.78%	6.21%
			0.73	0.39	0.42	0.44	0.40		
B10	17_{T1}		No Consolidation Test						
			"		"		"		
	19_{T2}	0	2.25%	4.69%	8.61%	14.47%	22.78%	18.11%	15.61%
			0.36	.178	.154	.121	.103		
	21_{T3}	0	2.90%	4.55%	6.51%	9.47%	13.60%	10.86%	8.89%
		-	1.37	0.94	1.39	2.00	3.32		

Sample Data-Boring B7

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
20_{T1}	0.54	1.136
22_{T2}	—	3.47
24_{T3}	1.46	~~0.39~~

Sample Data-Boring B8

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
19_{T1}	1.40	3.0
21_{T2}	0.54	12.9
23_{T3}	0.29	0.65
25_{T4}	4.83	1.85
26_{T5}	1.13	2.95

Sample Data-Boring B9

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
17_{T1}	4.63	2.34
19_{T2}	0.34	1.01
21_{T3}	3.6	10.16

Sample Data-Boring B10

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
17_{T1}	1.28	2.0
19_{T2}	1.26	~~9.13~~
21_{T3}	1.37	4.28

Sample Data-Boring B11

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviator Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
16_{T1}	0.778	1.74
18_{T2}	0.98	2.56
20_{T3}	0.42	0.73
22_{T4}	1.426	2.412

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

SUPPLEMENTARY BORINGS B7 TO B11

10' 8' 6' 4' 2' 0 10'
SCALE IN FEET

Sheet 12 of 92

MODJESKI & MASTERS
ENGINEERS

OCTOBER, 1953

SUPPLEMENTARY DRAWING No. 9

CXIX-25-D

DR. TOY | TR. | CK. H.J.E.

Contract 1

1 9-2 5 L 0 8 2 X

BORING NO. B12 - PIER 12

Sta. 116+07.28

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24" cas.

Classification	Stratum Elev.
	-48.0
Soft gray silt; river mud	-80.0
Soft gray silt with lenses of very fine gray sand	-98.0
Fine-med. gray sand; traces of silt	-103.0
Coarse gray sand; silt	-104.0
Soft gry. silt; organ. veg.	-105.5
Fine-med gray sand	-108.0
Soft gray silt; traces of organic vegetation	-118.0
Very fine green-gray silty clay and sand	-123.0
Fine to coarse gray sand	-129.0
Green silty clay; trace of very fine sand	-148.0
Very fine green silty sand; shells	-155.0
Very fine green silty sand; clay with traces of shells; pkts. fine sand	-158.0
Stiff green silty clay	-170.5
Stiff grn. clay; shells	-172.0
Fine to med. green silty sand; shells	-175.0

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
53.5	1	Push	0	
58.5	2	Push	0	
63.5	3	Push	0	
68.5	4	Push	0	
73.5	5	Push	0	107.5% 89.8
78.5	6	Push	0	
85.5	7	Push	10	
90.5	8	Push	11	86.7% 94.3
95.5	9	Push	12	
100.5	10	17	23	
105.5	11A 11B	4	37	
110.5	12	Push	37	
115.5	13	4	48	
120.5	14	8	50	
125.5	15	13	70	
130.5	16_{T1}	10	116	31% 115.3 (62% Sd)
135.5	17	9	167	
140.5	18_{T2}	9	198	28.6% 119.9 (52.5% Sd)
145.5	19	9	206	
150.5	20_{T3}	32	387	20.8% 127.5 (82% Sd)
155.5	21	9	239	
160.5	22	13	249	
165.5	23	14	316	
170.5	24A 24B	9	305	
174.5	25	35	313	

BORING NO. B13 - PIER 13

Sta. 113+39.19

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24" cas.

Classification	Stratum Elev.
	-52.0
Very soft gray silt; river mud	-79.0
Coarse gray sand	-81.5
Medium-fine gray sand; river mud	-84.0
Soft gray silt	-86.5
Coarse gray sand; trace of silt	-89.0
Soft gray silt; very fine lenses of sand	-94.0
Coarse gray sand	-104.0
Soft fine gray silt; sand; traces of organ. vegetation	-109.0
Very soft light gray silt and sand	-115.0
Brn.-gry. silt-sand; decay. organ. veg.	-116.5
Fine gry. sand-silt; traces of organ. veg.	-120.0
Green silty clay; some very fine sand	-144.0
Very fine grn. silty sand	-149.0
Very fine grn. silty sand-silt; med. lrg. shells	-153.0
Very fine grn. silty clay tr. of fine shells	-159.0
Stiff green clay; pkts. of fine sand	-171.0
Med. grn. silty sand; large shells	-173.0

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
61.5	1	Push	0	
66.5	2	Push	0	
71.5	3	Push	0	97.0% 92.0
76.5	4	Push	0	
81.5	5A 5B	15	6	
86.5	6A 6B	5	7	
91.5	7	17	9	71.0% 98.6
96.5	8	5	58	
101.5	9	15	22	
106.5	10	5	69	
111.5	11	3	15	
116.5	12A 12B	15	60	
121.5	13	8	68	
125.5	14_{T1}	8	112	31.2% 122.2 (54.5% Sd)
130.5	15	13	113	
135.5	16_{T2}	11	162	35.8% 114.5 (18.4% Sd)
140.5	17	10	192	
145.5	18_{T3}	8	196	29.2% 121.5 (80% Sd)
150.5	19	10	213	
155.5	20	9	116	
160.5	21	10	214	
166.5	22	13	195	
172.5	23	77	300	

BORING NO. B14 - PIER 14

Sta. 108+71.10

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24" cas.

Classification	Stratum Elev.
	-61.7
Soft gray silt with fine sand lenses; river mud	-85.0
Medium gray running sand	-93.0
Soft green silty clay with some sand	-107.0
Very soft fine gray silty sand	-117.0
Green silty fine sand with decomposed shells possibly some clay	-128.0
Greenish silty clay	-138.0
Green silty very fine sand	-141.0
Green silty clay with some sand	-148.0
Green soft fine silty sand	-154.0
Green medium silty clay	-160.0
Stiff green clay	-171.6

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
65.5	1	Push	0	
70.5	2	Push	0	
75.5	3	Push	0	
80.5	4	Push	0	48.5% 108.3
85.5	5	Jetted	0	
90.5	6	Jetted	50	
95.5	7	8	87	
100.5	8	10	26	
105.5	9	11	54	
110.5	10	15	61	28.9% 122.2
115.5	10	15	117	
120.5	11_{T1}	8	127	33.6% 117.7 (70% Sd)
125.5	12	15	131	
130.5	13_{T2}	11	134	39.6% 114.4 (29% Sd)
135.5	14	11	139	
140.5	15_{T3}	12	147	28% 121.3 (51% Sd)
145.5	16	11	156	
150.5	17	12	159	
155.5	18	7	163	
160.5	20	17	172	
165.5	21	15	259	
171.5	22	88	273	

BORING NO. B15 - PIER 15

Sta. 104+03.01

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24" cas.

Ref. Elev. -20

Classification	Stratum Elev.
	-47.9
Greenish gray silty fine sand	-65.9
Greenish gray silty fine sand; very slight traces of shells	-86.9
Greenish gray silty fine sand	-92.9
Gray sandy silt; possibly some clay	-116.9
Green clay with sand lenses	-132.9
Green clay with sand lenses and few shells	-142.9
Green silty sand with few large shells	-153.9
Green clay with very fine sand lenses; shells	-159.9
Very stiff green clay	-167.9

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
53.5	1	6	8	
58.5	2	6	6	25.8% 125.2
63.5	3	7	12	
69.5	4	7	14	
73.5	5	7	22	
78.5	6	10	33	
84.5	7	11	49	
89.5	8	8	69	25.1% 126.0
94.5	9	10	84	
100.5	10	11	129	
105.5	11	11	137	
110.5	12	12	138	
115.5	13_{T1}	10	121	28.4% 121.2 (98.2% Sd)
120.5	14	13	252	
125.5	15_{T2}	12	308	30% 120.5 (78% Sd)
130.5	16	37	416	
135.5	17_{T3}	14	478	34.6% 117.0 (33% Sd)
141.5	18	13	427	
147.5	19	8	640	
151.5	20	9	345	
156.5	21	11	313	
161.5	22	83	385	
167.5	23	47	412	

BORING NO. B16 - PIER 16

Sta. 99+34.55

Casing $3\frac{1}{2}$" I.D.
Spoon 3" O.D.
300# Hammer
18" Drop samp.-24" cas.

Classification	Stratum Elev.
	-26.8
Loose wet gray silty sand and shells	-39.8
Very fine green silty sand; some large shells	-48.8
Gray silty sand; traces of fine shells	-51.4
Brown-gray clay	-56.4
Green clay; fine sand; trace of fine shells	-63.8
Very fine greenish silty sand; silt; traces of shells	-75.8
Very fine greenish silty sand; silt; tr. shells	-79.8
Very fine green silt and sand	-94.8
Very fine gray sandy silt; some clay	-103.8
Gray silty clay with some fine sand	-138.8
Green silty clay with some sand	-141.8
Very fine green silty clay; some sand	-148.8
Very fine dark green silty sand	-153.8
Very fine dark green silty sand; silt; some fine shells	-160.8
Stiff green clay	-161.8

Sample Elev.	Sample No.	Hammer Blows-Sampler	Hammer Blows-Casing	Moisture Percentage and Weight (lbs/cu.ft.)
35.3	1	5	73	29.0% 122.2
40.3	2	19	29	
45.3	3 3A	17 24	68 32	
51.3	4A 4B	6	59	
56.3	5A 5B	6	69	
61.3	6	6	67	
67.3	7	5	89	
71.3	8	6	69	
76.3	9	7	87	
81.3	10	5	84	
86.3	11	6	92	
91.3	12	7	74	
96.3	13	7	81	
101.3	14	6	82	
106.3	15	9	123	
110.3	16_{T1}	8	130	28.7% 120.6 (56% Sd)
115.3	17	8	190	
120.3	18_{T2}	10	180	36.6% 116.0 (60.5% Sd)
125.3	19 19A	10 11	183 140	
130.3	20_{T3}	9	203	32.0% 117.2 (60.5% Sd)
135.3	21	9	182	
140.3	22	9	207	
145.3	23	8	189	
150.3	24	20	190	
156.3	25	9	190	
161.3	26	15	182	

CONSOLIDATION IN PERCENT & COEFFICIENT OF CONSOLIDATION (SQ. CM/SEC) x10^-3

Pressure, Tons per sq. ft. (upper line of each sample: % Consol.; lower line: Coef. Consol. x10-3)

Hole	Sample	0	0.5	1	2	4	8	2	0.5
B11	16_{T1}	0	2.10%	3.41%	5.89%	10.18%	18.68%	13.18%	11.35%
			4.43	3.21	1.06	.65	.35		
	18_{T2}	0	3.10%	4.33%	6.77%	9.76%	13.68%	13.20%	10.91%
			1.40	.73	.96	1.00	1.08		
	20_{T3}	0	0.56%	1.46%	2.68%	4.00%	5.80%	4.65%	3.94%
			.90	1.46	2.16	3.11	2.40		
	22_{T4}	0	1.64%	3.39%	5.68%	8.35%	11.08%	8.64%	5.95%
			12.65	2.90	.59	.48	.71		
B12	16_{T1}	0	2.64%	4.39%	7.43%	11.08%	16.07%	14.32%	11.62%
			1.92	.87	.68	.45	.48		
	18_{T2}	0	1.84%	3.16%	6.24%	9.60%	13.84%	12.49%	10.10%
			3.59	.64	-	.20	.225		
	20_{T3}	0	2.49%	3.75%	5.41%	7.79%	11.85%		
			9.53	3.95	5.92	2.13	11.92		
B13	14_{T1}	0	1.62%	2.73%	4.29%	6.62%	9.79%	8.10%	6.33%
			2.04	2.02	2.84	2.10	2.39		
	16_{T2}	0	3.52%	4.89%	7.06%	10.26%	16.34%	13.70%	10.47%
			5.13	.93	.34	.23	.19		
	18_{T3}	0	2.81%	3.86%	5.31%	7.04%	9.29%		
			34.5	49.7	96.9	131.2	184.7		
B14	11_{T1}	0	2.40%	3.88%	6.48%	10.90%	17.34%	13.20%	10.10%
			1.23	.53	.62	.32	.27		
	13_{T2}	0	2.27%	3.85%	6.35%	9.31%	13.53%	10.64%	6.83%
			.99	.36	.23	.26	.24		
	15_{T3}	0	1.97%	4.08%	4.84%	8.11%	13.17%	9.86%	7.74%
			5.71	4.47	1.33	1.07	0.93		
B15	13_{T1}	0	1.42%	2.64%	5.14%	8.02%	12.12%	10.33%	8.24%
			1.23	.81	.62	.74	.53		
	15_{T2}	0	4.87%	6.30%	8.38%	11.15%	15.78%	13.01%	11.46%
			6.33	5.54	5.98	15.96	8.32		
	17_{T3}	0	3.59%	4.25%	6.94%	11.60%	19.55%	15.18%	10.65%
			-	.57	.23	.14	.10		
B16	16_{T1}	0	5.51%	7.14%	9.40%	13.11%	18.69%	15.94%	14.41%
			1.01	1.38	1.17	1.24	1.07		
	18_{T2}	0	3.29%	5.06%	7.80%	12.58%	19.6%	15.87%	12.37%
			.45	.32	.36	.23	.18		
	20_{T3}	0	1.23%	2.29%	4.49%	7.16%	10.41%	8.91%	7.56%
			3.03	2.11	2.04	1.94	2.21		

← % Consol.
← Coef. Consol. x10-3

Sample Data-Boring B12

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviation Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
16_{T1}	0.75	1.64
18_{T2}	0.75	2.28
20_{T3}	0.40	.92

Sample Data-Boring B13

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviation Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
14_{T1}	1.12	2.83
16_{T2}	1.07	2.41
18_{T3}	0.30	0.98

Sample Data-Boring B14

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviation Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
11_{T1}	1.06	2.40
13_{T2}	2.54	1.99
15_{T3}	1.31	1.96

Sample Data-Boring B15

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviation Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
13_{T1}	1.037	2.30
15_{T2}	0.81	2.12
17_{T3}	1.00	2.41

Sample Data-Boring B16

Sample No.	Shear (Tons/sq.ft.) from Unconfined Compression	Triaxial Deviation Stress (Tons/sq.ft.) Under 2 Tons/sq.ft. Lateral Pressure
16_{T1}	1.09	2.01
18_{T2}	0.81	2.27
20_{T3}	1.08	2.62

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS

RAPPAHANNOCK RIVER BRIDGE
AT GREYS POINT, VIRGINIA

SUPPLEMENTARY BORINGS B12 TO B16

10 8 6 4 2 0 10'
SCALE IN FEET

Sheet 83 of 92

MODJESKI & MASTERS, ENGINEERS CXIX-25-D

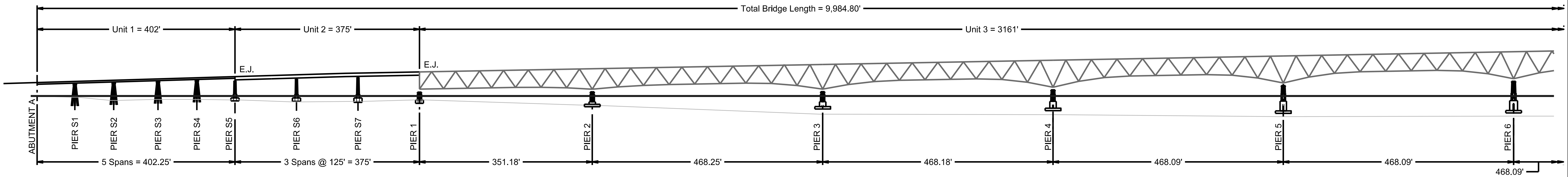
OCTOBER, 1953 SUPPLEMENTARY DRAWING NO. 10

DR. TOM TR. CK. H.L.B.

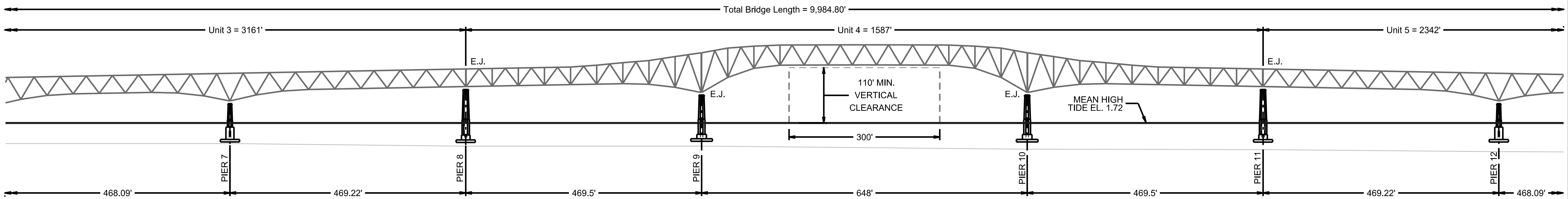
Contract 1

1 9 - 2 5 - 0 8 3 X

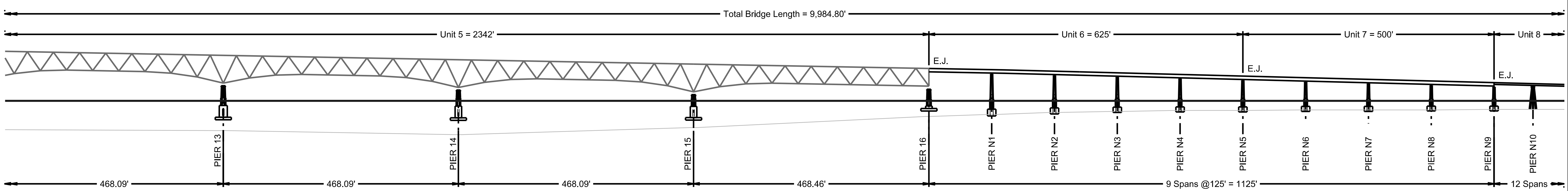
Appendix B – Report Drawings



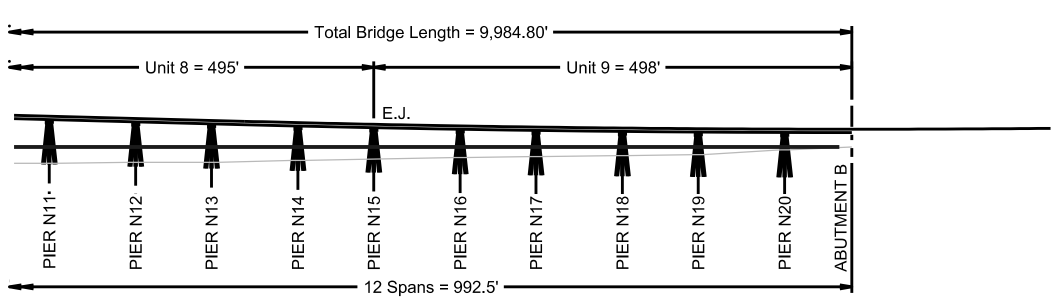
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



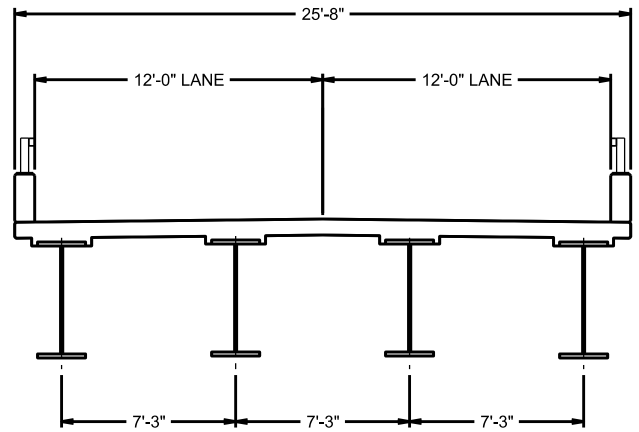
ELEVATION - PIERS 13 THRU N10



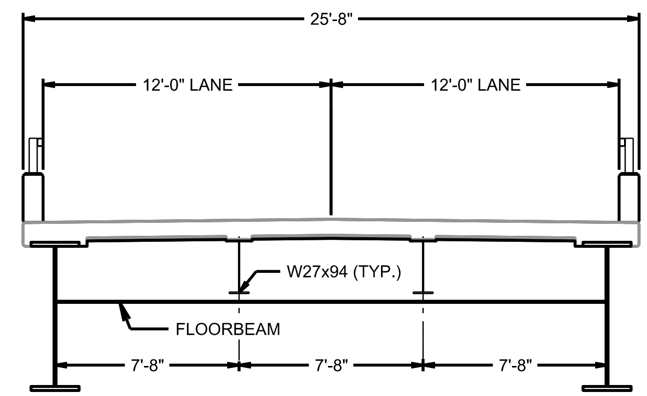
ELEVATION - PIER N11 THRU ABUTMENT B

CONCEPT

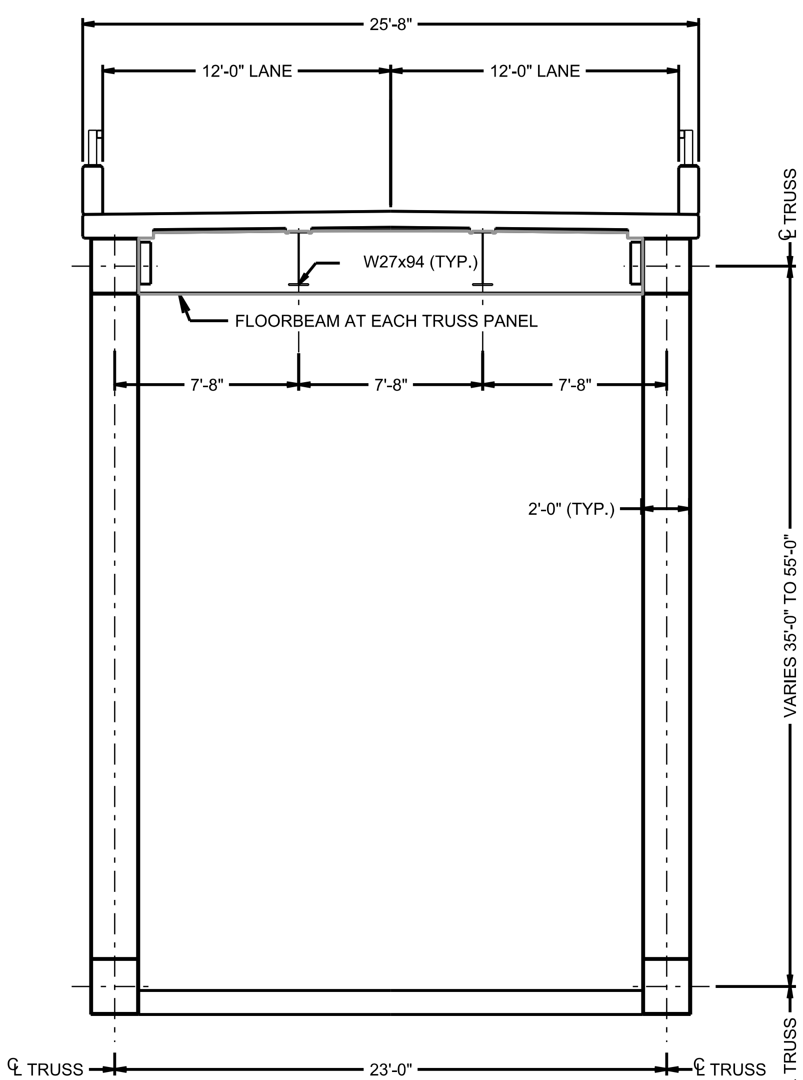
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE A	DRAWING 5-1



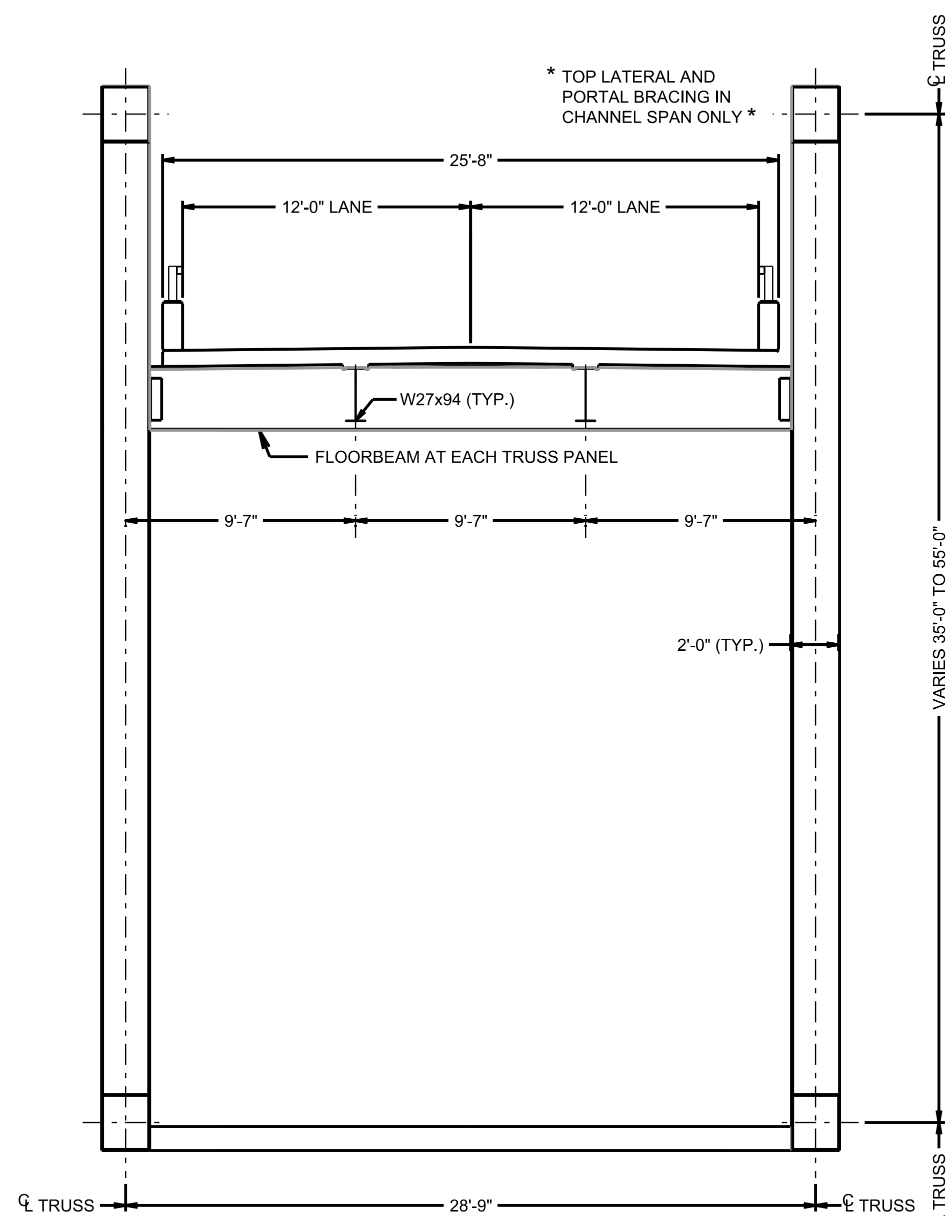
TYPICAL SECTION - BEAM UNITS



TYPICAL SECTION - GIRDER UNITS



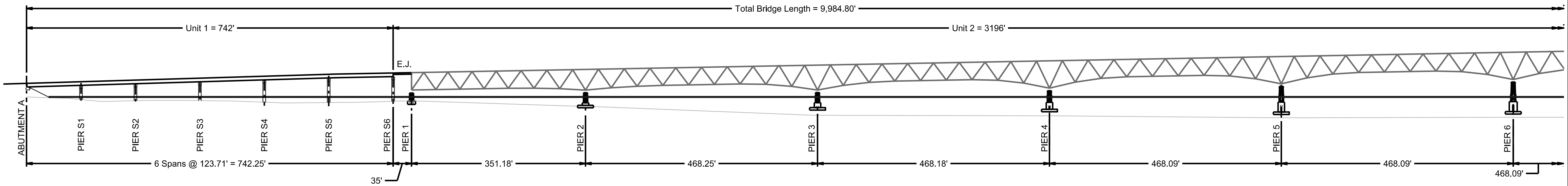
TYPICAL SECTION - DECK TRUSS
APPROACH UNITS



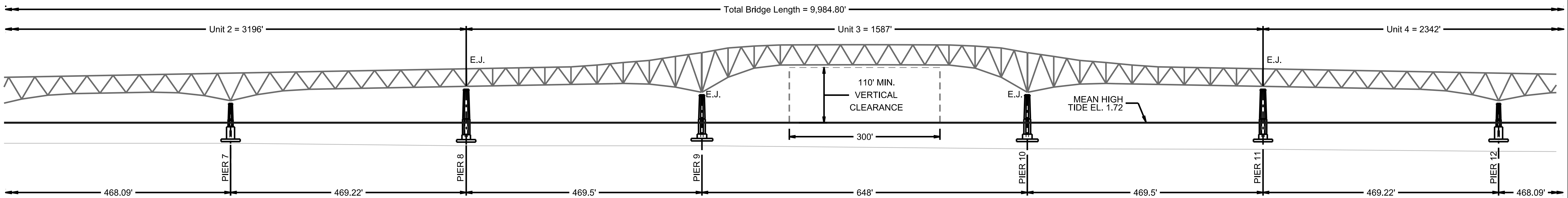
TYPICAL SECTION - THRU TRUSS
CHANNEL UNIT

CONCEPT

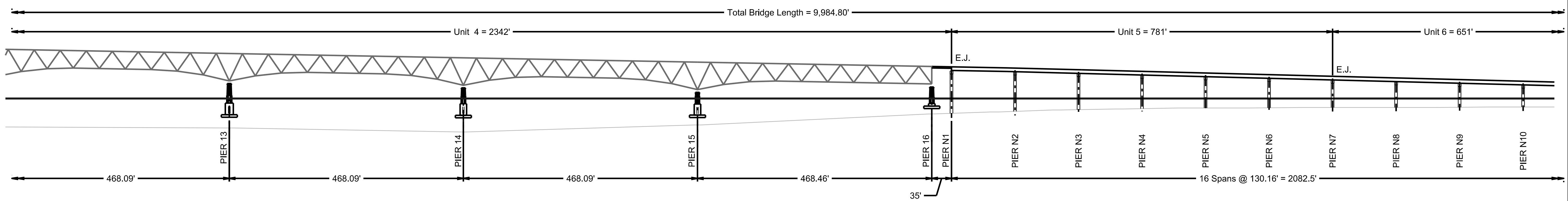
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS ALTERNATIVE A	DRAWING 5-2



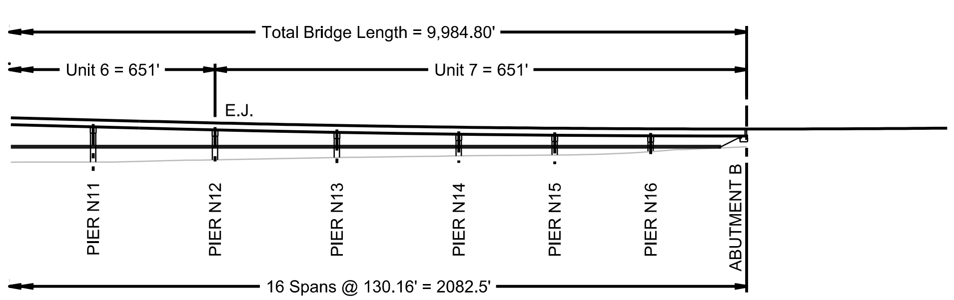
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



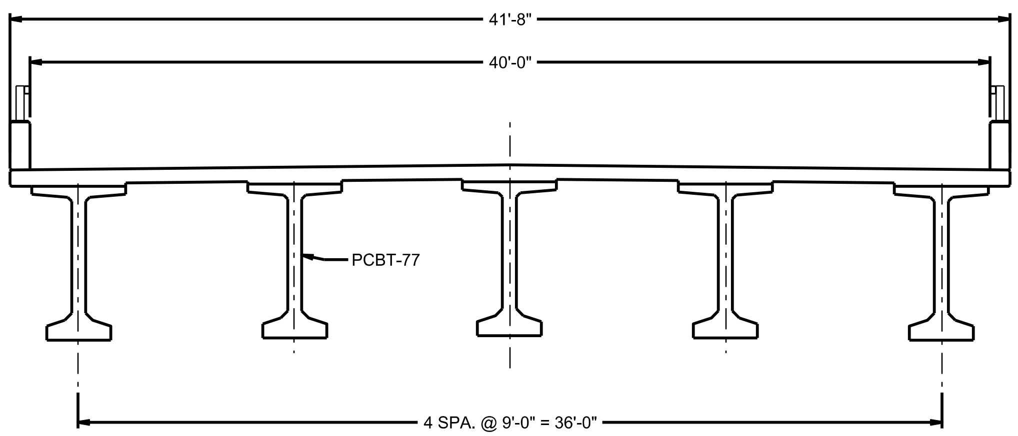
ELEVATION - PIERS 13 THRU N10



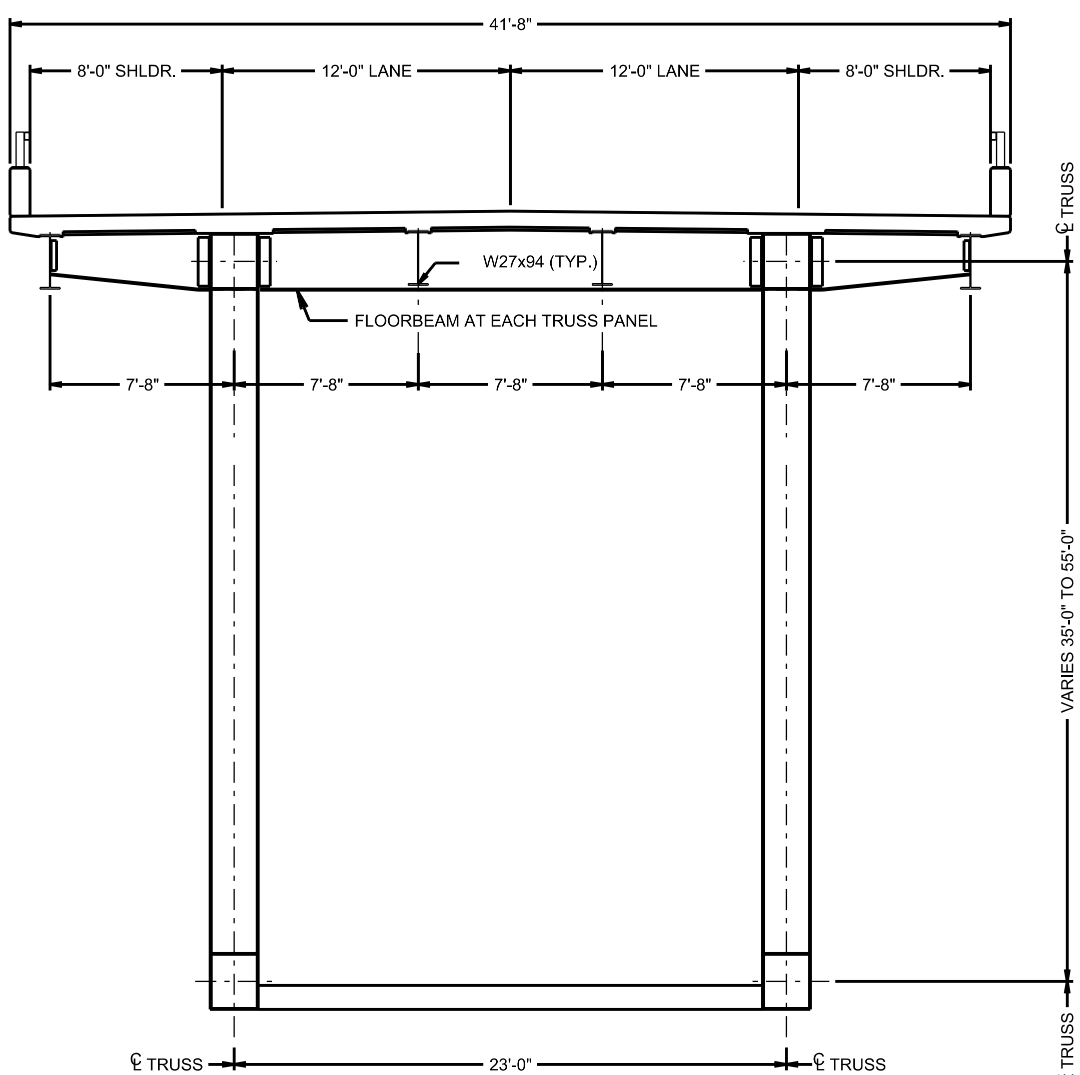
ELEVATION - PIER N11 THRU ABUTMENT B

CONCEPT

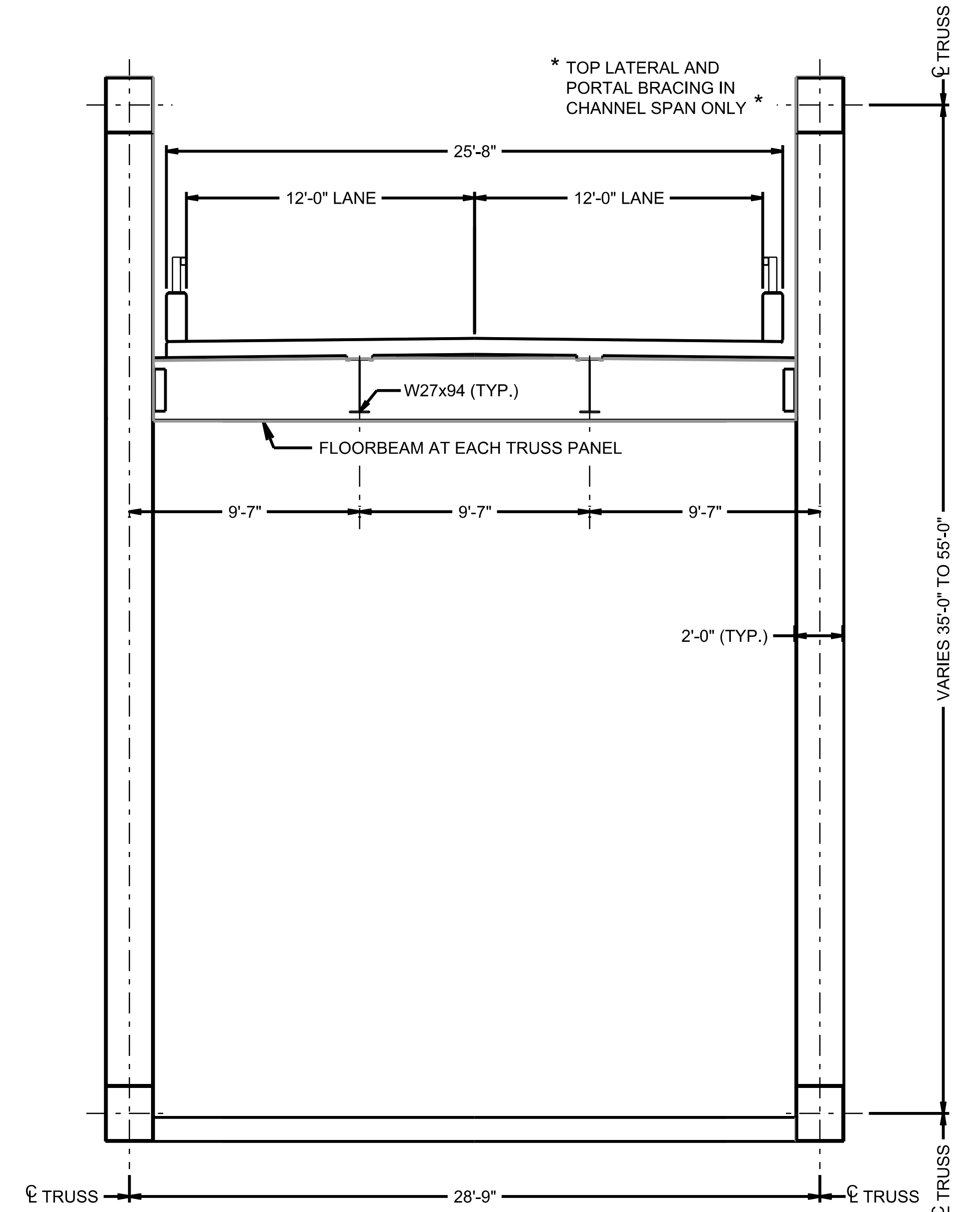
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE B	DRAWING 5-3



TYPICAL SECTION - BEAM & GIRDER UNITS



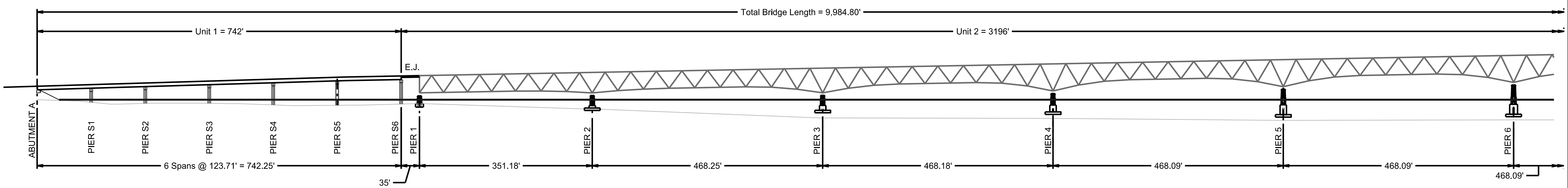
TYPICAL SECTION - DECK TRUSS
APPROACH UNITS



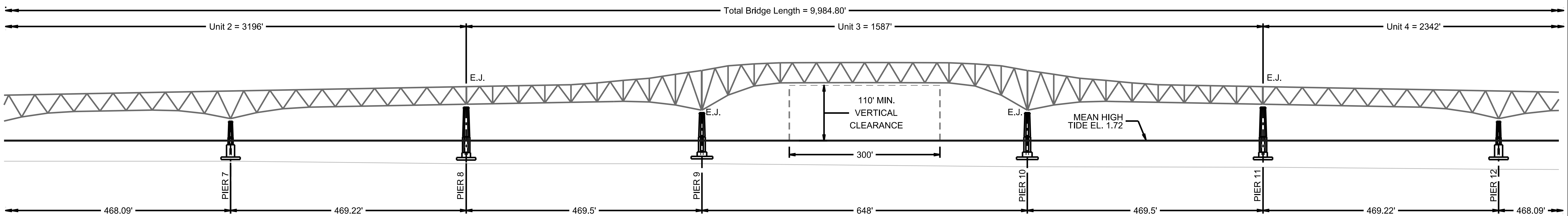
TYPICAL SECTION - THRU TRUSS
CHANNEL UNIT

CONCEPT

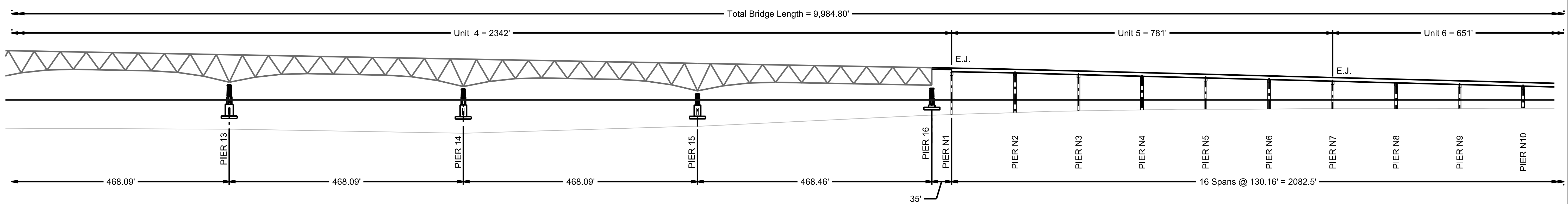
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS ALTERNATIVE B	DRAWING 5-4



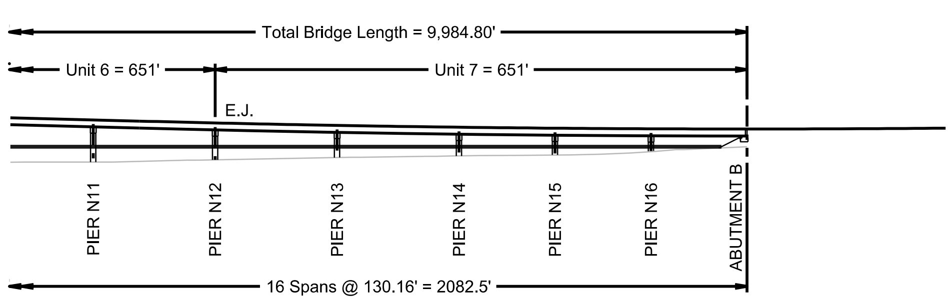
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



ELEVATION - PIERS 13 THRU N10



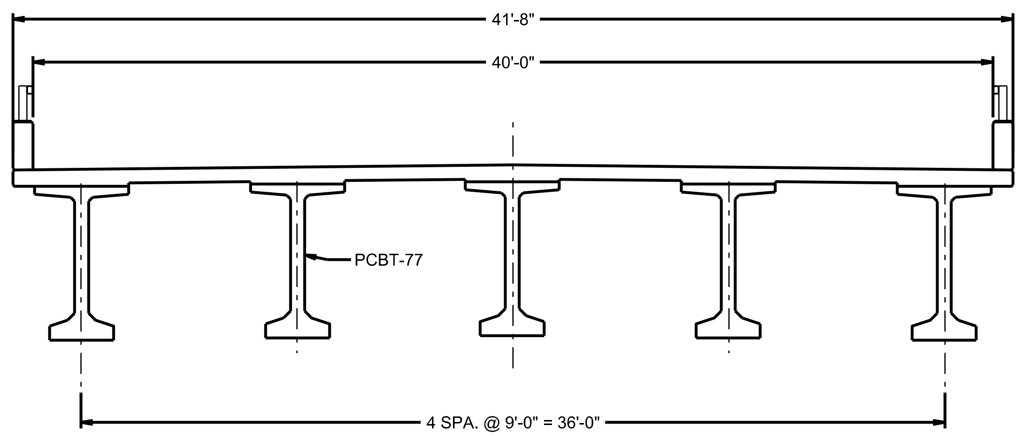
ELEVATION - PIER N11 THRU ABUTMENT B

CONCEPT

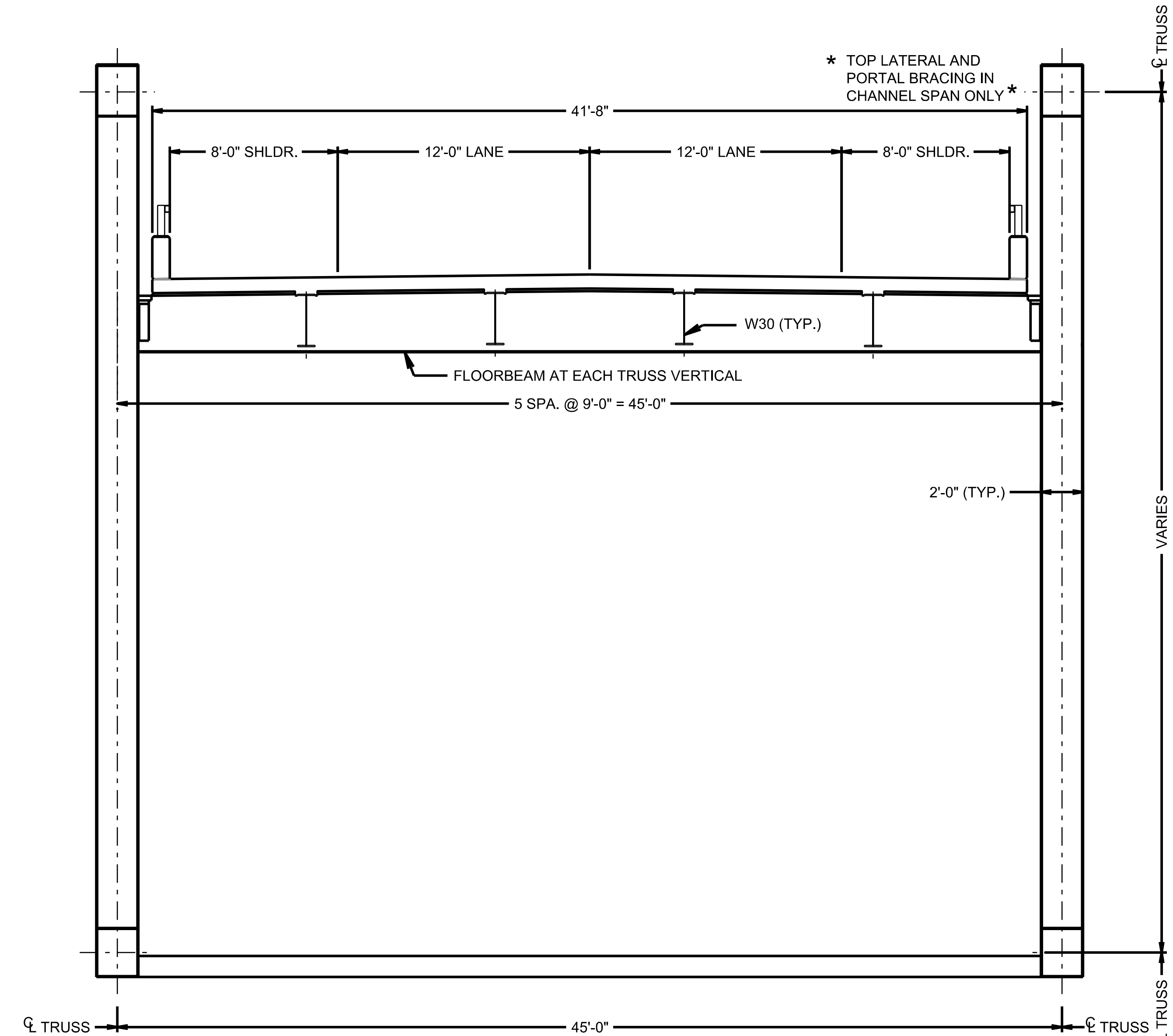
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

PROPOSED ELEVATION
ALTERNATIVE C

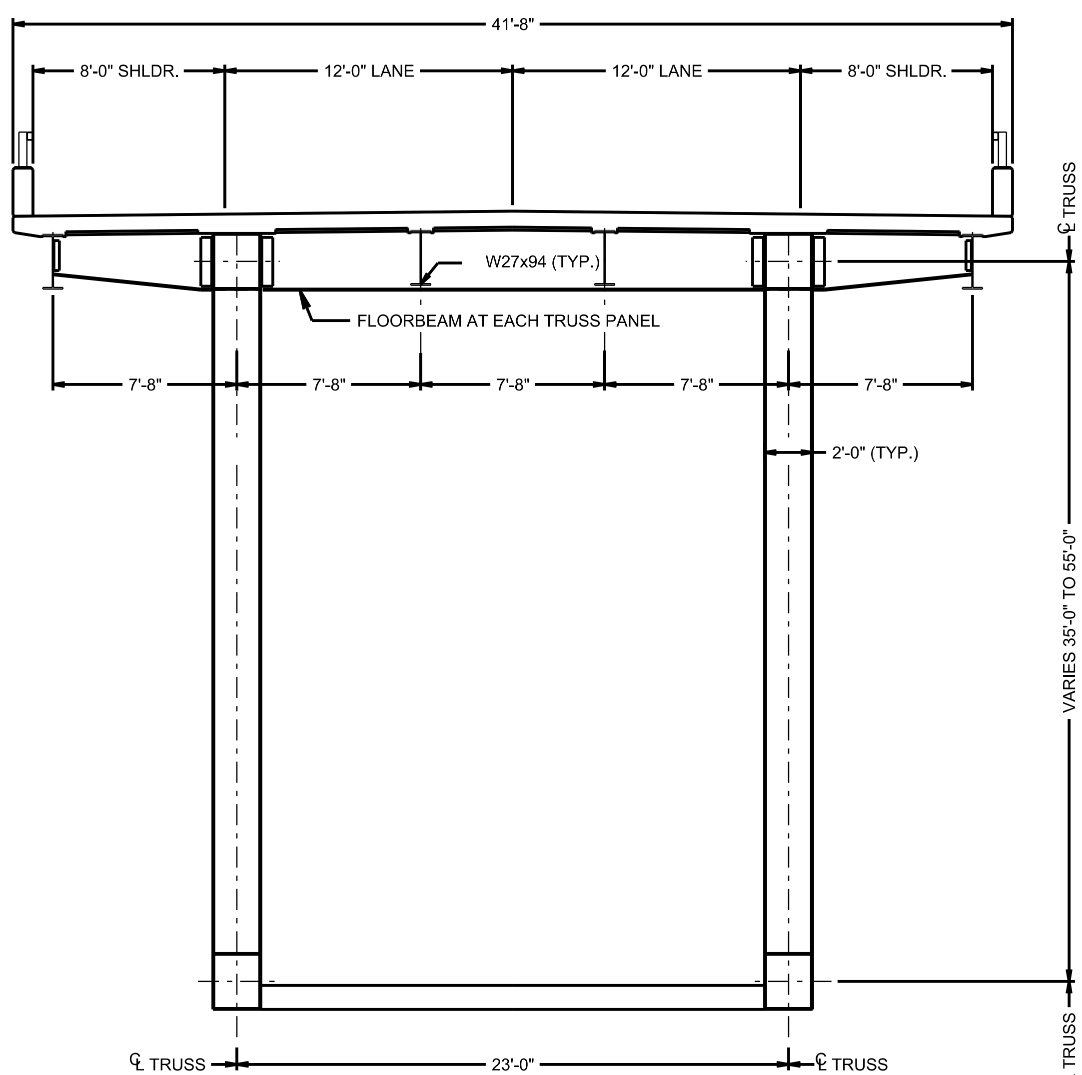
DRAWING
5-5



TYPICAL SECTION - BEAM & GIRDER UNITS



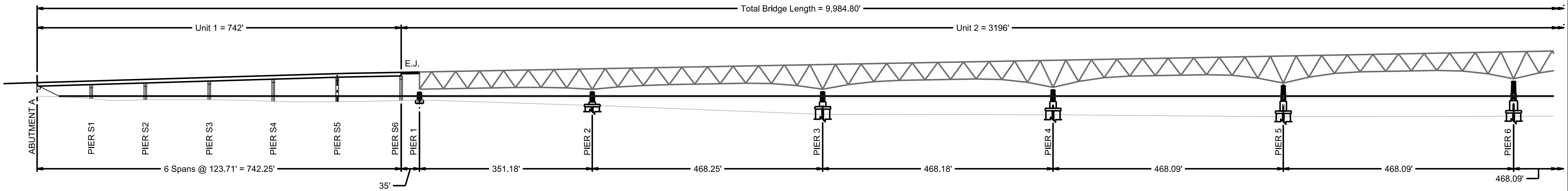
TYPICAL SECTION - THRU TRUSS
CHANNEL UNIT



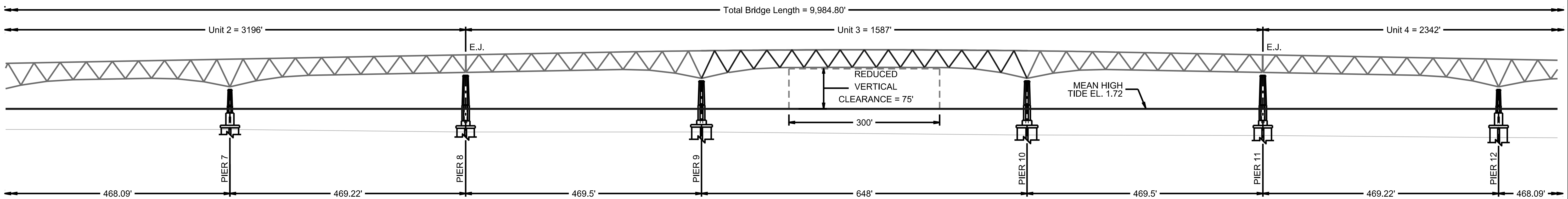
TYPICAL SECTION - DECK TRUSS
APPROACH UNITS

CONCEPT

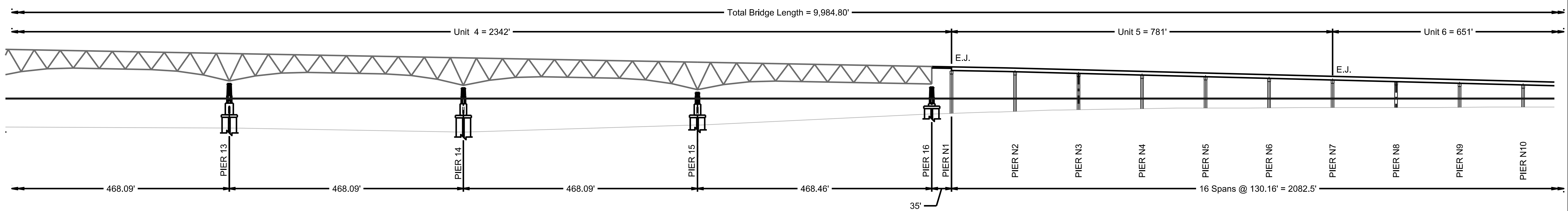
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS ALTERNATIVE C	DRAWING 5-6



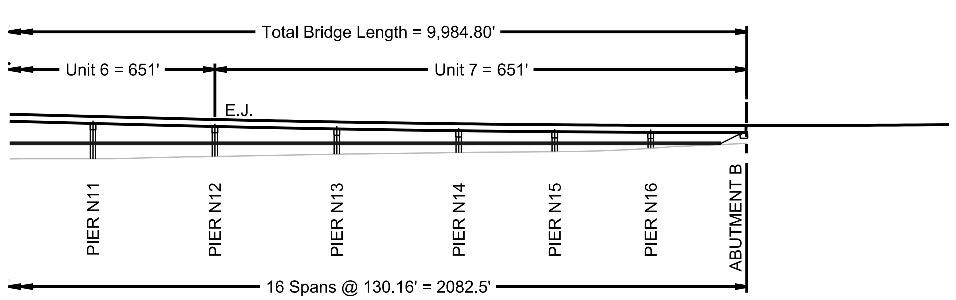
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



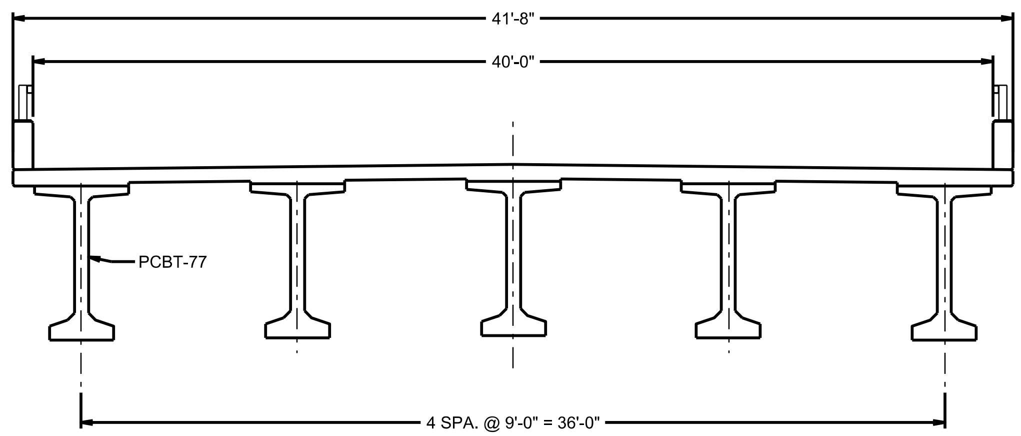
ELEVATION - PIERS 13 THRU N10



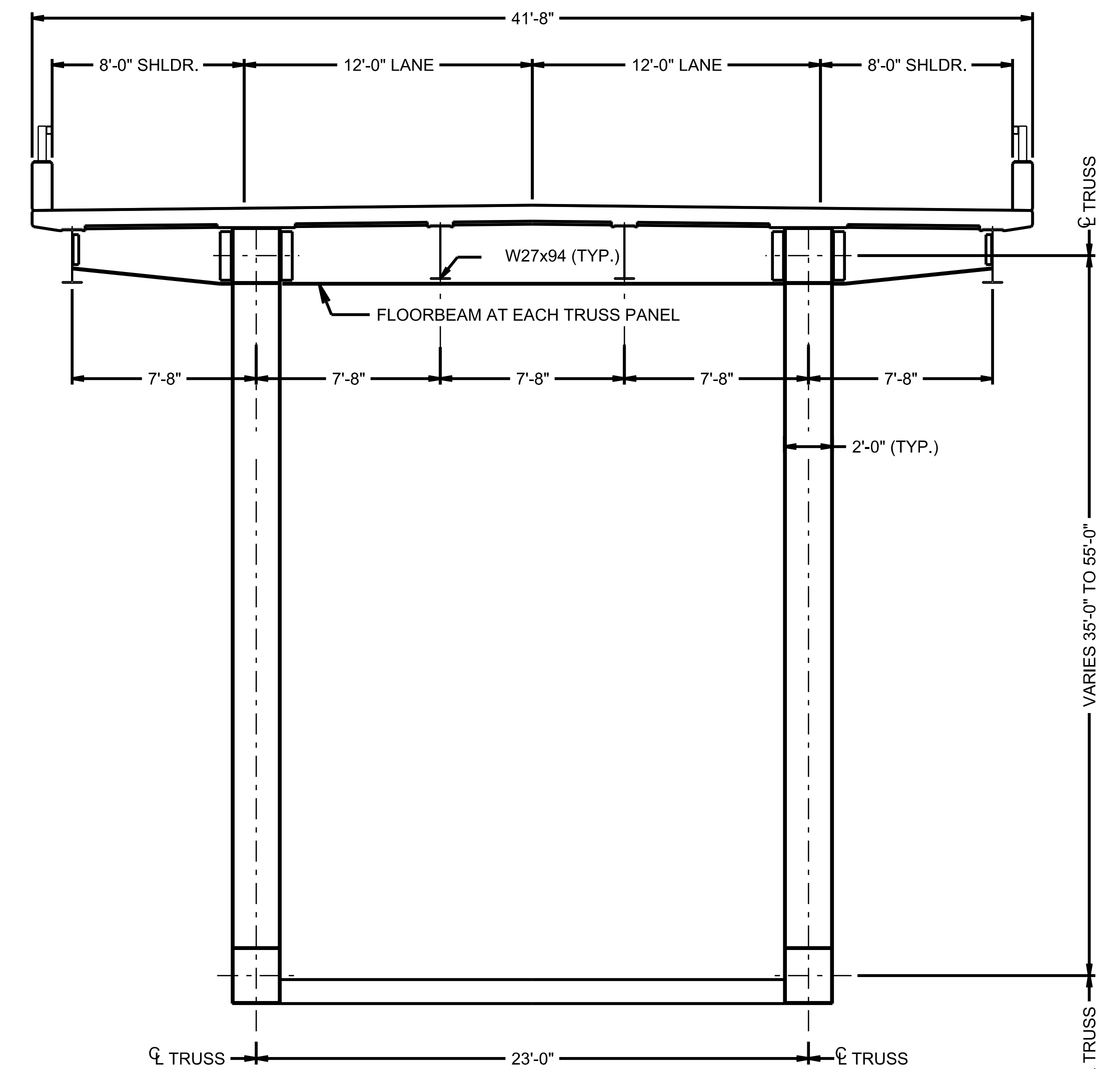
ELEVATION - PIER N11 THRU ABUTMENT B

CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE D	DRAWING 5-7



TYPICAL SECTION - BEAM & GIRDER UNITS

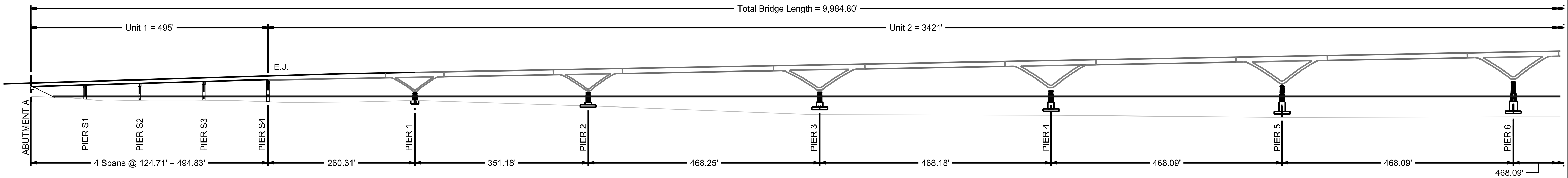


TYPICAL SECTION - DECK TRUSS
APPROACH & CHANNEL UNITS

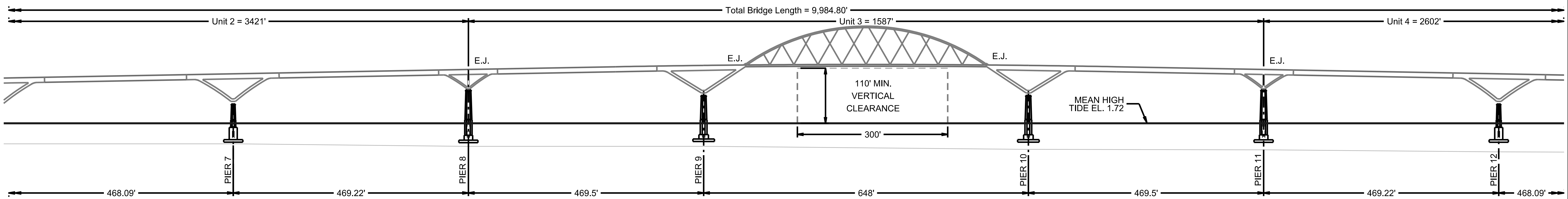
CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

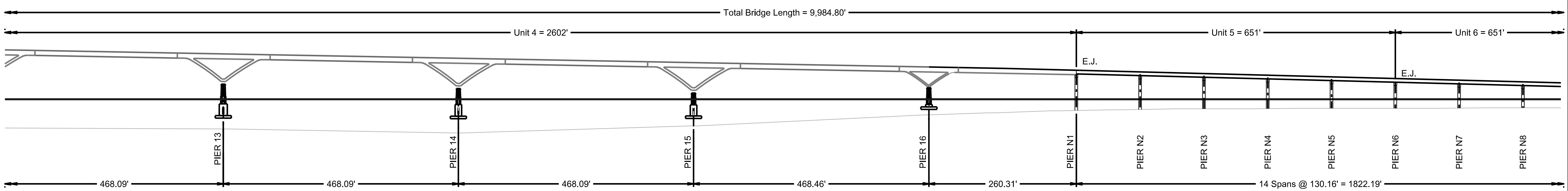
PROPOSED DETAILS ALTERNATIVE D	DRAWING 5-8



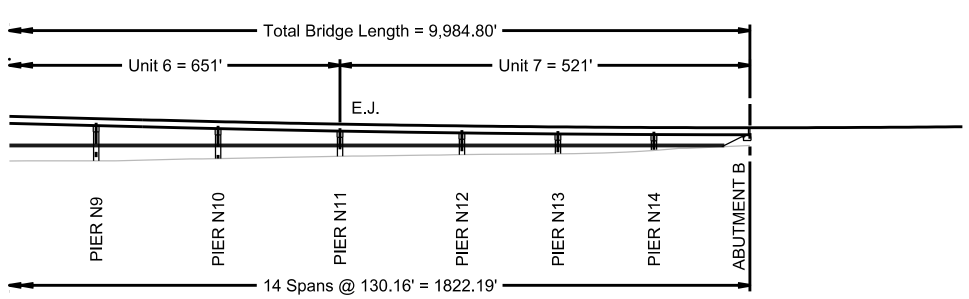
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



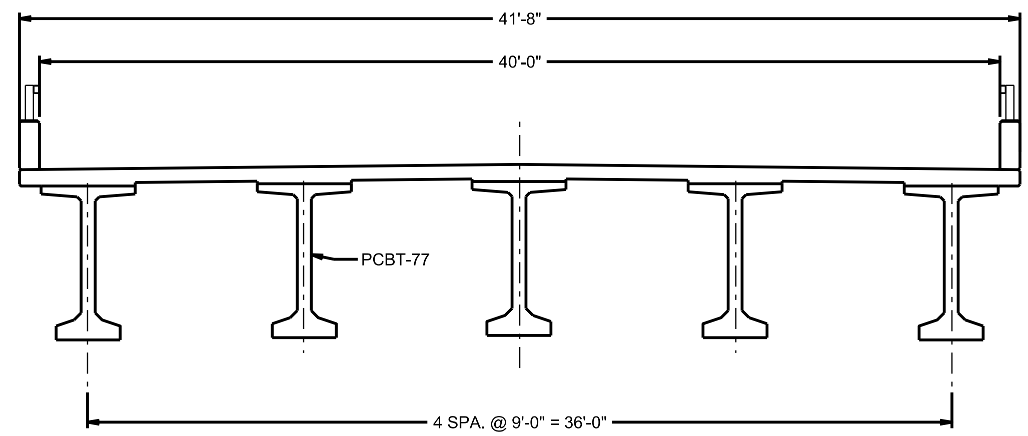
ELEVATION - PIERS 13 THRU N8



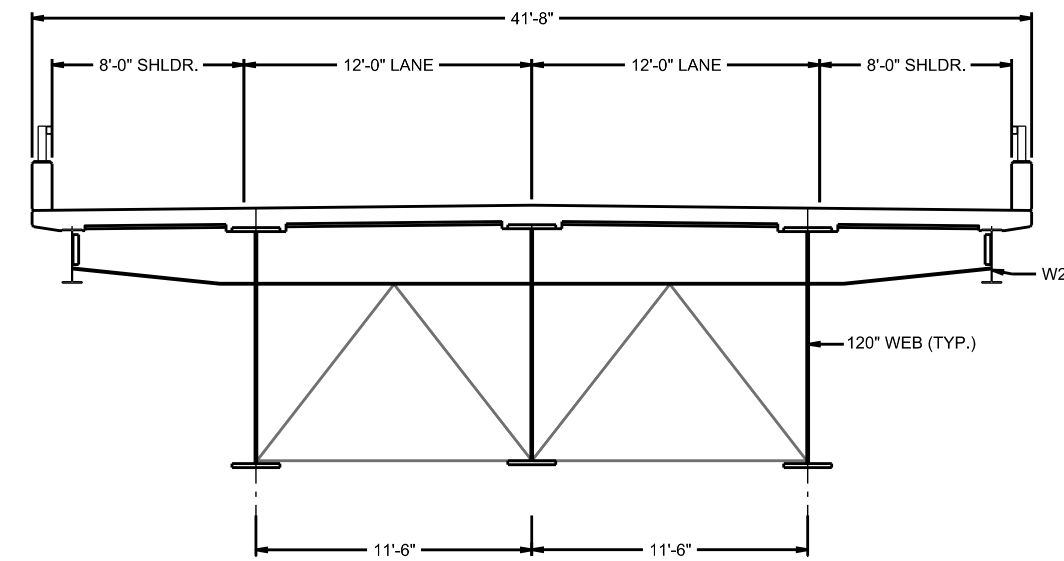
ELEVATION - PIER N9 THRU ABUTMENT B

CONCEPT

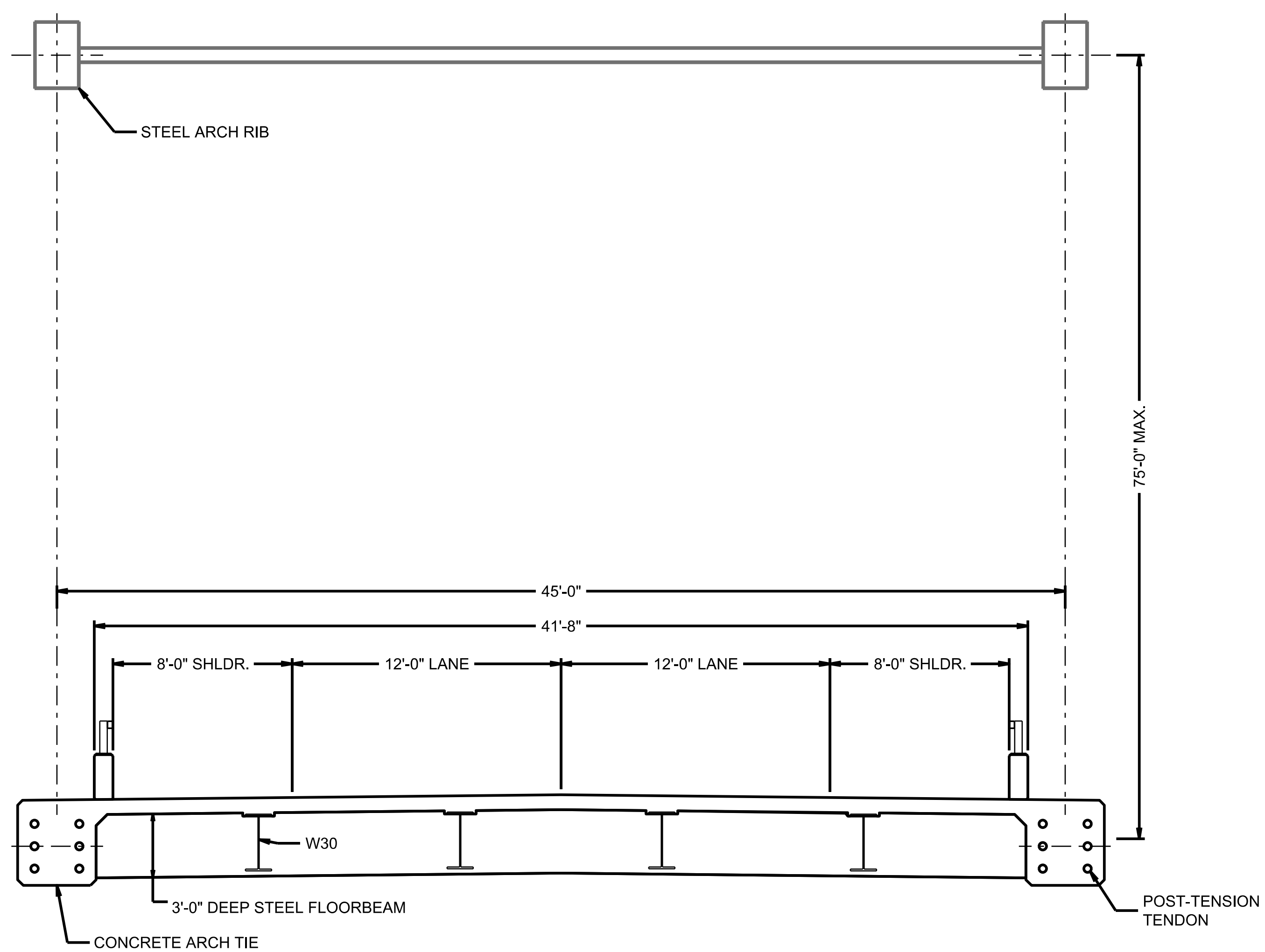
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE E	DRAWING 5-9

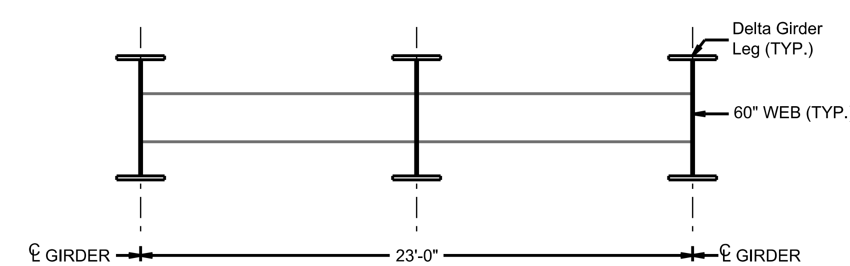


TYPICAL SECTION - BEAM & GIRDER UNITS



TYPICAL SECTION - STEEL DELTA GIRDERS
APPROACH UNITS



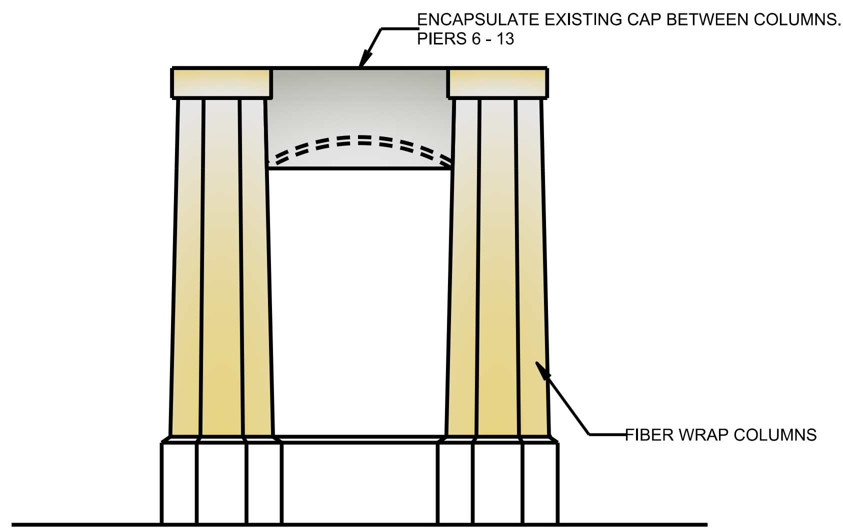


TYPICAL SECTION - TIED ARCH
CHANNEL UNIT

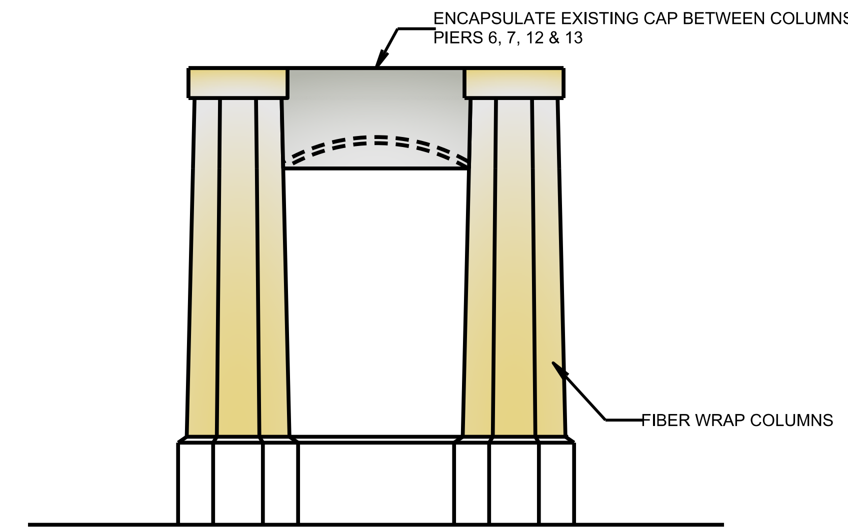
CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

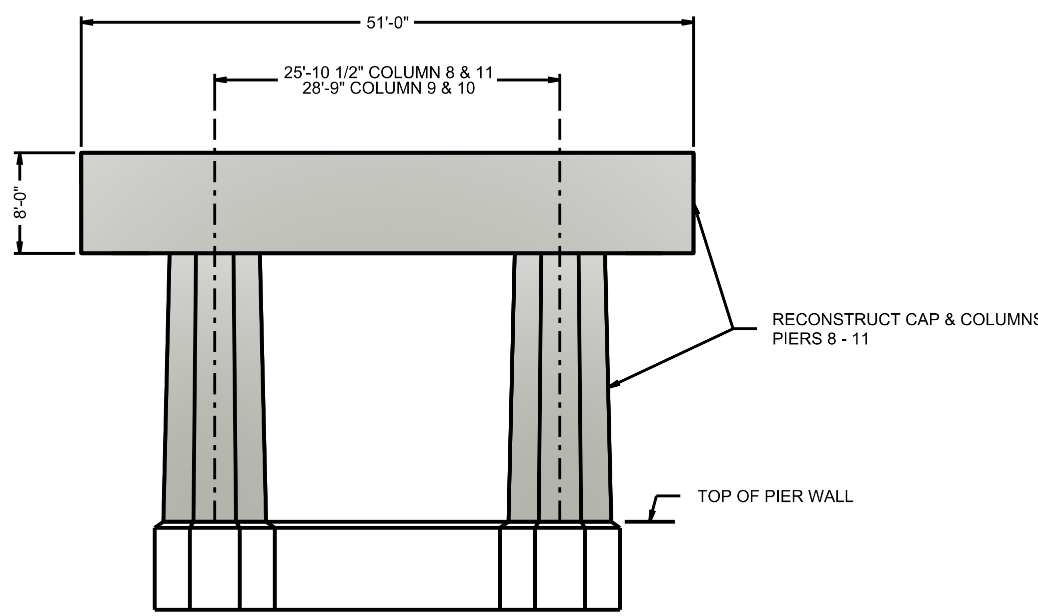
PROPOSED DETAILS ALTERNATIVE E	DRAWING 5-10



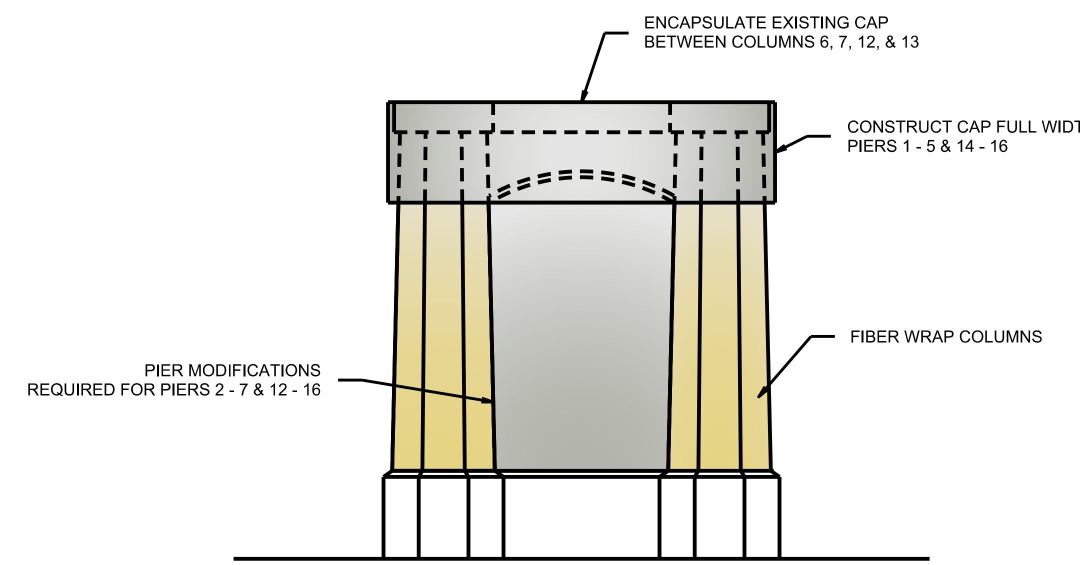
PIER MODIFICATIONS - SUPERSTRUCTURE REPLACEMENT - ALTERNATIVES A, B, & D



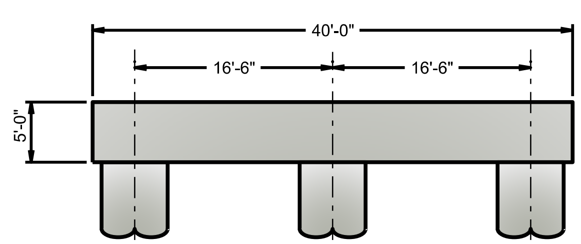
PIER MODIFICATIONS - SUPERSTRUCTURE REPLACEMENT - ALTERNATIVE C



PIER MODIFICATIONS - SUPERSTRUCTURE REPLACEMENT - ALTERNATIVES C & E



PIER MODIFICATIONS - SUPERSTRUCTURE REPLACEMENT - ALTERNATIVE E



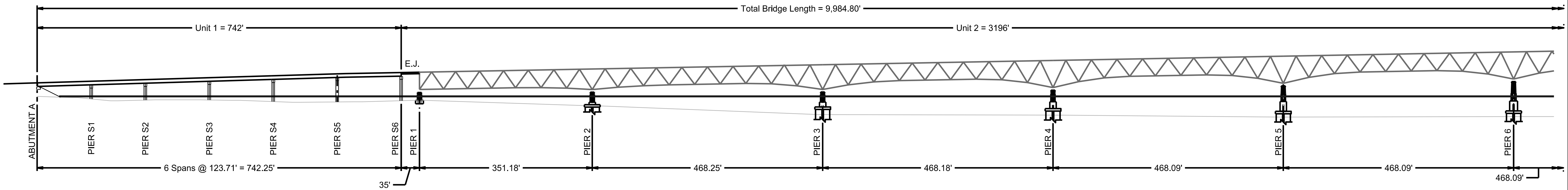
REPLACEMENT PIERS IN BEAM & GIRDER UNITS - SUPERSTRUCTURE REPLACEMENT - ALTERNATIVES B, C, D, & E

NEW CONCRETE

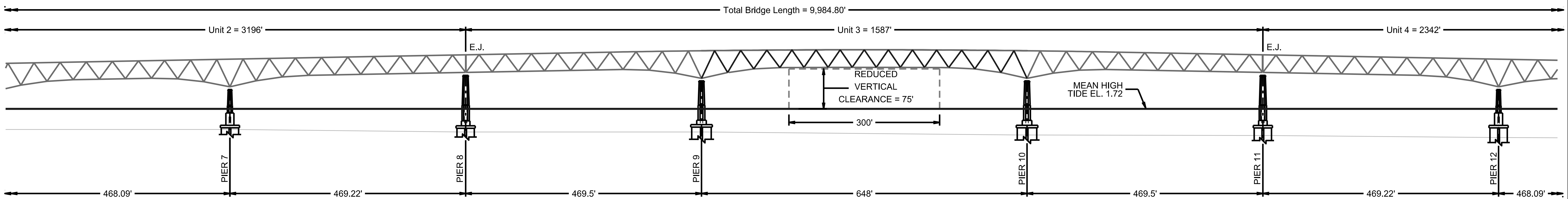
FIBER WRAP CONCRETE

CONCEPT

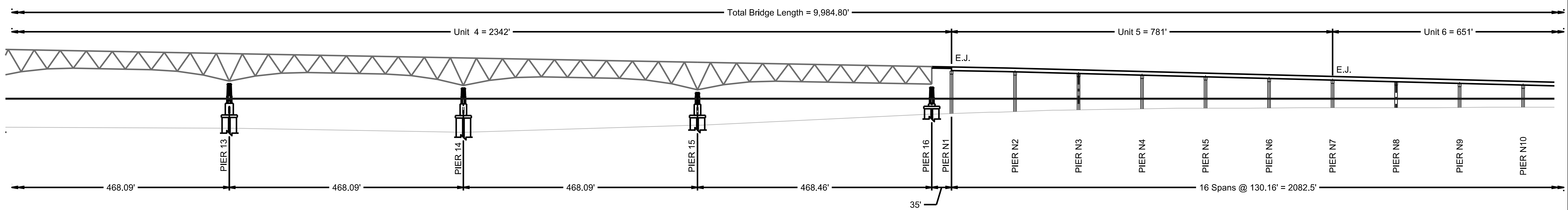
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS PIER MODIFICATIONS	DRAWING 5-11



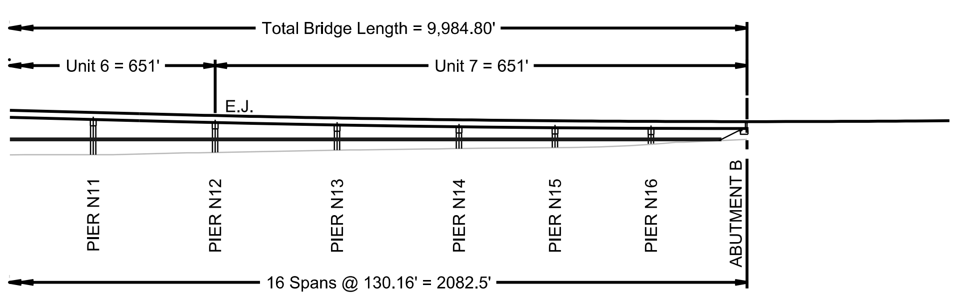
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



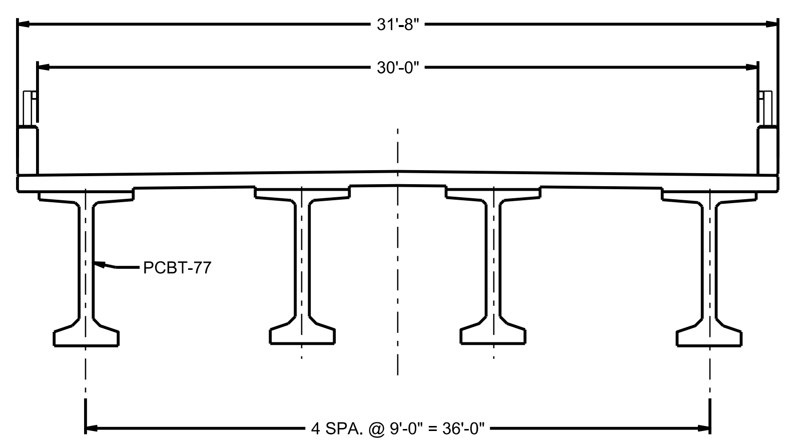
ELEVATION - PIERS 13 THRU N10



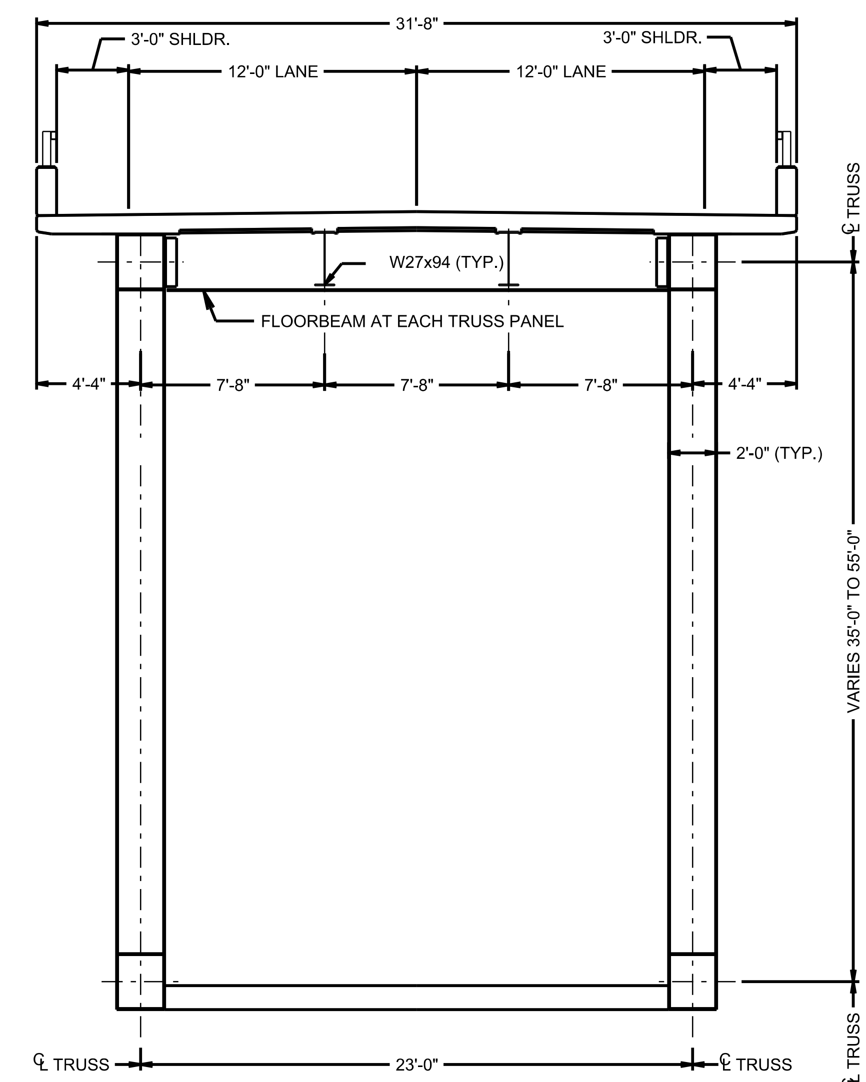
ELEVATION - PIER N11 THRU ABUTMENT B

CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE D1	DRAWING 5-12



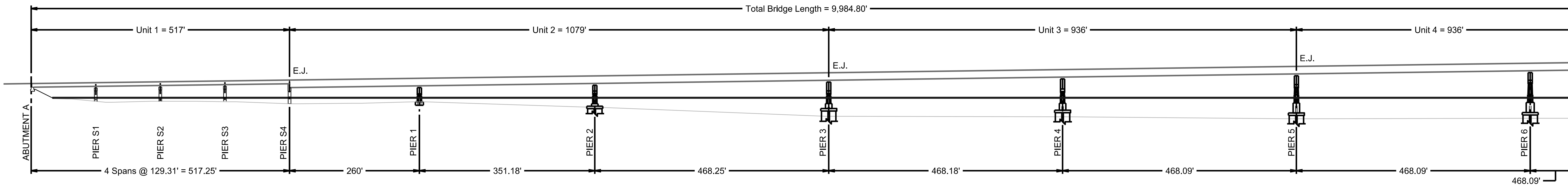
TYPICAL SECTION - BEAM & GIRDER UNITS



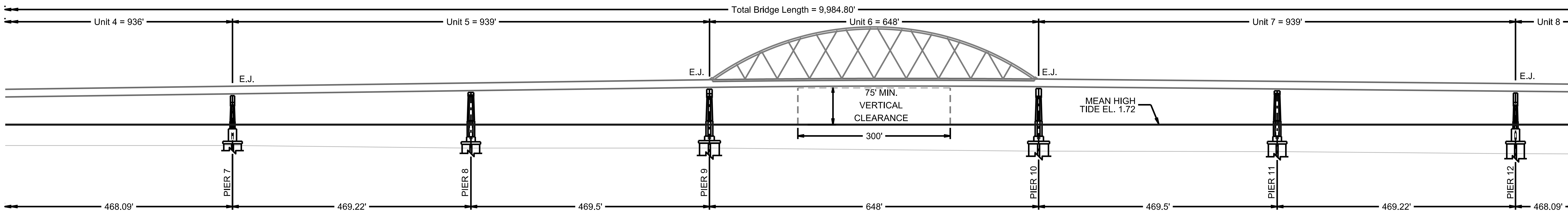
TYPICAL SECTION - DECK TRUSS
APPROACH & CHANNEL UNITS

CONCEPT

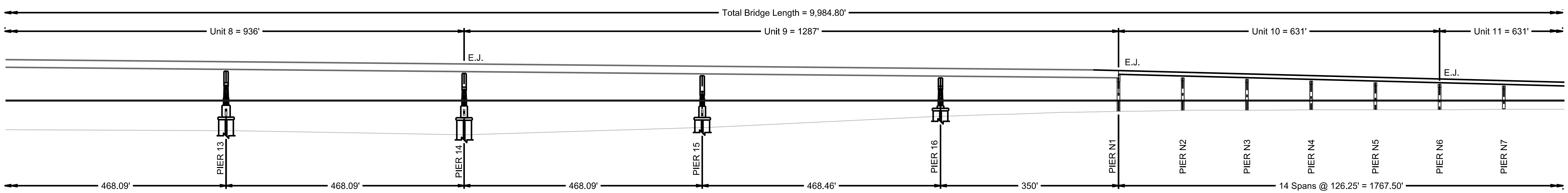
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS ALTERNATIVE D1	DRAWING 5-13



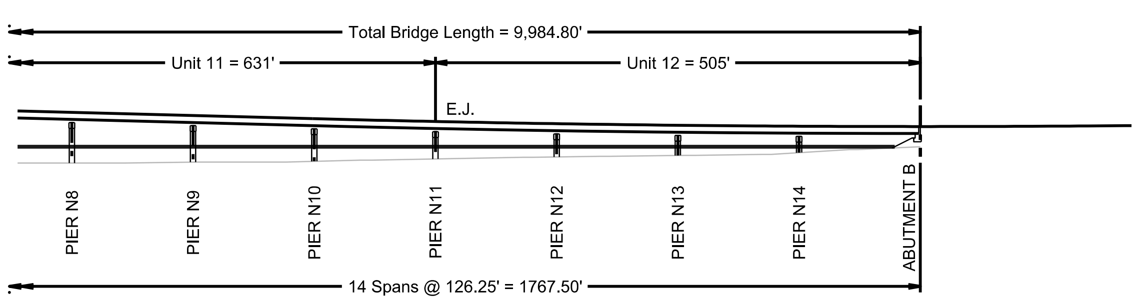
ELEVATION - ABUTMENT A THRU PIER 6



ELEVATION - PIERS 7 THRU 12



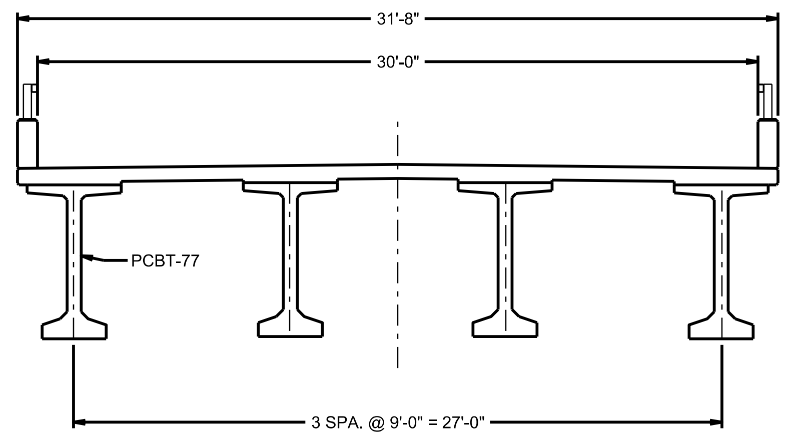
ELEVATION - PIERS 13 THRU N7



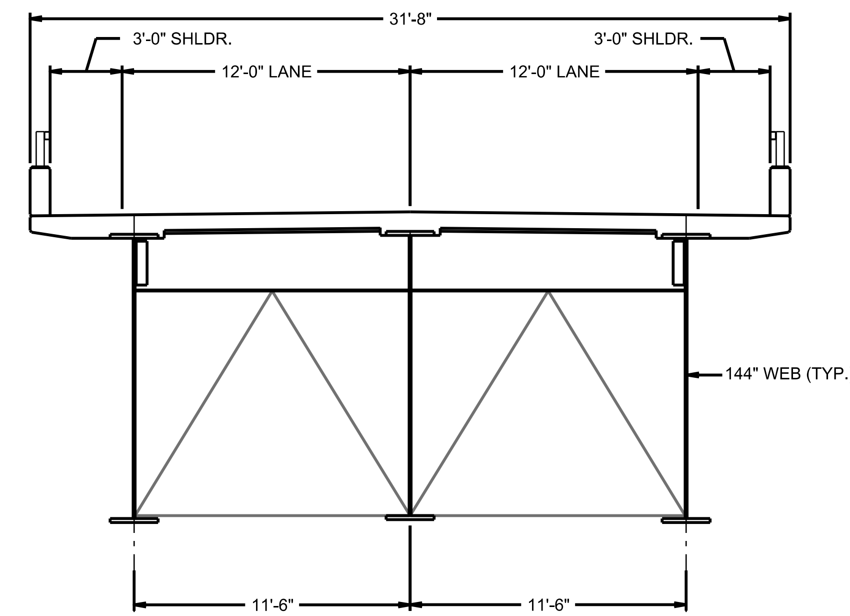
ELEVATION - PIER N8 THRU ABUTMENT B

CONCEPT

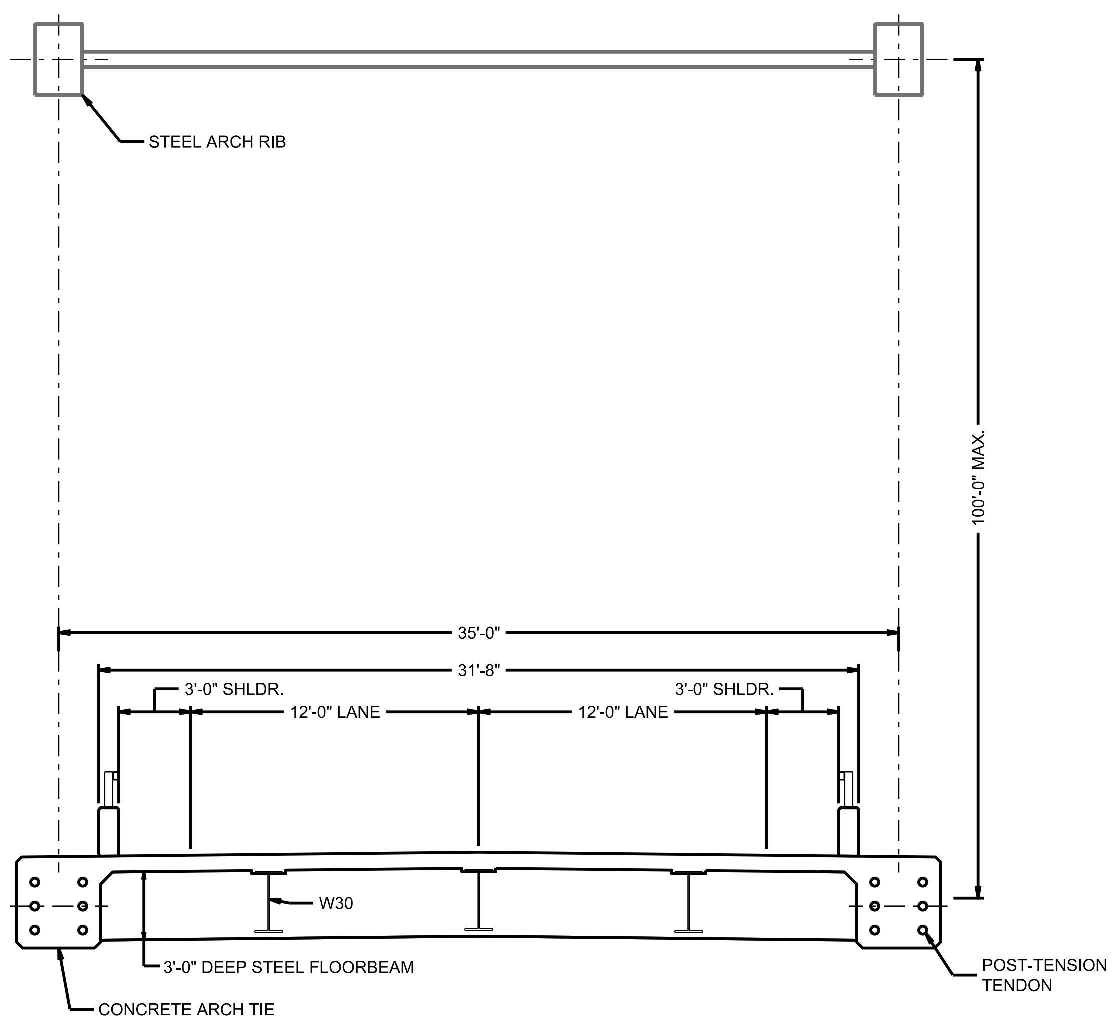
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION ALTERNATIVE F	DRAWING 5-14



TYPICAL SECTION - BEAM & GIRDER UNITS



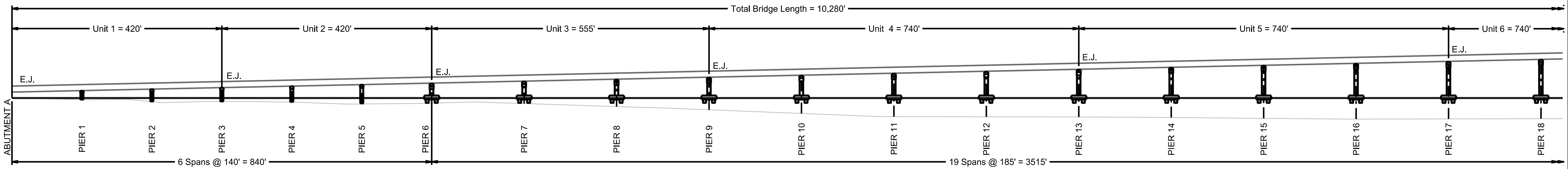
TYPICAL SECTION - STEEL PLATE GIRDER
APPROACH UNITS



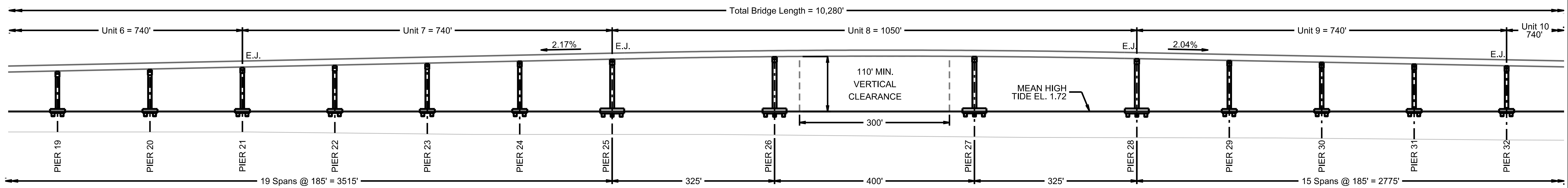
TYPICAL SECTION - TIED ARCH
CHANNEL UNIT

CONCEPT

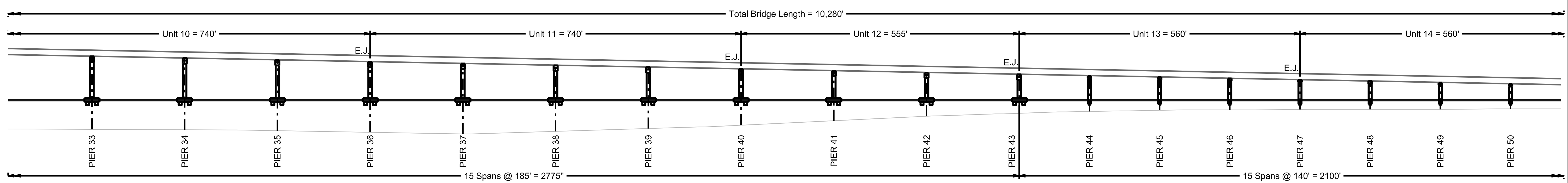
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED DETAILS ALTERNATIVE F	DRAWING 5-15



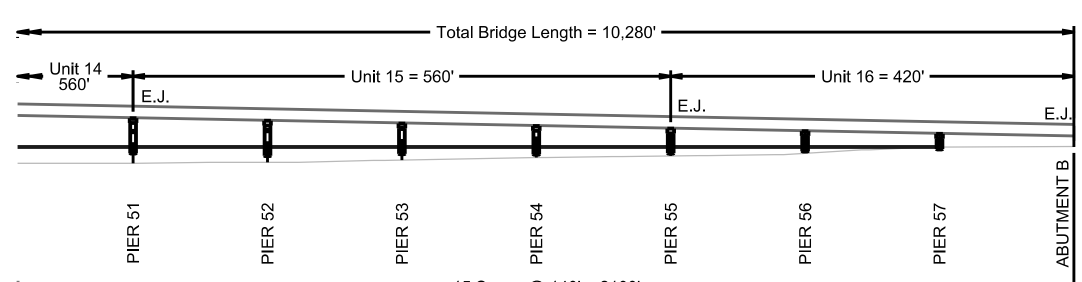
ELEVATION - ABUTMENT A THRU PIER 18



ELEVATION - PIERS 19 THRU 32



ELEVATION - PIERS 33 THRU 50



ELEVATION - PIER 51 THRU ABUTMENT B

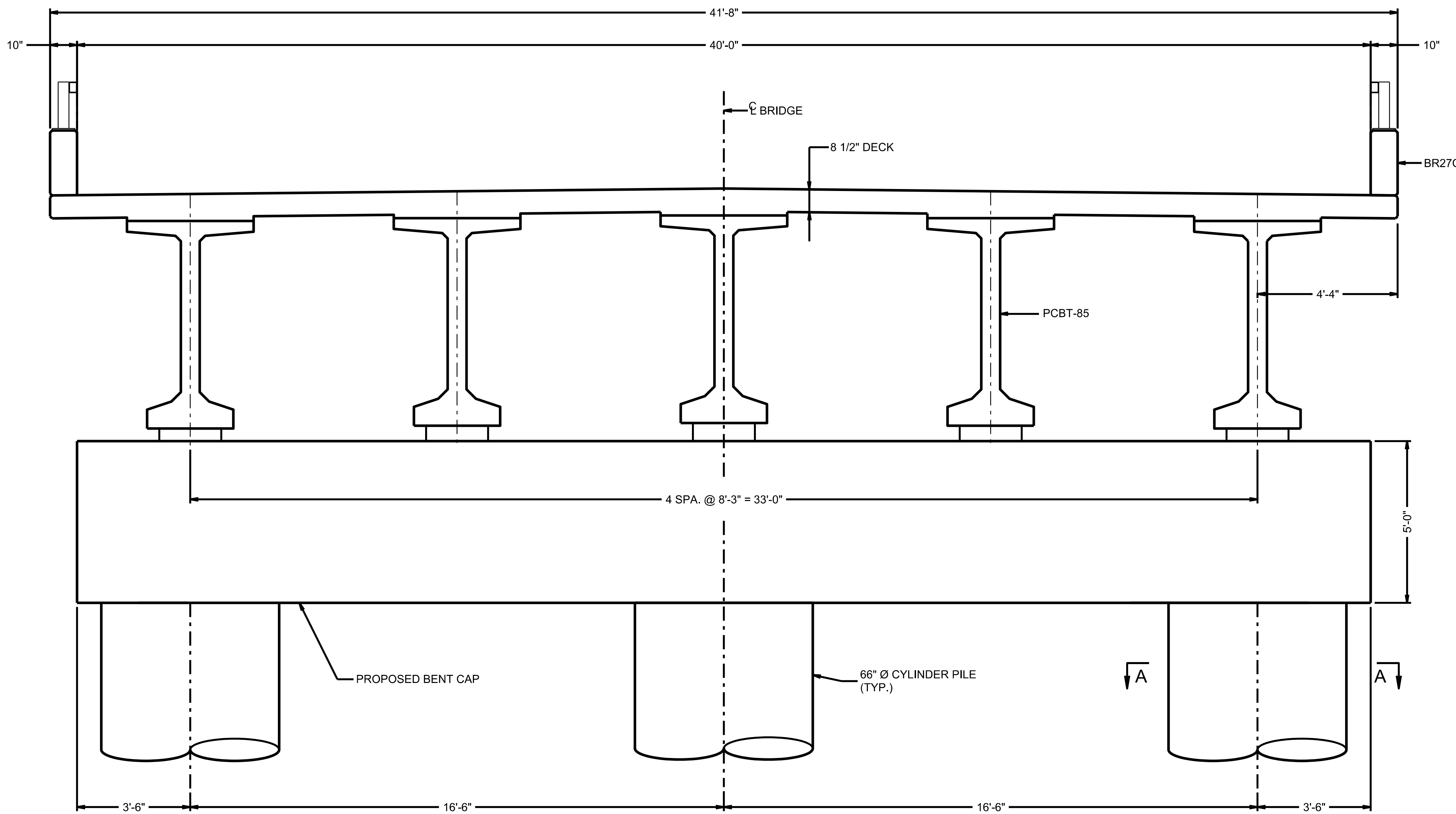
SOUTH APPROACH	MAIN CHANNEL	NORTH APPROACH
140' SPANS PIER HEIGHT RANGE = 18.8' - 34' PIER HEIGHT AVERAGE = 26.4' TOTAL # PIERS = 5 185' SPANS PIER HEIGHT RANGE = 36.4' - 104.7' PIER HEIGHT AVERAGE = 70.6' TOTAL # PIERS = 19	325' & 400' SPANS PIER HEIGHT RANGE = 108.8' - 110.0' PIER HEIGHT AVERAGE = 109.4' TOTAL # PIERS = 4	140' SPANS PIER HEIGHT RANGE = 14.4' - 54.4' PIER HEIGHT AVERAGE = 34.4' TOTAL # PIERS = 14 185' SPANS PIER HEIGHT RANGE = 56.6' - 105.7' PIER HEIGHT AVERAGE = 81.1' TOTAL # PIERS = 15

CONCEPT

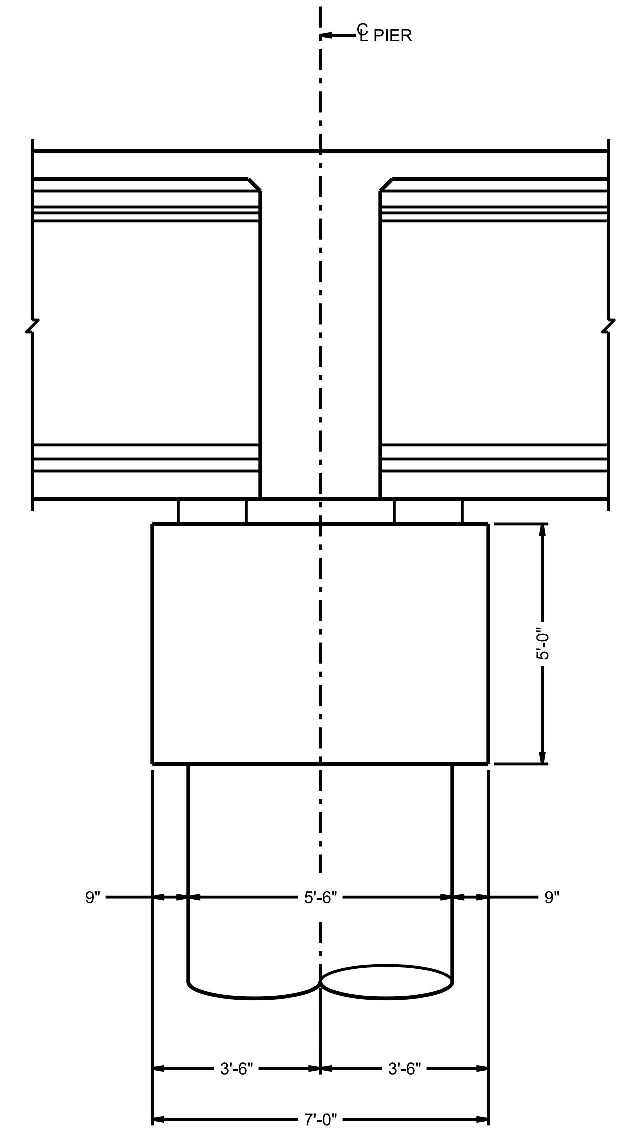
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

PROPOSED ELEVATION
REPLACEMENT OPTION 1

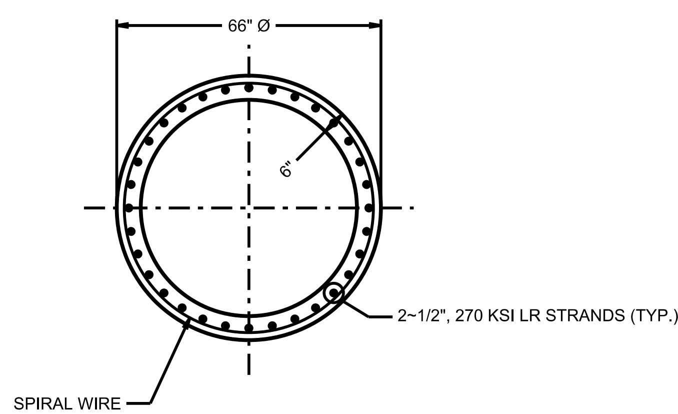
DRAWING
6-1



ELEVATION



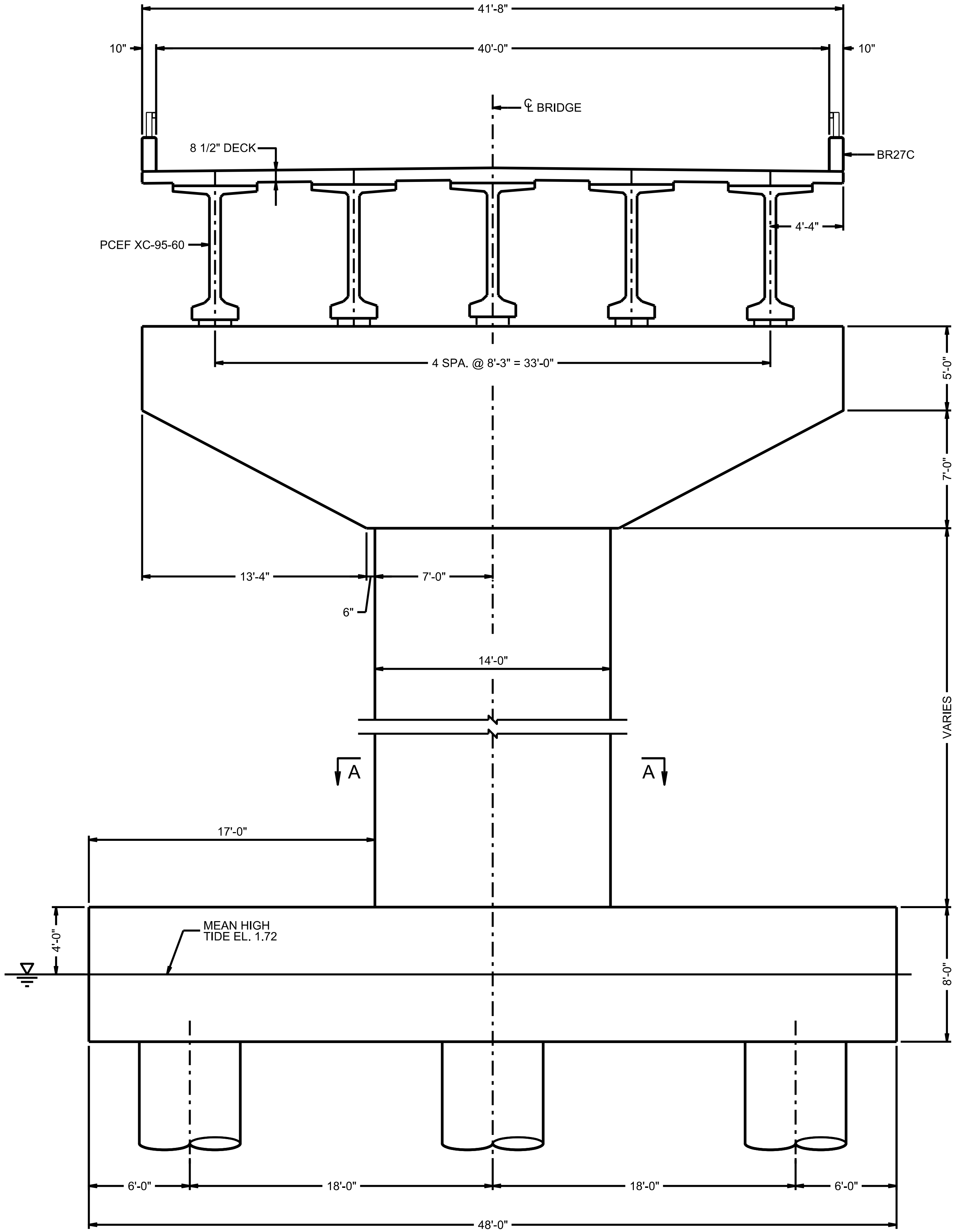
END VIEW



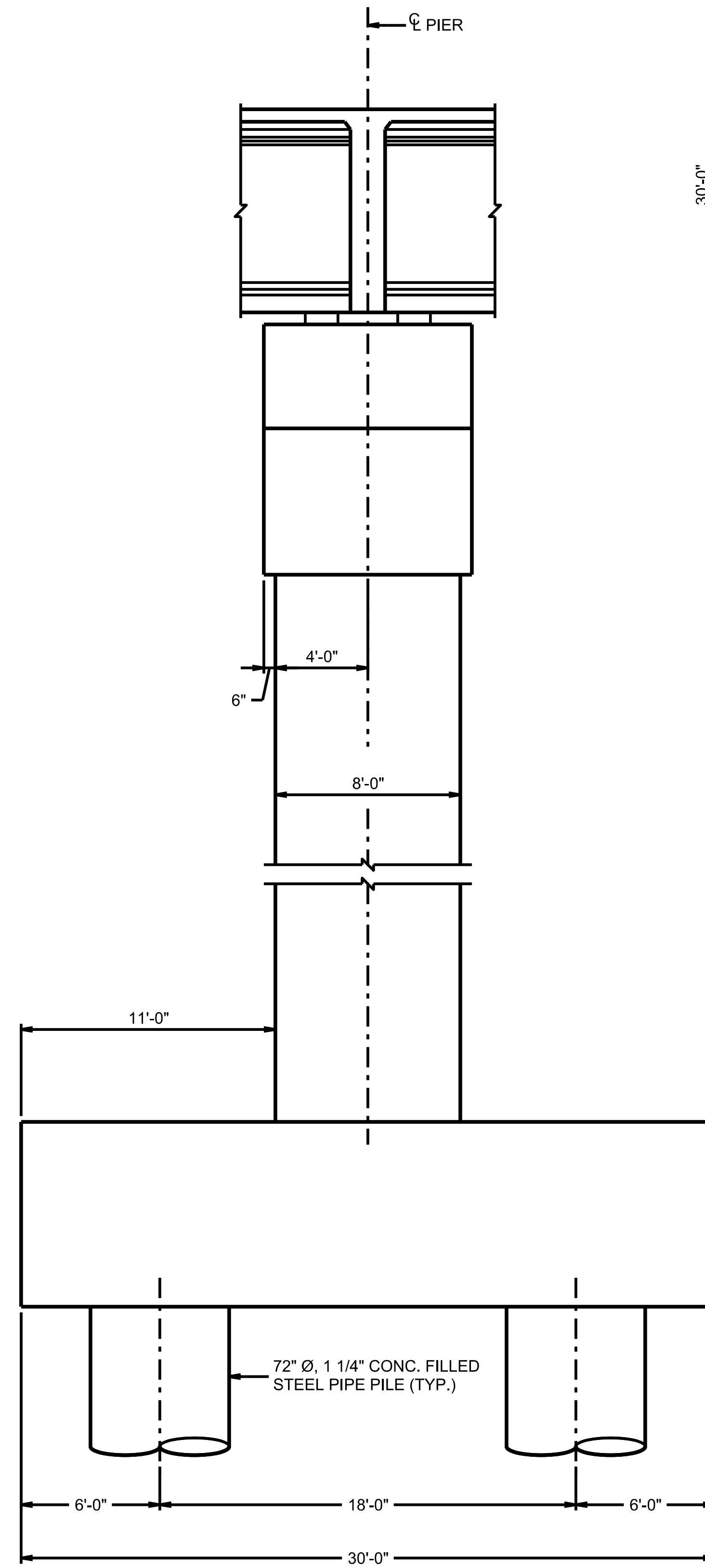
SECTION A-A

CONCEPT

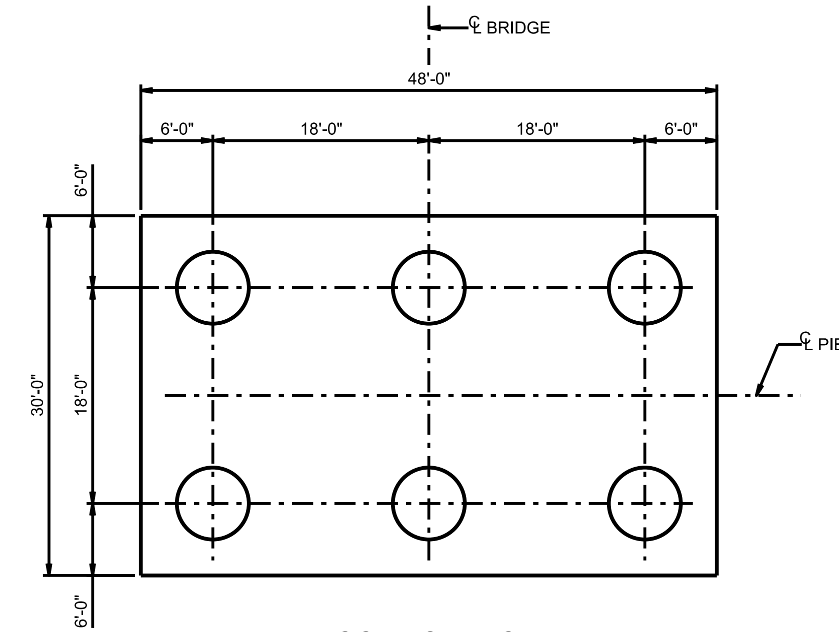
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
150' APPROACH SPANS REPLACEMENT OPTION 1	DRAWING 6-2



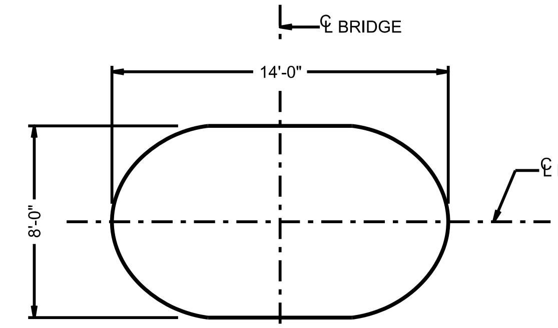
ELEVATION



END VIEW



FOOTING LAYOUT

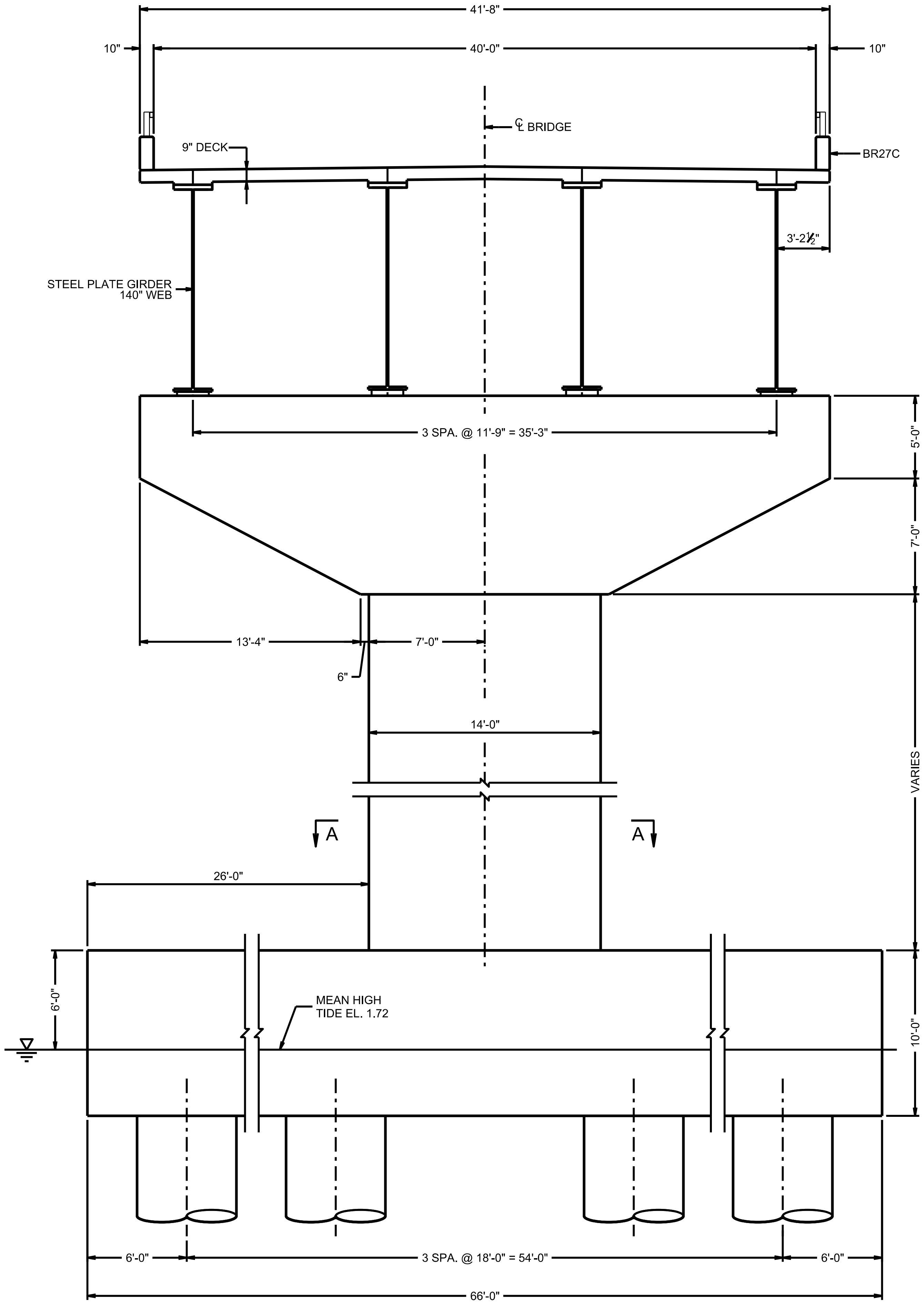


SECTION A-A

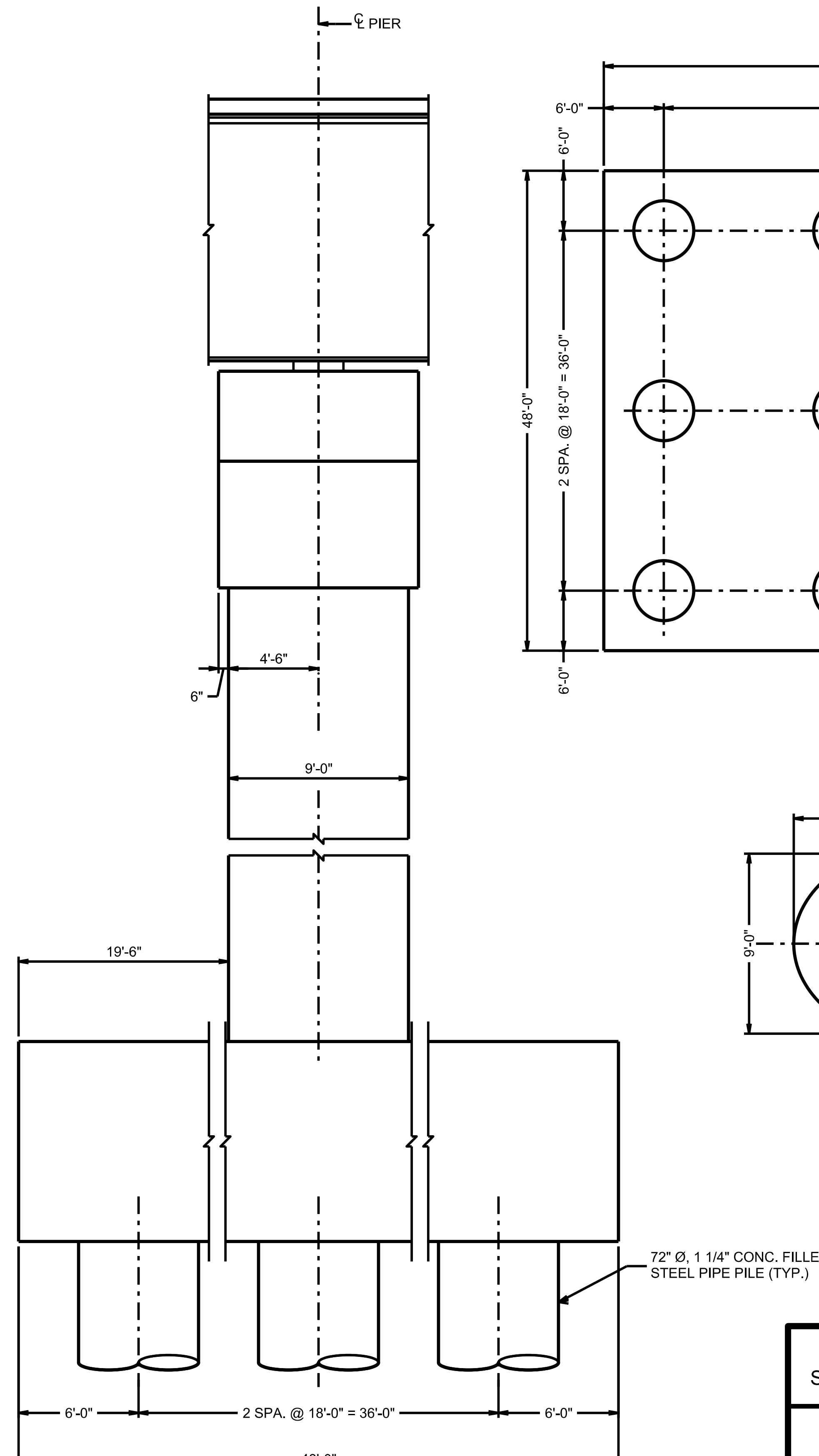
CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

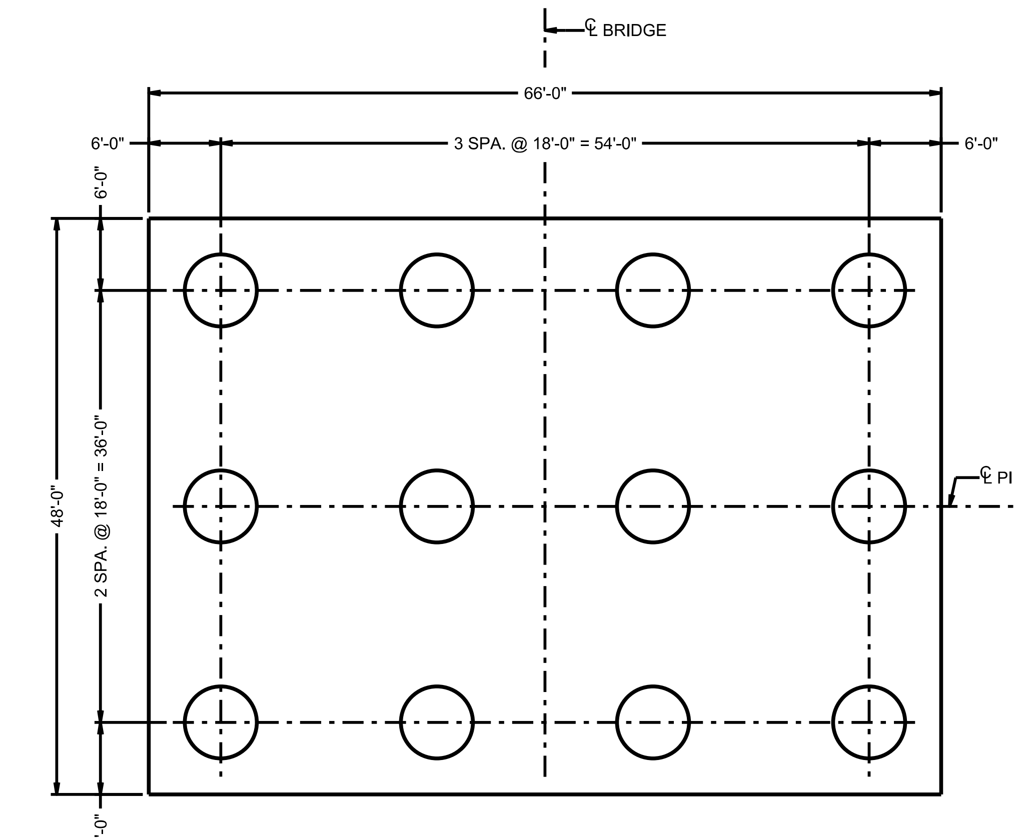
185' APPROACH SPANS REPLACEMENT OPTION 1	DRAWING 6-3



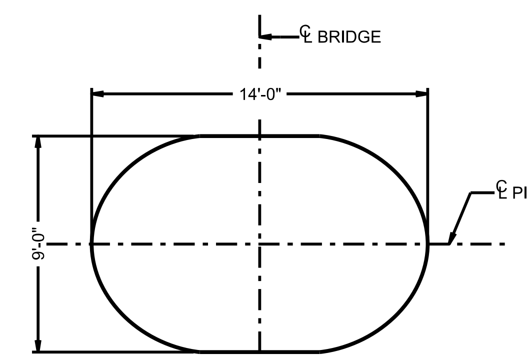
ELEVATION



END VIEW



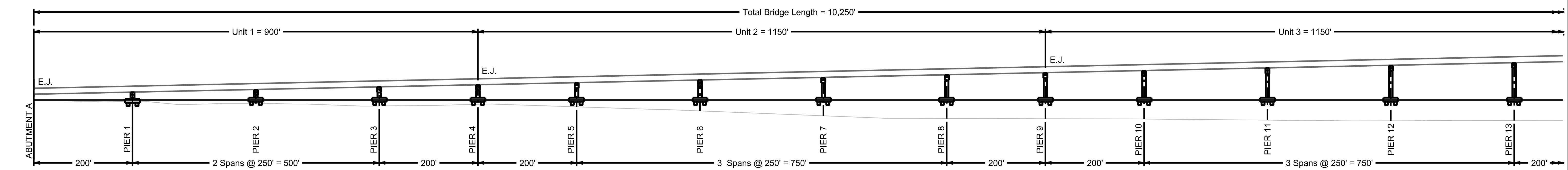
FOOTING LAYOUT



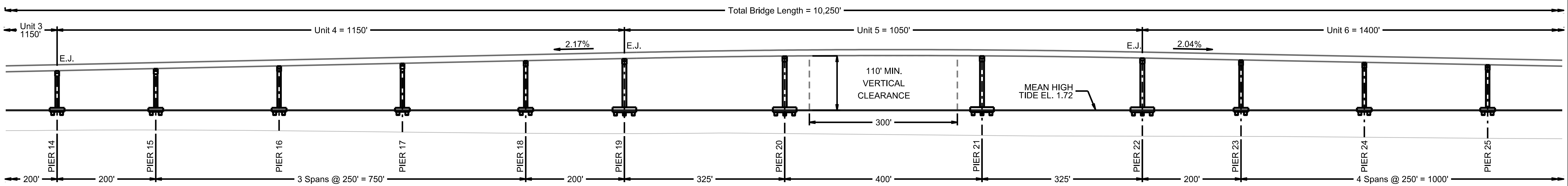
SECTION A-A

CONCEPT

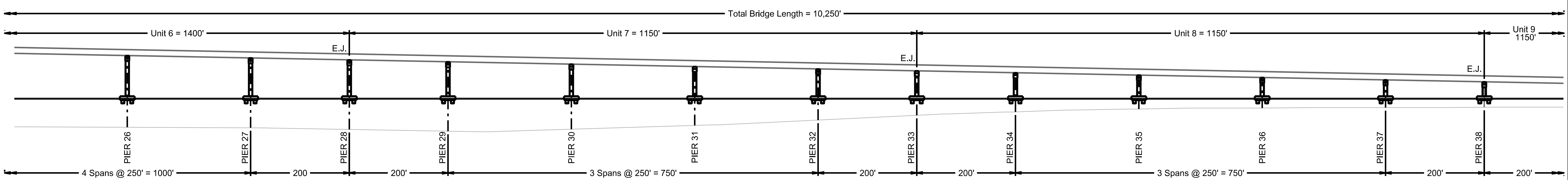
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
325' & 400' CHANNEL SPANS REPLACEMENT OPTION 1 & 2	DRAWING 6-4



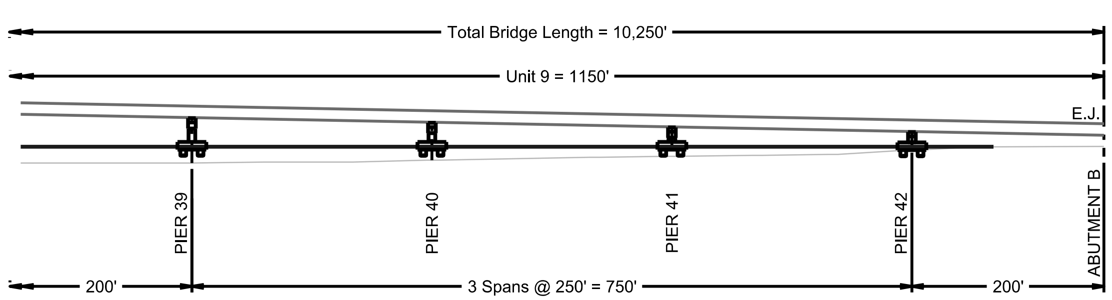
ELEVATION - ABUTMENT A THRU PIER 13



ELEVATION - PIERS 14 THRU 25



ELEVATION - PIERS 26 THRU 38



ELEVATION - PIER 39 THRU ABUTMENT B

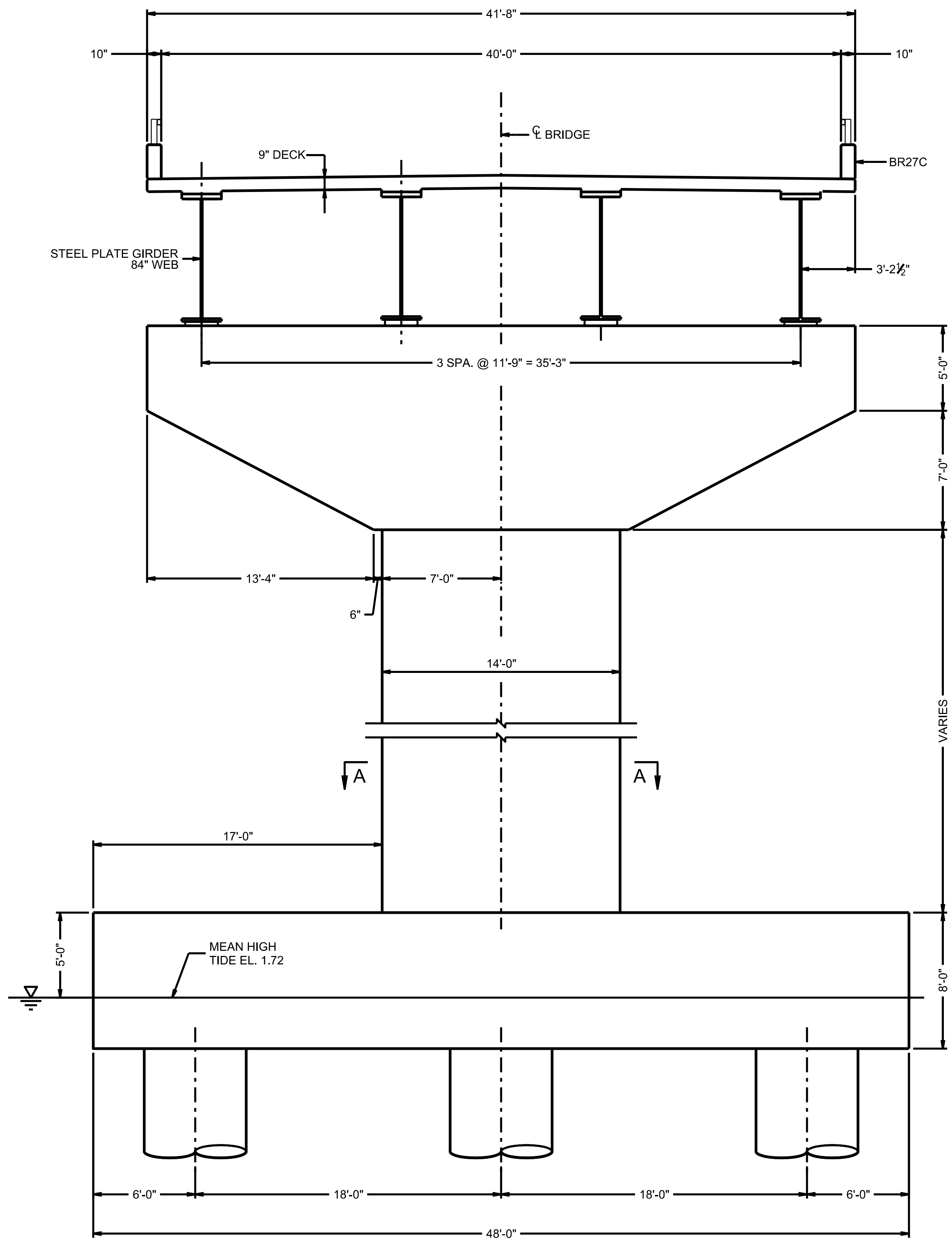
SOUTH APPROACH	MAIN CHANNEL	NORTH APPROACH
200' SPANS = 7 250' SPANS = 10 PIER HEIGHT RANGE = 18.6' - 104.4' PIER HEIGHT AVERAGE = 61.5' TOTAL # PIERS = 18	325' & 400' SPANS PIER HEIGHT RANGE = 108.8' - 110.0' PIER HEIGHT AVERAGE = 109.4' TOTAL # PIERS = 4	200' SPANS = 6 250' SPANS = 13 PIER HEIGHT RANGE = 14.6' - 105.4' PIER HEIGHT AVERAGE = 60.0' TOTAL # PIERS = 20

CONCEPT

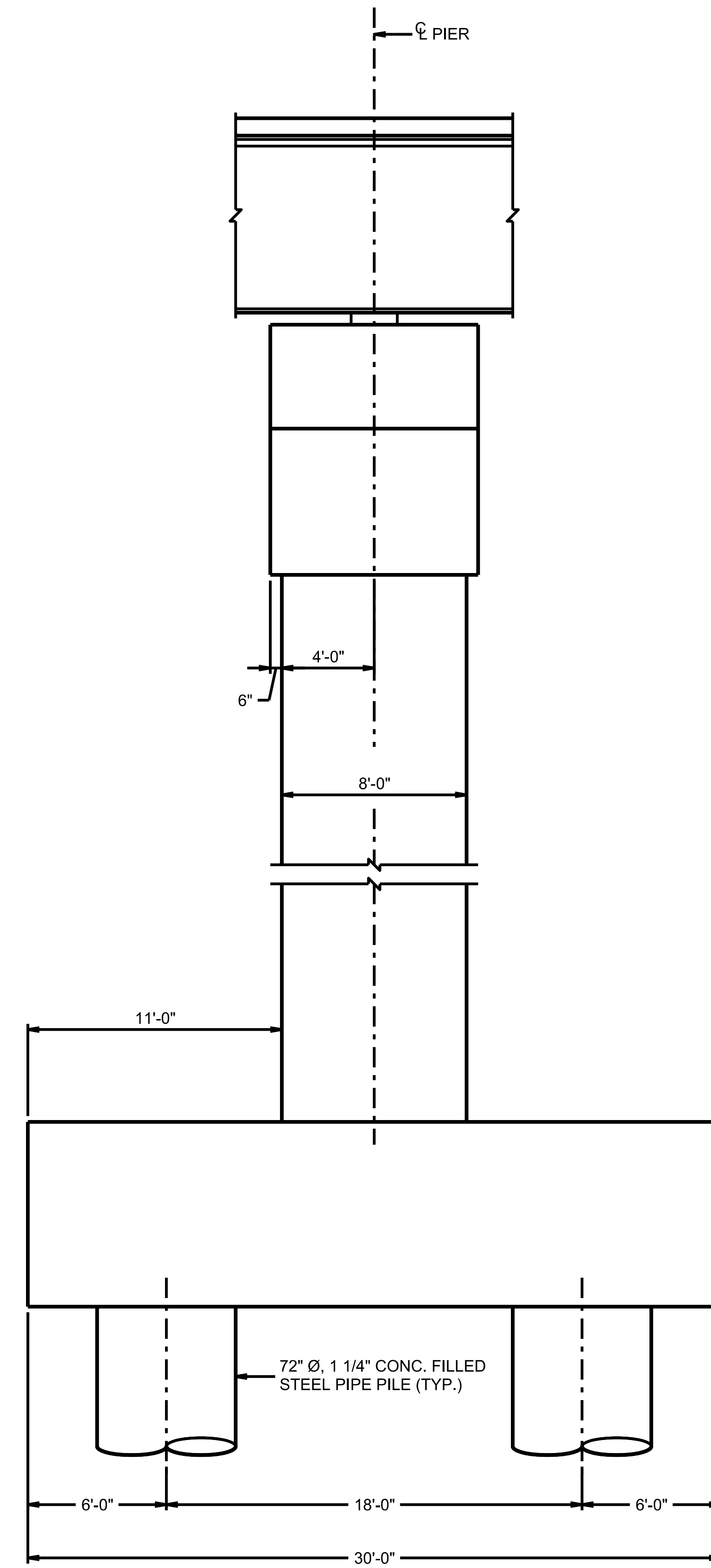
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

PROPOSED ELEVATION
REPLACEMENT OPTION 2

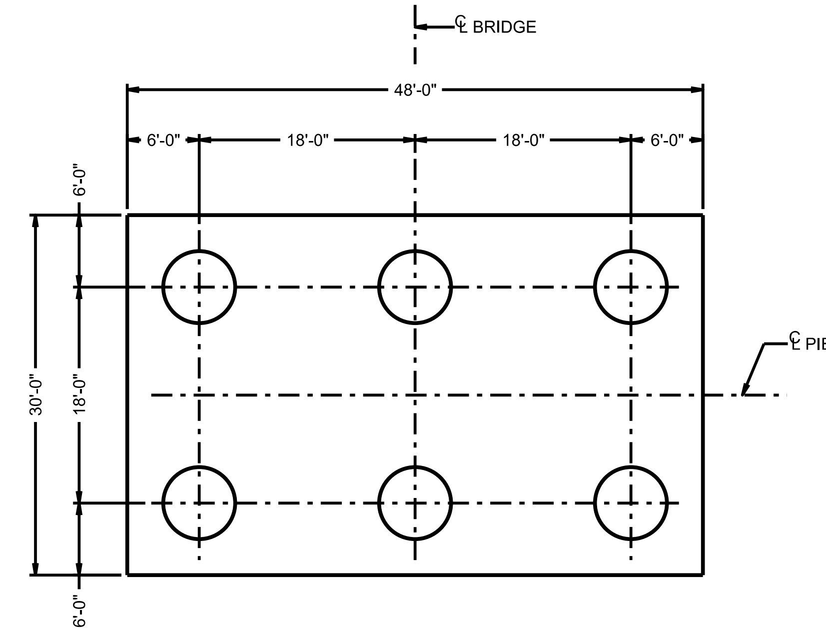
DRAWING
6-5



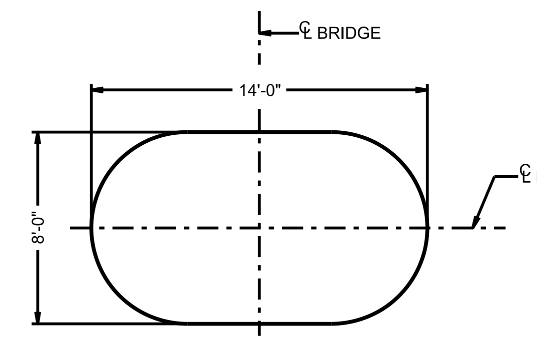
ELEVATION



END VIEW



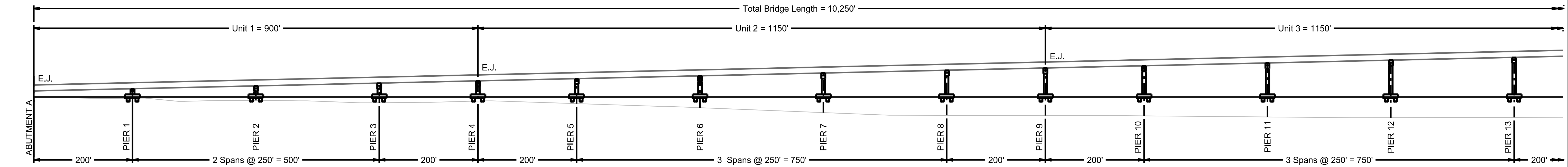
FOOTING LAYOUT



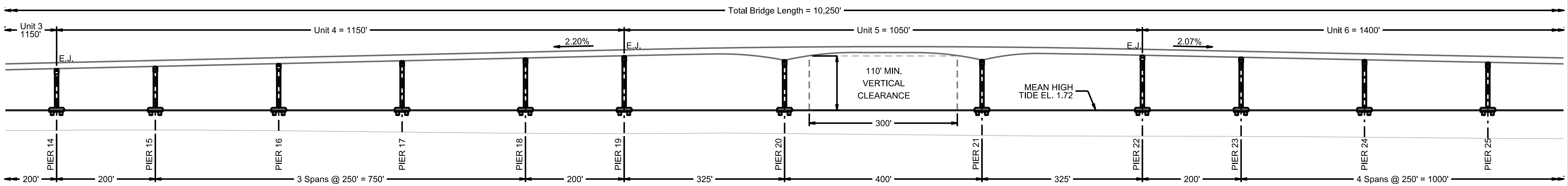
SECTION A-A

CONCEPT

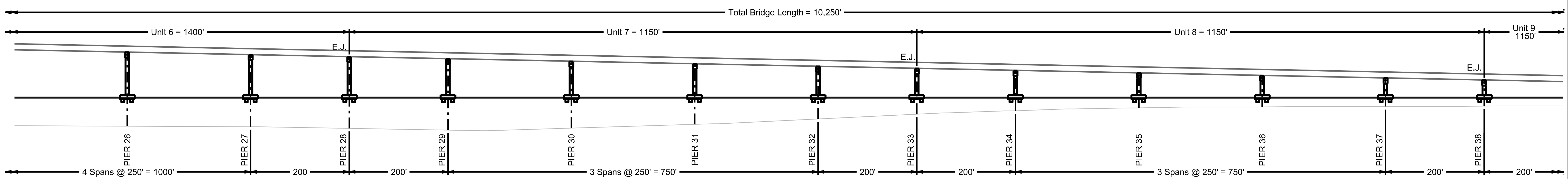
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
200 & 250' APPROACH SPANS REPLACEMENT OPTION 2	DRAWING 6-6



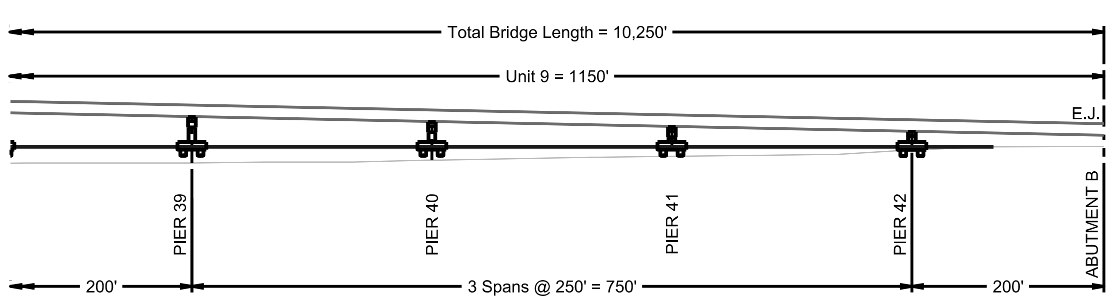
ELEVATION - ABUTMENT A THRU PIER 13



ELEVATION - PIERS 14 THRU 25



ELEVATION - PIERS 26 THRU 38



ELEVATION - PIER 39 THRU ABUTMENT B

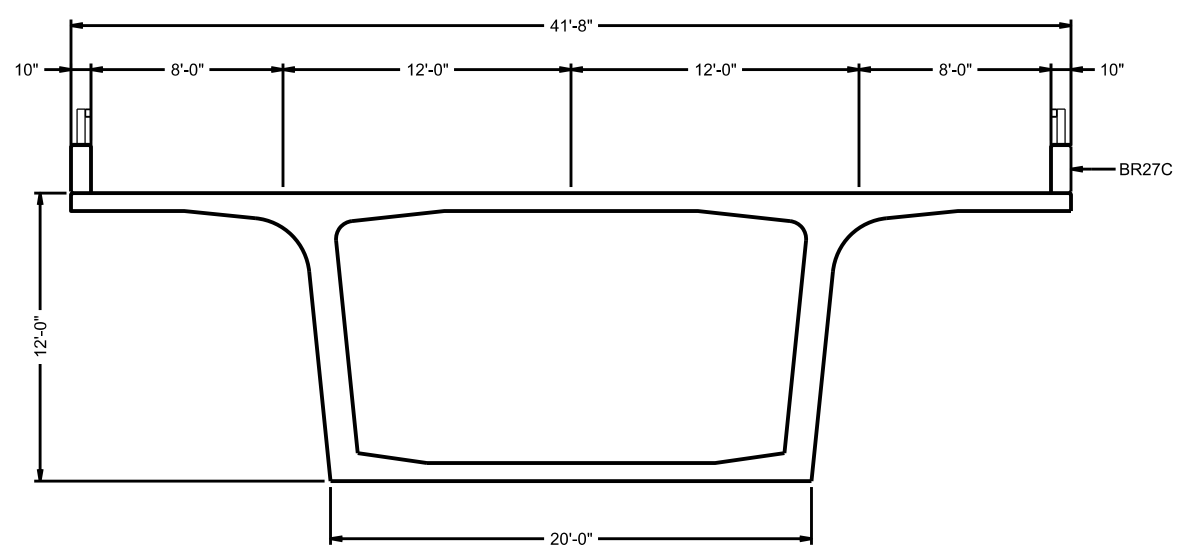
SOUTH APPROACH	MAIN CHANNEL	NORTH APPROACH
200' SPANS = 8	325' & 400' SPANS	200' SPANS = 8
250' SPANS = 11	PIER HEIGHT RANGE = 108.8' - 110.0'	250' SPANS = 13
PIER HEIGHT RANGE = 18.6' - 104.4'	PIER HEIGHT AVERAGE = 109.4'	PIER HEIGHT RANGE = 14.6' - 105.4'
PIER HEIGHT AVERAGE = 61.5'	TOTAL # PIERS = 4	PIER HEIGHT AVERAGE = 60.0'
TOTAL # PIERS = 18	VARIABLE DEPTH SEGMENTS = 101	TOTAL # PIERS = 20
TYPICAL SEGMENTS (10'-0") = 416	PIER SEGMENTS = 8	TYPICAL SEGMENTS (10'-0") = 464
PIER SEGMENTS = 36		PIER SEGMENTS - 40

CONCEPT

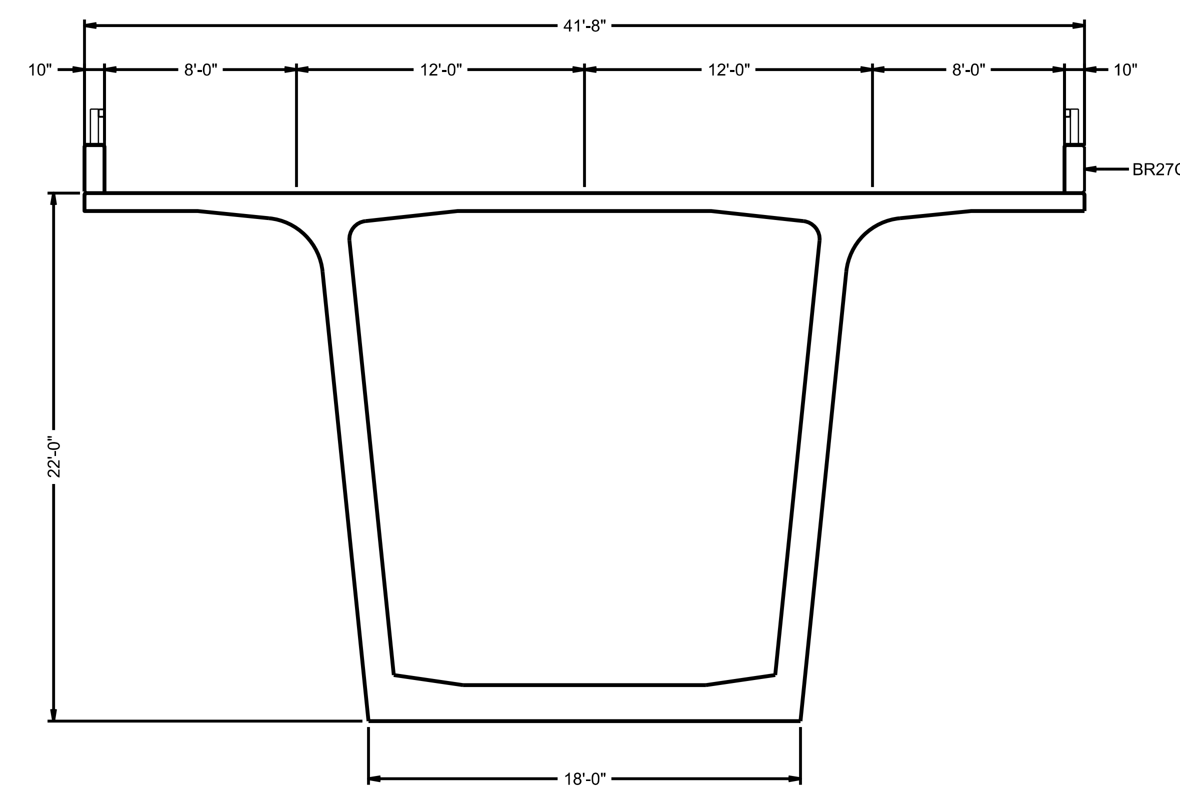
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER
SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY

PROPOSED ELEVATION
REPLACEMENT OPTION 3

DRAWING
6-7



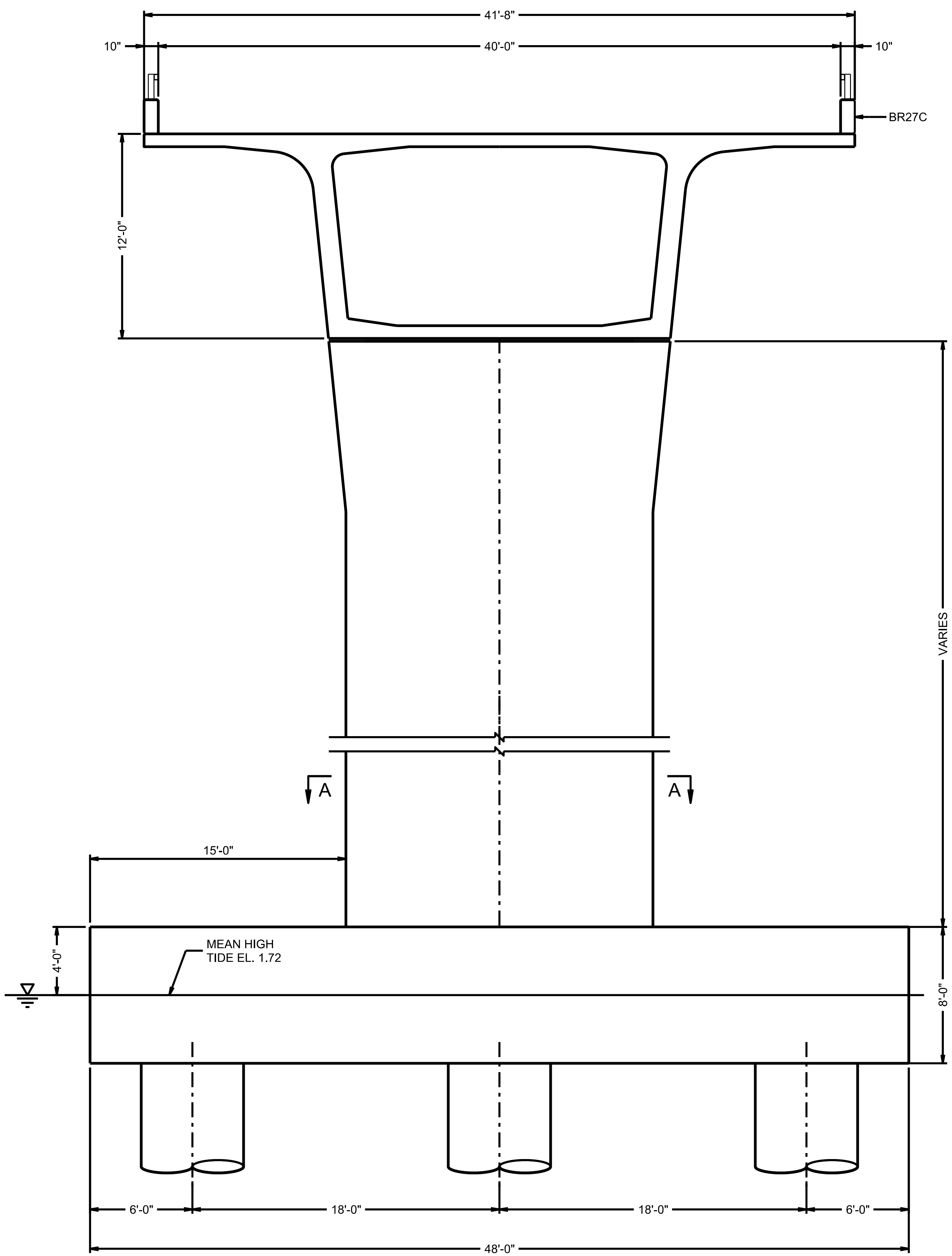
TYPICAL SECTION



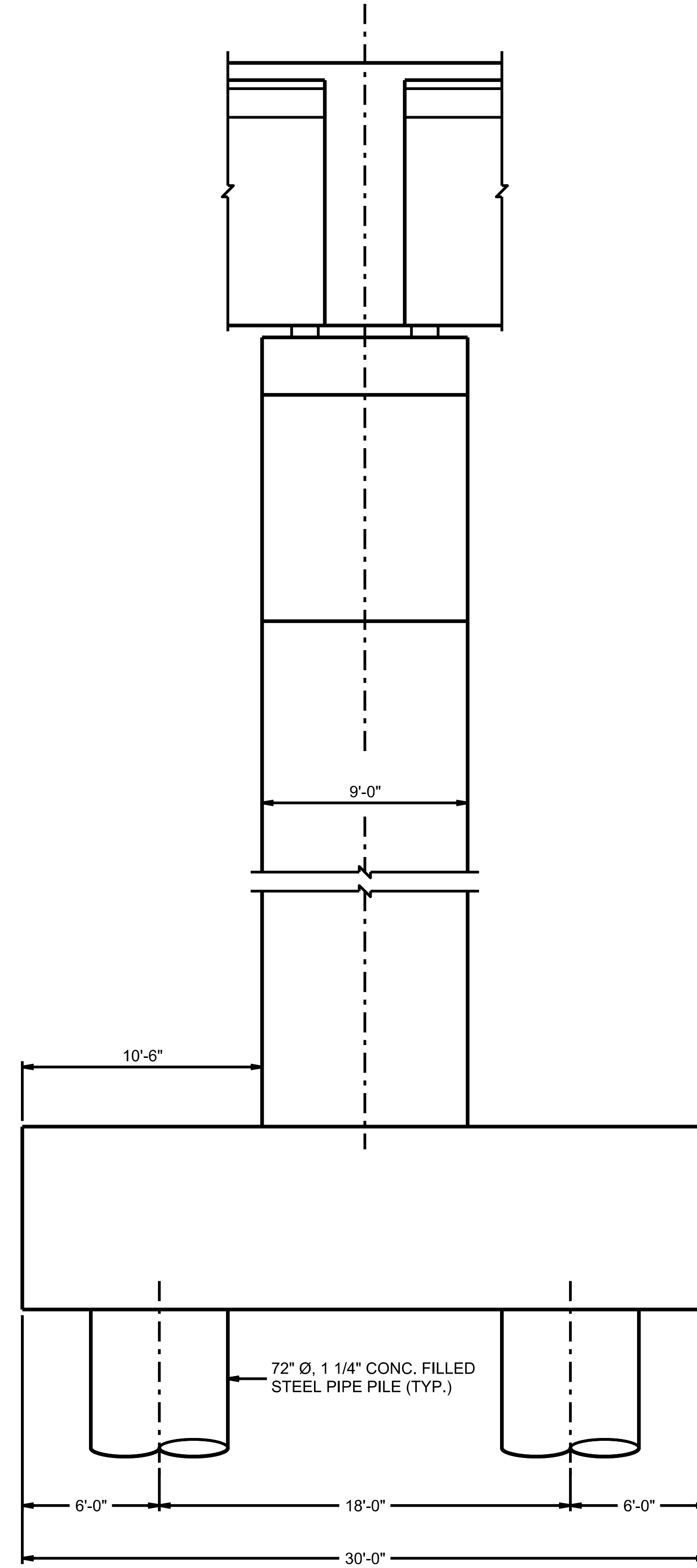
TYPICAL SECTION
AT CHANNEL SPAN PIERS

CONCEPT

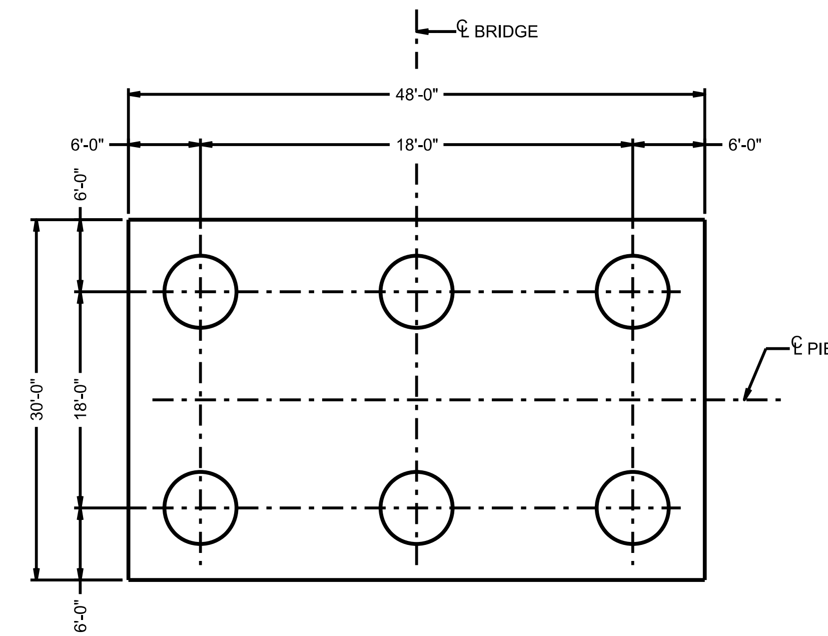
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED SECTIONS REPLACEMENT OPTION 3	DRAWING 6-8



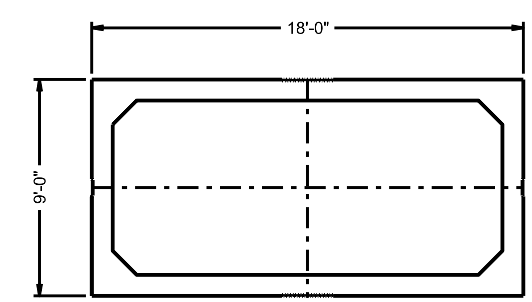
ELEVATION



END VIEW



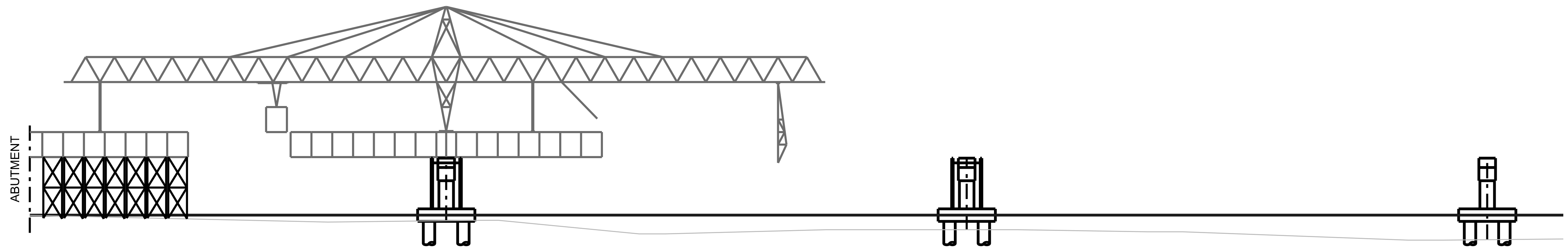
FOOTING LAYOUT



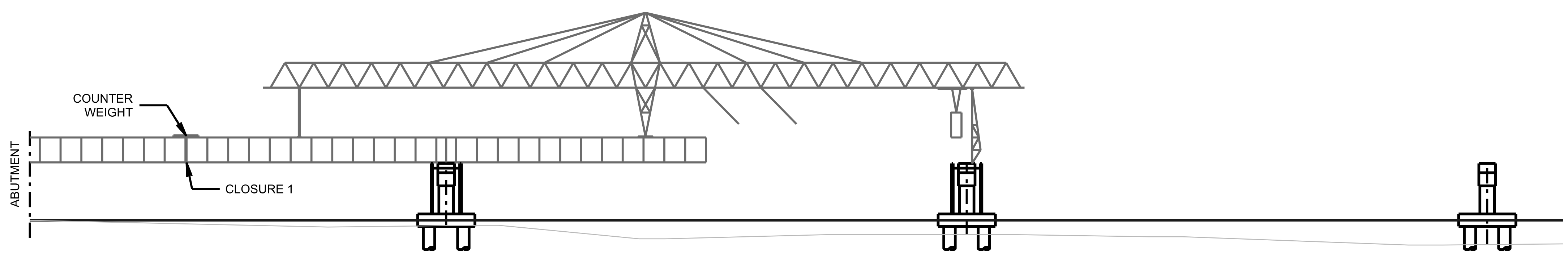
SECTION A-A

CONCEPT

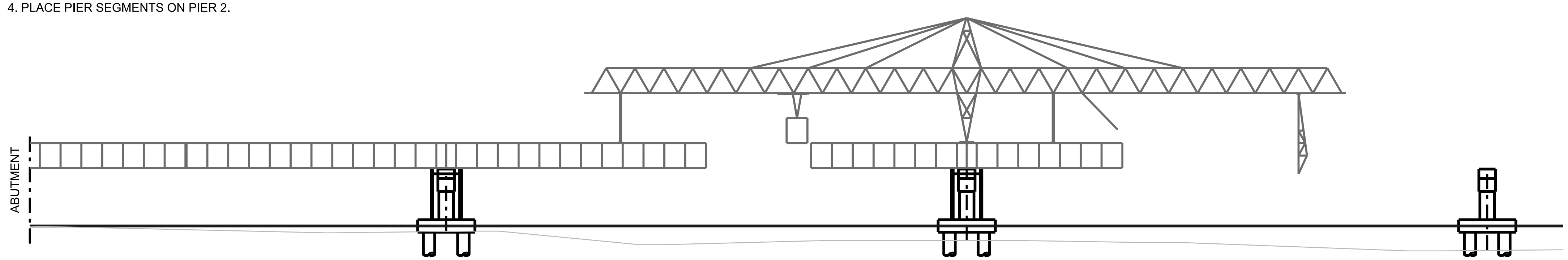
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED SECTIONS REPLACEMENT OPTION 3	DRAWING 6-9



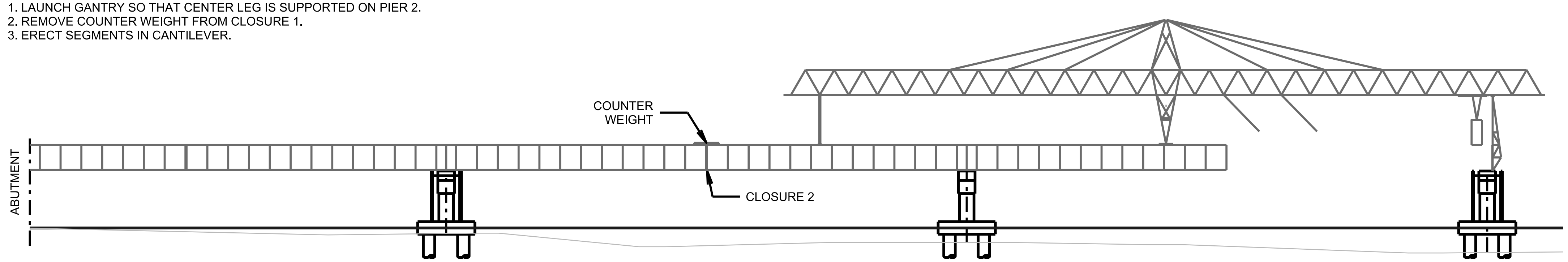
PHASE 1:
1. INSTALL FALSEWORK AT ABUTMENT A.
2. PLACE SEGMENTS ON FALSEWORK AND SLIDE INTO PLACE.
3. PLACE PIER TOWERS ON PIERS 1 AND 2.
4. PLACE PIER 1 PIER SEGMENTS.
5. LAUNCH GANTRY ONTO PIER 1.
6. ERECT SEGMENTS IN CANTILEVER FROM PIER 1, STRESSING CANTILEVER AND DRAPED TENDONS.



PHASE 2:
1. POUR CLOSURE 1 AND STRESS CONTINUITY TENDONS.
2. PLACE COUNTER WEIGHT OVER CLOSURE.
3. LAUNCH GANTRY SO THAT THE FRONT SUPPORT LEG IS ON PIER 2.
4. PLACE PIER SEGMENTS ON PIER 2.



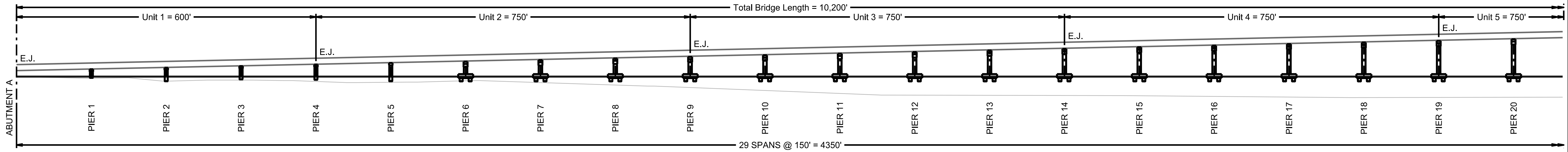
PHASE 3:
1. LAUNCH GANTRY SO THAT CENTER LEG IS SUPPORTED ON PIER 2.
2. REMOVE COUNTER WEIGHT FROM CLOSURE 1.
3. ERECT SEGMENTS IN CANTILEVER.



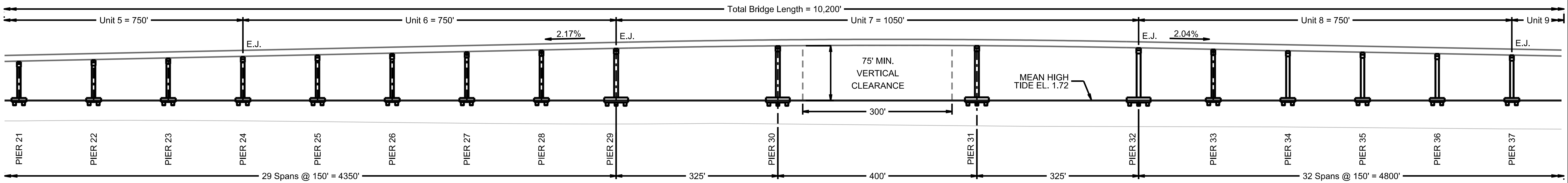
PHASE 4:
1. POUR CLOSURE 2 AND STRESS CONTINUITY TENDONS.
2. REPEAT PHASE II AND III FOR REMAINING PIERS THROUGH COMPLETION OF SUPERSTRUCTURE.

CONCEPT

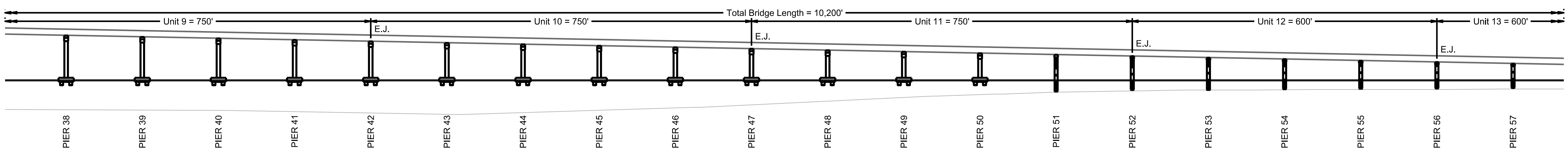
NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED CONSTRUCTION SEQUENCE REPLACEMENT OPTION 3	DRAWING 6-10



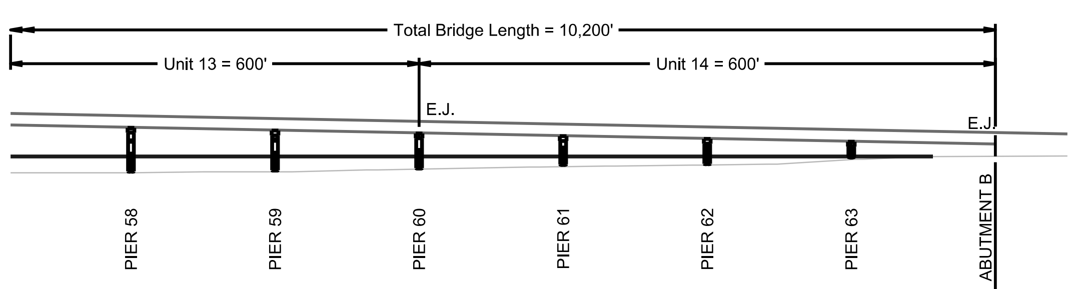
ELEVATION - ABUTMENT A THRU PIER 20



ELEVATION - PIERS 21 THRU 37



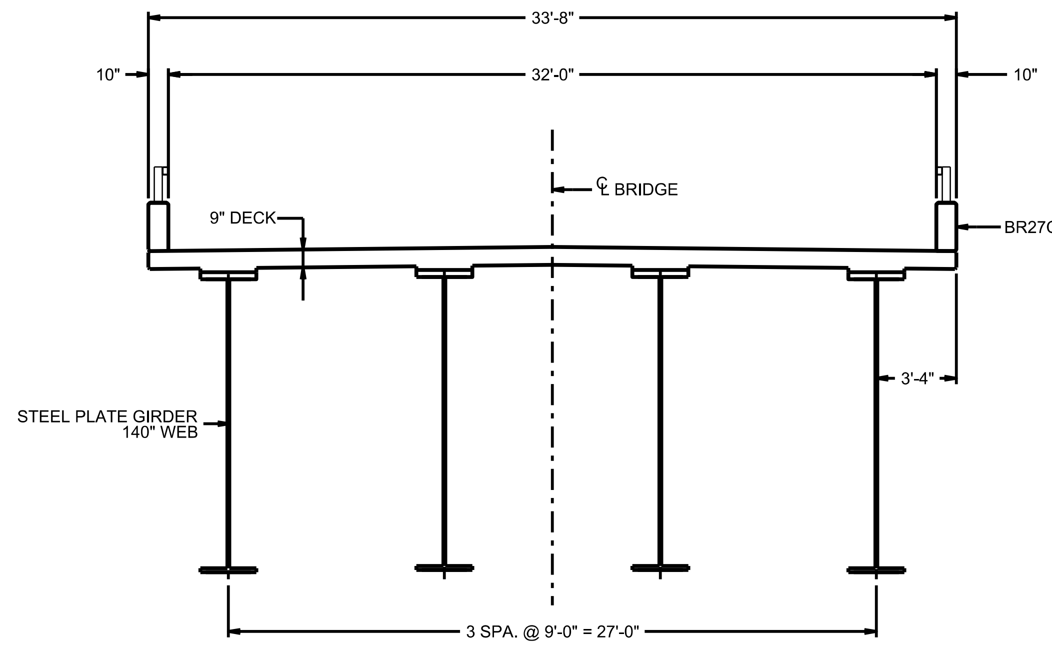
ELEVATION - PIERS 38 THRU 57



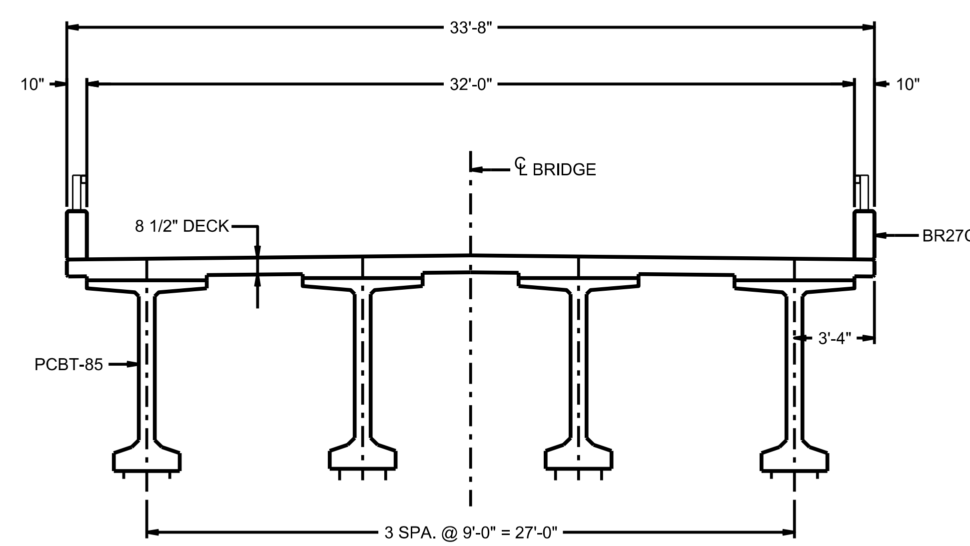
ELEVATION - PIER 58 THRU ABUTMENT B

CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
PROPOSED ELEVATION REPLACEMENT OPTION 7A	DRAWING 6-11



TYPICAL SECTION - 325' & 400'
CHANNEL SPANS UNIT



TYPICAL SECTION - 150'
APPROACH UNITS

CONCEPT

NORRIS BRIDGE OVER RAPPAHANNOCK RIVER SUPERSTRUCTURE REPLACEMENT CONCEPT STUDY	
TYPICAL SECTIONS REPLACEMENT OPTION 7A	DRAWING 6-12

Appendix C - Detailed Cost Estimates

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Temporary Works for Rapid Replacement
Method 2A: Slide In All Spans

Bid Description	Bid Quantity	Units		Total Cost	Unit Cost
TEMP TRANSFER BENTS; PIERS 1-16 (TRUSS SPANS)	***30***	***EA***	***$***	***82,549,323***	***$2,751,644***
48-IN BATTER PIPE PILES (0.75 WALL)	83,650	LF	$	61,960,452	$741
TEMP REINF CONCRETE CAPS	6,231	CY	$	4,684,729	$752
TEMP CONCRETE PEDESTALS	640	CY	$	1,124,539	$1,757
TEMP FENDERS @TEMP BENTS	90	EA	$	985,176	$10,946
SLIDE BEAMS	60	EA	$	8,265,460	$137,758
JACKING PLATFORMS	30	EA	$	908,850	$30,295
TEMP RAILINGS & FALL PROT	60	EA	$	1,269,748	$21,162
REMOVE ALL TEMP WORK	30	EA	$	3,350,369	$111,679
TEMP TRANSFER BENTS S. APPROACH	***7***	***EA***	***$***	***5,189,401***	***$741,343***
66-IN PCPS CYLINDER PILES	2,380	LF	$	1,151,615	$484
TEMP ABUTMENT 'A'	1	EA	$	67,008	$67,008
REINF CONCRETE CAPS (INCL PERM PORTION)	1,519	CY	$	1,423,340	$937
TEMP FENDERS @TEMP BENTS	14	EA	$	151,090	$10,792
FURN & INSTALL RAILS, ROLLERS & JACKS	70	EA	$	1,514,213	$21,632
JACKING PLATORMS	7	EA	$	212,067	$30,295
TEMP RAILINGS & FALL PROT	14	EA	$	222,206	$15,872
SAWCUT CAPS AFTER SLIDE	446	SF	$	68,475	$153
REMOVE ALL TEMP WORK	7	EA	$	379,388	$54,198
TEMP TRANSFER BENTS N. APPROACH	***19***	***EA***	***$***	***13,851,974***	***$729,051***
66-IN PCPS CYLINDER PILES	6,460	LF	$	3,124,450	$484
TEMP ABUTMENT 'B'	1	EA	$	67,008	$67,008
REINF CONCRETE CAPS (INCL PERM PORTION)	4,123	CY	$	3,787,946	$919
TEMP FENDERS @TEMP BENTS	38	EA	$	410,101	$10,792
FURN & INSTALL RAILS, ROLLERS & JACKS	190	EA	$	4,109,840	$21,631
JACKING PLATFORMS	19	EA	$	575,603	$30,295
TEMP RAILINGS & FALL PROT	38	EA	$	603,130	$15,872
SAWCUT CAPS AFTER SLIDE	1,212	SF	$	185,963	$153
REMOVE ALL TEMP WORK	19	EA	$	987,932	$51,996
TEMP APPROACH ROADWAYS (INSTALL & REMOVE)	***10,196***	***SF***	***$***	***138,205***	***$14***
CLEAR & GRUB	0	AC	$	4,946	$19,783
EMBANKMENT ALLOWANCE	1,200	CY	$	32,317	$27
10-IN DGA BASE	315	CY	$	21,205	$67
6-IN ASPHALT BASE COURSE	398	TONS	$	31,432	$79
3-IN ASPHALT TOP COURSE	133	TONS	$	11,484	$86
GUARDRAIL	860	LF	$	18,112	$21
REMOVE TEMP ROADWAYS	10,196	SF	$	18,710	$2
SLIDE OVER TRUSS SPANS (7,090 LF; 15 SPANS)	***1***	***LS***	***$***	***8,296,075***	***$8,296,075***
PURCH JACKS & APPURTS	60	EA	$	6,018,855	$100,314
INSTALL JACKS	60	EA	$	381,555	$6,359
PREP FOR SLIDE	15	EA	$	924,645	$61,643
PERFORM SLIDE OPERATION	3	EA	$	432,456	$144,152
REMOVE JACKS & CONTROLS	30	EA	$	538,564	$17,952
SLIDE OVER S. APPROACH SPANS (777 LF; 6 SPANS)	***1***	***LS***	***$***	***2,430,048***	***$2,430,048***
PURCH JACKS & APPURTS	14	EA	$	1,562,926	$111,638
INSTALL JACKS	14	EA	$	381,555	$27,254
PREP FOR SLIDE	6	EA	$	215,750	$35,958
PERFORM SLIDE OPERATION	1	EA	$	144,152	$144,152
REMOVE JACKS & CONTROLS	7	EA	$	125,665	$17,952
SLIDE OVER N. APPROACH SPANS (2,117 LF; 18 SPANS)	***1***	***LS***	***$***	***5,321,033***	***$5,321,033***
PURCH JACKS & APPURTS	38	EA	$	3,870,003	$101,842
INSTALL JACKS	38	EA	$	241,652	$6,359
PREP FOR SLIDE	18	EA	$	589,461	$32,748
PERFORM SLIDE OPERATION	1	EA	$	278,827	$278,827
REMOVE JACKS & CONTROLS	19	EA	$	341,090	$17,952
TOTAL DIRECT COSTS			**$**	**117,776,059**	
INDIRECT ALLOWANCE	15.00%		$	17,666,409	
SUBTOTAL			**$**	**135,442,468**	
OVERHEAD/PROFIT/RISK	21.00%		$	28,442,918	
SUBTOTAL			**$**	**163,885,386**	

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Temporary Works for Rapid Replacement
Method 2B: Slide In Truss Span Only

Bid Description	Bid Quantity	Units		Total Cost		Unit Cost
TEMP TRANSFER BENTS; PIERS 1-16 (TRUSS SPANS)	***30***	***EA***	***$***	***82,549,323***	***$***	***2,751,644***
48-IN BATTER PIPE PILES (0.75 WALL)	83,650	LF	$	61,960,452	$	741
TEMP REINF CONCRETE CAPS	6,231	CY	$	4,684,729	$	752
TEMP CONCRETE PEDESTALS	640	CY	$	1,124,539	$	1,757
TEMP FENDERS @TEMP BENTS	90	EA	$	985,176	$	10,946
SLIDE BEAMS	60	EA	$	8,265,460	$	137,758
JACKING PLATFORMS	30	EA	$	908,850	$	30,295
TEMP RAILINGS & FALL PROT	60	EA	$	1,269,748	$	21,162
REMOVE ALL TEMP WORK	30	EA	$	3,350,369	$	111,679
TEMP BRIDGE S. APPROACH - FURN/INST/REM	***778***	***LF***	***$***	***4,161,667***	***$***	***5,349***
24-IN BATTER PIPE PILES (0.63 WALL)	6,308	LF	$	1,870,239	$	296
TEMP ABUTMENT 'A'	1	EA	$	33,504	$	33,504
PC PILE CAPS	7	EA	$	350,999	$	50,143
FURN & INSTALL TEMP BRIDGE ON S. APPROACH	778	LF	$	1,332,798	$	1,713
REMOVE ALL TEMP BRIDGE ELEMENTS	778	LF	$	574,127	$	738
TEMP BRIDGE N. APPROACH - FURN/INST/REM	***2,118***	***LF***	***$***	***11,150,497***	***$***	***5,265***
24-IN BATTER PIPE PILES (0.63 WALL)	16,916	LF	$	4,997,150	$	295
TEMP ABUTMENT 'B'	1	EA	$	33,504	$	33,504
PC PILE CAPS	19	EA	$	952,709	$	50,143
FURN & INSTALL TEMP BRIDGE ON N. APPROACH	2,118	LF	$	3,658,649	$	1,727
REMOVE ALL TEMP BRIDGE ELEMENTS	2,118	LF	$	1,508,485	$	712
TEMP APPROACH ROADWAYS	***10,196***	***SF***	***$***	***138,205***	***$***	***14***
CLEAR & GRUB	0	AC	$	4,946	$	19,783
EMBANKMENT ALLOWANCE	1,200	CY	$	32,317	$	27
10-IN DGA BASE	315	CY	$	21,205	$	67
6-IN ASPHALT BASE COURSE	398	TONS	$	31,432	$	79
3-IN ASPHALT TOP COURSE	133	TONS	$	11,484	$	86
GUARDRAIL	860	LF	$	18,112	$	21
REMOVE TEMP ROADWAYS	10,196	SF	$	18,710	$	2
SLIDE OVER TRUSS SPANS (7,090 LF; 15 SPANS)	***1***	***LS***	***$***	***8,282,269***	***$***	***8,282,269***
PURCH JACKS & APPURTS	60	EA	$	6,018,855	$	100,314
INSTALL JACKS	60	EA	$	381,555	$	6,359
PREP FOR SLIDE	15	EA	$	924,645	$	61,643
PERFORM SLIDE OPERATION	3	EA	$	432,456	$	144,152
REMOVE JACKS & CONTROLS	30	EA	$	524,757	$	17,492
TOTAL DIRECT COSTS			**$**	**106,281,961**		
INDIRECT ALLOWANCE	15.00%		$	15,942,294		
SUBTOTAL			**$**	**122,224,255**		
OVERHEAD/PROFIT/RISK	21.00%		$	25,667,093		
SUBTOTAL			**$**	**147,891,348**		

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative A: 24-foot Width Full Length - Truss

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	7,493	$700	$5,245,282
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	1,873,320	$4.00	$7,493,280
	Bridge Deck Grooving	SY	26,627	$7.00	$186,387
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	1,632,603	$2.10	$3,428,466
	Structural Steel Rolled Beam, ASTM A709, 50W, Duplex	LB	1,316,429	$2.10	$2,764,501
	Structural Steel Truss, ASTM A709, 50W/70W, Duplex	LB	14,259,264	$2.40	$34,222,232
	Tooth Expansion Joint Device	LF	154	$2,200	$338,800
	Modular Expansion Joint Device	LF	154	$4,000	$616,001
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$61,339,111
				Cost/SQFT	$239.31
Substructure	Concrete, 4000 psi (Cap), Class A4	CY	261	$1,200	$313,333
	Corrosion Resistant Reinforcing Steel, Class I	LB	52,230	$2.10	$109,683
	Dowel Holes w/Anchor for Reinforcing Steel	LF	3,200	$100.00	$320,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	110	$1,000	$110,000
	HLMR Bearing with Lock-Up Device	EA	40	$75,000	$3,000,000
				Subtotal	$5,453,016
				Cost/SQFT	$21.27

Mobilization & Demolition	
Mobilization	$3,339,606.36
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$13,823,856
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency" Subtotal	$45,715,421
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$6,316,570
Owner PM/CM (12%)	$15,159,769
Design Build Engineering (12%)	$15,159,769
Subtotal	$37,636,107
Total Project Cost	**$311,928,634**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative B: 40-foot Width Approach Spans & 24-foot Width Channel Span Unit - Truss

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	11,279	$700	$7,895,190
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,819,720	$4.00	$11,278,880
	Bridge Deck Grooving	SY	41,556	$7.00	$290,895
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Truss, ASTM A709, 50W/70W, Duplex	LB	20,972,239	$2.40	$50,333,373
	Prestressed Concrete Girder (77" - 130 ft)	EA	110	$50,000	$5,500,000
	Tooth Expansion Joint Device	LF	250	$2,200	$550,000
	Modular Expansion Joint Device	LF	167	$4,000	$666,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$83,559,166
				Cost/SQFT	$259.18
Substructure	66" Diameter Conc. Filled Steel Pipe Piles (Piers)	LF	9,900	$900	$8,910,000
	16" Diameter Conc. Filled Steel Pipe Piles (Abutment)	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	22	$3,000	$66,000
	Concrete, 4000 psi (Cap), Class A4	CY	1,402	$1,200	$1,682,222
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	320,880	$2.10	$673,848
	Dowel Holes w/Anchor for Reinforcing Steel	LF	3,200	$100.00	$320,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	220	$1,000	$220,000
	HLMR Bearing with Lock-Up Device	EA	40	$75,000	$3,000,000
				Subtotal	$19,542,070
				Cost/SQFT	$60.61

Mobilization & Demolition	
Mobilization	$5,155,062
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$15,639,312
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency" Subtotal	$53,340,334
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$8,604,044
Owner PM/CM (12%)	$20,649,706
Design Build Engineering (12%)	$20,649,706
Subtotal	$50,903,456
Total Project Cost	**$370,945,461**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative C: 40-foot Width Approach & Channel Span Units - Deck / Through Truss

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	12,118	$700	$8,482,927
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	3,029,620	$4.00	$12,118,480
	Bridge Deck Grooving	SY	44,378	$7.00	$310,644
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Truss, ASTM A709, 50W/70W, Duplex	LB	23,918,769	$2.40	$57,405,046
	Prestressed Concrete Girder (77" - 130 ft)	EA	110	$50,000	$5,500,000
	Tooth Expansion Joint Device	LF	250	$2,200	$550,000
	Modular Expansion Joint Device	LF	167	$4,000	$666,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$92,077,926
				Cost/SQFT	$221.30
Substructure	66" Diameter Conc. Filled Steel Pipe Piles	LF	9,900	$900	$8,910,000
	16" Diameter Conc. Filled Steel Pipe Piles	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	22	$3,000	$66,000
	Concrete, 4000 psi (Cap), Class A4	CY	1,891	$1,200	$2,269,422
	Concrete, 4000 psi (Column), Class A4	CY	780	$1,200	$935,733
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	590,290	$2.10	$1,239,609
	Dowel Holes w/Anchor for Reinforcing Steel	LF	4,200	$100.00	$420,000
	Pier 8-11 Demolition (Columns/Cap)	EA	4	$200,000.00	$800,000
	Column Fiber Wrap	Column	28	$50,000.00	$1,400,000
	Elastomeric Bearing	EA	220	$1,000	$220,000
	HLMR Bearing with Lock-Up Device	EA	40	$75,000	$3,000,000
				Subtotal	$22,330,765
				Cost/SQFT	$53.67

Mobilization & Demolition	
Mobilization	$5,720,435
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$16,204,685
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency[2] Subtotal	$55,714,899
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$9,316,414
Owner PM/CM (12%)	$22,359,393
Design Build Engineering (12%)	$22,359,393
Subtotal	$55,035,200

Total Project Cost	**$389,324,596**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative D: 40-foot Width Approach & Channel Span Units - Deck Truss

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	12,118	$700	$8,482,927
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	3,029,620	$4.00	$12,118,480
	Bridge Deck Grooving	SY	44,378	$7.00	$310,644
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Truss, ASTM A709, 50W/70W, Duplex	LB	23,257,686	$2.40	$55,818,447
	Prestressed Concrete Girder (77" - 130 ft)	EA	110	$50,000	$5,500,000
	Tooth Expansion Joint Device	LF	167	$2,200	$366,667
	Modular Expansion Joint Device	LF	167	$4,000	$666,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$90,307,993
				Cost/SQFT	$217.05
Substructure	66" Diameter Conc. Filled Steel Pipe Piles	LF	9,900	$900	$8,910,000
	16" Diameter Conc. Filled Steel Pipe Piles	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	22	$3,000	$66,000
	Concrete, 4000 psi (Cap), Class A4	CY	1,402	$1,200	$1,682,222
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	320,880	$2.10	$673,848
	Dowel Holes w/Anchor for Reinforcing Steel	LF	3,200	$100.00	$320,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	220	$1,000	$220,000
	HLMR Bearing with Lock-Up Device	EA	36	$75,000	$2,700,000
				Subtotal	$19,242,070
				Cost/SQFT	$46.25

Mobilization & Demolition	
Mobilization	$5,477,503
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$15,961,753
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency[2] Subtotal	$54,694,588
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$9,010,320
Owner PM/CM (12%)	$21,624,769
Design Build Engineering (12%)	$21,624,769
Subtotal	$53,259,857

Total Project Cost	**$381,427,384**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative E: 40-foot Width Approach & Channel Span Units - Delta Girders / Tied-Arch

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	12,064	$700	$8,445,013
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	3,016,080	$4.00	$12,064,320
	Bridge Deck Grooving	SY	44,378	$7.00	$310,644
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Delta Girder, ASTM A709, 50W, Duplex	LB	29,716,690	$2.60	$77,263,395
	Structural Steel Tied Arch, ASTM A709, 50W/70W, Duplex	LB	1,991,668	$3.20	$6,373,338
	Concrete, 10000 psi (Arch Tie)	CY	744	$1,000.00	$743,704
	Post-Tensioning - 0.6" ASTM A416, Grade 270 (Arch Tie)	LB	8,525	$6.00	$51,149
	Prestressed Concrete Girder (77" - 130 ft)	EA	90	$50,000	$4,500,000
	Tooth Expansion Joint Device	LF	250	$2,200	$550,000
	Modular Expansion Joint Device	LF	167	$4,000	$666,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$118,012,392
				Cost/SQFT	$283.63
Substructure	66" Diameter Conc. Filled Steel Pipe Piles	LF	8,100	$900	$7,290,000
	16" Diameter Conc. Filled Steel Pipe Piles	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	18	$3,000	$54,000
	Concrete, 4000 psi (Cap), Class A4	CY	1,848	$1,200	$2,217,311
	Concrete, 4000 psi (Column), Class A4	CY	1,344	$1,200	$1,613,333
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	705,830	$2.10	$1,482,243
	Dowel Holes w/Anchor for Reinforcing Steel	LF	2,600	$100.00	$260,000
	Pier 8-11 Demolition (Columns/Cap)	EA	4	$200,000.00	$800,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	180	$1,000	$180,000
	HLMR Bearings	EA	66	$10,000	$660,000
				Subtotal	$19,226,887
				Cost/SQFT	$46.21

Mobilization & Demolition	
Mobilization	$6,861,964
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$17,346,214
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency" Subtotal	$60,509,323
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$10,754,741
Owner PM/CM (12%)	$25,811,378
Design Build Engineering (12%)	$25,811,378
Subtotal	$63,377,497

Total Project Cost	**$426,433,435**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative D1: 30-foot Width Approach & Channel Span Units - Deck Truss

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	9,210	$700	$6,447,026
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,302,510	$3.50	$8,058,785
	Bridge Deck Grooving	SY	33,283	$7.00	$232,983
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Truss, ASTM A709, 50W/70W, Duplex	LB	17,675,719	$2.50	$44,189,297
	Prestressed Concrete Girder (77" - 130 ft)	EA	88	$50,000	$4,400,000
	Tooth Expansion Joint Device	LF	127	$2,200	$278,667
	Modular Expansion Joint Device	LF	127	$4,000	$506,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$71,157,586
				Cost/SQFT	$171
Substructure	66" Diameter Cylinder Piles (SS prestressed)	LF	9,900	$900	$8,910,000
	16" Diameter Cylinder Piles (SS prestressed)	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	22	$3,000	$66,000
	Concrete, 4000 psi (Cap), Class A4	CY	1,117	$1,200	$1,340,000
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	263,840	$2.10	$554,064
	Dowel Holes w/Anchor for Reinforcing Steel	LF	3,200	$100.00	$320,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	176	$1,000	$176,000
	HLMR Bearing with Lock-Up Device	EA	36	$75,000	$2,700,000
				Subtotal	$18,736,064
				Cost/SQFT	$45

Mobilization & Demolition	
Mobilization	$4,494,683
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$14,978,933
Rapid Replacement Temporary Works	
Subtotal	$147,961,122
Contingency	
20% Contingency" Subtotal	$50,566,741
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$7,771,966
Owner PM/CM (12%)	$18,652,719
Design Build Engineering (12%)	$18,652,719
Subtotal	$46,077,404

Total Project Cost	**$349,477,850**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Superstructure Replacment
Alternative F: 30-foot Width Approach & Channel Span Units - Tied-Arch

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	9,598	$700	$6,718,402
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,399,430	$3.50	$8,398,005
	Bridge Deck Grooving	SY	33,283	$7.00	$232,983
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,050	$225	$4,511,250
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	27,453,762	$2.50	$68,634,406
	Structural Steel Tied Arch, ASTM A709, 50W/70W, Duplex	LB	3,078,003	$3.20	$9,849,610
	Concrete, 10000 psi (Arch Tie)	CY	2,091	$1,000	$2,090,667
	Post-Tensioning - 0.6" ASTM A416, Grade 270 (Arch Tie)	LB	17,263	$6.00	$103,576
	Prestressed Concrete Girder (77" - 130 ft)	EA	72	$50,000	$3,600,000
	Tooth Expansion Joint Device	LF	285	$2,200	$627,001
	Modular Expansion Joint Device	LF	127	$4,000	$506,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$107,805,478
				Cost/SQFT	$259
Substructure	66" Diameter Cylinder Piles (SS prestressed)	LF	8,100	$900	$7,290,000
	16" Diameter Cylinder Piles (SS prestressed)	LF	4,000	$200	$800,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	18	$3,000	$54,000
	Concrete, 4000 psi (Cap), Class A4	CY	2,818	$1,500	$4,227,431
	Concrete, 4000 psi (Column), Class A4	CY	4,384	$1,500	$6,576,389
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	1,568,700	$2.10	$3,294,270
	Dowel Holes w/Anchor for Reinforcing Steel	LF	2,600	$100.00	$260,000
	Column Fiber Wrap	Column	32	$50,000.00	$1,600,000
	Elastomeric Bearing	EA	144	$1,000	$144,000
	HLMR Bearings	EA	75	$10,000	$750,000
				Subtotal	$27,266,089
				Cost/SQFT	$66

Mobilization & Demolition	
Mobilization	$6,753,578
Dismantle & Remove Existing Bridge	$10,484,250
Subtotal	$17,237,828
Rapid Replacement Temporary Works	
Subtotal	$0
Contingency	
20% Contingency' Subtotal	$30,461,879
Project Development and Administration	
ROW Estimate	$1,000,000
Owner PE (5%)	$9,138,564
Owner PM/CM (12%)	$21,932,553
Design Build Engineering (12%)	$21,932,553
Subtotal	$54,003,670

Total Project Cost	**$236,774,945**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 1: Prestressed Concrete Bulb T Girder Spans (140' & 185') & Steel Channel Spans

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	11,237	$700	$7,866,004
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,809,290	$3.50	$9,832,515
	Bridge Deck Grooving	SY	45,689	$7.00	$319,822
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,640	$225	$4,644,000
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	4,375,100	$2.35	$10,281,485
	Prestressed Concrete Girder (96")	EA	170	$95,000	$16,150,000
	Prestressed Concrete Girder (77")	EA	105	$50,000	$5,250,000
	Tooth Expansion Joint Device	LF	708	$2,200	$1,558,333
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$58,435,071
				Cost/SQFT	$136.41
Substructure	66" Diameter Cylinder Piles (SS prestressed)	LF	9,450	$900	$8,505,000
	72" Diameter Conc. Filled Steel Pipe Piles	LF	52,620	$1,100	$57,882,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	32	$3,000	$96,000
	Precast Concrete Shells (Footings)	EA	38	$80,000	$3,040,000
	Concrete, 4000 psi (Cap), Class A4	CY	5,665	$700	$3,965,729
	Concrete, 4000 psi (Column), Class A4	CY	7,871	$800	$6,296,414
	Concrete, 4000 psi (Footing), Class A4	CY	19,200	$700	$13,440,000
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	6,745,080	$2.10	$14,164,668
	Elastomeric Bearings	EA	550	$1,000	$550,000
	HLMR Bearings	EA	16	$10,000	$160,000
	Bridge Fender System	LS	1	$2,000,000	$2,000,000
				Subtotal	$112,369,811
				Cost/SQFT	$262.32

Mobilization & Demolition	
Mobilization	$8,540,244.11
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$21,645,557
Contingency	
15% Contingency' Subtotal	$28,867,566
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$11,065,900
Owner PM/CM (12%)	$26,558,161
Design Build Engineering (12%)	$26,558,161
Subtotal	$67,182,221

Total Project Cost $288,500,226

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 2: Steel Plate Girder Spans (250' typ & 400' channel span)

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	11,863	$700	$8,304,405
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,965,860	$3.50	$10,380,510
	Bridge Deck Grooving	SY	45,556	$7.00	$318,889
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,580	$225	$4,630,500
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	27,375,300	$2.35	$64,331,955
	Tooth Expansion Joint Device	LF	417	$2,200	$916,667
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$91,415,836
				Cost/SQFT	$214.03
Substructure	72" Diameter Conc. Filled Steel Pipe Piles	LF	54,120	$1,100	$59,532,000
	66" Diameter Conc. Filled Steel Pipe Piles	LF	660	$960	$633,600
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	32	$3,000	$96,000
	Precast Concrete Shells (Footings)	EA	42	$80,000	$3,360,000
	Concrete, 4000 psi (Cap), Class A4	CY	5,121	$700	$3,584,747
	Concrete, 4000 psi (Column), Class A4	CY	6,850	$800	$5,480,151
	Concrete, 4000 psi (Footing), Class A4	CY	20,907	$700	$14,634,667
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	6,753,090	$2.10	$14,181,489
	HLMR Bearings	EA	208	$10,000	$2,080,000
	Bridge Fender System	LS	1	$2,000,000	$2,000,000
				Subtotal	$107,852,654
				Cost/SQFT	$252.51

Mobilization & Demolition	
Mobilization	$9,963,425
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$23,068,737
Contingency	
15% Contingency' Subtotal	$33,350,584
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$12,784,391
Owner PM/CM (12%)	$30,682,537
Design Build Engineering (12%)	$30,682,537
Subtotal	$77,149,465

Total Project Cost $332,837,277

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 3: Precast Segmental Box Girder, Balanced Cantilever Spans (250' typ & 400' channel span)

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Superstructure Concrete, 6000 psi	CY	31,001	$1,100	$34,101,165
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Corrosion Resistant Reinforcing Steel, Class III	LB	5,622,285	$3.50	$19,677,999
	Longitudinal Post-Tensioning Strands (½" dia.)	LB	1,669,925	$4.00	$6,679,701
	Transverse Post-Tensioning Strands (0.6" dia.)	LB	363,050	$5.00	$1,815,249
	Post-Tensioning Bars (1-3/8" dia.)	LB	106,779	$8.00	$854,235
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,500	$225	$4,612,500
	Erection Equipment (overhead gantry & haulers)	LS	1	$5,000,000.00	$5,000,000
	Tooth Expansion Joint Device	LF	400	$2,200	$880,000
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$76,153,760
				Cost/SQFT	$178.30
Substructure	72" Diameter Conc. Filled Steel Pipe Piles	LF	58,920	$1,100	$64,812,000
	66" Diameter Conc. Filled Steel Pipe Piles	LF	900	$960	$864,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	32	$3,000	$96,000
	Precast Concrete Shells (Footings)	EA	42	$80,000	$3,360,000
	Concrete, 5500 psi (Cap), Class A4	CY	3,117	$700	$2,182,133
	Concrete, 5500 psi (Column), Class A4	CY	2,987	$800	$2,389,558
	Concrete, 5500 psi (Footing), Class A4	CY	20,907	$700	$14,634,667
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	5,221,928	$2.10	$10,966,049
	Disk Bearings	EA	104	$10,000	$1,040,000
	Bridge Fender System	LS	1	$2,000,000.00	$2,000,000
	Vertical Post-Tensioning Strands (½" dia.)	LB	96,061	$8.00	$768,488
	Post-Tensioning Bars (1-3/8" dia.)	LB	61,067	$12.00	$732,808
				Subtotal	$106,115,703
				Cost/SQFT	$248.45

Mobilization & Demolition	
Mobilization	$9,113,473
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$22,218,786
Contingency	
15% Contingency" Subtotal	$30,673,237
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$11,758,074
Owner PM/CM (12%)	$28,219,378
Design Build Engineering (12%)	$28,219,378
Subtotal	$71,196,831

Total Project Cost $306,358,318

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 7: Prestressed Concrete Bulb T Girder Spans (150' typ) & Steel Plate Girder Channel Spans

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	11,150	$700	$7,804,790
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,787,430	$3.50	$9,756,005
	Bridge Deck Grooving	SY	45,333	$7.00	$317,333
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,480	$225	$4,608,000
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	4,375,100	$2.35	$10,281,485
	Prestressed Concrete Girder (85")	EA	305	$75,000	$22,875,000
	Tooth Expansion Joint Device	LF	625	$2,200	$1,375,000
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$59,550,525
				Cost/SQFT	$140.11
Substructure	66" Diameter Cylinder Piles (SS prestressed)	LF	7,650	$900	$6,885,000
	72" Diameter Conc. Filled Steel Pipe Piles	LF	49,520	$1,100	$54,472,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	32	$3,000	$96,000
	Precast Concrete Shells (Footings)	EA	48	$80,000	$3,840,000
	Concrete, 4000 psi (Cap), Class A4	CY	6,654	$700	$4,657,963
	Concrete, 4000 psi (Column), Class A4	CY	9,946	$800	$7,956,676
	Concrete, 4000 psi (Footing), Class A4	CY	16,427	$700	$11,498,667
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	6,844,770	$2.10	$14,374,017
	Elastomeric Bearings	EA	600	$1,000	$600,000
	HLMR Bearings	EA	16	$10,000	$160,000
	Bridge Fender System	LS	1	$2,000,000	$2,000,000
				Subtotal	$108,810,323
				Cost/SQFT	$256.00

Mobilization & Demolition	
Mobilization	$8,418,042.37
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$21,523,355
Contingency	
15% Contingency" Subtotal	$28,482,630
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$10,918,342
Owner PM/CM (12%)	$26,204,020
Design Build Engineering (12%)	$26,204,020
Subtotal	$66,326,381

Total Project Cost $284,693,214

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 8: Precast Segmental Box Girder, Span-By-Span (150' typ & 400' channel span)

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Superstructure Concrete, 6000 psi	CY	24,489	$1,100	$26,937,761
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Corrosion Resistant Reinforcing Steel, Class III	LB	4,505,153	$3.50	$15,768,035
	Longitudinal Post-Tensioning Strands (½" dia.)	LB	1,567,313	$4.00	$6,269,252
	Transverse Post-Tensioning Strands (0.6" dia.)	LB	361,279	$5.00	$1,806,395
	Post-Tensioning Bars (1-3/8" dia.)	LB	106,259	$8.00	$850,068
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,400	$225	$4,590,000
	Erection Equipment (overhead gantry & haulers)	LS	1	$3,000,000.00	$3,000,000
	Tooth Expansion Joint Device	LF	600	$2,200	$1,320,000
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$63,074,422
				Cost/SQFT	$148.40
Substructure	72" Diameter Conc. Filled Steel Pipe Piles	LF	62,960	$1,100	$69,256,000
	66" Diameter Conc. Filled Steel Pipe Piles	LF	900	$960	$864,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	32	$3,000	$96,000
	Precast Concrete Shells (Footings)	EA	63	$80,000	$5,040,000
	Concrete, 5500 psi (Cap), Class A4	CY	4,676	$700	$3,273,200
	Concrete, 5500 psi (Column), Class A4	CY	4,387	$800	$3,509,866
	Concrete, 5500 psi (Footing), Class A4	CY	20,427	$700	$14,298,667
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	5,616,667	$2.10	$11,795,000
	Elastomeric Bearings	EA	128	$2,500	$320,000
	Bridge Fender System	LS	1	$2,000,000.00	$2,000,000
	Vertical Post-Tensioning Strands (½" dia.)	LB	141,281	$8.00	$1,130,248
	Post-Tensioning Bars (1-3/8" dia.)	LB	89,814	$12.00	$1,077,772
				Subtotal	$114,930,753
				Cost/SQFT	$270.40

Mobilization & Demolition	
Mobilization	$8,900,259
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$22,005,571
Contingency	
15% Contingency[2] Subtotal	$30,001,612
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$11,500,618
Owner PM/CM (12%)	$27,601,483
Design Build Engineering (12%)	$27,601,483
Subtotal	$69,703,584
Total Project Cost	**$299,715,942**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.

Replacement of Norris Bridge
Route 3 over Rappahannock River
Conceptual Engineer's Estimate

Conceptual Cost Estimate: Total Replacement
Option 7A: Prestressed Concrete Bulb T Girder Spans (150' typ) & Steel Plate Girder Channel Spans

	Description	Unit Measure	Quantity	Unit Cost	Total Cost
Superstructure	Concrete, Class A4, Bridge Approach Slab	CY	44	$500	$22,222
	Deck Concrete Low Shrinkage, Class A4 Modified	CY	10,314	$700	$7,219,754
	Approach Slab: Reinforcing Steel	LB	8,222	$1.30	$10,689
	Deck: Corrosion Resistant Reinforcing Steel, Class III	LB	2,578,490	$3.50	$9,024,715
	Bridge Deck Grooving	SY	36,267	$7.00	$253,867
	Concrete Barrier Railing (BR27C-12 Steel Railing)	LF	20,480	$225	$4,608,000
	Structural Steel Plate Girder, ASTM A709, 50W, Duplex	LB	4,242,100	$2.35	$9,968,935
	Prestressed Concrete Girder (85" - 150ft)	EA	244	$75,000	$18,300,000
	Tooth Expansion Joint Device	LF	505	$2,200	$1,111,001
	Bridge Conduit System	LS	1	$2,500,000	$2,500,000
				Subtotal	$53,019,183
				Cost/SQFT	$154
Substructure	66" Diameter Cylinder Piles (SS prestressed)	LF	8,100	$900	$7,290,000
	72" Diameter Conc. Filled Steel Pipe Piles	LF	46,880	$1,100	$51,568,000
	Static Load Test	EA	4	$500,000	$2,000,000
	Dynamic Pile Test (PDA and Monitoring)	EA	63	$3,000	$189,000
	Precast Concrete Shells (Footings)	EA	45	$80,000	$3,600,000
	Concrete, 4000 psi (Cap), Class A4	CY	4,205	$700	$2,943,447
	Concrete, 4000 psi (Column), Class A4	CY	6,412	$800	$5,129,818
	Concrete, 4000 psi (Footing), Class A4	CY	15,627	$700	$10,938,667
	Concrete, 3000 psi (Abutments), Class A3	CY	450	$600	$270,000
	Corrosion Resistant Reinforcing Steel, Class I	LB	5,417,520	$2.10	$11,376,792
	Elastomeric Bearings	EA	488	$1,000	$488,000
	HLMR Bearings	EA	16	$10,000	$160,000
	Bridge Fender System	LS	1	$2,000,000	$2,000,000
				Subtotal	$97,953,724
				Cost/SQFT	$285

Mobilization & Demolition	
Mobilization	$7,548,645
Dismantle & Remove Existing Bridge	$13,105,313
Subtotal	$20,653,958
Contingency	
15% Contingency[2] Subtotal	$25,744,030
Project Development and Administration	
ROW Estimate	$3,000,000
Owner PE (5%)	$9,868,545
Owner PM/CM (12%)	$23,684,507
Design Build Engineering (12%)	$23,684,507
Subtotal	$60,237,559
Total Project Cost	**$257,608,453**

Notes:
1) The cost for Mobilization is calculated as approximately 5% of the total conceptual cost for the alternate.
2) Quantities above do not include Roadway Approachs, Construction Survey, Deck Drainage System, Lighting, Navigation Lighting, Utility Relocation, etc.
3) Superstructure Corrosion Resistant Reinforcing Steel, Class III: deck, railings, diaphragms.
4) Substructure Corrosion Resistant Reinforcing Steel, Class I: all substructure; prestressed girders reinforcement extending into concrete deck slab.