Structural Study of Existing Lafayette Bridge No. 9800

(TH 52 over the Mississippi River in Saint Paul, Minnesota)

Minnesota Department of Transportation

S.P. 6244-9800 (Study)
Mn/DOT Agreement No. 86425

March 1, 2007
Project No. 13559.000

TKDA
ENGINEERS • ARCHITECTS • PLANNERS

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1. EXECUTIVE SUMMARY

The scope of work for the Lafayette Bridge Study requires the investigation of four bridge widening options. Options are defined in terms of three criteria: horizontal alignment, deck geometry, and whether or not river construction operations (piles, footings, pier shafts, cofferdams, etc.) are allowed. The options are named Option 1a, 1b, 2, and 3. The distinguishing characteristics of each option are as follows:

- **Option 1a:** Symmetrical widening about the existing centerline, bridge cross sections meet full geometric standards, foundation work in the river is not allowed.
- **Option 1b:** Symmetrical widening about the existing centerline, bridge cross sections meet full geometric standards, foundation work in the river is allowed.
- **Option 2:** Symmetrical widening about the existing centerline, bridge cross sections do not meet full geometric standards, foundation work in the river is not allowed.
- **Option 3:** Northbound bridge on a new alignment east of the existing bridge, southbound bridge centered on the existing centerline, bridge cross sections meet full geometric standards, foundation work in the river is required.

All the options require the removal of the existing steel superstructure over the river spans. The decision to remove the existing steel superstructure of the river spans was made by the Mn/DOT Bridge Office on the basis of fatigue cracking problems associated with these spans, along with the difficulty in widening the current configuration. Three new superstructure alternatives were investigated for the river spans: multiple steel plate girders, steel box girders, and post-tensioned concrete box girders. The steel superstructure of the existing approach spans had to be evaluated for its suitability for inclusion in the reconstructed bridge deck.

Over the course of the bridge study, four important controlling and interdependent factors emerged:

- Geometric constraints due to clearance requirements for the existing navigation channel below the bridge, for the flight corridor for runway 14-32 of the Saint Paul Downtown Airport (Holman Field) above the bridge, and the existing power line crossing above the bridge deck at Pier 11. These clearance requirements impact the available superstructure depth.

- Driving new piles for river pier widening schemes has serious interference issues in the vicinity of the existing river piers. These interference problems often preclude the installation of new piles in the location where they are needed.

- The limitations imposed by the need to maintain traffic during construction.

- The factor of safety implied by the old allowable stress design method (ASD) is unreliable, because it is only applied to a design stress. It does not address the variability of the loads and of the construction materials. For a number of load combinations, the ASD code permits stresses in excess of the allowable basic unit stress. This practice tends to obscure the real factor of safety even further. On the other hand, results obtained on the basis of the load and resistance factors of the LRFD code, which is based on the theory of probability, provide a much more reliable measure of the actual factor of safety. In addition, Grade 40 reinforcement has a smaller ASD factor of safety than Grade 60 reinforcement (2.0 vs. 2.5). The original bridge is reinforced with Grade 40 reinforcement and therefore is affected by the smaller 2.0 factor.

From the investigation, the following results emerged:

Options 1a and 2 are not feasible since the existing foundations are inadequate to safely carry the new design loads associated with the respective deck geometry.

Incorporation of the existing steel girders of the approach spans is feasible, but would require accepting the following consequences: remedial procedures for fatigue-prone details, adjustments to the girder locations and modifications of the diaphragms, interference problems for pier widening in some locations, maintenance on eleven expansion devices, and limited structural capacity due to corrosion in the top flanges of the girders.

The negative impacts of traffic staging on motorists would be much smaller for Option 3 than for Option 1b. The latter option would significantly reduce the number of lanes available during construction and contribute to traffic jams.

Providing the piers in the river with the capacity to withstand barge collision forces would require creating a single foundation unit. Incorporating the existing piles would require the removal of existing dead loads from these piles, followed by the addition of the new dead loads in the course of the construction sequence. Both requirements can be met for a multiple steel girder alternative under Option 1b. For Option 3, it would be necessary to remove the existing piers, including their footings, in order to be able to add piles and to create a single foundation unit.

For Option 1b, only a multiple-steel-girder alternative for the river spans emerged as a workable solution. The river pier construction would require a cofferdam as described in the preceding paragraph. The construction of the northbound bridge would have to be accomplished in stages. It would provide the minimum number of traffic lanes. The multiple steel girders alternative is the only one of three alternatives that would avoid interference issues at the footing level, provide a stable superstructure at all stages, and would not compromise the stability of the remaining existing bridge.

For Option 3, all three superstructure alternatives are viable. The river pier construction would also require a cofferdam. New piles would have to be added to the existing footings. The existing river piers would be removed down to the cofferdam seal to allow the construction of a footing that would incorporate both existing and new piles.

For the approach spans, both new superstructure alternatives (new steel girders or prestressed concrete beams) can be used. This outcome is dependent on three decisions: to construct new footings and pier columns wherever necessary, to eliminate some piers while relocating others to more suitable locations, and to locate the river span transition at Piers 8 and 11.

The Lafayette Bridge site is unique with regard to the impacts on construction scheduling. Crane operations will have to be coordinated with the air traffic controllers at Holman Field. Work in the Mississippi River will have to be coordinated with river navigation and work over the railroad tracks will require flagging services and be limited to established work windows. Work in the vicinity of the high voltage power lines can be expected to require power outages, which often can be scheduled only at certain times of the year. Traffic on Kellogg Boulevard and Warner Road, if maintained during construction, would have less impact on the construction schedule, but would still require special treatment. All of these conditions will extend the time and increase the cost of construction. More tedious construction solutions, such as reusing the existing steel, would be even more time consuming and costly.

The findings of this study point in the direction of extensive new construction, starting with new footings. The extent of new construction would afford the opportunity to consider a comprehensive aesthetic concept for the new Lafayette Bridge.

Two decision matrices can be found at the end of this executive summary. They show pertinent issues for the river spans and the approach spans, including estimated construction costs. These costs only include the construction of the bridge structure and do not include the costs associated with additional required construction beyond the bridge for maintaining traffic during staged construction phases, or the cost of permanent connections from the bridge to the existing transportation system. Likewise, possible resolutions of the clearance and interference issues with the 115 kV power line and the associated construction costs are not addressed in the report. Also, the cost of utility relocations, including the cost of relocating or temporarily supporting the 24-inch-diameter water main hung from the superstructure for the river spans is not included. The cost of additional right-of-way, temporary or permanent easements, railroad force account work, engineering, and administrative costs are also not included.

The estimates of probable construction costs included in the decision matrices have been adjusted for additional costs associated with staged construction methods. They include contingency allowances for differences between preliminary and final quantities and unit prices, aesthetic enhancements, and additional minor pay items that can be anticipated to be required for the final bidding documents. The estimated construction costs are also based on current unit prices and do not consider inflation, since it is quite possible that the construction year could change from its current schedule. Therefore, inflation allowances should be added based on the most recent planned construction date. A more detailed discussion of the methodology and computation of the estimates of probable constructions costs is included in a separate document titled "Construction Cost Study Technical Report, Supplement for the Structural Study of the Existing Lafayette Bridge No. 9800," dated March 1, 2007.

Inherent in every proposed project are certain risk factors. Examples of these are labor strikes, river flooding, material shortages, unknown subsurface conditions, and so forth. No attempt has been made to assign risk factors or potential costs associated with them in this report. That effort is typically part of Mn/DOT's total project planning process. These risk factors would be expected to impact all alternatives equally and would not affect the choice of alternatives.

The decision matrices evaluate the river spans and approach spans separately. For a particular option, each approach alternative could be combined with each surviving river span alternative. The selected approach span cost must be combined with the selected river span cost in order to obtain the total bridge construction cost. Table 1 is a matrix that shows the combined estimated bridge construction costs for the various combinations of approach spans and river spans.

The preferred alternative for replacing the existing Lafayette Bridge is based on the alignment of Option 3 and either new steel girders for the approach spans or prestressed concrete beams, but not reuse of existing steel.

Table 1. Combined Alternatives Cost Summary Matrix

ALIGNMENT OPTION 1b				
Approach Span Alternatives	**River Span Alternatives**			
	Multiple Steel Girders		**Variable Depth Steel Box Girder**	**Post-Tensioned Segmental Conc. Box Girder**
	Constant Depth	**Variable Depth**		
Reuse Existing Steel Girders	$85 M	$86 M	NA	NA
New Steel Girders	$78 M	$79 M	NA	NA
New Prestressed Concrete Beams	$72 M	$73 M	NA	NA
ALIGNMENT OPTION 3				
Approach Span Alternatives	**River Span Alternatives**			
	Multiple Steel Girders		**Variable Depth Steel Box Girder**	**Post-Tensioned Segmental Conc. Box Girder**
	Constant Depth	**Variable Depth**		
Reuse Existing Steel Girders	$79 M	$80 M	$83 M	$79 M
New Steel Girders	$75 M	$76 M	$79 M	$75 M
New Prestressed Concrete Beams	$68 M	$69 M	$72 M	$68 M

Table 2. Decision Matrix: River Spans

OPTIONS AND ALTERNATIVES	FEASIBILITY AND COST: Feasible?	FEASIBILITY AND COST: 2007 Estimated Construction Cost	CONSTRUCTION ISSUES: Number of Construction Seasons	CONSTRUCTION ISSUES: Number of Traffic Lanes During Construction	CONSTRUCTION ISSUES: Foundation Work in Shipping Channel?	CONSTRUCTION ISSUES: Interference with Tower of 115 kV Power Line?	CLEARANCE REQUIREMENTS: Meets Vertical Clearance Requirement for 115 kV Power Line?	CLEARANCE REQUIREMENTS: Provides Required Clearances for Navigation Channel?	CLEARANCE REQUIREMENTS: Meets Required Clearances for Holman Field?	MAINTENANCE ISSUES: Anticipated Future Maintenance Costs	MAINTENANCE ISSUES: Degree of Difficulty to Replace Deck	MAINTENANCE ISSUES: 75-Year Structural Life?
Option 1a - River Spans												
Constant-Depth Steel Girder	No	NA			(No)							
Variable-Depth Steel Girder	No	NA			(No)							
Steel Box Girder	No	NA			(No)							
PT Concrete Box Girder	No	NA			(No)							
Option 1b - River Spans												
Constant-Depth Steel Girder	Yes	\$36 M	4	2	Yes	No	No	Yes	Yes	Medium	Medium	Yes
Variable-Depth Steel Girder	Yes	\$37 M	4	2	Yes	No	No	Yes	Yes	Medium	Medium	Yes
Steel Box Girder	No	NA										
PT Concrete Box Girder	No	NA										
Option 2 - River Spans												
Constant-Depth Steel Girder	No	NA			(No)							
Variable-Depth Steel Girder	No	NA			(No)							
Steel Box Girder	No	NA			(No)							
PT Concrete Box Girder	No	NA			(No)							
Option 3 - River Spans												
Constant-Depth Steel Girder	Yes	\$32 M	3	4	Yes	Yes	No	Yes	Yes	Medium	Medium	Yes
Variable-Depth Steel Girder	Yes	\$33 M	3	4	Yes	Yes	No	Yes	Yes	Medium	Medium	Yes
Steel Box Girder	Yes	\$36 M	3	4	Yes	Yes	No	Yes	Yes	Medium	High	Yes
PT Concrete Box Girder	Yes	\$32 M	3	4	Yes	Yes	No	Yes	Yes	Low	Complex	Yes

Table 3. Decision Matrix: Approach Spans

OPTIONS AND ALTERNATES	FEASIBILITY AND COST: Feasible?	FEASIBILITY AND COST: 2007 Estimated Construction Cost	CONSTRUCTION ISSUES: Number of Construction Seasons	CONSTRUCTION ISSUES: Number of Traffic Lanes During Construction	CONSTRUCTION ISSUES: Work Over Railroad Tracks?	CONSTRUCTION ISSUES: Requires New Pier Foundations?	CLEARANCE REQUIREMENTS: Meets Railroad Clearance Requirements?	CLEARANCE REQUIREMENTS: Meets Required Clearances for Holman Field?	MAINTENANCE ISSUES: Number of Expansion Joints	MAINTENANCE ISSUES: Anticipated Future Maintenance Costs	MAINTENANCE ISSUES: Degree of Difficulty to Replace Deck	MAINTENANCE ISSUES: 75-Year Structural Life?
Option 1a - Approach Spans												
Reuse Existing Steel Girders	(1)	NA										
New Steel Girders	(1)	NA										
New PCBs	(1)	NA										
Option 1b - Approach Spans												
Reuse Existing Steel Girders	Yes	$49 M	4	2	Yes	Yes	Yes	Yes	11	High	Typical	Yes
New Steel Girders	Yes	$42 M	3 to 4	2	Yes	Yes	Yes	Yes	8	Medium	Typical	Yes
New PCBs	Yes	$36 M	3 to 4	2	Yes	Yes	Yes	Yes	8	Low	Typical	Yes
Option 2 - Approach Spans												
Reuse Existing Steel Girders	(2)	NA										
New Steel Girders	(2)	NA										
New PCBs	(2)	NA										
Option 3 - Approach Spans												
Reuse Existing Steel Girders	Yes	$47 M	4	4	Yes	Yes	Yes	Yes	11	High	Typical	Yes
New Steel Girders	Yes	$43 M	3	4	Yes	Yes	Yes	Yes	8	Medium	Typical	Yes
New PCBs	Yes	$36 M	3	4	Yes	Yes	Yes	Yes	8	Low	Typical	Yes

(1) For the approach spans, Options1a and 1b are identical.
(2) Option 2 would be feasible for the approach spans, but does not work for the river spans.

2. INTRODUCTION

2.1. Organization of the Report

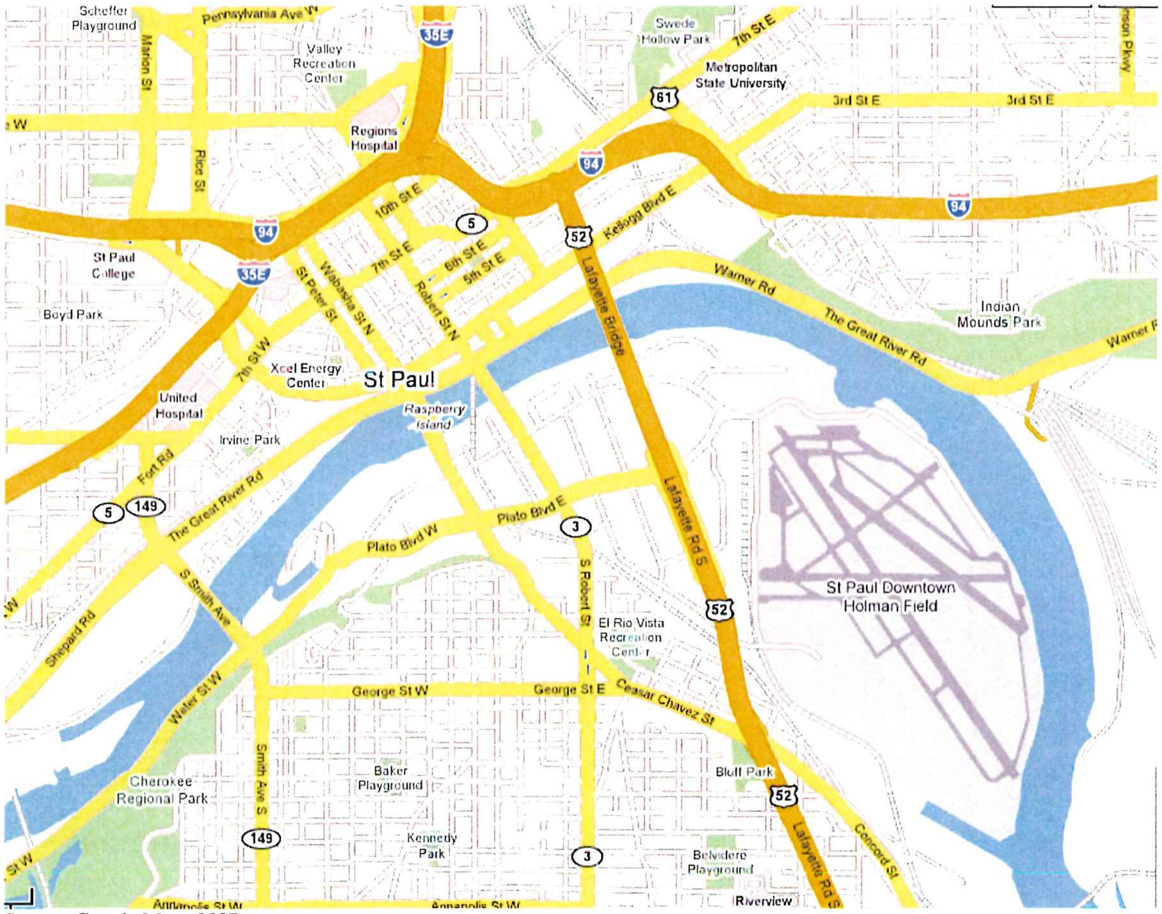
This report has six distinct parts:

- The executive summary (Section 1).
- The introduction of the existing bridge and of the scope and parameters of the structural study (Section 2).
- The discussion of aesthetic opportunities, design criteria, construction, and maintenance issues (Sections 3 and 4). The content of these sections is general in nature and applies to all options. For these reasons, it was kept in the main body of the report and was not relegated to an appendix. However, the reader may wish to skip Sections 2 and 3 (as well as Sections 5 and 6) and proceed immediately to the discussion of Options 1a, 1b, 2, and 3 (Sections 7 through 10).
- The evaluation of the structural components of the existing piers and steel approach spans (Sections 5 and 6). These two sections contain the most technical information in the report. They provide the basis for the evaluation of the options in the following sections.
- The discussion of the options and their viability (Sections 7 through 10). This part of the report presents results and findings. All other parts of the report provide supporting information to that end. As stated above, this portion of the report can be read gainfully without prior reading of the detailed technical discussion of the previous sections.
- The appendices contain CAD drawings and other supporting information.

2.2. Background

The Minnesota Department of Transportation (Mn/DOT) Bridge Office retained TKDA to evaluate a range of bridge widening options for Bridge No. 9800, also known as the Lafayette Bridge, in Saint Paul, Minnesota. Figure 1 shows the location of the Lafayette Bridge with respect to other features in adjacent areas. Prior to the start of this study, Mn/DOT had identified four bridge deck widening options as the best candidates for further evaluation, and it was those four options which were the basis of this study. Options 1a, 1b, and 2 have a common alignment, while Option 3 has a different alignment (see Figures 1A and 2A in Appendix A). The river spans of the existing bridge, with their twin two-girder and floor beam steel superstructure, have had a history of fatigue problems, and Mn/DOT has determined that these spans should be removed and replaced.

Upon completion of this study, Mn/DOT will undertake the environmental process, from which the preferred bridge widening option will be selected.



Source: Google Maps 2007

Figure 1. Lafayette Bridge Location

2.3. Bridge Widening Options Evaluated

The bridge deck widening options which provided the overall general framework for this study can be defined in terms of guidelines for the reconstruction of the river spans. In essence, each option links the desired future deck geometry to permissible river pier foundation operations. Thus, the key characteristics for each option are:

- The desired geometric standards of the bridge deck.
- The permissibility of river pier construction (allowed or not allowed).
- The choice of horizontal alignment (existing or new).

The following paragraphs introduce the options in greater detail:

Option 1a would require full widening to current geometric standards (two 12-foot traffic lanes, one 12-foot auxiliary lane, and two 12-foot shoulders in each direction) along the existing alignment. This option would not allow foundation work (cofferdams, piles, footings, or pier shafts) in the river.

Option 1b would require full widening to current geometric standards (two 12-foot traffic lanes, one 12-foot auxiliary lane, and two 12-foot shoulders in each direction) along the existing alignment. This option would allow foundation work (cofferdams, piles, footings, or pier shafts) in the river. Existing river piers would be widened with additional foundations, pier shafts, and cap extensions. Existing approach span piers would either be widened or replaced with new piers.

Option 2 would involve partial widening to substandard geometry along the existing alignment. This option's superstructure width is limited to the maximum width the existing river pier foundations can support. Consequently, this option would not allow foundation work (cofferdams, piles, footings, or pier shafts) in the river.

Option 3 would require full widening to current geometric standards (two 12-foot traffic lanes, one 12-foot auxiliary lane, and two 12-foot shoulders in each direction) along a new alignment. The existing Lafayette Bridge would be converted to a one-way structure for southbound traffic. For northbound traffic, a structure similar to the southbound structure would be constructed. This option would require new river piers and approach span piers for the northbound structure.

2.4. Superstructure Alternatives Evaluated

For all the options under consideration, the existing superstructure in the river spans would need to be completely removed and replaced. For the river spans, the following three replacement superstructure alternatives were evaluated:

- Welded plate steel girder spans (multiple girders) (see Figure 2).
- Post-tensioned concrete box girder spans (see Figure 3).
- Steel box girder spans (see Figure 4).

For the approach spans, the following superstructure alternatives were evaluated:

- Salvaging the existing steel beam spans.
- New prestressed concrete beam spans (see Figure 5).
- New steel beam spans (see Figure 6).

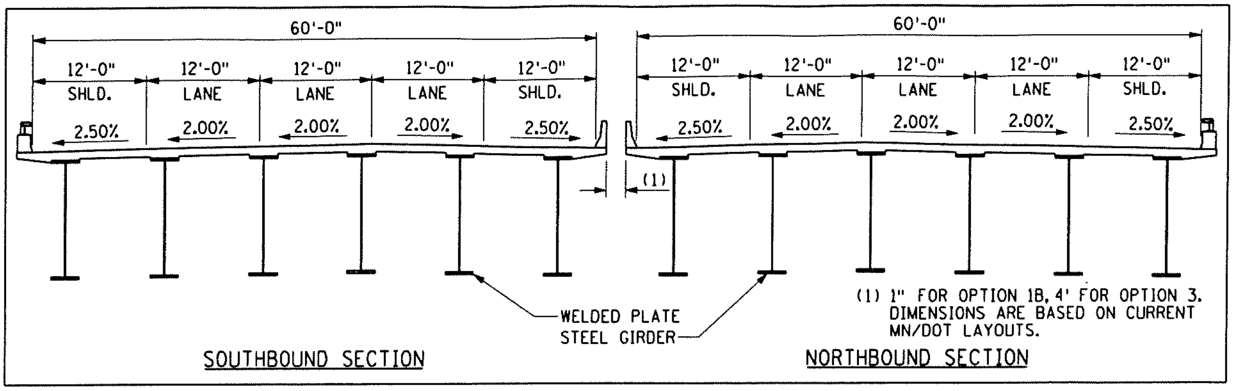


Figure 2. River Span, Welded Plate Steel Girder Spans

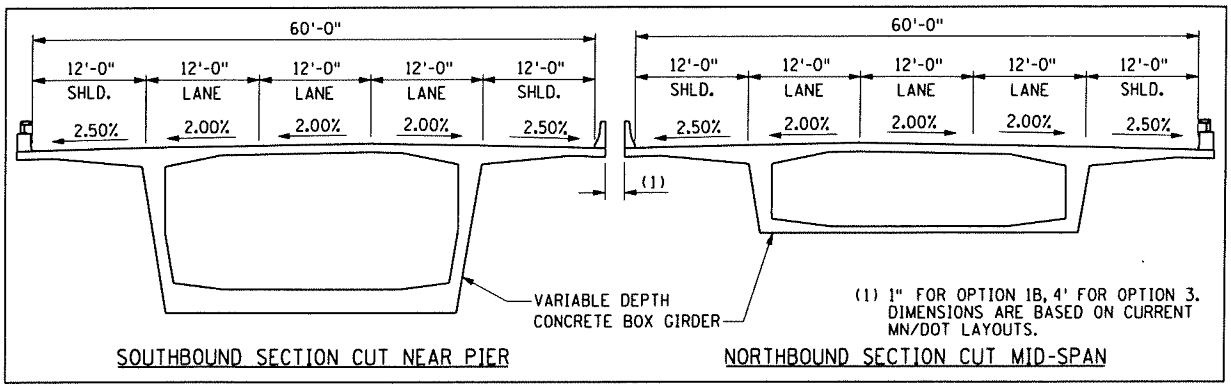


Figure 3. River Span, Post-Tensioned Concrete Box Girder Spans

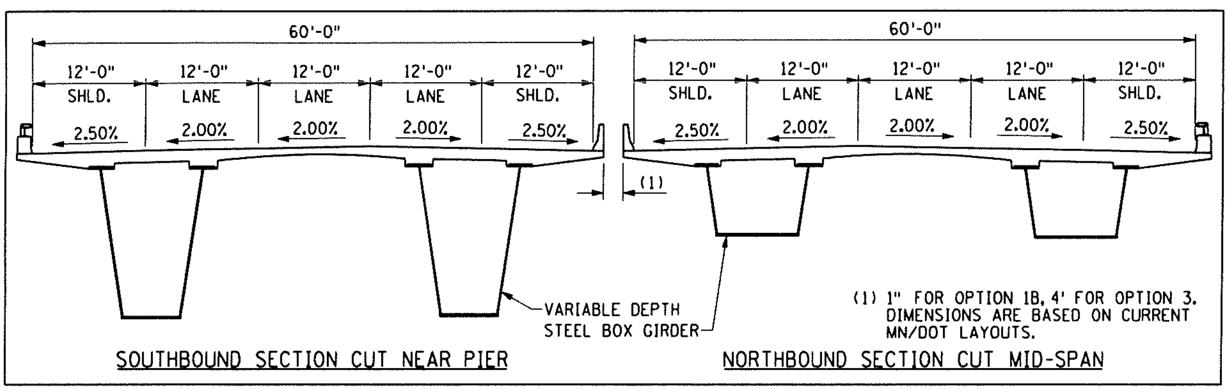


Figure 4. River Span, Steel Box Girder Spans

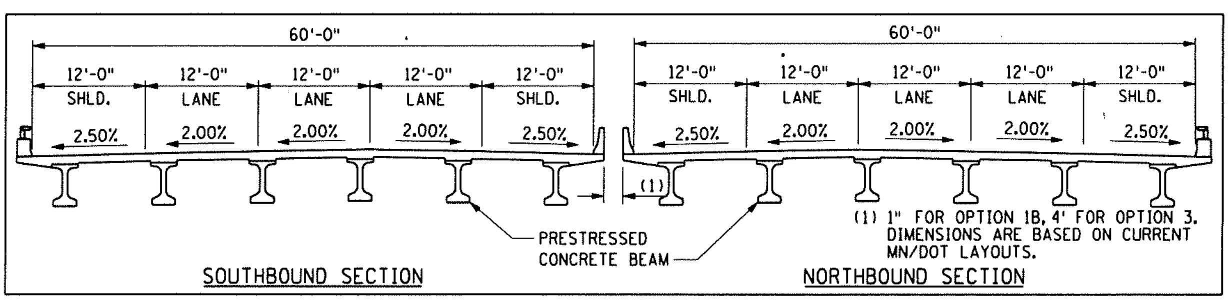


Figure 5. Approach Span, Prestressed Concrete Beam Spans

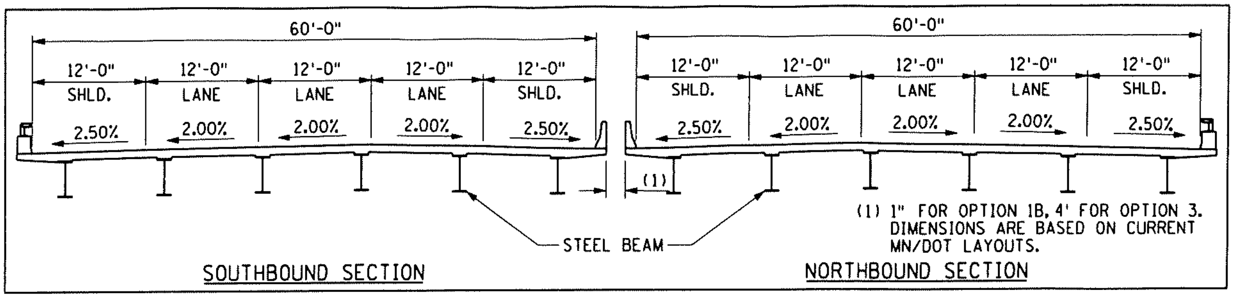


Figure 6. Approach Span, Steel Beam Spans

2.5. Design Codes and Specifications

The design specifications that are applicable to this study are:

- *AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications.* The live load design vehicle is the HL-93 truck.
- *AASHTO Manual for Condition Evaluation and Load Resistance Factor Rating (LRFR) of Highway Bridges.*
- Mn/DOT *LRFD Bridge Design Manual.*

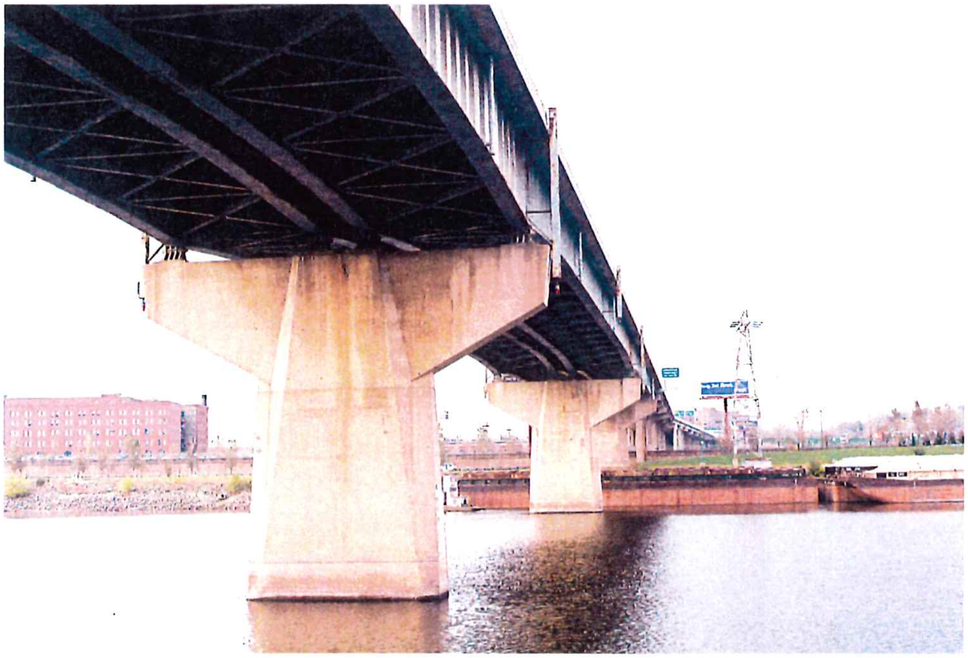
The pertinent horizontal alignments for the bridge options were provided by Mn/DOT.

The existing Lafayette Bridge was designed in accordance with the 1961 *AASHTO Design Specifications for Highway Bridges*, 8th Edition. The design live load was HS20-44. The design method used was the Allowable Stress Design (ASD) method. Current Mn/DOT standards call for the Load and Resistance Factor Design (LRFD). The evaluation of the existing bridge done for this study yielded a number of cases where the ASD approach predicted a "safe" structural component, while the LRFD approach showed a code violation. The LRFD approach is based on a probabilistic model in which loads are typically increased by applying a factor larger than 1, while resistances are typically decreased by applying a factor smaller than 1. The resulting factor of safety of an LRFD design has a higher degree of reliability than that of an ASD design. The factor of safety for the latter is elusive. For a comparison of the ASD and LRFD codes, see Appendix B.

2.6. Existing Bridge

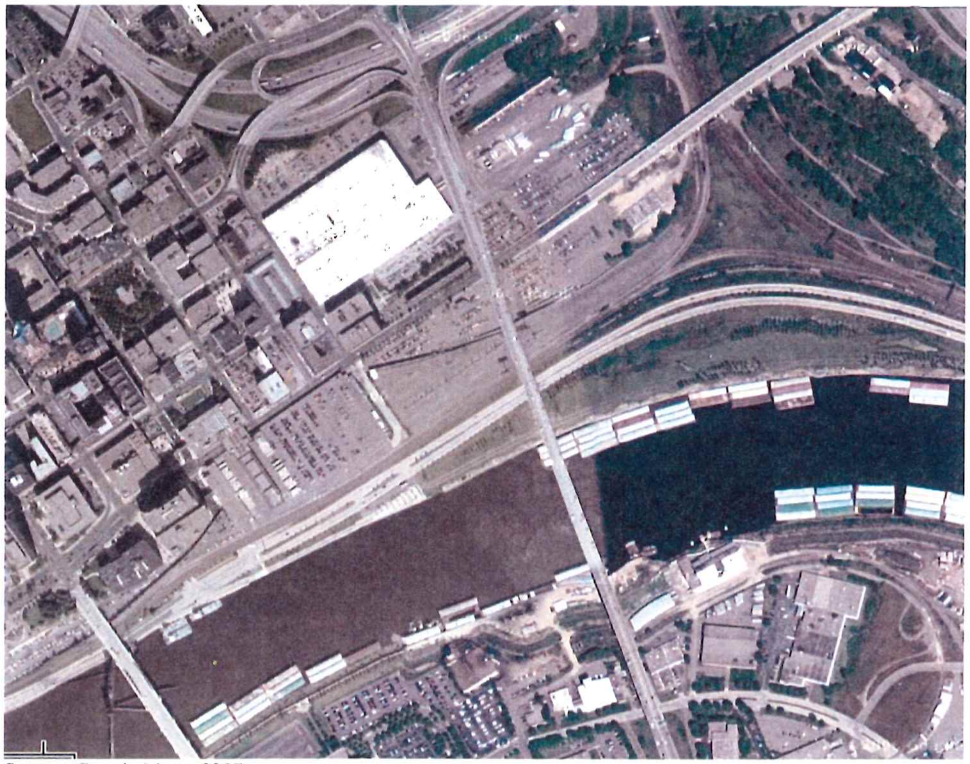
Originally opened in 1968, the Lafayette Bridge is located on the east end of downtown Saint Paul, Minnesota. The bridge carries four lanes of TH 52 over the Mississippi River, several city streets, a barge terminal, Canadian Pacific Railroad tracks, and several contract parking lots (see Figure 7). The bridge structure consists of two structurally independent superstructures separated by a split median barrier. The combined typical out-to-out width of the bridge decks is

67 feet-4 inches. The twin superstructures are supported by either a single pier or a common abutment. The river piers are single-shaft hammerhead piers, as can be seen in Photograph 1; the piers for the approach spans are two-legged pier frames with large cantilevering pier caps, as shown in Photograph 2. (See the existing bridge plans in Appendix C.)



Photograph 1. River Span Piers and Superstructure Looking North

Photograph 2. Approach Span Pier



Source: Google Maps, 2007

Figure 7. Lafayette Bridge Aerial Photograph

The bridge has twenty-nine steel beam spans, with a total bridge length of 3,366 feet. The substructures consist of twenty-eight piers and two abutments. The piers are numbered from 1 to 28, with the numbering starting at the south end. A thorough review of the geometry of the existing bridge (vertical alignment, pier location and orientation, span lengths, structure depths, and expansion joint locations) revealed important horizontal clearance and interference-constraints. Some of these constraints are still applicable, while others have been removed during the life span of the bridge or are no longer a source of concern. (See Figures 6A and 13A in Appendix A.)

The following material properties were used in the original design:

Structural Concrete:

Concrete Strength: $f_C' = 4{,}000$ psi

Allowable Concrete Stress: $f_C = 1{,}600$ psi

Reinforcement Bars, Grade 40 (Intermediate Grade):

Allowable Stress in Reinforcement: $f_S = 20{,}000$ psi

Structural Steel:
("MHD plus specification number" refers to the Minnesota Highway Department specification in force at the time of the original design.)

MHD 3306 (A36)	F_Y = 36,000 psi f_S = 20,000 psi
MHD 3309 (A242)	F_Y = 50,000 psi f_S = 27,000 psi
MHD 3310 (A441)	F_Y = 50,000 psi f_S = 27,000 psi (for 0.75 inch and under)
MHD 3310 (A441)	F_Y = 46,000 psi f_S = 25,000 psi (over 0.75 inch to 1.50 inches inclusive)
MHD 3310 (A441)	F_Y = 42,000 psi f_S = 23,000 psi (over 1.50 inch to 4.00 inches inclusive)

Steel Grade A441 was used for the superstructure over the river spans. The 46-inch welded steel girders in the approach spans are typically Steel Grade A36, except for some negative moment regions, which are Steel Grade A242. The cover plated rolled beams at the north end are typically Steel Grade A36.

All structural steel was originally painted with a shop coat of red lead per MHD 3506, followed by two field coats of aluminum per MHD 3527 and 3528. In 1987, the superstructure steel was repainted with an organic zinc-rich primer and a vinyl top coat. The entire steel framing system was sandblasted in accordance with SSPC No. 10, "Near White Blast Cleaning."

2.6.1. Vertical Profile

The existing bridge profile consists of a +3.50% back gradient (starting at the South Abutment), a 930-foot-long symmetrical parabolic crest curve, and a -0.31% ahead gradient. The high point of the vertical curve is located at Station 214+29.33, which is very close to Pier 10. At nearly the same location, the old plans show the centerline flight line of the NW-SE (or 13-31) runway approach zone to Saint Paul Downtown Airport (Holman Field). The centerline of the flight line crosses the centerline of TH 52 at a 31° 29' 15" angle. The South Extremity Flight Line is located at Station 204+90.00; the North Extremity Flight Line is located at Station 227+35.00. The end of the approach runway at Holman Field is located roughly 3,600 feet from the bridge. The approach clear zone is defined by a plane with a 40:1 slope starting at the end of the runway. By selecting the vertical profile described above, the original bridge designers accomplished the following goals:

- The south approach grade of +3.50% provides a minimum vertical clearance of 51.3 feet for the shipping channel, while staying outside of the clear zone of the runway approach.

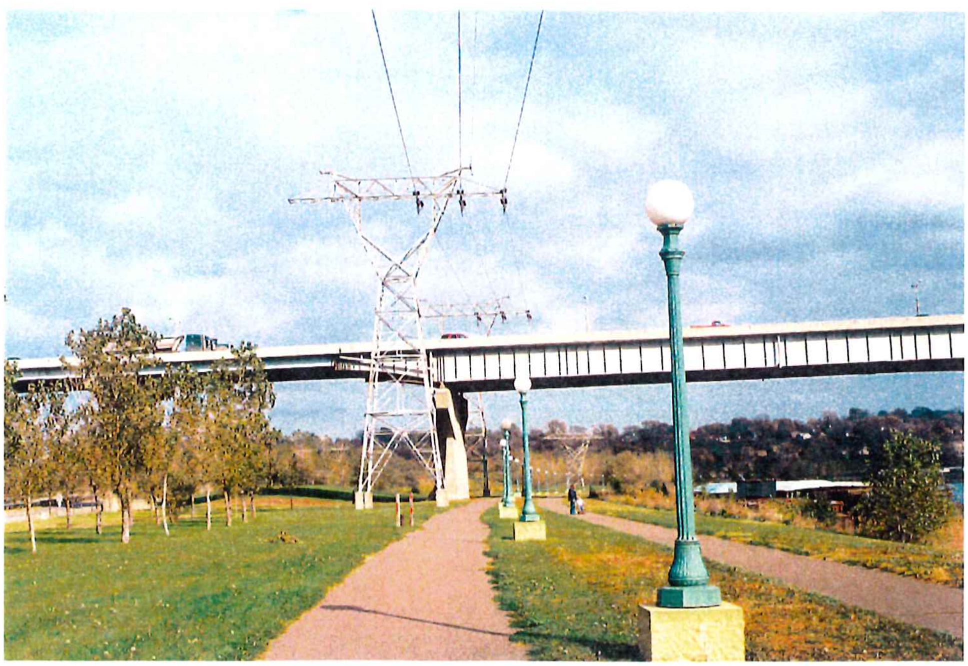
- The south approach grade, in conjunction with the pier location in the river, provides a 350-foot-wide (and 51.3-foot-high) shipping channel south of Pier 10.

The somewhat unfavorable north approach grade of -0.31% was most likely controlled by the tie-in elevation at the North Abutment.

Today, Runway 13-31 is no longer utilized for approaches from the northwest or for departures in that direction.

2.6.2. Pier Locations and Orientation

Wherever possible, the existing piers were placed at right angles to the roadway, except for the constrained area on the north end of the bridge. In this area, most of the piers have a regular skew angle that matches the street layout. However, some skew angles vary due to the railroad and street alignments underneath the bridge. Most of the original railroad tracks have since been eliminated. The only railroad tracks remaining are located in Span 14 (north of the newly aligned Warner Road). Warner Road was originally located south of Pier 11. Today, this corridor is occupied by a recreational trail and Warner Road has been relocated in a corridor between Piers 12 and 13. See Photos 3 and 4.



Photograph 3. Recreational Trail Looking East

Photograph 4. Warner Road Looking East

2.6.3. Span Lengths

The span lengths of the approach spans south of the Mississippi River vary from 39 feet-0 inches to 107 feet-0 inches. The river spans have lengths of 270 feet-0 inches, 362 feet-0 inches, and 250 feet-6 inches.

North of the river, in the region between Piers 11 and 16, the piers have a variety of skew angles and the corresponding span lengths are irregular. The respective span lengths vary from 87 feet-0 inches to 142 feet-6 inches. The irregular pier layout in this region was mainly controlled by railroad clearance requirements and the location of Warner Road. Since the construction of the bridge in 1965, most of the railroad tracks have been removed and Warner Road has been relocated.

The remaining approach spans to the north have lengths varying from 63 feet-0 inches to 113 feet-4 inches, but the span lengths of the structural units north of Pier 16 are generally well balanced.

2.6.4. Structure Depths

The existing bridge has three structure depths. The river spans have parabolic haunches. The symmetrical 70-foot-long parabolic haunches are located at Piers 9 and 10. The structure depth at these two piers is 15 feet-8 inches. The constant depth portion of the river spans is 12 feet-10 inches. At Piers 8 and 11, the 40-foot cantilever has a parabolic transition to bring the structure depth from 12 feet-10 inches to 4 feet-10 inches (46-inch girder webs) (see Photograph 5). The approach spans, with the exception of Spans 24 through 29, have a structure depth of 4 feet-10 inches. The remaining spans at the north end have a structure depth of 3 feet-9 inches (36-inch rolled beams with cover plates).

Photograph 5. Steel Beam Superstructure Depth Transition

2.6.5. Number of Expansion Joints

The existing bridge has eleven expansion joints. They are located at: the South Abutment; the hinges south of Pier 1, Pier 5, and Pier 8; the hinge north of Pier 11; the hinge south of Pier 13; the hinge north of Pier 16; the hinge south of Pier 20; the hinge north of Pier 23; the hinge south of Pier 28; and the North Abutment. The expansion joints near Piers 5, 8, 11, 13, 16, 20, 23,and 28 have finger joints. In relationship to the bridge's length (3,366 feet), the number of expansion joints is high. Each expansion joint requires continuous maintenance.

2.6.6. Bridge Configuration and Subsequent Modifications

There are eight approach spans south of the river spans and eighteen approach spans to the north. The south approach spans and twelve of the eighteen north approach spans are supported by 46-inch-deep welded plate girders. The remaining north approach spans have either 36-inch-deep rolled beams with welded cover plates (used in the original construction) or 36-inch-deep welded plate girders (used for subsequent bridge deck widening construction).

The river spans of each of the twin bridge structures consist of a non-redundant two-girder system. The girders are variable-depth fracture critical welded steel plate girders, bridging three spans and a 40-foot cantilever section at both ends. The center river span is 362 feet, and the two end spans are 270 feet and 252 feet. The entire twenty-nine spans were designed as a continuous structure, with nine hinges located at various points along the structure. Finger-type expansion joints at all hinges and expansion devices at the abutments accommodate the necessary temperature movements of the bridge structure.

The bridge has undergone several modifications throughout its lifetime. The barriers were replaced and an overlay was added in 1980. In 1981, the twelve northernmost approach spans were widened to the west to accommodate a new on-ramp, at which time Piers 18 through 28 were extended to provide the support for additional lines of girders. The piers were extended using a variety of modifications ranging from cantilevering pier cap extensions to constructing additional pier columns to support greater pier cap extensions. Similarly, in 1991, the north approach spans were widened to the east to accommodate a new off-ramp. New shafts and caps were added to extend Piers 18 to 28. Photograph 6 shows the extended piers at the north approach spans. In addition, new hammerhead piers for a curved exit ramp were added adjacent to Piers 23 to 28.

Additional modifications and repairs have been made to the superstructure but are not relevant within the context of this study.

Photograph 6. Pier Extensions - North Approach

2.6.7. Deck and Site Drainage

There are floor drains on each side of the existing bridge deck at nearly every pier location. The runoff water from the bridge deck flows through a series of downspouts and sloping troughs to locations where it is discharged on splash blocks or paved surfaces at the base of the columns. The discharged water from the bridge deck is then directed toward catch basins located under the bridge between the southbound and northbound bridges, where it is combined with surface runoff from the parking lots and other areas below the bridge. Photograph 7 shows the drainpipes and a catch basin at the south approach spans. The individual catch basins are linked to be a longitudinal storm sewer system that follows the longitudinal centerline of the median.

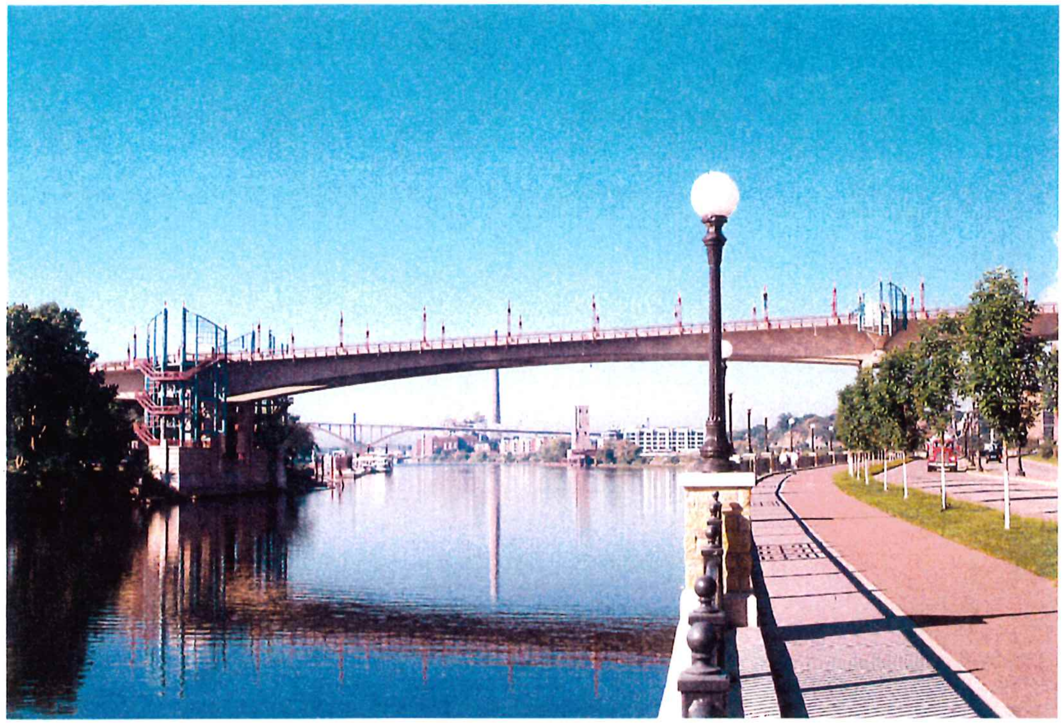
Photograph 7. Drainpipes and Catch Basin - South Approach

3. AESTHETIC OPPORTUNITIES

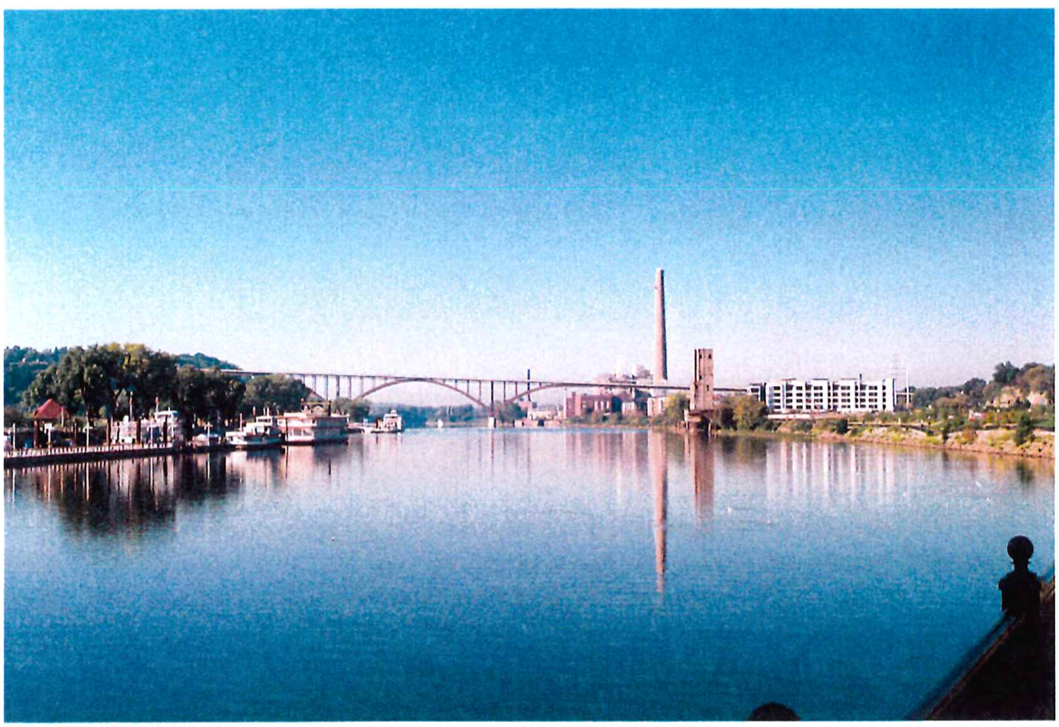
3.1. General Setting and Adjacent Bridges

The Lafayette Bridge is one of four roadway bridges across the Mississippi River in the vicinity of downtown Saint Paul. The other three bridges are located upstream from the Lafayette Bridge. Each of these three bridges has different characteristics and a different setting. Closest to the Lafayette Bridge is the Robert Street Bridge, which was built in the 1920s in the art deco style (see Photograph 8). It consists of eight arch spans of variable span lengths. Located farther upstream, the Wabasha Street Bridge is a four-span post-tensioned box girder bridge with a variable structure depth (see Photograph 9). It was built in the 1990s. The bridge farthest away is the Smith Avenue Bridge, also known as the "High Bridge." Its main spans are tied arch spans: one full arch span flanked by a semi-arch on each side. (See Photograph 10.) The High Bridge was built in the 1980s.

Photograph 8. Robert Street Bridge



Photograph 9. Wabasha Street Bridge



Photograph 10. Smith Street Bridge

3.2. Impact of Clearance Requirements

The clearance requirements for the nearby airport do not allow tall projections above the bridge deck. Therefore, a cable-stayed bridge or an extradosed post-tensioned concrete bridge cannot be considered for this location. Similarly, an arch bridge is inadmissible because of clearance restrictions above and below the bridge deck. Girder bridges appear to be the only structure type that can meet the clearance requirements at this site.

3.3. General Aesthetic Features

The need for extensive reconstruction affords the opportunity to develop a comprehensive aesthetic concept for the Lafayette Bridge, which should comprise all bridge components. One of the most important aesthetic features will be the silhouette of the Lafayette Bridge against the river valley to the east and the city skyline to the west. The locations of the river piers have been predetermined. The vertical bridge profile will allow small adjustments. The following simplified elevation views show the overall proportions of the river spans (see Figures 8, 9 and 10). These figures also convey the opportunity to consider different concepts in girder design which will affect the overall bridge aesthetic qualities.

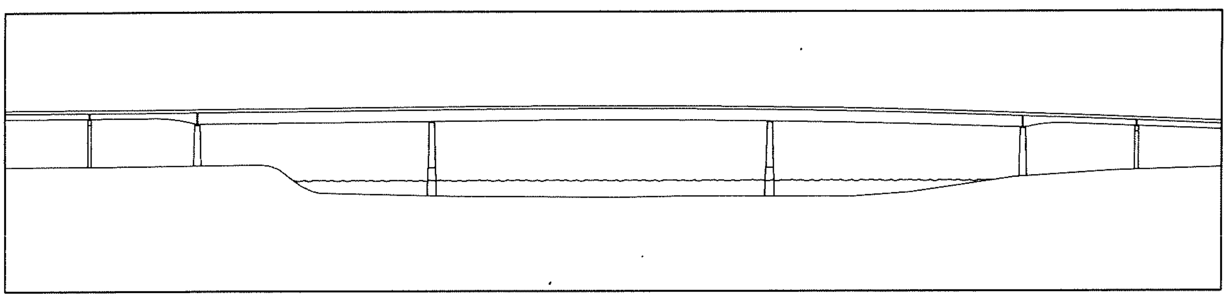


Figure 8. Elevation, Constant-Depth Steel Girder

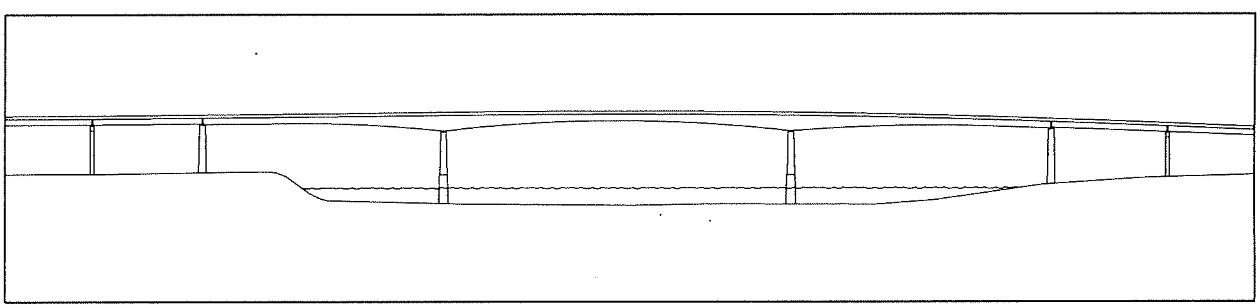


Figure 9. Elevation, Steel Box Girder and Variable-Depth Steel Girder

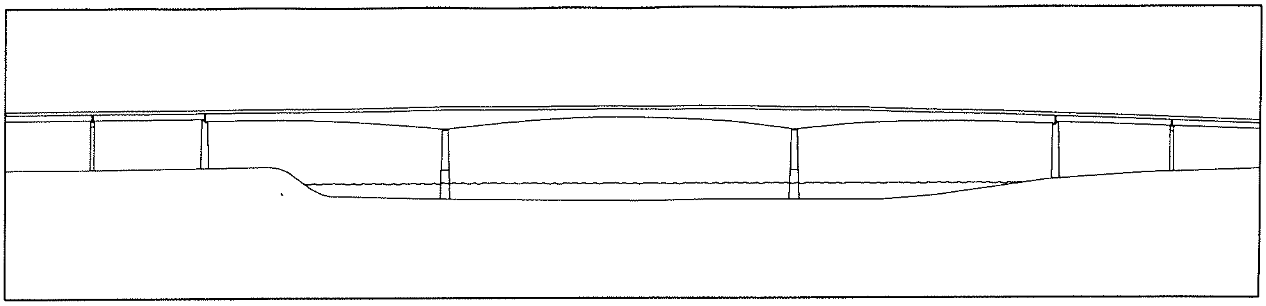


Figure 10. Elevation, Concrete Box Girder

The transition details from the river spans to the approach spans require special attention. Abrupt, large structure depth differentials should be avoided for the transitions.

The overall appearance of the bridge could be further enhanced through features of the piers and by the color scheme. The attractiveness of the bridge at night could be enhanced with an architectural lighting system.

3.4. Specific Aesthetic Features

3.4.1. Piers of the Approach Spans

All of the approach span piers are tall; i.e., they are more than 25 feet high. Most of the piers for the approach spans are located in commercial zones. In addition, the bridge widening projects of 1981 and 1991 have diminished the aesthetics of the original piers. The reconstruction project will provide an opportunity to improve the aesthetic qualities of these piers. While structural efficiency will play an important role, the introduction of aesthetic features for the piers would help to enhance their appearance.

3.4.2. River Piers

The river piers have larger proportions than the approach piers. Some features, such as a common pile cap, are required structurally. The superstructure type and the construction sequence will greatly influence the shape of the river piers. Aside from meeting structural and hydraulic requirements, the river piers may be shaped to fit the overall aesthetic concept.

3.4.3. Superstructure and Railings

In the absence of pedestrian traffic on the bridge, the importance of architectural features at the deck level is reduced. The appearance of the bridge as seen from below or from afar is much more important. The structure type will have a great visual impact. Depending on the superstructure type, the view from below will either show individual girders and diaphragms or, in the case of box girders, arching surfaces.

The choice of the outside traffic railings will impact the overall appearance of the bridge. Aesthetic enhancement of the traffic barriers will be limited to the outside faces of the barriers.

4. DESIGN, CONSTRUCTION, AND MAINTENANCE

4.1. Design Criteria

The design criteria listed below were utilized for this study. Some of the information, such as the barge tow data, was taken from design criteria used for the design of the Wabasha Street Bridge, which is located approximately 0.65 miles upstream of the Lafayette Bridge and was constructed in 1996. Slight adjustments were made in hydraulic elevation data to account for water surface profile differences between the two sites. The barge dimensions vary somewhat from those in the *AASHTO LRFD Bridge Design Specifications* but are representative of the actual barge traffic navigating this stretch of the Mississippi River at the time of the Wabasha Bridge design. This data should provide reasonable criteria from which to base preliminary designs. During the design phase of the project, all of the design criteria should be substantiated to be certain they are current and applicable.

4.1.1. Geometrics

- Permanent Deck Dimensions
  - Two 12-foot lanes northbound and southbound.
  - One 12-foot auxiliary lane northbound and southbound.
  - 12 foot-0 inch outside shoulders (or as required due to capacity restraints).
  - 12 foot-0 inch interior shoulders (or as required due to capacity restraints).

- Temporary Lane Widths for Staged Construction
  - One 13-foot lane northbound and southbound.

- Railings
  - Outside: Structural Tube Railing (Design T-1) and Concrete Parapet (Type P-2).
  - Inside: Split Median Barrier and Glare Screen Type F.

- Alignment
  - Per preliminary layouts received from Mn/DOT Metro Office (Options 1a, 1b, and 2—January 11, 2006; Option 3—September 21, 2006).

- Horizontal Clearances

  - Mississippi River: 350-foot navigation channel.
  - Railroad: Without crashwall = 25 feet-0 inches;
    With crashwall = 9 feet-0 inches.
  - Warner Road: 2 feet-0 inches beyond gutterline.

- Vertical Clearances

  - Over railroad: 23 feet-0 inches.
  - Mississippi River: 51.3 feet above 2% flowline or 59.6 feet above normal pool.

4.1.2. Water Surface Elevations

(M.S.L. 1929 Adj.)

- Normal pool: Elevation 686.75.
- 2% flowline: Elevation 695.05.
- Design high water (Q_{100}): Elevation 705.65.
- High water (vessel collision design): Elevation 701.30.
- Maximum observed high water (1965): Elevation 709.20.
- Cofferdam seal high water design: Elevation 700.00.

4.1.3. Barge Impact

- Design Vessel (Barge)

  - Length (L_B): 200 feet.
  - Width (B_M): 35 feet.
  - Depth (D_V): 14 feet.
  - Empty (light) draft (D_E): 2.0 feet.
  - Loaded draft (D_L): 10 feet.
  - Depth of bow (D_B): 15-foot bow rake length (R_L) – 20 feet.
  - Head log height (H_L): 2.0 feet.
  - Cargo capacity (C_C): 1,612 tons.
  - Empty displacement (W_E): 300 tons.
  - Loaded displacement (W_L): 1,912 tons.
  - Dead weight tonnage (DWT): 325 tons.

- Towboat
  - Length: 90 feet.
  - Width: 25 feet.
  - Typical draft: 9.0 feet.
  - Loaded displacement: 200 tons.

- Tow Configuration (Loaded Barges)
  - Upstream direction: 3 barges wide by 5 barges long.
  - Downstream direction: 2 barges wide by 4 barges long.

- Vessel Speed
  - Upstream direction: 4 mph.
  - Downstream direction: 8 mph.

- Current Velocity
  - Parallel to channel: 6.0 fps.
  - Cross current: 0.5 fps.

4.1.4. Design Loading

(Per AASHTO and Mn/DOT requirements, as stated below.)

- Dead Loads
  - Unit weight of reinforced concrete: 150 pcf.
  - Unit weight of structural steel: 490 pcf.
  - Initial wearing course: 2-inch low slump concrete.
  - Future wearing course: 20 psf.
  - Barrier:
    Exterior: 425 plf.
    Interior: 582 plf.

- Live Load Plus Impact
  - HL-93.
  - Dynamic load allowance: Per AASHTO 3.6.2.
  - Live load surcharge: Per LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.

- Wind Loads

  - Design wind speed: 100 mph.
  - Base wind pressures on superstructure: Per Table 3.8.1.2.2-1.
  - Vertical wind pressure on bridge deck: 20 psf applied at windward quarter point of bridge deck width.
  - Base wind pressures on substructure: 40 psf and per Table 3.8.1.3-1.
  - Base wind pressure on live load: 100 plf acting 6 feet-0 inches above the bridge deck.

- Earth Loads

  - Compacted earth backfill: 120 pcf.
  - Submerged earth: 125 pcf.
  - Horizontal earth pressure: 33 pounds/cubic foot equivalent fluid pressure for engineered fill.

- Centrifugal Force

  - Applied in accordance with AASHTO 3.6.3.

- Longitudinal Forces

  - Per AASHTO 3.4.5.

- Earthquake Effects

  - Seismic Performance Zone 1.

- Forces From Stream Current and Floating Ice

  - For stream pressure, the current velocity shall be taken as 6.7 mph.
  - For stream pressure, the water surface elevation shall be taken as 707.23.
  - For dynamic ice force, the strength of the ice shall be taken as 200 psi over a thickness of 18 inches.
  - For dynamic ice force, the water surface elevation shall be taken as 700.32 (two-thirds of the distance from the flowline elevation to the 100-year high water elevation).
  - For channel piers, sheet ice exerting a force of 20,000 plf applied parallel to the centerline of bridge at elevation 686.50 shall be used.

- Thermal Forces

  - Mean temperature: 45°F.
  - Concrete Superstructures:
    Thermal coefficient: 0.000006/°F.
    Seasonal variation for design of structure:
    Temperature Range for Procedure A:
    Temperature rise of 35°F.
    Temperature fall of 45°F.
    Temperature Range for Procedure B: 120°F.
    Post-tensioned concrete box girder structures with integral substructures, longitudinal frame action design forces to be generated from a $\Delta T = 120°F$.
  - Steel Superstructures:
    Thermal coefficient: 0.0000065/°F.
    Seasonal variation for design of structure:
    Temperature rise of 75°F.
    Temperature fall of 75°F.

4.1.5. Bearing Assemblies

- Temperature range for design of bearings: 150°F (-30°F to 120°F).

- Per load tables in the Mn/DOT *LRFD Bridge Design Manual.*

- Pot bearings for loads that exceed the loads in the Mn/DOT *LRFD Bridge Design Manual.*

4.1.6. Expansion Joints

- Temperature range for design of joint openings: 150°F (-30°F to 120°F).

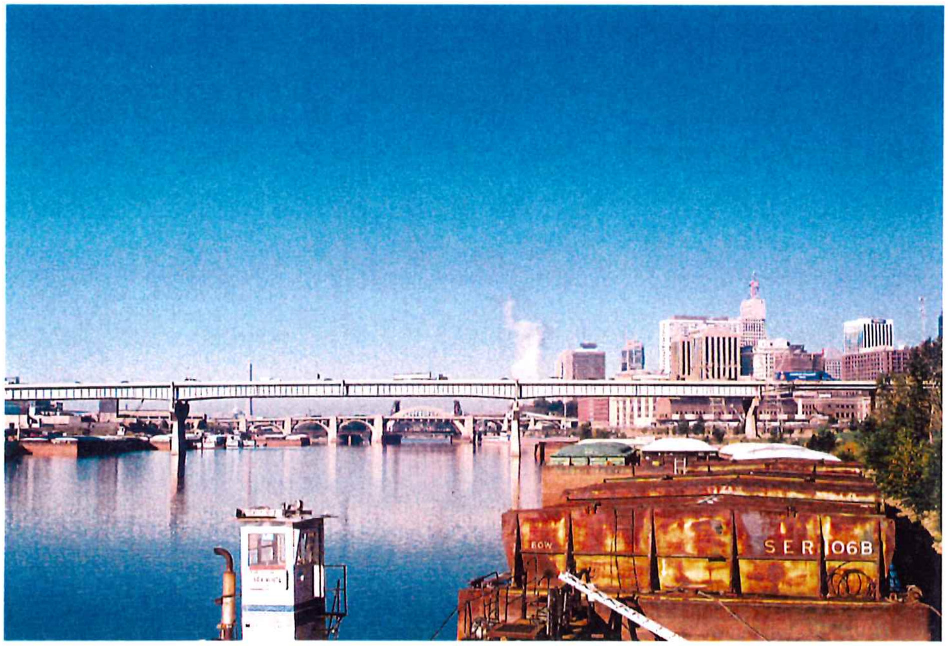
- Strip seals: Movements of 1/4 inch to 4 inches.

- Modular expansion joints: Movements greater than 4 inches.

4.2. Geometric Constraints

4.2.1. Vertical Constraints

The available structure depth of the river spans is constrained from below by a navigation channel and from above by a runway clear zone for Holman Field. The navigation channel, which is 350 feet wide and provides a 51.3-foot minimum vertical clearance above the 2% flood elevation, is located between Piers 9 and 10. Photograph 11 shows the navigation channel under the existing bridge. In a change from the original criterion, the governing runway clear zone is now associated with Runway 14-32. Runway 13-31, which previously controlled the elevation of the high point of the crest curve, is no longer used.



Photograph 11. Lafayette Bridge Navigation Channel

The current vertical constraints are a power line near Pier 11 and the clear zone for Runway 14-32. In the vicinity of Pier 11, an overhead power line crosses the Lafayette Bridge at roughly 90 degrees. (See Photos 12 and 13.) The voltage of this power line is 115 kV, which would require a minimum vertical clearance of 25 feet. The existing bridge does not meet this stringent clearance requirement. However, this power line constitutes the most critical obstruction within the runway clear zone for Runway 14-32.

Photograph 12. Xcel Energy Power Lines Looking East

Photograph 13. Xcel Energy Power Lines Looking West

The clearance requirements for the power line with regard to the bridge deck, plus the clearance requirements for Runway 14-32 and the navigation channel, result in a triple constraint on the available superstructure depth. Variable-depth superstructure types would violate the shipping channel clearance requirements because the location of the power line restricts the magnitude of the required profile grade raise.

The vertical clearance requirements for railroad tracks and roadways can be easily met. With the exception of three tracks located north of Warner Road, all other railroad tracks underneath the Lafayette Bridge have been removed.

The elimination of the clear zone for Runway 13-31 over the Lafayette Bridge allows modification of the vertical profile. Two vertical profiles have been developed: Figures 11 and 12 on the following pages. Figure 13 highlights the interference problems with the 115 kV power line. (See Figures 4A and 5A in Appendix A.)

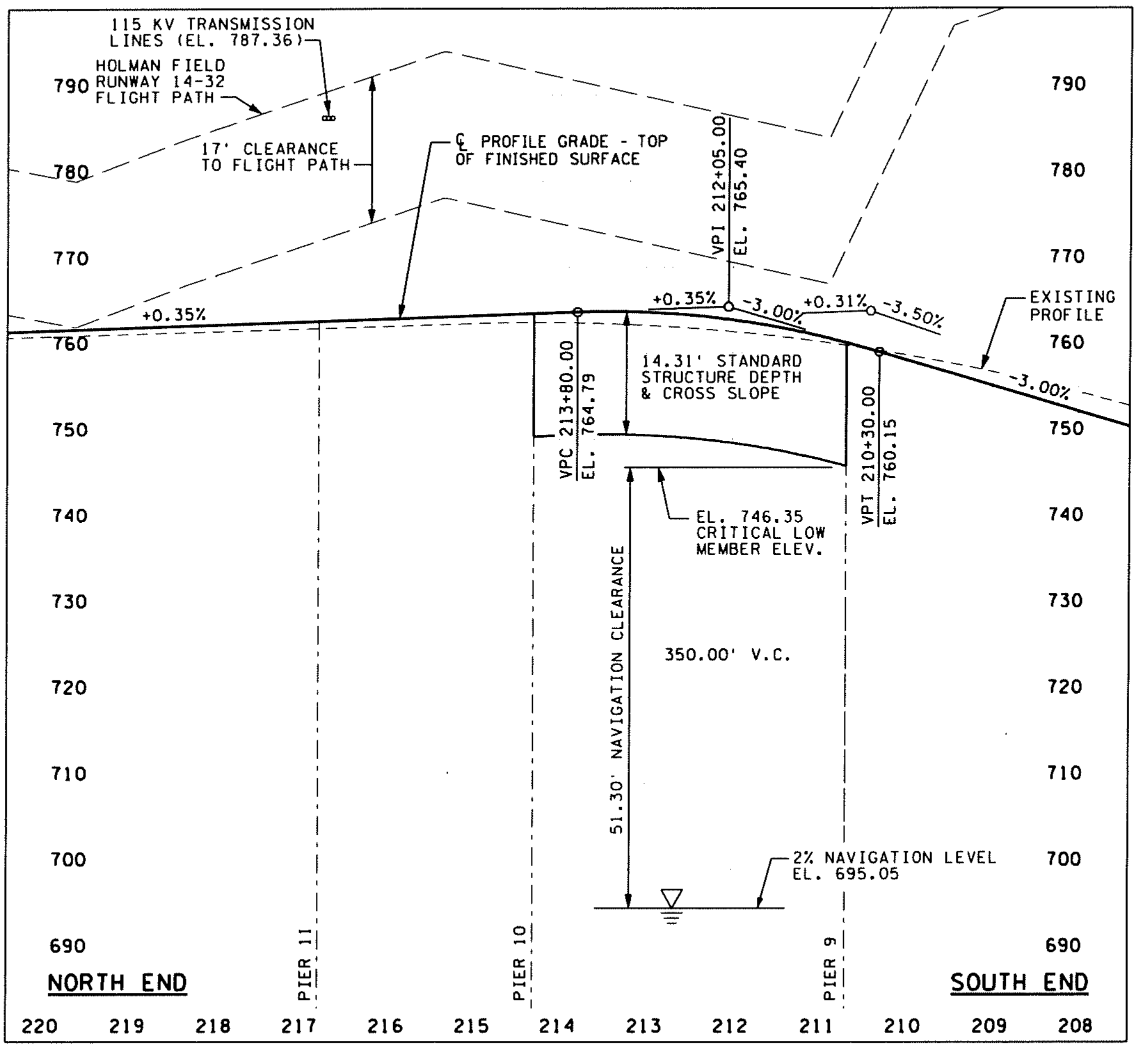


Figure 11. Vertical Profile, Standard Depth

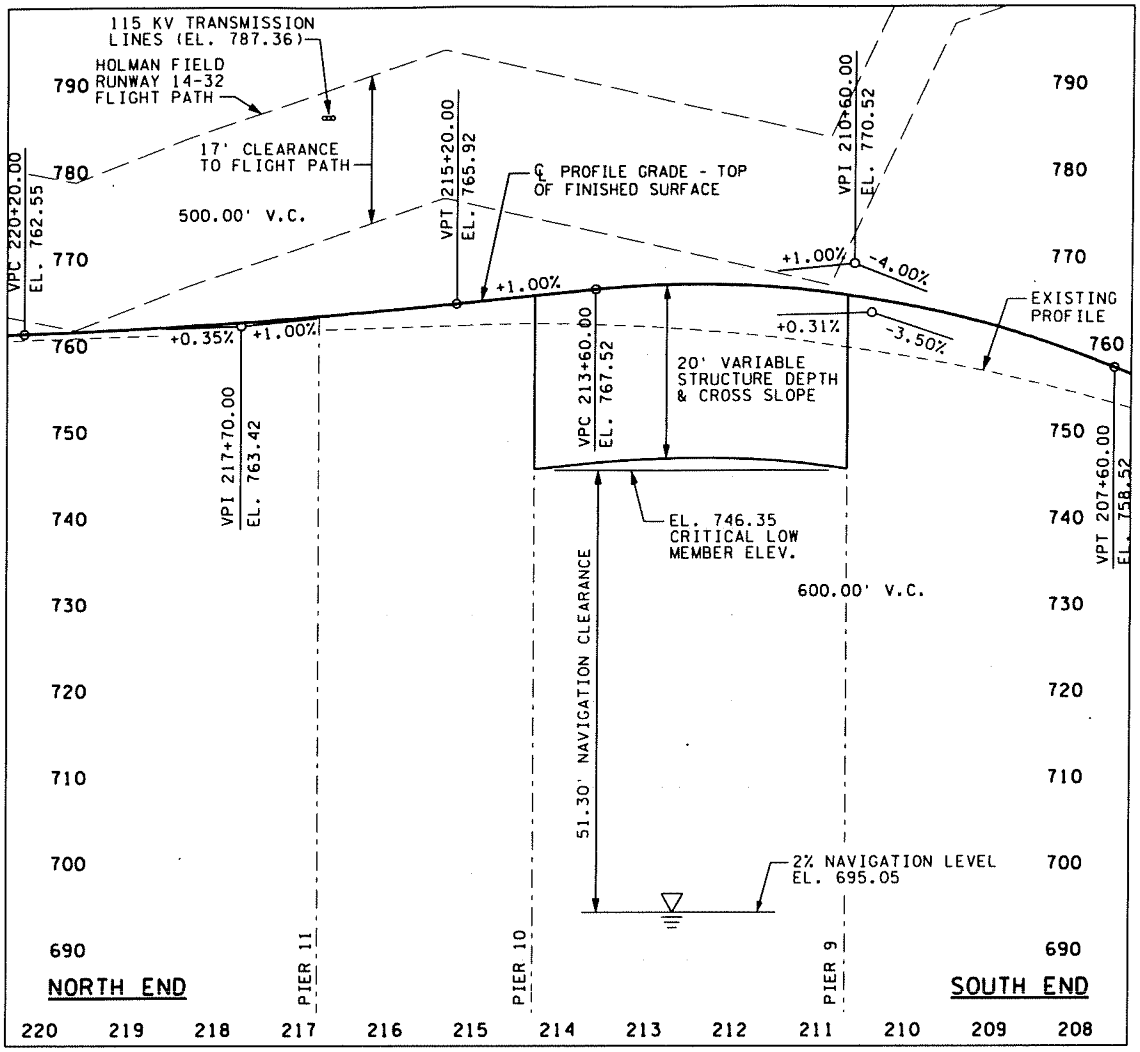


Figure 12. Vertical Profile, Variable Depth

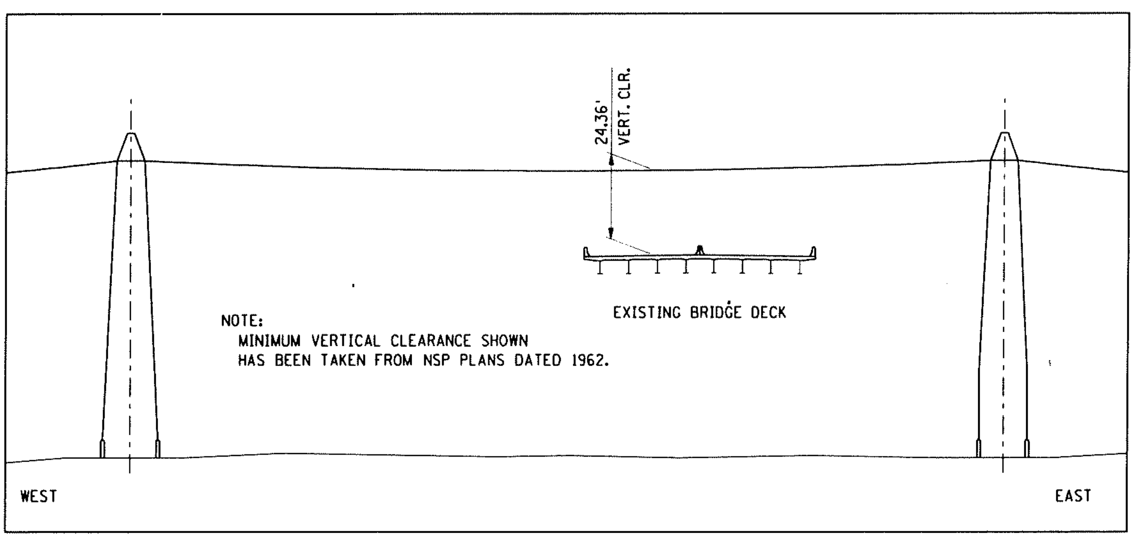


Figure 13. Transmission Line Section

At the north end, the vertical profile needs to match the existing condition. (For a detailed discussion of the existing vertical profile, see Section 2.6.1.)

4.2.2. Horizontal Constraints

Geometric Roadway Layout: The general geometric layouts for the options to be studied were provided by Mn/DOT. The alignments, particularly at the north end, consider the pertinent right-of-way issues and the intersection geometry.

Overhead Power Line and Runway Clear Zone: These two constraints have both horizontal and vertical impacts. The overhead power line crosses the Lafayette Bridge at roughly 90 degrees near Pier 11. Runway 14-32 has an azimuth of 324.8 degrees, while the alignment of the Lafayette Bridge has an azimuth of 338.97 degrees. The relationship of these constraints is shown in Figures 14 and 15. The figures show the relationship between the eastern bridge limits and the interference with both the power line and the runway clear zone. Simply stated, the farther east the bridge is located, the greater the interference problems with the flight path.

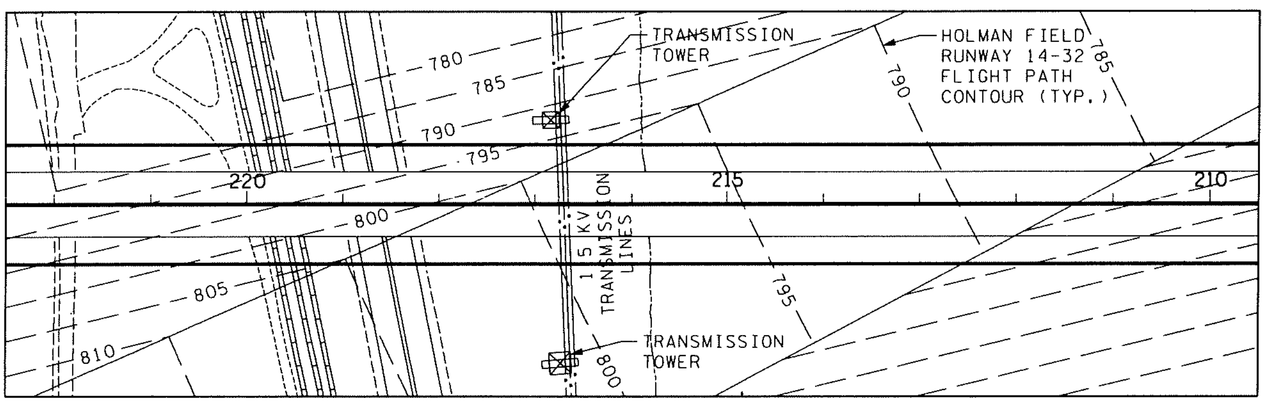


Figure 14. Option 1b, Flight Path, Xcel Plan

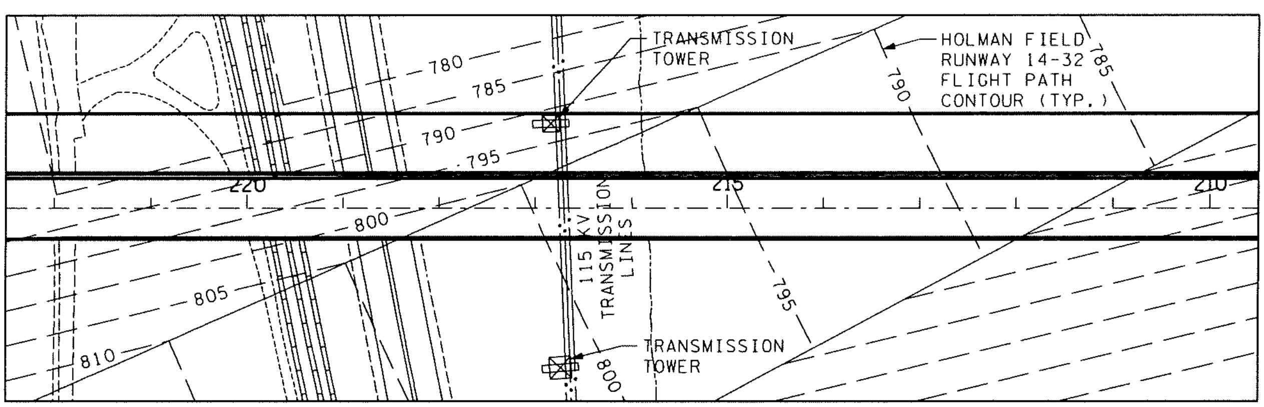


Figure 15. Option 3, Flight Path, Xcel Plan

River Piers: The location of the current shipping channel, together with the pile configuration of the river pier footings, greatly limits the options available for new pier locations. For hydraulic reasons, any new river piers should line up with the existing river piers. The requirement to avoid interfering with existing piles will determine the necessary horizontal clearances between existing and new foundations.

Approach Piers: Wherever possible, the piers in the approach spans were either oriented at 90 degrees with respect to the centerline of the bridge or were oriented to match the skew angle of the roadway grid. However, there are a number of piers with irregular skews. Most of the irregular skews were caused by the presence of railroad tracks; most of those railroad tracks have since been removed. Warner Road also has been relocated. The elimination of a number of horizontal constraints affords the opportunity to redefine suitable pier locations and orientations.

The orientation of some skewed piers (Piers 11 and 14) negates symmetrical pier widening schemes due to interference with roadways or railroad tracks.

4.3. Maintenance of Traffic

Maintenance of traffic during construction was a key requirement for all of the bridge widening options. At a minimum, at least one lane of traffic in each direction will need to be kept open at all times during construction. This requirement can be easily met under Option 3, which would entail constructing a separate bridge for the northbound traffic.

The non-redundant nature of the piers provides some technical challenges for maintenance of traffic under Option 1b. The degree of difficulty would be different for the river piers and the approach span piers. The typical approach span pier on the Lafayette Bridge has two square columns and a variable-depth cap beam with large cantilevers. Such a pier frame gives the appearance of two tall and connected hammerhead piers with slender columns. Breaking the continuity of the pier cap during construction to allow two lanes of traffic to stay open would create two unstable hammerhead piers. (See Section 8 for a concept for dealing with this instability.)

Option 1b would require that traffic be shifted during the course of construction: first to the west side of the bridge (while the existing east superstructure is removed and a new structure is constructed), then to the new partial east structure (while the new west structure is constructed), and then again to the west side of the bridge while the east side is completed.

4.4. Constraints on Construction Operations

Reconstruction of the Lafayette Bridge will be impacted by the need to maintain other modes of traffic and utility services during construction. The presence of Holman Field, the Mississippi River, the Canadian Pacific Railway, and busy streets such as Kellogg Boulevard and Warner Road will cause construction schedules to be extended due to work stoppages. Crane operations will have to be coordinated with the air traffic controllers at Holman Field and booms will need to be lowered for incoming and outgoing flights. Work in the Mississippi River will need to be coordinated with river navigation and the channel will have to be protected from falling debris. Work over the railroad tracks will require flagging services and will have to be limited to established work windows. Work in the vicinity of the high voltage power lines can be expected to require power outages, which can often be scheduled only at certain times of the year. Traffic on Kellogg Boulevard and Warner Road, if maintained during construction, would have less impact on the construction schedule but would still require special treatment. Existing commercial buildings, such as the old Gillette property, are in close proximity to the bridge. A 24-inch outside diameter water main ascends the south face of Pier 8, spans the Mississippi River, and descends again at the north face of Pier 11. All of these conditions will extend the time and increase the cost of construction, especially for the options that require a more tedious reconstruction solution.

4.5. Accessibility for Bridge Inspection

For the purposes of this study, all of the options under consideration were required to provide accessibility for bridge inspection by way of a snooper truck. All of the horizontal bridge layouts were developed by Mn/DOT. From the point of view of bridge maintenance staff, an 9-foot gap between the northbound and southbound bridge decks is desirable. However, in a design progress meeting, the observation was made that the under-bridge arm of a snooper truck can reach as far as 75 feet underneath the bridge deck. Such a reach would provide access to all parts of the bridge.

The gap widths shown in the figures of this report are based on the alignments provided by Mn/DOT. The required gap width and the corresponding alignments will have to be determined prior to the final design.

5. EVALUATION OF EXISTING SUBSTRUCTURE COMPONENTS

5.1. Methodology

The premise for the presentations in the following sections is as follows: it is possible to design a new superstructure of adequate capacity for any one of the desired alternates, but there are two important limitations which need to be investigated. First, all existing substructure components (piles, footings, and pier columns or shafts) need to have adequate capacity to carry the new required loads.

Second, it has to be possible to construct the new superstructure while meeting the traffic maintenance requirements.

Each existing piles and footings have an LRFD design capacity, which is independent from a specific load case. In the case of an existing column, its LRFD design capacity can be shown in an interaction diagram, which also is independent from specific load cases. These design capacities are common to all of the options and alternatives. For this reason, the general discussion of the LRFD design capacity of the existing piles, footings and pier columns is presented in Sections 5.2, 5.3, and 5.4, respectively.

For a specific option it was necessary to determine the appropriate superstructure loads and check whether the force effects exceed LRFD design capacities, or not. The results of these investigations are reported in the sections covering each option. Equally, construction and traffic maintenance issues are also discussed under the sections for each option.

5.2. LRFD Design Capacity of Existing Piles

5.2.1. Existing Information

The following information was available for evaluation of the existing piles:

- Bittner, K.F., "Bridge Scour Analysis for the St. Paul Project," Memorandum for Record, State of Minnesota Department of Transportation, 1988.

- Drawings Nos. 1 through 9, "Bridge 9800 T.H. 56 (Lafayette Road) over Streets, Mississippi River, & Railroads in St. Paul, Minnesota Project No. I-094-3(94)-241," May 1964, State of Minnesota, Department of Highways.

- Drawings Nos. 1 through 14, "Construction Plan for Bridge 9800 Piers 5, 6, 8, 7 (Contract B), Minnesota Project No. U-044-1(31)," August 1962, State of Minnesota, Department of Highways.

- Drawings Nos. 1 through 27, "Construction Plan for Bridge 9800 Piers 1 Thru 7, 8, and 11 Thru 13 (Contract C), Minnesota Project No. U-044-1-(31)," April 1964, State of Minnesota, Department of Highways.

- Drawings Nos. 1 through 56, "Construction Plan for Br. Widening - Br. 9800 Located on T.H. 3 (Lafayette Road) over Streets, Mississippi River, & Railroads in St. Paul, Minnesota Project No. 6283-16(94=392)," July 1981, Minnesota Department of Transportation.

- Drawings Nos. 1 through 105, "Construction Plan for Br. 9800 Widening Located on Lafayette St. (T.H. 3) from Kellogg Blvd. to T.H. 94, Minnesota Project No. 6283-9800B(T.H. 3 = 112)," January 1992, Minnesota Department of Transportation.

- Hendrickson, Andrea, "Bridge Scour Rating – Lafayette Bridge #9800," Office Memorandum, State of Minnesota Department of Transportation, 1992.

- "Pile Driving Reports – Pile Installation Records for Production Piles at Each Support Location," May 1963 through June 1963 (Contract B), August 1964 through November 1964 (Contract C), and April 1965 through November 1965 (Contract D).

- "Test Pile Reports – Pile Installation Records for Test Piles at Each Support Location," December 1962 through February 1963 (Contract B), August 1964 through October 1964 (Contract C), March 1965 through July 1965 (Contract D), and April 1992 through May 1992.

- Collins Engineers, Inc., "Water Depth Soundings at Pier 9 of MinnDOT Bridge No. 9800," January 1994.

5.2.2. Subsurface Conditions

The geotechnical information available for review of the site consists of the original boring logs and the boring logs for the 1991 bridge widening project. The original subsurface investigations consisted of fourteen borings taken prior to the original construction. Two additional borings were made for the 1991 widening. The boring logs are summarized in Table 4.

Table 4. Summary of Boring Logs

Boring Number	Bottom of Boring Elevation (feet)	Location
Original Bridge		
T-1	552.5	Near Pier 11
T-2	562.6	North of North Abutment
T-3	556.3	Between Piers 18 and 19
T-4	561.6	Between Pier 28 and North Abutment
T-5	557.9	Near Pier 24
T-6	554.7	Between Piers 15 and 16
T-7	549.9	Pier 8
T-8	549.2	Near Pier 2
T-9	549.1	Between Piers 4 and 5
T-10	547.9	Pier 10
T-11	549.9	Pier 10
T-12	565.3	Pier 9
T-14	677	North Abutment
1991 Widening		
T-15	667.1	North Abutment 189' Rt. of Median
T-17	636.9	Near Pier 24 115' Rt. of Median

In general, the borings along the bridge alignment indicate that the soil is primarily alluvial deposits from the ground surface down to bedrock elevation. The gradation of the alluvial deposits varies from fine to coarse, depending on depth and location. Mixing with gravel was also observed. Layers of fine grained soil, with up to 40% organic content, exist within the fill. The fine grained layers seem to increase in sand content and decrease in organic content toward the north end of the bridge. Typically, the fine grained layers exist as isolated pockets of less than about 15 feet in thickness; however, layer thicknesses of up to about 30 feet were observed. The bedrock is classified as Oneota Dolomite and was observed at the lower limit of each boring. Core samples were taken from the borings and indicate the bedrock to be hard and dense. The top of bedrock elevation is higher toward the north end of the bridge compared to the south end.

5.2.3. Foundation Information

Based on the pile driving records and construction plans, we observe that two types of piles were used for the original foundation construction. Piers 1 to 7 and Piers 12 to 28 are founded on 12BP53 steel H-piles, while Piers 8 to 11 are founded on 14BP73 steel H-piles. The original design for the pier foundations specified an end bearing pile with an allowable pile capacity of 78 tons for the 12BP53 piles and 110 tons for the 14BP73 piles based on Grade 36 steel. The North Abutment is founded on 12BP53 friction piles with a design capacity of 50 tons, while the South Abutment is founded on a spread footing. All pile groups contain battered piles. The amount of batter varies between substructure units and also within a given substructure foundation. For instance, the pile batter is 3 horizontal to 12 vertical for the North Abutment; while the pile batter used in the pier footings varies from 1.5 horizontal to 12 vertical, to 4.25 horizontal to 12 vertical. Pile tips were used on all the test piles as well as several of those driven in the foundations of Piers 26, 27, and 28.

The pier expansions in 1981 and 1991 utilized 12.75-inch-diameter 0.25-inch wall thickness cast-in-place (CIP) friction piles. The design capacities varied from 45.2 tons to 71.1 tons for the 1981 widening, and from 32.4 tons to 58.6 tons for the 1991 widening. The steel shell piles were driven with closed ends.

5.2.4. Review of Pile Installation and Testing Information

Test pile and production pile records taken at each substructure unit during the original construction and subsequent expansions were available for review. For the test piles, full length driving records from the original construction are graphically presented in Figures 16–19. For the pier foundation piles, the average pile tip elevation versus bedrock elevation is shown in Figure 20. The range of pile lengths between the shortest driven pile and the longest driven pile associated with each pier foundation is also indicated on Figure 20. A summary of the test pile installation indicating test pile number, pile type, pile length, pile installation contractor, original contract (Contract B through Contract D), pile hammer information, and driving resistance summary is shown in Table 5. Similarly, a summary of the production pile installation indicating pier number, pile type, average pile length, pile length scatter, pile installation contractor, original contract, and pile hammer information is shown in Table 6.

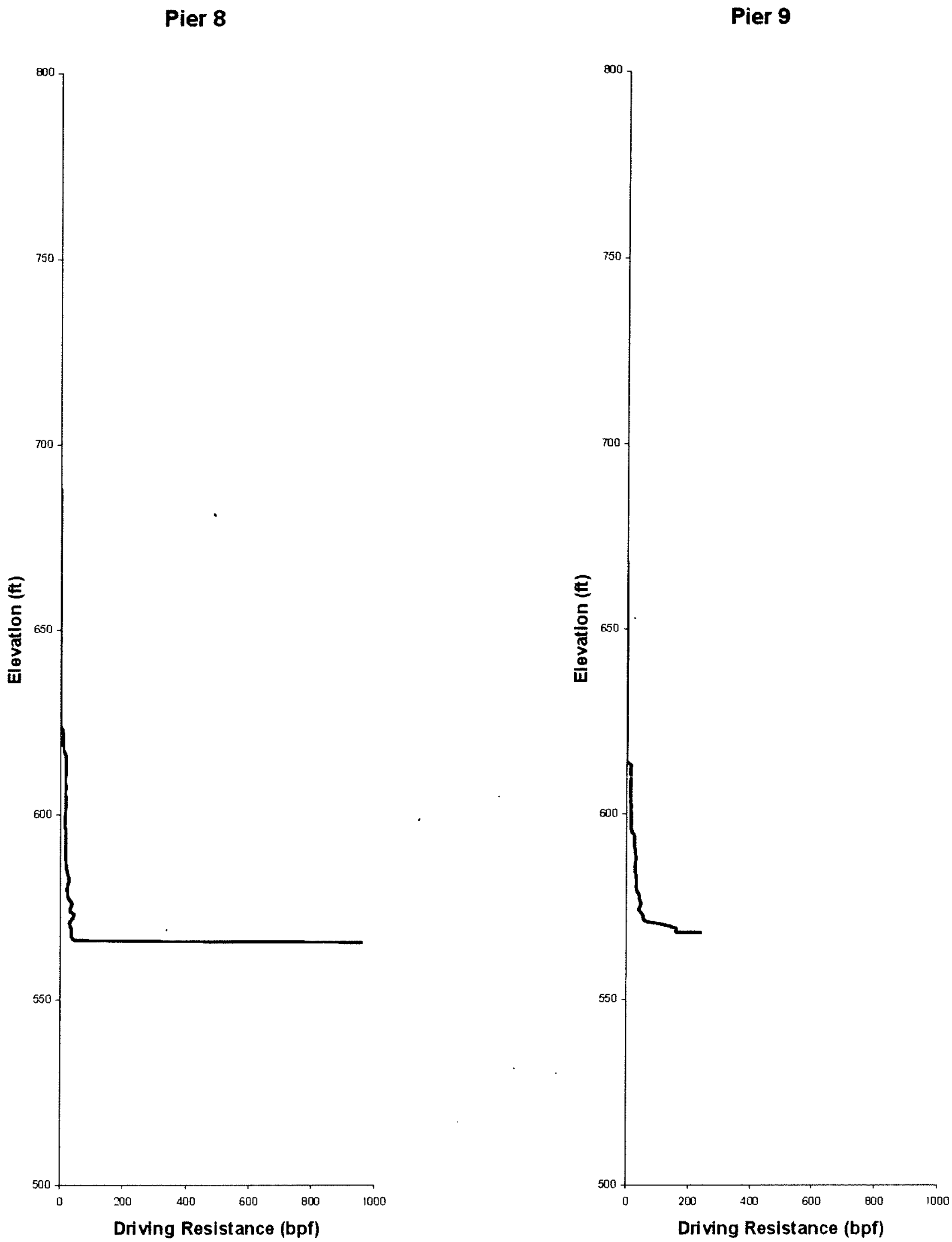


Figure 16. Test Pile Driving Resistance vs. Elevation – Contract B

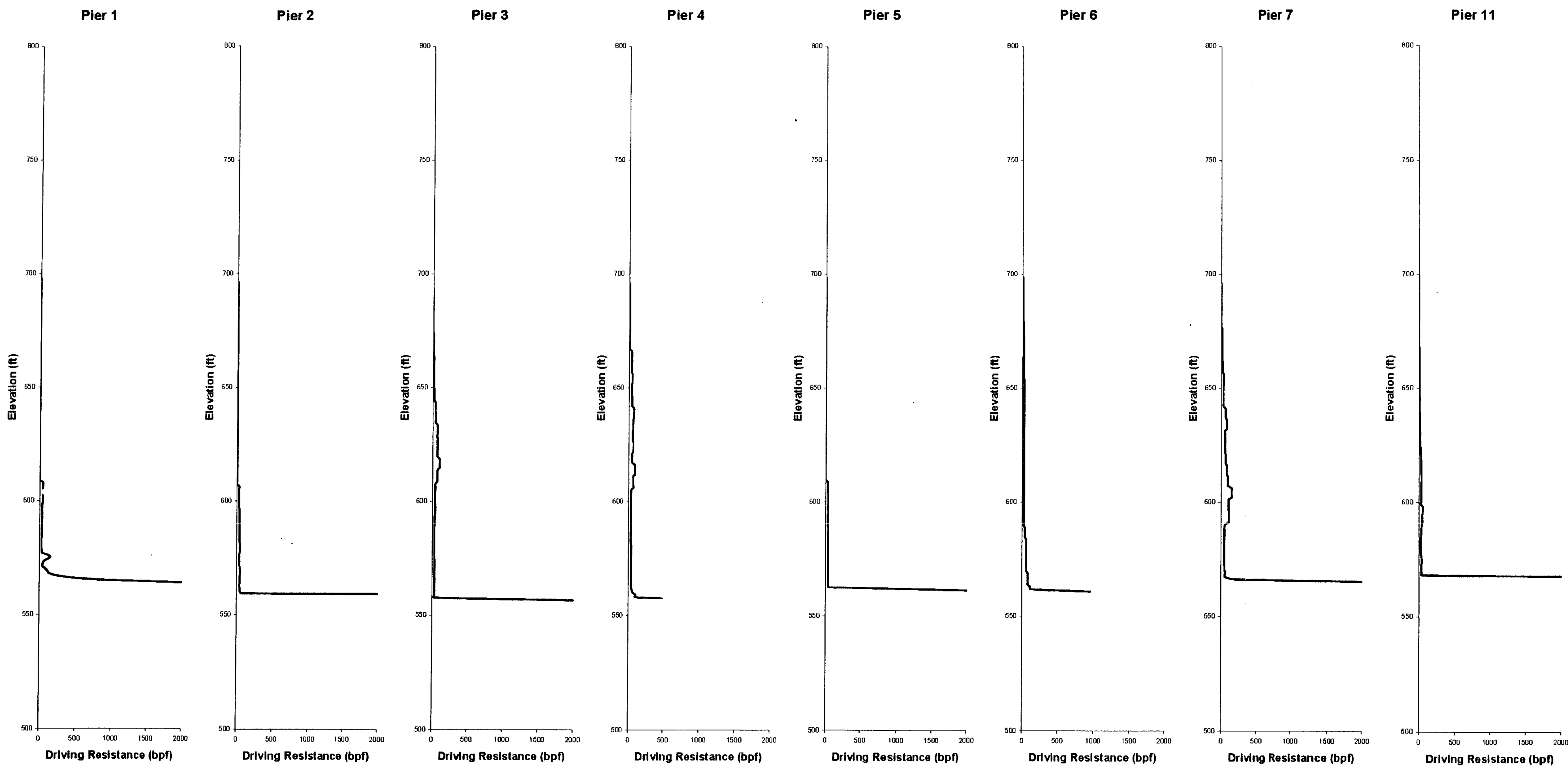


Figure 17. Test Pile Driving Resistance vs. Elevation – Contract C

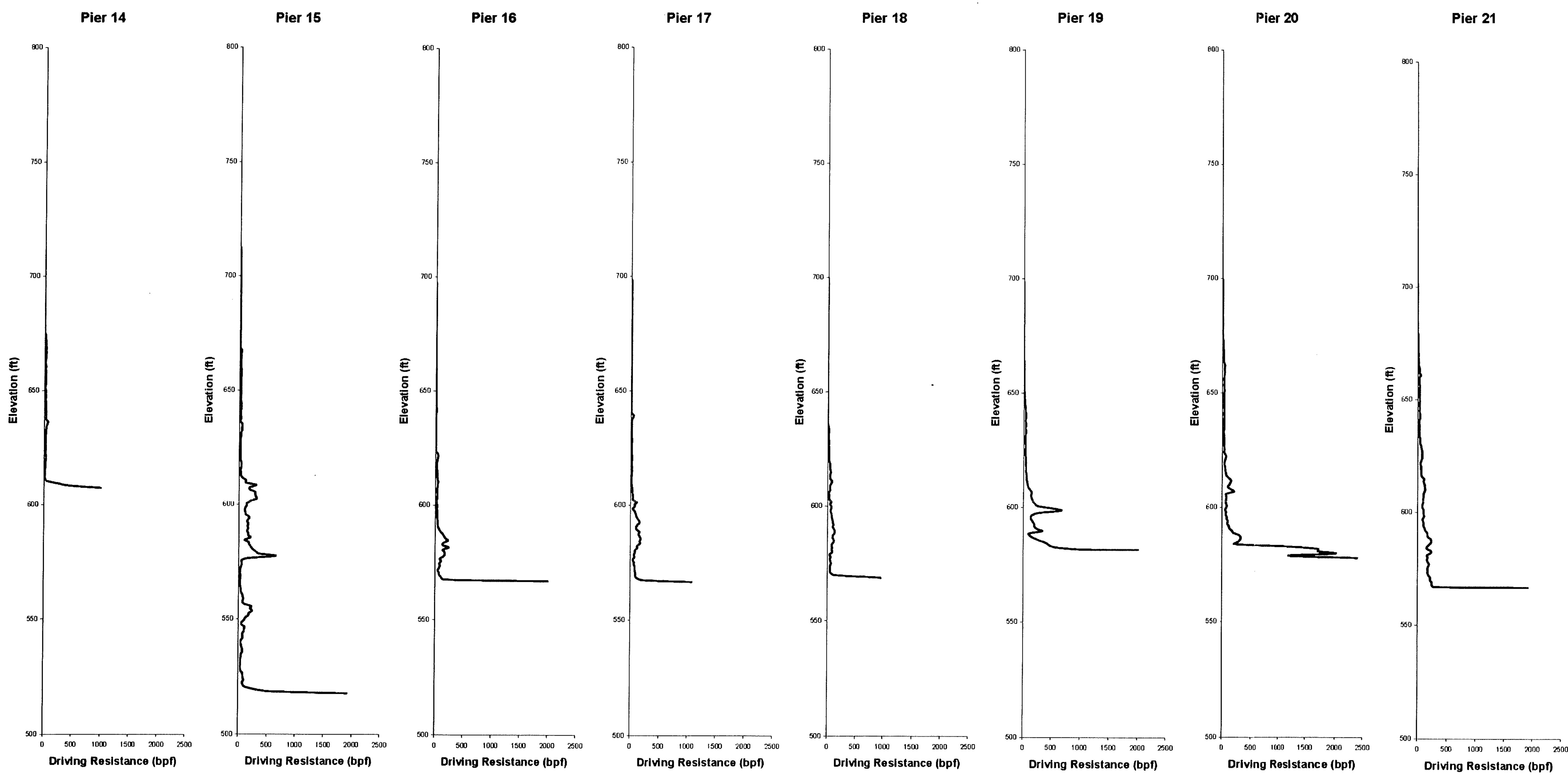


Figure 18. Test Pile Driving Resistance vs. Elevation – Contract D

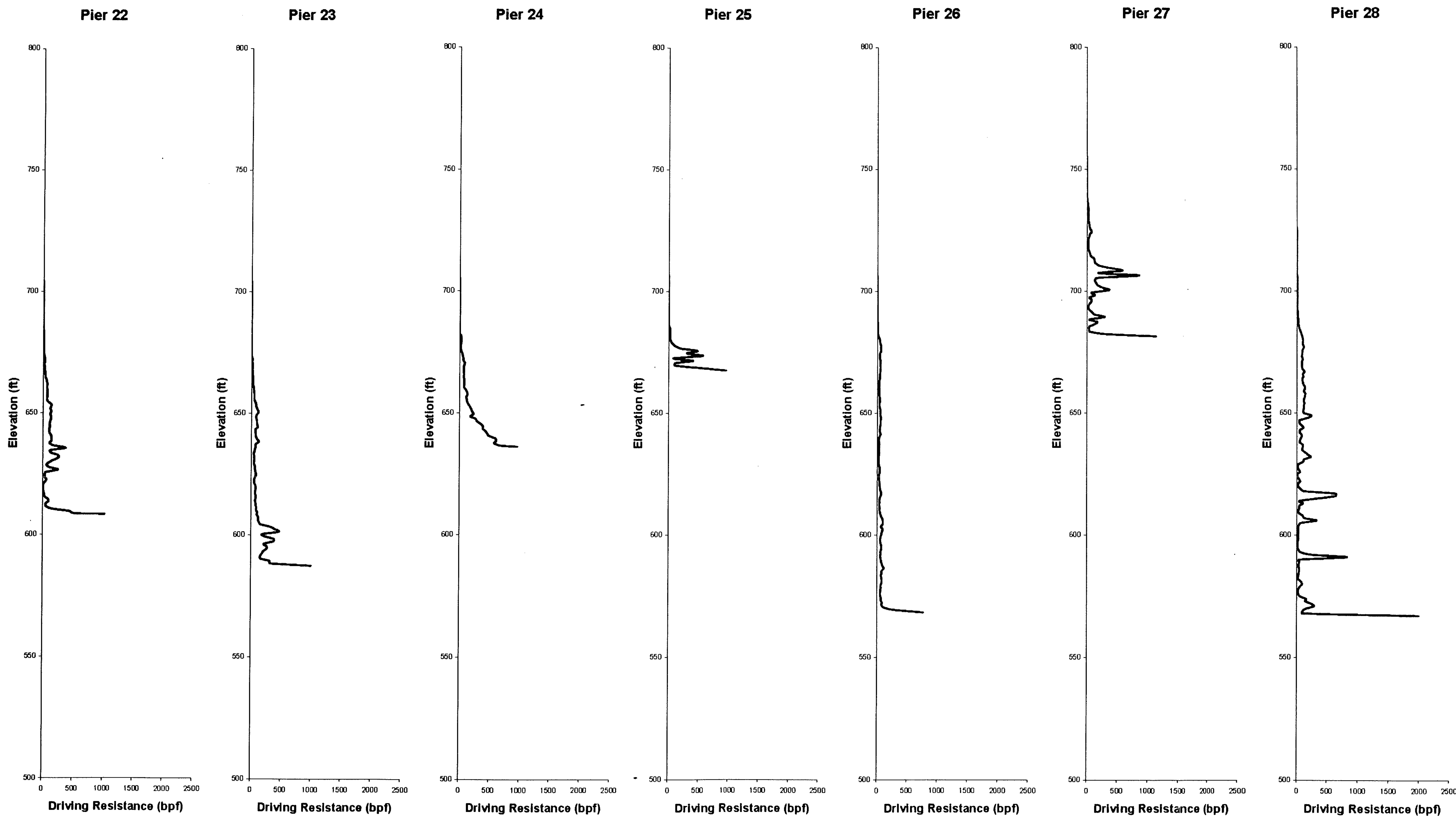


Figure 20. Test Pile Driving Resistance vs. Elevation – Contract D

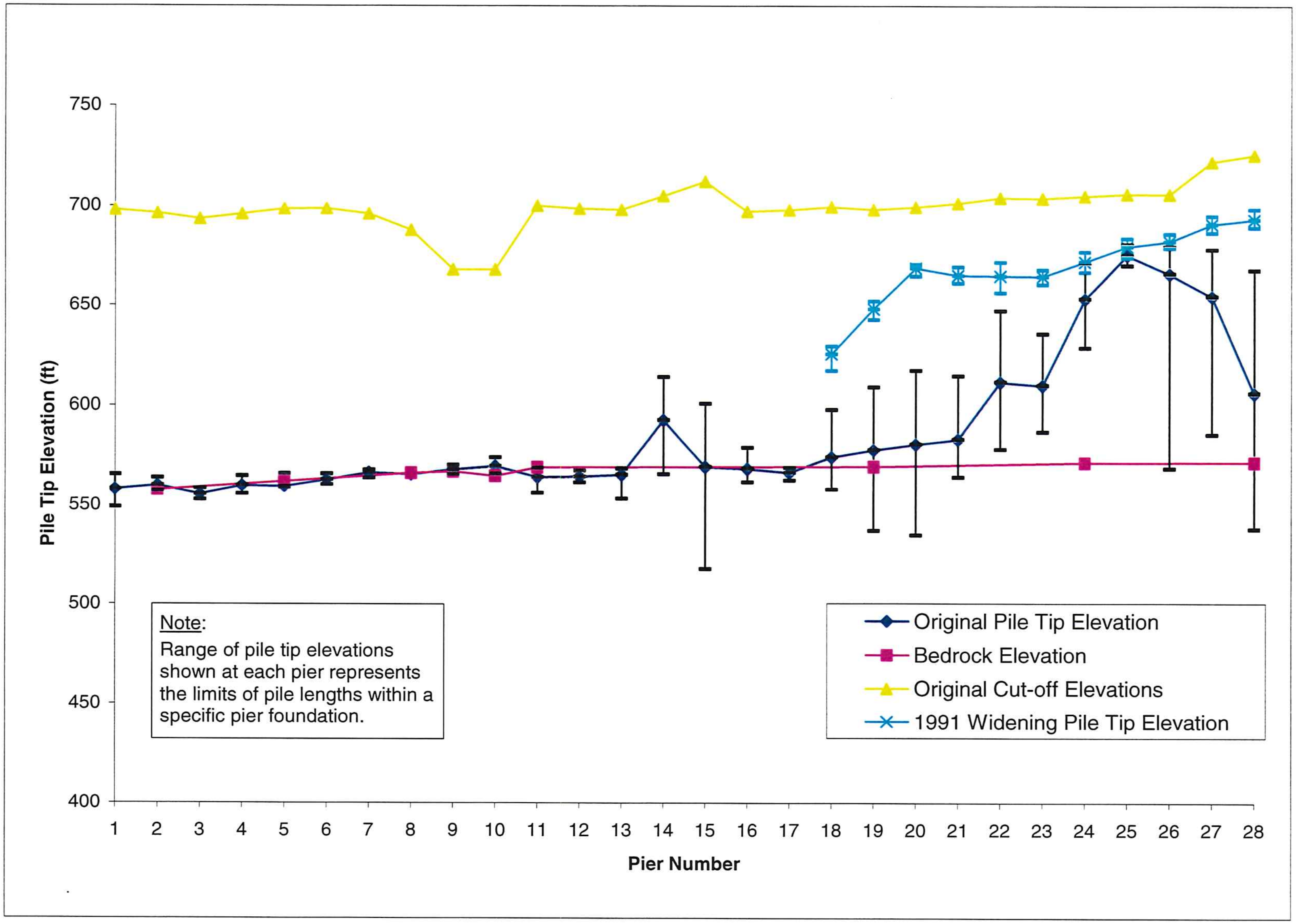


Figure 20. Pile Tip Elevations

Table 5. Test Pile Summary

Pier	Test Pile #	Pile Type	Length (ft)	Contractor	Contract	Hammer	Stroke	Rated Energy (ft-lbs)	Driving Resistance			Refusal	Cut-Off Elevation	Tip Elevation	Notes
									<25 bpf	25 bpf to 50 bpf	50 bpf to 150 bpf				
1	1	12BP53	134.6	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	N/A	Grade - El. 575	El. 575 - El. 565	El. 560	698.11	563.51	
2	2	12BP53	137.3	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	N/A	Grade - El. 560	El. 560 - El. 558	El. 558	696.43	559.13	
3	3	12BP53	137	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	3.25ft	16,000	N/A	N/A	Grade - El. 608	N/A	693.58	N/A	Driven to a depth of 85 feet with the S-8 Hammer
3	3	12BP53	137	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	4ft	26,000	N/A	El. 608 - El. 556	N/A	El. 556	693.58	556.58	Driven from 85 ft to refusal with the Link-Belt hammer
4	4	12BP53	139	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	3.25ft	16,000	Grade - El. 667	El. 667 - El. 642	El. 642 - El. 606	N/A	696.04	N/A	Driven to a depth of 90 feet with the S-8 Hammer
4	4	12BP53	139	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	4ft	26,000	N/A	El. 606 - El. 561	El. 561 - El. 557	El. 557	696.04	557.04	Driven from 90 ft to refusal with the Link-Belt hammer
5	5	12BP53	137.4	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	Grade - El. 614	El. 614 - El. 561	N/A	El. 561	698.57	561.17	
6	6	12BP53	138	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	22,000 - 26,000	N/A	N/A	El. 584 - El. 562	El. 561	698.74	560.74	
7	7	12BP53	131	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	3.25ft	16,000	Grade - El. 657	El. 657 - El. 642	El. 642 - El. 591	N/A	696.13	N/A	Driven to a depth of 105 feet with the S-8 Hammer
7	7	12BP53	131	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	4ft	26,000	N/A	El. 591 - El. 574	El. 574 - El. 567	El. 565	696.13	565.13	Driven from 105 ft to refusal with the Link-Belt hammer
8	1	14BP73	122.6	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	Grade - El. 578	El. 578 - El. 566	N/A	El. 565	688	565.4	Incorrect pile capacity formula
9	2	14BP73	100.1	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	Grade - El. 594	El. 594 - E. 574	El. 574 - El. 570	El. 568	668	567.9	Incorrect pile capacity formula
10	N/A	14BP73	N/A	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	N/A	N/A	N/A	N/A	668	N/A	Test pile driving record not available.
11	8	14BP73	132.5	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	Grade - El. 624	El. 624 - El. 568	N/A	El. 568	700.23	567.73	
12	1	12BP53	N/A	Sheehy Bridge Const. Co.	C	N/A	N/A	N/A	N/A	N/A	N/A	N/A	698.83	N/A	Test pile driving record not available.
13	2	12BP53	N/A	Sheehy Bridge Const. Co.	C	N/A	N/A	N/A	N/A	N/A	N/A	N/A	698.36	N/A	Test pile driving record not available.
14	3	12BP53	105	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 642	El. 642 - El. 610	N/A	El. 607	712.37	607.37	
15	4	12BP53	194.7	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 620	El. 620 - 608.5	El. 608 - El. 577	El. 518	712.5	517.8	Extremely hard layer between El. 608 & El. 577 - > 150 bpf
16	5	12BP53	130.7	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 624	El. 624 - El. 589	El. 589 - El. 569	El. 567	697.53	566.83	Extremely hard driving between El. 588 & El. 579 - > 150 bpf
17	6	12BP53	131.8	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 608	El. 608 - El. 602	El. 602 - El. 568	El. 567	698.36	566.56	Extremely hard driving between El. 593 & El. 583 - > 150 bpf
18	7	12BP53	131.3	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 624	El. 624 - El. 603	El. 603 - El. 570	El. 569	699.88	568.58	
19	8	12BP53	117.1	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000 -26,000	Grade - El. 648	El. 648 - El. 616	El. 616 - El. 605	El. 581	698.58	581.48	Extremely hard driving beginning at El. 605 to refusal - > 150 bpf
20	9	12BP53	123	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	Grade - El. 664	El. 664 - El. 618	El. 618 - El. 591	El. 577	699.85	576.85	Extremely hard driving between El. 608 & El. 606 - > 150 bpf
21	10	12BP53	135.3	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 665	El. 665 - El. 631	El. 631 - El. 592	El. 567	701.84	566.54	Extremely hard driving beginning at El. 592 to refusal - > 150 bpf
22	11	12BP53	96.2	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 23,000	Grade - El. 669	El. 669 - El. 665	El. 665 - El. 611	El. 608	704.56	608.36	
23	12	12BP53	117	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000	Grade - El. 664	El. 664 - El. 655	El. 655 - El. 605	El. 587	704.29	587.29	Extremely hard driving beginning at El. 605 to refusal - > 150 bpf
24	16	12BP53	69.5	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000	Grade - El. 676	El. 676 - El. 673	El. 673 - El. 653	El. 636	705.48	635.98	Extremely hard driving beginning at El. 653 to refusal - > 150 bpf - No pile tip used.
25	17	12BP53	39	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000	Grade - El. 683	El. 683 - El. 679	El. 679 - El. 677	El. 667	706.48	667.48	Extremely hard driving between El. 677 & El. 671 - > 150 bpf - No pile tip used.
26	18	12BP53	138	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000	Grade - El. 681	El. 681 - El. 610	El. 610 - El. 569	El. 568	706.41	568.41	
27	19	12BP53	74	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	Grade - El. 702	El. 702 - El. 684	El. 684 - El. 679	El. 649	722.57	648.57	Extremely hard driving between El. 679 & El. 668 - > 150 bpf.
28	20	12BP53	159	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	Grade - El. 688	El. 688 - El. 685	El. 685 - El. 619	El. 567	725.92	566.92	Extremely hard driving within finite layers prior to bedrock.

Table 6. Production Pile Summary

Pier	Pile Type	Average Length (ft)	Pile Length Scatter	Contractor	Contract	Hammer	Stroke	Rated Energy (ft-lbs)	Cut-off Elevation	Average Tip Elevation
1	12BP53	139.9	9.40 / -7.37	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	698.11	559.13
2	12BP53	136.2	3.00 / -3.70	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	696.43	N/A
3	12BP53	138.1	2.79 / -3.03	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	693.58	556.58
4	12BP53	135.9	4.37 / -4.74	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	696.04	N/A
5	12BP53	139.0	0.60 / -6.43	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	698.57	557.04
6	12BP53	135.6	2.48 / -2.75	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	698.74	563.14
7	12BP53	129.5	2.68 / -1.27	Sheehy Bridge Const. Co.	C	Link-Belt Diesel 520 DAPH	4ft	26,000	696.13	566.63
8	14BP73	122.2	1.54 / -2.11	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	688	565.8
9	14BP73	99.9	2.32 / -2.22	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	668	568.1
10	14BP73	98.2	3.66 / -4.33	Industrial Construction Co.	B	McKiernan Terry S-8 SASH	3.25ft	26,000	668	569.8
11	14BP73	135.7	8.33 / -4.37	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	700.23	564.53
12	12BP53	134.1	2.87 / -2.94	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	698.85	564.75
13	12BP53	132.6	12.37 / -2.92	Sheehy Bridge Const. Co.	C	McKiernan Terry S-8 SASH	3.25ft	26,000	698.36	565.76
14	12BP53	112.4	26.89 / -21.36	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000 - 26,000	712.37	599.97
15	12BP53	143.0	51.70 / -31.54	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	712.5	569.5
16	12BP53	128.9	6.58 / -10.55	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	697.53	568.63
17	12BP53	131.5	3.92 / -2.31	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	698.36	566.86
18	12BP53	125.4	16.30 / -23.55	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	699.88	574.48
19	12BP53	120.5	40.88 / -31.19	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000 -26,000	698.58	578.08
20	12BP53	118.9	46.06 / -36.60	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	699.85	580.95
21	12BP53	118.5	18.72 / -31.48	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	24,000	701.84	583.34
22	12BP53	92.6	33.58 / -35.41	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000 - 24,000	704.56	611.96
23	12BP53	94.1	23.09 / -25.85	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000	704.29	610.19
24	12BP53	52.0	24.50 / -18.27	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	23,000	705.48	653.48
25	12BP53	30.9	5.41 / -5.44	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	N/A	706.48	675.58
26	12BP53	40.2	97.34 / -13.53	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000	706.41	666.21
27	12BP53	67.8	69.02 / -23.60	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	722.57	654.77
28	12BP53	119.6	68.40 / -61.57	Johnson Bros. Const. Co.	D	Link-Belt Diesel 520 DAPH	4ft	22,000 - 24,000	725.92	606.32

With the exception of the north end, starting at Pier 14, it is clear from the figures that all of the original H-piles were driven to the bedrock layer. South of Pier 14, typical pile driving operations encountered little driving resistance prior to striking the bedrock. The point at which the pile engaged the bedrock is marked by a sudden and dramatic increase in driving resistance as indicated by the figures. North of Pier 14, Figure 20 shows an extremely large scatter of pile lengths. Review of the test boring logs indicates an intermediate layer of brown sand with gravel, which appears dense enough to cease H-pile driving operations, even with the addition of pile tips. The Pier 28 test pile driving record provides a good illustration of the pile driving operation, as this test pile was driven through the intermediate layer to bedrock. Beginning at about elevation 620 and terminating at about elevation 580, extremely hard driving resistance (+500 bpf) was reported. Examination of Boring Log T-4 indicates the presence of the brown sand with gravel layer beginning at about elevation 624 and terminating at elevation 581.

The intermediate layer is first encountered on Boring Log T-1 near Pier 11 and is found in all the boring logs proceeding northward. One contradiction in the findings is obvious. Since the boring logs indicate the presence of the intermediate layer near Pier 11, it is unclear how the piles at Piers 12 and 13 could all be easily driven to bedrock. The phenomenon may be explained by two observations. First, the load bearing quality of the intermediate layer is not as great near Pier 11 as it is near Pier 14. Boring Log T-1 reports a driving resistance of 49 bpf for the soil boring through this layer; whereas Boring Log T-6 reports a driving resistance of 60 to 75 bpf at the same depth. Therefore, the intermediate layer becomes more dense toward the north end of the bridge and becomes more difficult to drive a pile through. Second, since the piers were constructed under three separate contracts, different equipment was used to install the piling by different contractors. The hammer used to drive piling for Piers 12 and 13 was larger than the hammer used for the remaining piles for Piers 14 to 28. Table 5 indicates that an 8000-pound McKiernan Terry S-8 hammer with a rated energy of 26,000 ft-lbs was used to drive the production piles at Piers 12 and 13; whereas a 5000-pound Link-Belt Diesel hammer with a rated energy of 22,000 ft-lbs – 26,000 ft-lbs was used to drive the production piles at Piers 14 to 28. Therefore, since the larger hammer was able to "hit" the piles harder, the piles at Piers 12 and 13 were more easily driven to bedrock than those located farther to the north.

The foundation recommendations for the 1981 and 1991 pier extensions stipulated the use of CIP piles with lower design loads and shorter pile lengths. The CIP piles could be safely driven to attain their required design capacity in the denser soil layer mentioned earlier. Figure 20 illustrates that the CIP piles were indeed driven to a much shorter length with much less scatter between the maximum and minimum length piles for each foundation.

The only pile load test information available dealt with the North Abutment test pile. The pile in question was a 12BP53 friction pile with a total length of 52.1 feet and a computed bearing resistance of 63.8 tons. The pile was driven by an 8000-pound McKiernan Terry S-8 hammer with a rated energy of 26,000 ft-lbs. The pile test was conducted by constructing a load frame apparatus around the exposed end of the pile such that varying loads could be applied and the resulting deflections measured via a dial gage over time as the load was transferred from the pile to the surrounding soil matrix. One load/unload cycle was completed over the course of two days. The load application procedure specified an initial load of 40 tons, with an increasing load in 10-ton increments applied every hour until 100 tons was applied to the pile. The 100-ton load was left on the pile for 24 hours and then removed in 10-ton increments. The resulting deflections were measured at 15-minute intervals throughout the course of the test. The results indicate a total pile deflection of 0.175 inch under the 100-ton load, with a total rebound of 0.167 inch upon full removal of the loading. The behavior is consistent with that of an end bearing pile where the bulk of the deflection under load is due to elastic shortening.

5.2.5. Conclusions

During the design progress meeting on March 7, 2006, it was agreed that, since the H-piles were driven to bedrock, the bearing capacity is not a geotechnical issue. As long as it can be ascertained that the piles were driven to bedrock and were not damaged during the driving operations, the structural capacity of the pile is the limiting aspect for the determination of pile resistance ($\phi * Q_n$). Figure 20 shows the pile lengths versus the approximate bedrock elevations taken from the soil borings.

In accordance with the LRFD specifications, the structural capacity of an H-pile is limited to $\phi * A_S * f_Y$, where $\phi = 0.50$. Thus, the capacity of a BP14x73 pile, on the basis of a yield strength of 36 ksi, is 193 tons. Correspondingly, the structural capacity of a BP12x53 pile is 140 tons.

For those piles not driven to bedrock, estimates of pile capacity were made based on the pile driving record. Table 7 illustrates the average calculated nominal capacity for the non-end bearing piles. The capacities result from the two dynamic formulae which were considered. First, the Mn/DOT LRFD formula for steel H-piling driven with a power driven hammer:

$$\phi P_n = \phi\left[\frac{10.5E}{S+0.2} * \frac{W+0.1M}{W+M}\right]$$

Where:

ϕ = Resistance factor = 0.4.
P_n = Nominal bearing capacity in pounds.
W = Mass of the striking part of the hammer in pounds.
H = Height of fall in feet.
S = Average penetration in inches per blow.
M = Total mass of the pile plus driving cap.
E = Energy per blow for each stroke of the hammer.

and second, the FHWA-modified Gates formula:

$$\phi P_n = \phi\left[1.75\sqrt{E}\log(10N)-100\right]$$

Where:

ϕ = Resistance factor = 0.4.
P_n = Nominal bearing capacity in pounds.
E = Energy per blow for each stroke of the hammer.
N = Number of hammer blows per inch of permanent set.

The resulting average calculated nominal capacity for non-end bearing piles at each pier is shown in Table 7.

Table 7. Average Calculated Nominal Capacity for Non-End Bearing Piles

Pier	Mn/DOT LRFD Formula ϕP_n (tons)	FHWA-Modified Gates ϕP_n (tons)
14	94	111
15	92	115
18	100	128
19	96	120
20	101	129
21	102	129
22	101	102
23	99	104
24	121	105
25	128	102
26	106	87
27	108	99
28	103	161

No reduction in pile dimensions due to corrosion is necessary for the river piers since the piles are completely submerged at all times. However, the approach span foundations do contain piles with portions that exist above the water table, and seasonal fluctuations of water table elevation may yield portions of the pile which could be susceptible to corrosion. For these piles, however, no reduction in pile nominal capacity due to corrosion was considered based on standard Mn/DOT practice.

For the structural evaluation of the foundations with non-end bearing piles, the Mn/DOT LRFD formula pile capacity values listed in Table 7 were used for the remainder of the report.

The scour analysis for Piers 9 and 10 shows that pier scour can extend well below the bottom of the tremie seal. Figure 21 shows a hydrograph of scour at the river cross section at the Lafayette Bridge. during the 1969 flood-over for a duration of approximately two weeks during rising water. Figure 22 shows a hydrograph of the same area for approximately two weeks during falling water. The temporary loss of the soil below the tremie seal would significantly decrease the design capacity of the piles, since the formula listed above ($\phi * A_S * f_Y$) is based on a continually braced pile. Therefore, it is extremely important to implement the scour protection measures listed in the 1988 memorandum by Kevin Bittner (see Section 5.2.1 for the full citation).

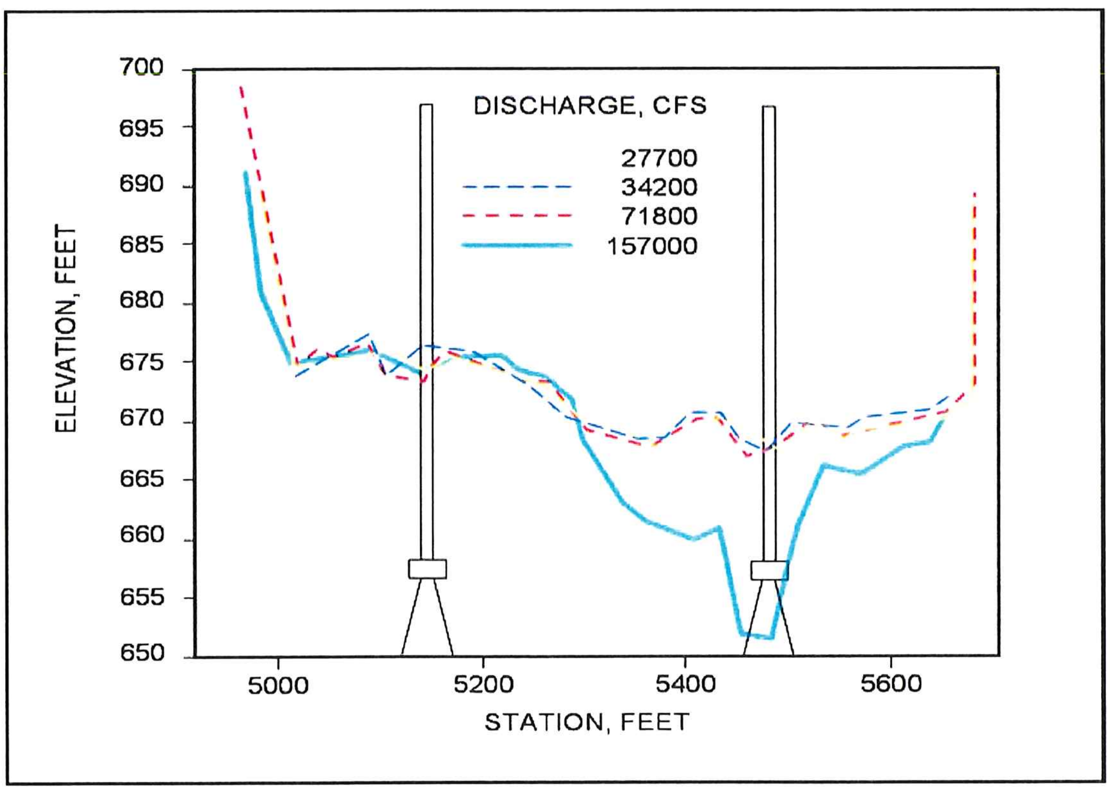


Figure 21. 1969 Flood Hydrograph - Water Elevation Rising

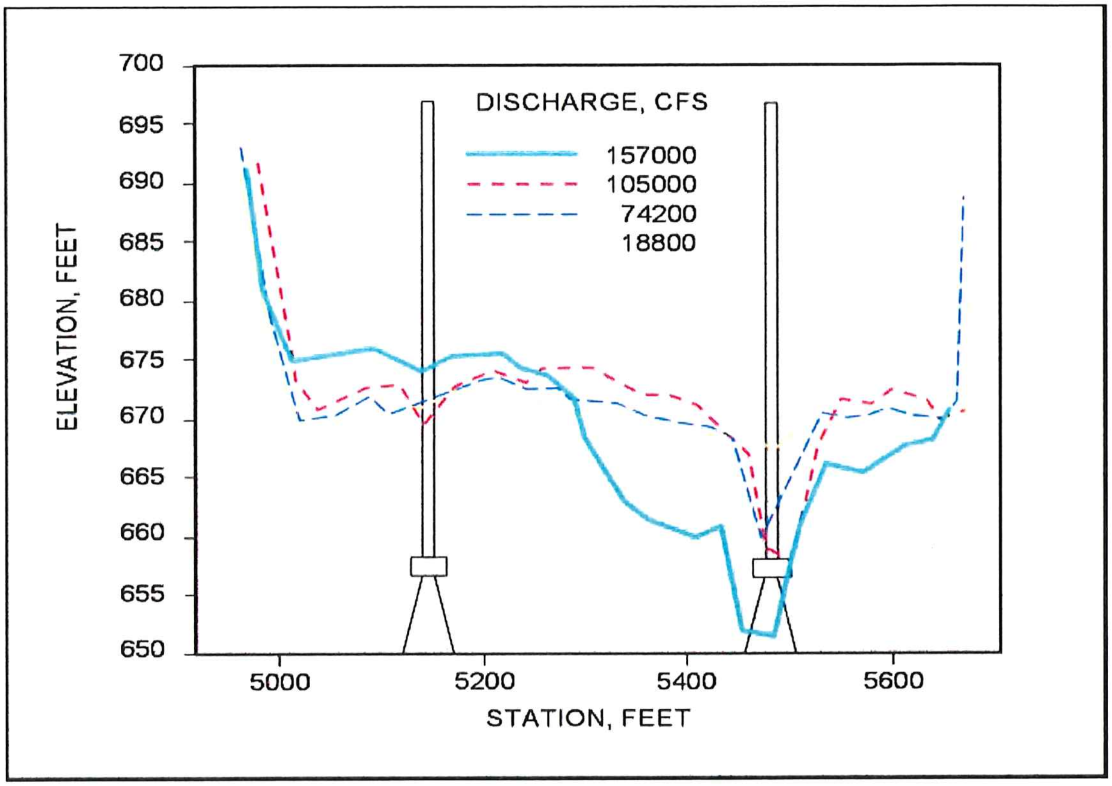


Figure 22. 1969 Flood Hydrograph - River Elevation Falling

Based on recommendations by Braun Intertec, the future foundation modifications may be more efficiently supported on CIP friction piles similar to those used for the 1981 and 1991 widenings than on end bearing piles as used for the original design. Nominal pile capacities of 100 tons may be expected for a 12-inch-diameter pile and 140 tons for a 16-inch-diameter pile. Since the river pier foundations are susceptible to scour, additional pile length may be added at these locations to ensure sufficient pile embedment to safely support the loadings under an extreme flooding event. The use of the shorter friction piles would have a number of beneficial effects. For instance, the shorter pile length requires less material, resulting in significant material savings over end bearing piles. Additionally, the shorter pile length allows for more favorable pile locations, such that interference with the existing battered piles would be avoided. This geometric limitation is especially important at the river piers where foundation modifications will require the construction of a cofferdam and the existing piles occupy a large restricted pile driving zone into which new piles may not penetrate, as shown in Figure 23. The use of shorter piles would also allow for new pile locations closer to the existing locations, which would minimize the eccentricity between the centroid of the pile group and the centroid of the applied loadings based on the specified alignments.

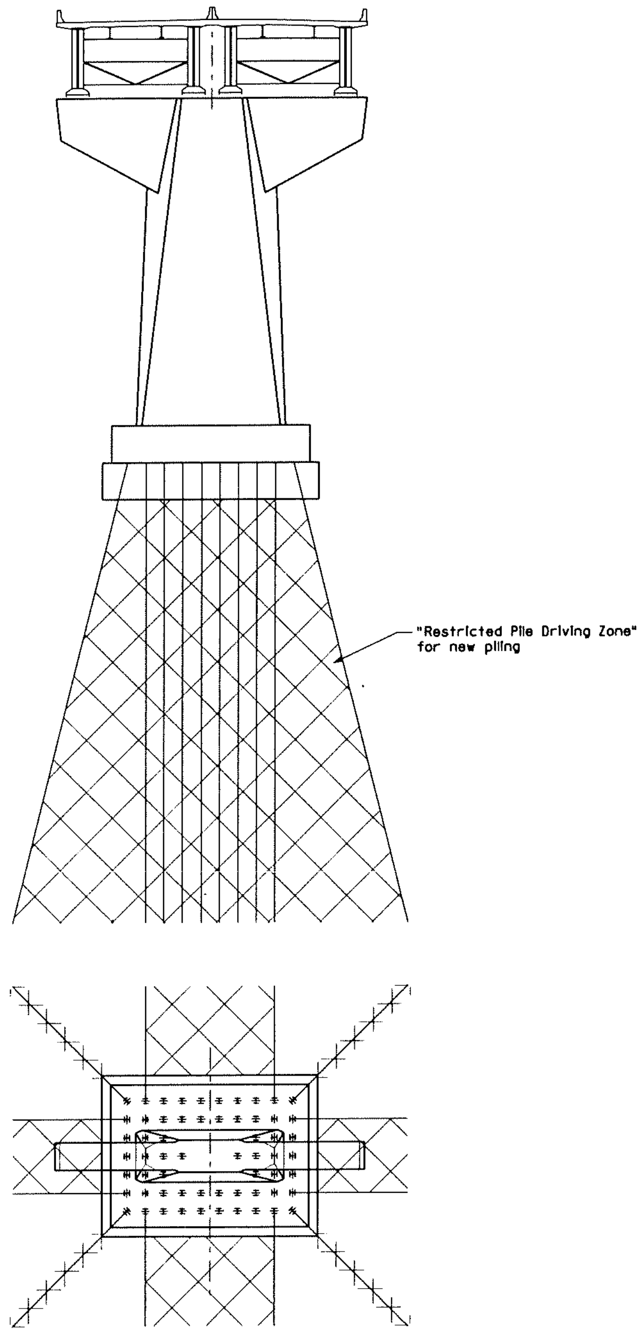


Figure 23. Existing Pile Interference Zone

5.3. LRFD Design Capacity of Existing Footings

5.3.1. General Remarks

For the discussion of the capacity of an existing footing, a few general geometric comparisons of the existing bridge versus the proposed bridge(s) will provide a useful context.

- The original cross section of the Lafayette Bridge provided two curb-to-curb roadway widths of 29 feet-0 inches. The typical total out-to-out deck width was 67 feet-4 inches.
- This deck geometry provided a total of four design traffic lanes (two in each direction).
- Subsequent barrier and median modifications increased the available shoulder widths but did not increase the number of design lanes.
- The new deck geometry for all options (with the exception of Option 2) produces ten design traffic lanes (five in each direction). The corresponding total out-to-out deck width of the bridge will be 126 feet-8 inches, plus the width of the gap between the two bridge decks.

The comparisons show that the overall deck width will increase by 88%, while the number of design traffic lanes will increase by 250%. This 2.5-fold increase in the number of design traffic lanes represents a drastic change in the applied loads to the footings. In the case of river piers, the width of the gap between the bridges introduces an additional eccentricity and an associated overturning moment. Regardless of the design code, such geometric increases produce dead loads and live loads that exceed the capacity of the existing footing(s). Additional foundation capacity will have to be added.

Dead Load: The increase in overall deck width also results in an increase in dead load. In addition to the deck geometry, the superstructure type and the weight of the pier itself will affect the dead load of a given footing.

Live Load: The magnitude of the live load on given pier footing is a function of the number of design traffic lanes, their multiple presence factors, the load type (HS-20 vs. HL-93), and the eccentricity of the design traffic lanes. The factors for simultaneously loaded design traffic lanes are different for the ASD code and the LRFD code, as follows:

Number of Lanes	ASD	LRFD
1 Lane	1.0	1.2
2 Lanes	1.0	1.0
3 Lanes	0.90	0.85
>3 Lanes	0.75	0.65

In the case of a single river pier footing, the comparison yields 3.0 design traffic lanes (4 times 0.75) for the existing ASD code and 6.5 design traffic lanes (10 times 0.65) for the LRFD code.

For a discussion of the effects of the ASD versus the LRFD code provisions on live load, refer to Appendix B.

5.3.2. Design Methodology for Footing Design

The existing footings were designed according to the allowable stress design method (ASD) and are reinforced with Grade 40 reinforcement bars. Very often the governing load case was a load case that allowed 25% or 40% overstress. A typical footing design per ASD would use the moment and shear design approach according to the beam analogy. This approach ignores the fact that in most cases the design region is a "disturbed" region and not a "Bernoulli" region. The beam analogy only applies to the latter regions. It also reduces a three-dimensional problem to a two-dimensional one. The compounding effects of a two-dimensional design model and the provisions of the ASD design code decrease the factor of safety. This decrease is exposed when a strut-and-tie model in conjunction with the LRFD code is applied.

A strut-and-tie model of the footing approximates the actual force flow through the footing much more closely. Under the LRFD code, the design loads and the strength of a structural component are determined in a much more transparent fashion.

The loads from the existing superstructure were applied to the existing footings. Design loads were factored per the LRFD code. The footings were analyzed as strut-and-tie models. The results showed that the existing footing reinforcement did not meet the requirements of the LRFD code. These results do not mean that the footings will fail; it merely implies that the probability of failure has increased and is outside the calibrated range of the code.

5.3.3. River Pier Footings

The term "river piers" has been applied to Piers 8 through 11 since they support the river spans of the Lafayette Bridge, even though only Piers 9 and 10 are actually located in the Mississippi River. However, since all four piers are hammerhead piers on a single footing, Piers 8 and 11 were also classified as river piers. The true river piers, however, required a cofferdam seal during construction. Piers 8 and 11 did not require a seal during their construction.

The existing footings were reinforced with uncoated Grade 40 reinforcement. The allowable tensile stress in the reinforcement was 20,000 pounds per square inch. Grade 60 reinforcement would have had an allowable tensile stress of 24,000 pounds per square inch. In comparison, the ratio of yield strength versus allowable tensile strength is 2.0 for Grade 40 reinforcement and 2.5 for Grade 60 reinforcement. These ratios show that, other things being equal, an ASD design with Grade 40 reinforcement has a smaller safety cushion than an ASD design with Grade 60 reinforcement.

The footings were designed for all required load combinations of the code applicable at that time. With the exception of load case I, all other load cases allowed overstresses ranging from 25% to 50%. Barge collision forces were not considered for the foundation design.

LRFD design checks for the existing footings on the basis of a strut-and-tie model show that the footing reinforcement is inadequate when piles are loaded to their LRFD design capacity. The most pronounced inadequacies occur at the footing corners. Since this is the region where the highest pile loads occur, this is also the most critical area. The compression struts transferring the loads to the individual piles produce tensile stresses in the footing reinforcement. In the case of corner piles, the tensile stresses need to be resolved with respect to the direction of both the longitudinal and the transverse reinforcement. Since the classical moment design method neglects the three-dimensional nature of the force flow, the provided transverse reinforcement is inadequate by at least a factor of two.

In addition to the inadequacies of the reinforcement at the footing corners, the main reinforcement provided for piles located along the footing perimeter does not meet LRFD design requirements.

5.3.4. River Pier Footing Modifications

The existing superstructure dead loads, most notably in the river spans, are low in comparison with a modern, redundant steel superstructure. Several factors contributed to the fairly light existing superstructure:

- The superstructure consists of a non-redundant, two-girder system with floor beams and stringers.
- The use of intermediate stiffeners allowed thinner web plates and resulted in a weight reduction.

- The original deck slab was only 7 1/2 inches thick and there was no weight allowance for a future wearing course.

All three new superstructure alternatives would be heavier than the original river span superstructure. The new steel superstructure alternatives would be heavier for a number of reasons. First, without intermediate stiffeners, the girder webs would have to be thicker. Second, the redundancy provided by multiple girders would result in more dead load. Third, for the steel box alternative, the weight of the bottom flanges and the weight of the variable-depth deck slab would add significant weight. Fourth, in the case of the multiple steel girder alternative, the minimum 9-inch-thick deck slab plus the future wearing course allowance would create a superstructure heavier than the original superstructure. A concrete box girder bridge would be significantly heavier than any comparable steel superstructure.

In addition to the extra dead load, the LRFD code load provisions require larger live load reactions at the piers. For the original ASD design, the maximum live load reaction per traffic lane was produced by either an HS-20 truck or a 640-pound per lineal foot lane load plus a 26-kip concentrated load. In contrast, the LRFD code specifies the concurrent application of 90% of both the 640-pound per lineal foot lane load plus the effects of two HS-20 trucks. For a comparison of the ASD and LRFD codes, see Appendix B.

Based on the increased loads mentioned above, the need to provide additional pile capacity is evident. When the effects of barge collision forces are considered, the need for extra piles increases even more. The feasibility of adding piles depends greatly on the geometric constraints and on the superstructure alternative. A key factor is the need to unload the existing piles before they can be reloaded. Since the existing hammerhead piers contribute significant dead load to the piles, the complete removal of the existing river piers would be advantageous. See Sections 8 and 10 for further discussion of this topic.

5.3.5. Approach Span Pier Footings

For the approach spans, each pier column is supported by a rectangular footing. The footing size and the number of piles vary from pier to pier. The number of piles per footing ranges from six to nine.

LRFD design checks for the existing footings on the basis of a strut-and-tie model show problems similar to those for the river piers. The reinforcement provided for the corner piles is inadequate.

5.3.6. Approach Pier Footing Modifications

Each approach pier typically has two separate, symmetrically arranged footings. Regardless of the option, piles will need to be added. These can be added as part of an underpinned footing, as part of a footing extension, or as part of a newly constructed footing.

Construction of an underpinned footing would be labor intensive and would not reliably address the inadequate capacity of the existing reinforcement. Furthermore, the construction method required for an underpinned footing would be very difficult and costly.

A simple footing extension which would splice on to the existing footing reinforcement will not work because it is impossible to add reinforcement bars in the most critical areas of the existing footing.

The existing footings, like all the other original reinforced concrete components, are reinforced with Grade 40 reinforcement bars, which afford a lower factor of safety when interpreted by the LRFD design method, resulting in a decreased additional capacity. Therefore, the existing pier footings should be removed. After the removal of the existing footings, additional piles could be driven as required. All new footings could be constructed in locations that are based on an efficient pier frame layout. They would incorporate newly-driven and existing piles as needed. They also would be reinforced with Grade 60 reinforcement.

5.3.7. Conclusions

Arriving at a workable modification scheme for foundations that meet LRFD design requirements and can be constructed within the limitations imposed by staged construction was one of the key challenges of this study. Only two of the four options (Options 1b and 3) are feasible, and each has its challenges. The greater design loads (larger dead load from the superstructure, an increased number of traffic lanes, larger live loads per LRFD) and, in the case of the piers in the river, the necessity to account for barge collision loads all result in the need to strengthen the footings.

5.4. LRFD Design Capacity of Existing Pier Shafts and Columns

5.4.1. General Remarks

The general remarks regarding deck geometry and design traffic lanes made in Section 5.3.1 also apply to pier shafts and columns.

5.4.2. Pier Columns

The pier columns, like all the other reinforced concrete components of the original bridge, are reinforced with Grade 40 reinforcement bars. Typically, the provided reinforcement is adequate to meet LRFD design requirements. The columns of the skewed piers attract design forces in excess of the column capacity. However, the fate of the existing columns is not decided by their own structural capacity, but by two other factors.

The first factor is the structural adequacy of the footing supporting the column. As the discussion of the previous section showed, the existing footings are often structurally inadequate. The second factor is the usefulness of column location, such that a new pier will have a meaningful, structurally-efficient geometry.

5.4.3. Pier Shafts

In the case of the river piers, the existing piers contribute significant dead loads to their foundations. Regardless of the alignment option, the existing pier shaft would interfere with the installation of additional new piles. The removal of the pier shaft is necessary for the removal of the pier footing underneath. The construction of a completely new pier would afford the opportunity to choose the pier form most suitable for the superstructure alternative under consideration. For piers in the river, the ability to withstand barge collision forces and to minimize pier contraction scour are additional important considerations.

5.4.4. Conclusions

The structural adequacy of the piers columns and shafts is only one aspect of the usefulness of the columns in the future pier frame. Equally important is the location of a given column.

6. EVALUATION OF EXISTING STEEL APPROACH SPANS

The Lafayette Bridge's twenty-nine spans are comprised of twenty-six approach spans constructed from either 46-inch welded plate girders or 36-inch rolled sections, and three main river spans which are non-redundant variable-depth plate girders. Given the non-redundancy of the main river spans and their historic fatigue issues[1], they will be replaced as part of the bridge widening project. The approach spans, however, possess the desirable redundancy and have not experienced significant fatigue cracking problems[2]. The question therefore arises, can the approach spans be reused as part of the rehabilitation?

The viability of reuse of the approach spans depends on the resolution of a number of factors, including the remaining fatigue life of the approach spans, incompatible geometry between the existing and proposed structure, ongoing maintenance issues and costs, reuse/rehabilitation of the existing hinge and expansion joints, and transition from the approach spans to the main river spans.

6.1. Fatigue Evaluation

The fatigue evaluation for this study was conducted according to the guidelines presented in Section 7, Fatigue Evaluation of Existing Steel Bridges, in the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*. The manual defines "load-induced" and "distortion-induced" as the two types of fatigue which must be considered. Load-induced fatigue is defined as fatigue damage that is "due to the in-plane stresses in the steel plates that comprise bridge member cross sections. These in-plane stresses are those typically calculated by designers during bridge design or evaluation." Distortion-induced fatigue is defined as fatigue damage that is "due to secondary stresses in the steel plates that comprise bridge member cross sections....These secondary stresses are minimized through proper detailing."[3]

The demands on older bridges in terms of traffic volumes and vehicle weights are increasing every year. As a result, fatigue damage has become a service issue for in-service steel bridges[4]. Given the development of design codes and materials, steel bridges from different eras will be susceptible to different types of fatigue damage. For instance, bridges constructed prior to the advent of modern fatigue design requirements, pre-mid-1970s, are prone to both load-induced and distortion-induced fatigue. Bridges constructed between the mid 1970s and 1985 are basically immune to load-induced fatigue damage; however, distortion-induced fatigue is possible. Bridges built post-1985 should not be

[1] J.W. Fisher, *Fatigue and Fracture in Steel Bridges*, John Wiley and Sons, New York, NY, 1984, 336 pp.
[2] Minnesota Department of Transportation, Metro District Maintenance Operations, Bridge Inspection, *Fracture Critical Bridge Inspection In-Depth Report, Bridge #9800 (Lafayette Bridge) TH 52 over the Mississippi River in St. Paul, Minnesota*, Oakdale, MN, 2004.
[3] American Association of State Highway and Transportation Officials, *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, First Edition*, Washington, D.C., 2003.
[4] J.W. Fisher, *Fatigue and Fracture in Steel Bridges*

susceptible to fatigue damage of any kind if properly constructed[5]. Given the era in which the Lafayette Bridge was constructed, both load-induced and distortion-induced fatigue must be considered.

6.1.1. Fatigue Categories

All the components of a steel bridge, from base metal to bolts or welds, may be classified into one of eight fatigue categories labeled A through E'. The fatigue categories group various details and components which are known to represent a known fatigue resistance to repetitive load of a given magnitude based on experimental tests.

For example, a bridge component with a very high fatigue resistance, such as base metal in a rolled beam, would be considered a Category A detail. Details which show very poor fatigue resistance, such as welded cover plates, or those which are considered undesirable and are discouraged from use, such as intermittent fillet welds, are classified as either E or E'. The boundaries of these categories are defined by the Constant Amplitude Fatigue Threshold (CAFT). The CAFT represents the maximum live load and impact stress range a particular detail can resist without suffering any fatigue damage. In other words, as long as the live load and impact stress range stays below the CAFT, the detail will have an infinite fatigue life. The fatigue detail categories and corresponding CAFT are shown in Table 8.

Table 8. Fatigue Detail Categories and CAFT

Detail Category	CAFT (ksi)
A	24.0
B	16.0
B'	12.0
C	10.0
C'	12.0
D	7.0
E	4.5
E'	2.6

The live loading used for a fatigue evaluation is an HS-15 truck with a 15% dynamic load allowance, and the stress range is the difference between the maximum and minimum stress produced by one lane loading of this vehicle. The minimum stress may be compressive; however, if the maximum tensile stress produced by the fatigue live loading is less than twice the compressive stress due to dead loads, the

[5] R.J. Connor, R. Dexter, and Hassam Mahmoud, *NCHRP Synthesis 354: Inspection and Management of Bridge with Fracture-Critical Details*, Transportation Research Board, National Research Council, Washington, D.C., 2005, 76 pp.

detail need not be checked for fatigue. Since bridges must carry a live loading that is completely random in terms of both frequency and amplitude, the fatigue truck represents 50% of the maximum fatigue loading for evaluation. In other words, the stress ranges produced from an analysis whose live loading is based on the fatigue truck must be doubled. Alternatively, the analysis results may be compared to one-half the CAFT values reported in Table 8. The latter comparison is the method used for this evaluation.

6.1.2. Identification of Fatigue-Prone Details

Prior to the analysis, careful review of the existing record drawings and corresponding shop drawings highlighted a number of potential fatigue-prone details. These details included welded cover plates, longitudinally welded gusset plates, and transverse stiffeners which were not connected to both flanges.

The first potential fatigue-prone detail exists at the ends of the welded cover plates on the rolled beams in Spans 24-29, which were classified Category E or E', depending on the flange thickness of the rolled beam. The cover plates are variable in width and thickness depending on the beam size and location; however, all the cover plates are attached by a 5/16-inch fillet weld which travels all the way around the plate. The cover plates are located in both the positive and negative moment portions of the span. For the cover plates located over the supports at Piers 24 to 27, only the top flange cover plate has the potential for fatigue issues.

The second potential fatigue-prone detail is the welded gusset plates connecting the wind bracing in Span 15 and the drainage system downspouts to the fascia beams throughout the bridge length. The geometry of the welded attachments at both locations were classified as Category E. The gusset plates connecting the drainage system downspouts to the fascia beam web are all located within a few feet of a pier. Given the presence of shear connectors at these locations, the steel beam and concrete deck are assumed to act compositely, placing the neutral axis near the steel beam's top flange. The geometry indicates that only compressive stress ranges would be created by live loads at these locations, rendering this detail immune to fatigue problems.

The third potential fatigue-prone detail deals with out-of-plane distortions at the diaphragm connection plate/girder connection. Currently, the diaphragm connection plate is not attached to the tension flange in both the positive and negative moment regions, leaving a small "web gap" at the location where the connection plate corner is clipped to provide clearance around the longitudinal flange-web weld. The current geometry is known to be problematic and prone to fatigue cracking due to the out-of-plane rotations. Evidence of this type of cracking is currently apparent in at least one location.

The remaining details on the bridge were all classified as Category C or better.

6.1.3. Residual Fatigue Life Levels

Should the results of the fatigue evaluation yield live load and stress impact ranges which exceed the CAFT for a particular detail, the remaining fatigue life may be estimated. Based on the variability inherent in experimentally derived fatigue lives, the *AASHTO Manual for Condition Evaluation and Load Resistance Factor Rating (LRFR) of Highway Bridges* describes three levels of residual fatigue life: minimum, evaluation, and mean. The minimum level corresponds to the standard design level, or to a stress range limit two standard deviations below the mean level based on experimental tests. For residual fatigue life calculations, the minimum level is typically considered too conservative. The evaluation level is slightly less conservative, and the mean level corresponds to the most likely fatigue life.

6.1.4. Programs Used for the Fatigue Evaluation

The numerical analysis conducted for the fatigue evaluation of the existing steel approach spans was done using the commercially available steel design and analysis software program MDX. MDX is a Windows-based design program which is capable of producing both grillage and finite element models for line-girder and system analysis of steel bridge structures. The program is capable of producing full designs for new bridges and performing rating calculations of new or existing bridges. Analysis and design of rolled beams with and without cover plates, welded and bolted plate girders, and box girder sections are all possible with MDX.

MDX is a powerful tool; however, it must be used carefully and the results must be closely checked. As with any numerical solution, the results are an idealization of the actual behavior and are subject to interpretation. One method to verify numerical output is to run an independent analysis with another program. For this project, the MDX live load calculations were verified using STAAD. A simple continuous beam element model was created in STAAD and a moving live axle loading representing the fatigue truck was moved incrementally across the model such that the maximum moments generated at various locations could be collected. The moments were then proportioned to the individual girders using the distribution factors developed in Section 4 of the AASHTO code.

Figure 24 and Table 9 show a comparison of the results between the two programs. Table 9 indicates the maximum support reactions calculated at the interior supports of each model and the percent difference between the two values. Figure 24 is a comparison of the maximum calculated positive moment due to the fatigue truck live load with dynamic load allowance in Span 14.

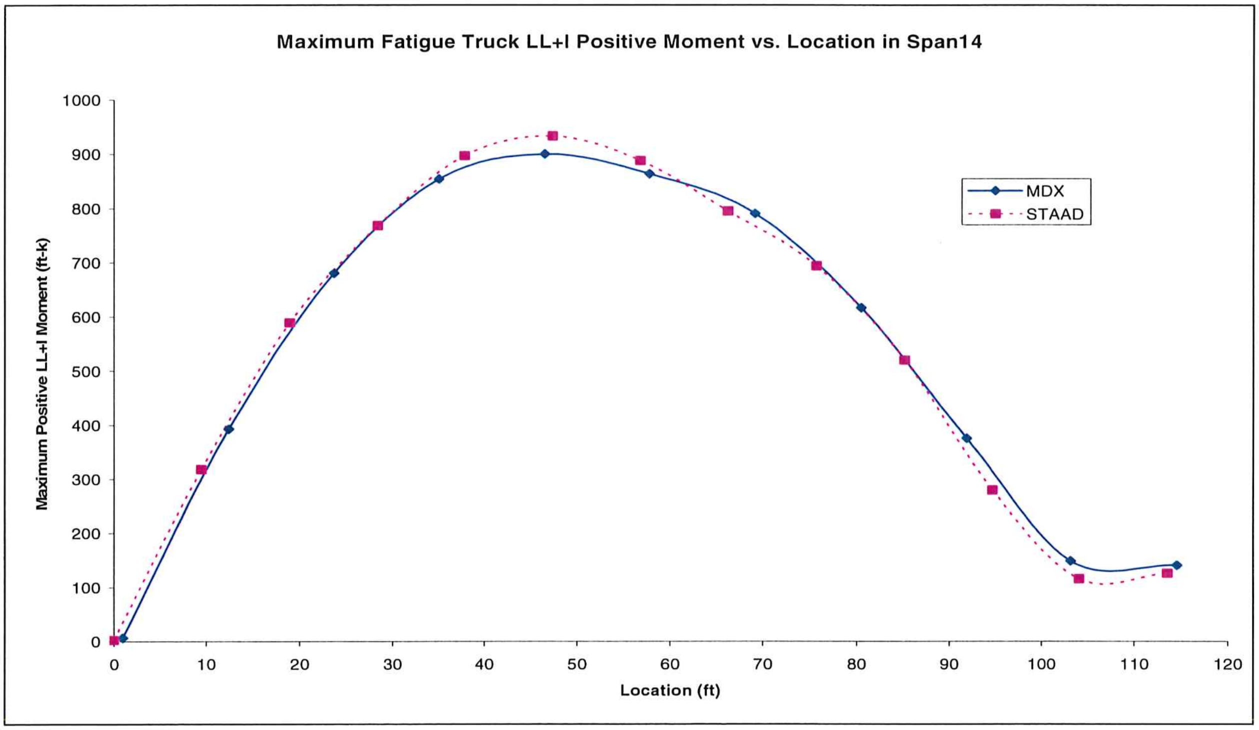


Figure 24. Model Comparison of Span 14 Maximum Positive Moment Versus Location

Table 9. Maximum Support Reaction Model Comparison

Location	Support Reaction - MDX (k)	Support Reaction - STAAD (k)	% Difference
Pier 13	49.45	48.40	2.12
Pier 14	52.35	52.14	0.40
Pier 15	52.25	52.08	0.33
Pier 16	48.23	47.59	1.33
Pier 17	52.01	51.39	1.18
Pier 18	51.25	51.22	0.05
Pier 19	50.67	50.40	0.53
Pier 20	49.68	47.90	3.58
Pier 21	52.30	52.30	0.00
Pier 22	52.46	52.19	0.51
Pier 23	48.89	47.55	2.74

Figure 24 and Table 9 illustrate the satisfactory agreement between the two results, lending credibility to the MDX analysis.

Unfortunately, MDX is not without limitations, and the user is forced to operate within the framework of the software's constraints. For instance, the maximum number of spans which may be analyzed at one time is twenty. Since the current bridge consists of twenty-nine spans, the entire bridge may not be analyzed at one time. Additionally, for a line girder analysis, MDX requires that the end support conditions be identical. The user is given the option of two choices: either fixed-fixed or pinned-pinned. If alternate support conditions are required, a system model is required.

6.1.5. Fatigue Evaluation Modeling

Prior to performing an analysis to determine how the existing steel superstructure may perform should it be incorporated into a new structure, an evaluation of the structure's past must be conducted to determine if any of the fatigue resistance has been consumed at any point. Therefore, an analysis of the existing structure, incorporating any modifications which have been made through the years, must be conducted. Since MDX cannot analyze all twenty-nine spans with one analysis, the bridge was broken into three distinct sections. Section 1 begins at the South Abutment and ends at Pier 8, Section 2 begins at Pier 11 and terminates at Pier 24, and Section 3 begins at Pier 22 and terminates at the North Abutment. Section 3 comprises the curved sections of the bridge and was therefore not analyzed with Section 2 to limit the model to a manageable size.

Once the structure's fatigue history is known, the evaluation may proceed to determine how the existing superstructure may be incorporated into the new bridge. For this analysis, the structure was again broken into the three distinct modeling regions used for the historical evaluation. Additionally, the approach span transition type and the proposed cross section must be incorporated into the analysis.

Three alternate transition types between the main river spans and the approach spans are to be considered as part of the rehabilitation. Since the selection of approach span transition type will have an influence on the stress ranges in the remainder of the approach spans, the three transition types must be considered for the fatigue analysis. The three transition types are numbered Type 1, Type 2, and Type 3. Type 1 is an extension of the existing approach span from an existing field splice such that the approach span terminates at the river pier with a simple support; Type 2 is a hinge joint with transfer girder transition similar to the existing transition; and Type 3 is a transition at a bolted splice in the existing approach span rendering the approach span fully continuous with main river span.-Both Type 2 and Type 3 transitions provide live load continuity between the approach spans and the river spans. For the Type 2 transition, an alternative cantilever dimension from the river span may be considered. However, in order to minimize the impact on the existing approach span girders, the existing 40'-0" cantilever was maintained.

For the re-use evaluation, three river span structure types were considered: multiple steel plate girder, variable depth concrete box girder, and variable depth steel box girder. Each river span type has different section properties and will have a different impact on the fatigue behavior of the existing approach spans for the transition types providing continuity. Since the exact dimensions and section properties of the river spans are beyond the scope of this study, an approximate method is required to assess the approach spans without knowing the exact nature of the river span. The approximate procedure involved fixing the end support of the model at the river pier for the Section 1 and Section 2 analyses. Fictitious girder sizes were added in the new river or approach span voids created by the specific transition type. The ramifications of the over-restraint provided by the fully fixed support were to overestimate the negative moment at the end support and underestimate the positive moment near the middle of the transition span. Therefore, the analysis of the calculated stress range was not considered valid in the span adjacent to the river span.

The approximate procedure required the use of a system model, as dissimilar end support conditions were required for analysis of Section 1. For continuity, the system model was used for the remainder of the evaluation.

The approach/river span transition types are illustrated in Figure 25. For each transition type, the end support conditions, areas of new, fictitious members, and valid results locations are indicated.

The end support methodology of modeling the continuity with a fixed support and ignoring the results in the end span was also used at the end support of the Spans 13–24 model, away from the river span, and at the interior support of the Spans 23–29 model.

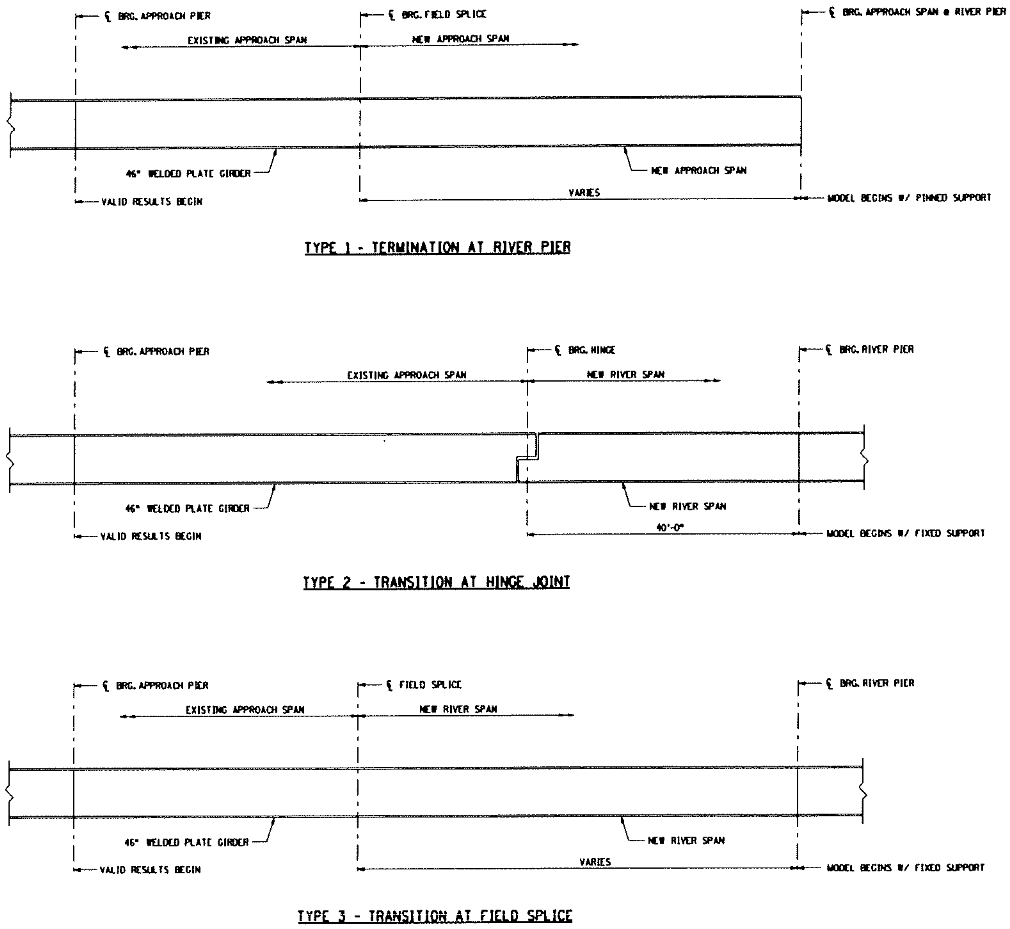


Figure 25. Schematic of Approach/River Span Transition Types

An additional consideration for the numerical model construction was the applicable cross section. Since the proposed cross section will require significant widening to meet the current geometric requirements, the beam spaces and girder locations may be very different in the finished product from what currently exist on the bridge. However, since the current geometry represents the "as designed" condition, the existing cross section was felt to be an adequate representation of the maximum live load stress range the girders may see if the existing approaches are salvaged. Therefore, the model was constructed based on the existing bridge cross section.

Using the existing cross section to define the limits of the deck, the proportion of live load each beam line is required to carry may be calculated. According to the *LRFD Specification*, the live load may be placed as close as 2'-0" from the gutterline. Historically, however, the vast majority of the vehicle loading has taken place from vehicles contained within the lanes as striped on the bridge. Since the exact value of the maximum live load stress range is highly dependent on the live load distribution factor calculated for each girder, the application of the fatigue truck loading was considered confined within the limits of the lane striping on the current bridge deck for the historical evaluation of the existing approach spans. However, since the final location of the girders within the proposed cross section is unknown, the outer limits of the entire bridge deck were used to define the live load distribution factors for the re-use evaluation.

Additionally, for the analyses of Section 1 and Section 2, the additional stiffness provided by the barrier sections was incorporated into the analysis, both in terms of section properties to resist applied loadings and in terms of relative stiffness to attract live load. For the analysis of Section 3, only the additional stiffness provided by the fascia girder in terms of section properties was included in the analysis, since the live load distribution factors were calculated according to the procedures of the *LRFD Specification.*

It should be noted that the most accurate representation of the stress ranges developed at load-induced fatigue prone details can only come from actual field-measured data. Due to alternative load paths and inherent approximations, numerical studies tend to overestimate actual measured stress ranges by as much as a factor of 2.

6.1.6. Analysis Results

For this evaluation, load-induced fatigue was checked with a numerical analysis to determine the live load stress range at each fatigue detail located on the bridge. The stress range was compared to known threshold values for the detail category for infinite fatigue life. If the calculated stress range was less than the threshold value, the detail is said to have infinite fatigue life. If the calculated stress range was greater than the threshold value, the remaining fatigue life is finite and an estimation of remaining life was calculated. If several details within the span were greater than the CAFT, the residual fatigue life was based on the largest calculated stress range, as it would produce the shortest residual fatigue life. Distortion-induced fatigue was checked by carefully reviewing the design and shop drawings for details which are known to be susceptible to distortion-induced fatigue.

Examination of the original construction documents reveals that the bridge has had two deck configurations throughout its lifetime, which define two distinct eras in the structure's fatigue history. Originally, the bridge was constructed with a 6.5-inch-thick concrete deck and continuous curb type barrier sections defining the limits of the bridge deck. The bridge operated in service for approximately twelve years with this deck configuration. In 1980, the deck geometry was altered with the placement of a 2.5-inch overlay, yielding a 9-inch total deck thickness, and the replacement of the curb sections with a type J barrier. Since the deck thickness and barrier type will influence the distribution of live load to the individual girders, for the historical evaluation, both of the two distinct eras of the bridge's past were evaluated.

The results for the historical fatigue evaluation are presented in Table 10. For a given era, if the stress range at a particular detail was less than the CAFT, it was determined that no fatigue resistance had been consumed and no information is presented in Table 10. If the calculated stress range was greater than the CAFT, the remaining fatigue life was calculated based on an estimation of the number of fatigue cycles during the particular era. Results are presented as a present day compilation of both historical fatigue eras of the bridge. In other words, the results indicate the current estimated remaining fatigue life of the existing structure.

Table 10. Results of Historical Fatigue Evaluation

		Estimated Residual Fatigue Life (Years)		
Section	**Span Range**	Minimum	Evaluation	Mean
1	1-8	-----	-----	-----
2	12-24	-----	-----	-----
3	24-29	-20.71	-9.73	6.74

The analysis results for the incorporation of the existing steel into the new bridge are presented in Tables 11, 12, and 13 (see below). For each transition type considered, the live load stress range at each fatigue detail was calculated. If the stress range at a particular detail was less than the CAFT, it is said to have an infinite fatigue life. If the calculated stress range was greater than the CAFT, the residual fatigue life was calculated. The residual fatigue life shown in Tables 11, 12, and 13 is based on the combination of the results from the analysis of the re-use evaluation and the results from the historical fatigue evaluation.

For the re-use evaluation, it should be noted that at several locations the critical fatigue detail occurs at the end of the diaphragm connection plate to web fillet weld. MDX assumes the diaphragm connection plate to be full depth between the flanges. However, the existing connection plates were cut 1 inch short of the tension flange in both the positive and negative moment regions. As a result, the MDX-calculated stress range is approximately 2% larger than it should be. Based on the section properties and calculated moment range at these locations, the correct stress range was manually calculated.

Section 1, Spans 1–8: Spans 1–8 are comprised of 46-inch-deep welded plate girders. The model limits are shown as the highlighted regions in Figure 26. The shaded portion represents the spans considered for the model. The lightly shaded spans are those for which the results are considered valid, and the darkly shaded span represents the end span for which the results are not valid.

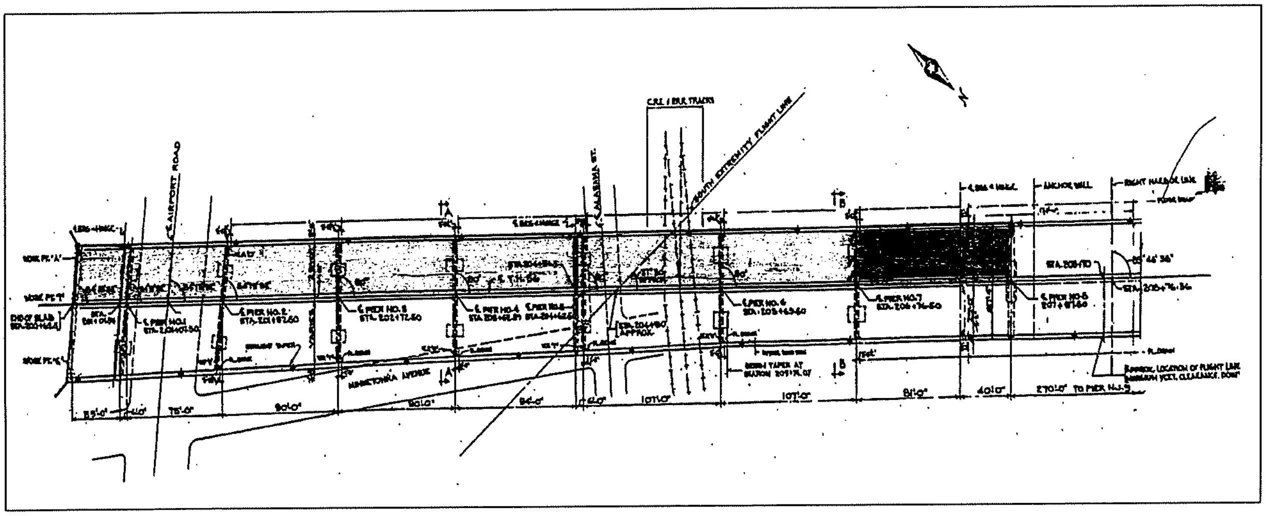


Figure 26. Limits of Section 1 MDX Model, Spans 1–8

The analysis results are presented in Table 11. Calculated stress ranges are greater than the CAFT for details in both the interior and exterior fascia beams, with the critical element occurring within the interior fascia beam. The calculated stress ranges are less than the CAFT for all details within the interior beams. For transition Type 1, the critical element is the base metal at the shear connector fillet weld connection located at the centerline of Pier 7, 611.58 feet from the centerline bearing of the south abutment. The detail is classified as Category C and has a CAFT of 5.00ksi. The calculated stress range at this detail is 6.62ksi. The critical element for transition Types 2 and 3 occurs at the end of the diaphragm connection plate to web fillet weld located 547.38 feet from the centerline bearing of the South Abutment. The detail is classified as Category C' and has a CAFT of 6.00ksi. The calculated stress ranges at these details are 6.41ksi for Type 2 and 6.41ksi for Type 3.

Table 11. Spans 1–8 Estimated Residual Fatigue Life

Transition	Critical Element	Estimated Residual Fatigue Life (Years)		
		Minimum	Evaluation	Mean
Type 1	Base Metal	17.00	20.40	18.95
Type 2	Connection Plate	18.75	22.50	20.90
Type 3	Connection Plate	18.75	22.50	20.90

Section 2, Spans 12–24: Spans 12–24 are constructed of 46-inch-deep welded plate girders and 36-inch-deep rolled beam sections with cover plates. The transition between the two different beam types occurs at the hinged expansion joint in Span 24. The model limits are shown as the highlighted regions in Figure 27. The shaded portion represents the spans considered for the model. The analysis results are considered valid for the 46-inch-deep welded plate girders contained within the lightly shaded portion, only.

The analysis results are presented in Table 12. Calculated stress ranges are greater than the CAFT for details in both the interior and exterior fascia beams, with the critical element occurring in the interior fascia beam.

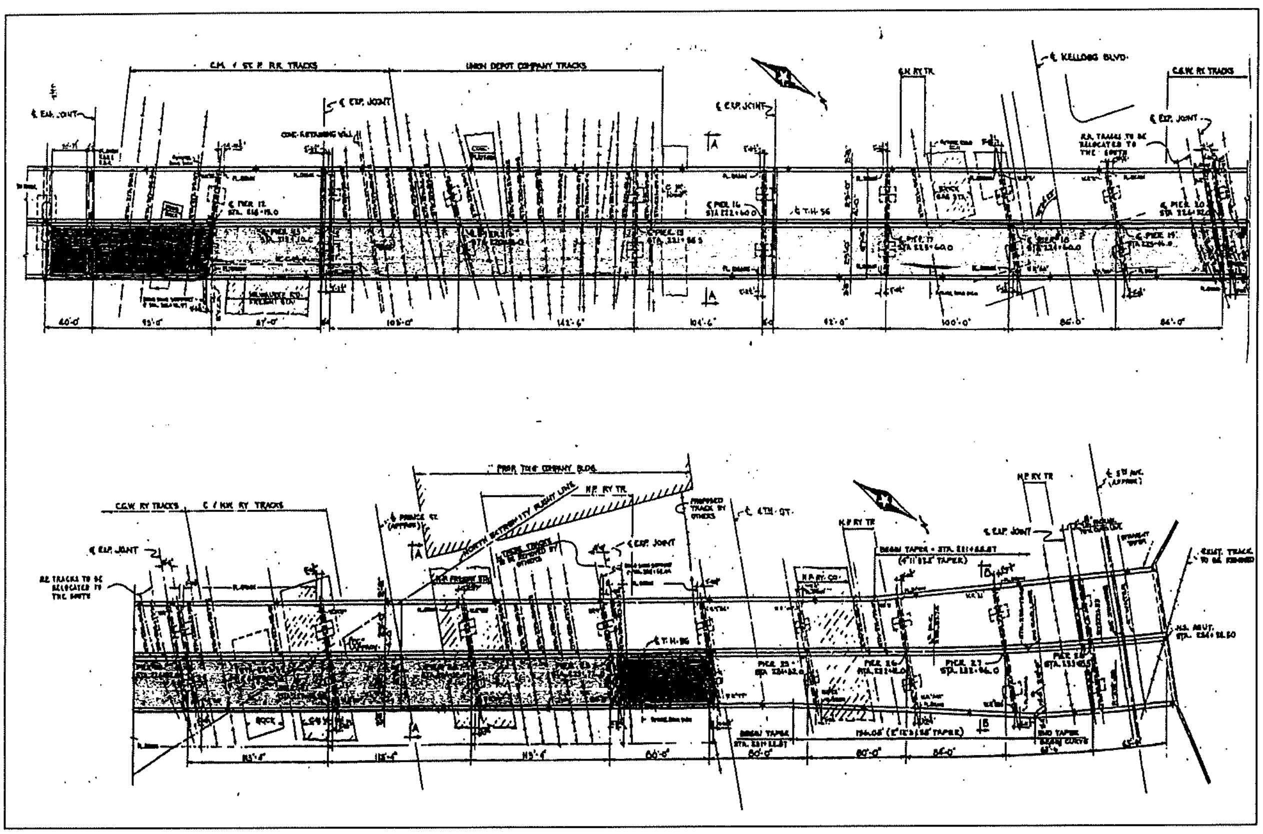


Figure 27. Limits of Section 2 MDX Model – Spans 12–24

The calculated stress ranges are less than the CAFT for all details within the interior beams. For transition Type 1, the critical element occurs at the end of the diaphragm connection plate to web fillet weld located 176.17 feet from the centerline of Pier 11. The detail is classified as Category C' and has a CAFT of 6.00ksi. The calculated stress range at this detail is 7.01ksi. The critical element for transition Types 2 and 3 occurs at the end of the diaphragm connection plate to web fillet weld located 560.83 feet from the centerline of Pier 11. The detail is classified as Category C' and has a CAFT of 6.00ksi. The calculated stress range at this detail is 6.85ksi for both Type 2 and Type 3.

Table 12. Spans 12-24 Estimated Residual Fatigue Life

Transition	**Critical Element**	**Estimated Residual Fatigue Life (Years)**		
		Minimum	Evaluation	Mean
Type 1	Connection Plate	14.36	17.23	16.01
Type 2	Connection Plate	15.36	18.43	17.12
Type 3	Connection Plate	15.36	18.43	17.12

Section 3, Spans 23–29: Spans 23–29 are constructed of 46-inch-deep welded plate girders and 36-inch-deep rolled beam sections with cover plates. The transition between the two different beam types occurs at the hinged expansion joint in Span 24. The model limits are shown as the highlighted regions in Figure 28. The shaded portion represents the spans considered for the model. The analysis results are considered valid for the 36-inch-deep rolled beam sections with cover plates contained within the lightly shaded portion, only.

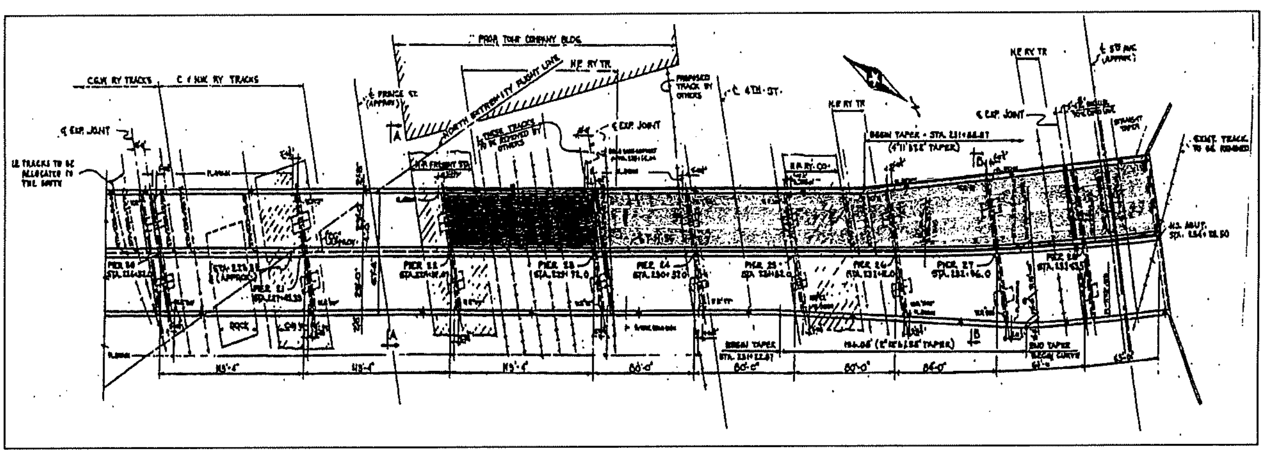


Figure 28. Limits of Section 3 MDX Model, Spans 23–29

The analysis results are presented in Table 13. Calculated stress ranges are greater than the CAFT for details within all beams in the cross section, with the critical element occurring in the interior fascia beam. For Spans 23–29, the critical element occurs at the weld at the end of the bottom flange cover plate located 155.33 feet from the centerline of Pier 22. The detail is classified as Category E' and has a CAFT of 1.30ksi. The calculated stress range at this detail is 4.74ksi. Since the minimum and evaluation levels have already been eclipsed from the historical evaluation, only the remaining mean level is calculated.

Table 13. Spans 23–29 Estimated Residual Fatigue Life

Transition	**Critical Element**	**Estimated Residual Fatigue Life (Years)**		
		Minimum	Evaluation	Mean
Type 3	Cover Plates			1.67

6.1.7. Remedial Measures

Load-Induced Fatigue

The results indicate that a number of common details are prone to load-induced fatigue. Among these are the fillet welded connections between the transverse stiffeners and diaphragm connection plates to the beam web, the bottom flange cover plates on the rolled beam sections, the base metal at the shear connector welds, and the welded lateral gusset plates required for the wind bracing in Span 15 and at the drainage system downspout braces throughout the bridge. Several methods of increasing the fatigue performance of welded steel bridge details are available, including grinding, gas-tungsten arc (GTA) re-melting, and peening.

Grinding of the weld increases fatigue resistance by physically changing the geometry of the weld toe to decrease the local stress concentration. Grinding of the weld toe is not an effective method of increasing fatigue life. Experimental test results indicate considerable scatter and inconsistency[6]. If used, considerable care must be taken to achieve beneficial results, which may prove difficult for a field application. Additionally, weld toe grinding is only effective on surface defects. Grinding will have no beneficial impact on subsurface defects. Therefore, grinding of the weld toe is not recommended.

GTA re-melting is an effective method of increasing the fatigue resistance of fillet welded details. GTA has been successfully used as a retrofit method on both cracked and un-cracked cover plate end weld details, with service life extensions in excess of twenty years[7]. If done properly, GTA can effectively remove existing cracks up to 3/16 inch

[6] J.W. Fisher, H. Hausammann, M.D. Sullivan, and A.W. Pense, *NCHRP Report 206:Detection and Repair of Fatigue Damage in Welded Highway Bridges,* Transportation Research Board, National Research Council, Washington, D.C., 1979.
[7] H. Takamori and J.W. Fisher, *Tests of Large Girders Treated To Enhance Fatigue Strength,* Transportation Research Board, National Research Council, Washington, D.C., 2000.

deep and 3 inches long from base and weld metal. Significant benefits may be attributed to the removal of existing weld defects near the weld toe, such as removal of slag intrusions and undercuts, and improved weld toe geometry. The application of the overhead field weld for the cover plates, however, will prove difficult and costly. Therefore, GTA re-melting is not recommended.

The final method of increasing the fatigue resistance of welded details is by peening of the weld toe. The peening process works by plastically deforming the weld and base metal in the area of the weld toe. This deformation has two beneficial effects in terms of fatigue resistance. First, it modifies the geometry of the weld toe, reducing the stress intensity created by the kink between the weld and the base metal. Second, it creates a residual compressive stress field which acts to reduce the magnitude of the tensile stress range. A number of peening processes are available, including shot, air-hammer, and ultrasonic. The shot peening process utilizes small spherical media which are fired at the base material to produce the beneficial deformations, with the size, mass, and velocity of the shot all having an influence on the result. Given that the remedial measures will be required to be applied in the field, the shot peening process is not applicable as it must be performed in a shop. As a result, shot peening is the least desirable peening method. Air-hammer peening is a process which utilizes a hand-operated pneumatically controlled hammer to deform the base metal. This process has been used to successfully increase the fatigue life of existing structures[8]. The pneumatic hammer typically operates at a frequency between 50Hz and 100Hz. The relatively low frequency can make the hammer difficult to control and to maintain proper alignment on the weld toe with a field application. Other drawbacks include the high level of noise created by the process, and the high degree of vibration which may cause health risks to the operator if prolonged use is required. The final peening process available is known as Ultrasonic Impact Treatment (UIT). UIT is similar to the air-hammer process; however, the impact frequency is roughly 27kHz. The high frequency make the noise and vibration levels much lower than the air-hammer process, which makes the UIT process much easier to control in the field. The tools are small, light, and easy to use, which makes the process suitable for a field application. Therefore, the UIT process is recommended to mitigate the load-induced fatigue at the transverse stiffener to web and diaphragm connection plate to web fillet weld ends and at the fillet welded cover plates.

Research indicates that the expected effects of the UIT process would be to increase the fatigue category of the transverse stiffener to web and diaphragm connection plate to web fillet weld ends from Category C' to

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[8] J.W. Fisher, *Fatigue and Fracture in Steel Bridges*

Category B. The fillet welded cover plate would be expected to increase from Category E or E' to Category B, assuming the UIT process is applied in the presence of full dead loads[9]. The increase in detail categories would place the calculated stress ranges below the CAFT. thus rendering the remaining fatigue life infinite. UIT is a proprietary technology which is marketed by Applied Ultrasonics of Birmingham, Alabama. Applied Ultrasonics leases the technology to the contractor and provides instruction and oversight to make sure the technology is applied correctly. The bid price for each contractor would be identical and the technology would be returned to Applied Ultrasonics upon completion of the project. The cost of UIT is approximately $300 per detail.

The results of the fatigue analysis also indicate that the base metal at the shear connector welds is prone to load-induced fatigue. These locations occur at the negative moment regions over the piers where the welded connections experience a tensile stress range due to the fatigue truck loading. Only details on the fascia girders were flagged, given their larger live load distribution factor as compared to the interior beams. Post-weld treatment such as UIT would probably improve the fatigue resistance of the detail, as cracking modes do initiate from the weld toe. However, no known research has been done to validate or quantify the benefits for these details. A more reasonable approach is to reduce the live load stress range at these details by modification of the existing geometry. Since the interior girders are required to carry a much smaller portion of the live load as compared to the fascia girders, remediation for the interior fascia is not required. Once the deck is made continuous, the existing interior fascia will be treated as a typical interior girder and will be required to carry a much smaller portion of the live load. The reduction in live load will be sufficient to lower the live load stress range below the CAFT for these details. Similarly, geometric modifications will work to lower the live load stress range for the exterior fascia girders as well. The existing deck cantilevers 4 feet-8 inches beyond the centerline of the fascia girder. If the current beam spacing is maintained, the limits of the proposed deck would produce a cantilever of only 8 inches beyond the centerline of the fascia. The smaller cantilever will act to reduce the magnitude of the fascia girder's live load stress range below CAFT for this detail.

The final load-induced fatigue prone detail which requires remediation is the longitudinally loaded welded attachments located at the lateral gusset plates connecting the wind bracing in Span 15 and the drainage system downspouts to the fascia beam throughout the bridge length. The geometry of the welded attachments at both locations would be

[9] J.W. Fisher and S. Roy, *Enhancing Fatigue Strength by Ultrasonic Impact Treatment,* International Conference on Fatigue and Fracture in the Infrastructure, August, 2006.

classified as Category E. To alleviate, the wind bracing should be removed from Span 15 completely. Once removed, the web should be ground smooth at the weld connection locations. The proposed fix would raise the fatigue detail category from E to that of the base metal alone. The gusset plates connecting the drainage system downspouts to the fascia beam web are all located within a few feet of a pier. Given the presence of shear connectors at these locations, the steel beam and concrete deck are assumed to act compositely, placing the neutral axis near the steel beam's top flange. The geometry indicates that only compressive stress ranges would be created by live loads at these locations, rendering the details immune to fatigue problems. Therefore, no remedial measures are required at these details.

Distortion-Induced Fatigue

Remediation of the distortion-induced fatigue prone details must also be considered should the approach spans be reused. Currently, inadequate web gaps at the diaphragm connection plates is the only type of distortion-induced fatigue detail which exists on the bridge. Unfortunately, however, the number of locations which require remediation is sizeable, as the details are located throughout the original structure as well as the 1981 widening.

Currently, the diaphragm connection plate is not attached to the tension flange, leaving a small "web gap" at the location where the connection plate corner is clipped to provide clearance around the longitudinal flange-web weld. The current geometry is known to be problematic and prone to fatigue cracking due to the out-of-plane rotations, and evidence of this type of cracking is currently apparent in at least one location. To alleviate the potential of further cracking due to distortion-induced fatigue, a positive connection between the end of the diaphragm connection plate and the beam flange must be created. The connection detail could be done when the deck is removed to adjust the beam elevations to match the proposed profile. Either welding or bolting the plate to the flange could be used; however, a bolted detail is preferred given the problems associated with field welding. Figure 29 shows an example of the bolted type of detail. Either detail would increase the stiffness of the connection greatly and move any stress concentrations caused by distortions away from the web gap, thus removing any potential fatigue problems.

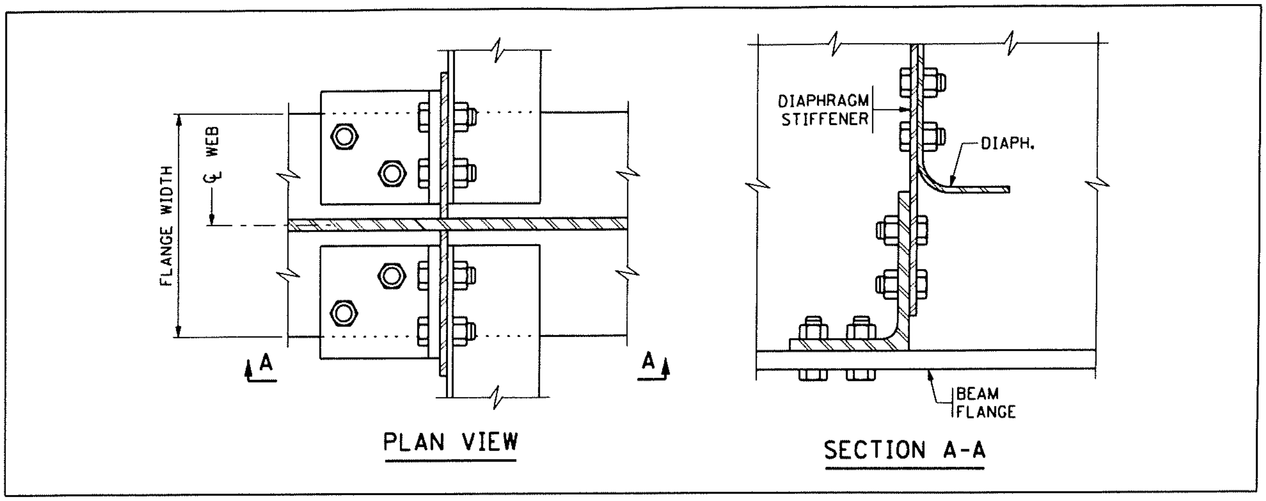

Figure 29. Flange Stiffener Detail

6.1.8. Fatigue Evaluation Conclusions

Approach spans contain fatigue prone details which do indicate a finite life. In order to evaluate the residual fatigue life of the approach spans to help determine their viability for reuse, a numerical evaluation using MDX was conducted. The bridge was broken into three distinct regions for analysis. Within the three regions of analysis, variable approach span/river span transition types were considered where appropriate. The lack of continuity created by severing a portion of the existing bridge for analysis was handled by creating a fixed end support at the truncation location and only considering valid results one span removed from the end span. Based on the variable end support conditions, a system analysis was required.

Based on the results of the evaluation, several conclusions may be drawn.

- The approach/river span transition type has a significant influence on the residual fatigue life. The behavior is not surprising, as the moment is very dependent on the support condition, especially near the end span. For this evaluation, three transition types were considered. The Type 1 transition is an extension of the existing approach span from an existing field splice such that the approach span terminates at the river pier with a simple support. Type 2 is a hinge joint with transfer girder transition similar to the existing transition; and Type 3 is a transition at a bolted splice in the existing approach span rendering the approach span fully continuous with main river span. Due to the rotational restraint provided by the fixed end support, the positive moment near the middle of the end span is reduced, and the negative moments at the end support and interior supports are increased. Figure 30 provides an illustration of the effects of approach/river span transition on the design live load plus dynamic load allowance moment for the interior fascia girder in Spans 7 and 8.

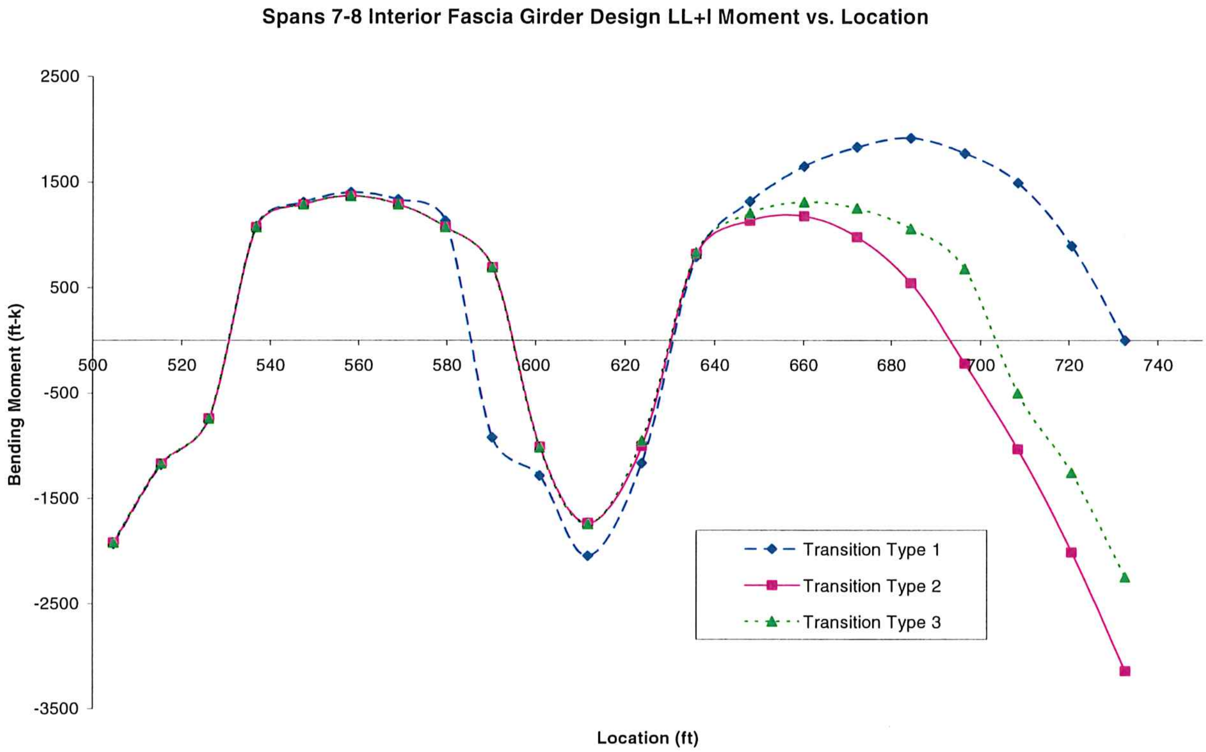

Figure 30. Design LL+I Moment Versus Location for Interior Fascia Girder in Spans 7–8

Given that the existing approach spans were designed with hinge joint transitions, modification of the transition type alters the live load the girders are required to carry. As a result, the Type 1 transition shows the shortest residual fatigue life, and the Type 3 transition shows the longest residual fatigue life. Also, the location of the critical element depends on the approach/river span transition. The critical location for the Type 1 transition occurred at the first interior support away from the transition span, and the critical element was the base metal at the longitudinally loaded fillet welded shear connectors, which are present in the negative moment regions over the piers. This detail is classified as a Category C detail. The critical detail location for the Type 2 and Type 3 transition occurs at the end of the diaphragm connection plate/web fillet weld located near the midpoint of Span 7, which is classified as a Category C' detail.

- Even though several existing details are prone to load-induced fatigue, mitigation methods are available such that infinite fatigue life may be expected. Weld toe modifications and geometric alterations were discussed as remedial measures to increase the resistance of the fatigue prone details. Of the weld toe modifications available, the peening process is most suitable for the applications required on this project. The peening process is required at the fillet welded connections between the transverse stiffeners and diaphragm connection plates to the beam web, and the bottom flange cover plates on the rolled beam sections. Several peening processes are currently available; however, Ultrasonic Impact Treatment is assumed to be the easiest and most economical. The results of the UIT would be to increase the fatigue resistance at these details sufficiently to render them immune to future fatigue issues.

Geometric modifications are also required to mitigate load-induced fatigue in the base metal at the shear connector welds located at the negative moment regions over the piers. Since the fatigue prone details only occurred in the fascia girders and not the interior beams, the geometry of the proposed cross section will render the details immune to future load-induced fatigue. The interior fascia girders will act as typical interior beams once the deck is made continuous. Given that the limits of the proposed deck are narrower than the outer limits of the existing twin superstructures, the length of the deck cantilever will be much smaller on the rehabilitated bridge should the existing beam spacing be held. The result will sufficiently reduce the live load stress range the fascia girders are required to carry such that infinite fatigue life may be expected.

- Longitudinally loaded welded lateral attachments exist at two details on the existing bridge: as gusset plate connections in the lateral wind bracing system in Span 15 and as downspout support brackets near the piers throughout the length of the bridge. As remedial measures, the lateral gusset plates which are part of the lateral wind bracing system in Span 15 should be removed. The details are classified as Category E and are located in a region of high live load stress range near the bottom flange at mid-span. Once the gusset plates are removed and the web ground smooth, the detail will be classified as that of the base metal alone and will be immune to load-induced fatigue cracking. The lateral support brackets which are part of the drainage system do not require any remedial measures and may be left in place. The downspouts are all located within a few feet of the piers in the negative moment region. Given the presence of shear connectors at these locations, the steel beam and concrete deck are assumed to act compositely, placing the neutral axis near the steel beam's top flange. The geometry indicates that only compressive stress ranges would be created by live loads at these locations, rendering the details immune to fatigue problems.

- Inadequate web gaps at the diaphragm connection plates are the one type of distortion-induced fatigue prone detail which exists on the approach spans, and evidence of cracking at this location already exists. Remedial measures for this detail involve creating a positive connection between the end of the connection plate and the tension flange of the beam. Either a welded or bolted connection is appropriate; however, a bolted connection is preferred since it does not require field welding. Execution of the connection detail may take place when the deck is removed and the top flange is fully exposed.

- Based on the results of the evaluation, the remaining fatigue life of the approach spans will not eliminate their reuse as part of the bridge rehabilitation; however, their reuse will not come without a price. The estimated costs associated with mitigation of the fatigue prone details are as follows and include both materials and labor:

UIT at diaphragm connection plates - Transition Type 1	\$6,300
UIT at diaphragm connection plates - Transition Type 2	\$4,700
UIT at diaphragm connection plates - Transition Type 3	\$4,700
UIT at transverse stiffener welds – Transition Type 1	\$3,200
UIT at transverse stiffener welds – Transition Type 2	\$2,000
UIT at transverse stiffener welds – Transition Type 3	\$1,600
UIT of cover plate welds	\$15,000
Removal of wind bracing	\$15,000
Diaphragm connection plate/flange connection	\$30,000

6.2. Geometric Evaluation

Remaining fatigue life is not the only variable which needs to be considered for reuse of the existing approach spans. To be viable for reuse, the existing beams will have to be placed at efficient locations within the new cross section. In order to create a bridge cross section which meets current geometric design standards, significant modifications would be required to both the bridge alignment and cross section. To meet the traffic volume demands, a minimum roadway width that is much wider than the existing width would need to be maintained throughout the length of the bridge. In order to accommodate the wider superstructure, alterations to both the current framing plan and the diaphragm geometry would be required. Consequently, the alignment would also need to be altered to accommodate the wider superstructure. Since several widening options are under consideration for this project, no one alignment will satisfy the geometric requirements of each option. Therefore, two different alignments must be considered.

6.2.1. Existing Cross Section

The existing bridge cross section is highly variable from the South Abutment to the North Abutment. For purposes of comparison, the bridge was broken into three distinct segments. Segment 1 consists of Spans 1 to 6, Segment 2 is made up of Spans 7 to 17, and Segment 3 consists of Spans 18 to 29.

The existing bridge cross section consists of two independent superstructures separated by a split median barrier with a 1-inch polystyrene sealed longitudinal joint.

The main characteristic which defines the cross section in Segment 1 is the straight taper on the east side of the deck. Within this segment, the framing consists of nine girders for Spans 1 to 5 and eight girders for Span 6. The nine-girder section is composed of eight parallel girders with constant spacing and one tapered girder on the east fascia with variable spacing as shown in Figure 31. The first interior girder on the east side of the cross section terminates at the hinge joint 6 feet-0 inches short of Pier 5, resulting in the eight-girder section which continues to Pier 6.

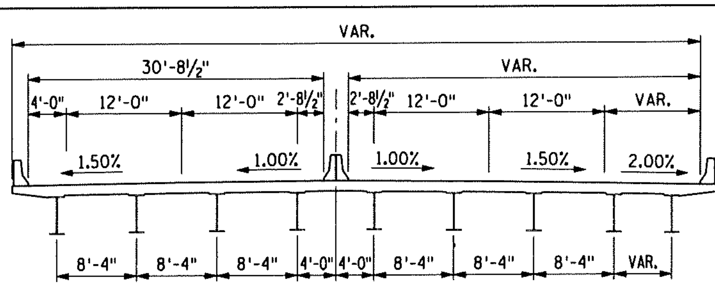

Figure 31. Existing Section, Spans 1–6

Segment 2 comprises Spans 7 to 17 and marks the constant width section of the bridge. Within this segment, the existing approach span framing consists of eight parallel girders as shown in Figure 32. Each superstructure unit has a 30 foot-8 1/2 inch roadway surface with Type J barriers on the exterior side and a Type J split median barrier along the interior. The resulting total superstructure width is 67 feet-4 inches.

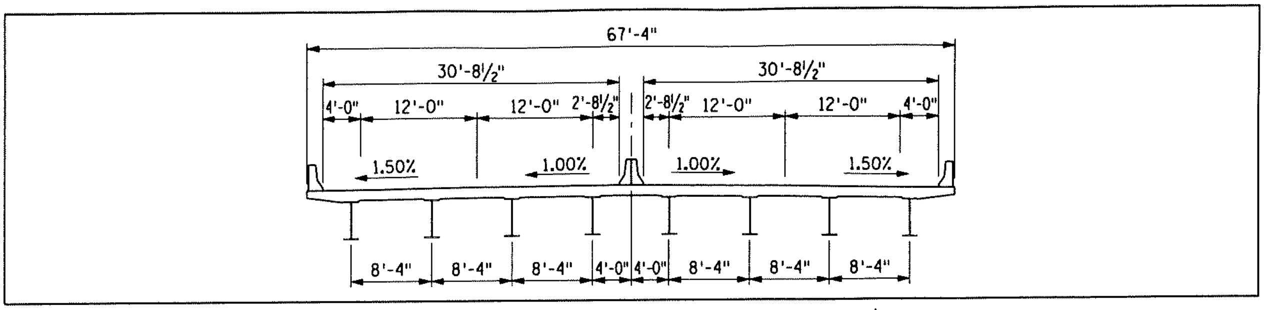

Figure 32. Existing Section, Spans 7–17

Segment 3 is comprised of Spans 18 to 29 and has the most highly variable cross section of the three. Segment 3 has changed in geometry over the years to accommodate new on and off ramps via two separate widenings, one to the north in 1981 and one to the south in 1991. As a result, the framing of this segment is very complicated. The core framing of the segment is the eight interior girders which run continuously and have nearly constant spacing through the segment. The fascia girders follow the edge of deck, with additional beams spliced in place to accommodate the widening deck.

6.2.2. Proposed Cross Section

Similar to the existing cross section, the proposed structure will consist of two independent superstructures; however, the width of each superstructure will be much greater. Both northbound and southbound cross sections will consist of three 12 foot-0 inch lanes with one 12 foot-0 inch shoulder on either side, resulting in a total roadway width of 60 feet-0 inches in each direction as shown in Figure 33. Since the resulting structure will be much wider, the existing taper on the east side of Segment 1 will no longer be required. Consequently, the cross section within Segments 1 and 2 will be identical, with a framing consisting of straight parallel girders. To accommodate the on and off ramps on the north end, the variability of the cross section within Segment 3 will still be required.

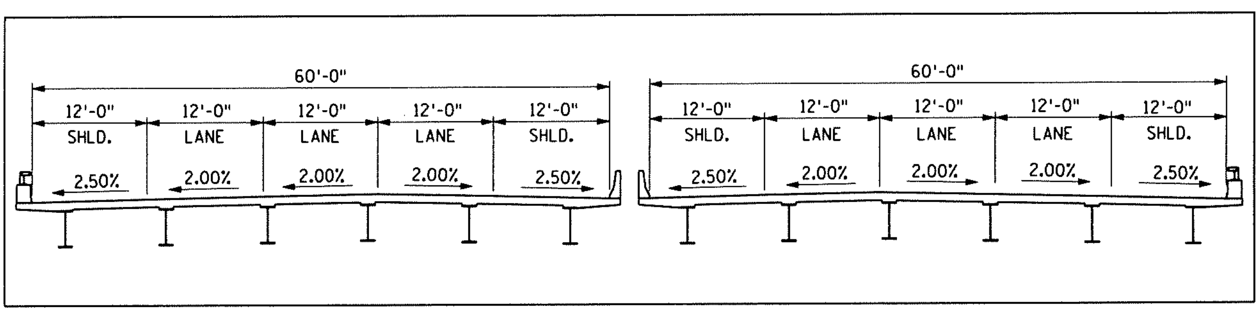

Figure 33. Proposed Section, Spans 1–17

6.2.3. Alignment for Options 1a, 1b, and 2

Options 1a, 1b, and 2 consider symmetric widening on either side of the current bridge centerline. Since the existing structure consists of two independent superstructures separated by a 1-inch longitudinal joint, the axis of symmetry will be the center of the longitudinal joint. Given that the current alignment is straight within Segments 1 and 2, no deviation from the existing alignment would be required for the proposed alignment.

Segment 3, however, is another matter. In order to avoid interference with the Gillette building within Segment 3, a series of horizontal curves beginning at Pier 19 and ending at the north abutment would need to be used. A superposition of the proposed limits of the bridge deck over the existing for Segment 3 is shown in Figure 34 and illustrates the divergence of alignments.

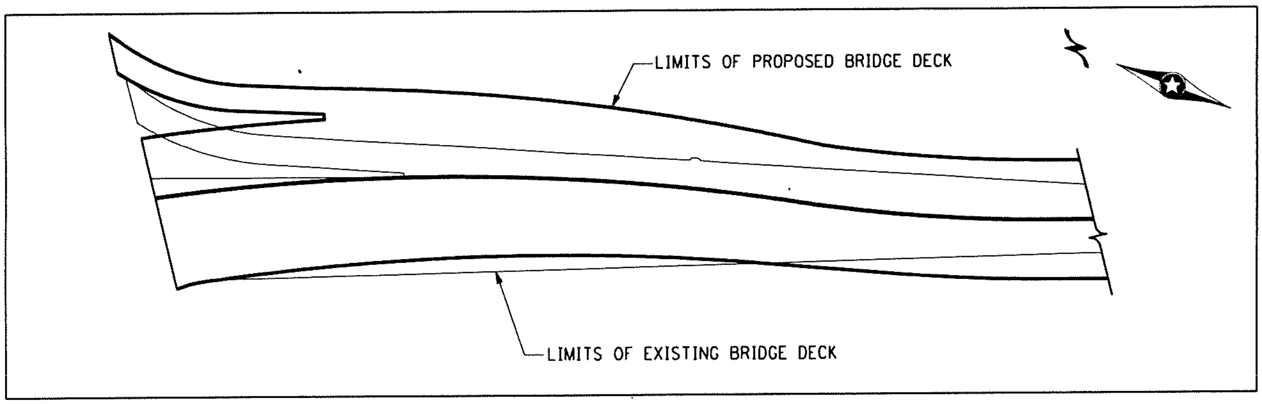

Figure 34. Superposition Plan, Spans 18–29, Option 1b

6.2.4. Geometric Implications for Options 1a, 1b, and 2

Based on comparison of the proposed geometry for Options 1a, 1b, and 2 to the existing geometry, recommendations may be made regarding the reuse of the existing steel approach spans within the various segments. Since it will be necessary to maintain at least one lane of traffic in each direction throughout the rehabilitation, all of the existing steel will not be able to be removed and reused at one time. As a result, the existing approach span girders may be incorporated into both the northbound and southbound structures. Additionally, the existing fascia girders need not be reused as fascia girders for these options.

For Segment 1, Figure 35 illustrates a comparison between the existing framing and the proposed deck limits. The existing tapered east fascia girder does not follow the proposed alignment and would therefore be ill-suited for reuse. Additionally, since the existing first interior girder does not traverse the entire length of the segment, significant modifications would be required to extend the girder line to Pier 6. Therefore, reuse of the existing east fascia girder and east first interior girder is not recommended. The alignment of the seven remaining parallel girders, however, is well-suited for the proposed geometry and reuse of these girders should be considered.

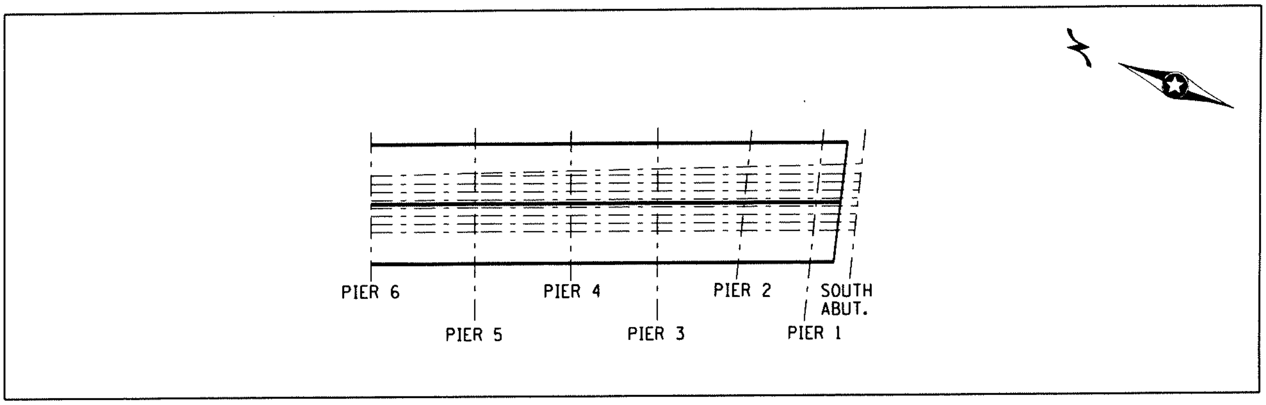

Figure 35. Pier Superposition Plan, Spans 1–6, Option 1b

Comparison between the existing and proposed sections for Segment 2 indicates favorable geometry for reuse of the existing approaches. Since the limits of the proposed cross section fall outside the limits of the existing cross section and both alignments are parallel within the entire segment, reuse of all girders within this segment should be considered.

Segment 3 represents the most challenging segment with regard to reuse of the existing steel approach spans. Figure 36 shows a comparison between the existing framing and the proposed bridge deck limits for this segment. The figure illustrates that the current framing does not match well within the boundaries of the proposed structure, nor does it follow the proposed curved alignment. Therefore, the steel approach spans are not recommended for reuse within this segment.

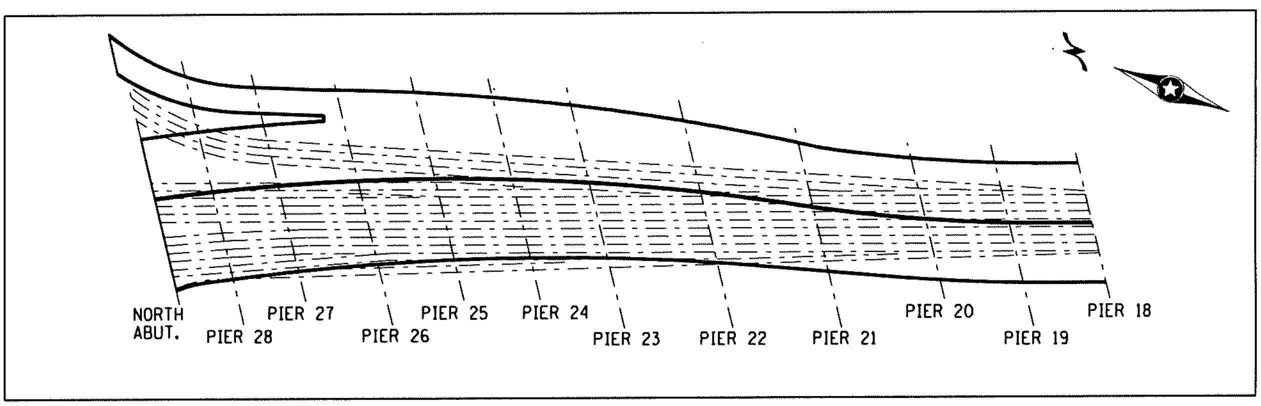

Figure 36. Pier Superposition Plan, Spans 18–29, Option 1b

In addition to geometric issues regarding girder alignment within the cross section, girder elevation will also play an important role in the cost associated with reuse of the existing approach spans. For comparison, Segment 2 provides the best opportunity to illustrate which components will require modification as a result of the change in bridge cross section, as both the proposed and existing alignments for this segment are parallel.

Currently, each deck of Segment 2 consists of two 12 foot-0 inch lanes with a 4 foot-0 inch shoulder on the exterior lane and a 2 foot-8 1/2 inch shoulder on the inside lane, resulting in a roadway that is 30 feet-8 1/2 inches wide as shown in Figure 32. The crown of each deck is located along the interior gutter line and slopes away at 1% for 14 feet-8 1/2 inches, and then 1.5% for the reaming 16 feet-0 inches. The proposed bridge cross section is shown in Figure 33 and consists of a single deck with three lanes of traffic and two 12-foot shoulders resulting in a roadway width of 60 feet-0 inches. The crown of the proposed deck is located between the two interior lanes and slopes away at 2.0% to the edges of the lane boundaries. The cross slope dips to 2.5% at the shoulder.

Figure 37 shows a superposition of the proposed cross section over the existing cross section for the southbound traffic lanes and indicates the incompatible geometry which must be resolved should the existing approach spans be reused. Assuming that the existing profile is maintained, the figure indicates that most of the beams will be required to move vertically to accommodate the new deck geometry. This movement will have no impact on the individual beams; however, fabrication of new diaphragms will be required.

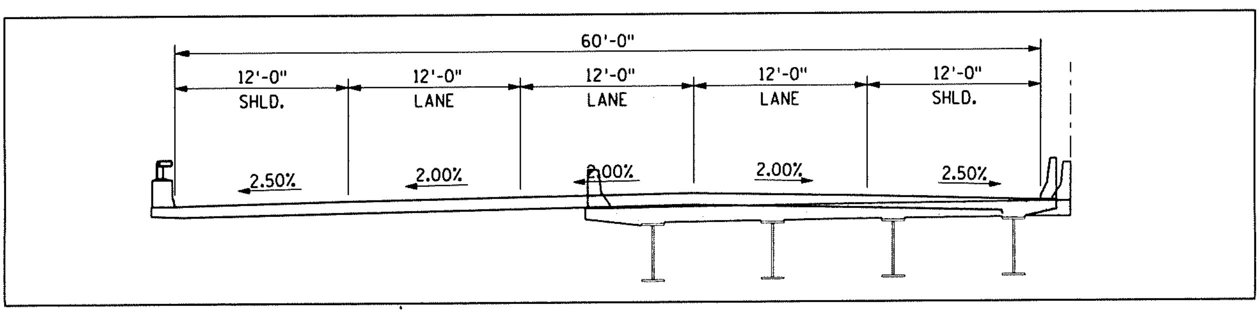

Figure 37. Superposition Section, Spans 7–17, Option 1b

6.2.5. Alignment for Option 3

Option 3 considers widening to the full geometric requirements through the creation of an entirely new northbound structure. The alignment of the new southbound structure would match that of the existing bridge through Segments 1 and 2. Similar to the alignment for Options 1a, 1b, and 2, the proposed alignment for Option 3 would diverge from the existing alignment within Segment 3. However, the divergence not nearly as severe as it is necessary to accommodate the on and off ramps, and not to avoid interference with the Gillette building. A superposition of the proposed limits of the bridge deck over the existing for Segment 3 is shown in Figure 38 and illustrates the divergence of alignments.

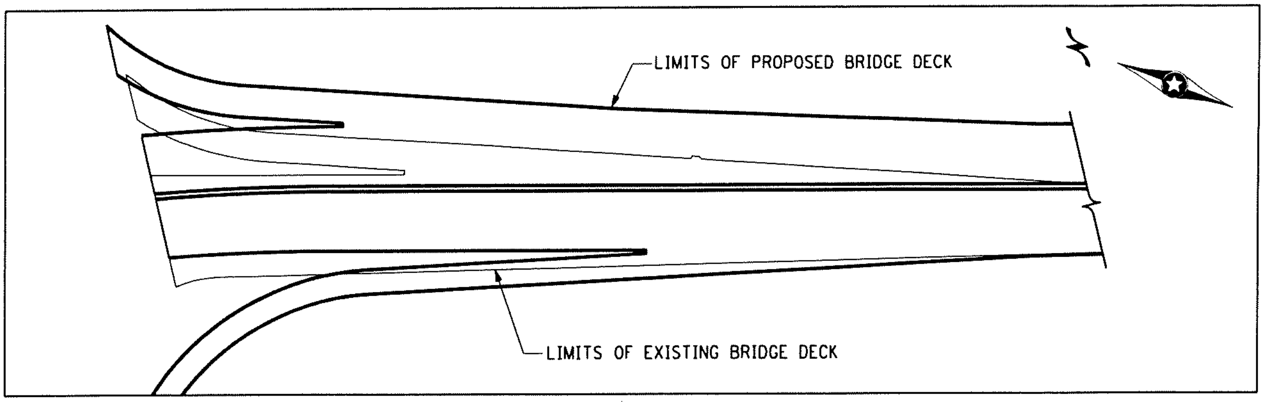

Figure 38. Superposition Plan, Spans 18–29, Option 3

6.2.6. Geometric Implications for Option 3

Based on comparison of the proposed geometry for Option 3 to the existing geometry, recommendations may be made regarding the reuse of the existing steel approach spans within the various segments. Since an entirely new bridge would be constructed under this option and it will be necessary to maintain at least one lane of traffic in each direction throughout the rehabilitation, reuse of the existing steel approach spans will only be considered for the new southbound structure

For Segment 1, Figure 39 illustrates a comparison between the existing framing and the proposed deck limits. Examination of the figure reveals that the existing tapered east fascia girder does not follow the proposed geometric limits and would therefore be ill-suited for reuse. Consequently, the existing first interior girder would have to take the place of the fascia girder on the proposed structure. Since the existing first interior girder does not traverse the entire length of the segment, significant modifications would be required to extend the girder line to Pier 6. Also, since the fascia girder must be designed to carry a larger portion of the live load, it is doubtful the capacity of the existing first interior girder would be sufficient to safely act as a fascia. Therefore, fabrication of a new east fascia girder will be required, as reuse of the existing east first interior girder is not recommended. The alignment of the seven remaining parallel girders, however, is well-suited for the proposed geometry and reuse of these girders should be considered.

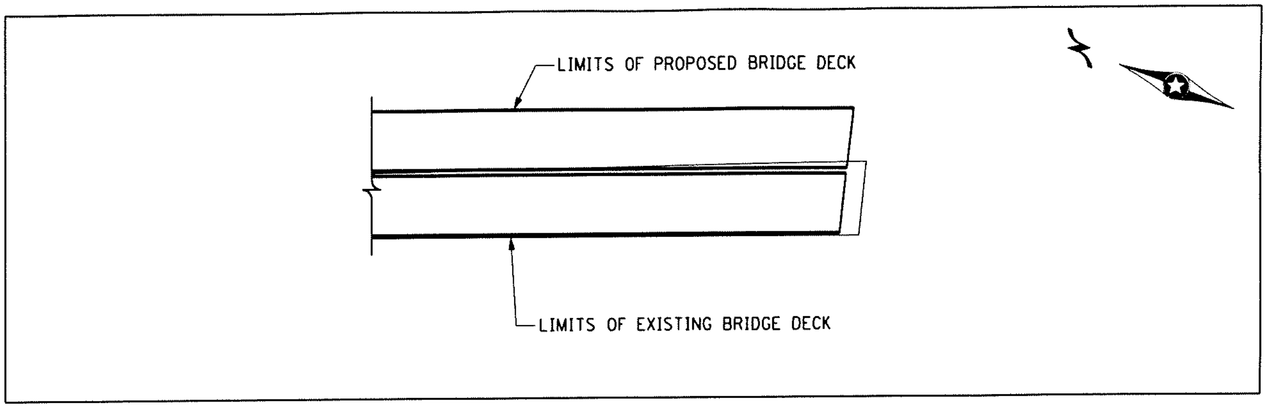

Figure 39. Pier Superposition Plan, Spans 1–6, Option 3

Comparison between the existing and proposed sections for Segment 2 indicates favorable geometry for reuse of the existing approaches. Since the limits of the proposed cross section fall within the limits of the existing cross section and both alignments are parallel within the entire segment, reuse of all girders within this segment should be considered.

Segment 3 represents the most challenging segment with regard to reuse of the existing steel approach spans. Figure 40 shows a comparison between the existing framing and the proposed bridge deck limits for this segment. The figure illustrates that the current framing does not match well within the boundaries of the proposed structure, especially the girders added as part of previous widening projects. However, the difference between the existing and proposed geometry of the structure is not so severe as to limit the reuse of all of the girders, and portions of the core eight interior girders may be considered for reuse as indicated.

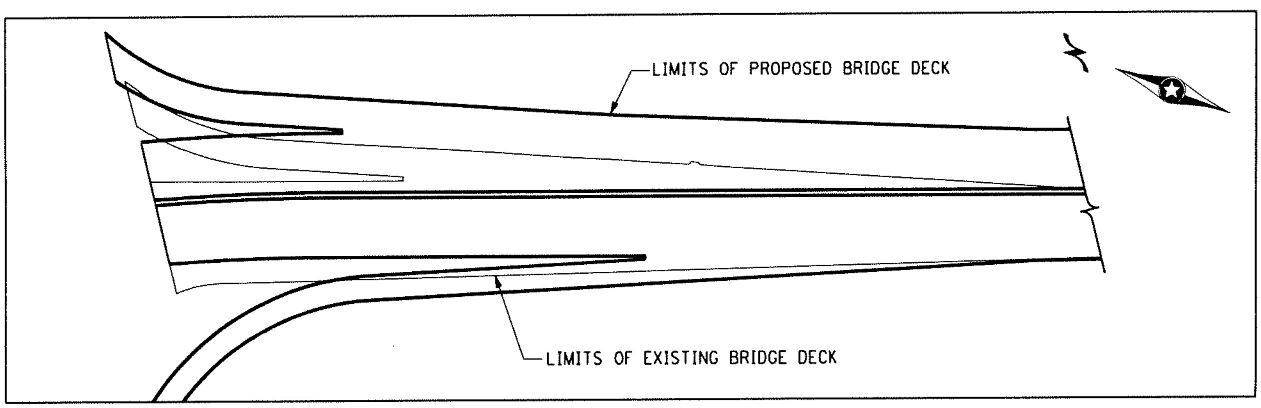

Figure 40. Pier Superposition Plan, Spans 18–29, Option 3

Similar to Options 1a, 1b, and 2, geometric issues regarding girder alignment within the cross section are not the only geometric constraints which require consideration. Girder elevation will also play an important role in the cost associated with reuse of the existing approach spans, and identical arguments to those for the previous options may be made. However, an additional geometric constraint will influence the location of the individual beams within the proposed cross section. For Option 3, the limits of the proposed southbound deck fall within the limits of the existing twin decks. If the current girder spacing is maintained, the resulting deck cantilever beyond the fascia girder will be insufficient. To alleviate, the girder spacing must be decreased. Figure 41 shows the proposed cross section for the southbound lanes with the existing girders at new spacing, which better meets the geometric limits of the deck. The required new diaphragms and connection plates are also indicated.

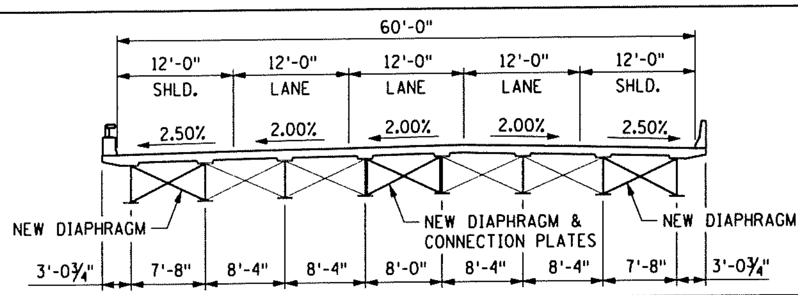

Figure 41. Superposition Section, Spans 7–17, Option 3

6.2.7. Conclusions

Difference in geometry between the existing and proposed structures is a limiting factor for sections of the existing approach spans. The bulk of the approach spans may be considered for reuse within Segments 1 and 2 for all options considered. Segment 3, however, is another matter. Given the curved alignment and bridge deck limits for Options 1a, 1b, and 2, all steel within Segment 3 is not recommended for reuse under those options. Alternatively, the bulk of the girders within this segment may be considered for reuse for Option 3. Only portions of the core eight girders and all of the girders associated with the past widenings are not recommended for reuse under Option 3.

Regardless of option, the change in bridge width and cross-slope will require fabrication of new diaphragms. Additionally, since the existing bridge deck is split by a longitudinal joint, reuse of the existing interior fascia girders will require the fabrication and installation of new diaphragm connection plates.

Depending on option, the location of the girders within the cross section may be very different in the proposed structure than the existing structure. Since it will be necessary to maintain at least one lane of traffic in each direction throughout the rehabilitation, all of the existing steel will not be able to be removed and reused at one time for Options 1a, 1b, and 2. As a result, the existing approach span girders may be incorporated into both the northbound and southbound structures and the existing fascia girders need not be reused as fascia girders for these options. Since an entirely new bridge will be constructed for Option 3, reuse of the existing steel approach spans was only considered for the new southbound structure for this option. Given the geometric limits of the southbound structure, the location of the individual beams within the cross section must be maintained.

6.3. Serviceability Evaluation

Should the approach spans be reused, ongoing maintenance will be required and must be considered as a long-term cost over the life of the structure when compared to alternative approach span structure types. Several maintenance issues will influence the long-term cost of maintaining the structure, including visual inspections, painting, joint performance, and corrosion.

Visual inspections will be required regardless of the approach span structure type selected. The cost of each inspection will not be distinctly different between the various approach span alternatives. However, the time required to perform the inspection may vary which will impact traffic flow and provide a variable social cost. Due to the large number of structural components, the time required to perform the inspections will be greatest for the existing approaches. If new steel approach spans are fabricated, the potential exists to reduce the number of joints and eliminate a large number of stiffeners. Consequently, fewer structural elements will be present on the bridge resulting in a shorter inspection. Either steel option, however, will take a greater amount of time to inspect when compared to a Prestressed Concrete Beam (PCB) alternative. Although the number of beams per span will be greater, the total number of structural elements will be less resulting in the shortest amount of time required per inspection of the three alternatives. Overall, however, great variability in the cost associated with performing a visual inspection is not expected and should not be a deciding factor for the purpose of this study.

Currently, the lifespan of the paint system used on steel bridges is less than the design life of the structure. Therefore, if either the existing approach spans are reused, or new steel approaches constructed, painting will be required at some point within the life of the structure. The Prestressed Concrete Beam (PCB) option is the most favorable characteristic with regard to routine painting as it is not required for this alternative. Since the current estimated cost of painting the existing approach spans is $860,000, a major long-term savings may be realized with the PCB alternative.

The existing approaches were constructed with large number of hinged expansion joints. These joints have proven to be a maintenance issue throughout the life of the bridge, and every effort to reduce their number should be considered. The inspection reports indicate that these areas are highly prone to increased rates of corrosion, and many of them are simply not properly functioning. The condition of the hinge near Pier 8 is shown in Photograph 14. Should the existing approaches be reused, it is unlikely the number of hinge joints could be reduced as the remaining portions of the beams and girders were not designed accordingly. Even if the bridge joints are replaced with strip-seal joints, and the rocker bearing replaced with either elastomeric or pot type bearings, the potential for future maintenance problems at these locations still exists. Therefore, if the existing approach spans are incorporated in to the future structure, the number of hinged expansion joints may not be reduced.

Photograph 14. Hinged Expansion Joint

The final maintenance issue associated with the reuse of the existing steel approach spans is corrosion, and corrosion will prove problematic. The existing beams indicate active corrosion is already affecting the bridge, especially on the fascia girders (4). Should the existing approaches be reused, major steps must be taken to halt the corrosion and quantify the areas which are already affected. Prior to final design, estimates of section loss will be required such that adequate capacity may be insured in the final product. Due to its highly variable nature, it would be safe to assume that the extent of the section loss associated with corrosion is highly variable both from span to span and within each span. For a safe design, the capacity of the section must be based on the section which is most highly corroded. Therefore, the capacity of the section may be highly compromised. The estimated cost associated with corrosion removal from the existing steel approach spans is unknown as the extent of the corrosion is not known. Should the existing approach spans be reused, painting would be required and the corrosion removal would be included in the surface preparation specifications for the paint system.

6.4. Conclusions

A number of factors influence the viability of re-using the existing approach spans. Although a number of issues will have to be resolved, the idea that all of the approach spans may be reused as part of the rehabilitation project is not fatally flawed by any one factor. For consideration of reuse as part of the current rehabilitation project, the remaining fatigue life, geometric incompatibilities, and long term maintenance were all considered. Based on the results of the investigation, the following conclusions may be drawn:

- Fatigue details do exist within the approach spans. Both load-induced and distortion-induced fatigue details were identified by both numerical analysis and careful plan review for known problematic details. Several mitigation recommendations including geometric modifications and post-weld treatments are required should the existing approaches be reused. Should the mitigation recommendations be implemented, the resulting structure should expect an infinite fatigue life.

- Geometric incompatibilities between the existing and proposed structures will dictate how much of the existing approach spans may be reused. Overall, the bulk of the existing approaches may be incorporated in to the new structure. For comparison, the bridge was divided in to three distinct segments. The individual segments are: Segment 1 - Spans 1 to 6, Segment 2 - Spans 7 to 17, and Segment 3 - Spans 18 to 29, with Segment 3 being most challenged. Within Segment 3, reuse of the existing approaches is not recommended for Options 1a, 1b, and 2, and only portions of the core eight girders are considered viable for Option 3. For all segments and options, incompatible bridge deck cross-slopes will force the fabrication of new diaphragms and some new diaphragm connection plates.

- The current condition of the existing approaches will have a major influence on their viability for reuse. Several long-term maintenance issues were considered, but the most influential by far is corrosion. The inspection reports indicate the interior fascia girders are most heavily affected with "severe corrosion" including some section loss. Until the extent of the section loss due to corrosion is quantified, their feasibility for reuse is in doubt.

7. OPTION 1A

7.1. Description of Option 1a

Option 1a would require symmetrical widening of the superstructure to full geometric standards while allowing no foundation work in the river. Widening under this option would need to proceed symmetrically about the centerline of TH 52. The curb-to-curb width of each of the proposed replacement bridge decks would be 60 feet-0 inches, compared with the 30 foot-8 1/2 inch width of each existing bridge deck. The replacement bridge decks would therefore almost double the overall deck width, and under this option all the extra loads would have to be supported by the existing foundations.

Each proposed bridge had to be designed with five design lanes, whereas the existing bridge only required a total of four design lanes.

7.2. Evaluation of Option 1a

For the evaluation of this option, the following methodology was used:

- The weight of the retrofitted pier cap was determined.
- The superstructure dead load included only the weight of the traffic barriers, wearing course, future wearing course, and a 9-inch-deep deck slab. No allowance was made for the weight of girders, webs, etc.

In comparison with the original superstructure, any proposed superstructure alternative will be heavier. The reasons for this increase are twofold. First, the existing superstructure consists of a fairly light, non-redundant two-girder system with floor beams. Second, the original bridge deck was only 7 1/2 inches thick, whereas a new deck slab will require a thickness of 9 inches plus a weight allowance for future overlays. The LRFD live loads are also significantly higher than the loads used for the original design. (See the discussion of ASD versus LRFD design in Appendix B.)

7.3. Conclusions

Even without an allowance for the weight of girders, webs, etc., the design loads in the piles exceeded the pile capacity. A number of factors make this option not feasible:

- The extra dead load due to the extensive widening.
- Larger pier support reactions due to live load (90% of the combined effects of continuous lane loading and two HS-20 trucks per design lane). Refer to the more detailed discussion of ASD versus LRFD design in Appendix B.
- A larger load factor: γ_p of 1.50 for limit state Strength IV (particularly adverse for unbalanced loads during construction).
- The eccentricity of the applied dead load during construction, with the associated large overturning forces and moments.

- The possible asymmetry of the increased live load. Asymmetry of the live load could either be the result of the staged construction or, during the final stage, due to the presence of traffic in only one direction. Furthermore, the widened structure would accommodate additional design traffic lanes.

8. OPTION 1B

8.1. Description of Option 1b

Option 1b requires symmetrical widening of the superstructure to full geometric standards while allowing foundation work in the river. Widening under this option would need to proceed symmetrically about the centerline of TH 52. Each of the two proposed bridge decks would have an out-to-out width of 62 feet-9 1/2 inches if 12-foot-wide shoulders are used.

8.2. Construction and Traffic Staging For Option 1b

The construction stages for Option 1b have been numbered consecutively, from Stage 1 through Stage 3. Each construction stage is associated with a concurrent traffic control stage. Construction would start on the east side of the bridge. Construction and traffic staging depend on the superstructure alternative for the river spans. The influence of the approach span construction is negligible. Central to all the construction and traffic staging concepts are interference problems (at the pier cap level and at the foundation level), the construction requirements for a specific superstructure type for the river spans, and the future ability of the completed piers to withstand the forces from barge collisions.

Before reconstruction could begin, all traffic would need to be moved to the west (or southbound) bridge. During Stage 1 construction, the existing east (or northbound) superstructure would be removed. (See Figures 42, 43, and 44.) It would be highly desirable to build the new northbound bridge in its entirety, but in the river spans there are obstacles in the way of accomplishing this goal. After the completion of Stage 1 construction, all traffic would be shifted to the new structure. During Stage 2 construction, the new west (or southbound) bridge would be constructed. (See Figures 45, 46, and 47.) Upon its completion, all traffic would be shifted back to the west bridge in order to allow the completion of the construction of the east bridge under Stage 3 construction. (See Figures 48, 49, and 50.) The traffic staging at the north end is shown in Figures 51, 52 and 53.

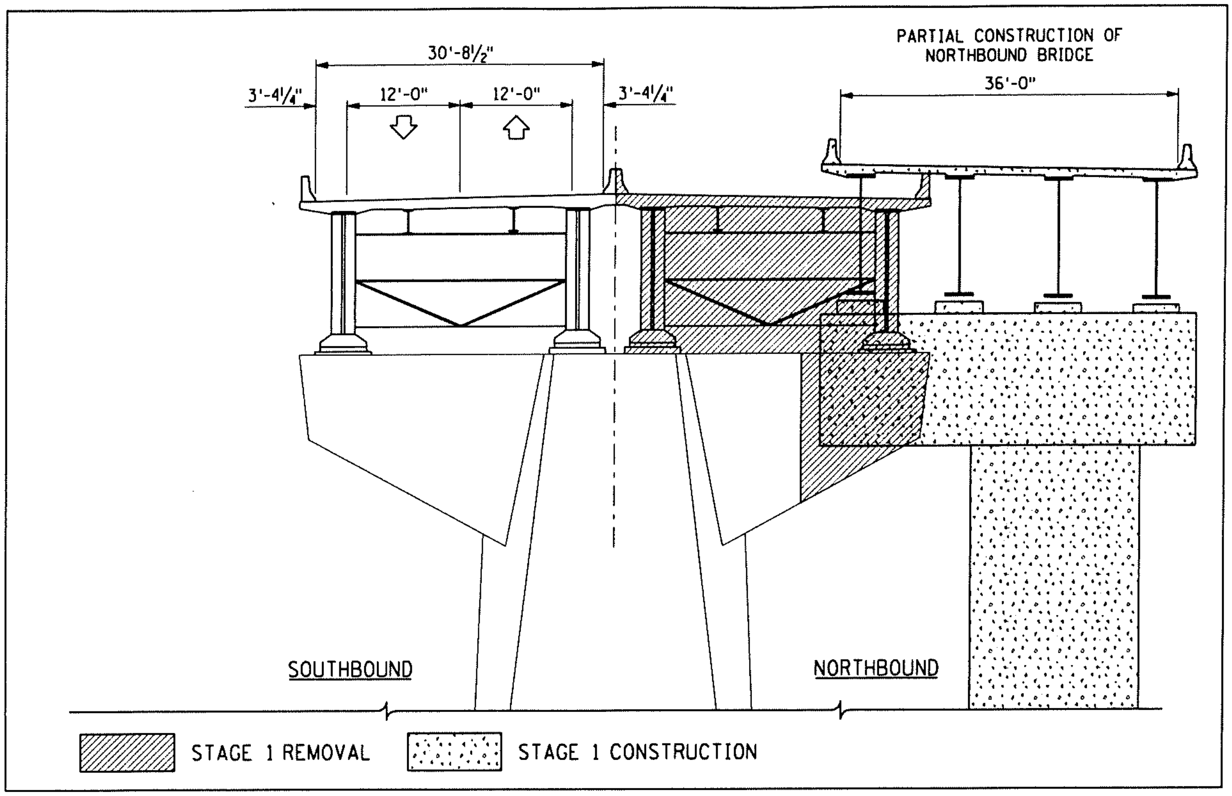

Figure 42. Option 1b, River Pier, Stage 1, Steel Girder

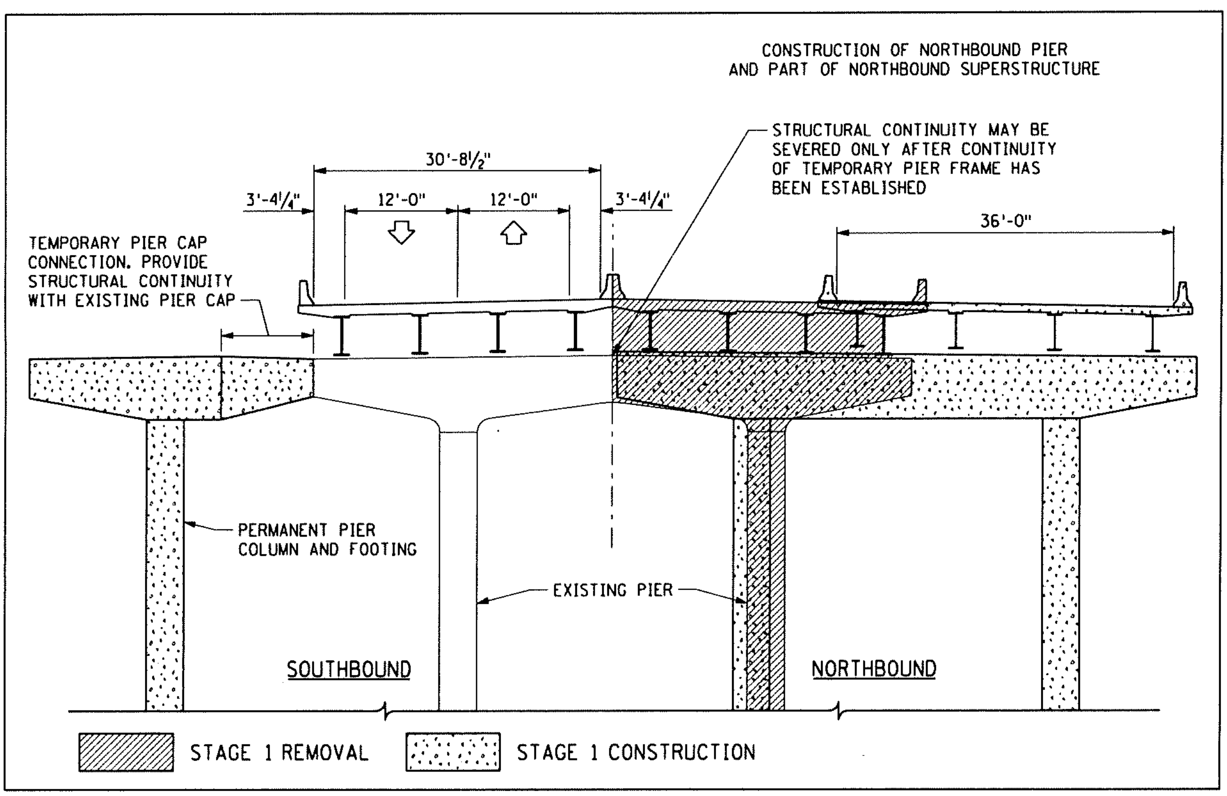

Figure 43. Option 1b, Piers 7 and 11, Stage 1, Steel Girder

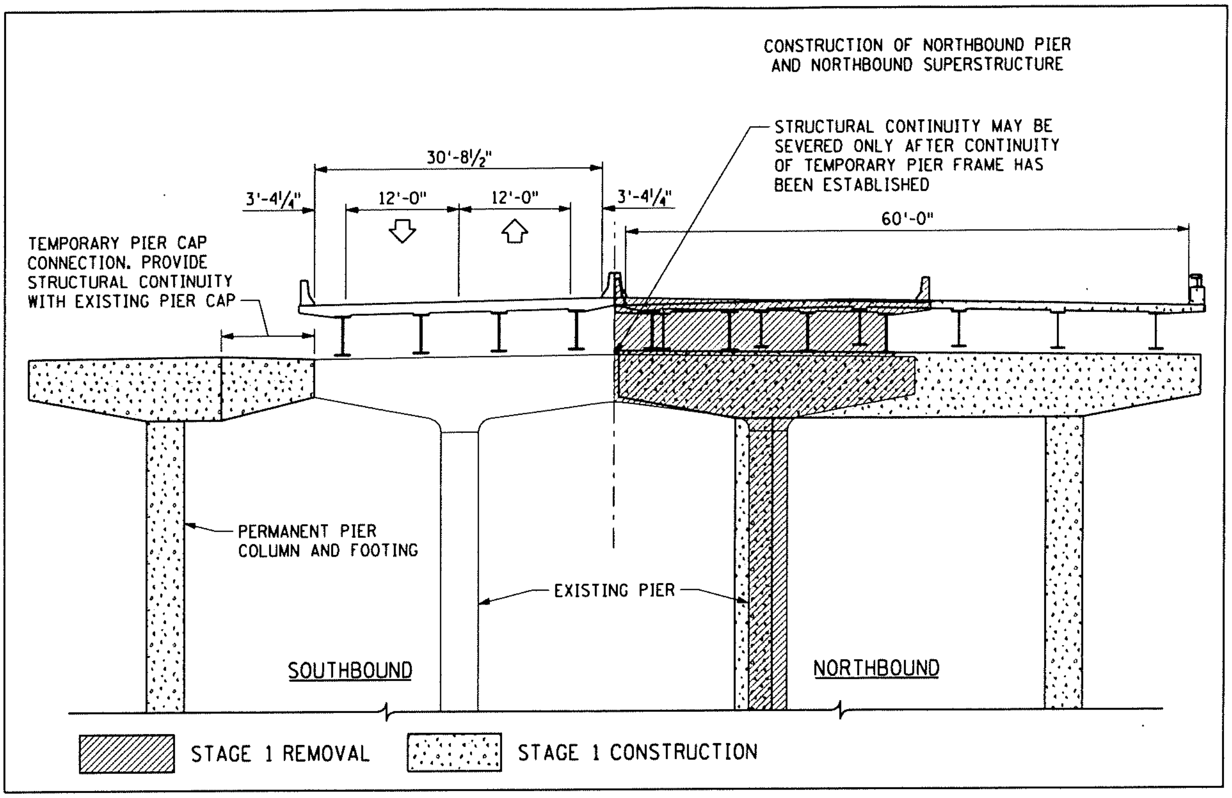

Figure 44. Option 1b, Approach Pier, Stage 1, Steel Girder

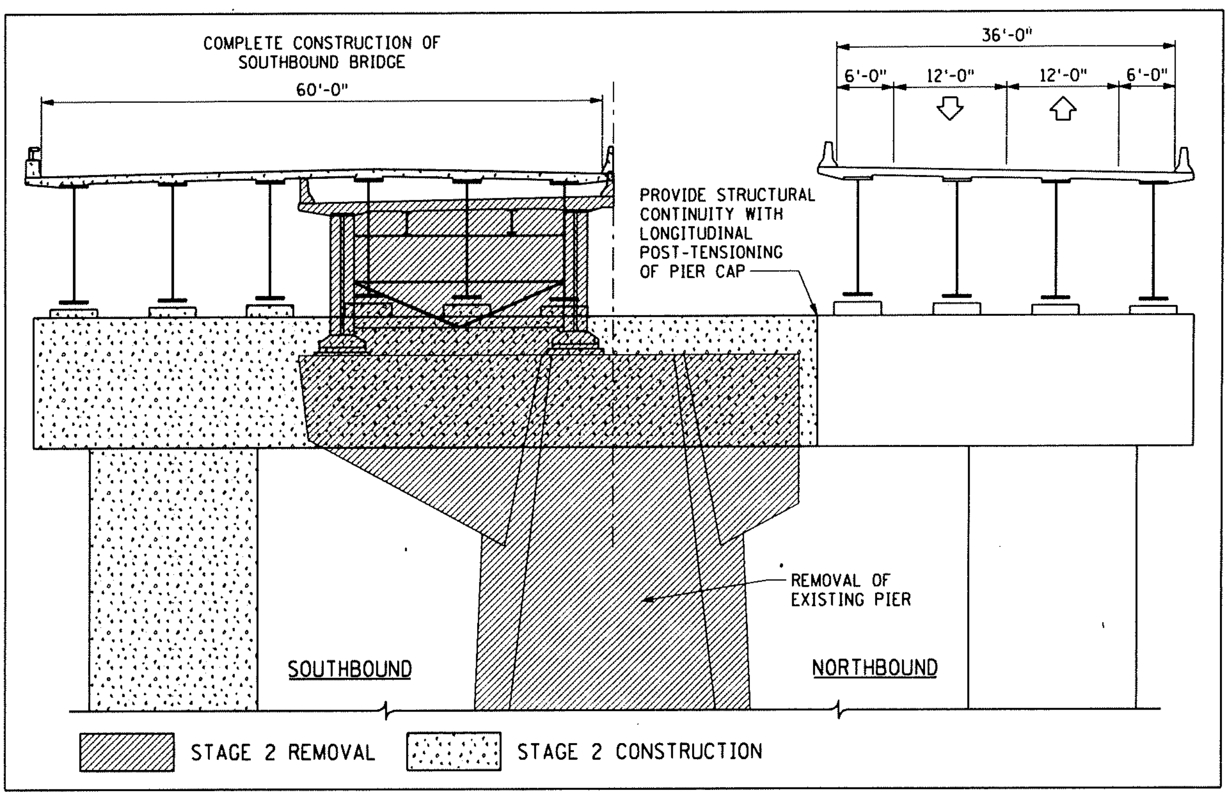

Figure 45. Option 1b, River Pier, Stage 2, Steel Girder

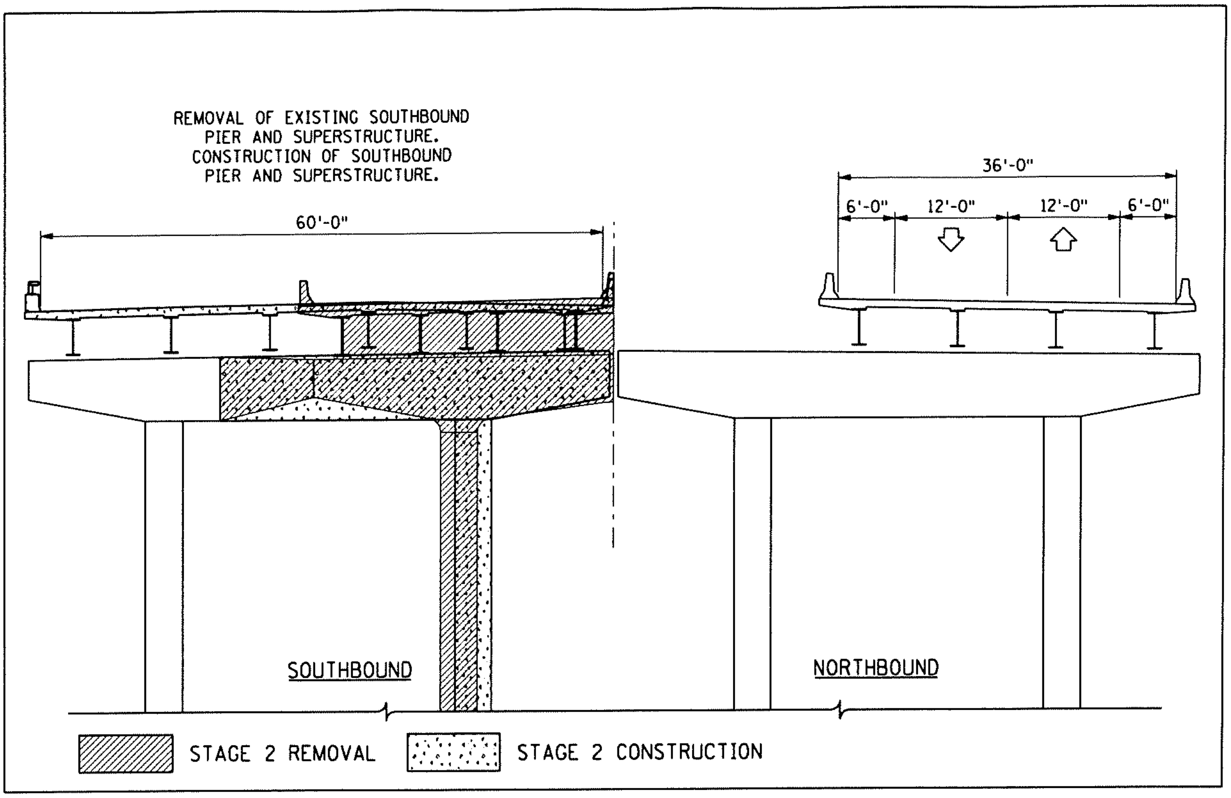

Figure 46. Option 1b, Piers 7 and 11, Stage 2, Steel Girder

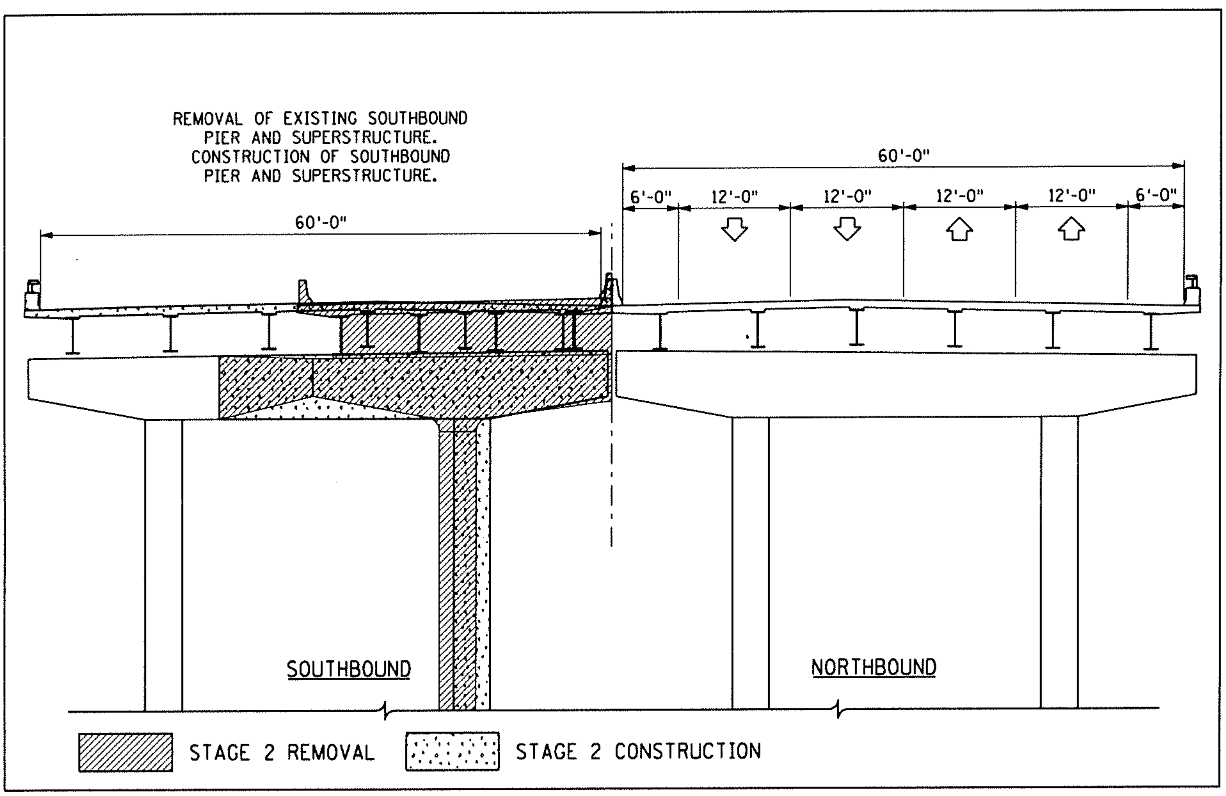

Figure 47. Option 1b, Approach Pier, Stage 2, Steel Girder

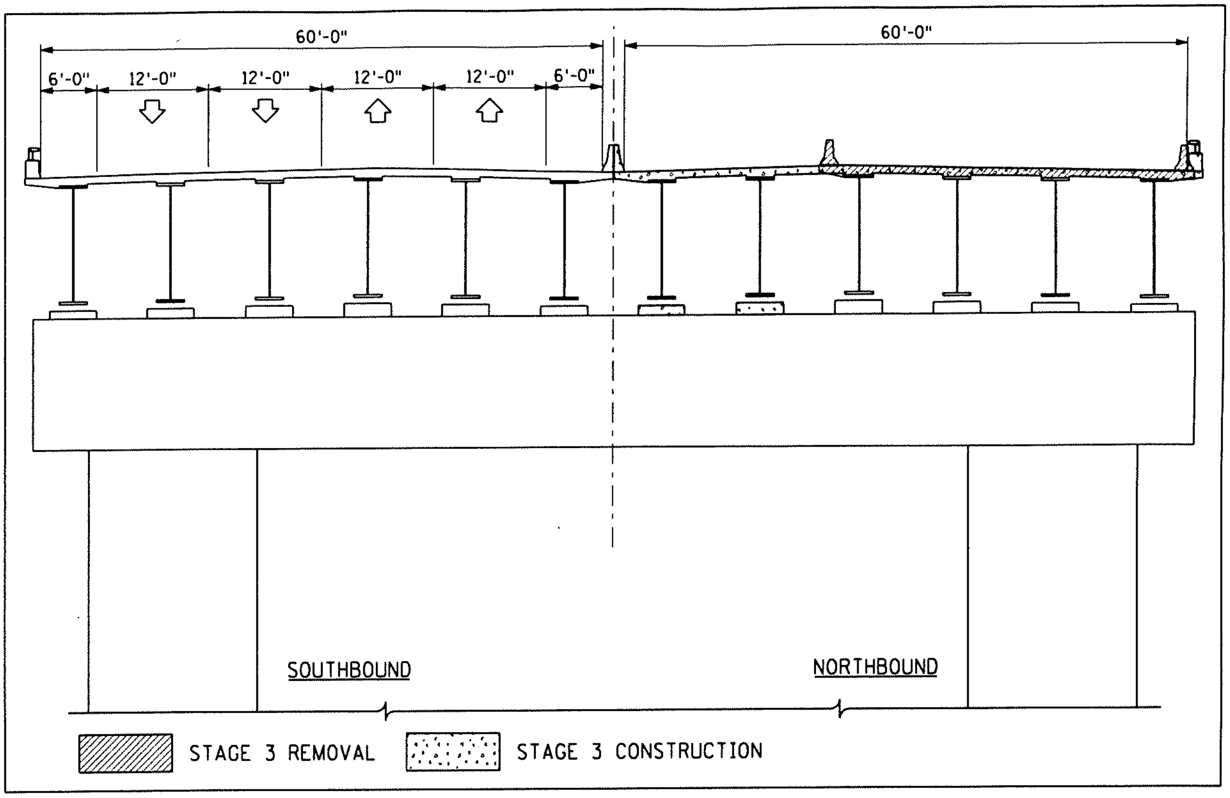

Figure 48. Option 1b, River Pier, Stage 3, Steel Girder

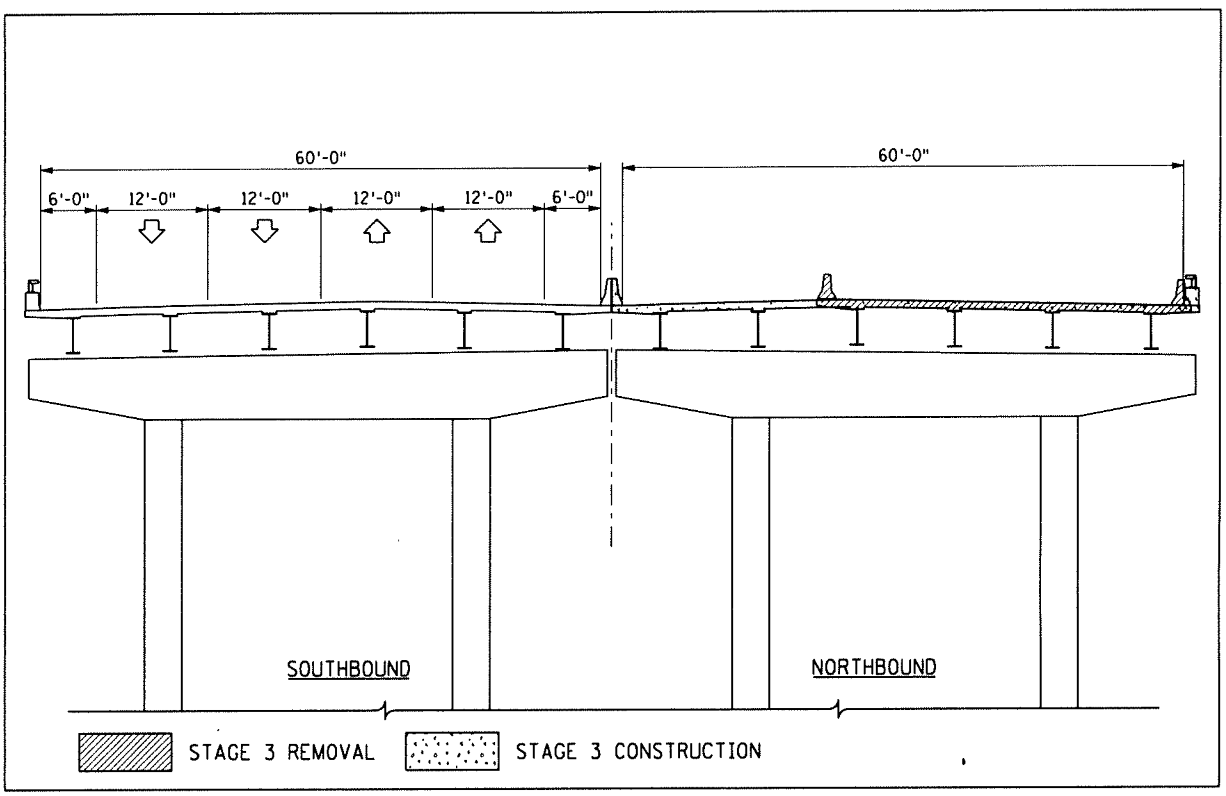

Figure 49. Option 1b, Piers 7 and 11, Stage 3, Steel Girder

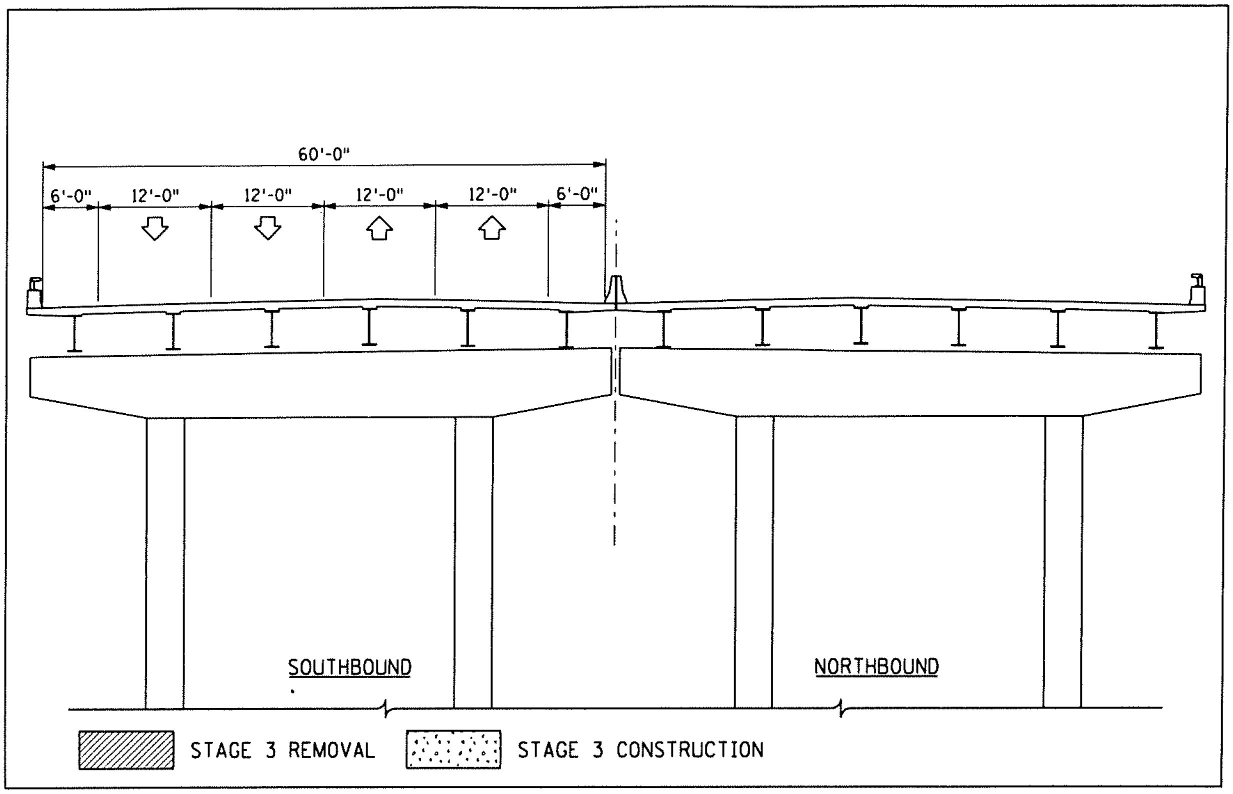

Figure 50. Option 1b, Approach Pier, Stage 3, Steel Girder

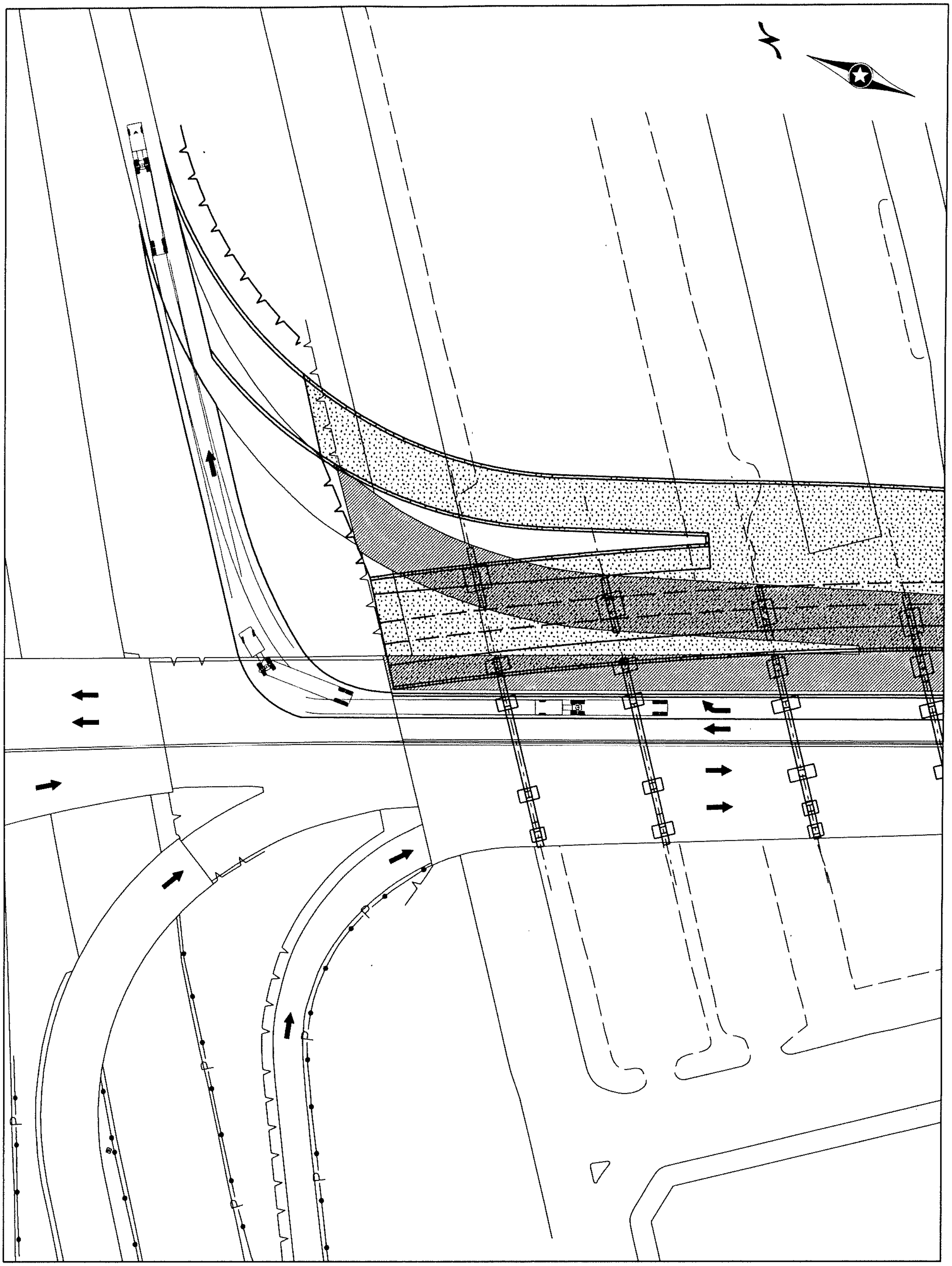

Figure 51. Option 1b, Roadway Approach, Stage 1

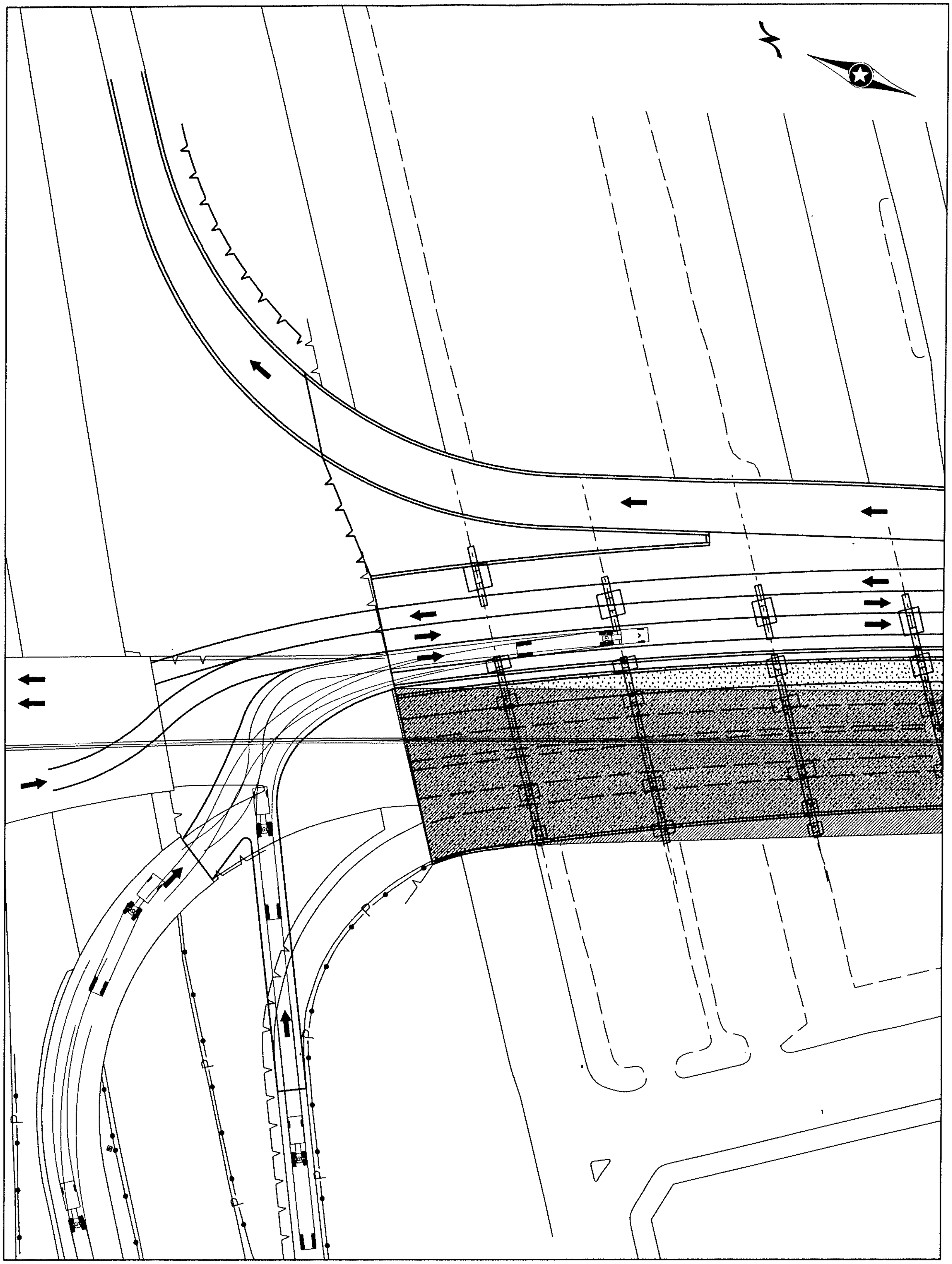

Figure 52. Option 1b, Roadway Approach, Stage 2

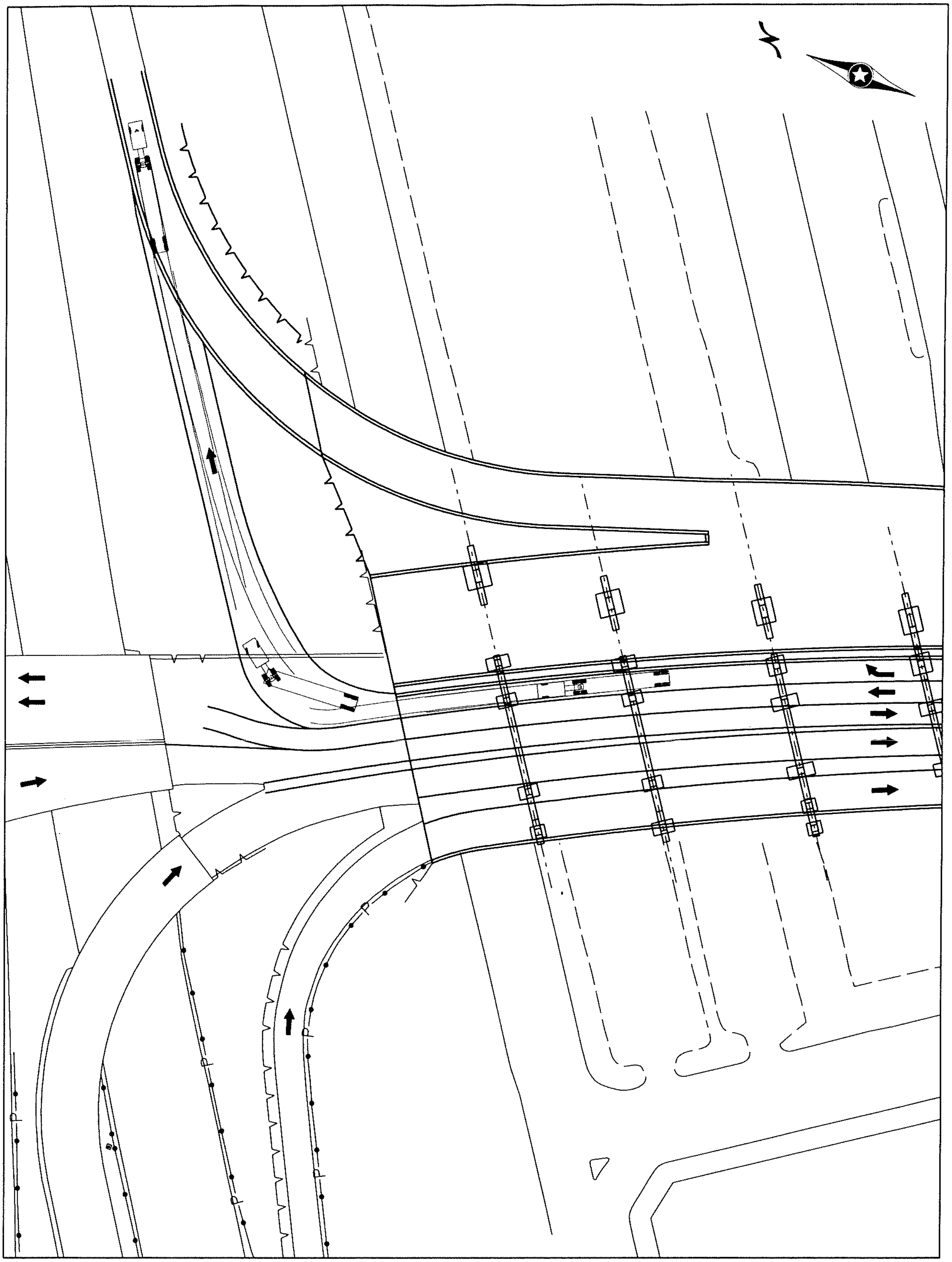

Figure 53. Option 1b, Roadway Approach, Stage 3

This construction and traffic staging sequence is far from ideal but cannot be avoided under Option 1b for several reasons. First, there are limitations on adding new piles immediately adjacent to the existing footings due to interference problems. The existing battered 100-foot-long perimeter piles of each pier footing enclose a large area, which cannot be utilized by new piles. By using 16-inch-diameter CIP concrete piles (instead of HP 14x73), a shorter pile length could be used. This choice would shrink the interference region of the existing piles and allow piles to be placed closer to the existing footing.

Second, the existing piles need to be unloaded before they can be reloaded and used to support loads in unison with new piles. A viable construction sequence needs to accomplish this important task.

Third, each superstructure alternative for the river spans has different pile requirements, both in terms of the number of piles and the preferred pile location. In addition, the pier type chosen should provide a logical load path for the superstructure loads.

Fourth, the new pier should be able to withstand barge impact loads, which implies that the existing piles should be utilized to increase the strength of the foundation against barge impact loads.

Other important factors for construction and traffic control are:

- The requirement to provide at least one lane of traffic in each direction.
- Dealing with the non-redundant two-girder systems of the existing river span superstructure, which restrict the limits for the removal and reconstruction operations.
- The non-redundant features of the existing single shaft hammerhead piers, which could lead to large unbalanced loads during construction.

As can be seen from the discussion above, construction and traffic staging depend largely on the pier type and on the superstructure alternative under consideration.

8.2.1. Construction and Traffic Staging for the Multiple Steel Girder Alternative

During Stage 1 construction (the construction of the new east, or northbound, superstructure), only a portion of the new superstructure could be constructed in the river spans and in the first adjacent approach span. The rest of approach spans would be constructed in their entirety. Upon completion of this portion of the bridge, all traffic would be shifted to it. After the traffic shift, the existing west (or southbound) bridge would be removed and reconstructed in its entirety, after which all traffic would be shifted to the west bridge. After this traffic shift, the deck of the river spans would be removed, the remaining girders and beams would be erected, and the construction of the east (or northbound) bridge would be completed.

Figures 42 through 53 show the construction and traffic staging sequence for the multiple steel girder alternative and its approach spans. (See Figure 7A in Appendix A.)

8.2.2. Construction and Traffic Staging for the Steel Box Girder Alternative

The steel box girder alternative would use a pier type very similar to that of the multiple steel girder alternative. As can be seen in Figure 8A (in Appendix A), it would not be possible to build a complete superstructure during the first stage. However, the stability of the steel box girder alternative would be highly dependent on the interaction of two box girder spines, which in turn would be stabilized by a continuous deck slab and by external diaphragms at the piers. Probable asymmetrical traffic loads on a deck slab wide enough for two traffic lanes would induce large torsion in the thin walls of the box section. For these reasons, the steel box girder alternative was eliminated as a suitable alternative for Option 1b.

8.2.3. Construction and Traffic Staging for a Post-Tensioned Concrete Box Girder Alternative

At the deck level, a single-cell post-tensioned concrete box girder for the proposed northbound bridge would fit without interfering with the existing river pier. At the footing level, however, there are serious issues, which are discussed in detail in Section 8.3.3. (See Figure 9A in Appendix A.)

8.3. River Pier Details

8.3.1. River Piers for the Multiple Steel Girder Alternative

The term "river piers" is used for Piers 8 through 11. Technically, only Piers 9 and 10 are located in the Mississippi River, while Piers 8 and 11 are located on land beyond the harbor line. These four piers support the deeper river span superstructure. All four piers are single-shaft hammerhead piers on a single footing. The pier shaft is sculpted to yield a multi-faceted geometric shape. Since Piers 9 and 10 flank the navigation channel, they may be hit by barges. For this reason, a combined footing would be advantageous for withstanding the large barge collision loads. The harnessing of separate footings into one unit would be accomplished by means of a crash wall. The crash wall would have a three-fold function: it would harness the footings into one unit and help to distribute the superstructure loads to all piles, it would protect the pier columns, and it would minimize turbulence in the vicinity of the piers.

As stated earlier, there are pile interference issues in the regions immediately adjacent to the river pier footings. At first glance, the widening concept of Option 1b would seem to exacerbate the interference problems, because all the widening would be clustered about the existing footing. However, by introducing a pier frame, which would bracket the existing pier, a number of beneficial effects could be accomplished.

First, all of Stage 1 construction could be supported independently from the existing foundation. Second, the subsequent construction operations would result in a completely symmetrical foundation layout. Third, the existing piles would be unloaded and then reloaded during Stage 2 construction, because the existing hammerhead pier (pier cap and shaft) would be removed. Once the existing pier has been removed, the new pier frame and crash wall could be constructed, Thus all parts of the footing, both existing and new, would be connected to act as a unit by a continuous crash wall. The continuous crash wall would help to distribute a barge collision load to all piles. Once the pier cap in its entirety has been poured, it would be post-tensioned. The final result of this construction concept would be a structurally efficient pier supported on a symmetrical foundation.

Piers 8 and 11 would require a similar construction sequence, but there would be no need for a connecting crash wall. The new pier frame would be supported by two individual footings. The existing pier would be completely removed.

8.3.2. River Piers for the Steel Box Girder Alternative

The pier concept for the steel box girder alternative would be very similar to that of the multiple steel girder alternative. However, this superstructure type is not suitable for staged construction in this setting.

8.3.3. River Piers for the Post-Tensioned Concrete Box Girder Alternative

Complications at the foundation level are responsible for the elimination of the post-tensioned concrete box girder alternative. As can be seen in Figure 9A (in Appendix A), there are two major obstacles in the way of providing a sufficiently strong pile foundation for a post-tensioned concrete box girder. First, it would not be possible to place a sufficient number of new piles in the area directly below the center of the box section. Second, the existing foundation piles could not be unloaded because of the requirement to maintain traffic.

8.4. Approach Pier Details

The majority of the following remarks apply to both Options 1b and 3. Since the alignments for these two options are different, the impacts on the available horizontal clearances will be different. All of the existing piers, other than the four piers identified as river piers, are two-legged frames with large cantilevers. The pier columns are rectangular and the pier caps are of variable depth throughout. Structurally, the pier frames were created by connecting the pier caps of two very slender hammerhead piers. Breaking the continuity of the pier cap would introduce serious stability problems unless countermeasures are taken. This aspect was one of the serious challenges for construction staging. At the north end, the pier caps have been extended to accommodate the widening of the bridge deck.

When the Lafayette Bridge was first designed, the horizontal clearance requirements for the approach spans were much more numerous. Several pier locations and span configurations were unfavorable. The modifications at the north end modified or added to the piers as necessary for widening the bridge. At the north end, the horizontal alignment for Option 1b would shift to the east in order to avoid the Gillette Building. This shift, plus the location of the new entrance and exit ramps at the north end, leaves many of the existing columns in a useless location.

Therefore, when dealing with the piers in the approach spans, it is important to evaluate two key aspects: the geometric compatibility and the structural adequacy of a pier. The term "geometric compatibility" is used to describe the general location and orientation of a pier, the location of its columns, and the adjacent span lengths that are the result of a given pier location. Put in simpler terms, evaluating the geometric compatibility of a pier means investigating whether an existing pier and its columns are in the right place, whether its location produces practical span lengths, and if there would be a more favorable location for the pier and its columns.

The structural inadequacy of pier footings and pier columns afforded greater flexibility in choosing better pier locations. The existing piers were designed according to the allowable stress design method (ASD) and reinforced with Grade 40 reinforcement bars. Very often the governing load case was a load case that allowed 25% or 40% overstress. For a more detailed discussion of the LRFD design approach for pier footings, see Sections 5.3.4 and 5.3.5, and the discussion of ASD versus LRFD design in Appendix B.

Based on the preceding observations, a strategy was developed to determine meaningful pier locations. Implicit in this strategy were two criteria. The first criterion was that the best location for the river span transition is at Piers 8 and 11. This criterion is discussed in greater detail in Section 8.7. The second criterion is the realization that the new approach span alternatives should not be saddled with the numerous shortcomings of the existing pier locations. (The only alternative that is tied to the existing pier layout is the alternative that reuses the existing steel girders. This topic is discussed extensively in Section 6). Once these two criteria were applied, the following pier layouts were developed (see Figure 6A in Appendix A):

- South of the Mississippi River, the existing South Abutment, which is supported by a spread footing, would be replaced by new abutment on piling. The new South Abutment would be moved closer to Alabama Street. Relocating the abutment would allow the elimination of existing Pier 1. All other piers on the south side of the Mississippi River would be reconstructed along their respective centerlines.

- North of the Mississippi River is an area where the greatest variations in span lengths and pier skew angles occurs. This is the area where numerous railroad tracks used to be located. Most of these tracks have been removed and this change allows a more radical approach to relocating piers. It would be possible to simplify the span arrangements from Pier 11 through Pier 16. The ensuing span arrangement would work for both the new steel girder alternative and the prestressed concrete beam alternative. Starting with Pier 18, the existing pier locations would be kept.

8.5. River Span Superstructure Alternatives

The three river span superstructure alternatives were introduced earlier within the context of construction and traffic staging. In a following section, the superstructure alternatives were evaluated with the foundation requirements in mind. In this section, additional important characteristics of the three alternatives will be discussed.

The multiple steel girder alternative has the following features:

- The superstructure, in comparison with a concrete alternative, would be relatively light. The lesser weight would be beneficial because fewer foundation piles would be required to support the bridge.

- The multi-girder system would afford great flexibility for construction staging.

- Whether the shoulder width is 10 feet or 12 feet, with multiple girders either deck geometry could be easily accommodated.

The steel box girder alternative has the following features:

- On the basis of the required number of lanes and shoulders, the bridge slab would be supported by two single-cell steel box girders with sloped webs of variable depth.

- In terms of weight, the steel box girder alternative would be very similar to the multiple steel girder alternative.

- The constraints of the construction sequence expose the Achilles heel of the steel box girder alternative in this specific application: lack of stability and limitations on the magnitude of the slab overhang.

- Providing a 12-foot shoulder would be more difficult because, regardless of the shoulder width, each bridge deck would have to be supported by two box girder spines. Adding 4 feet to the overall bridge deck width would increase the distance between the boxes by the same amount, because the magnitude of the slab cantilevers on the outside is limited.

- Due to the large transverse span lengths between the girder flanges, the thickness of the deck slab would have to be variable.

The post-tensioned concrete box alternative has the following features:

- On the basis of the required number of lanes and shoulders, the bridge superstructure would consist of a single-cell concrete box girder with sloped webs of variable depth.

- In terms of weight, a post-tensioned concrete box girder superstructure would be significantly heavier than a steel girder superstructure. Due to the constraints on pile locations, the piles required for this alternative cannot be installed where they are needed.

- At the deck level, this alternative would allow the construction of a complete bridge without interference.

- Providing a 12-foot shoulder would be more difficult, because of the increased cell dimensions.

8.6. Approach Span Alternatives

For the approach spans, three alternatives had to be investigated: reusing the existing steel girders, new steel girders, and new prestressed concrete beams. The issues associated with reusing the existing girders are presented in Section 6. In this section, the remaining two alternatives will be discussed. The pier layout described in Section 8.4 would work for both remaining alternatives. The framing plan layouts for both alternatives would pose no serious challenges. The sharply curved ramp structures would not be suitable for prestressed concrete beams.

8.7. River Span Transition Details

The existing river span superstructure is continuous and extends 40 feet beyond Piers 8 and 11. At the end of each 40-foot cantilever is a hinged support for the approach span girders. The girder depth near the piers is variable. The girder depths of the river spans and those of the approach spans are significantly different. Typically, there is a 70-foot parabolic transition of the web depth on each side of a pier, with the exception of the 40-foot cantilevered side. The cantilever portion is shaped to produce a smooth transition of the girder depth.

From an aesthetic point of view, the proper choice of the transition location is very important. Thereby, a variable-depth superstructure would serve a dual function. It would enhance the appearance of the main river spans by shaping the underside of the bridge like intrados of an arch. Equally important, it would help to minimize the depth differential between the constant-depth approach spans and the river spans. See Figures 54, 55, and 56.

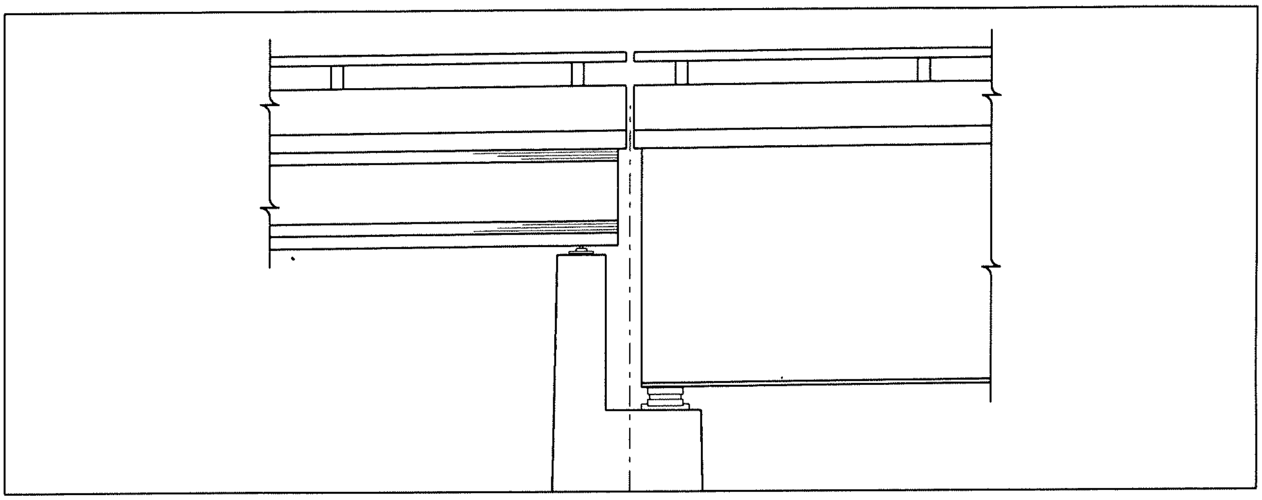

Figure 54. Detail Elevation, Constant-Depth Steel Girder

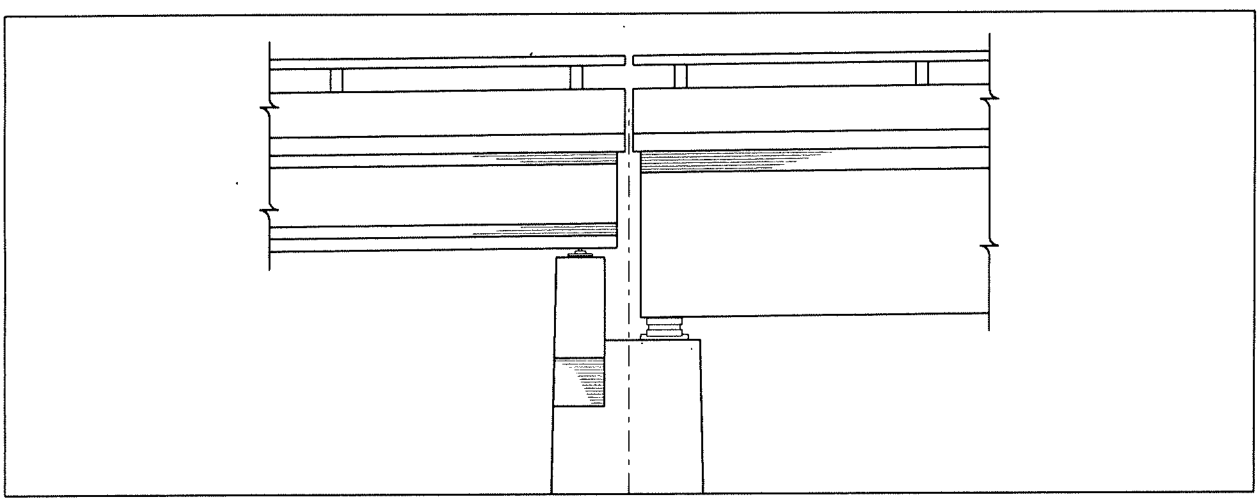

Figure 55. Detail Elevation, Steel Box and Variable-Depth Steel Girder

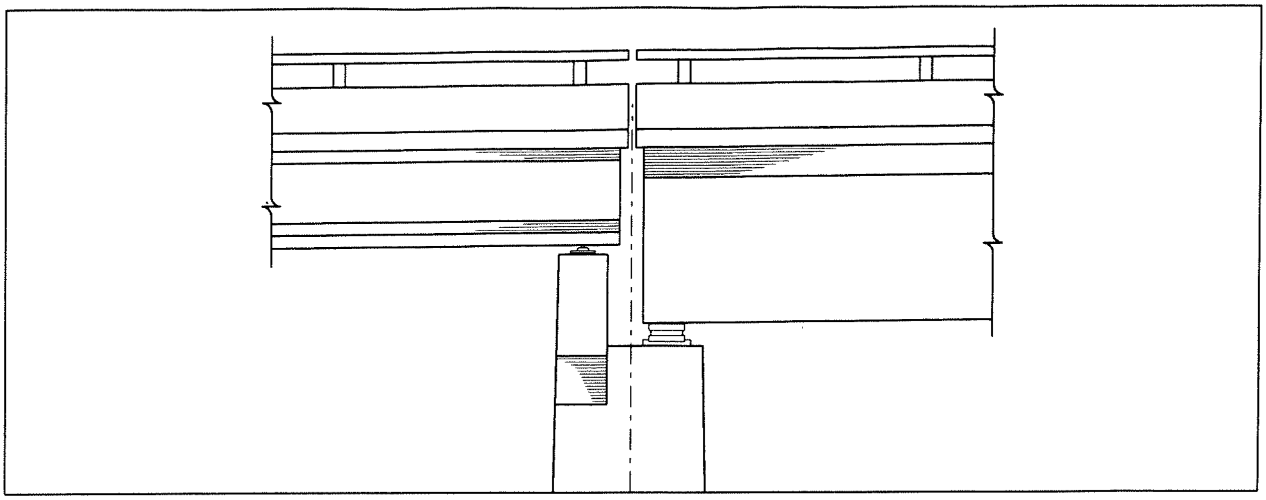

Figure 56. Detail Elevation, Concrete Box

8.8. Conclusions

Option 1b would work with only one of the three river span superstructure alternatives, the multiple steel girder alternate. For the approach spans, all three alternatives would work. A very important consideration is that the roadway alignment of Option 1b would not impact the tall masts of the 115kV power line. Neither mast would need to be relocated, nor would the current vertical clearance be worsened by the new construction. The following paragraphs list the pros and cons for each alternative:

The multiple steel girder alternative for the river spans offers many advantages:

- The bridge superstructure would be a redundant system.
- The superstructure weight would be less than that of a comparable concrete structure, which would result in fewer piles.
- Twelve-foot-wide shoulders could be easily accommodated.
- The two-column pier frame with a post-tensioned concrete pier cap and a crash wall would provide an efficient support system for the superstructure.
- Under the construction sequence for this alternative, the existing piles first would have most of the existing dead load removed. Then the new dead load would be applied as the existing piles are incorporated in a common footing.
- The transition details for the approach spans would work well.

The disadvantages are:

- Stage 1 construction would not produce a complete bridge, but require a third construction stage.

9. OPTION 2

9.1. Description of Option

Option 2 investigated the feasibility of widening the bridge on the existing alignment. The maximum allowable deck geometry for this option would be limited by the structural capacity of the existing foundation, since no foundation work in the river would be allowed.

9.2. Discussion of Results

In order to put the results for this option in their proper context, a number of observations need to be made:

- The in-place superstructure consists of a non-redundant two-girder system with floor beams, which contributed towards a comparatively light superstructure. A redundant steel girder system without intermediate stiffener plates would be heavier than the existing superstructure. (A post-tensioned concrete box girder structure would be too heavy for this option under any circumstances.)

- The original deck slab was only 7 inches thick, and the original design did not include an allowance for a future wearing course.

- The applicable design live loads at the time of the original bridge design were lighter than the live loads of the current LRFD design specifications. (Refer to the more detailed discussion of ASD versus LRFD design in Appendix B).

- Without additional piles, the existing foundation would not be able to withstand barge collision loads.

Once the curb-to-curb roadway width exceeds 32 feet, a new multiple-steel-girder superstructure produces pile loads larger than allowed by the LRFD code. The asymmetry of the design loads during construction would also limit the extent a new bridge deck can be safely widened. Thus, there are three factors that are mainly responsible for making Option 2 not feasible:

- The increase in dead load (additional deck width, higher girder weights, and a thicker deck slab).

- The higher live load reactions at the piers.

- The asymmetry of the design loads during construction.

Without additional piles, which are not allowed for this option, only a very minimal increase in the deck width (30 feet-8 1/2 inches versus 32 feet-0 inches) would be possible. Such a deck width would provide only two traffic lanes, plus two 4-foot shoulders. It would not provide sufficient space for three traffic lanes and substandard shoulders. Therefore, Option 2 is not feasible.

10. OPTION 3

10.1. Description of Option 3

Option 3 requires widening of the superstructure to full geometric standards, while allowing foundation work in the river. Widening under this option would center the proposed southbound bridge on the existing alignment. The proposed northbound bridge would be built on a new alignment at a downstream location (east of the existing bridge). Each of the two proposed bridge decks would have an out-to-out width of 62 feet-9 1/2 inches.

10.2. Construction and Traffic Staging

Option 3 has two construction stages. Each stage is associated with a concurrent traffic control stage. Construction would start on the east side of the bridge. Central to all the construction and traffic staging concepts are interference problems (at the foundation level), the construction requirements for a specific superstructure type for the river spans, and the future ability of the completed piers to withstand the forces from barge collisions.

Reconstruction could begin immediately without significant impact to traffic, which would continue to use the complete existing bridge. During Stage 1 construction, the existing east (or northbound) superstructure would be constructed, while all traffic would remain on the existing bridge (see Figures 57 and 58). After the completion of Stage 1 construction, all traffic would be shifted onto the new structure. During Stage 2 construction, the new west (or southbound) bridge would be constructed (see Figures 59 and 60). Upon its completion, traffic could use both bridges. This construction and traffic staging sequence is far superior to the staging sequence for Option 1b.

Construction and traffic staging are completely independent of the superstructure alternative for the river spans. However, Option 3 has two serious drawbacks. First, by shifting the proposed bridge eastward, the bridge would extend farther into the flight corridor for runway 14-32. More seriously yet, the tower of the 115 kV power line located east of the bridge would have to be relocated. Consequently, the changes made for the power line would impact the flight clear zone. Second, while Piers 9 and 10 for the proposed northbound bridge can be designed for all applicable loads, the existing foundations for these piers would have to be removed down to the cofferdam seal and reconstructed to include additional new foundation piles.

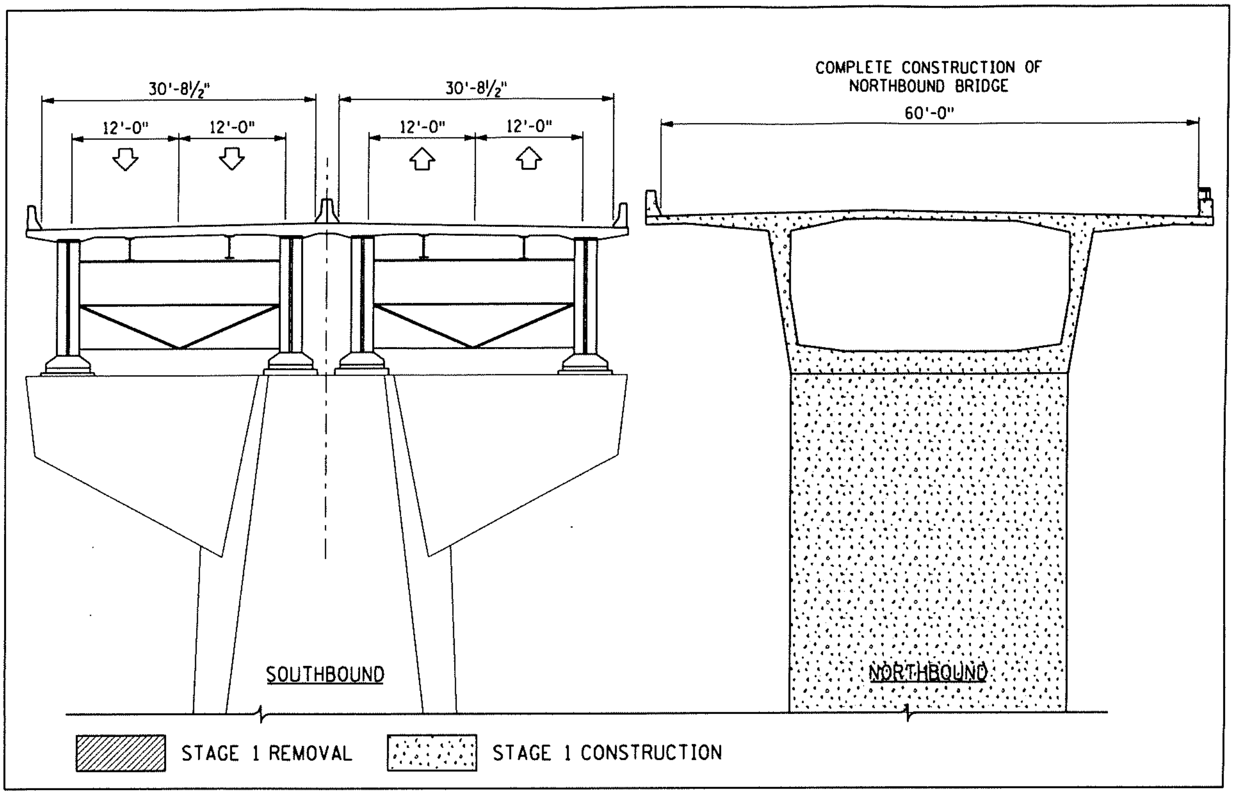

Figure 57. Option 3, River Pier, Stage 1, PT Concrete Box Girder

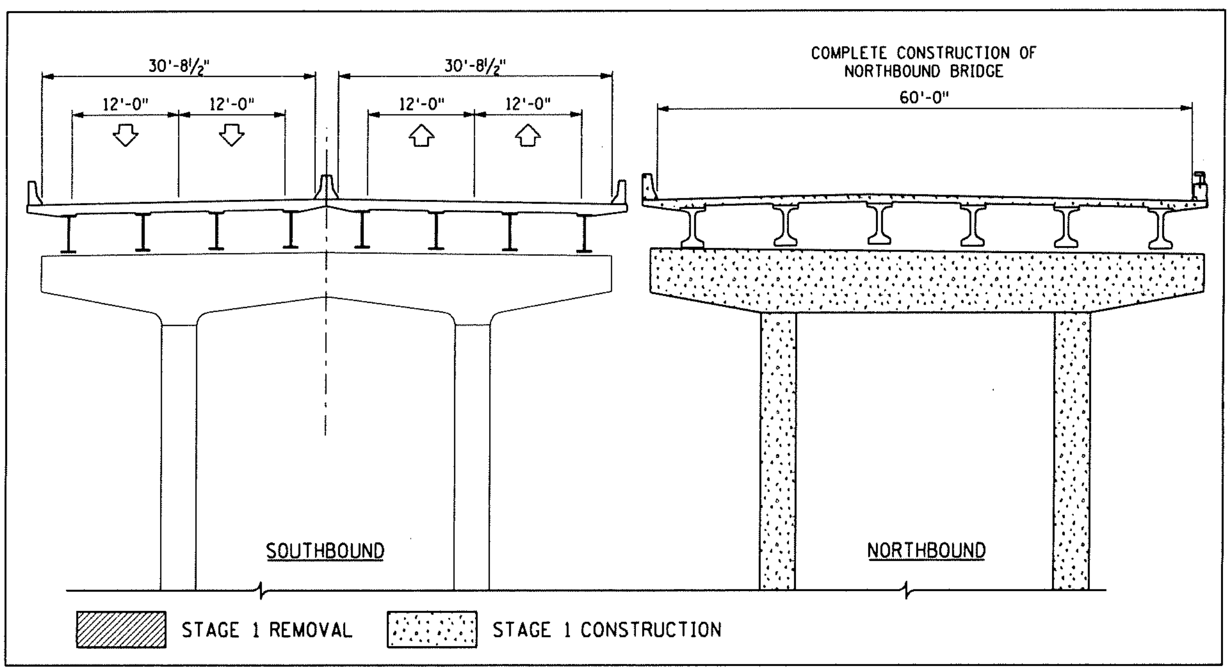

Figure 58. Option 3, Approach Pier, Stage 1, PCB Alternative

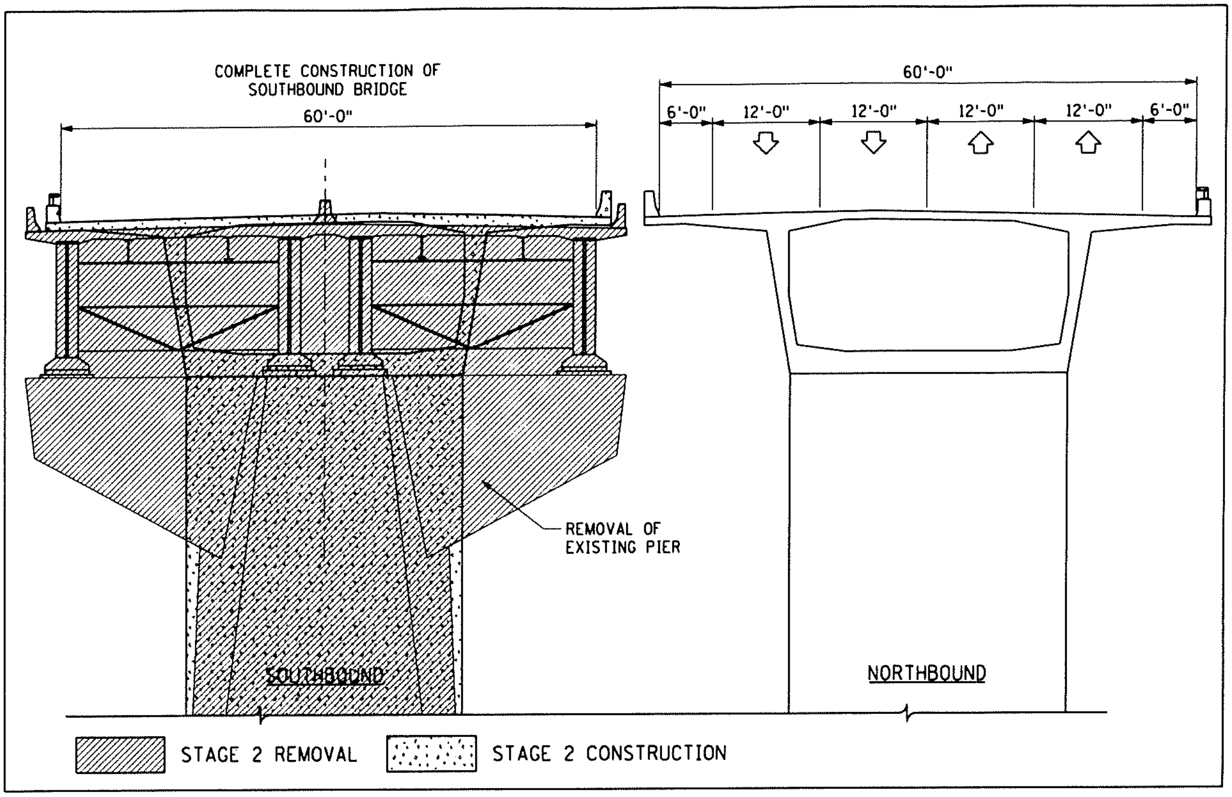

Figure 59. Option 3, River Pier, Stage 2, PT Concrete Box Girder

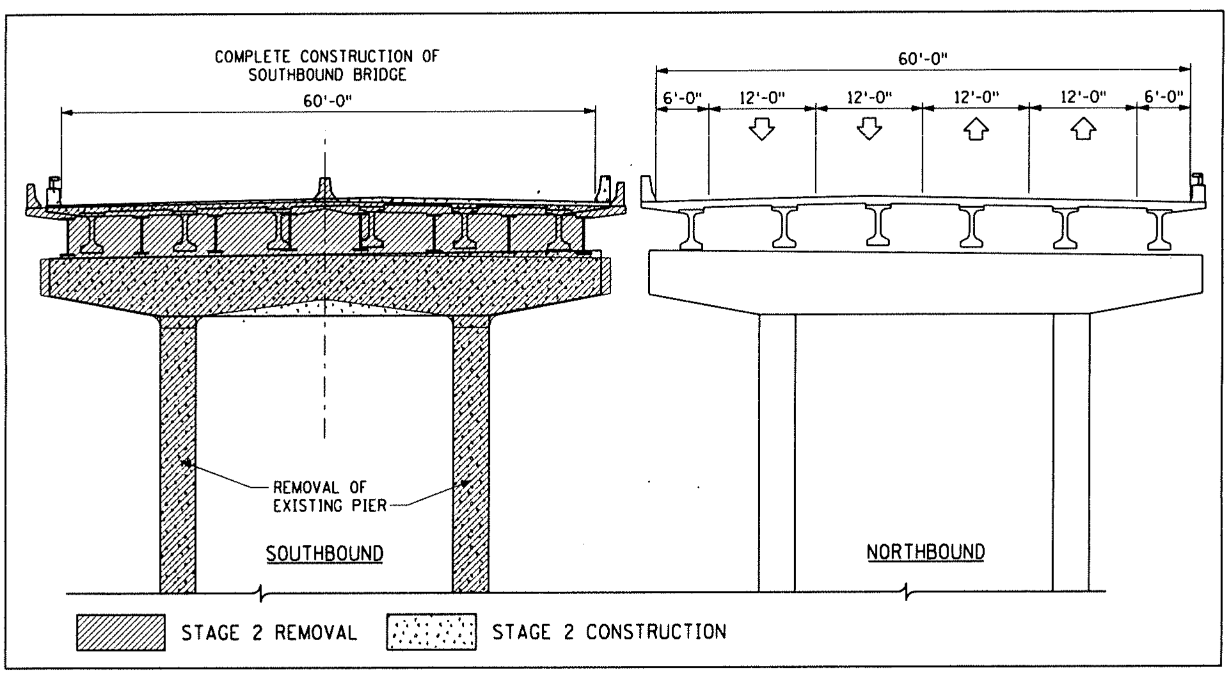

Figure 60. Option 3, Approach Pier, Stage 2, PCB Alternative

10.3. River Pier Details

For a general discussion of the river pier details, refer to Section 8.3.

10.3.1. River Piers for the Multiple Steel Girder Alternative

Piers 9 and 10 for the proposed northbound bridge could be shaped like the corresponding existing piers of the southbound structure. By connecting the existing pier footings with the new footings and introducing a continuous crash wall, the combined footing could handle a longitudinal barge collision load but not a transverse barge collision load applied to the existing pier. Only the complete removal of the piers down to the seal would allow the addition of piles and the construction of a pier footing with adequate reinforcement. Piers 8 and 11 do not have these issues. (See Figure 10A in Appendix A.)

10.3.2. River Piers for the Steel Box Girder Alternative

The pier concept for the steel box girder alternative would be very similar to that of the multiple steel girder alternative. The issues are very similar as well. (See Figure 11A in Appendix A.)

10.3.3. River Piers for the Post-Tensioned Concrete Box Girder Alternative

A hammerhead-shaped pier is not very useful for a post-tensioned concrete box girder. A single-shaft pier wall matching the width of the box girder soffit or twin walls would be much more suitable for the balanced cantilever construction. The single-shaft pier wall, due to its smaller cross section, would have a smaller impact on the river flow than a twin wall. On the other hand, temporary support towers would be needed for the balanced cantilever construction. A twin wall pier would provide the necessary stability for the balanced cantilever construction, but the impact on the river flow would be more pronounced.

The construction of the new southbound structure would require the complete removal of the existing piers down to the piles, the addition of new piles, and the construction of a footing with reinforcement meeting the LRFD design requirements. (See Figure 12A in Appendix A.)

10.4. Approach Pier Details

The general observations made in Section 8.4 for Option 1b apply to Option 3 as well. At the north end, the horizontal alignment for Option 3 does not impact the Gillette Building. The location of the new entrance and exit ramps at the north end leaves many of the existing columns in a useless location for the new piers. This is illustrated in Figures 61 and 62, which show the conditions at Piers 26 and 27.

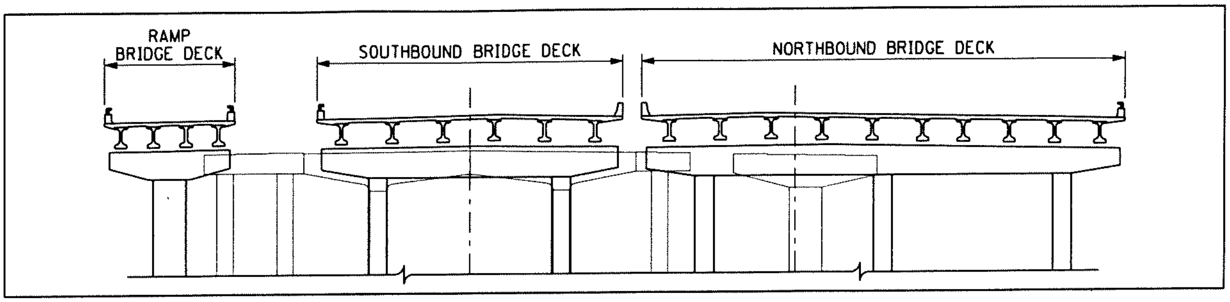

Figure 61. Option 3, Pier 26, Existing vs. Proposed Piers

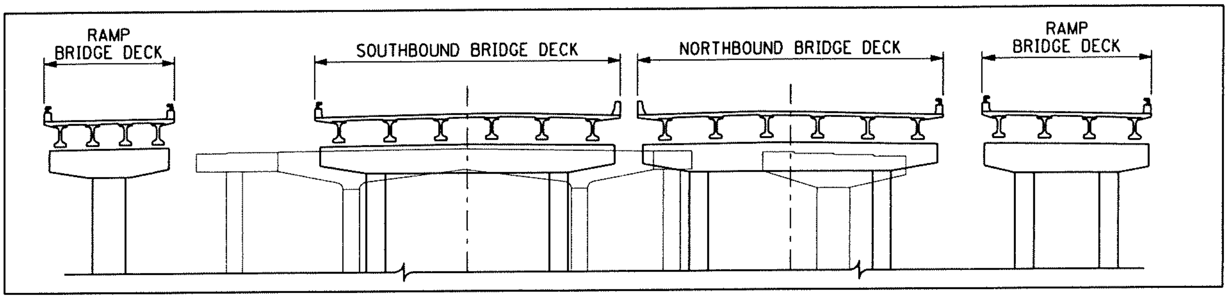

Figure 62. Option 3, Pier 27, Existing vs. Proposed Piers

10.5. River Span Superstructure Alternatives

The three river span superstructure alternatives were introduced earlier within the context of construction and traffic staging. In a following section, the superstructure alternatives were evaluated with the foundation requirements in mind. In this section, additional important characteristics of the three alternatives will be discussed.

The multiple steel girder alternative has the following features:

- The superstructure, in comparison with a concrete alternative, would be relatively light. The lesser weight would be beneficial, because fewer foundation piles would be required to support the bridge.

- The multi-girder system would afford great flexibility for construction staging.

- And deck geometry can be easily accommodated with multiple girders.

The steel box girder alternative has the following features:

- On the basis of the required number of lanes and shoulders, the bridge slab would be supported by two single-cell steel box girders with sloped webs of variable depth.

- In terms of weight, the steel box girder alternative is very similar to the multiple steel girder alternative.

- Erection and construction of the steel box girder alternative would be more challenging than the erection of multiple steel girders, but two single-cell boxes and external diaphragms at the piers would constitute a stable structural system.

- The magnitude of the deck slab cantilevers will have to be limited in order to prevent detrimental bending stresses in the outside web.

- Due to the large transverse span lengths between the girder flanges, the thickness of the deck slab would have to be variable.

The post-tensioned concrete box alternative has the following features:

- On the basis of the required number of lanes and shoulders, the bridge superstructure would consist of a single-cell concrete box girder with sloped webs of variable depth.

- In terms of weight, a post-tensioned concrete box girder superstructure would be significantly heavier than a steel girder superstructure. Due to the constraints on pile locations, the piles required for this alternative could only be installed if the existing pier concrete is removed.

- At the deck level, this alternative would allow the construction of a complete bridge without interference.

- A post-tensioned concrete box girder allows large cantilevers for the deck slab.

10.6. Approach Span Alternatives

For the approach spans, three alternatives had to be investigated: reusing the existing steel girders, new steel girders, and new prestressed concrete beams. The issues associated with reusing the existing girders are presented in Section 6. In this section, the remaining two alternatives will be discussed. The pier layout described in Section 10.4 would work for both alternatives. The framing plan layouts for both alternatives would pose no serious challenges. The sharply curved ramp structures would not be suitable for prestressed concrete beams.

10.7. River Span Transition Details

The existing river span superstructure is continuous and extends 40 feet beyond Piers 8 and 11. At the end of each 40-foot cantilever is a hinged support for the approach span girders. The girder depth near the piers is variable. The girder depths of the river spans and those of the approach spans are significantly different. Typically, there is a 70-foot parabolic transition of the web depth on each side of a pier, with the exception of the 40-foot cantilevered side. The cantilever portion is shaped to produce a smooth transition of the girder depth.

From an aesthetic point of view, the proper choice of the transition location is very important. Thereby, a variable-depth superstructure would serve a dual function. It would enhance the appearance of the main river spans by shaping the underside of the bridge like intrados of an arch. Equally important, it would help to minimize the depth differential between the constant-depth approach spans and the river spans.

10.8. Conclusions

Option 3 would work for all three river span superstructure alternatives: the multiple steel girder alternative, the steel box alternative, and the post-tensioned concrete box girder alternative. For the approach spans, all three alternatives would work. A very important consideration is that the roadway alignment of Option 3 would impact the tall masts of the 115kV power line. Neither mast would need to be relocated, nor would the current vertical clearance be worsened by the new construction.

The following paragraphs list the pros and cons for each river span alternative:

The multiple steel girder alternative for the river spans offers many advantages:

- The bridge superstructure would be a redundant system.
- The superstructure weight would be less than that of a comparable concrete structure, which would result in fewer piles.
- Any deck geometry could be easily accommodated.
- The transition details for the approach spans would work well for a variable-depth girder system.
- Replacement of the bridge decks would be fairly easy.

Its disadvantages are:

- Unless the existing piers in the Mississippi River (Piers 9 and 10) are completely removed and rebuilt, they would not meet the requirements of the LRFD design code.
- Anticipated maintenance costs would be higher.

The steel box girder alternative for the river spans offers some advantages:

- The superstructure weight would be less than that of a comparable concrete structure, which would result in fewer piles.
- Any deck geometry could be accommodated.
- The transition details for the approach spans would work well for a variable-depth girder system.
- Replacement of the bridge decks would be fairly easy.

Its disadvantages are:

- Unless the existing piers in the Mississippi River (Piers 9 and 10) are completely removed and rebuilt, they would not meet the requirements of the LRFD design code.
- Anticipated maintenance costs would be higher.

The post-tensioned box girder alternative for the river spans offers many advantages:

- The bridge superstructure would be a redundant system.
- Anticipated maintenance costs would be lower in comparison with steel structures.
- The transition details for the approach spans would work well for a variable-depth girder system.

Its disadvantages are:

- Unless the existing piers in the Mississippi River (Piers 9 and 10) are completely removed and rebuilt, they would not meet the requirements of the LRFD design code.
- Replacement of the bridge decks would be very challenging because of the presence of post-tensioned forces, both longitudinally and transversely.
- Form traveler clearance requirements will have to be considered with respect to the navigation channel.
- The deck width for a single-cell box girder is limited without providing stiffening elements (ribs) for the slab portions in the compression zone.
- The superstructure weight would be greater than that of a comparable steel structure, which would results in more piles.

APPENDIX A
Appendix Figures

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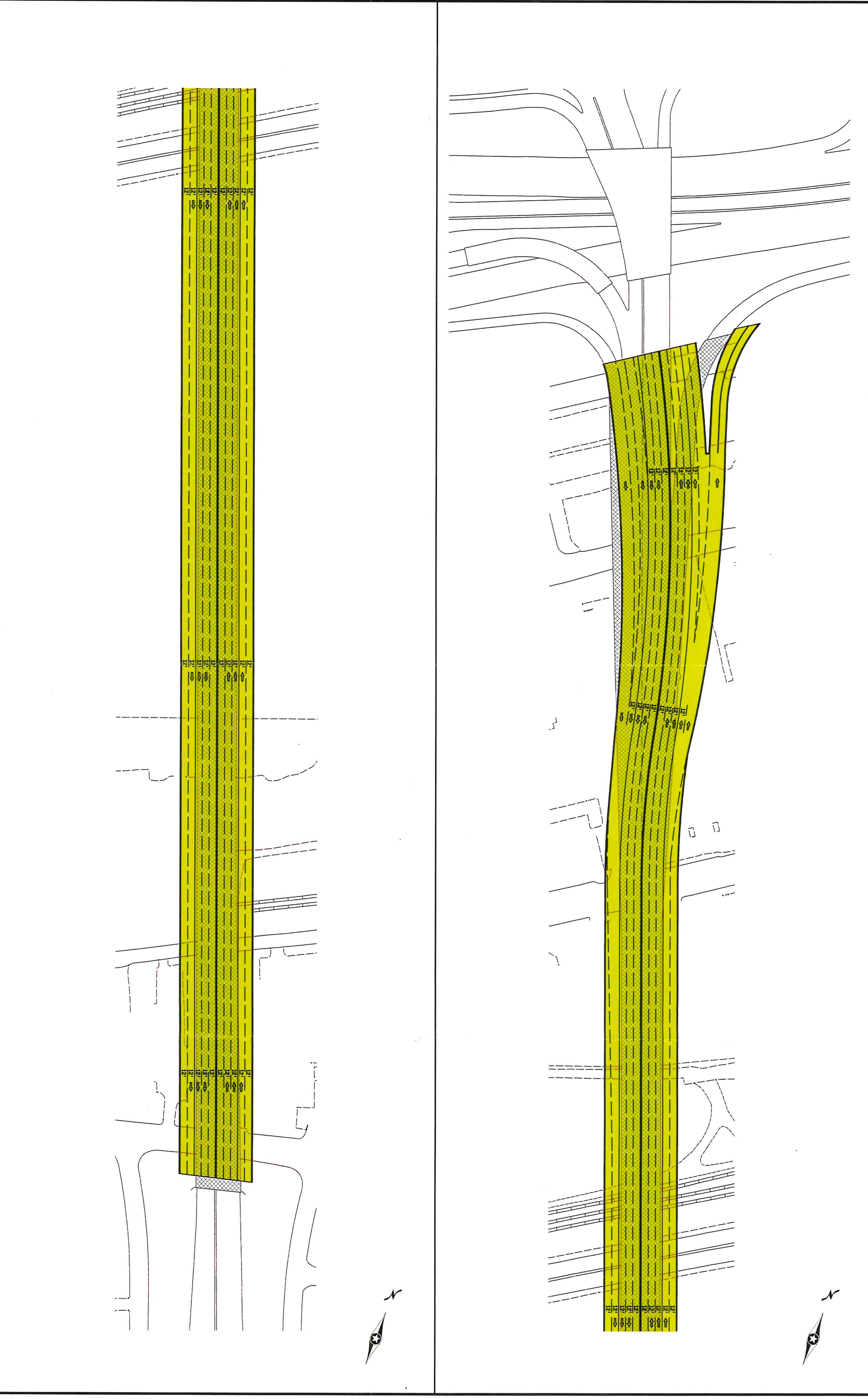

MINNESOTA DEPARTMENT
OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE:
BRIDGE LAYOUT FOR OPTION 1B

FIGURE 1A

Bridge No.
9800

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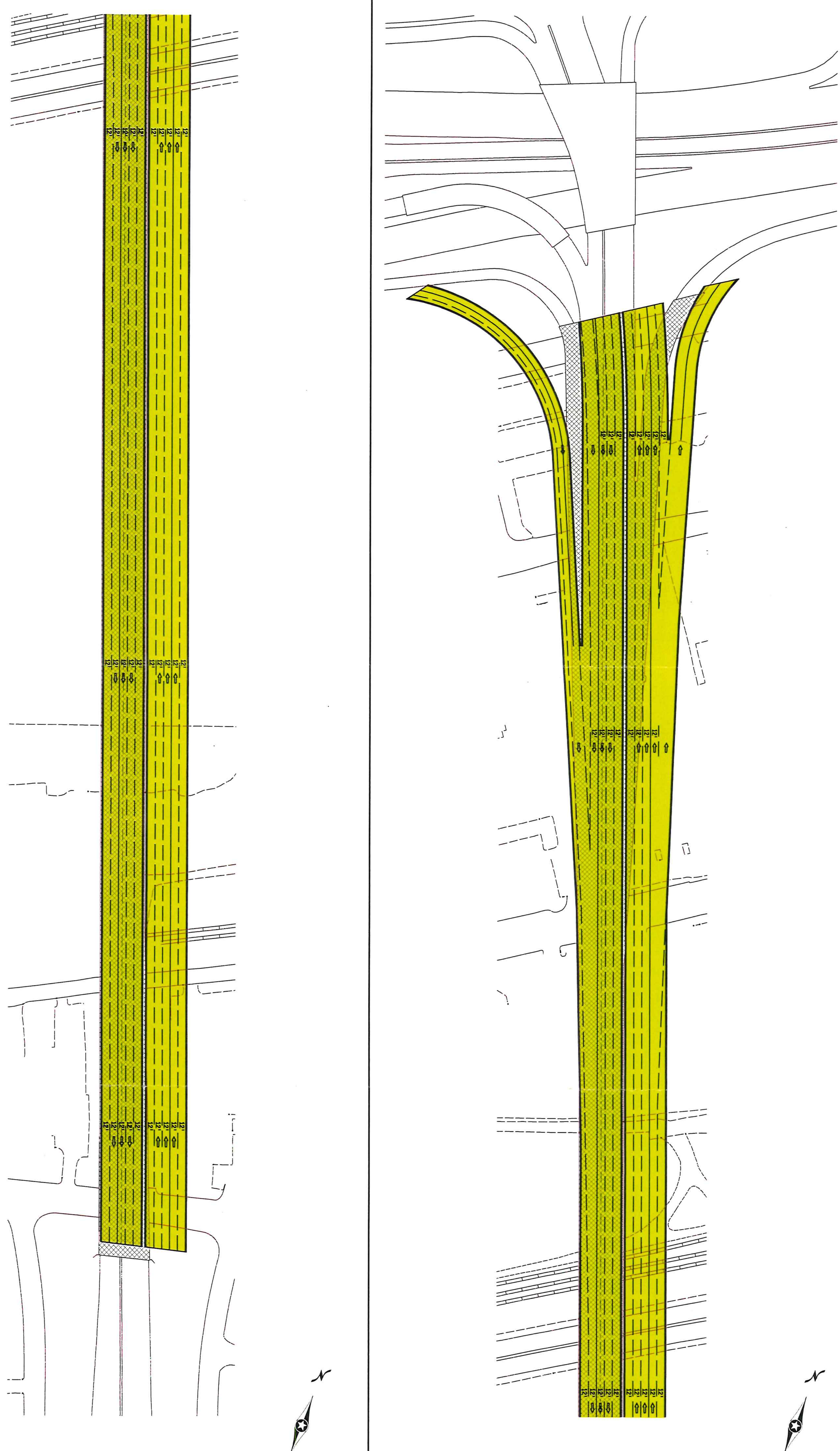

MINNESOTA DEPARTMENT
OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE:
BRIDGE LAYOUT FOR OPTION 3

FIGURE 2A

Bridge No.
9800

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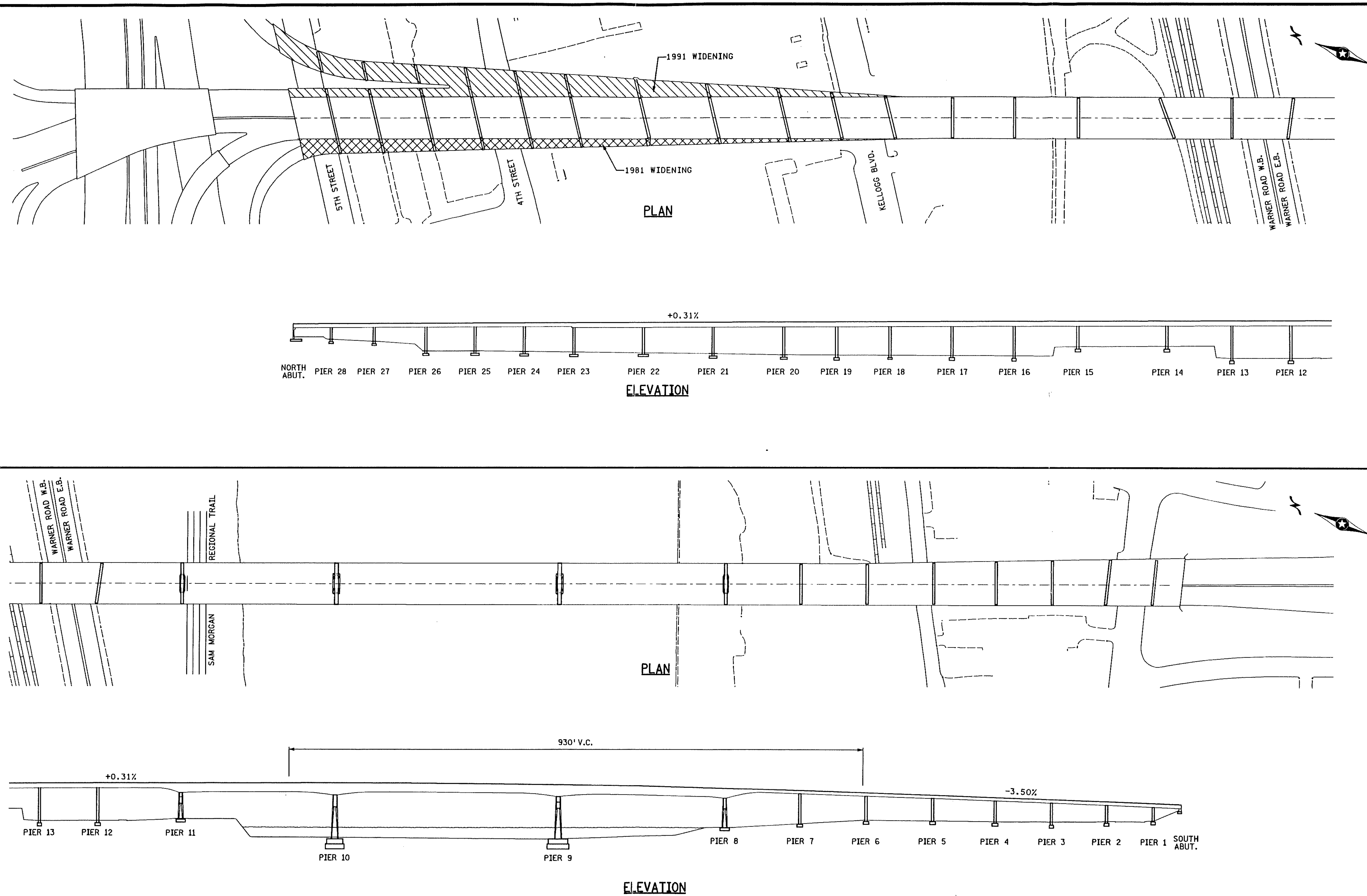

MINNESOTA DEPARTMENT OF TRANSPORTATION LAFAYETTE BRIDGE STUDY	TITLE: PLAN AND ELEVATION OF EXISTING BRIDGE	FIGURE 3A	Bridge No. 9800

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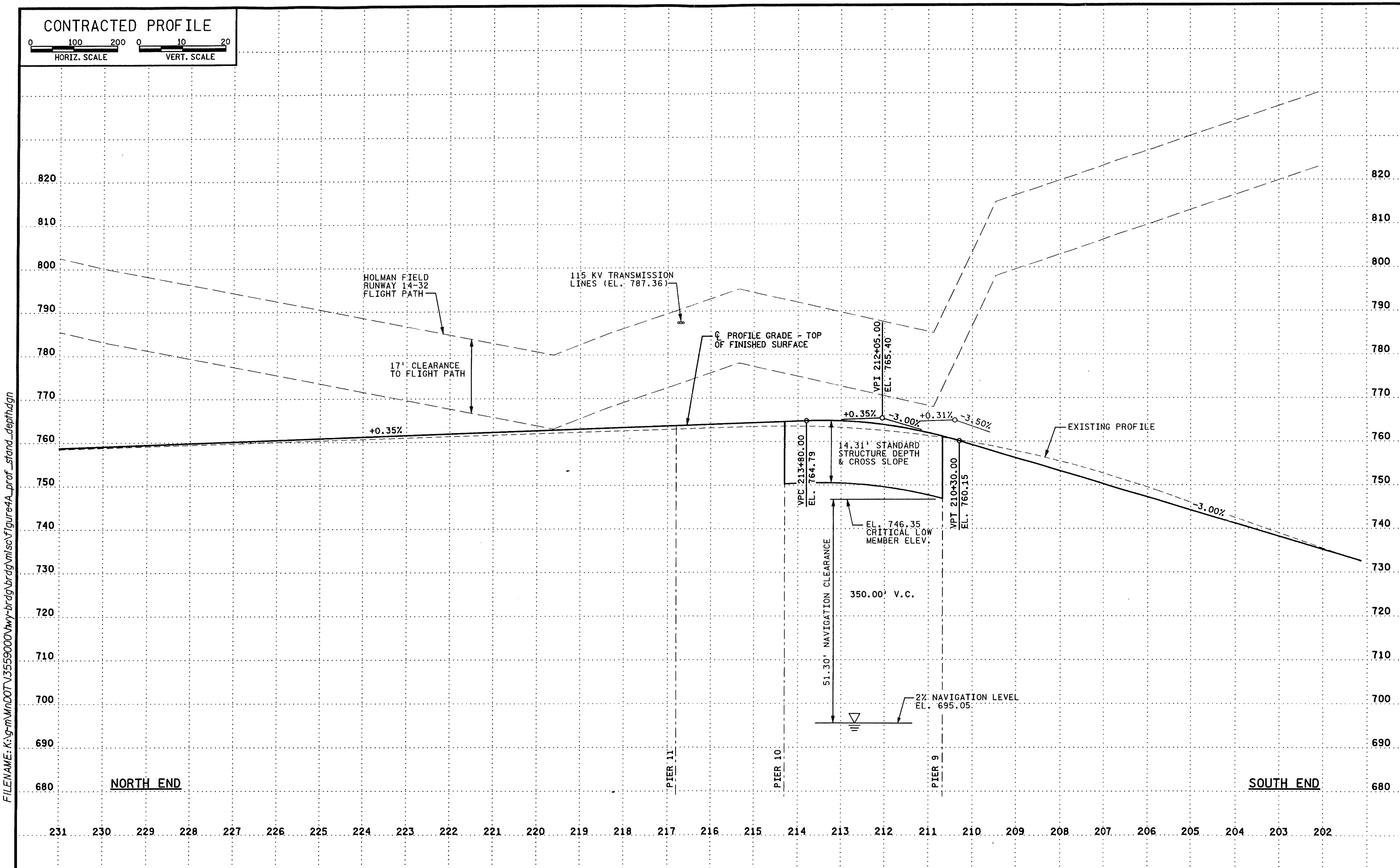

MINNESOTA DEPARTMENT OF TRANSPORTATION LAFAYETTE BRIDGE STUDY	TITLE: VERTICAL PROFILE CONSTANT-DEPTH RIVER SPAN	FIGURE 4A	Bridge No. 9800

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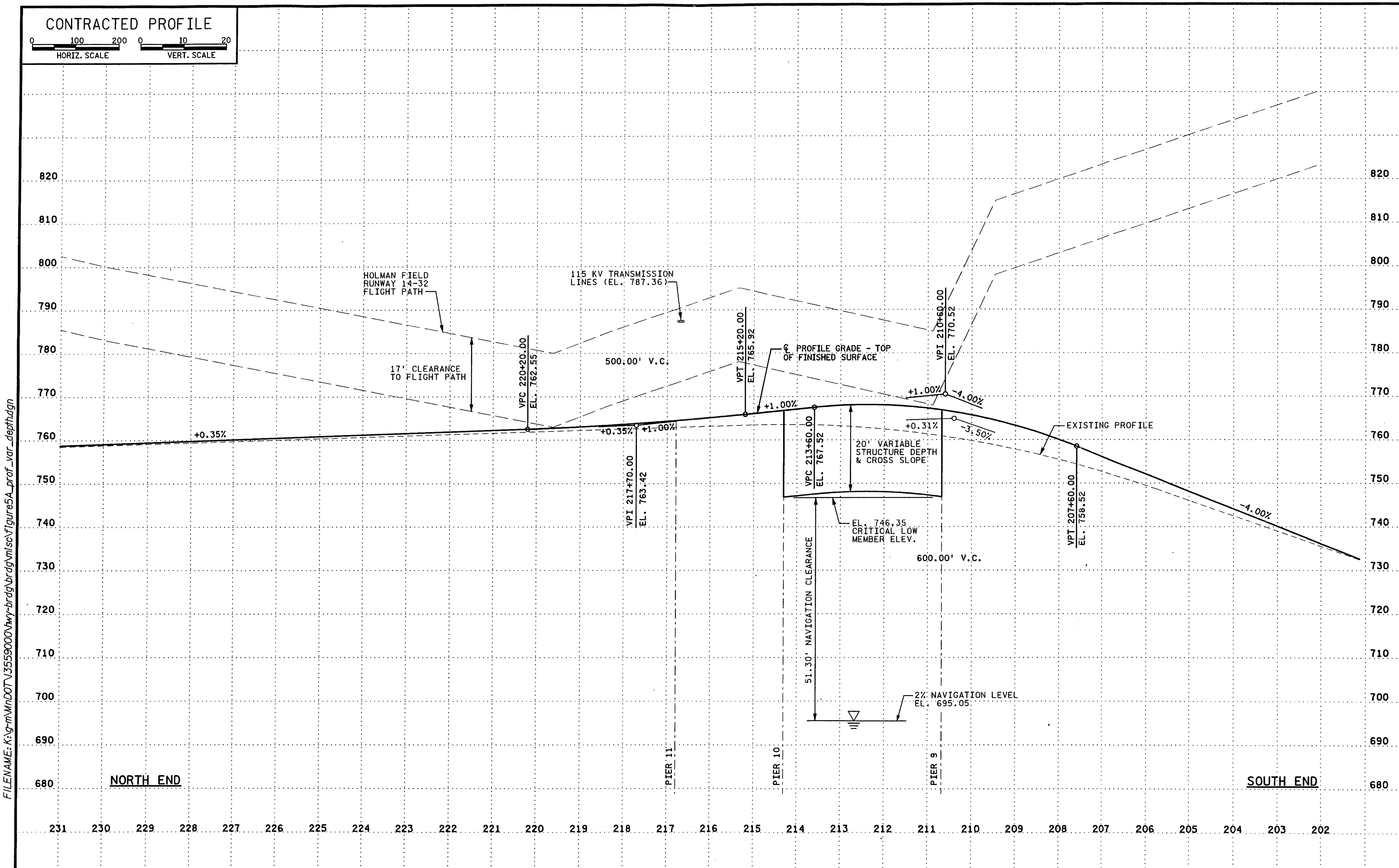

MINNESOTA DEPARTMENT OF TRANSPORTATION LAFAYETTE BRIDGE STUDY	TITLE: VERTICAL PROFILE VARIABLE-DEPTH RIVER SPAN	FIGURE 5A	Bridge No. 9800

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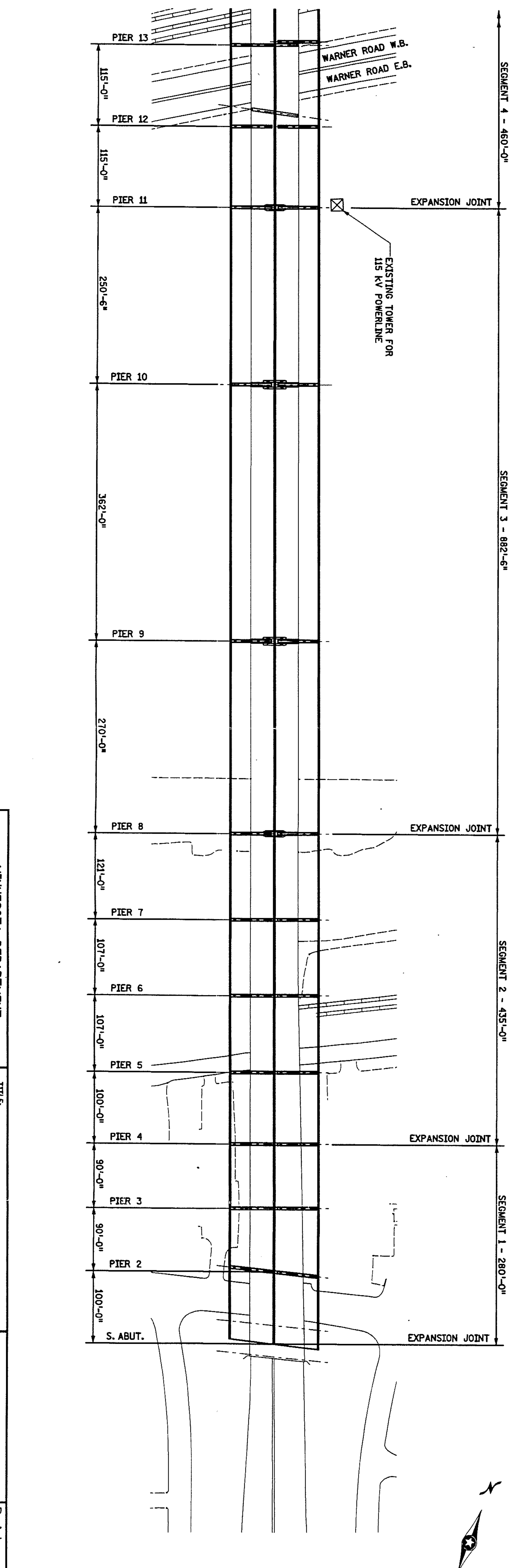

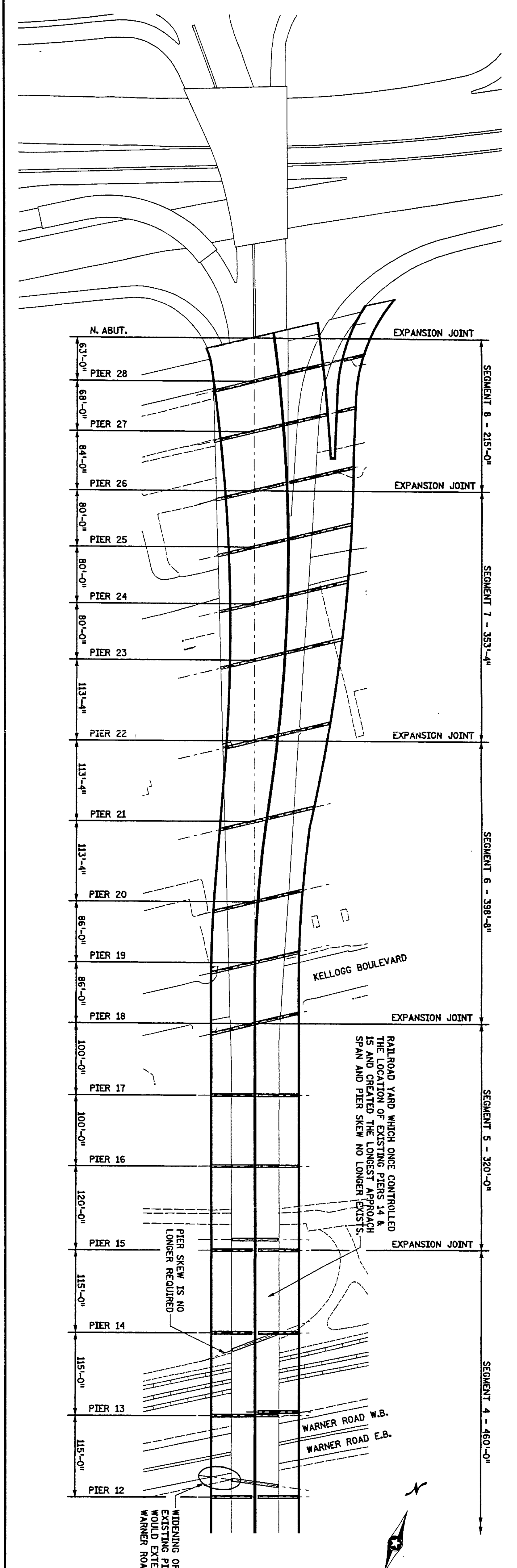

MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: PIER LAYOUT FOR OPTION 1B

FIGURE 6A

Bridge No. 9800

ALL FOUNDATION CONSTRUCTION TO BE ACCOMPLISHED WITHIN A

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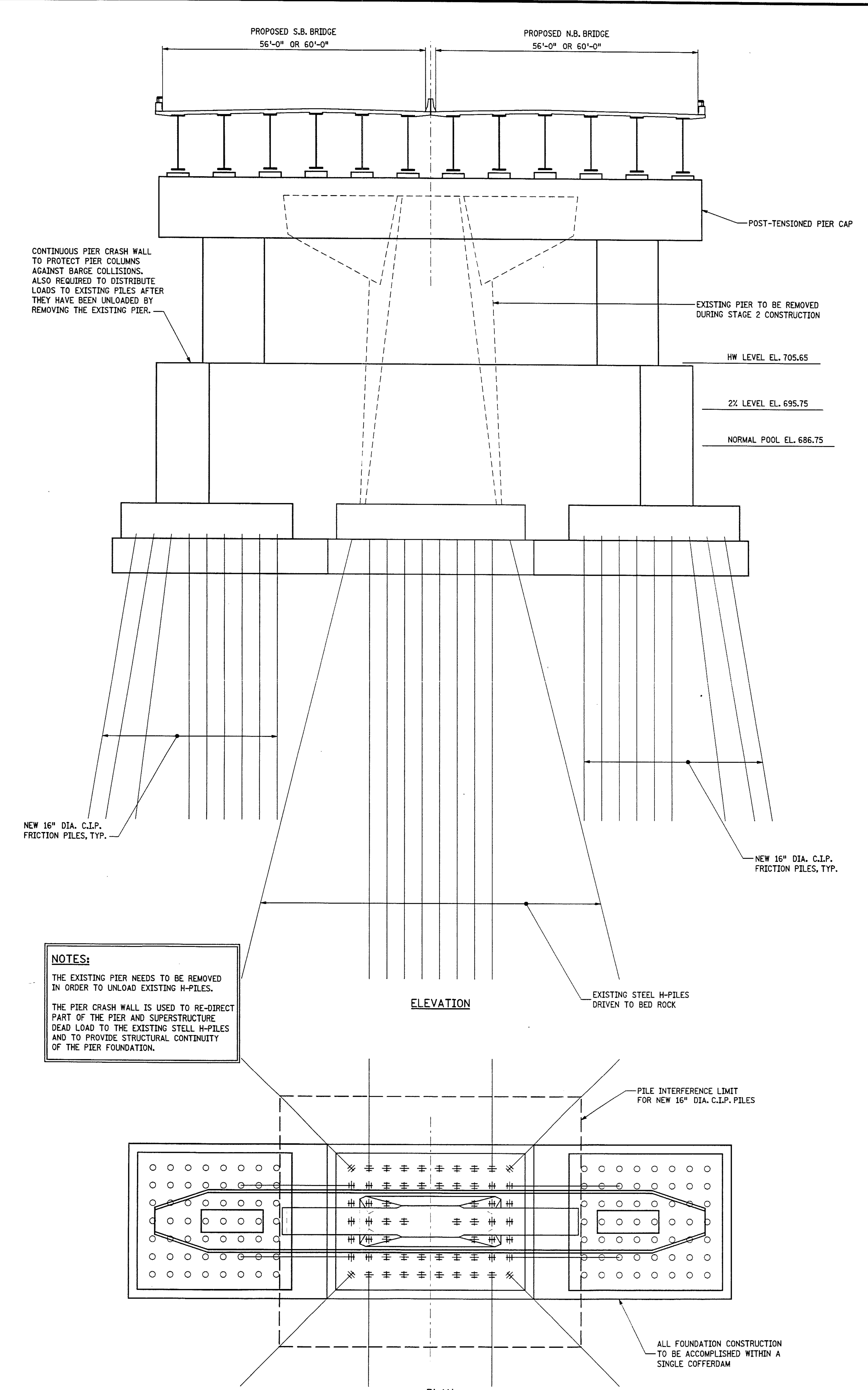

MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: OPTION 1B
RIVER PIER CONSTRUCTION
STEEL GIRDER ALTERNATIVE

FIGURE 7A

Bridge No. 9800

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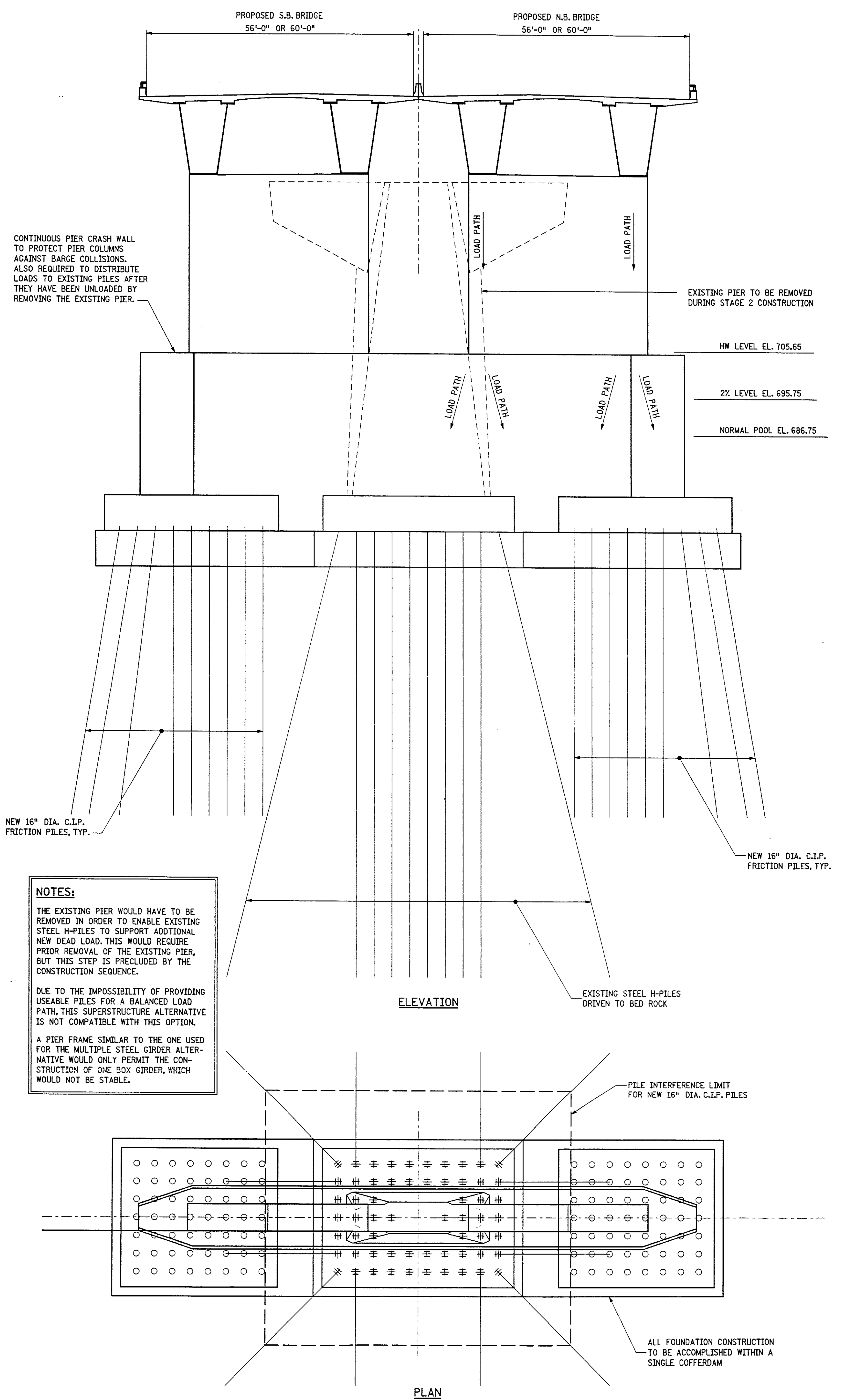

NOTES:

THE EXISTING PIER WOULD HAVE TO BE REMOVED IN ORDER TO ENABLE EXISTING STEEL H-PILES TO SUPPORT ADDTIONAL NEW DEAD LOAD. THIS WOULD REQUIRE PRIOR REMOVAL OF THE EXISTING PIER, BUT THIS STEP IS PRECLUDED BY THE CONSTRUCTION SEQUENCE.

DUE TO THE IMPOSSIBILITY OF PROVIDING USEABLE PILES FOR A BALANCED LOAD PATH, THIS SUPERSTRUCTURE ALTERNATIVE IS NOT COMPATIBLE WITH THIS OPTION.

A PIER FRAME SIMILAR TO THE ONE USED FOR THE MULTIPLE STEEL GIRDER ALTERNATIVE WOULD ONLY PERMIT THE CONSTRUCTION OF ONE BOX GIRDER, WHICH WOULD NOT BE STABLE.

MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: OPTION 1B
RIVER PIER CONSTRUCTION
STEEL BOX ALTERNATIVE

FIGURE 8A

Bridge No. 9800

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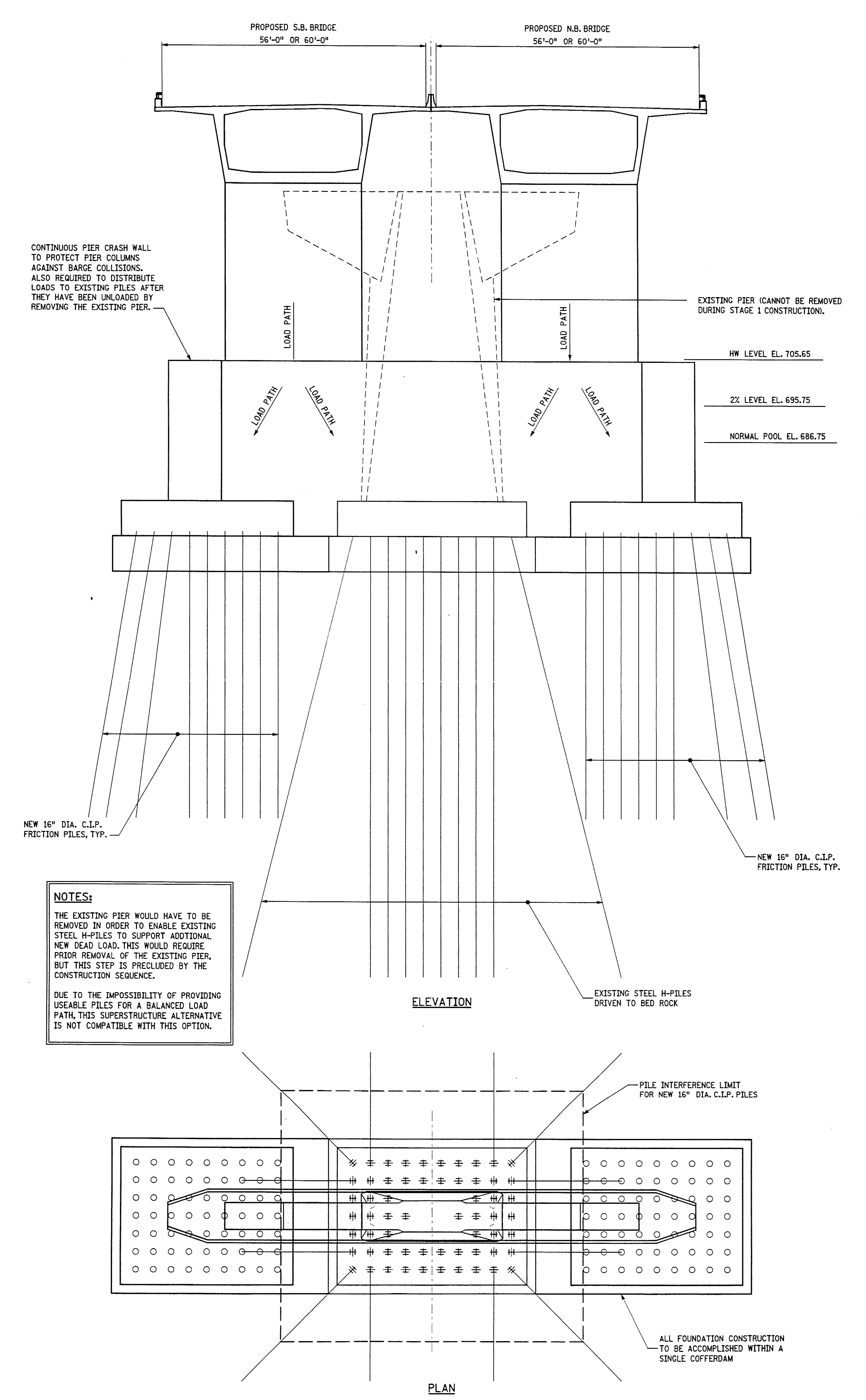


NOTES:

THE EXISTING PIER WOULD HAVE TO BE REMOVED IN ORDER TO ENABLE EXISTING STEEL H-PILES TO SUPPORT ADDTIONAL NEW DEAD LOAD. THIS WOULD REQUIRE PRIOR REMOVAL OF THE EXISTING PIER, BUT THIS STEP IS PRECLUDED BY THE CONSTRUCTION SEQUENCE.

DUE TO THE IMPOSSIBILITY OF PROVIDING USEABLE PILES FOR A BALANCED LOAD PATH, THIS SUPERSTRUCTURE ALTERNATIVE IS NOT COMPATIBLE WITH THIS OPTION.

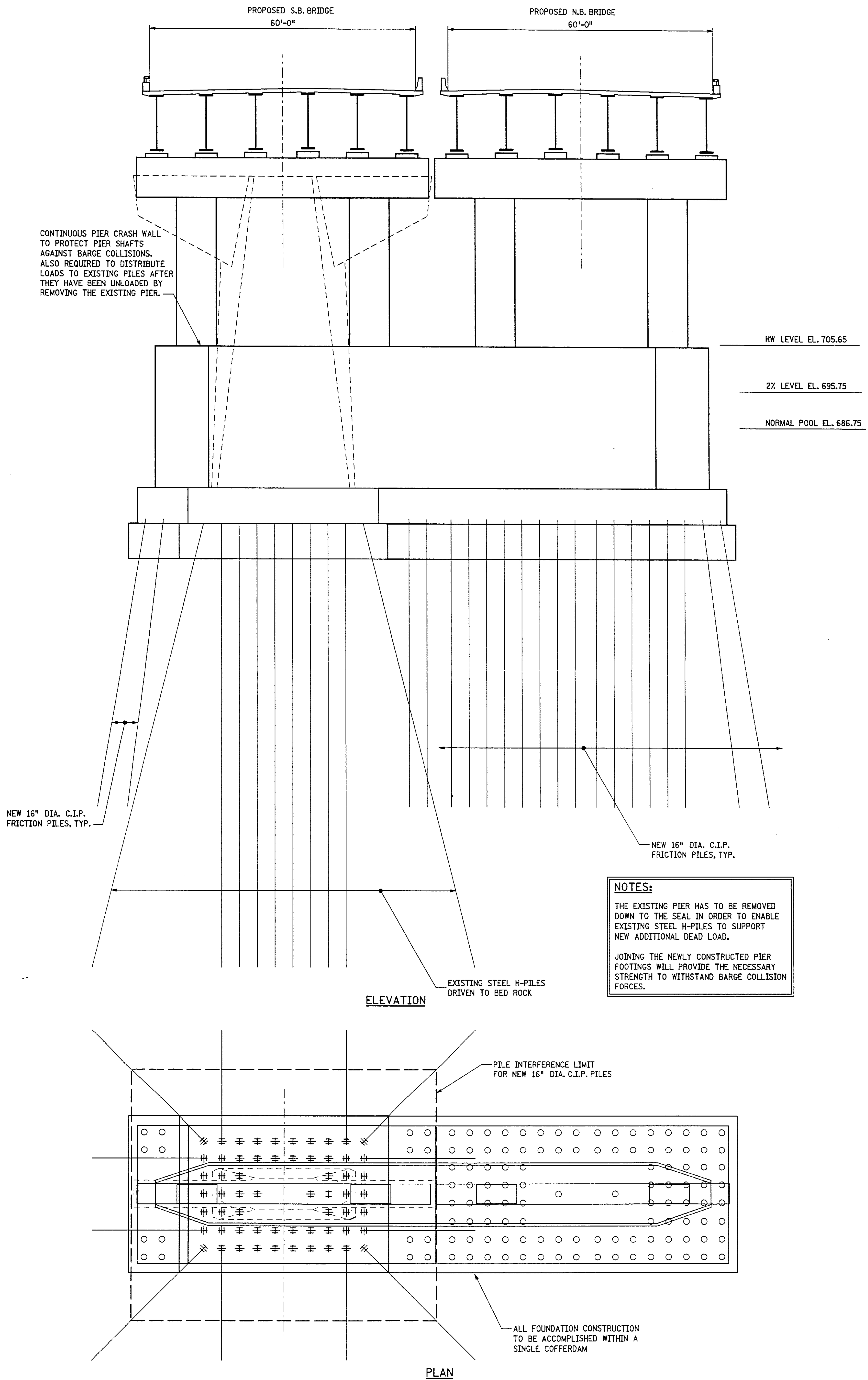
MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: OPTION 1B
RIVER PIER CONSTRUCTION
PT CONCRETE BOX ALTERNATIVE

FIGURE 9A

Bridge No. 9800

DATE: 2/28/2007 TIME: 1:47:58 PM
FILENAME: K:\g-m\MnDOT\3559000\hwy-brdg\brdg\misc\figure10A_riverpier_steelgirder_opt3.dgn



NOTES:

THE EXISTING PIER HAS TO BE REMOVED DOWN TO THE SEAL IN ORDER TO ENABLE EXISTING STEEL H-PILES TO SUPPORT NEW ADDITIONAL DEAD LOAD.

JOINING THE NEWLY CONSTRUCTED PIER FOOTINGS WILL PROVIDE THE NECESSARY STRENGTH TO WITHSTAND BARGE COLLISION FORCES.

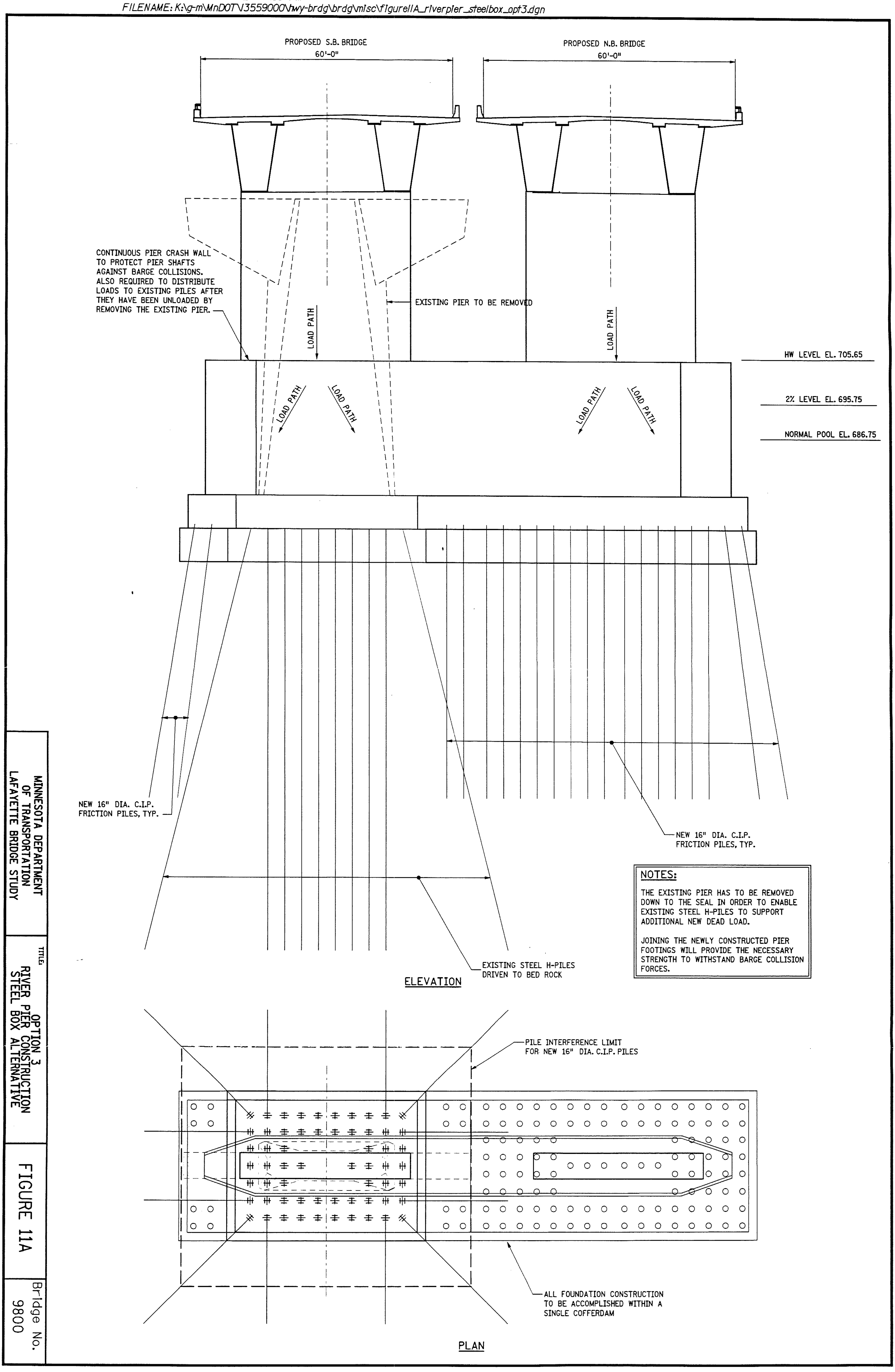
MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: OPTION 3
RIVER PIER CONSTRUCTION
STEEL GIRDER ALTERNATIVE

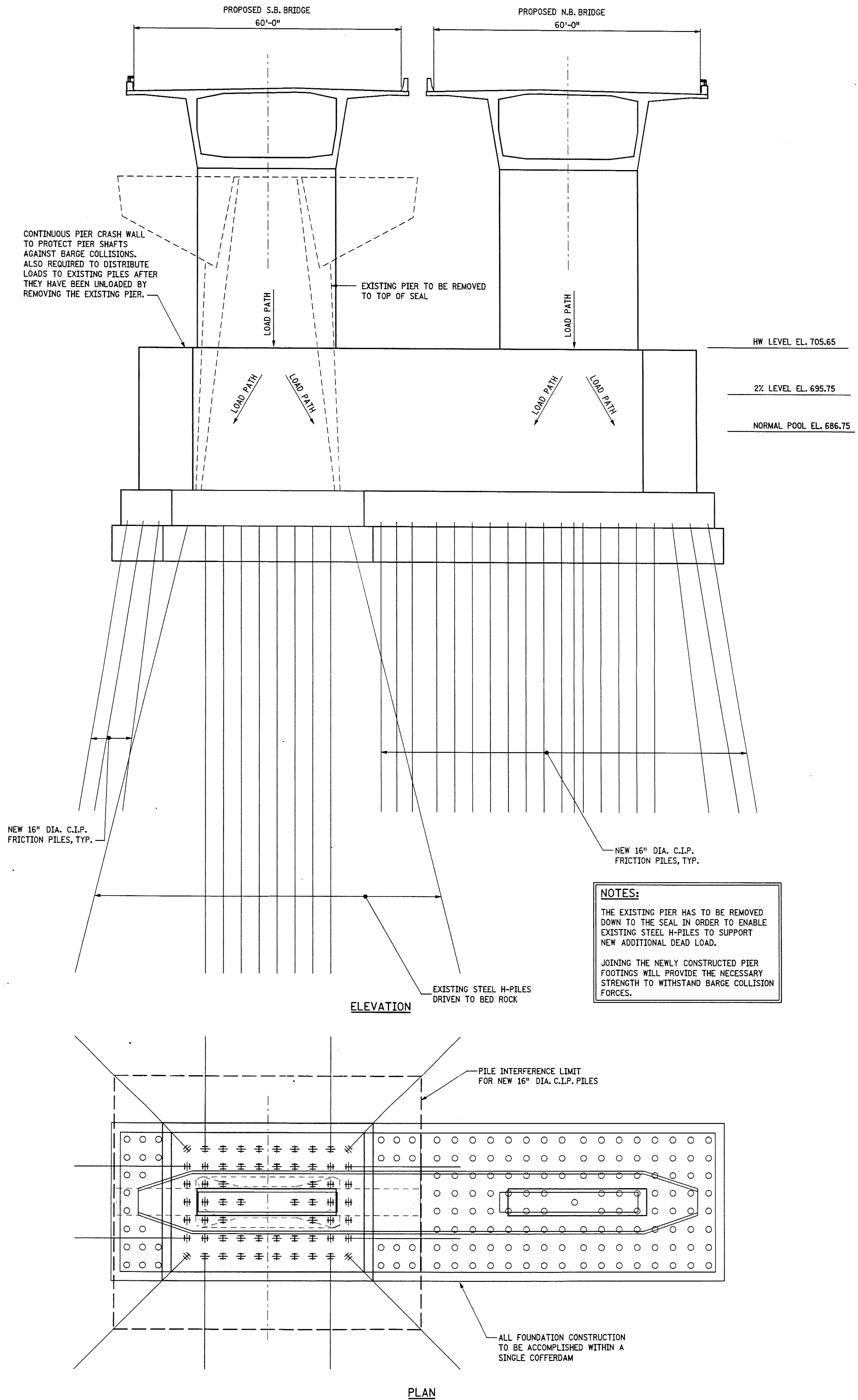
FIGURE 10A

Bridge No. 9800

DATE: 2/28/2007 TIME: 1:47:59 PM
FILENAME: K:\g-m\MnDOT\3559000\hwy-brdg\brdg\misc\figure11A_riverpier_steelbox_opt3.dgn



DATE: 2/28/2007 TIME: 1:48:00 PM
FILENAME: K:\g-m\MnDOT\3559000\hwy-brdg\brdg\misc\figure12A_riverpier_concretebox_opt3.dgn



NOTES:

THE EXISTING PIER HAS TO BE REMOVED DOWN TO THE SEAL IN ORDER TO ENABLE EXISTING STEEL H-PILES TO SUPPORT NEW ADDITIONAL DEAD LOAD.

JOINING THE NEWLY CONSTRUCTED PIER FOOTINGS WILL PROVIDE THE NECESSARY STRENGTH TO WITHSTAND BARGE COLLISION FORCES.

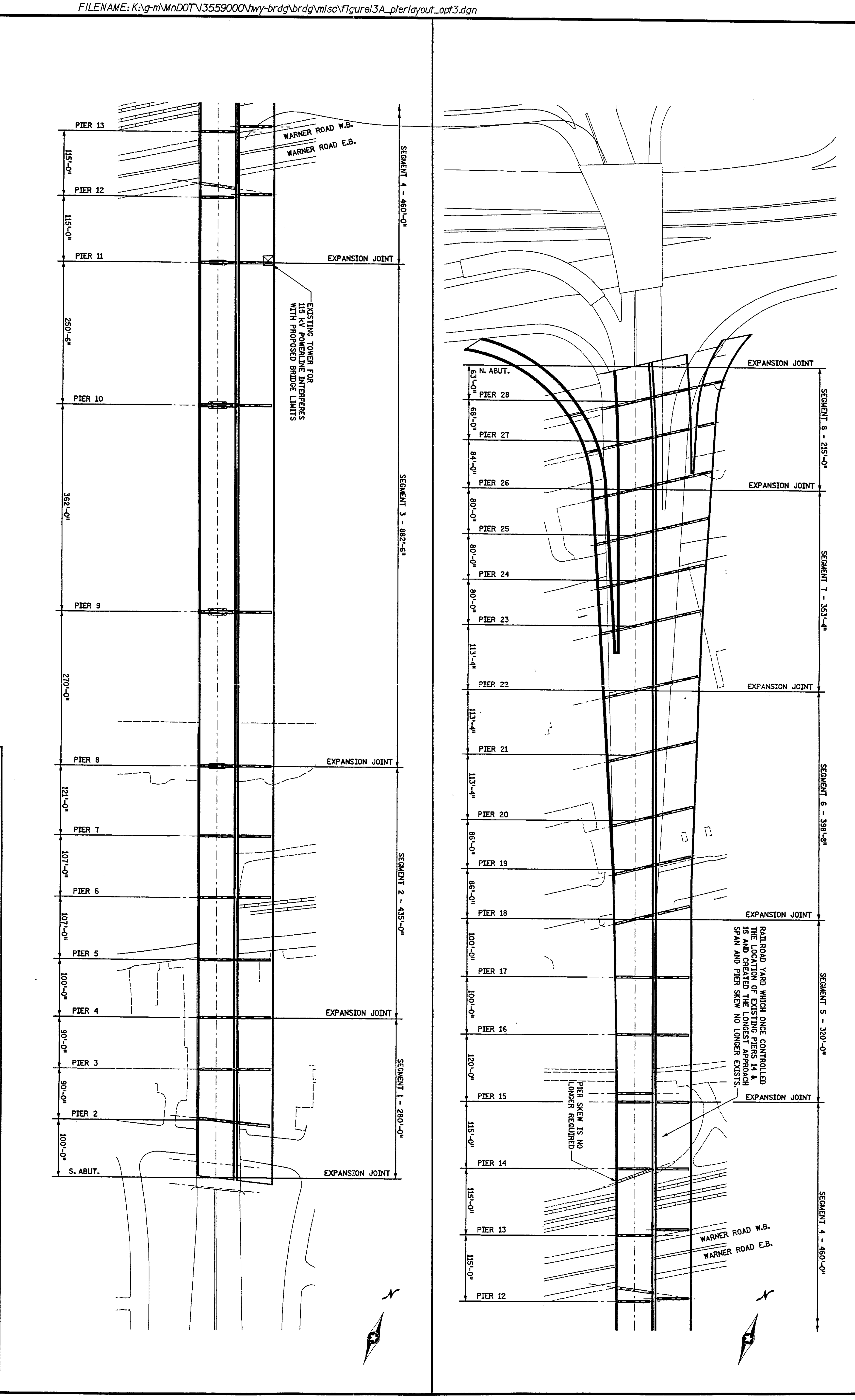
MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: OPTION 3
RIVER PIER CONSTRUCTION
PT CONCRETE BOX ALTERNATIVE

FIGURE 12A

Bridge No. 9800

DATE: 2/28/2007 TIME: 1:48:01 PM
FILENAME: K:\g-m\MnDOT\J3559000\hwy-brdg\brdg\misc\figure13A_pierlayout_opt3.dgn



MINNESOTA DEPARTMENT OF TRANSPORTATION
LAFAYETTE BRIDGE STUDY

TITLE: PIER LAYOUT FOR OPTION 3

FIGURE 13A

Bridge No. 9800

APPENDIX B

The ASD Code Versus the LRFD Design Code

Structural Study of Existing Lafayette Bridge No. 9800
(TH 52 Over the Mississippi River in Saint Paul, Minnesota)
Minnesota Department of Transportation
S.P. 6244 9800 (Study)
Mn/DOT Agreement No. 86425

APPENDIX B
The ASD Code Versus the LRFD Design Code Applied to Pier Footings

1. General Observations

The concept of a "safe" design is approached very differently by the ASD and LRFD codes. The older ASD code, as its name implies, operated with allowable stresses. An allowable stress in turn was determined on the basis of a factor of safety, which was applied to a stress level associated with a nominal structural failure. As a rule, the design loads were not adjusted for variability. All loads, except for wind loads in certain load combinations, had a multiplier of 1.0; i.e., they were interpreted as service loads. The capacity of a structural member for each governing load case had to be checked on the basis of allowable stresses.

The ASD code also accounted for the unlikelihood that all loads of a load combination would occur simultaneously at their peak level. For such load combinations an "overstress" provision was permissible and the allowable base stress could be exceeded by a certain percentage (25%, 33%, 40% or 50%).

The newer LRFD code is based on the theory of probability. It distinguishes different limit state categories: strength limit states, service limit states, extreme event limit states, and fatigue limit states. For all load combination limit states, the design loads are multiplied by a load factor, which is typically equal to or greater than 1.0. The nominal structural capacity of a structural component is multiplied by a resistance factor, which is typically equal to or smaller than 1.0. The load factors depend on the load combination limit state and have been determined on the basis of probability. Resistance factors depend on the type of structural resistance mechanism. Thus, the LRFD code attempts to address the variability of the loads in specific load combinations in a much more rigorous fashion than the overstress provision of the ASD code.

2. Differences in Pier Live Loads (Service Load Level)

Since the methodologies of the two codes are different, the results can be expected to be different as well. The differences in the applied live loads for a superstructure design are not as pronounced as they are for a pier, since the ASD code provided for special live loads in the negative moment region of a continuous girder. In the case of a pier design, the differences are pronounced. The respective live loads at the service load level compare as follows:

Pier Reactions for Two 100-Foot-Long Simple Spans				
		2 Lanes	**3 Lanes**	**4 Lanes**
ASD Code:				
	HS-20 Lane Load	180.0	243.0	270.0
	HS-20 Truck Load	130.6	176.3	195.8
	Governing Live Load	180.0	243.0	270.0
LRFD Code:				
	HL-93 Loading	273.3	348.5	355.3
Ratio LRFD vs. ASD:		**1.52**	**1.43**	**1.32**

A typical calculation for the values in the previous table is shown below:

LIVE LOAD REACTION AT PIERS:

(Assume Two 100-ft. Simple Spans) $L_{SPAN} := 100.00ft$ $N_L := 2$

ASD Code:

$F_{LR} := 1.0$

HS-20 Lane Load:

$$R_{PIER\_L} := N_L \cdot \left(0.640 \frac{kip}{ft} \cdot L_{SPAN} + 26.0kip\right)$$

$R_{PIER\_L} = 180.0\,kip$

HS-20 Truck Load:

$$R_{PIER\_T} := N_L \cdot \left(32.0kip + 32.0kip \cdot \frac{86.00ft}{100.00ft} + 8.0kip \cdot \frac{72.00ft}{100.00ft}\right)$$

$R_{PIER\_T} = 130.6\,kip$

Governing Live Load:

$$R_{PIER\_HS} := \max(R_{PIER\_L}, R_{PIER\_T})$$

$R_{PIER\_HS} = 180.0\,kip$

LRFD Code:

HL-93 Load: $MPF := 1.0$ $\gamma_L := 1.0$ *(Service Load Factor)*

(Lane Load, plus two Trucks, 50 feet apart)

$$R_{PIER\_HL} := N_L \cdot MPF \cdot \gamma_L \cdot (0.9) \cdot \left[0.64 \frac{kip}{ft} \cdot L_{SPAN} + \left[32.0kip \cdot \left(\frac{100.0ft + 86.0ft}{100.0ft}\right) + 8.0kip \cdot \frac{72.0ft}{100.0ft}\right] \ldots \atop + 8.0kip \cdot \frac{50.0ft}{100.0ft} + 32.0kip \cdot \left(\frac{36.0ft + 22.0ft}{100.0ft}\right)\right]$$

$R_{PIER\_HL} = 273.3\,kip$

The 100-foot spans are representative values for the approach spans. The differences between the codes are largest when only two lanes are loaded. Such a load case produces a live load that is 52% higher. When four lanes are loaded, the difference is 32%. It is important to consider that the differences would be smaller for HS-25 live loads. However, the original design used HS-20 live load.

For the river spans of the Lafayette Bridge (span lengths of 362'-0" and 270'-0"):

Pier Reactions for 362-Foot and 270-Foot-Long Simple Spans				
		2 Lanes	**3 Lanes**	**4 Lanes**
ASD Code:				
	HS-20 Lane Load	456.5	616.2	684.7
	HS-20 Truck Load	140.3	189.4	210.4
	Governing Live Load	456.5	616.2	684.7
LRFD Code:				
	HL-93 Loading	586.9	748.3	763.0
Ratio LRFD vs. ASD:		**1.29**	**1.21**	**1.11**

3. Effects of Asymmetrical Dead Loads

Since the LRFD code focuses on limit states, the effects of asymmetrical dead load is recognized implicitly by the code. A larger load factor, $\gamma_p = 1.50$ for limit state Strength IV, accounts for particularly adverse or asymmetrical loads during construction. The ASD code had no comparable provisions.

4. The Use of Grade 40 Reinforcement

The existing piers of the Lafayette Bridge were reinforced with uncoated Grade 40 reinforcement. The allowable tensile stress in the reinforcement was 20,000 pounds per square inch. In contrast, Grade 60 reinforcement would have had an allowable tensile stress of 24,000 pounds per square inch. Thus, the ratio of yield strength versus allowable tensile strength is 2.0 for Grade 40 reinforcement and 2.5 for Grade 60 reinforcement. These ratios show that, other things being equal, an ASD design with Grade 40 reinforcement has a smaller safety cushion than an ASD design with Grade 60 reinforcement.

The piers were designed for all required load combinations of the code applicable at that time. With the exception of load case I, all other load cases permitted overstresses ranging from 25% to 50%. Barge collision forces were not considered for the foundation design.

5. Differences in the Footing Design Methodology

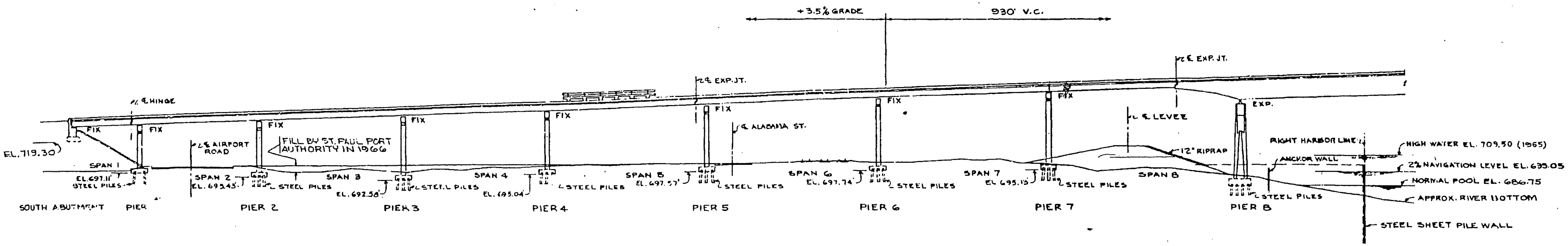
The typical ASD design used the moment and shear design methodology to design pier footings. This approach assumes that the footing will behave like a beam. It ignores the fact that in most cases the design region is a "disturbed" region and not a "Bernoulli" region. At each pile location, a large concentrated force enters the footing. The force flow caused by these large concentrated loads is different from the force flow assumed by the beam analogy. Furthermore, the beam analogy also reduces a three-dimensional problem to a two-dimensional one.

The combined effects of these phenomena result in a decrease in the factor of safety. This decrease is exposed when a strut-and-tie model in conjunction with the LRFD code is applied.

LRFD design checks for the existing footings on the basis of a strut-and-tie model show that the footing reinforcement is inadequate when piles are loaded to their LRFD design capacity. The most pronounced inadequacies occur at the footing corners, where the column loads enter the corner piles. Since this is the region where the highest pile loads occur, this is also the most critical area. The compression struts transferring the column loads to the individual piles produce tensile stresses in the footing reinforcement. In the case of corner piles, the tensile stresses need to be resolved with respect to the direction of both the longitudinal and the transverse reinforcement. Since the classical moment design method neglects the three-dimensional nature of the force flow, the provided transverse reinforcement is typically inadequate.

In addition to the inadequacies of the reinforcement at the footing corners, the main reinforcement provided for piles located along the footing of the river pier perimeter does not meet LRFD design requirements.

APPENDIX C
Existing Plans of the Lafayette Bridge

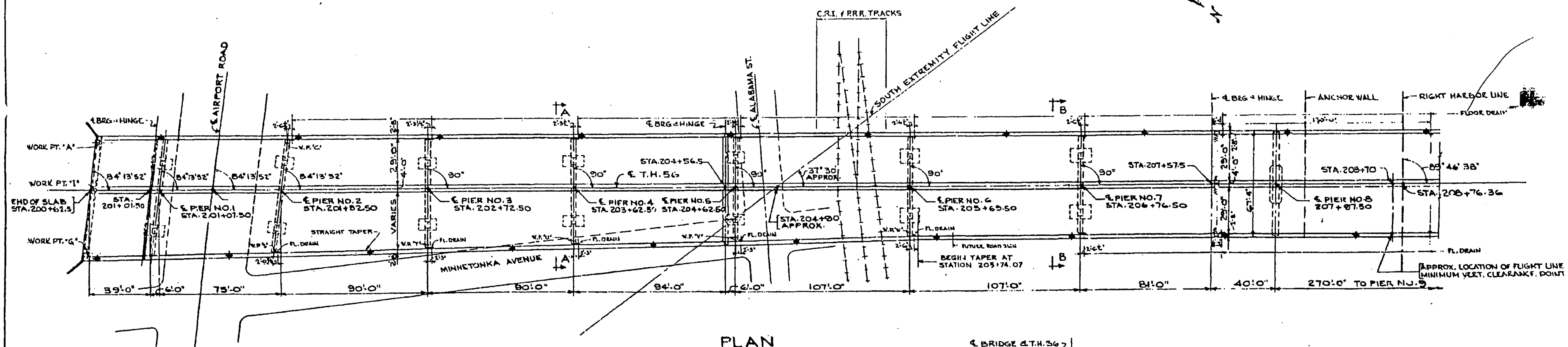


ELEVATION

~~SEGMENT NO. 3~~ SOUTH PORTION

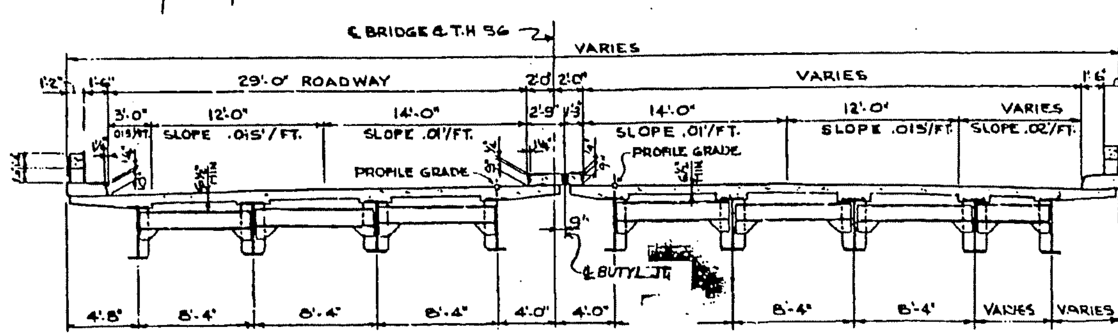
SCALE: 1" = 30'

(LIGHT STANDARDS NOT SHOWN)



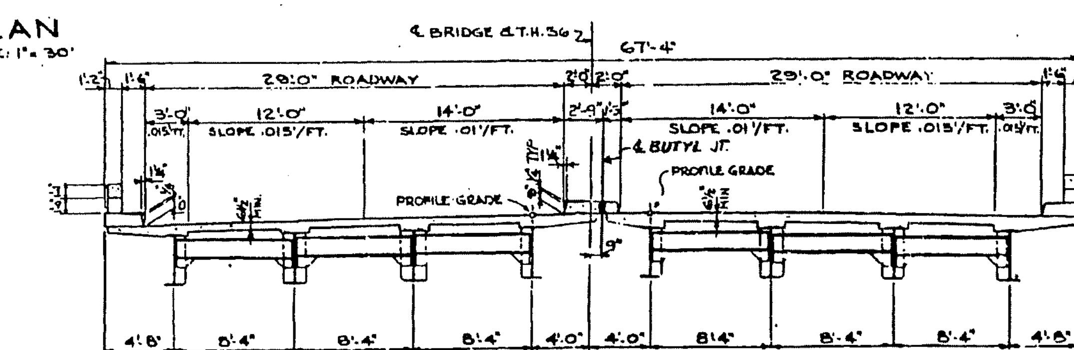
PLAN

SCALE: 1" = 30'



TRANSVERSE SECTION A-A THRU BRIDGE DECK

SCALE: 1" = 6'-0"



TRANSVERSE SECTION B-B THRU BRIDGE DECK

SCALE: 1" = 6'-0"

NOTE: SLAB THICKNESS VARIES WHERE BEAM SPACING DIFFERS FROM 8'-4". SEE SUPERSTRUCTURE SH. NO. 81.

PREPARED BY
ELLERBE & COMPANY
ARCHITECTS - ENGINEERS
ST. PAUL, MINNESOTA

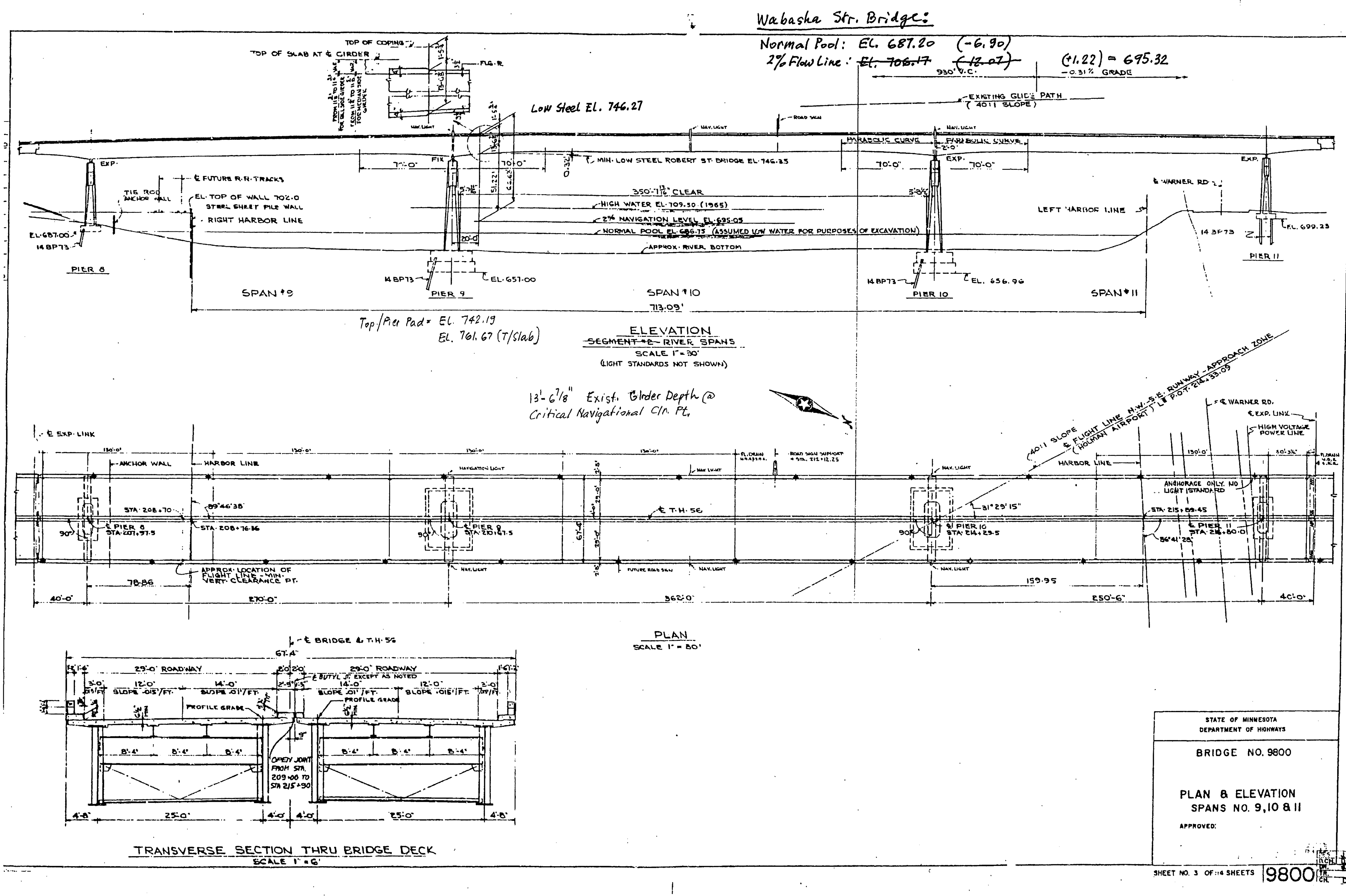
STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

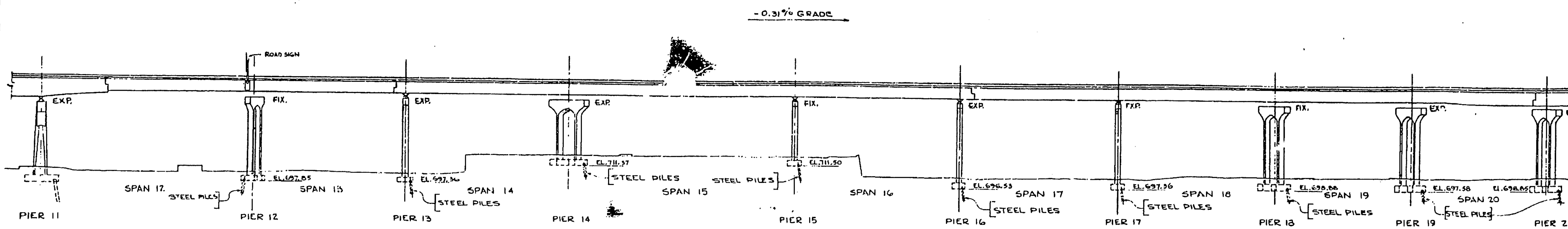
BRIDGE NO. 9800

PLAN AND ELEVATION
SPANS 1 THRU 8

APPROVED:

SHEET NO. 2 OF 114 SHEETS 9800


Wabasha Str. Bridge:
Normal Pool: EL. 687.20 (-6.90)
2% Flow Line: ~~EL. 706.17~~ ~~(12.07)~~ (+1.22) = 695.32
Low Steel EL. 746.27
TOP OF COPING
TOP OF SLAB AT ℄ GIRDER
930' V.C.
-0.31% GRADE
EXISTING GLIDE PATH (40:1 SLOPE)
MIN. LOW STEEL ROBERT ST. BRIDGE EL. 746.25
℄ FUTURE R.R. TRACKS
TIE ROD ANCHOR WALL
EL. TOP OF WALL 702.0 STEEL SHEET PILE WALL
RIGHT HARBOR LINE
350'-1 13/16" CLEAR
HIGH WATER EL. 709.50 (1965)
2% NAVIGATION LEVEL EL. 695.05
NORMAL POOL EL. 686.75 (ASSUMED LOW WATER FOR PURPOSES OF EXCAVATION)
APPROX. RIVER BOTTOM
PARABOLIC CURVE
LEFT HARBOR LINE
℄ WARNER RD.
EL. 687.00
14 BP73
PIER 8
PIER 9
EL. 657.00
PIER 10
EL. 656.96
PIER 11
EL. 699.25
SPAN #9
SPAN #10
SPAN #11
713.09'
Top/Pier Pad = EL. 742.19
EL. 761.67 (T/Slab)
ELEVATION
~~SEGMENT #2~~ RIVER SPANS
SCALE 1" = 30'
(LIGHT STANDARDS NOT SHOWN)
13'-6 7/8" Exist. Girder Depth @ Critical Navigational C/L Pt.
℄ EXP. LINK
ANCHOR WALL
HARBOR LINE
STA. 208+70
STA. 208+76.86
89°46'38"
℄ PIER 8 STA. 207+97.5
℄ PIER 9 STA. 210+67.5
℄ T.H. 56
℄ PIER 10 STA. 214+23.5
81°29'15"
STA. 215+89.45
℄ PIER 11 STA. 216+80.0
86°41'28"
ANCHORAGE ONLY. NO LIGHT STANDARD
APPROX. LOCATION OF FLIGHT LINE - MIN. VERT. CLEARANCE PT.
40:1 SLOPE & FLIGHT LINE N.W.-S.E. RUNWAY - APPROACH ZONE (HOLMAN AIRPORT) ℄ P.O.T. 214+33.09
℄ WARNER RD.
℄ EXP. LINK
HIGH VOLTAGE POWER LINE
78.86
270'-0"
362'-0"
159.95
250'-6"
40'-0"
PLAN
SCALE 1" = 80'
℄ BRIDGE & T.H. 56
67'-4"
29'-0" ROADWAY
SLOPE .015'/FT.
SLOPE .01'/FT.
PROFILE GRADE
℄ BUTYL JT. EXCEPT AS NOTED
OPEN JOINT FROM STA. 209+00 TO STA 215+90
8'-4"
25'-0"
4'-8"
TRANSVERSE SECTION THRU BRIDGE DECK
SCALE 1" = 6'
STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS
BRIDGE NO. 9800
PLAN & ELEVATION
SPANS NO. 9, 10 & 11
APPROVED:
SHEET NO. 3 OF 14 SHEETS
9800

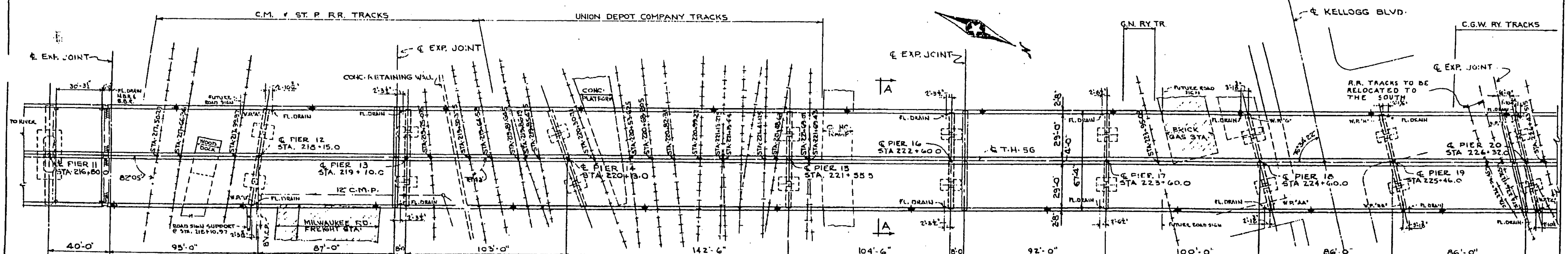


ELEVATION

~~PART SEGMENT "3"~~ - NORTH PORTION

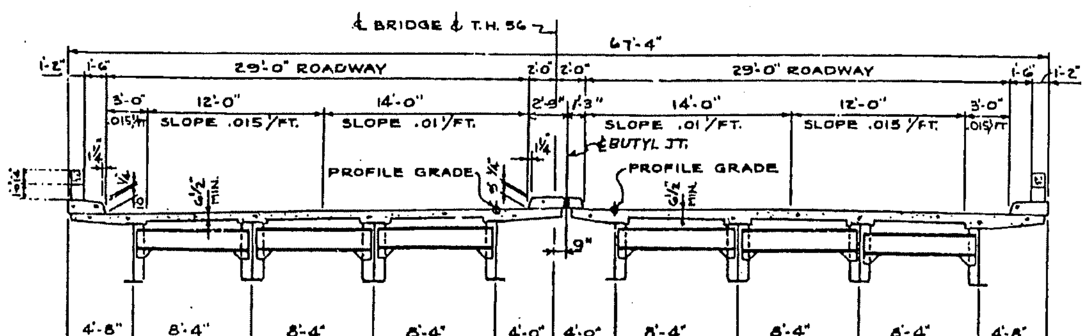
SCALE 1" = 30'

(LIGHT STANDARDS NOT SHOWN)



PLAN

SCALE 1" = 30'



TRANSVERSE SECTION A-A THRU BRIDGE DECK

SCALE: 1" = 1'-0"

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

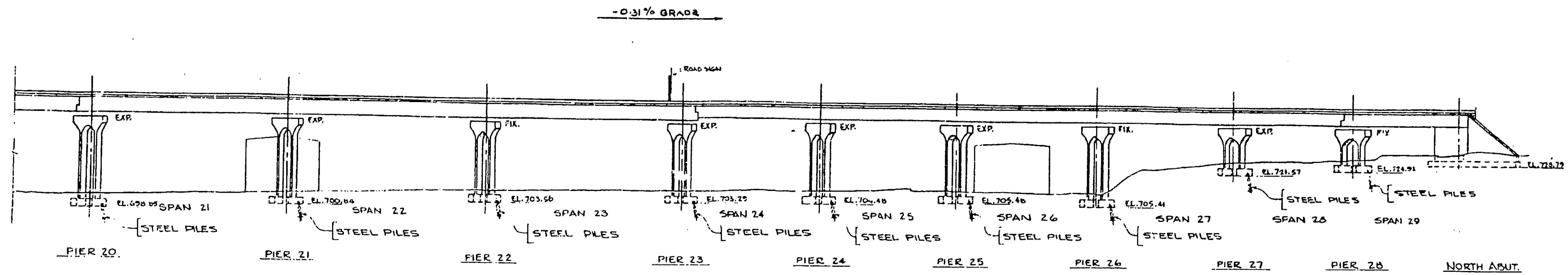
BRIDGE NO. 9800

PLAN & ELEVATION
SPANS 12 THRU 20

APPROVED:

SHEET 4 OF 11 SHEETS

9800

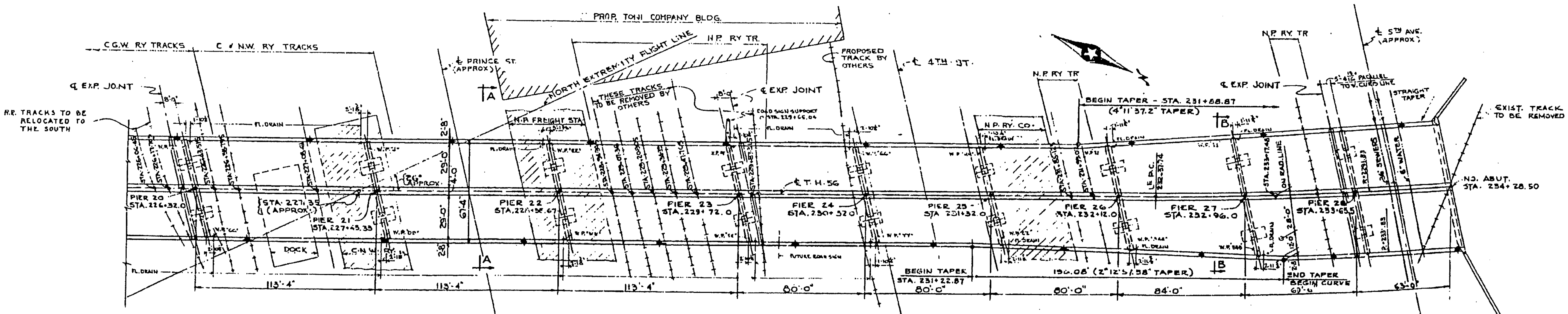


ELEVATION

~~PART SEGMENT #3~~ NORTH PORTION

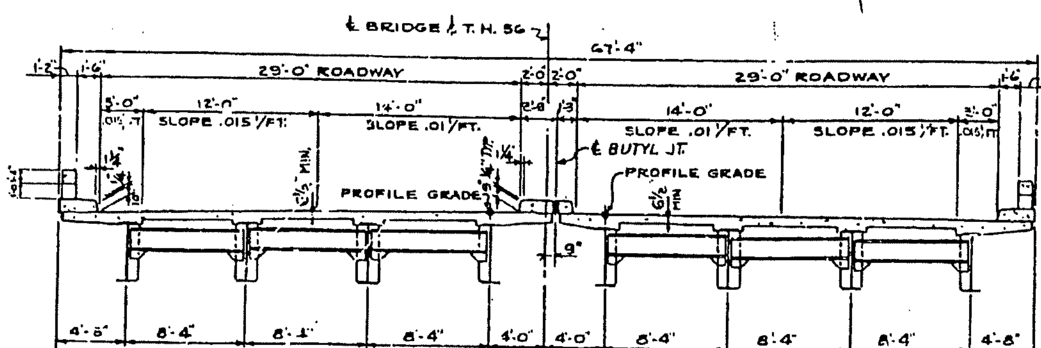
SCALE 1" = 30'

(LIGHT STANDARDS NOT SHOWN)



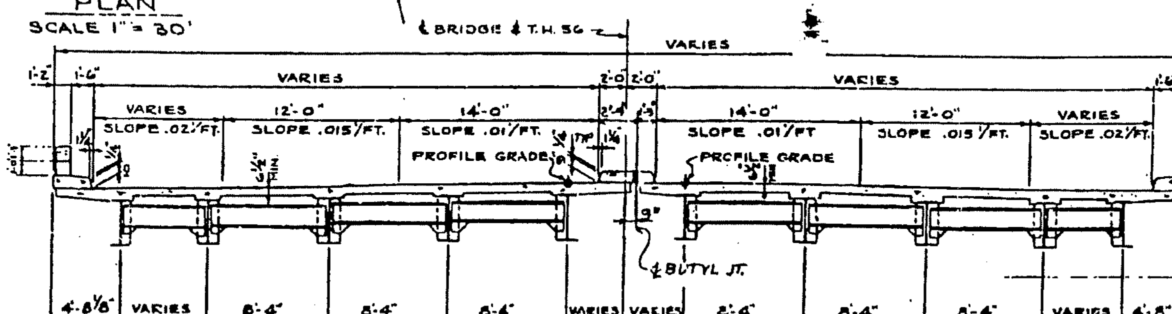
PLAN

SCALE 1" = 30'



TRANSVERSE SECTION A-A THRU BRIDGE DECK

SCALE: 1" = 6'-0"



TRANSVERSE SECTION B-B THRU BRIDGE DECK

SCALE: 1" = 6'-0"

NOTE: SLAB THICKNESS VARIES WHERE BEAM SPACING DIFFERS FROM 8'-4". SEE SUPERSTRUCTURE SH. NO. 96

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

BRIDGE NO. 9800

PLAN & ELEVATION
SPANS 21 THRU 29

APPROVED

DES. B.G.
D.CH. D.B.
DR. J.S.
TR.

SHEET 5 OF 14 SHEETS

9800