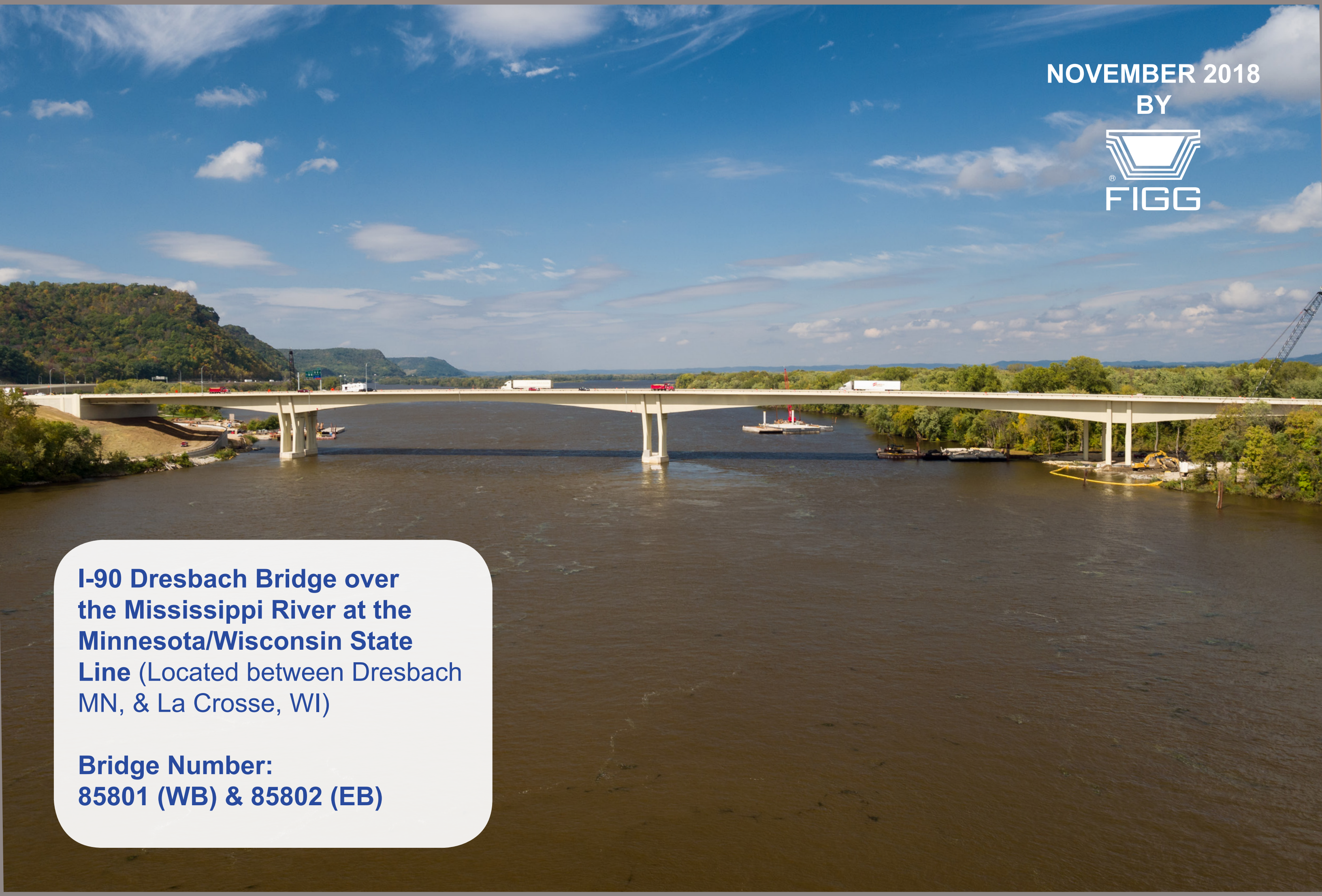
INSPECTION & MAINTENANCE MANUAL
DRESBACH BRIDGE



NOVEMBER 2018
BY
FIGG

I-90 Dresbach Bridge over the Mississippi River at the Minnesota/Wisconsin State Line (Located between Dresbach MN, & La Crosse, WI)

Bridge Number:
85801 (WB) & 85802 (EB)

PREPARED FOR THE

MINNESOTA DEPARTMENT OF TRANSPORTATION

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ABBREVIATIONS

The following is a list of abbreviations used in this document:

Abut.	-	Abutment
BRG	-	Bearing
CIP	-	Cast-in-Place
CS.	-	Closure Segment
Conc.	-	Concrete
DS	-	Downstation
EB	-	Eastbound
Elev.	-	Elevation
Est.	-	Estimated
FFBW	-	Front Face of Backwall
I.D.	-	Inside Diameter
Max.	-	Maximum
Min.	-	Minimum
No.	-	Number
P.G.L.	-	Profile Grade Line
P.I.	-	Point of Inflection
PT	-	Post-Tensioning or Pier Table
PTFE	-	polytetrafluoroethylene
Seg.	-	Segment
Spa.	-	Spaces
Sta.	-	Station
Symm.	-	Symmetric
Typ.	-	Typical
US	-	Upstation
WB	-	Westbound

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LIMITATIONS

INTRODUCTION

1. INTRODUCTION

1.1 Purpose of the Manual

This Manual provides special procedures for conducting inspections to determine the physical condition of the post-tensioned concrete segmental unit of the Interstate 90 Bridge at the Mississippi River (IM 0903(187)) between Dresbach, Minnesota and La Crosse, Wisconsin. It also outlines a program of routine preventative and corrective maintenance procedures for the bearings and expansion joints in the post-tensioned concrete segmental unit (unit 1) of the bridge.

This Manual does not provide special procedures for conducting inspections to determine the physical condition of the prestressed girder unit (unit 2) of the Interstate 90 Bridge at the Mississippi River. Unit 2 is a standard Minnesota Department of Transportation girder bridge. Refer to the *Bridge Inspection Field Manual* for more information regarding inspection of this unit.

The inspection procedure portions of this manual are intended for use by individuals qualified as bridge inspectors in accordance with the State of Minnesota standards. The objective of this manual is to serve as a supplement to the Minnesota Department of Transportation standard procedures for detailed inspections of the Interstate 90 Bridge at the Mississippi River.

1.2 Scheduled Inspections and Timely Maintenance

Programmed systematic inspections are required by Federal and State Directives and are essential in order to detect any problem that may require future monitoring and corrective maintenance or repair. In addition, these inspections help assure that appropriate action is initiated in a timely manner. Bridge inspections and associated maintenance will ensure that the structure performs satisfactorily over its intended service life. Early detection of maintenance needs and the performance of maintenance in a timely and effective manner are essential to protect the substantial investment of public funds.

DESCRIPTION OF THE BRIDGE

2. DESCRIPTION OF THE BRIDGE

2.1 General Description

The Interstate 90 Bridge at the Mississippi River is located between Dresbach, Minnesota and La Crosse, Wisconsin (Figure 2.1) on Interstate 90 over the Mississippi River. Unit 1 of the structure is a cast-in-place, post-tensioned, segmental concrete box girder bridge built in the balanced cantilever method of construction. Unit 2 is a prestressed concrete girder bridge. Each bridge is comprised of twin structures separated by a minimum of 10'-8". One structure carries westbound traffic and the other carries eastbound traffic.

Figure 2.1 – Project Location

Unit 1 of both structures is 1666'-6" long and carries two 12'-0" through lanes, a 6'-0" inside shoulder, and a variable width outside shoulder. In addition, span 1 of westbound structure carries a 14'-0" deceleration ramp lane and spans 1 through 3 of the eastbound structure carries a variable width acceleration ramp lane. The typical widths of the unit 1 westbound and eastbound box girders are 45'-4" and 53'-4", respectively. To accommodate the acceleration and deceleration lanes, both structures widen in span 1 and the eastbound structure narrows in span 4. On both structures, the unit 1 box girder varies in depth from 25‘-0” at the piers to 11‘-0” at the center of the main spans and in the end spans. Unit 1 of both structures is a 4-span continuous unit with expansion joints located at the west abutment and pier 4. Unit 1 was cast predominantly in balanced cantilever with a portion of the end spans cast on falsework. The westbound unit 1 span lengths are: 294’-8 7/8”, 508’-0”, 508’-0”, and 345’-1”. The eastbound unit 1 span lengths are: 315’-8 7/8”, 508’-0”, 508’-0” and 324’-1”.

Unit 2 of both structures is 926’-1” long and consists of four 82” prestressed concrete beams and a 45’-4” wide deck. Each structure carries two 12‘-0” through lanes, a 6‘-0” inside shoulder, and a 12‘-0” outside shoulder. Unit 2 of both structures is a 6-span continuous unit with expansion joints located at pier 4 and the east abutment. The westbound and eastbound unit 2 span lengths are: 152’-6 1/8”, 154’-0”, 154’-0”, 154’-0”, 154’-0”, and 153’-1 1/2”.

A type Mod. P-2 (TL-4) parapet with T-1 railing is along the outside edge of each deck and a type Mod. P-4 (TL-4) barrier is along the inside edge of each deck.

A general plan and elevation of the westbound structure is shown in Figure 2.2, and a general plan and elevation of the eastbound structure is shown in Figure 2.3. Typical sections for both structures are shown in Figures 2.4 and 2.5.

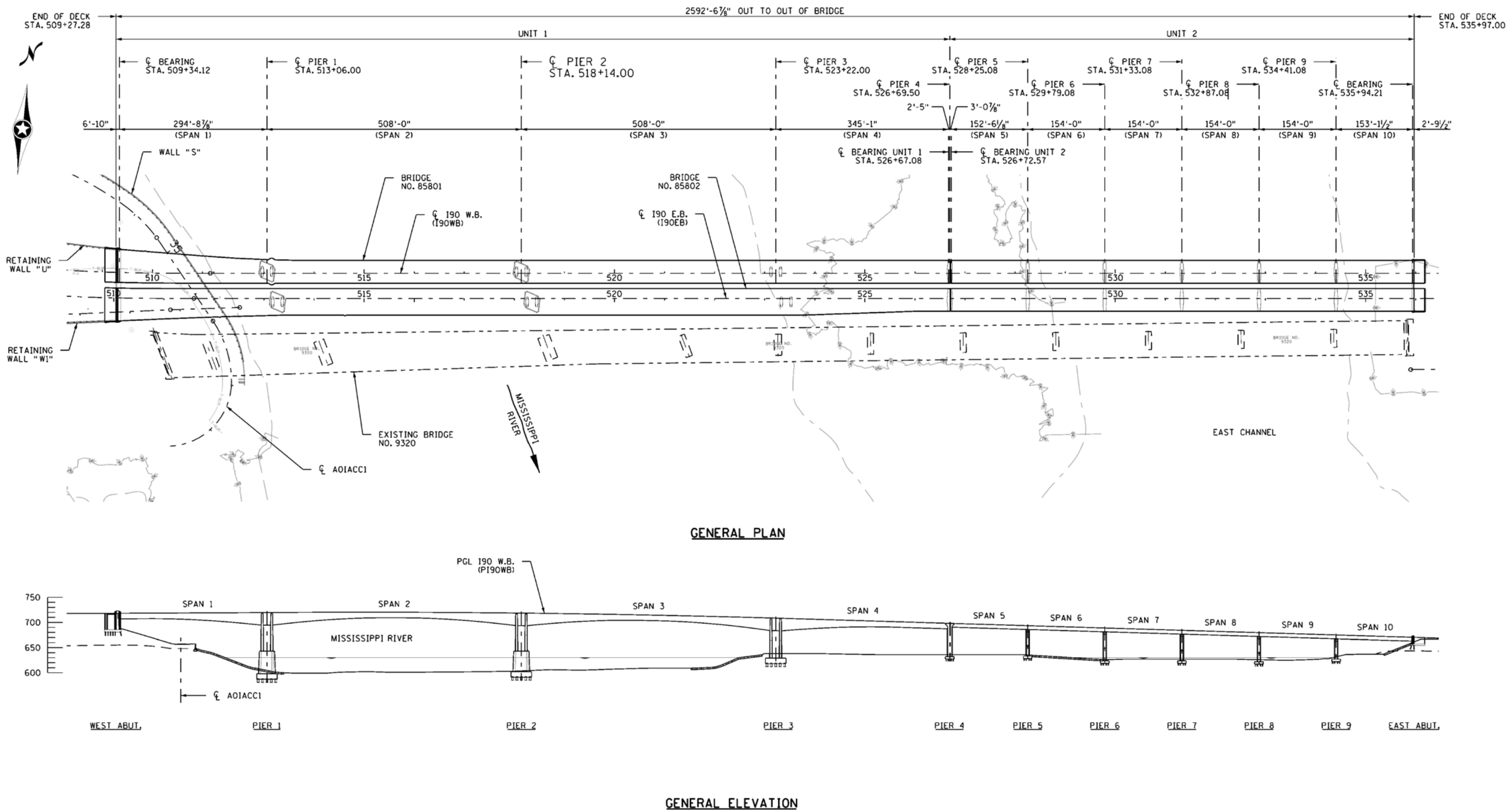


Figure 2.2 – Westbound General Plan and Elevation

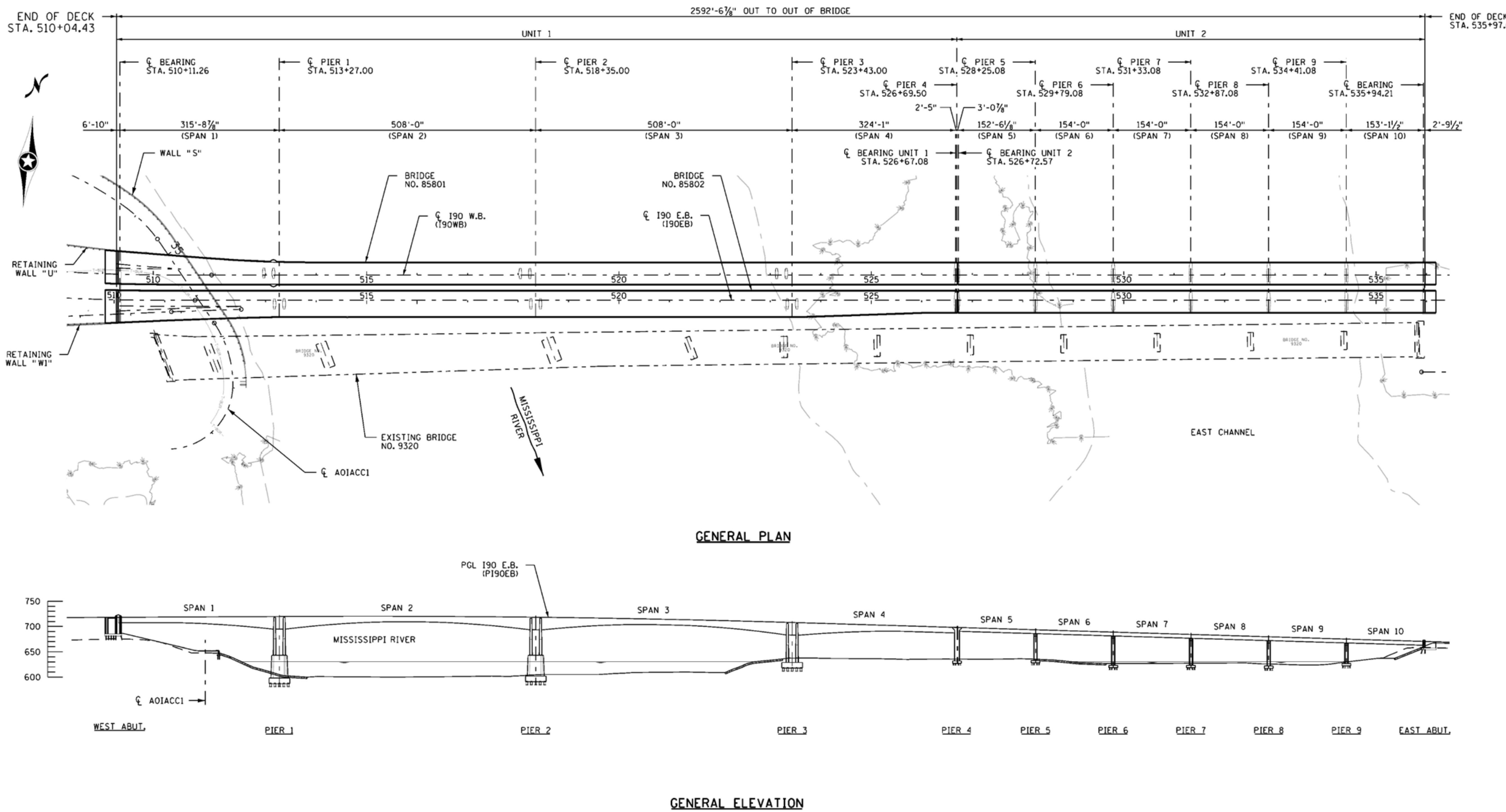


Figure 2.3 – Eastbound General Plan and Elevation

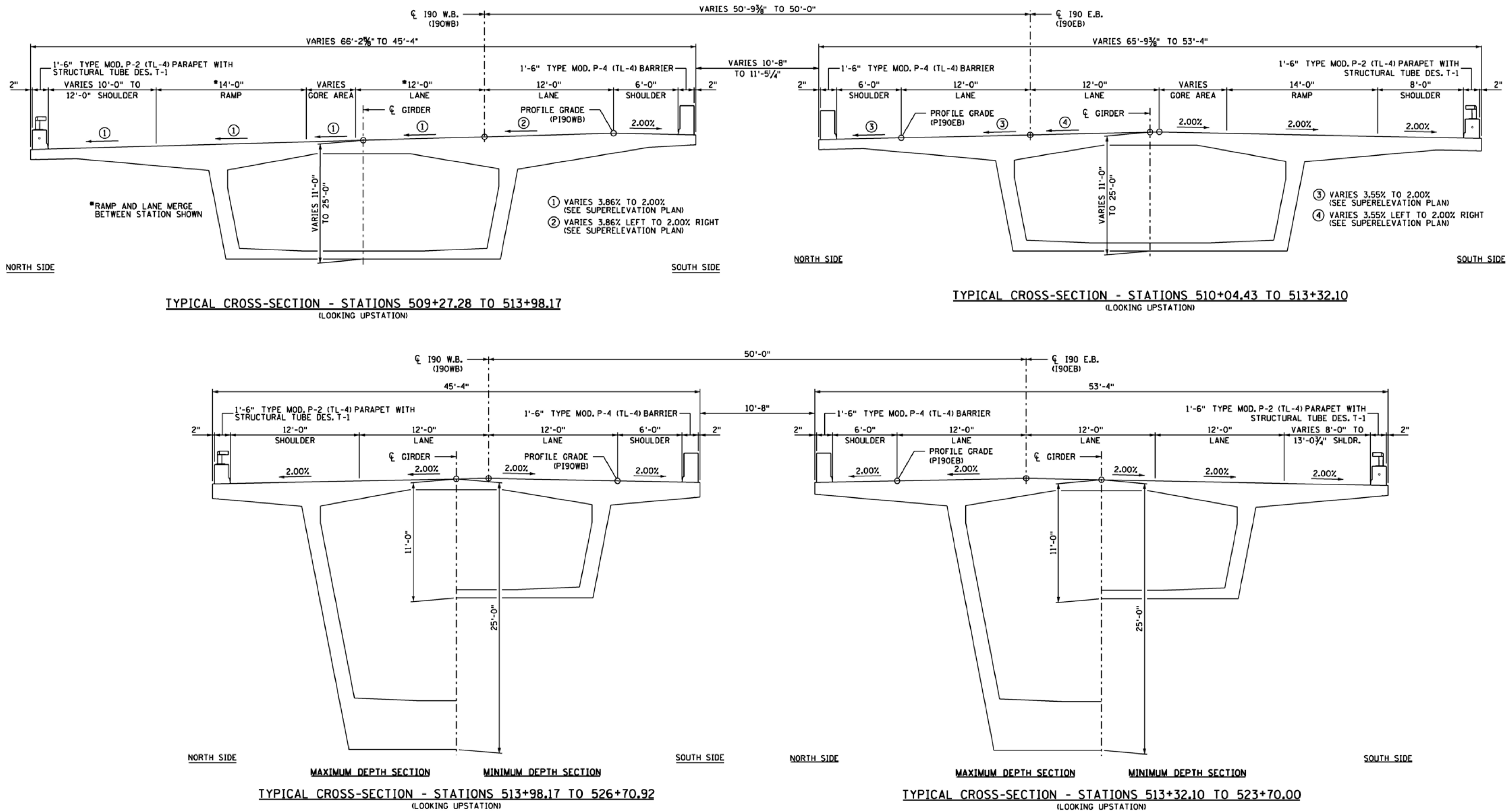


Figure 2.4 – Typical Sections I

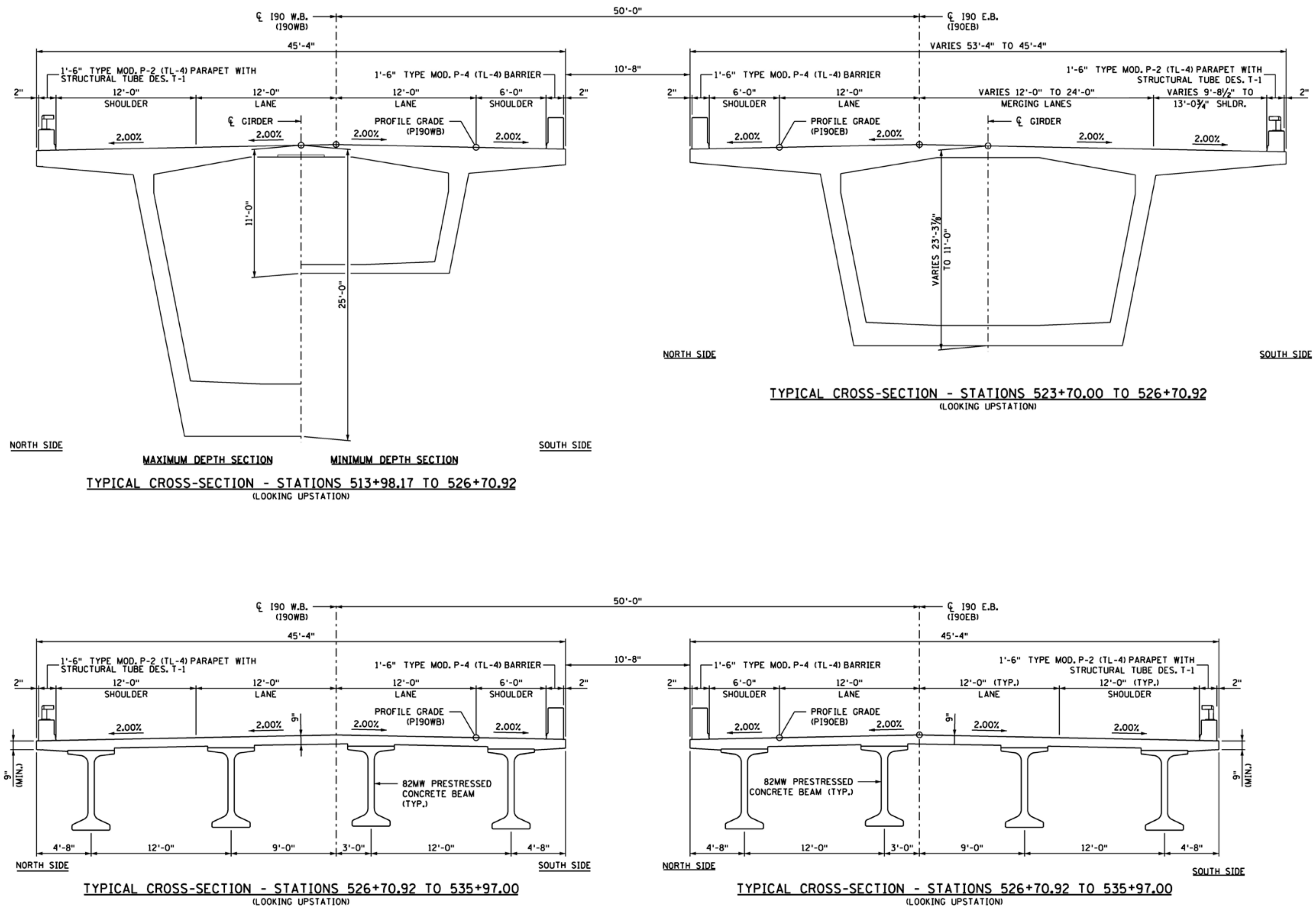


Figure 2.5 – Typical Sections II

2.2 Basics of Cast-In-Place Segmental Bridge Construction

Unit 1 of the Interstate 90 Bridge at the Mississippi River features a cast-in-place, variable depth, post-tensioned concrete segmental box girder built in the balanced cantilever method of construction. In this method, the bridge is cast in small increments, called segments, using a traveling form system. Each segment is a complete transverse section of the bridge. The length of each typical segment in the longitudinal direction is 16’-3”. The depth of each segment varies along the length of the bridge (deeper at the piers, thinner at mid-span) in order to maximize the efficiency of the girder.

In the balanced cantilever method of construction, the cast-in-place segments make up the majority of the structure. The segments are cast sequentially, on alternating sides of each pier, so that the out-of-balance forces in the foundations are minimized. As segments are cast on each side, the length of the cantilever increases symmetrically about the pier. Ultimately, two cantilevers from adjacent piers close in the center to create an interior span. The cantilevers close with cast-in-place on falsework sections to complete the end spans. Each cantilever consists of 14 pairs of segments, and each of the three unit 1 piers supports two balanced cantilevers for a total of 84 segments per structure. For the purpose of identifying segments, all cast-in-place segments are permanently labeled on the interior web of the box girder. The segment labeling system and layout is illustrated in Figures 2.6 and 2.7 for the westbound and eastbound bridges, respectively.

The remaining length of each unit 1 structure consists of three cast-in-place on falsework pier tables, two cast-in-place on falsework end sections, and four 7’-0” long closure pours. The closure pours occur between the cantilevers tips in the two interior spans and between the cantilevers and end sections in the two end spans.

The pier tables are the first portion of the bridge superstructure constructed. They form the starting point for the balanced cantilevers and act as the launching point for the traveling form system. The pier tables are integral with the twin wall piers and are cast in three lifts atop the piers using falsework. The 46’-0” longitudinal length of the pier table is provided for sufficient room on top to support the traveling form system used to cast the segments. As illustrated in Figures 2.6 and 2.7, the pier table is offset to one side of the pier so that the cantilevers are never more than 8’-0” (approximately one-half segment length) out of balance.

The cast-in-place on falsework end sections provide additional length to the end spans beyond what can be supported by the cantilever to achieve an efficient end span length. The westbound span 1 and span 4 cast-in-place end sections are 44’-6 7/8” and 86’-11” long, respectively. The eastbound span 1 and span 4 cast-in-place end sections are 65’-6 7/8” and 65’-11” long, respectively. Each cast-in-place end section is cast in three lifts using falsework supported from the ground.

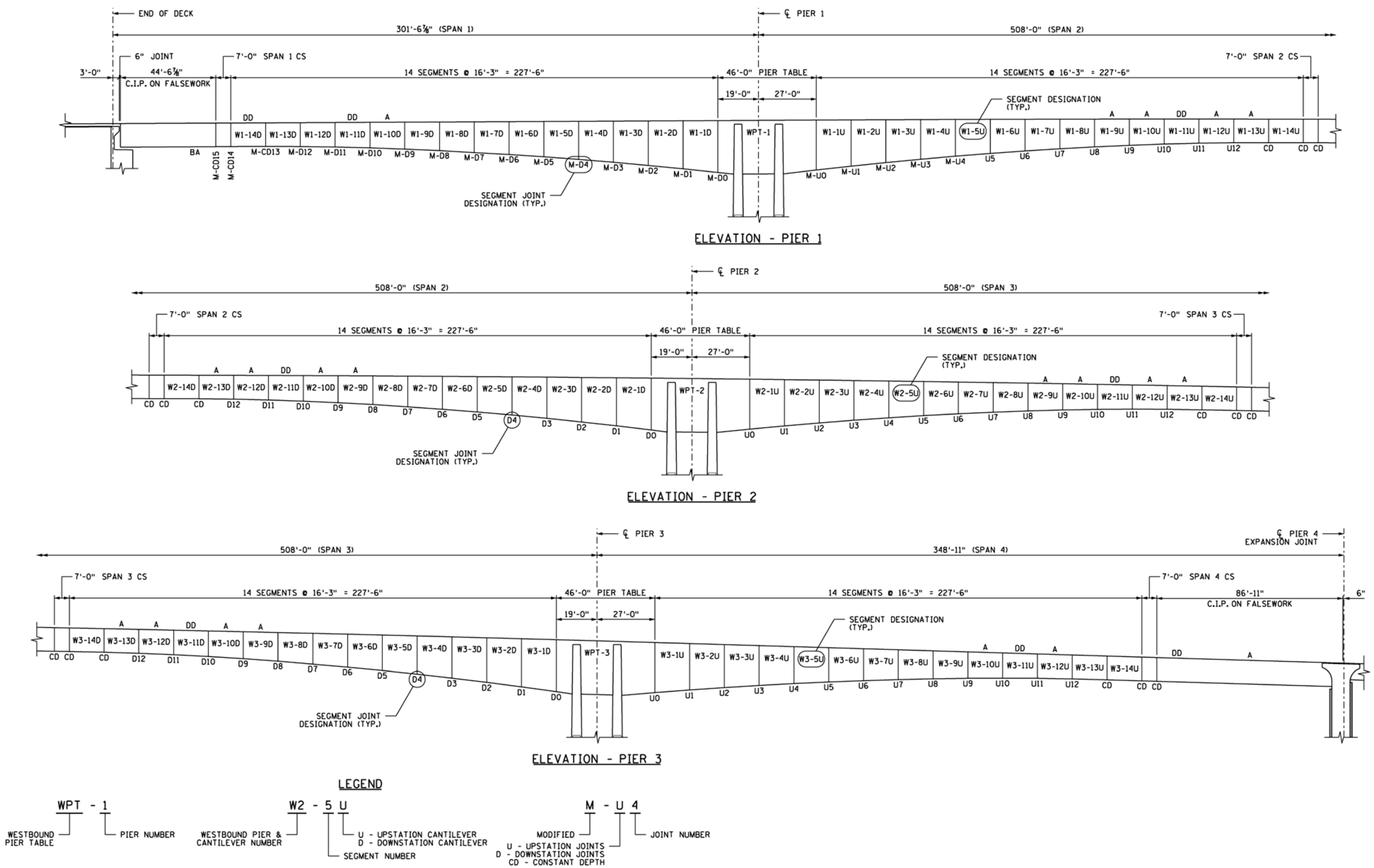


Figure 2.6 – Westbound Segment Designation

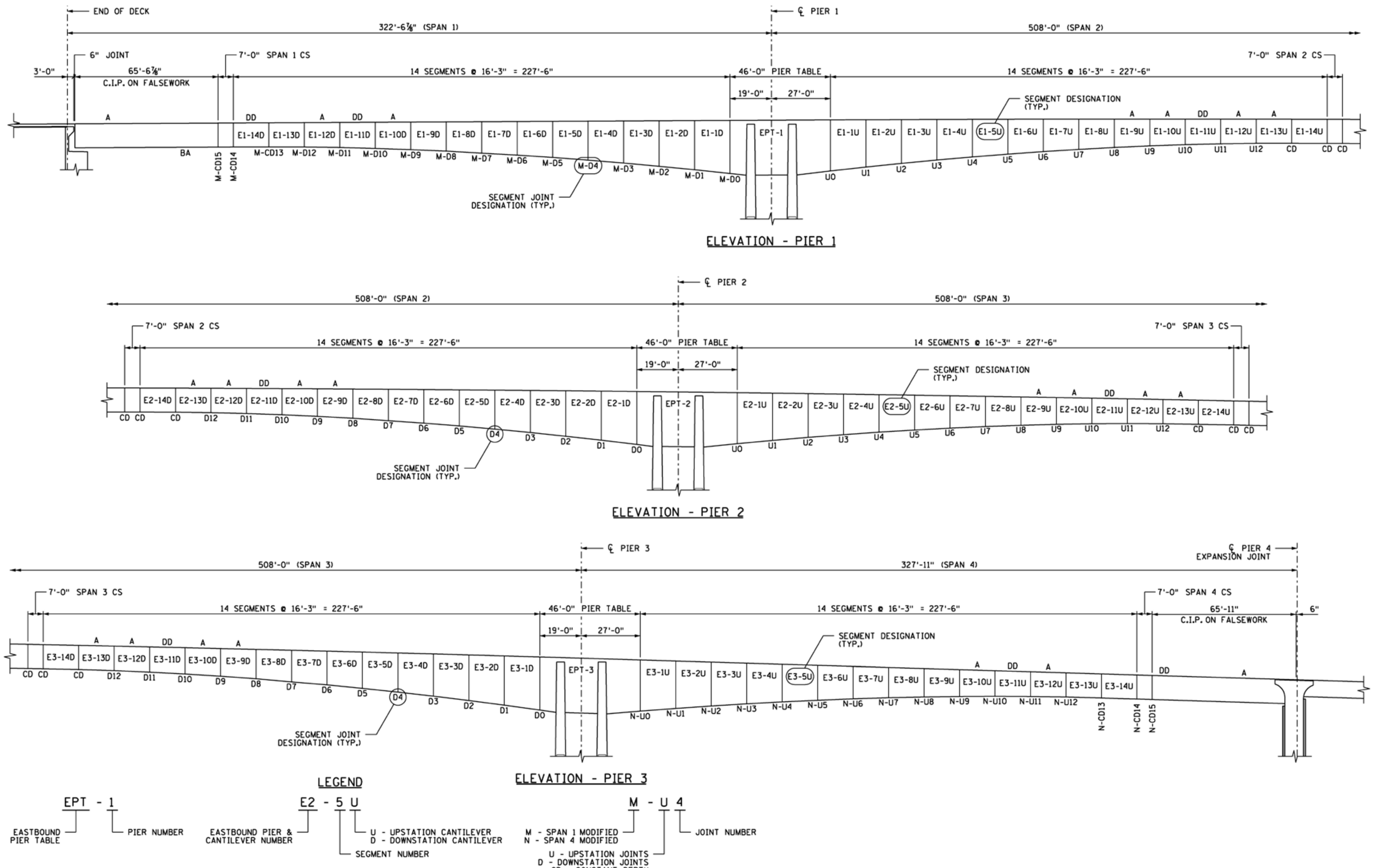


Figure 2.7 – Eastbound Segment Designation

The traveling form system consists of self-launching steel trusses that are supported from the bridge superstructure and can support the weight of the forms, reinforcing, and concrete needed to cast a segment. The segment forms and supporting trusses at the end of each cantilever are collectively referred to as a form traveler. Form travelers allow construction of the bridge superstructure without the need for falsework supported from the ground. The rear of the form traveler is tied down using high strength post-tensioning bars anchored to the bottom of the bridge deck through formed blockouts and the front is supported on the bridge deck. Formwork for the bottom soffit, webs, and top slab are all supported from the end of the last segment and from a truss or beam at the leading edge of the traveler. The forms can be manipulated to achieve the variable depth profile and variable top slab geometry, to cast to the desired roadway geometry, and to correct for camber and anticipated deflections.

A casting cycle is as follows: 1) reinforcing steel and post-tensioning hardware are placed in the forms, 2) the forms are surveyed into position, 3) the segment is cast, 4) the transverse and longitudinal cantilever post-tensioning tendons in the segment are stressed so that the segment is self-supporting, and finally, 5) the form traveler is launched forward on launching rails onto the newly-cast segment and positioned to begin the next casting cycle.

When all cantilever 3 segments and the span 4 cast-in-place end section have been cast, the end section and the cantilever are joined with a 7'-0" closure pour in span 4. To create continuity between cantilever 3 and the end section, draped external post-tensioning running the length of the span is stressed. The draped post-tensioning tendons are external to the bridge cross-section but internal to the girder cell. Bottom slab post-tensioning tendons are also stressed at this time. When all the cantilever 2 segments have been cast, cantilever 3 and cantilever 2 are joined with a 7'-0" closure pour in span 3. Continuity is created between the cantilevers by stressing the draped and bottom slab post-tensioning tendons. When all the cantilever 1 segments have been cast, cantilever 1 and cantilever 2 are joined with a 7'-0" closure pour in span 2. Again, to create continuity between the cantilevers, the draped and bottom slab post-tensioning tendons are stressed. Finally, when the span 1 cast-in-place end section has been cast, the end section and cantilever 1 are joined with a 7'-0" closure pour in span 1 and the draped and bottom slab post-tensioning are stressed. The eastbound structure was constructed using the above sequence. The westbound structure was constructed using the above sequence with the exception that span 1 is closed before span 2, along with the need for counterweights needed on cantilever 1 to reduce out-of-balance moments.

In summary, the fundamental aspect of segmental construction as used for unit 1 of the Interstate 90 Bridge at the Mississippi River is that cast-in-place segments are post-tensioned together to form the bridge superstructure.

2.3 Structure Components

The components of the structure include the substructure, bearings, superstructure (including the integral wearing surface), approach slabs, and expansion joint devices.

2.3.1 Substructure

The substructure is composed of conventional cast-in-place reinforced concrete footings, piers, and abutments. The substructure serves to transfer all superstructure loads, both vertical and horizontal, to the underlying bearing strata.

2.3.1.1 Pier Foundations

The foundations serve to transmit all loads from the piers to the underlying bearing strata.

The eastbound pier 1 foundation is comprised of 30 - 42" diameter cast-in-place concrete piles, spaced 8'-9" on center, capped with a 41'-0" by 49'-9", 11'-0" deep cast-in-place footing. The footing sits on top of a 14'-6" thick concrete seal slab.

The eastbound pier 2 foundation is comprised of 25 - 42" cast-in-place concrete piles, typically spaced 8'-9" on center, capped with a 41'-0" by 45'-9", 10'-0" deep cast-in-place footing. The footing sits on top of a 14'-0" thick concrete seal slab.

The eastbound pier 3 foundation is comprised of 22 - 42" cast-in-place concrete piles, spaced 8'-9" on center, capped with a 41'-0" square, 10'-0" deep cast-in-place footing. The footing sits on top of a 6'-0" thick concrete seal slab.

The eastbound pier 4 foundation is comprised of 28 - HP 14x117 steel piles, spaced approximately 4'-0" on center, capped with a 20'-10" by 23'-4", 6'-0" deep cast-in-place footing. The footing sits on top of a 3'-6" thick concrete seal slab.

Eastbound piers 5 through 9 foundations are comprised of 18 - HP 14x117 steel piles, capped with a 17'-4" by 23'-4", 5'-0" deep cast-in-place footing. The footing sits on top of a 4'-3" thick (foundations 5 and 9) or 5'-3" thick (foundations 6, 7, and 8) concrete seal slab.

The westbound pier 1 foundation is comprised of 28 - 42" diameter cast-in-place concrete piles, spaced 8'-9" on center, capped with a 41'-0" by 49'-9", 11'-0" deep cast-in-place footing. The footing sits on top of a 14'-6" thick concrete seal slab.

The westbound pier 2 foundation is comprised of 25 - 42" cast-in-place concrete piles, typically spaced 8'-9" on center, capped with a 41'-0" by 45'-9", 10'-0" deep cast-in-place footing. The footing sits on top of a 14'-0" thick concrete seal slab.

The westbound pier 3 foundation is comprised of 22 - 42” cast-in-place concrete piles, spaced 8’-9” on center, capped with a 41’-0” square, 10’-0” deep cast-in-place footing. The footing sits on top of a 6’-0” thick concrete seal slab.

The westbound pier 4 foundation is comprised of 28 - HP 14x117 steel piles, spaced approximately 4’-0” on center, capped with a 20’-10” by 23’-4”, 6’-0” deep cast-in-place footing. The footing sits on top of a 3’-6” thick concrete seal slab.

Westbound piers 5 through 9 foundations are comprised of 18 - HP 14x117 steel piles, capped with a 17’-4” by 23’-4”, 5’-0” deep cast-in-place footing. The footing sits on top of a 4’-3” thick (foundations 5 and 9) or 5’-3” thick (foundations 6, 7, and 8) concrete seal slab.

For foundation plan views see Figure 2.8 through Figure 2.11. For elevation views of the pier foundations, see Figure 2.12 through Figure 2.15.

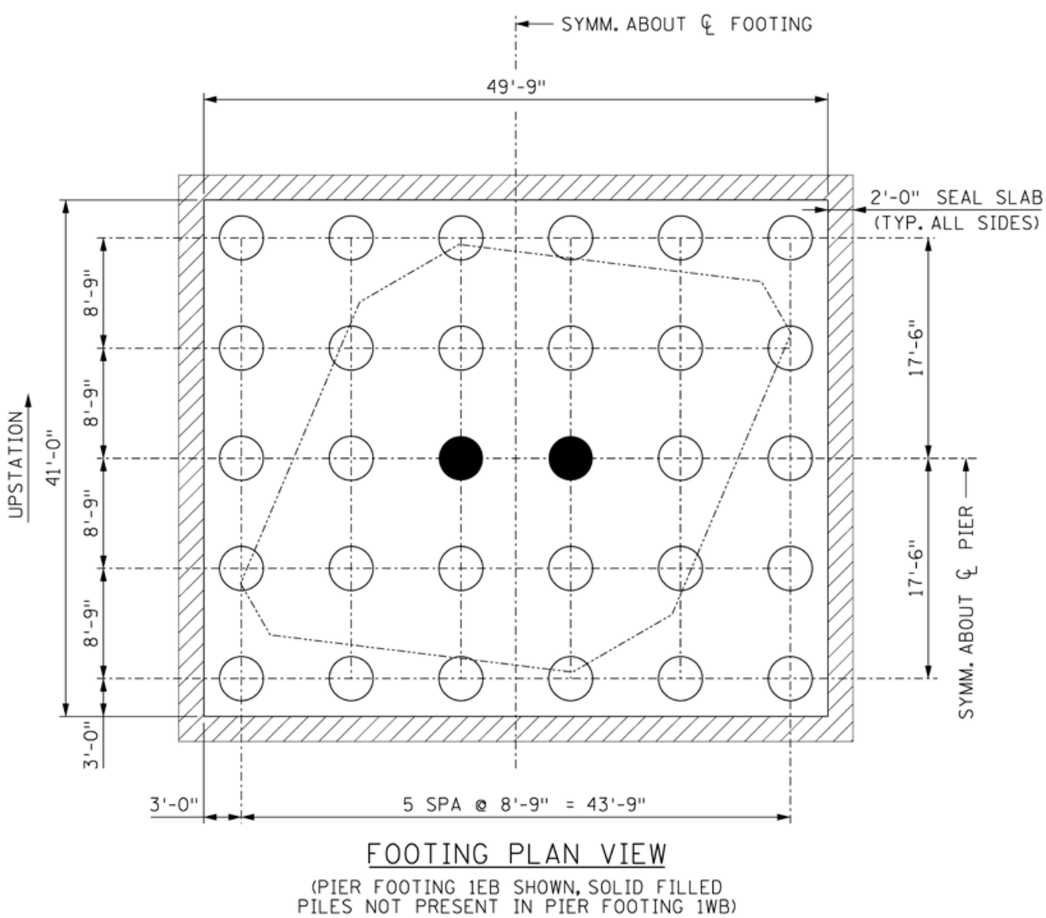


Figure 2.8 – Pier 1 Foundation Plan View

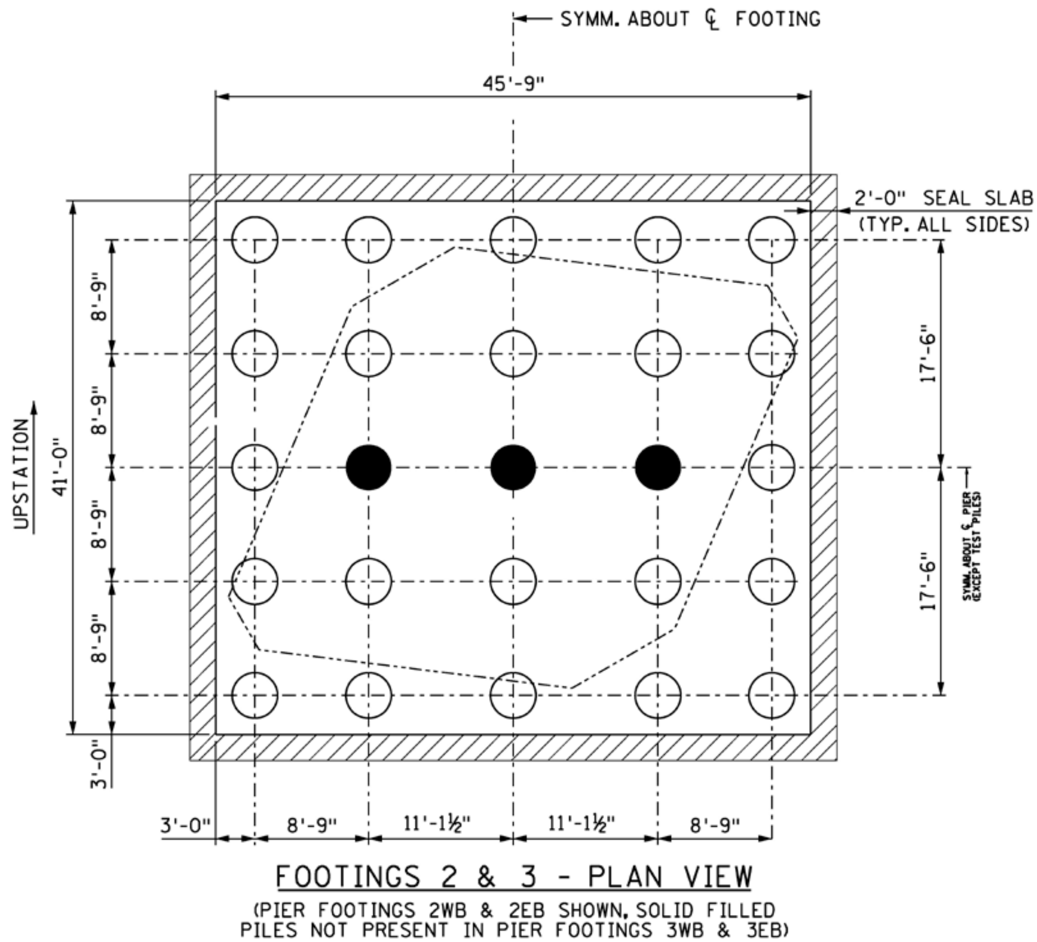


Figure 2.9 – Piers 2 and 3 Foundation Plan View

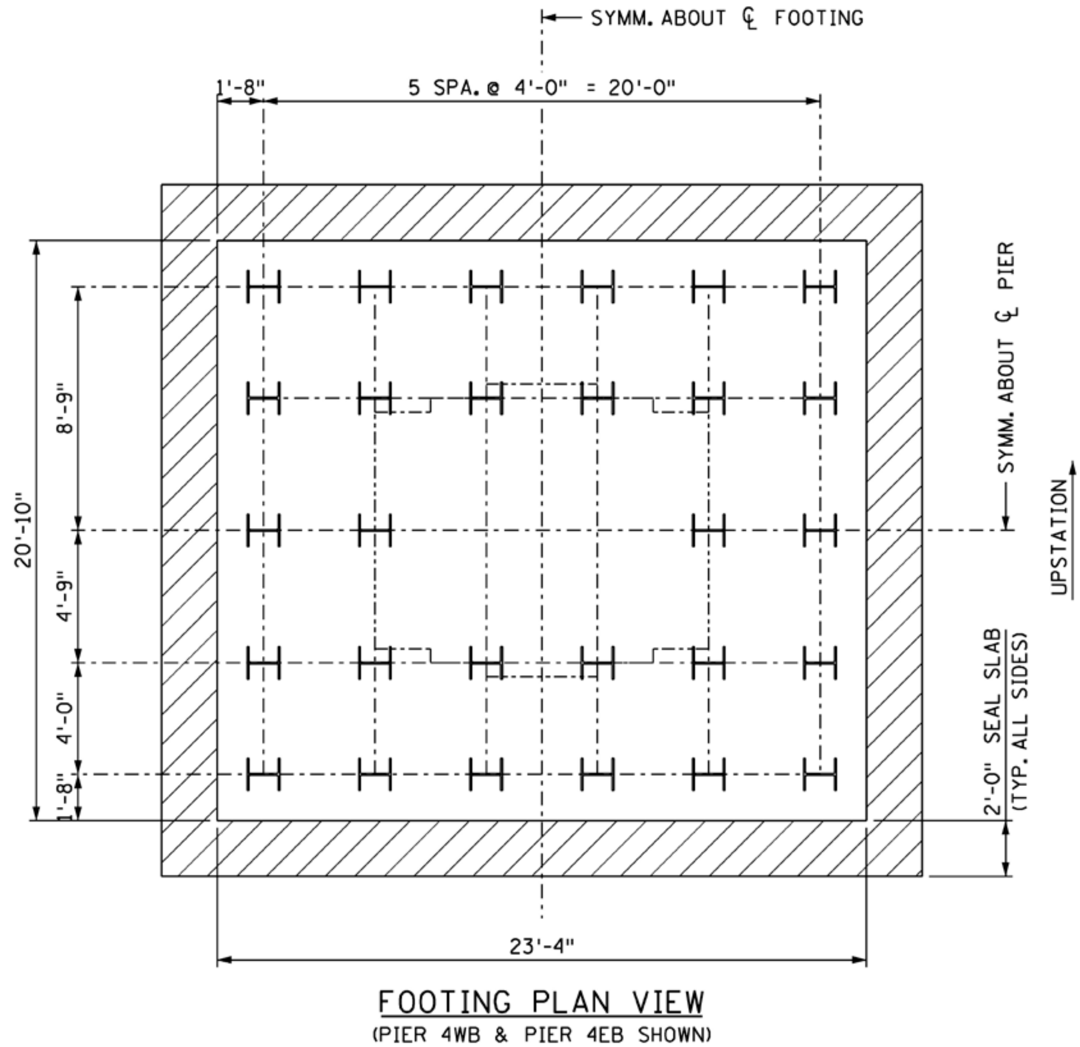


Figure 2.10 – Pier 4 Foundation Plan View

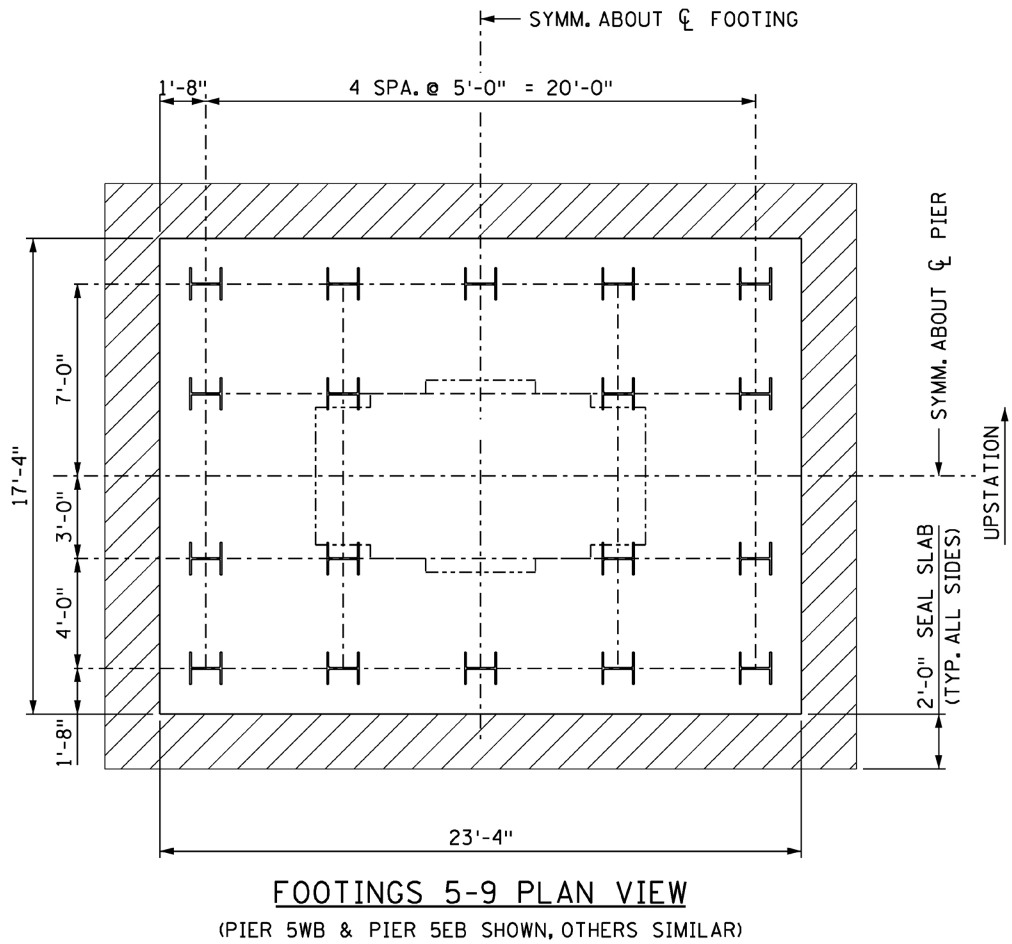


Figure 2.11 – Piers 5-9 Foundation Plan View

2.3.1.2 Piers

Piers transmit all superstructure loads to the foundations. Elevation views of the piers are shown in Figures 2.12 through 2.15. Piers 1, 2, and 3 consist of twin walls constructed with cast-in-place reinforced concrete. Piers 1 and 2 twin walls are cast atop solid reinforced concrete pedestals, which are supported on the footings. Pier 3 twin walls are supported directly on the footing. Pier 4 is a cast-in-place reinforced concrete transition pier between units 1 and 2. Disc bearings supporting the unit 1 superstructure are grouted integral with the bearing pedestal on pier 4 (downstation). Piers 5 through 9 are typical cast-in-place reinforced concrete piers. Piers 4 through 9 are supported directly on footings. Elastomeric bearings supporting the unit 2 superstructure are grouted integral with the bearing pedestals on piers 4 (upstation) through 9.

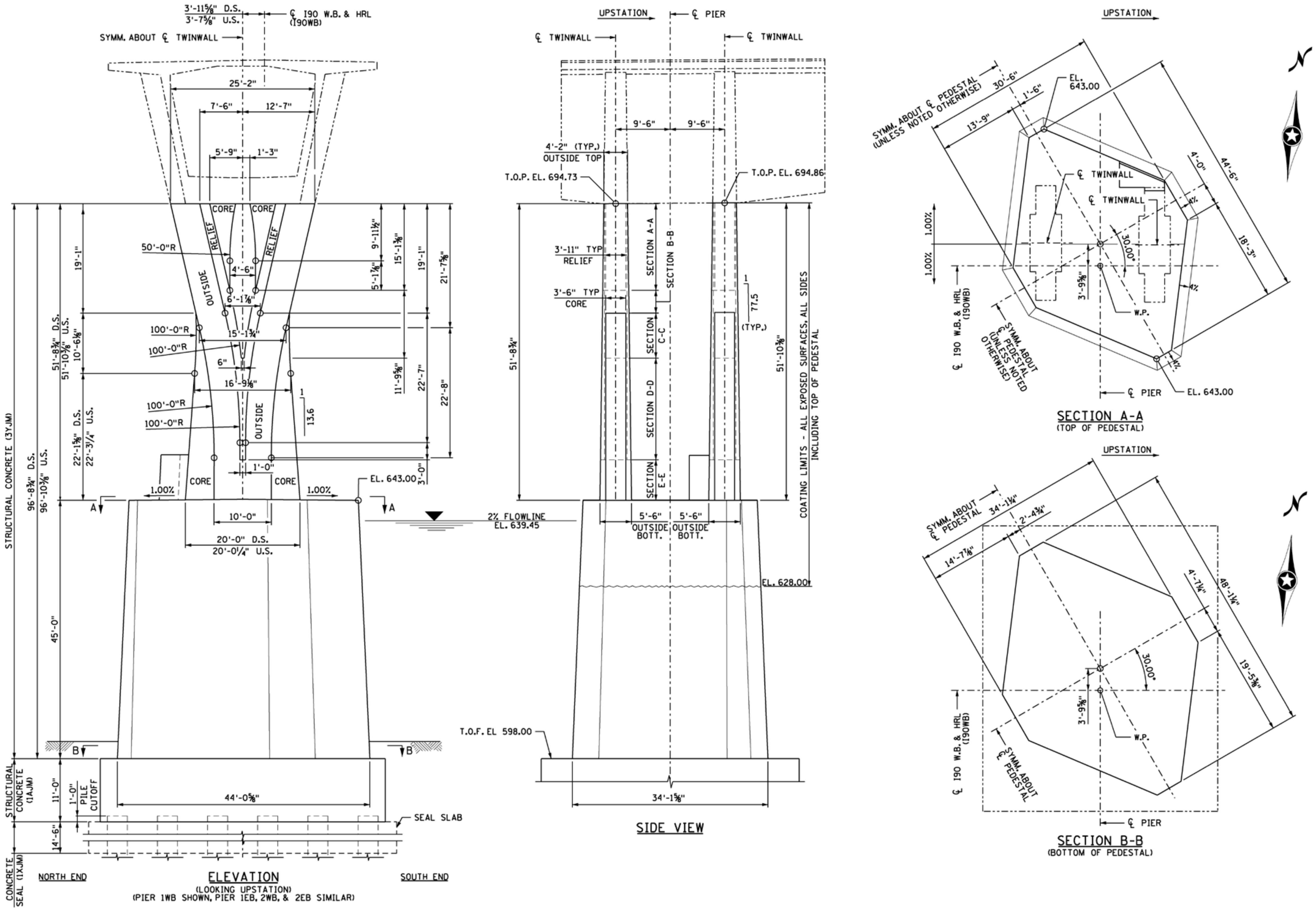


Figure 2.12 – Piers 1 and 2

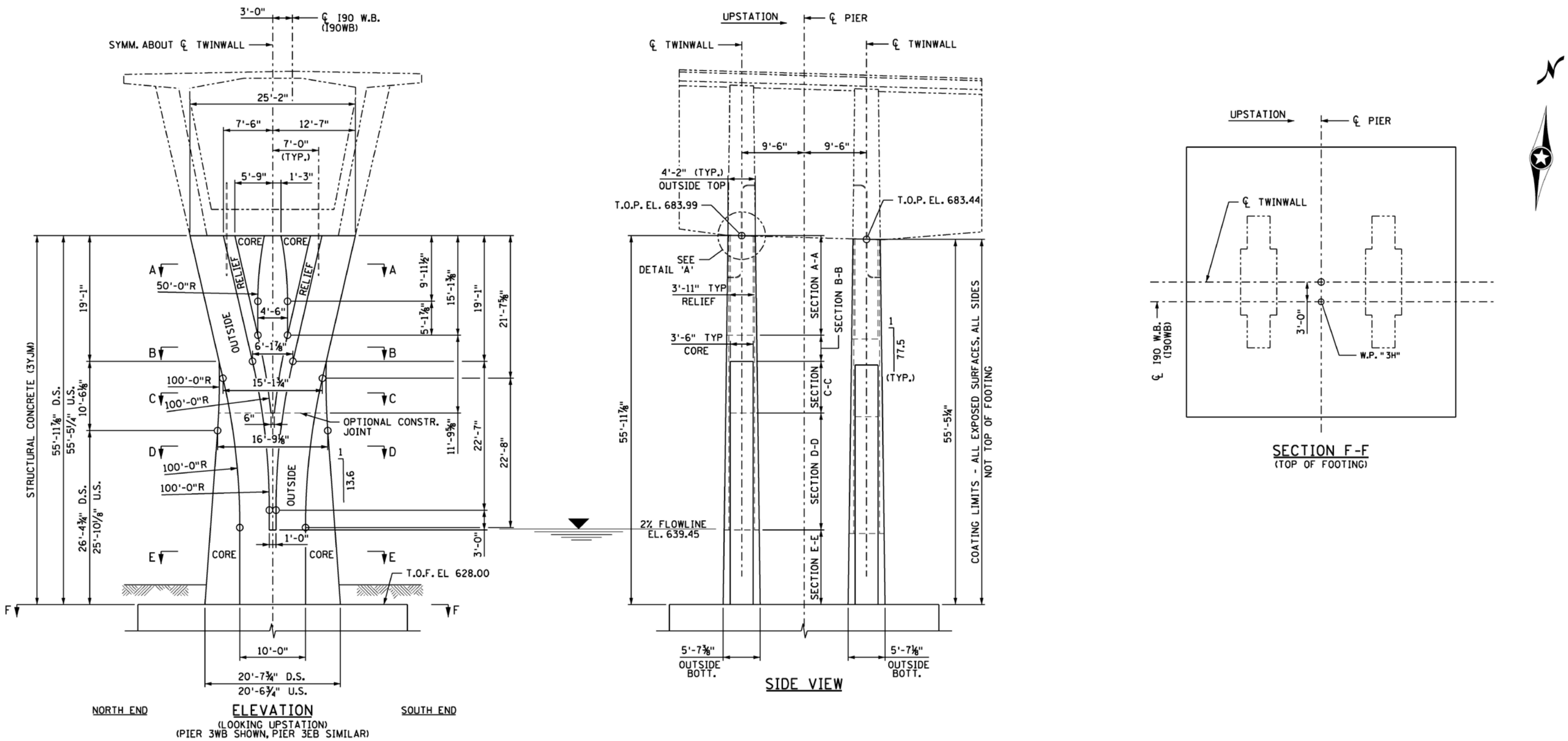
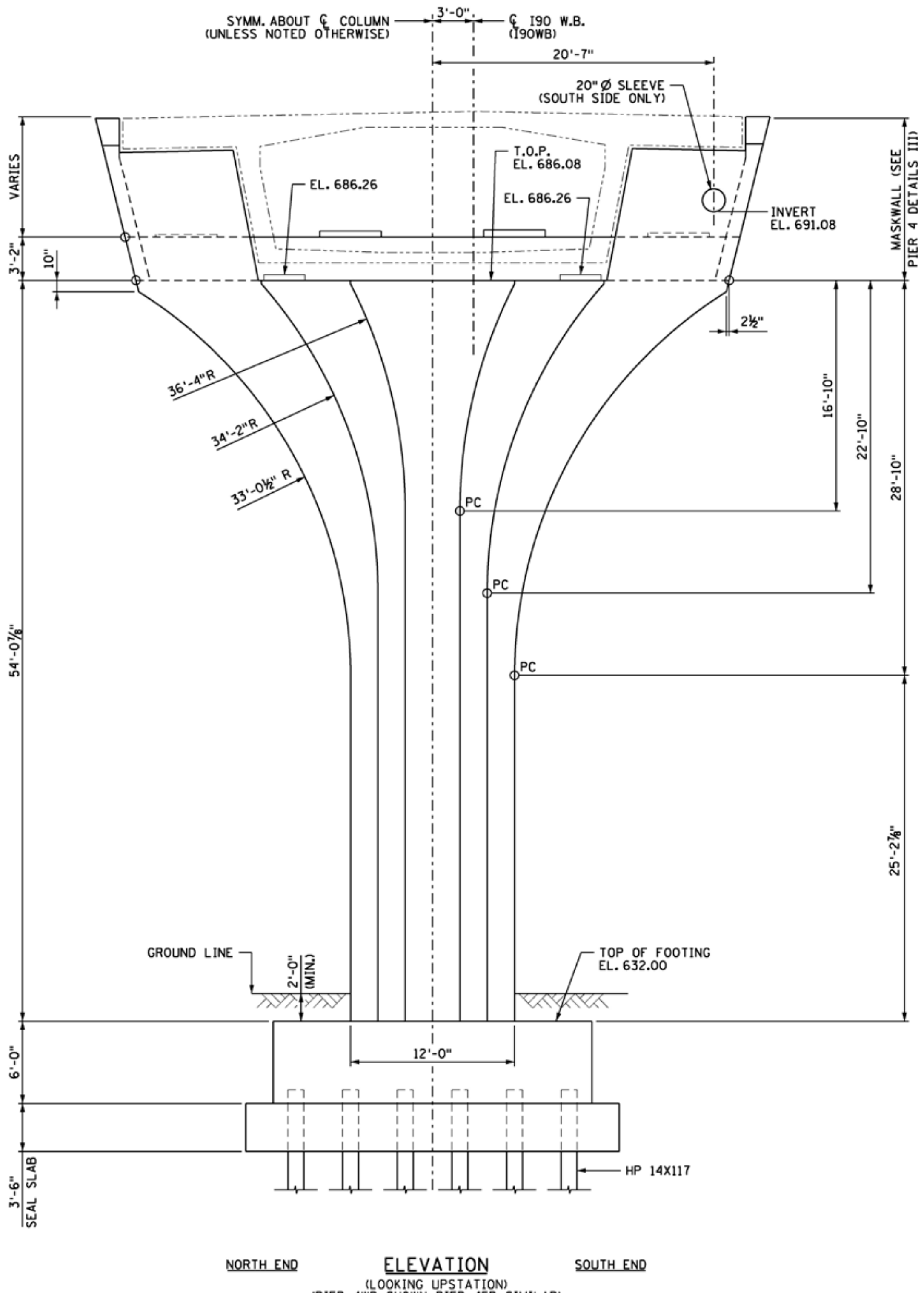


Figure 2.13 – Pier 3

FIGG



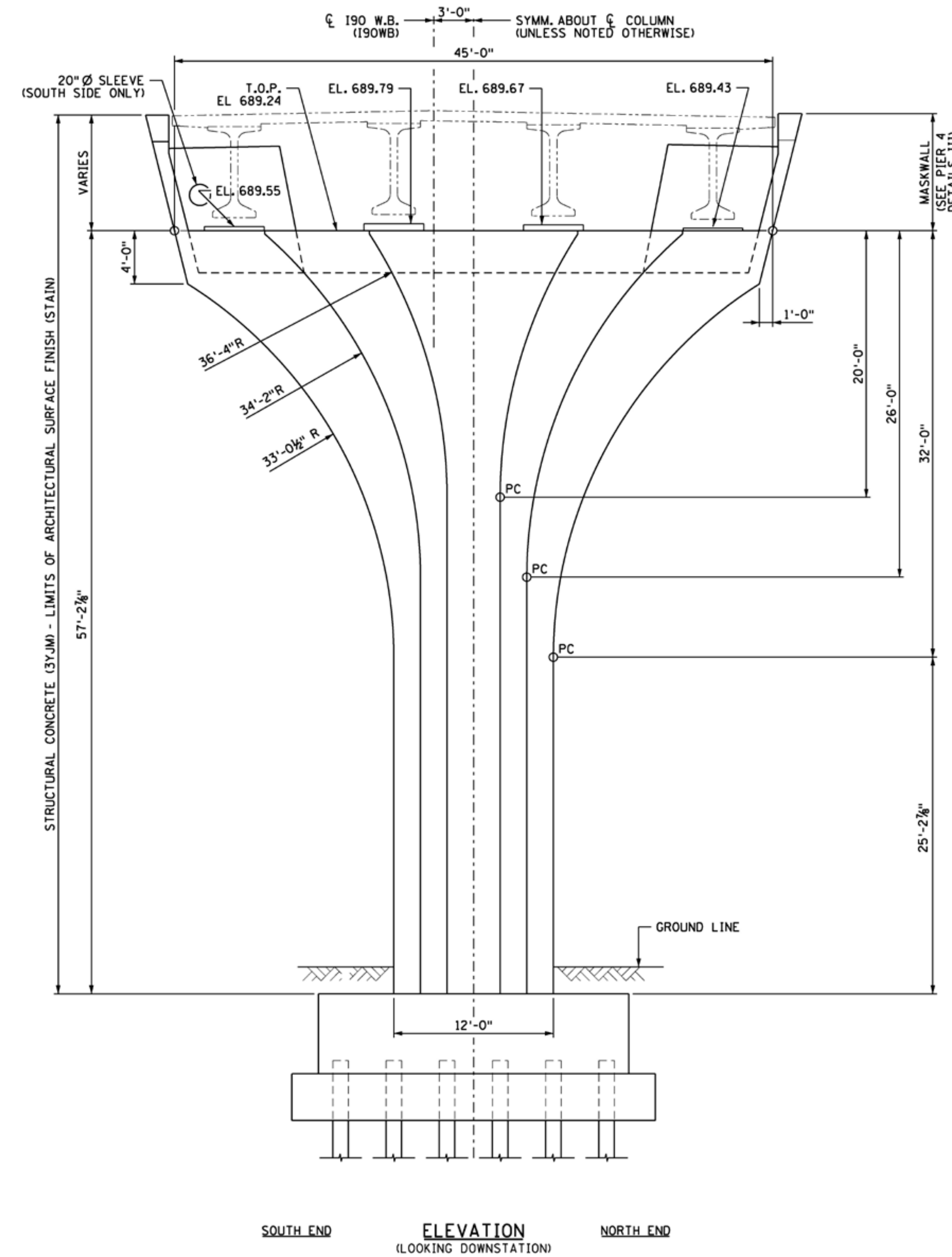


Figure 2.14 – Pier 4

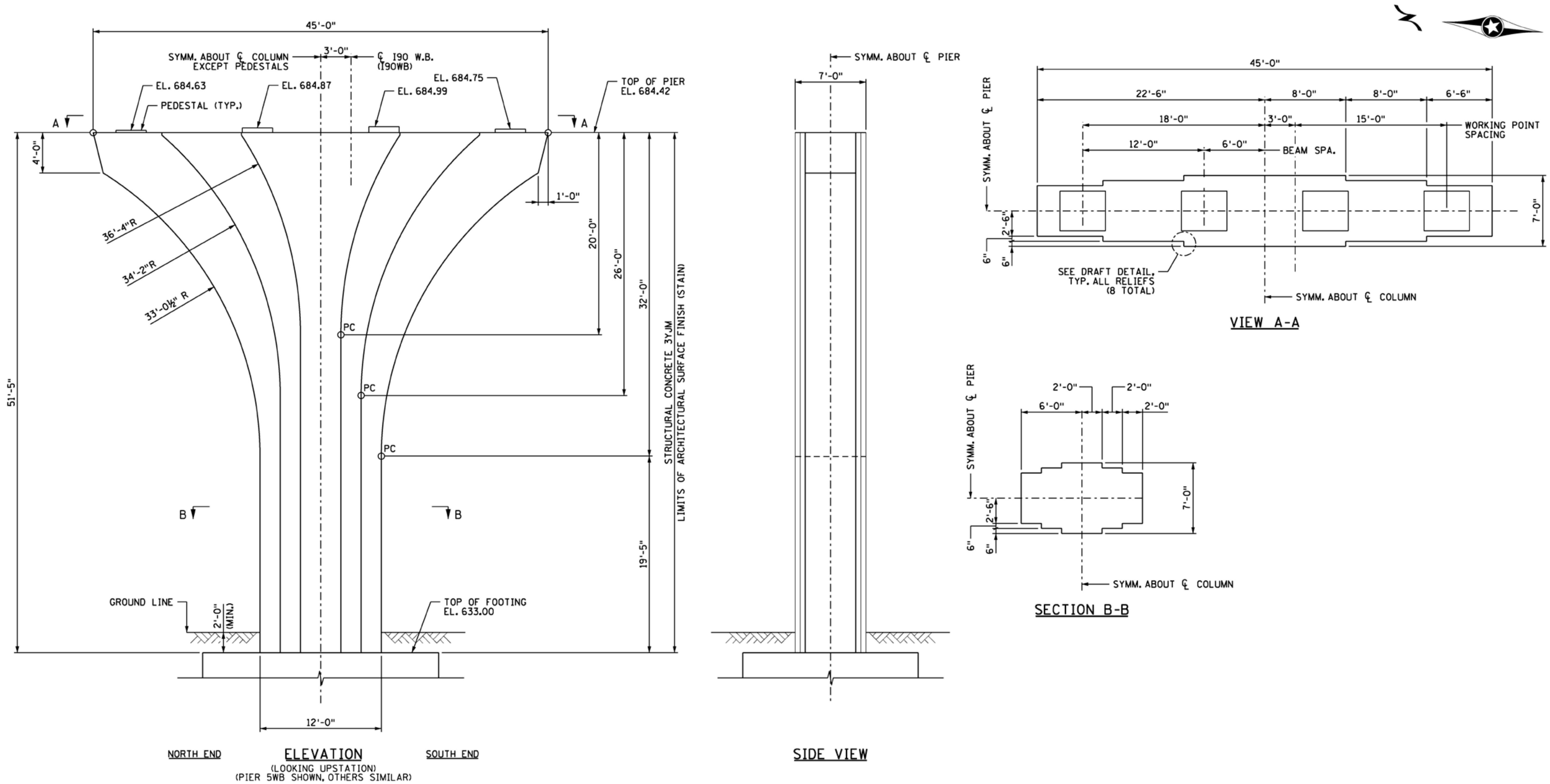


Figure 2.15 – Piers 5-9

2.3.1.3 Abutments

Each end of the westbound and eastbound bridges is supported by a single abutment. Structurally, the abutments transmit the superstructure loads to the substrata, resist lateral pressure from fill behind their backwalls and wingwalls, and support the approach slabs.

Each abutment is common to both the westbound and eastbound bridges. Both abutments utilize 16” diameter cast-in-place concrete piles to transmit loads into the underlying soil strata. The west abutment is supported on 96 piles, while the east abutment is supported on 28 piles. The west abutment is shown in Figures 2.16 and 2.17. The east abutment is shown in Figure 2.18.

The west abutment includes wingwalls approximately 32’ in length with decorative pilasters at each end. These walls tie in with the MSE walls downstation of the abutment. The east abutment includes wingwalls approximately 21’ long with a 5’-0” long pilaster connected to the abutment seat. The west abutment includes maskwalls on the front of the abutment seat that gives the appearance of the superstructure disappearing into the abutment, while restricting access to the top of the abutment seat. The west abutment contains a locked access hatch in the median via a lid slab that allows maintenance and inspection staff to access the abutment seat and the box girders.

Disc bearings supporting unit 1 are grouted integral with the bearing pedestal on the west abutment. Elastomeric bearings supporting unit 2 are grouted integral with the bearing pedestal on the east abutment.

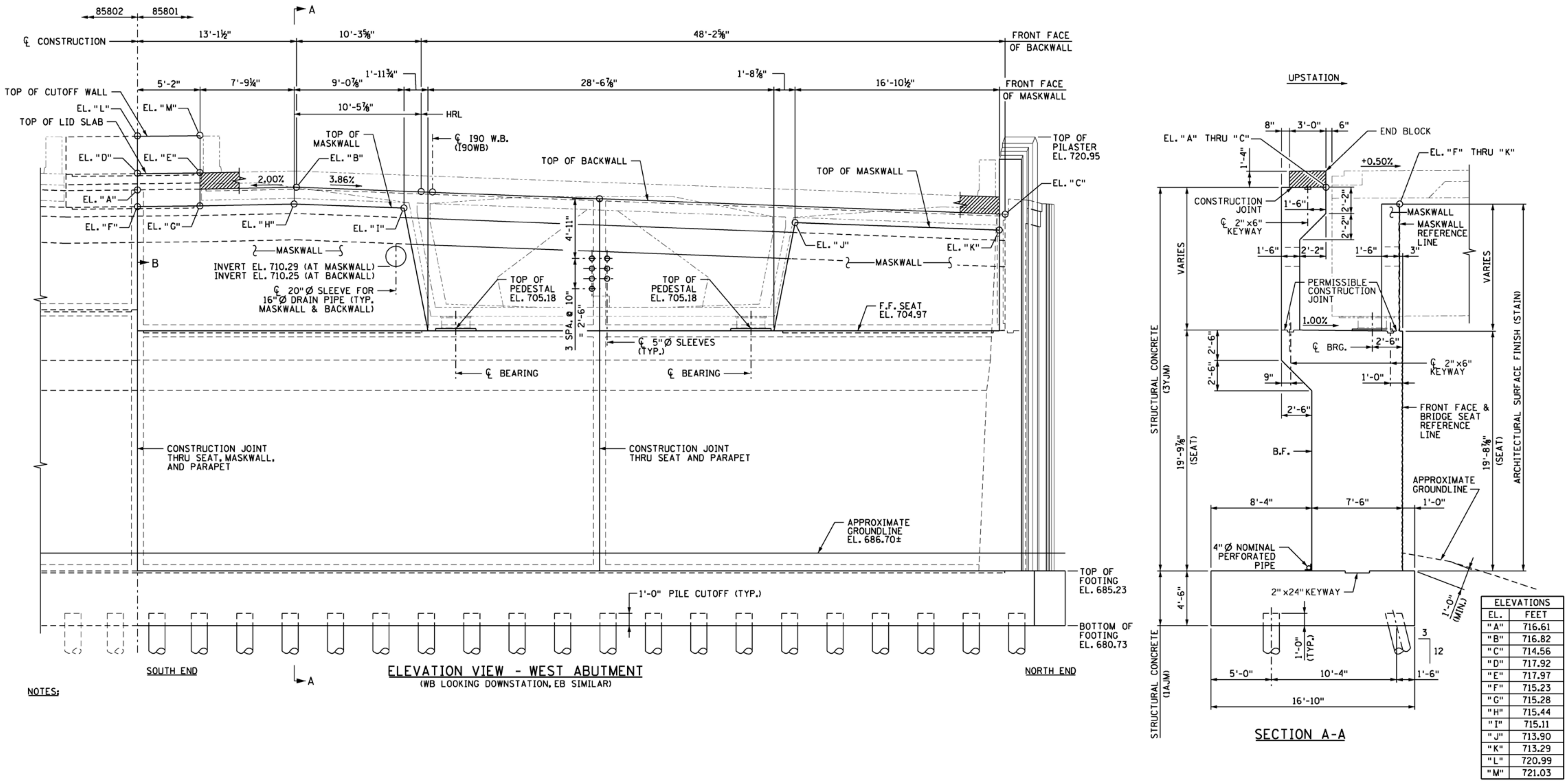
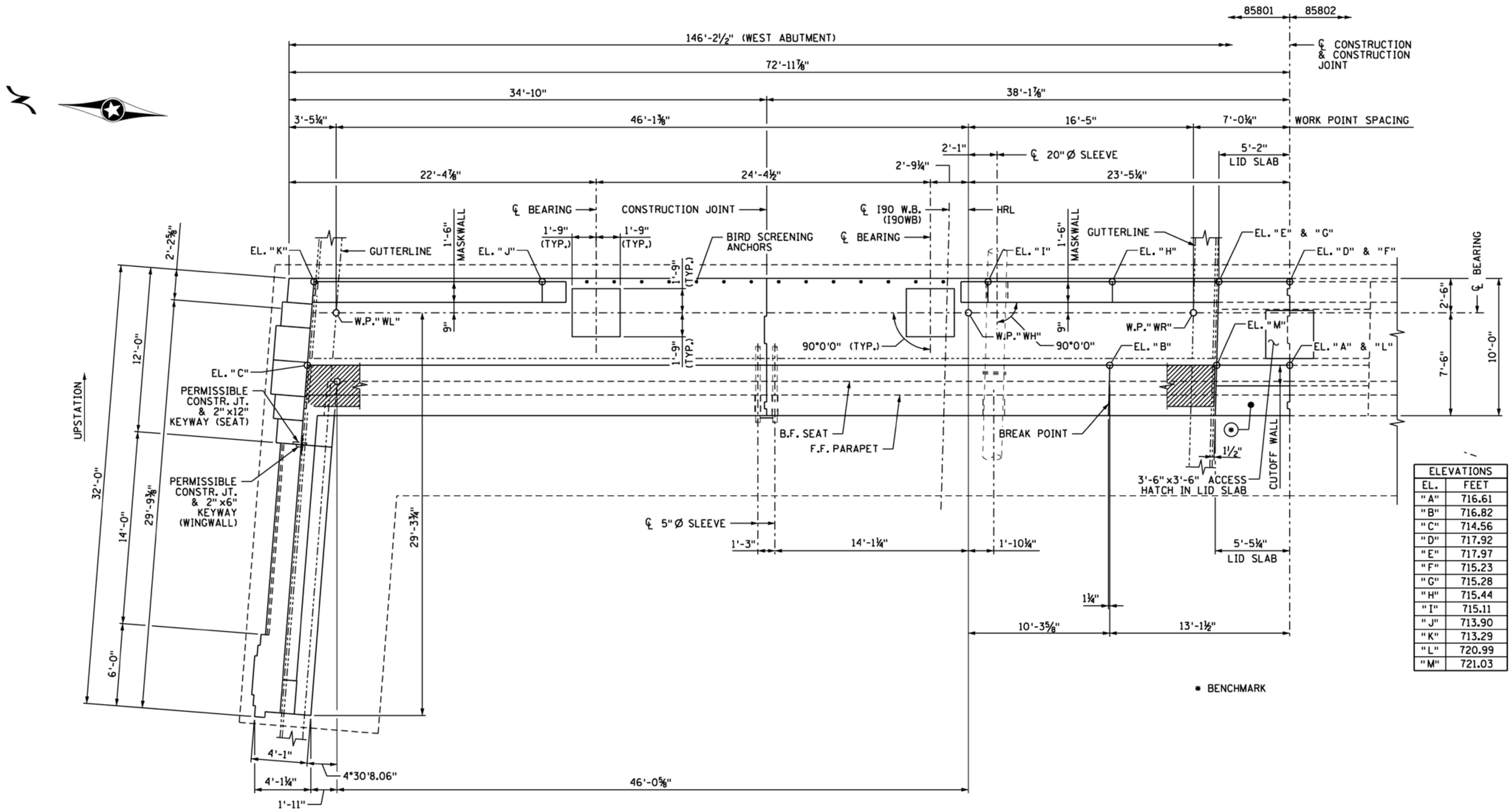


Figure 2.16 – West Abutment Elevation and Section View



ELEVATIONS	
EL.	FEET
"A"	716.61
"B"	716.82
"C"	714.56
"D"	717.92
"E"	717.97
"F"	715.23
"G"	715.28
"H"	715.44
"I"	715.11
"J"	713.90
"K"	713.29
"L"	720.99
"M"	721.03

PLAN VIEW - WEST ABUTMENT
(WB SHOWN, EB SIMILAR)

Figure 2.17 – West Abutment Plan View

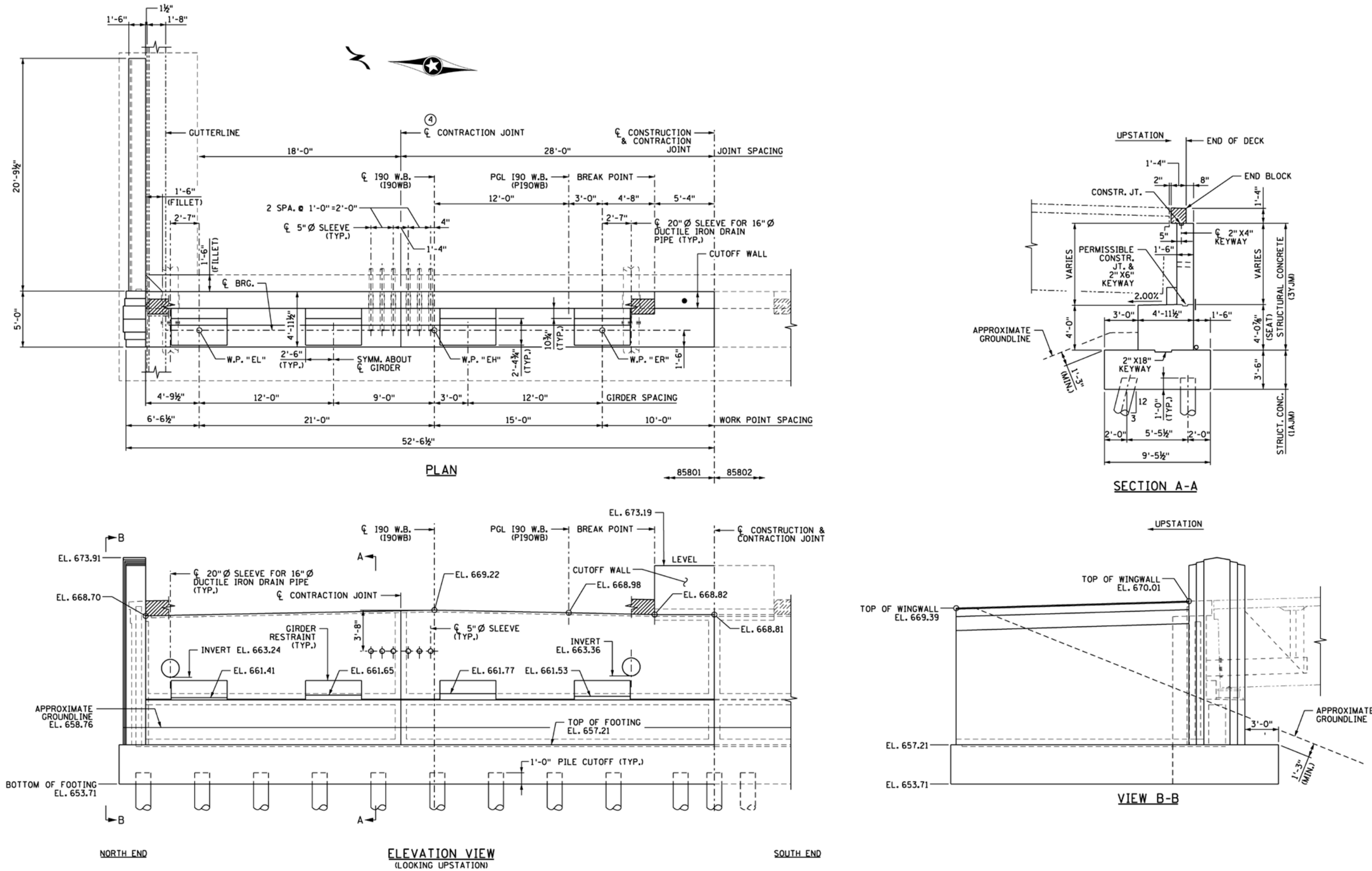


Figure 2.18 – East Abutment Elevation, Section, and Plan View

2.3.2 Bearings

All loads from the superstructure are transferred to the substructure though bearings, except at piers 1 through 3, where the superstructure is integral with the substructure. The bearings must simultaneously support the superstructure and allow for movement of the superstructure due to thermal expansion and contraction, and concrete creep and shrinkage.

2.3.2.1 Disc Bearings

There are two types of disc bearings used on unit 1 of the Interstate 90 Bridge at the Mississippi River: guided bearings and non-guided bearings.

Disc bearings are comprised of steel plates and a polyether urethane disc element that allows rotation. Sliding bearings also have plates with a PTFE (polytetrafluoroethylene) to stainless steel sliding interface. The bearings are designed using provisions to allow for future replacement, if required.

The guided bearings are located at the west abutment and at pier 4 (downstation) in the north bearing position, and are always paired transversely with a non-guided bearing. The guided bearings are designed to allow full longitudinal movement while restricting transverse movement. Steel guide bars welded to the top plate guide the direction of movement and transmit all transverse loads to the foundation. The upper bearing plate is covered with a PTFE (polytetrafluoroethylene) sheet that mates with a stainless steel sliding surface bonded to the top plate and guide bars. The stainless steel sliding surface is sized and positioned to account for the predicted structural movements.

The non-guided bearings are located in the south bearing position and are identical to the guided bearing except that there are no guide bars restricting transverse movement. They allow free movement and rely on the paired guided bearings to transmit all transverse loads.

Disc bearings are shown on the Guided and Non-Guided Disc Bearing Inspection Sheets in Appendix B.

2.3.2.2 Elastomeric Bearings

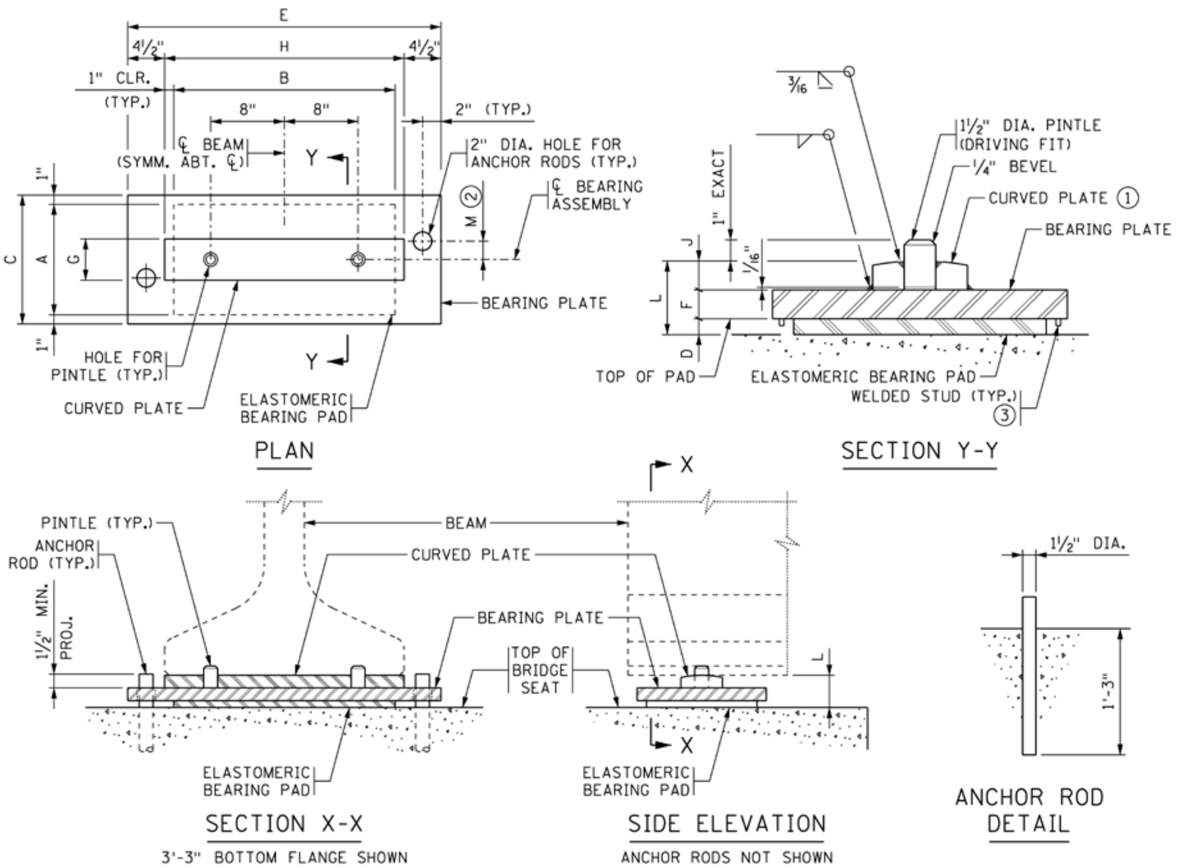
There are three types of elastomeric bearings used on unit 2 of the Interstate 90 Bridge at the Mississippi River: fixed elastomeric bearings, expansion elastomeric bearings, and sliding elastomeric bearings.

Fixed elastomeric bearings are located at piers 8 and 9. These bearings are standard MnDOT fixed bearings for prestressed concrete beams. They allow rotation of the beam ends and provide a fixed longitudinal connection to transfer longitudinal superstructure loads to the substructure.

Expansion elastomeric bearings are located at piers 5, 6, 7, and the east abutment. These bearings are standard MnDOT expansion bearings for prestressed concrete beams. They allow rotation of the beam ends and allow longitudinal movement through shear deformation of the elastomeric bearing pad. The elastomeric bearing pads consist of a neoprene matrix reinforced with steel sheets. Typically, ¾” layers of neoprene alternate with 1/8” steel sheets. Under vertical loads, the steel reinforcing sheets prevent lateral bulging of the elastomer allowing higher loading than an unreinforced pad.

Sliding elastomeric bearings are located only at pier 4 (upstation). The sliding elastomeric bearings are similar to the expansion elastomeric bearings, with the addition of a PTFE (polytetrafluoroethylene) pad on top of the elastomeric pad. The PTFE pad is bonded to a stainless steel recessed top plate, which is in turn bonded to the elastomeric pad. The PTFE side of the pad bears on a stainless steel sliding plate attached to the prestressed concrete beams. This allows unrestrained translation between the superstructure and substructure along the sliding plane between the PTFE pad and the stainless steel sliding plate.

Fixed, expansion, and sliding elastomeric bearings are shown in Figures 2.19 through 2.21.



ASSEMBLY TYPE	LOCATION	BEAM SIZE	BEARING PAD SIZE			SHAPE FACTOR	BEARING PLATE SIZE			CURVED PLATE SIZE			ANCHOR ROD OFFSET		ASSY. HEIGHT	CURVED PLATE
			A	B	D		C	E	F	G	H	J	+/- ②	M	L	R ①
F1	PIER 8	82MW	16"	36"	3/4"	7.4	18"	47"	1 3/4"	8"	38"	1 1/4"	-	0"	3 3/4"	20"
F1	PIER 9	82MW	16"	36"	3/4"	7.4	18"	47"	1 3/4"	8"	38"	1 1/4"	-	0"	3 3/4"	20"

NOTES:

ELASTOMERIC MATERIALS AND PAD CONSTRUCTION SHALL COMPLY WITH MnDOT SPEC. 3741.

ALL STEEL PLATES SHALL COMPLY WITH MnDOT SPEC. 3306.

ANCHOR RODS SHALL COMPLY WITH MnDOT SPEC. 3306. GALVANIZE PER Mn/DOT SPEC. 3394.

PINTLES SHALL COMPLY WITH MnDOT SPEC. 3309.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION PER MnDOT SPEC. 3394, EXCEPT AS NOTED.

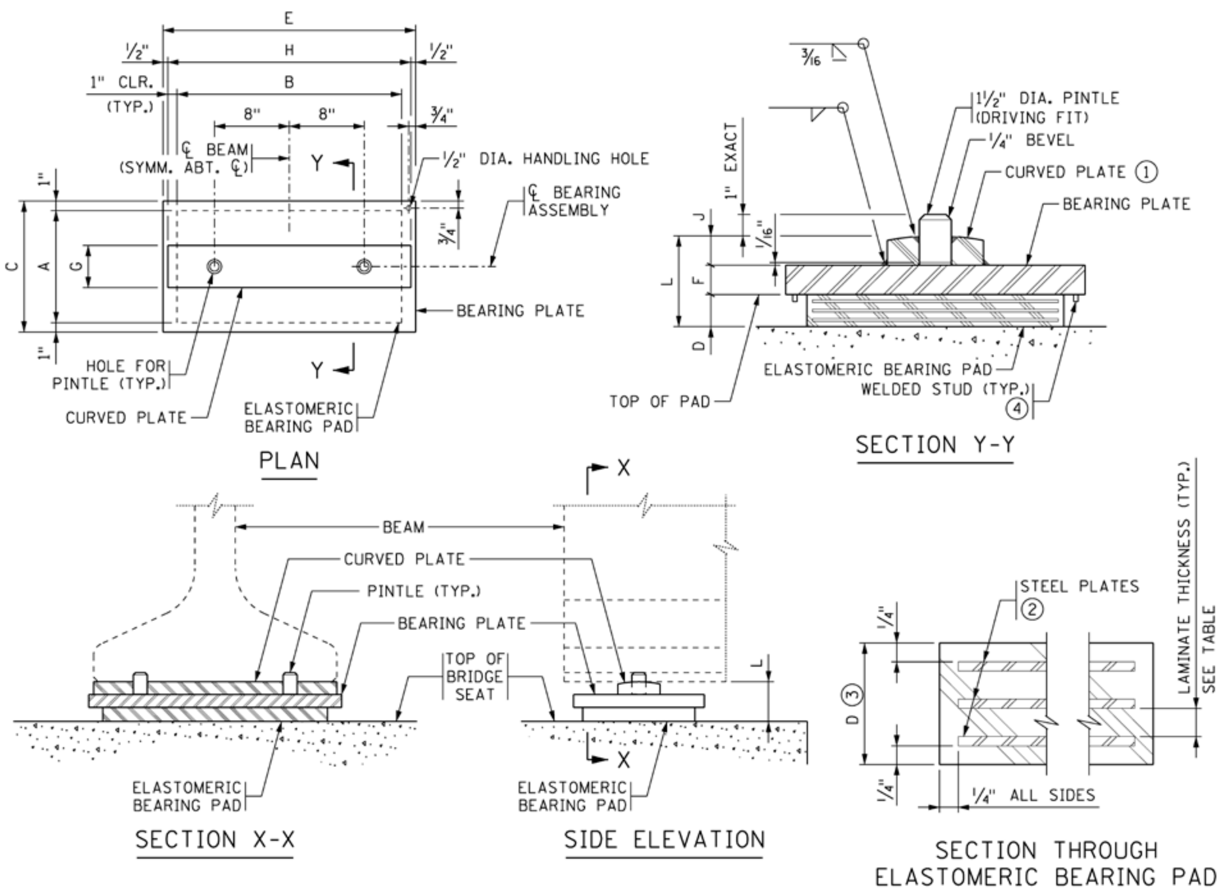
PAYMENT FOR BEARING ASSEMBLY SHALL INCLUDE ALL MATERIAL ON THIS DETAIL.

① THE MIN. RADIUS SHALL BE 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS SHALL BE 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.

② "+" DENOTES OFFSET AS SHOWN. "-" DENOTES OFFSET OPPOSITE OF SHOWN.

③ 5/16" DIA. x 3/8" KNOCK-OFF WELD STUDS INSTALLED ON BEARING PLATE AROUND PERIMETER OF BEARING PAD. CENTERLINE STUD TO EDGE OF PAD DIMENSION = 1/2", MAX. STUD SPACING = 4", AND MAX. SPACING TO PAD CORNER = 2".

Figure 2.19 – Fixed Elastomeric Bearing



TABLE

ASSEMBLY TYPE	LOCATION	BEAM SIZE	BEARING PAD SIZE			STEEL PLATES		LAMINATES		SHAPE FACTOR	BEARING PLATE SIZE			CURVED PLATE SIZE			ASSY. HEIGHT	CURVED PLATE
			A	B	D	NO.	THICK.	NO.	THICK.		C	E	F	G	H	J	L	R ①
E1	PIER 5	82MW	16"	36"	5⅞"	7	⅛"	6	¾"	7.4	18"	38"	1¾"	8"	38"	1¼"	8⅞"	20"
E1	PIER 6	82MW	16"	36"	5⅞"	7	⅛"	6	¾"	7.4	18"	38"	1¾"	8"	38"	1¼"	8⅞"	20"
E1	PIER 7	82MW	16"	36"	5⅞"	7	⅛"	6	¾"	7.4	18"	38"	1¾"	8"	38"	1¼"	8⅞"	20"
E1	ABUT.	82MW	20"	36"	7⅝"	9	⅛"	8	¾"	8.6	22"	38"	1¾"	8"	38"	1¼"	10⅝"	20"

NOTES:

ELASTOMERIC MATERIALS AND PAD CONSTRUCTION SHALL COMPLY WITH MnDOT SPEC. 3741.

ALL STEEL PLATES SHALL COMPLY WITH MnDOT SPEC. 3306.

PINTLES SHALL COMPLY WITH MnDOT SPEC. 3309.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION PER MnDOT SPEC. 3394, EXCEPT AS NOTED.

PAYMENT FOR BEARING ASSEMBLY SHALL INCLUDE ALL MATERIAL ON THIS DETAIL.

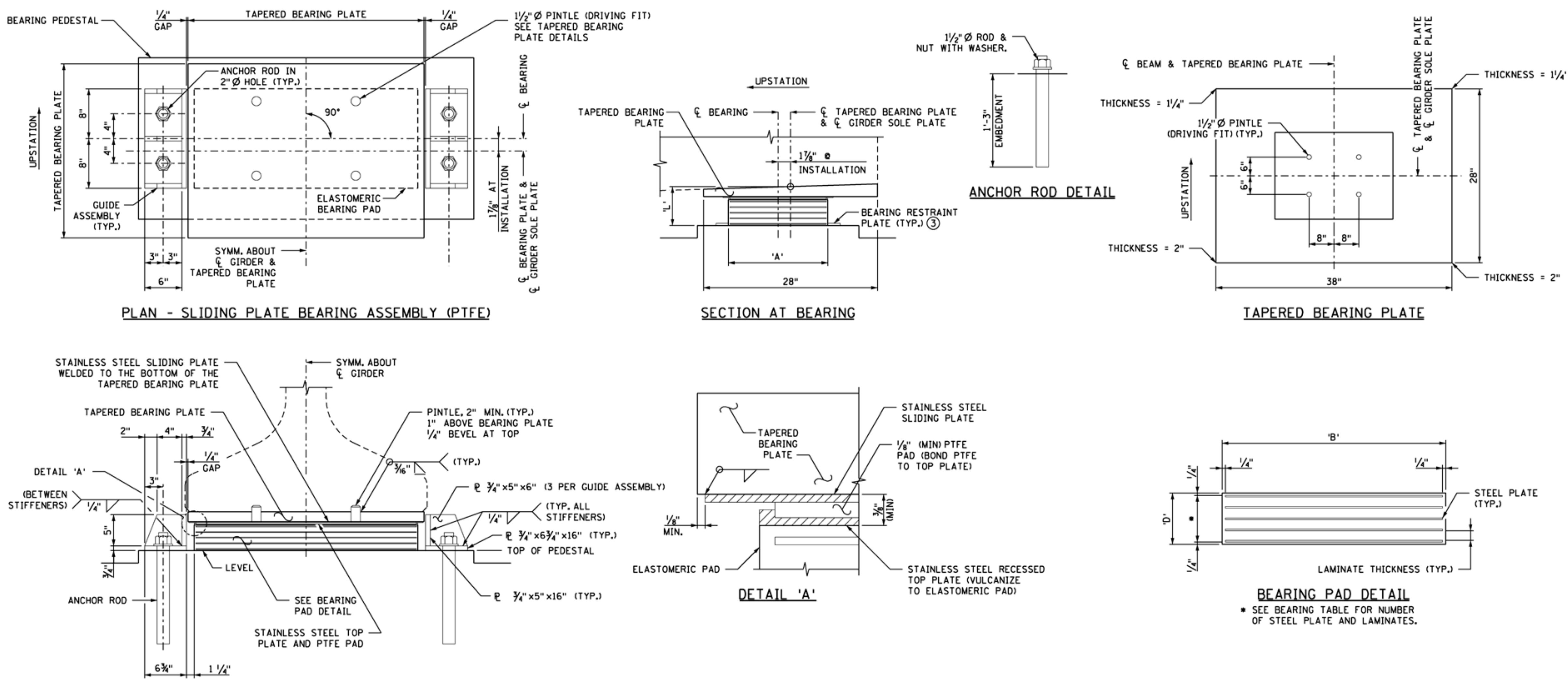
① THE MIN. RADIUS SHALL BE 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS SHALL BE 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.

② DO NOT GALVANIZE THESE PLATES.

③ THE TOTAL THICKNESS SHOWN INCLUDES THE STEEL PLATES.

④ 5/16" DIA. x ⅜" KNOCK-OFF WELD STUDS INSTALLED ON BEARING PLATE AROUND PERIMETER OF BEARING PAD. CENTERLINE STUD TO EDGE OF PAD DIMENSION = ½". MAX. STUD SPACING = 4", AND MAX. SPACING TO PAD CORNER = 2".

Figure 2.20 – Expansion Elastomeric Bearing



ELEVATION - SLIDING PLATE BEARING ASSEMBLY (PTFE)

LOCATION	NO.	VERTICAL DEAD LOAD (DC+DW) PER BEARING (KIPS)	MINIMUM VERTICAL SERVICE LIMIT LOAD PER BEARING (KIPS)	MAXIMUM VERTICAL SERVICE LIMIT LOAD PER BEARING (KIPS)	MAXIMUM LATERAL SERVICE LIMIT LOAD PER BEARING (KIPS)	LRFD 14.7.9.2 STRENGTH DESIGN LATERAL LOAD (KIPS)	MAXIMUM SERVICE ROTATION (RADIANS)	LONGITUDINAL RANGE OF STRUCTURE MOVEMENT (IN) (+ DENOTES UPSTATION MOVEMENT)				
								CREEP & SHRINKAGE	75°F TEMP. FALL	TOTAL CONTRACTION (4)	75°F TEMP. RISE	TOTAL EXPANSION (5)
PIER 4 (U.S.)	4	247	247	397	3	16	0.01	+2.8	+3.5	+7.4	−3.5	−3.5

BEARING TABLE															
ASSEMBLY TYPE	LOCATION	GIRDER	TYPE	BEAM FLANGE WIDTH	BEARING PAD SIZE			STEEL PLATES		LAMINATES		SHAPE FACTOR	TAPERD BEARING PLATE SIZE			ASSY. HEIGHT
					'A'	'B'	'D' (2)	NO.	THICK.	NO.	THICK.		WID.	LEN.	'J' (1)	'L' (1)
S1	PIER 4 (U.S.)	ALL	PTFE	39"	16"	36"	4⅛"	5	⅛"	4	¾"	7.4	38"	28"	1⅝"	6¼"

NOTES:

ELASTOMERIC MATERIALS AND PAD CONSTRUCTION SHALL COMPLY WITH MnDOT SPEC. 3741.

ALL STEEL PLATES SHALL COMPLY WITH MnDOT SPEC. 3306.

ALL STAINLESS STEEL PLATES SHALL COMPLY WITH MnDOT SPEC. 3312.

STAINLESS STEEL SLIDING SURFACE SHALL COMPLY WITH THE PROJECT SPECIAL PROVISIONS.

ANCHOR RODS SHALL COMPLY WITH MnDOT SPEC. 3306. GALVANIZE PER MnDOT SPEC. 3394.

PINTLES SHALL COMPLY WITH MnDOT SPEC 3309.

GALVANIZE GUIDE ASSEMBLIES AFTER FABRICATION PER MnDOT SPEC. 3394 EXCEPT AS NOTED.

PTFE PADS SHALL BE UNFILLED, CONFORMING TO ASTM D4894 OR D4895.

NOTES (CONT.):

TAPERED BEARING PLATE SHALL BE GALVANIZED PER MnDOT SPEC. 3394. TOUCH UP GALVANIZATION AFTER WELDING STAINLESS STEEL.

PAYMENT FOR BEARING ASSEMBLY (PTFE) SHALL INCLUDE ALL MATERIAL ON THIS DETAIL, FABRICATED AND IN PLACE.

THIS DRAWING DEPICTS A SCHEMATIC OF THE BEARING DEVICES REQUIRED AT PIER 4 (U.S.). THE DESIGN OF THE BEARINGS IS THE RESPONSIBLITIY OF THE BEARING SUPPLIER. SHOP DRAWINGS FOR THE SLIDING BEARINGS SHALL BE SUBMITTED FOR REVIEW BY THE ENGINEER.

MATERIALS, DESIGN, AND FABRICATION PER THE PROJECT SPECIAL PROVISIONS.

NOTES (CONT.):

THE BEARINGS REQUIRED AT PIERS 5 THROUGH 9 AND THE EAST ABUTMENT ARE SHOWN ON MnDOT STANDARD BRIDGE DETAILS B310 AND B311. SEE STANDARD BRIDGE DETAILS IV SHEET.

(1) DIMENSIONS 'J' AND 'L' ARE THICKNESS OF TAPERED BEARING PLATE AND BEARING ASSEMBLY AT CENTERLINE OF TAPERED BEARING PLATE ASSUMED WHEN COMPUTING PIER ELEVATIONS. CONTRACTOR SHALL ADJUST PIER AND/OR PEDESTAL HEIGHT TO ACCOUNT FOR ACTUAL SUPPLIED BEARING HEIGHTS.

(2) THE TOTAL THICKNESS SHOWN INCLUDES THE STEEL PLATES.

(3) SEE BEARING DETAILS IV FOR BEARING RESTRAINT PLATE DETAILS.

(4) 1.3TU + CR + SH

(5) 1.0TU

BEARING PLACEMENT:

PIER 4 (U.S.): PLACE BEARING PADS AS SHOWN ON PIER DETAILS SHEET, NO ADJUSTMENTS FOR TEMPERATURE SHALL BE MADE.

Figure 2.21 – Sliding Elastomeric Bearing

2.3.3 Superstructure

The unit 1 superstructure consists of a post-tensioned single-cell concrete box girder. The unit 2 superstructure consists of four prestressed concrete beams with a conventionally reinforced deck.

2.3.3.1 Box Girder

The unit 1 superstructure is a single-cell, variable depth, cast-in-place segmental post-tensioned concrete box girder. The westbound box girder cross-section, shown in Figure 2.22, has a typical width of 45'-4". The eastbound box girder cross-section, shown in Figure 2.23, has a typical width of 53'-4". Both structures vary in depth from 25'-0" at the piers to 11'-0" at midspan and in the cast-in-place on falsework end sections. The box girder is continuous along the entire length of unit 1 with modular expansion joints at the west abutment and pier 4.

In addition to the typical box girder section, anchor blocks, deviation diaphragms, expansion joint diaphragms and pier table diaphragms are included within the interior of the box girder to transfer post-tensioning forces to the box girder and superstructure loads into the bearings and twin wall piers.

The anchor blocks, shown in Figure 2.24, are located in the interior bottom corners of the segments and transmit the bottom slab post-tensioning anchorage forces to the box girder. In the westbound structure they are located in the following segments: W1-10D, W1-9U, W1-10U, W1-12U, W1-13U, W2-13D, W2-12D, W2-10D, W2-9D, W2-9U, W2-10U, W2-12U, W2-13U, W3-13D, W3-12D, W3-10D, W3-9D, W3-10U,W3-12U and in the span 4 cast-in-place end section. In the eastbound structure the anchor blocks are located in the following segments: E1-12D, E1-10D, E1-9U, E1-10U, E1-12U, E1-13U, E2-13D, E2-12D, E2-10D, E2-9D, E2-9U, E2-10U, E2-12U, E2-13U, E3-13D, E3-12D, E3-10D, E3-9D, E3-10U, E3-12U, and in the spans 1 and 4 cast–in-place end sections.

The deviation diaphragms, shown in Figure 2.25 and Figure 2.26, provide a means to establish the draped post-tensioning tendon geometry and transfer a portion of the gravity loads in the box girder to the post-tensioning tendons. The draped tendons pass through curved steel pipes cast into the deviation diaphragms. There are two types of deviation diagrams, type I and type II, located in both westbound and eastbound unit 1. In the westbound structure, the type I deviation diaphragms are located in segments W1-11D, W1-11U, W2-11D, W2-11U, W3-11D, and W3-11U and the type II deviation diaphragms are located in segment W1-14D and in the span 4 cast-in-place end section. In the eastbound structure, the type I deviation diaphragms are located in segments E1-11D, E1-11U, E2-11D, E2-11U, E3-11D, and E3-11U and the type II deviation diagrams are located in segment E1-14D and in the span 4 cast-in-place end section. The type II deviation diaphragms are transversely post-tensioned across the bottom using 1-3/8" diameter P.T. bars.

The expansion joint diaphragms (Figure 2.27 and Figure 2.28) are located at the ends of unit 1. The type I expansion joint diaphragm is located over pier 4 and the type II expansion joint diaphragm is located over the west abutment. The diaphragms transfer the superstructure loads into the bearings and also transfer the post-tensioning anchorage forces into the box girder. The diaphragms feature a blockout in the top that contains the modular expansion joint. The diaphragms are post-tensioned vertically with 1-3/8” diameter P.T. bars and transversely with both 4x0.6” diameter strand tendons and 12x0.6” diameter strand tendons.

The pier table diaphragms (Figure 2.29) are located over piers 1-3 and consist of twin, 5’-0” thick walls integral with each twin wall pier. There are portals though each diaphragm wall to allow inspection access through the diaphragms between spans. The diaphragms transmit the superstructure loads to the piers and also transfer the post-tensioning anchorage forces into the box girder. The pier table diaphragms are post-tensioned vertically with 1-3/8” diameter P.T. bars and transversely with 4x0.6” diameter, 12x0.6” diameter, and 19x0.6” diameter strand tendons.

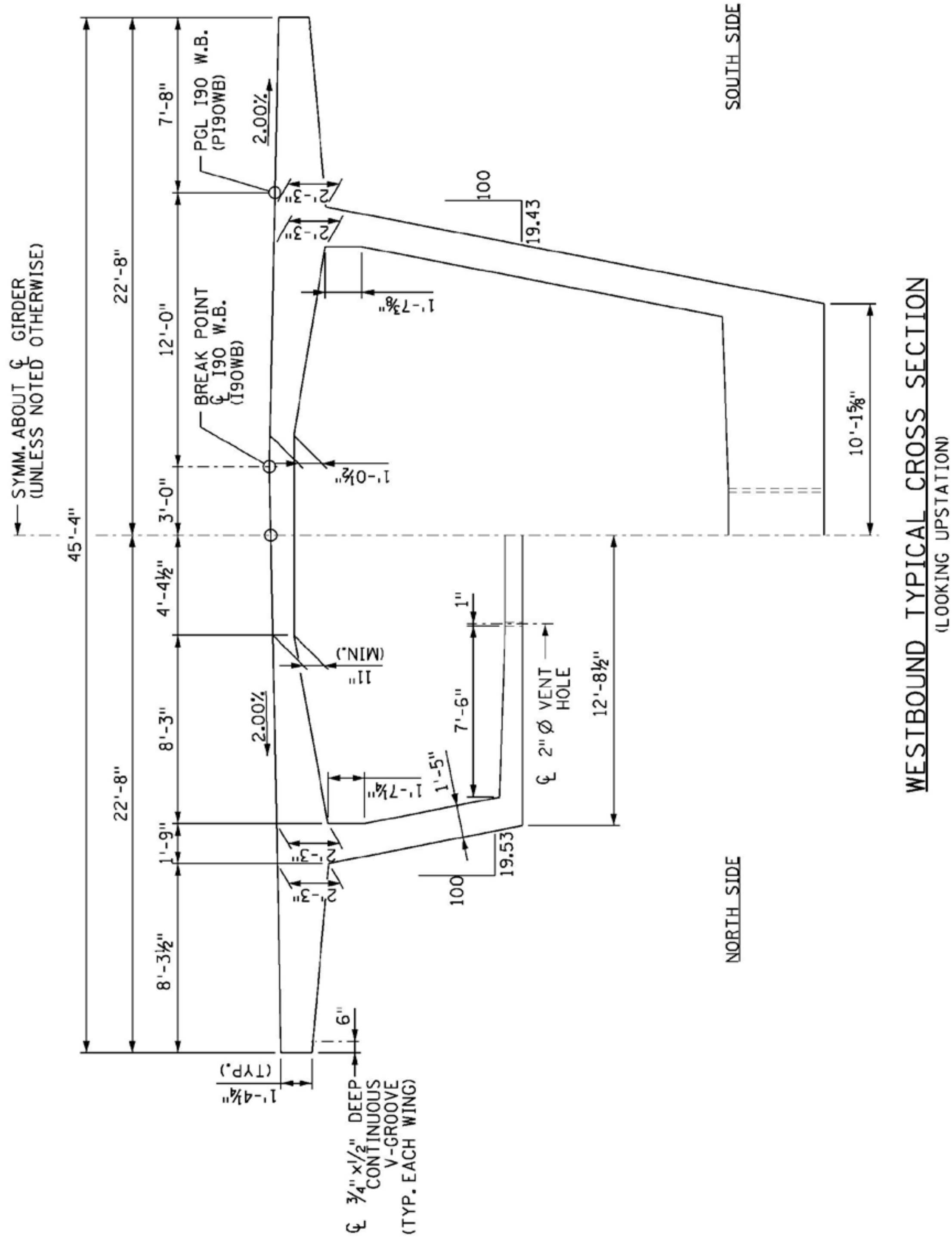


Figure 2.22 – Westbound Typical Box Girder Cross-Section

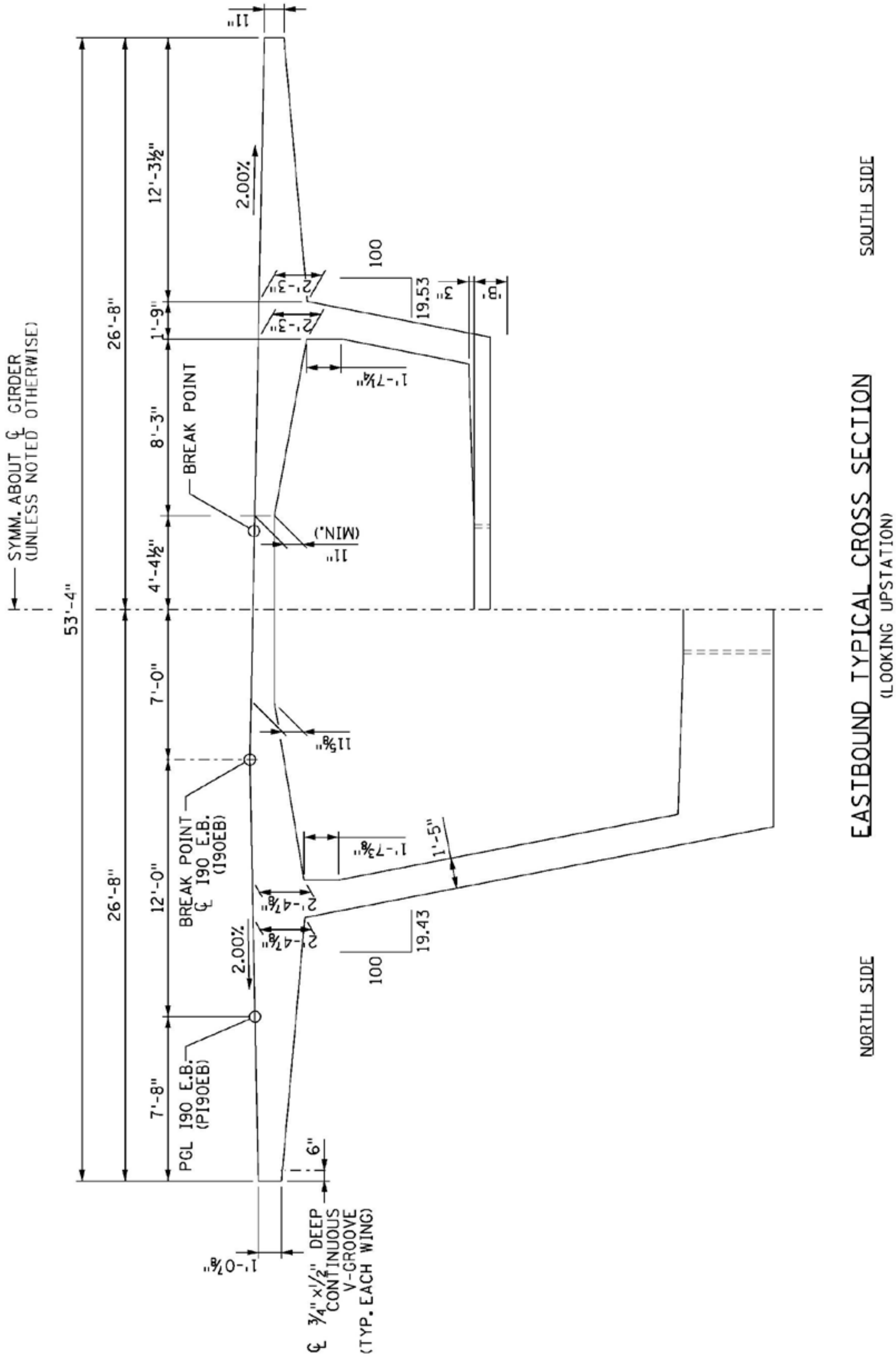


Figure 2.23 – Eastbound Typical Box Girder Cross-Section

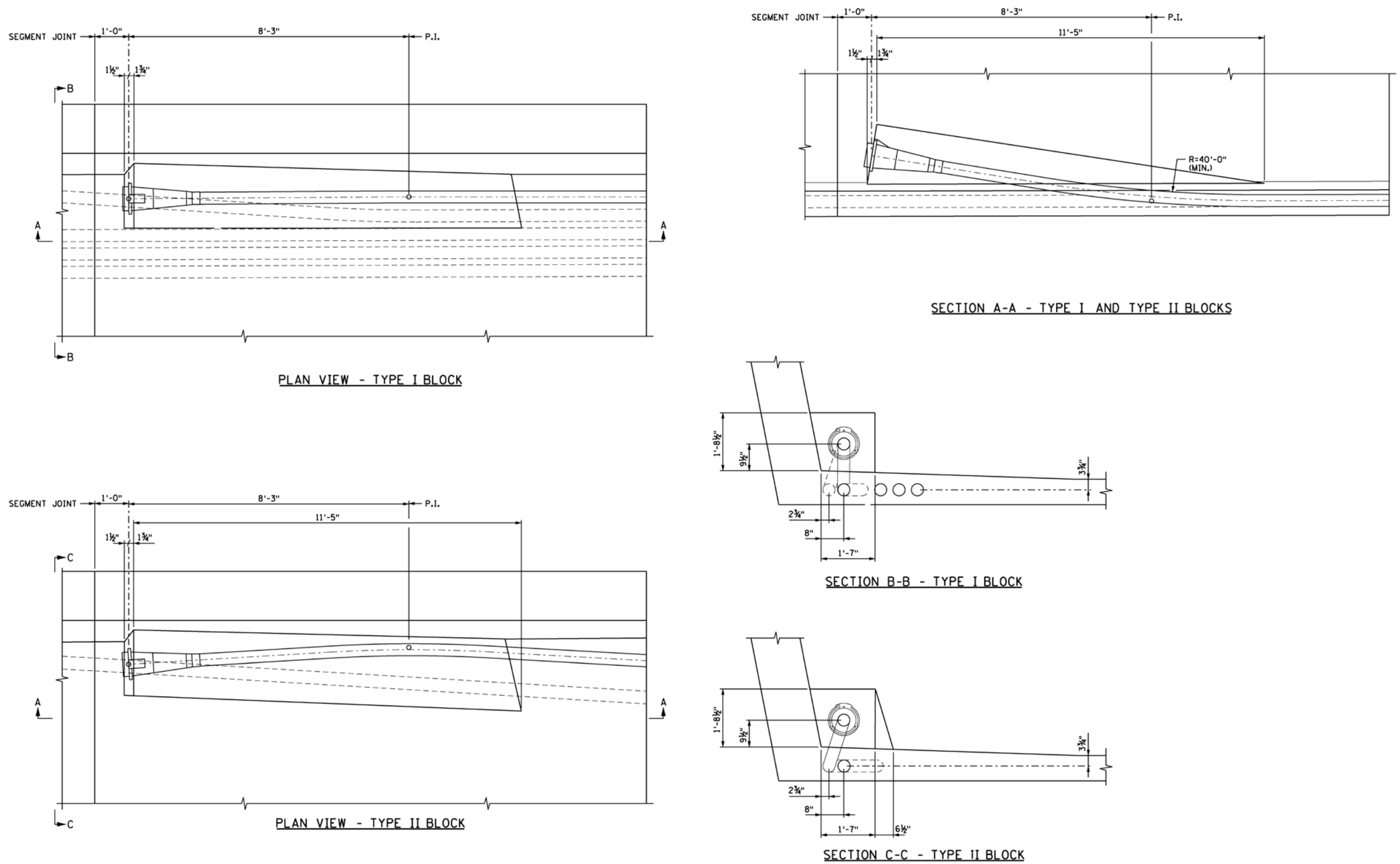


Figure 2.24 – Anchor Block

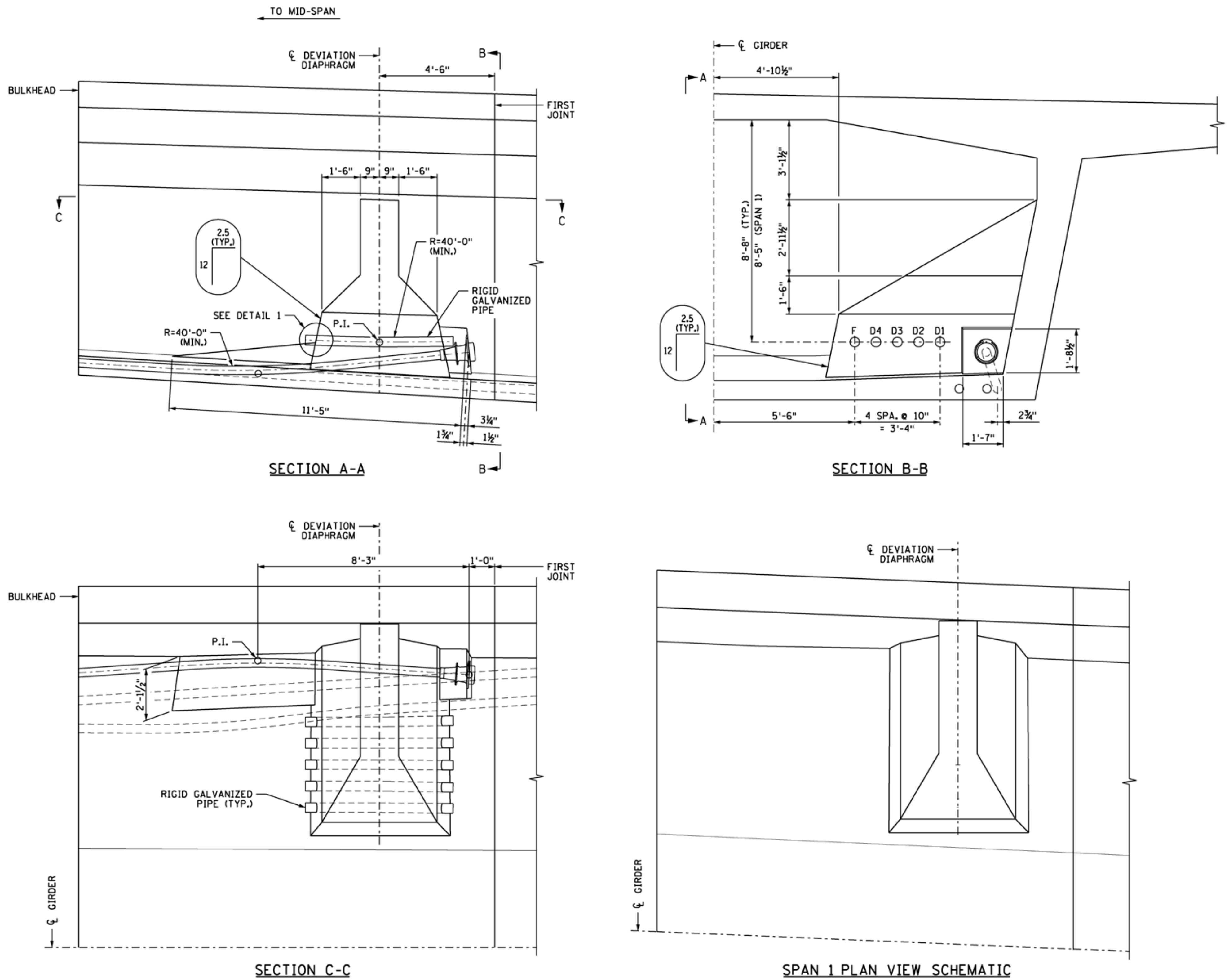


Figure 2.25 – Type I Deviation Diaphragm

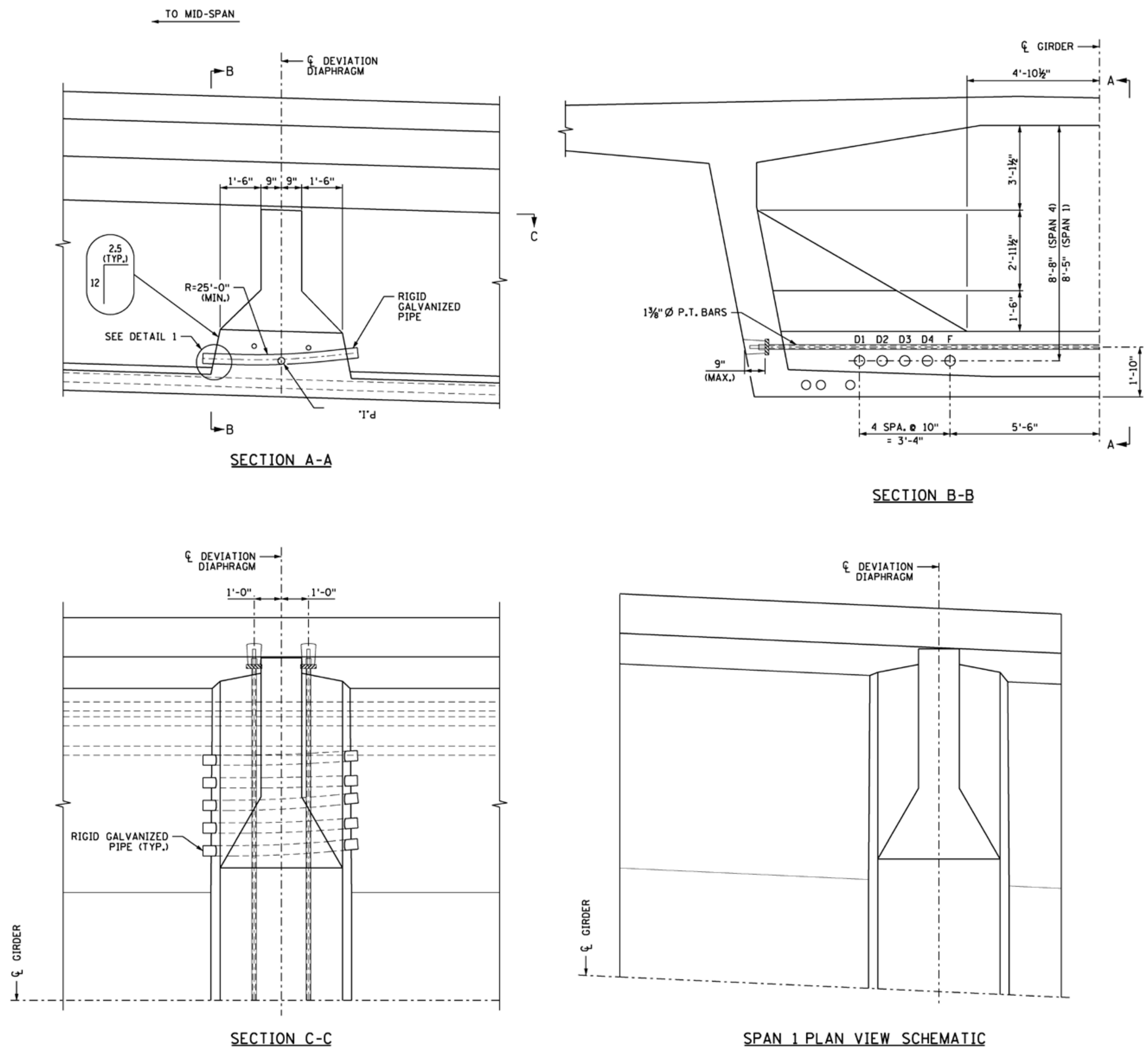


Figure 2.26 – Type II Deviation Diaphragm

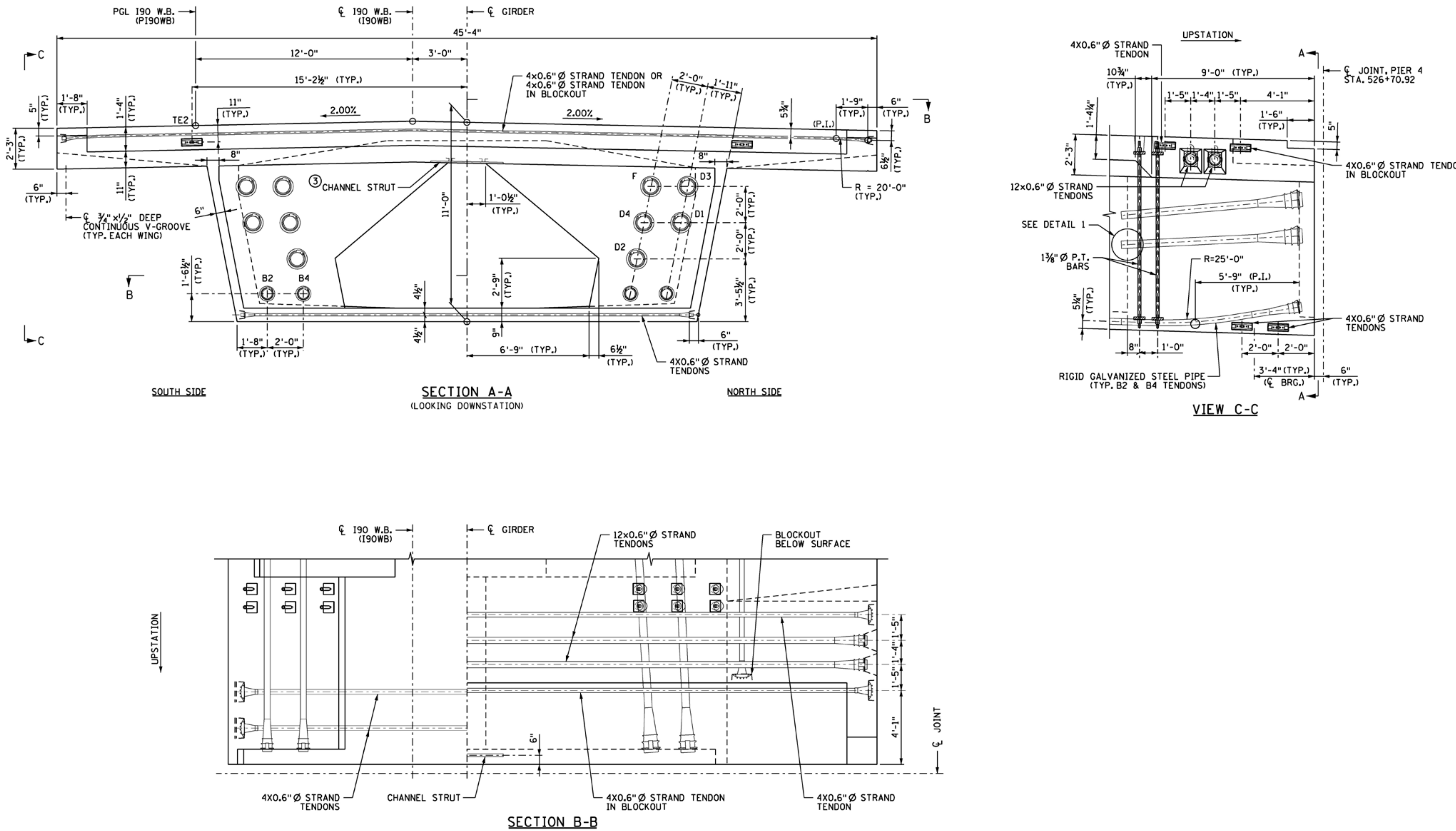


Figure 2.27 – Type I Expansion Joint Diaphragm

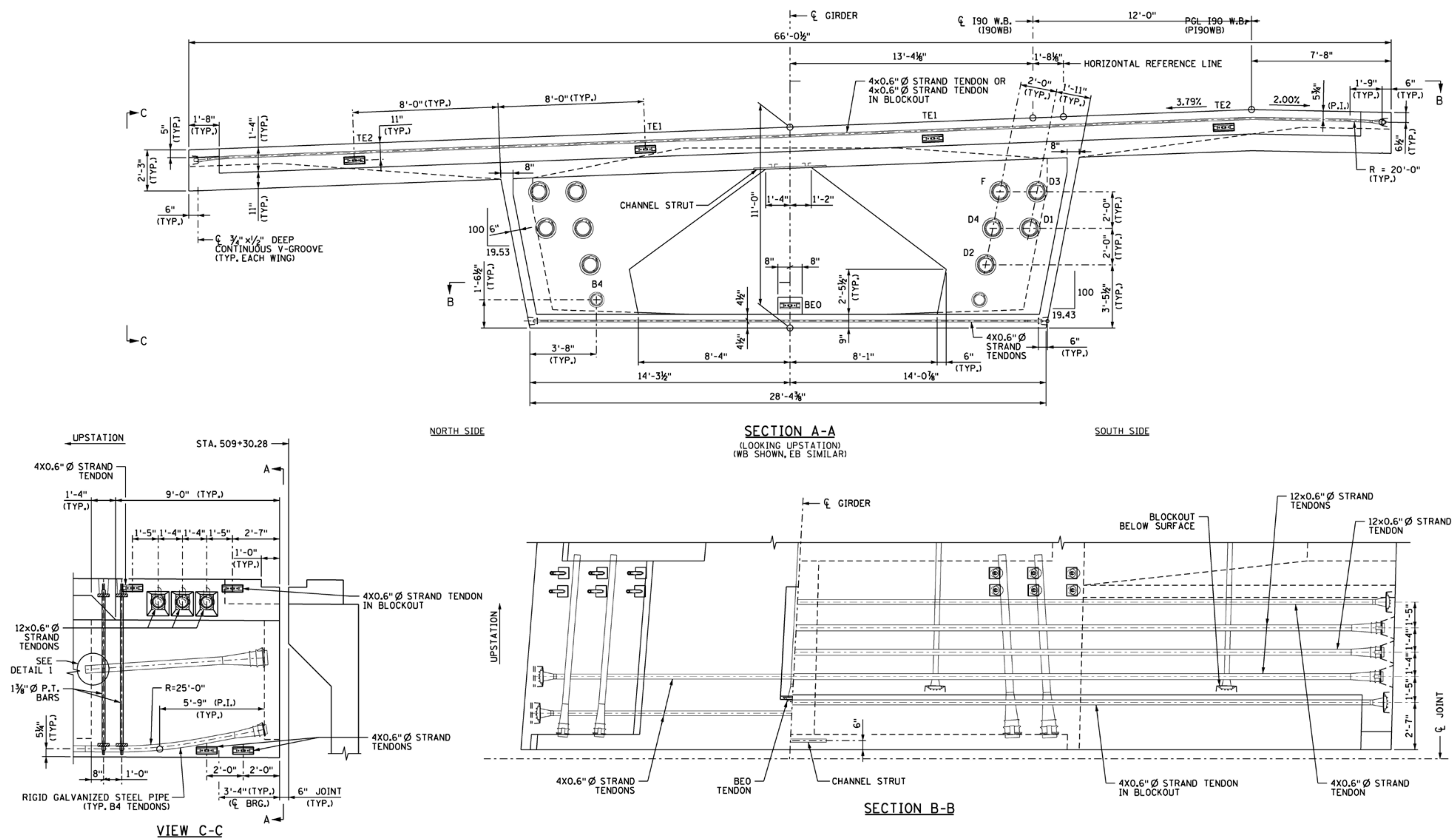


Figure 2.28 – Type II Expansion Joint Diaphragm

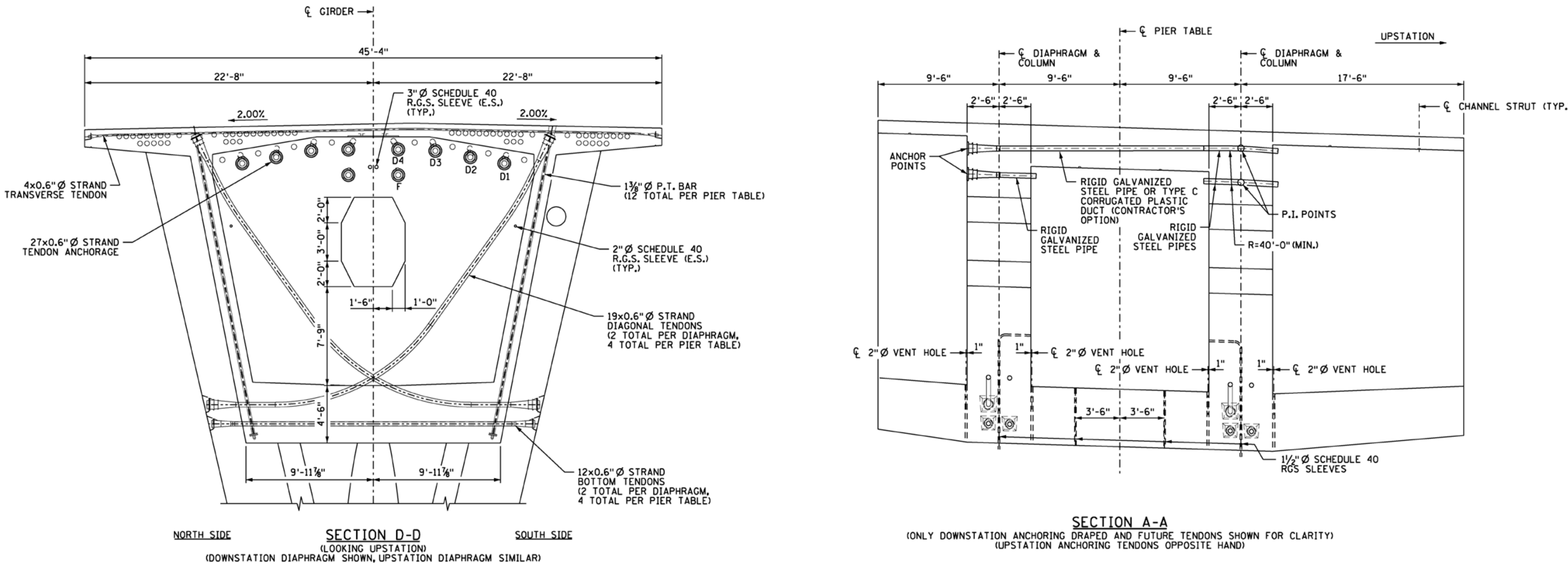


Figure 2.29 – Pier Table Diaphragm

2.3.3.2 Post-Tensioning System

Post-tensioning is used to apply a compressive force to the concrete after placement and sufficient strength gain. This compression serves to counteract tensile stresses in the concrete due to self-weight and external loads, thus reducing or preventing cracking. Post-tensioning also serves as the primary flexural reinforcing in the box girder in the event that the bridge is loaded past its service capacity.

Post-tensioning is provided either by tendons, consisting of multiple 0.6” diameter seven-wire strands, or 1-3/8” diameter post-tensioning bars. Some tendons are internal to the concrete section, and some are external to the concrete, protected by high-density polyethylene (HDPE) ducts located within the box girder cell. Bars are always cast internal to the concrete section. Ducts cast into the concrete allow for the installation of strand tendons. Anchorages at the ends of both internal and external tendons transfer tendon force to the concrete. Three-part steel wedges secure the individual strands to the anchor head, which bears on a circular bearing plate cast into the concrete. The anchorage for a threaded post-tensioning bar consists of a threaded nut bearing on a flanged, circular bearing plate.

The Interstate 90 Bridge at the Mississippi River is post-tensioned longitudinally and transversely for its full length, and vertically in the expansion joint diaphragms, pier table diaphragms, and a portion of the eastbound span 1 cast-in-place end section adjacent to the west abutment. Longitudinal post-tensioning consists of cantilever, draped, and bottom slab tendons. The 19x0.6” diameter strand cantilever tendons run internal to the top deck and are anchored at the segment joints. Pairs of cantilever tendons are stressed with each segment cast and are centered over the pier. Bottom slab tendons are also 19x0.6” diameter strand tendons and run internal to the bottom slab and are anchored in anchor blocks. The main span bottom slab tendons are centered in both main spans and anchor in segments to either side. The end span bottom slab tendons are approximately centered on the end span closure and anchor in the segments to one side and in the cast-in-place end section on the other side. Draped tendons are 27x0.6” diameter strand tendons and run external to the concrete section, but inside the box girder cell. They anchor high in the pier tables and end diaphragms, run down through the deviation diaphragms, and low between deviation diaphragms across the center of each span.

Plan views of the cantilever and bottom slab tendons, as well as an elevation view of the draped tendons are provided in Figures 2.26 through 2.43.

After the post-tensioning is stressed, grout is injected into the ducts to provide corrosion protection. Additionally, the grout provides a structural bond between the strand/bar and the concrete section for internal post-tensioning and to bond the tendon/bar with the post-tensioning duct. Anchorages for all bottom slab and draped tendons are capped, grouted, and then capped with a secondary epoxy grout pourback that is sealed with two layers of elastomeric membrane. Anchorages for the cantilever tendons are capped, grouted and embedded with the following segment pour concrete. The top anchorage for the PT bars is poured back flush with the deck with an epoxy grout, while the bottom anchorage is embedded in the concrete. Grouting and anchorage protection details are illustrated in Figures 2.44 and 2.45.

Typical transverse post-tensioning consists of 4x0.6” diameter strand flat tendons internal to the bridge deck spaced approximately on 2’-8 1/2” centers (6 tendons per typical segment). The anchorages are cast in blockouts located in the wing tips that allow for stressing access. After stressing the tendons, the anchorages are capped, the blockouts poured back, and the tendons grouted. Transverse post-tensioning details are provided in Figure 2.46.

Additional transverse post-tensioning is present in special bridge elements, such as pier tables, end diaphragms, and deviators. For example, pier tables contain 12x0.6” and 19x0.6” diameter strand tendons in the diaphragm, end diaphragms contain 12x0.6” diameter strand tendons in the top slab adjacent to the expansion joint, and deviators in end spans use 1-3/8” diameter PT bars running transversely in the deviator rib. Additional details can be found in the Contract Plans.

2.3.3.3 Future Post-Tensioning System

Provisions for additional future post-tensioning tendons have been included in the design and are shown in Figures 2.29 through 2.32 and Figures 2.36 through 2.39. This future post-tensioning is included to provide additional post-tensioning force, if desired, during the life of the bridge to compensate for higher design loads. The future post-tensioning is required to be installed if the pedestrian bridge supported from the westbound and eastbound bridge is built.

The future tendons are 27x0.6” diameter strand draped tendons. They anchor high in the pier tables and end diaphragms, run down through the deviation diaphragms, and low between deviation diaphragms across the center of each span. It is recommended that HDPE ducts be used around the external portions of the future tendons, similar to the existing draped tendons. The anchorage hardware and deviation pipes are already in place, and the tendons only need to be installed, stressed, grouted and anchorage protection provided similar to the existing draped tendons.

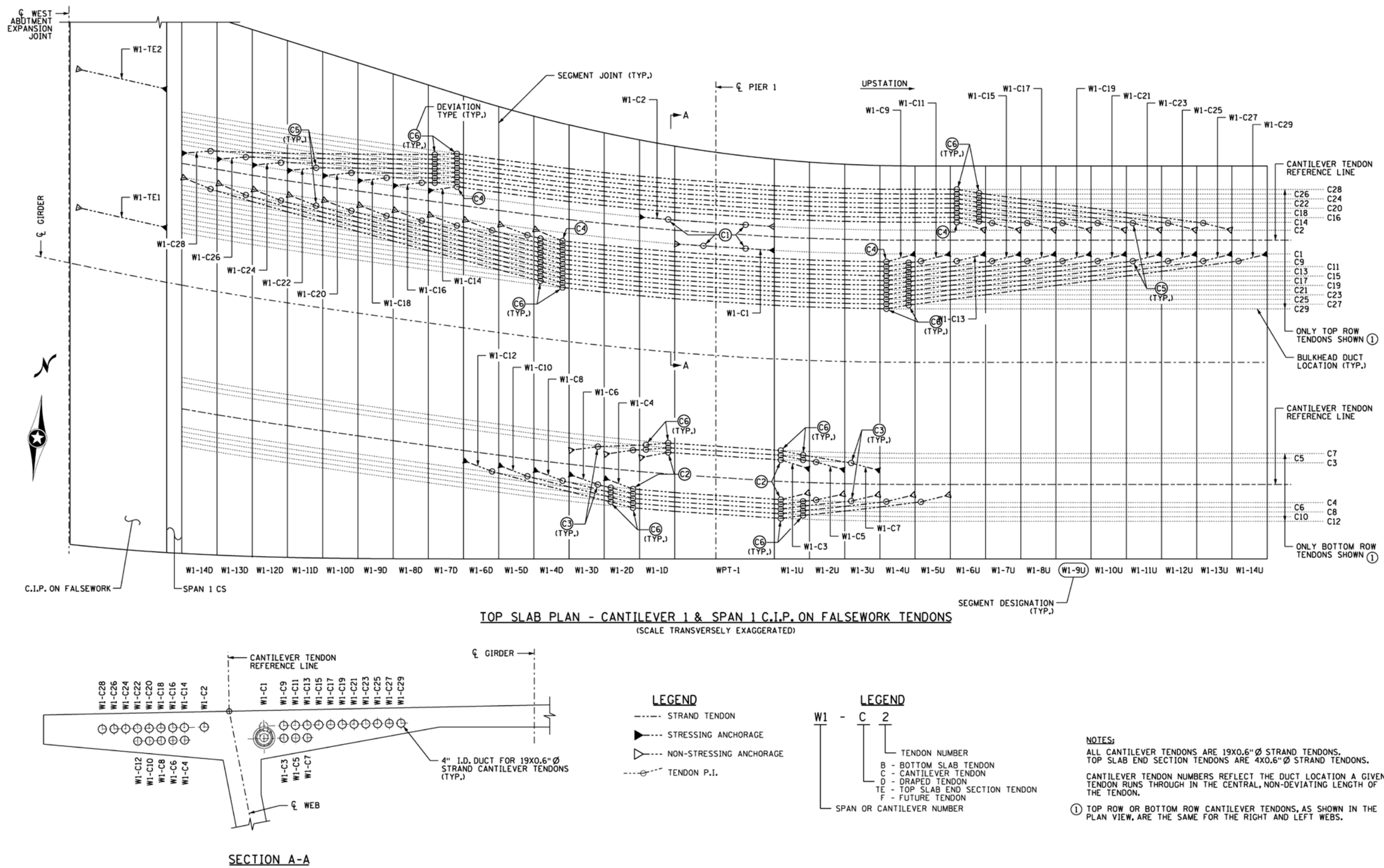


Figure 2.30 – Westbound Cantilever 1 and Span 1 C.I.P. Cantilever Post-Tensioning Layout

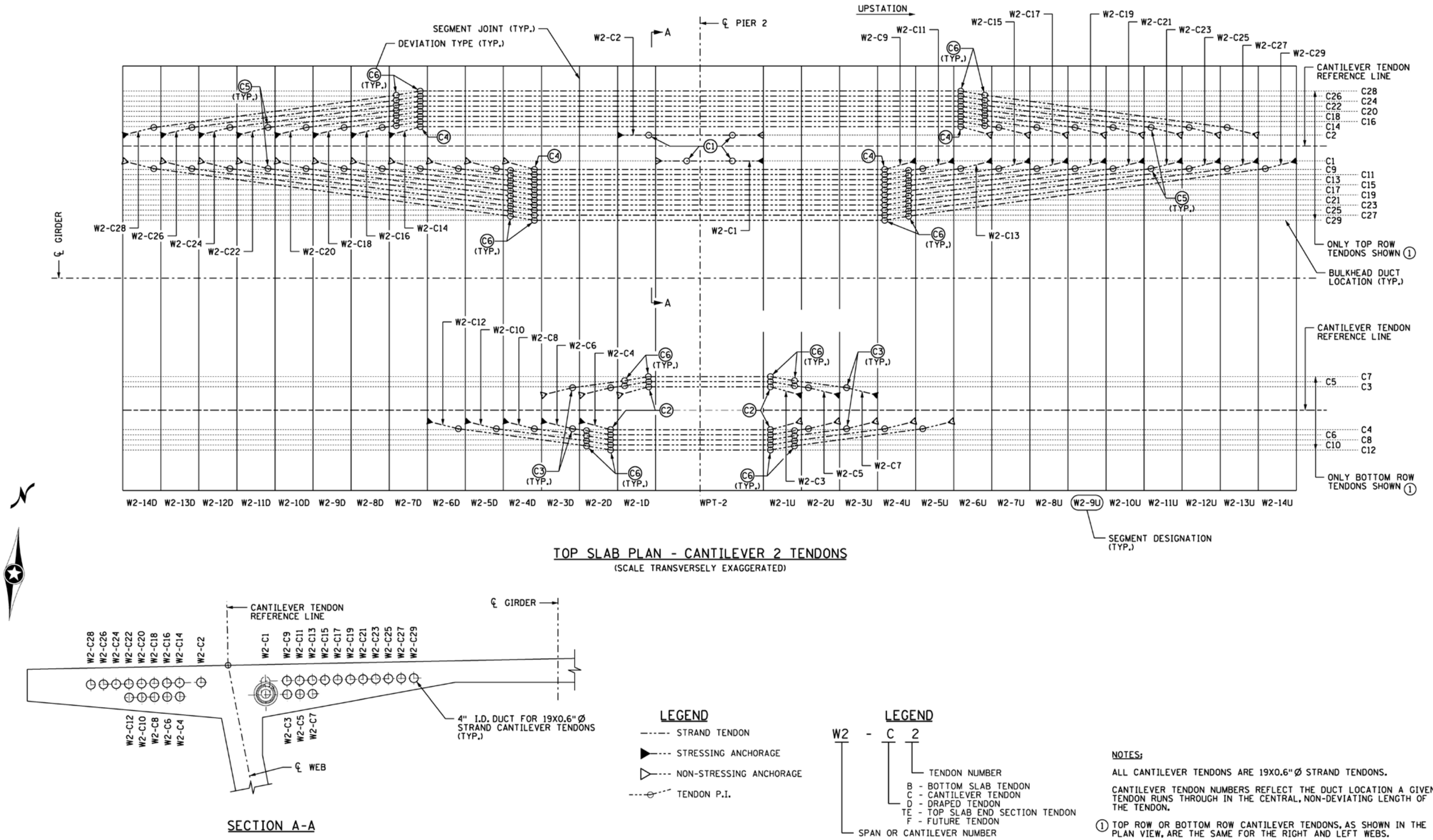


Figure 2.31 – Westbound Cantilever 2 Cantilever Post-Tensioning Layout

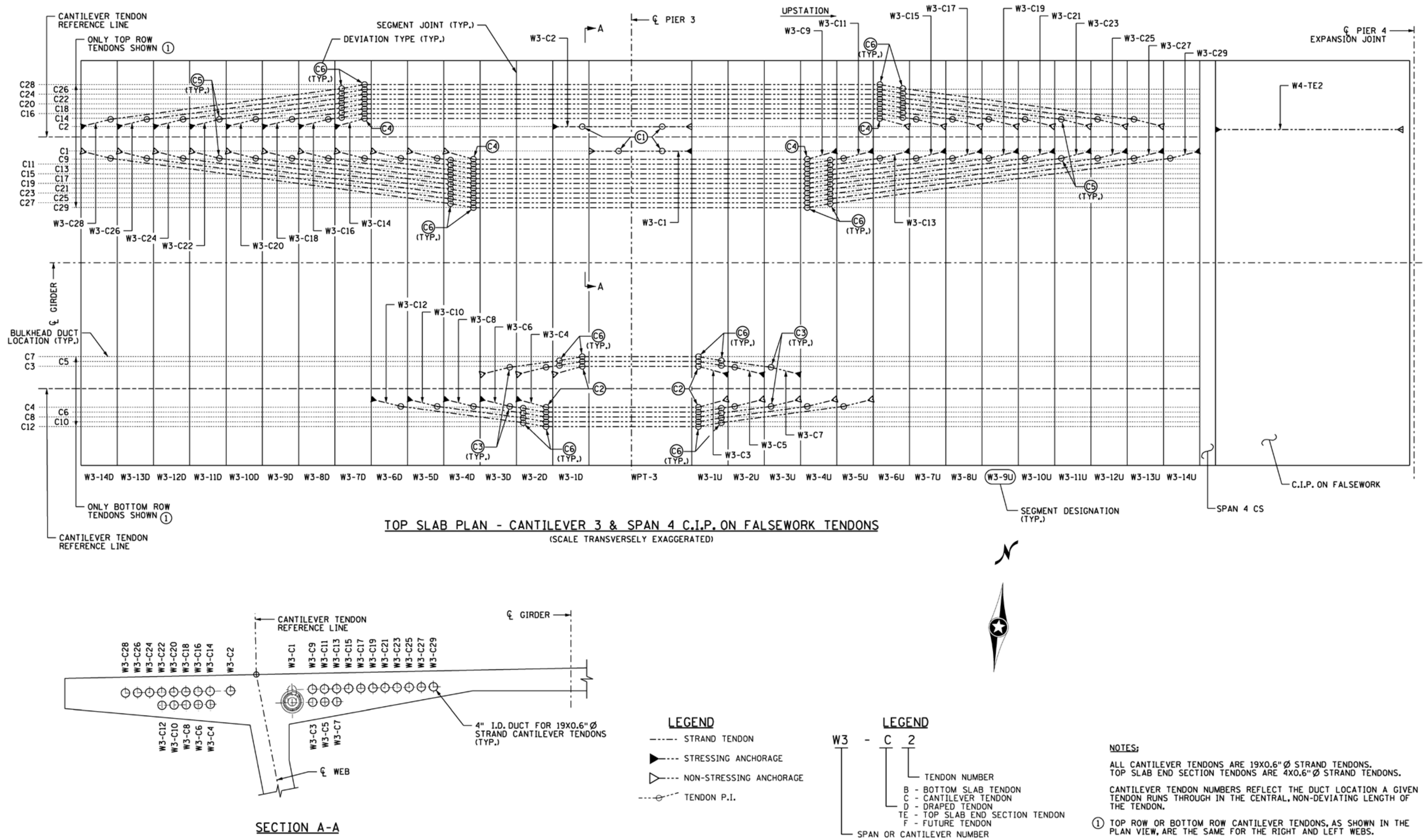


Figure 2.32 – Westbound Cantilever 3 and Span 4 C.I.P. Cantilever Post-Tensioning Layout

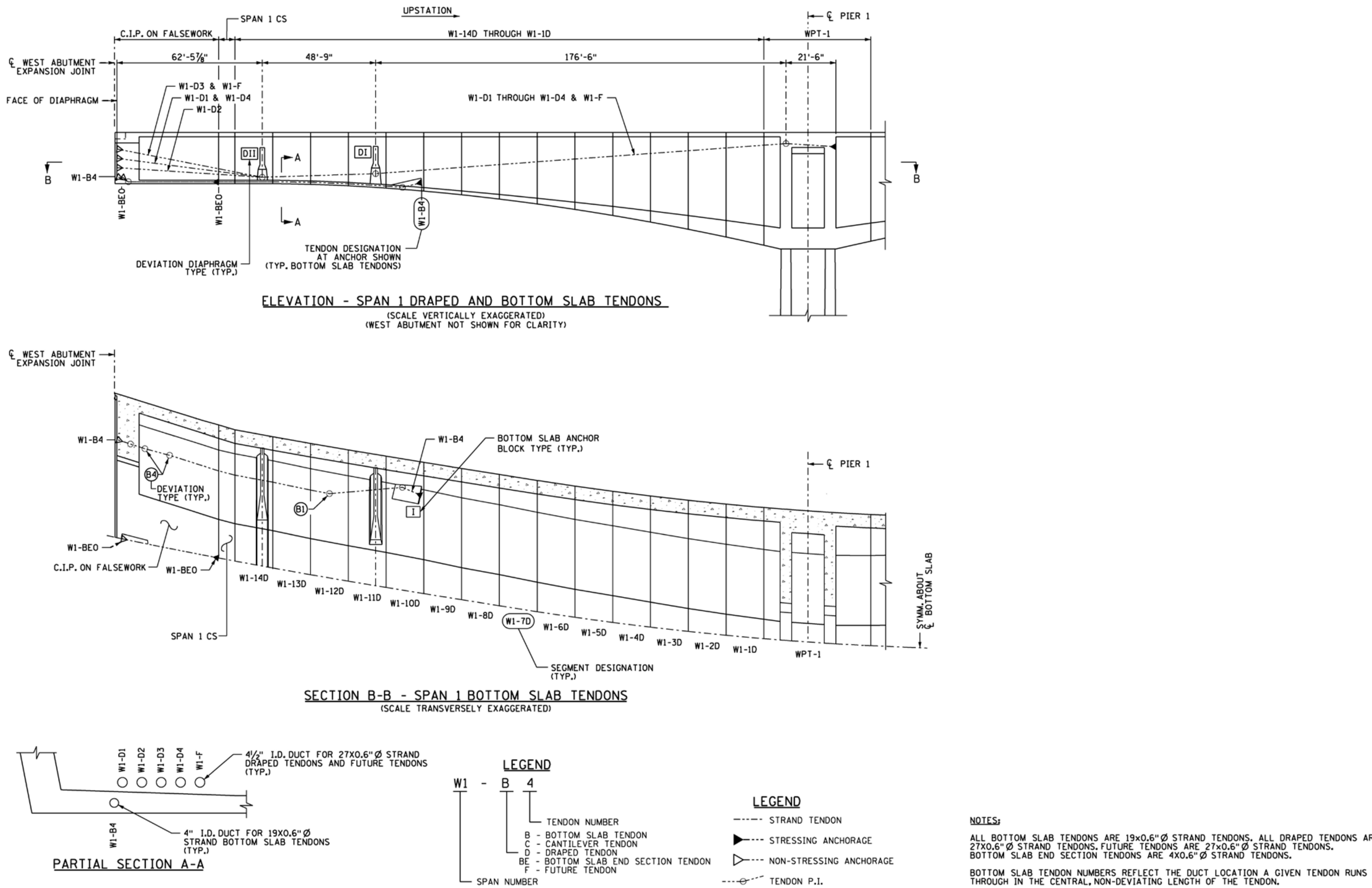


Figure 2.33 – Westbound Span 1 Draped and Bottom Slab Post-Tensioning Layout

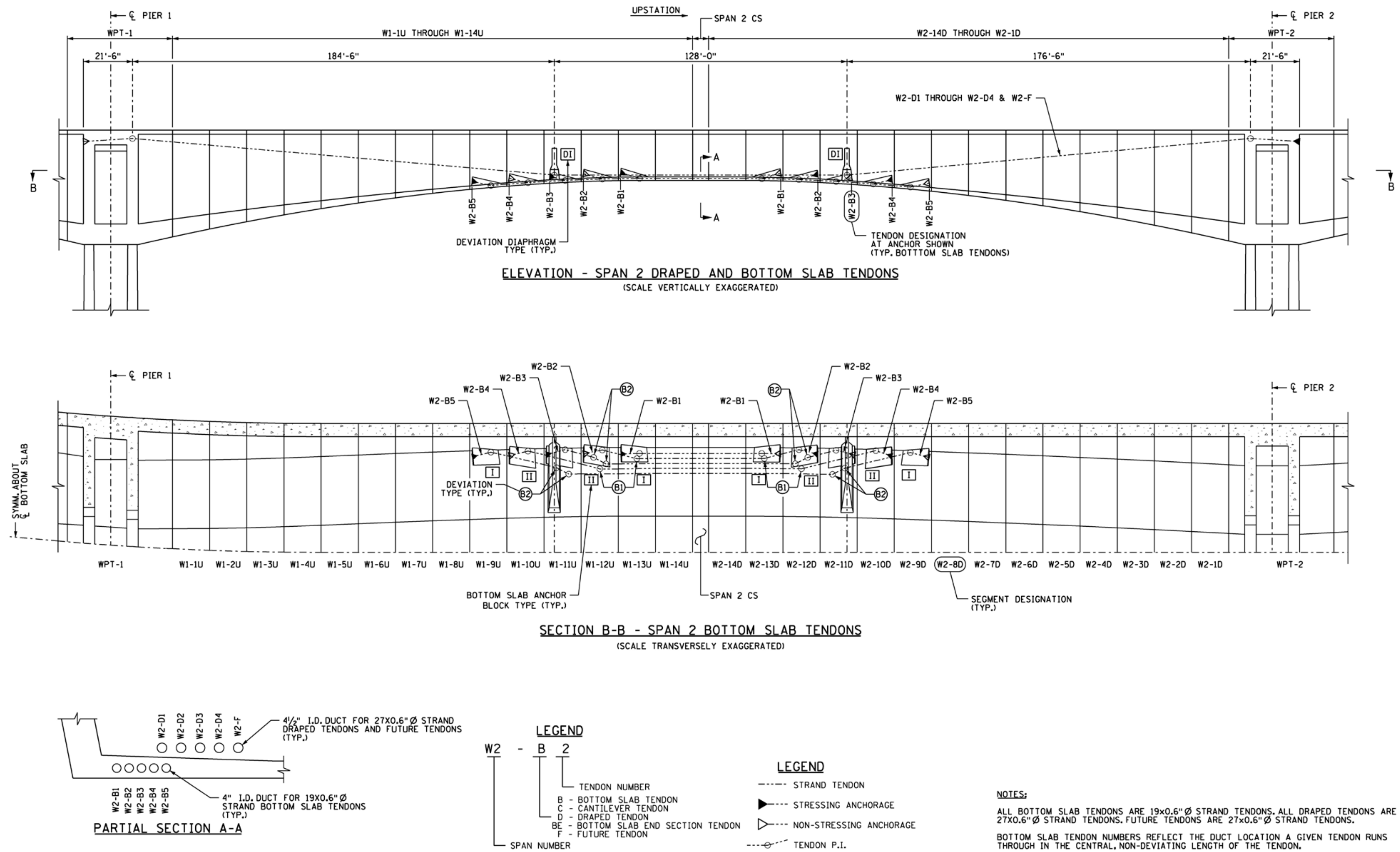


Figure 2.34 – Westbound Span 2 Draped and Bottom Slab Post-Tensioning Layout

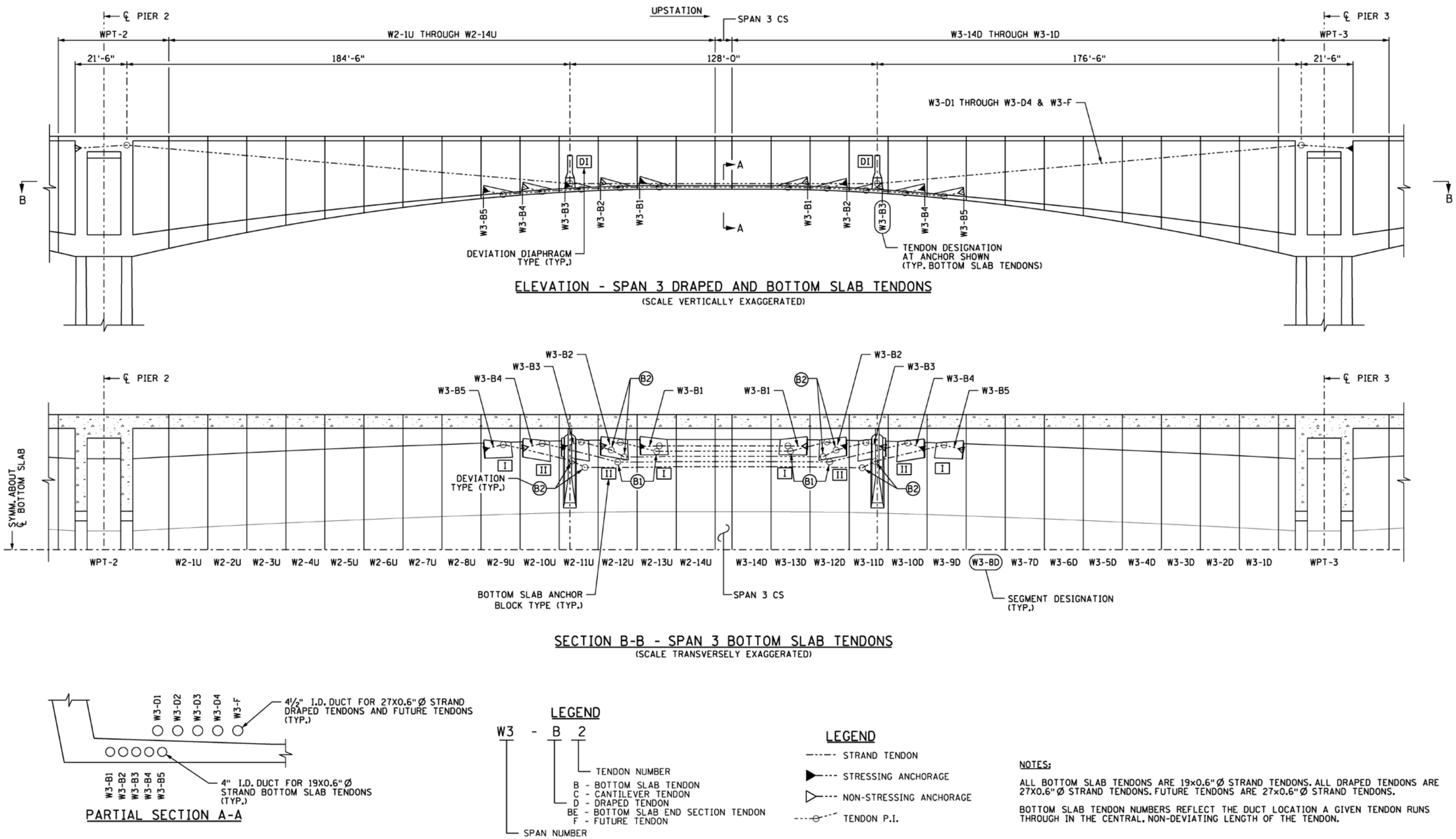


Figure 2.35 – Westbound Span 3 Draped and Bottom Slab Post-Tensioning Layout

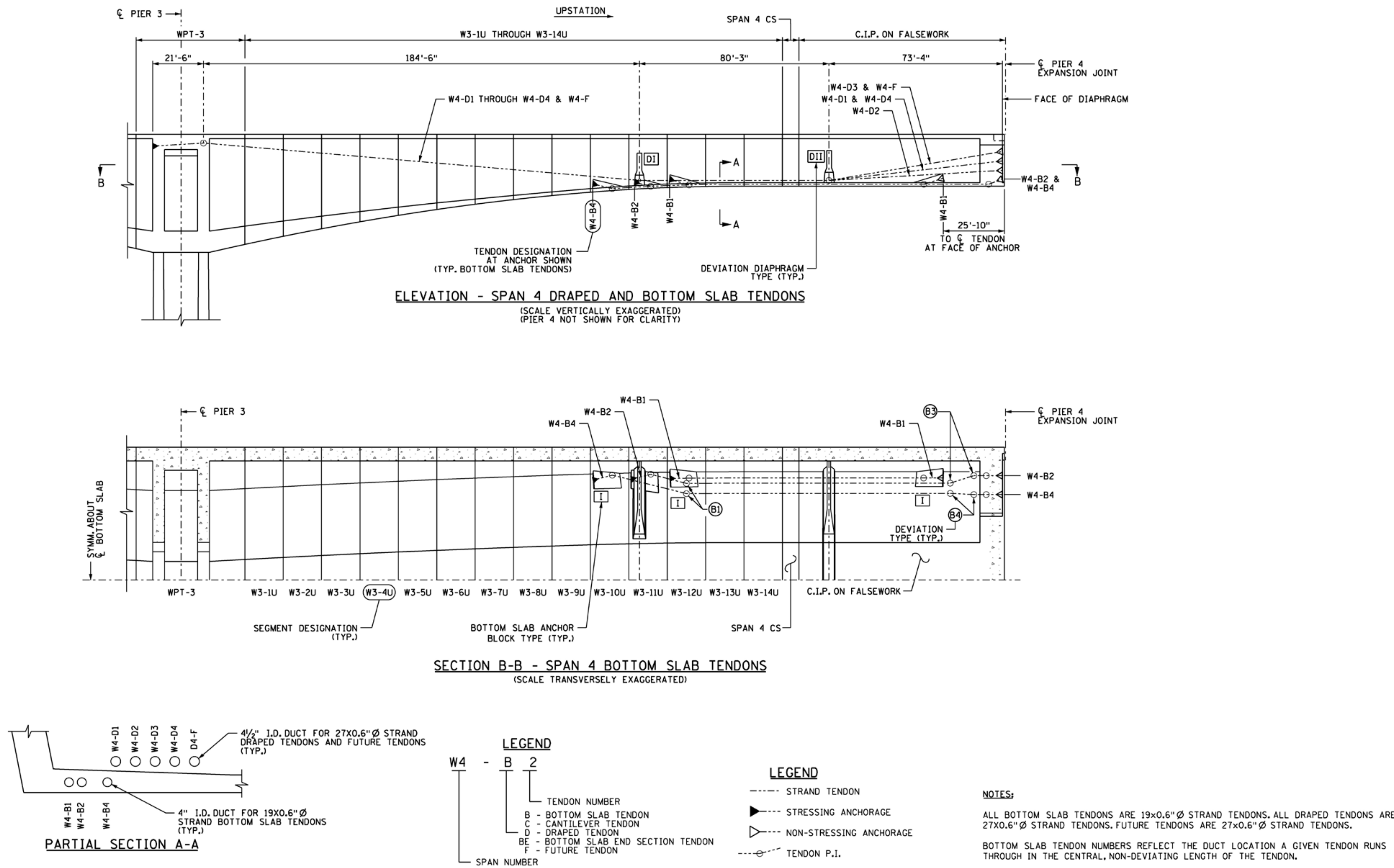


Figure 2.36 – Westbound Span 4 Draped and Bottom Slab Post-Tensioning Layout

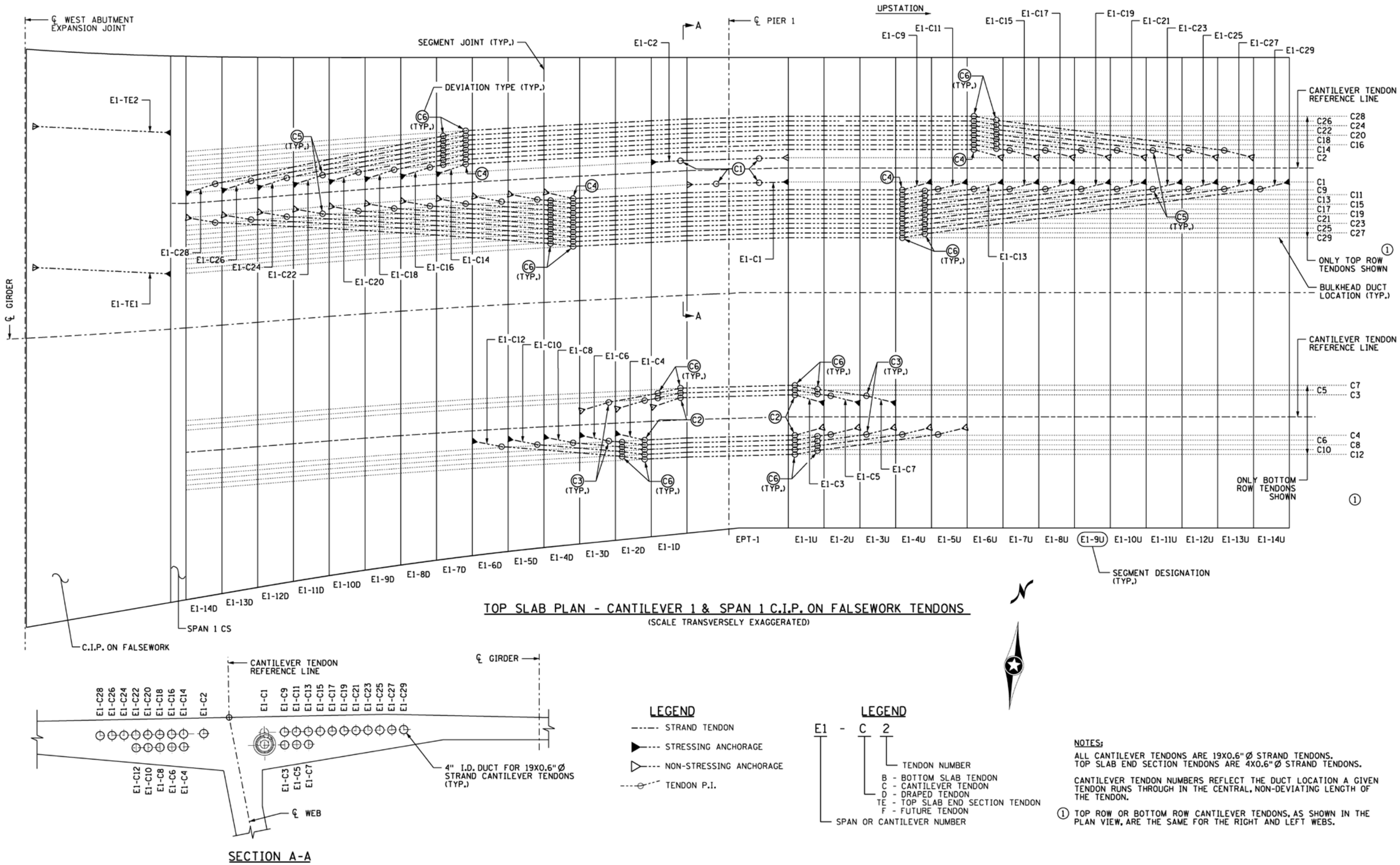


Figure 2.37 – Eastbound Cantilever 1 and Span 1 C.I.P. Cantilever Post-Tensioning Layout

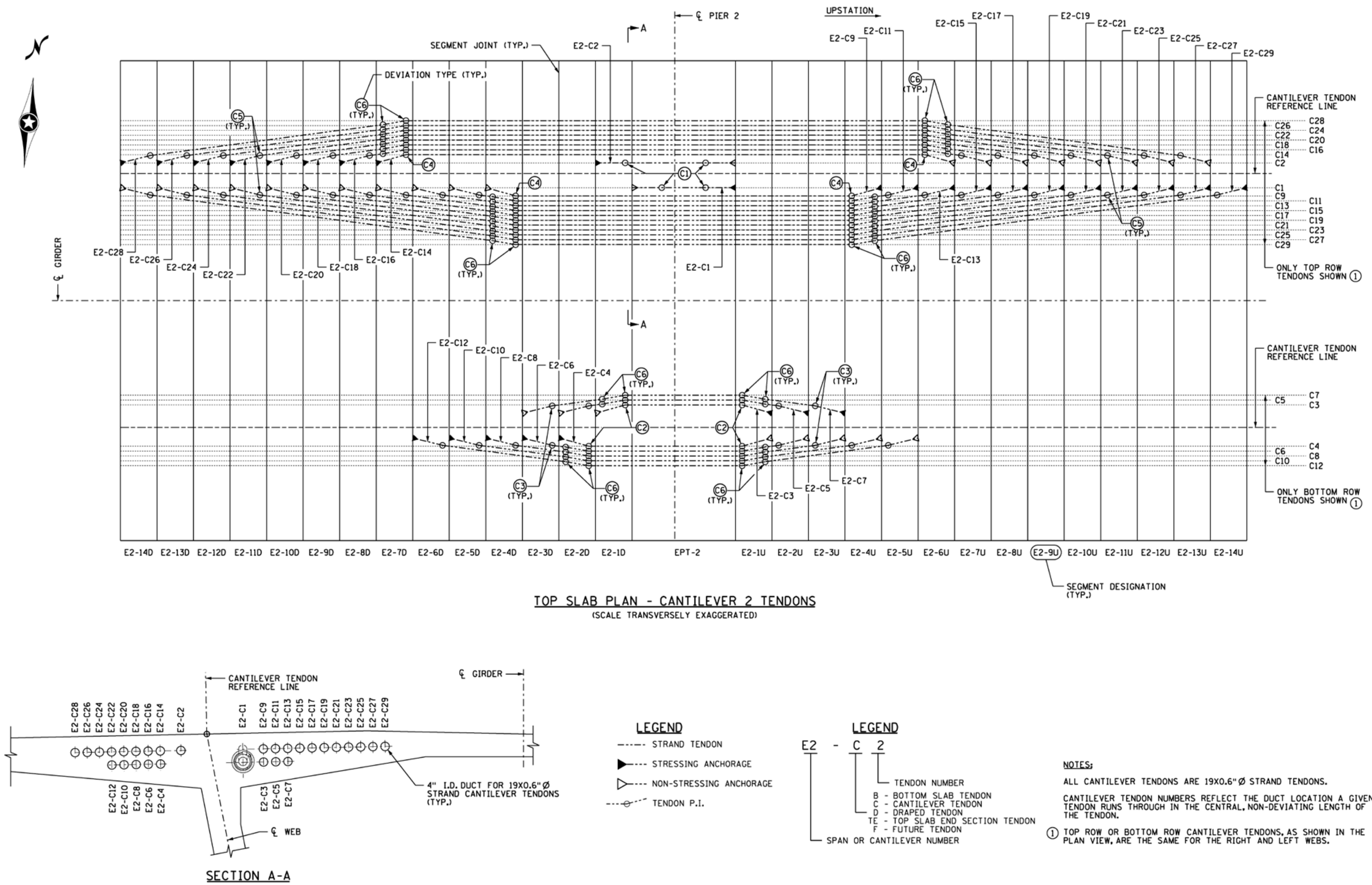


Figure 2.38 – Eastbound Cantilever 2 Cantilever Post-Tensioning Layout

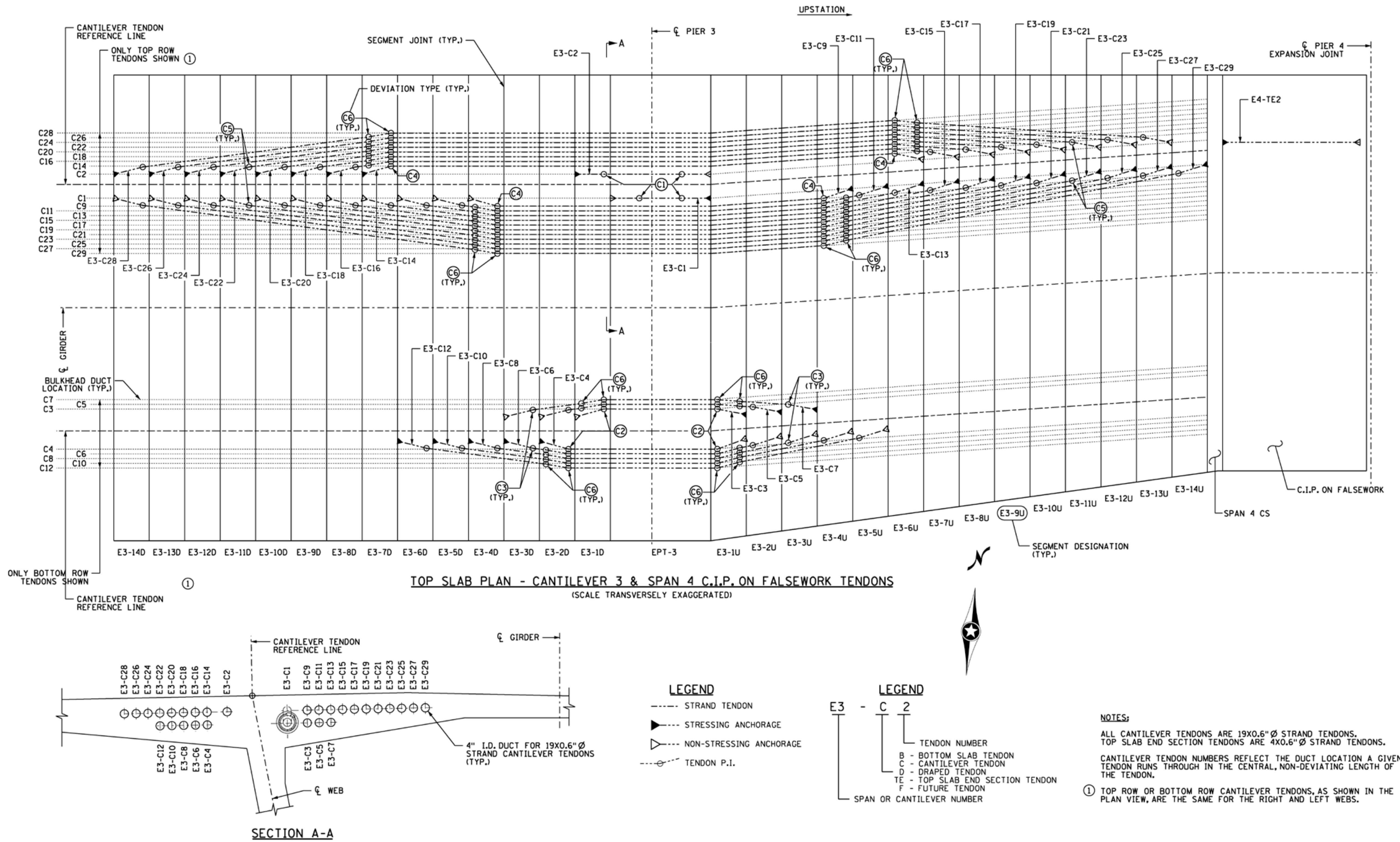


Figure 2.39 – Eastbound Cantilever 3 and Span 4 C.I.P. Cantilever Post-Tensioning Layout

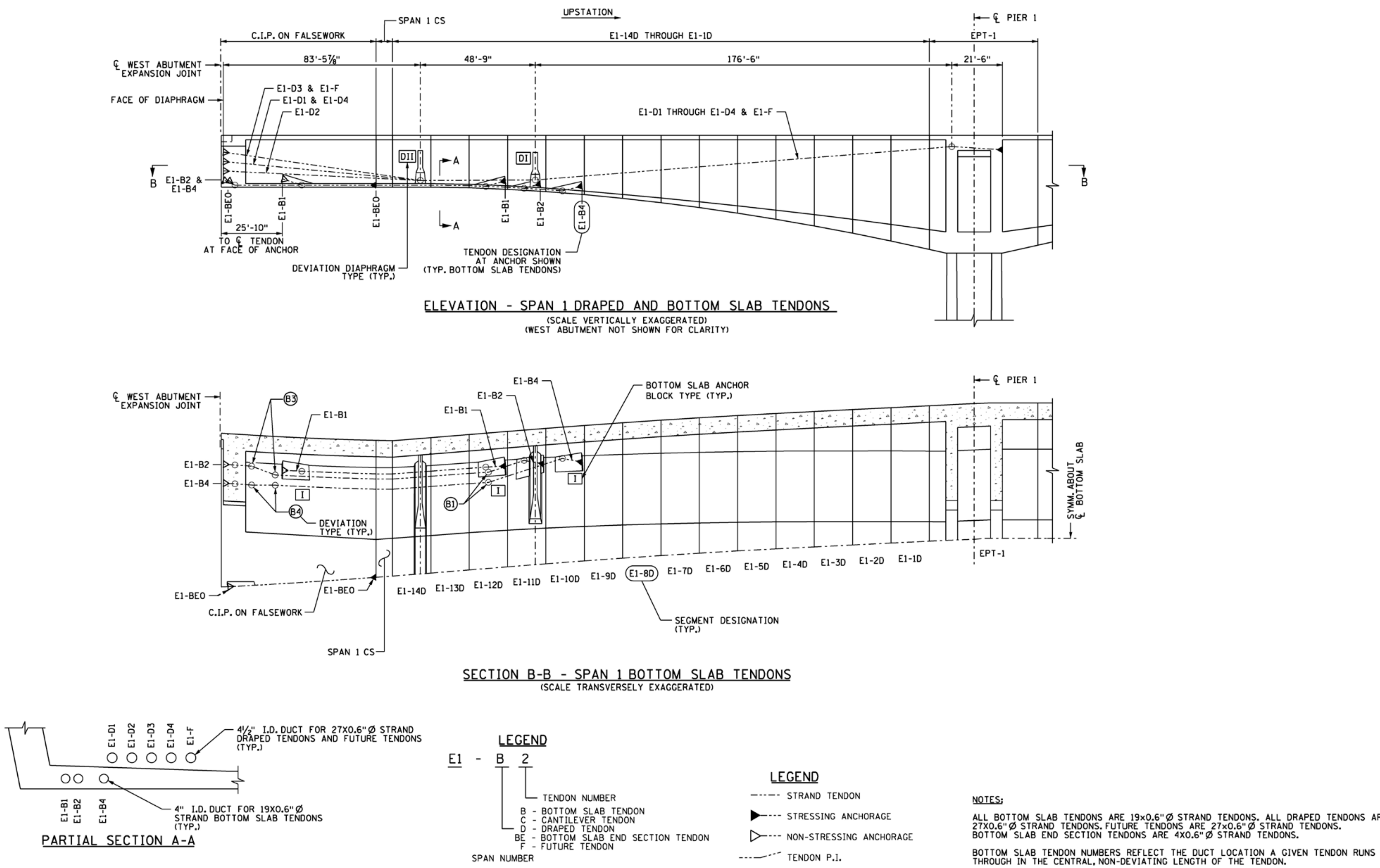


Figure 2.40 – Eastbound Span 1 Draped and Bottom Slab Post-Tensioning Layout

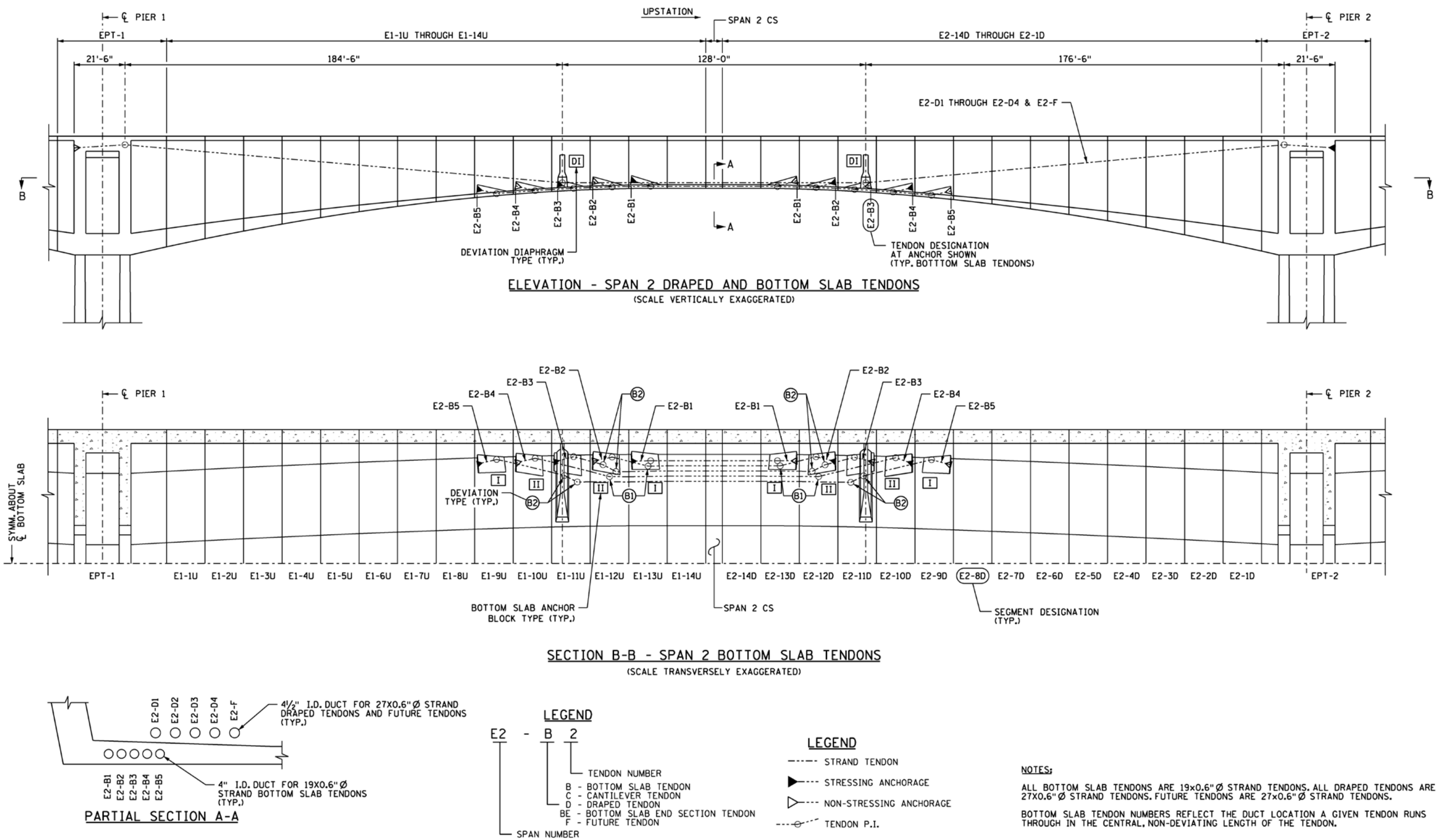


Figure 2.41 – Eastbound Span 2 Draped and Bottom Slab Post-Tensioning Layout

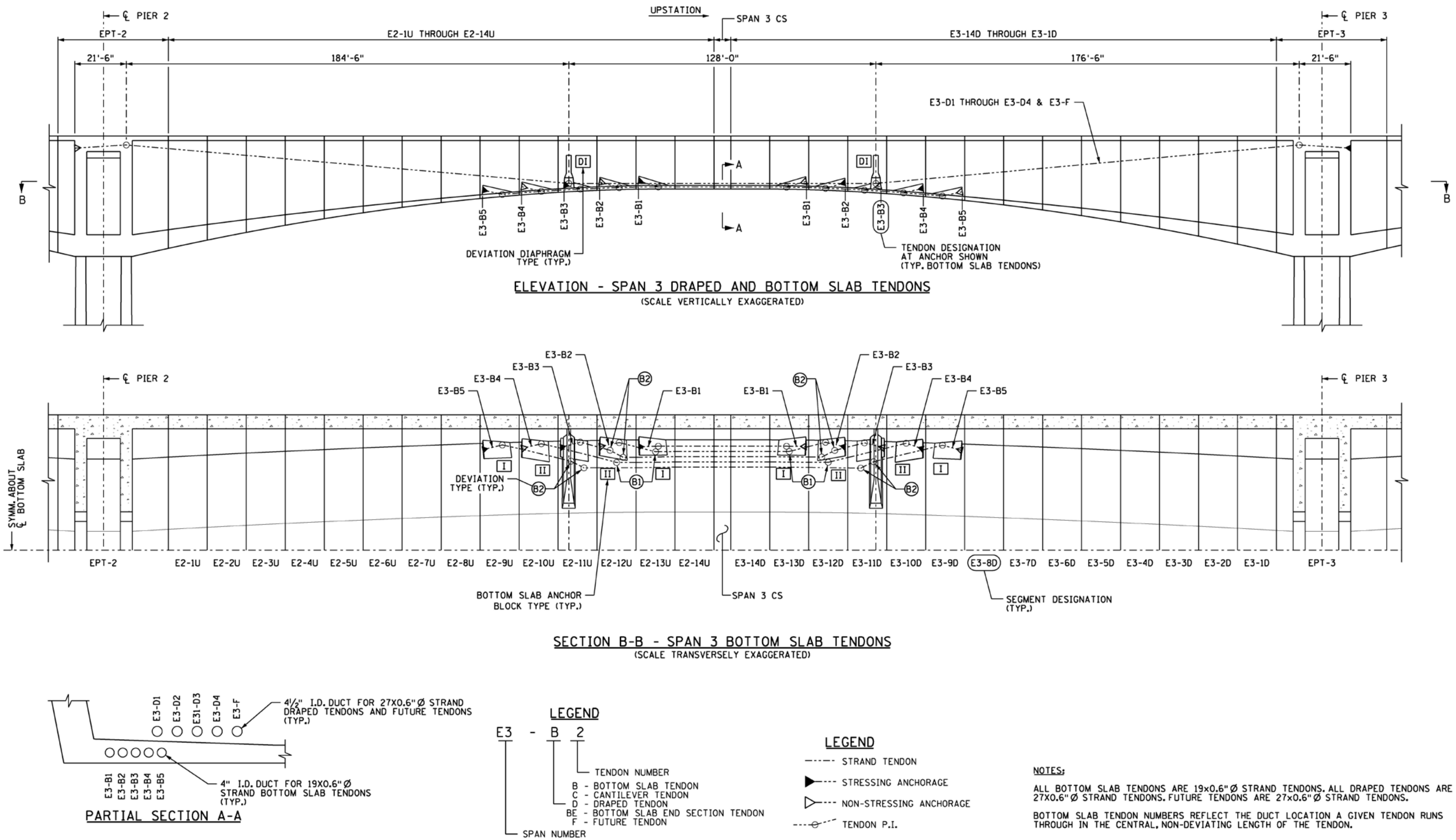


Figure 2.42 – Eastbound Span 3 Draped and Bottom Slab Post-Tensioning Layout

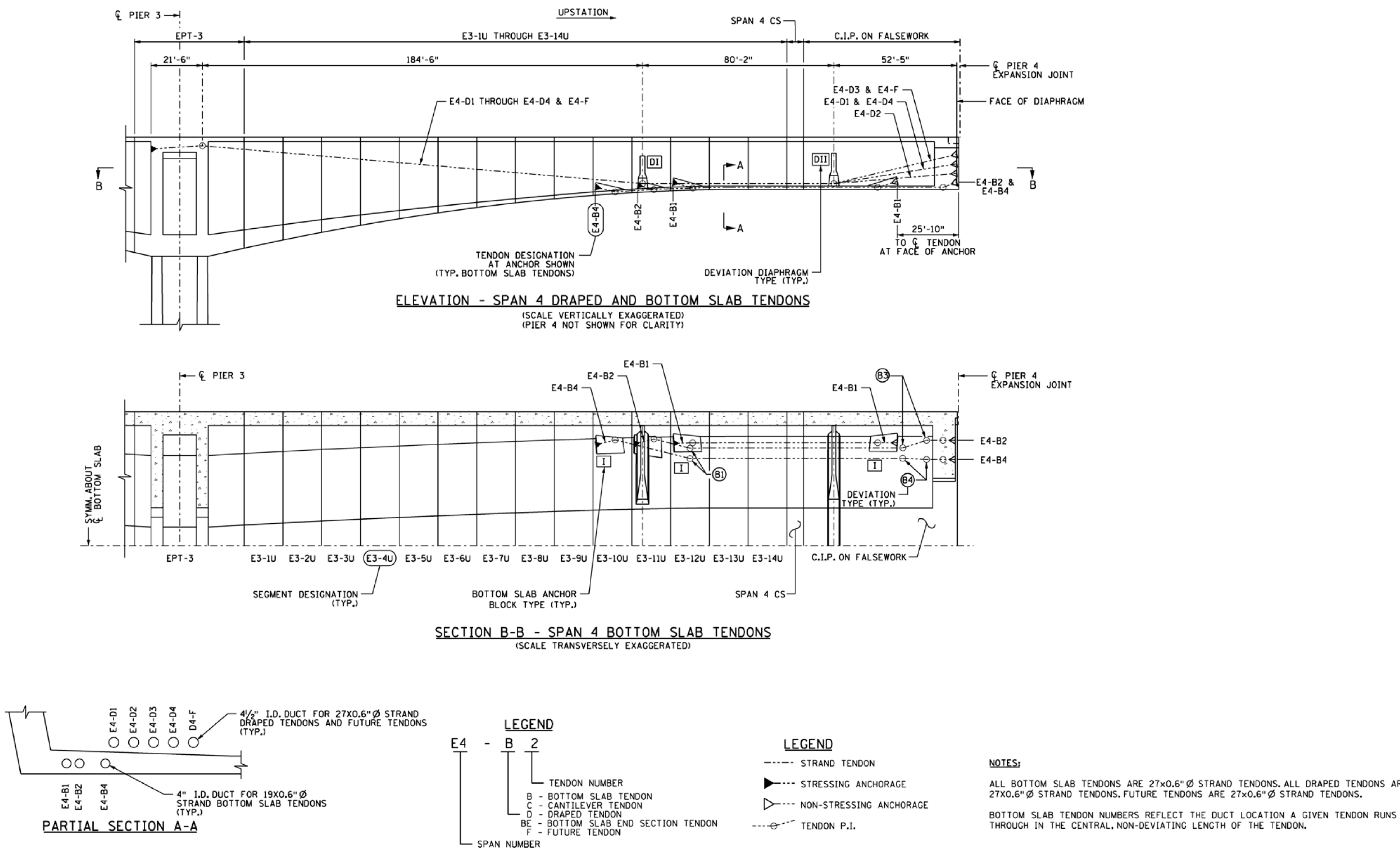


Figure 2.43 – Eastbound Span 4 Draped and Bottom Slab Post-Tensioning Layout

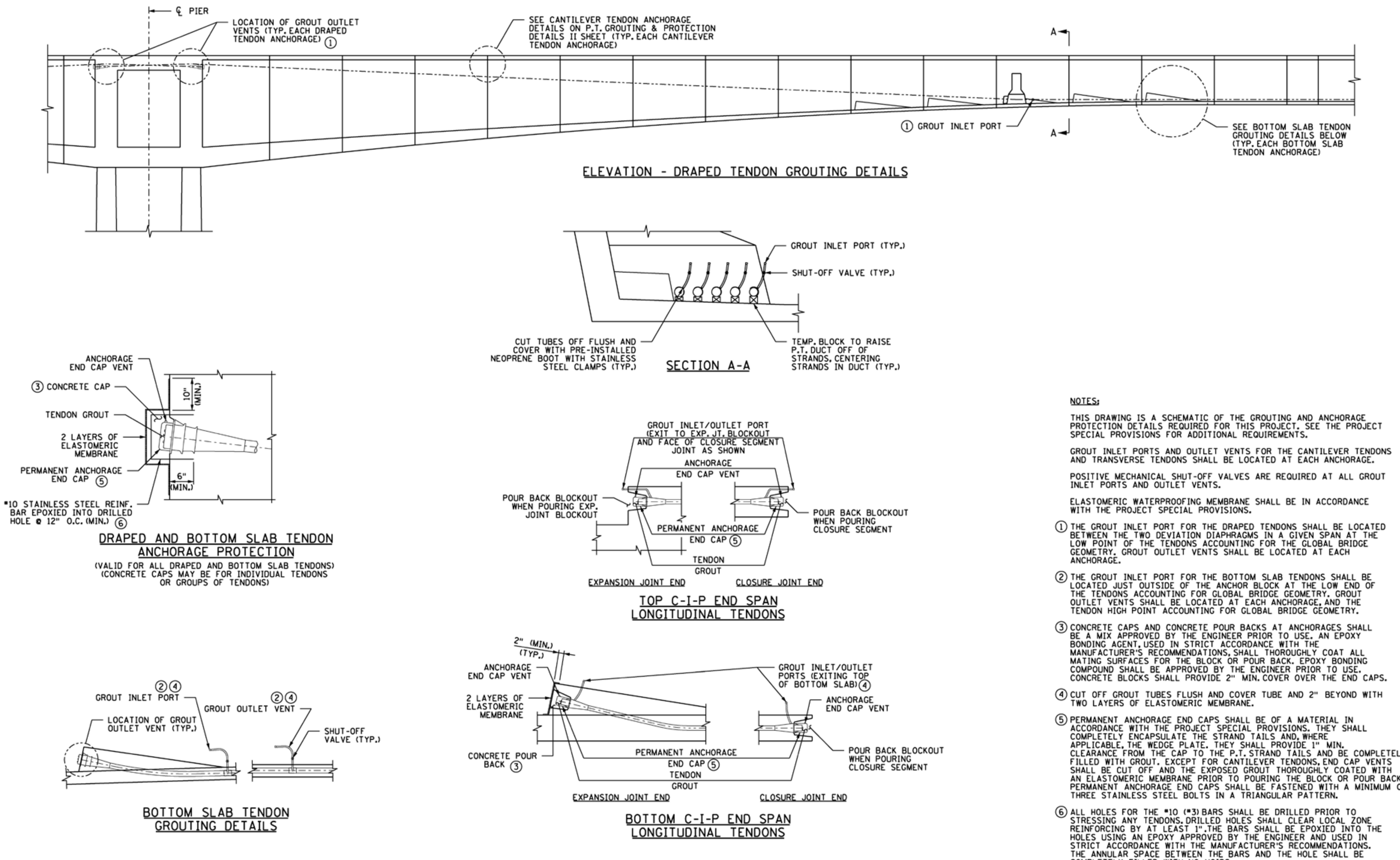
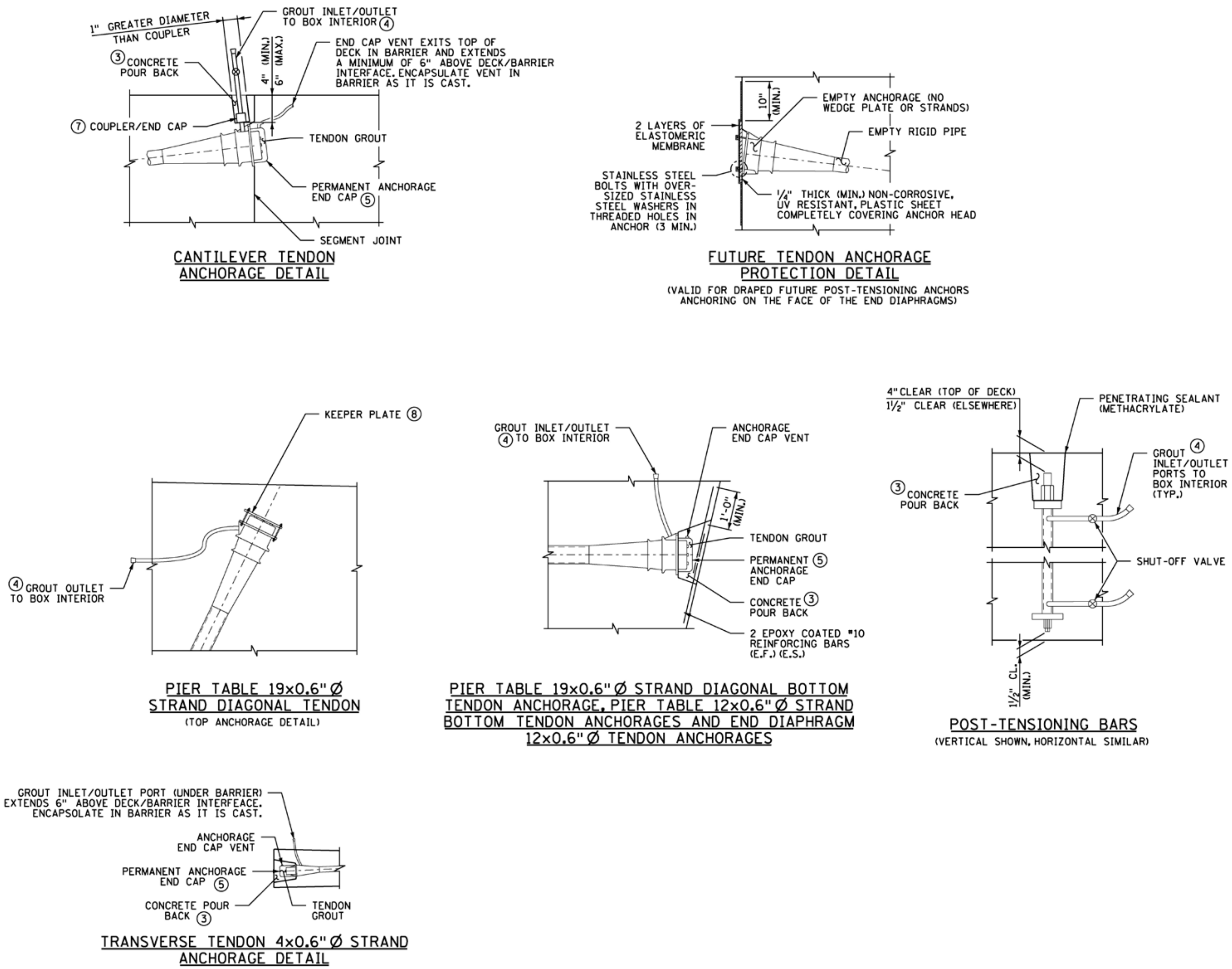


Figure 2.44 – Grouting and Anchorage Protection Details I



NOTES:

THIS DRAWING IS A SCHEMATIC OF THE GROUTING AND ANCHORAGE PROTECTION DETAILS REQUIRED FOR THIS PROJECT. SEE THE PROJECT SPECIAL PROVISIONS FOR ADDITIONAL REQUIREMENTS.

GROUT INLET PORTS AND OUTLET VENTS FOR THE CANTILEVER TENDONS AND TRANSVERSE TENDONS SHALL BE LOCATED AT EACH ANCHORAGE.

POSITIVE MECHANICAL SHUT-OFF VALVES ARE REQUIRED AT ALL GROUT INLET PORTS AND OUTLET VENTS.

ELASTOMERIC WATERPROOFING MEMBRANE SHALL BE IN ACCORDANCE WITH THE PROJECT SPECIAL PROVISIONS.

① THE GROUT INLET PORT FOR THE DRAPED TENDONS SHALL BE LOCATED BETWEEN THE TWO DEVIATION DIAPHRAGMS IN A GIVEN SPAN AT THE LOW POINT OF THE TENDONS ACCOUNTING FOR THE GLOBAL BRIDGE GEOMETRY. GROUT OUTLET VENTS SHALL BE LOCATED AT EACH ANCHORAGE.

② THE GROUT INLET PORT FOR THE BOTTOM SLAB TENDONS SHALL BE LOCATED JUST OUTSIDE OF THE ANCHOR BLOCK AT THE LOW END OF THE TENDONS ACCOUNTING FOR GLOBAL BRIDGE GEOMETRY. GROUT OUTLET VENTS SHALL BE LOCATED AT EACH ANCHORAGE, AND THE TENDON HIGH POINT ACCOUNTING FOR GLOBAL BRIDGE GEOMETRY.

③ CONCRETE CAPS AND CONCRETE POUR BACKS AT ANCHORAGES SHALL BE A MIX APPROVED BY THE ENGINEER PRIOR TO USE. AN EPOXY BONDING AGENT, USED IN STRICT ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS, SHALL THOROUGHLY COAT ALL MATING SURFACES FOR THE BLOCK OR POUR BACK. EPOXY BONDING COMPOUND SHALL BE APPROVED BY THE ENGINEER PRIOR TO USE. CONCRETE BLOCKS SHALL PROVIDE 2" MIN. COVER OVER THE END CAPS.

④ CUT OFF GROUT TUBES FLUSH AND COVER TUBE AND 2" BEYOND WITH TWO LAYERS OF ELASTOMERIC MEMBRANE.

⑤ PERMANENT ANCHORAGE END CAPS SHALL BE OF A MATERIAL IN ACCORDANCE WITH THE PROJECT SPECIAL PROVISIONS. THEY SHALL COMPLETELY ENCAPSULATE THE STRAND TAILS AND, WHERE APPLICABLE, THE WEDGE PLATE. THEY SHALL PROVIDE 1" MIN. CLEARANCE FROM THE CAP TO THE P.T. STRAND TAILS AND BE COMPLETELY FILLED WITH GROUT. EXCEPT FOR CANTILEVER TENDONS, END CAP VENTS SHALL BE CUT OFF AND THE EXPOSED GROUT THOROUGHLY COATED WITH AN ELASTOMERIC MEMBRANE PRIOR TO POURING THE BLOCK OR POUR BACK. PERMANENT ANCHORAGE END CAPS SHALL BE FASTENED WITH A MINIMUM OF THREE STAINLESS STEEL BOLTS IN A TRIANGULAR PATTERN.

⑥ ALL HOLES FOR THE #10 (#3) BARS SHALL BE DRILLED PRIOR TO STRESSING ANY TENDONS. DRILLED HOLES SHALL CLEAR LOCAL ZONE REINFORCING BY AT LEAST 1". THE BARS SHALL BE EPOXIED INTO THE HOLES USING AN EPOXY APPROVED BY THE ENGINEER AND USED IN STRICT ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS. THE ANNULAR SPACE BETWEEN THE BARS AND THE HOLE SHALL BE COMPLETELY FILLED WITH NO VOIDS.

⑦ GROUT INLET AND OUTLET PORTS FOR CANTILEVER TENDONS MAY BE P.V.C. OR STAINLESS STEEL. THEY SHALL BE BONDED INTO ANCHORAGES WITH A BONDING COMPOUND. AFTER INSPECTING THE ANCHORAGES IN ACCORDANCE WITH THE PROJECT SPECIAL PROVISIONS, GROUT BACK THE DRILLED HOLE. AFTER THE GROUT SETS, REMOVE THE UPPER PORTION OF THE INLET/OUTLET AND THE COUPLER AND REPLACE WITH A PERMANENT THREADED END CAP FASTENED WITH A BONDING COMPOUND. POUR BACK THE RECESS IN ACCORDANCE WITH NOTE 3.

⑧ UPPER NON-STRESSING ANCHORAGES FOR DIAGONAL 19x0.6"Ø STRAND TENDONS ARE COMPLETELY EMBEDDED WITH NO BLOCKOUT. KEEPER PLATES OR SOME OTHER POSITIVE MECHANICAL MEANS SHALL BE USED TO SECURE THE TENDONS TO THE TOP ANCHORAGES FROM INSTALLATION UNTIL STRESSING.

Figure 2.45 – Grouting and Anchorage Protection Details II

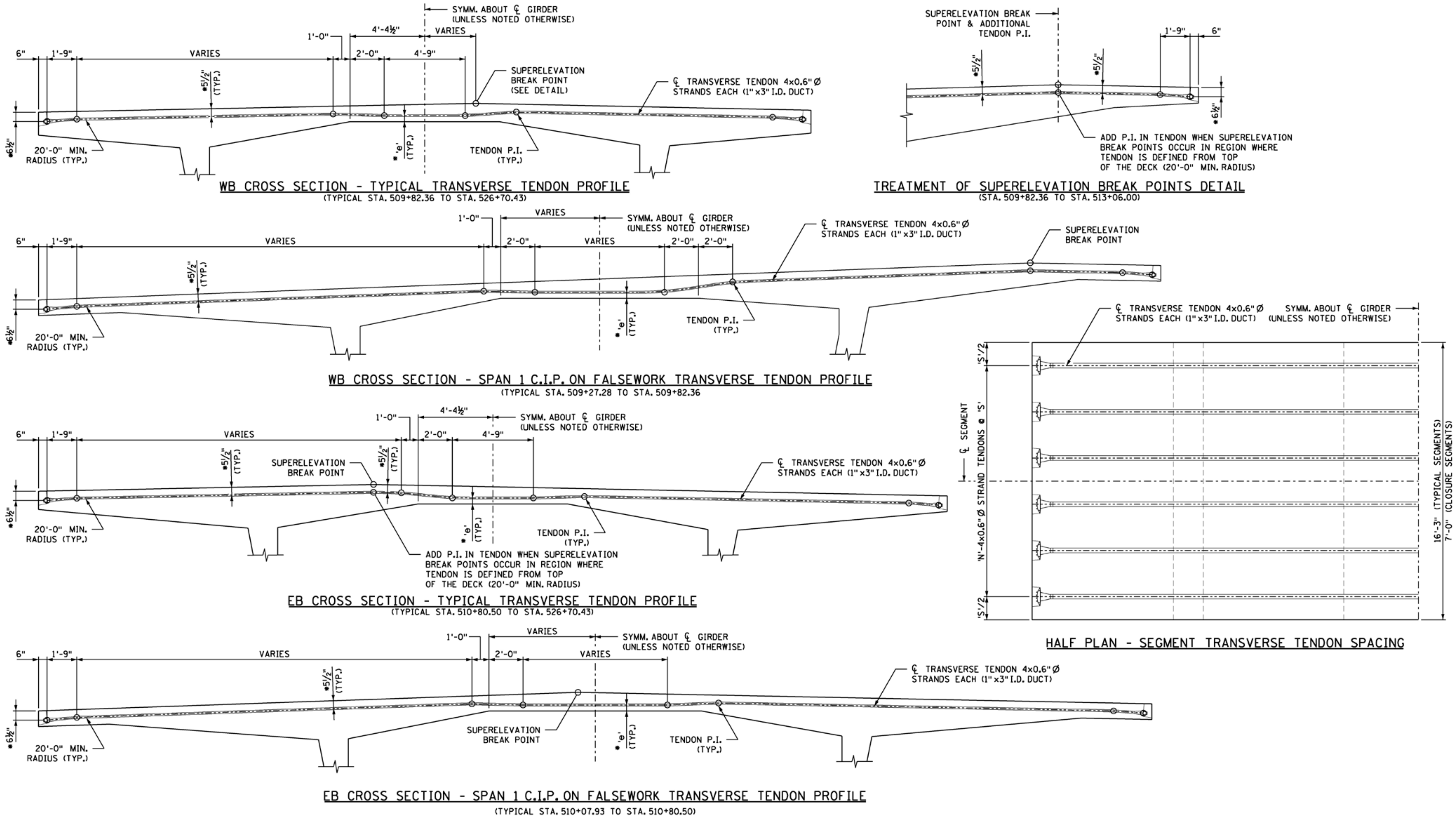


Figure 2.46 – Transverse Post-Tensioning Details

2.3.3.4 Prestressed Girder

The unit 2 superstructure is comprised of four 82" prestressed concrete beams supporting a typical conventionally reinforced concrete deck. The width of both westbound and eastbound unit 2 is 45'-4".

2.3.4 Expansion Joint Devices

Modular expansion joints are used at the west abutment between the abutment backwall and the end diaphragm and at pier 4 between the end diaphragm and unit 2. Modular expansion joints bridge the gap between the abutment and unit 1 and unit 1 and unit 2, support wheel loads, and allow for the bridge to expand and contract. The joints also contain elastomeric glands that keep water on the bridge deck and prevent it from seeping down between the west abutment and box girder onto the bearing seat or between the two units onto pier 4.

The joints at the west abutment provide for 15" of total movement while the joints at pier 4 provide for 27" of total movement.

2.3.5 Integral Wearing Surface

A 2" thick concrete wearing surface is cast integral with the unit 1 bridge decks, which further protects the deck against abrasion and chloride intrusion. Up to a ½" of the integral wearing surface was removed from the unit 1 bridge decks during the grinding and tining operation to achieve the final riding surface. There is an allowance of 20 pounds per square foot of bridge deck for the placement of a future wearing surface, if needed. The future wearing surface can be placed on top of the 1-½" of remaining integral wearing surface, or the 1-½" integral wearing surface can be removed prior to placing the future wearing surface.

2.3.6 Approach Slabs

Approach slabs are provided at the west and east abutment to improve vehicle ride quality on and off of the bridge by spanning the roadway approach fill, isolating the roadway from any settlement behind the abutment backwall. The approach slabs are approximately 22'-0" long, vary in width, and match the grade and profile of the bridge.

2.3.7 Drainage System

Both units 1 and 2 include drainage scuppers and piping under the deck that carry the bridge deck drainage to either end of the bridge for treatment. The westbound bridge drain pipes are located on the south side of Span 1 WB running into the west abutment, on the south side of Span 2 WB through Span 10 WB running into the east abutment, and a short run on the north side of Span 10 WB into the east abutment. The eastbound bridge drain pipes are located on the north and south sides of Span 1 EB running into the west abutment, on the north side of Span 2 EB through Span 10 EB running into the east abutment, and a short run on the south side of Span 10 EB into the east abutment. The fiberglass pipes are connected to a flanged ductile iron pipe cast into each abutment backwall and are supported on a hanger system thread into a cast in concrete anchor in the deck and drilled in anchors in the webs or precast beams. To accommodate the bridge movements and differences in thermal expansion properties between the fiberglass pipe and the concrete bridge, the piping is supported on rollers to allow the pipe to move independently of the bridge.

2.3.8 Utilities

The westbound bridge includes inserts cast into the deck for a future utility hanger system and six capped sleeves that pass through both abutment backwalls. These sleeves connect with handholes in the approach roadways that allow for the installation of up to six future utilities. The eastbound bridge includes one empty 3" diameter rigid steel conduit (RSC) for future use, one 3" diameter RSC conduit occupied by a 1.5" diameter NMC and a fiber optic trunk cable (48SM), and one 4" diameter RSC for power cables, supported on a hanger system off the deck (inside of unit 1 and between precast beams in unit 2). The hanger system and abutment backwall penetrations allow for the installation of two additional future 4" diameter RCS conduits.

2.3.9 Access and Lighting

An under-bridge inspection vehicle (snooper) may be used to inspect the outside of the box girder beneath deck level. A snooper is required for inspection of the pier cap and bearings at pier 4, while access to visually inspect the west abutment bearings may be achieved via ladder, snooper, or man-lift. Access to piers within the river must be by boat or from above using a snooper. Divers may inspect the underwater portions of the pier foundations.

Access to the interior of the box girders can be achieved at several locations. A snooper or man-lift is required to access the bottom slab access openings located in spans 1 and 4. An access opening is located in the median at the west abutment via the lid slab, which will require a ladder down to the abutment seat. Access to the box interior may then be gained between the abutment backwall and the end diaphragm. Additionally, access into the bridges may be gained via snooper or man-lift on the upstation side of Pier 4 by climbing over the precast beam pier cap ledge. However, note that all access points have locked security doors.

Maintenance lighting is provided in each span and is controlled by three-way switches located at the pier table and end diaphragm at the end of each span. This allows personnel to turn on the lights upon entering a span and turn off the lights upon exiting at the other end of the span. Flashlights or floodlights (with extension cords) should be considered to adequately inspect the box girder interior. Power receptacles are located throughout the spans at select light locations. These outlets can be used to provide additional AC-powered lighting for inspection purposes.

2.3.10 Future Pedestrian Bridge Support System

The unit 1 superstructure and the foundations in unit 2 have been designed to accommodate a future pedestrian bridge (by others). Hangers have been cast into the segments at the intersection of the top slab and web to support a future pedestrian bridge between the eastbound and westbound structures. Due to the increase in loading imposed on unit 1 EB & WB, the future tendons must be installed, stressed, capped, and grouted prior to installing the future pedestrian bridge.

2.4 Additional References

In addition to this manual, there are a number of other documents that could be useful for the inspection and maintenance of the Interstate 90 Bridge at the Mississippi River. These include:

Contract Plans and Special Provisions – The Contract Plans and Special Provisions are an excellent reference.

Load Rating Manual and Calculation Aids – The load rating manual contains information that may be used to calculate the effect of loads applied to the structure and compare it to the structure capacity using simple hand calculations. Information is included for longitudinal shear and torsion, and moment in the box girder and transverse moments in the deck.

Shop Drawings – Detailed shop drawings exist for the expansion joints, bearings, the post-tensioning system and other bridge elements.

Design Documentation – Documentation from the design phase of the project exists including correspondence, meeting minutes, design calculations, etc.

Construction Documentation – Documentation of the project construction, which includes correspondence, meeting minutes, diaries, inspection reports, material test reports, contractor submittals, photographs, etc.

Initial Inspection Documentation – Documentation of the initial inspection was prepared by the Minnesota Department of Transportation and is included in Appendix A of this Manual. This information is helpful in identifying bridge conditions that have changed since the first inspection.

CHAPTER 3

INSPECTION PROCEDURES

3. INSPECTION PROCEDURES

3.1 General Information

The following subsections discuss potential concrete conditions, inspection procedures related to substructure, disc bearings, superstructure, and modular expansion joint devices, deformation monitoring, and inspection forms.

3.2 Potential Concrete Conditions

As is the case with all concrete structures, cracking (which may include efflorescence) and spalling (including delamination) of concrete may occur in any of the various concrete members of a post-tensioned concrete bridge. To avoid duplicating text, these potential deficiencies will be discussed prior to discussing deficiencies that are more likely to be limited to a single element or only a few elements of the bridge. During systematic inspections, the bridge inspector must be constantly alert in order to detect cracking or spalling that might indicate that a maintenance activity should be initiated.

3.2.1 Cracking of Concrete

Some small cracks are to be expected in all concrete structures. These are commonly caused by shrinkage and temperature changes in the concrete and do not necessarily indicate any structural problems with the structure. The significance of these cracks is that they can allow entry of water and chemicals into the concrete that could eventually initiate corrosion of the reinforcing.

According to recommendations by the American Concrete Institute, cracks in reinforced concrete should not be considered a significant factor in causing corrosion of embedded reinforcing steel until the width exceeds:

<u>Environment</u>	<u>Width</u> (1 mil = 0.001 in.)
Dry Air	16 mils (0.016 in)
Moist Air	12 mils (0.012 in)
Exposure to Deicing Chemicals	7 mils (0.007 in)

The following is a guide for the various elements of the Dresbach Bridge:

Guide for Determining the Significance of Cracks

Element of Substructure:	Cracks May Be Significant If Width Exceeds:
Footing	10 mils (0.010 in)
Piers	12 mils (0.012 in)
Pier Caps	10 mils (0.010 in)
Abutment	10 mils (0.010 in)
Concrete Box Girder:	
Deck	7 mils (0.007 in)
Web	12 mils (0.012 in)
Bottom Slab	12 mils (0.012 in)
Abutment and Pier Diaphragms	16 mils (0.016 in)

In the early stages of corrosion, rust stains (usually dark gray or rust colored) can be observed at cracks. This staining may also appear due to water seeping through the pores of concrete, and it may indicate corrosion of the reinforcing even though no crack is visible. Later, there is more prominent cracking in a direction parallel to the reinforcing and a delamination of the concrete at the level of the steel. This is due to the expansive forces generated by the iron oxides (rust), which occupy a greater volume than the original reinforcing steel without corrosion.

Stress Related Cracks

This type of cracking is caused by stressing concrete beyond its tensile capacity. There are several reasons why this may occur.

First, note that conventional reinforced concrete structures must crack before the reinforcing becomes effective in carrying tensile forces. The philosophy adopted in the design of this structure was to limit crack widths to within acceptable limits. Piers, abutments and footings on the Dresbach Bridge are conventional reinforced concrete construction. In post-tensioned concrete structures, concrete is precompressed, which substantially limits cracking. The concrete superstructure of the Dresbach Bridge is post-tensioned in the longitudinal direction, and the deck is transversely post-tensioned. The abutment and pier diaphragms are also post-tensioned but behave structurally more like a conventional reinforced concrete member. Superstructure webs and bottom slab are conventionally reinforced in the plane oriented transverse to the bridge axis.

Secondly, a temporary overstress may cause cracking. This occurs sometimes during construction. Appendix A is reserved for the initial inspection report that documents the bridge condition at the end of construction. This information should serve as a benchmark to determine if further cracking is occurring in the structure. Temporary overstressing may also occur after the bridge is in service. Small cracks that show no propagation or increase in size may indicate such an occurrence.

Finally, although not likely, cracking may indicate that some type of structural concern is developing. In these cases, a corrective maintenance action is necessary.

Types of Potential Cracking in the Superstructure

The purpose of this section is to provide the inspector with a basic understanding of potential modes of cracking in the superstructure.

It should be noted that if cracking occurs, it is often due to the sum of several effects. Therefore, it is possible that any cracking observed in the field may not fit neatly into one of the following categories but may vary substantially and represent the sum of several modes of cracking.

Direct tension would cause a series of parallel cracks transverse to the section. Cracks would form in all elements of the girder and could extend around the entire section. (Figure 3.1)

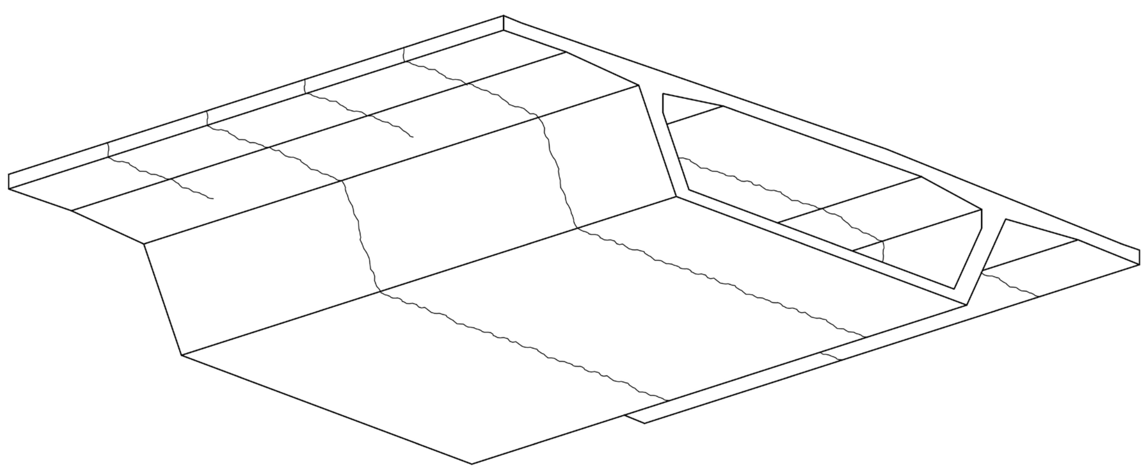


Figure 3.1 – Superstructure Potential Crack Pattern for Direct Tension

Flexural cracking from a positive moment near midspan would result in a series of parallel cracks transverse to the section. Unlike direct tension cracking, cracks would only occur in the bottom slab and lower portions of webs (Figure 3.2).

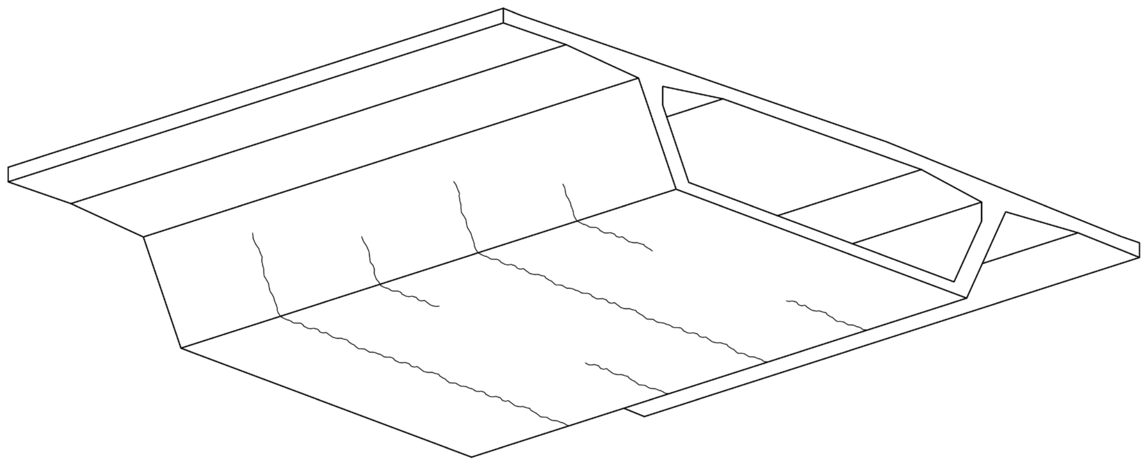


Figure 3.2 – Superstructure Potential Crack Pattern for Positive Moment

Flexural cracking from a negative moment near the pier would result in cracking similar to that for a positive moment, but cracks would only occur in the deck and upper portions of webs (Figure 3.3).

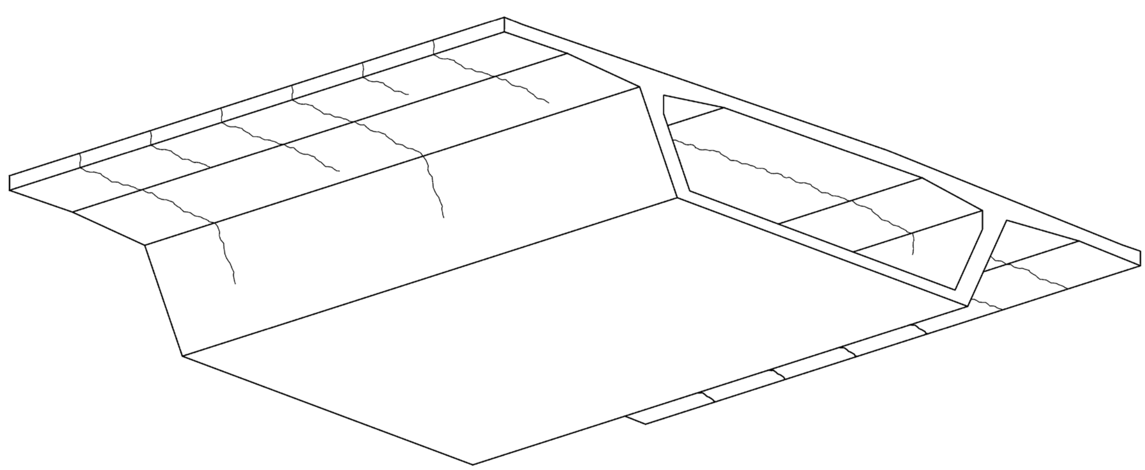


Figure 3.3 – Superstructure Potential Crack Pattern for Negative Moment

Excessive shear generates inclined cracking in the webs. Flexural shear cracks would begin as vertical flexural cracks near the bottom or top of the member and become more inclined as they move toward the mid-depth of the member (Figure 3.4).

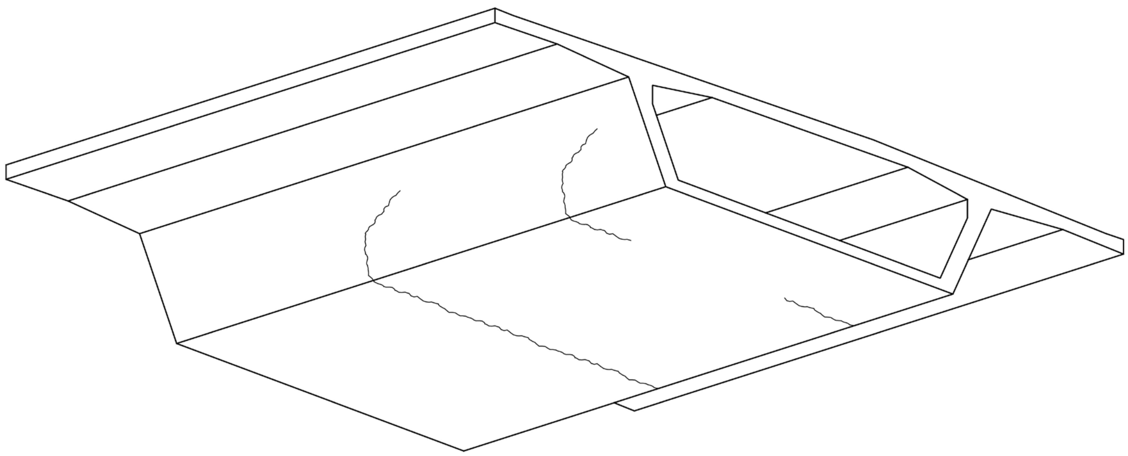


Figure 3.4 – Superstructure Potential Crack Pattern for Flexural Shear

Pure shear cracks would initiate in the webs near the neutral axis of the section close to the supporting pier or abutment (Figure 3.5). These cracks are generally inclined at approximately 45 degrees from vertical.

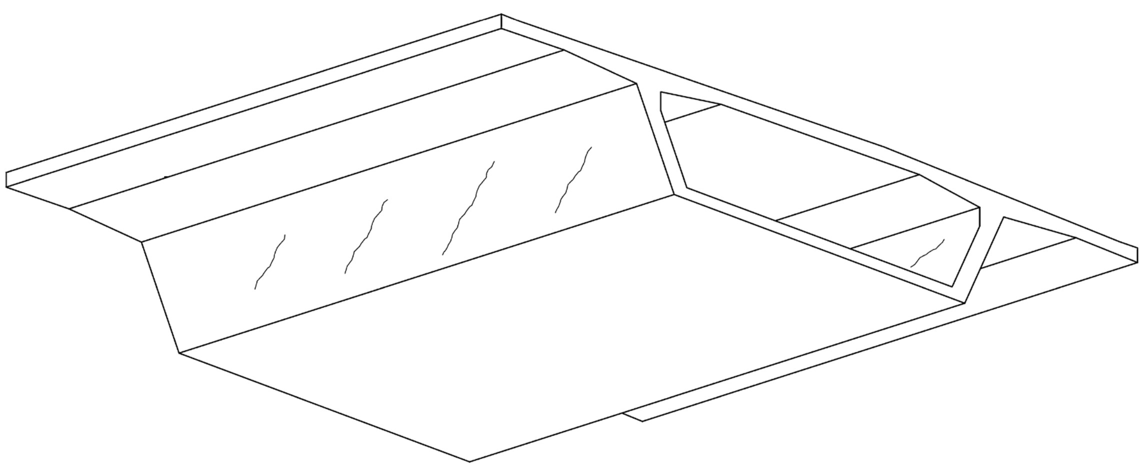


Figure 3.5 – Superstructure Potential Crack Pattern for Pure Shear

Pure torsion would produce inclined cracks that continuously wrap around the entire box section, excluding the cantilever wings (Figure 3.6). However, with the combined effects of shear and torsion, it is possible for one web to be cracked while the other web sees little or no cracking.

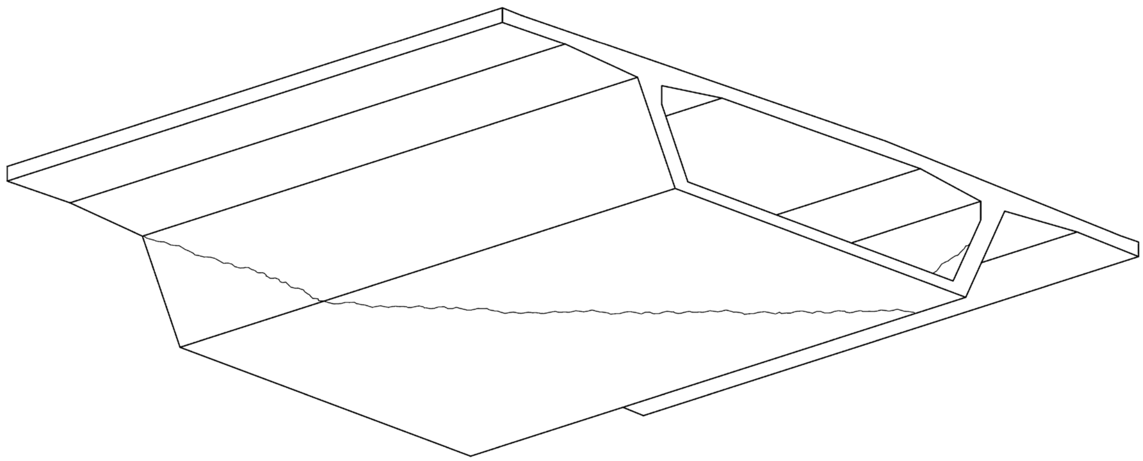


Figure 3.6 – Superstructure Potential Crack Pattern for Pure Torsion

Thermal cracks, due to a difference in temperature between adjacent elements of different thickness, would produce longitudinal cracks near the change in cross-section (Figure 3.7).

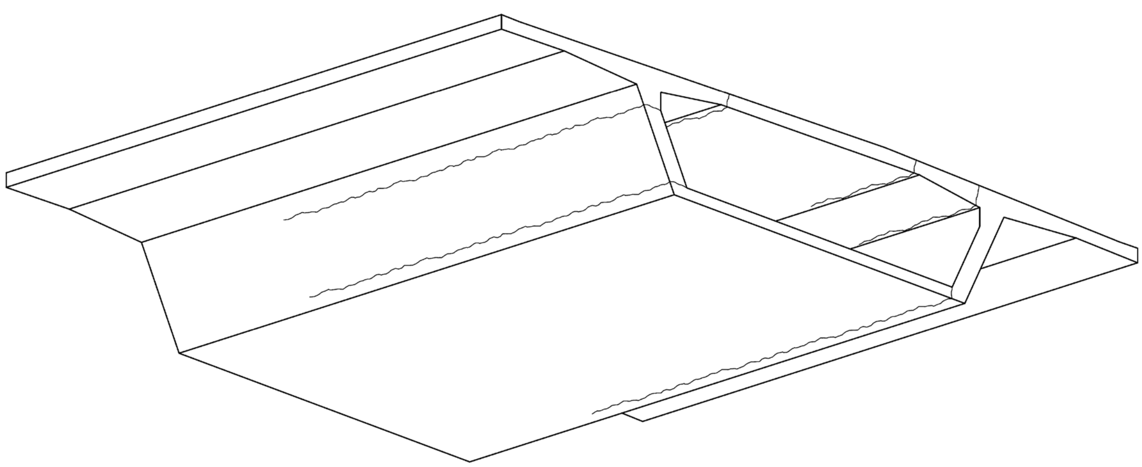


Figure 3.7 – Superstructure Potential Crack Pattern for Thermal Effects

Transverse deck flexure cracking would correspond to bending moments in the top or bottom slabs of the girder and could produce longitudinal cracks at the top of the slab near the webs (negative bending) or at mid-span of the slab at the bottom (positive bending) (Figure 3.8).

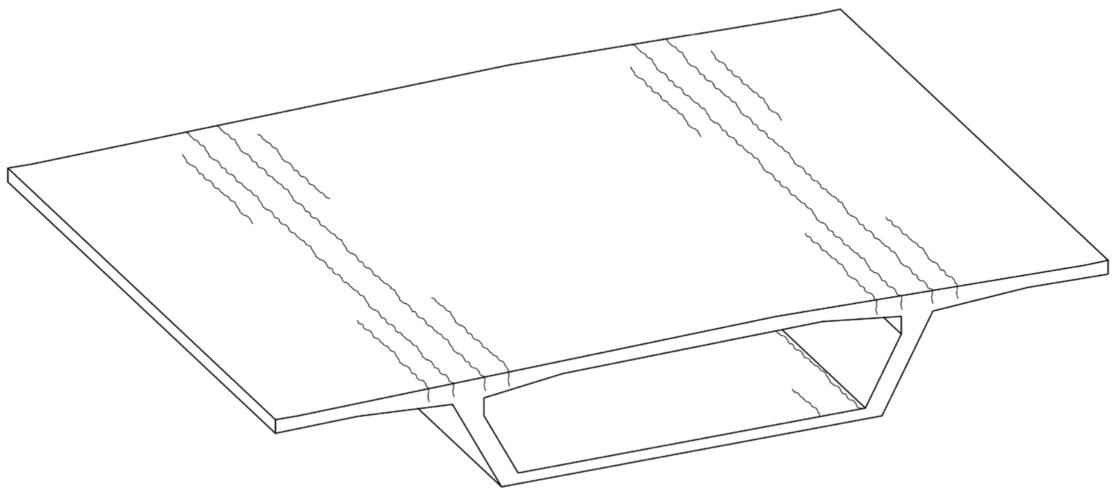


Figure 3.8 – Superstructure Potential Crack Pattern for Transverse Deck Flexure

Concentrated tendon loads on bottom slab anchor blocks can produce cracks in the blocks, transverse cracks in the web or bottom slab behind the anchorage, and longitudinal cracks in the bottom slab (where tendon paths deviate).

Types of Potential Cracking in the Substructure

While cracking of the pier columns is unlikely, flexural cracking is the most potential form of cracking in pier columns. This mode of cracking results in a parallel series of cracks transverse to the pier axis (horizontal cracks).

Pier cap cracks are most likely to occur between the bearings, with cracks aligned parallel to the pier axis (vertical cracks) and across the pier cap top surface in the longitudinal direction.

The potential for footing cracks exist due to thermal effects from casting the footings. Since the footings are considered a mass concrete element, differentials within the footing during construction could cause some cracking of the element. These cracks would run vertically around the footing, from the top of the element to the limit of the footing pour.

There is a potential for vertical cracking in the abutment due to the effects of shrinkage of the concrete during the curing process.

Recording Cracks

Regardless of the cause, an inspector should map accurately the location and size of all significant cracks along with any indication of increases in size. Significant cracks should be accurately sketched onto the survey sheets. Interior superstructure cracks may be traced directly on the concrete along with the month and year of observation.

Significant cracks are defined as those that are either indicative of a significant structural concern or will result in a recommendation for short or long-term maintenance. This means hairline cracks in mildly reinforced concrete are most likely not significant. However, hairline cracks in pre-compressed zones of post-tensioned concrete may be significant and should be reported if the inspector considers it indicative of a potential structural concern.

3.2.2 Spalling of Concrete

Spalls are defined as depressions resulting from the detachment of member fragments from the larger mass of concrete. Spalling may be caused by defects built into the concrete during construction, excessive stresses in the concrete resulting from a concentrated load, an impact load, a chemical reaction between components of the concrete mixture, or expansive forces within concrete resulting from embedded reinforcement corrosion or freezing water.

The same survey sheets used to record cracking should be used for spalls. The following scale, developed by the Federal Highway Administration, should be used for classifying spalls:

Light:	0” to 3/16” coarse aggregate not exposed
Medium:	3/16” to 3/8” coarse aggregate exposed
Heavy:	3/8” to 1” coarse aggregate projecting from the surface
Severe:	over 1” loss of coarse aggregate particles

Two additional items should be included when reporting a spall. First, it should be noted whether new surfaces created by a spall are corrosion stained. This helps determine if corrosion may have caused the spall. Second, it should be noted whether any reinforcing or post-tensioning ducts are exposed by the spall.

Spalling of the deck is especially critical. Potholes may be hazardous for driving and can increase the impact loading, which in turn propagates the spall. Deck spalls also allow entry of de-icing chemicals and water into the top slab. Deck spalls should be addressed immediately if encountered.

3.3 Substructure Inspection

The major components of the substructure include foundations, piers, and abutments. The inspection of these components is discussed below.

3.3.1 Foundations

The foundations at the abutments and Piers 3, 4, 5, and 9 are below, or partially below, grade. Therefore, these foundations may not be accessible for direct inspection. Information on crack and spall inspection is given for completeness and in the event that suspected problems require excavation for inspection of the footings.

The foundations at Piers 1 and 2 are below the normal pool elevation of the main Mississippi River and the foundations at Piers 6, 7, and 8 are below the normal pool elevation of the east channel. Therefore, these foundations will not be accessible for direct inspection.

3.3.1.1 Abutment Foundations

Any suspected movement of the abutment should be checked by survey. If exposed by excavation, abutment surfaces should be inspected for cracks and spalls along with the degree of severity. Any evidence of corrosion staining should also be noted.

3.3.1.2 Pier Foundations

As with abutment foundations, any suspected movement should be checked by survey. Movement of the pier foundation would be apparent in the pier column supported by the footing. Cast-in-place footings surround/encase the top of the pile foundations. Footing surfaces should be inspected for cracks, spalls, and signs of corrosion staining if visible. Indications of distress in the footing may be a sign of problems with the piles.

3.3.2 Piers

Piers 1, 2, and 3 consist of twin walls constructed with cast-in-place reinforced concrete. The twin walls of Piers 1 and 2 are cast atop solid reinforced concrete pedestals, which are supported on the footings. The twin walls of Pier 3 are supported directly on the footing. Pier 4 is a cast-in-place reinforced concrete transition pier supported directly on a footing. Piers 5 through 9 are typical cast-in-place reinforced concrete piers supported directly on footings.

3.3.2.1 Pier Columns

The pier column should be inspected for cracks, spalls, and evidence of corrosion. Flexural cracking near the pier mid-section is the most probable type of cracking.

3.3.2.2 Pier Caps

Pier caps should be inspected for cracks, spalls and evidence of corrosion. Cracking between bearings, with cracks aligned in the longitudinal direction or vertically on the pier faces, are the most likely type of cracking. Disc bearings are located on Pier 4 (downstation) and elastomeric bearings are located on Pier 4 (upstation) through Pier 9.

3.3.3 Abutments

All elements of the abutments should be inspected for cracks, spalls and evidence of corrosion. Any cracks with efflorescence around them indicate that water has previously penetrated into the element.

3.4 Disc Bearing Inspection

Guided and non-guided sliding disc bearings are used on the unit 1 portion of the bridge. The inspection of the bearings should be carried out by accessing the west abutment bearing seat and the pier 4 column cap via a ladder or via an under-bridge inspection truck.

3.4.1 Sliding Disc Bearings (Guided and Non-Guided)

All steel elements should be checked for corrosion and the condition of the zinc coating noted, the concrete adjacent to the top plate and grout adjacent to the bottom masonry plate should be checked for spalls or cracks, and the bolts attaching the top plate and bottom masonry plate should be checked and tightened if necessary. The polyether urethane disc element should be checked for potential cracking and UV exposure damage.

Disc bearings are designed for a maximum rotation of 0.02 radians (1.15 degrees). The actual rotation of the bearings can be estimated by measuring, to within 1/32”, the vertical distance between the bearing plates at each of the four corners. The distance between the four corners should also be recorded. The bearing rotation along each side of the bearing plate in radians is then calculated as the difference in height between two corners divided by the distance between the two corners, both measured in inches. Excessive rotation or great differences in disc element height between bearings may indicate structural issues or foundation settlement.

The horizontal location of the sliding plate relative to the bearing plate should be measured at all sliding bearings. These measurements and concrete temperatures are entered on the bearing inspection survey sheets given in Appendix B for each bearing. The appendix illustrates how to calculate total movement required in each direction based on current date and temperature. These values are compared to measurements taken to ensure that future movement capacity is available.

Lastly, the sliding surfaces should be inspected for delamination of the stainless steel layer or the PTFE layer, checked to ensure free movement is still occurring, and the stainless surface should be checked for cleanliness and for scratches that may indicate foreign matter is lodged between the sliding surfaces. If a bearing is suspected to be ‘frozen’, a disc bearing manufacturer or equivalent expert shall be consulted to assess the bearing.

3.5 Superstructure Inspection

The major components of the superstructure include the concrete box girder and the post-tensioning system.

3.5.1 Concrete Box Girder

Inspection of the concrete box girder includes the following: deck, webs, bottom slab, cast-in-place segment joints, diaphragms, and anchor blocks.

3.5.1.1 Deck

The deck is the most critical element for protecting the structure from infiltration by water and de-icing chemicals. Check for cracks, spalls, and any evidence of corrosion. Inspect under cantilever wings and inside the box for signs of efflorescence or corrosion staining. Both indicate water seepage through the deck. Observe carefully the bottom of cantilever wings near outside edges for cracking and seepage near transverse post-tensioning anchorages. Check for any evidence of segment joint openings or leakage. This is particularly important because post-tensioning ducts are spliced at these locations, which provides a potential path for water to reach a tendon if joints do not remain closed and sealed. Deck drains should also be checked to make sure they are unblocked and draining freely. Note any areas of the deck where water tends to pond.

Pay special attention to the deck of the closure segments. The closure segments include a high performance PPC overlay that should be inspected and compared to the baseline condition. Inspect the top slab access hatches and their PPC overlay for cracking and signs of leakage.

3.5.1.2 Webs and Bottom Slab

Inspect for cracks, spalls, and any evidence of corrosion. If it is deemed necessary to do so, the bottom slab could be inspected when a positive thermal gradient exists in the box girder. The middle or late afternoon of a sunny day provides a potential scenario for checking of thermal gradient effects. For further explanation of thermal gradients, refer to Section 3.7.4. Also, check that no water is ponding inside the box girder and that bottom slab vents are unblocked and draining freely.

3.5.1.3 Cast-In-Place Segment Joints

Cast-in-place segment joints should be inspected for evidence of joint opening in the form of cracks along the joint or water seepage. In the event a joint is suspected of leaking, it should be noted and sealed using approved MnDOT procedures.

3.5.1.4 Diaphragms

Diaphragms are highly stressed critical structural elements. They should be carefully inspected for cracks, spalls and any sign of corrosion. Potential areas for cracks are at the junction between the diaphragm and webs, deck, or bottom slab. In addition, inspect around PT anchors and on the diaphragm faces as these are other areas subject to potential cracking.

3.5.1.5 Anchor Blocks

Bottom slab anchor blocks are highly stressed and should be closely inspected for cracks, spalling, and evidence of corrosion. Cracking would most likely develop parallel and above the duct in the block and in the bottom slab and web directly behind the anchorage.

3.5.2 Post-Tensioning

Post-tensioning tendons are critical structural elements that keep the concrete stresses within allowable limits. They comprise the main longitudinal reinforcing for the superstructure and transverse reinforcing for the deck. The following subsections discuss important points related to the inspection of external draped tendons, internal tendons, and anchorages.

3.5.2.1 External Draped Tendons

Inspection of external tendons is easily accomplished from inside the box girder. Check for cracks or unusual deformations in the polyethylene ducts and ensure that couplings between the rigid pipes and polyethylene ducts adjacent to piers, expansion joints, and deviation diaphragms are watertight. Look for any signs of corrosion from tendons and the steel pipes embedded in the diaphragms. Any physical damage to the tendon, the polyethylene ducts, or the neoprene tendon dampers should be noted. Nonlinearities of the external tendon polyethylene ducts may exist due to deformations of the duct between a duct support system used prior to grouting. Any such nonlinearities are the duct only and are not cause for concern. Inspectors should consult the MnDOT Bridge Office to determine if additional inspection is required for external draped tendons.

3.5.2.2 Internal Tendons

Inspection of internal tendons is not directly possible. However, signs of significant deficiencies such as concrete cracks parallel to the tendons, spalling of concrete alongside the tendons, and rust stains on the concrete can be detected. Internal tendon and post-tensioning bar locations should be checked for these possible indicators.

3.5.2.3 Anchorages

Post-tensioning anchorages are a critical component of the post-tensioning system to help ensure forces are applied to the structure as designed by the engineer. Also, anchorages are typically the post-tensioning component most susceptible to corrosion; therefore, inspection of anchorage areas is critical. There are numerous types of anchorages and anchorage protection systems on the Dresbach Bridge; a brief description of each follows with a guideline for inspection.

Cantilever Tendon Anchorages: Cantilever tendons anchor between segment joints at the top of the web. These anchorages are protected by an epoxy grout-filled non-metallic (plastic) permanent end cap. Since the anchorages are not exposed, inspection should include observing the condition of the surrounding concrete and noting any signs of corrosion such as staining or spalling.

Draped and Bottom Slab Anchorages: These anchorages are located at the top of pier diaphragms (Figure 3.9), along the webs of expansion joint diaphragms, and at intermediate locations along the bottom slab where tendon anchorages are required. A grout-filled non-metallic (plastic) permanent end cap, encased in a secondary epoxy grout pourback with a protective elastomeric membrane, protects draped and bottom slab anchorages. Inspection of these anchorages should include a thorough assessment of the elastomeric membrane covering these anchorages and the pourback. Cracks in the elastomeric membrane, pourback, or separation of elastomeric membrane should be noted. Inspection of these anchorages should also include observing the condition of surrounding concrete and noting any signs of corrosion such as staining or spalling.

Figure 3.9 – Draped Tendon Anchorages

Draped and bottom slab anchorages at expansion joint diaphragms are located on the outside face of the diaphragm. Special consideration of tendon anchorages located at expansion joint diaphragms beneath expansion joints is necessary (West Abutment and Pier 4). These areas are likely locations for infiltration of water and chlorides from the bridge deck. Given their more exposed location, inspection of these anchorages should include a thorough assessment of elastomeric membranes covering the secondary pourbacks, such as cracks in the elastomeric membrane or separation of elastomeric membrane, which should be noted. Inspection should also include observing the condition of surrounding concrete, checking the soundness of blocks (i.e. no separation), and noting any signs of corrosion such as staining or spalling.

Transverse Tendon Anchorages: These anchorages are protected by grout-filled non-metallic (plastic) permanent end caps, which are surrounded by concrete either integral to the girder top slab or by a secondary epoxy grout pourback. Anchorages with secondary pourbacks are those where the anchorage was blocked-out for stressing access. Inspection of transverse tendon anchors can be done from the deck, using a mirror extended beyond the wingtip. Inspection should include observing the condition of surrounding concrete and noting any signs of corrosion such as staining or spalling.

Pier Table Tendon Anchorages: These anchorages are located near the bottom slab on the exterior face of the pier tables. These anchors are protected by an epoxy grout-filled non-metallic (plastic) permanent end cap inside an epoxy grout blockout. Inspection of the pier table diagonal tendon anchors can be done from outside the girder. Inspection should include observing the condition of surrounding concrete and noting any signs of corrosion such as staining or spalling.

Post-Tensioning Bar Anchorages: For the vertical post-tensioning bars that anchor in the deck of the pier table and end diaphragms, the anchorage blockouts were poured-back with an epoxy-based grout. Inspection of these anchorages should include careful inspection of the pourbacks for signs of separation from surrounding concrete, as well as soundness of the pourbacks (note: visible location of pourbacks will be restricted by the bridge coating). Transverse post-tensioning bars anchor in the deviation diagrams near the bottom slab on the exterior face of the deviation diagrams. The block outs for these bars were poured back with an epoxy-based grout after stressing, and the pourbacks should be inspected in the same manner as the vertical post-tensioning bar pourbacks.

3.6 Expansion Joint Device Inspection

The following subsections discuss inspection of the modular expansion joints and the associated bridge barrier sliding plates.

3.6.1 Modular Expansion Joints

Each modular expansion joint should be examined for freedom of movement (signs of distress to the rails, support beams, or surrounding concrete indicate lack of freedom of movement), proper opening (spaces between rails should be equal), proper vertical alignment, debris accumulation, and watertightness of the seals and the joint between the concrete and the exterior rails. Predicted opening width depends on the age of the structure and average temperature of the superstructure concrete substrate at the time of the inspection. Measurement of the joint opening should be performed using the joint survey sheet in Appendix B. Concurrent concrete temperature and date must also be recorded to ensure proper interpretation of the data. Calculated minimum and maximum joint openings can then be compared to measured openings to determine if enough movement capacity exists in each direction.

Each expansion joint assembly should be inspected for loose or damaged parts, accumulation of trash, soil, rocks, and/or other foreign material that may restrict free movement or apply pressure on the seals. Stones lodged in the joints can create localized stresses that may cause cracking or spalling in the surrounding deck concrete or damage the neoprene seals, allowing the joint to leak.

The condition of the neoprene seals in the joint should be observed. This includes inspecting the joint from the underside within the box girder, for evidence of water infiltration, signs that the seals are pulling away from the steel rails, abrasion, shriveling, hardening or cracking, and UV damage. Moisture or staining on the underside of the deck or on the abutment backwall may indicate the seals are leaking. Also check the joint between the exterior rails and concrete for delamination or leaks.

Concrete in the vicinity of expansion joint assemblies should be sounded and examined for evidence of cracking, voids, or delamination of the deck. This includes the entire thickened portion of the box girder cantilever wings.

3.6.2 Bridge Barrier Sliding Plates

The bridge barrier incorporates galvanized steel base plates with sliding cover plates that bridge the gap in the barrier over the expansion joints. The sliding cover plates are bolted in place and removable with the aid of a truck-mounted lift, if necessary. Inspection should identify any steel plate damage that could interfere with free movement of the sliding plate. The integrity of the counter-sunk anchor bolts and surrounding concrete as well as any indications of rush should also be noted.

3.7 Deformation Monitoring

The following sections discuss the deformation monitoring for substructure settlement, pier movement, superstructure creep and shrinkage, and superstructure thermal effects.

3.7.1 Substructure Settlement

It is not anticipated that any special monitoring for settlement will be required. However, should any suspicious or unusual observations be found during the superstructure inspection (see Section 3.7.3), further investigations should be undertaken. This should include surveys of the suspected footing and pier to determine if there are non-uniform or excessive settlements.

3.7.2 Piers

Piers do not specifically need to be monitored for deformation unless a problem is suggested by the results of other inspection or monitoring. This would include unusual or unexpected longitudinal movement indicated from the bearing monitoring program (see Section 3.4). Surveying to determine relative movements at the top and base of the pier would then be appropriate.

3.7.3 Superstructure Creep and Shrinkage

Creep and shrinkage are both phenomena that occur in concrete structures over a long period of time. The effects are even greater in post-tensioned concrete structures and generally take approximately thirty years to stabilize, with the majority of effects taking place in the first three or four years. Creep is additional deflection in a structure on a long-term basis due to a sustained load such as self-weight and post-tensioning. A post-tensioned concrete structure experiences greater creep effects compared to a conventionally reinforced concrete structure because of the precompression applied over the cross-section. Shrinkage is simply a reduction in volume of the concrete, which begins during curing and continues for approximately ten years, regardless of the loads applied.

Creep and shrinkage will produce a longitudinal shortening of the structure over time, accounting for the permanent displacements observed at the expansion joints, bearings, and tops of piers. The effects of creep and shrinkage are most important when considering the movement capacity of bearings and expansion joints in combination with a uniform temperature fall since the effects are additive.

Both creep and shrinkage are critical aspects of concrete segmental design due to the process of staged construction. Since Spans 2 and 3 are built in cantilever and then connected at midspan, the initial self-weight moment diagram is different than that for the portion of the bridge cast entirely on falsework. The moments will be larger at the piers and smaller at midspan for the segmental structure. However, over a period of time due to creep and shrinkage, the moments will shift and redistribute toward the moments one would see for a bridge cast on falsework. This produces larger midspan positive moments over time that were accounted for in the design. One result of this shift in moments is an increase in downward vertical deflections of the bridge spans with time.

The structure was built with an initial camber in the opposite direction of expected creep deflections. Theoretically, if the bridge was initially erected with the calculated camber and the bridge behaves as predicted, then vertical alignment should match the design roadway alignment after all long-term deformations have occurred. However, variations between the as-built elevations and predicted elevations always exist and need to be taken into account if comparing survey results to the plan vertical roadway alignment.

To monitor the deformation of the superstructure, perform a visual inspection of the deck by sighting down the bridge barrier. In addition, visually inspect the bottom of the box girder near the middle of the spans and the top of the deck over the piers for substantial traverse cracking. Transverse cracking in these two locations can be indicative of larger displacements occurring in the structure. If larger displacements are suspected, a deck profile survey should be performed and compared to the baseline survey that is included in Appendix A.

3.7.4 Superstructure Thermal

Two types of thermal effects in the superstructure are important. First, when the overall average bridge temperature changes, the bridge shortens with a drop in temperature and elongates with temperature rise. This affects the bearing and expansion joint displacements and applies a horizontal force at the top of piers with fixed bearings. The other thermal effect is when a differential temperature exists through the structure depth (thermal gradient). This typically occurs when sunshine on the bridge deck warms the top slab, while webs and bottom slab remain cooler. This results in a positive longitudinal moment applied to the superstructure, corresponding to an increase in compression at the top of the section and an increase in tension in the bottom.

Monitoring structure deformations due to average bridge temperature change is done by measuring the concrete temperature and corresponding movement of the bridge at sliding bearings and expansion joints. The date of the readings is also noted, since creep and shrinkage of the bridge has an effect on longitudinal movements. The primary concern is that bearings and expansion joints have sufficient movement capacity during the coldest and warmest parts of the season. The coldest seasonal movements normally control since direction of movement is the same as creep and shrinkage movements. Movement capacities are verified by using the measurements shown on the bearing and expansion joint survey sheets (see Appendix B). Using information provided in Appendix B on the survey sheets, additional required movement capacities are calculated and compared to actual remaining movement capacities.

It is not necessary to monitor structure deformations due to thermal gradient.

3.8 Inspection Forms

Appendix B contains forms to record observations and measurements taken during inspections. Also included is information to assist with calculating anticipated movement capacities required for bearings and expansion joints.

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MAINTENANCE PROCEDURES

4. MAINTENANCE PROCEDURES

4.1 General

Experience in highway operations has shown that continuous and systematic maintenance of a bridge will extend its service life and reduce its operating expenses. Nevertheless, maintenance of bridges and their approaches is one of the most neglected aspects of highway operation.

Regular inspection of the components of the Dresbach Bridge, as discussed in Chapter 3, should be made to locate areas that need attention before they develop into problems needing repair. In addition to scheduled inspections, maintenance personnel should be alerted to conditions that should be monitored on a routine basis to avoid further damage.

Items of concern noted during an inspection or by maintenance personnel should be addressed in a reasonable period of time. When a condition is identified during an inspection, the following questions should be considered:

- Is the symptom one that occurred in the past or is it still evolving?

- If the symptom is no longer evolving, is the structure functioning as intended in the original design and is maintenance needed? Symptoms discovered during construction or soon after completion often fit into this category.

- If the symptom is still evolving:
    - What are the potential situations that may develop?
    - Should a special monitoring procedure be established?
    - Should the area of the bridge in which the symptom is located be inspected at more frequent intervals?
    - Is maintenance, repair, or rehabilitation required?

In many cases, knowledge of the design and construction of concrete segmental bridges is necessary to properly diagnose symptoms, foresee the effect of specific symptoms on the structural performance of the bridge, and recommend appropriate actions, if any.

The following definitions are provided for clarity:

Maintenance: Maintenance is defined as corrective or preventative action that sustains the current level of performance.

Repair: Repair is defined as corrective action that significantly improves the level of performance without exceeding the original level of performance.

Rehabilitation: Rehabilitation is defined as a larger corrective action that results in restoring the bridge structural integrity to its original level of design standards or higher.

Maintenance activities are divided into routine maintenance (those activities carried out on a day-to-day basis) and periodic maintenance (those activities which are planned over a longer term on the basis of identified needs).

Routine maintenance items may include:

- Expansion joint device cleaning
- Deck drain cleaning

Periodic maintenance items may include:

- Traffic railing repairs (due to vehicular damage)
- Removal of existing integral overlay and placement of future overlay

4.2 Concrete General

The following sections outline maintenance procedures for cracking and spalling of concrete.

4.2.1 Cracking

In evaluating the significance of cracks that have occurred in any concrete structure, it is first necessary to determine the cause of such cracks. Section 3.2.1 discusses potential causes of concrete cracking.

When a crack is determined to be stress related, the cause of overstressing of the concrete should be identified and a determination made as to whether or not the overstressing is critical enough to warrant corrective action. Some overstressing is anticipated during the service life of a bridge and corrective action to eliminate this condition may not be necessary.

As mentioned in Section 3.2.1, non-stress related cracks in concrete provide a path for water and chlorides to infiltrate the concrete, which contributes to deterioration of the concrete or corrosion of embedded reinforcing steel. If a crack passes entirely through the deck of a concrete box girder, it may also allow water to reach the interior of the box. The effect of cracking on the long-term performance of a concrete structure is related to the environment in which the cracked concrete is located and the degree to which cracking has occurred.

4.2.1.1 Crack Width Criteria

The significance of various crack widths was discussed in Section 3.2.1. The following information is repeated here for convenience.

According to recommendations by the American Concrete Institute, cracks in reinforced concrete should not be considered to be a significant factor in causing corrosion of embedded reinforcing steel until the width exceeds:

Environment	Width (1 mil = 0.001 in.)
Dry Air	16 mils (0.016 in)
Moist Air	12 mils (0.012 in)
Exposure to Deicing Chemicals	7 mils (0.007 in)

The following is a guide applying the above criteria to the various elements of the Dresbach Bridge:

Element of Substructure:	Cracks May Be Significant If Width Exceeds:
Footings	10 mils (0.010 in)
Piers	12 mils (0.012 in)
Pier Caps	10 mils (0.010 in)
Abutments	10 mils (0.010 in)

Concrete Box Girder:

Deck	7 mils (0.007 in)
Web	12 mils (0.012 in)
Bottom Slab	12 mils (0.012 in)
Abutment and Pier Diaphragms	16 mils (0.016 in)

4.2.1.2 Crack Repair

The first task for repairing cracks is to determine the probable cause(s) of the cracking. If it is determined that a structural repair, such as adding reinforcing or post-tensioning is necessary, the work should be done at the same time as the crack repair to prevent re-cracking of the concrete.

The need for sealing cracks should be determined by the Minnesota Department of Transportation based on the above discussion of crack widths and recognizing the relative importance of cracks to the long-term serviceability of the structure based on crack location, orientation, and width. Suggested criteria for crack sealing are shown below.

- Top slab of box girder - Seal cracks that are 7 mils or more in width, or that are showing signs of leakage such as efflorescence. Seal any leaking segment joints.

- Expansion Joint regions - Seal cracks on both interior and exterior surfaces that are exposed to the expansion joint opening and are 7 mils or more in width.

- Footings and abutments – Seal cracks that are 10 mils or more in width.

- Box girder webs, bottom slab, and piers - Seal cracks that are 12 mils or more in width.

- Abutment and pier diaphragms – Seal cracks that are 16 mils or more in width.

There are several methods available for sealing cracks. One of the most commonly used methods is epoxy pressure injection. This has the advantage of restoring the tensile capacity of the concrete in addition to sealing the crack. Epoxy pressure injection is discussed in detail below. For horizontal surfaces, such as the deck or interior of the bottom soffit, a gravity-fed penetrating low viscosity epoxy or methacrylate sealer will work well. These types of products are especially good for sealing leaks in epoxy segment joints. Concrete surface sealers are also available that can be applied by spray or brush/roller. However, the penetration of these sealers into the concrete is limited, and therefore reapplication may be necessary on a regular basis.

Epoxy Pressure Injection

Epoxy resins generally can be used to repair cracks and bond fractured sections. It may be convenient to rout or widen cracks and then fill with latex mortar overlay. However, cracks should not be intentionally widened in post-tensioned elements.

Cracks ranging in width from 5-6 mils (0.005” to 0.006”) to ¼” can be successfully filled. Cracks wider than ¼” generally require a system incorporating a mineral filler. As noted above, some cracks extending downward from nearly horizontal surfaces may be filled by gravity using special materials. The minimum width of crack that can be filled by gravity is a function of the viscosity of the fill material.

Cracks that are to be filled with epoxy should be free of dust, oil, disintegrating material, or any debris that could block the flow of resin. A minimum preparation as described in the following steps is required with epoxy.

- The surface along the crack must be clean. A chemical, mechanical, or water blasting method can be used to remove dried mud, grease, or other foreign material. Concrete coatings, such as paint, should be removed. Care must be exercised to keep debris from contaminating the crack.

- Loose debris at or near the surface of the crack should be blown out with an air jet of 75 to 100 psi that is free of oil and moisture. Any large, loose particles should be removed by hand.

- The crack should be prepared in strict accordance with epoxy manufacturer's recommendations.

- If drilling is necessary, vacuum drill bits are advisable to prevent drill dust from sealing off narrow cracks. Cracks to be filled with latex modified mortar should be prepared by routing to a depth of at least two inches and then by cleaning with air blasting, high-pressure water jetting, or sandblasting.

The epoxy system should be a type approved by the Minnesota Department of Transportation. The grade and class should satisfy job conditions and requirements. The system should be capable of bonding to wet surfaces unless it can be assured that the crack is dry. Epoxy injection should be performed per epoxy manufacturer's specifications.

Entry ports for pressure injection should be properly spaced along cracks. While guidelines are given for proper spacing, good judgment should be the final criterion. Guidelines for port spacing in partial depth cracks are as follows:

- Space ports at the desired depth of penetration. This allows the resin to travel as far into the crack as along the face of the crack. If port-to-port travel at this spacing is not obtained, establish intermediate ports.

- If the cracks are less than 5 mils (0.005") wide, entry ports should not be spaced more than 6 inches apart.

- If the cracks are more than 24 inches in depth, full penetration may be difficult to achieve because of equipment limitations. Intermediate ports should be established to monitor the flow of epoxy.

Guidelines for port spacing in cracks that extend the full member depth are as follows:

- For members 12 inches or less in thickness, place ports in the crack along one side spaced at a distance equal to the thickness of the member.

- For members greater than 12 inches and less than 24 inches in thickness, ports are placed in the crack on all available sides. Space the ports less than the thickness of the member.

- For members greater than 24 inches in thickness, place ports along the crack on all available sides. The ports are spaced according to the guidelines set forth for partial depth cracks.

The first and last entry ports are respectively established at or near the bottom and top of any vertical crack, or at the ends of any horizontal crack in a vertical or horizontal member.

After applying and allowing for proper cure of the epoxy surface sealer around the ports and over the full length of the crack, the epoxy injection can begin. Injection should start at the lowest injection port and continue until epoxy flows from the adjacent port. Cap the first port and inject into the second port until epoxy flows from the third port. Continue the process until epoxy flows from the last port. Note, if flow does not occur, maintain pressure on the non-flowing port for approximately 2 minutes, which will produce a pressurized topical seal.

Where appearance is important, lines or spills of epoxy must be avoided or removed along with surface seals from exposed surfaces.

4.2.2 Spalling

The following sections discuss the significance of spalling and the necessary steps required to repair and patch spalled concrete.

4.2.2.1 Significance

As discussed in Chapter 3, spalling of concrete may occur near expansion joints in the superstructure, near bearings on pier caps, at locations where cracking has become severe, at locations of expansive chemical reactions within the concrete, or areas of extensive corrosion along embedded reinforcing steel.

Before repairing spalls, the probable cause of the deterioration should be determined. If a structural repair such as adding reinforcing steel is needed, it should be done at the same time as the spall repair, so that the spall does not later recur.

The urgency of repairing spall locations depends on the severity of the spall and its effect on corrosion of reinforcing or post-tensioning steel. Extensive spalling in an area where concrete stresses are critical should be immediately investigated to determine if the safe load-carrying capacity of the bridge is significantly reduced. Spalls are considered large when their size exceeds approximately ¾” in depth or 6” in plan dimension. Section 3.2.2 discusses the inspection for spalls. The following information is repeated here for convenience.

The following scale, developed by the Federal Highway Administration, should be used for classifying spalls:

Light:	0” to 3/16” coarse aggregate not exposed
Medium:	3/16” to 3/8” coarse aggregate exposed
Heavy:	3/8” to 1” coarse aggregate projecting from the surface
Severe:	over 1” loss of coarse aggregate particles

4.2.2.2 Spall Repair and Patching

Extensive information is available regarding spalls, delamination, and procedures for repair. Any conventional, approved patching procedure may be used for repair. The following points should be given careful attention when patching spalls.

<u>Post-Tensioning Tendons</u>

Prior to any concrete removal, the locations of embedded post-tensioning tendons and/or bars should be evaluated. Extreme care should be taken to avoid damage to the post-tensioning and its ductwork.

<u>Concrete Removal</u>

The basic requirement for success of any repair is proper preparation of the existing concrete. Regardless of the position of the bridge member, or the type of repair to be made, all unsound and disintegrated concrete must be removed.

Concrete contaminated with chlorides must be given special consideration for removal. A durable repair requires removal of all concrete that shows evidence of active or potential corrosion. Usually an area of active steel corrosion and chloride-contaminated concrete is considerably larger than the area of spalled or delaminated concrete. If only the area of spalled or delaminated concrete is removed and repaired, a continuing repair program will probably be required. However, a durable repair is obtained if the concrete that contributes to active corrosion is removed and the repair is properly protected with a waterproof membrane, bonded topping, or overlay.

<u>Reinforcing Steel</u>

Delaminated areas and spalls sometimes extend to or beyond the reinforcing steel. Care shall be exercised in the use of saws and other power tools to avoid damage to the steel. If removal of material has exposed more than half the perimeter of a reinforcing bar, it is recommended that the bar be completely exposed with sufficient clearance under the bar to ensure encasement and bond. Reinforcing steel should be clean. Loose, scaly rust should be removed but thin, tightly bonded rust need not be removed. Bits of mortar, if hard and sound, can be left as long as they adhere tightly to the steel. Steel in areas to be patched with materials other than Portland cement concrete should be clean and dry with no loose scale, rust, mortar, or other material. Sandblasting is the most effective cleaning method.

The reinforcing bars located in the deck and barriers is stainless steel. The remaining reinforcing steel in the superstructure and in the substructure above footings is epoxy coated. Any damage to the epoxy coating should be repaired with an approved repair kit prior to placement of patching materials.

Where reinforcing steel is to be cut and replaced, lap splices, with accepted lap lengths as determined by a Bridge Engineer, are required.

Patch Edges

Top edges of the deck areas to be patched should be sharp to ensure good contact between the substrate and patch material. Edges of deck patch areas are best if they are sawcut to eliminate feathered edges. To provide a keyed patch, bevel the cut away from the area to be repaired. If a saw cut is made, an undercut edge can be made by placing one wheel of the saw carriage on a plank. The saw cut should not overrun the area to be patched nor should it be so deep as to cut reinforcing. Patch areas other than on the deck may be done by tilting the saw blade to accomplish the keyed patch. Some non-deck area patches may only require hand tools.

Tools

Chipping tools must be selected so that they will not damage surrounding areas. Where unnecessarily heavy equipment, or sharp-edged tools are used, possible damage to the surrounding concrete can create additional areas of potential failure.

Where only partial-depth patching is required, additional limitations on air hammer operators should be made to avoid breaking through the deck or fracturing the concrete below the partial-depth patch area. Special care should be applied in removing unsound concrete from around reinforcing steel and embedded anchorages (such as for expansion joint assemblies). This care prevents loss of bond in the remaining sound concrete.

Surface Preparation

After removal of unsound concrete, the area should be cleaned to remove loose particles and dust. The chipped surface often retains particles that have been broken but not dislodged. Air blasting may be effective, but the compressor should be equipped with a functioning oil trap to prevent contamination. Alternatively, these particles can be removed by high-pressure water jetting or sandblasting.

All concrete surfaces on which new materials are to be bonded must be clean. Tar and asphalt (if present) should be removed by mechanical methods, and the surface then sandblasted. Strong detergents may be useful in removing surface oil contamination. However, oils that have penetrated the surface shall be removed by chipping or scarification and these particles shall be properly disposed of.

Final cleaning should be performed immediately prior to placement of new material to ensure that contamination does not interfere with a good bond. A two-part epoxy bonding agent will insure good bond if the patch material is placed before the bonding agent cures.

Patching

The patching material should be a MnDOT approved patching material and should be placed and cured as per the manufacturer's recommendations or standard practices. All products should be used per manufacturer’s recommendations.

Shotcrete Repairs

For shallow spalls or scaling (the local flaking or peeling of the near surface portion of concrete or mortar), a technique known as "shotcreting" may be employed. Shotcrete is especially adaptable to patching large expanses of shallow scaled or spalled beams, pier caps, barriers, and undersides of decks. ACI Committee 506 gives detailed recommendations.

Surface preparation is similar to that required for other repair methods. One additional requirement is that concrete be removed to form a shape that will not entrap rebounding materials. Stainless steel anchor bolts may be used to tie new material to the old concrete.

4.3 Substructure Maintenance

The following sections outline maintenance procedures for substructure settlement and scour.

4.3.1 Substructure Settlement

Because of the deep pile foundations, it is not anticipated that any significant substructure settlement will occur. However, a vertical alignment survey will identify any problem areas if needed based on observations described in Sections 3.7.1 and 3.7.3. A survey was performed at the end of construction to serve as a baseline for surveys in the future.

If observations described in Sections 3.7.1 and 3.7.3 are noted, then elevation variations of survey markers placed over the pier segments and expansion joint segments should be determined to indicate the settlement of each pier and abutment. Only differential settlements will affect the total load bearing capacity of the superstructure. If the differential settlement between adjacent supports (abutment or pier) exceeds ¾”, a detailed analysis will be required to determine if corrective maintenance is necessary. If differential settlement of a given support in the transverse direction (as measured at the two exterior bearing locations) exceeds ¼”, a detailed analysis will be required to determine if corrective maintenance is necessary.

If corrective maintenance is required, elevations must be surveyed on a monthly basis at the support experiencing excessive settlement and at the adjacent piers. Plots of settlement with time should be established to help evaluate whether the settlement is still occurring. Corrective maintenance may consist of either one or a combination of the following actions:

- Adjust bearing height as a means to correct superstructure elevation. Contact bearing manufacturer for means of shimming bearings without compromising the bearing’s integrity. Jacking the bridge should be carried out in accordance with the procedures described in Section 4.4.3. Shims should be protected from corrosion (e.g. galvanized or stainless steel).

- Install and stress additional post-tensioning tendons. (The future post-tensioning locations provided may be used for this.)

- Grout injection or other techniques to stabilize foundations.

4.3.2 Correction for Scour

Westbound and eastbound Piers 1 through 9 are subject to scour action from the Mississippi River.

Results of the MnDOT foundation inspections for scour, including underwater inspections, should be reviewed with the Engineer of Record to determine if any corrective actions may be required.

4.4 Disc Bearing Maintenance

All bearings should be regularly cleaned of debris and maintained so they function as designed. The stainless steel sliding surface should be gently wiped clean. Sand, dust, debris, etc., often accumulate around the bearings and at times become moisture saturated leading to corrosion and potentially, freezing of the bearing. The corrosion process is accelerated if the moisture contains deicing chemicals. If a bearing freezes, the bridge is restrained from its natural expansion and contraction movements. This restraint can produce tensile and compressive forces in the box section that may lead to damage to the bearings and surrounding concrete.

4.4.1 Releasing Frozen Bearings

If a bearing becomes frozen, so that movement of the bearing is no longer possible, it may be necessary to replace the sliding portions of the bearing or the entire bearing if damage has occurred. The bearing replacement procedures outlined in Section 4.2.3 can be used to perform the necessary repairs. It is recommended the bearing manufacturer is consulted before replacing the bearing in part or in full.

4.4.2 Compensation for Excess Movement

As part of the periodic inspections, the MnDOT Bridge Office should calculate the maximum anticipated movements of the sliding bearings. These are calculated using the forms provided in Appendix B of this manual. The maximum anticipated movements are checked against measured capacities to ensure that the sliding bearing surfaces will maintain complete contact when fully extended due to long-term creep and shrinkage and temperature contractions.

If the calculated anticipated maximum horizontal bearing movements exceed the capacity for a given bearing, more stringent bearing inspections should be planned to monitor the bearing movements and corrective action should be planned if it appears bearing damage will occur.

Corrective action is not recommended for most cases where theoretical bearing sliding capacity is exceeded because movement beyond the bearing capacity may not result in damage to the bearing. For instance, if the movement capacity is exceeded by 1” (1” of the PTFE sliding surface is not in contact with the stainless steel surface), the bearing stresses will only increase slightly through the sliding surface still in contact. Since the stainless steel surface extends to the end of the top plate, damage to the PTFE surface is unlikely. Therefore, exceeding capacity by 1” would likely not result in impaired function and so repair would not be advised.

In the rare case where the bearing surface is greatly reduced or damaged, it may be advised to replace the guide plate and/or the top plate with a longer plate to ensure full contact and function. An Engineer familiar with bearing design should design the replacement parts. The bearing replacement procedures outlined in Section 4.2.3 can be used to install the new guide plate and/or top plate.

If it is determined through field measurements that the disc bearing has experienced rotation beyond the design capacity (see Section 3.2), the cause for the excessive rotation should be identified and corrected, if appropriate. For instance, the rotation may be due to a pier rotation caused by foundation settlement. Disc bearings have inherently more rotation capacity than the design criteria stated so repair may not be required, especially if there is no distress in the bearing plates, sliding plates, or surrounding concrete. It is recommended that the bearing manufacturer be contacted for an assessment prior to performing any repairs.

4.4.3 Bearing Replacement

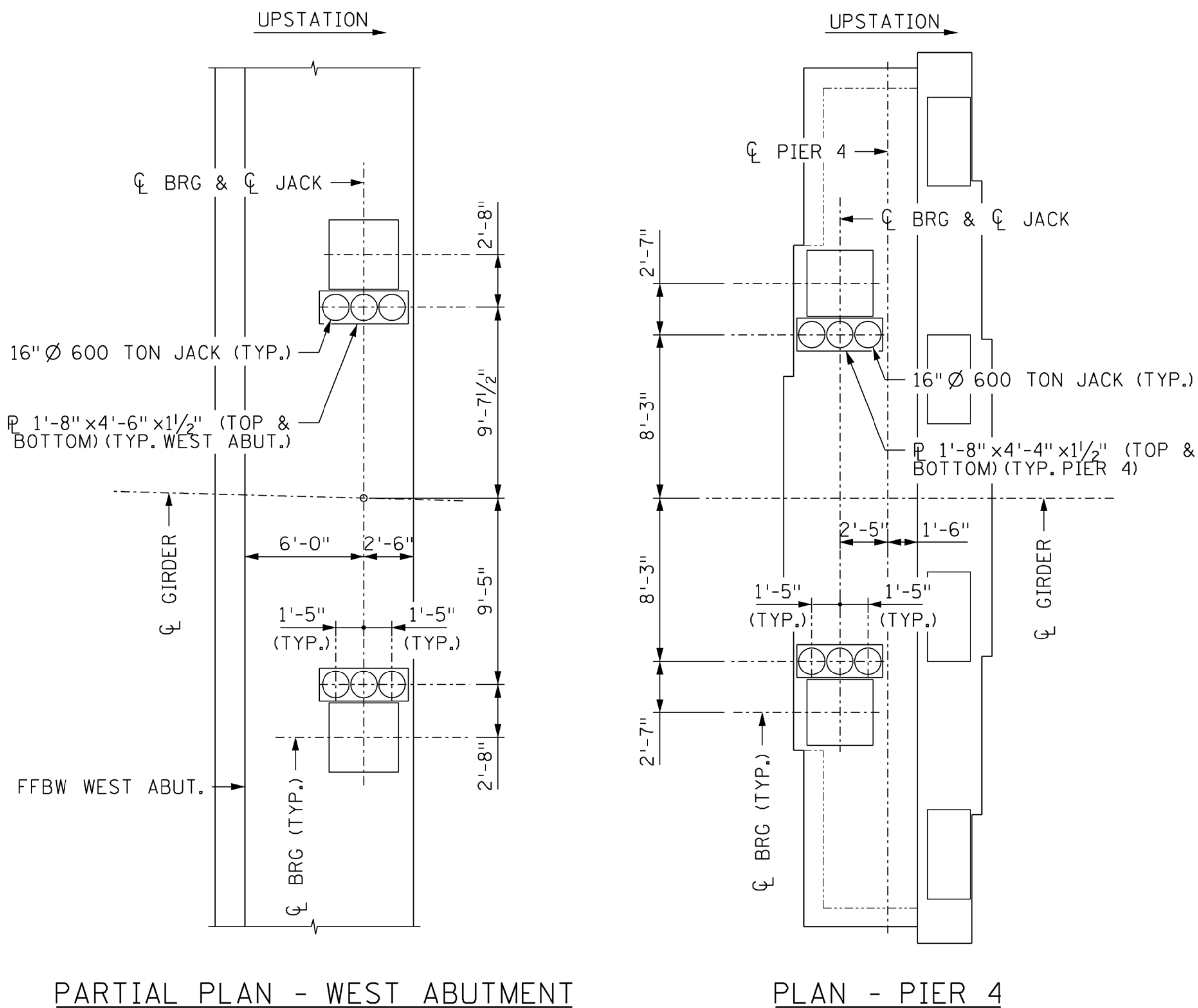
In the event that a bearing becomes unserviceable to the point of needing replacement, the following procedure may be used.

The west abutment or pier 4 should be locked to the superstructure to prevent transverse and longitudinal movement prior to and during jacking of the superstructure. The method for locking these movements shall be designed through careful engineering and must allow for small longitudinal and transverse deflections in the system in order to realign the embedded masonry and sole plates.

Permit loads shall not be allowed on the structure during the entire bearing replacement operation.

Jack the superstructure vertically a maximum of 1/2” using the jacks and jack arrangement shown in Figure 4.1. Steel bearing plates are needed to distribute the jacking forces to the concrete and lock rings on the jacks are advised during replacement to suspend the load. All traffic shall be suspended during jacking and can resume on the structure after the lock rings have been set and the load is no longer supported by the hydraulic system. In accordance with Figure 4.1, all minimum edge distances must be maintained during jacking operations. Jacking should be completed one pier or abutment at a time.

Once the structure has been lifted off the bearings, the masonry plate bolts can be removed and the entire bearing assembly can be removed from the bearing pedestal. The top sole plate can now be unbolted and removed as well. The bottom and top plates are cast into the substructure and superstructure with 6” long stud anchors, and are not typically removed unless severely damaged. If replacement of the embedded plates is required, they must be cut and chipped from the concrete. The removal and replacement of these plates is difficult and is not recommended unless under the supervision of an engineer familiar with this type of replacement.



NOTES:

ALL BEARING COMPONENTS EXCEPT THE SOLE PLATE CAN BE REPLACED BY JACKING THE BRIDGE A MAXIMUM OF ½". JACKS FOR BOTH BEARINGS SHALL BE ENGAGED SIMULTANEOUSLY OFF A COMMON MANIFOLD (6 JACKS TOTAL).

DURING REPLACEMENT, NO PERMIT LOADS SHALL BE ALLOWED ON THE BRIDGE, THE TEMPERATURE SHALL BE ABOVE 0°F AND THE WIND SPEEDS SHALL BE LESS THAN 30 MPH.

Figure 4.1 – Bearing Replacement

Once removed, the reconditioned or new bearing components, of identical height and dimensions as the original bearings, can be replaced in the reverse order to that which they were removed. Finally, the structure can be lowered onto the bearings while maintaining a maximum transverse differential of 1/8”. If the bottom or top plates were replaced, check that there is not a differential in bearing heights that may twist or rotate the superstructure. If such a differential exists, the bearing components shall be shimmed or sized such that the superstructure will rest in its original position prior to jacking. Once the bearing replacement is complete, the paint system on the bearings should be touched up where any painted surfaces have been chipped during placement operations.

Traffic shall be suspended while the structure is lowered onto the bearings. Bearing replacement operations should not occur when the wind speed exceeds 30 miles per hour or the temperature is below 0^{o}F. Lubricated sliding plates should be between the jacks and the superstructure at pier 4 and at the west abutment to ensure the bridge is allowed to move normally and to prevent excessive lateral loads on the jack system.

4.4.4 Bearing Protective Coating

All external steel surfaces were zinc metalized for corrosion control using the arc spray application method. The protective coating is bonded to the steel surface and should not flake or crack under normal wear if the application was performed properly. However, impacts during bearing installation or during the construction of the superstructure may have damaged the coating enough to initiate flaking or cracking. Any observed flaking, cracking or debonded material should be removed with a paint scraper, all corrosion should be blast-cleaned from the steel and patched with a zinc-rich paint system to a thickness greater than 4 mils in accordance with ASTM A 780.

4.5 Superstructure Maintenance

The following sections address the superstructure maintenance for the concrete box girder and post-tensioning system.

4.5.1 Post-Tensioned Concrete Box Girder

In general, cracks, spalls and other defects noted in the post-tensioned concrete box girder should be evaluated to determine if they are stress related and have structural implications. Conditions that might fall into this category include cracking in precompressed tensile zones such as longitudinal or transverse cracks in the deck and transverse cracks in the webs or bottom slab of the box girder. If it is determined that the condition may be a structural concern, an engineer experienced in the design and construction of post-tensioned and/or segmental bridges should be consulted. If a defect is more likely the result of force distribution in reinforced concrete, shrinkage, construction handling, or other circumstance that does not present a concern for the ability of the structure to carry loads as intended, repairs may be performed as discussed in Section 4.2. Items unique to the individual portions of the box girder are discussed in more detail below.

4.5.1.1 Deck

The most important step for maintaining the bridge deck is to prevent moisture penetration. This helps minimize the potential for corrosion and freeze-thaw damage. Areas of suspected leakage noted during inspection, including segment joints, should be sealed either by epoxy pressure injection or by using a gravity-fed low viscosity sealer. Prior to the bridge opening to traffic, the bridge deck was cleaned and sealed.

Impediments to proper bridge deck drainage should be corrected. Deck drains should be regularly cleaned of debris to eliminate any area on the deck in which water may tend to pond.

The Dresbach Bridge deck was designed to include a 2-1/2" integral wearing surface. Typically, at least 2" of the integral wearing surface remains after surface grinding (required for ride optimization). A concrete sealer, methacrylate, was applied after planing the deck and should have penetrated any small cracks in the surface and formed a durable seal. In accordance with MnDOT Standard Practice, the bridge deck and expansion joints are washed/flushed annually. If inspection of the deck reveals any areas where surface cracks have appeared in the riding surface, a methacrylate (or similar) sealer should be applied in that area. Special attention should be paid to key areas such as closure segments, top slab temporary access holes, closure pours, anchorage pourbacks, and pourbacks around expansion joint devices. Shoulder areas of the bridge were feathered for drainage. The information about grinding during construction should be considered when determining the proper preparation procedures for a future wearing surface.

During design, it was assumed that an overlay wearing surface could be added after removal of the integral wearing surface per the Design Criteria. The Dresbach Bridge was designed to accommodate a future wearing surface of up to 20 pounds per square foot to replace the integral wearing surface. The integral wearing surface should be replaced when chloride concentrations at a depth of 1/2" reach a chosen corrosion threshold level determined by the Minnesota Department of Transportation. Replacement should also be considered if the deck becomes damaged.

4.5.1.2 Webs and Bottom Slab

The majority of the webs and bottom slab can be maintained as typical reinforced concrete elements as described in Section 4.5.1.

4.5.1.3 Closure Segments

Maintenance activities for the cast-in-place closure segments are the same as for the cast-in-place superstructure segments.

4.5.1.4 Diaphragms

The diaphragms can be maintained as other structural concrete elements. However, they typically contain a large amount of post-tensioning and function to transfer significant forces between the superstructure and substructure. Therefore, caution should be used when performing maintenance operations other than simple crack sealing in the diaphragm areas, to avoid disturbing the ability of the diaphragms to function as intended.

4.5.1.5 Anchor Blocks

When minor cracks appear over time, they can be repaired in the same way as typical cracks. However, if there is a tendency for cracks to grow in size and length over time, special repair work should be employed. This could consist of an additional reinforced concrete or reinforced epoxy mortar block, cast-in-place along the crack and fixed to the existing concrete with reinforcing dowels and epoxy. An engineer experienced in the design and construction of post-tensioned bridges should be consulted on this type of crack repair.

To patch a spalled concrete area near an anchor, the typical repair procedures discussed in Section 4.2.2.2 can be used. The repair should be performed without removing any more concrete than necessary to prevent concrete failure under anchor pressure. In case of a significant spall, an additional concrete block should be constructed in the same way as previously described. An engineer experienced in the design and construction of post-tensioned bridges should be consulted on this type of spall repair.

4.5.2 Post-Tensioning

The maintenance procedures for the post-tensioning system are outlined below.

4.5.2.1 Ducts and Post-Tensioning Steel

<u>Ducts</u>

All of the post-tensioning tendons in the Dresbach Bridge are contained in ducts, with the space between the post-tensioning steel and duct filled with grout.

The majority of the tendons are located internal to the superstructure concrete, and thus cannot be inspected directly. If a duct for an internal tendon is exposed and damaged due to a spall or repair work, the damaged area should be investigated immediately to be sure that there has been no damage to the post-tensioning steel. If the damage is limited to the duct, it is not necessary to repair the duct before patching the concrete covering the duct. However, use of a two-part epoxy-bonding agent is strongly recommended when repairing the concrete over the tendon to ensure good bond.

The draped tendons are external to the concrete. The ducts for these tendons are black high-density polyethylene ducts connecting between galvanized steel pipes embedded in the expansion joint, pier, or deviation diaphragms.

Any crack observed in a polyethylene pipe must be carefully evaluated to determine the cause. If cracking is found to be related to expansive forces exerted on the pipe, then the pipe should be cut open to determine the cause. At this time, the tendon should be inspected for corrosion and, if necessary, action should be taken as outlined below for post-tensioning steel. Once the cause of the corrosion of the steel or the reaction within the grout is determined and corrected, all the affected grout should be removed. The duct should then be resealed using sections of high-density polyethylene pipe, a flexible mastic sealer and a Tedlar® tape, and the void grouted with epoxy grout. Neoprene patches used with stainless steel bands are also acceptable for repairing small damaged areas of duct.

If cracking is found to be solely due to deterioration of the polyethylene material, the cracks should be filled with a flexible mastic material. If the cracking is extensive, then the pipe should be wrapped with a protective Tedlar® tape or removed and replaced.

If corrosion is found on the steel duct pipes exiting from the deviation blocks or diaphragms, the surface of the pipe should be cleaned and a protective coating applied to the exposed portions of the pipe around its full circumference. Materials for the coating could be bituminous, epoxy, or paint.

If the joint between the polyethylene and steel duct pipes does not appear to be watertight, a bituminous or epoxy coating should be applied to the joint or a new neoprene coupling and stainless steel banding clamps should be installed.

Voids in the grout within the duct can be detected by tapping on the tendon duct with a rubber mallet and listening for a hollow sound; note that the sound will change as you get closer to a fixed point (i.e. a deviation rib or diaphragm), but that does not necessarily indicate that there is a void in that location. Small voids in the tendon grout are common and are generally not a cause for concern. However, if a grout void is found in a tendon duct that is large enough to be a concern to the Minnesota Department of Transportation for corrosion protection of the strand, then a small opening may be cut in the duct over the void to inspect the tendon strands. If the steel strands are in satisfactory condition, the void should be filled with grout or epoxy. Injection should proceed from a hole punched in the duct on the downhill side of the void to a vent location at the uphill end of the void. While drilling the injection ports, care must be taken to avoid damage to the post-tensioning or duct. After injection is completed, the injection and vent ports must be sealed. Alternatively, voids may be vacuum grouted.

Post-Tensioning Steel

Distortion of the polyethylene duct enclosing external draped tendons should be further investigated to determine if damage has occurred to the tendon. Nonlinearities of the external tendon polyethylene ducts as described in Section 3.5.2.1 are a result of the construction grouting operations and are not cause for further investigation. If evidence of post-tensioning steel corrosion is reported for either external or internal tendons, an investigation should be initiated to determine the cause. An evaluation should also be made to determine if the corrosion has affected the structural capacity of the tendon.

Corrective action appropriate to the given situation, such as sealing concrete cracks, should be performed where instances of tendon corrosion are confirmed, to prevent further damage. If there is evidence that there has been a significant loss of the post-tensioning force in the prestressing steel in any tendon, it is essential that the situation be evaluated by an engineer who is knowledgeable in the design of post-tensioned concrete bridges. Installing and stressing additional tendons, such as the future post-tensioning tendons, may be required if the loss of force is substantial.

4.5.2.2 Anchorages

As noted in Section 3.5.2.3, many of the post-tensioning anchorages are embedded in the superstructure concrete, covered with a secondary pour after stressing and grouted. Anchorages for the draped external tendons and bottom slab continuity tendons located inside the girder are covered with a grout-filled non-metallic (plastic) grout cap and an epoxy-grout block that is coated with an elastomeric membrane.

If, during the inspection of a non-metallic (plastic) grout cap or secondary pourback covering a post-tensioning anchorage, evidence of corrosion of embedded steel is observed, the cause of the corrosion should be determined and corrected. Non-metallic (plastic) grout caps and secondary concrete pourbacks protecting the tendon anchorages should remain free of cracking to minimize the potential for infiltration of water and chlorides. This is particularly important for anchorages located at expansion joints due to the greater likelihood of water entry into the box girder at these locations. Cracks in the grout caps or secondary concrete pourbacks should be sealed with an elastomeric coating. During construction, anchorage protection devices were sealed with a liquid cold-applied elastomeric waterproofing membrane system.

Any grout caps or secondary concrete pourbacks covering post-tensioning anchorages that are deteriorated, such that some or all of the anchorage protection has de-bonded from the anchorage or spalled-off, should either be repaired or replaced. One recommended replacement for a defective grout cap would be to remove the grout cap, remove any loose or damaged grout, clean the grout with a wire brush to remove rust, install a new grout cap, inject the cap with tendon grout, pour back a secondary concrete pourback, and then cover with 2 layers of elastomeric membrane. Such a remedial repair may require doweling mild reinforcement near the anchor and using an epoxy bonder to seal and adhere the pourback to the anchorage. To replace a secondary concrete pourback, remove the deteriorated concrete, being very careful not to damage the steel anchor head or plastic end cap. Clean the plastic end cap with a wire brush, then apply an epoxy binder and immediately cast a new protective secondary concrete pourback. After curing, two layers of elastomeric membrane as shown in Figure 2.44 and 2.45 should be applied to the concrete caps.

If there is substantial deterioration of the elastomeric membrane on the secondary concrete pourback over a post-tensioning anchorage such that the underlying concrete is easily visible, the surface of the block should be cleaned and re-covered with two layers of elastomeric membrane.

4.5.2.3 Physical Damage to a Tendon

It is unlikely that a post-tensioning tendon will suffer physical damage. However, all personnel responsible for routine maintenance or inspection of this bridge must be aware of the critical nature of physical damage to a tendon. Because the superstructure is supported by the tendons, damage to a tendon could cause a substantial reduction in load carrying capacity. This type of damage would include corrosion of a post-tensioning tendon or loss of post-tensioning force.

Any significant damage to a tendon must be immediately evaluated by an engineer who is knowledgeable in post-tensioned concrete bridge design. Installing and stressing additional tendons, such as the future post-tensioning tendons, may be required if there is a substantial loss of post-tensioning force.

4.5.2.4 Future Post-Tensioning Details

As discussed in Section 2.3.3.3, the Dresbach Bridge features provisions for additional longitudinal draped post-tensioning tendons to be installed in the future, if needed. These future tendons are external tendons located inside the box girder. They are 27 x 0.6” diameter strand tendons that anchor in the diaphragms at pier and expansion joint segments. Spans 1 through 4 have one future tendon per web.

These tendons may be installed and stressed if needed due to increased live loads, loss of post-tensioning force in other tendons, or other conditions. An analysis should be made by an engineer who is knowledgeable in post-tensioned concrete bridge design to verify stresses prior to installation of the future tendons.

Refer to the details given in the contract plans for installation of the future tendons. Anchorages and galvanized steel deviation pipes, with appropriate reinforcing, were cast into the superstructure during the original construction. High-density polyethylene ducts must be installed between the ends of the galvanized steel pipes exiting from the diaphragms, and the pipes extending from the bottom soffit deviation diaphragms in the spans. The union between galvanized steel pipe and the polyethylene duct shall be joined with neoprene couplings and stainless steel banding clamps. This will form continuous tendon ducts from diaphragm to diaphragm in which the prestressing steel can be installed. After installation of anchor heads and wedges, the tendons may be stressed and grouted. The tendon anchorages should have protective grout-filled non-metallic (plastic) cap grout caps (or similar) as shown in Figure 2.44 and 2.45. Future tendon anchors in place in the pier and expansion joint segments were manufactured by SDI. Should these anchors ever be used, the remaining anchorage system must be compatible with the SDI future tendon anchors already installed.

4.6 Expansion Joint Device Maintenance

4.6.1 Cleaning

If a joint becomes filled with incompressible material (dust, sand, debris, etc.) the adjacent deck may crack or spall during contraction of the joint. To insure free movement, expansion joints should be cleaned systematically of all foreign materials. This can be accomplished using pressurized water or air. Additionally, it is recommended that all debris be removed from the surrounding bridge deck to prevent it from accumulating in the joint again.

In addition to cleaning the seals at the deck surface, the stainless steel surface of support bars beneath the modular joints should be periodically cleaned to prevent accumulation of debris, which may interfere with joint movement.

Maintaining a clean joint will greatly improve the service life of the joint and other elements of the structure. It will also reduce the long-term maintenance costs of the structure.

4.6.2 Measuring Movement Capacity

As part of the periodic inspections, the Structures Division should calculate the maximum and minimum anticipated expansion joint openings. This is done using the forms included in Appendix B.

The maximum and minimum anticipated joint openings can then be compared with the maximum and minimum openings allowed for the particular joint, as shown on the form next to the expansion joint sketch. Normally, the maximum opening will control, since creep and shrinkage movements permanently increase the joint opening over time. If minimum opening controls, further analysis should be done to be sure that creep and shrinkage movements of the structure are as expected.

If analysis indicates that the maximum allowable joint opening will be exceeded, the joint should be inspected on a very cold winter day and the analysis repeated. Table B.2 in Appendix B can be used to determine if creep and shrinkage movements will cause the joint to overextend, and if so, in approximately which year. The wintertime joint monitoring should be performed for suspect joints each year that creep and shrinkage movements are still occurring. If it is confirmed that a joint will significantly exceed its capacity, the joint should be scheduled for replacement. Joint replacement is discussed in Section 4.3.4.

4.6.3 Replacement of Seals

Neoprene seals that leak should be replaced. Information concerning the replacement of neoprene seals, recommended procedure, and any special tools required may be obtained from the expansion joint shop drawings and from the joint manufacturer, DS Brown.

Note that evidence of water infiltration below the expansion joint does not always indicate neoprene seal failure. It is possible for the bond between the transverse edge beams and the secondary concrete pour to weaken over time causing water infiltration. This can be exacerbated by material being lodged between the rails. Delamination between the joint beams and adjacent concrete can be sealed with penetrating sealers or epoxy injection.

4.6.4 Replacement of Joint Hardware

If individual components of the expansion joints require replacement, it should be coordinated though the expansion joint manufacturer, DS Brown.

Sliding plates at the bridge barrier can be replaced by removing counter-sunk bolts that thread into concrete inserts. These bolts attach the sliding plates to the bridge barrier. New plates can then be attached to the existing concrete barrier with the existing inserts, or new drilled inserts as needed.

Modular expansion joints can be entirely replaced by first removing the volume of concrete that fills the blockouts in the end diaphragm and abutment backwall that contain the expansion joint device. Removal of the concrete and modular expansion joints may be phased to maintain traffic. These blockouts measure 1’-4” deep by 3’-0” wide at the West Abutment and 1’-4” deep by 4’-6” wide at Pier 4. These blockouts were cast for the full width of the bridges, minus the barrier widths. It may be necessary to remove the sliding plates in the bridge barrier prior to removing the blockout concrete. Care should be exercised when removing this concrete to avoid damaging the mild reinforcing steel. Also, there is a transverse post-tensioning tendon embedded in the blockout concrete on the unit 1 superstructure side of the joint that would need to be carefully cut out and removed as the concrete is removed.

After removing the blockout concrete, any of the mild reinforcing steel extending from the structure into the blockout that is damaged during concrete removal should be replaced with bars doweled and epoxied into the structure. The replacement expansion joint unit can be placed in the blockout along with replacements for the transverse mild reinforcing steel and the transverse post-tensioning tendon contained within the blockout as shown in the contract plans.

After adjusting the opening and elevation of the expansion joint, new concrete can be placed in the blockout and cured.

Replacement expansion joints will not need to have the same movement capacity as the original joints due to the permanent creep and shrinkage of the concrete that occur primarily in the first few years after the structure is completed. Appendix B describes the calculation of future anticipated movements at each joint location. The information contained in the expansion joint inspection sheet of Appendix B can also be used to determine the appropriate width for setting the replacement joint for a given structure temperature.

INITIAL INSPECTIONS RESULTS
(PROVIDED BY MNDOT)

APPENDIX A

Initial inspection forms to be filled out at a later date by the Minnesota Department of Transportation.

RECORDING FORMS AND CALCULATIONS

APPENDIX B – RECORDING FORMS AND CALCULATIONS

B.1 Contents

This section of the appendix contains a number of items useful for the inspection of unit 1 of the Interstate 90 Bridge at the Mississippi River, including inspection sheets and information on how to calculate expected bearing and expansion joint movements.

B.2 Survey and Inspection Sheets

Several types of survey sheets are available for use in inspections. Survey sheets are included to aid inspection for the following bridge components: external surfaces, internal surfaces, railings, diaphragms, bottom slab anchor blocks, bearings, piers, expansion joints, and abutments. An index of the survey sheets available is included on page B-9. The appropriate number of copies of each form should be made to use in a given field inspection.

The surface survey forms depict the interior and exterior surfaces of the box girder superstructure. Each form shows either the interior or exterior portion of a girder. Items found during inspections, such as cracks, spalls, leaking joints, or defective post-tensioning tendons, should be diagrammed on the appropriate form along with accompanying notes. These forms can be applied to either the westbound or eastbound bridge. The bridge to be inspected can be noted at the top of the page. The location of possible areas of concern can be referenced from abutments or pier centerlines.

The diaphragm survey sheets depict the concrete surfaces of the superstructure areas that contain diaphragms (expansion joint, pier, and deviation). The diaphragm surfaces and the remaining interior surfaces of these areas are shown on the diaphragm forms. The exterior surfaces of these areas are shown on the exterior surface survey sheets for the typical sections. Bottom slab anchor block survey sheets are also provided for anchor block inspection at the box girder areas that anchor bottom slab tendons.

Survey sheets for substructure inspection include abutment and pier survey sheets. The pier forms include the pier column and pedestal (as needed).

Bearing and expansion joint inspection sheets are used to record information about the condition of these components and their measured dimensions. The forms are also used to calculate the anticipated movement capacity required at that location for direct comparison with the actual capacity. The calculation of anticipated movements is discussed in detail in Section B.3.

B.3 Anticipated Movement Calculations

As discussed in Chapter 4, part of the bearing and expansion joint inspection is to calculate the maximum anticipated longitudinal movement of the bearing or joint, and compare it to the measured movement capacity at that location. This can be accomplished using the bearing and expansion joint inspection sheets with the field data completed, in conjunction with Tables B.1 and B.2 and the procedures described below.

B.3.1 Sliding Bearings

The table labeled “Displacement Allowance” at the bottom of the guided and non-guided bearing inspection sheets is used to calculate the anticipated bearing movements.

The columns of the table labeled L_{DN} and L_{UP} should be filled in with the dimensions measured in the field. The temperature of the bridge concrete can be determined using a concrete surface thermometer or laser thermometer. Temperatures should be recorded at the approximate mid-height of the webs in the shade. Note that bridge temperatures can vary throughout the course of the day, so temperatures should be recorded at the same time the bearing plate dimensions are measured. It is recommended that the average of 3, or more, temperatures along the mid-height of the shaded web be used for the bridge temperature.

Next, look up the thermal movement and remaining creep and shrinkage values in Table B.1 and enter these values into Columns 1 and 4, respectively. The thermal movement values are the amount of movement due to a one-degree Fahrenheit temperature drop. The creep and shrinkage values are the expected remaining movement in each year. Note that creep and shrinkage values are listed on November 13th for the eastbound structure and September 1st for the westbound structure. Interpolation of the tables can be used for inspections at other times of the year if it would make a significant difference. Remaining creep and shrinkage for the year 2041 or later is zero.

The amount that the bearing is expected to move upstation and downstation from the current setting due to temperature is calculated next. This is done using the measured concrete temperature, the thermal movement from Column 1, and the formulas for Columns 2 and 3 as shown on the inspection sheet. Note the sign convention in Column 1 is important when computing Columns 2 and 3. The results are in inches of movement for the design temperature extremes of -30°F and 120°F. Enter these values in Columns 2 and 3 in the table.

The maximum anticipated bearing movements in the downstation and upstation directions are then calculated using the formulas for Columns 5 and 6 given below the table. Write these values in Columns 5 and 6 of the table. The anticipated upstation and downstation movements can then be compared with the available movement capacities of the bearings (L_{DN} and L_{UP}) measured at that particular location. If L_{DN} and L_{UP} are greater than or equal to Columns 5 and 6, respectively, then the bearing movement capacity is adequate.

An example of these calculations is included for reference.

B.3.2 Expansion Joints

The table labeled "Displacement Allowance" at the bottom of the expansion joint inspection sheet is used to calculate the anticipated joint movements.

The first column of the table should be filled in with the dimension "A" as measured in the field. The second column should be filled in with the dimension "X" as measured in the field. The temperature of the bridge concrete, which can be determined using a concrete surface thermometer or laser thermometer, should be recorded on the inspection sheet. Temperatures should be recorded at the approximate mid-height of the webs in the shade. Note that bridge temperatures can vary throughout the course of the day, so temperatures should be recorded at the same time the "A" and "X" dimensions are measured. It is recommended that the average of 3, or more, temperatures along the mid-height of the shaded web be used for the bridge temperature.

The first item calculated is the amount the joint is expected to open from its current setting if the temperature drops to the design minimum of -30°F. This is calculated using the measured concrete temperature and the Column 3 formula located just below the table. The result is in inches of joint opening. Write this value in Column 3 of the table. Similarly, the anticipated joint closing for the maximum design temperature of 120°F is calculated using the Column 4 formula below the table and entered in Column 4.

The remaining amount of creep and shrinkage joint opening movement is found in Table B.2 and entered in Column 5. The values are given for each year. Note that creep and shrinkage values are listed on November 13th for the eastbound structure and September 1st for the westbound structure. Interpolation of the tables can be used for inspections at other times of the year if it would make a significant difference. Remaining creep and shrinkage for the year 2041 or later is zero.

The maximum anticipated joint opening is then calculated as the sum of the measured dimension "X" (Column 1), the joint opening for -30°F (Column 3), and the remaining creep and shrinkage movement (Column 5). Write this value in Column 6 of the table (Max.).

The minimum anticipated joint opening is calculated as the measured dimension “X” (Column 2) plus the joint opening for 120°F (Column 4). Write this value in Column 7 of the table (Min.).

The maximum and minimum anticipated joint openings can then be compared with the maximum and minimum opening allowed for that particular joint, as shown on the inspection sheet next to the expansion joint sketch. Normally, the maximum opening will control because creep and shrinkage movements affect this opening and are permanent. If the anticipated values both fall between the allowable values, the joint has adequate movement capacity.

An example of these calculations follows for reference.

Table B.1 – Bearing Displacements

Location		Westbound West Abut.	Westbound Pier 4
Thermal Movement (IN/°F Fall)		0.056	-0.061
Remaining Creep and Shrinkage Movements on September 1 (Inches)	2015	N/A	N/A
	2016	3.66	-3.86
	2017	2.80	-2.99
	2018	2.28	-2.45
	2019	1.91	-2.06
	2020	1.63	-1.75
	2021	1.39	-1.50
	2022	1.21	-1.30
	2023	1.06	-1.15
	2024	0.94	-1.02
	2025	0.83	-0.90
	2026	0.73	-0.79
	2027	0.64	-0.69
	2028	0.56	-0.60
	2029	0.48	-0.53
	2030	0.43	-0.47
	2031	0.38	-0.41
	2032	0.33	-0.36
	2033	0.29	-0.31
	2034	0.24	-0.27
	2035	0.20	-0.22
	2036	0.17	-0.18
	2037	0.13	-0.14
	2038	0.09	-0.10
	2039	0.06	-0.07
	2040	0.03	-0.03
	2041	0.00	0.00

Location		Eastbound West Abut.	Eastbound Pier 4
Thermal Movement (IN/°F Fall)		0.058	-0.059
Remaining Creep and Shrinkage Movements on November 13 (Inches)	2015	4.01	-3.69
	2016	2.99	-2.85
	2017	2.42	-2.34
	2018	2.02	-1.96
	2019	1.71	-1.67
	2020	1.46	-1.43
	2021	1.26	-1.24
	2022	1.11	-1.09
	2023	0.98	-0.97
	2024	0.86	-0.86
	2025	0.76	-0.75
	2026	0.67	-0.66
	2027	0.58	-0.57
	2028	0.51	-0.50
	2029	0.45	-0.45
	2030	0.40	-0.40
	2031	0.35	-0.35
	2032	0.30	-0.30
	2033	0.26	-0.26
	2034	0.22	-0.22
	2035	0.18	-0.18
	2036	0.14	-0.14
	2037	0.11	-0.11
	2038	0.07	-0.07
	2039	0.04	-0.04
	2040	0.01	-0.01
	2041	0.00	0.00

NOTE: Positive value indicates superstructure movement towards upstation direction (east).

Table B.2 – Expansion Joint Displacements

		Westbound	
Location		Expansion Joint at West Abut.	Expansion Joint at Pier 4
Thermal Movement (IN/°F Fall)		0.056	0.109
Remaining Creep and Shrinkage Movements on September 1 (Inches)	2015	N/A	N/A
	2016	3.67	5.92
	2017	2.81	4.76
	2018	2.29	4.00
	2019	1.92	3.44
	2020	1.63	2.99
	2021	1.40	2.61
	2022	1.21	2.31
	2023	1.07	2.06
	2024	0.94	1.84
	2025	0.83	1.64
	2026	0.73	1.46
	2027	0.64	1.30
	2028	0.56	1.14
	2029	0.49	1.01
	2030	0.43	0.90
	2031	0.38	0.80
	2032	0.33	0.70
	2033	0.29	0.60
	2034	0.24	0.51
	2035	0.20	0.43
	2036	0.17	0.35
	2037	0.13	0.27
	2038	0.09	0.20
	2039	0.06	0.13
	2040	0.03	0.06
	2041	0.00	0.00

		Eastbound	
Location		Expansion Joint at West Abut.	Expansion Joint at Pier 4
Thermal Movement (IN/°F Fall)		0.058	0.106
Remaining Creep and Shrinkage Movements on November 13 (Inches)	2015	4.02	5.87
	2016	3.00	4.70
	2017	2.43	3.95
	2018	2.02	3.39
	2019	1.71	2.94
	2020	1.47	2.58
	2021	1.26	2.27
	2022	1.11	2.03
	2023	0.98	1.82
	2024	0.87	1.62
	2025	0.76	1.45
	2026	0.67	1.29
	2027	0.58	1.14
	2028	0.51	1.01
	2029	0.45	0.90
	2030	0.40	0.80
	2031	0.35	0.70
	2032	0.30	0.61
	2033	0.26	0.52
	2034	0.22	0.44
	2035	0.18	0.36
	2036	0.14	0.28
	2037	0.11	0.21
	2038	0.07	0.14
	2039	0.04	0.08
	2040	0.01	0.01
	2041	0.00	0.00

NOTE: Positive value indicates increasing Dim 'A' and Dim 'X'.

NON-GUIDED DISC BEARING INSPECTION SHEET

BRIDGE: WB **EB** W. ABUTMENT / **PIER 4 (DN STA.)**

EXAMPLE ONLY

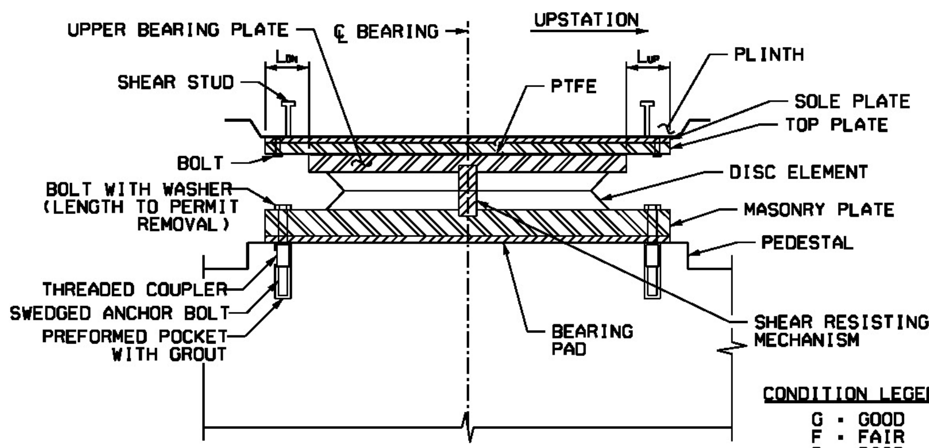
BEARING MANUFACTURER: D.S. BROWN

AMBIENT TEMPERATURE @ INSPECTION: 25 °F

CONCRETE TEMPERATURE @ INSPECTION - MID WEB (IN SHADE): 20 °F

INSPECTED BY: J. Dvorak

DATE INSPECTED: February 15, 2016



SIDE VIEW

CONDITION LEGEND

G - GOOD
F - FAIR
P - POOR
S - SEVERE

ELEMENT	CONDITION	COMMENTS
BEARING PLATE WITH PTFE	**G** F P S	
SOLE PLATE	**G** F P S	
POLYETHER URETHANE DISC	**G** F P S	
TOP & MASONRY PLATES	**G** F P S	Stainless steel surfaces should be cleaned.
BEARING PAD	**G** F P S	

DISPLACEMENT ALLOWANCE (INCHES)								MOVEMENT CAPACITY ADEQUATE
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MIN. DISTANCE FROM BRG TO END OF STAINLESS		MOVEMENT REQUIRED		L_{DN} > ⑤
①	②	③	④	L_{DN}	L_{UP}	⑤ DN STA.	⑥ UP STA.	L_{UP} > ⑥
-0.059	+5.90"	+2.95"	-3.48"	9.75"	13.25"	5.90"	6.43"	**OK** / NG*

① & ④ - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.1

IF ① > 0: ② = (CONC. TEMP. + 30°F) X ① ⑤ = ② + ④ (CONTRACTION)
③ = (120°F - CONC. TEMP.) X ① ⑥ = ③ (EXPANSION)

IF ① < 0: ② = (CONC. TEMP. - 120°F) X ① ⑤ = ② (EXPANSION)
③ = (-30°F - CONC. TEMP.) X ① ⑥ = ③ - ④ (CONTRACTION)

COMMENTS: Used linear interpolation to compute Item 4.

* IF MOVEMENT CAPACITY IS NO GOOD (NG) REFER TO SECTION 4.2.2

BRG-1

EXPANSION JOINT INSPECTION SHEET

BRIDGE: WB EB WEST ABUTMENT / PIER 4

EXAMPLE ONLY

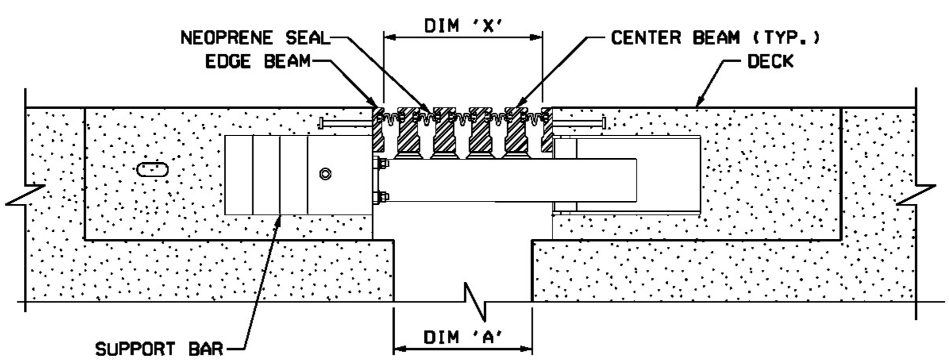
EXPANSION JOINT MANUFACTURER: D.S. BROWN

AMBIENT TEMPERATURE @ INSPECTION: 25 °F

CONCRETE TEMPERATURE @ INSPECTION - MID WEB (IN SHADE): 20 °F

INSPECTED BY: J. Dvorak

DATE INSPECTED: February 15, 2016



SECTION

WEST ABUTMENT

MAX. ALLOWABLE 'X' = 28 1/4"
MIN. ALLOWABLE 'X' = 12 1/2"

PIER 4

MAX. ALLOWABLE 'X' = 50 7/8"
MIN. ALLOWABLE 'X' = 22 1/2"

CONDITION LEGEND

G - GOOD
F - FAIR
P - POOR
S - SEVERE

ELEMENT	CONDITION	COMMENTS
CENTER BEAMS	G F P S	
DECK SURFACE @ JOINT	G F P S	
UNIFORMITY OF SEAL OPENINGS	G F P S	
SUPPORT BARS	G F P S	
NEOPRENE SEALS	G F P S	Debris should be cleaned from seals.
WELDS BETWEEN SUPPORT BARS & CENTER BEAMS	G F P S	
SLIDING PLATES @ BARRIERS	G F P S	
BOND BETWEEN EDGE BEAMS AND CONCRETE	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)								MOVEMENT CAPACITY ADEQUATE IF:
MEASURED DIM 'A'	MEASURED DIM 'X' ①	THERMAL MOVEMENT ②	TEMP. CORRECTION FALL ③	TEMP. CORRECTION RISE ④	REMAINING CR. & SH. ⑤	ANTICIPATED ACTUAL 'X' MAX. ⑥	ANTICIPATED ACTUAL 'X' MIN. ⑦	MIN. > MIN. ALLOWABLE MAX. < MAX. ALLOWABLE
15.50"	17.63"	+0.056	+2.80"	-5.60"	+3.77	24.20"	12.03"	OK / NG*

② & ⑤ - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.2
③ = (CONC. TEMP. + 30°F) X ② ⑥ = ① + ③ + ⑤ (JOINT EXPANSION)
④ = (CONC. TEMP. - 120°F) X ② ⑦ = ① + ④ (JOINT CONTRACTION)

COMMENTS: Used linear interpolation to compute Item 5. Joint should be re-inspected in the warmer months, as additional creep & shrinkage movement may occur. Maximum bridge temperature of 111°F is needed for Min. 'X' = 12.5" without any creep & shrinkage.

* IF MOVEMENT CAPACITY IS NO GOOD (NG) REFER TO SECTION 4.3.2

SURVEY SHEETS AND INSPECTION FORMS

INDEX OF SHEETS

SHEET DESIGNATION

TYPICAL BOX GIRDER SURVEY SHEETS:

DIAPHRAGM SURVEY SHEETS:

SUBSTRUCTURE SURVEY SHEETS:

BEARING AND EXPANSION JOINT INSPECTION SHEETS:

I-90 DRESBACH BRIDGE

SPANS 1 - 4 INTERIOR SURFACES SURVEY SHEET

SPAN NO.: 1 / 2 / 3 / 4

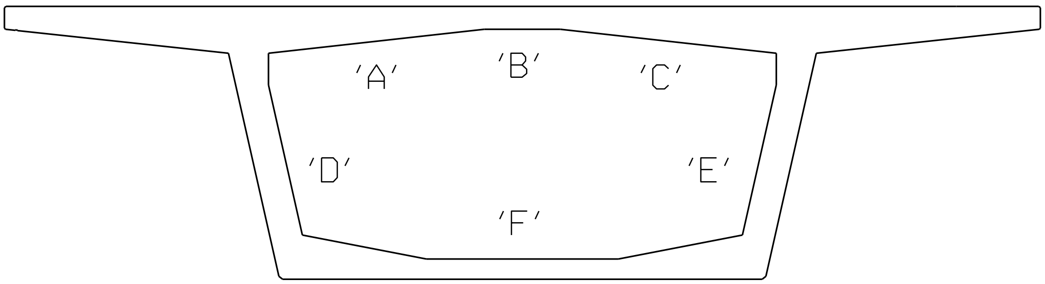
INSPECTED BY: ____________________

BRIDGE: WB / EB

DATE INSPECTED: ____________________

ORGANIZATION: ____________________

DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ____________________



LOOKING UPSTATION

SURFACE INDEX:

'D'	
'F'	
'E'	

PLAN - WEBS AND BOTTOM SLAB

SURFACE INDEX:

'A'	
'B'	
'C'	

REFLECTED PLAN - TOP SLAB

SPANS 1 - 4 TOP DECK SURFACES SURVEY SHEET

SPAN NO.: 1 / 2 / 3 / 4

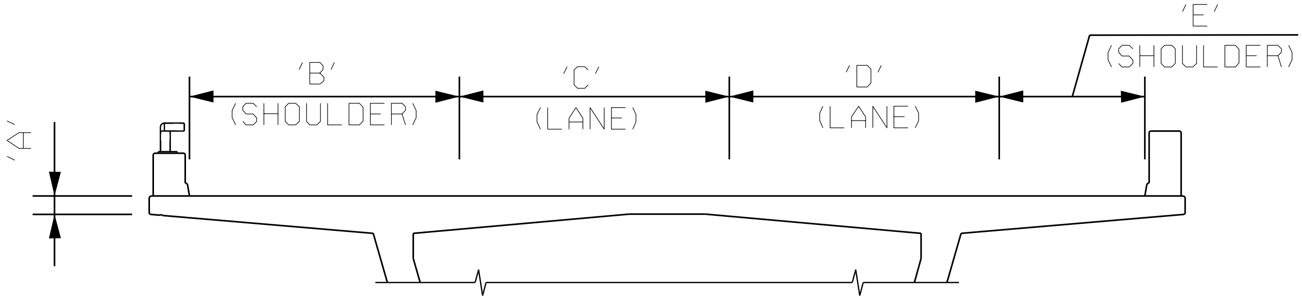
INSPECTED BY:____________________

BRIDGE: WB / EB

DATE INSPECTED:____________________

ORGANIZATION:____________________

DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.):____________________



LOOKING UPSTATION

NOTE: LOCATION OF BARRIERS SHOWN FOR WB BRIDGE.
EB BRIDGE OPPOSITE.

SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
TUBE RAIL & BARRIER	
BARRIER	

I-90 DRESBACH BRIDGE

SPANS 1 - 4 EXTERIOR SURFACES SURVEY SHEET

SPAN NO.: 1 / 2 / 3 / 4

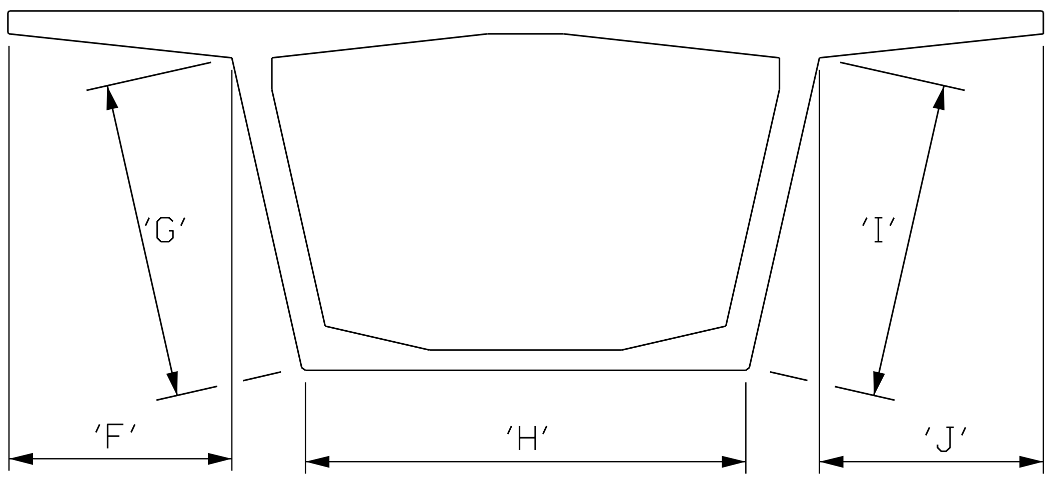
INSPECTED BY: ____________________

BRIDGE: WB / EB

DATE INSPECTED: ____________________

ORGANIZATION: ____________________

DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ____________________



LOOKING UPSTATION

SURFACE INDEX:

'F'	
'G'	
'H'	
'I'	
'J'	

I-90 DRESBACH BRIDGE

BOTTOM SLAB ANCHOR BLOCK SURVEY SHEET

SPAN NO.: ______

BRIDGE: WB / EB

INSPECTED BY: ______________________

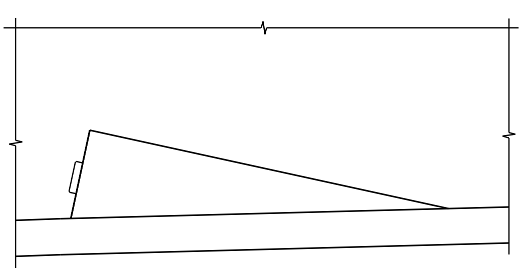
DATE INSPECTED: ______________________

ORGANIZATION: ______________________

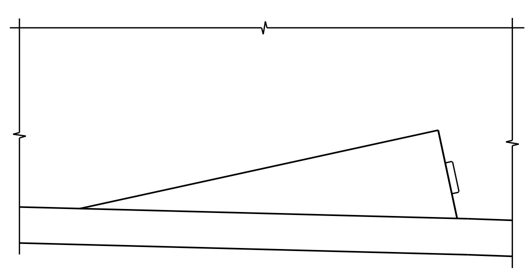
DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ______________________

ELEVATION

VIEW A-A



VIEW B-B



VIEW C-C

I-90 DRESBACH BRIDGE

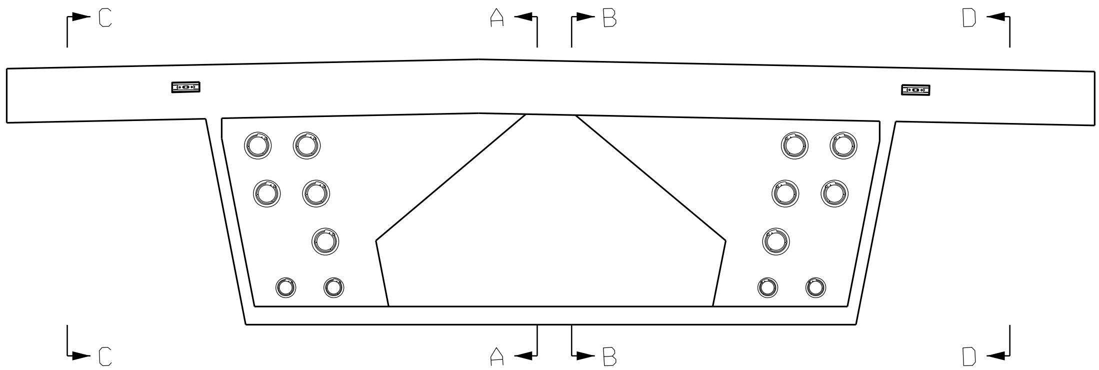
PIER 4 DIAPHRAGM & BOX SURVEY SHEET

INSPECTED BY: ______________________

BRIDGE: WB / EB

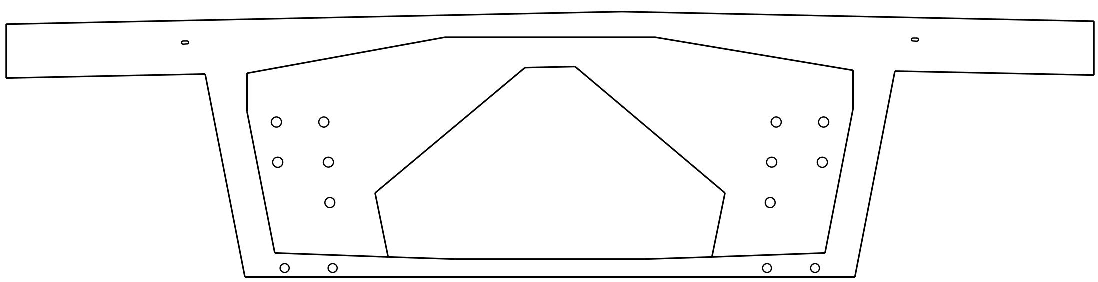
DATE INSPECTED: ______________________

ORGANIZATION: ______________________



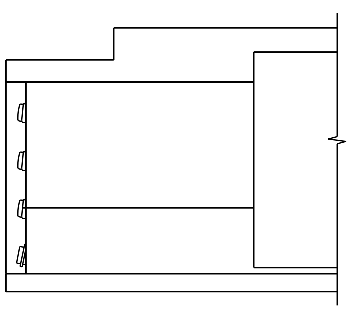
ELEVATION

(LOOKING TOWARD THE BRIDGE)
(WB BRIDGE SHOWN, EB BRIDGE SIMILAR)

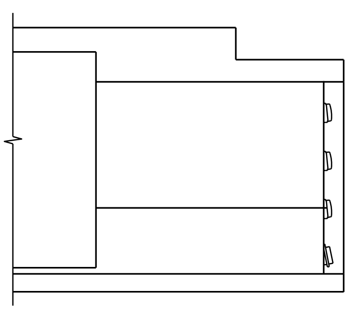


ELEVATION

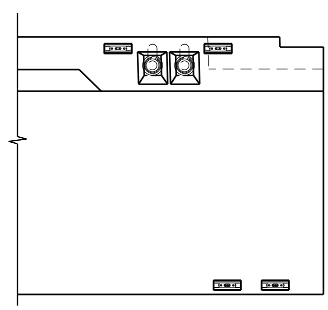
(LOOKING TOWARD THE ABUTMENT)
(WB BRIDGE SHOWN, EB BRIDGE SIMILAR)



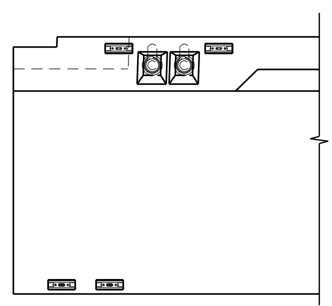
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

BRIDGE NO.: 85801 (WB) / 85802 (EB)

I-90 DRESBACH BRIDGE

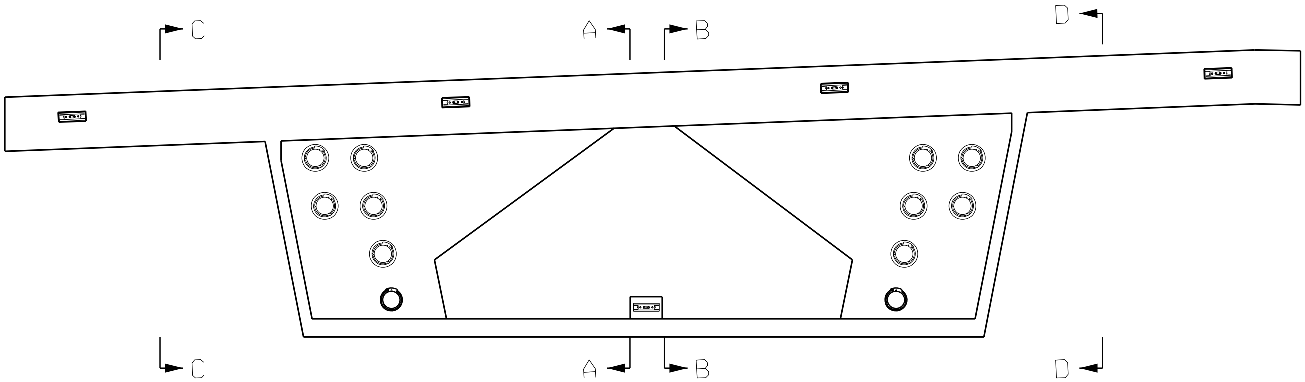
WEST ABUTMENT DIAPHRAGM & BOX SURVEY SHEET

INSPECTED BY: ______________________

BRIDGE: WB / EB

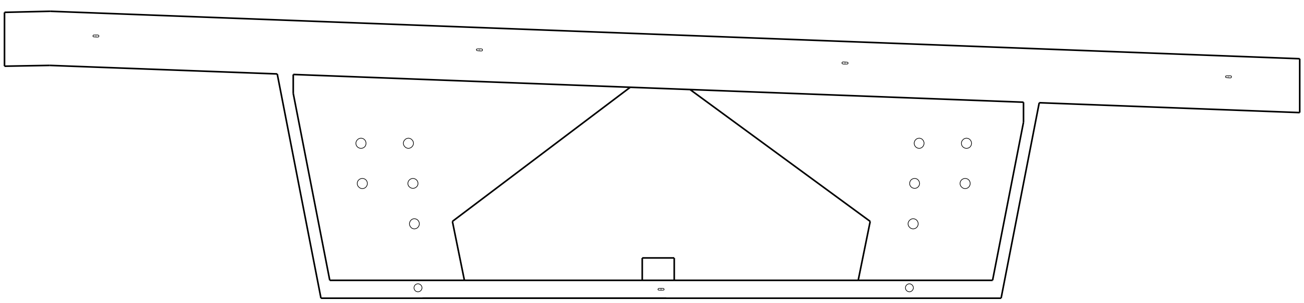
DATE INSPECTED: ______________________

ORGANIZATION: ______________________



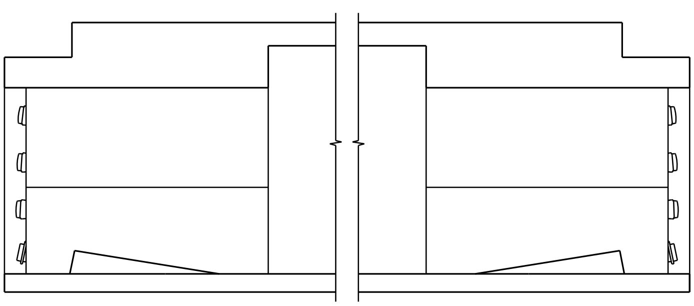
ELEVATION

(LOOKING TOWARD THE BRIDGE)
(WB BRIDGE SHOWN, EB BRIDGE SIMILAR)



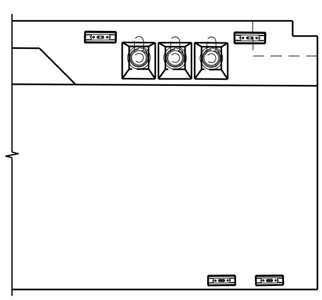
ELEVATION

(LOOKING TOWARD THE ABUTMENT)
(WB BRIDGE SHOWN, EB BRIDGE SIMILAR)

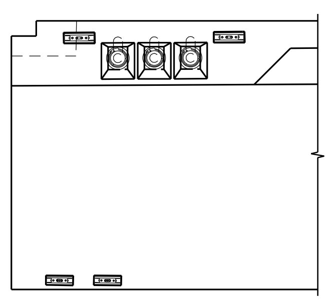


SECTION A-A

SECTION B-B



SECTION C-C



SECTION D-D

I-90 DRESBACH BRIDGE

PIERS 2 & 3 DIAPHRAGM & BOX INTERIOR SURVEY SHEET

PIER NO.: 2 / 3

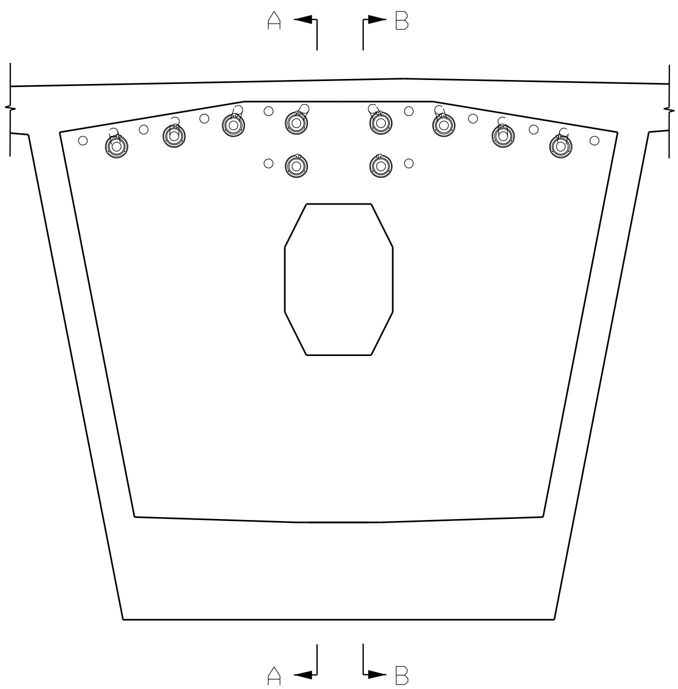
BRIDGE: WB / EB

DIAPHRAGM: UP STA / DWN STA

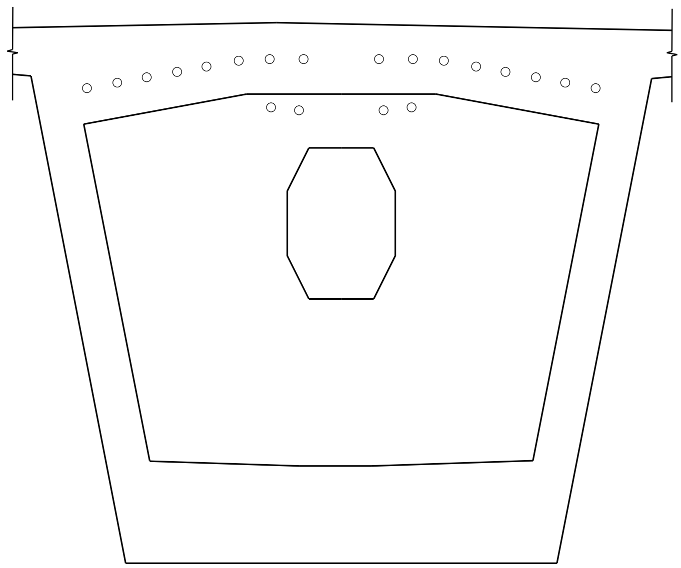
INSPECTED BY: ____________________

DATE INSPECTED: ____________________

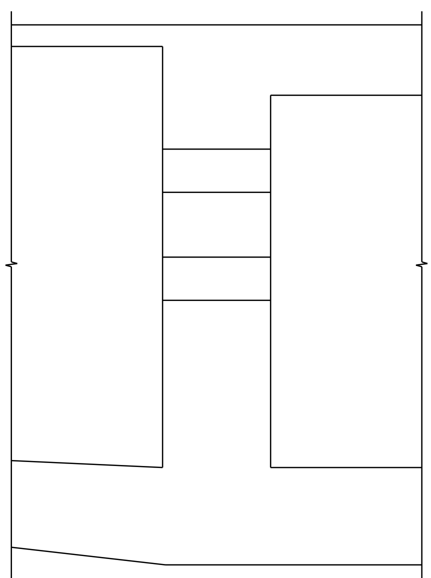
ORGANIZATION: ____________________



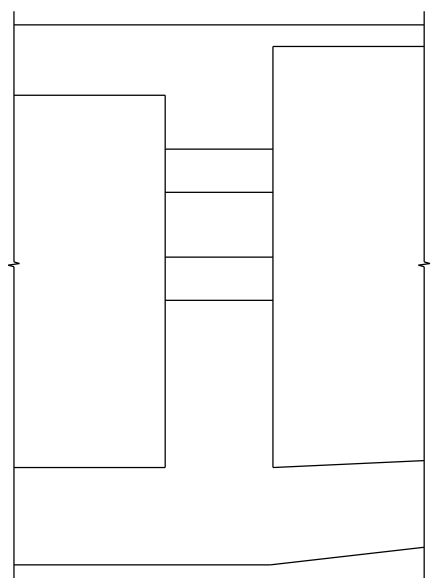
ELEVATION
(EXTERIOR FACE)



ELEVATION
(INTERIOR FACE)



SECTION A-A



SECTION B-B

BRIDGE NO.: 85801 (WB) / 85802 (EB)

I-90 DRESBACH BRIDGE

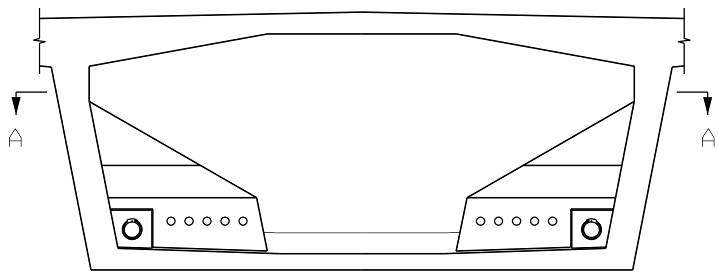
TYPE I DEVIATION DIAPHRAGM SURVEY SHEET

SPAN NO.: 1 / 2 / 3 / 4 INSPECTED BY: ____________________

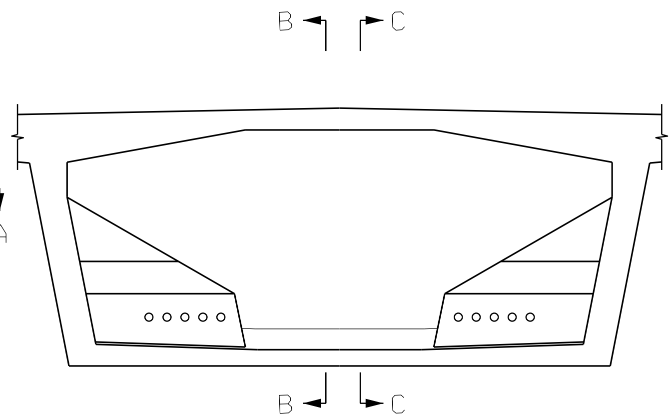
BRIDGE: WB / EB DATE INSPECTED: ____________________

ORGANIZATION: ____________________

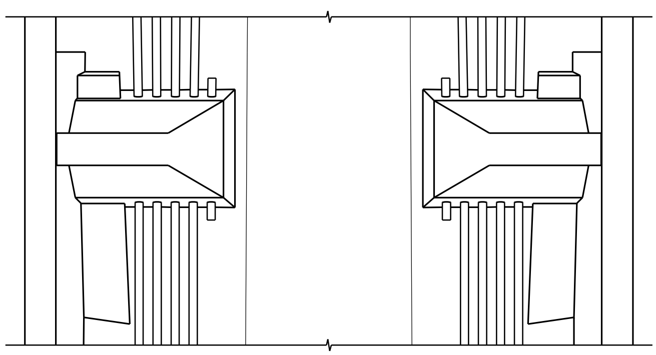
DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ____________________



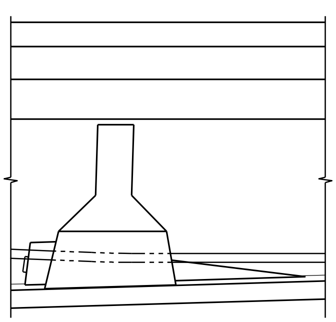
ELEVATION
(LOOKING UP STA / DWN STA)



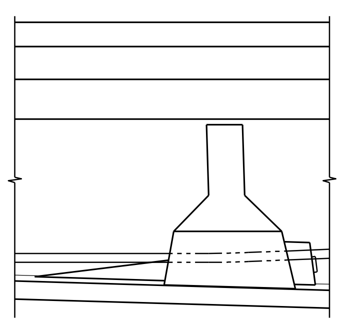
ELEVATION
(LOOKING DWN STA / UP STA)



SECTION A-A



SECTION B-B



SECTION C-C

I-90 DRESBACH BRIDGE

TYPE II DEVIATION DIAPHRAGM SURVEY SHEET

SPAN NO.: 1 / 4

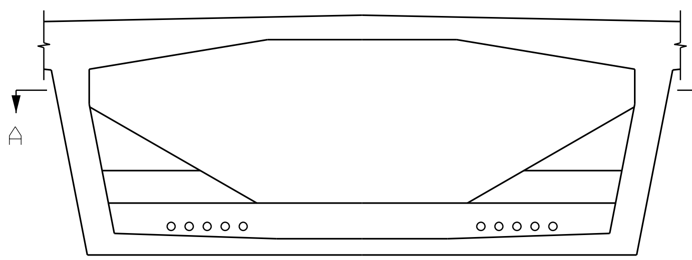
INSPECTED BY: ______________________

BRIDGE: WB / EB

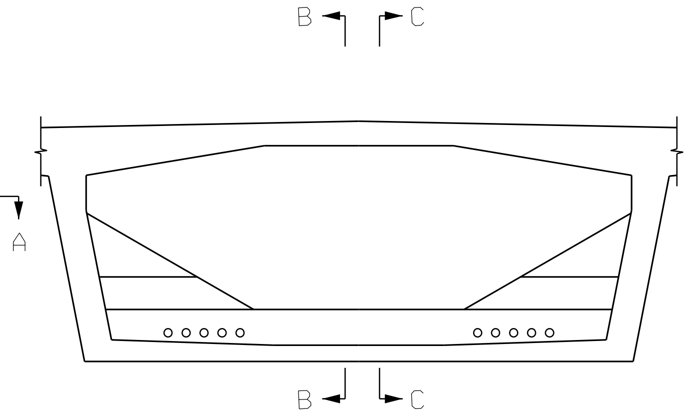
DATE INSPECTED: ______________________

ORGANIZATION: ______________________

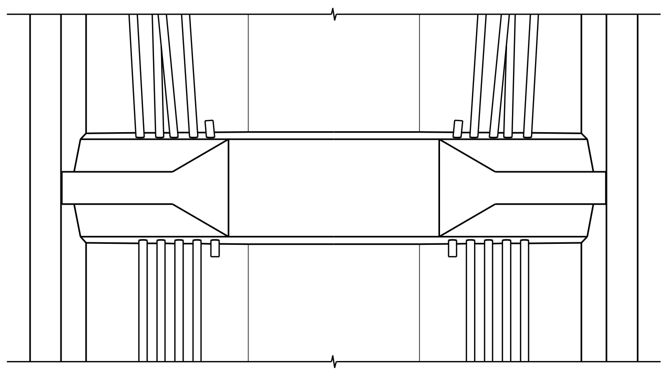
DIST. FROM: EXP. JT. / PIER: ______________________



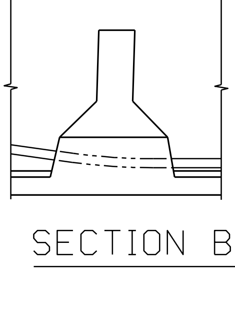
ELEVATION
(LOOKING UP STA / DWN STA)



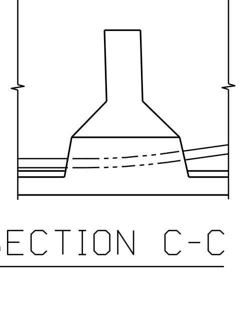
ELEVATION
(LOOKING DWN STA / UP STA)



SECTION A-A



SECTION B-B



SECTION C-C

I-90 DRESBACH BRIDGE

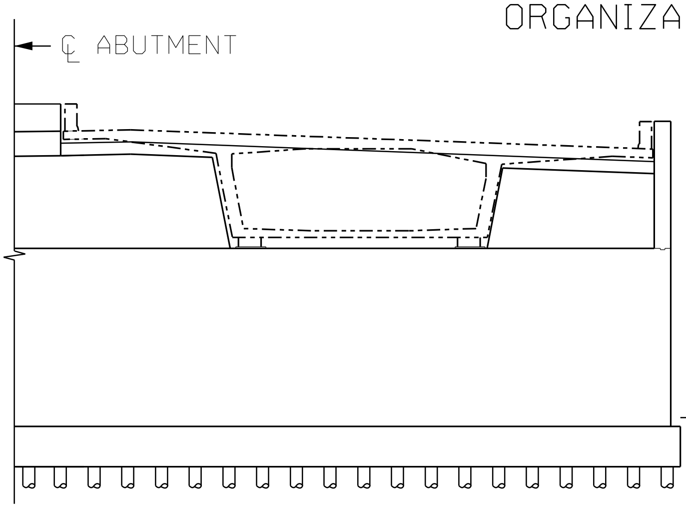
WEST ABUTMENT SURVEY SHEET

BRIDGE: WB / EB

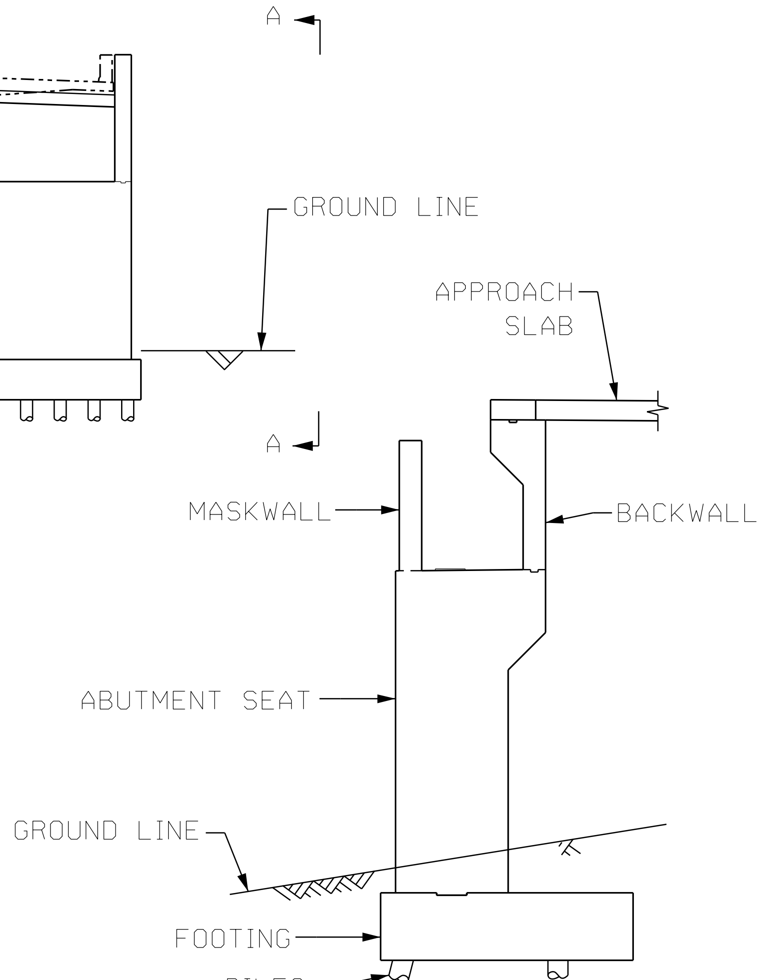
INSPECTED BY:____________________

DATE INSPECTED:____________________

ORGANIZATION:____________________



ELEVATION
(EB BRIDGE SHOWN,
WB BRIDGE OPPOSITE)



VIEW A-A

CONDITION LEGEND

G = GOOD

F = FAIR

P = POOR

S = SEVERE

ELEMENT	CONDITION	COMMENTS
ABUTMENT SEAT	G F P S	
BACKWALL	G F P S	
APPROACH SLAB	G F P S	
MASKWALL	G F P S	

I-90 DRESBACH BRIDGE

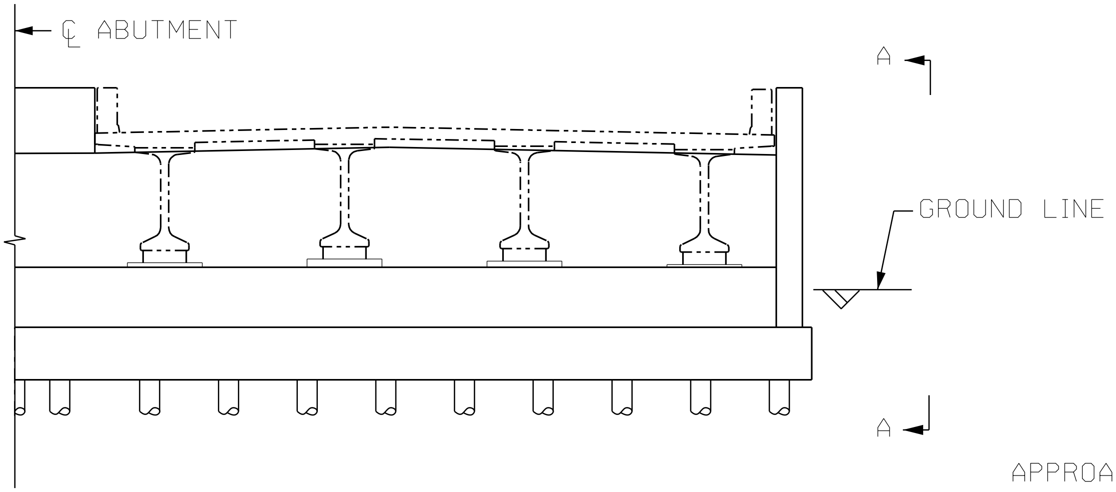
EAST ABUTMENT SURVEY SHEET

BRIDGE: WB / EB

INSPECTED BY:______________________

DATE INSPECTED:____________________

ORGANIZATION:______________________



ELEVATION
(EB BRIDGE SHOWN,
WB BRIDGE OPPOSITE)

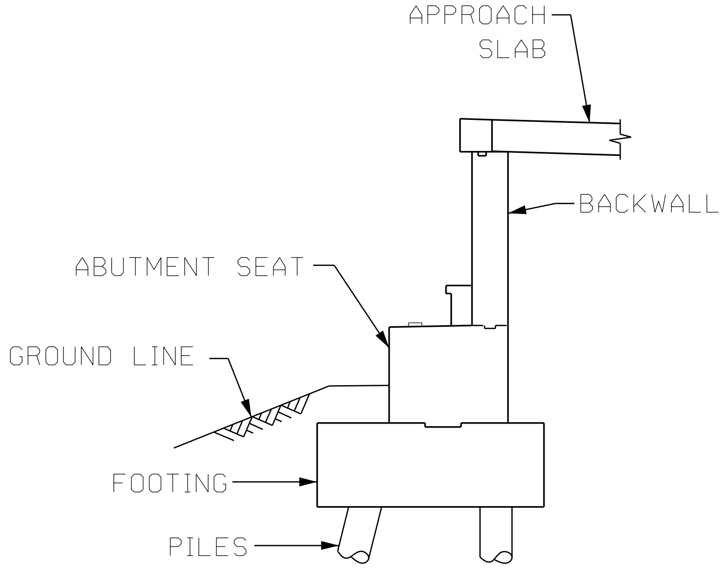
CONDITION LEGEND

G = GOOD

F = FAIR

P = POOR

S = SEVERE



VIEW A-A

ELEMENT	CONDITION	COMMENTS
ABUTMENT SEAT	G F P S	
BACKWALL	G F P S	
APPROACH SLAB	G F P S	

I-90 DRESBACH BRIDGE

PIERS 1 - 3 SURVEY SHEET

PIER NO.: 1 / 2 / 3

BRIDGE: WB / EB

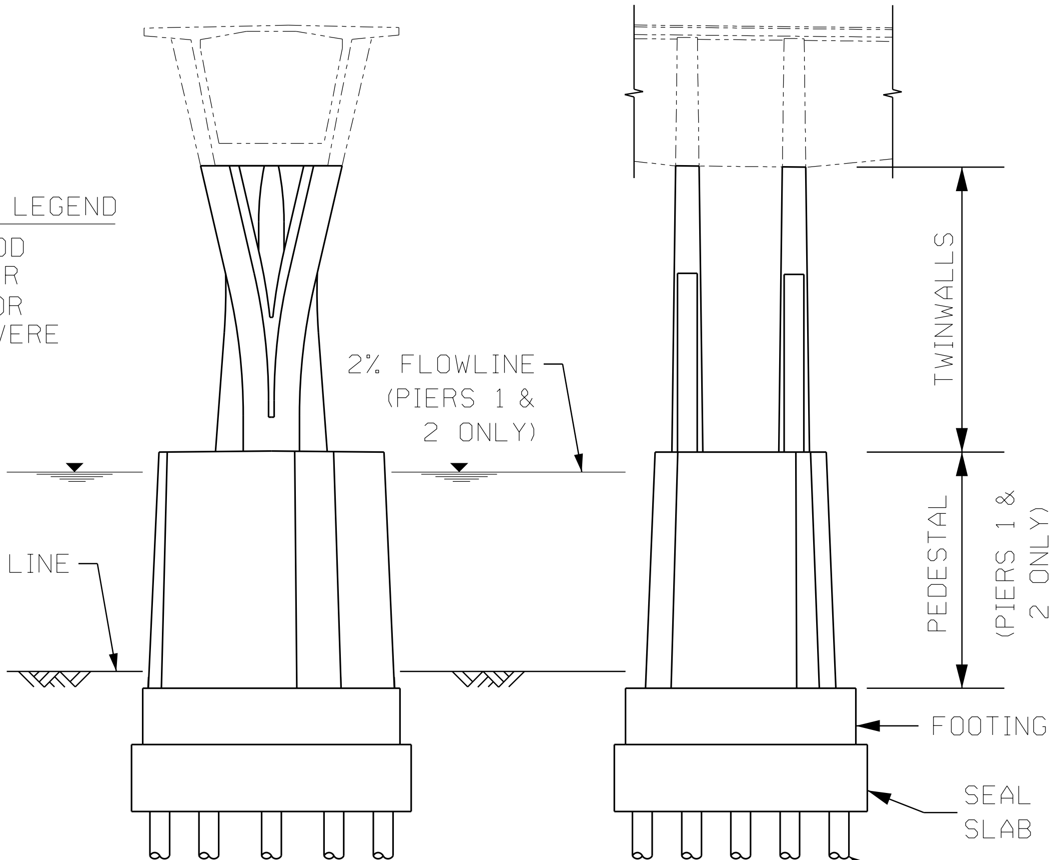
INSPECTED BY:____________________

DATE INSPECTED:____________________

ORGANIZATION:____________________

CONDITION LEGEND

G = GOOD
F = FAIR
P = POOR
S = SEVERE



ELEVATION VIEW

(UP STA / DWN STA)
(INT FACE / EXT FACE)

SIDE VIEW

(NORTH / SOUTH)

ELEMENT	CONDITION	COMMENTS
TWINWALL	G F P S	
PEDESTAL (PIERS 1 & 2 ONLY)	G F P S	

I-90 DRESBACH BRIDGE

PIER 4 SURVEY SHEET

PIER NO.: 4

BRIDGE: WB / EB

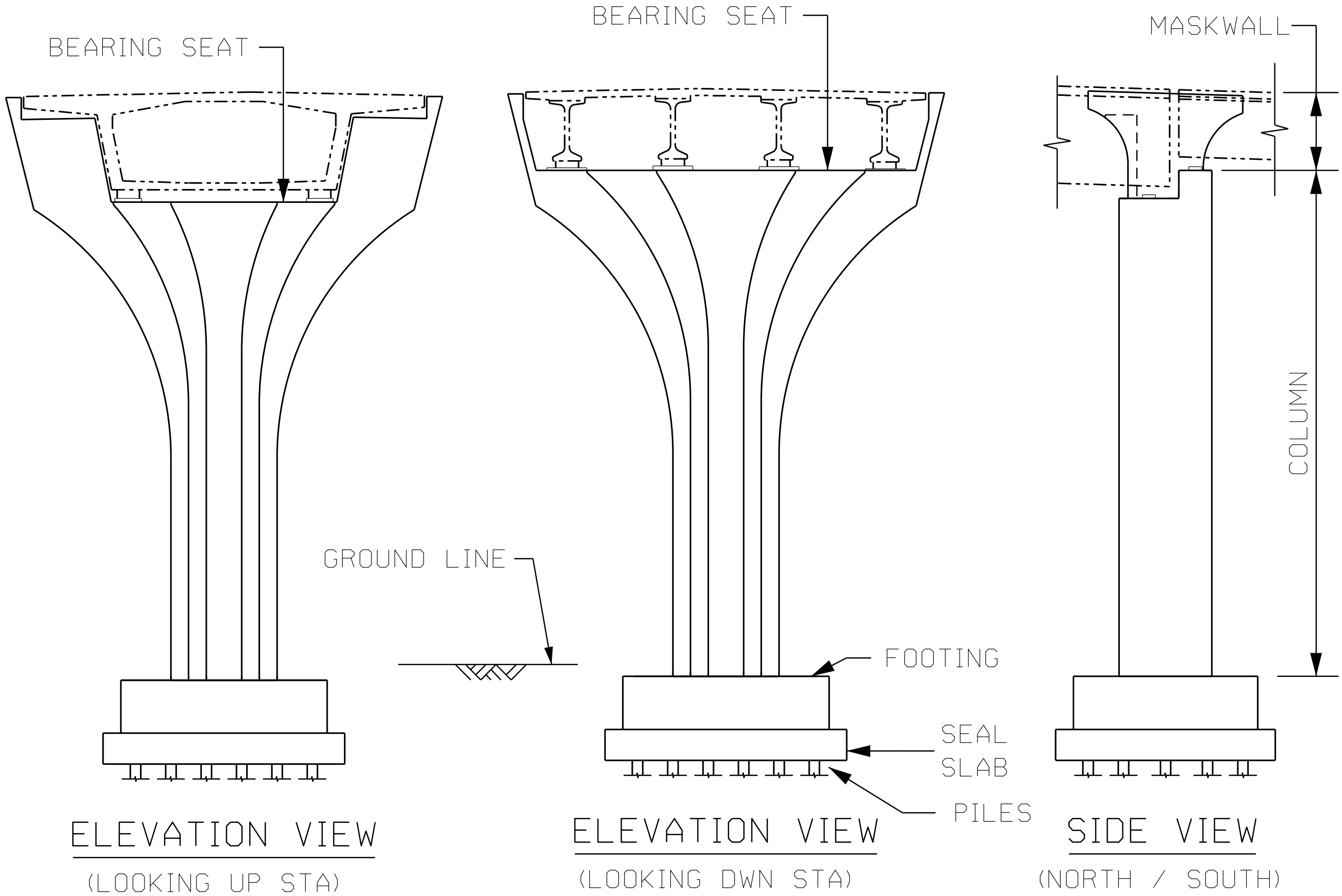
INSPECTED BY:________________

DATE INSPECTED:________________

ORGANIZATION:________________

CONDITION LEGEND

G = GOOD
F = FAIR
P = POOR
S = SEVERE



ELEMENT	CONDITION	COMMENTS
COLUMN	G F P S	
MASKWALL	G F P S	
BEARING SEAT	G F P S	

I-90 DRESBACH BRIDGE

PIERS 5 - 9 SURVEY SHEET

PIER NO.:_____

BRIDGE: WB / EB

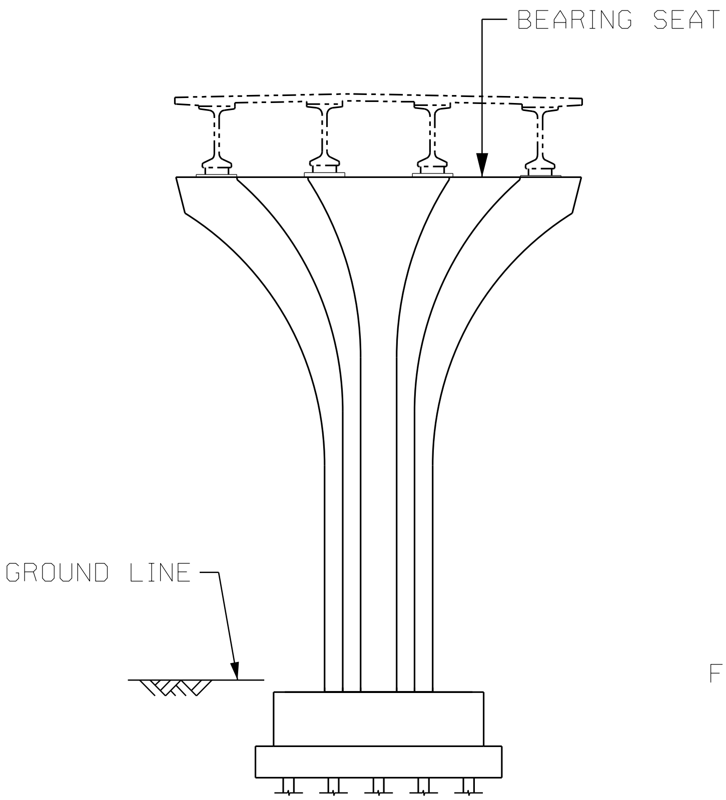
INSPECTED BY:________________

DATE INSPECTED:________________

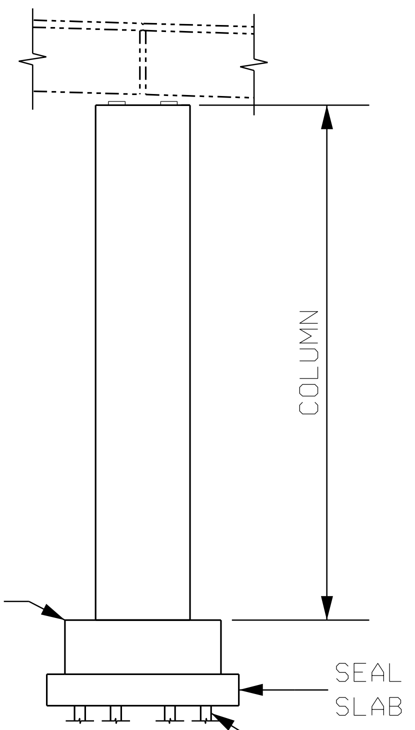
ORGANIZATION:________________

CONDITION LEGEND

G = GOOD
F = FAIR
P = POOR
S = SEVERE



ELEVATION VIEW
(LOOKING UP STA / LOOKING DWN STA)



SIDE VIEW
(NORTH / SOUTH)

ELEMENT	CONDITION	COMMENTS
COLUMN	G F P S	
BEARING SEAT	G F P S	

NON-GUIDED DISC BEARING INSPECTION SHEET

BRIDGE: WB EB W. ABUTMENT / PIER 4 (DN STA.)

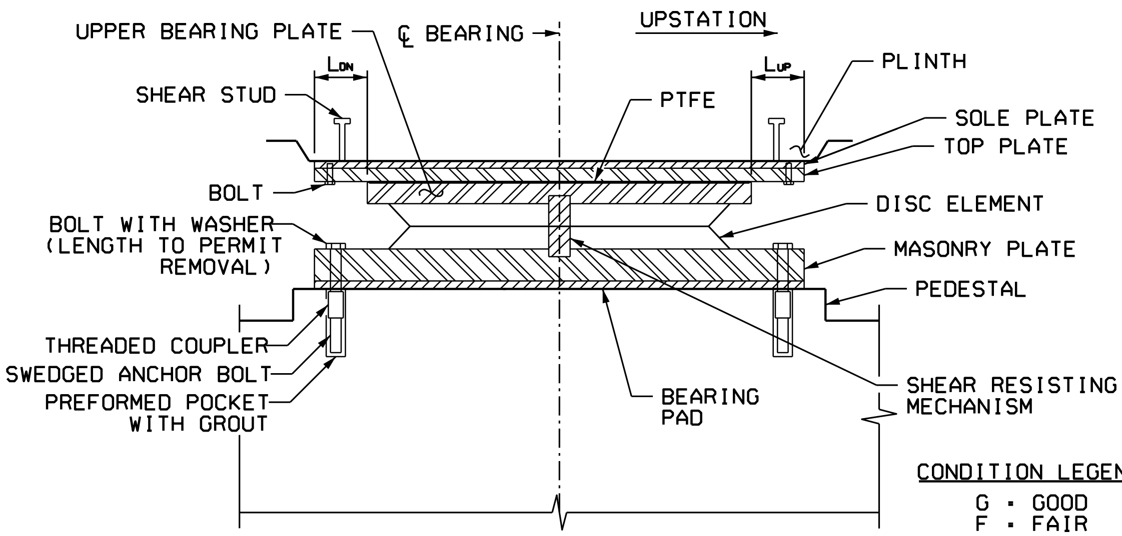
BEARING MANUFACTURER: D.S. BROWN

AMBIENT TEMPERATURE @ INSPECTION: ____________°F

CONCRETE TEMPERATURE @ INSPECTION - MID WEB (IN SHADE): ____________°F

INSPECTED BY: ____

DATE INSPECTED: ____



CONDITION LEGEND

G - GOOD
F - FAIR
P - POOR
S - SEVERE

SIDE VIEW

ELEMENT	CONDITION	COMMENTS
BEARING PLATE WITH PTFE	G F P S	
SOLE PLATE	G F P S	
POLYETHER URETHANE DISC	G F P S	
TOP & MASONRY PLATES	G F P S	
BEARING PAD	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)								MOVEMENT CAPACITY ADEQUATE
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MIN. DISTANCE FROM BRG TO END OF STAINLESS		MOVEMENT REQUIRED		L_{DN} > ⑤
①	②	③	④	L_{DN}	L_{UP}	⑤ DN STA.	⑥ UP STA.	L_{UP} > ⑥
								OK / NG*

① & ④ - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.1

IF ① > 0: ② = (CONC. TEMP. + 30°F) X ① ⑤ = ② + ④ (CONTRACTION)
③ = (120°F - CONC. TEMP.) X ① ⑥ = ③ (EXPANSION)

IF ① < 0: ② = (CONC. TEMP. - 120°F) X ① ⑤ = ② (EXPANSION)
③ = (-30°F - CONC. TEMP.) X ① ⑥ = ③ - ④ (CONTRACTION)

COMMENTS: ____

____

____

* IF MOVEMENT CAPACITY IS NO GOOD (NG) REFER TO SECTION 4.2.2

GUIDED DISC BEARING INSPECTION SHEET

BRIDGE: WB EB W. ABUTMENT / PIER 4 (DN STA.)

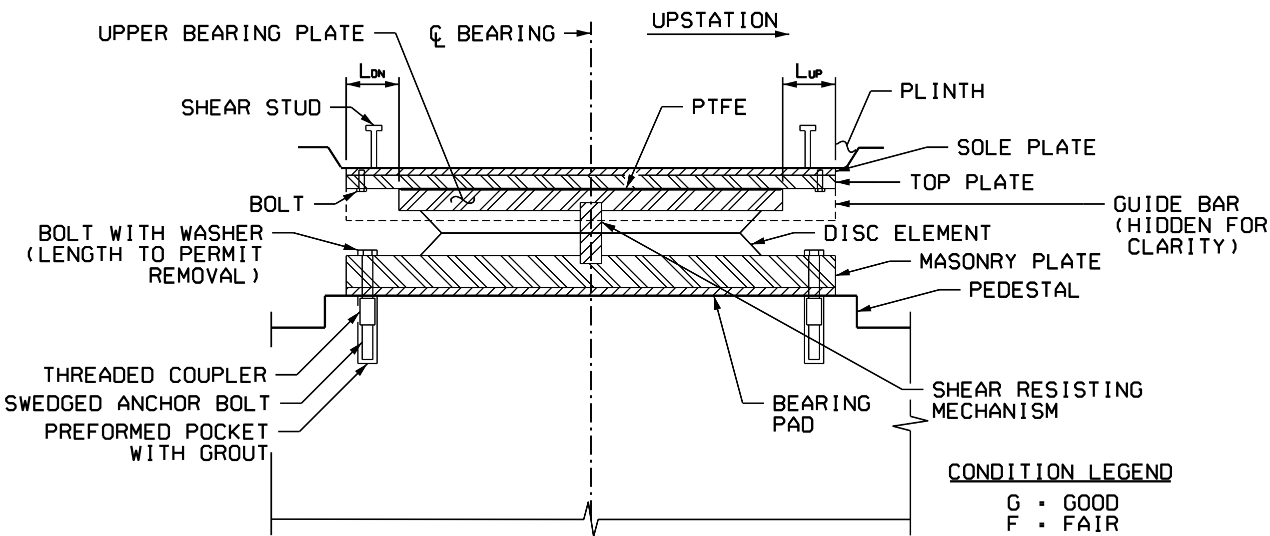
BEARING MANUFACTURER: D.S. BROWN

AMBIENT TEMPERATURE @ INSPECTION: ____________°F

CONCRETE TEMPERATURE @ INSPECTION - MID WEB (IN SHADE): ____________°F

INSPECTED BY: ______________________________

DATE INSPECTED: ______________________________



CONDITION LEGEND

G - GOOD
F - FAIR
P - POOR
S - SEVERE

SIDE VIEW

ELEMENT	CONDITION	COMMENTS
BEARING PLATE WITH PTFE	G F P S	
SOLE PLATE	G F P S	
POLYETHER URETHANE DISC	G F P S	
TOP & MASONRY PLATES	G F P S	
BEARING PAD	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)								MOVEMENT CAPACITY ADEQUATE
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MIN. DISTANCE FROM BRG TO END OF STAINLESS		MOVEMENT REQUIRED		
①	②	③	④	L_{DN}	L_{UP}	⑤ DN STA.	⑥ UP STA.	L_{DN} > ⑤ L_{UP} > ⑥
								OK / NG*

① & ④ - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.1

IF ① > 0: ② = (CONC. TEMP. + 30°F) X ① ⑤ = ② + ④ (CONTRACTION)
③ = (120°F - CONC. TEMP.) X ① ⑥ = ③ (EXPANSION)

IF ① < 0: ② = (CONC. TEMP. - 120°F) X ① ⑤ = ② (EXPANSION)
③ = (-30°F - CONC. TEMP.) X ① ⑥ = ③ - ④ (CONTRACTION)

COMMENTS: ______________________________

* IF MOVEMENT CAPACITY IS NO GOOD (NG) REFER TO SECTION 4.2.2

EXPANSION JOINT INSPECTION SHEET

BRIDGE: WB EB WEST ABUTMENT / PIER 4

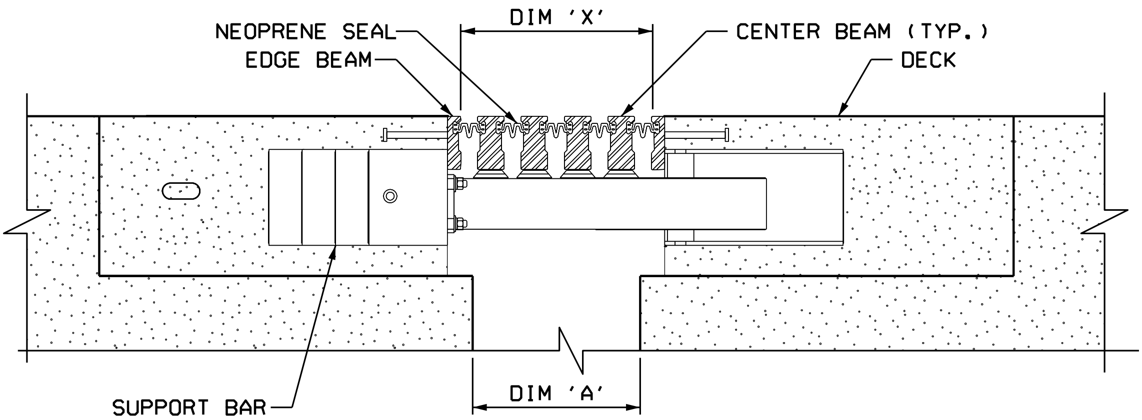
EXPANSION JOINT MANUFACTURER: D.S. BROWN

AMBIENT TEMPERATURE @ INSPECTION: __________°F

CONCRETE TEMPERATURE @ INSPECTION - MID WEB (IN SHADE): __________°F

INSPECTED BY: ______________________________

DATE INSPECTED: ______________________________



SECTION

CONDITION LEGEND

G - GOOD
F - FAIR
P - POOR
S - SEVERE

WEST ABUTMENT

MAX. ALLOWABLE 'X' = 28 1/4"
MIN. ALLOWABLE 'X' = 12 1/2"

PIER 4

MAX. ALLOWABLE 'X' = 50 7/8"
MIN. ALLOWABLE 'X' = 22 1/2"

ELEMENT	CONDITION	COMMENTS
CENTER BEAMS	G F P S	
DECK SURFACE @ JOINT	G F P S	
UNIFORMITY OF SEAL OPENINGS	G F P S	
SUPPORT BARS	G F P S	
NEOPRENE SEALS	G F P S	
WELDS BETWEEN SUPPORT BARS & CENTER BEAMS	G F P S	
SLIDING PLATES @ BARRIERS	G F P S	
BOND BETWEEN EDGE BEAMS AND CONCRETE	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)								MOVEMENT CAPACITY ADEQUATE IF:
MEASURED DIM 'A'	MEASURED DIM 'X' ①	THERMAL MOVEMENT ②	TEMP. CORRECTION FALL ③	TEMP. CORRECTION RISE ④	REMAINING CR. & SH. ⑤	ANTICIPATED ACTUAL 'X' MAX. ⑥	ANTICIPATED ACTUAL 'X' MIN. ⑦	MIN. > MIN. ALLOWABLE MAX. < MAX. ALLOWABLE
								OK / NG*

② & ⑤ - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.2

③ = (CONC. TEMP. + 30°F) X ②   ⑥ = ① + ③ + ⑤ (JOINT EXPANSION)

④ = (CONC. TEMP. - 120°F) X ②   ⑦ = ① + ④ (JOINT CONTRACTION)

COMMENTS: ______________________________

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* IF MOVEMENT CAPACITY IS NO GOOD (NG) REFER TO SECTION 4.3.2