INTERSTATE 35W

FIGG

Inspection & Maintenance Manual

I-35W St. Anthony Falls Bridge

Over the Mississippi River, Minneapolis, MN
Bridge No. 27409 (SB) & Bridge No. 27410 (NB)

August 2008

FIGG

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ABBREVIATIONS

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

Abut.	=	Abutment
C.J.	=	Construction Joint
Conc.	=	Concrete
Cr.	=	Creep
DN	=	Down
Elev.	=	Elevation
Est.	=	Estimated
FFBW	=	Front Face of Backwall
I.D.	=	Inside Diameter
In.	=	Inch
Lbf.	=	Pound Force
Lbs.	=	Pounds
Max.	=	Maximum
NB	=	Northbound
No.	=	Number
P.G.L.	=	Profile Grade Line
P.S.	=	Pier Segment
P.T.	=	Post-Tensioning
PTFE	=	Polytetrafluoroethylene
SB	=	Southbound
Sh.	=	Shrinkage
Spa.	=	Spaced
Sta.	=	Station
Symm.	=	Symmetric
Typ.	=	Typical
Wt	=	Weight
@	=	At
ϕ	=	Diameter

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LIMITATIONS

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CHAPTER 1

INTRODUCTION

1. INTRODUCTION

1.1 Purpose of the Manual

This manual provides special procedures for conducting inspections to determine the physical condition of the prestressed concrete box girder St. Anthony Falls Bridge, Interstate 35 West over the Mississippi River, Minneapolis, Minnesota. It also outlines a program of routine preventative and corrective maintenance procedures for the bridge and its components.

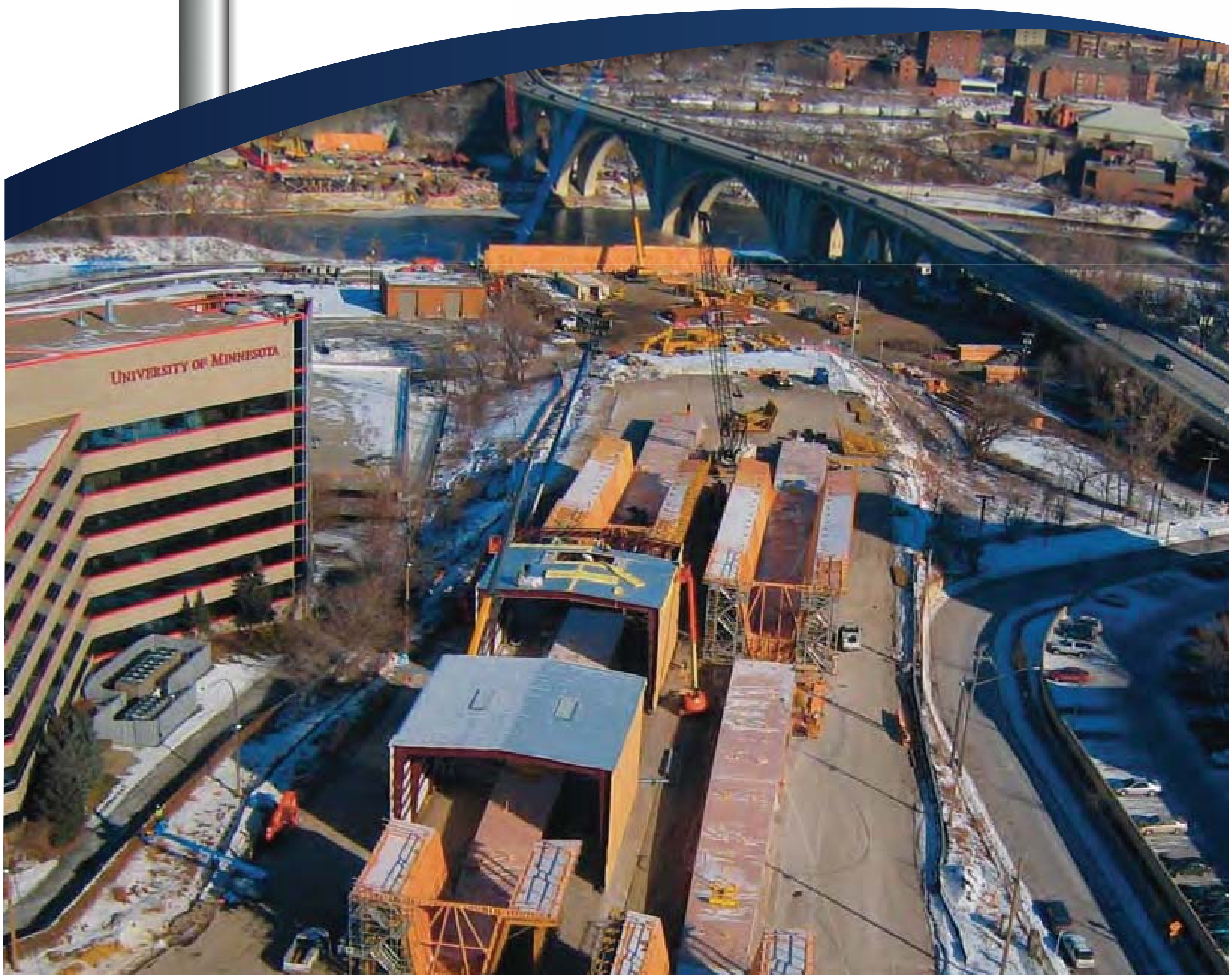
The inspection portions of this manual are intended for use by individuals qualified as bridge inspectors in accordance with the State of Minnesota standards and access is limited to only those who have a need to know as determined by the Minnesota Department of Transportation. The objective of this manual is to serve as a supplement to the Minnesota Department of Transportation standard procedures for detailed inspections of the St. Anthony Falls (I-35W) Bridge.

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 condition that may require future monitoring, corrective maintenance or repair. Inspection frequency as outlined in the Federal and State Directives should be adhered to. These inspections help ensure 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 providing the longest possible service life for the St. Anthony Falls (I-35W) Bridge.

Chapter 2

Description of the Bridge



2. DESCRIPTION OF THE BRIDGE

2.1 General Description

The St. Anthony Falls (I-35W) Bridge is a post-tensioned concrete box girder bridge located in Minneapolis, Minnesota, north of the Minneapolis/St. Paul International Airport (Figure 2.1) on Interstate 35 West over the Mississippi River. The bridge is comprised of two parallel structures 1227'-9" (NB) and 1220'-3" (SB) in length. Each structure features two box girders carrying five 12 ft traffic lanes, a 13 ft outside shoulder and a 14 ft inside shoulder. A 2-bar steel bridge railing is used along the outside shoulder overlooking the Mississippi River and a 4'-6" high concrete bridge rail is used along the inside shoulder. The main span 90'-4" (typical) wide twin box girder assembly varies in depth from 25'-2 1/2" at Piers 2 and 3 to 11'-2 1/2" at midspan of Span 2, Abutment 1 and Pier 4 (for Spans 1 through 3). Span 2 was constructed with precast segments erected in cantilever, while all other spans were cast-in-place on falsework.

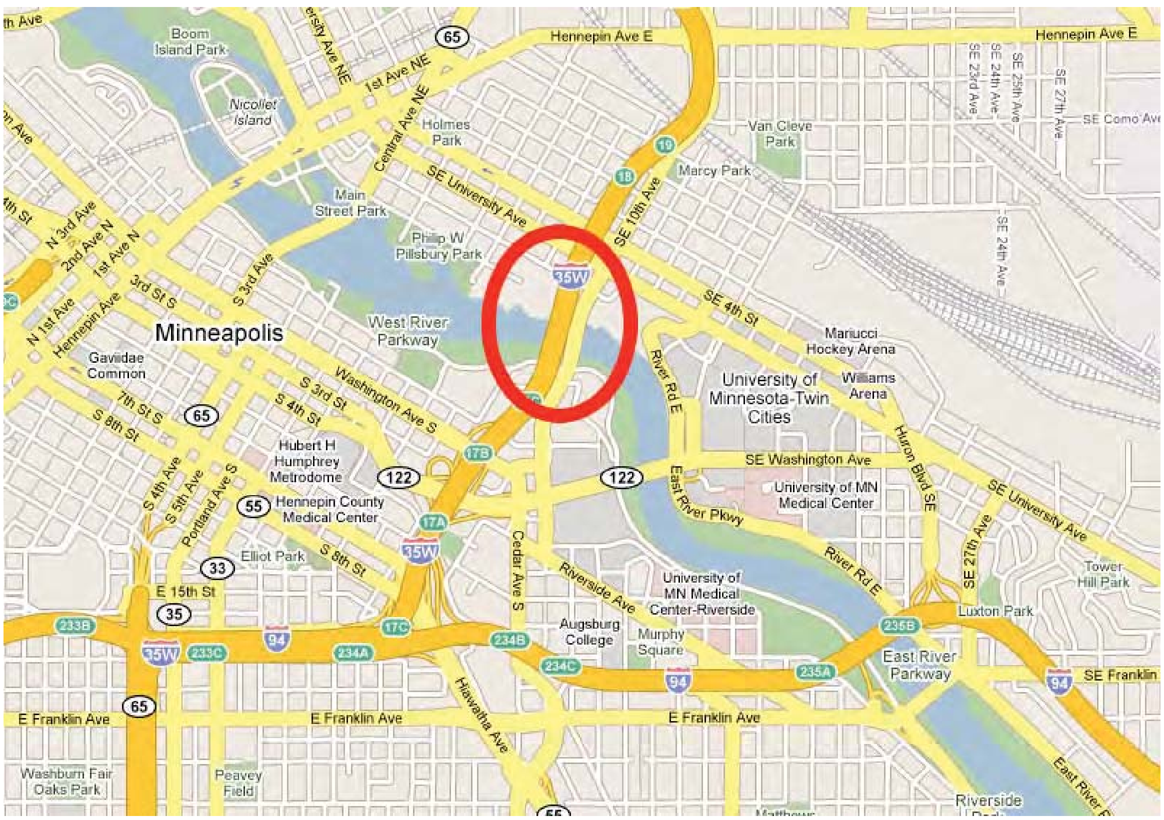


Figure 2.1 – Project Location

The main span unit is a 3-span continuous structure with expansion joint devices located at Abutment 1 and Pier 4. Span 4 is a single span with fixed bearing connection at Pier 4 and an integral monolithic superstructure/abutment at Abutment 5. A general plan and elevation of the project is shown in Figure 2.2 and the bridge typical section in Figure 2.3.

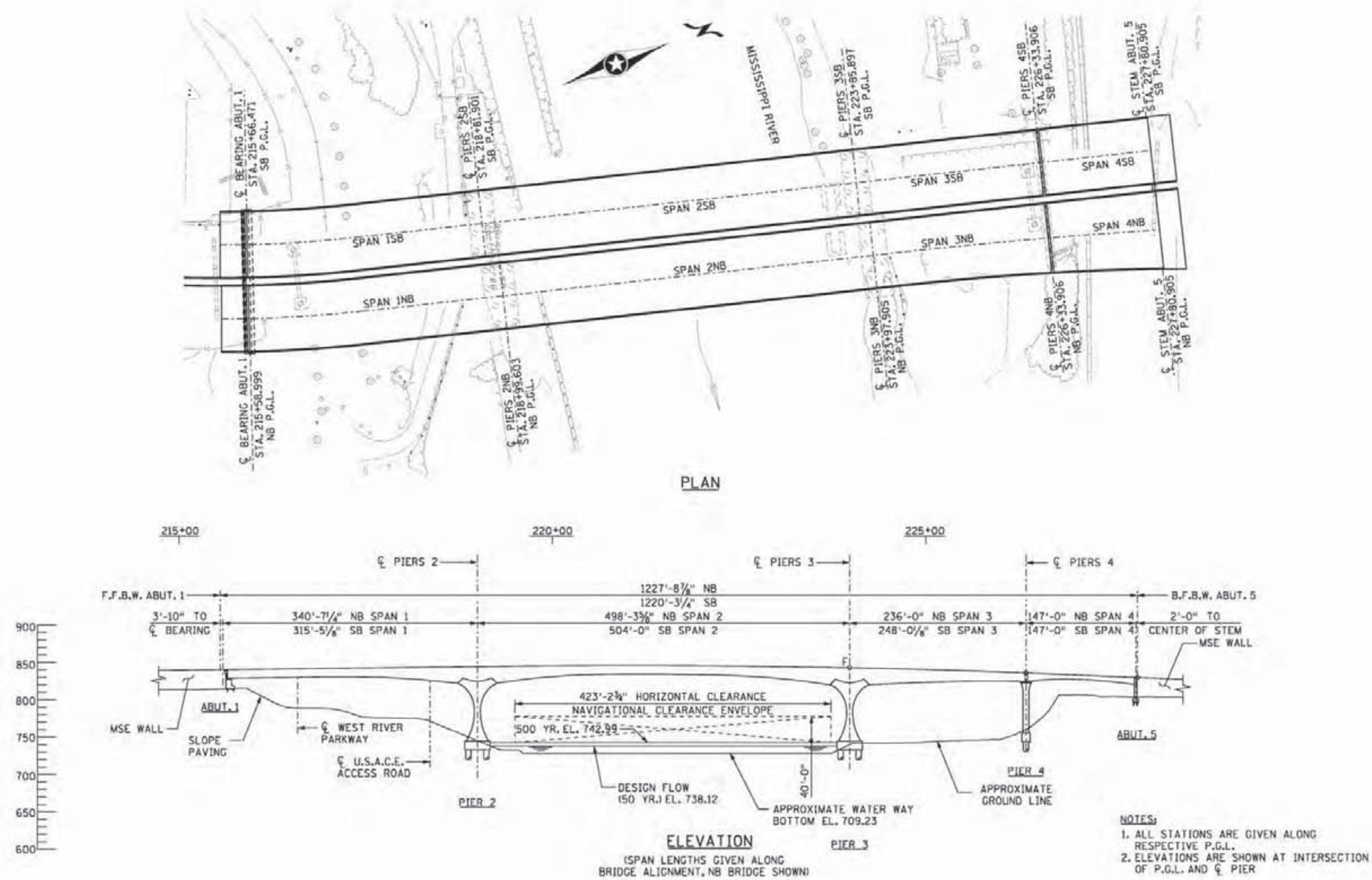


Figure 2.2 – General Plan and Elevation

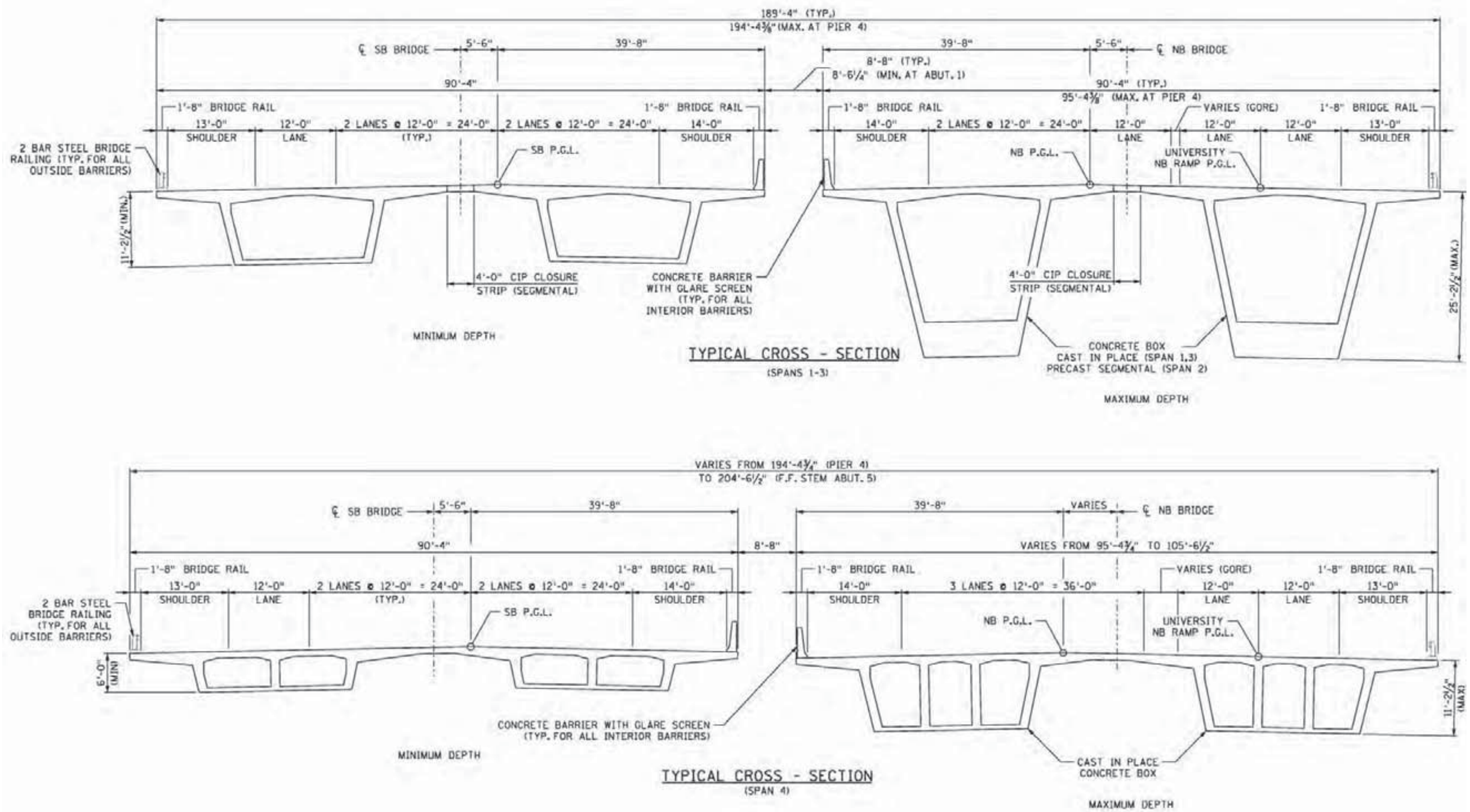


Figure 2.3 – Bridge Typical Section

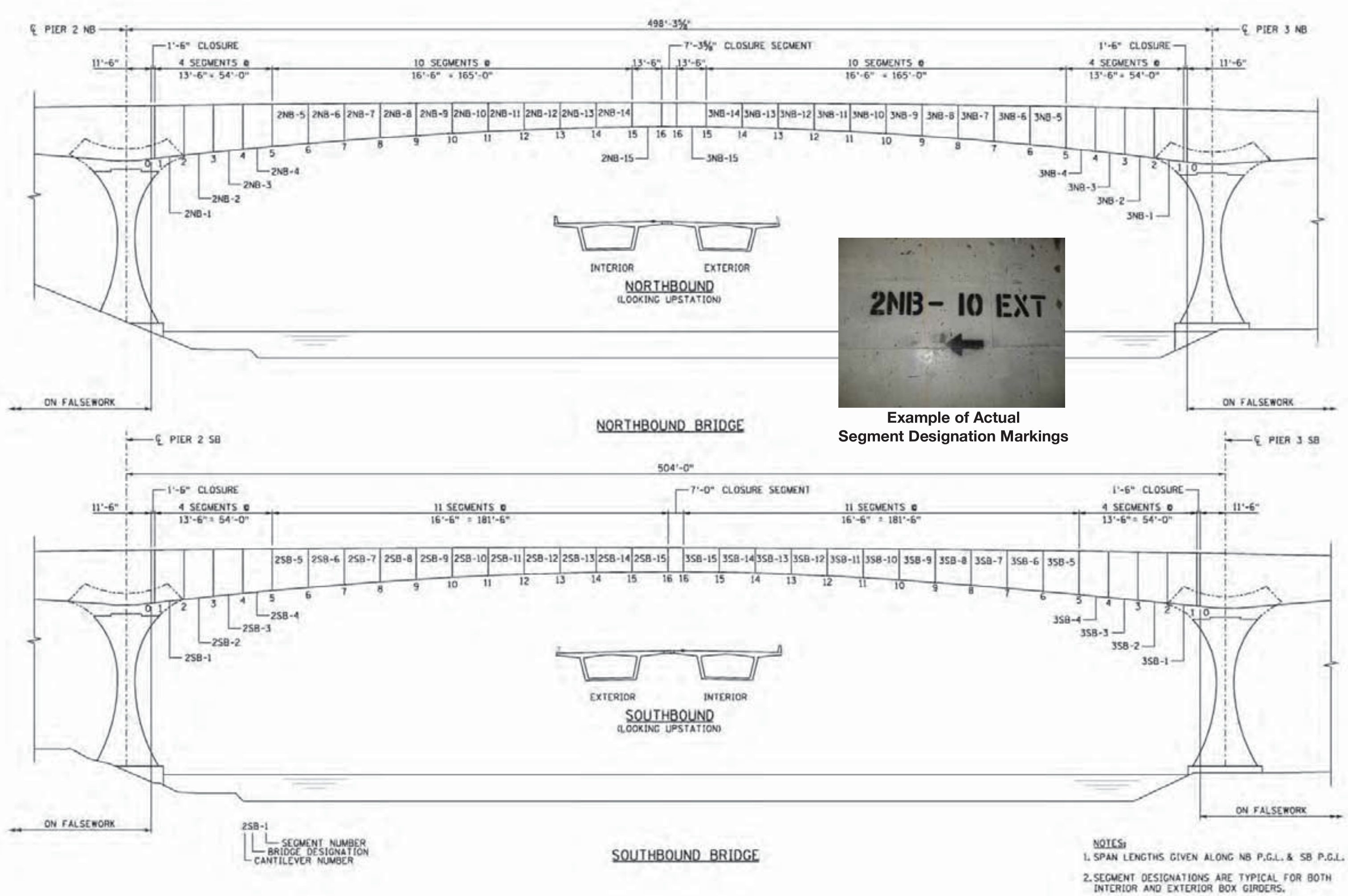
2.2 Basics of Box Girder Bridge Construction

The St. Anthony Falls (I-35W) Bridge features precast and cast-in-place post-tensioned concrete box girder construction. The precast segments in Span 2 were cast in two distinct lengths in the longitudinal direction of 13'-6" and 16'-6". Segments cantilevering from a given pier were match-cast with a cast-in-place closure joint near the pier and a main span cast-in-place closure segment at midspan. For the purpose of identifying precast segments, both during construction and after the bridge is completed, all precast segments have been permanently marked on the interior of the box girder with a unique designation (Figure 2.4).

Segments were precast south of the bridge on the old I-35W approach road bed in a casting yard where segments could be cast in a controlled environment during winter months (Figure 2.5). After segments were cast and cured, they were transported to the Bohemian Flats area along the Mississippi River (Figure 2.6) for storage until they were ready to be erected. The formwork used for long-line casting of segments, called a casting bed (8 total – one for each cantilever), included individual bulkheads for each segment, retractable core form, bottom soffit, web, and wing forms that were made of structural steel and timber (Figure 2.7). The bottom soffit was fixed timber and the core form was adjustable in order to vary the depth of the segments. The long line method of precasting (Figure 2.8) used for the bridge involved casting a single segment, referred to as a *wet-cast* segment, between the bulkhead form and a previously cast segment, referred to as a *match-cast* segment. The desired vertical and horizontal geometry, with compensation for the predicted camber, was cast into each segment by proper pre-positioning of the formwork. Once properly cured, the wet-cast segment became the next match-cast segment. Non-continuous transverse deck post-tensioning was stressed and grouted in the casting yard after match casting but prior to storage and subsequent erection (Figure 2.9) and continuous PT was stressed and grouted after erection and placement of longitudinal closures.

The cast-in-place side spans (Spans 1 and 3) were completed on falsework while the main span box girders (Span 2) were being precast. By utilizing precast and cast-in-place construction concurrently, the schedule duration was optimized. Cast-in-place concrete box girder construction for Spans 1 and 3 commenced with the erection of falsework to support the formwork, including the large diaphragms located over the piers. Once the formwork was in place, the contractor placed the reinforcement for the bottom slab and webs for the entire cast-in-place span, then poured the concrete. After placing concrete for bottom slab and webs, additional forms were placed to form the 'core' and top slab of the box girder. The reinforcement was installed for the top slab and the concrete subsequently placed.

Following completion of the side spans and pier table, erection of the main span began in cantilever. The first pair of segments at each pier (interior and exterior girder assemblies) was erected by crane and supported by cantilever beams extending off the pier top. The beams supported the bottom soffit of the segments, which allowed for the "starter segment" position to be carefully and precisely aligned. A 1'-6" cast-in-place closure pour between the cast-in-place span and the precast starter segment was then formed and concrete placed.



Example of Actual Segment Designation Markings

Figure 2.4 – Segment Designation

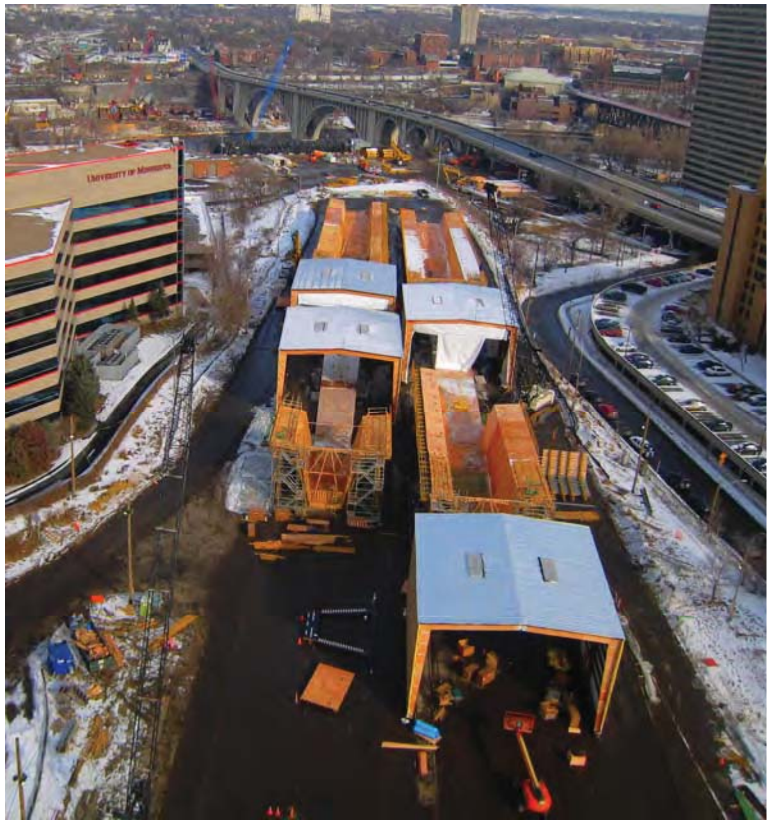


Figure 2.5 – Casting Yard

Figure 2.6 – Segment Storage

Figure 2.7 – Casting Bed

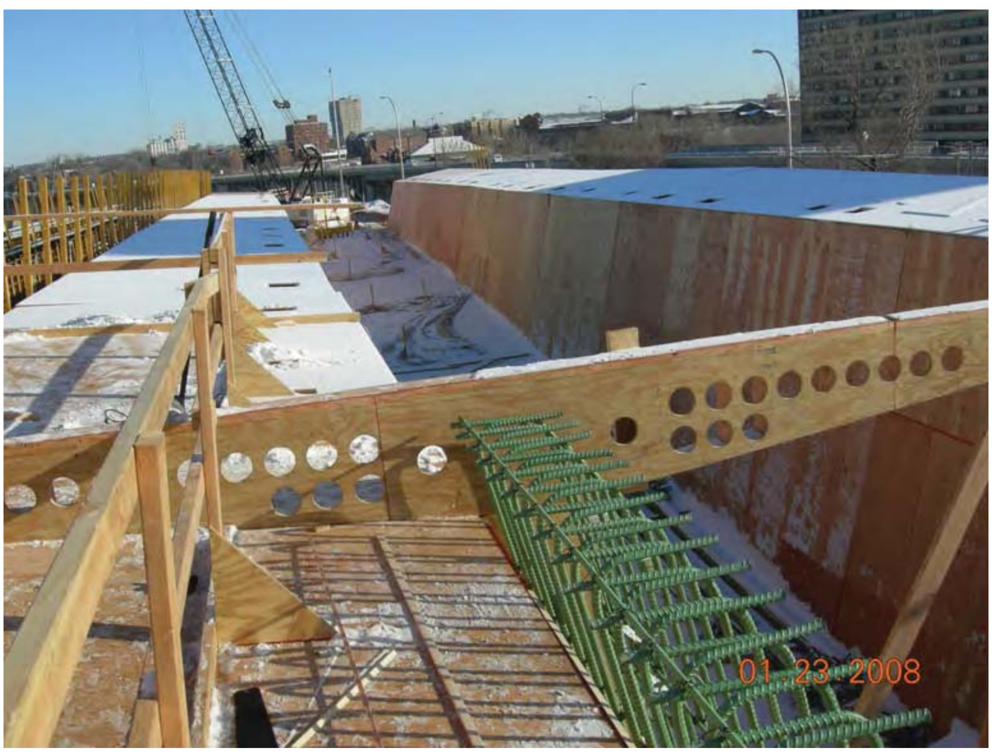


Figure 2.8 – Long Line Casting Bed

Figure 2.9 – Segment Casting

The remaining segments were then erected. Once a segment was lifted and aligned by barge mounted crane (Figure 2.10), post-tensioning bars were coupled to the previous segment. A special epoxy formulated for precast segmental erection was then applied to both segment faces to seal the joints and the post-tensioning bars were stressed (Figure 2.11).

After positioning the segment with post-tensioning bars, multi-strand structural cantilever tendons were installed and stressed (Figure 2.12). The previous segment's 4 ft. wide longitudinal closure strip between the parallel precast box girders was cast and the continuous transverse post-tensioning stressed as the precast segment erection advanced.

Once the precast segments and longitudinal closure strips were completed, a 7 ft. cast-in-place closure joint was formed and poured at mid-span to join the two cantilevers. After the cast-in-place closures were properly cured, longitudinal tendons located inside the box girder (draped tendons and bottom slab tendons) were stressed. Figure 2.13 shows the external draped tendons in the bridge.

Cast-in-place Span 4 is a multi-cell box girder with intermediate diaphragms, and exterior shape matching the main-span unit box girder. Span 4 was constructed after Span 3 was built. The concrete for Span 4 was placed similar to the other cast-in-place spans (bottom slab and webs followed by the top slab) and the longitudinal post-tensioning stressed from the integral Abutment 5 backwall. The post-tensioning anchors for Span 4 are embedded in the Abutment 5 backwall concrete and at the Span 4 Expansion Joint located at Pier 4.

2.3 Structure Components

The components of the structure can be categorized as substructure, bearings, superstructure (box girder, post-tensioning system and integral wearing surface), approach slabs, and expansion joint devices.

Figure 2.10 – Crane Erecting Segments (Span 2)

Figure 2.11 – Epoxy Joint Between Segments

Figure 2.12 – Multi-Strand Cantilever PT Strand

Figure 2.13 – Span 1 External Draped Tendons

2.3.1 Substructure

The substructure is composed of cast-in-place reinforced concrete piers, abutments and their foundations. The Piers are founded on 7'-0" or 8'-0" cast-in-place drilled shaft foundations and Abutment 5 is founded on 4'-0" cast-in-place drilled shafts. Abutment 1 is founded on HP 14X117 driven steel H-Piles. The foundations serve to transfer all superstructure loads, both vertical and horizontal, to the load bearing strata.

2.3.1.1 Foundations

NB and SB structures at Piers 2 and 3 are supported by a group of eight cast-in-place drilled shafts, 84" or 96" in diameter, with cast-in-place concrete footings. Four cast-in-place drilled shafts, 96" in diameter with cast-in-place concrete footings, form the foundation system for Piers 4 NB and SB (Figure 2.14). Special foundation layouts were implemented to avoid existing storm drains that are located beneath the pier footings as shown in Figure 2.15.

A group of 102 A572 Grade 50 HP 14X117 driven steel piles at Abutment 1 and a group of 40 cast-in-place drilled shafts 48" in diameter at Abutment 5 comprise the foundation systems for the bridge abutments.

2.3.1.2 Piers

Piers transmit all superstructure loads to the foundations. Piers 2 and 3 columns are hour-glass shaped with a 26'-0" long base tapering to an 8'-0" mid-section and widening to 31'-8" at the top of pier (Figure 2.16). The top of the pier has extensions that enclose the bearing seat assembly. Transversely, the pier cap widens from 16'-0" to 19'-6 1/4" at the top of pier to accommodate the pier extensions (Figure 2.17). Piers 4 columns are also hour-glass shaped with a 10'-2 1/4" (NB) or 11'-0" (SB) long base tapering to a 7'-0" mid-section widening to 12'-0" at the top of pier (Figure 2.18). Transversely, the pier cap widens at the top of pier to align with the bottom soffit of the bridge segments (Figure 2.19). All piers were constructed using cast-in-place reinforced concrete.

2.3.1.3 Abutments

Abutments transmit superstructure loads to the soil substrata and resist fill pressure behind the backwall. Abutments 1 and 5 are shown in Figures 2.20 and 2.21. These figures also show the foundation arrangement for the abutments.

Mechanically stabilized earth (MSE) walls with stone-filled gabion basket facing (Gabion© Walls) are used on both the North and South approaches. These walls start at the abutments and serve as wingwalls. The abutments are cast-in-place reinforced concrete with 20'-0" long approach slabs that rest on top of the abutment backwalls and mitigate the effect of potential settlement at the roadway to bridge interface (Figure 2.22).

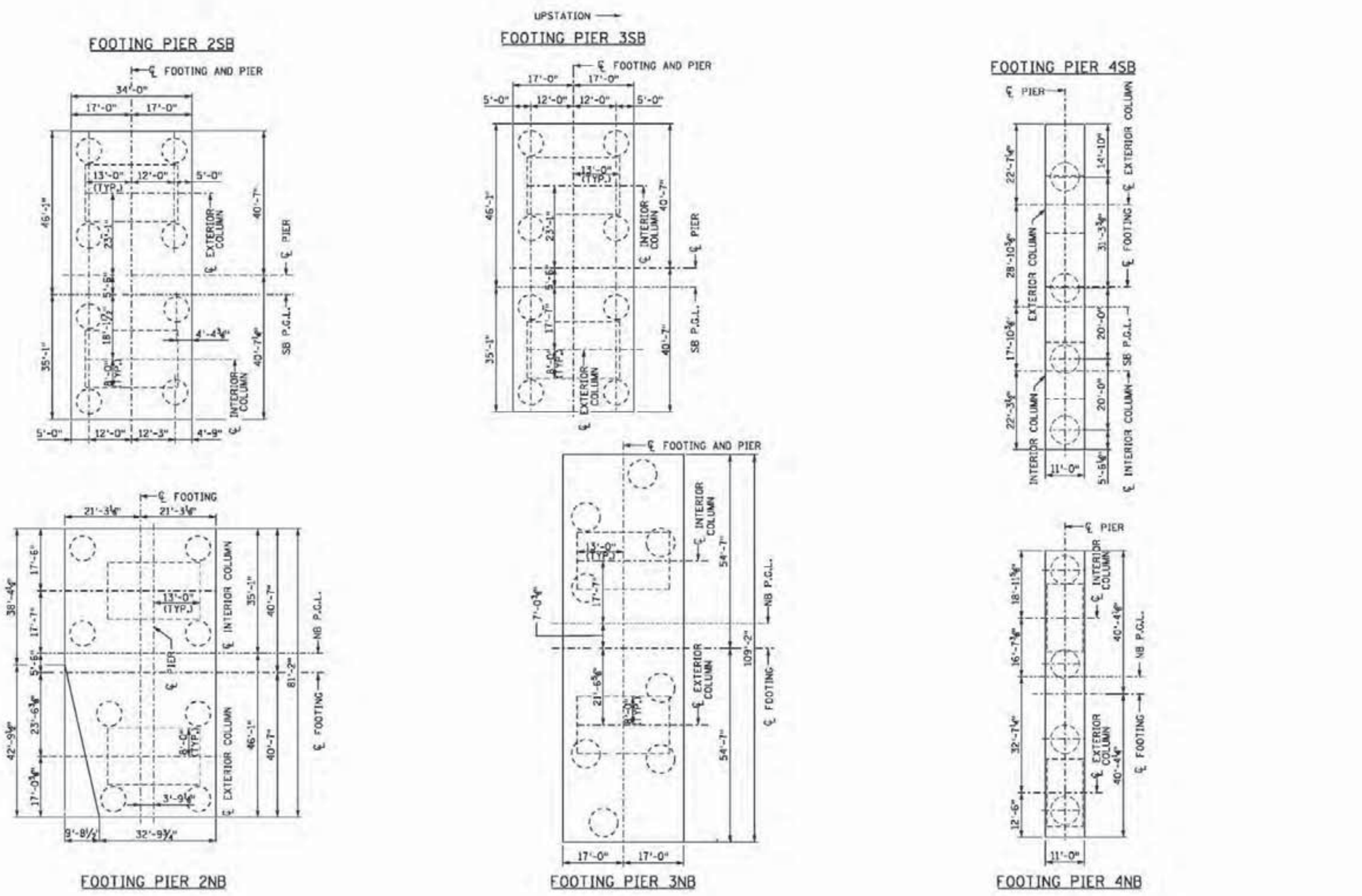


Figure 2.14 – Pier Footing Section

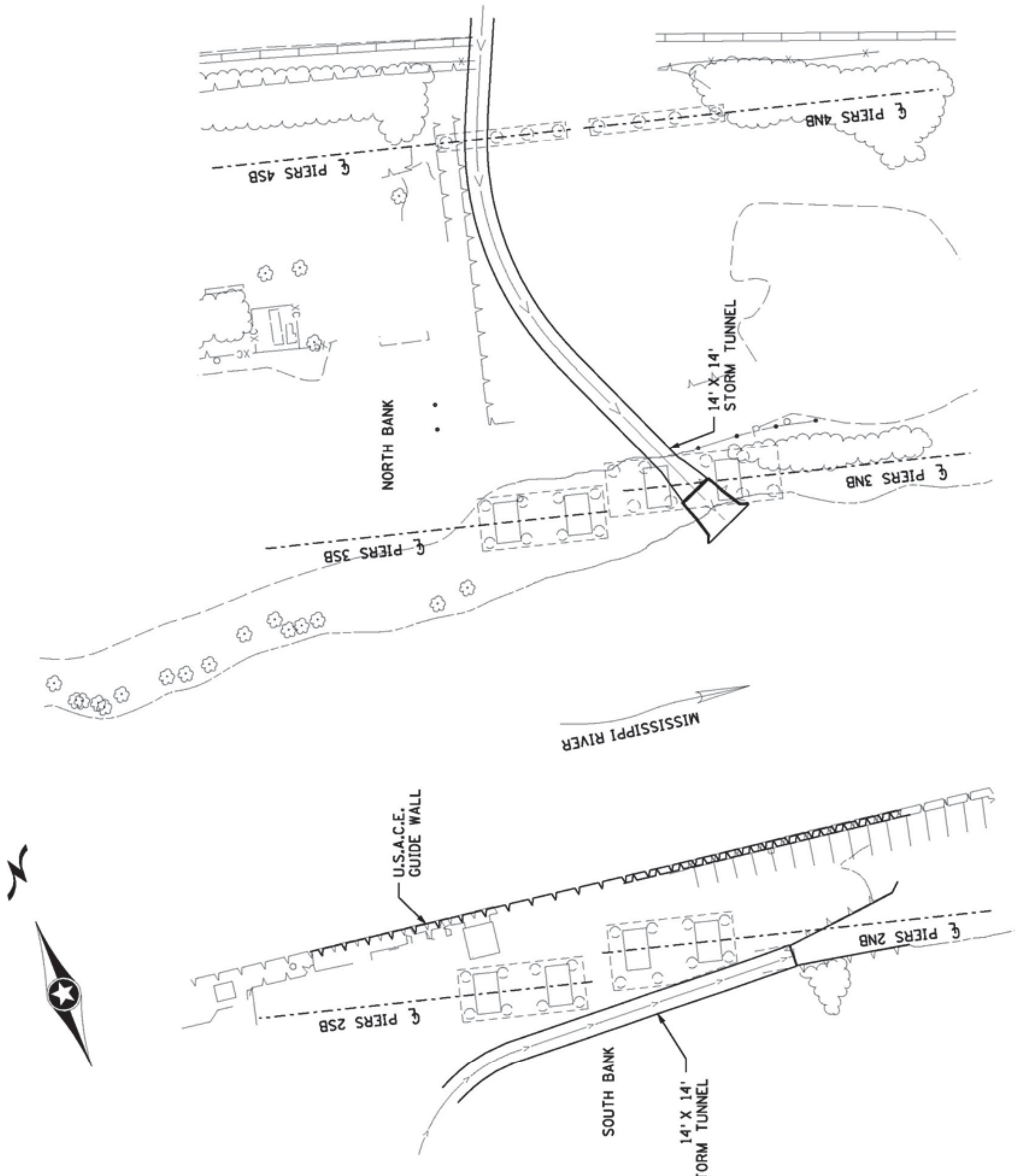


Figure 2.15 – Storm Drain Layout

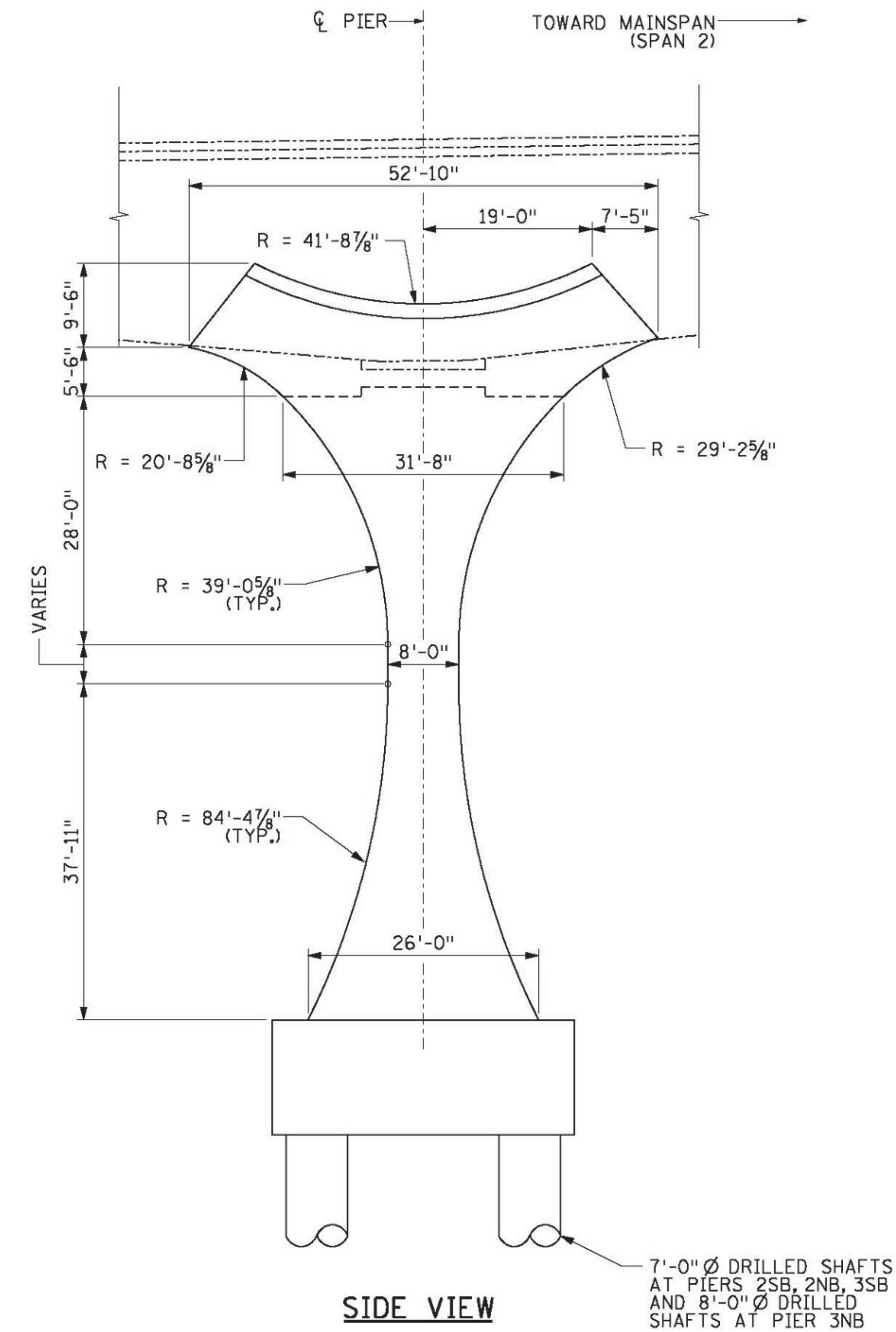


Figure 2.16 – Piers 2 and 3 Side View

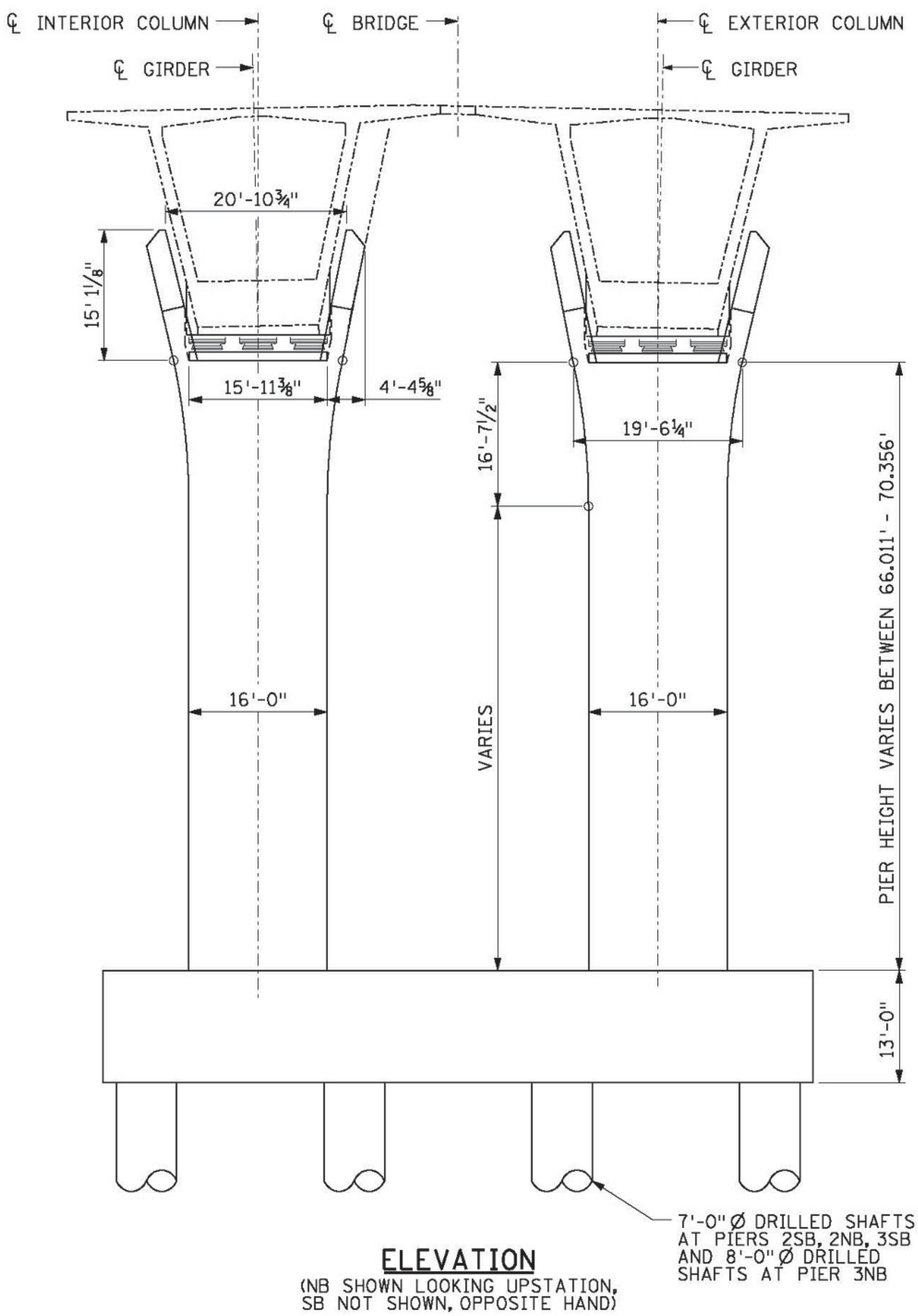


Figure 2.17 – Piers 2 and 3 Elevation

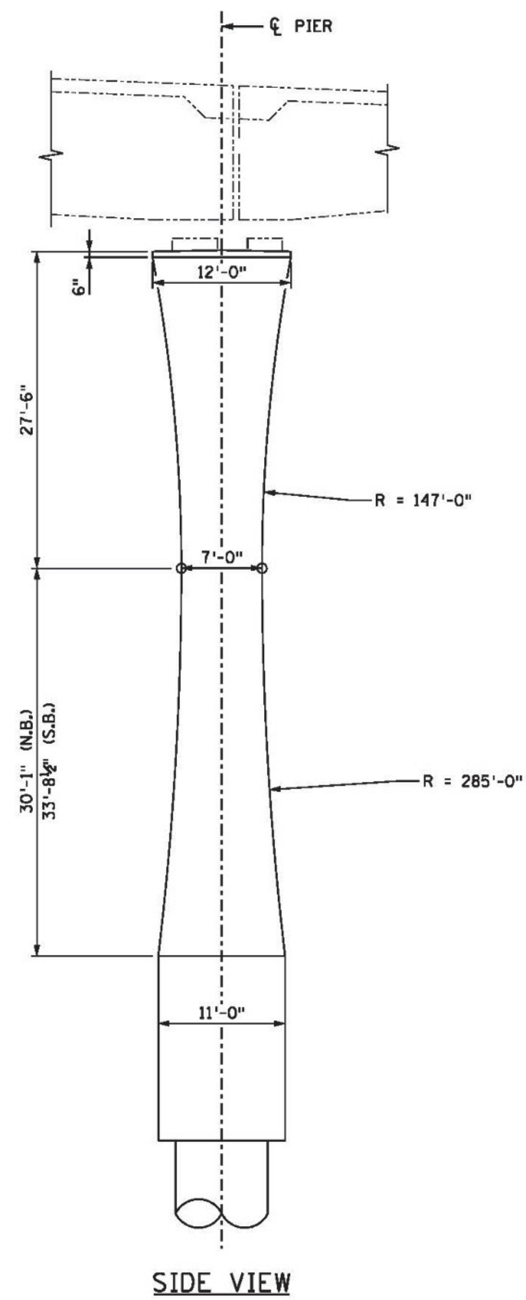


Figure 2.18 – Pier 4 Side View

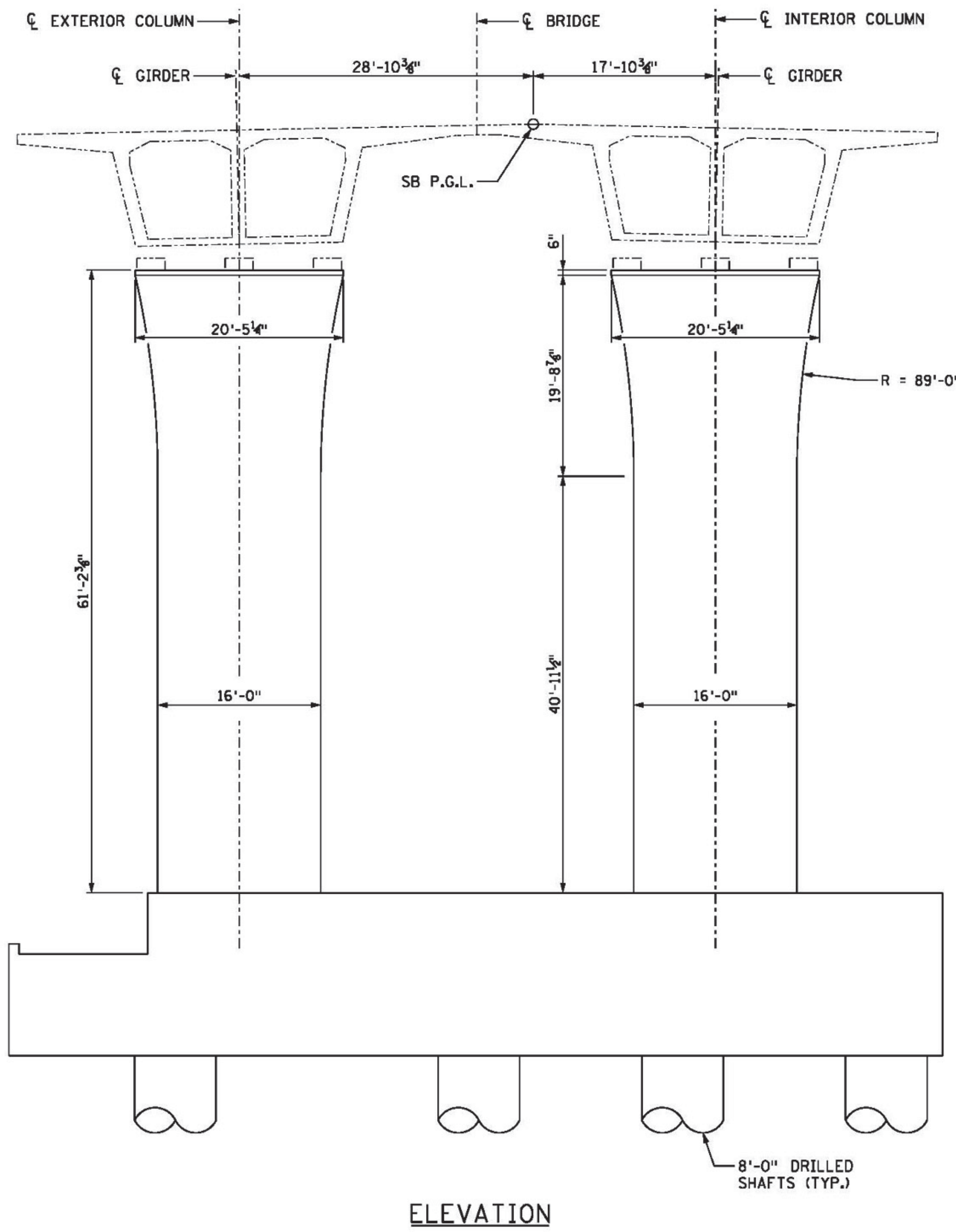


Figure 2.19 – Pier 4 Elevation
(SB Shown, NB Similar)

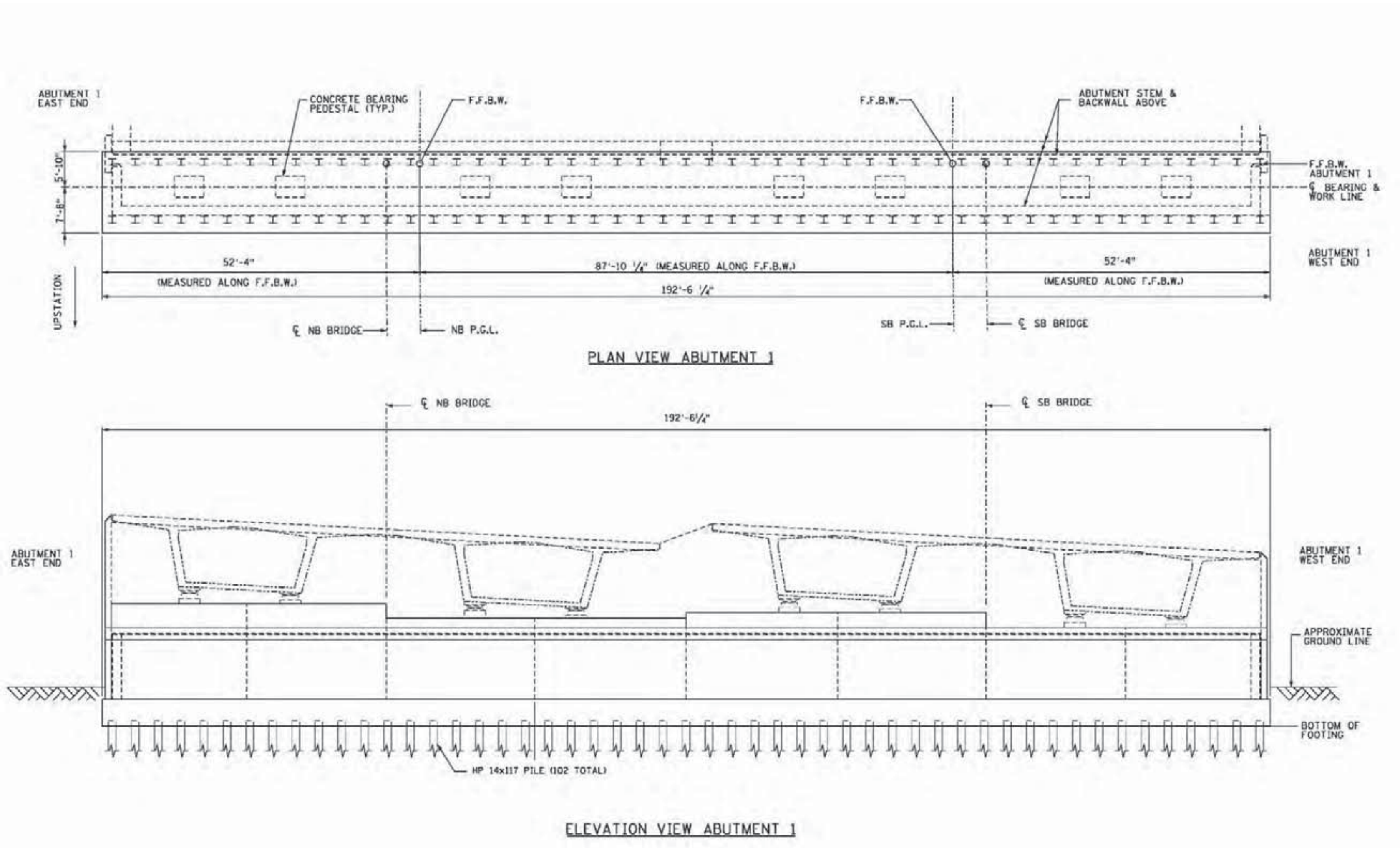


Figure 2.20 – Abutment 1 Foundation Layout

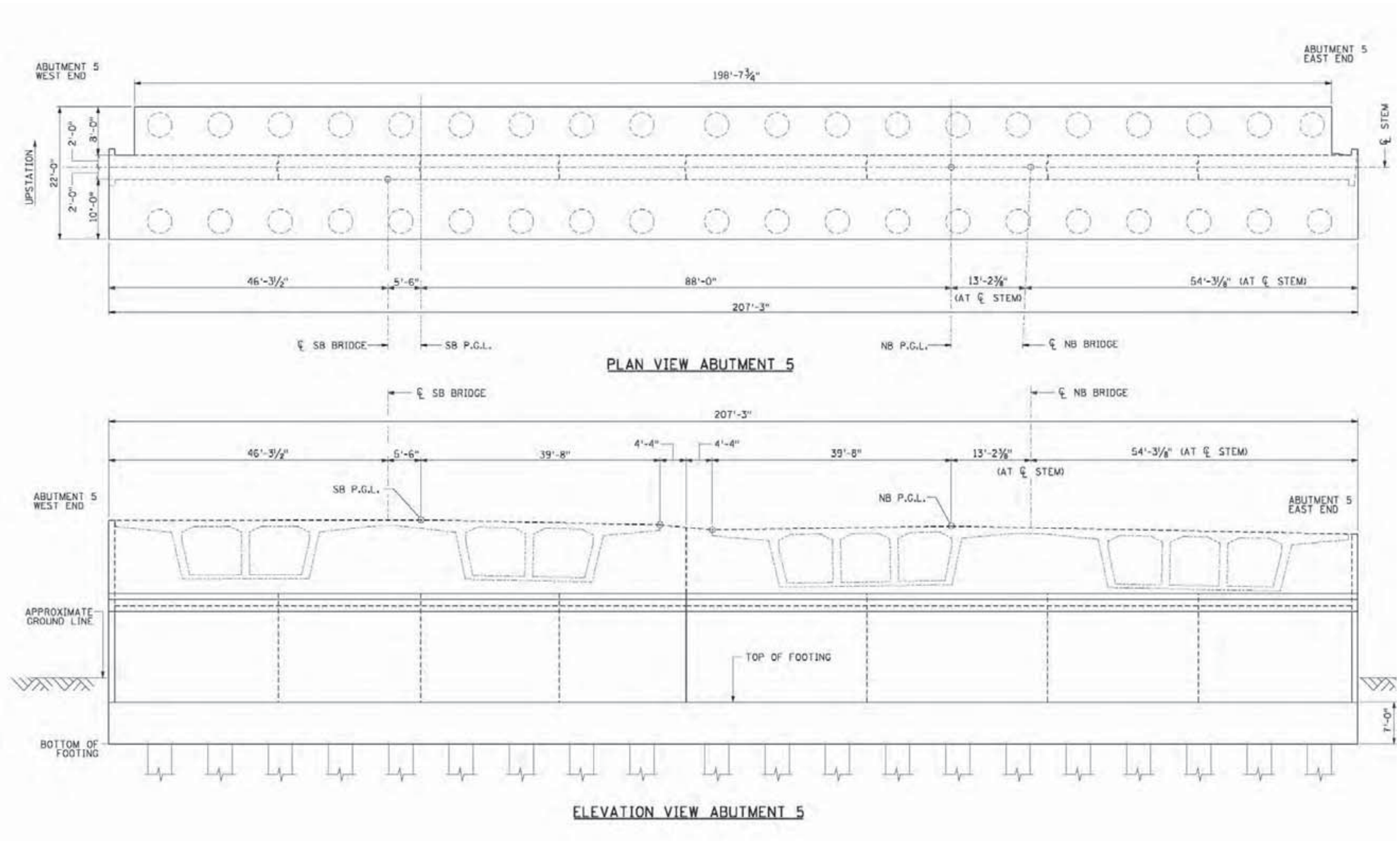


Figure 2.21 – Abutment 5 Foundation Layout

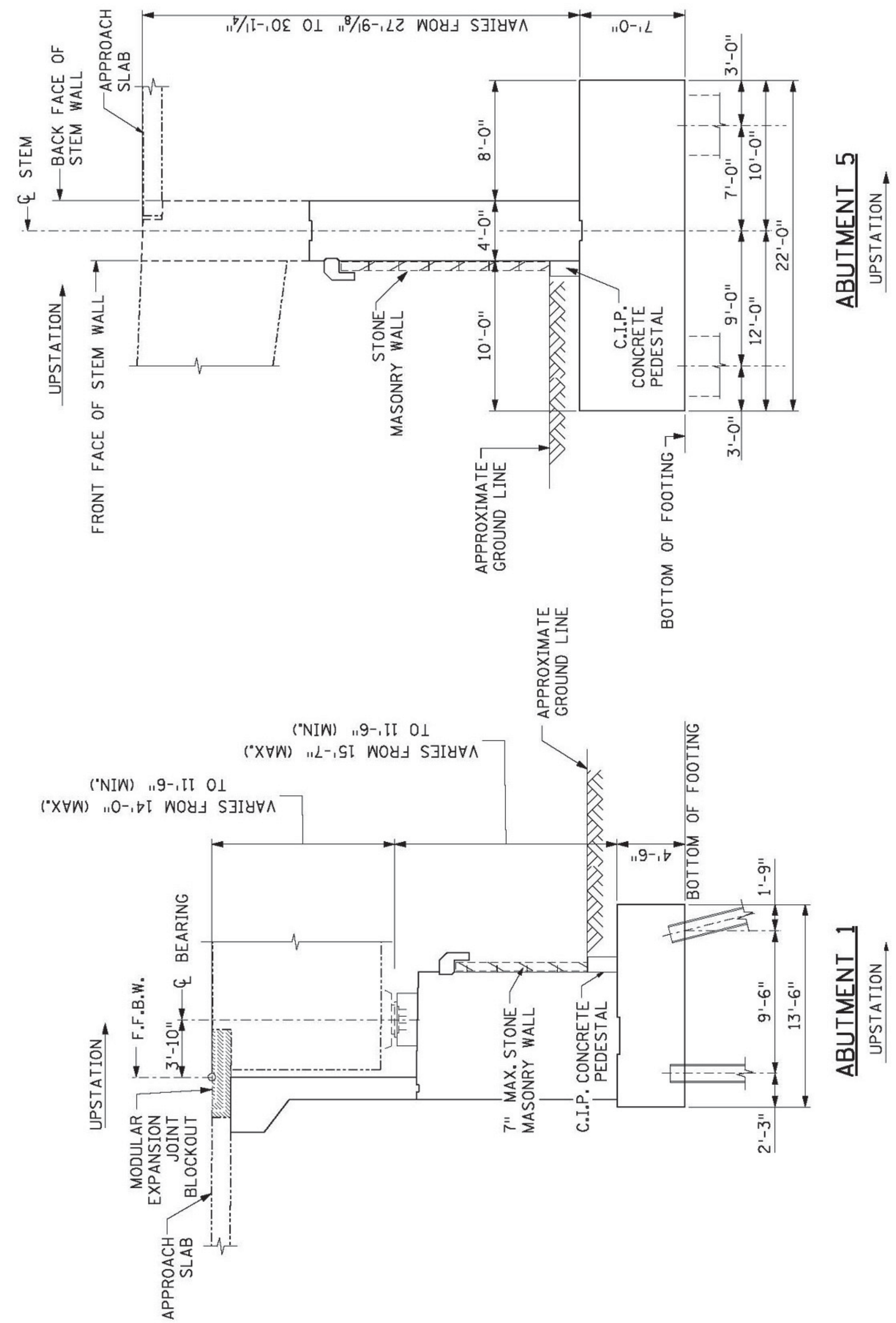


Figure 2.22 – Abutment Section View

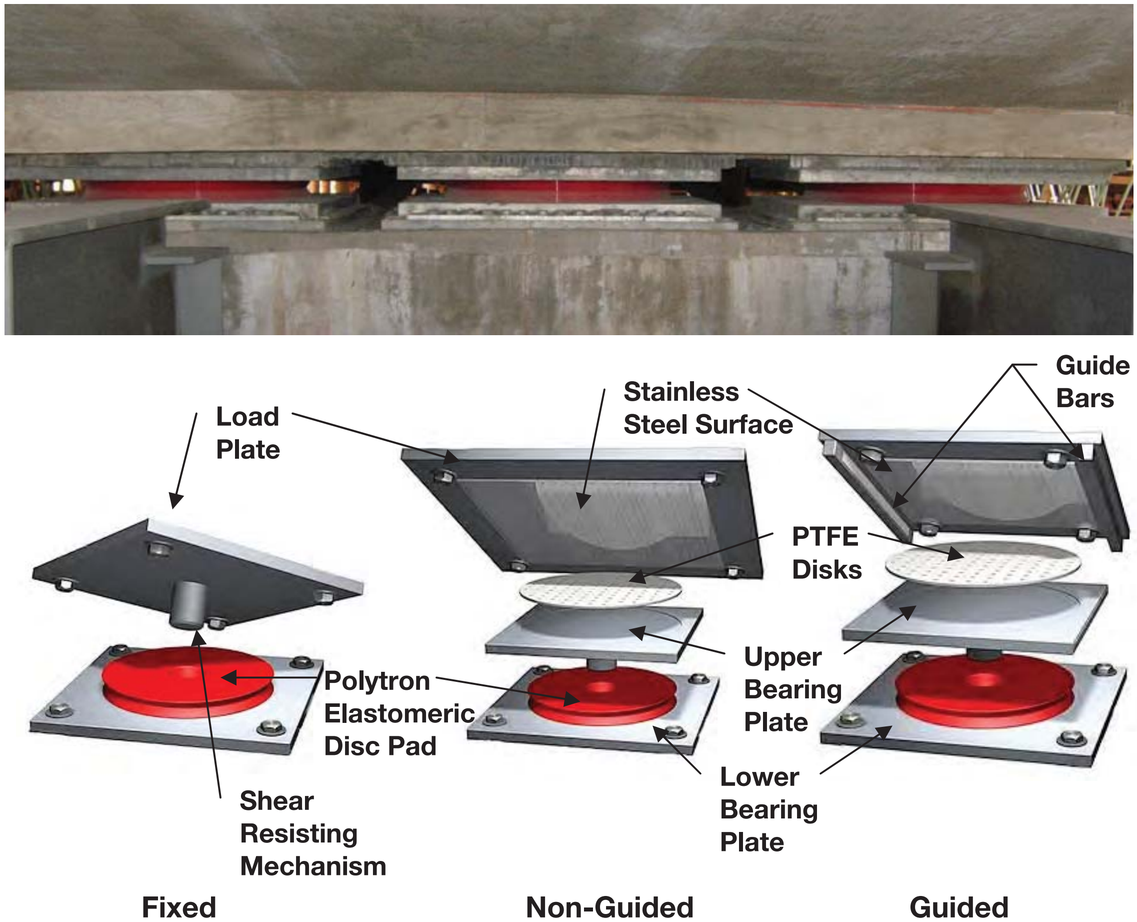


Figure 2.23 – Disktron© Bearings

2.3.2 Disktron© Bearings

With the exception of Abutment 5, all loads from the superstructure are transferred to the substructure through bearings. Non-guided and guided bearings allow for movement of the superstructure due to thermal expansion, thermal contraction, creep, and shrinkage of the concrete. Where fixed bearings are utilized, the flexibility of the substructure accommodates these superstructure movements. Figure 2.23 shows examples of the Disktron© bearings utilized on the St. Anthony Falls (I-35W) Bridge: fixed, non-guided and guided bearings.

All bearings for the St. Anthony Falls (I-35W) Bridge are Disktron© Bearings manufactured by RJ Watson, Inc. The major steel components of the bearing are the load plate, upper and lower bearing plates, the guide bars, shear resisting mechanism and Polytron© elastomeric disc pad, which gives the bearing its rotational capabilities. Expansion bearings also have PTFE disks and stainless steel surfaces to allow movement.

Should replacement of the bearing assembly be necessary, see Section 4.4.3 for the bearing replacement procedure.

2.3.2.1 Non-Guided Disktron© Bearings

The non-guided Disktron© bearings on the St. Anthony Falls (I-35W) Bridge utilize a PTFE (polytetrafluoroethylene) sheet and mirror finish stainless steel which results in a low coefficient of friction. The sheet is bonded to the upper bearing plate, providing a bearing surface for the stainless steel surface, which is welded to the load plate.

This arrangement allows unrestrained longitudinal and transverse translation between the superstructure and substructure along the sliding plane between the PTFE disk and the stainless steel surface. Non-guided Disktron© Bearings are used in combination with guided bearings to avoid binding that might be caused by having two guided bearings at a given support. Non-guided bearings are located at Pier 4 and at Abutment 1 along the outside webs of the box girders.

2.3.2.2 Guided Disktron© Bearings

Guided bearings differ from the non-guided bearings in that the load plate located on top of the upper bearing plate has guide bars that are attached to the load plate allowing for rotation in any direction and longitudinal displacement, but limiting displacement in the transverse direction. A PTFE disk and stainless steel surface is provided between the load plate (with guide bars) and upper bearing plate. These guided bearings are used at locations where required for superstructure longitudinal movement and where lateral loads are transferred to the substructure. Guided bearings are located at the same substructure locations as the non-guided bearings (Abutment 1 and Pier 4) but along the inside webs.

2.3.2.3 Fixed Disktron© Bearings

Fixed Disktron© bearings for this project are similar to the non-guided Disktron© bearings except that there is no sliding surface or upper bearing plate. These bearings restrain both longitudinal and transverse movement but allow rotation at the bearing. Fixed bearings are used at locations where longitudinal movements can be accommodated by the pier flexibility. Specifically, fixed bearings are used at Piers 2 and 3 in addition to the upstation side of Pier 4 for both the northbound and southbound bridge bearings.

2.3.3 Superstructure

The superstructure of each bridge consists of two box girders joined at the deck, a post-tensioning system, health-monitoring system and provisions for future post-tensioning system in the main-span unit.

2.3.3.1 Box Girder

The superstructure is comprised of two units; Spans 1 through 3 comprise the Main-Span Unit over the Mississippi River valley, and Span 4 is a single span over a transportation corridor on the northern bluff. The main span unit is comprised of two single-cell box girders that are each a variable depth, cast-in-place (Spans 1 and 3) and precast segmental (Span 2), post-tensioned concrete box girder, which together form a total deck width of 90'-4". The box girder cross-section varies in depth from a minimum of 11'-2 1/2" at midspan, Abutment 1 and Pier 4, to a maximum of 25'-2 1/2" at Piers 2 and 3. The main span unit structure is 3 continuous spans with expansion joint devices located at Abutment 1 and Pier 4.

Span 4 is comprised of two box girders that are each a multi-cell variable depth, cast-in-place, post-tensioned concrete box girder that together form a total deck width of 90'-4" that widens through Span 4NB by approximately 15'-7" to accommodate an off ramp at grade. The box girder cross-section for Span 4 has a minimum depth at midspan of 6'-0" with an 11'-2 1/2" deep section at Pier 4 and 9'-8" deep section at Abutment 5. Span 4 is a single span with a fixed bearing at Pier 4 and an integral monolithic superstructure/abutment at Abutment 5.

Additional description of this superstructure was given in Section 2.2.

In addition to the typical box girder section, deviation diaphragms, pier diaphragms, and expansion joint diaphragms are also included in the bridge superstructure. Deviation diaphragms (Figures 2.24 and 2.25) provide a location to change the path for the external longitudinal prestressing tendon geometry. Ribs at the bottom corners of the segment transfer forces from the longitudinal prestressing tendons into the webs of the box girder.

Pier diaphragms (Figure 2.26) are located at Piers 2 and 3 and rest on fixed Disktron© bearings. These diaphragms were cast-in-place and contain anchorages for the longitudinal draped tendons. Diaphragms within the box girder transfer shear and torsion forces to the bearings. The pier diaphragms are post-tensioned transversely with bars and are also mildly reinforced.

Expansion Joint diaphragms (Figure 2.27) are located at Abutment 1 and Pier 4 of the superstructure and feature blockouts to accommodate the modular expansion joints. Diaphragms within the expansion joint box girder contain anchorages for the longitudinal prestressing tendons and are used to transfer shear and torsion forces to the bearings. Expansion Joint segments are post-tensioned vertically with bars as well as mildly reinforced.

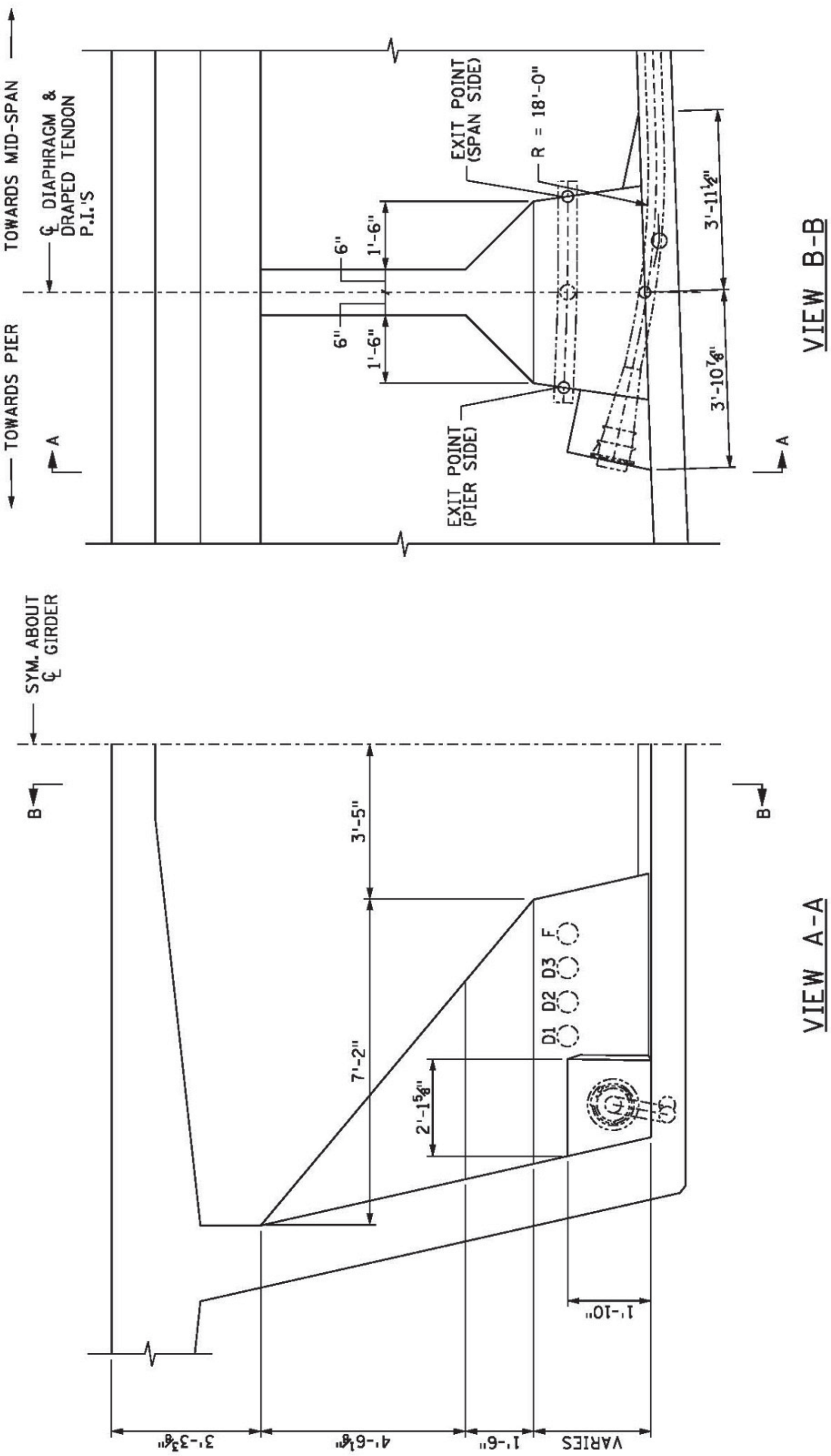


Figure 2.24 – Precast Span Deviation Diagramm

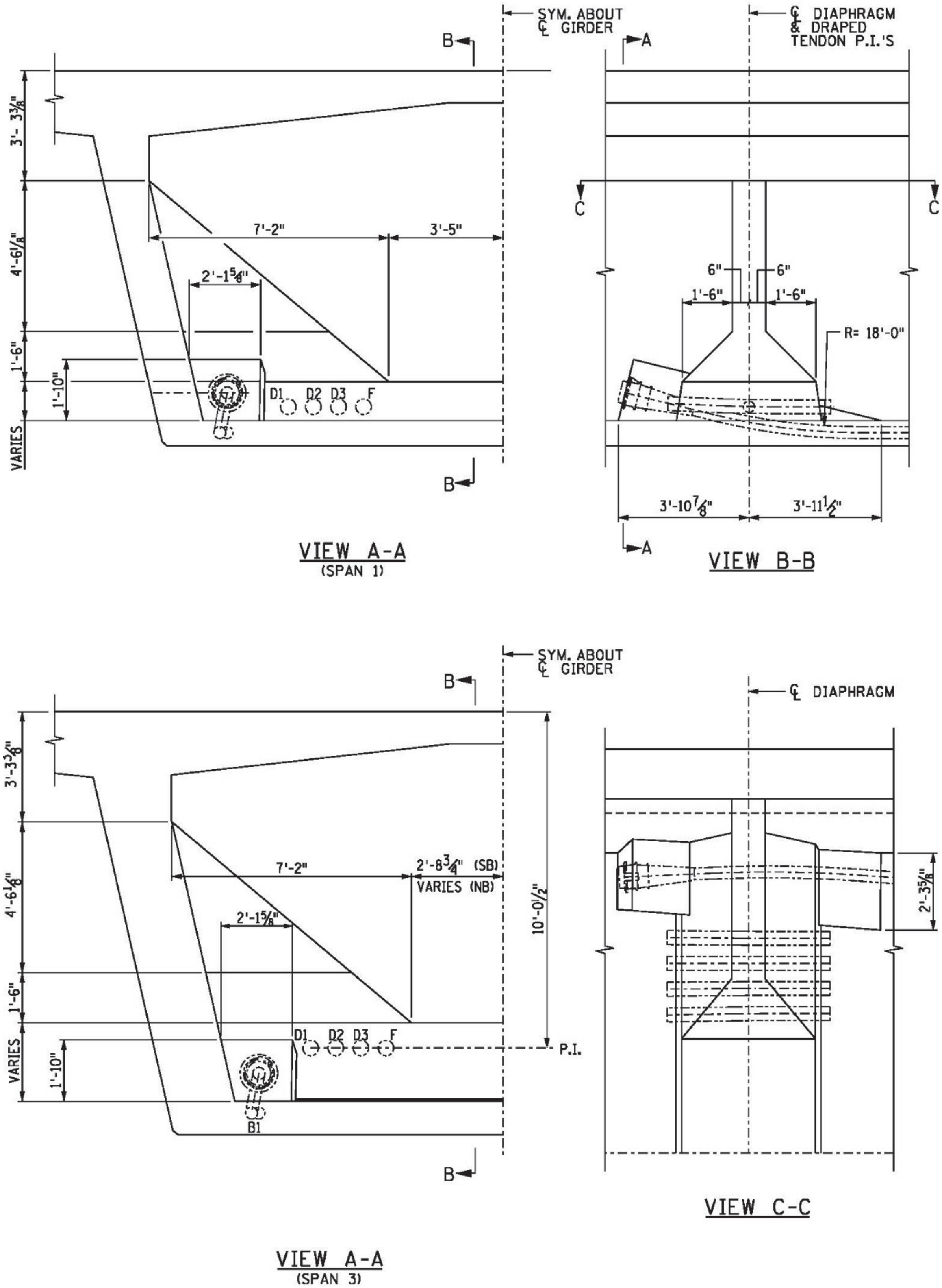


Figure 2.25 – CIP Span Deviation Diaphragm

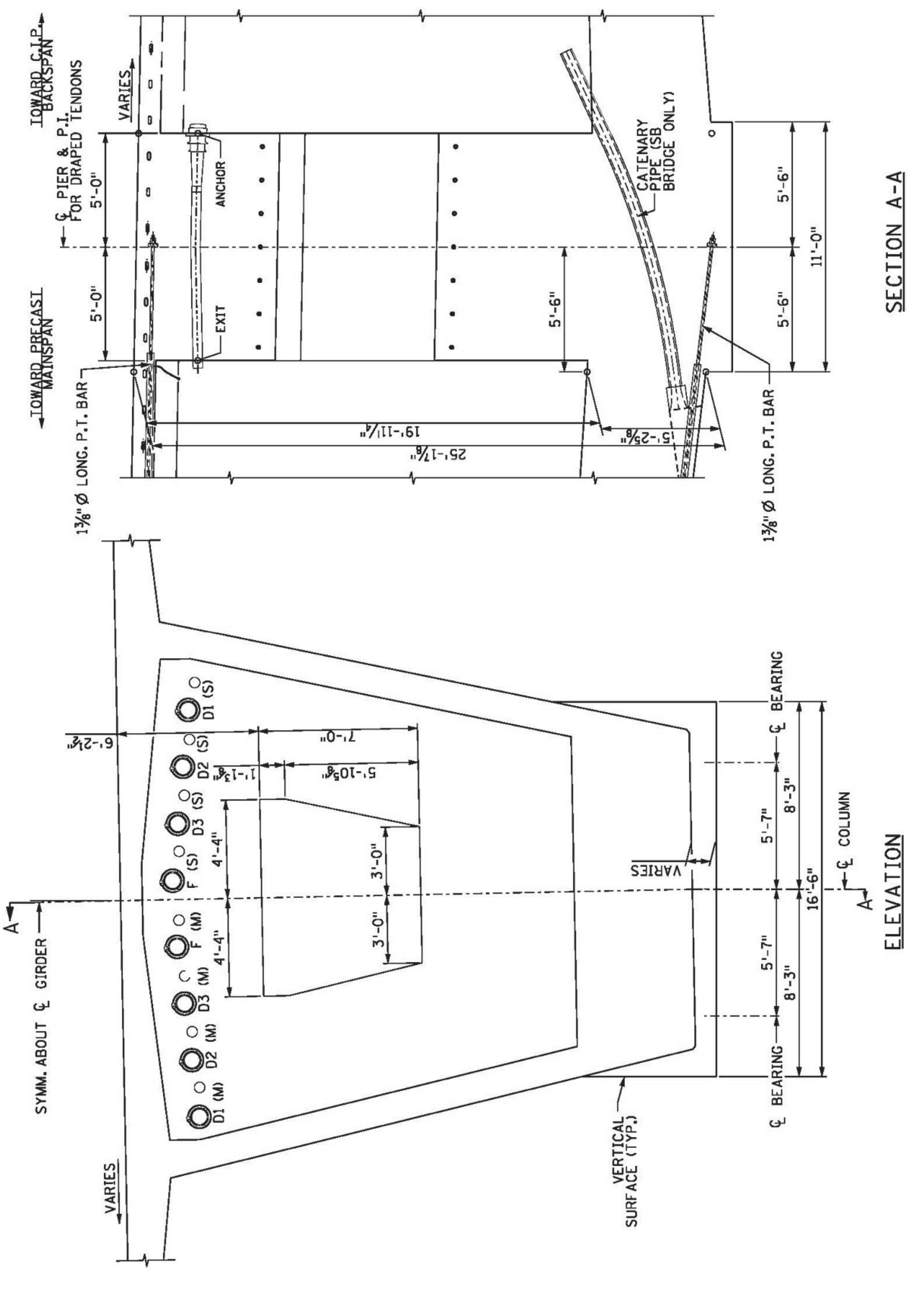


Figure 2.26 – Pier Diaphragm

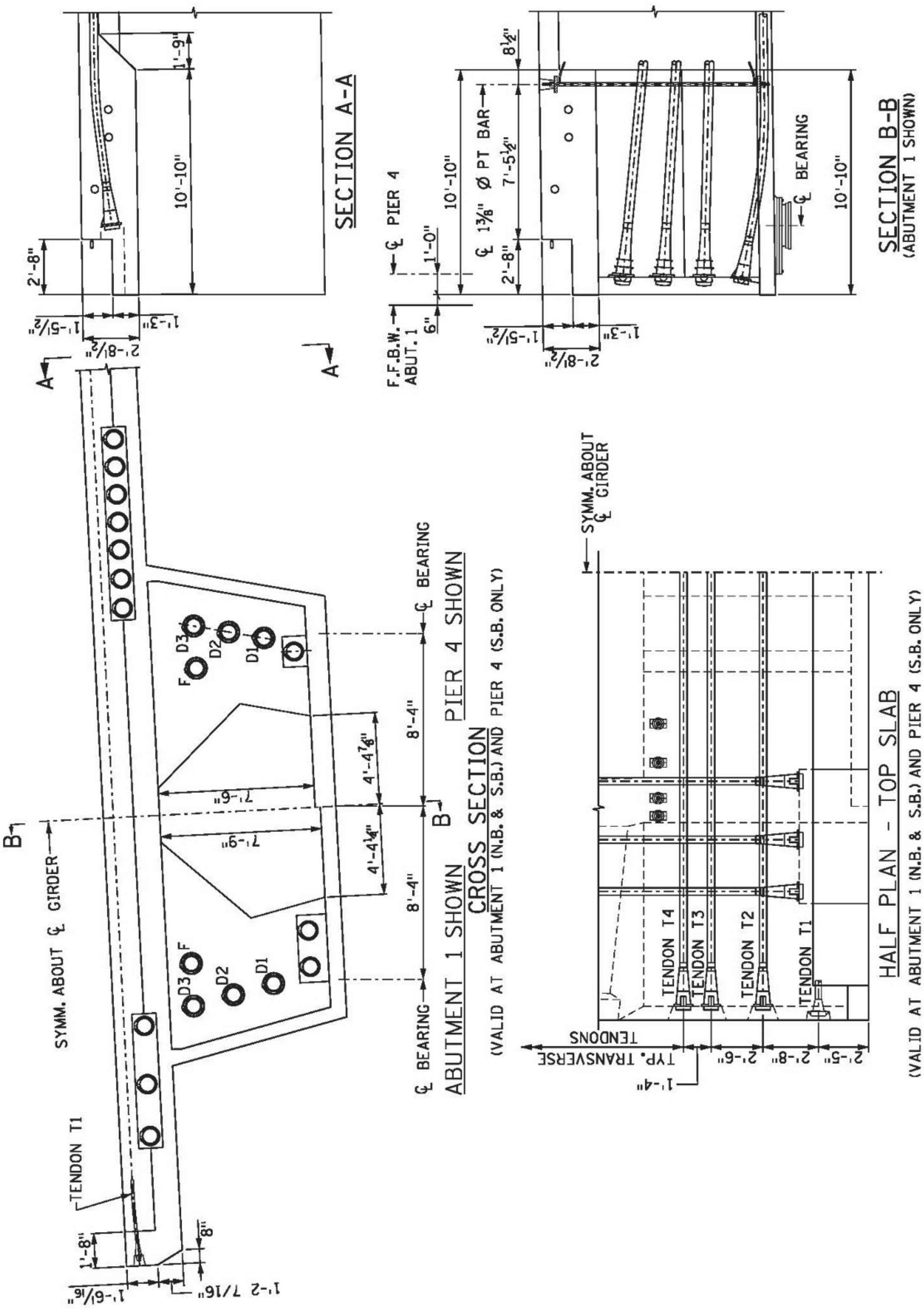


Figure 2.27 – Expansion Joint Diagram

2.3.3.2 Post-Tensioning System

Post-tensioning is used to apply a compressive force to the concrete. This compression serves to counteract tensile stresses in the concrete due to self-weight and external loads, thus keeping the concrete in compression. Post-tensioning also serves as the primary reinforcement in case the bridge is loaded beyond its service capacity. The St. Anthony Falls (I-35W) Bridge was designed using the Load and Resistance Factor Design (LRFD) method in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications, 4th Edition, 2007 and MnDOT specification and special provisions developed specifically for this project.

Post-tensioning is provided either by tendons, consisting of multiple seven-wire strands, or post-tensioning bars. Some tendons are internal to the concrete section, and some are external to the concrete, protected by high-density polyethylene ducts located inside the box girder cell. Bars are always internal to the concrete section. Ducts cast into segments provide for installation of internal bars and tendons.

Anchorages at the ends of both internal and external tendons transfer tendon forces to the concrete. Steel wedges secure the individual strands to the anchor head, which bears on a bearing plate cast into the concrete. The anchorage for a PT bar is similar except that a nut on the threaded post-tensioning bar transfers bar force to the bearing plate. Tendon and post-tensioning bar anchorage details are shown in Figure 2.28.

After post-tensioning is stressed, grout is injected into the ducts to provide corrosion protection and to bond the tendon or bar to surrounding concrete. External tendons are enclosed in polyethylene ducts, which are injected with grout to provide additional corrosion protection. Anchorages for top slab, bottom slab and draped tendons at the expansion joint diaphragms (near girder ends) are capped, grouted, and then capped with an epoxy grout cap, which is sealed with an elastomeric membrane (Figure 2.29).

The St. Anthony Falls (I-35W) Bridge is post-tensioned longitudinally and transversely. Longitudinal post-tensioning is comprised of cantilever, draped, and bottom slab tendons. Plan views of cantilever and bottom slab tendons in addition to an elevation view of the draped tendons are provided in Figures 2.32 through 2.37. Cantilever tendons run internal to the deck and groups of these tendons are centered over the main piers. Draped tendons run external to the concrete section, but inside the box girder. These tendons anchor in the pier and expansion diaphragms, extend down to deviator diaphragms, low across the center of each span and back up to anchor in the next diaphragm. Bottom slab tendons run internal to the bottom slab. Groups of these tendons are centered on each interior span and located near the expansion joint of Spans 1 and 3, and at the mid-span of Span 2.

Transverse post-tensioning for the bridge is internal to the deck and consists of a typical 4 x 0.6” strand transverse tendon, spaced nominally at 2’-0” on center along the precast segments and cast-in-place box girders (Figure 2.30 & 2.31). An additional single strand tendon, anchoring at the interior web/top slab interface, is placed between adjacent segments 9 through 15 and the mid-span closure across the longitudinal cast-

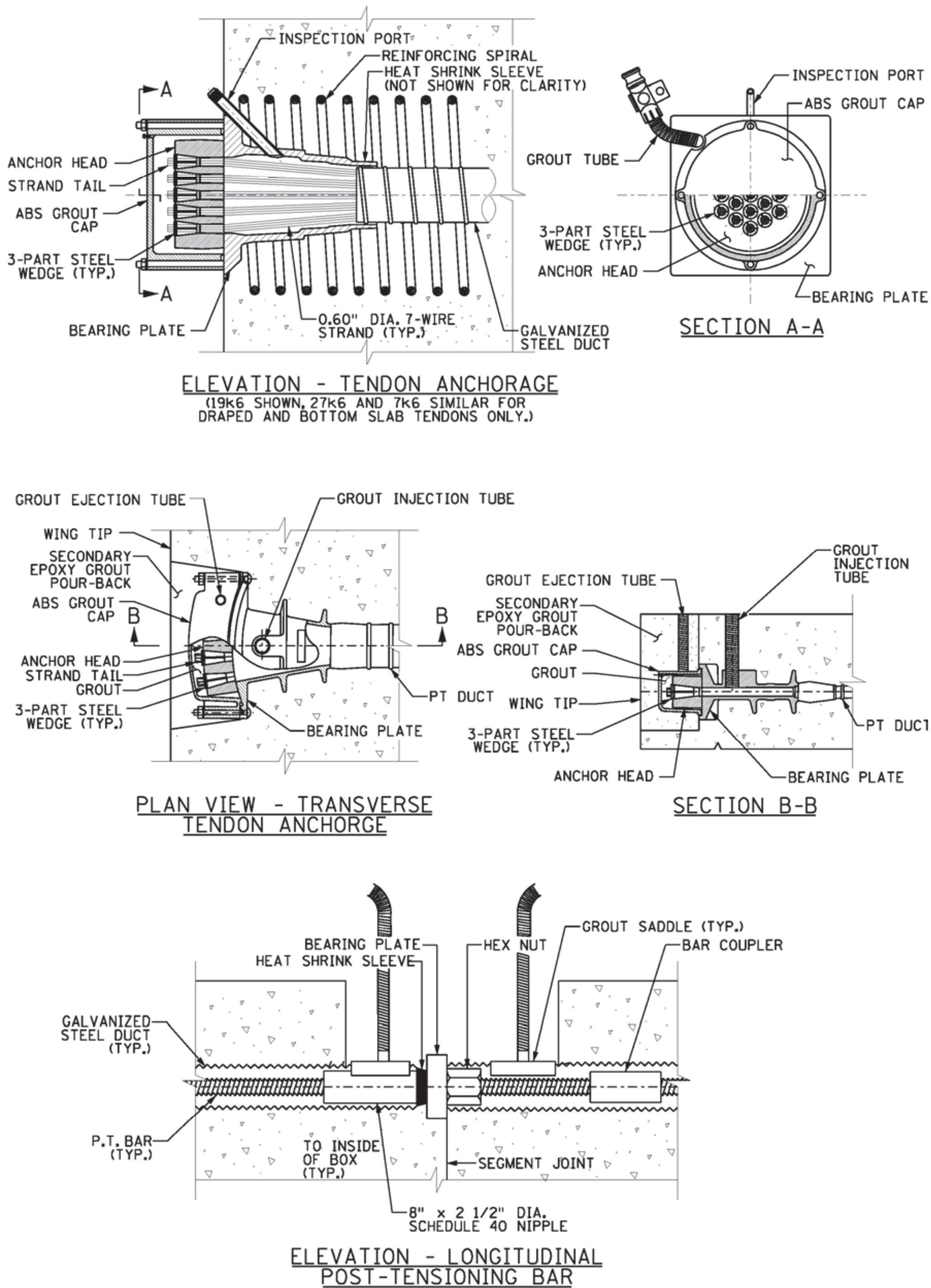


Figure 2.28 – Anchorage Details

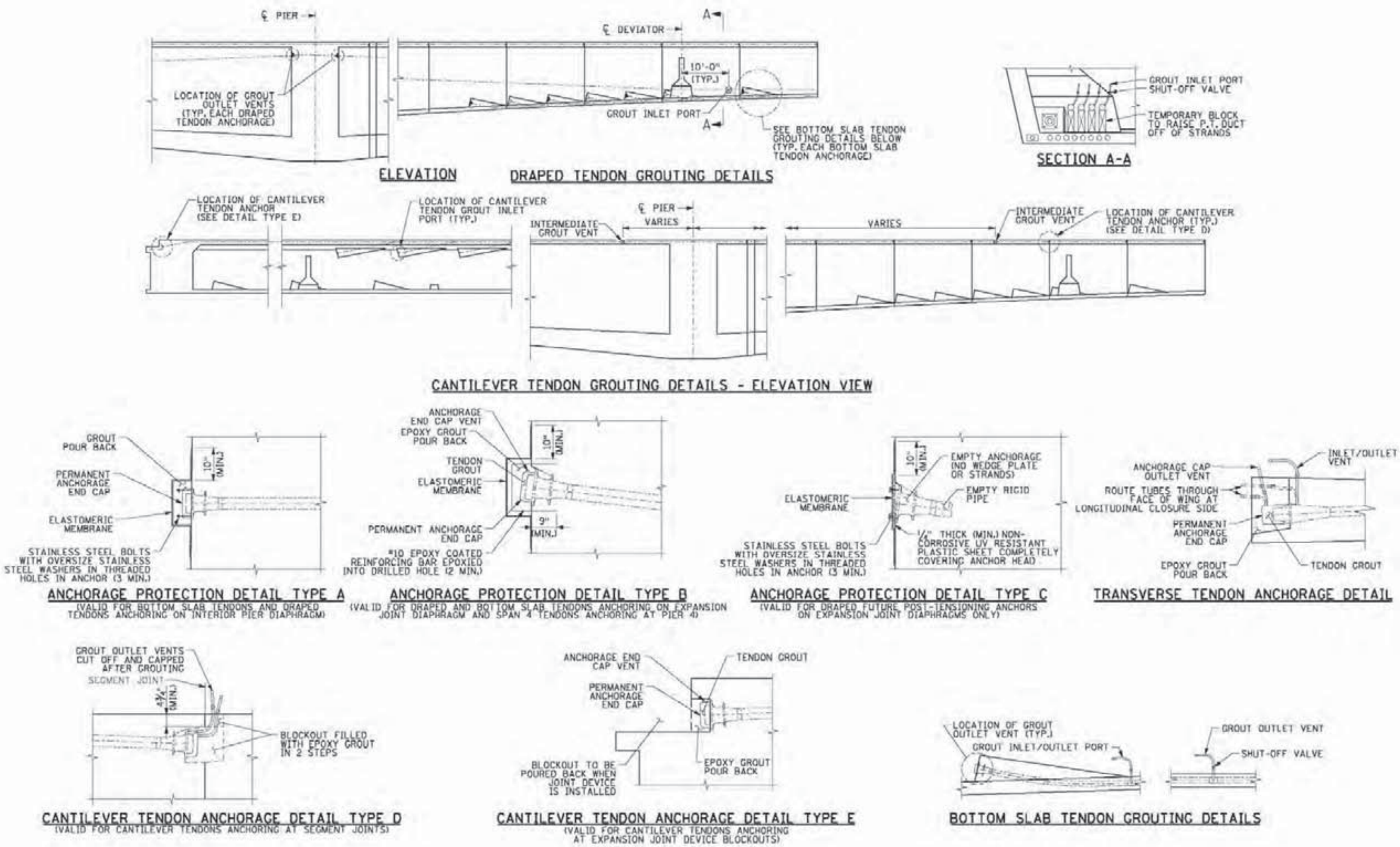


Figure 2.29 – Anchorage Protection Details

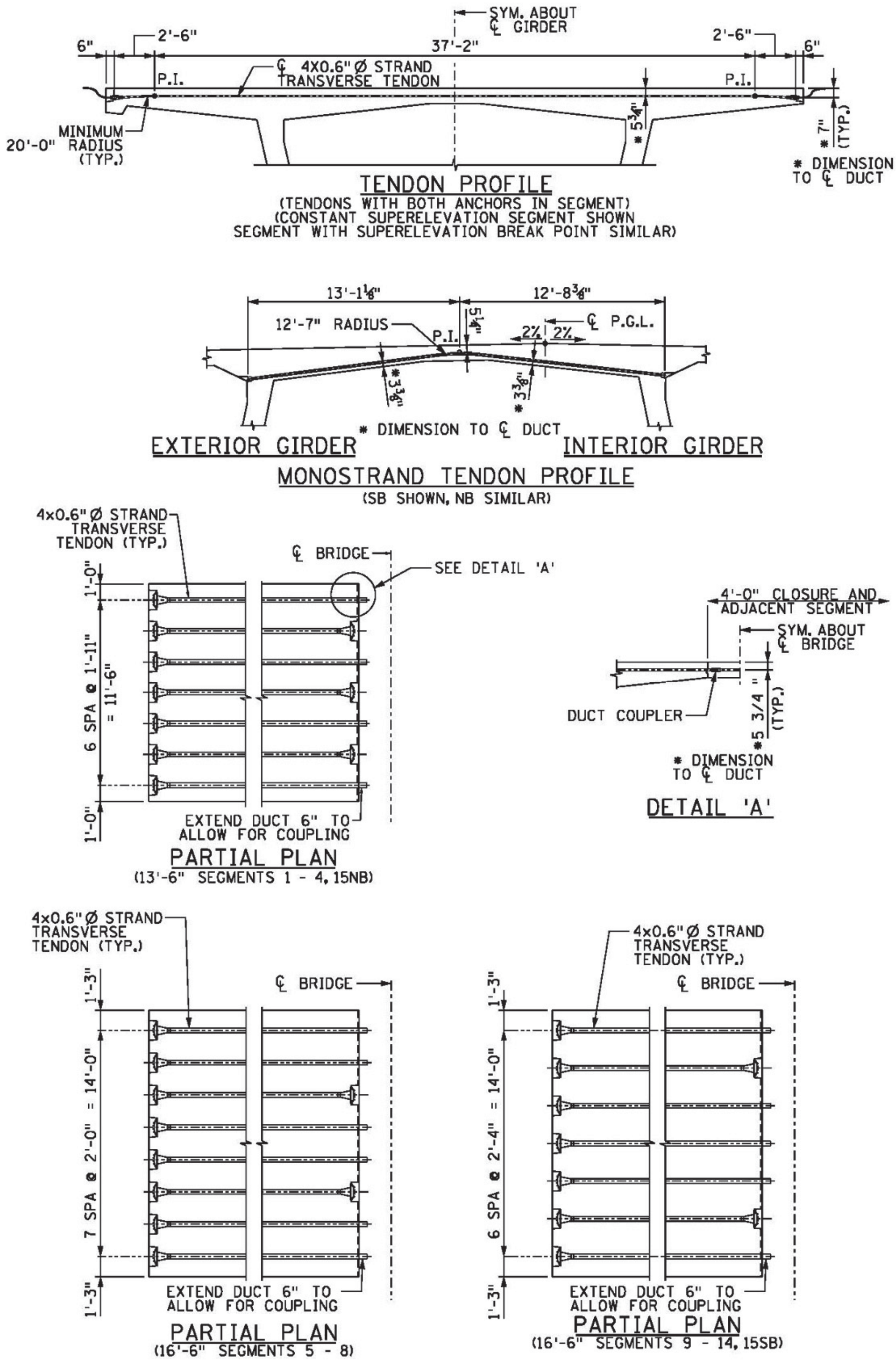


Figure 2.30 – Transverse Post-Tensioning 1

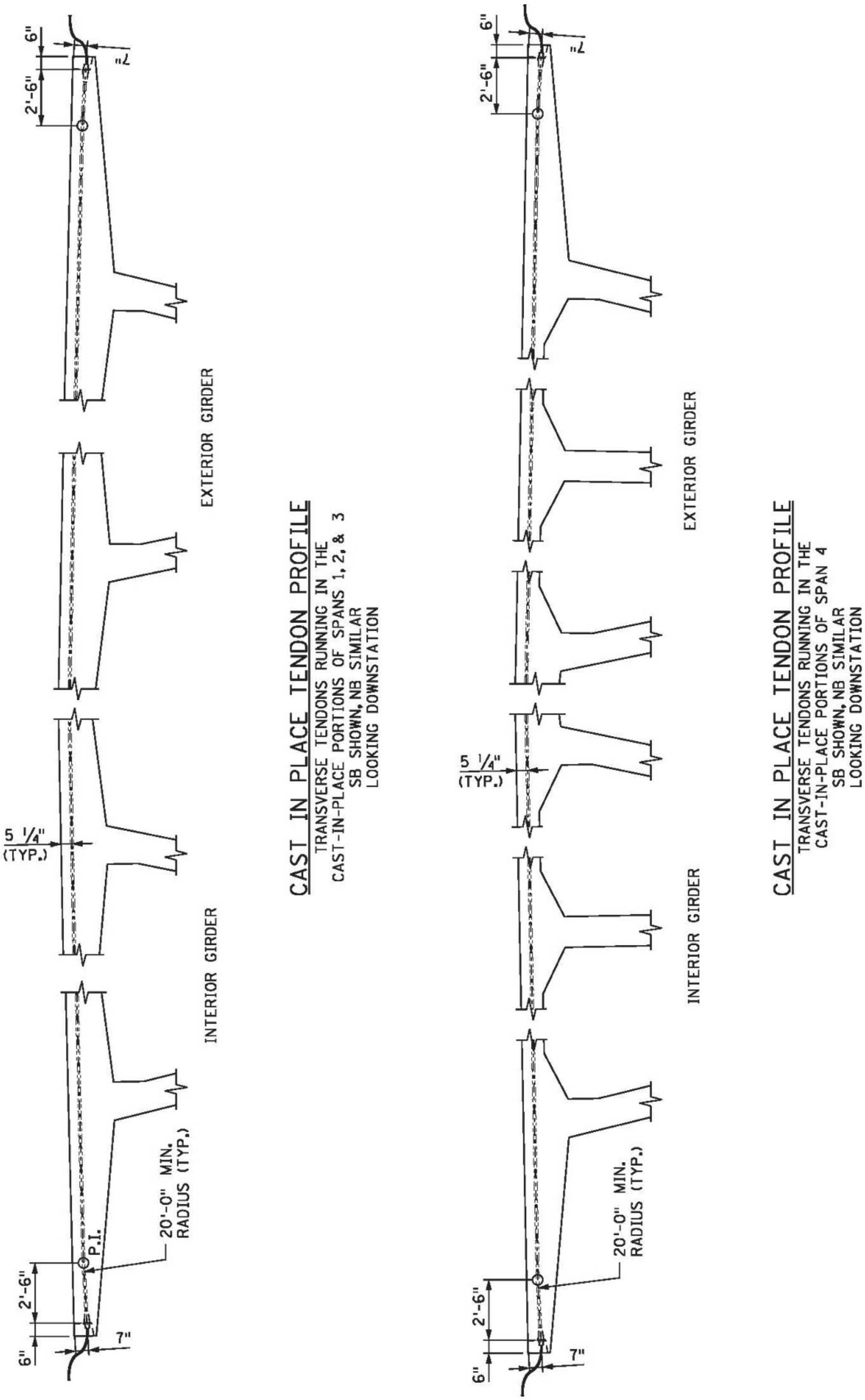


Figure 2.31 – Transverse Post-Tensioning 2

in-place closure in Span 2. Expansion joint diaphragm transverse post-tensioning consists of three 19 x 0.6” strand tendons and one 4 x 0.6” strand tendon (Figure 2.27) plus an additional 4 x 0.6” strand tendon in the expansion joint blockout. One 4 x 0.6” strand tendon is utilized at cast-in-place Pier-Closures and three 4 x 0.6” strand tendons are utilized at the Mid-Span Closure Segments.

For erection of the Span 2 segments, permanent post-tensioning bars were used to attach segments to the cantilever prior to installing the cantilever tendons. Post-tensioning bars also served to provide the necessary pressure to tightly join the segments and squeeze out excess epoxy. Six 1 3/8” diameter bars are located in the deck and typically two (sometimes four) 1 3/8” diameter bars are located in the bottom slab.

2.3.3.3 Future Post-Tensioning System

Provisions for additional future post-tensioning tendons have been included in the design for Spans 1 through 3. This future post-tensioning is to provide additional compression, if desired, during the life of the bridge to compensate for higher design loads, a damaged tendon, or for other reasons.

The future tendons are 27 x 0.6” diameter strand draped external tendons that anchor high in the pier and expansion joint diaphragms, run down to the deviator segments, and low across the center of each span. It is recommended that polyethylene ducts be used around external portions of the tendons, similar to the draped tendons. The anchorage hardware (manufactured by DSL) and deviation pipes are already in place. The tendons need only to be installed, stressed, and grouted.

2.3.3.4 Future Pedestrian Bridge System

Provisions for a Pedestrian Bridge have been included in the Southbound Structure. Hanger locations have been pre-planned and sleeves installed in the superstructure segments. Catenary cable guide pipes in the SB Piers 2 and 3 pier diaphragms were also provided to accommodate a future anchorage location. Refer to the as-built plans for exact locations of the hanger sleeves and catenary guide pipes in the Southbound structure box girders.

2.3.4 Expansion Joint Devices

Two modular type expansion joints manufactured by D.S. Brown were used on the St. Anthony Falls (I-35W) Bridge, one at Abutment 1 between the superstructure and abutment backwall and one at Pier 4, between Span 3 and Span 4.

Expansion joint devices span over the discontinuities in the bridge deck, providing support for wheel loads across joint openings while allowing the bridge to expand and contract. The joints are sealed to direct any residual deck drainage away from the bearings and abutment seat. Figure 2.38 shows the expansion joint device.

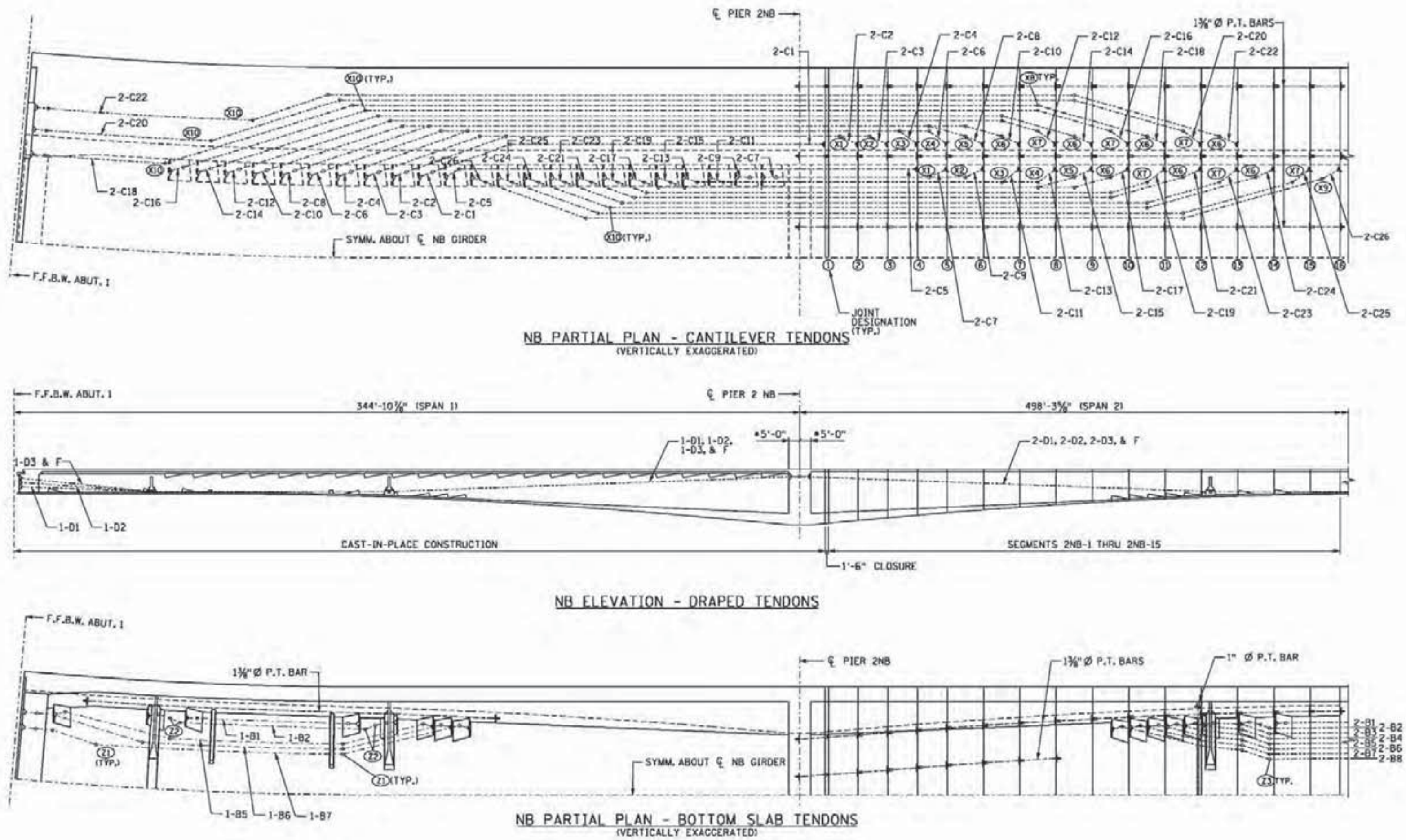


Figure 2.32 – NB Bridge Post-Tensioning Layout 1

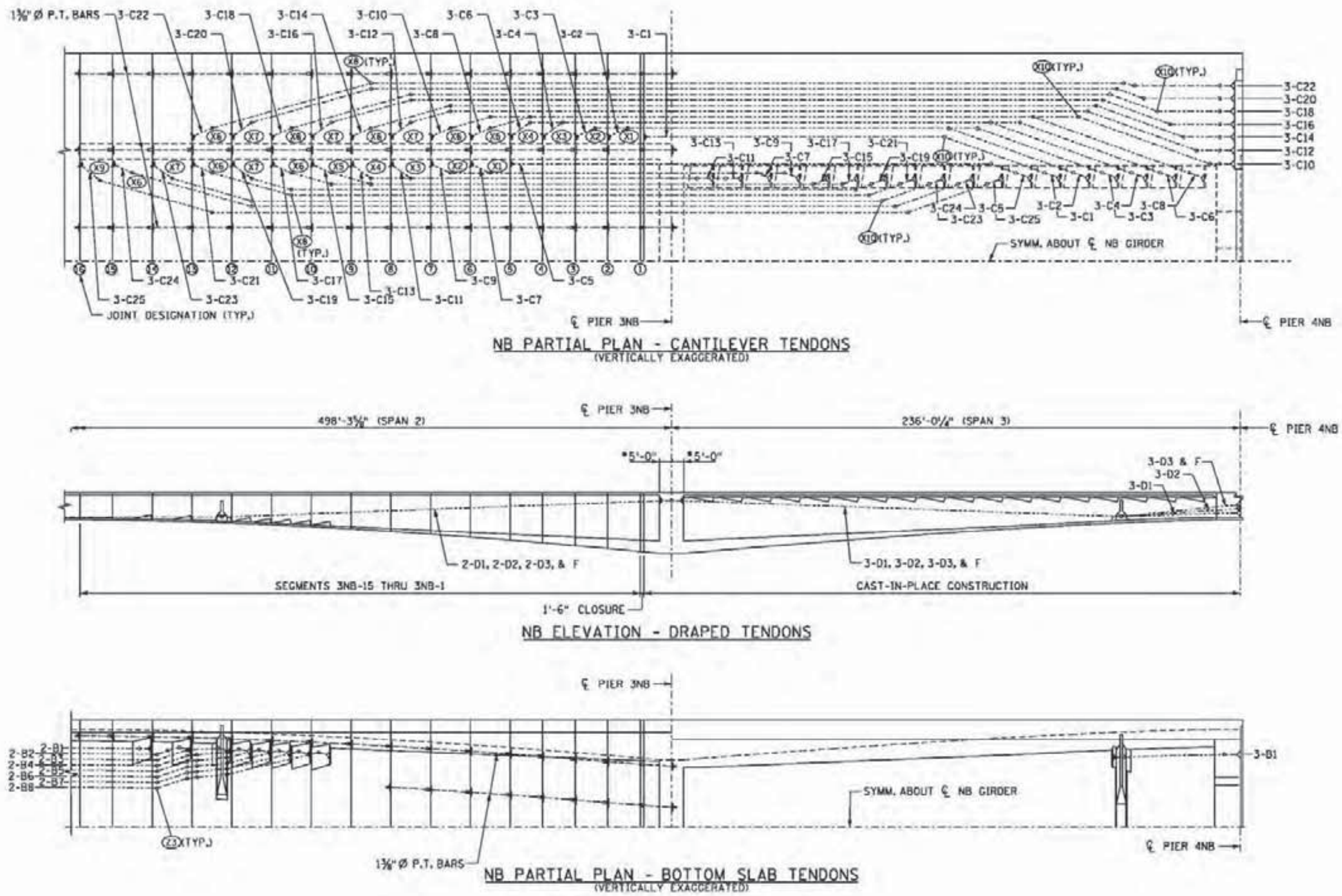


Figure 2.33 – NB Bridge Post-Tensioning Layout 2

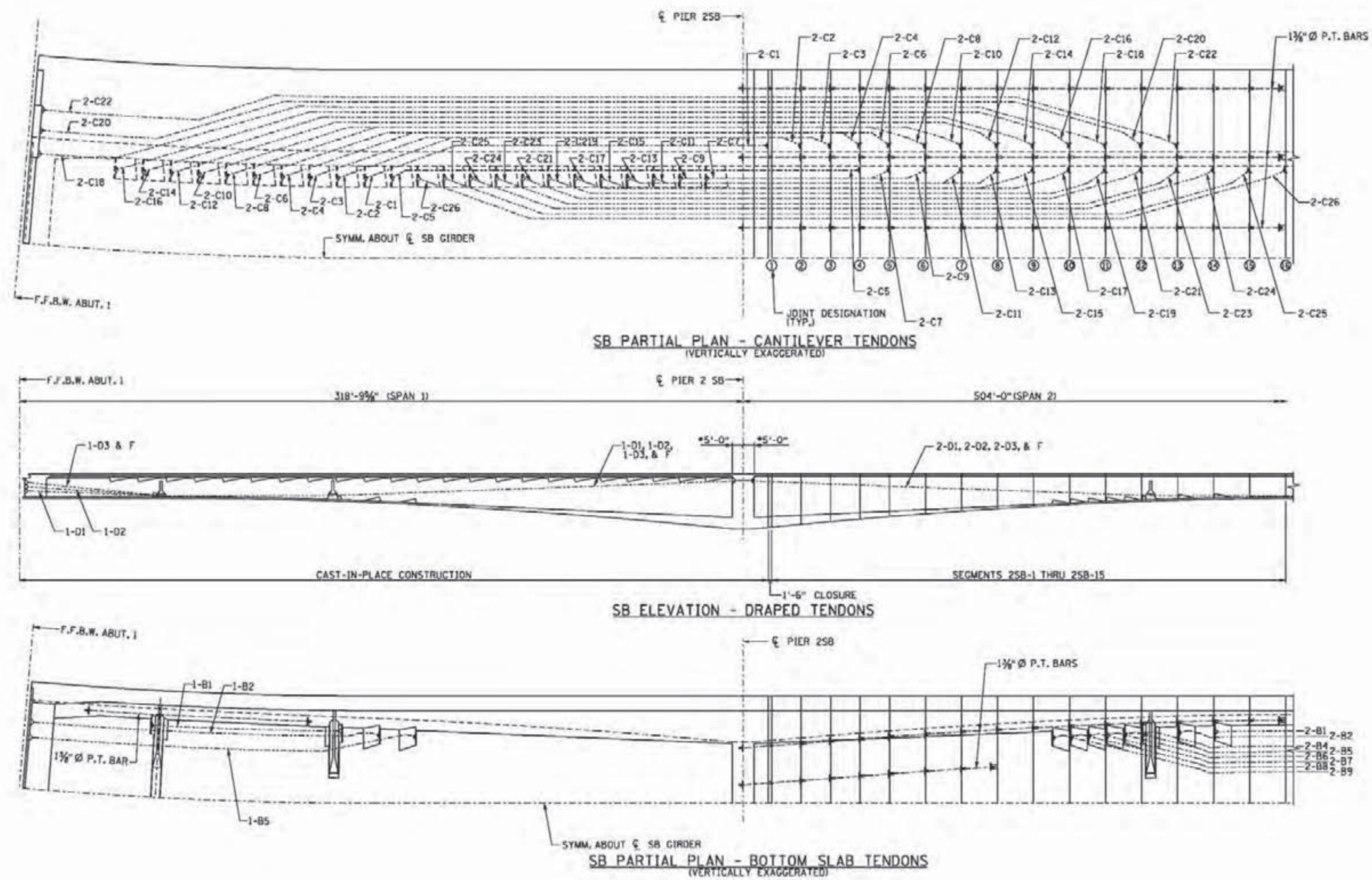


Figure 2.34 – SB Bridge Post-Tensioning Layout 1

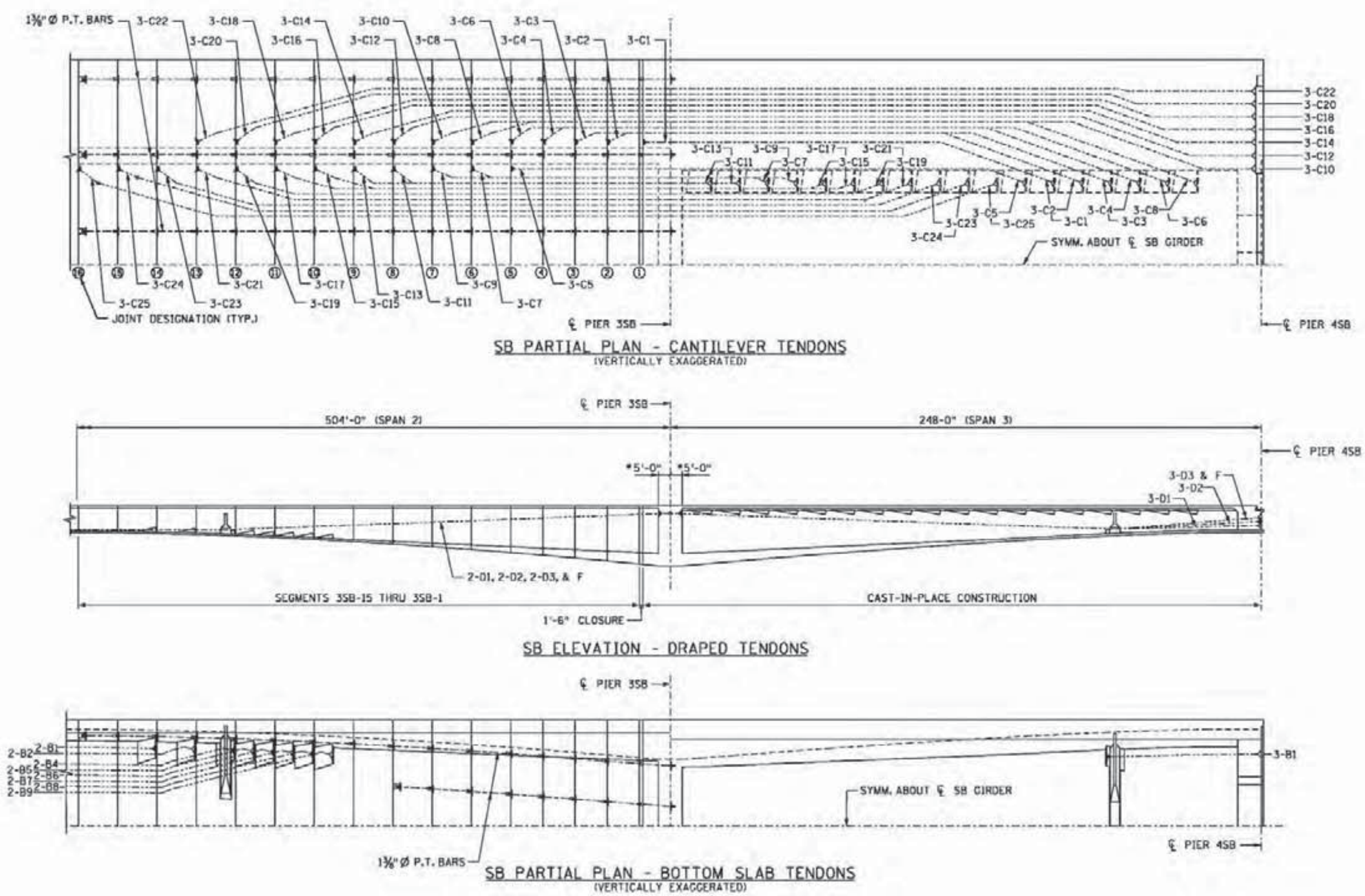


Figure 2.35 – SB Bridge Post-Tensioning Layout 2

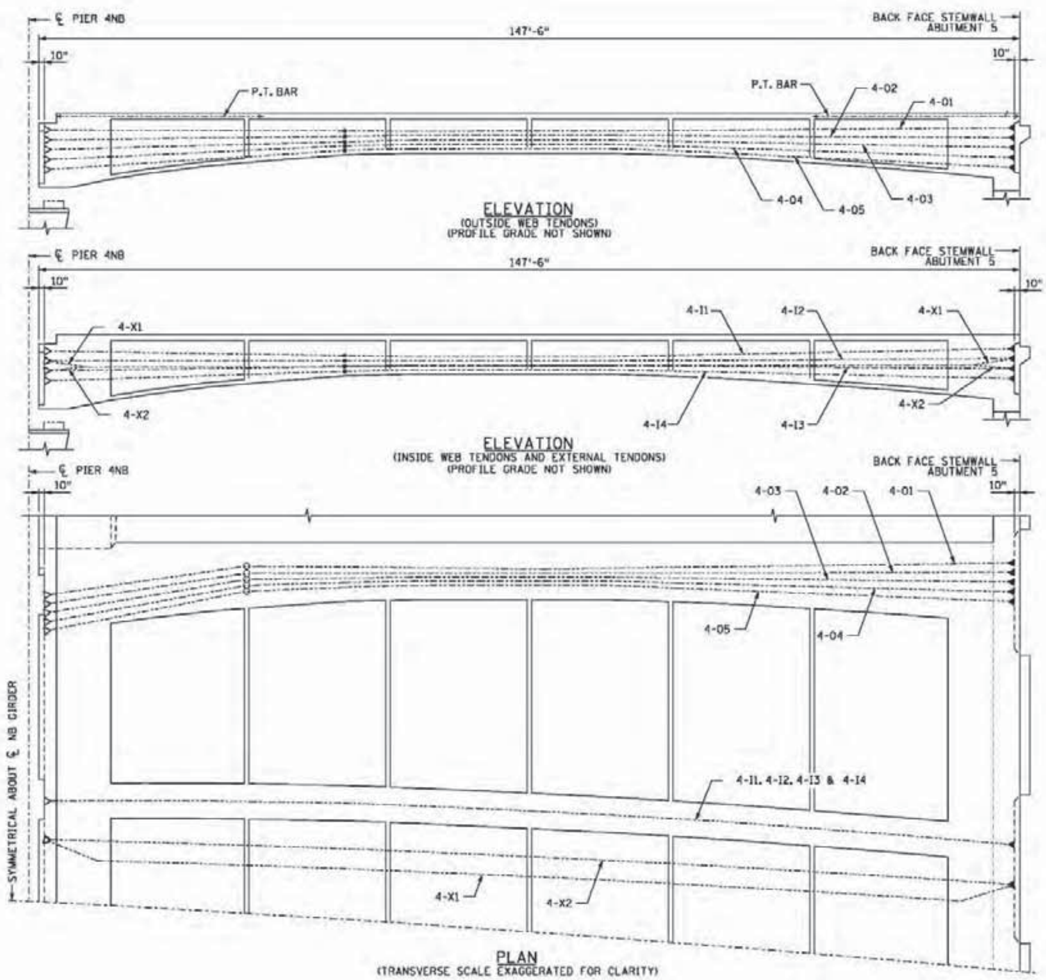


Figure 2.36 – Span 4NB Post Tensioning Layout

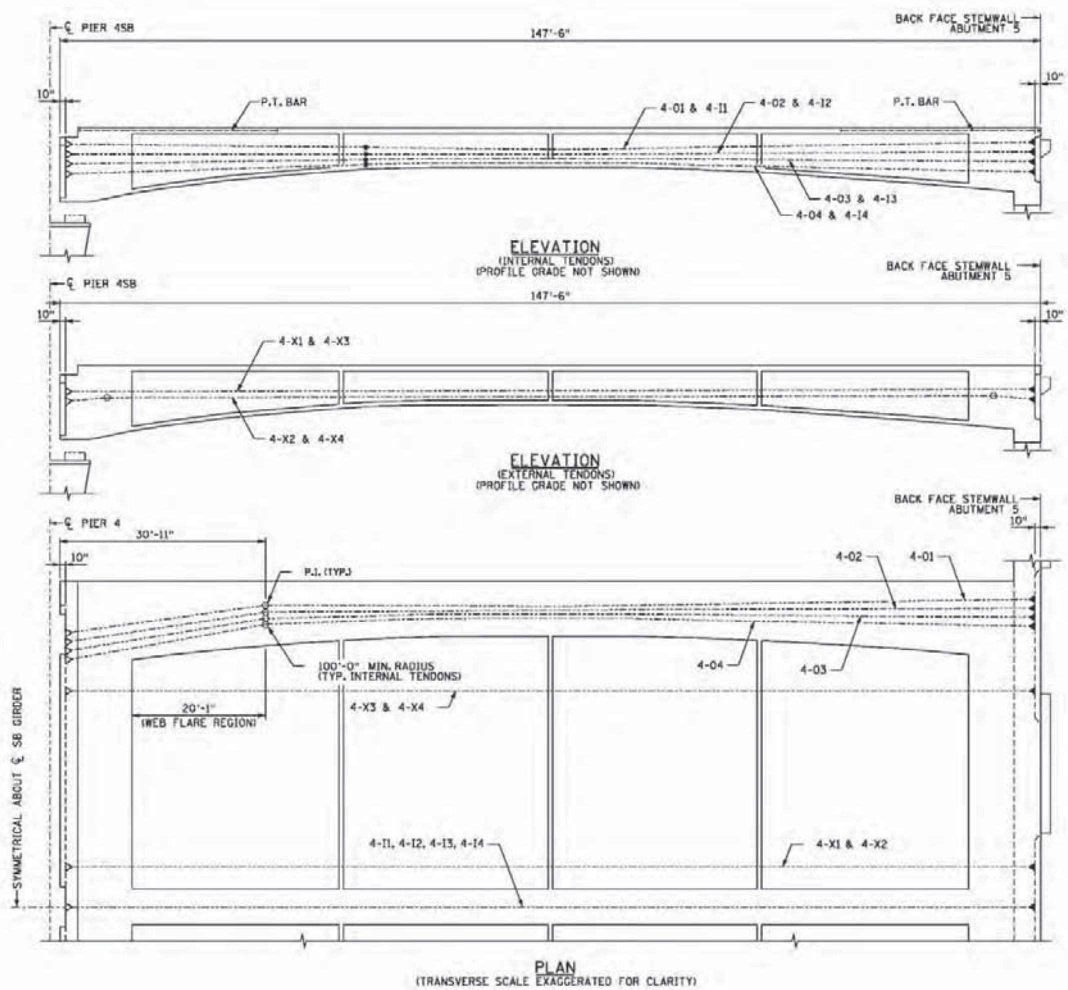


Figure 2.37 – Span 4SB Post-Tensioning Layout

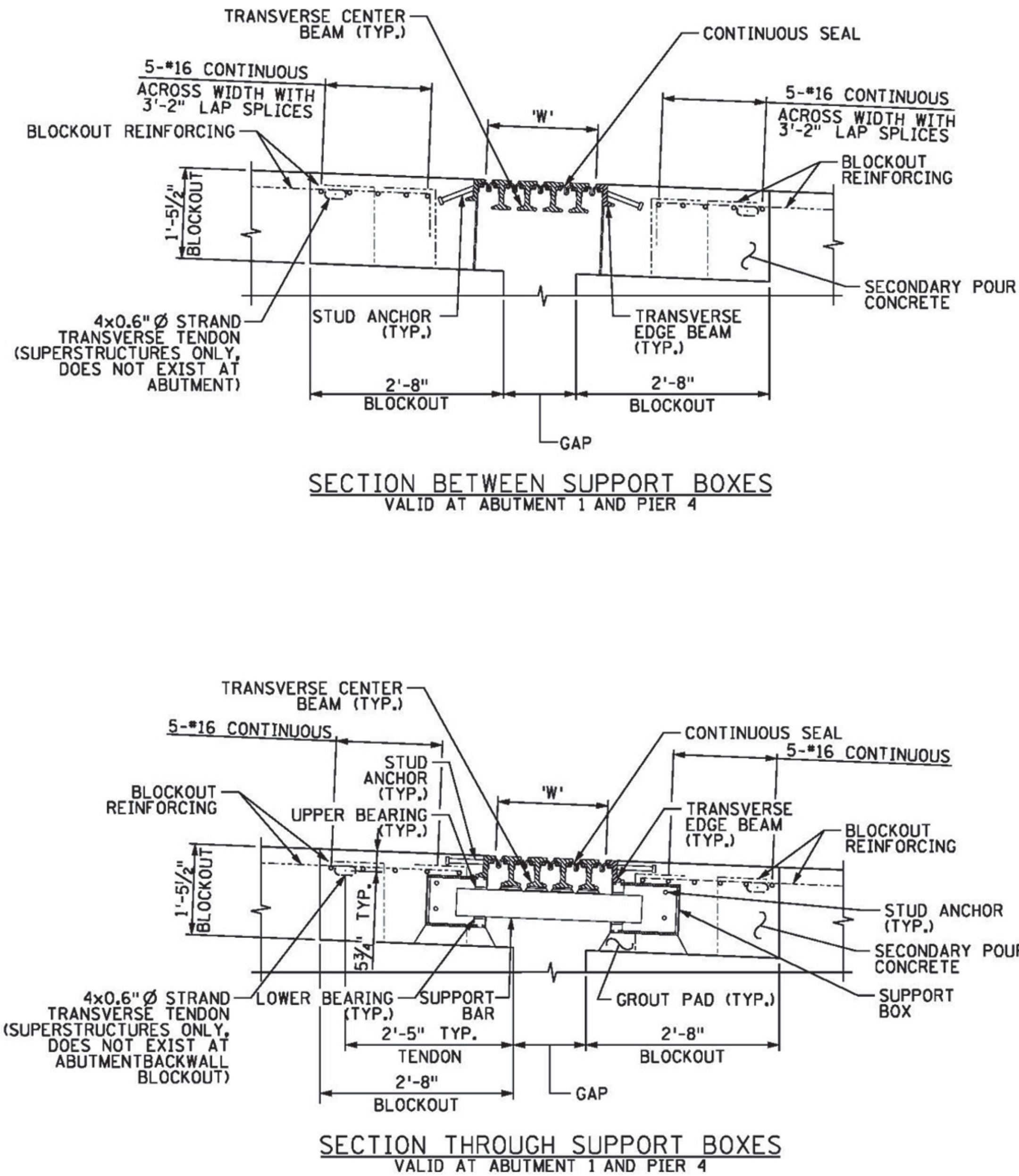


Figure 2.38 – Expansion Joint Details

2.3.5 Integral Wearing Surface

A 2 1/2”-thick wearing surface was cast integral to the top slab. An integral wearing surface further protects the deck against abrasion and chloride intrusion. To optimize the ride of the bridge, the contractor initially ground the finished deck, typically not more than 1/2”. In Appendix A are the results from the radar survey performed by AET to determine the final concrete cover in the deck at strategic locations.

Degadeck® Plus was applied to the entire bridge deck as a flood coat using brooms and squeegees. Areas around anti-icing puck blockouts and bridge drains were pretreated to prevent the material from entering these systems. Mixed batches were poured onto the deck and worked into the grooves left from grinding operations. Dry, clean, sand was broadcast using broadcast spreaders into the wet, uncured resin. The material was allowed to fully cure before construction and live traffic was allowed to cross over applied areas.

2.3.6 Approach Slabs

Approach slabs are provided at the ends of the bridge to improve vehicle ride quality onto and off of the bridge that may otherwise be compromised due to potential roadway settlement before the bridge. The approach slabs are 20’-0” long and match the profile grade of the bridge.

2.3.7 Bridge Lighting System

In addition to the roadway lighting on the St. Anthony Falls (I-35W) Bridge, the project also includes unique lighting arrangements that add to the visual quality of the structure. The lighting enhancements include LED lighting along the exterior girder wings to “wash light” the wings and box girder webs. The piers are lighted with visual “up lighting” from the plazas up toward the box girders. Gateway Monuments at the ends of the structure also incorporate “up lighting” to make for a unique, nighttime visual experience for individuals crossing the bridge. The roadway lighting is pole mounted LEDs, a first for a major US Bridge. Expanded information for the light fixtures and their locations is outlined in Chapter 4 (Section 4.7) and Appendix D.

2.3.8 Health Monitoring System

The St. Anthony Falls (I-35W) Bridge incorporates an extensive network of sensors, monitoring equipment and gauges. The Health Monitoring System is monitored by the Minnesota Department of Transportation and the University of Minnesota. The system is intended to provide long-term structural behavior monitoring to allow for enhanced analysis of the long-term health of the structure. See Appendix D and the contract plans for specific details on the Health Monitoring System. Included in the health monitoring system is a bridge security system that will alert the monitoring station if unauthorized entrance to the bridge occurs. The system also includes a closed-circuit video monitoring capabilities.

2.3.9 Anti-Icing System

The St. Anthony Falls (I-35W) Bridge has an integral anti-icing system. This automated system (by Boschung) will distribute an anti-icing fluid over the bridge deck and approach roadways to prevent ice from forming when the system sensors indicate the potential for ice exists. Distribution points are located along the bridge deck flush with the roadway. These “pucks” are spray heads that spread the fluid onto the bridge deck. A network of piping is located inside the box girder to distribute the fluid from the pump house to the spray heads. See Appendix D and the Boschung as-built plans for specific details on the Anti-Icing System.

2.4 Construction Modifications

Although the St. Anthony Falls (I-35W) Bridge was predominantly built in accordance with the released-for-construction plans and shop drawings, some modifications were made to certain structure details. The As-Built plans should be consulted for complete details.

2.5 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 St. Anthony Falls (I-35W) Bridge. These include:

As-Built Plans - The as-built plans are an excellent reference. These include modifications made to the details shown in the released-for-construction plans during construction (see Section 2.4).

Load Rating 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 only simple hand calculations. Information is included for longitudinal shear, torsion, and moment in the box girder and transverse moments in the deck.

Shop Drawings - Detailed shop drawings exist for expansion joints, bearings, railings, bridge drain components, electrical & lighting components, bridge coatings, ITS components, health monitoring, anti-icing and other bridge elements.

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

Construction Documentation - Documentation of the project construction was compiled by the Design-Build Team including as-built drawings, correspondence, meeting minutes, diaries, material test reports, contractor submittals, photographs, etc.

Chapter 3

Inspection Procedures

3. INSPECTION PROCEDURES

3.1 General Information

This chapter presents general topics related to inspection of the St. Anthony Falls (I-35W) Bridge including access and lighting, special precautions, required equipment, and pre-inspection meetings.

3.1.1 Access and Lighting

The bridge site features security systems including video surveillance, locks, and intrusion sensors. Proper arrangement must be made for authorized access prior to deploying inspection personnel to the site.

An under-bridge inspection crane may be used to inspect the outside of the box girder beneath deck level. This equipment or a ground-based manlift is also required for inspection of the pier caps and bearings. Access to the Pier 2 and 3 bearings is through the Main Span Bearing Access Hatches located on either side of the piers under the bottom soffit of the segments (Figure 3.1). Access to the base of abutments may be achieved on foot. Access to piers on the bank of the river is from the plaza beneath the bridge or from above using an under-bridge inspection crane. The plazas are designed to support an H-5 vehicle (2 kip front axle, 8 kip rear axle with 14'-0" axle spacing) or equivalent for maintenance and inspections.

Access to the interior of the box girders is through bottom slab access hatches located in Span 1, Span 3 and Span 4 (Figure 3.2). An under-bridge inspection crane or ground-based man lift is required. Cranes or lifts with the minimum capabilities should be utilized to reach the bottom slab access hatches: Span 1 - vertical lift of ~35'-0", Span 3 – snorkel lift with ~135'-0" boom, Span 4 – vertical lift of ~20'-0" (note: Span 4 access restricted by chain-link fence without a gate). The bottom slab access hatches have a nominal weight of 120 lbs and the force required to open the hatch is 65 lbf. At the main span pier diaphragms, ramps provide access over and through the diaphragm openings. Portable extension ladders are inside the box girders for inspection purposes.

Maintenance lighting is provided in each box girder and is controlled by three-way switches located within each span near the access location. 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 bridge. There is also a timer on the lights which automatically turns the lights off after 6 hours. The maintenance lighting is sufficient for mobility, but flashlights or floodlights should also be considered to adequately inspect the box girder interior. Power outlets are located at every other light fixture throughout the bridge on the interior face of the box girder web. These outlets can be utilized to provide additional AC-powered lighting for inspection purposes.

3.1.2 Special Precautions

Standard safety procedures used for inspecting all bridges should be undertaken while inspecting the St. Anthony Falls (I-35W) Bridge. Air quality and temperature should be checked to verify that it is safe for personnel to enter the girder.

A minimum of two inspection personnel should be inside the box girder at any given time. At least one of them should be in radio contact with personnel outside the girder.

It is important that inspectors inside the box girder wear hard hats, due to the box interior's height at midspan, the reduced height of the pier diaphragm openings, and utilities suspended from the top slab. Care should also be taken when walking through the box girders, as anchorages, deviation ribs, electric junction boxes, etc. located in the bottom slab may pose a potential tripping and or obstruction hazard.

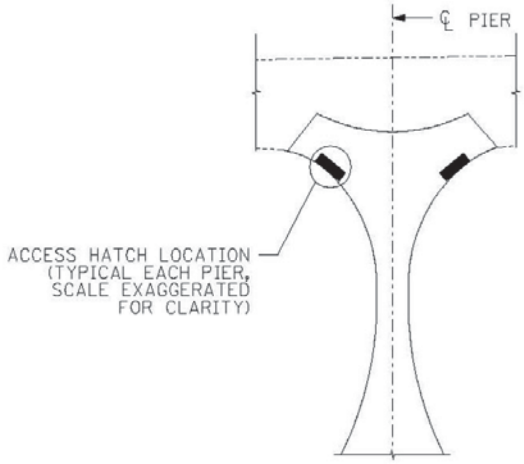


Figure 3.1a –Main Pier Bearing Access Locations

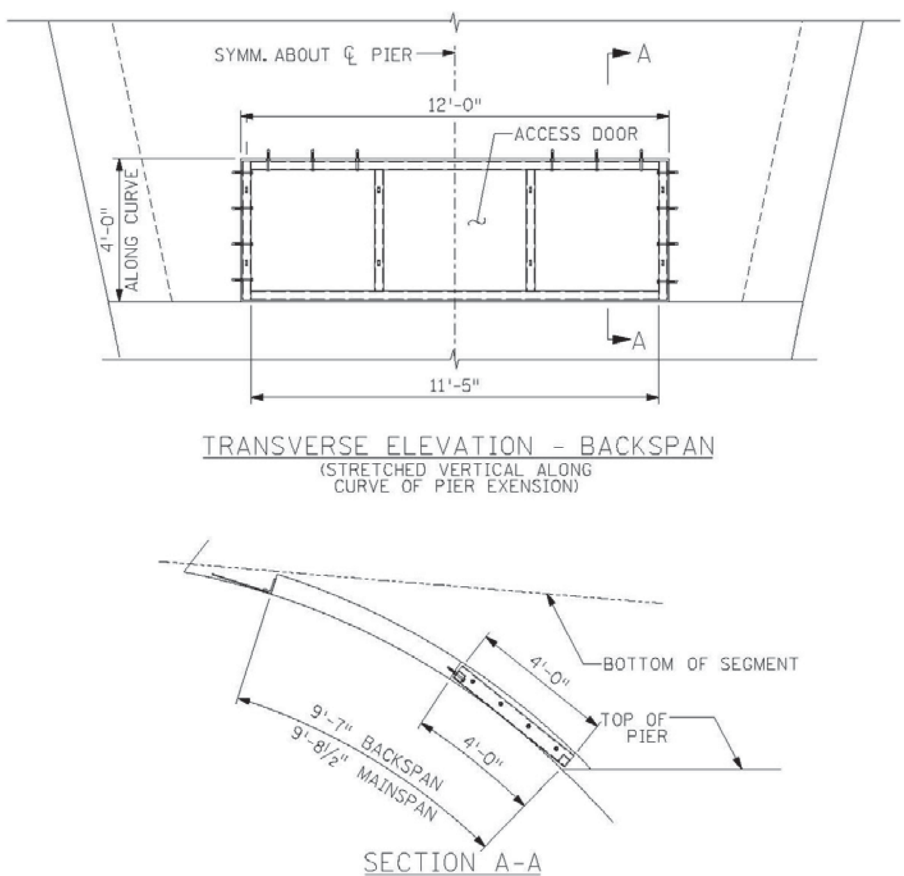


Figure 3.1b – Main Pier Bearing Access Details

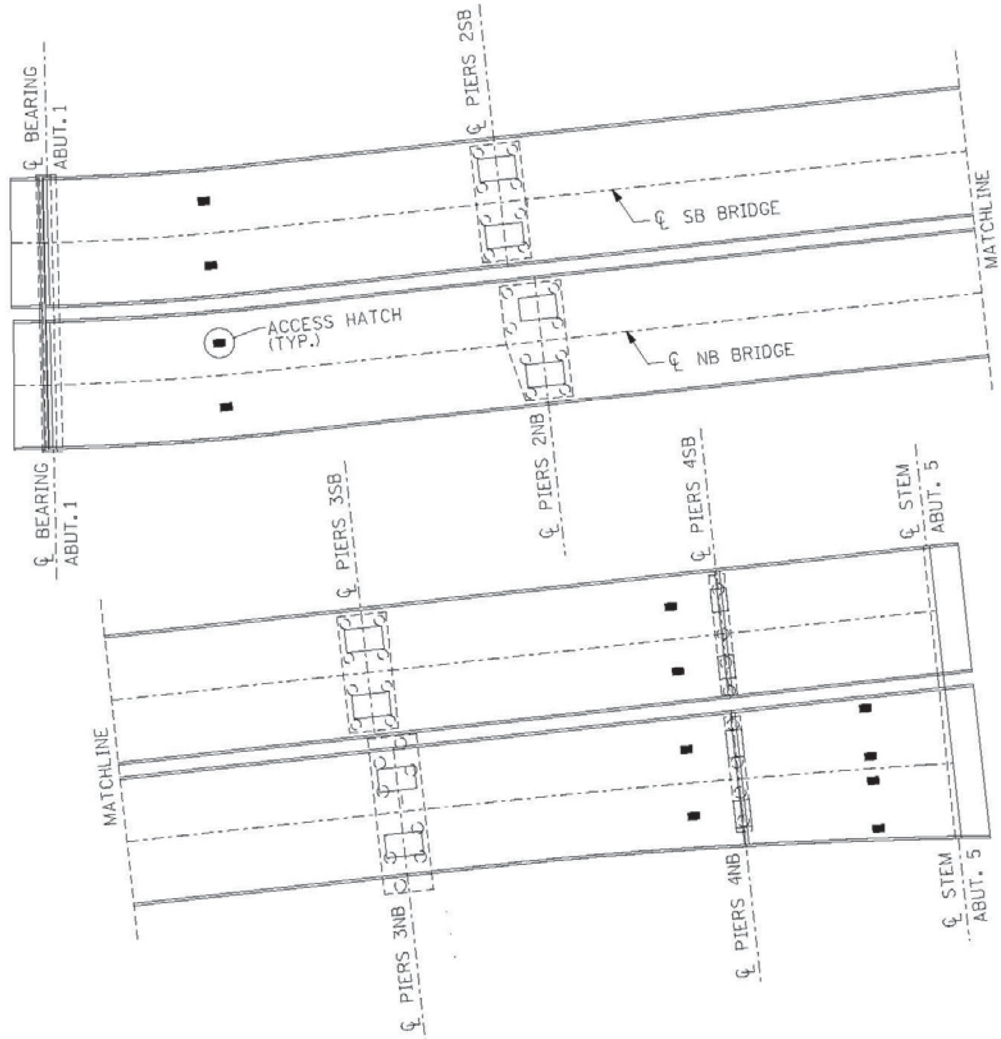


Figure 3.2a –Bottom Slab Bridge Access Locations

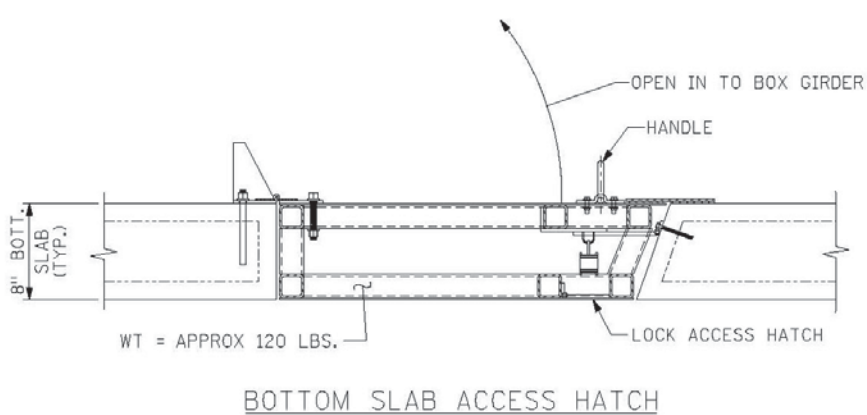


Figure 3.2b –Bottom Slab Bridge Access Details

3.1.3 Required Equipment

The standard items listed in the Federal Highway Administration Bridge Inspector's Reference Manual will be required for the inspection. This Bridge Inspector's Reference Manual and sufficient copies of the survey sheets from Appendix B will also be needed. Specialized equipment is limited to an under-bridge inspection crane and/or ground-based manlift for access to the pier caps, bearings, box girder interior, and exterior of the superstructure beneath deck level. A ground-based manlift will be necessary to inspect the piers and bearings. Traffic control will most likely be required while the under-bridge inspection crane is positioned on the bridge deck and while the deck itself is being inspected.

The following is a suggested list of equipment that may be needed during regular inspections of the St. Anthony Falls (I-35W) Bridge:

Personal Comfort/Safety

- Hardhats
- Work Gloves
- Eye Protection
- Safety Vests
- Flashlights
- Extra Flashlight Batteries
- Life Jackets
- Fall Protection Harness & Lanyards
- Hand-Held Radios
- Drinking Water

Access to Box Girder Interior and Piers 2/3 Bearings

- Ladder / Under-Bridge Crane / Ground-Based Manlift
- Master Service Door Padlock Keys

Recording/Measuring/Testing Devices

- Permanent Markers (or paint sticks / crayons)
- Pencils
- Pens
- Clipboards
- Scratch Paper
- Copies of Survey Sheets from Appendix B of this manual
- Mn/DOT Standard Bridge Inspection Forms
- Crack Width Comparator Cards
- Still Cameras w/Flash (Film and/or Digital)
- Video Cameras, videotapes (optional)
- Measuring Tape (U.S. Customary)
- Concrete Surface Thermometer
- Rock Hammer
- Rubber Mallet

3.1.4 Pre-Inspection Meeting

Immediately prior to an inspection, all personnel who will be involved should be briefed about any unique aspects of the upcoming inspection. This includes specialized inspection procedures and safety concerns as listed in this manual. It should also include any concerns noted in the last inspection and any maintenance or modifications completed since the last inspection.

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 which 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:

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 for the various elements of the St. Anthony Falls (I-35W) 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 Cap	10 mils (0.010 in)
Abutment	10 mils (0.010 in)

Concrete Box Girder:

Deck	7 mils (0.007 in)
Web Exterior	12 mils (0.012 in)
Web Interior	16 mils (0.016 in)
Bottom Slab Exterior	12 mils (0.012 in)
Bottom Slab Interior	16 mils (0.016 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 St. Anthony Falls (I-35W) Bridge are conventional reinforced concrete construction. In post-tensioned concrete structures, concrete is precompressed, which substantially limits cracking. The concrete superstructure of the St. Anthony Falls (I-35W) 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 conditions 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 3.3)

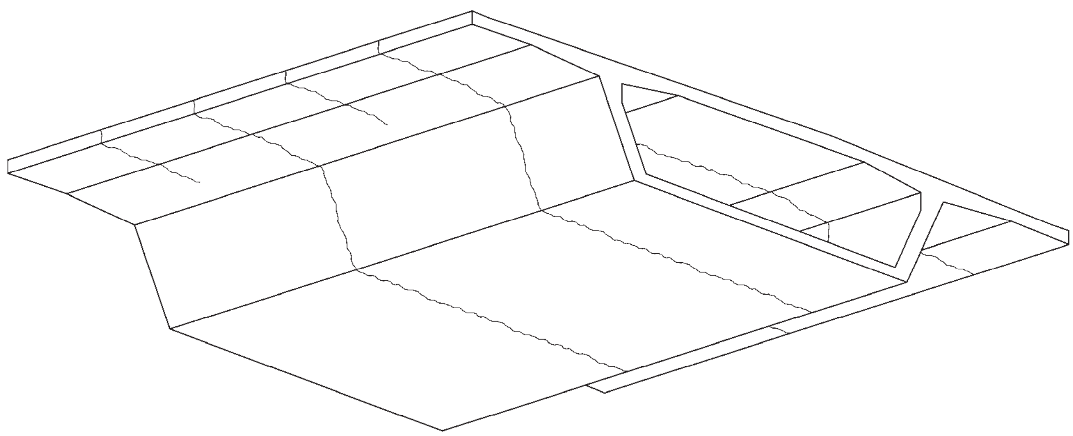


Figure 3.3 – Superstructure Potential Crack Pattern for Direct Tension

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 3.4).

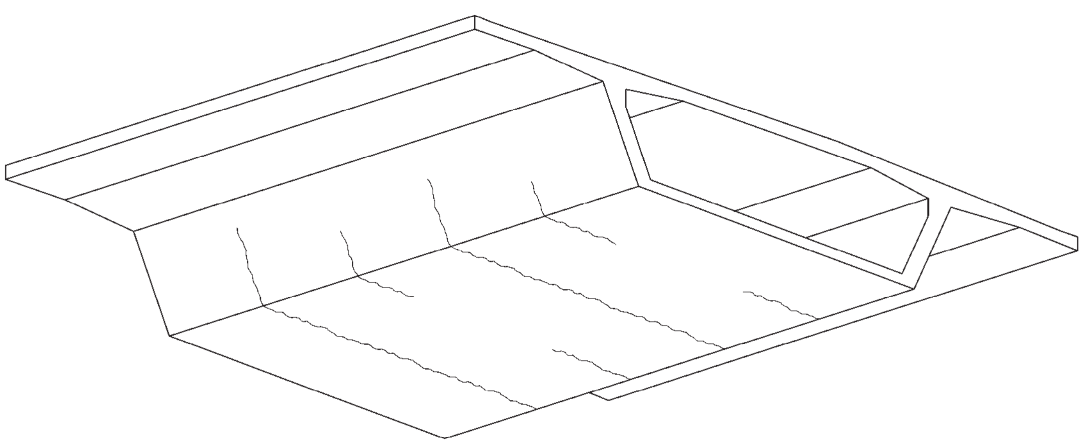


Figure 3.4 – Superstructure Potential Crack Pattern for Positive Moment

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 3.5).

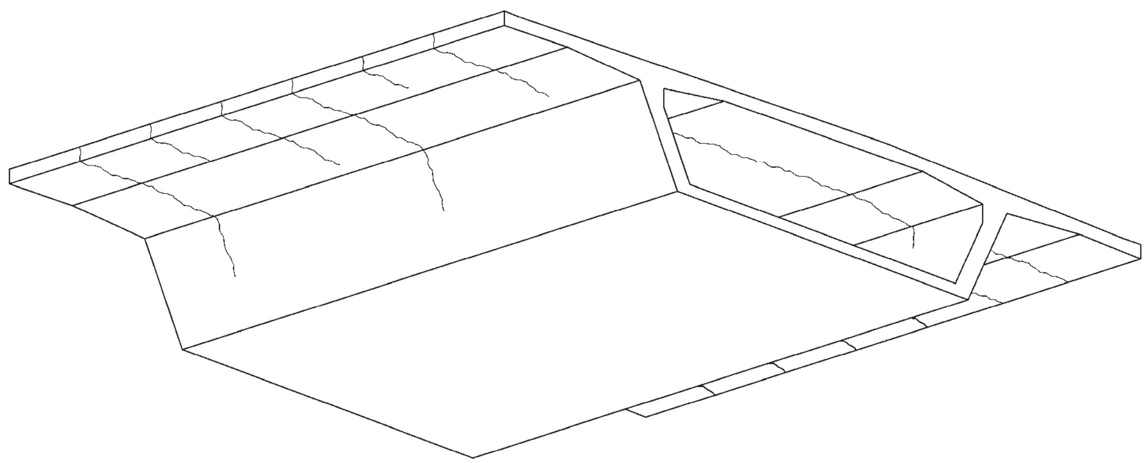


Figure 3.5 – Superstructure Potential Crack Pattern for Negative Moment

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 3.6).

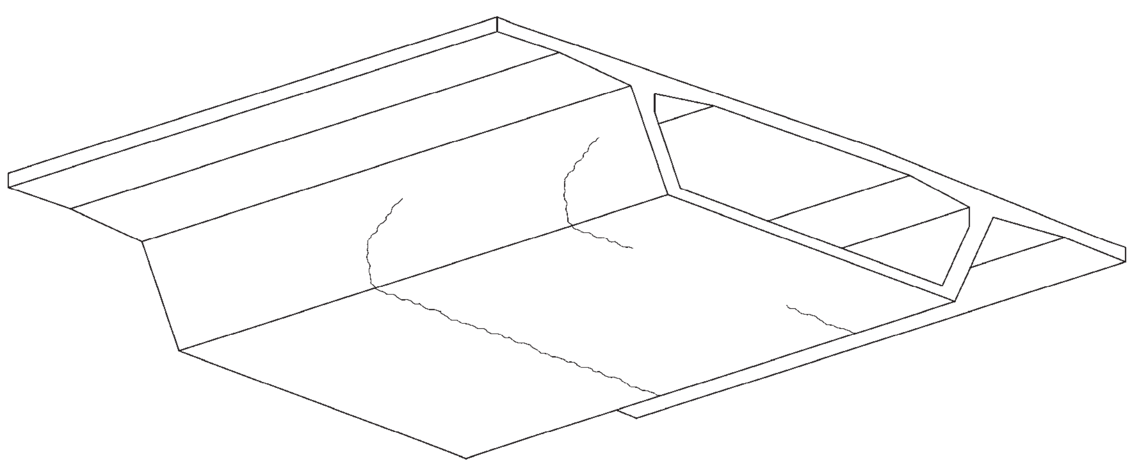


Figure 3.6 – Superstructure Potential Crack Pattern for Flexural Shear

Pure shear cracks would initiate in the webs near the neutral axis of the section close to the supporting pier or abutment (Figure 3.7). These cracks are generally inclined at approximately 45 degrees from vertical.

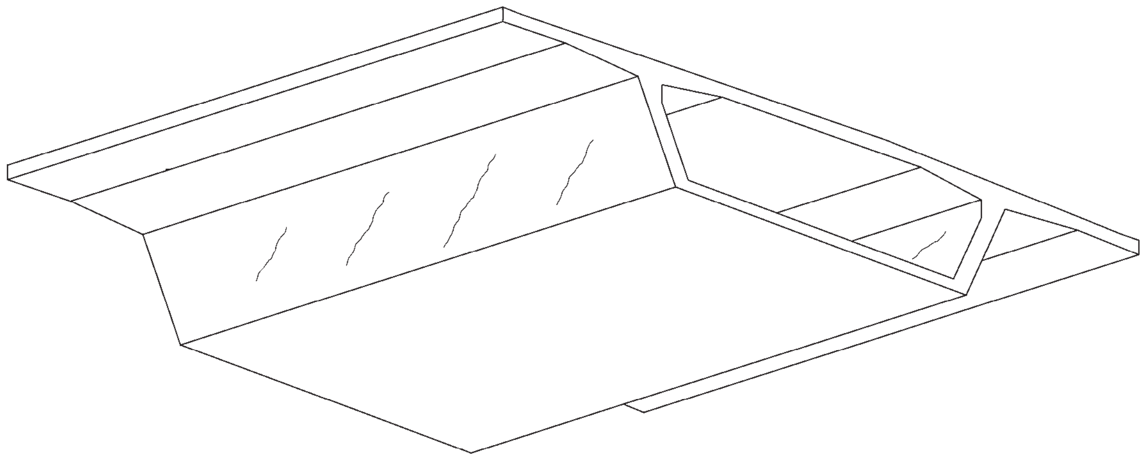


Figure 3.7 – Superstructure Potential Crack Pattern for Pure Shear

Pure torsion would produce inclined cracks that continuously wrap around the entire box section, excluding the cantilever wings (Figure 3.8). 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.

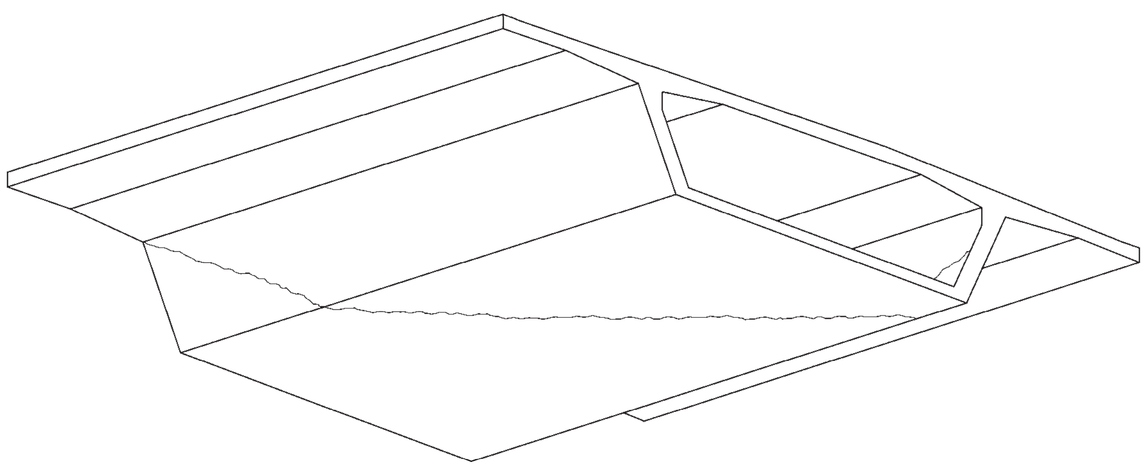


Figure 3.8 – Superstructure Potential Crack Pattern for Pure Torsion

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 3.9).

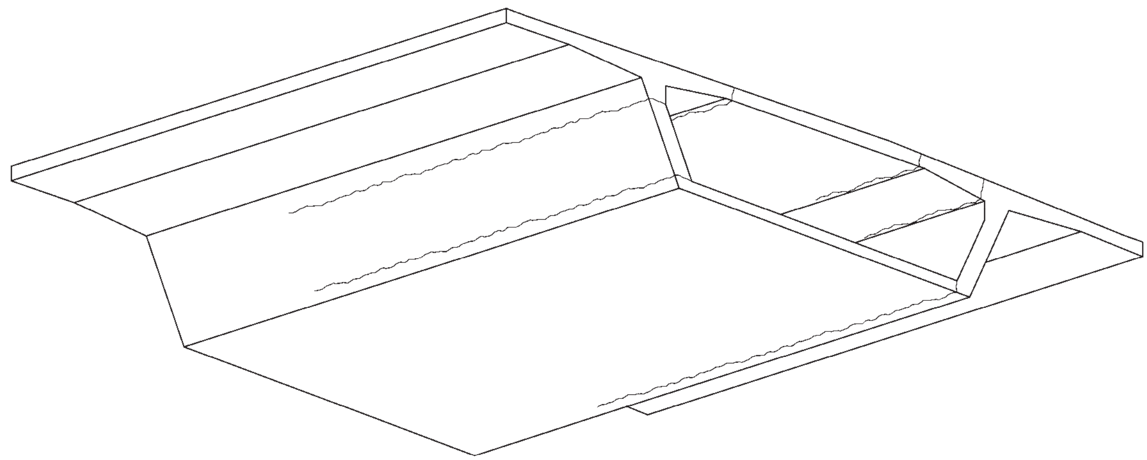


Figure 3.9 – Superstructure Potential Crack Pattern for Thermal Effects

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 3.10).

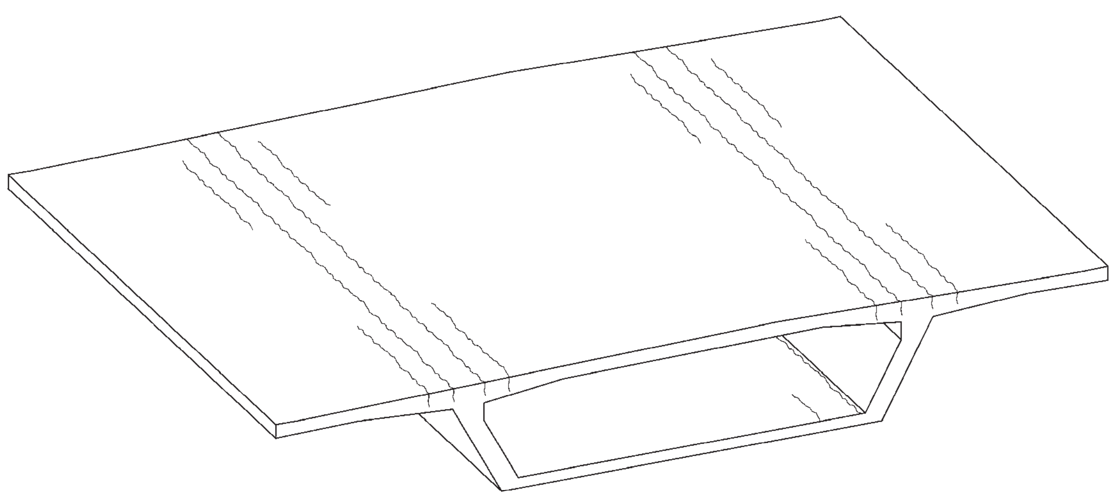


Figure 3.10 – Superstructure Potential Crack Pattern for Transverse Deck Flexure

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 in unlikely, flexural and thermal 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 (Figure 3.11).

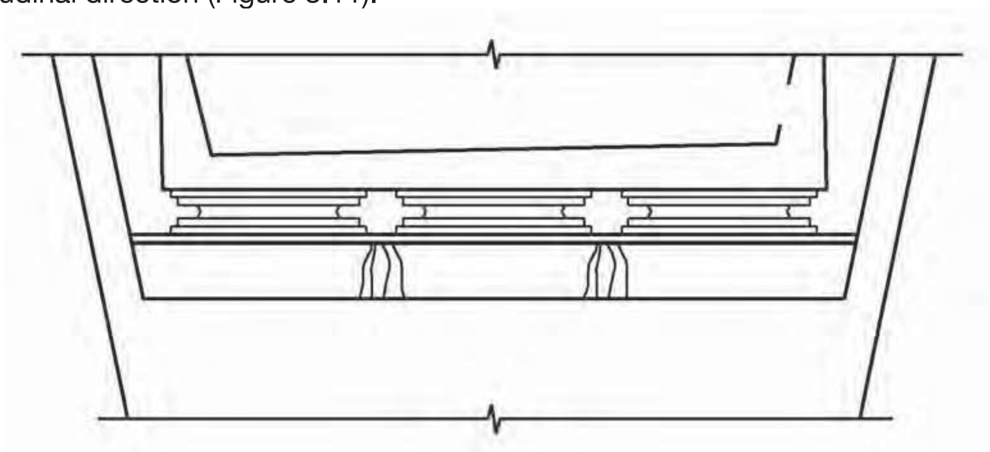


Figure 3.11 – Pier Cap Potential Crack Pattern

The potential for footing cracks exists 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 2, 3 and 4 are below, or partially below, grade. Therefore, the 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.

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 drilled shaft foundations. Footing surfaces should be inspected for cracks, spalls, and signs of corrosion staining. Indications of distress in the footing may be a sign of problems with the drilled shafts.

3.3.2 Piers

The main piers for the St. Anthony Falls (I-35W) Bridge are longitudinally hour-glass shaped with "extensions" that enclose the bearing seat assembly. Transversely, the pier cap widens at the top of pier to accommodate the pier extensions. They are constructed of conventional cast-in-place reinforced concrete.

Pier 4 columns are also longitudinally hour-glass shaped with a pier cap that widens transversely at the top of pier to align with the bottom soffit of the bridge segments. They are constructed of conventional cast-in-place reinforced concrete.

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.

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. Stone masonry walls facing the abutment walls should be inspected for cracks and spalling of the mortar between the rock. The cast-in-place concrete coping above the masonry wall and the caulking between the coping and the abutment wall should also be inspected for cracking.

3.4 Disktron© Bearing Inspection

There are three types of bearings (fixed, non-guided, and guided) used on the St. Anthony Falls (I-35W) Bridge and they have been divided into two groups for the purpose of inspection: fixed and sliding. Two types of sliding bearings are used: guided and non-guided.

3.4.1 Fixed Disktron© Bearings

Each Disktron© bearing should be inspected for the following: corrosion of any elements; condition of protective coating; concrete condition above and below bearings, particularly the area above the sole plate and below the masonry plate; and condition of the Disktron© bearing assemblies. Any distress in the Polytron© disk or cracked steel indicates a need for service or possibly replacement.

Disktron© bearings were designed for a maximum rotation of 0.02 radians (1.15 degrees). The actual rotation of bearings can be estimated by measuring, to within 1/32", the vertical distance between the masonry plate and the sole plate at the four corners of the masonry plate. For convenience, horizontal dimensions of the masonry plate should also be recorded. The bearing rotation in a given vertical plane is then calculated as the difference in vertical clearances divided by the horizontal distance between two corners (units of radians).

3.4.2 Sliding Disktron© Bearings (Guided and Non-guided)

The inspection should cover all points listed for fixed Disktron© bearings. The horizontal location of all sliding bearings should be measured as part of bearing inspection. Bearing movements at Abutment 1 and Pier 4 are also monitored by linear potentiometers as part of the Bridge Health Monitoring System. These measurements and concrete temperatures are taken and entered onto the bearing inspection survey sheets given in Appendix B for each bearing. The appendix explains how to calculate total movement required in each direction based on a current date and temperature. These values are compared to measurements taken to ensure that future movement capacity is available. If it is not adequate, the State Bridge Engineer should be notified of the situation.

In addition, the PTFE and stainless steel surfaces should be checked to ensure that free movement is still occurring. Large scratches in either surface indicate that foreign matter could be lodged between surfaces, which could possibly impede movement. PTFE shavings may also be visible if the bearing has exceeded its design movement. If a bearing is suspected to be "frozen", the Disktron© bearing manufacturer or similar expert should 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, epoxied segment joints, closure 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. Other critical areas to check for seepage are cast-in-place closure joints. Check for any evidence of segment joint openings or leakage. 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.

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 Epoxied Segment Joints

Excess epoxy was removed prior to coating the box girders, but some excess epoxy may remain visible along the underside of the box girder (wings, webs and bottom soffit) at the joints between segments from the epoxy joining of segments during construction. This epoxy is cured fully during the erection of the segments, so it does not represent a problem or durability issue with the structure. Epoxied 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 Cast-in-Place Closure Joints

The inspection procedure used for other superstructure elements is also applicable to cast-in-place closure joints.

3.5.1.5 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.

3.5.1.6 Anchor Blocks

Top slab and bottom slab anchor blocks are also 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 top or 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/wraps through the intermediate diaphragms in Span 4 should also 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.

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 St. Anthony Falls (I-35W) Bridge; a brief description of each follows with a guideline for inspection.

Cantilever Tendon Anchorages: Cantilever tendons in the cast-in-place section of the bridge anchor in blocks along the top deck/web interface on the inside of the box girder (Figure 3.12) or in the expansion joint blockouts at the end of Spans 1 and 3. Their anchorages are protected by a grout-filled non-metallic (plastic) cap inside an epoxy grout pourback with an elastomeric membrane seal. Additional cantilever tendons anchor on the face of expansion joint blockouts at Abutment 1 and Pier 4. These anchorages are protected by a two-layer epoxy grout-filled blockout. 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. Cantilever tendons for the precast segments anchor between the two segment joints at the top of the web. These anchorages are protected by a two-layer epoxy grout-filled blockout. Since the anchorages in the precast segments 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 3.13), 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) 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 or 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.

Draped and bottom slab anchorages at expansion joint diaphragms are located on the outside face of the diaphragm (Figure 3.14). Special consideration of tendon anchorages located at expansion joint diaphragms beneath expansion joints is necessary (Abutment 1 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) 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.

Figure 3.12 – Internal Tendon Anchor Blocks

Figure 3.13 –Draped Tendon Anchorages

Abutment 1

Pier 4

Figure 3.14 – Expansion Joint Anchors Prior to Epoxy Grout Pourback and Seal

Figure 3.15 – Pier Diaphragm PT Bar Anchor with Epoxy Grout Pourback

Post-Tensioning Bar Anchorages: For the vertical post-tensioning bars that anchor in the deck of the expansion joint 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). Longitudinal post-tensioning bars anchor in the pier table and at the bulkhead face of each precast segment and are protected by a grout-filled galvanized steel duct. The mid-span cast-in-place closure segment, in conjunction with plastic grout caps, protects post-tensioning bars anchored at the cantilever tips. Additionally, there are transverse post-tensioning bars located above Piers 2 and 3, that run from web to web, through the pier segment diaphragm (Figure 3.15). 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 at the vertical post-tensioning bars pourbacks. Inspection of longitudinal post-tensioning bar anchorages should include observing the condition of surrounding concrete (top and bottom) and noting any signs of corrosion such as staining or spalling.

3.6 Expansion Joint Device Inspection

The following subsections discuss inspection of the modular expansion joints and the associated bridge railing joints.

3.6.1 Modular Expansion Joints

Each modular expansion joint should be examined for freedom of movement (signs of distress to rails and support beams indicate a lack of freedom of movement), proper opening (spaces between rails should be approximately equal), proper vertical alignment, debris accumulation, and water tightness of the seals. Proper opening size depends on the age of the structure and the temperature of the structure at the time of inspection. Measurement of joint opening should be performed using the joint survey sheet in Appendix B. Measurement can be done by hand or by using data from the linear potentiometers that are part of the health monitoring system. Concrete temperature and date must also be recorded at the time of measurement to ensure proper interpretation of the data. The Bridge Division should perform a calculation of anticipated maximum and minimum joint openings on the survey sheet in Appendix B.

Each modular expansion joint assembly should be inspected for loose or damaged parts, accumulation of trash, soil, rocks and other foreign material that may restrict free movement of the joint. Stones lodged in the joints can create localized stresses that may cause cracking and spalling of the deck or damage neoprene seals, allowing the joint to leak. Large amounts of debris may cause jamming of the joint, which renders the joint ineffective.

The condition of neoprene seal elements in the joint assembly should be observed. This includes inspecting joints from the underside, within the box girder to determine the integrity of the seal in preventing water infiltration from the deck. Inspect for evidence of the seal pulling away from steel rails, for abrasion, shriveling, or for other physical deterioration of the seal, which may cause leaks. Also check for stains or other signs of leakage underneath the deck. Moisture or efflorescence on the underside of the deck are possible indications of leakage. Careful attention should be given to detect looseness or breakage of any metal parts of the joint assembly or fasteners.

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 box girder cantilever wings.

3.6.2 Bridge Railing Joints

The detail for the bridge railing at expansion joints incorporates a sliding cover plate. Inspection should identify any steel plate damage that could interfere with free movement of the sliding plate and verify the structural integrity of concrete anchor cap screws and bridge railing concrete adjacent to the joint. Rust or signs of rust on the sliding cover should be noted.

3.7 Deformation Monitoring

The following sections discuss the deformation monitoring for substructure settlement, pier movement, superstructure creep, superstructure 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 readings be found at the piers during the superstructure alignment surveys (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 Span 2 is 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 when comparing survey results to the plan vertical roadway alignment.

The estimated total amount of long-term vertical superstructure deflection (relative to elevations at the end of construction) due to concrete creep in 2018 is shown below for typical spans. These are total vertical deflections at midspan expected to occur in 2018, 10 years after the end of construction, which was the summer of 2008. There should be no significant additional increase in vertical deflections due to creep after approximately ten years, or 2018. If measured vertical superstructure deflections differ substantially from these values, further investigation is warranted.

Span 1NB	-2 3/8”	(down)
Span 1SB	-7/8”	(down)
Span 2NB	1 5/8”	(up)
Span 2SB	5/8”	(up)
Span 3NB	-3/4”	(down)
Span 3SB	-3/4”	(down)
Span 4NB	-3/8”	(down)
Span 4SB	-3/8”	(down)

Uniform procedures for regular structure profile surveys should be established to ensure consistent and meaningful data. These should include the following:

- Use the same benchmark with known elevation as a starting point for all surveys.

- Close the level loop by either returning to the same benchmark or by ending on a second benchmark of known elevation. If satisfactory closure is not obtained, the survey should be repeated.

- Surveys should only be performed when thermal gradient effects are minimized. This means completing survey work in a timeframe where thermal gradients are minimal, usually in the morning within two hours of sunrise.

The results of the deck profile surveys that were performed prior to opening the bridge to traffic are included in Appendix A and should be used as a baseline for monitoring long-term vertical deflections.

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. However, these deformations will adversely affect results of the vertical alignment survey. As noted in Section 3.7.3, the vertical alignment survey should be done when no significant thermal gradient exists by taking surveys within two hours of sunrise, which will limit or eliminate the thermal effect from solar radiation.

3.8 Unique I-35W Bridge Elements

Other Bridge elements such as the Pier Extensions should not be overlooked during inspection. While Gateway Monuments and Pier Plazas should be reviewed, note that they are structurally independent from the bridge. Generally, these items should be evaluated for potential damage, stability of the element or its connection to the structure, signs of corrosion, and need for general maintenance.

3.9 Lighting, Electrical, and Other Components

There are numerous miscellaneous components to the St. Anthony Falls (I-35W) Bridge that require inspection and maintenance. These are not necessarily unique to this bridge, but will be mentioned briefly so that they are included in planning inspection activities.

Lighting – Lighting for the St. Anthony Falls (I-35W) Bridge consists of interior box girder maintenance lights, roadway lights and bridge aesthetic lighting. Bridge aesthetic lighting includes box girder wing lights, pier up-lighting, abutment lights, and monument lighting. These should be observed at night to be sure each fixture is functioning properly. Detailed information on the light fixtures used may be found in Appendix D.

- Maintenance Lighting: The maintenance lights are located inside the box girder and are located on the bottom of the top slab of the box girder web. The power conductors are routed in conduit along the inside face of the box girder web. The maintenance lights for each span are 3-way switched at each end of the span. The maintenance lights are controlled by a timer switch which turns off the lights after 6 hours.

- Roadway Lighting: The roadway lights are LEDs that are mounted on poles located behind the barrier on the inside shoulder of each bridge. The roadway lights are powered from both feedpoint W5J and feedpoint W5L. Feedpoint W5J is located on the right northbound shoulder at the merge point of the Washington Avenue on ramp. Feedpoint W5L is located at the corner of the northbound off ramp and University Avenue. The roadway lights are switched by a photoeye.

- Aesthetic Box Girder Wing Lighting: The aesthetic box girder wing lights are LEDs located on the outside wing of the outside box girder on both bridges. Spacing of the fixtures varies from 3’-4 ½” to 11’-7” along the bridge. Separate conduits are provided for the power and control wires. Power to the aesthetic wing lighting is fed from the north end of the project. The controller is located in the control building near the pump house along 2nd Street and is remotely accessible.

- Aesthetic Pier Up-Lighting: The pier lights are LEDs mounted in concrete wells at the base of each column of Piers 2 and 3. Each curved column face has three lights totaling 48 for the project. They are powered and controlled by the same cables as the wing lights.

- Abutment Wall Floodlighting: Downlighting for the abutment masonry walls is provided by two fixtures at each abutment location, one below the southbound exterior girder and one below the northbound exterior girder, for a total of 4 fixtures. Power is provided from lighting service cabinets located in the southbound and northbound exterior girders and the lighting system is controlled by computer timers.

- Aesthetic Gateway Monument Lighting: The gateway monument lights are at the base of each monument. Gateway monument lighting consists of four sets of four lights, 16 total for the project. They are powered and controlled by the same cables as the abutment lighting.

Electrical – The main power feeds for the project are:

- South Roadway Lighting – on the right shoulder of I-35W northbound at the merge point of the Washington Avenue on ramp (W5J).

- North Roadway Lighting, Bridge Deck Lighting, and Box Girder Interior Power and Lighting – on the corner of the I-35W northbound off ramp and University Avenue (W5L).

- All Bridge Aesthetic Lighting and Health Monitoring System inside Box Girders – near the pump house on 2nd Street (NFP). The anti-icing system is powered from the same location but on a separate meter.

- Pier 2 Plaza and Walkway Lighting – under Northbound Span 1 between the West River Parkway and the USACOE driveway near the lift station (FPSL).

- Pier 3 Plaza and Walkway Lighting – near the northeast corner of the Pier 3 plaza (FPNL).

All conductors are located inside conduit. Any suspected damage to this or any other components of the electrical system should be evaluated by a certified electrician.

Railings – Note damage to any of the railing systems. Cracked areas and/or spalls due to vehicular impact on the traffic railing should be evaluated and repaired according to the procedures described in Chapter 4. Open barrier railing panels should be stable, with all bolts secure and no signs of rust. Any damage extending below the top of the deck should be evaluated with respect to the transverse post-tensioning anchorages.

Vermin Guards – Vermin guards are the steel frame system and bird deterrent mechanisms located at each expansion joint and on tops of the pier extensions. They serve to keep birds and other small animals from entering the interior of the superstructure. These should be inspected to be sure their effectiveness is maintained.

Overhead Signs – The overhead signs located at Pier 4 on the St. Anthony Falls (I-35W) Bridge should be inspected per Standard MnDOT procedures for these elements. Special attention should be given where the sign is bolted to the bridge superstructure elements and any cracked areas and/or spalls in the region where the sign is bolted to the barrier elements should be noted.

Anti-Icing System – The Boschung Anti-Icing System should be inspected to ensure continued reliable service based on the manufacture's recommendations for inspection and maintenance as outlined in the manufacture's provided manual.

3.10 Survey Sheets

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 as noted in Section 3.7.3 and 3.7.4.

Chapter 4

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. Therefore, maintenance of bridges and their approaches should be considered a vital phase of highway operation.

Regular inspection of all components of the St. Anthony Falls (I-35W) Bridge should be made as discussed in Chapters 1 and 3 to locate areas that need attention that can be accomplished with routine maintenance procedures before they develop into repair situations. In addition to scheduled inspections, maintenance personnel in the course of their daily operations should be alert to conditions that need to be corrected. This simple effort contributes to the safe operating condition of the bridge.

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 asked:

- 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 of construction quite 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:

<u>Maintenance</u>: Maintenance is defined as corrective or preventive action that sustains current performance level or capacity.

<u>Repair</u>: Repair is defined as corrective action that significantly improves current or existing performance level or capacity without exceeding the original level or capacity.

Rehabilitation: Rehabilitation is defined as 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 frequent basis) and periodic maintenance (those activities which are planned over a longer term on the basis of identified needs). Periodic maintenance is subdivided into corrective maintenance (those activities necessary to correct conditions identified from scheduled inspections) and preventive maintenance (those activities carried out to prevent or minimize future problems).

Items that include routine maintenance may include:

- Expansion joint device cleaning
- Deck drain cleaning
- Open railing cleaning and/or painting (due to vandalism or corrosion)
- Traffic railing repairs (due to vehicular damage)
- Bridge Lighting

The above items are anticipated as the most common type of maintenance required on the St. Anthony Falls (I-35W) Bridge. Other items that are not anticipated to be common maintenance items but may need to be performed, such as epoxy injection of cracks, bearing replacement, re-sealing of the concrete wearing surface and installing and stressing future post-tensioning tendons, are included in this chapter.

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:

<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 applying the above criteria to the various elements of the St. Anthony Falls (I-35W) Bridge:

<u>**Guide for Determining the Significance of Cracks**</u>

Element of Substructure:	**Cracks May Be Significant If Width Exceeds:**
Footing	10 mils (0.010 in)
Piers	12 mils (0.012 in)
Pier Cap	10 mils (0.010 in)
Abutment	10 mils (0.010 in)
Concrete Box Girder:	
Deck	7 mils (0.007 in)
Web Exterior	12 mils (0.012 in)
Web Interior	16 mils (0.016 in)
Bottom Slab Exterior	12 mils (0.012 in)
Bottom Slab Interior	16 mils (0.016 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 epoxy 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.

- Box girder exterior webs and bottom soffit, and pier columns - Seal cracks that are 12 mils or more in width.

- Box girder interior webs and bottom soffit - 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. Additional technical information is provided in Appendix E. 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.

<u>Epoxy Pressure Injection</u>

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 cleaning by air, water, 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 affect 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.

Post-Tensioning Tendons

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.

Concrete Removal

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.

Reinforcing Steel

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.

All reinforcing steel in the superstructure is to remain 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 placed and cured as per the manufacturer's recommendations or standard practices from the provided list in Appendix E, or a

Mn/DOT approved patching material. All products should be used per manufacturer's recommendations. Additional technical product information is provided in Appendix E.

<u>Shotcrete Repairs</u>

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

Because of the deep drilled shaft foundations, it is not anticipated that any significant substructure settlement will occur. However, a vertical alignment survey will identify any problem areas. A survey was performed at the end of construction to serve as a baseline for surveys in the future and is included in this manual on the enclosed DVD. Determining the elevation variations of survey markers placed over the pier segments and expansion joint segments will 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.

4.4 Disktron© 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 the corrosion and/or subsequent "freezing" of the bearing. The corrosion process is accelerated if the moisture contains deicing chemicals. With the bearing "frozen", the bridge is restrained from expanding or contracting with temperature change. This restraint transmits compressive or tensile stresses to the girder and pier and ultimately creates a potential for damage and cracking of concrete.

4.4.1 Compensation for Excess Movement

As part of the periodic inspections, the Bridge Division should calculate the maximum anticipated longitudinal movements of the sliding bearings. These are calculated using the forms included in Appendix B of this manual. The maximum anticipated movements are checked to ensure that the sliding bearing surfaces will maintain complete contact when fully extended due to creep, shrinkage, and thermal movements.

If the calculated anticipated maximum horizontal bearing movements exceed the capacity for a given bearing, it may mean that corrective action should be planned. However, it is not recommended that corrective action be taken unless a bearing has sustained damage that renders it unable to function satisfactorily. There are two reasons for this.

First, some additional longitudinal movement capacity is provided by the flexibility of the piers even though they are 'pinned' at Piers 2 & 3 and Pier 4/Span 4. Thus, the top of a pier may deflect to the north or south a certain amount due to the force applied by the temperature rise and fall in combination with creep and shrinkage because of Span 2 – but there is no relative bearing movement at the Pier 2 & 3 or Pier 4/Span 4 bearings. The pier flexibility was conservatively ignored in the design of the bearings. Because of uncertainties regarding the actual bearing friction coefficients and the movement history of the bridge, it is difficult to quantify the pier deflection contribution to movement capacity at any given time. If it appears that a given bearing will not have sufficient movement capacity without contribution from pier flexibility, the Minnesota Department of Transportation Bridge Division should investigate the situation further before deciding on a course of action.

The second reason that corrective action is not recommended unless bearing function is impaired is that movement beyond the bearing capacity may not result in significant damage to the bearing. For instance, a bearing exceeding its movement capacity by 1" may experience damage to the first 1" of PTFE sliding surface. The damage most likely will not significantly impair bearing performance. The bearing should be monitored to ensure that movement is not impaired, and for any additional deterioration such as continued loss of PTFE, damage to the stainless steel surface, or loss of elastomer from the Disktron© due to uneven loading.

If it is determined that compensation for excess movement is needed at a given bearing location, the top plate and stainless steel plate can be replaced with longer plates. The relevant procedures outlined in Section 4.4.3 can be followed to accomplish this. Recess the new plate into the embedded sole plate with the same recess dimensions as the existing top plate.

If it is determined by field measurements that a bearing has experienced rotation beyond the design capacity (see Section 3.4), the cause for the excessive bearing rotation should be identified and corrected, if appropriate. For instance, the rotation may be due to a pier rotation caused by uneven foundation settlement. If the bearing has failed, the bearing should be scheduled for replacement. Note that the replacement bearing should have tapered plates or other compensation for the rotations that have occurred at that location, if any.

4.4.2 Releasing "Frozen" Bearings

The Teflon and stainless steel surfaces of sliding bearings should be cleaned of any debris that might restrict free movement. If a bearing becomes "frozen", so that movement of the bearing is no longer possible, it may be necessary to replace portions of the bearing or the entire bearing. The relevant procedures outlined in Section 4.4.3 can be used to perform the necessary repairs. Again, it is recommended that the bearing manufacturer be consulted.

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 in conjunction with the Bearing Replacement Procedure that is outlined in the construction plan sheets B-48 and B-49.

Bearing replacement operations shall not occur while wind gust speeds are greater than 30 miles per hour, or the concrete substrate temperature is 40°F or below. The concrete temperature requirement may be waived if all the minimum edge distances can be met as required by the construction plans.

If the bridge will be supported by the jacks long enough that significant thermal movement of the box girder could occur, lubricated sliding plates should be used between the jacks and the box girder at supports with expansion bearings, and the jacks should be braced against horizontal movement at all supports.

Piers with fixed bearings shall be locked to the superstructure to prevent longitudinal and transverse movements prior to jacking and throughout jacking operations. All other supports shall be locked to the superstructure transversely throughout the replacement operations. The method for locking these movements shall be determined by careful engineering and must allow for small longitudinal and transverse deflections to the system in order to realign the embedded masonry and sole plates.

For Piers 2 and 3, jack the superstructure vertically a maximum of ½" using the jacks designated "A" and "B" with the jack arrangements shown in the contract plans. Note that the entire steel cover at the pier tops is designed to be unbolted and temporarily removed for easier access. Steel distribution plates are required to distribute the jacking forces to the concrete of the superstructure. In accordance with the contract plans, all minimum edge distances must be maintained during the jacking operations. Following the initial jacking procedure, replace the upper and lower bearing plates and the Polytron© disks for the outside bearings. Then place jacks designated “C” in the contract plans into position. Jack up groups “A” and “C” an additional 1/8” above the ½” initially performed with jacks “A” and “B”. Replace the upper and lower bearing plates and the Polytron© disk for the center bearing. At Abutment 1 and Pier 4, jack the superstructure vertically a maximum of ½” using the jacks and jack arrangement shown in the contract plans. Jacking shall be done one support location at a time, but combining both pier columns per direction heading (i.e. Pier 2NB interior and exterior girders). The bearings are designed to be replaceable after jacking the box girder as described above. Jacking at Abutment 1 and Pier 4 may need to be further restricted to avoid damage to the expansion joints.

Once the structure has been jacked up, the expansion bearings can be removed by first unbolting and sliding the load plate (and guide bars for the guided bearing) out. This plate can slide out in any direction for the non-guided bearings, but it can only slide longitudinally for the guided bearings. Then the entire upper bearing plate/shear-resisting mechanism and Polytron© pad assembly can be removed together, by sliding the assembly off the lower bearing plate. A similar procedure is used to remove the fixed bearings, except that there is no upper bearing plate to remove.

The top load plate and lower bearing plate are attached to the concrete with multiple anchor studs welded to each plate and can only be removed by chipping away the concrete and cutting the studs from each plate or by chipping the studs out of the concrete entirely. For this reason, removal of these plates is not recommended. If it is deemed necessary to replace one of these plates, an engineer experienced in this type of replacement should be contacted.

Once the bearings have been removed, the reconditioned or new bearing components can be placed in the reverse order to that in which they were removed. Finally, the structure can be lowered onto the bearings, while maintaining a maximum transverse differential of 1/8". As the structure is lowered, it should be verified that there is not a difference in height between the left and right bearings that could cause the box girder to twist as it is set down. If such a difference occurs, the structure shall not be lowered onto the bearings until the height differential is corrected. Once the bearing replacement is complete, the corrosion protection system for the bearings shall be touched up where any surfaces have been chipped during placement operations.

Vertical movement of the superstructure should be monitored with dial gauges located at the bearing centerline for the left and right bearing to ensure that the transverse differential is no more than 1/8”.

Jacks at a given pier should be connected through manifolds to a common series of pumps to ensure uniform pressure (within 5%). Either steel shims or jacks with threaded safety nuts should be used such that the superstructure cannot drop more than 1/8” should a hydraulic failure occur during bearing replacement.

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 precast 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 epoxy 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. Additional information on these products can be found in Appendix E.

Impediments to proper bridge deck drainage should be corrected. Deck drains (exist in Span 2 only) should be regularly cleaned of debris to eliminate any area on the deck in which water may tend to pond.

The St. Anthony Falls (I-35W) 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 Mn/DOT 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 pours and anchor pour backs. 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 St. Anthony Falls (I-35W) Bridge was designed to accommodate a future wearing surface of up to 2 1/2" (30 pounds per square foot to replace the 2 1/2" 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. The corrosion potential sensors installed as part of the bridge health monitoring system may assist in determining the appropriate time for wearing surface replacement. 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 Segment

Maintenance activities for the cast-in-place closure segments are the same as for the precast superstructure segments. However, due to the fact that these elements were cast on-site and because of the differential age and curing of the closure segments versus the precast segments, the closure segments may be more prone to shrinkage cracks than the precast segments; needed repairs may be performed as discussed in Section 4.2.

4.5.1.4 Diaphragms

Again, 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 Bottom and Top Slab 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 St. Anthony Falls (I-35W) 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. In Span 4, through the intermediate diaphragms, neoprene tendon dampers/wraps protect the polyethylene pipe and the replacement of or addition to this protection should be executed if an inspection deems it necessary.

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 or top 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. Additional technical information is provided in Appendix E.

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, and pour back a secondary concrete pourback, which would be covered 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 2.29 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 St. Anthony Falls (I-35W) 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 3 have one future tendon per web; Span 4 does not have provisions for future post-tensioning, additional PT has been provided and initially stressed to provide the same level of increase capacity that may be provided for in the future PT for Spans 1, 2 and 3.

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 2.28 and 2.29. Future tendon anchors in place in the pier and expansion joint segments were manufactured by DSI. Should these anchors ever be used, the remaining anchorage system must be compatible with the DSI future tendon anchors already installed.

4.6 Expansion Joint Device Maintenance

The following sections describe the maintenance for the modular expansion joint devices.

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 assure 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 them from accumulating in the joint again.

In addition to cleaning the seals at the deck surface, the stainless steel surfaces 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 Bridge 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 affect this opening and are permanent. If the minimum opening controls, then further analysis should be done to be sure that the movements of the entire 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.1 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 4.6.4.

4.6.3 Replacement of Seals

Neoprene seals that leak should be replaced. Information concerning the replacement of neoprene seals, recommended procedures, and any special tools required may be obtained from the joint manufacturer, D.S. Brown.

4.6.4 Replacement of Joint Hardware

If individual components of the expansion joints require replacement, it should be coordinated through the expansion joint manufacturer, D.S. Brown.

Sliding plates at the bridge railing can be replaced by removing counter-sunk bolts that thread into drilled concrete inserts. These bolts attach the sliding plates to the bridge railing. New plates can then be attached to the existing concrete rail 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 blockout in the expansion joint segments and Abutment 1 backwall that contains the expansion joint device. Removal of the concrete and modular expansion joints may be phased to maintain traffic. These blockouts measure 2'-8" along the bridge alignment for the expansion joint segments as shown in Figure 2.38. The blockouts are 1'-5 1/2" deep for both Abutment 1 and Pier 4 Expansion Joints. It may be necessary to remove the sliding plates in the bridge railing 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 expansion joint segments that will need to be carefully cut 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 was damaged during concrete removal should be replaced with bars doweled and epoxied into the structure. The replacement expansion joint unit can then 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, the new concrete can be placed in the blockout and cured.

Replacement expansion joints may not need to have the same movement capacity as the original joints. This is because movements due to creep and shrinkage of the concrete are irreversible, and 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 survey sheet of this appendix can also be used to determine the appropriate width for setting the replacement joint for a given structure temperature.

4.7 Lighting, Electrical, and Other Components

The St. Anthony Falls (I-35W) Bridge includes various lighting applications as described herein. All electrical inspections, maintenance, and repairs are recommended to be done by a certified electrician. All maintenance and repairs shall be done in accordance with the National Electrical Code (NEC). Refer to sections 4.7.1.1 through 4.7.1.4 for additional information, requirements, and contact information.

4.7.1 Lighting and Electrical Systems

The location of power feeds for the St. Anthony Falls (I-35W) Bridge electrical service are discussed in Section 3.9 of this manual.

4.7.1.1 Electrical System Description

The electrical distribution service for bridge lighting is located on the north side. The distribution service feedpoint consists of meter, main disconnect, contactors, and circuit breakers enclosed in a cabinet and rated at 200A, 240/480V, 1 phase, 3 wire. From that point, 240/480 volt circuits are run to the bridge deck lighting and 240/480 feeders are brought into each box girder, at which a step-down transformer drops the voltage to a 120/240V system, which is fed to branch circuit panelboards for the interior box lighting and maintenance power. A second distribution service feedpoint rated at 200A, 240/480V, 1 phase, 3 wire is provided for power to the aesthetic lighting. Feeders are brought out to the two outer box girders and feed 240/480 volt circuit breaker panels to power the wing and pier lighting. Details for the feedpoints is located in Appendix D.

The electrical system consists of various lighting applications: maintenance lighting, roadway lighting, aesthetic box girder wing lighting, aesthetic pier up lighting, aesthetic gateway monument lighting, and plaza pathway lighting. Maintenance lighting is powered from the interior box panels at a single-phase voltage of 120V. Roadway and aesthetic lighting are powered at a single –phase voltage of 240V. The plaza pathway lighting is powered at a single-phase voltage of 120V.

Within the box girder, there are convenience receptacles available for maintenance work and repairs. They are spaced approximately 100' on center. Receptacles are alternately connected to 20A, 120V circuits.

4.7.1.2 Electrical Equipment Items

Roadway Lights: Bridge roadway lighting is located on the inside barrier side of the bridge and is powered from conduit in the railing. At each abutment, the lighting fixtures are located along the outside barrier. The fixtures have a white 6000k long-life Light Emitting Diodes (LED) source and are powered off of 240V circuits. They are manufactured by Beta LED, model BLD-ARE-T*-AA, The Edge LED Area Light – Type III & V. The fixtures are mounted on a 40' tall galvanized steel pole with a circular cross-section.

Maintenance Lights: Maintenance lighting is located inside the box girder. Each span consists of maintenance lighting along with convenience outlets. The maintenance lights and the convenience outlets are fed from the circuit breaker panels in the box girder. They consist of a 42 watt compact fluorescent lamp with a glass globe and die-cast aluminum guard. A 120V circuit powers the fixtures. They are manufactured by Lightolier Exceline/RL model CSD-UNV42PS/L.

Gateway Monument Lights: Gateway monument lights are located at each abutment with eight fixtures for each monument. The monument fixture is a KIM Lighting model DBF-18/150PMH240V/WH-P Metal Halide lamp. Fixtures have grazing type distribution for mounting close to the structure and operate at 240 volts.

Pier Up Lights: Pier lights are located at the base of the piers below the observation plazas. The pier fixture is a narrow graze optic type with 180 Luxeon© high flux Light Emitting Diodes (LED) and red, green, and blue sources. Fixtures have a horizontal rectangular beam projection type distribution and operate at 240 volts. The fixture is manufactured by TIR Systems Ltd., model Destiny SL.

Box Girder Wing Lights: Box Girder wing lights are also located under the outside box girder wing. Wing fixtures are either 45 deg flood or narrow spread, linear cove type distribution. Both fixtures use Luxeon© Light Emitting Diodes (LED) with sources in red, green, and blue. The fixture operates at 240 volts and is manufactured by TIR Systems Ltd., model Destiny CV.

Lighting Control: The lighting controls consist of lighting contactors and a photoeye. The photoeye controls all roadway lighting on the St. Anthony Falls (I-35W) Bridge. Aesthetic lighting is operated through a controller using DMX communications.

Maintenance lighting is controlled with 0-6 hour interval timers. Maintenance lighting is controlled by 3-way switches at each end of each span.

4.7.1.3 Electrical Maintenance

During the life of the electrical system on the St. Anthony Falls (I-35W) Bridge, electrical maintenance will need to be performed.

Inspection: An inspection shall be performed regularly on the electrical system. The inspection shall cover but not be limited to:

- Lamp inspection
- Fuse inspection
- Photoeye inspection

During the routine inspection, the lens on the photoeye may need to be cleaned of dirt and debris and realigned to achieve the desired time of operation.

During the regular inspections, if any lamps or fuses are not in working condition, a certified electrician or a qualified maintenance person shall replace the items as needed. To ensure safety is maintained during all maintenance and repairs, qualified personnel should perform all required tasks.

Lockdown Procedure: During any periods of time where maintenance work or repairs are done on the electrical system, the electrician or qualified maintenance person must follow proper lockdown procedures. To achieve proper lockdown procedures, follow OSHA standards for "Lockout/Tag Procedures".

4.7.1.4 Contacts for Electrical Items

Electrical Engineer:

TKDA, Inc.
444 Cedar Street, Suite 1500
Saint Paul, MN 55101-2140
Ph: (651) 292-4400
Fax: (651) 292-0083

Lighting Specialist:

Randy Burkett Lighting Design, Inc.
609 East Lockwood Avenue, Suite 201
St. Louis, Missouri 63119
Ph: (314) 961-6650
Fax: (314) 961-7640

4.7.2 Aesthetic Design Elements

Aesthetic design elements were used to increase the aesthetic quality of the bridge. The final "Visual Quality Management Manual" that was provided to Mn/DOT summarizes the decisions made by the Visual Quality Advisory Team. These elements include the TRI-MIX PLASTER MIX Cementitious Concrete Surfacer® bridge coating and the Gabion® Stone Masonry Wall along the approach roadways. Product Data Sheets for the paints applied to the bridge coating and railing are in Appendix E.

The Gabion® Stone Masonry Wall should be maintained according to the manufacturer's recommendations.

The color used for the TRI-MIX PLASTER MIX Cementitious Concrete Surfacer® bridge coating on St. Anthony Falls (I-35W) Bridge is:

"SNOWBOUND WHITE" (Sherwin Williams)

4.7.3 Boschung Anti-Icing System

The Anti-Icing system provided by Boschung presents its own maintenance requirements above that of the structural bridge system. It is recommended that the owner consult the specific Operation and Maintenance manual provided by the supplier, Boschung, to Mn/DOT for maintenance of the anti-icing system. A copy of the index for this manual is included in Appendix E.

To replace a spray disk (sometimes referred to as a "puck"), first determine what portion of the disk needs to be replaced. The upper portion of the disk can be replaced by simply removing the screws and attaching a new piece to the embedded disk bottom.

If the entire disk needs replacement, first completely remove the original disk components. After disabling the system, disconnect the stainless steel nipple that is threaded into the bottom of the disk and connects to the pressure tubing under the deck. Then remove the sealant and epoxy around the old disk, the disk itself, and the remaining epoxy under the disk. Remove all sealant and epoxy from the concrete blockout area, while being careful to avoid damaging the concrete. Repair any local concrete damage in accordance with the recommendations for spall repair. If the damaged concrete area extends into where the transverse post-tensioning tendons are located, a bridge engineer familiar with post-tensioned structures should be consulted. After removal of the old disk, install the new disk using the same procedures and materials as shown in the Boschung As-Built Plan Set (Drawing No. 2081-H-020) included in Appendix E.

4.7.4 Health Monitoring System

The bridge is instrumented with a state of the art Health Monitoring system to monitor the structural behavior of the bridge during its service life. A data acquisition system collects and stores the information via fiber optic cables from Mn/DOT's Operations Center and the University of Minnesota Department Of Engineering (U of MN). The gathered information is managed in a partnership among Mn/DOT, the Federal Highway Administration (FHWA) and the University of Minnesota (U of MN). Information obtained from these sensor systems will compliment routine bridge inspections providing a more in-depth understanding of the structure. This will assist Mn/DOT engineers with documenting conditions and making maintenance decisions. Additional information about the individual components of the system can be found in Appendix D. The following is description of the system components:

- Vibrating wire strain gauges embedded or mounted at select locations in the bridge superstructure, the drilled shafts and Pier 2.
- Temperature sensors in the superstructure.
- Accelerometers near the center of each span in each box girder record the structure's response to live loads.
- Sensors based on fiber optic cables located in the Span 2 box girder provide a means to collect strains over the longer gage lengths across the span and can be used to determine curvatures and bridge deformations.
- Sensors embedded in the top surface of the bridge deck concrete measure the potential for corrosion in sacrificial steel bars at various depths in the concrete below the deck surface.
- Linear potentiometers at the expansion joints measure longitudinal bridge movements.

Appendix A

Initial Inspection Results (Provided by MnDOT)

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

INTERSTATE 35W

Appendix B

Recording Forms & 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 the St. Anthony Falls (I-35W) Bridge, including survey sheets and information on how to calculate expected bearing and expansion joint movements.

B.2 Survey 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 and top slab anchor blocks, bearings, piers, expansion joints, and abutments. An index of the survey sheets available is included on page B.7. 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. Separate forms are provided for Spans 1 and 3, Span 2, and Span 4 for inspector convenience. 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 northbound or southbound bridge as well as either the interior or exterior girder. The bridge and girder to be inspected can be noted at the top of the page. The location of possible areas of concern can be referenced from abutments, pier centerlines, and/or segment joints.

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. Top and bottom slab anchor block survey sheets are also provided for anchor block inspection at the box girder areas that anchor top or bottom slab tendons.

Expansion joint and bearing survey 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.

Survey sheets for substructure inspection include abutment and pier survey sheets. The pier forms include the pier column, pier cap, and footings.

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 expansion joint and bearing survey sheets with the field data completed, in conjunction with Table B.1.

B.3.1 Sliding Bearings

The table labeled "Displacement Allowance" at the bottom of the guided and non-guided bearing survey 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 or from correlated readings from the bridge health monitoring system. The temperature of the bridge concrete can be determined using a concrete surface thermometer or from sensors in the health monitoring system. Temperatures should be recorded at a location at the approximate mid-height of the webs in the shade. Note that bridge temperatures can vary widely throughout the course of the day, so temperatures should be recorded at the same time the bearing plate dimensions are measured. It is recommended the average of at least 3 temperatures along the mid-height of the shaded web be used for the bridge temperature.

Next, look up the coefficient of 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 (in inches) due to a one-degree Fahrenheit change in temperature. The creep and shrinkage values are the expected remaining movement (in inches) for September in each year. 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 2036 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 coefficient from Column 1, and the formulas for Columns 2 and 3 as shown on the survey sheet. 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 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 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 survey 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, should be recorded on the survey sheet. Temperatures should be recorded at a location at the approximate mid-height of the webs in the shade. Note that bridge temperatures can vary widely 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 the average of at least 3 temperatures along the mid-height of the shaded web be used for the bridge temperature. Alternately, the dimensions “A” and “X”, as well as the bridge temperature, can be determined from correlated readings from the bridge health monitoring system.

The first item calculated is the amount that the joint is expected to open (in inches) from the current setting if the temperature drops to the design minimum of –30°F. This is calculated using the measured concrete temperature, the coefficient of thermal movement, and the Column 4 formula located just below the table. The result is in inches of joint opening. Write this value in Column 4 in the table. The anticipated joint closing for the maximum design temperature of 120°F is similarly calculated using the Column 5 formula below the table, and entered in Column 5.

The remaining amount of creep and shrinkage for the joint opening movement is found in Table B.1 and entered in Column 6. The values are given for September of each year. 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 2036 or later is zero.

The maximum anticipated joint opening is then calculated as the sum of the measured dimension “X” (Column 2), the joint opening for –30°F (Column 4), and the remaining creep and shrinkage movement (Column 6). Write this value in the seventh column of the table (Max.).

The minimum anticipated joint opening is calculated as the measured dimension "X" (Column 2) minus the joint opening for 120°F (Column 5), and the remaining creep and shrinkage movement (Column 6). Write this value in the last column of the table (Min.).

The maximum and minimum anticipated joint openings can then be compared with the maximum and minimum openings allowed for that particular joint, as shown on the form beneath the equations. Normally, the maximum opening will control because creep and shrinkage movement effects are permanent. If the anticipated values both fall between the allowable values, the joint has adequate movement capacity.

An example of these calculations is included for reference.

Table B.1 – Bearing & Expansion Joint Displacements

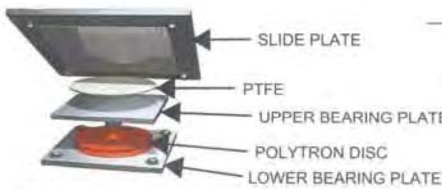
Location		Abut. 1NB	Abut. 1SB	Pier 4NB	Pier 4SB
Thermal Movement (IN./ °F)		**0.042**	**0.039**	**0.044**	**0.046**
Remaining Creep and Shrinkage Movement on September 15 (Inches)	2009	2.83	2.64	2.75	2.73
	2010	2.30	2.14	2.23	2.21
	2011	1.96	1.83	1.89	1.88
	2012	1.67	1.56	1.62	1.61
	2013	1.39	1.29	1.36	1.34
	2014	1.13	1.05	1.12	1.10
	2015	0.87	0.80	0.89	0.86
	2016	0.62	0.55	0.66	0.62
	2017	0.36	0.31	0.43	0.39
	2018	0.11	0.06	0.21	0.17
	2019	0.00	0.00	0.09	0.10
	2020	0.00	0.00	0.08	0.09
	2021	0.00	0.00	0.08	0.08
	2022	0.00	0.00	0.07	0.07
	2023	0.00	0.00	0.06	0.06
	2024	0.00	0.00	0.06	0.06
	2025	0.00	0.00	0.05	0.05
	2026	0.00	0.00	0.04	0.04
	2027	0.00	0.00	0.04	0.04
	2028	0.00	0.00	0.03	0.03
	2029	0.00	0.00	0.03	0.03
	2030	0.00	0.00	0.03	0.03
	2031	0.00	0.00	0.02	0.02
	2032	0.00	0.00	0.02	0.02
	2033	0.00	0.00	0.01	0.01
	2034	0.00	0.00	0.01	0.01
	2035	0.00	0.00	0.01	0.01
	2036	0.00	0.00	0.00	0.00

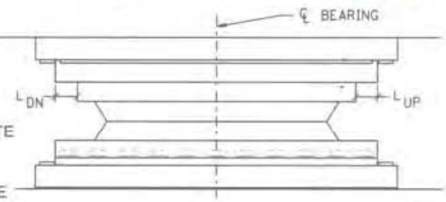
NOTE: Positive remaining creep and shrinkage values indicate superstructure movement toward final structure position (Day 10000).

I-35W - ST. ANTHONY FALLS BRIDGE

MULTI-DIRECTIONAL DISKTRON BEARING SURVEY SHEET

BEARING MANUFACTURER: R.J. WATSON INSPECTED BY: JKS
BRIDGE: NB / SB DATE INSPECTED: 9/15/2010
PIER / ABUT. NO.: 1 ORGANIZATION: FIGG
BEARING NO.: 4
AMBIENT TEMP. @ INSPECTION: 70 °F Example Only
CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): 60 °F





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

ELEMENT	CONDITION	COMMENTS
SOLE PLATE	G F P S	
SLIDE PLATE	G F P S	
PTFE	G F P S	
UPPER BEARING PLATE	G F P S	
POLYTRON DISC	G F P S	
LOWER BEARING PLATE	G F P S	
REMOVABLE BAR	G F P S	
PERMANENT BAR	G F P S	
MASONRY PLATE	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)							
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MEASURED DISP. DOWNSTATION AND UPSTATION		MOVEMENT REQUIRED	
①	②	③	④	L DN	L UP	⑤DN STA	⑥UP STA
0.039	3.51"	2.34"	2.14"	12	8	5.65"	2.34"

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

AT ABUTMENT 1: ② = (30°F + CONC. TEMP.) X ① ⑤ = ② + ④
③ = (120°F - CONC. TEMP.) X ① ⑥ = ③

AT PIER 4: ② = (120°F - CONC. TEMP.) X ① ⑤ = ②
③ = (30°F + CONC. TEMP.) X ① ⑥ = ③ + ④

COMMENTS: ______ CHECK: $L_{DN} \geq$ ⑤
CHECK: $L_{UP} \geq$ ⑥

BRIDGE NO.: 27409 (SB) / 27410 (NB) BRG-1

I-35W - ST. ANTHONY FALLS BRIDGE

EXPANSION JOINT SURVEY SHEET

JOINT MANUFACTURER: DS BROWN INSPECTED BY: JKS

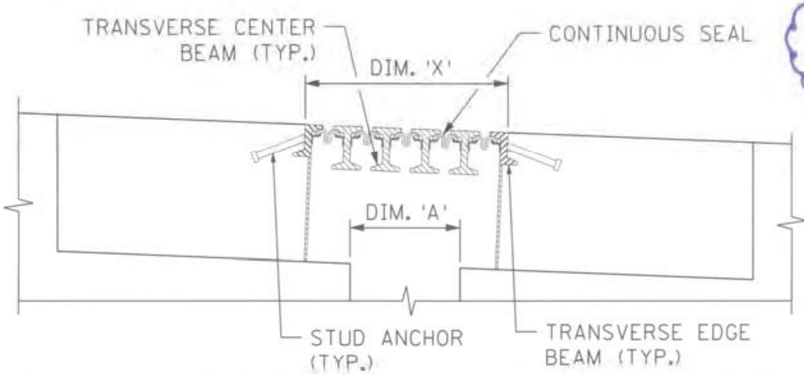
BRIDGE: NB / SB DATE INSPECTED: 9/15/2010

ABUT. 1 / PIER 4 ORGANIZATION: FIGG

AMBIENT TEMP. @ INSPECTION: 70 °F

CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): 60 °F

Example Only



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	
WELDS BETWEEN SUPPORT BARS & CENTER BEAMS	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)							
MEASURED DIM. 'A'	MEASURED DIM. 'X'	THERMAL MOVEMENT	TEMP. CORRECTION		REMAINING CR. & SH.	ANTICIPATED 'X'	
			FALL	RISE		MAX.	MIN.
1	2	3	4	5	6	2 + 4 + 6	2 - 5
7"	18"	0.044	3.96"	2.64"	2.23"	24.19"	15.36"

4 = (30°F + CONC. TEMP.) X 3

5 = (120°F - CONC. TEMP.) X 3

3 & 6 - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.1

CHECK: 1 $\geq$ 5

CHECK: $12\frac{1}{2}$" $\leq$ ANTICIPATED 'X' $\leq$ $27\frac{1}{2}$"

BRIDGE NO.: 27409 (SB) / 27410 (NB) EJ

SURVEY SHEETS AND CALCULATIONS

INDEX OF FORMS AVAILABLE

FORM	DESIGNATION
TYPICAL BOX GIRDER SURVEY SHEETS:	
CIP Spans 1 & 3 Interior Surfaces	INT-1
Precast Span 2 Interior Surfaces	INT-2
CIP Span 4NB Interior Surfaces	INT-3
CIP Span 4SB Interior Surfaces	INT-4
Spans 1 – 3 Top Deck Surfaces	EXT-1
Span 4 Top Deck Surfaces	EXT-2
Spans 1 – 3 Exterior Surfaces	EXT-3
Span 4 Exterior Surfaces	EXT-4
Top Slab Anchor Block	BLK-1
Bottom Slab Anchor Block	BLK-2
DIAPHRAGM SURVEY SHEETS:	
Abutment 1 Diaphragm & Box	DIA-1
Abutment 5NB Diaphragm	DIA-2
Abutment 5SB Diaphragm	DIA-3
Span 4NB Intermediate Diaphragm	DIA-4
Span 4SB Intermediate Diaphragm	DIA-5
Span 3 Expansion Joint Diaphragm & Box	DIA-6
Span 4NB Expansion Joint Diaphragm & Box	DIA-7
Span 4SB Expansion Joint Diaphragm & Box	DIA-8
Pier Diaphragm & Box Interior	DIA-9
Type I Deviation Diaphragm	DIA-10
Type II Deviation Diaphragm	DIA-11
BEARING AND EXPANSION JOINT SURVEY SHEETS:	
Multi-Directional Disktron Bearing	BRG-1
Uni-Directional Disktron Bearing	BRG-2
Fixed Disktron Bearing	BRG-3
Expansion Joint	EJ
SUBSTRUCTURE SURVEY SHEETS:	
Abutments 1 & 5	ABUT
Piers 2 & 3	PIER-2/3
Pier 4	PIER-4

I-35W - ST. ANTHONY FALLS BRIDGE

CIP SPANS 1 & 3 INTERIOR SURFACES SURVEY SHEET

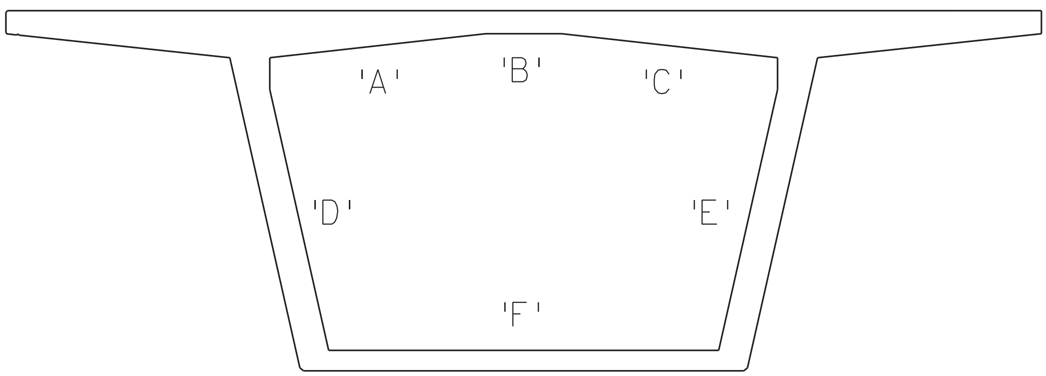
CIP SPAN NO.: 1 / 3 INSPECTED BY: ____________________

BRIDGE: NB / SB DATE INSPECTED: ____________________

BOX GIRDER: INT. / EXT. ORGANIZATION: ____________________

LOCATION: SHALLOW / INTERMEDIATE / DEEP

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



LOOKING UPSTATION

SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
'F'	

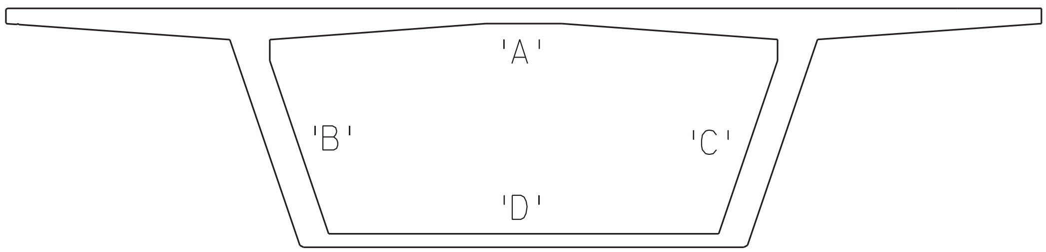
I-35W - ST. ANTHONY FALLS BRIDGE

PRECAST SPAN 2 INTERIOR SURFACES SURVEY SHEET

PRECAST SPAN NO.: 2 | INSPECTED BY: ____________________

BRIDGE: NB / SB | DATE INSPECTED: ____________________

BOX GIRDER: INT. / EXT. | ORGANIZATION: ____________________



LOOKING UPSTATION

SEG. # *	SURFACE INDEX:			
	'A'	'B'	'C'	'D'

* SEGMENT NUMBER NOTED AS DESCRIPTION PAINTED ON WEB OF SEGMENT

I-35W - ST. ANTHONY FALLS BRIDGE

CIP SPAN 4NB INTERIOR SURFACES SURVEY SHEET

INSPECTED BY: ____________________

CIP SPAN NO.: 4NB

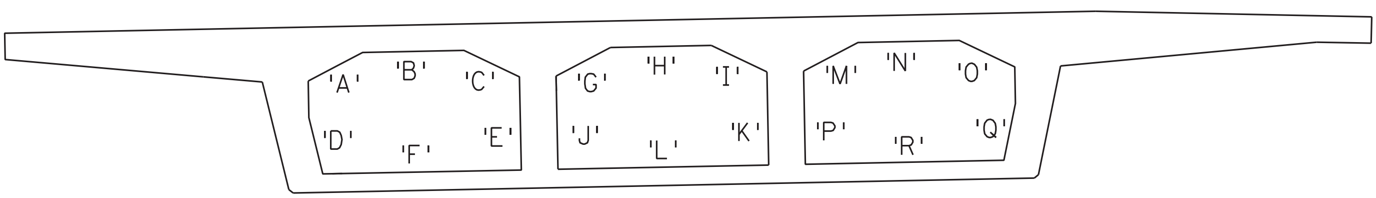
DATE INSPECTED: ____________________

BOX GIRDER: INT. / EXT.

ORGANIZATION: ____________________

LOCATION: SHALLOW/ INTERMEDIATE / DEEP

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



LOOKING UPSTATION

SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
'F'	
'G'	
'H'	
'I'	
'J'	
'K'	
'L'	
'M'	
'N'	
'O'	
'P'	
'Q'	
'R'	

I-35W - ST. ANTHONY FALLS BRIDGE

CIP SPAN 4SB INTERIOR SURFACES SURVEY SHEET

INSPECTED BY: ____________________

CIP SPAN NO.: 4SB

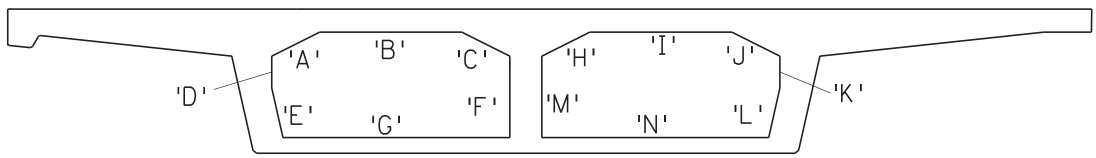
DATE INSPECTED: ____________________

BOX GIRDER: INT. / EXT.

ORGANIZATION: ____________________

LOCATION: SHALLOW / INTERMEDIATE / DEEP

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



LOOKING UPSTATION

SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
'F'	
'G'	
'H'	
'I'	
'J'	
'K'	
'L'	
'M'	
'N'	

I-35W - ST. ANTHONY FALLS BRIDGE

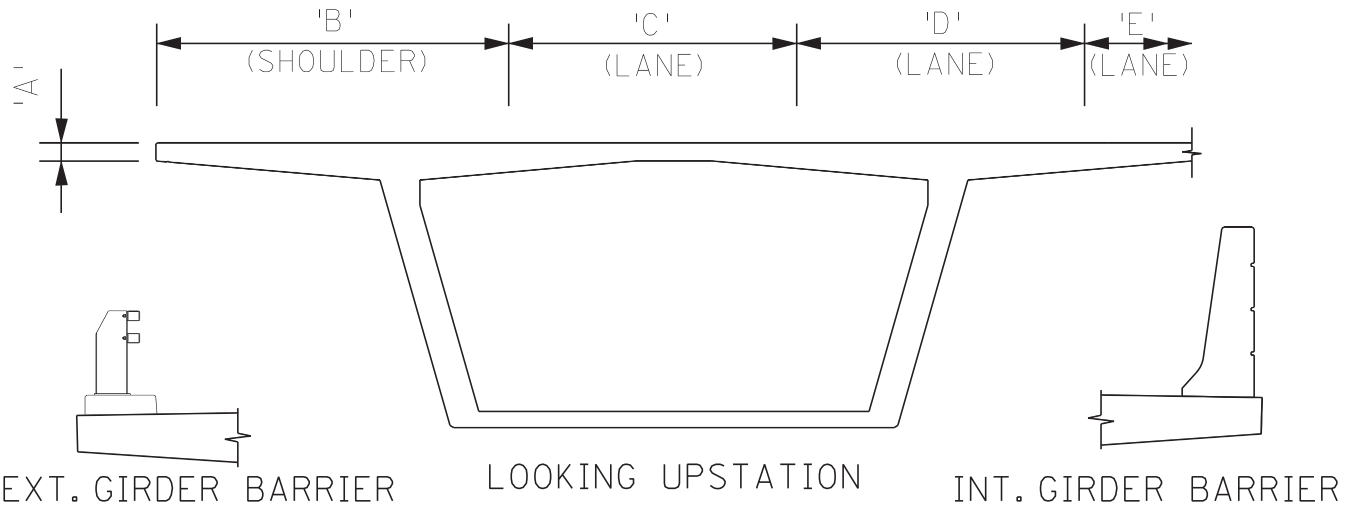
SPANS 1 - 3 TOP DECK SURFACES SURVEY SHEET

CIP SPAN NO.: 1 / 2 / 3 INSPECTED BY: ____________

BRIDGE: NB / SB DATE INSPECTED: ____________

BOX GIRDER: INT. / EXT. ORGANIZATION: ____________

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



SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
BARRIER	

I-35W - ST. ANTHONY FALLS BRIDGE

SPAN 4 TOP DECK SURFACES SURVEY SHEET

INSPECTED BY: ______________________

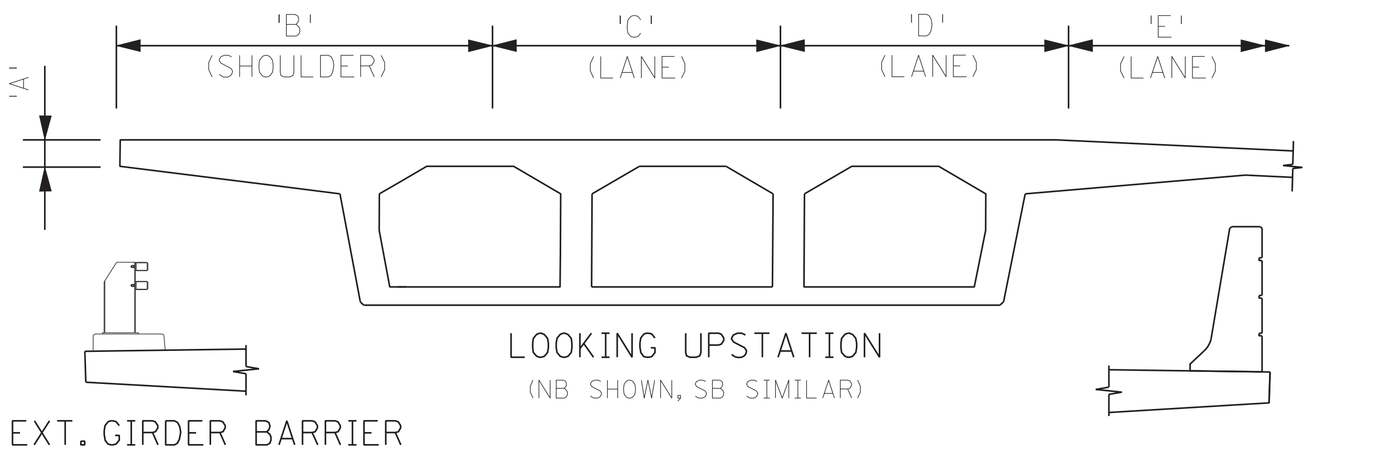
BRIDGE: NB / SB

DATE INSPECTED: ______________________

BOX GIRDER: INT. / EXT.

ORGANIZATION: ______________________

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



SURFACE INDEX:

'A'	
'B'	
'C'	
'D'	
'E'	
BARRIER	

I-35W - ST. ANTHONY FALLS BRIDGE

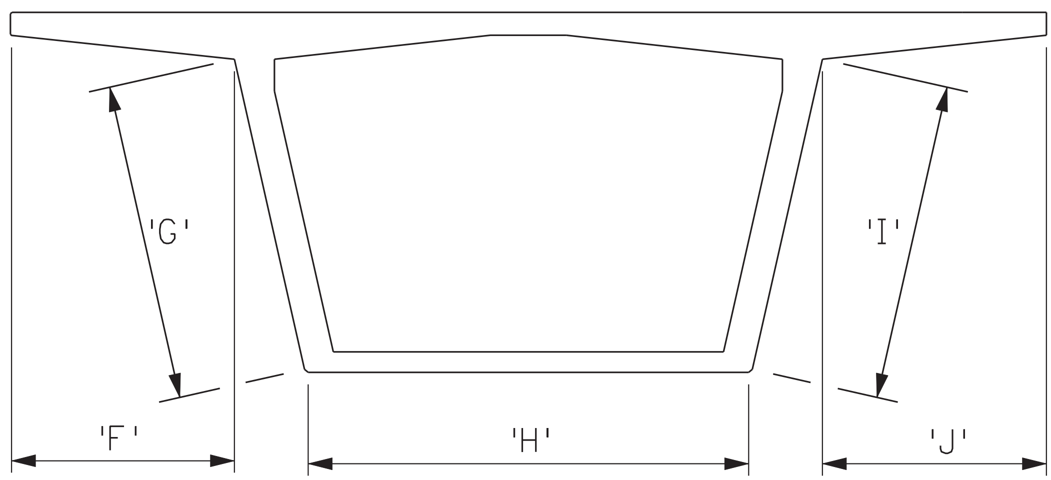
SPANS 1 - 3 EXTERIOR SURFACES SURVEY SHEET

CIP SPAN NO.: 1 / 2 / 3 INSPECTED BY: ____________

BRIDGE: NB / SB DATE INSPECTED: ____________

BOX GIRDER: INT. / EXT. ORGANIZATION: ____________

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



LOOKING UPSTATION

SURFACE INDEX:

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

I-35W - ST. ANTHONY FALLS BRIDGE

SPAN 4 EXTERIOR SURFACES SURVEY SHEET

INSPECTED BY: ____________________

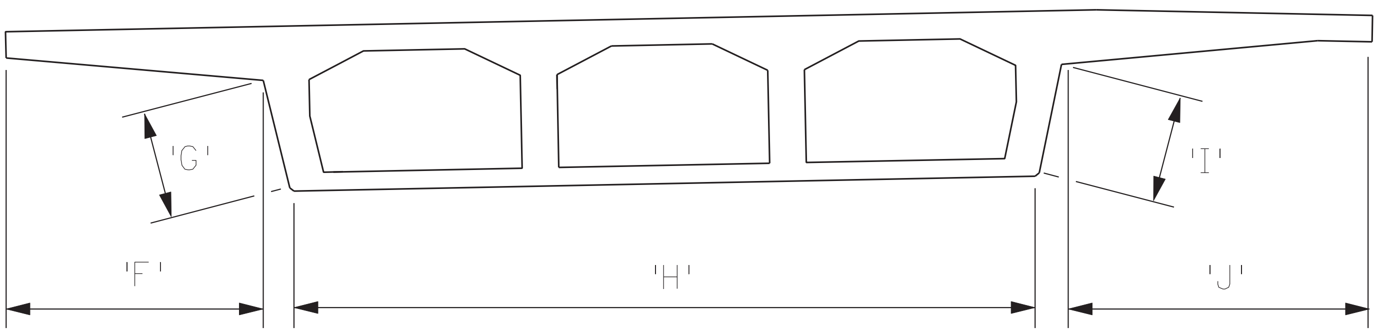
BRIDGE: NB / SB

DATE INSPECTED: ____________________

BOX GIRDER: INT. / EXT.

ORGANIZATION: ____________________

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



LOOKING UPSTATION
(NB SHOWN, SB SIMILAR)

SURFACE INDEX:

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

I-35W - ST. ANTHONY FALLS BRIDGE

TOP SLAB ANCHOR BLOCK SURVEY SHEET

SPAN NO.: ______

INSPECTED BY: ______________________

BRIDGE: NB / SB

DATE INSPECTED: ______________________

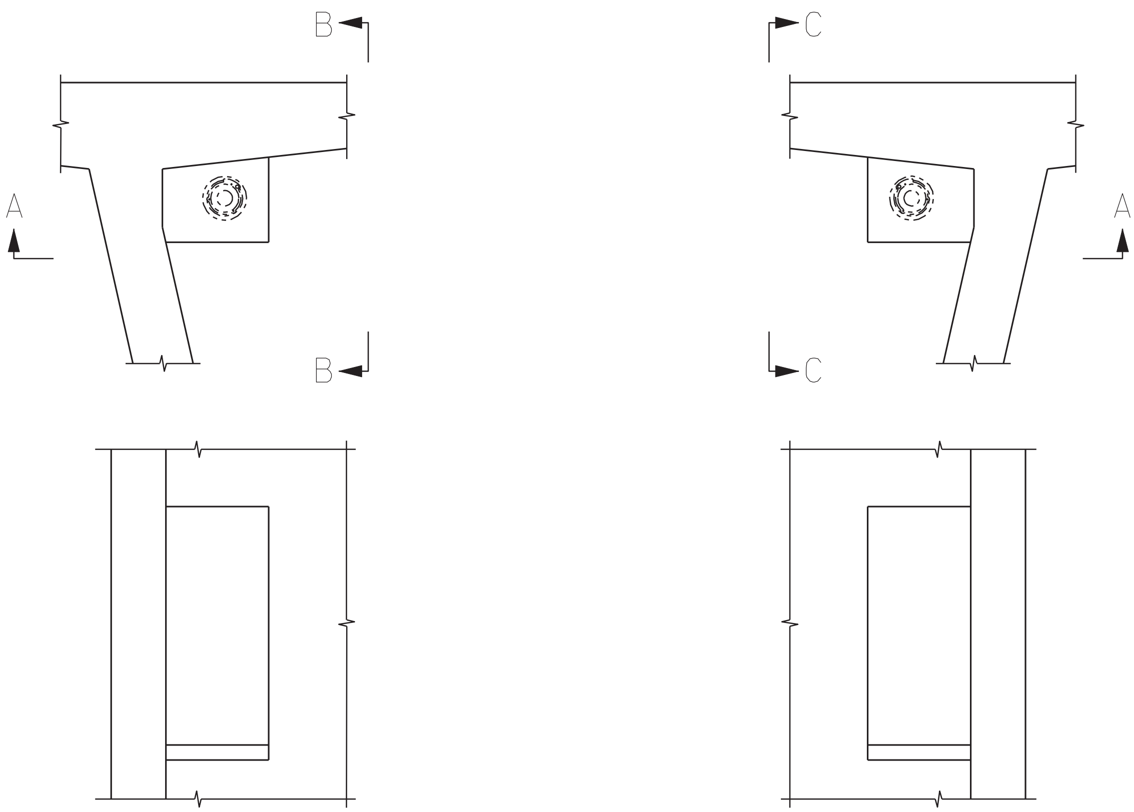
BOX GIRDER: INT. / EXT.

ORGANIZATION: ______________________

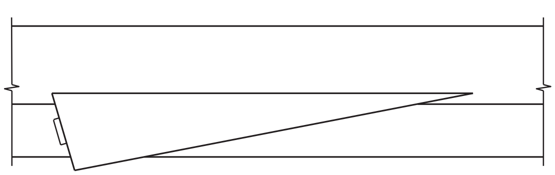
LOCATION: SHALLOW / INTERMEDIATE / DEEP

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

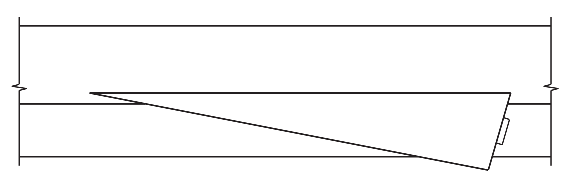
ANCHOR NO.: ______



VIEW A-A



VIEW B-B



VIEW C-C

I-35W - ST. ANTHONY FALLS BRIDGE

BOTTOM SLAB ANCHOR BLOCK SURVEY SHEET

SPAN NO.: ______ INSPECTED BY: ______________________

BRIDGE: NB / SB DATE INSPECTED: ______________________

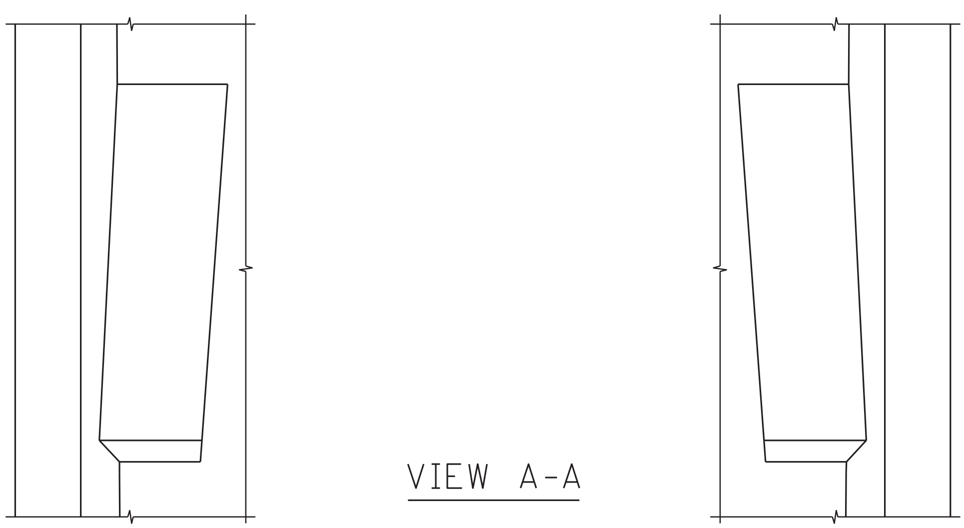
BOX GIRDER: INT. / EXT. ORGANIZATION: ______________________

LOCATION: SHALLOW / INTERMEDIATE / DEEP

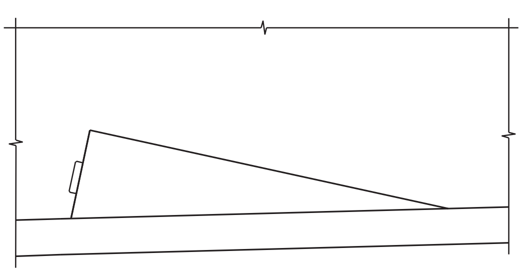
DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ______________________

ANCHOR NO.: ______

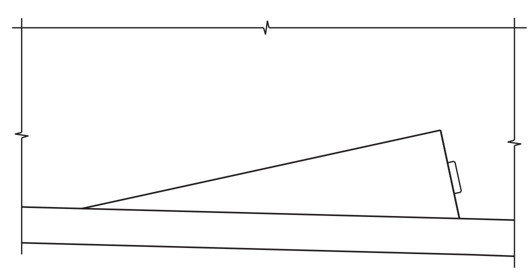
ELEVATION



VIEW A-A



VIEW B-B



VIEW C-C

I-35W - ST. ANTHONY FALLS BRIDGE

ABUTMENT 1 DIAPHRAGM & BOX SURVEY SHEET

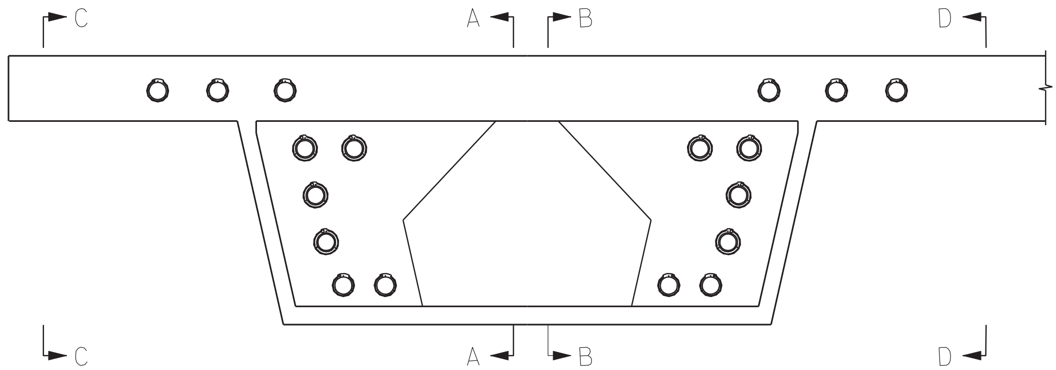
INSPECTED BY: ______________________

BRIDGE: NB / SB

DATE INSPECTED: ______________________

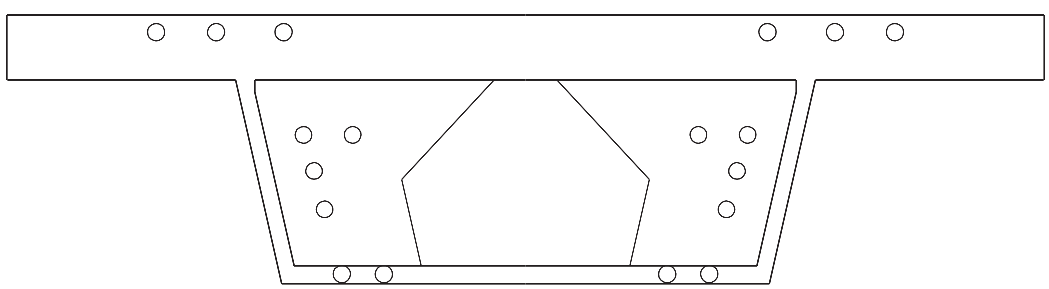
BOX GIRDER: INT. / EXT.

ORGANIZATION: ______________________



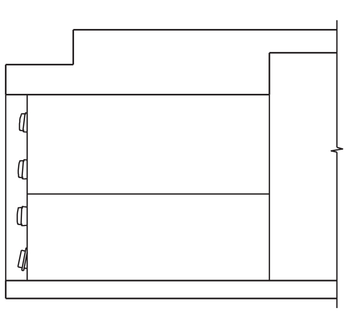
ELEVATION

(LOOKING TOWARD THE BRIDGE)

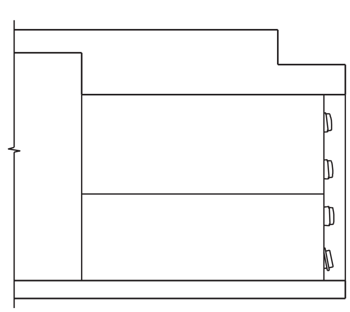


ELEVATION

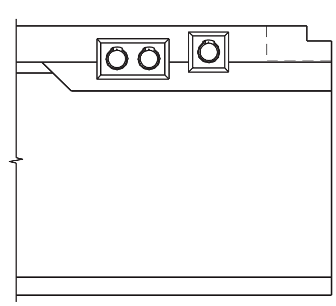
(LOOKING TOWARD THE ABUTMENT)



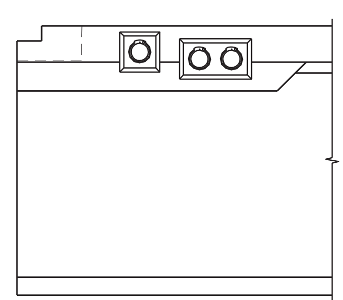
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

I-35W - ST. ANTHONY FALLS BRIDGE

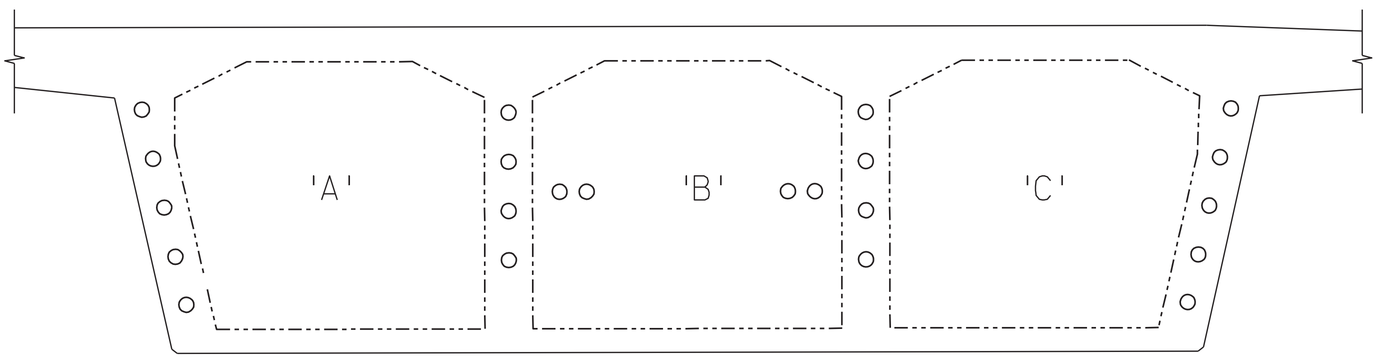
ABUTMENT 5NB DIAPHRAGM SURVEY SHEET

BOX GIRDER: INT. / EXT.

INSPECTED BY: ______________________

DATE INSPECTED: ______________________

ORGANIZATION: ______________________



ELEVATION

(LOOKING UPSTATION)

SURFACE INDEX:

'A'	
'B'	
'C'	

I-35W - ST. ANTHONY FALLS BRIDGE

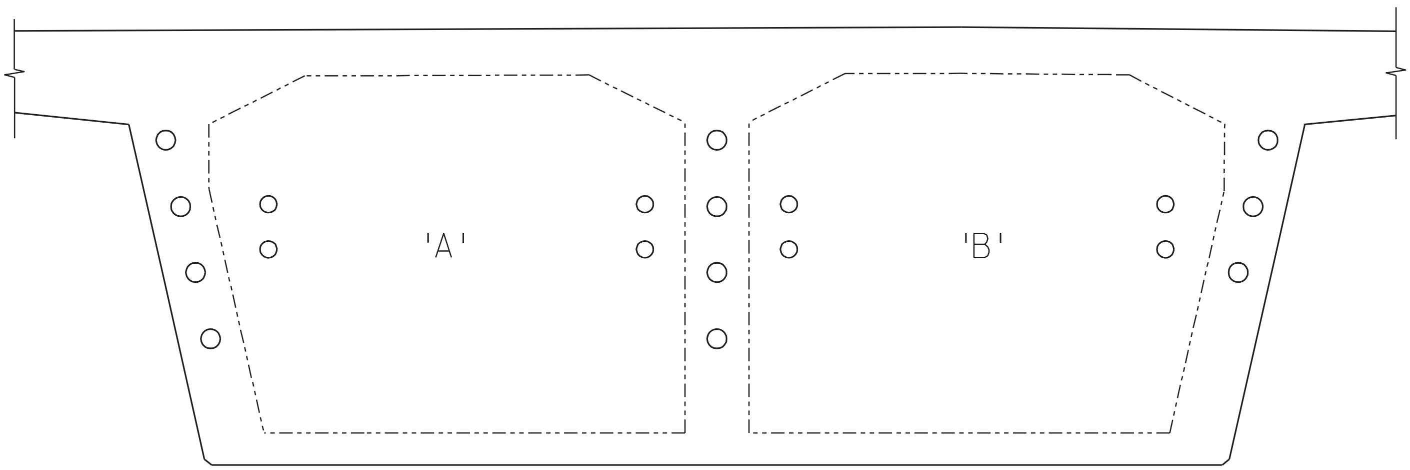
ABUTMENT 5SB DIAPHRAGM SURVEY SHEET

BOX GIRDER: INT. / EXT.

INSPECTED BY: ______________________

DATE INSPECTED: ______________________

ORGANIZATION: ______________________



ELEVATION
(LOOKING UPSTATION)

SURFACE INDEX:

'A'	
'B'	

I-35W - ST. ANTHONY FALLS BRIDGE

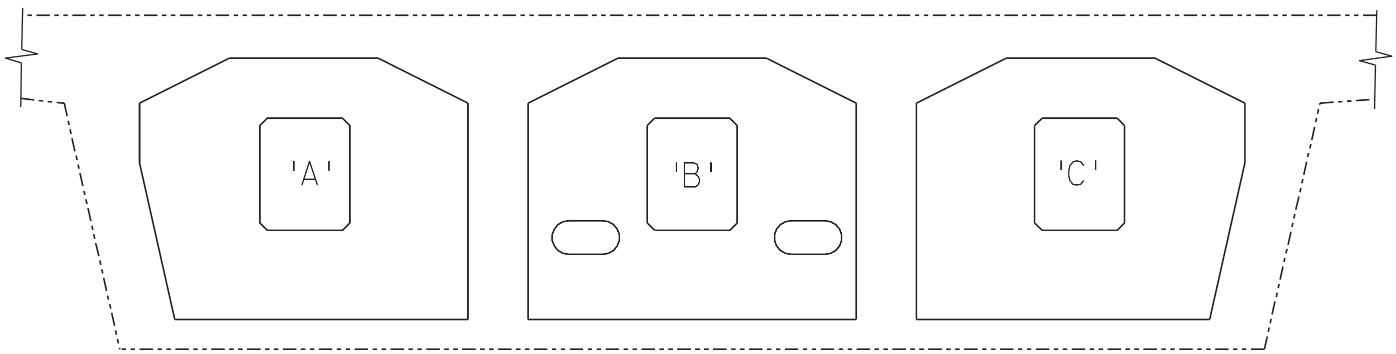
SPAN 4NB INTERMEDIATE DIAPHRAGM SURVEY SHEET

DIA.: A / B / C / D / E

INSPECTED BY: ____________________

DATE INSPECTED: ____________________

ORGANIZATION: ____________________



ELEVATION
(LOOKING UPSTATION)

SURFACE INDEX:

'A'	
'B'	
'C'	

I-35W - ST. ANTHONY FALLS BRIDGE

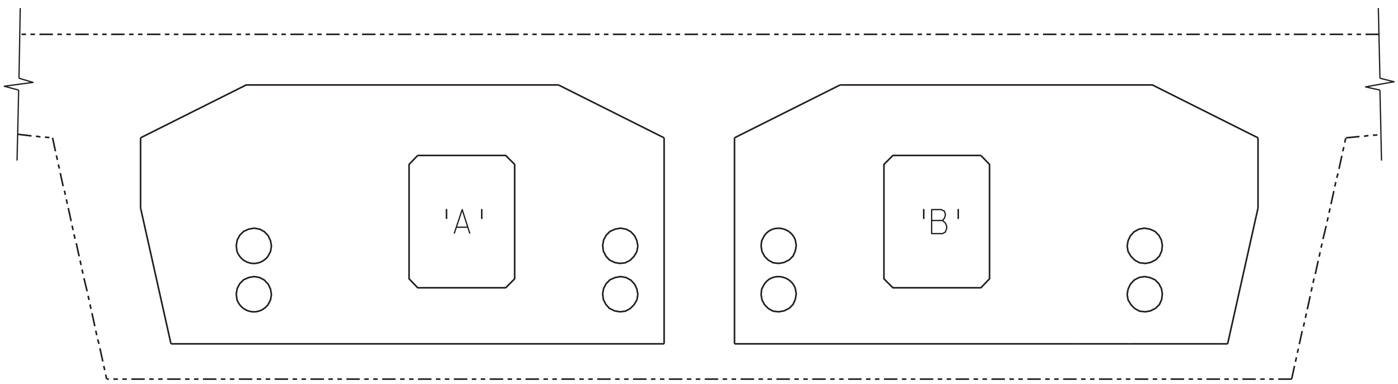
SPAN 4SB INTERMEDIATE DIAPHRAGM SURVEY SHEET

DIA.: F / G / H

INSPECTED BY: ____________________

DATE INSPECTED: ____________________

ORGANIZATION: ____________________



ELEVATION

(LOOKING UPSTATION)

SURFACE INDEX:

'A'	
'B'	

I-35W - ST. ANTHONY FALLS BRIDGE

SPAN 3 EJ DIAPHRAGM & BOX SURVEY SHEET

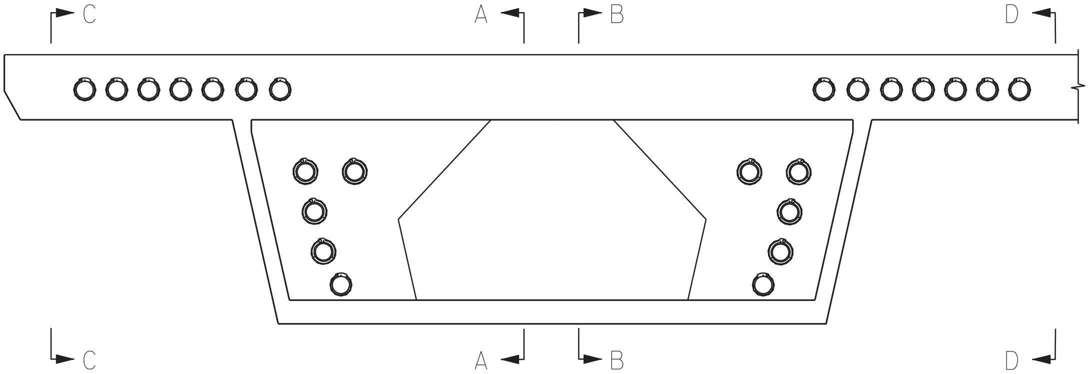
INSPECTED BY: ____________________

BRIDGE: NB / SB

DATE INSPECTED: ____________________

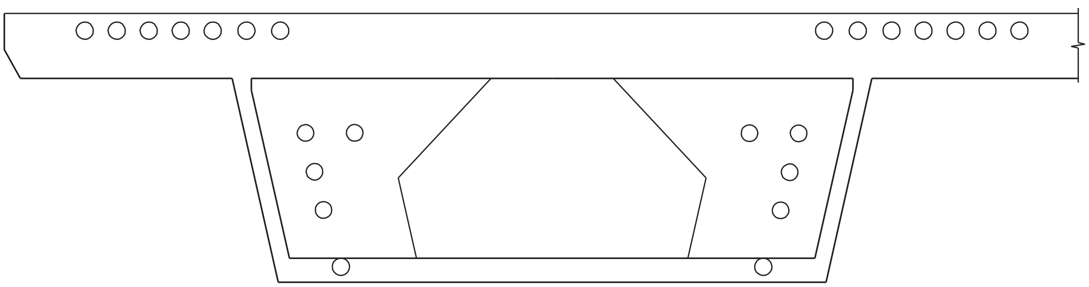
BOX GIRDER: INT. / EXT.

ORGANIZATION: ____________________



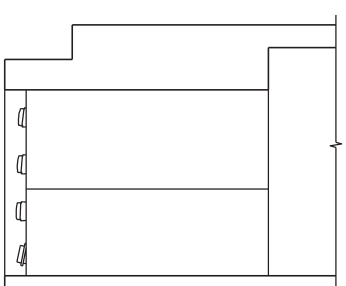
ELEVATION

(LOOKING TOWARD SPAN)

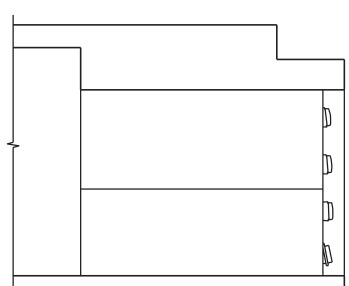


ELEVATION

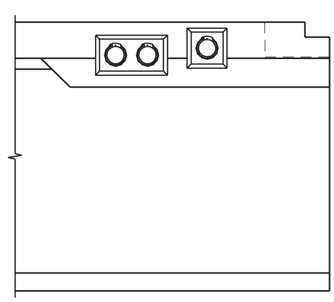
(LOOKING TOWARD JOINT)



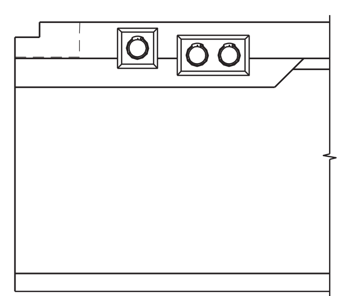
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

I-35W - ST. ANTHONY FALLS BRIDGE

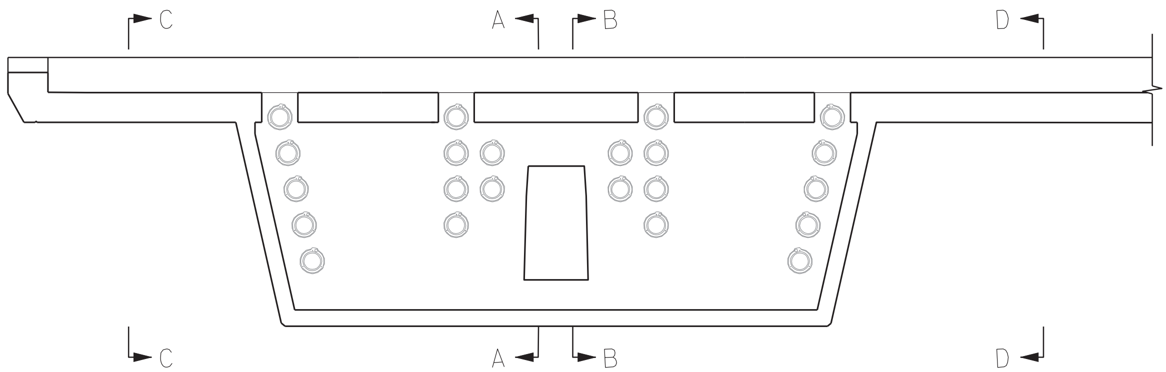
SPAN 4NB EJ DIAPHRAGM & BOX SURVEY SHEET

INSPECTED BY: ______________________

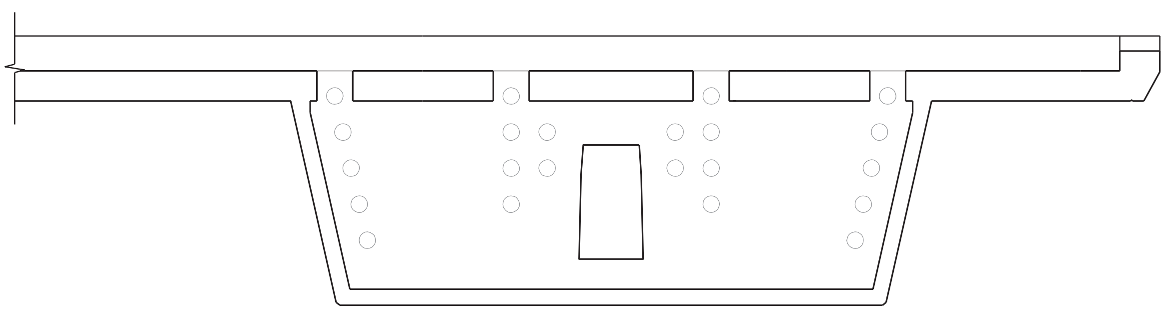
DATE INSPECTED: ______________________

BOX GIRDER: INT. / EXT.

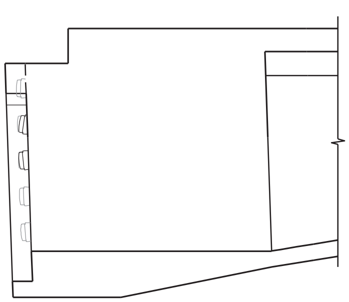
ORGANIZATION: ______________________



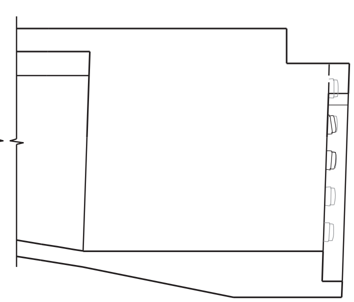
ELEVATION
(LOOKING TOWARD SPAN 4NB)



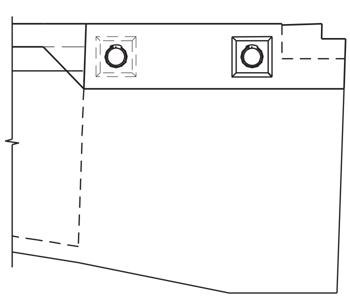
ELEVATION
(LOOKING TOWARD SPAN 3NB)



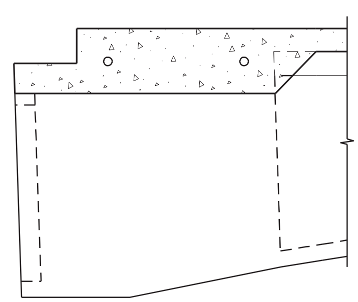
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

I-35W - ST. ANTHONY FALLS BRIDGE

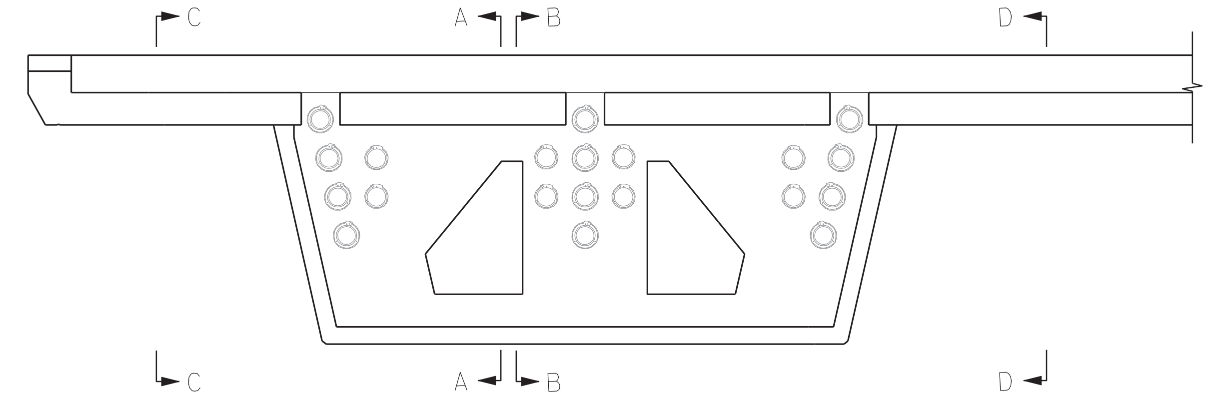
SPAN 4SB EJ DIAPHRAGM & BOX SURVEY SHEET

INSPECTED BY: ______________________

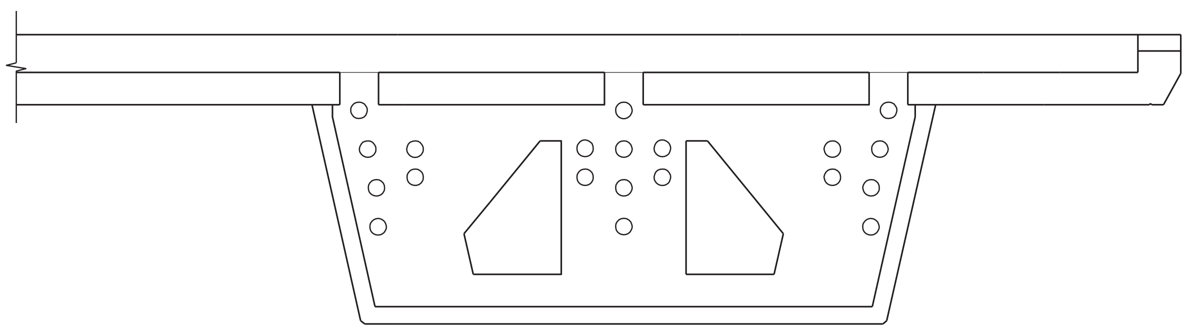
DATE INSPECTED: ______________________

BOX GIRDER: INT. / EXT.

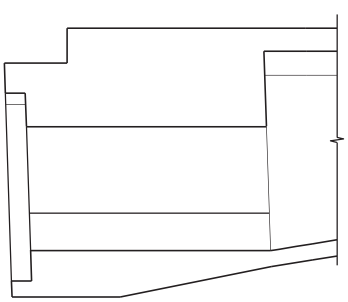
ORGANIZATION: ______________________



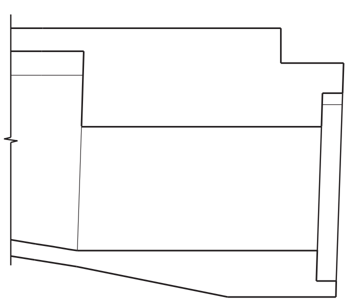
ELEVATION
(LOOKING TOWARD SPAN 4SB)



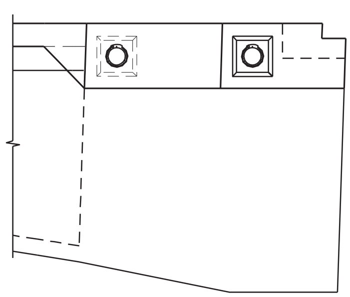
ELEVATION
(LOOKING TOWARD SPAN 3SB)



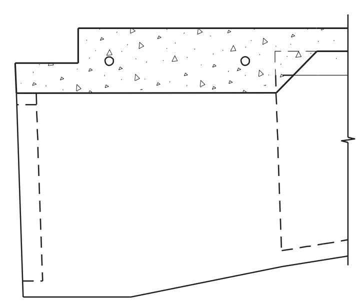
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

I-35W - ST. ANTHONY FALLS BRIDGE

PIER DIAPHRAGM & BOX INTERIOR SURVEY SHEEET

PIER NO.: ______

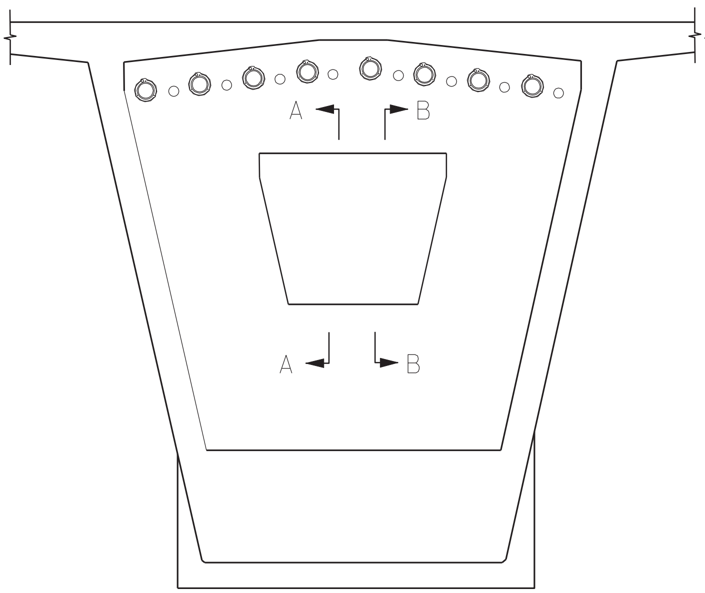
BRIDGE: NB / SB

BOX GIRDER: INT. / EXT.

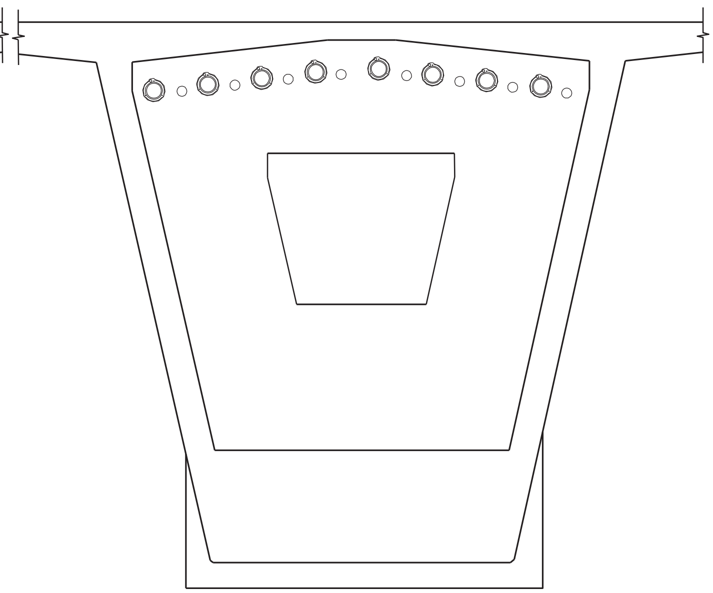
INSPECTED BY: ____________________

DATE INSPECTED: ____________________

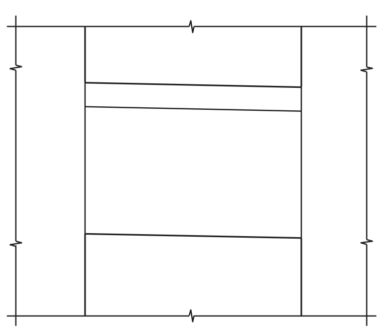
ORGANIZATION: ____________________



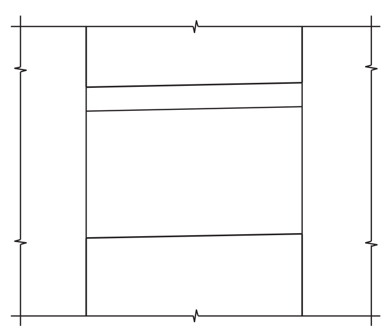
ELEVATION
(UPSTATION)



ELEVATION
(DOWNSTATION)



SECTION A-A



SECTION B-B

I-35W - ST. ANTHONY FALLS BRIDGE

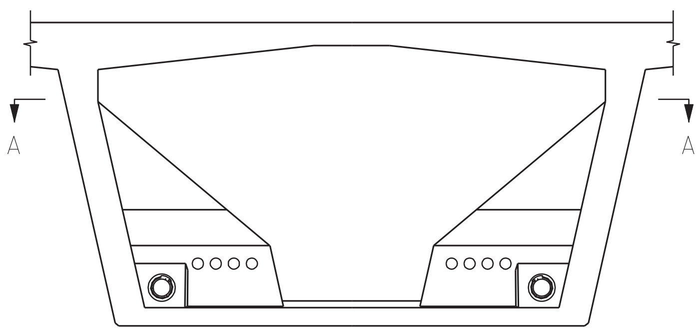
TYPE I DEVIATION DIAPHRAGM SURVEY SHEET

SPAN NO.: ____________ INSPECTED BY: ________________________

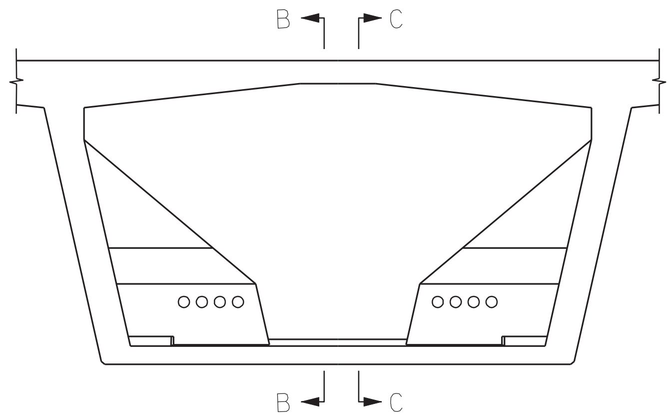
BRIDGE: NB / SB DATE INSPECTED: ________________________

BOX GIRDER: INT. / EXT. ORGANIZATION: ________________________

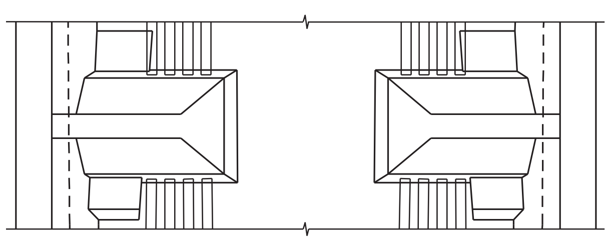
DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ____________________



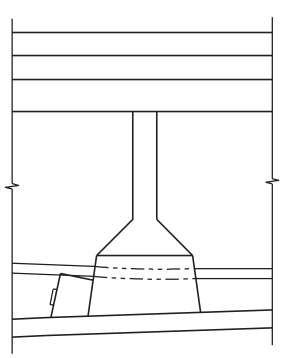
ELEVATION
(LOOKING UP STA / DWN STA)



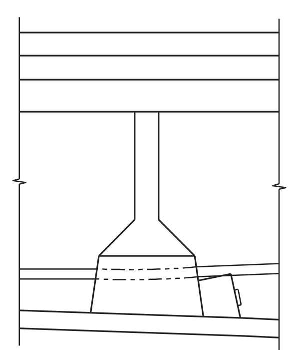
ELEVATION
(LOOKING DWN STA / UP STA)



SECTION A-A



SECTION B-B



SECTION C-C

I-35W - ST. ANTHONY FALLS BRIDGE

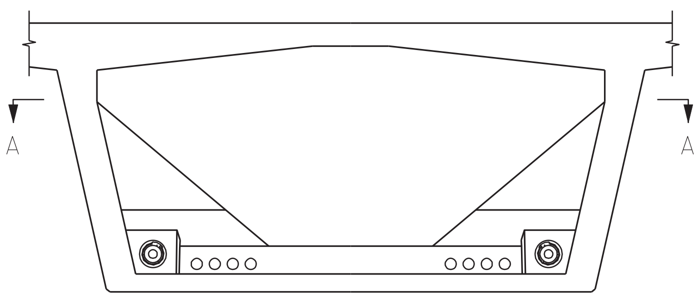
TYPE II DEVIATION DIAPHRAGM SURVEY SHEET

SPAN NO.: ____________ INSPECTED BY: ________________________

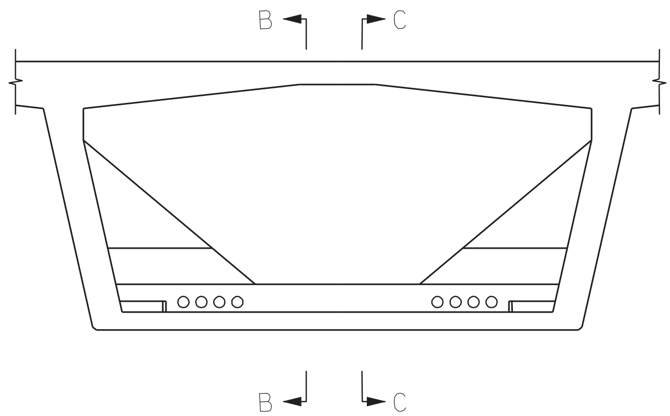
BRIDGE: NB / SB DATE INSPECTED: ______________________

BOX GIRDER: INT. / EXT. ORGANIZATION: ________________________

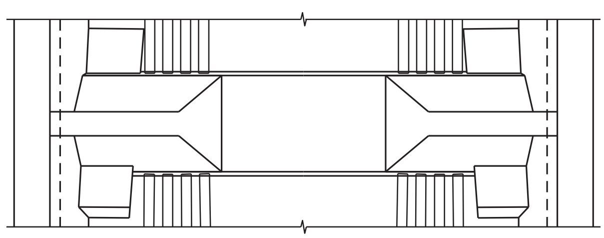
DIST. FROM: EXP. JT. / PIER (UP STA. / DWN STA.): ____________________



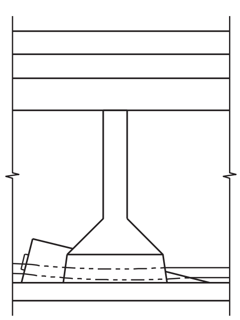
ELEVATION
(LOOKING UP STA / DWN STA)



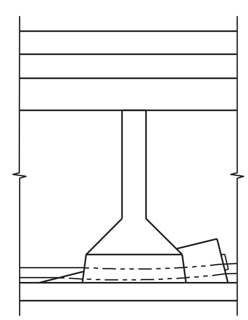
ELEVATION
(LOOKING DWN STA / UP STA)



SECTION A-A



SECTION B-B



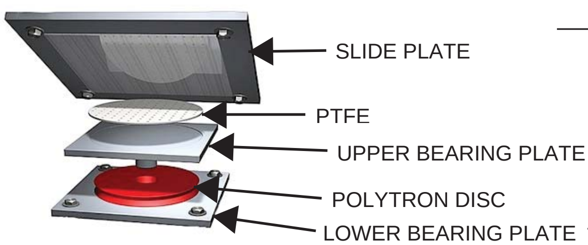
SECTION C-C

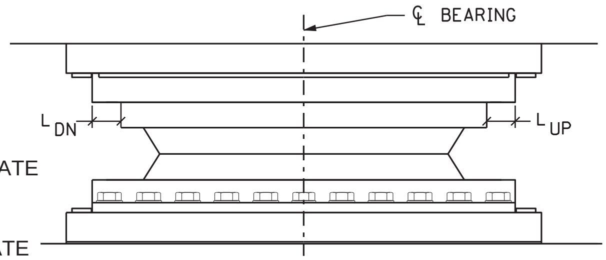
I-35W - ST. ANTHONY FALLS BRIDGE

MULTI-DIRECTIONAL DISKTRON BEARING SURVEY SHEET

BEARING MANUFACTURER: R.J. WATSON
BRIDGE: NB / SB
PIER / ABUT. NO.: ________
BEARING NO.: ________
AMBIENT TEMP. @ INSPECTION: ________°F
CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): ________°F

INSPECTED BY: ________________
DATE INSPECTED: ________________
ORGANIZATION: ________________





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

ELEMENT	CONDITION	COMMENTS
SOLE PLATE	G F P S	
SLIDE PLATE	G F P S	
PTFE	G F P S	
UPPER BEARING PLATE	G F P S	
POLYTRON DISC	G F P S	
LOWER BEARING PLATE	G F P S	
REMOVABLE BAR	G F P S	
PERMANENT BAR	G F P S	
MASONRY PLATE	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)							
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MEASURED DISP. DOWNSTATION AND UPSTATION		MOVEMENT REQUIRED	
1	2	3	4	L_{DN}	L_{UP}	5 DN STA	6 UP STA

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

AT ABUTMENT 1: 2 = (30°F + CONC. TEMP.) X 1   5 = 2 + 4
3 = (120°F - CONC. TEMP.) X 1   6 = 3

AT PIER 4: 2 = (120°F - CONC. TEMP.) X 1   5 = 2
3 = (30°F + CONC. TEMP.) X 1   6 = 3 + 4

COMMENTS: ____
____

CHECK: $L_{DN} \geq$ 5
CHECK: $L_{UP} \geq$ 6

I-35W - ST. ANTHONY FALLS BRIDGE

UNI-DIRECTIONAL DISKTRON BEARING SURVEY SHEET

BEARING MANUFACTURER: R.J. WATSON INSPECTED BY: ____________________

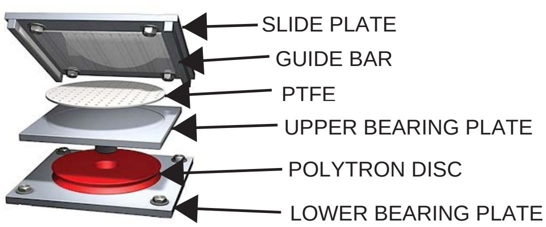
BRIDGE: NB / SB DATE INSPECTED: ____________________

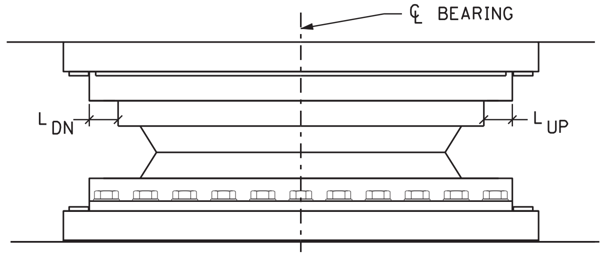
PIER / ABUT. NO.: ________ ORGANIZATION: ____________________

BEARING NO.: ________

AMBIENT TEMP. @ INSPECTION: ________ °F

CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): ________ °F





CONDITION LEGEND

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

ELEMENT	CONDITION	COMMENTS
SOLE PLATE	G F P S	
SLIDE PLATE	G F P S	
PTFE	G F P S	
UPPER BEARING PLATE	G F P S	
GUIDE BAR	G F P S	
POLYTRON DISC	G F P S	
LOWER BEARING PLATE	G F P S	
REMOVABLE BAR	G F P S	
PERMANENT BAR	G F P S	
MASONRY PLATE	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)							
THERMAL MOVEMENT	TEMP. MOVEMENT		REMAINING CR. & SH.	MEASURED DISP. DOWNSTATION AND UPSTATION		MOVEMENT REQUIRED	
1	2	3	4	L_{DN}	L_{UP}	5 DN STA	6 UP STA

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

AT ABUTMENT 1: 2 = (30°F + CONC. TEMP.) X 1 5 = 2 + 4

3 = (120°F - CONC. TEMP.) X 1 6 = 3

AT PIER 4: 2 = (120°F - CONC. TEMP.) X 1 5 = 2

3 = (30°F + CONC. TEMP.) X 1 6 = 3 + 4

COMMENTS: ______________________________

____

CHECK: $L_{DN} \geq$ 5

CHECK: $L_{UP} \geq$ 6

BRIDGE NO.: 27409 (SB) / 27410 (NB) BRG-2

I-35W - ST. ANTHONY FALLS BRIDGE

FIXED DISKTRON BEARING SURVEY SHEET

BEARING MANUFACTURER: R.J. WATSON INSPECTED BY: ____________________

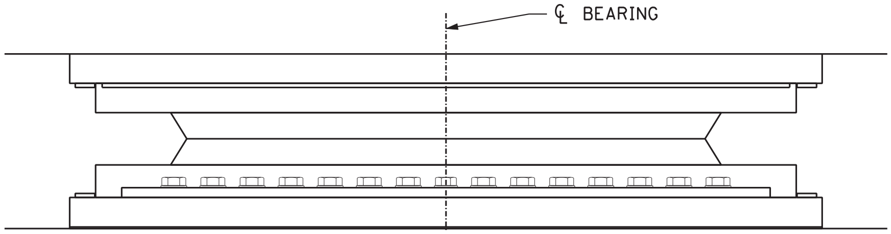
BRIDGE: NB / SB DATE INSPECTED: ____________________

PIER NO.: ________ ORGANIZATION: ____________________

BEARING NO.: ________

AMBIENT TEMP. @ INSPECTION: ________°F

CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): ________°F



CONDITION LEGEND

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

ELEMENT	CONDITION	COMMENTS
SOLE PLATE	G F P S	
UPPER BEARING PLATE	G F P S	
POLYTRON DISC	G F P S	
LOWER BEARING PLATE	G F P S	
REMOVABLE BAR	G F P S	
LOWER MASONRY PLATE	G F P S	
PERMANENT BARS	G F P S	

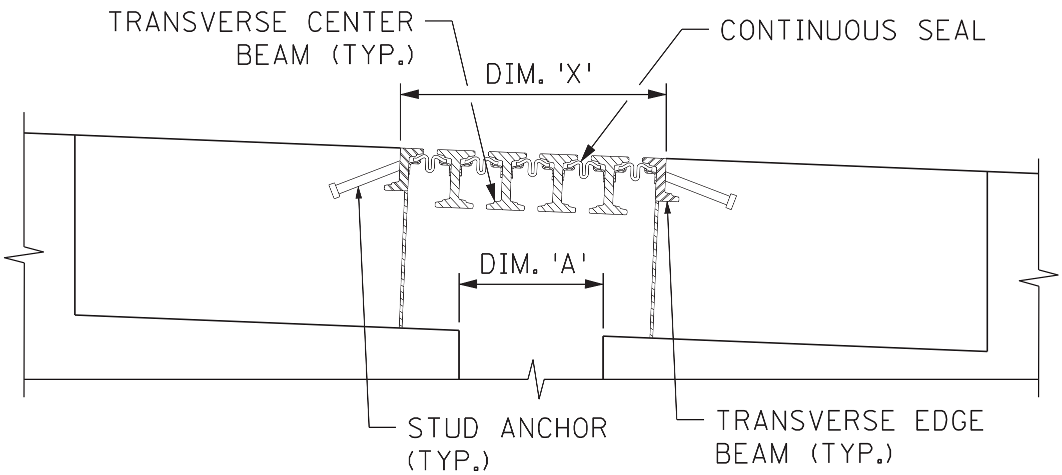
COMMENTS: ____________________

I-35W - ST. ANTHONY FALLS BRIDGE

EXPANSION JOINT SURVEY SHEET

JOINT MANUFACTURER: DS BROWN
BRIDGE: NB / SB
ABUT. 1 / PIER 4
AMBIENT TEMP. @ INSPECTION: ________ °F
CONCRETE TEMP. @ INSPECTION - MID WEB (IN SHADE): ________ °F

INSPECTED BY: ____________________
DATE INSPECTED: ____________________
ORGANIZATION: ____________________



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	
WELDS BETWEEN SUPPORT BARS & CENTER BEAMS	G F P S	

DISPLACEMENT ALLOWANCE (INCHES)							
MEASURED DIM. 'A'	MEASURED DIM. 'X'	THERMAL MOVEMENT	TEMP. CORRECTION		REMAINING CR. & SH.	ANTICIPATED 'X'	
			FALL	RISE		MAX.	MIN.
1	2	3	4	5	6	2 + 4 + 6	2 - 5

4 = (30°F + CONC. TEMP.) X 3

5 = (120°F - CONC. TEMP.) X 3

3 & 6 - SEE INSPECTION & MAINTENANCE MANUAL TABLE B.1

CHECK: 1 $\geq$ 5

CHECK: $12\frac{1}{2}$" $\leq$ ANTICIPATED 'X' $\leq$ $27\frac{1}{2}$"

I-35W - ST. ANTHONY FALLS BRIDGE

ABUTMENT SURVEY SHEET

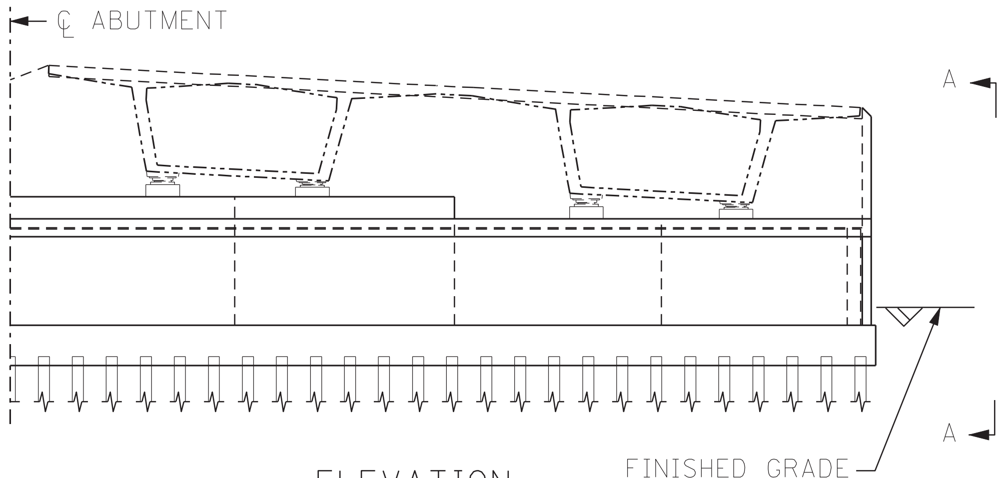
ABUTMENT NO.: 1 / 5

BRIDGE: NB / SB

INSPECTED BY: ______________________

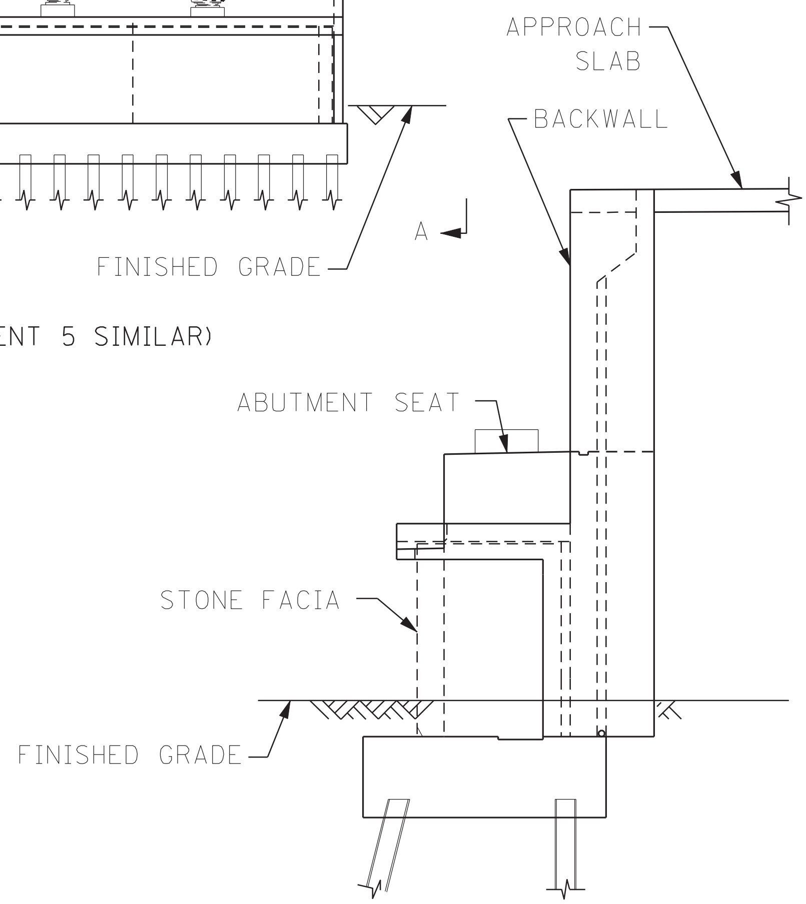
DATE INSPECTED: ______________________

ORGANIZATION: ______________________



ELEVATION

(ABUTMENT 1 SHOWN, ABUTMENT 5 SIMILAR)



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	

I-35W - ST. ANTHONY FALLS BRIDGE

PIERS 2 & 3 SURVEY SHEET

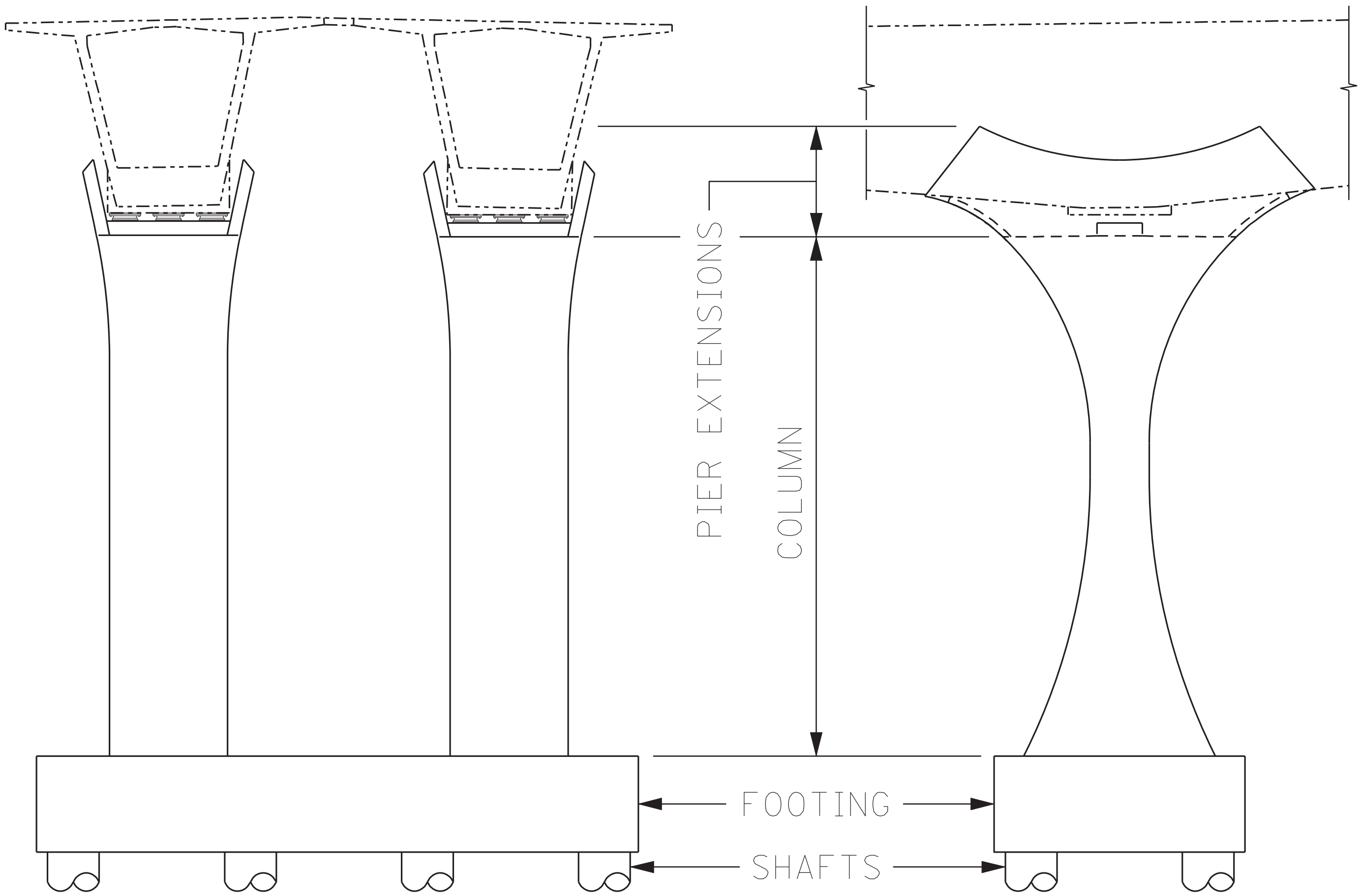
PIER NO.: 2 / 3

BRIDGE: NB / SB

INSPECTED BY: ______________________

DATE INSPECTED: ______________________

ORGANIZATION: ______________________



ELEVATION VIEW
(LOOKING UP STA / DWN STA)

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

SIDE VIEW
(INT / EXT)
(EAST / WEST)

ELEMENT	CONDITION	COMMENTS
PIER EXTENSIONS	G F P S	
COLUMN	G F P S	
FOOTING	G F P S	
BEARING SEAT	G F P S	

PIER 4 SURVEY SHEET

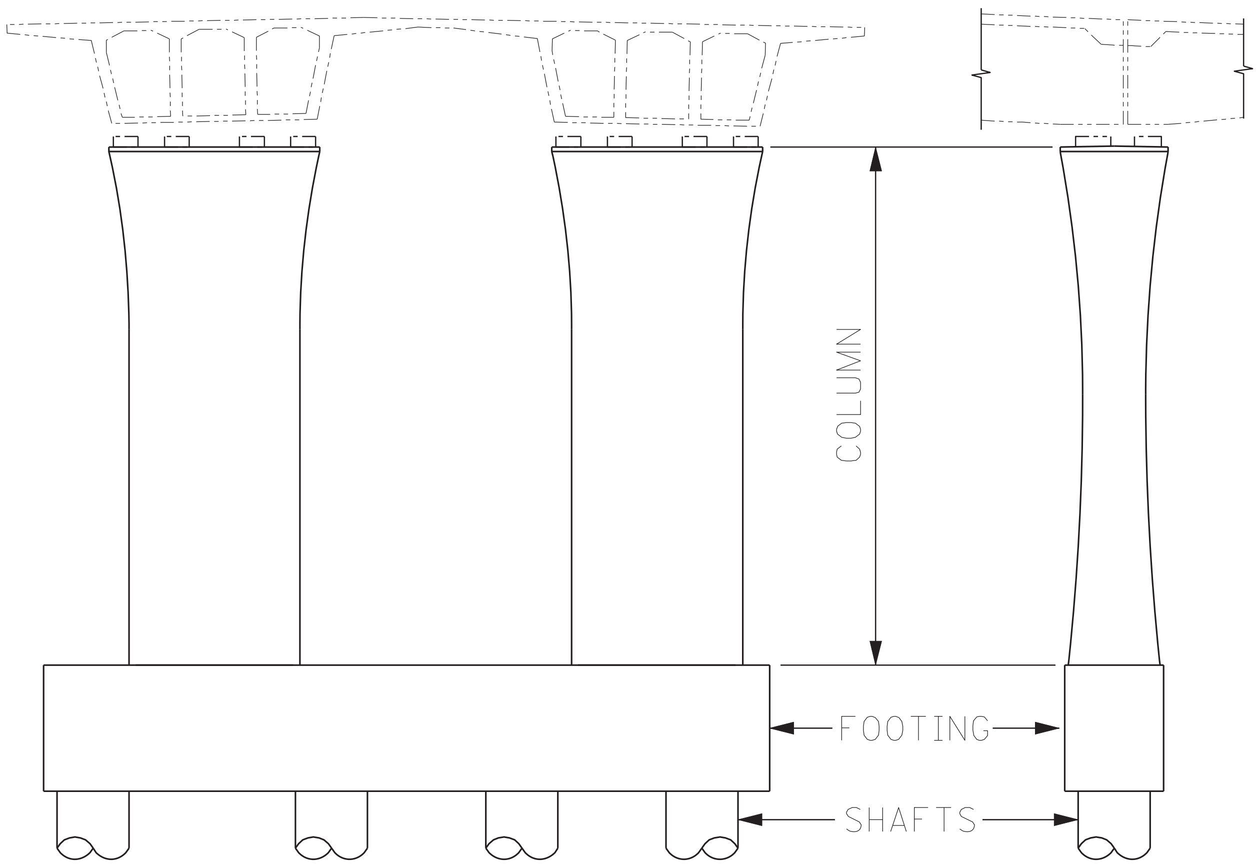
PIER NO. 4

BRIDGE: NB / SB

INSPECTED BY: ______________________

DATE INSPECTED: ______________________

ORGANIZATION: ______________________



ELEVATION VIEW

UP STA
DWN STA

CONDITION LEGEND

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

SIDE VIEW

(INT / EXT)
(EAST / WEST)

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

Appendix C

Casting & Erection Dates

Table C.1 – Casting and Erection Dates for Span 1

Concrete Pour	Span 1NB – Interior	Span 1NB – Exterior	Span 1SB – Interior	Span 1SB - Exterior
Superstructure – Phase 1	04/03/2008	04/02/2008	04/04/2008	04/07/2008
Superstructure – Phase 2	04/12/2008	04/10/2008	04/15/2008	04/15/2008
Superstructure – Phase 3 / 4	04/19/2008	04/18/2008	04/21/2008	04/22/2008
Pier 2 Diaphragm – 1st Pour	04/27/2008	04/27/2008	05/07/2008	05/07/2008
Pier 2 Diaphragm – 2nd Pour	04/28/2008	04/28/2008	05/10/2008	05/10/2008
Pier 2 Diaphragm – 3rd Pour	05/07/2008	05/07/2008	05/10/2008	05/10/2008
Superstructure Deck to 1st CJ	05/07/2008	05/07/2008	05/11/2008	05/11/2008
Superstructure Deck to 2nd CJ	05/12/2008	05/12/2008	05/18/2008	05/18/2008
Top Deck Final Pour	05/16/2008	05/16/2008	05/19/2008	05/19/2008

NOTE: “CJ” refers to control joint.

For Spans 1, Phase 1 consists of casting the soffit and webs from Abutment 1 to Control Joint 1-1. Phase 2 consists of casting soffit and webs from Control Joint 1-1 to Control Joint 1-2. Phase 3 consists of casting soffit and webs from Control Joint 1-2 to Control Joint 1-3. Phase 4 consists of casting the pier box girder soffit and webs. See Figure C.1 for clarification.

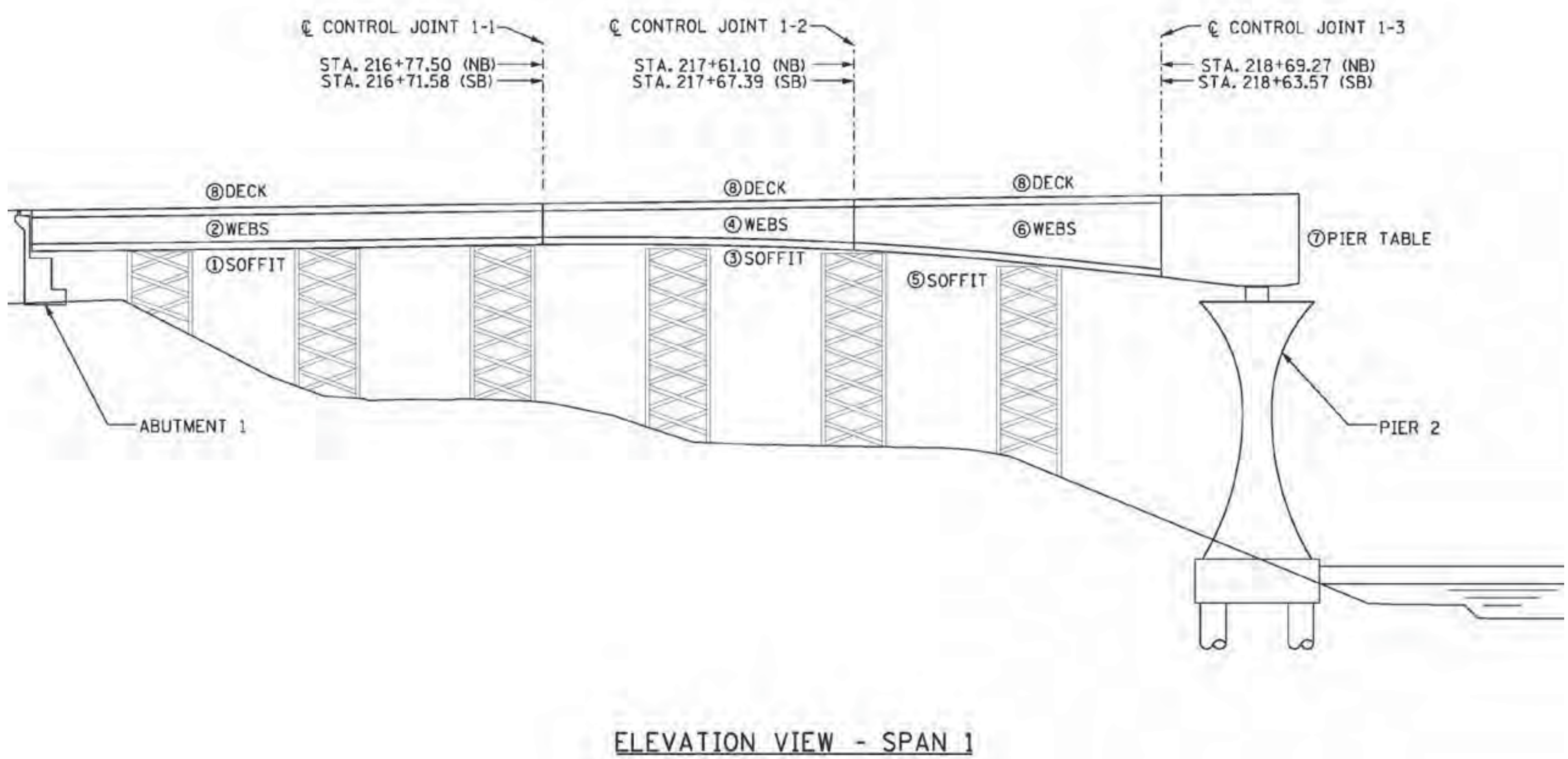


Figure C.1 – Span 1 Construction Phases

Table C.2A – Casting and Erection Dates for Span 2NB

Span 2NB - Interior			Span 2NB - Exterior		
Segment Number	Casting Date	Erection Date	Segment Number	Casting Date	Erection Date
CLOSURE	05/30/2008	05/30/2008	CLOSURE	05/30/2008	05/30/2008
2NB-1*	02/19/2008	05/26/2008	2NB-1**	01/31/2008	05/25/2008
2NB-2	03/1/2008	06/01/2008	2NB-2	02/26/2008	06/01/2008
2NB-3	03/10/2008	06/04/2008	2NB-3	03/06/2008	06/04/2008
2NB-4	03/14/2008	06/07/2008	2NB-4	03/12/2008	06/07/2008
2NB-5	03/20/2008	06/09/2008	2NB-5	03/18/2008	06/09/2008
2NB-6	03/24/2008	06/12/2008	2NB-6	03/21/2008	06/12/2008
2NB-7	03/28/2008	06/14/2008	2NB-7	03/26/2008	06/14/2008
2NB-8	04/01/2008	06/15/2008	2NB-8	03/29/2008	06/15/2008
2NB-9	04/07/2008	06/17/2008	2NB-9	04/05/2008	06/17/2008
2NB-10	04/10/2008	06/19/2008	2NB-10	04/8/2008	06/19/2008
2NB-11	04/14/2008	06/26/2008	2NB-11	04/12/2008	06/26/2008
2NB-12	04/19/2008	06/27/2008	2NB-12	04/17/2008	06/27/2008
2NB-13	04/25/2008	06/28/2008	2NB-13	04/25/2008	06/28/2008
2NB-14	04/30/2008	07/02/2008	2NB-14	04/29/2008	07/02/2008
2NB-15	05/03/2008	07/03/2008	2NB-15	05/02/2008	07/03/2008
CLOSURE		07/16/2008	CLOSURE		07/16/2008
3NB-15	06/05/2008	07/05/2008	3NB-15	06/06/2008	07/05/2008
3NB-14	06/03/2008	07/04/2008	3NB-14	06/04/2008	07/04/2008
3NB-13	05/30/2008	07/03/2008	3NB-13	05/31/2008	07/03/2008
3NB-12	05/24/2008	07/02/2008	3NB-12	05/28/2008	07/02/2008
3NB-11	05/20/2008	07/01/2008	3NB-11	05/21/2008	07/01/2008
3NB-10	05/15/2008	06/30/2008	3NB-10	05/17/2008	06/30/2008
3NB-9	05/12/2008	06/29/2008	3NB-9	05/14/2008	06/29/2008
3NB-8	05/08/2008	06/28/2008	3NB-8	05/09/2008	06/28/2008
3NB-7	05/05/2008	06/26/2008	3NB-7	05/06/2008	06/26/2008
3NB-6	04/30/2008	06/25/2008	3NB-6	05/02/2008	06/25/2008
3NB-5	04/26/2008	06/22/2008	3NB-5	04/29/2008	06/22/2008
3NB-4	04/21/2008	06/21/2008	3NB-4	04/23/2008	06/21/2008
3NB-3	04/16/2008	06/18/2008	3NB-3	04/17/2008	06/18/2008
3NB-2	04/5/2008	06/16/2008	3NB-2	04/10/2008	06/16/2008
3NB-1	03/27/2008	06/05/2008	3NB-1	04/02/2008	06/05/2008
CLOSURE	06/10/2008	06/10/2008	CLOSURE	06/10/2008	06/10/2008

NOTE: "CLOSURE" refers to closure joint pours.

* 2NB-1INT bottom slab and webs cast 2/6/08 & top slab cast 2/19/08
** 2NB-1EXT bottom slab and webs cast 1/31/08 & top slab cast 2/14/08

Table C.2B – Casting and Erection Dates for Span 2SB

Span 2SB - Interior			Span 2SB - Exterior		
Segment Number	Casting Date	Erection Date	Segment Number	Casting Date	Erection Date
CLOSURE	06/01/2008	06/01/2008	CLOSURE	06/01/2008	06/01/2008
2SB-1*	02/16/2008	05/29/2008	2SB-1**	02/21/2008	05/29/2008
2SB-2	02/29/2008	06/03/2008	2SB-2	03/05/2008	06/03/2008
2SB-3	03/08/2008	06/07/2008	2SB-3	03/11/2008	06/07/2008
2SB-4	03/13/2008	06/09/2008	2SB-4	03/15/2008	06/09/2008
2SB-5	03/19/2008	06/11/2008	2SB-5	03/20/2008	06/11/2008
2SB-6	03/24/2008	06/13/2008	2SB-6	03/25/2008	06/13/2008
2SB-7	03/27/2008	06/14/2008	2SB-7	03/28/2008	06/14/2008
2SB-8	03/31/2008	06/16/2008	2SB-8	04/02/2008	06/16/2008
2SB-9	04/05/2008	06/17/2008	2SB-9	04/07/2008	06/17/2008
2SB-10	04/09/2008	06/19/2008	2SB-10	04/11/2008	06/19/2008
2SB-11	04/14/2008	06/28/2008	2SB-11	04/15/2008	06/28/2008
2SB-12	04/18/2008	07/06/2008	2SB-12	04/21/2008	07/06/2008
2SB-13	04/24/2008	07/07/2008	2SB-13	04/25/2008	07/07/2008
2SB-14	04/28/2008	07/08/2008	2SB-14	04/30/2008	07/08/2008
2SB-15	05/01/2008	07/09/2008	2SB-15	05/03/2008	07/09/2008
CLOSURE		07/24/2008	CLOSURE		07/24/2008
3SB-15	06/05/2008	07/10/2008	3SB-15	06/04/2008	07/10/2008
3SB-14	06/03/2008	07/09/2008	3SB-14	06/01/2008	07/09/2008
3SB-13	05/31/2008	07/08/2008	3SB-13	05/30/2008	07/08/2008
3SB-12	05/27/2008	07/07/2008	3SB-12	05/23/2008	07/07/2008
3SB-11	05/20/2008	07/06/2008	3SB-11	05/19/2008	07/06/2008
3SB-10	05/16/2008	07/05/2008	3SB-10	05/14/2008	07/05/2008
3SB-9	05/13/2008	07/04/2008	3SB-9	05/10/2008	07/04/2008
3SB-8	05/09/2008	07/01/2008	3SB-8	05/07/2008	07/01/2008
3SB-7	05/06/2008	06/29/2008	3SB-7	05/05/2008	06/29/2008
3SB-6	05/01/2008	06/25/2008	3SB-6	04/29/2008	06/25/2008
3SB-5	04/28/2008	06/24/2008	3SB-5	04/23/2008	06/24/2008
3SB-4	04/21/2008	06/23/2008	3SB-4	04/18/2008	06/23/2008
3SB-3	04/16/2008	06/22/2008	3SB-3	04/15/2008	06/22/2008
3SB-2	04/09/2008	06/21/2008	3SB-2	04/04/2008	06/21/2008
3SB-1	03/29/2008	06/16/2008	3SB-1	03/26/2008	06/16/2008
CLOSURE	06/18/2008	06/18/2008	CLOSURE	06/18/2008	06/18/2008

NOTE: “CLOSURE” refers to closure joint pours.

* 2SB-1INT bottom slab and webs cast 2/4/08 & top slab cast 2/16/08

** 2SB-1EXT bottom slab and webs cast 2/8/08 & top slab cast 2/21/08

Table C.3 – Casting and Erection Dates for Span 3

Concrete Pour	Span 3NB – Interior	Span 3NB – Exterior	Span 3SB – Interior	Span 3SB - Exterior
Superstructure – Phase 1	04/30/2008	04/29/2008	05/18/2008	05/20/2008
Superstructure – Phase 2	04/30/2008	04/30/2008	05/23/2008	05/29/2008
Superstructure – Phase 3	05/13/2008	05/13/2008	05/30/2008	05/30/2008
Pier 3 Diaphragm – 1st Lift	05/17/2008	05/17/2008	06/04/2008	06/04/2008
Pier 3 Diaphragm – 2nd Lift	05/20/2008	05/20/2008	N/A	N/A
Superstructure Deck to 1st CJ	05/27/2008	05/27/2008	06/08/2008	06/08/2008
Top Deck Final Pour	05/30/2008	05/30/2008	06/12/2008	06/12/2008

NOTE: "CJ" refers to control joint.

For Spans 3, Phase 1 consists of casting the soffit from Pier 4 to Control Joint 3-1. Phase 2 consists of casting soffit from Control Joint 3-1 to Control Joint 3-3 and webs from Pier 4 to Control Joint 3-2. Phase 3 consists of casting webs from Control Joint 3-2 to Control Joint 3-3. See Figure C.2 for clarification.

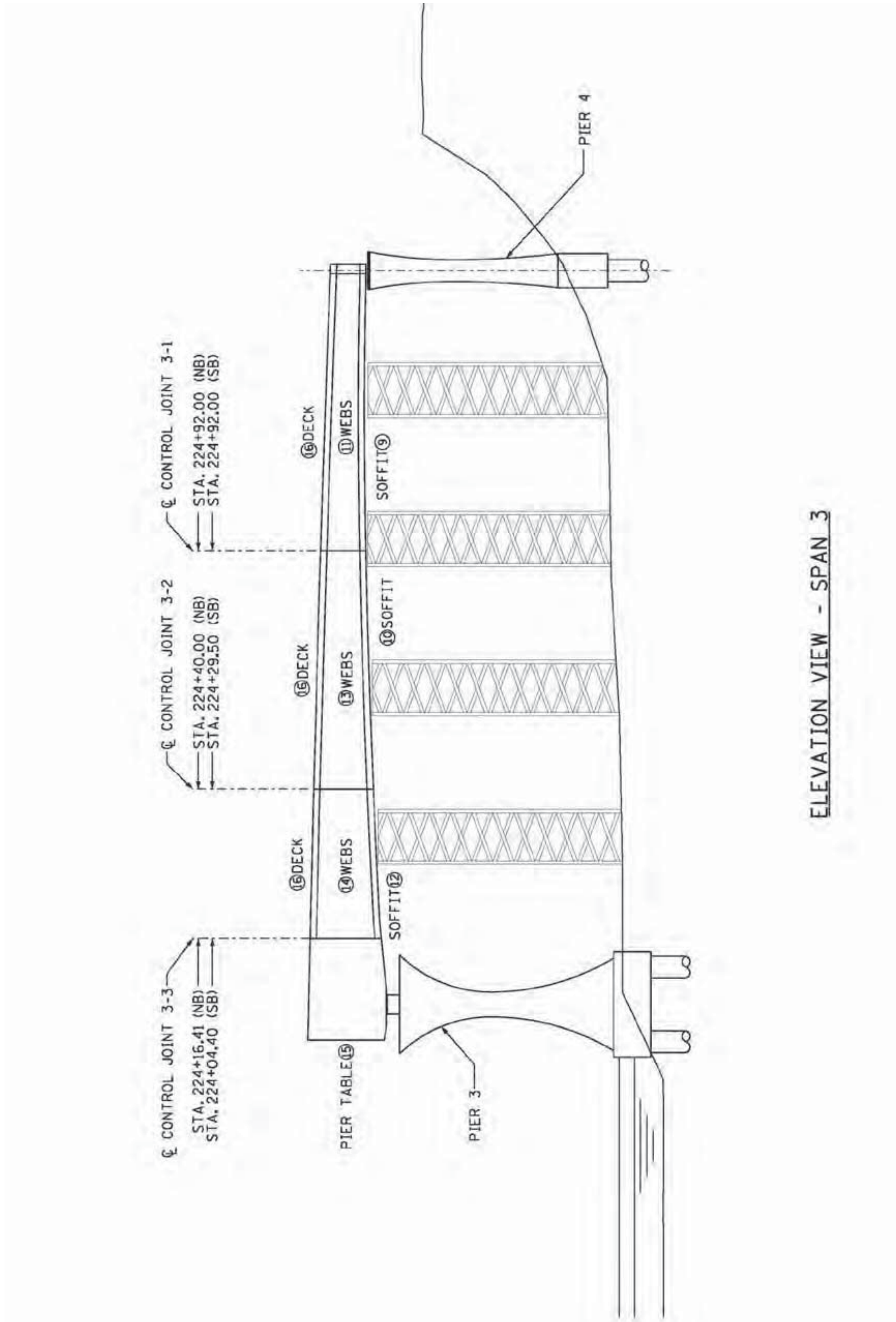


Figure C.2 – Span 3 Construction Phases

Table C.4 – Casting and Erection Dates for Span 4

Concrete Pour	Span 4NB – Interior	Span 4NB – Exterior	Span 4SB – Interior	Span 4SB - Exterior
Superstructure – Phase 1	07/16/2008	07/16/2008	07/24/2008	07/24/2008
Superstructure Deck to 1st CJ	07/28/2008	07/28/2008	08/02/2008	08/02/2008
Pier 4 Diaphragm – 1st Lift	07/28/2008	07/28/2008	08/04/2008	08/04/2008
Pier 4 Diaphragm – 2nd Lift	08/08/2008	08/08/2008	08/09/2008	08/09/2008
Top Deck Final Pour	08/08/2008	08/08/2008	08/09/2008	08/09/2008

NOTE: "CJ" refers to control joint.

For Spans 4, Phase 1 consists of casting the soffit and webs from Abutment 5 to Pier 4. See Figure C.3 for clarification.

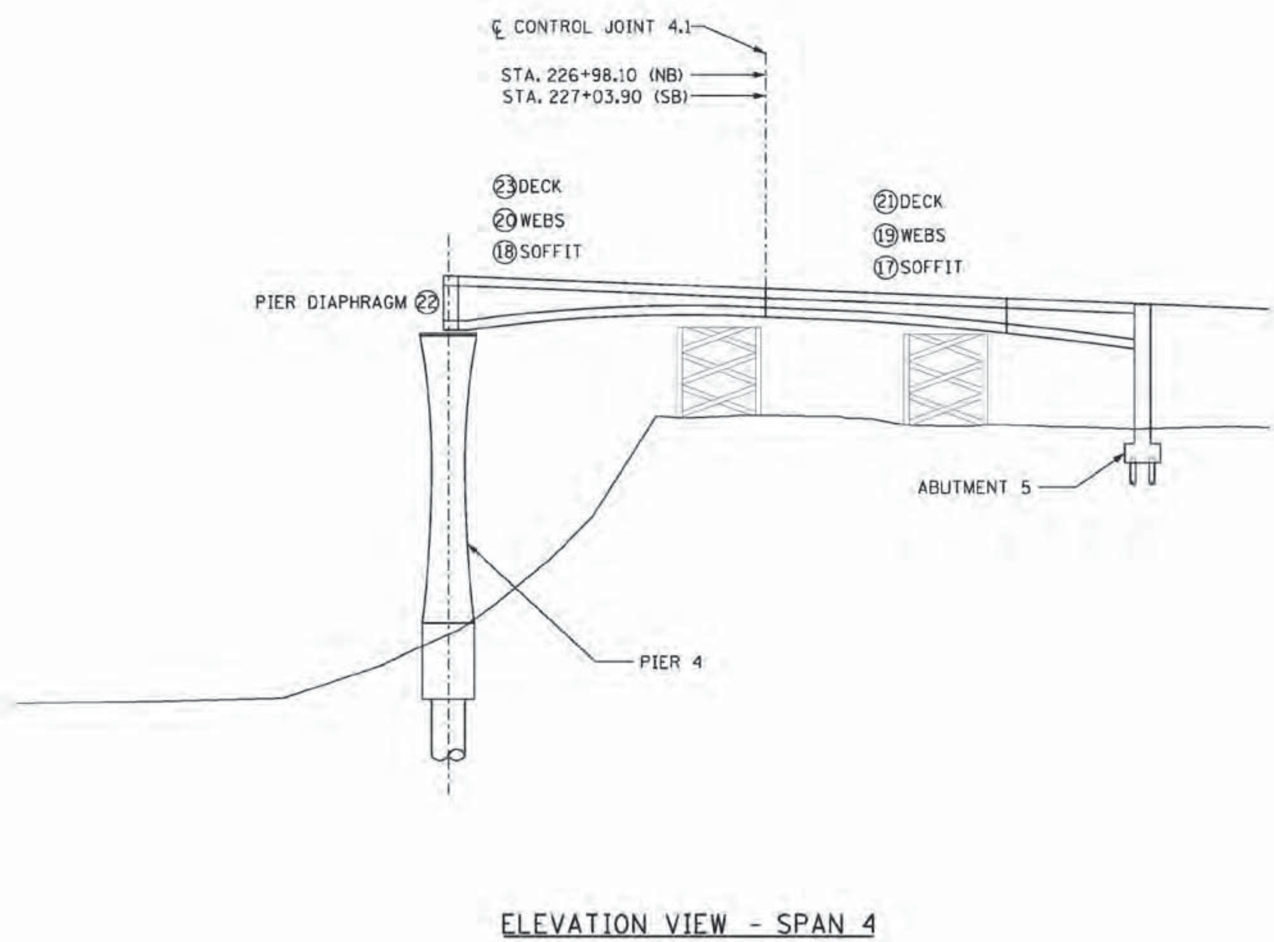
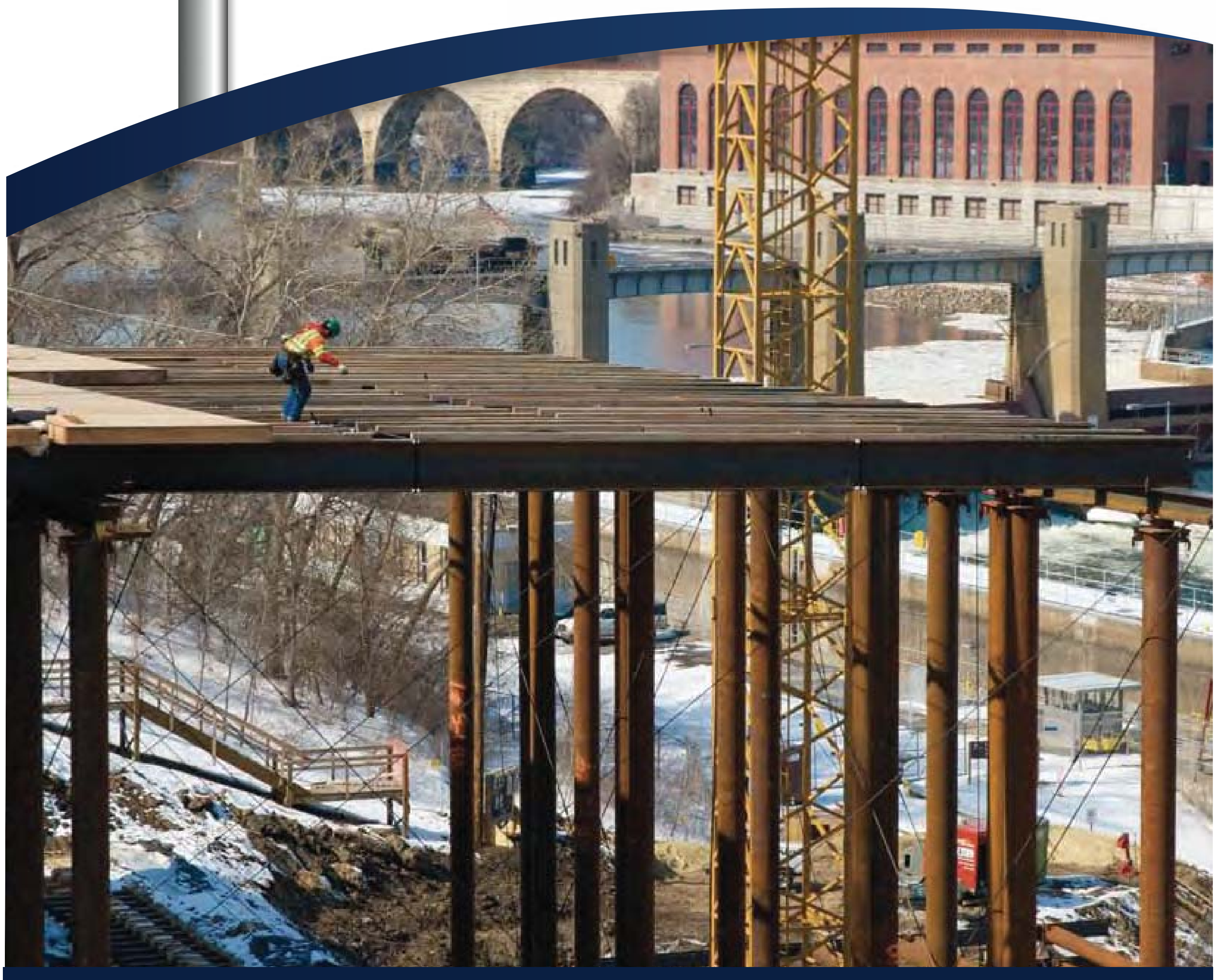


Figure C.3 – Span 4 Construction Phases

Appendix D

Electrical Systems & Utilities



APPENDIX D – ELECTRICAL SYSTEMS AND UTILITIES

- Lighting System

Light Fixtures - Flatiron Manson		
	Designer	**Light Fixture Type / Catalog #**
Main Bridge		
Bridge Deck Lighting LED Fixtures	Beta Lighting, Inc.	1. BLD-ARE-T3-AA-204-LED-A-UL-SV 2. BLD-ARE-T5-AA-170-LED-A-UL-SV
Aesthetic Wingtip Web Wash Lighting Fixtures	Philips / TIR	1. DES-CV-45DEG-LL3-RGB-WHT (36") 2. DES-CV-45DEG-LL4-RGB-WHT (48") 3. DES-CV-45DEG-LL5-RGB-WHT (60") 4. DES-CV-NCO-LL3-RGB-WHT (36")
Gateway Monument Lighting Fixtures	KIM Lighting	Type:DBF-18/150PMH240V/WH-P
Pier Uplighting Fixtures	Philips / TIR	DES-SL-NGO-RGB-SLR
Abutment Wall Floodlighting Fixtures	KIM Lighting	KN-AFL11-80WH
Pathway Lighting, Panorama Bollard Luminaires	KIM Lighting	VRB1/70PMH240/PS-P
Pathway Lighting Pole	Cooper Lighting	MH-TR-23-250W

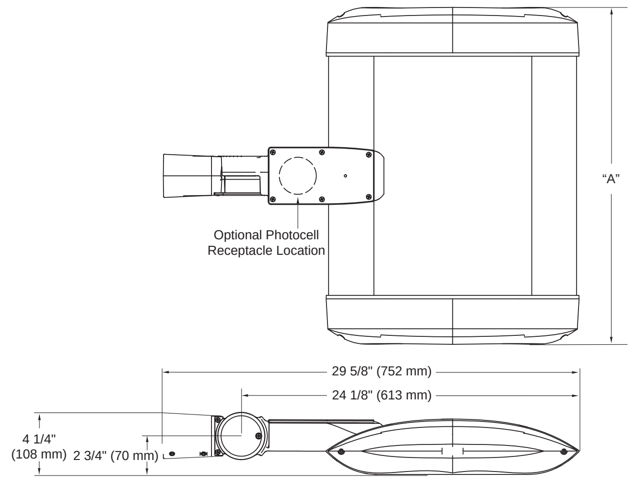
- Electrical Feedpoint Diagrams
- Anti-icing System
- Health Monitoring System

Bridge Deck Lighting

BLD-ARE-T3-AA The Edge™ LED Area Light — Type III

Rev. Date: 02/29/08

Beta Catalog Number: BLD - ARE - T3 - AA - - LED-B - - - R



LED Performance Generation B Specs

Light Bars	Initial Delivered Lumens – Catalog Description	Initial Delivered Lumens – Type III Optic	System Watts 120-277V	System Watts 347-480V	Dim. "A"/in.
1	1,700	1,600	28	30	11.75
2	3,400	3,200	55	59	11.75
3	5,100	4,800	79	84	13.75
4	6,800	6,400	104	109	15.75
5	8,500	8,000	128	133	17.75
6	10,200	9,600	153	156	19.75
7	11,900	11,200	183	194	21.75
8	13,600	12,800	207	218	23.75
9	15,300	14,400	232	242	25.75
10	17,000	16,000	257	266	27.75
11	18,700	17,600	281	290	29.75
12	20,400	19,200	306	313	31.75

Product Family
BLD

Housing Indicator
ARE

Optics
T3[1]

Mounting
AA[2]

Initial Delivered Lumens (00's)
- ☐ 017
- ☐ 034
- ☐ 051
- ☐ 068
- ☐ 085
- ☐ 102
- ☐ 119
- ☐ 136
- ☐ 153
- ☐ 170
- ☐ 187
- ☐ 204

LED Performance
LED-B

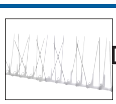
Voltage
- ☐ UL (120–277V Universal)
- ☐ UH (347–480V Universal)
- ☐ 12
- ☐ 27
- ☐ 34

Color Options
- ☐ BZ
- ☐ BK
- ☐ WH
- ☐ SV
- ☐ PB

Factory-Installed Options
If choosing more than one option, please type in manually on the lines provided above.
- ☐ EM–Emergency[3]
- ☐ F–Fuse
- ☐ P–Photocell[4]
- ☐ R–NEMA Photocell Receptacle

Notes:

Field-Installed Accessories



Bird Spikes
☐ XA-BRDSPK

1-IESNA Type III distribution
2-Adjustable arm; consult factory
3-Consult factory
4-Must specify voltage other than UL or UH

General Description
Slim, low profile design minimizes wind load requirements. Fixture sides are rugged cast aluminum with integral, weather-tight LED driver compartments and high performance aluminum heatsinks. Adjustable mounting arm is rugged die cast aluminum and mounts to a 2" tenon. Includes leaf/debris guard. Five year limited warranty on fixture.

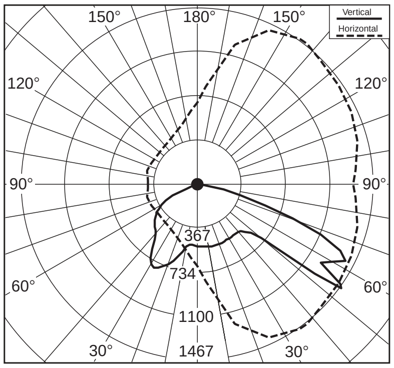
Electrical
Modular design accommodates varied lighting output from high brightness, white, 6000K, minimum 75 CRI, long life LED sources. 120-277V 50/60 Hz, Class 1 LED drivers are standard. 347-480V 50/60 Hz driver is optional. LED drivers have power factor >90% and THD <20% of full load. Integral weather-tight electrical box with terminal strip for easy power hook-up.

Finish
Exclusive Colorfast DeltaGuard® finish features an E-Coat epoxy primer with an ultra-durable silver powder topcoat, providing excellent resistance to corrosion, ultraviolet degradation and abrasion. Bronze, black, white and platinum bronze powder topcoats are also available. The finish is covered by our 10 year limited warranty.

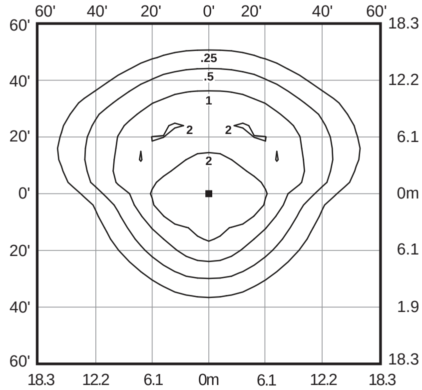
Labels
UL listed in the U.S. and Canada for wet locations and enclosure classified IP66 per IEC 529.

Patents
Pending

Output gains using Generation B LEDs can be achieved by multiplying footcandle levels by 1.1.



Independent Testing Laboratories certified test. Report No. ITL 59234. Candlepower distribution curve of 2 light bar luminaire with 2863 initial delivered lumens.



Isofootcandle plot of 6 light bar Type III LED luminaire at 20' A.F.G. Initial delivered lumens at 8589. Initial FC at grade.

Isofootcandles plots shown are initial at grade.

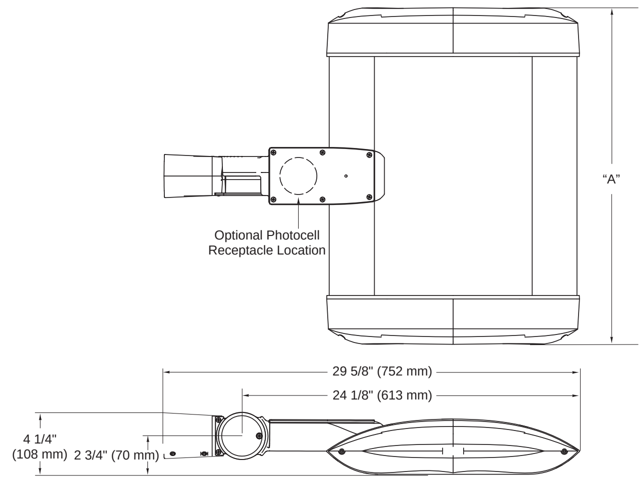
LED Area Light EPA Calculations

	LIGHT BARS										
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Adj Fitter Mount 0 deg											
1 fixture	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.70	0.74
2 fixtures (180°)	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.39	1.48
2 fixtures (90°)	0.98	1.02	1.07	1.11	1.15	1.19	1.23	1.27	1.31	n/a	n/a
3 fixtures (90°)	1.64	1.68	1.72	1.76	1.80	1.84	1.89	1.93	1.97	n/a	n/a
4 fixtures (90°)	1.97	2.05	2.13	2.21	2.29	2.38	2.46	2.54	2.62	n/a	n/a

BLD-ARE-T5-AA The Edge™ LED Area Light — Type V

Rev. Date: 02/29/08

Beta Catalog Number: ***BLD - ARE - T3 - AA - - LED-B - - -***



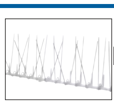
LED Performance Generation B Specs

Light Bars	Initial Delivered Lumens – Catalog Description	Initial Delivered Lumens – Type V Optic	System Watts 120-277V	System Watts 347-480V	Dim. "A"/in.
1	1,700	1,700	28	30	11.75
2	3,400	3,400	55	59	11.75
3	5,100	5,100	79	84	13.75
4	6,800	6,800	104	109	15.75
5	8,500	8,500	128	133	17.75
6	10,200	10,200	153	156	19.75
7	11,900	11,900	183	194	21.75
8	13,600	13,600	207	218	23.75
9	15,300	15,300	232	242	25.75
10	17,000	17,000	257	266	27.75
11	18,700	18,700	281	290	29.75
12	20,400	20,400	306	313	31.75

Product Family	*Housing Indicator*	*Optics*	*Mounting*	*Initial Delivered Lumens (00's)*	*LED Performance*	*Voltage*	*Color Options*	*Factory-Installed Options*
BLD	**ARE**	**T5**[1]	**AA**[2]	☐ 017 ☐ 034 ☐ 051 ☐ 068 ☐ 085 ☐ 102 ☒ 119 ☐ 136 ☐ 153 ☐ 170 ☐ 187 ☐ 204	LED-B	☐ UL (120–277V Universal) ☐ UH (347–480V Universal) ☐ 12 ☐ 27 ☐ 34	☐ BZ ☐ BK ☐ WH ☐ SV ☐ PB	If choosing more than one option, please type in manually on the lines provided above. ☐ EM–Emergency[3] ☐ F–Fuse ☐ P–Photocell[4] ☒ R–NEMA Photocell Receptacle

Notes:

Field-Installed Accessories



Bird Spikes
☐ XA-BRDSPK

1-IESNA Type V distribution
2-Adjustable mounting arm; consult factory
3-Consult factory
4-Must specify voltage other than UL or UH

General Description
Slim, low profile design minimizes wind load requirements. Fixture sides are rugged cast aluminum with integral, weather-tight LED driver compartments and high performance aluminum heatsinks. Adjustable mounting arm is rugged die cast aluminum and mounts to a 2" tenon. Includes leaf/debris guard. Five year limited warranty on fixture.

Electrical
Modular design accommodates varied lighting output from high brightness, white, 6000K, minimum 75 CRI, long life LED sources. 120-277V 50/60 Hz, Class 1 LED drivers are standard. 347-480V 50/60 Hz driver is optional. LED drivers have power factor >90% and THD <20% of full load. Integral weather-tight electrical box with terminal strip for easy power hook-up.

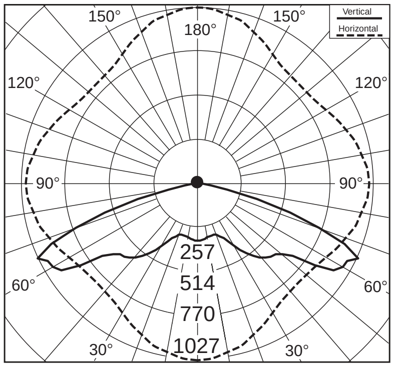
Finish
Exclusive Colorfast DeltaGuard® finish features an E-Coat epoxy primer with an ultra-durable silver powder topcoat, providing excellent resistance to corrosion, ultraviolet degradation and abrasion. Bronze, black, white and platinum bronze powder topcoats are also available. The finish is covered by our 10 year limited warranty.

Labels
UL listed in the U.S. and Canada for wet locations and enclosure classified IP66 per IEC 529.

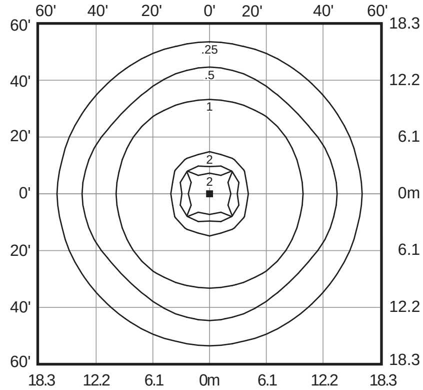
Patents
Pending

BLD-ARE-T5-AA **The Edge™ LED Area Light — Type V** Rev. Date: 02/29/08

Output gains using Generation B LEDs can be achieved by multiplying footcandle levels by 1.06.



Independent Testing Laboratories certified test. Report No. ITL 59237. Candlepower distribution curve of 2 light bar Luminaire with 3138 initial delivered lumens.



Isofootcandle plot of 6 light bar Type V LED luminaire at 20' A.F.G. Initial delivered lumens at 9414. Initial FC at grade.

Isofootcandles plots shown are initial at grade.

LED Area Light EPA Calculations

	LIGHT BARS										
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Adj Fitter Mount 0 deg											
1 fixture	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.70	0.74
2 fixtures (180°)	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.31	1.39	1.48
2 fixtures (90°)	0.98	1.02	1.07	1.11	1.15	1.19	1.23	1.27	1.31	n/a	n/a
3 fixtures (90°)	1.64	1.68	1.72	1.76	1.80	1.84	1.89	1.93	1.97	n/a	n/a
4 fixtures (90°)	1.97	2.05	2.13	2.21	2.29	2.38	2.46	2.54	2.62	n/a	n/a

Aesthetic Lights

DESTINY ®

Destiny CV

features

- Exceptional asymmetric throw from a setback as close as 6"
- Narrow cove optic provides smooth gradient with no hot spot
- Also available with wide cove optic or flood optics
- Adjustable mounting brackets
- Uniform color mixing with 12 Luxeon® LEDs per linear foot
- DMX512 compatibility for dynamic color control with 1' resolution
- TIR® Thermal management system protects electronic components from heat damage
- Robust construction suitable for outdoor applications
- Operational and environmental benefits of LED technology

standard specifications

OPTICS	Narrow Cove, Wide Cove, 22° and 45° flood optics
LIGHT SOURCE	12 Luxeon high flux LEDs per foot
DISTRIBUTION	Narrow spread linear cove
SETBACK DISTANCE	Minimum 6” from luiminaire center line
FINISH	Clear anodized aluminum. Optional silver, black or white powdercoat finishes.
POWER SUPPLY	90 VAC to 264 VAC integral power supply, auto ranging (50-60 Hz)

standard order codes

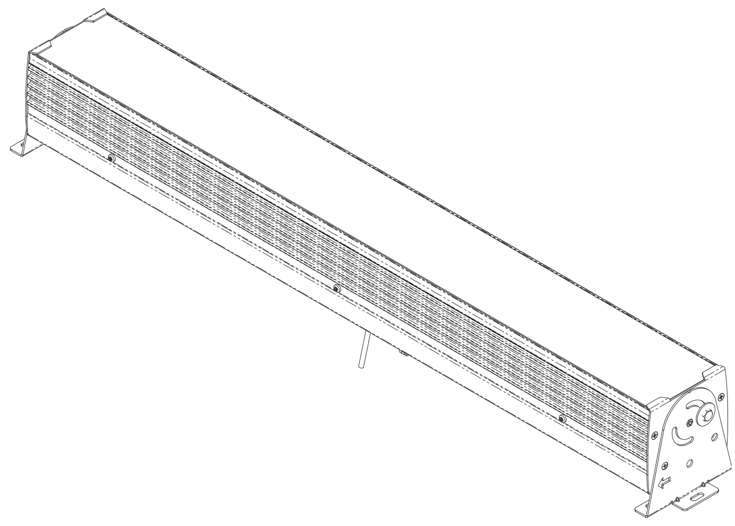
DES —	**CV** —	______ —	______ —	______ —	______ —	**DMX**
SERIES	PRODUCT	OPTIC	LENGTH	LED LIGHT COLOR	FINISH	NETWORK
Destiny®	**Destiny CV**	**NCO** Narrow Cove Optic **WCO** Wide Cove Optic **22°** 22° beam angle **45°** 45° beam angle	**LL5** 5’ nom **LL4** 4’ nom **LL3** 3’ nom	**RGB** 4 red, 4 blue 4 green, per foot **RED** 12 red per foot **GRN** 12 green per foot **BLU** 12 blue per foot **ABR** 12 amber per foot **WWH** 12 warm white per foot, 3300K **CWH** 12 cool white per foot, 5500K	**ANO** Clear anodized **SLR** Silver powdercoat **BLK** Black powdercoat **WHT** White powdercoat **CUS** Custom color	**DMX** DMX Network

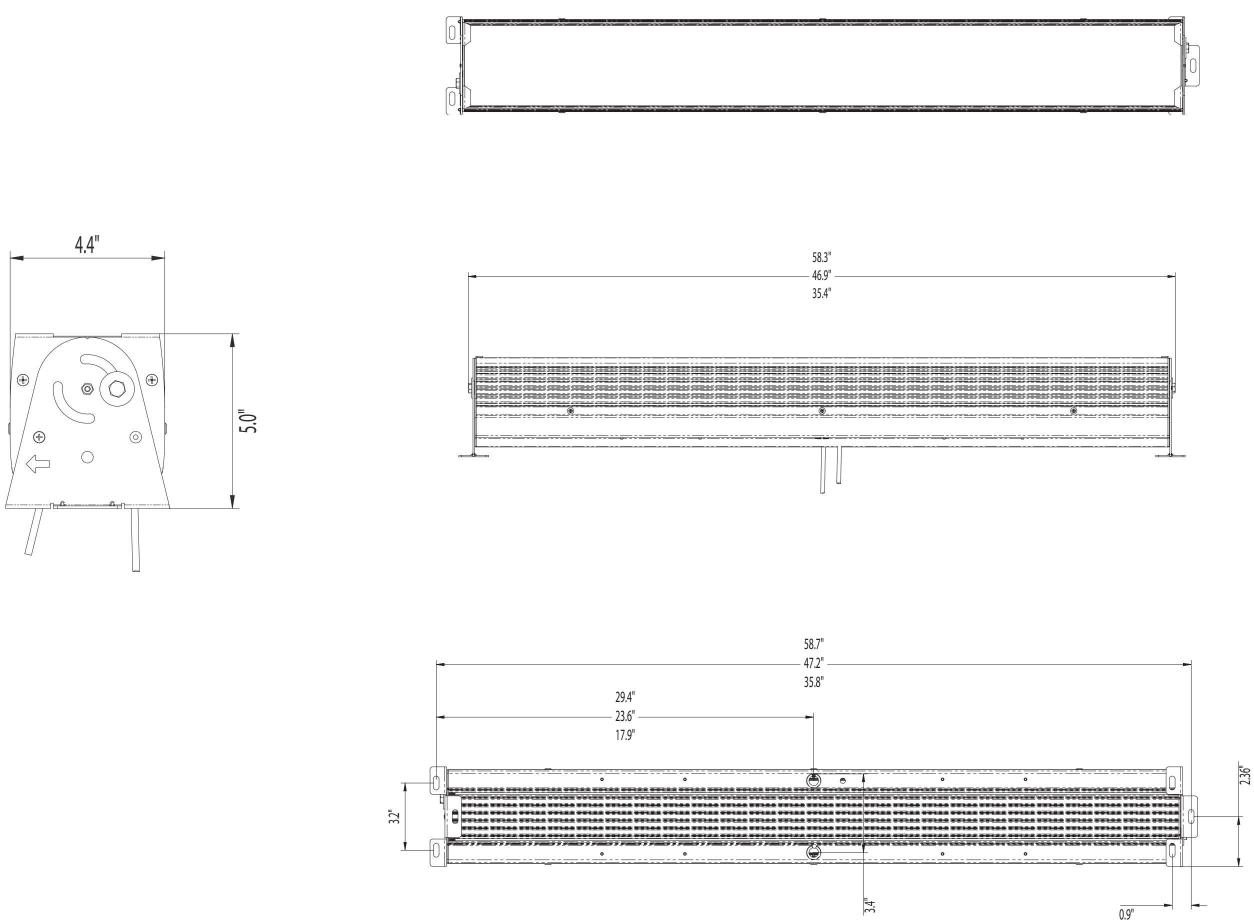
CERTIFIED LUXEON LUMINAIRE MANUFACTURER

TIR Systems Ltd. is a member of the Luxeon Lighting Network

TIR Systems Ltd.
7700 Riverfront Gate
Burnaby BC
Canada V5J 5M4
1 800 663 2036
T 604 294 8477
F 604 294 3733
www.tirsys.com

STANDARD





TIR Systems Ltd.
7700 Riverfront Gate
Burnaby BC
Canada V5J 5M4

1 800 663 2036
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F 604 294 3733
www.tirsys.com

technical specifications			
	mechanical	HOUSING	Extruded aluminum; PSU and controller are integral to the luminaire
		MOUNTING	Wall, ceiling or floor mount; stainless steel bracket
	electrical	INPUT VOLTAGE	90 VAC to 264 VAC
		MAX INPUT POWER	see below
		CONNECTIONS	AC: Industrial grade AC cable; DATA: Individually shielded 24 AWG twisted pair + bare drain input and output DMX 10’ standard cable whip length
	environmental	TEMPERATURE RANGE	-40ºF to 104ºF (-40ºC to 40ºC) operating temperature -4ºF to 104ºF (-20ºC to 40ºC) starting temperature
		CERTIFICATION	CUL/UL/CE
		INGRESS PROTECTION	IP66 Rated

MAX INPUT POWER:

MODEL 5’ LENGTH (LL5)	OUTPUT COLOR (ON FULL)	LUMINAIRE INPUT POWER	LUMINAIRE INPUT CURRENT (100 VAC)
RGB	Red	40W	0.4 A
RGB	Green	40W	0.4 A
RGB	Blue	40W	0.4 A
RGB	White	110W	1.1 A
Single Color option	Red, Green, Blue Amber, White	110W	1.1 A

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45° illuminance distribution

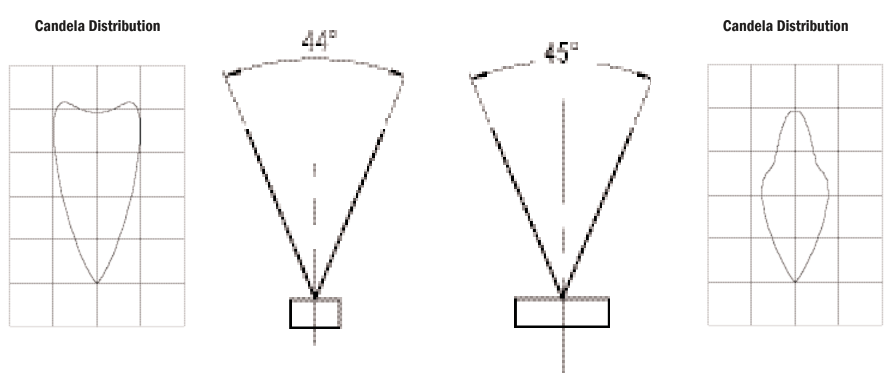
Throw Distance (ft)

Vertical Illuminance (fc)

Throw Distance (ft)	8' SETBACK -3'	-2'	-1'	0'	+1'	+2'	+3'	20' SETBACK -3'	-2'	-1'	0'	+1'	+2'	+3'
- 4'	2.2	4.1	5.6	5.8	5.6	4.1	2.2	2.4	2.4	2.3	2.2	2.3	2.4	2.4
- 3'	4	7.3	8.9	8.7	8.9	7.3	4	2.7	2.7	2.5	2.5	2.5	2.7	2.7
- 2'	6.2	11.5	13.8	13.6	13.8	11.5	6.2	3.1	3	2.9	2.9	2.9	3	3.1
- 1'	7.8	14.9	17.4	17.3	17.4	14.9	7.8	3.3	3.3	3.3	3.3	3.3	3.3	3.3
0'	8.5	15.5	17.7	16	17.7	15.5	8.5	3.5	3.4	3.4	3.4	3.4	3.4	3.5
+ 1'	7.8	14.9	17.4	17.3	17.4	14.9	7.8	3.3	3.3	3.3	3.3	3.3	3.3	3.3
+ 2'	6.2	11.5	13.8	13.6	13.8	11.5	6.2	3.1	3	2.9	2.9	2.9	3	3.1
+ 3'	4	7.3	8.9	8.7	8.9	7.3	4	2.7	2.7	2.5	2.5	2.5	2.7	2.7
+ 4'	2.2	4.1	5.6	5.8	5.6	4.1	2.2	2.4	2.4	2.3	2.2	2.3	2.4	2.4

45° BEAM

End View **Face View**



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Gateway Monument Lights

IN-GRADE
DBF

Direct Burial Floodlight

35 - 175 Watt H.I.D. / 42 - 120 Watt PL

KIM LIGHTING

DBF

Direct Burial Floodlight

Table of Contents

The Direct Burial Floodlight (DBF) is a new Kim innovation, combining the features of in-grade accent lights with above-grade floodlights. The Kim DBF offers full floodlight optical systems in an architectural housing designed to project the lowest possible profile above grade. As such, the DBF offers a floodlighting solution for sensitive architectural designs where above-grade floodlights would detract from the purity of the building and site. Using the technology developed for Kim's renowned Lightvault below-grade luminaires, the DBF uses a composite housing for maximum below-grade durability and seal integrity. The above-grade housing is heavy cast bronze or aluminum and contains 9 optical systems from Kim's celebrated Architectural Floodlight product line.

ISO 9001:2000

KIM LIGHTING

SITE / AREA
PARKING STRUCTURE
ROADWAY
ARCHITECTURAL FLOOD
ACCENT
LANDSCAPE

MAILING ADDRESS:
P.O. BOX 60080
CITY OF INDUSTRY, CA
91716-0080

BUSINESS ADDRESS:
16555 EAST GALE AVENUE
CITY OF INDUSTRY, CA 91745
U.S.A.
PHONE 626 / 968-5666
FAX 626 / 369-2695

www.kimlighting.com

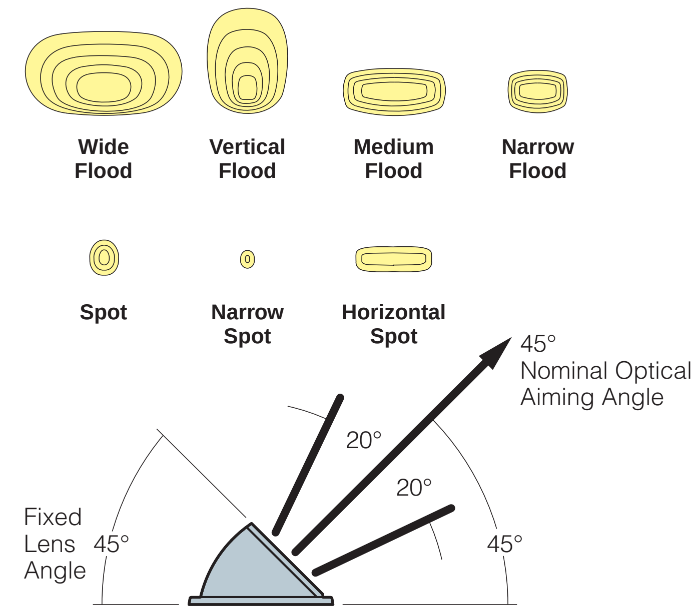
HUBBELL®

Hubbell Lighting, Inc.

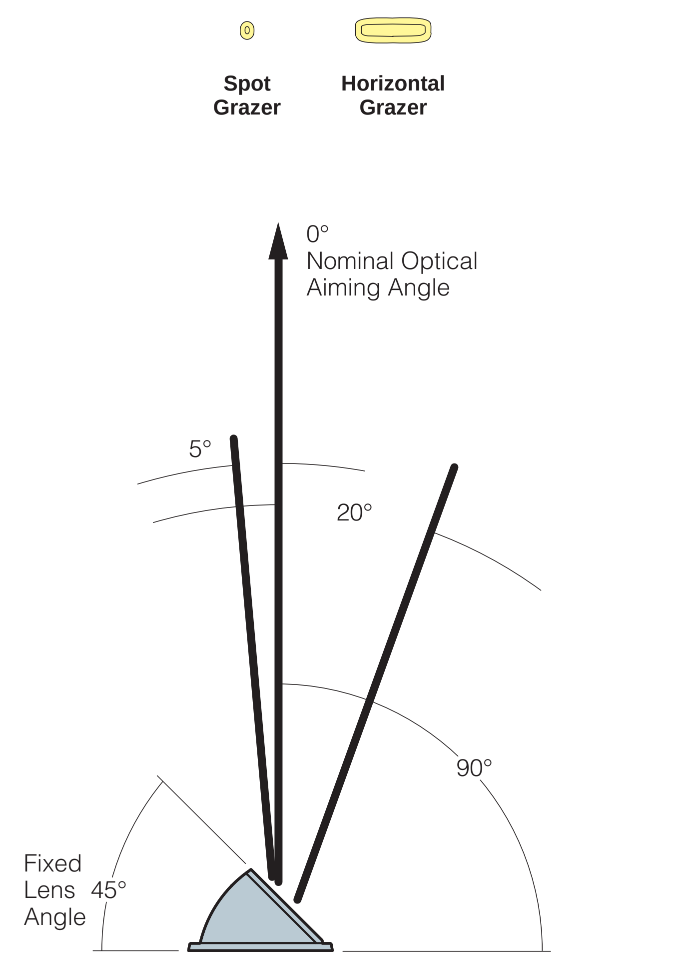
Printed in U.S.A.
5507204299
Version 10/04

9 Light Distributions

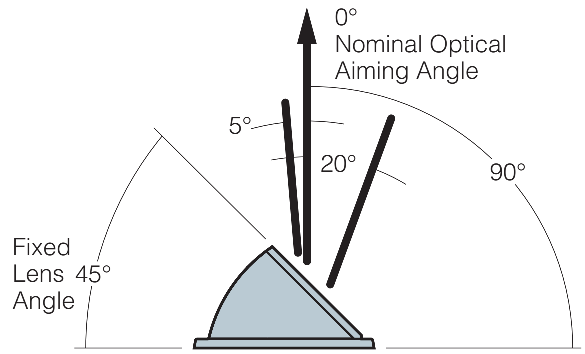
45° Nominal Optical Aiming Angle: ±20°



0° Nominal Optical Aiming Angle: +5°, -20°



Application
DBF18 and DBF19
0° Nominal Optical Aiming Angle



With a nominal optical aiming angle of 0° (straight up), DBF18 and 19 models are for mounting close to the building to create grazing and highlighting effects. Setbacks, usually only 2′ to 5′, should be determined in relation to reliefs and projections on the building facade and whether light should fall on those features. See page **10** for examples.

Mechanical Features

External Aiming

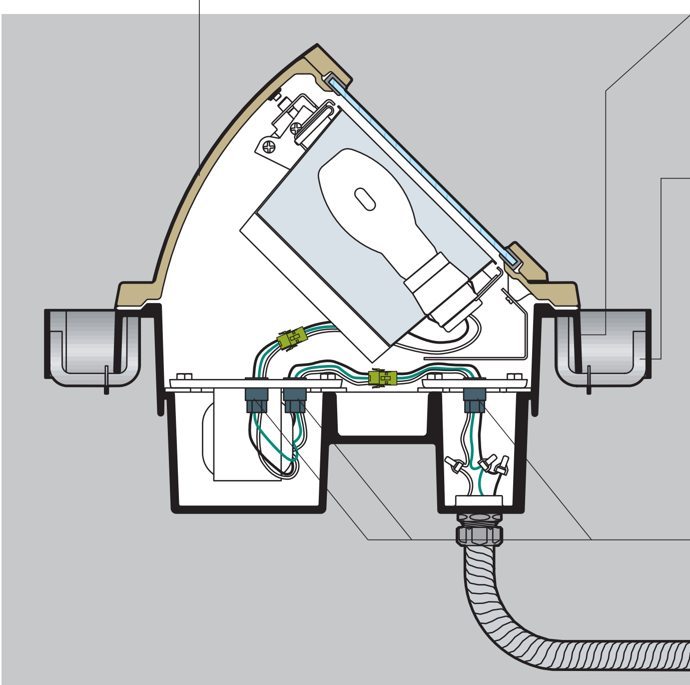
To provide the specifier or owner with true visual aiming ability, the DBF has been engineered with an external aiming feature. This means that fine-tuning of the lighting effect can be achieved at night, with the fixture energized and any optional attachments in position. Removing a waterproof plug on top of the housing accesses the aiming mechanism. An allen wrench is inserted into the housing and aiming adjustments are made while visually observing the changes in the lighting effect on the target surface or building feature. When aiming is complete, just re-insert the waterproof plug to seal the housing. The plug is tethered to the housing to prevent accidental loss when removed.

Bronze or Aluminum Above-grade Housings

Rugged one-piece, heavy-wall castings are used for the above-grade housing and lens frame. Bronze housing is natural, and will weather to a deep bronze color. Aluminum housing is finished with Super TGIC Powder Coat.

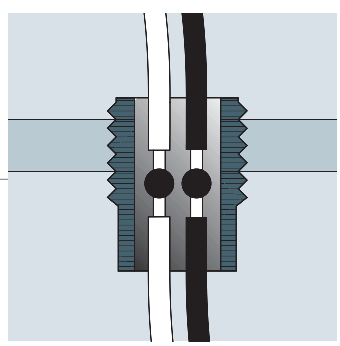
Composite Below-grade Housing

For maximum corrosion resistance, the below-grade housing is compression-molded, high-temperature, fiberglass impregnated composite. Captive brass inserts are molded in to receive the lens frame screws. Two ¾″ NPT conduit entries are provided in the bottom.



Optional Grout Mask

A galvanized steel Grout Mask is available to aid fixture installation in paved surfaces or a concrete pad. Grout Mask is tied into rebar and supports the fixture during concrete pour. See page **14**.



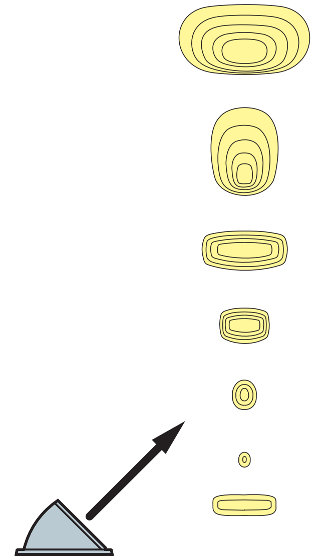
Anti-siphon Barriers

All wires entering and exiting the ballast compartment pass through anti-siphon barriers. This prevents any moisture from seeping into the ballast area through the wiring. Each anti-siphon barrier is a one-piece molded and threaded plug installed at the factory.

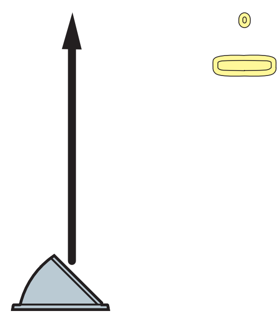
Optical Features

Nine Optical Systems

The DBF is available in nine optical configurations ranging from wide flood to narrow spot. For the DBF11-17 (45° nominal aiming), all reflectors are from Kim's AFL10 series of Architectural Floodlights. For the DBF18 and 19 (0° nominal aiming), new reflectors have been engineered for this configuration. All 9 reflectors are self-contained modules, fully interchangeable within the common housing. Each module is retained in the housing by quick-release hinges, and all wires have quick-disconnect plugs.



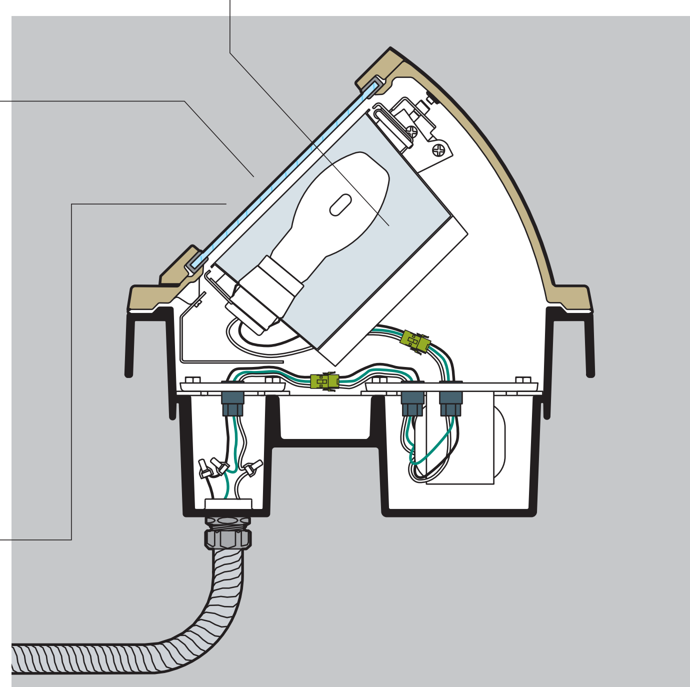
45° Nominal Aiming: ±20°



0° Nominal Aiming: +5°, -20°

Optional Lexan® SLX Lens Shield and Fixed Hoods

For additional side-glare control, a Fixed Hood or Full Shield is available. These glare control devices may only be used with the DBF11-17, as they will interfere with the straight-up light throw from the DBF18 and 19. For vandal-prone areas, the optional Lexan® SLX Lens Shield can be added and used in addition to the Fixed Hood or Full Shield.



Color Filters

Kim color filters are constructed of color media material sandwiched between two sheets of tempered glass, sealed around all edges. The filter is held 2″ away from the fixture lens by an extruded aluminum holder. Available in 5 colors. See page **13**.

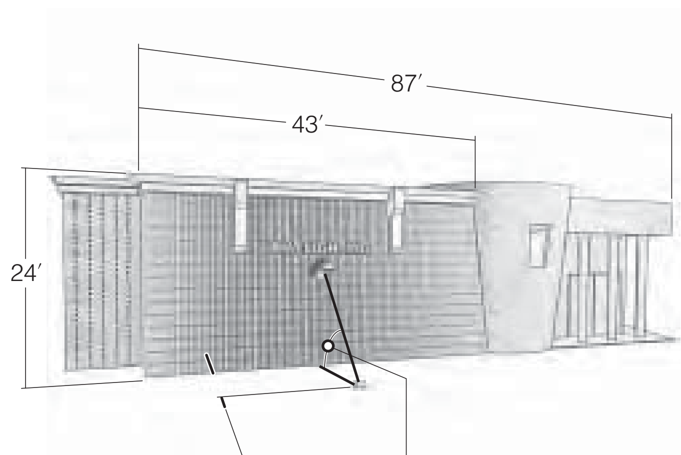
Virtual Performance Models

Architecture

The building created for these performance models is strictly for the purpose of showing the lighting effects of all 9 DBF models on one structure. It is a combination of geometric shapes and textures that simulate the various elements that are often found in architecture. By creating a single building for all 9 DBF optical systems, it is easier to compare the differences between the individual lighting effects. The building facade is divided into left and right sides, separated by a cone element. The left side is a textured wall with various elements that allow flood, spot, or grazing effects to be illustrated. The right side has three columns supporting an overhang to show the lighting effect on structural elements and recessed spaces. To create a finished lighting effect on this building, multiple fixtures would normally be used.

Lighting Effects

The lighting effects in the following nine illustrations are not renderings, but actual computer generated effects derived from real photometric data. Each illustration was created by a 3D design, modeling, and animation program capable of reading actual I.E.S. photometric files from each DBF fixture. The illustrations are very accurate and represent a breakthrough technology by Kim Lighting in helping specifiers visualize and apply the complex art of floodlighting.



Setback		**Aiming**	
DBF11-14	Setback 7′	DBF11-14	45°
DBF15-17	Setback 15′	DBF15-17	55°
DBF18-19	Setback 2′	DBF18-19	0°

DBF11
Wide Flood

Lamp: 175MH
Setback: 7′
Aiming: 45°

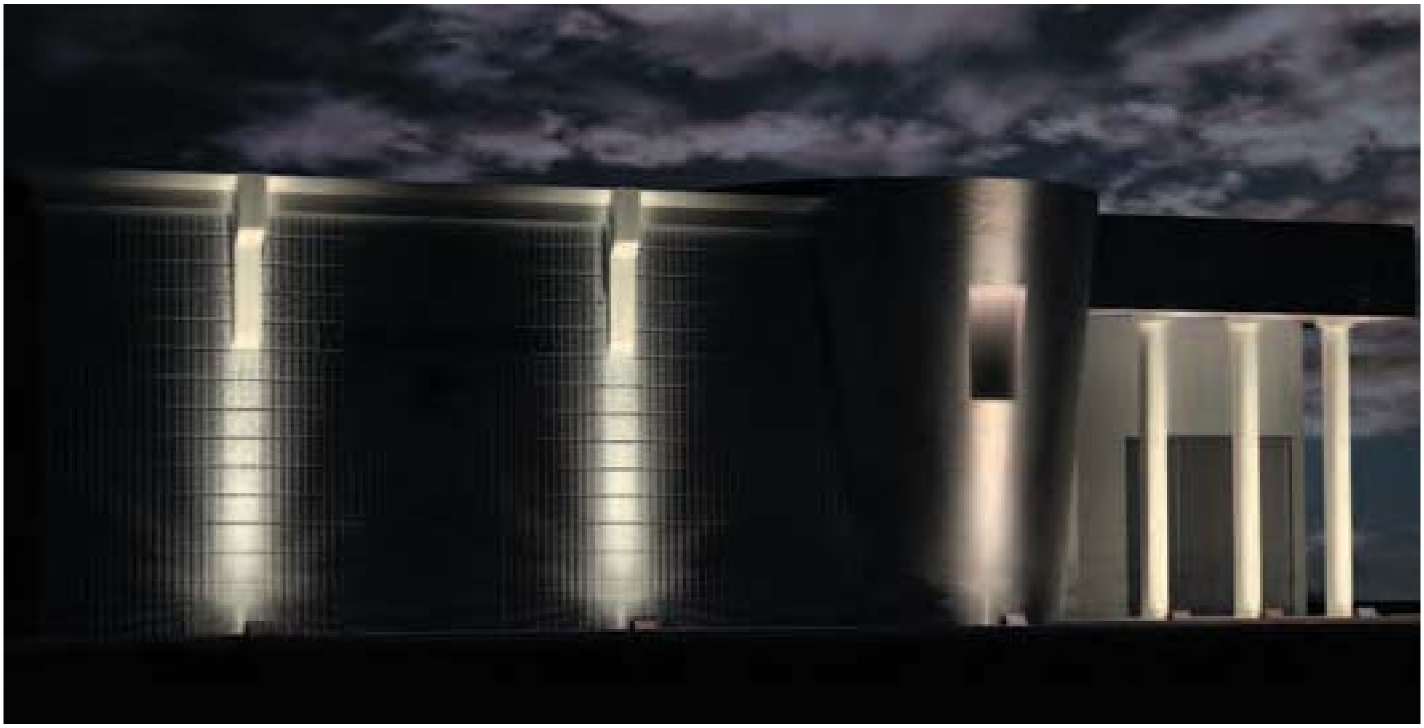
NOTE: This optical system has the greatest horizontal light throw. To uniformly light the illustrated wall, two fixtures would normally be used, and possibly a higher aiming angle to place more light at the top of the wall.

Virtual Performance Models

DBF18
Spot Grazer

Lamp: 175MH
Setback: 2′
Aiming: 0° (Straight up)

NOTE: This optical system is for creating dramatic highlights on building features. It is designed for mounting close to the structure, creating a high level of texture and contrast.

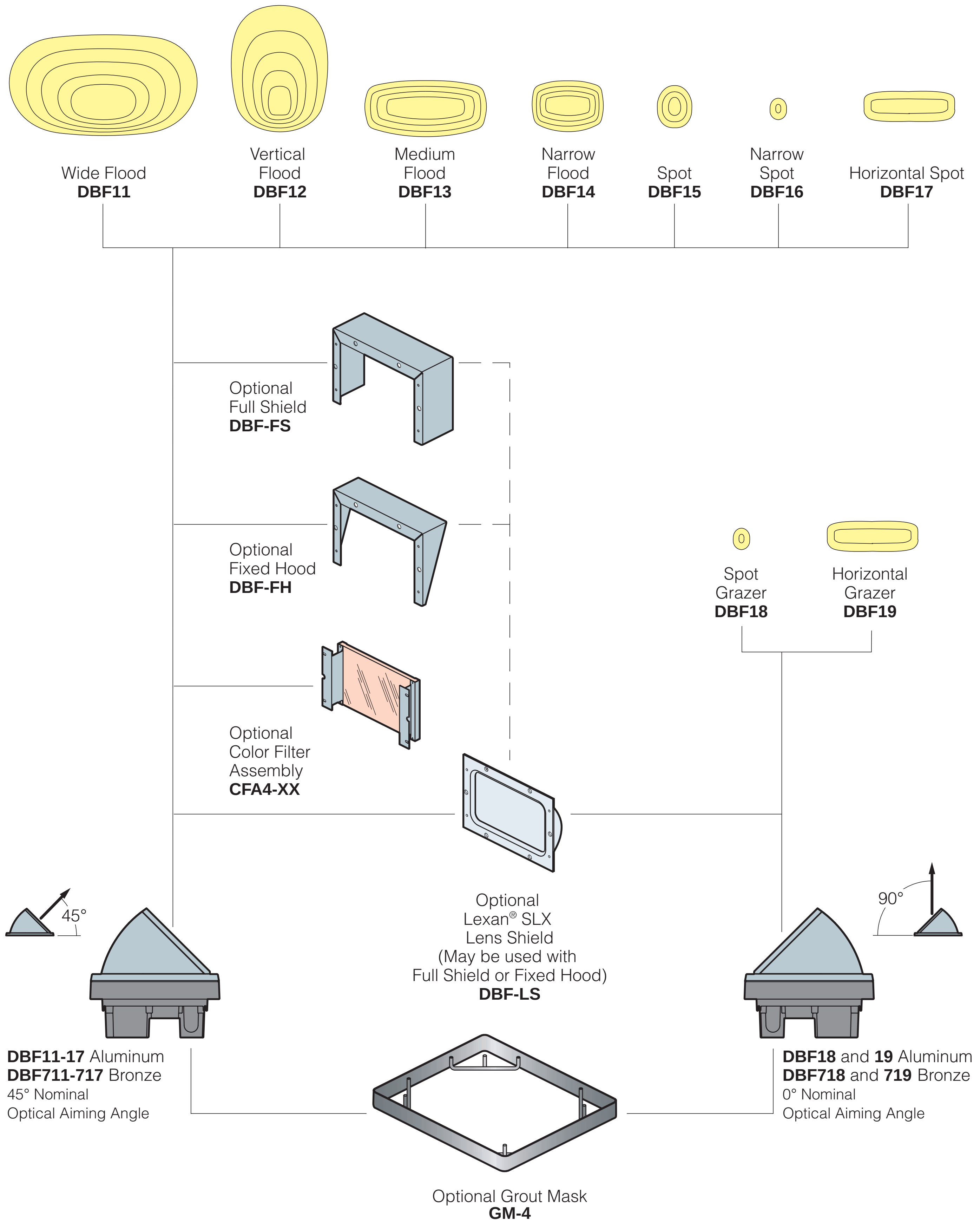


DBF19
Horizontal Grazer

Lamp: 175MH
Setback: 2′
Aiming: 0° (Straight up)

NOTE: This optical system is for creating maximum texture, and highlights on walls and building features. It can also be used for lighting the under side of building overhangs. The optical system is similar to the DBF17 Horizontal Spot, only for straight up aiming.

DBF


Wide Flood
DBF11
Vertical Flood
DBF12
Medium Flood
DBF13
Narrow Flood
DBF14
Spot
DBF15
Narrow Spot
DBF16
Horizontal Spot
DBF17
Optional Full Shield
DBF-FS
Optional Fixed Hood
DBF-FH
Optional Color Filter Assembly
CFA4-XX
Spot Grazer
DBF18
Horizontal Grazer
DBF19
45°
90°
Optional Lexan® SLX Lens Shield (May be used with Full Shield or Fixed Hood)
DBF-LS
DBF11-17 Aluminum
DBF711-717 Bronze
45° Nominal
Optical Aiming Angle
DBF18 and 19 Aluminum
DBF718 and 719 Bronze
0° Nominal
Optical Aiming Angle
Optional Grout Mask
GM-4

Ordering Information

Direct Burial Floodlight

Ordering Example:

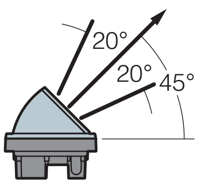
Fixture	Electrical Module	Optional Socket	Finish	Optional Fuse	Options
DBF15 /	175MH277 /	G12 /	BL-P /	SF	DBF-FH/BL-P
1	2	3	4	5	6-10
Standard Fixture and Options Ordered Assembled with Fixture					Options Ordered and Shipped Separately from Fixture

1 Fixture:

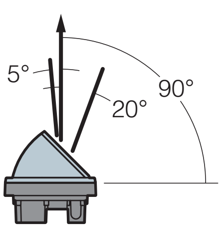
Cat. No. designates complete fixture and beam pattern. "**7**" designates bronze above-grade housing.

DBF 11
- 11 — Beam Pattern
- DBF — **Aluminum** Above-Grade Housing

DBF 711
- 11 — Beam Pattern
- 7 — **Bronze** Above-Grade Housing

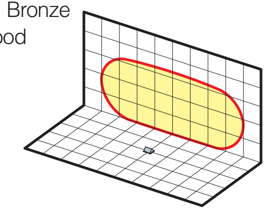


DBF11-17
DBF711-717
45° Nominal Optical Aiming Angle

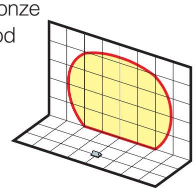


DBF18 and **19**
DBF718 and **719**
0° Nominal Optical Aiming Angle

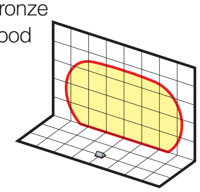
DBF11 Aluminum
DBF711 Bronze
Wide Flood



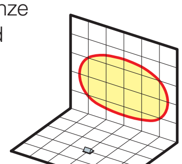
DBF12 Aluminum
DBF712 Bronze
Vertical Flood



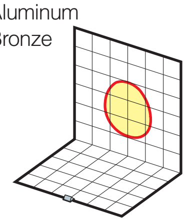
DBF13 Aluminum
DBF713 Bronze
Medium Flood



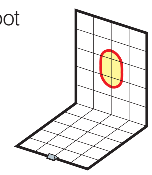
DBF14 Aluminum
DBF714 Bronze
Narrow Flood



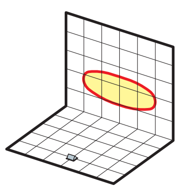
DBF15 Aluminum
DBF715 Bronze
Spot



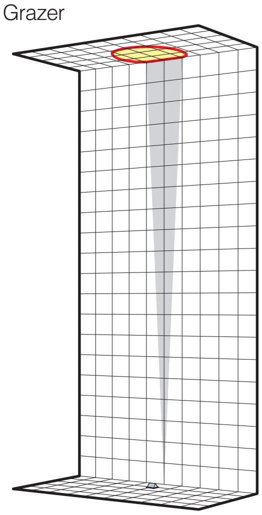
DBF16 Aluminum
DBF716 Bronze
Narrow Spot



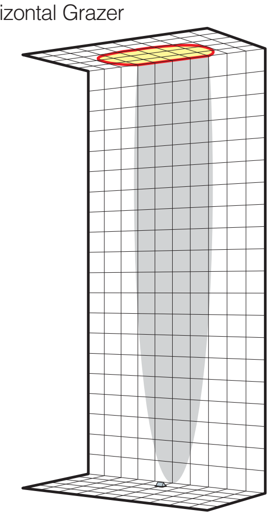
DBF17 Aluminum
DBF717 Bronze
Horizontal Spot



DBF18 Aluminum
DBF718 Bronze
Spot Grazer



DBF19 Aluminum
DBF719 Bronze
Horizontal Grazer



2 Electrical Module:

- **HPS** = High Pressure Sodium
- **MH** = Metal Halide
- **PMH** = Pulse Start Metal Halide
- **PL** = Fluorescent

Available with optional Electronic Ballast. Add **E** to Electrical Module number. e.g.: **150PMH277E**[4]

Lamp Watts	Lamp Type	Line Volts
175	MH	277

39PMH120	50PMH120	70PMH120	100PMH120	150PMH120	175MH120
39PMH208	50PMH208	70PMH208	100PMH208	150PMH208	175MH208
39PMH240	50PMH240	70PMH240	100PMH240	150PMH240	175MH240
39PMH277	50PMH277	70PMH277	100PMH277	150PMH277	175MH277
39PMH347	50PMH347	70PMH347	100PMH347	150PMH347	175MH347
		70PMH480[1]	100PMH480[1]	150PMH480[1]	175MH480[1]

35HPS120	50HPS120	70HPS120	100HPS120	150HPS120	42PL[2]
35HPS277	50HPS208	70HPS208	100HPS208	150HPS208	60PL[3]
	50HPS240	70HPS240	100HPS240	150HPS240	85PL[3]
	50HPS277	70HPS277	100HPS277	150HPS277	120PL[3]
		70HPS347	100HPS347	150HPS347	
		70HPS480[1]	100HPS480[1]	150HPS480[1]	

[1] Check with local codes for the use of medium base socket with 480 volt.
[2] Operates 42 watt compact fluorescent lamp. 120 through 277 volt / 50 or 60 Hz.
[3] Available in wide flood only (**DBF11** or **DBF711**). Operates 120 through 277 volt / 50 or 60 Hz.
[4] Optional Electronic Ballast is variable voltage ballast for use in 120 through 277 voltages and 50 or 60 Hz. For use with Metal Halide lamps only. Consult factory for other voltages.

3 Optional Socket:

Cat. No.: **G12**

Optional G12 base socket is available for 39, 70, and 150 Metal Halide T-6 Bipin lamps. Must use UV filtering lamp.

4 Finish:

Aluminum Above-Grade Housing[1]:

Color:	Black	Dark Bronze	Light Gray	Platinum Silver	White	Custom Colors[2]
Cat. No.:	**BL-P**	**DB-P**	**LG-P**	**PS-P**	**WH-P**	**CC-P**

[1]Finish: Super TGIC powder coat paint over clear anodizing and Titanated Zirconium conversion coating.
[2]Consult representative for custom colors.

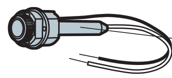
Bronze Above-Grade Housing

Cat. No.: **NB**
Natural Bronze

Will rapidly age to a rich deep bronze color, and may develop areas of verde patina.

5 Optional Fusing:

Line Volts:	120V	208V	240V	277V	347V	480V
Cat. No.:	**SF**	**DF**	**DF**	**SF**	**SF**	**DF**



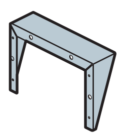
High temperature fuse holders factory installed inside the fixture housing. Single Fusing **(SF)** for 120V, 277V and 347V or Double Fusing **(DF)** for 208V, 240V and 480V.

6 Optional Fixed Hood:

Specify Finish for Aluminum Hood
Example: **DBF-FH/BL-P**

Copper Hood has natural finish, and will age to a deep bronze color. May develop areas of verde patina.

Example: **DBF7-FH**



Cat. No.: **DBF-FH**
Aluminum

Formed 3/32″ thick aluminum. Mounts to predrilled doorframe holes. Cannot be used with **DBF18** and **DBF19**. Ordered and shipped separately from fixture.

Cat. No.: **DBF7-FH**
Copper

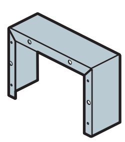
Formed 1/16″ thick copper. Mounts to predrilled doorframe holes. Cannot be used with **DBF718** and **DBF719**. Ordered and shipped separately from fixture.

7 Optional Full Shield:

Specify Finish for Aluminum Hood
Example: **DBF-FS/BL-P**

Copper Hood has natural finish, and will age to a deep bronze color. May develop areas of verde patina.

Example: **DBF7-FS**



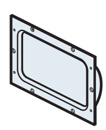
Cat. No.: **DBF-FS**
Aluminum

Formed 3/32″ thick aluminum. Mounts to predrilled doorframe holes. Should not be used with **DBF11** and **DBF12**. Use Fixed Hood instead. Cannot be used with **DBF18** and **DBF19**. Ordered and shipped separately from fixture.

Cat. No.: **DBF7-FS**
Copper

Formed 1/16″ thick copper. Mounts to predrilled doorframe holes. Should not be used with **DBF711** and **DBF712**. Use Fixed Hood instead. Cannot be used with **DBF718** and **DBF719**. Ordered and shipped separately from fixture.

8 Optional Lexan® SLX Lens Shield:

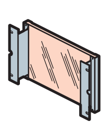


Cat. No.: **DBF-LS**
Clear Finish

3/16″ clear convex vacuum formed non-yellowing Lexan® SLX with gasket. Mounts over lens to predrilled door frame holes and may be used with **FH** Fixed Hood or **FS** Full Shield option. Ordered and shipped separately from fixture.

9 Optional Color Filter Assembly:

Specify Finish
Example: **CFA4-05/BL-P**



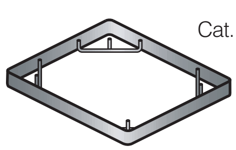
Cat. No.: **CFA4-XX**

Color Filter Assembly Cat. No. includes color filter and channel finish. Specify filter, substituting **XX** for color filter number (See below) and add finish.

Heavy wall aluminum extrusion with anti-reflection baffles and vertical channels that hold the color filter 2″ away from the fixture lens. Quick change-out of the color filter is possible by the removal of two channel screws. Support mounts to predrilled holes in fixture door frame. May be used in conjunction with **FH** Fixed Hood or **FS** Full Shield option. Ordered and shipped separately from fixture.

Color Filter:	Deep Straw	Rose Tint	Medium Red	Brilliant Blue	Primary Green
XX:	**15**	**05**	**27**	**69**	**91**

10 Optional Grout Mask:



Cat. No.: **GM-4**

For fixture support during concrete pour. Galvanized steel guard and rebar. Ties to paving rebar and provides 2″ of space for finishing up to fixture. Fixture must be with grout mask during pour. Ordered and shipped separately from fixture.

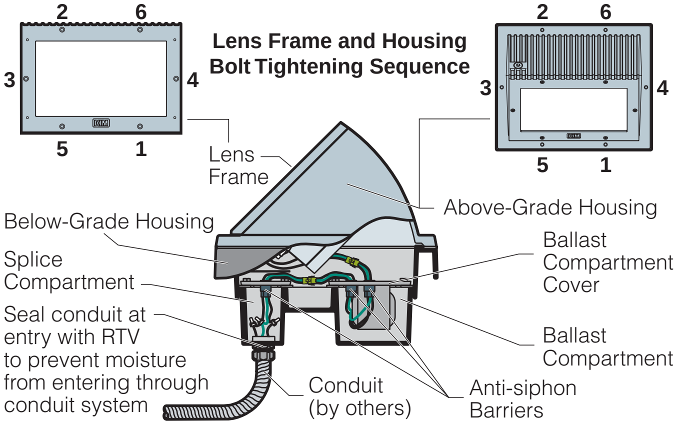
Installation Guide

Wiring and Assembly

After installing conduit and pulling the correct conductors into the splice compartment, seal the conduit by injecting silicone sealer into the open conduit end to completely block the entry of water.

Clean all gaskets, cover plates, housing flanges, and housing interior. Install gaskets, cover plates, and housings as outlined in the Installation Instructions. Install lamp and test for operation.

Energize the fixture. Allow the fixture to reach operating temperature (at least 30 minutes). Remove aiming plug and allow the fixture to "Breathe" for approximately 10 minutes; this will permit the moist air to be "Exhaled". While the fixture is still energized, replace the aiming plug and tighten as described in the Installation Instructions. This procedure should be repeated each time the fixture is opened for maintenance.



Isolate and Elevate.

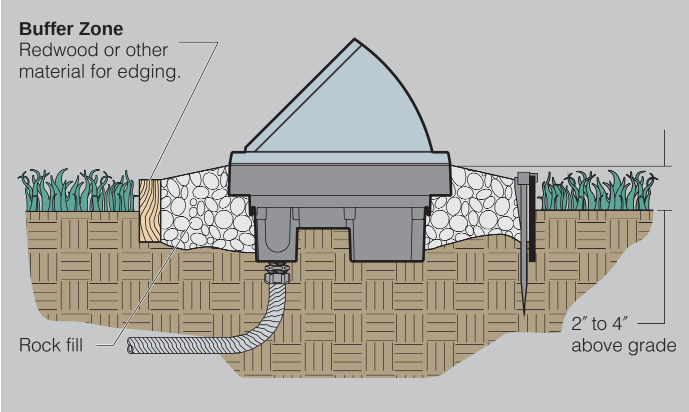
The fundamentals of a clean, maintainable installation.

Create a Buffer Zone

When fixtures are located in areas planted in ground cover or shrubbery, construct a buffer zone to prevent lens overgrowth and to create an edge for trimming. Elevate the fixtures for drainage and backfill with decorative rock. As the ground cover grows, the fixtures will look flush even though they are 2″ to 4″ above grade.

Advantages

- Prevents lens overgrowth.
- Provides a defined edge for trimming.
- Provides drainage away from the lens to maintain light output.
- Visually looks like a flush installation.



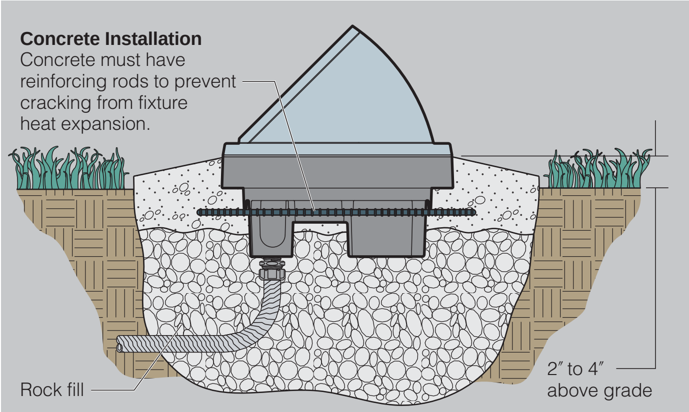
Install in Concrete

Another option for installations in ground cover, shrubbery or lawn areas is to encase the fixture in concrete. This creates the buffer zone as described above, with the additional advantage of greater fixture stability. Elevate the fixture 2″ to 4″ above grade, and slope the concrete away from the housing for drainage.

Advantages

- Cleaner, more stable installation, less susceptible to traffic and maintenance activity.
- Prevents lens overgrowth.
- Provides a defined edge for trimming.
- Provides drainage away from the lens to maintain light output.
- Visually looks like a flush installation.

NOTE: Always use adequate rebar surrounding the fixture to prevent cracking of the concrete due to heat expansion.

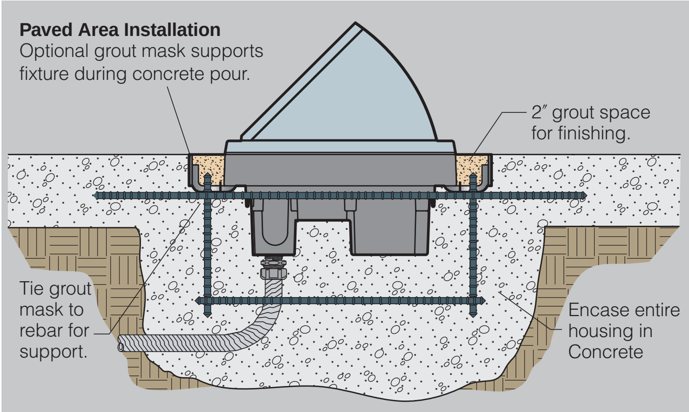


In Paved Areas

When below-grade luminaires are installed in paving, it is usually required that the below-grade housing be flush with finished grade. To make this installation easier, Kim offers an optional Grout Mask (page **15**) to support the housing at the proper height during the concrete pour. The Grout Mask is normally tied into the paving rebar for support.

Advantages

- Supports fixture at proper height during concrete pour.
- Provides 2″ grout space for finishing.
- Easily adapts to any paving material; concrete, brick, stone, etc.



Beam Spread Data

DBF15 Spot Beam Spread Chart

Lamp		Lamp Watts	Initial Lumens[1]	I.E.S. Type	Maximum Candlepower	Field Angle (10% of max.)	Beam Angle[2] (50% of max.)	Photometric Test No.
HIGH PRESSURE SODIUM								
35HPS	**ED-17 clear med. base**	35	2250	3H x 3V	5188 (0.0 x 3.0)	45.9 x 30.1	21.7 x 13.3	KL00568
50HPS	**ED-17 clear med. base**	50	4000	3H x 3V	9223 (0.0 x 3.0)	45.9 x 30.1	21.7 x 13.3	KL00567
70HPS	**ED-17 clear med. base**	70	6300	3H x 3V	14526 (0.0 x 3.0)	45.9 x 30.1	21.7 x 13.3	KL00566
100HPS	**ED-17 clear med. base**	100	9500	3H x 3V	21904 (0.0 x 3.0)	45.9 x 30.1	21.7 x 13.3	KL00510
150HPS	**ED-17 clear med. base**	150	16000	5H x 4V	21337 (1.0 x 1.0)	79.1 x 52.6	28.1 x 24.2	KL00372
METAL HALIDE								
39PMH	**T-6 clear G-12 base**	39	3400	3H x 2V	10695 (1.0 x -5.0)	45.8 x 21.4	19.8 x 9.3	KL00471
50PMH	**ED-17 clear med. base**	50	4250	3H x 2V	11706 (0.0 x 0.0)	43.4 x 23.6	22.5 x 11.6	KL00565
70PMH	**ED-17 clear med. base**	70	6200	3H x 2V	17077 (0.0 x 0.0)	43.4 x 23.6	22.5 x 11.6	KL00564
100PMH	**ED-17 clear med. base**	100	9300	3H x 2V	25615 (0.0 x 0.0)	43.4 x 23.6	22.5 x 11.6	KL00419
150PMH	**ED-17 clear med. base**	150	14000	3H x 3V	31464 (0.0 x 0.0)	42.9 x 33.9	23.8 x 14.5	KL00563
175MH	**ED-17 clear med. base**	175	14400	3H x 3V	32363 (0.0 x 0.0)	42.9 x 33.9	23.8 x 14.5	KL00424

DBF16 Narrow Spot Beam Spread Chart

Lamp		Lamp Watts	Initial Lumens[1]	I.E.S. Type	Maximum Candlepower	Field Angle (10% of max.)	Beam Angle[2] (50% of max.)	Photometric Test No.
HIGH PRESSURE SODIUM								
35HPS	**ED-17 clear med. base**	35	2250	1H x 2V	11406 (0.0 x 7.0)	16.8 x 26.3	10.6 x 13.5	KL00574
50HPS	**ED-17 clear med. base**	50	4000	1H x 2V	20277 (0.0 x 7.0)	16.8 x 26.3	10.6 x 13.5	KL00573
70HPS	**ED-17 clear med. base**	70	6300	1H x 2V	31937 (0.0 x 7.0)	16.8 x 26.3	10.6 x 13.5	KL00572
100HPS	**ED-17 clear med. base**	100	9500	1H x 2V	48159 (0.0 x 7.0)	16.8 x 26.3	10.6 x 13.5	KL00511
150HPS	**ED-17 clear med. base**	150	16000	1H x 3V	111233 (0.0 x 3.0)	13.2 x 31.4	5.5 x 14.7	KL00436
METAL HALIDE								
39PMH	**T-6 clear G-12 base**	39	3400	1H x 1V	73712 (0.0 x 5.0)	12.2 x 11.0	6.7 x 5.6	KL00479
50PMH	**ED-17 clear med. base**	50	4250	1H x 1V	75795 (0.0 x 3.0)	9.8 x 17.9	4.8 x 8.7	KL00571
70PMH	**ED-17 clear med. base**	70	6200	1H x 1V	110572 (0.0 x 3.0)	9.8 x 17.9	4.8 x 8.7	KL00570
100PMH	**ED-17 clear med. base**	100	9300	1H x 1V	165858 (0.0 x 3.0)	9.8 x 17.9	4.8 x 8.7	KL00420
150PMH	**ED-17 clear med. base**	150	14000	1H x 1V	138750 (0.0 x 3.0)	13.9 x 24.8	6.2 x 12.8	KL00569
175MH	**ED-17 clear med. base**	175	14400	1H x 2V	142714 (0.0 x 3.0)	13.9 x 24.8	6.2 x 12.8	KL00360

DBF17 Horizontal Spot Beam Spread Chart

Lamp		Lamp Watts	Initial Lumens[1]	I.E.S. Type	Maximum Candlepower	Field Angle (10% of max.)	Beam Angle[2] (50% of max.)	Photometric Test No.
HIGH PRESSURE SODIUM								
35HPS	**ED-17 clear med. base**	35	2250	6H x 5V	2372 (0.0 x 5.0)	107.2 x 89.6	75.0 x 9.1	KL00577
50HPS	**ED-17 clear med. base**	50	4000	6H x 5V	4216 (0.0 x 5.0)	107.2 x 89.6	75.0 x 9.1	KL00576
70HPS	**ED-17 clear med. base**	70	6300	6H x 5V	6640 (0.0 x 5.0)	107.2 x 89.6	75.1 x 9.1	KL00575
100HPS	**ED-17 clear med. base**	100	9500	6H x 5V	10013 (0.0 x 5.0)	107.2 x 89.6	75.0 x 9.1	KL00512
150HPS	**ED-17 clear med. base**	150	16000	6H x 5V	19425 (1.0 x 3.0)	103.3 x 85.9	74.0 x 7.2	KL00377
METAL HALIDE								
39PMH	**T-6 clear G-12 base**	39	3400	6H x 5V	3970 (1.0 x 3.0)	105.4 x 82.6	76.3 x 8.4	KL00485
50PMH	**ED-17 clear med. base**	50	4000	6H x 5V	4909 (1.0 x 3.0)	107.8 x 85.3	76.9 x 7.6	KL00498
70PMH	**ED-17 clear med. base**	70	5900	6H x 5V	7240 (1.0 x 3.0)	107.9 x 85.3	76.9 x 7.6	KL00497
100PMH	**ED-17 clear med. base**	100	8800	6H x 5V	10799 (1.0 x 3.0)	107.9 x 85.3	76.9 x 7.6	KL00392
150PMH	**ED-17 clear med. base**	150	12600	6H x 5V	12334 (0.0 x 1.0)	106.9 x 89.7	80.1 x 11.8	KL00401
175MH	**ED-17 clear med. base**	175	12800	6H x 5V	12530 (0.0 x 1.0)	106.9 x 89.7	80.1 x 11.8	KL00347

DBF18 Spot Grazer Beam Spread Chart

Lamp		Lamp Watts	Initial Lumens[1]	I.E.S. Type	Maximum Candlepower	Field Angle (10% of max.)	Beam Angle[2] (50% of max.)	Photometric Test No.
HIGH PRESSURE SODIUM								
35HPS	**ED-17 clear med. base**	35	2250	1H x 3V	8052 (0.0 x 47.5)	16.1 x 31.7	9.3 x 17.9	KL00454
50HPS	**ED-17 clear med. base**	50	4000	1H x 3V	14315 (0.0 x 47.5)	16.1 x 31.7	9.3 x 17.9	KL00441
70HPS	**ED-17 clear med. base**	70	6300	1H x 3V	22547 (0.0 x 47.5)	16.1 x 31.7	9.3 x 17.9	KL00437
100HPS	**ED-17 clear med. base**	100	9500	1H x 3V	33999 (0.0 x 47.5)	16.1 x 31.7	9.3 x 17.9	KL00371
150HPS	**ED-17 clear med. base**	150	16000	1H x 3V	66200 (0.0 x 47.5)	13.0 x 33.4	6.5 x 19.5	KL00375
METAL HALIDE								
39PMH	**T-6 clear G-12 base**	39	3400	1H x 1V	49146 (1.0 x 47.5)	9.9 x 15.4	5.7 x 7.2	KL00367
50PMH	**ED-17 clear med. base**	50	4000	1H x 2V	30524 (0.0 x 42.5)	12.4 x 18.2	6.9 x 11.6	KL00422
70PMH	**ED-17 clear med. base**	70	5900	1H x 2V	45023 (0.0 x 42.5)	12.4 x 18.2	6.9 x 11.6	KL00421
100PMH	**ED-17 clear med. base**	100	8800	1H x 2V	67153 (0.0 x 42.5)	12.4 x 18.2	6.9 x 11.6	KL00359
150PMH	**ED-17 clear med. base**	150	12600	1H x 2V	52380 (1.0 x 42.5)	16.8 x 26.4	8.6 x 13.4	KL00404
175MH	**ED-17 clear med. base**	175	12800	1H x 2V	53211 (1.0 x 42.5)	16.8 x 26.4	8.6 x 13.4	KL00350

[1] All **Initial Lumen** values shown are approximate and may vary from one manufacturer to another as well as one operating position to another. Consult lamp manufacturer's data for exact lumen and life data.
[2] **Beam Angle:** Horizontal and vertical beam spreads interpolated due to no valid I.E.S. standard.