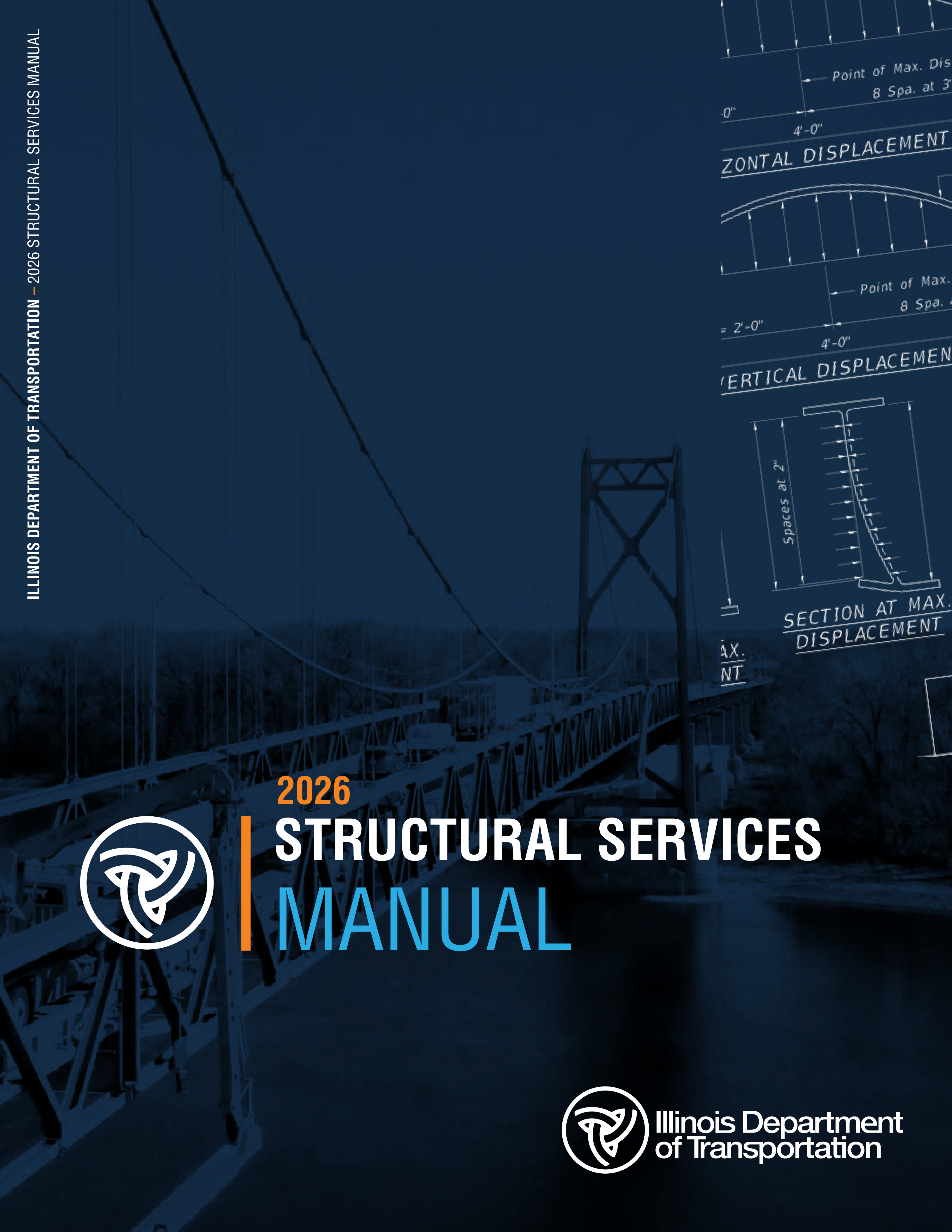
ILLINOIS DEPARTMENT OF TRANSPORTATION – 2026 STRUCTURAL SERVICES MANUAL



2026

STRUCTURAL SERVICES MANUAL

Illinois Department of Transportation

STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION BUREAU OF BRIDGES AND STRUCTURES	**FILE CLASSIFICATION:** Structural Services Manual **STRUCTURAL SERVICES MANUAL CHANGE LETTER NO.** 26-01
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The 2026 Structural Services Manual is available on the IDOT website. This manual shall replace the 2017 Structural Services Manual in its entirety.

The Illinois Structural Services Manual has undergone a comprehensive rewrite to ensure full compliance with the 2022 National Bridge Inspection Standards (NBIS) regulations. This updated edition features a complete reorganization of critical sections, specifically Section 3 (Bridge Inspection), to align with the new federal framework and the Specifications for the National Bridge Inventory (SNBI). Significant revisions include updated personnel qualification requirements for Program Managers and Team Leaders, enhanced procedures for Nonredundant Steel Tension Member (NSTM) inspections, and the integration of diverse interim policies into a unified standard. By incorporating these rigorous federal mandates, this manual provides the essential guidance necessary to improve statewide compliance and ensure the continued safety and reliability of Illinois' structural inventory.

Questions regarding the 2026 Structural Services Manual may be sent to dot.bbs.bridgemgmt@illinois.gov.

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Structural Services Manual

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Agency:

Illinois Department of Transportation

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Section 1 Introduction

As directed by the Illinois Department of Transportation (IDOT) Engineer of Bridges & Structures, it is the responsibility of the Engineer of Structural Services to develop, maintain, and administer the policies governing repair, inspection, and load rating of bridges and tunnels in Illinois. This applies to all bridges and tunnels carrying public traffic in the State of Illinois, regardless of maintenance responsibility. The IDOT *Structural Services Manual* is the vehicle by which these policies are controlled.

1.1 Organization and Functions

The Bureau of Bridges & Structures (BBS) is a part of the Office of Highway Project Implementation. The Engineer of Bridges & Structures, as head of the Bureau, is responsible for the planning, developing, and maintaining of the State's bridges. The Engineer of Structural Services is responsible for the bridge inspection program for the State of Illinois, structural investigations of existing bridges, the development or review of repair plans, determination of bridge load carrying capacity, establishment of posted weight limits, evaluation of overweight permit vehicle movements, and review and approval of Local Public Agency (LPA) projects.

The Section is comprised of four units: Bridge Investigations and Repair Plans Unit, Bridge Management and Inspection Unit, Local Bridge Unit, and the Structural Ratings and Permits Unit.

1.1.1 Bridge Investigations and Repair Plans Unit

Under the supervision of the Bridge Investigations and Repair Plans Unit Chief, this unit performs field investigations to identify the cause or extent of structural deficiencies, develops repair alternatives, and prepares plans to address deficiencies related to the deterioration of structural members or accidental or manmade damage. Field investigations performed by the unit also include those to evaluate reoccurring deficiencies associated with standard structural detailing practices to identify the elements contributing to the deficiencies and to offer solutions. The unit provides guidance to IDOT personnel, consulting engineers and other agencies engaged in the development of repair or maintenance projects, and this guidance is provided on a project specific basis and through the maintenance and updating of information contained in Section 2 Repairs of the IDOT *Structural Services Manual*, for which the unit is responsible. Repair, maintenance, and minor bridge rehabilitation projects prepared by IDOT personnel or consulting engineers are reviewed by the unit for comment and approval prior to being accepted for advertisement as a contract for letting. The unit also assists IDOT implementation personnel as required to resolve construction issues during the implementation of the projects reviewed or prepared by the unit. In order to comply with Federal Regulations and to ensure that IDOT obtains a proportional share of Federal funds, the unit tracks and assembles bridge construction cost information for submittal to the Federal Highway Administration (FHWA). For additional information related to unit procedures, Section 2 Repairs of the IDOT *Structural Services Manual* should be referenced.

1.1.2 Bridge Management and Inspection Unit

Under the supervision of the Bridge Management and Inspection Unit Chief, this unit is responsible for providing oversight of the bridge inspection procedures utilized by the various entities having maintenance responsibility for bridges carrying public traffic to ensure bridge safety as required by the National Bridge Inspection Standards (NBIS), 23 CFR Part 650 Subpart C. The Unit Chief serves as the Statewide NBIS Program Manager for bridge inspections in Illinois and is responsible for setting bridge inspection policy. The Unit Chief also ensures only qualified personnel inspect bridges in Illinois. This unit performs quality assurance reviews of agencies to ensure inspection programs meet the standards of the NBIS. These agencies include IDOT, the Illinois State Toll Highway Authority (ISTHA), Illinois Department of Natural Resources (IDNR), county, municipal and other entities having maintenance responsibility for bridges carrying public traffic. This unit is responsible for the publication of the IDOT *Structural Services Manual* and development and maintenance of Section 3 Bridge Inspection of the IDOT *Structural Services Manual* which summarizes the official bridge inspection policies for the State of Illinois. Unit personnel assist the Office of Planning and Programming during the maintenance and updating of the IDOT *Structure Information and Procedure Manual* (SIP Manual) to ensure compliance with the bridge safety and inventory provisions of the NBIS. The unit is responsible for ensuring bridge weight restrictions and closures are properly implemented. The unit is responsible for the IDOT Bridge Preservation Guide and assisting with IDOT’s Transportation Asset Management Plan (TAMP).

1.1.3 Local Bridge Unit

Under the supervision of the Local Bridge Unit Chief, this unit provides administrative and technical expertise to LPAs concerning local bridge matters and assists the Bureau of Local Roads and Streets during the development of policies and procedures for LPA bridges. LPA bridge rehabilitation and replacement projects utilizing Federal, State or Motor Fuel Tax funds, and other local projects requiring IDOT approval by State Statutes, are reviewed by the unit during project development to the degree necessary to ensure structural adequacy and compliance with IDOT policies and procedures. The unit provides services to counties, as required by State Statute, leading to the development of contract plans for bridge construction. Unit personnel conduct field inspections and perform analyses to determine the load carrying capacity of existing bridges in response to LPA requests, or to address changes in bridge conditions routinely reported by LPA inspection personnel. The unit establishes weight limits for LPA bridges to ensure highway safety and assists agencies in developing repairs to improve the condition of LPA bridges or to avoid the implementation of weight restrictions. LPAs coordinate with the unit, as necessary, to resolve construction issues and to evaluate permit requests for overweight vehicles. Unit personnel assist the Office of Planning and Programming during the maintenance and updating of the IDOT Structure Information and Procedure Manual to ensure compliance with the bridge safety and inventory provisions of the National Bridge Inspection Standards. For additional information related to unit procedures, the IDOT *Bureau of Local Roads and Streets Manual* should be referenced.

1.1.4 Structural Ratings and Permits Unit

Under the supervision of the Structural Ratings and Permits Unit Chief, this unit performs analysis and evaluations to determine the load carrying capacity of new and existing bridges under IDOT jurisdiction, as required by State Statute and Federal Regulations. When necessary, unit personnel perform field inspections of severely damaged or deteriorated bridges to obtain information regarding essential bridge elements for use in evaluating load carrying capacity. When necessary to address existing structural conditions, the unit issues directives to place weight restrictions on existing bridges under IDOT jurisdiction to ensure highway safety. The unit works cooperatively with the other units of the Structural Services Section for identifying repair alternatives to eliminate deficiencies that would otherwise require the implementation of a weight restriction. The Bureau of Operations routinely coordinates the review of overweight permit requests with the unit prior to authorizing the movement of overweight vehicles to ensure that highway infrastructure is not damaged. In order to ensure that bridge load carrying capacities can be determined in an expeditious manner, the unit maintains databases of structural information for use during the evaluation of overweight permit vehicle movements or the effect of damage or deterioration on bridge load carrying capacity. The movement of heavy construction equipment across existing bridges to facilitate construction projects is evaluated by this unit, as well as the feasibility of placing additional wearing surface on existing bridges located within the limits of roadway resurfacing projects. The unit is responsible for the development and maintenance of

Section 4 Load Rating of the IDOT *Structural Services Manual*. The unit reviews proposed legislative changes to the Illinois Vehicle Code to determine the effect of the changes on highway system bridges and coordinates with other Units and Bureaus for developing comments in regard to anticipated effects. Unit personnel assist the Office of Planning and Programming during the maintenance and updating of the IDOT *Structure Information and Procedure Manual* (SIP Manual) to ensure compliance with the bridge safety and inventory provisions of the National Bridge Inspection Standards (NBIS), and represents IDOT in matters pertaining to the maintenance, revision or updating of bridge rating specifications that may be proposed by the American Association of State Highway and Transportation Officials (AASHTO).

Section 2 Repairs

2.1 General

2.1.1 Purpose and Scope

This section has been developed to assist IDOT personnel and consulting engineers during the preparation of bridge repair plans. Serviceability and performance history were considered in the development of the procedures and details presented in this section. The inclusion of specific details in this section for the repair of every type of structural deterioration or damage that may be encountered is not possible. In preparing this section, information has been included for the types of repairs most often required to adequately maintain typical bridge structures during their service life. The details and procedures provided in this section are intended to address routine repair requirements and to provide a foundation to assist in the preparation of unusual repair details, and they represent the current practice. Details and practices not currently used can be found in previous updates of this section and are mainly intended for historical reference only.

IDOT's *Bridge Preservation Guide* also provides recommended schedule and condition-based criteria for Bridge Preservation Activities meant to extend the life of bridges. Many of the procedures and details in this manual are also useful in implementing the preservation activities discussed in the *Bridge Preservation Guide*. Examples of those activities include expansion joint replacement, bridge deck overlays and bearing repair and replacement.

To ensure that the repair procedure or detail being used performs as intended it is critical that the designer clearly understand the causes of the damage or deterioration being repaired.

All structure repair plans for maintenance contracts and Day Labor awards will be subject to the requirements of this section. The Design and Planning Sections of the IDOT *Bridge Manual* should be used as the primary reference when developing plans for bridge rehabilitations and the details and procedures provided in the Repair Section may be used during the final preparation of repair details associated with the rehabilitation project.

2.1.2 General Notes

The following general notes should be included in repair plans, when applicable. Additional general notes from the Design Section of the IDOT Bridge Manual shall also, if necessary, be included in repair plans, when applicable.

	NOTE	**APPLICATION**
1.	All structural steel shall conform to AASHTO Classification M-270 Gr. 36, unless otherwise noted.	When the structural steel elements are to be connected to an existing Grade 36 steel member.
2.	Fasteners shall be ASTM F 3125 Grade (A325) Type 1, (mechanically) galvanized bolts. Bolts $\frac{3}{4}''\phi$, holes $\frac{13}{16}''\phi$, unless otherwise noted.	Omit ( ) when placing in plans but replace items within the ( ) with A490 and Hot-dip as applicable to the project.
3.	Fasteners shall be ASTM F 3125 Grade (A325) Type 1, (mechanically) galvanized bolts. Bolts $\frac{7}{8}''\phi$, open holes $\frac{15}{16}''\phi$, unless otherwise noted.	Omit ( ) when placing in plans but replace items within the ( ) with A490 and Hot-dip as applicable to the project.
4.	Fasteners shall be ASTM F 3125 Grade (A325) Type 1, (mechanically) galvanized bolts. Flange splice holes shall be $\frac{15}{16}''\phi$ for $\frac{7}{8}''\phi$ bolts. Web splice holes shall be $\frac{13}{16}''\phi$ for $\frac{3}{4}''\phi$ bolts. (Place on the beam replacement sheet in lieu of General Note N2 or N2A.)	When installing new splice plates at existing splice locations that have different size bolts. Omit ( ) when placing in plans but replace items within the ( ) with A490 and Hot-dip as applicable to the project.
5.	The Contractor shall provide support and/or shoring systems for the slab and beam in the area of existing beam removal. See Special Provisions “Temporary Shoring and Cribbing” and “Temporary Slab Support System.” (Use on BEAM REPLACEMENT jobs.)	When replacing portions of non-composite beams or using a “zipper” beam replacement detail without removing the existing deck slab.
6.	The Contractor shall provide support and/or shoring systems for the beam in the area of existing pin (and link plate) replacement. See Special Provision “Temporary Support System.” (Use on PIN AND LINK PLATE REPLACEMENT and PIN REPLACEMENT jobs.)	When supporting hanging section of the span in order to replace pins and/or link plates.

7.	The Contractor shall grind all cracked welds parallel to the direction of the existing weld and not perpendicular to the weld. (May place next to specific detail)	
8.	Grind existing nicks, gouges and shallow cracks in the damaged beams as detailed. Grinding shall be done parallel to the longitudinal axis of the member. Ground surfaces shall be inspected for cracks using dye penetrant or magnetic particle testing prior to initiating any beam straightening operations. Any cracks that cannot be removed by grinding approximately 1/4″ deep shall be identified and reported to the Bureau of Bridges and Structures for further disposition. Ground surfaces shall be spot cleaned and painted with an aluminum epoxy mastic primer followed by a finish coat to match the color of the existing beam. Cost of grinding, testing and spot painting included with Beam Straightening.	All beam straightening projects where nicks, gouges or shallow cracks may be present on the damaged beams.
9.	Reinforcement bars designated (E) shall be epoxy coated.	Place in General Notes only.
10.	Prior to pouring the new concrete deck, all heavy or loose rust, loose mill scale, and other loose detrimental foreign material shall be removed from the surfaces in contact with concrete (SSPC-SP3 standards). Tightly adhered paint may remain unless otherwise noted. Removal shall be accomplished by methods that will not damage the steel, and the cost will be paid for according to Article 109.04 of the Standard Specifications. As directed by the Engineer, existing construction accessories welded to the top flange of beams and girders shall be removed. The weld areas shall be ground flush and inspected for cracks using magnetic particle testing (MT) or dye penetrant testing (PT) by qualified personnel approved by the Engineer. Any	

	cracks that cannot be removed by grinding ¼ inch deep shall be identified and reported to the Bureau of Bridges and Structures for further disposition. The cost of removing welded accessories, grinding and inspecting weld areas and grinding cracks will be paid for according to Article 109.04 of the Standard Specifications.	
11.	Tapered shims shall be added under the stools, as required by the Engineer, to make a smooth finger joint. Cost shall be included with Furnishing and Erecting Structural Steel. The finger plates shall be flame cut as provided in Article 505.04(k) of the Standard Specifications.	To ensure proper sitting of finger plate joint stools on supporting element.
12.	Plan dimensions and details relative to the existing structure have been taken from existing plans and are subject to nominal construction variations. The Contractor shall field verify existing dimensions and details affecting new construction and make necessary approved adjustments prior to construction or ordering of materials. Such variations shall not be cause for additional compensation for a change in scope of the work, however, the Contractor will be paid for the quantity actually furnished at the unit price bid for the work.	
13.	(Finger Plate) Expansion Joints shall be assembled in their final relative position with the ends in place for shop inspection and acceptance.	Finger plate expansion devices. Place note with finger plate details.
14.	Cost of removal and re-installation of all members necessary to complete the work as detailed on the plans and as specified in the Special Provisions shall be included with Furnishing and Erecting Structural Steel or Structural Steel Repairs.	
15.	The Inorganic Zinc Rich Primer / Acrylic / Acrylic Paint System shall be used for shop and field painting of new structural	When painting new steel is required. Cost included with Furnishing and Erecting Structural Steel or Structural Steel Repair.

	steel except where otherwise noted. The color of the final finish coat shall be ________________, Munsell No. ________. (Use with new structural steel.)	Colors to match existing steel: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1
16.	The Inorganic Zinc Rich Primer / Acrylic / Acrylic Paint System shall be used for shop and field painting of new structural steel except where otherwise noted. The color of the final finish coat shall be ________________, Munsell No. ________. Cost included with Pin (and Link Plate) Replacement. (Use on PIN AND LINK PLATE REPLACEMENT and PIN REPLACEMENT jobs.)	Colors to match existing steel: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1
17.	All existing steel surfaces behind link plates shall be cleaned and primed before installation of new link plates. Cost included with Pin and Link Plate Replacement. (Use with N13A on PIN AND LINK REPLACEMENT jobs.)	
18.	The existing structural steel coating contains lead. The Contractor shall take appropriate precautions to deal with the presence of lead on this project. (Use as applicable - see Bridge Office Memorandum 97.8; i.e. generally steel erected prior to 1986)	Steel structures erected prior to 1986 (or as determined from existing plans) with lead based primer. Consider using too on structures with hard to reach locations that have been previously cleaned and painted and in which lead residue may still be present.
19.	Existing structural steel that will be in contact with new structural steel shall be cleaned and painted prior to erection as required by the Special Provision "Cleaning and Painting Contact Surface Areas of Existing Steel Structures", and the Standard Specifications. The color of the final finish coat shall be , Munsell No. . Cost included with Structural Steel Repair.	When new structural steel is being provided and attached to existing structural steel as part of a repair project. Colors to match the existing steel: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1
20.	Existing structural steel that will be in contact with new structural steel shall be cleaned and painted prior to erection as	

	required by the Special Provision "Cleaning and Painting Contact Surface Areas of Existing Steel Structures". Cost included with Pin (and Link Plate) Replacement. (Use on PIN AND LINK PLATE REPLACEMENT and PIN REPLACEMENT projects.)	
21.	Cleaning & painting of contact surfaces shall meet the requirements for Primary Connections as specified in the special provision for "Cleaning and Painting Contact Surface Areas of Existing Steel Structures".	Place note on specific sheet where primary connections are being identified.
22.	No field welding is permitted except as specified in the contract documents.	
23.	All new structural steel shall be shop painted with the inorganic zinc rich primer per AASHTO M300, Type 1. Cost included with XXXXXXX(Use with limited use of structural steel, e.g. only joint plates. Will not need Specials for Painting.)	To be used in Day Labor projects.
24.	All new structural steel and bearing assemblies shall be hot-dip galvanized. See Special Provisions for "Hot Dip Galvanizing for Structural Steel".	To be used in all contracts for strengthening repairs, diaphragms and small repairs.
25.	Existing reinforcement bars extending into the removal area shall be cleaned, straightened, and incorporated into the new construction. Any reinforcement bars that are damaged during concrete removal shall be replaced with an approved bar splicer or anchorage system at the contractor's expense.	
26.	Load carrying components designated "CVN" shall conform to the Charpy-V-Notch Impact Energy Requirement, Zone 2.	Components designed for tensile stress require at least a minimum toughness to avoid crack propagation. These components include wide flange beams, tension flanges, webs of plate girders, all splice plate material except fill plates, and bracing designed for live load in curved or highly skewed (>60°) structures.

27.	The minimum thickness of the concrete overlay shall be __″ and varies as required to adjust for the existing profile grade and beam camber.	To be included with overlays placed on PPC I-Beams or PPC Deck Beams.
28.	The Contractor shall use extreme care during concrete removal so as not to damage the PPC I-Beam.	When concrete removal operations of the deck are being done directly over any portion of the PPC I-Beam.
29.	Diaphragm connection holes shall be $^{15}/_{16}$″ϕ for $^{3}/_{4}$″ϕ bolts. Two hardened washers shall be required at diaphragm connections.	
30.	The Pins and Link Plates shall conform to the minimum Charpy V-Notch Toughness of 25 ft.-lbs. at 40 °F.	
31.	The pins, link plates, bushings, nuts and silicone sealant are the items included in Pin and Link Plate Replacement. (Use on PIN AND LINK PLATE REPLACEMENT and PIN REPLACEMENT jobs. Include additional items as necessary, i.e. fill plates, plate washers, high strength bolts.)	
32.	Epoxy grout XX(E) bars in 9” min. holes according to Art. 584 of the Standard Specifications.	When grouting reinforcement into existing concrete. Place note near detail.
33.	Any pins that can be easily removed without damage to the pin shall be salvaged and the Bridge Engineer shall be contacted for disposition. Cost of salvage included with Pin and Link Plate Replacement. (Use for salvage of pins for Wyse-Genny)	
34.	Areas of deck repairs shown are estimated.	To be used when field measurements of deck slab damage cannot be obtained prior to development of plans.
35.	The Engineer shall show actual locations and size of deck repairs on As-built Plans.	
36.	The deck surface shall have its final finish tined according to Article 420.09(e)(1) of the Standard Specifications. Cost included	To be used when new concrete area is small, and no grooving is called for (mainly at joint replacement areas).

	with Concrete Superstructure. (Use on Day Labor Projects when Bridge Deck Grooving is not performed) (Use for all expansion joints when concrete is being poured for the portion of deck next to the joint.)	
37.	If the analysis submitted by the Contractor for the jacking/temporary support system to be used shows temporary stiffeners are required to prevent web crippling or buckling, the stiffeners shall be steel and bolted to the web. If stiffeners are not required, hardwood timbers shall be installed tightly between the top and bottom flange to prevent flange rotation.	If stiffeners are to be left in place, then they should be properly painted with paint that matches the existing steel. If stiffeners are removed, then holes on the web must be filled with HS bolts.
38.	Joint openings shall be adjusted according to Article 520.04 of the Std. Specs. when the deck is poured at an ambient temperature other than 50° F.	
39.	The number, location and orientation of support boxes shall be determined by the manufacturer. All boxes shall be located to miss the top flanges of the girders. Modular expansion joints shall be assembled in their final relative position with the ends in place for shop inspection and acceptance.	Modular expansion devices. Place note with modular joint details.
40.	Two 1/8 in. adjusting shims shall be provided for each bearing in addition to all other plates or shims and placed as shown on bearing details	For Type I bearings shims should be detailed between the bearing or bearing extension and the flange, and not extend beyond their mutual contact area. Place note on applicable bearing detail sheet.
41.	Prior to beginning any repair work, the contractor shall be responsible for providing a preloading system on the bridge deck over the existing damaged beam at the specified locations. The preloading system should produce a total maximum service load moment as shown at the centerline of the damaged area. Preloading shall be kept in place for at least three 3 days after completion of	Use with PPC I-Beam Repairs when pre-loading is required.

	concrete repair or until the concrete has reached an ultimate strength of 5,000 psi.	
42.	The contractor's proposed preloading system with computations sealed and signed by an Illinois Structural Engineer shall be submitted to the Bureau of Bridges and Structures for approval. The preloading system shall be placed shortly after bridge closure for repairs, and shall not be paid for separately but will be included in the unit price for PPC –I-Beam Repairs.	Use with PPC I-Beam Repairs when pre-loading is required.

2.2 Plan Preparation and Review

2.2.1 Responsibilities

The responsibility for the preparation of plans, special provisions and estimates for Maintenance Contracts and Day Labor awards rests with the Districts. The plans and special provisions may be prepared by District personnel or by a consulting engineer. Consulting engineers used for the preparation of repair plans and special provisions should be retained by listing the proposed project in the Professional Transportation Bulletin (PTB).

When repair plans are prepared by District personnel, the Bridge Investigations and Repair Plans Unit (Repairs Unit) of the Structural Services Section of the Bureau of Bridges & Structures will prepare the details and special provisions necessary for the repair of deteriorated structural elements which will be included in a project. The District should submit a request for the preparation of structural repair details to the Repairs Unit with the following information:

1. Structure Number
2. Route Number
3. Project Number
4. County Name
5. Marked Route
6. Feature Crossed
7. Inspection Information (including photos and sketches)
8. Description of Proposed Structural Repairs
9. Tentative Letting or Award Date
10. Contract Number or Day Labor Project Number (if available)

Upon completion of the structural repair details, the Repairs Unit will provide the District with a copy of the structural repair detail sheets which will be included with the plans prepared by the District.

When a consulting engineer prepares the repair plans, the consulting engineer is responsible for the preparation of all repair details, including the structural repair details, quantities and special provisions, necessary to complete the project.

2.2.2 Preparation Procedures

Bridge Condition Reports, available pictures/sketches, field reports, Load Rating Information (LRI) report (if available), existing fabrication plans and existing design plans should be consulted prior to commencing repair plan preparation. Electronic 11 in. x 17 in. plans with electronic signature and seal are archived for permanent record. The Computer Aided Design, Drafting, Modeling & Deliverables Manual (available at the primary IDOT CADD page) shall be referenced for proper font, font size, line weights and general presentation of plans. The plan presentation guidelines provided in the Design Section of the IDOT Bridge Manual should also be applied to repair plans keeping in mind that repair plans are intended to address specific deficiencies and/or improvements on a structure. As such they should include only information pertinent to the work being done and useful to the contractor. This simplifies the plan preparation and review.

2.2.3 Review Procedures

All Maintenance Contract and Day Labor plans prepared for the repair or retrofit of existing structures must be submitted to the Bureau of Bridges & Structures for review. The plans should be submitted for review prior to their submittal to the Bureau of Design and Environment for letting or to the Bureau of Operations' Day Labor Section for award. The plans should be submitted to the Bureau of Bridges & Structures sufficiently prior to the anticipated letting or award date to allow adequate time for review and revision of the plans. Review time will vary depending on the size and complexity of the project and on the current workload within the Bureau. Typically, project plans should be submitted for review a minimum of 90 days prior to the scheduled date for submittal of the plans, specifications and estimates (PSE) to the Bureau of Design and Environment for letting. In order for the Bureau of Bridges & Structures to be able to prioritize the review of projects submitted for review, it is important that each submittal for review be accompanied by information stating an anticipated letting or award date.

Note that the Repairs Unit reviews plans prepared by district personnel and design consultants, as well as developing plans done by their own in-house staff. Review and plan preparation priority is generally based on:

1. Emergency repairs to structures to correct structural issues.
2. Repairs to structures needed to prevent or lift a load posting.
3. Repairs to structures on a specific letting.
4. Repairs districts would like to implement as part of their normal maintenance schedule.

No bridge related repair plans for Maintenance Contracts or Day Labor awards are exempt from review by the Bureau of Bridges & Structures. All plans, including those for minor operations such as bridge rail and parapet repairs, slopewall repairs, rip rap placement, standard bridge rail installation and in-kind removal and replacement of HMA surfaces must be submitted for review prior to submittal to the Bureau of Design and Environment for Maintenance contract projects or the Bureau of Operations for Day Labor projects.

Plans prepared by District personnel or consulting engineers for bridge maintenance or repair projects will be reviewed by the Repairs Unit of the Structural Services Section of the Bureau of Bridges & Structures. Plan submittal is normally done electronically, must include a transmittal letter and should be sent to:

Unit Chief, Bridge Investigations and Repair Plans Unit at dot.bbs.repairs@illinois.gov

Plans submitted for review by consultant engineers must be in final form, ready to be placed on a letting or be sent out for award. Partially completed plans will be returned.

All Contract Maintenance and Day Labor repair plans prepared by District personnel must be sealed by the Bureau Chief of Bridges and Structures prior to submittal to the Bureau of Design and Environment for letting or the Bureau of Operations for award. Plans prepared by consulting engineers must include the Illinois structural seal of the Engineer of Record.

Plans prepared by consulting engineers for bridge replacement or rehabilitation projects will be reviewed by the Design Section of the Bureau of Bridges & Structures. Submittal procedures for these types of plans can be found in the IDOT *Bridge Manual*.

In most situations, maintenance and repair projects may be categorized as those projects for which a Type, Size and Location Plan (TSL) was not prepared. Project plans developed without the initial preparation of a TSL should be submitted to the Structural Services Section. After the submittal of the plans, if the scope of the project is determined to be beyond the scope of maintenance or repair, the Structural Services Section will forward the plans to the Design Section for review.

2.3 Inspections

2.3.1 Responsibilities

The District is responsible for the inspections required to produce repair plans prepared by IDOT personnel. The inspections should include the collection of information describing and locating the deterioration or damage. The Bureau of Bridges & Structures will provide assistance to the Districts during inspections, when requested to do so by the District. Assistance by Bureau of Bridges & Structures personnel will be limited to providing guidance to District personnel during the inspection, on the collection of data required to adequately determine the extent and nature of the repairs needed. Also, the Bureau of Bridges & Structures may provide the District with inspection equipment and operating personnel when inspection scheduling does not conflict with other Bureau responsibilities.

Detailed descriptions of structure inspection types, intervals and responsibilities can be found in Section 3.

2.3.2 General Deterioration

The information collected during the inspection of deteriorated structures should include a framing plan and detailed sketches of the affected members. The framing plan should indicate the locations of the deteriorated areas on the structure. The sketches should be dimensioned to show the extent and the location of the deterioration relative to a fixed point of reference, such as CL of bearing, CL of Pier, CL of splice or the end of the member. An estimate of the maximum amount of section loss should be given and the size and location of any corrosion holes, cracks and areas of section loss in the member should be shown on the sketch. Figure 2.3.2-1 shows a sample framing plan and sketch documenting the location and extent of structural deterioration on a steel girder. Photographs of all deteriorated locations requiring repairs should also be provided (See Section 2.3.5).

During an inspection, particular attention should be given to the structural elements in the area of existing deck drains, expansion joints and those areas behind diaphragms and cross-frames which are not readily visible. The presence of previous repairs in the area should be noted. Also, personnel inspecting tension members should be alert for the presence of tack welds, plug welds and intersecting welds. If they are found, they should be documented and promptly reported.

When inspecting abutments and piers to determine areas of unsound concrete, an effort should be made to determine the depth of concrete deterioration. The depth of unsound concrete can be determined using a chipping hammer, masonry drill or concrete cores. When the depth of deterioration exceeds 12 inches, or significantly reduces the effective size of the member, a structural evaluation is needed to determine repair requirements. Of particular importance is the extent, if any, and proximity of the deterioration to existing bearing plates. The length and size of all cracks in substructure units should be determined. Although hairline cracks are not repaired, they should be recorded in the inspection sketches and designated as "hairline". An example inspection sketch showing areas of deterioration is shown in Figure 2.3.2-2.

Inspections should be done considering the potential repair that may be necessary. Any information that will aid in the development of the repair details will greatly enhance the effectiveness of the repair and simplify the designer’s and contractor’s work.

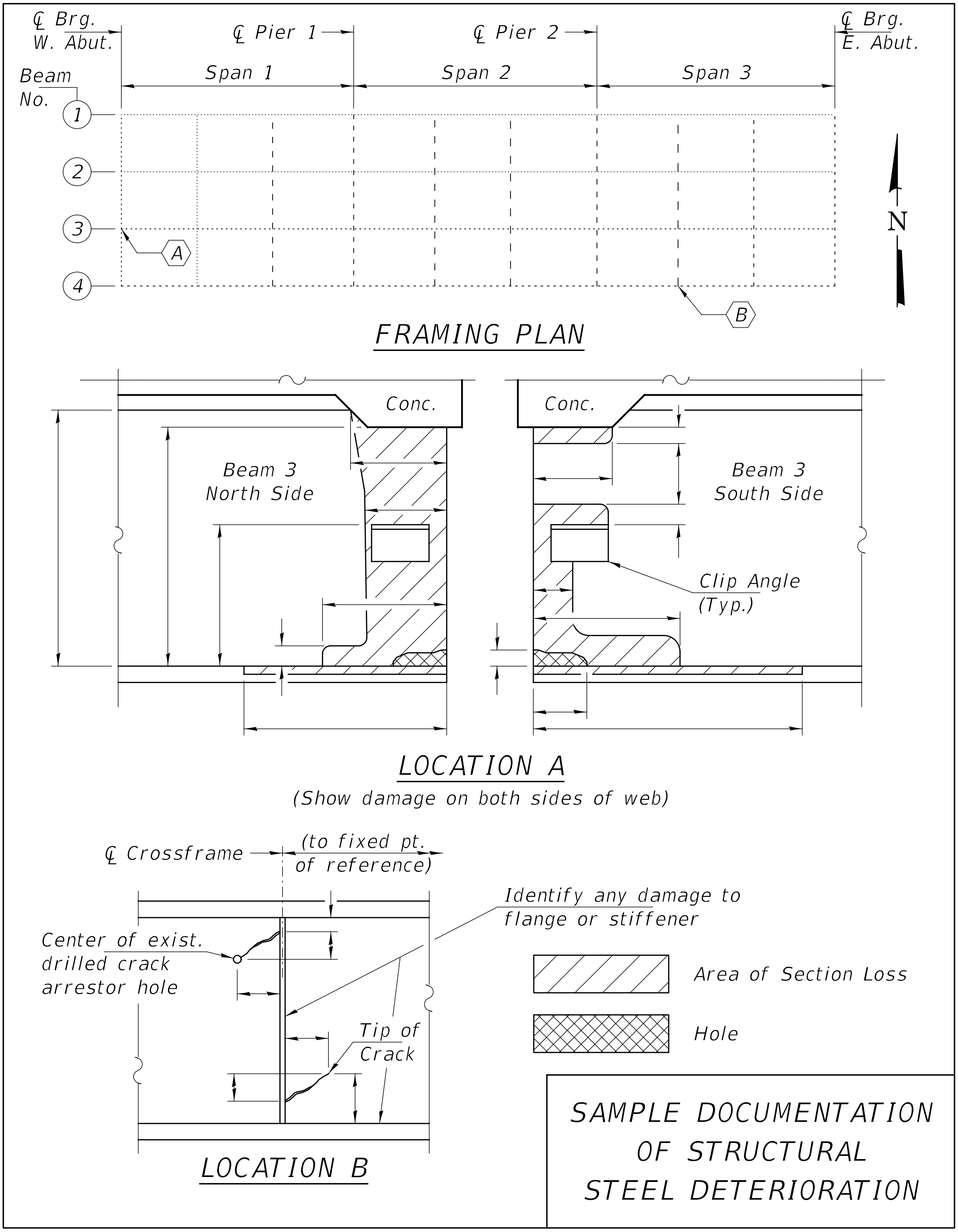


Figure 2.3.2-1: Sample Documentation of Structural Steel Deterioration

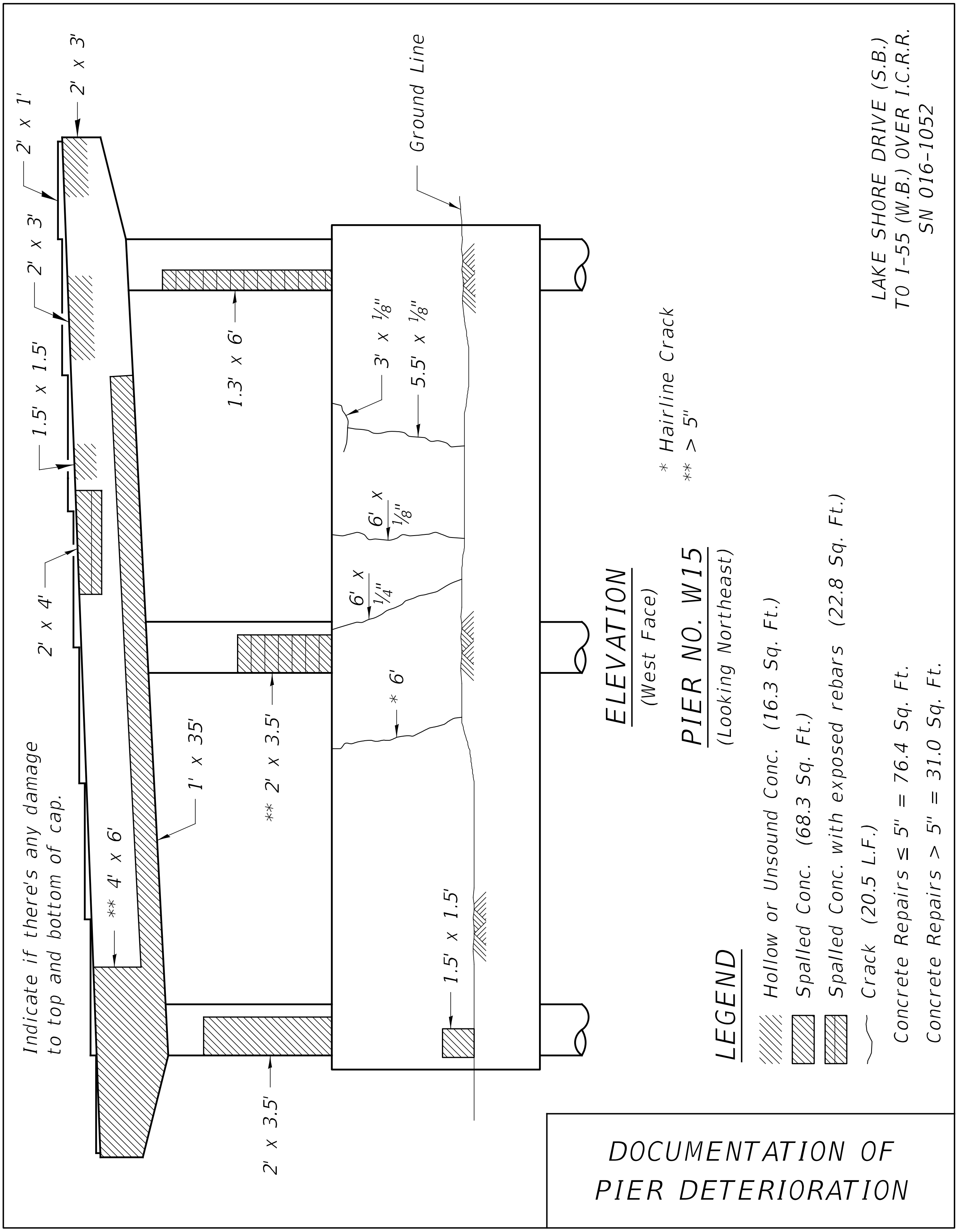


Figure 2.3.2-2: Documentation of Pier Deterioration

2.3.3 Impact Damage - Steel Structures

The inspection of steel beams or girders which have been struck by a vehicle is the first procedure necessary in determining whether to repair or replace a damaged beam. Depending on the extent of the damage, the beams may be replaced, straightened (note that the current practice allows an impacted beam to be straightened only once with a second impact on the same area resulting in the beam needing to be replaced) or straightened and strengthened. In many cases the need for beam replacement is obvious (e.g. cracked flange, excessive deformation, etc.); however, a beam may be damaged to a degree where visual observation alone is not sufficient to determine repair or replacement requirements. In order to more accurately determine if a beam can be straightened or must be replaced, measurements of impact deflection must be obtained. The measurements of the damage should be done in accordance with the guidelines presented in the National Cooperative Highway Research Program (NCHRP) Report 271 (Guidelines for Evaluation and Repair of Damaged Steel Bridge Members), with the following exceptions. NCHRP Report 271 indicates that deflection measurements should be obtained for the entire length of the beam where impact deflections have occurred. However, it has been found that it is not necessary to obtain deflection information for the entire deflected length of the beam when determining repair requirements. In order to reduce traffic lane closure time and the length of time inspection crews are exposed to traffic, measurements of vertical and horizontal beam flange deflection should typically be obtained at horizontal intervals of 3-inches for a distance of 2-feet on each side of the point of maximum displacement. This point is usually located at the point of impact where the highest local strains have occurred in the beam. Also, vertical and horizontal flange deflections should be obtained at horizontal intervals of 3-inches for a distance of 4-feet from the center of any deflected beam splice toward the adjacent substructure unit. Horizontal web deflections should also be measured at 2-inch vertical intervals for the depth of the web at the most severely deflected web section and adjacent to a deflected beam splice. Additional measurements of the web deflection should then be taken in a similar manner at a location 3-inches each side of the most severely deflected web section. The deflection measurements of the flange and web described above need not necessarily be the total deflection which has occurred relative to the beam's original position at the points being measured. The deflections being measured at 2 or 3-inch intervals must be measured relative to a common reference plane which is approximately parallel to the original location of the undamaged beam. Figure 2.3.3-1 illustrates the recording of localized impact deflections for steel beams and girders.

A newly developed technique to measure damage on impacted steel beams/girders is likely to be beneficial in the future. The new method involves the use of scanners to obtain three-dimensional measurements which when compared to original dimensions will provide the needed information with respect to deflections caused by an impact. The best results are obtained from structures which are geospatially located or have permanent markers installed. One of the greatest benefits of this method is the fact that measurements can be taken without encroaching into the traffic lanes.

In addition to the localized impact deflection information, an overall description of the extent of damage and existing beam conditions should be provided. This should consist of photographs, sketches and framing plans of the beams and diaphragms showing:

1. The location of impact on the beams which were hit referenced to a fixed point on the structure.
2. The extent of impact deflection along each damaged beam referenced to a fixed point on the structure or the point of impact.
3. The location of damaged diaphragms or cross-frames and the connecting angles or plates.
4. The location and dimensions of cracked welds and flange or web gouges.
5. The location and extent of damage to the concrete slab or fillet encasing the top flange of the beam.
6. Any factors or conditions (such as previous repairs, utilities, sign structures, clearances or beam deterioration not related to the impact) which must be considered during the preparation of repair plans.

An example framing plan recording impact damage information is shown in Figure 2.3.3-2. Figure 2.3.3-3 illustrates the documentation of damage to the concrete fillet encasing the top flange of a non-composite beam. Also, a cross section of each damaged beam should be provided showing:

1. The maximum horizontal and vertical impact deflection of the beam.
2. The maximum rotation of the flange.

Figure 2.3.3-4 illustrates the documentation of maximum beam distortion.

Similar documentation is to be provided for each beam if multiple beams have been impacted.

For repair procedure and details see Section 2.13.

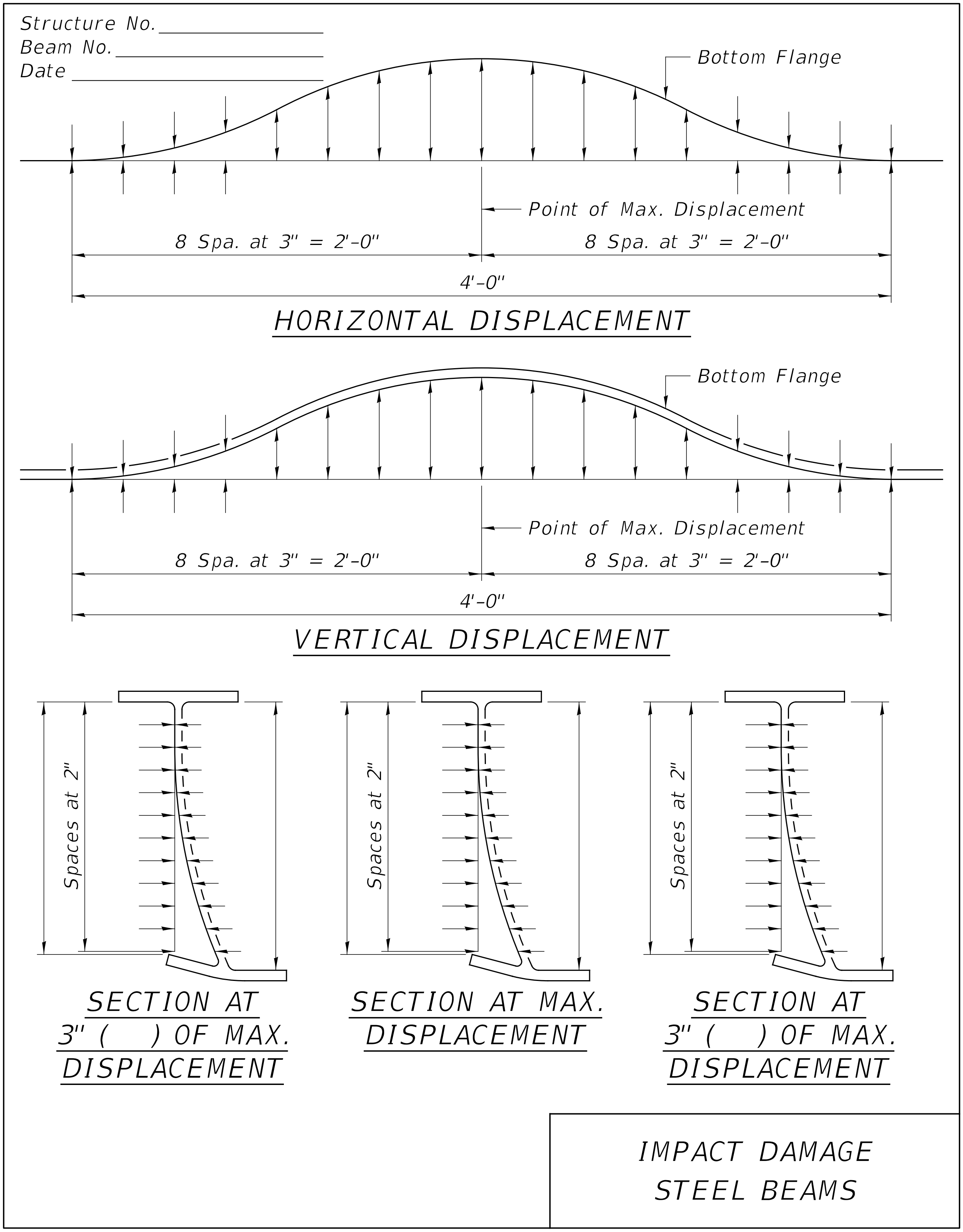


Figure 2.3.3-1: Impact Damage – Steel Beams

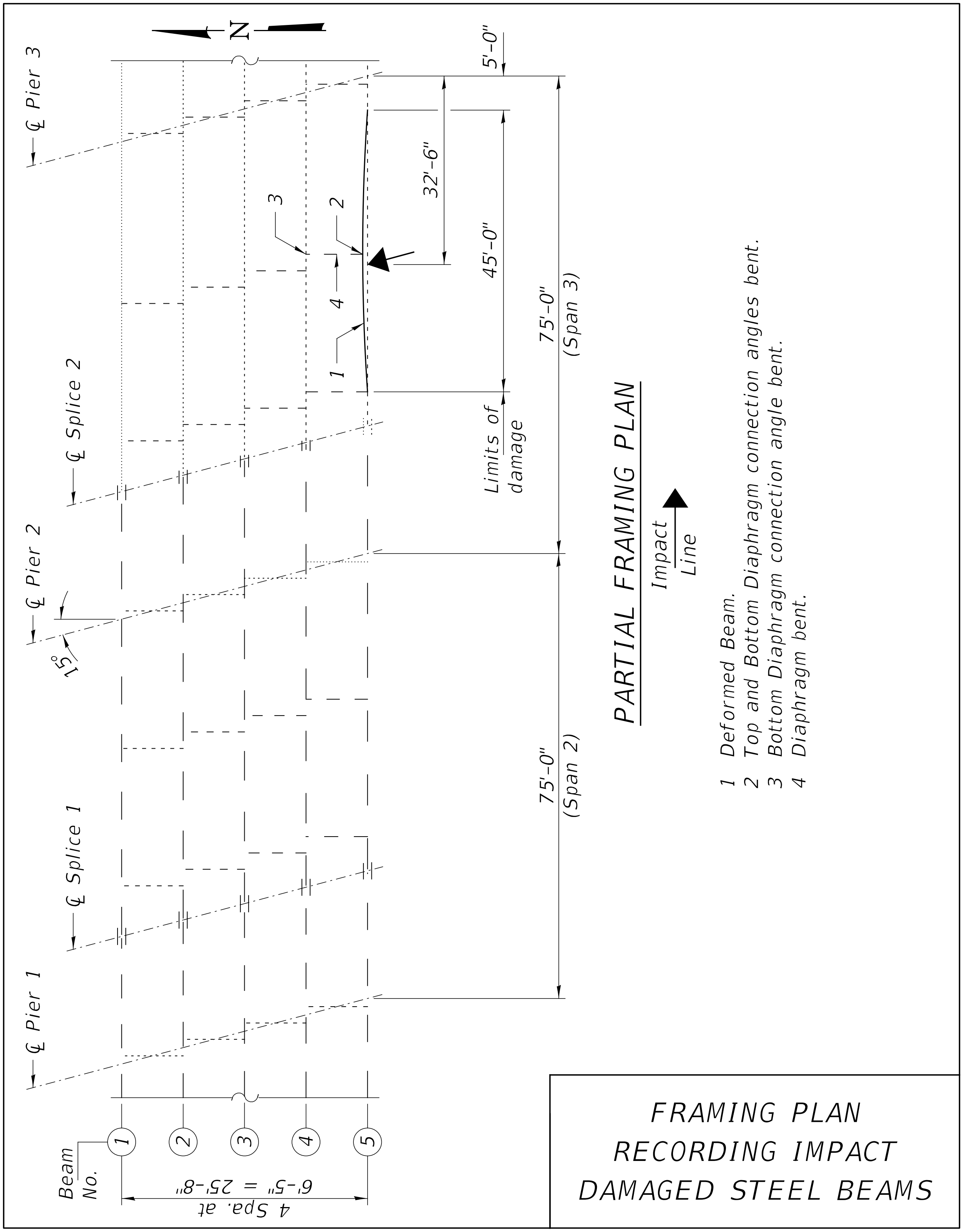


Figure 2.3.3-2: Framing Plan Recording Impact-Damaged Steel Beams

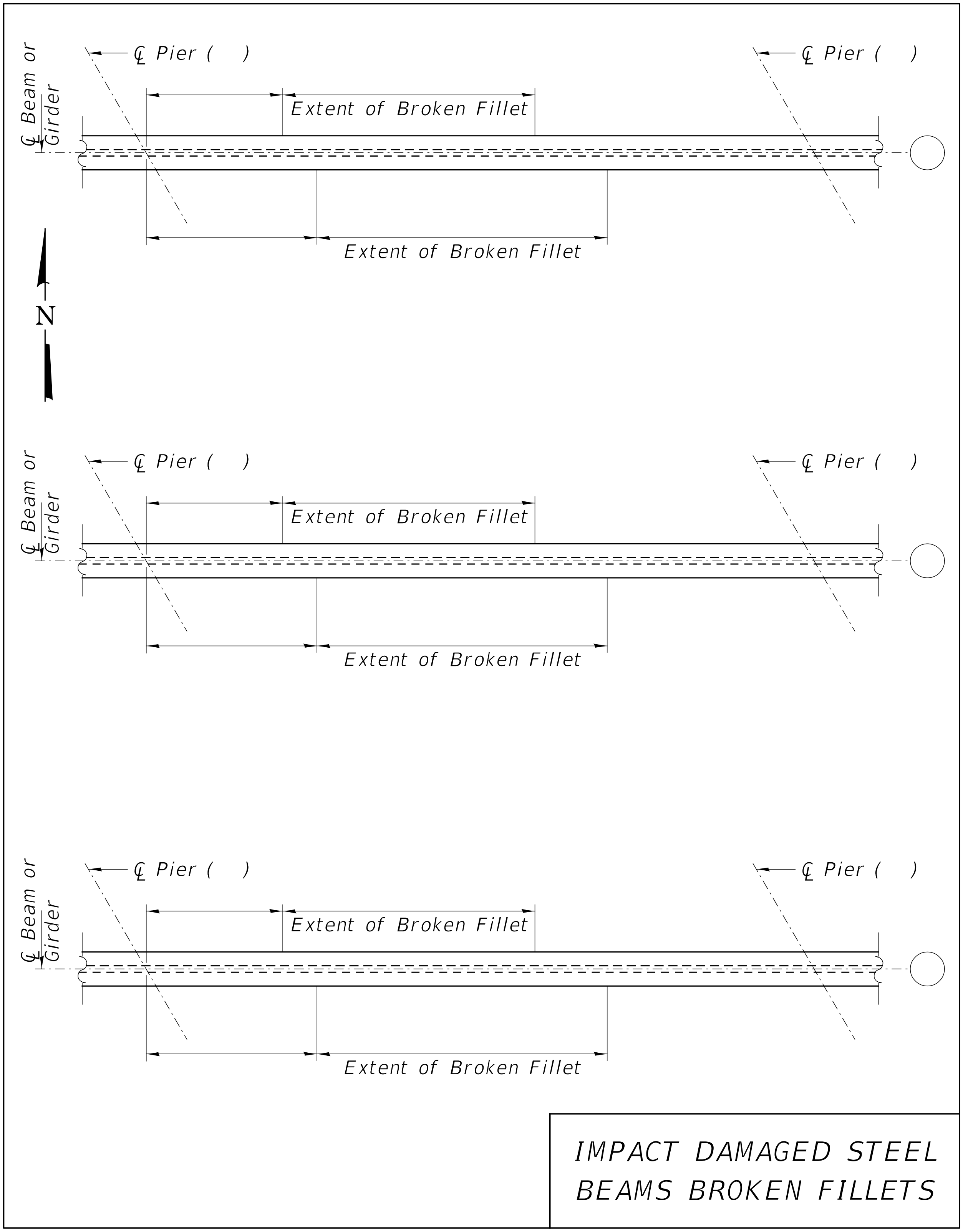


Figure 2.3.3-3: Impact-Damaged Steel Beams – Broken Fillets

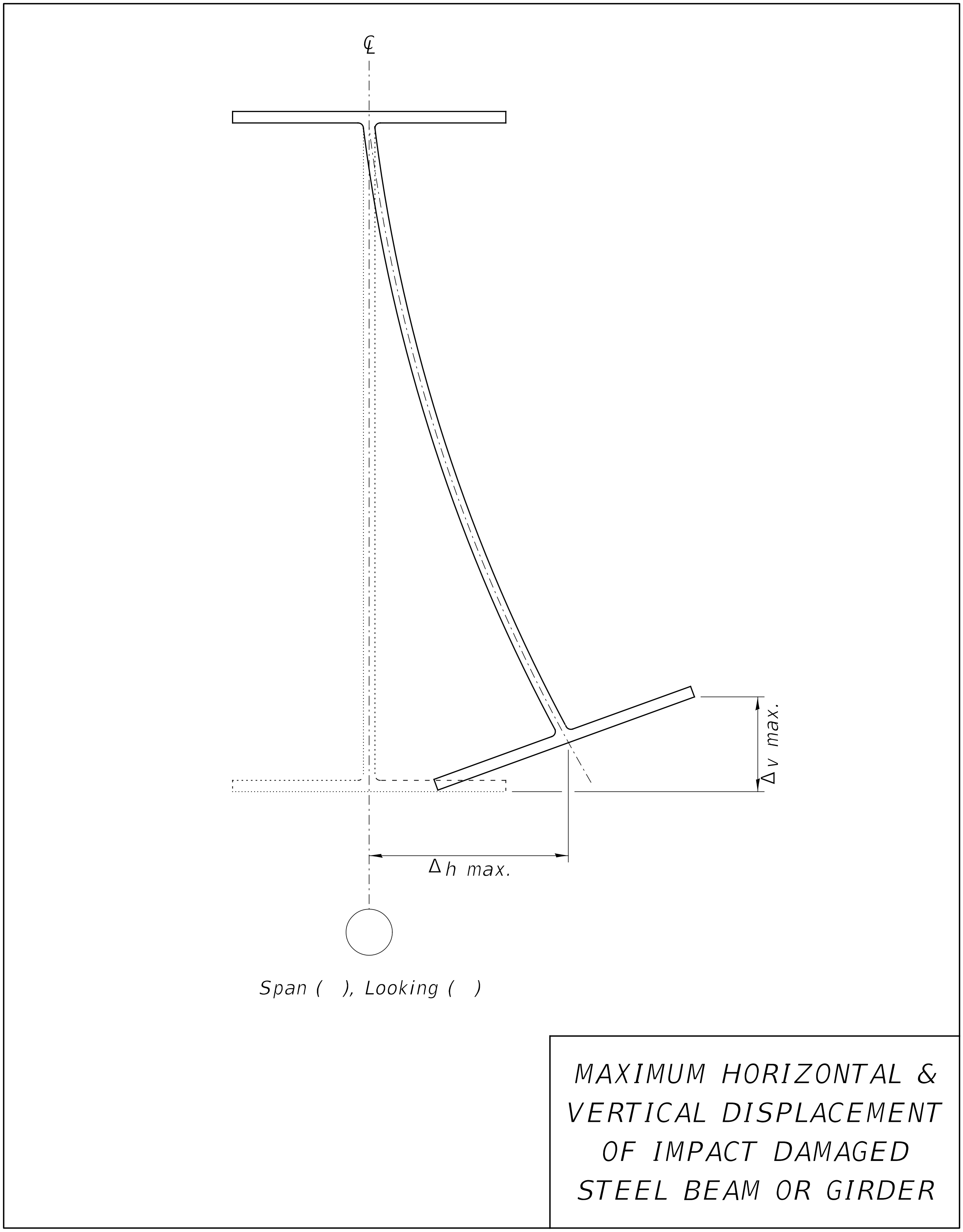


Figure 2.3.3-4: Max. Horiz. & Vert. Displacement of Impact-Damaged Steel Beam / Girder

2.3.4 Impact Damage - Concrete Structures

The inspection of impact damage to concrete structures should include information locating all cracks and areas of substantial section loss. This usually requires an elevation view of each side of the beam and a view of the bottom of the beam marked to show crack and section loss locations relative to a fixed point of reference as illustrated in Figure 2.3.4-1. The approximate width of cracks should be provided and the depth, width and length of areas of section loss should be defined. The location of exposed reinforcement bars or prestressing strands should be noted. Information giving the location, size and number of any severed reinforcement bars or prestressing strands should also be provided. Photographs of the damaged areas should be submitted with the inspection information. In addition to the above information, a framing plan showing information such as the point of impact and the location of damage to diaphragms should be included with the inspection data. An example framing plan recording impact damage on a concrete structure is illustrated in Figure 2.3.4-2.

A similar format can be used to report damage on concrete T girder structures.

For repair procedures and details see Section 2.14.

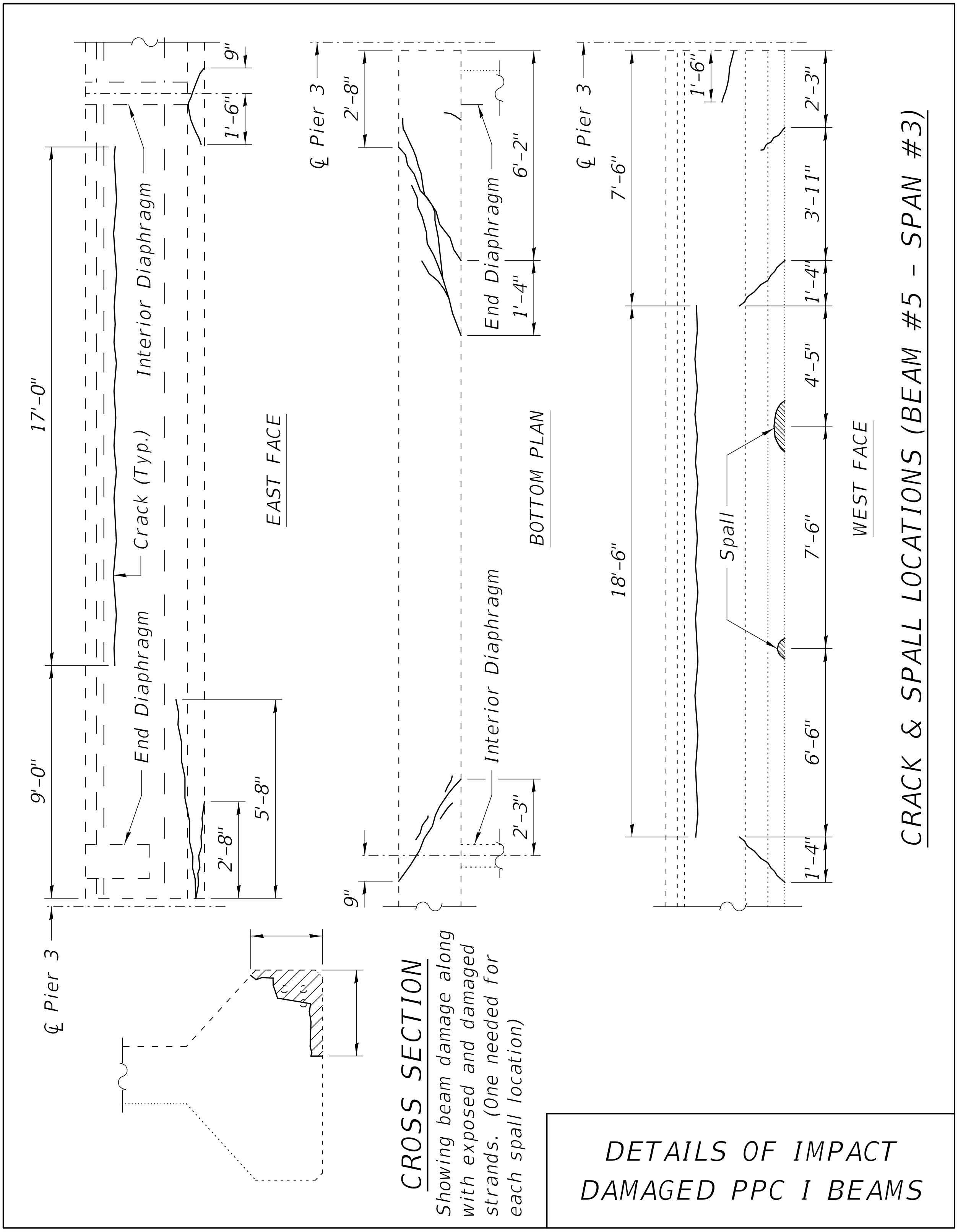


Figure 2.3.4-1: Details of Impact-Damaged PPC I-Beams

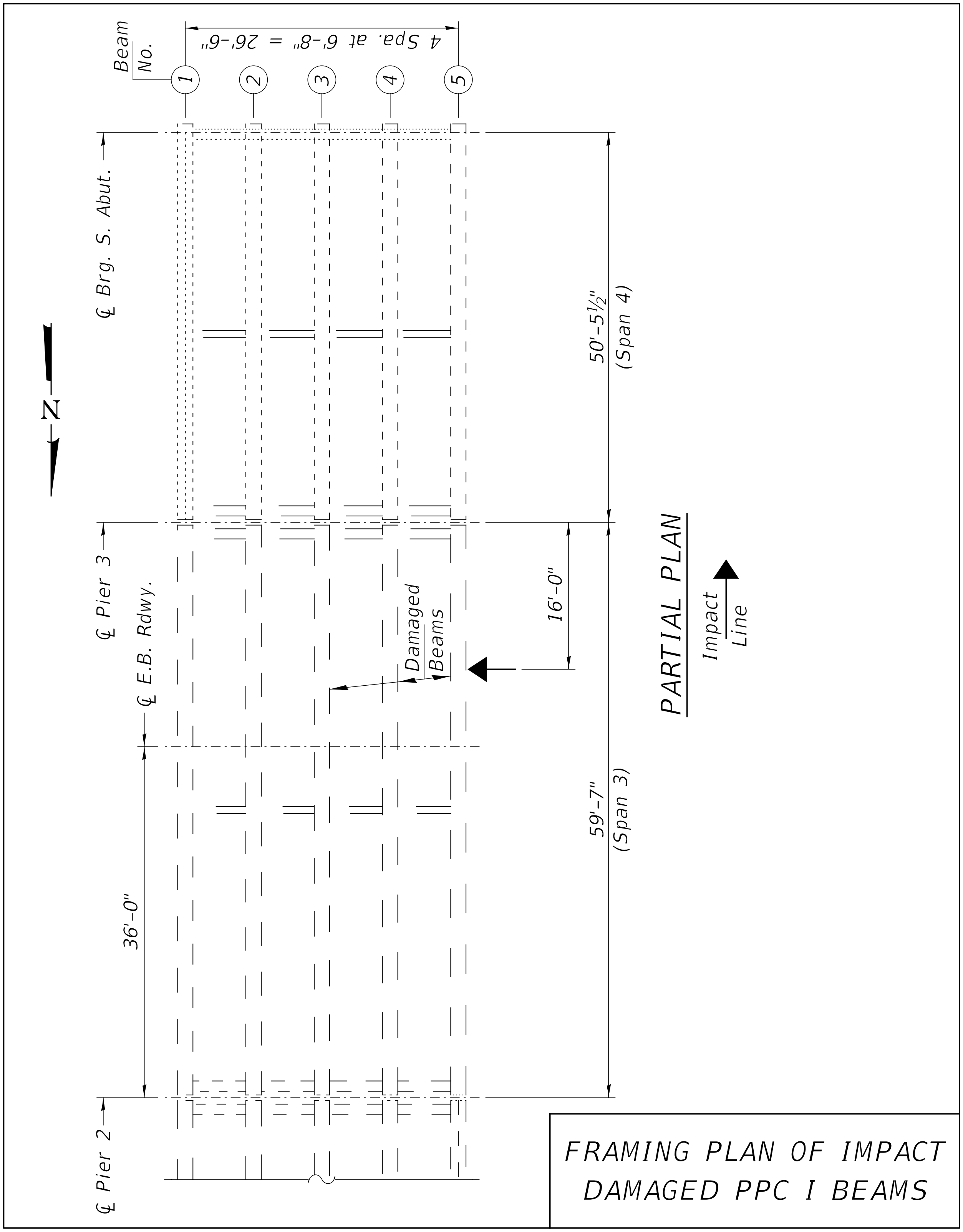


Figure 2.3.4-2: Framing Plan of Impact-Damaged PPC I-Beams

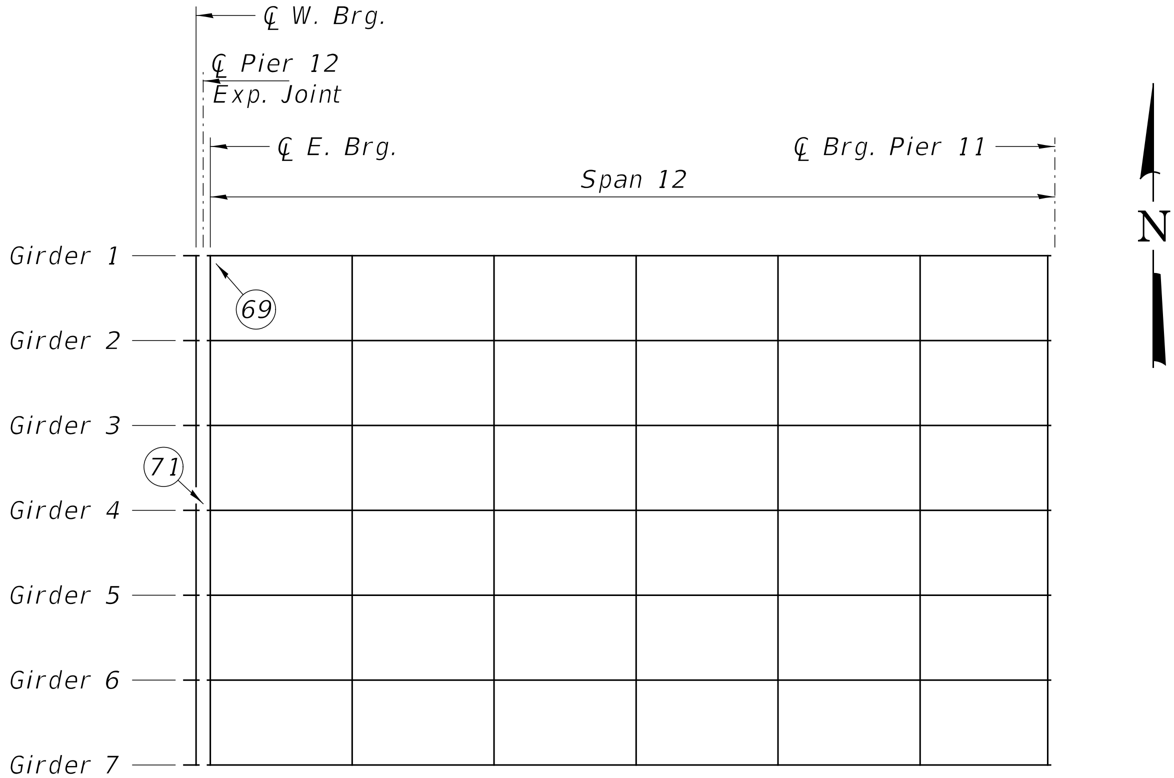
2.3.5 Inspection Photographs

In addition to the sketches described in Section 2.3.2, properly taken and labeled digital photographs are probably the single most effective way to convey damage information. New digital images can be manipulated to obtain very detailed information about deterioration or impact damage. A properly taken photo will show a clear view of the damage as well as an overall view of the damage location (showing associated members).

The proper labeling of the photograph is crucial in ensuring that the precise location of the damage is identified (see Figure 2.3.5-1). Normally the label will include:

1. The Structure Number (SN),
2. Date of photo
3. The specific member (i.e., Beam 1)
4. The overall location (i.e., “at south abutment”)
5. The specific location (i.e., “west face", “web”, stiffener” etc.)
6. The type of damage (i.e., “crack”, “section loss”, “deflection” etc.)
7. The direction in which the photograph was taken (i.e., “looking south”).

A partial framing plan identifying the picture greatly aids in the location of the damage especially when the orientation of the structure is not clear to the person taking the photograph (see Figure 2.3.5-2). Ideally the framing plan will use the same orientation and girder designation as on the existing plans.



Span	Member	General Location	Picture
12	Girder 1	@ P12	69
12	Girder 4	@ P12	71

Note:
See Fig. 2.3.5-2 for pictures 69 and 71.

PARTIAL FRAMING PLAN SHOWING PICTURE LOCATION AND DIRECTION

Figure 2.3.5-1: Partial Framing Plan Showing Picture Location and Direction

PICTURE 69 SN xxx-xxxx
Date xx/xx/xxxx
Girder 1
Span 12, Pier 12, East Bearing
Bottom of web/stiffener
Full section loss
Looking Northwest

PICTURE 71 SN xxx-xxxx
Date xx/xx/xxxx
Girder 4
Span 12, Pier 12, East Bearing
Bottom of girder web
Full section loss
Looking Southeast

SAMPLE OF PICTURES SHOWING DAMAGE

Figure 2.3.5-2: Sample of Pictures Showing Damage

2.3.6 Nondestructive Testing

When evidence of cracking in a steel member is found during an inspection, nondestructive testing should be performed to confirm the presence of the crack and to determine the extent of the cracking. The most frequently used forms of nondestructive testing employed during an inspection to identify cracking are dye penetrant testing and magnetic particle testing. Refer to Section 3.7 of the Structural Services Manual for detailed explanation of these and other non-destructive testing procedures.

The behavior of cracks is non-predictable. As such any crack found on a steel girder must be promptly reported in order to determine if immediate repair measures must be implemented. The most common temporary repair to minimize the possibility of crack propagation is the use of a crack arrestor hole as shown in Figure 2.12.1.1-1.

2.4 Bridge Deck Overlays

2.4.1 Overlay Types

Bridge deck overlays will typically consist of one of the following alternates:

1. Hot Mix Asphalt (HMA) concrete with a Full Lane Sealant Waterproofing System in accordance with Section 581 of the Standard Specifications.
2. Concrete overlays (latex, microsilica, Fly Ash, High-Reactivity Metakaolin (HRM))
3. Thin polymer
4. Reinforced concrete

The type of overlay used for a project is chosen by the District based on applicability, cost, availability and experience with the overlay types.

To aid in that selection, below are general considerations:

1. HMA overlays with a Full Lane Sealant Waterproofing System are intended for replacement of an existing HMA overlay, protection of a bare concrete deck or on structures in which hydro-scarification is not feasible (ie. precast structures). Note that in projects with an estimated lifespan of 5 years or less a HMA overlay without waterproofing may be considered.
2. Microsilica, Fly Ash, HRM and Latex overlays are intended to last 25 years and require the use of hydro-scarification of the existing deck surface for proper bonding.
3. Thin polymer overlays are recommended for decks with small areas and low patching quantities or when necessitated by the need to minimize additional dead load or need to avoid height adjustments at expansion devices.
4. Reinforced concrete overlays will significantly increase the dead load demand on the structure and may encroach on the minimum height required at the parapets.

Because expansion joints are not adjustable for changes in profile grade, a new expansion joint should, if at all possible, be paired with the installation of a new overlay, except on thin polymer overlays. See expansion joint replacement details (Section 2.5) and longitudinal joint closure details (Section 2.6) for treatment of overlay at these locations.

2.4.2 Bridge Condition Reports

The Bridge Condition Report requirements presented in the *IDOT Bridge Condition Report Procedures and Practices* apply to all Federal or State Funded projects. Bridge deck overlay projects using only Maintenance Funds do not require the preparation and submittal of a formal BCR. However, District personnel should use the procedures presented in the report for the survey and evaluation of an existing bridge deck when planning an overlay project using Maintenance Funds.

2.4.3 Deck Patching

Prior to the placement of an overlay, a deck survey is necessary to identify areas of the existing deck which require repair and the anticipated total quantity of repair, before the overlay can be placed. The date of the most recent delamination survey should be shown in the project plans on the sheet showing the deck patching locations or delamination survey plot. When more than one winter is expected to occur between the last delamination survey and the projected date of the commencement of deck patching operations, another delamination survey should be performed prior to the submittal of repair plans for review. It is preferred that the anticipated partial and full depth deck slab repair areas be shown on a plan view of the deck and included in the plans showing location and approximate size of the patches. When the deck slab repair areas cannot be predicted (because of the presence of an HMA overlay for example), then the deck survey data and how it relates to potential partial and full depth deck slab repairs should be shown in the plans. In all cases deck plan views should be shown of adequate size to allow the Engineer to record the location of the actual deck slab repairs on the as-built plans. The plans should include a note instructing the Engineer to record the deck repair areas in order to document as-built conditions for future reference. The note should also indicate that the quantities provided are estimated.

Except for locating the anticipated patching areas in a plan view, specific details for partial and full depth patching are not required to be shown in the plans. Instructions for deck patching are included in the special provision for “Deck Slab Repair”.

For decks originally constructed using precast prestressed deck planks as stay-in-place forms, special details are required for full depth patching. Because the repair of these planks is not feasible, removal of the affected planks is required. Details for removal and replacement of individual planks are shown in Figure 2.4.3-1 thru Figure 2.4.3-4. Details must be adjusted when multiple adjacent planks need replacement. Partial depth patching over precast prestressed deck planks is discouraged.

On structures over roadways or navigable waterways the installation of a new overlay that requires the use of hydro scarification will also require, in most cases, the installation of a protective shield system to prevent debris from falling on the traffic below. Because any partial depth patch has the potential to become full depth, the protective shield installation is a prudent measure to take.

When the existing superstructure consists of a reinforced concrete slab, reinforced concrete T-girder or other type of superstructure where the deck functions as a component of the main load carrying members of the bridge spans, the structural effects of partial and full depth deck patching must be considered. When partial or full depth patching is necessary on these types of structures, the size, locations and extent of the deck patching must be evaluated to determine if temporary shoring must be placed under portions of the superstructure until the deck patching concrete has obtained the required load carrying capacity.

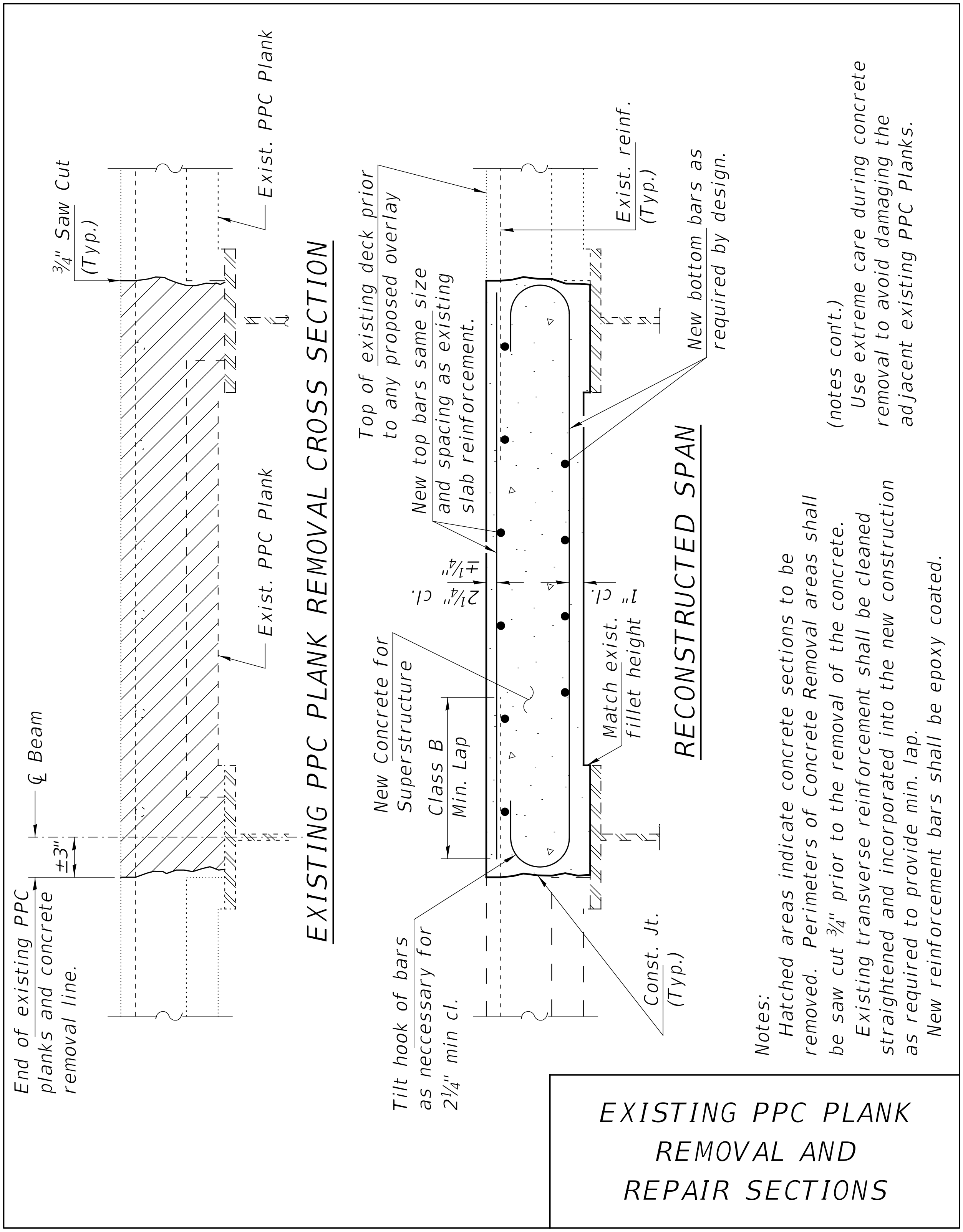


Figure 2.4.3-1: Existing PPC Plank Removal and Repair Sections

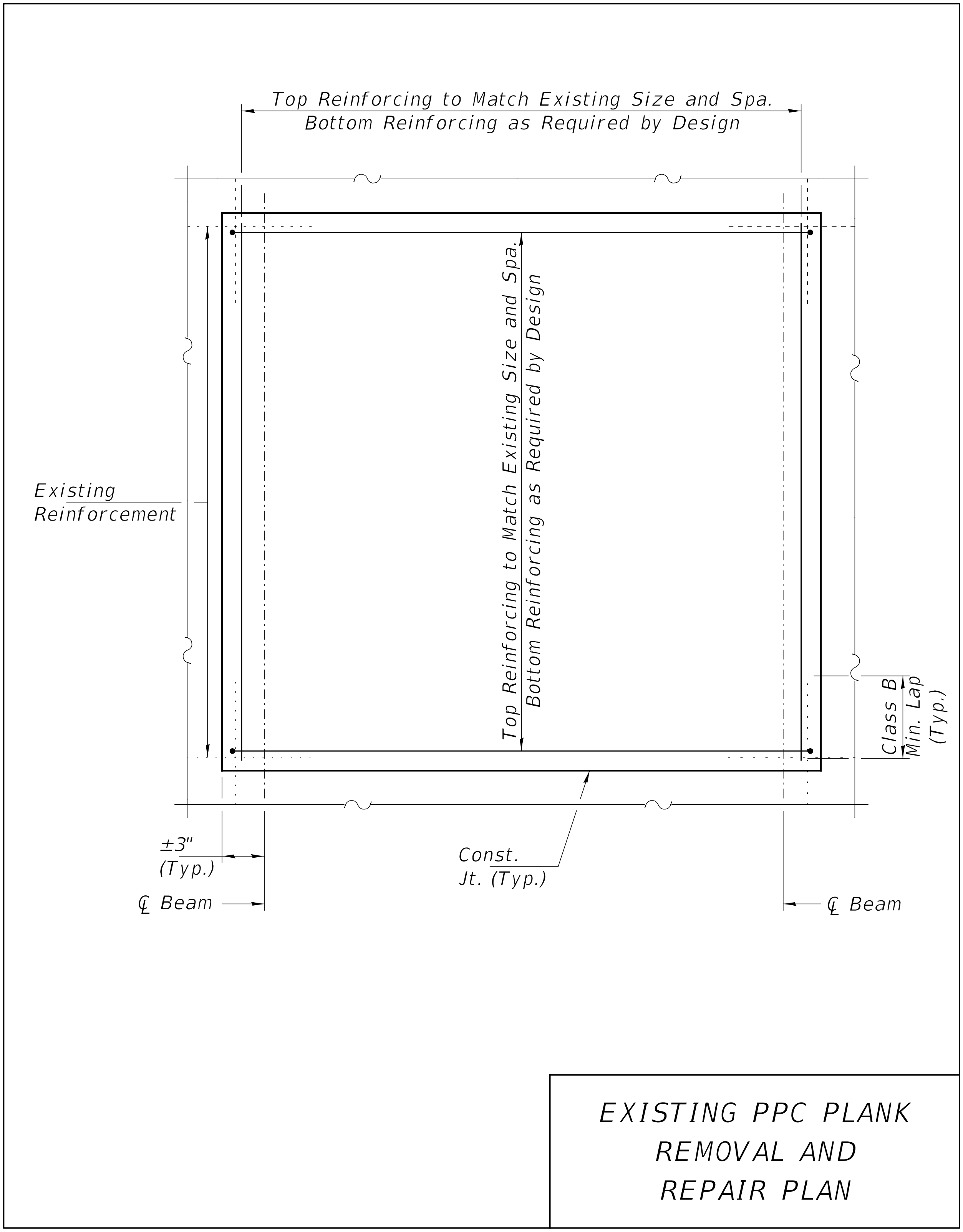


Figure 2.4.3-2: Existing PPC Plank Removal and Repair Plan

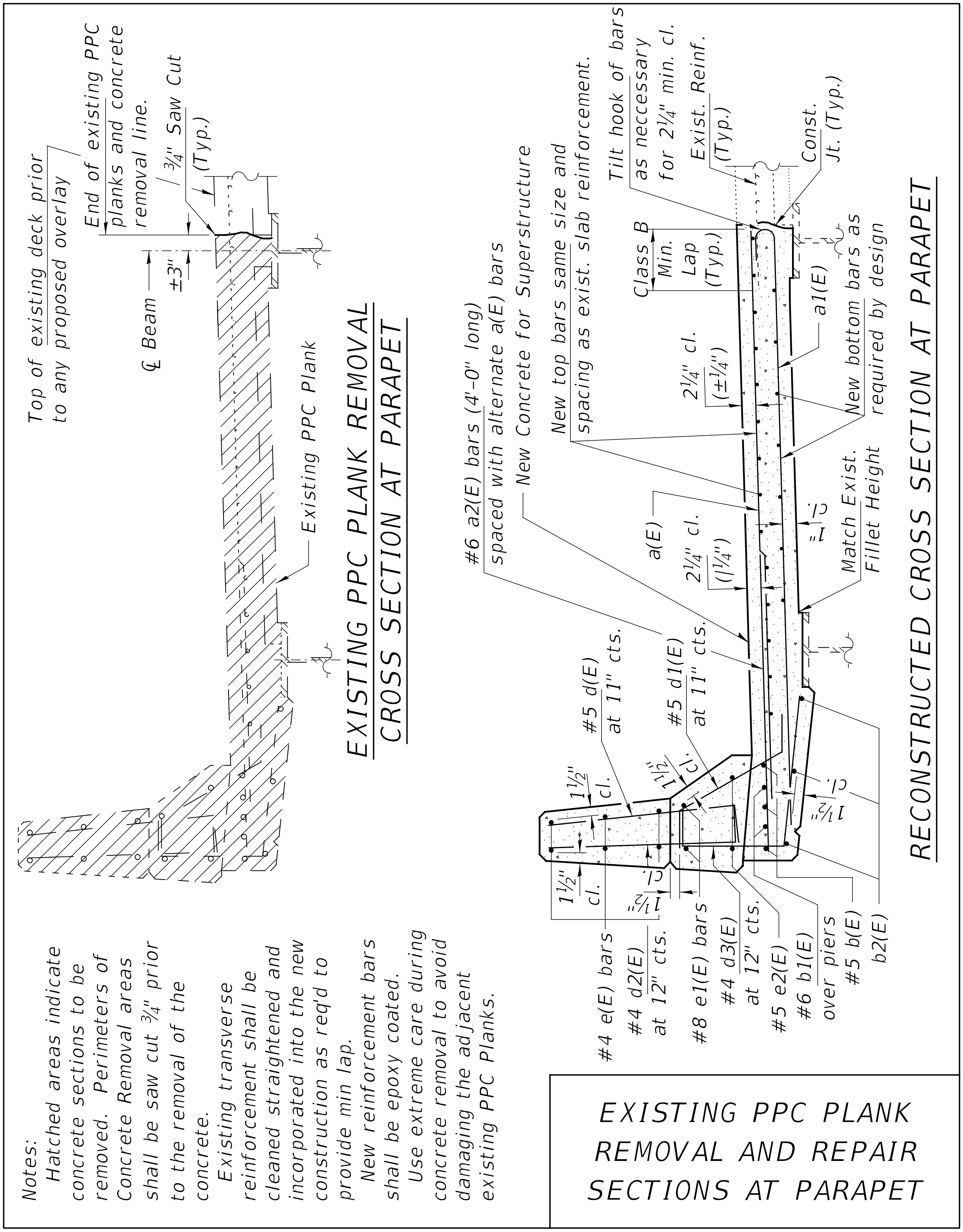


Figure 2.4.3-3: Existing PPC Plank Removal and Repair Sections at Parapet

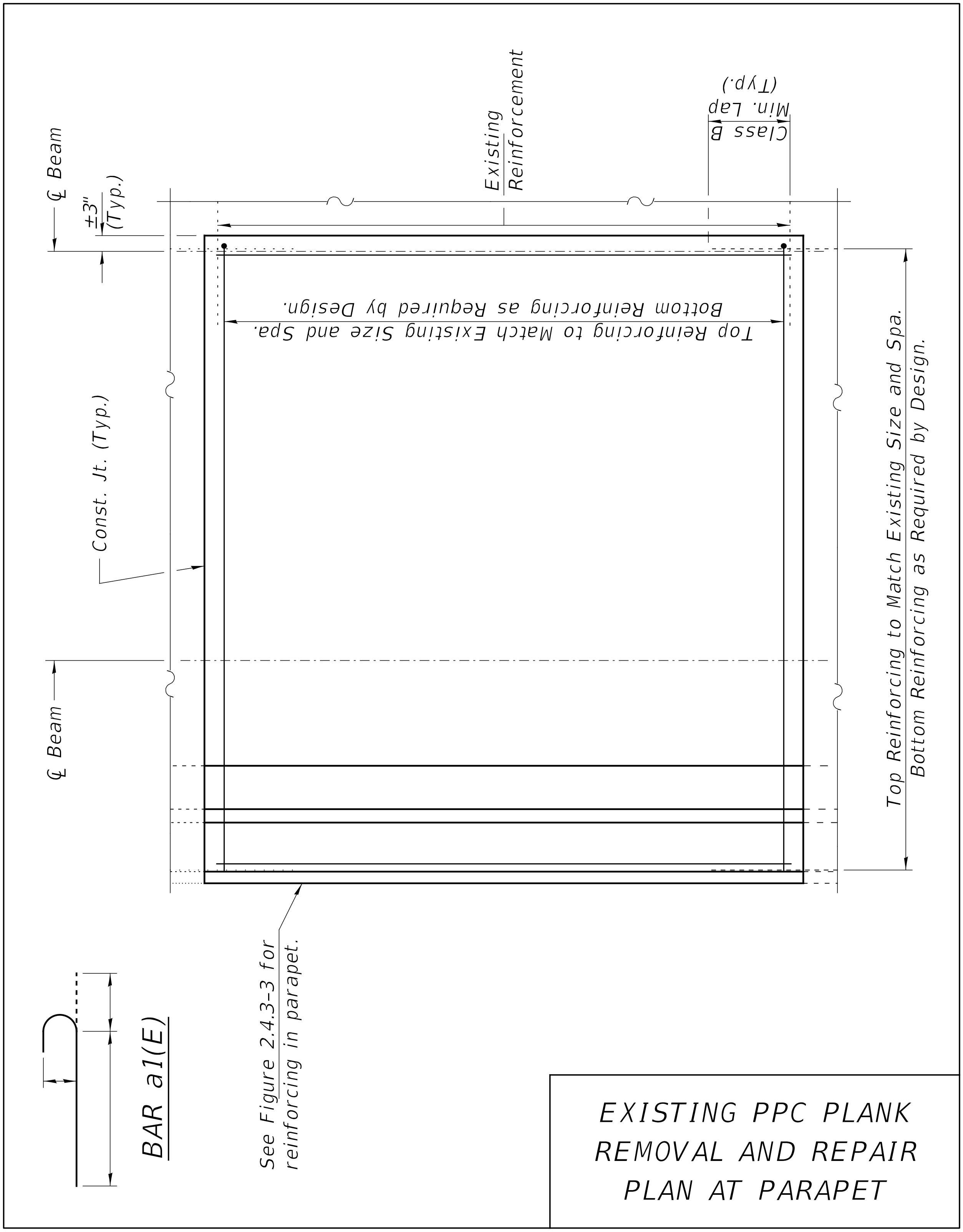


Figure 2.4.3-4: Existing PPC Plank Removal and Repair Plan at Parapet

2.4.4 Effect on Structure

The Bureau of Bridges & Structures should be contacted to evaluate the effect of overlay projects on the overall load carrying capacity of the structure. The load carrying capacity of a structure will be evaluated during the review of the BCR. When a BCR is not required for a project, the Ratings Unit of the Structural Services Section should be contacted to evaluate the effect of the overlay on the structure. This contact for evaluation is also necessary for bridges included with roadway resurfacing projects where the District plans to scarify the existing wearing surface on a bridge deck and/or place a new wearing surface on the bridge.

In most cases the required removal of the concrete on the top of the deck is limited to ¾” which will not expose the existing reinforcement. When a bridge deck overlay project requires the removal of the existing deck concrete over a large area to the level of the existing top reinforcement bars of a reinforced concrete slab bridge, special precautions are necessary to maintain the structural load carrying capacity of the structure. The Bureau of Bridges & Structures should be contacted by District personnel for assistance in determining what measures should be taken to ensure the structural integrity of reinforced concrete slab bridges when the method of deck removal specifies that the top reinforcement bars are to be exposed or significant deck patching is required. The preferred method used to address this is:

1. The existing reinforced concrete slab shall be scarified and overlaid in segments following a predetermined sequence. Details associated with the sequential concrete surface removal and overlaying of a reinforced concrete slab bridge are shown in Figure 2.4.4-1.

Alternatively:

1. The superstructure can be supported from below during the scarification and overlay operations.
2. The full and partial depth slab repairs can be completed and cured prior to scarification operations.

When full-depth patches are needed along the parapet for an extended length, a sequence of construction should also be used to prevent the overturning of the parapet.

The following notes should be placed near the details showing the pouring sequence:

1. At least 72 hours shall have elapsed from the end of the previous pour, and
2. The concrete shall have attained a minimum modulus of rupture of 650 psi or a minimum compressive strength of 3500 psi.

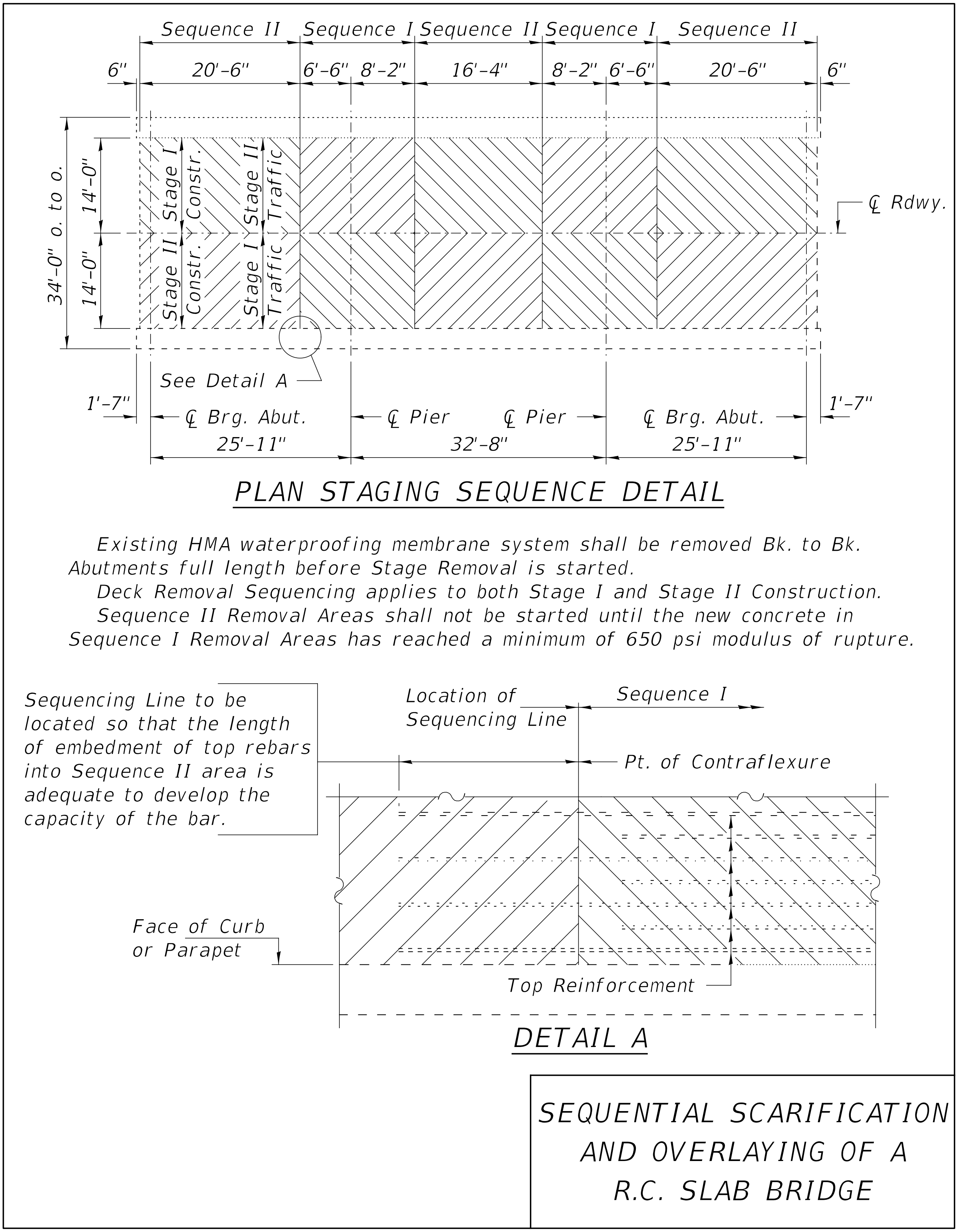
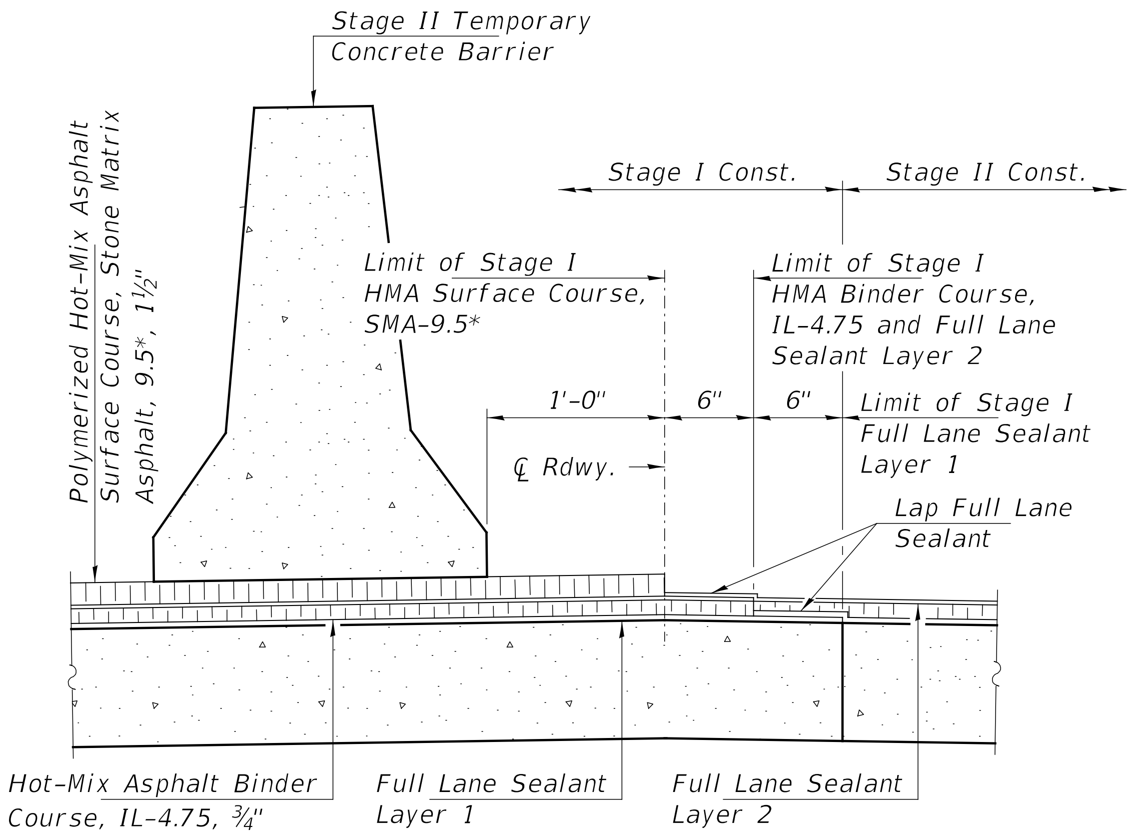


Figure 2.4.4-1: Sequential Scarification and Overlaying of a R.C. Slab Bridge

2.4.5 HMA Concrete Overlays

A Full Lane Sealant Waterproofing System is always required when using a HMA concrete overlay. The minimum thickness of the HMA surface course to be placed, not including the 3/4-inch binder course, should be 1 1/2 inches for an overall thickness of 2 1/4 inches. An existing HMA overlay, and waterproofing membrane system shall be removed and replaced in its entirety as it is unlikely that the HMA overlay can be removed without damaging the waterproofing membrane system. For treatment of the Full Lane Sealant Waterproofing System at the stage construction line refer to Figure 2.4.5-1.

An existing bare concrete deck should not be scarified prior to the application of the Full Lane Sealant Waterproofing System, but the deck should be cleaned using methods which will not roughen the surface. Waterproofing system should always be extended across construction joints in the deck at locations where the deck has been patched or replaced. When removing existing HMA concrete surfaces and existing waterproofing systems, the existing plans and other records should be consulted to determine if any hazardous materials, such as asbestos, were used. If hazardous materials were used in the existing overlay, containment of the materials during removal and proper disposal will be necessary.



WATERPROOFING STAGING

Full Lane Sealant Waterproofing System consists of Layer 1 of Full Lane Sealant, a layer of Hot-Mix Asphalt Binder Course, IL-4.75, and Layer 2 of Full Lane Sealant.

* Mix and N-number to be determined in design.

FULL LANE
WATERPROOFING SYSTEM
AT STAGE CONSTRUCTION

Figure 2.4.5-1: Full-Lane Waterproofing System at Stage Construction

2.4.6 "Hard" Concrete Overlays

Microsilica, Fly Ash, HRM and latex concrete should be placed with a minimum thickness of 2 1/4 inches on a hydro-scarified concrete surface. A maximum overlay thickness of 3 1/2 inches may be placed without the inclusion of reinforcement bars or wire mesh. When using a latex overlay consider the following:

Thicknesses greater than 3 1/2 inches will likely make the cost of this overlay prohibitive. Also, maximum slopes for placement should be limited to 3% because the material is more flowable and tends to back up behind the vibrating screed when placed uphill resulting in a wavy surface. Ways to prevent this is to require that placement should be done downhill, placement should be done in stages, or requiring that diamond grinding be done afterwards to improve smoothness.

Existing concrete decks, to which microsilica, Fly Ash, HRM or latex concrete is to be applied, should be scarified a minimum of 3/4 inch (using milling equipment for the top 1/4 inch and hydro-scarification for the bottom 1/2 inch) to remove all waterproofing membrane and to provide a very rough surface for the new overlay to adhere to. Detailed explanation of the scarification procedure is included in the special provisions for each overlay type.

All new microsilica, Fly Ash, HRM and latex overlays must receive Protective Coat similar to new concrete decks. Also, overlays must have their surface saw cut grooved. Typical grooving is done transversely to the CL of Roadway.

When applicable or at the request of the District, overlays shall receive diamond grinding per IDOT Bridge Manual Section 2.3.7.8. The thickness of the overlay to be placed must be increased by 1/4 inch to account for the amount to be removed by the grinding machine. Grooving in these cases must be done longitudinally. Additionally, steel rails for strip seals and steel elements of other expansion joint types should be placed 1/2 inch below top of overlay prior to grinding.

2.4.7 Reinforced Concrete Overlays

Reinforced concrete overlays may be a minimum of 4 inches in thickness. However, a minimum concrete overlay thickness of 5 inches should be used when the additional dead load will not adversely affect the load carrying capacity of the structure. Also, a minimum overlay thickness of 5 inches will facilitate the embedment of rail post anchors in the overlay if necessary. Reinforcement bars or wire mesh providing a minimum cross sectional area equivalent to #4 bars at 12-inch centers, in both the longitudinal and transverse directions, should be incorporated in the overlay to prevent and control cracking.

Except when placed on precast prestressed concrete deck beam or precast concrete bridge slab superstructures, the concrete surface on which the reinforced concrete overlay will be placed should be scarified a minimum of 3/4 inch. All existing wearing surface, waterproofing and foreign material must be removed from the surface receiving the overlay.

2.4.8 Overlaying Precast Superstructures

The top surface of precast prestressed concrete deck beams and precast bridge slab superstructures should be prepared by removing only the wearing surface and removing any waterproofing or foreign material using hand methods and shotblasting (use of milling equipment or heat is not allowed). The top surface of the original precast member should not be scarified.

The thickness of an overlay on a precast prestressed concrete deck beam superstructure will vary parallel to the centerline of the bridge in order to match the profile grade of the roadway and to account for the camber in the beams. A profile of the overlay should be shown in the plans to clarify the relationship of the overlay thickness to the profile grade and beam camber as illustrated in Figure 2.4.8-1.

Figure 2.4.8-2 shows the use of a preformed joint strip seal at an expansion joint reconstruction with concrete overlay.

In order to control cracks in the area of the overlay located over a fixed pier, a relief joint should be installed in the overlay over the pier as shown in Figure 2.4.8-3. This detail is used with an overlay on existing PPC Deck Beams.

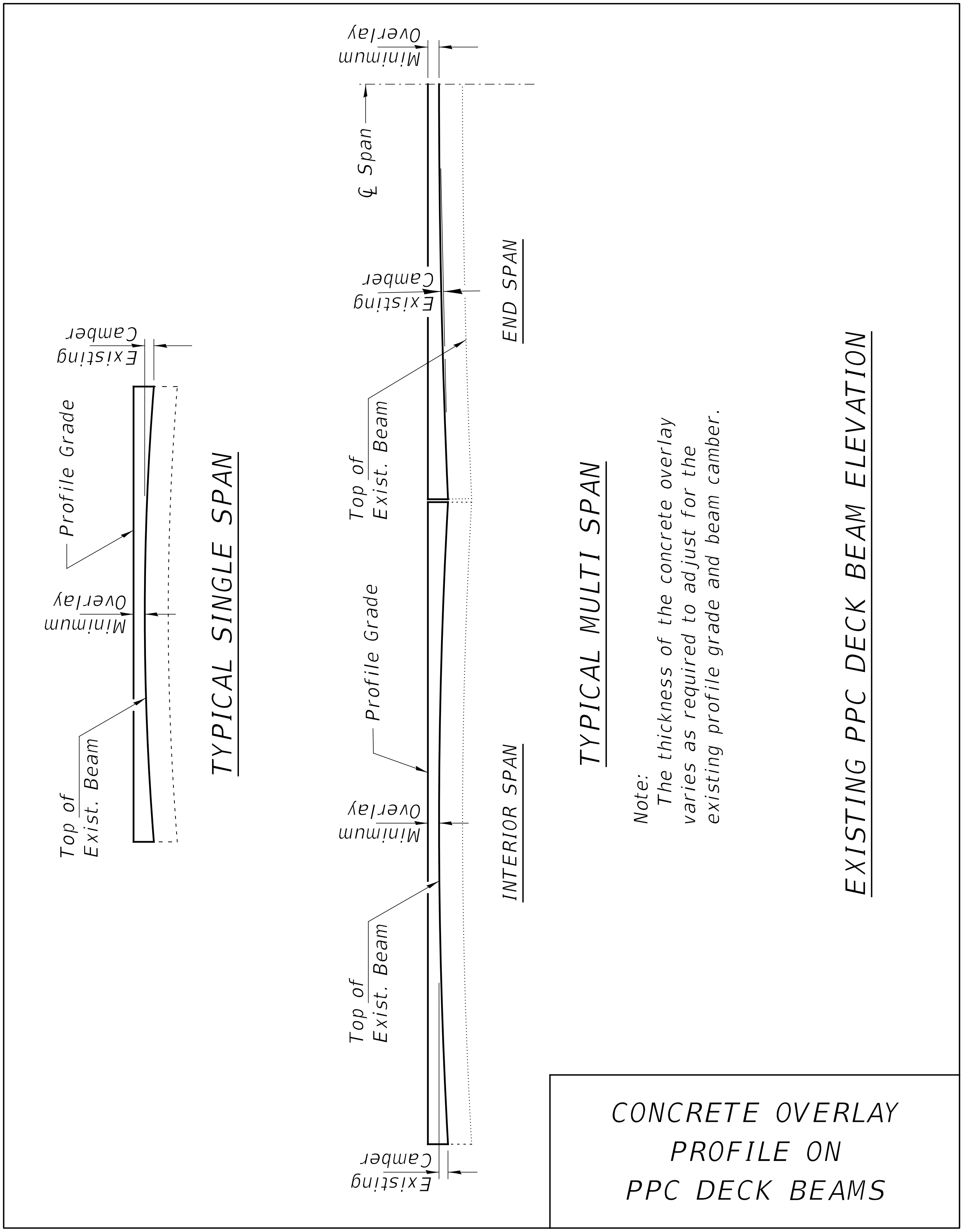


Figure 2.4.8-1: Concrete Overlay Profile on PPC Deck Beams

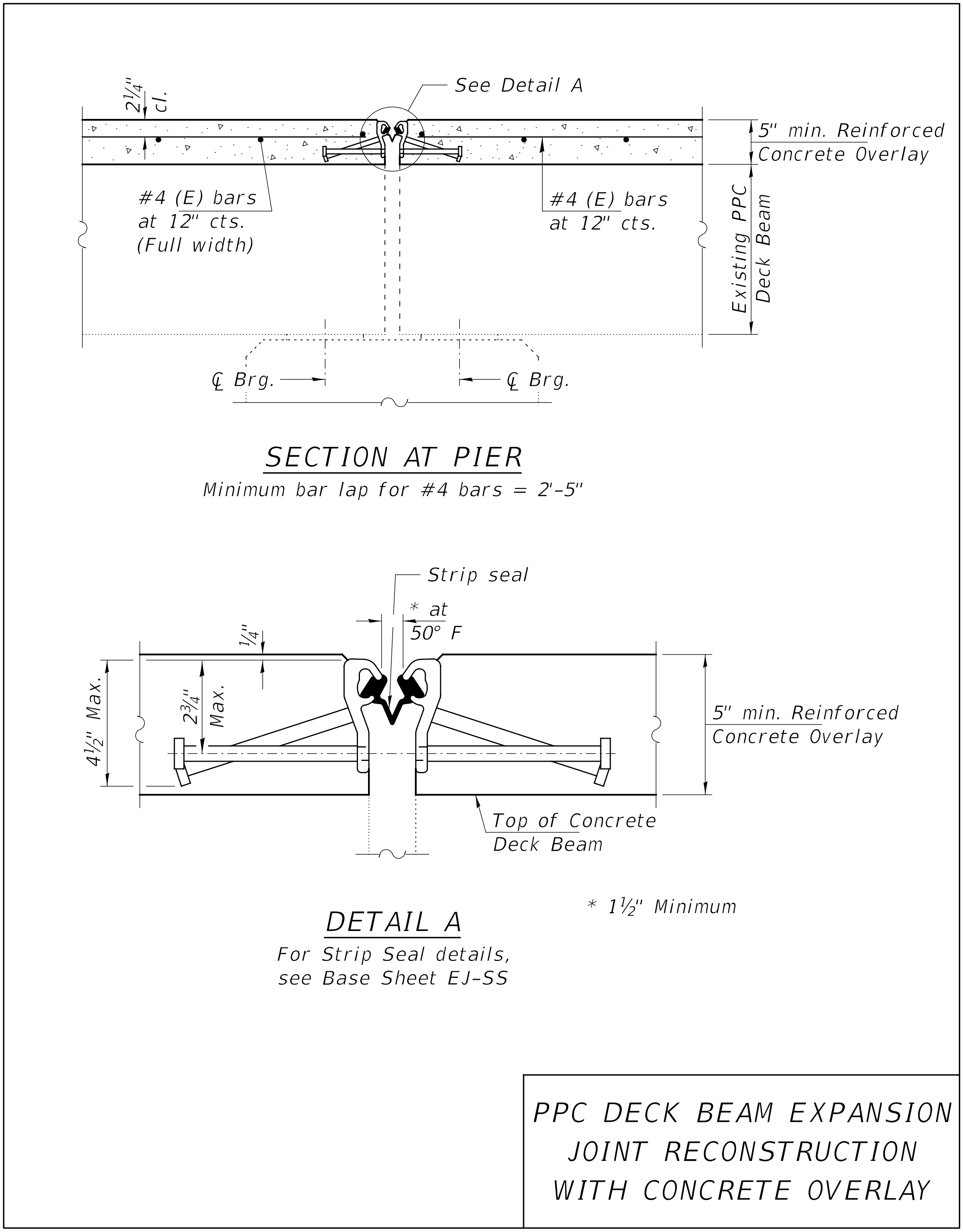


Figure 2.4.8-2: PPC Deck Beam Expansion Joint Reconstruction with Concrete Overlay

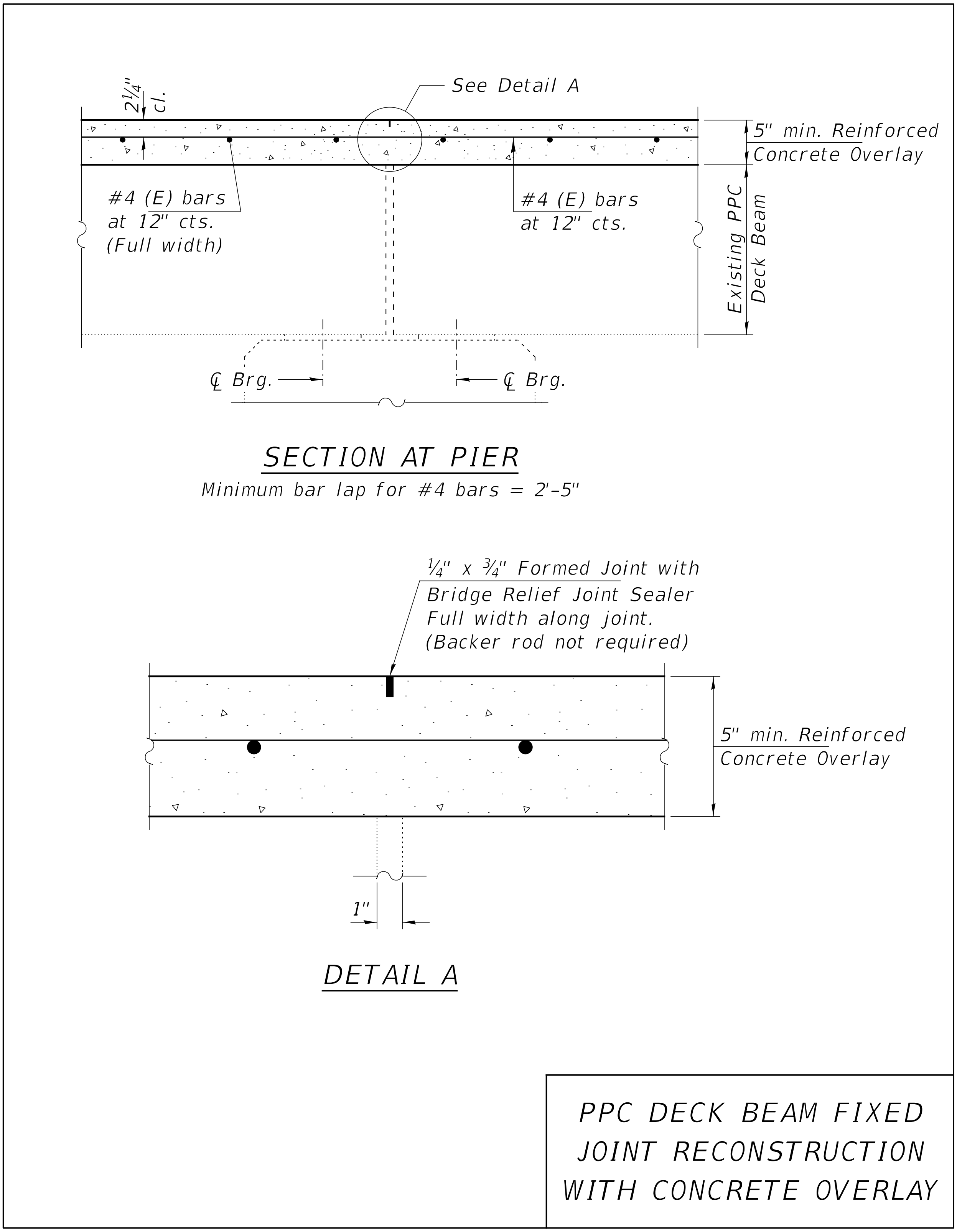


Figure 2.4.8-3: PPC Deck Beam Fixed Joint Reconstruction with Concrete Overlay

2.5 Expansion Joint Replacement

2.5.1 General

The replacement of expansion joints is a common maintenance operation necessary to prolong the life of a structure. This operation is usually required due to: the structural failure of the joint; the joint no longer provides protection for the structural elements below against exposure to drainage from the deck; the need to upgrade a joint when a new overlay is installed; or the joint opening has closed and is restricting the normal expansion of the structure. Before preparing plans to replace closed expansion joints it is necessary to determine the cause of the closure to avoid a recurrence after the new expansion joint is installed. If the cause of the joint closure is not apparent or easily correctable, the Repairs Unit of the Structural Services Section of the Bureau of Bridges & Structures should be contacted for assistance.

In some situations, it may be feasible to eliminate expansion joints in a bridge. When this option is feasible, it may be the most cost-effective alternative when considering the elimination of future maintenance of the joints themselves and the reduced maintenance of bridge components near the current expansion joints. Criteria for eliminating expansion joints and additional information including details for this work can be found in Section 2.5.8.

2.5.2 Joint Type Selection

When replacing an expansion joint, the selection chart provided in the Planning Section of the IDOT Bridge Manual (Figure 2.3.6.1.6-1) should be used to select the type of the replacement joint. Depending on the expansion length and deck skew, the options currently available are strip seal, finger plate and modular expansion joints. Joint details can be found in Section 3.6 of the IDOT Bridge Manual. Joint types not shown in the IDOT Bridge Manual or which have not been tested and approved for general use by IDOT should not be used unless the Bureau of Bridges & Structures and the Bureau of Materials and Physical Research have concurred in the use of the joint on an experimental basis. Some joint types which are not in the IDOT Bridge Manual, but can be considered are included in the Special Provision for Preformed Bridge Joint Seal. These joints are only intended to be used on bridge structures when the installation of the strip seal joint is not feasible or for short time use when the intent is only to replace the expanding material leaving the adjacent supporting elements in place. The specific type of joint desired by the District should be called for in the plans and the existing joint opening should be shown.

Consideration should be given to make joints that extend across a sidewalk ADA compliant. This will require the installation of additional sliding plates placed above the joint at the sidewalk location. Details are included with the strip seal base sheets.

2.5.3 Concrete Deck on Multi-Beam System

The most common type of expansion joint replacement involves the removal and replacement of a portion of a reinforced concrete deck slab which rests on a multi-beam support system. A minimum 2'-0" of deck slab and parapet removal and replacement is required (4 ft. removal is required for PPC I beam structure). The deck slab removal should be sufficient to include the entire thickened (corbel) area of the deck slab in the area of the joint. This will require more than the minimum 2'-0" of deck slab removal and replacement in many situations. This is the preferred method of joint replacement on structures that will also receive a new concrete overlay which are expected to last 25 years.

The longitudinal reinforcement bars in the area of slab, curb, sidewalk and parapet removal should remain in place and should be cleaned and incorporated into the new construction. The existing reinforcement bars placed parallel to the end of the deck should be removed and new reinforcement bars should be provided. Also, the existing vertical reinforcement bars in the curb and parapet areas should be removed and replaced.

The details, which should be incorporated in expansion joint replacements for deck slabs on multi-beam systems, are illustrated in Figure 2.5.3-1 for steel beams and Figures 2.5.3-2 and 2.5.3-3 for PPC I beams.

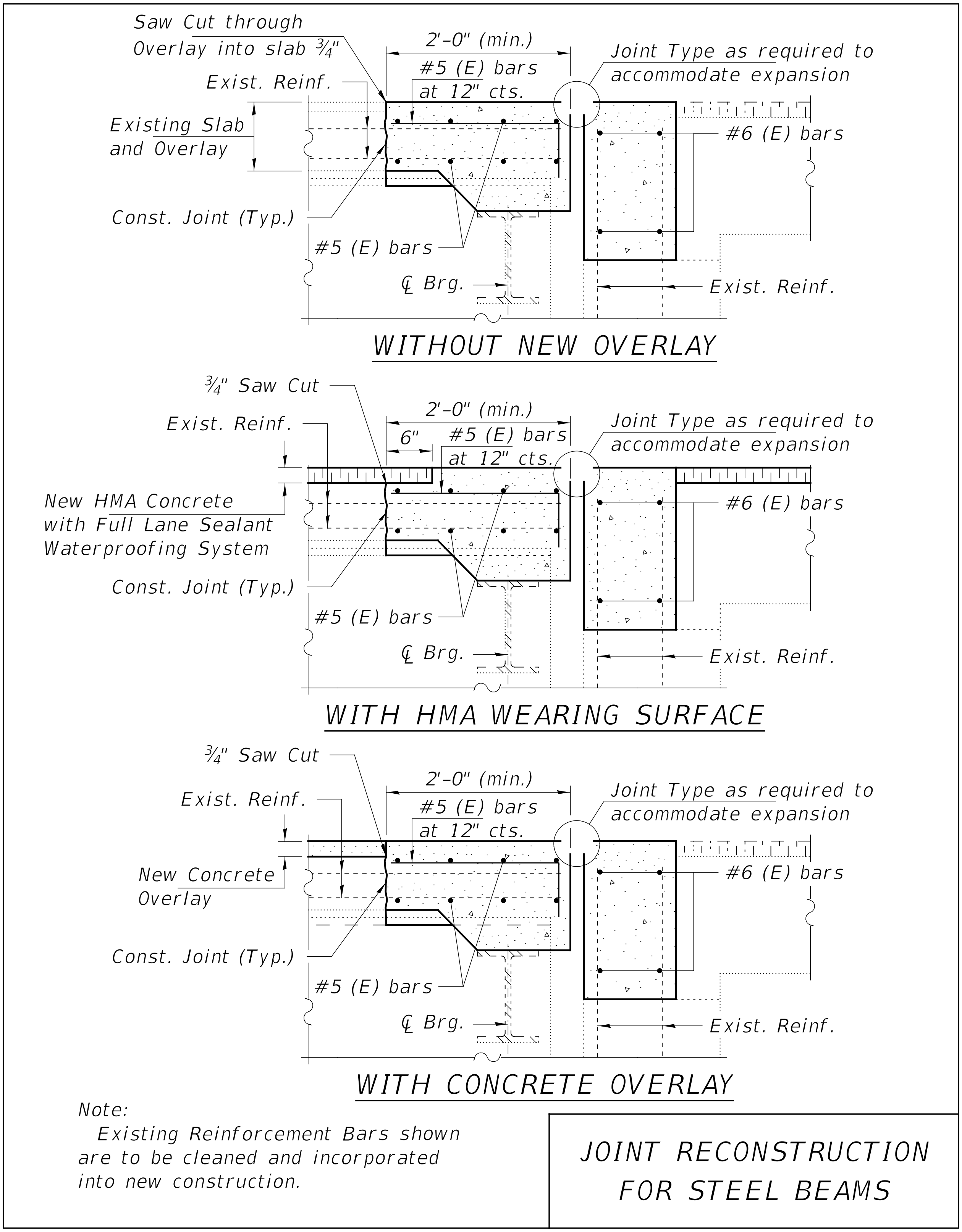


Figure 2.5.3-1: Joint Reconstruction for Steel Beams

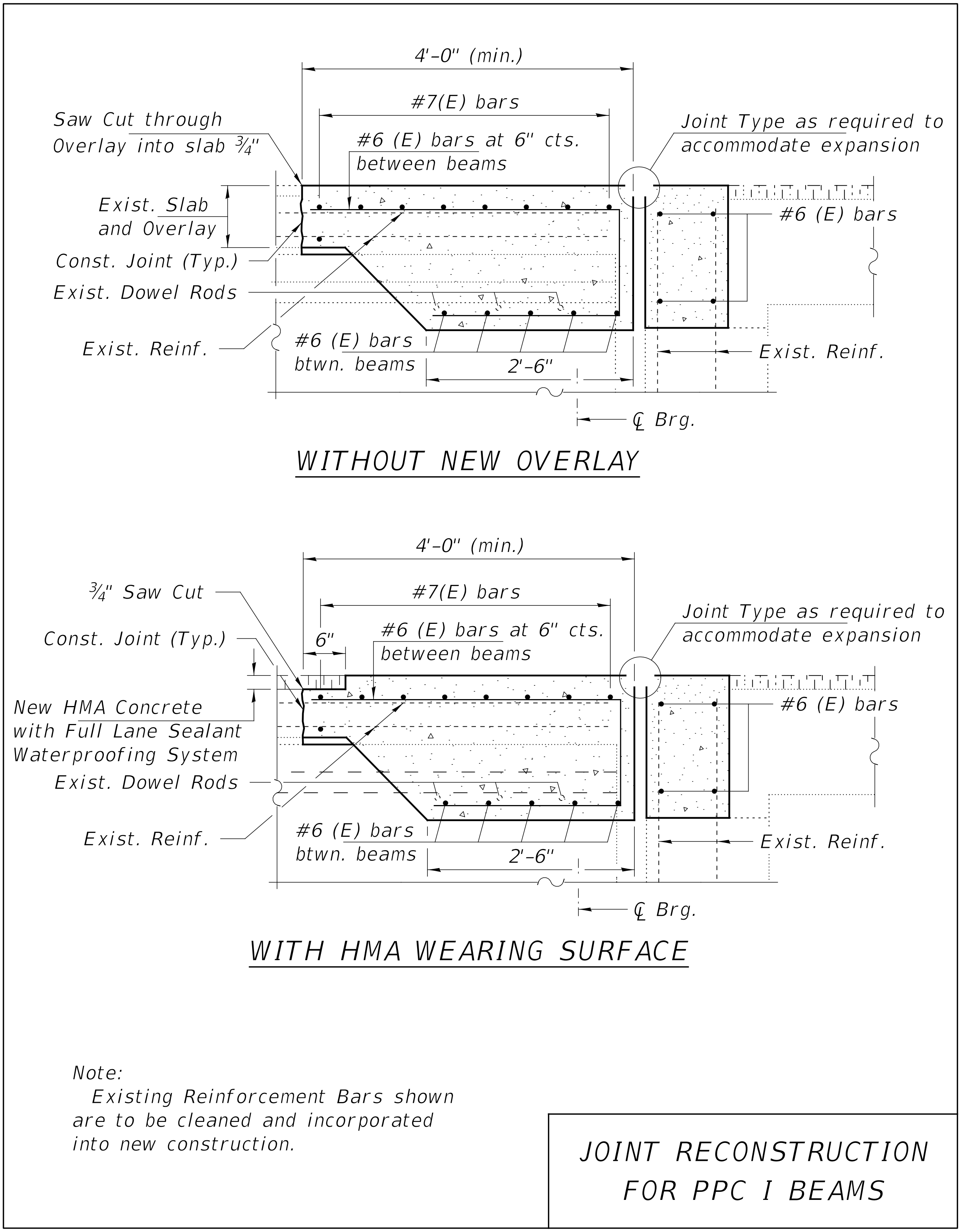


Figure 2.5.3-2: Joint Reconstruction for PPC I-Beams

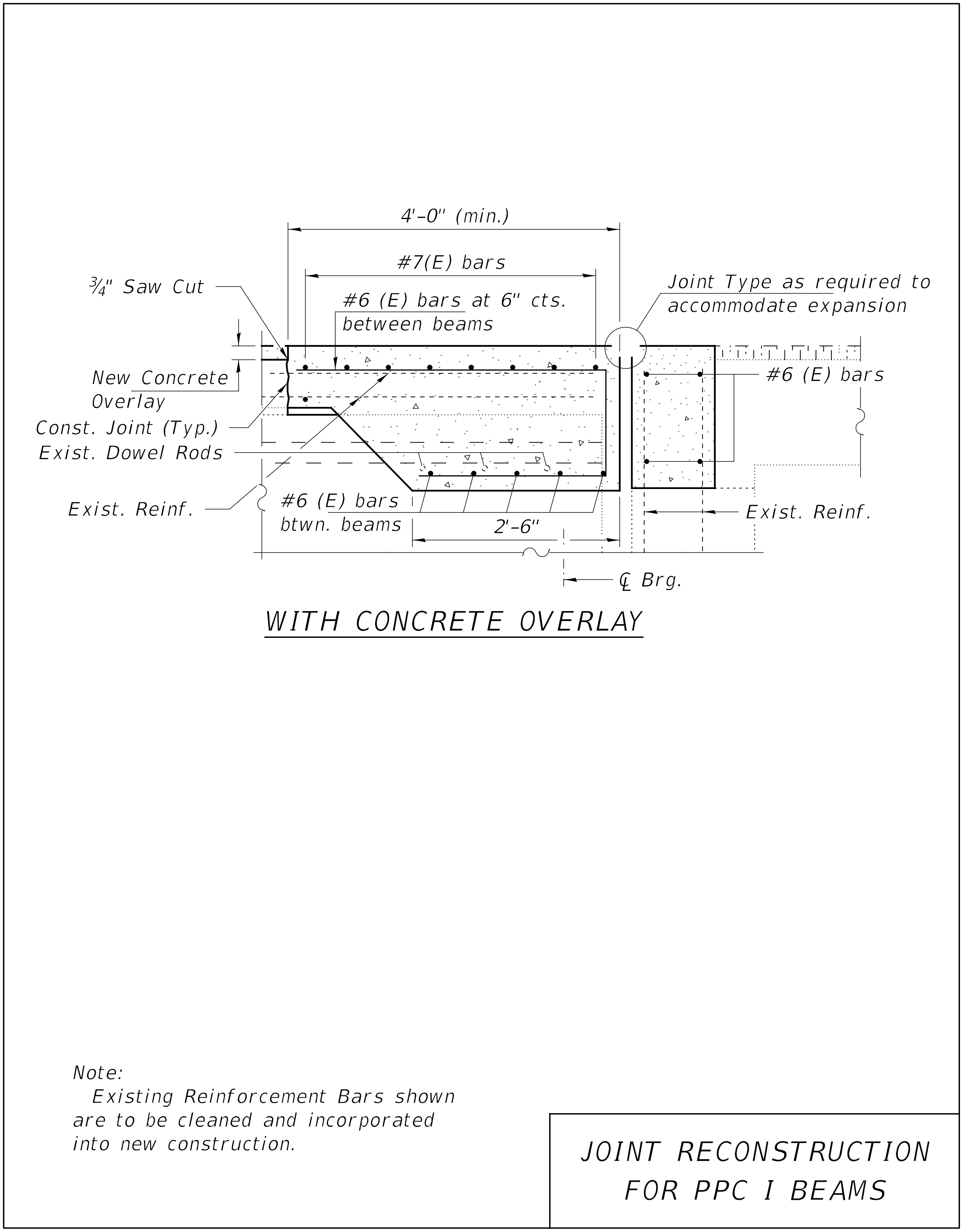


Figure 2.5.3-3: Joint Reconstruction for PPC I-Beams

2.5.4 Concrete Removal Limits and Reinforcement Bar Lapping

Regardless of the type of structure, expansion joint replacements should extend across the entire (out-to-out) width of the structure, and with the exception of finger plates and modular joints, it should also follow the skew of the structure out-to-out. The replacement joint should effectively seal the existing curb, sidewalk and parapet in a manner similar to new construction. Construction joints should be provided between the existing concrete slab or beam and the replacement concrete.

An approved mechanical splicing device or anchorage system should be used to install new longitudinal reinforcement bars in deck slabs and slab bridges when the existing reinforcement bars do not extend a sufficient distance into the new construction or cannot be salvaged. Provisions should also be made for lapping or splicing reinforcement bars which extend across a stage construction joint.

Refer to Figure 2.5.4-1 for a general illustration of concrete removal limits and reinforcement bar treatment requirements. A similar detail should be included in all repair plans for joint replacements.

The vast majority of joint replacements (as well as placement of new overlays) are done using stage construction. This requires the location of a stage construction line and a stage removal line typically located 12” away. Preferably the stage construction line should be located at the Centerline of Roadway, unless that location falls directly on top of an existing beam/girder line. In that case the location should be shifted as necessary to avoid the flange of the existing beam/girder.

Note that in structures with skews larger than 30˚, sliding plates must be installed at the parapets so provisions must be made to ensure the placement of all required plates. This may require the removal and replacement of a longer section of the parapet.

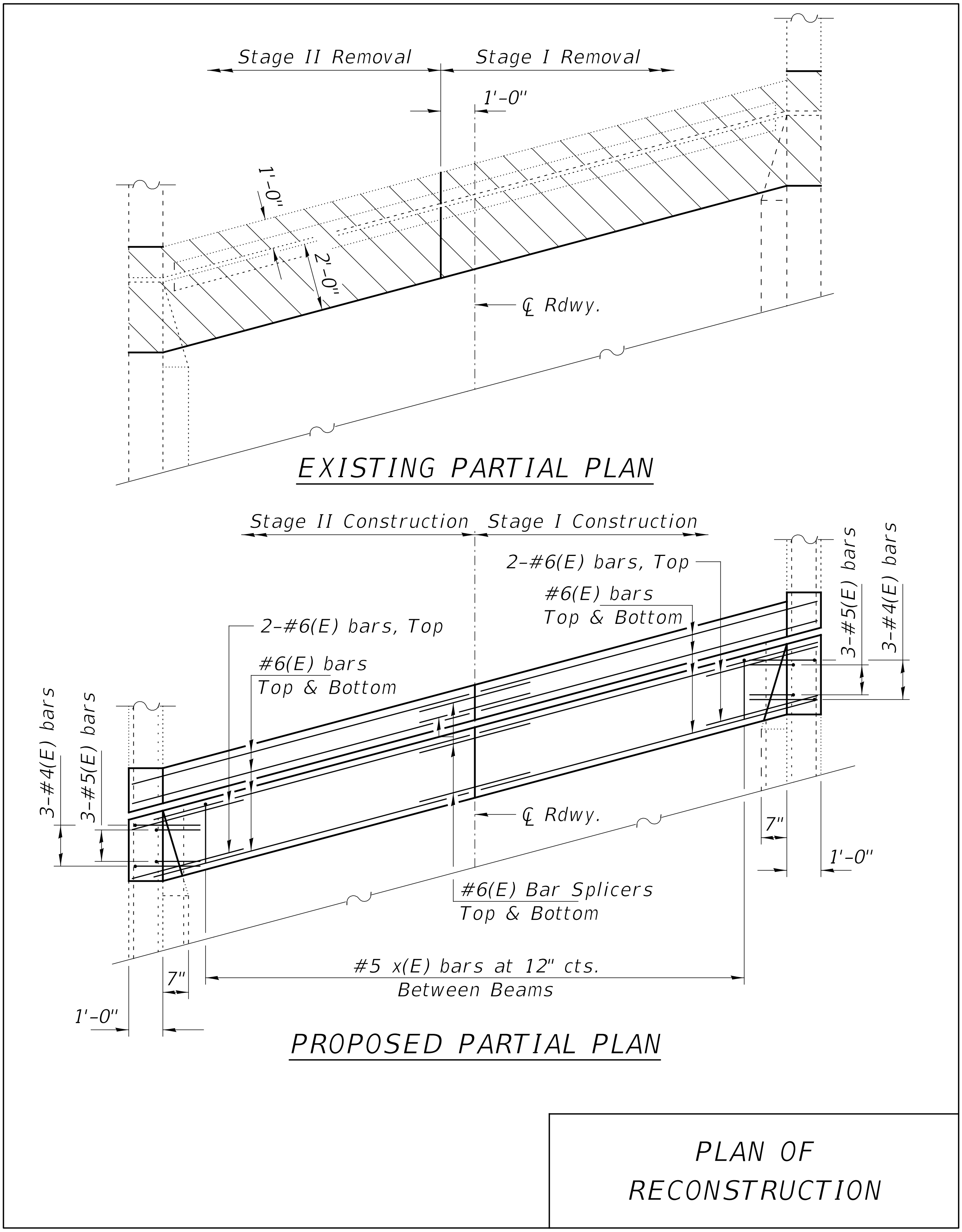


Figure 2.5.4-1: Plan of Reconstruction

2.5.5 Reinforced Concrete Deck Girder

Another type of superstructure often encountered during an expansion joint replacement project is the cast-in-place reinforced concrete deck girder (T-beam). The same slab removal and replacement requirements specified for multi-beam bridges should be used except that the depth of the deck slab removal in the area directly over the girders should be limited to the minimum depth of the deck slab rather than the thickened (corbel) end area of the slab. This difference in concrete removal limits is illustrated in Figure 2.5.5-1.

The existing reinforcement bars should be incorporated into the new construction or replaced in a manner similar to that specified for deck slabs on multi-beam systems. Figure 2.5.5-2 and Figure 2.5.5-3 illustrate details for expansion joint replacement associated with reinforced concrete deck girders.

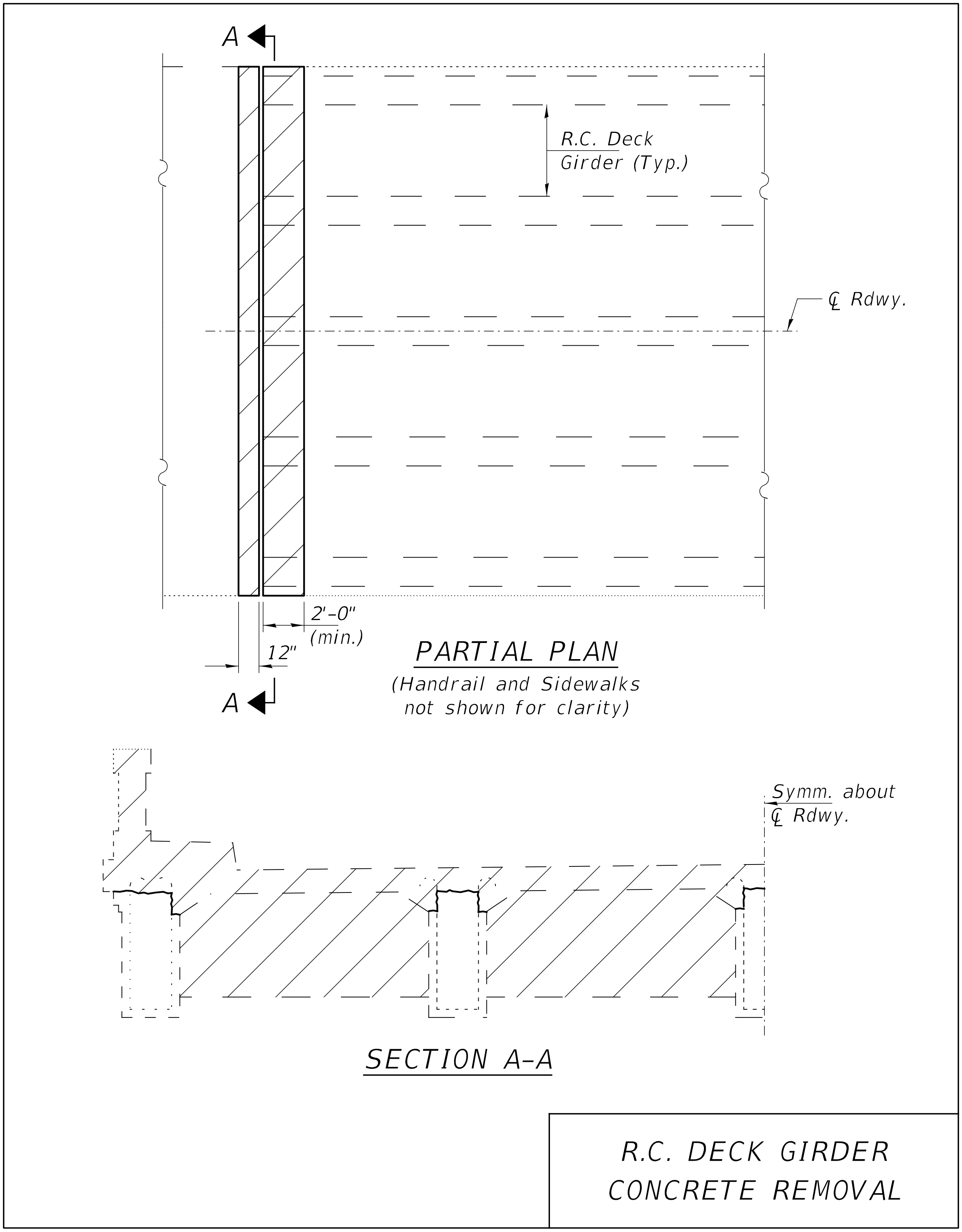


Figure 2.5.5-1: R.C. Deck Girder Concrete Removal

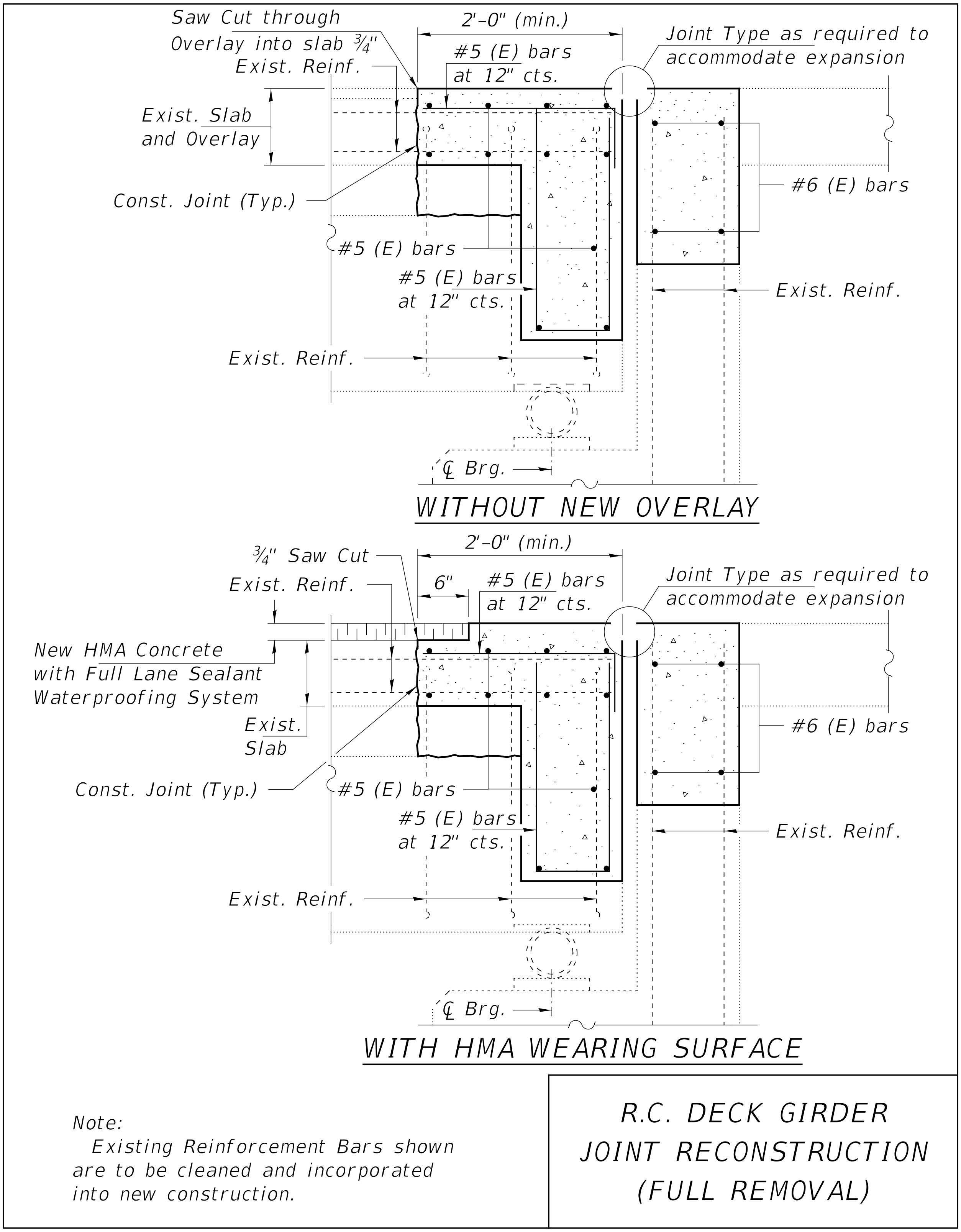
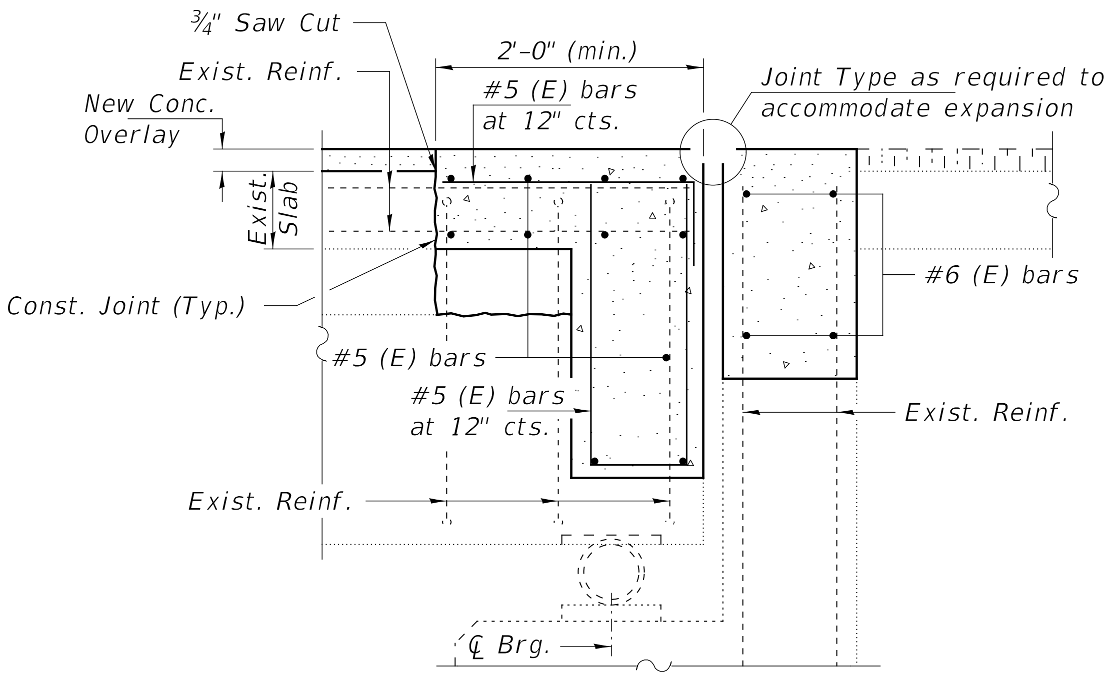


Figure 2.5.5-2: R.C. Deck Girder Joint Reconstruction (Full Removal)



WITH CONCRETE OVERLAY

Note:
Existing Reinforcement Bars shown are to be cleaned and incorporated into new construction.

R.C. DECK GIRDER
JOINT RECONSTRUCTION
(FULL REMOVAL)

Figure 2.5.5-3: R.C. Deck Girder Joint Reconstruction (Full Removal)

2.5.6 Reinforced Concrete Slab

Replacement of deck ends for a reinforced concrete slab also involves the replacement of existing roller bearings with new elastomeric bearings. This will require the installation of side retainers which is difficult because of the lack of space. It will also require the installation of a temporary support system. Consequently, a better option is to eliminate the joint by encasing the end of the deck in concrete, burying the existing roller bearings and making the abutment “integral”. For details see Section 2.5.8.

Although not preferred, a limited partial-depth removal area in the immediate area of the joint can be utilized if inspection indicates that the slab concrete that will remain in the area of the joint is sound and analysis shows that the reduced end area is structurally adequate to support the structure during the joint replacement operations without need for temporary support. Refer to Figure 2.5.6-1 and Figure 2.5.6-2 for details of partial slab removal and replacement.

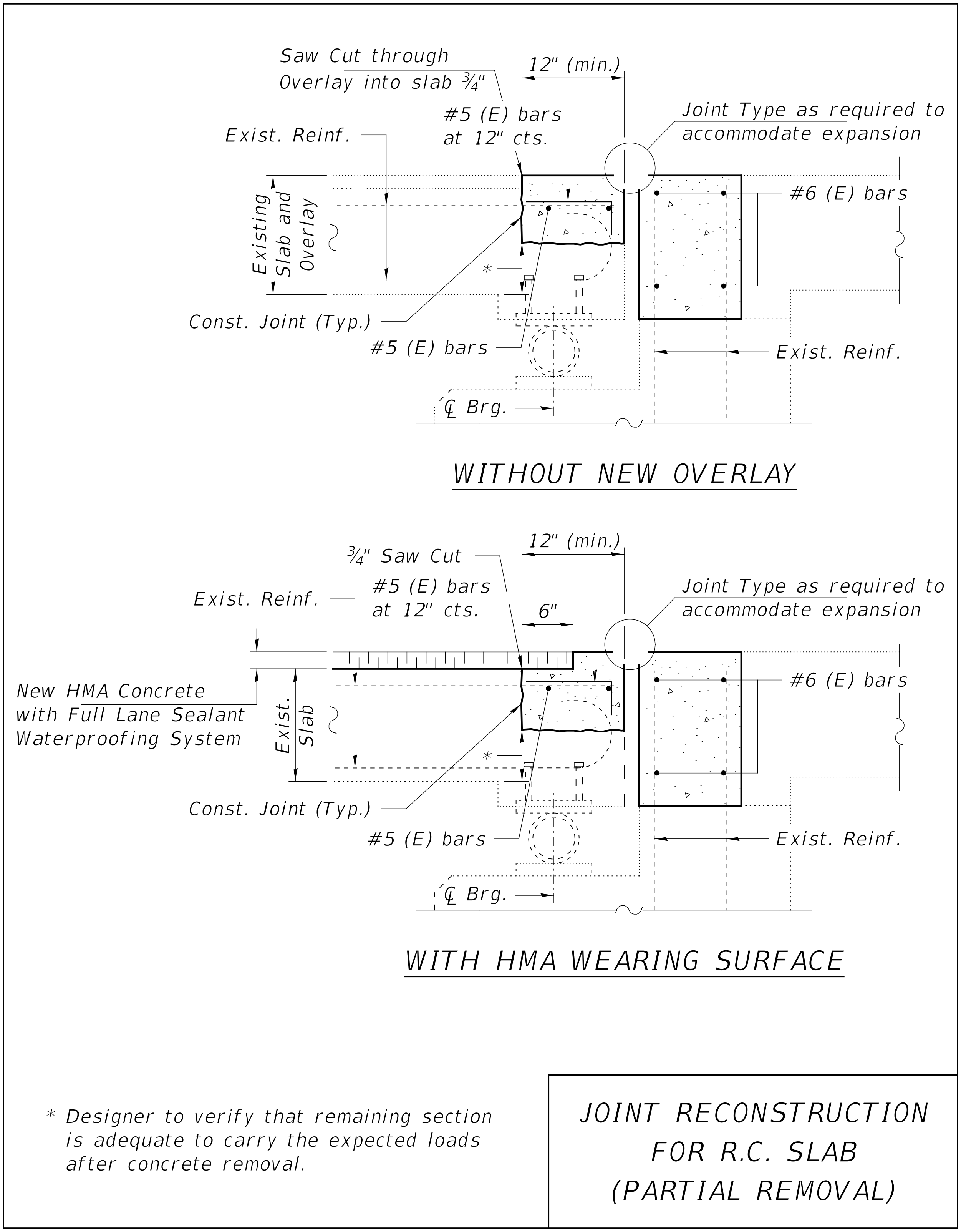


Figure 2.5.6-1: Joint Reconstruction for R.C. Slab (Partial Removal)

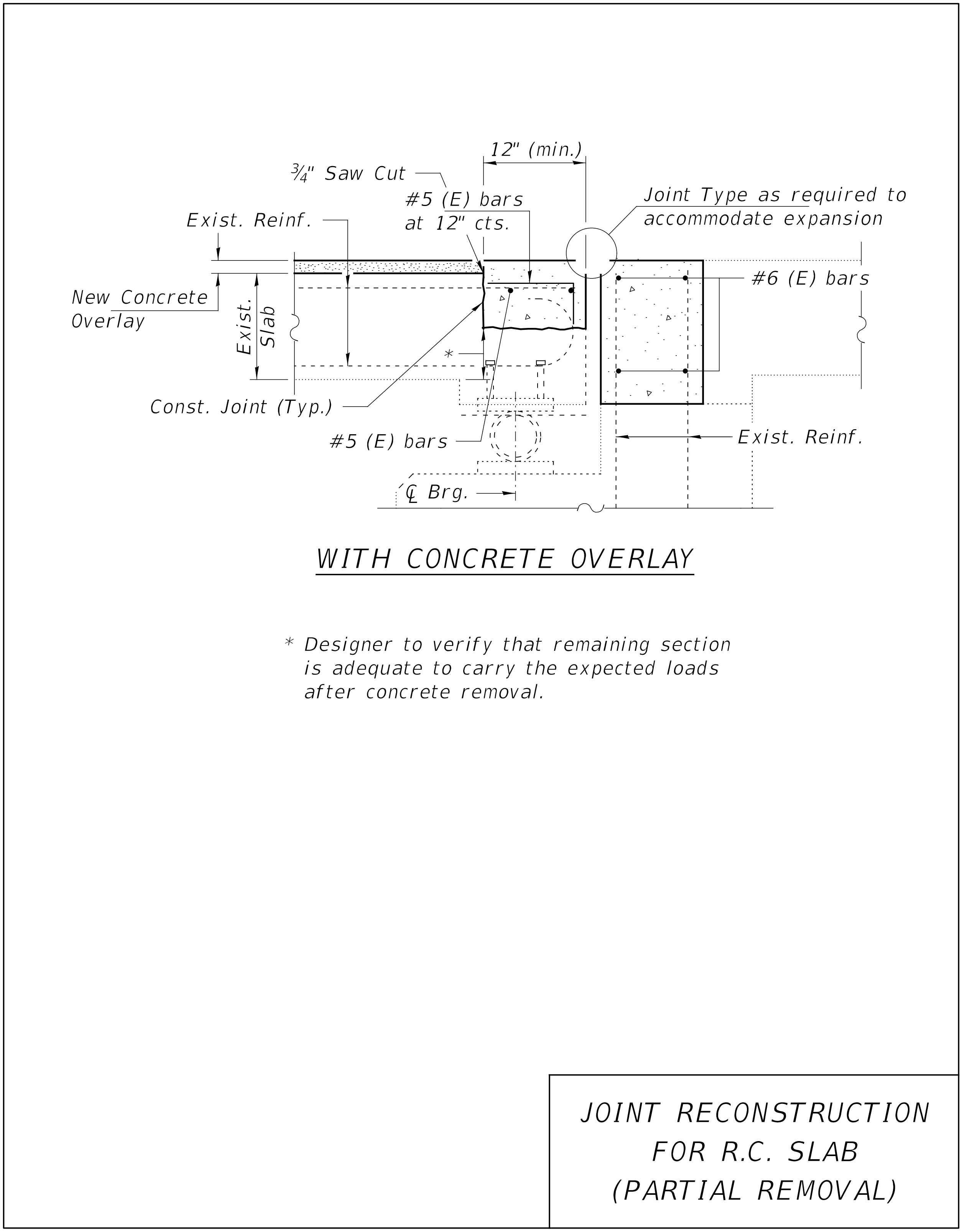


Figure 2.5.6-2: Joint Reconstruction for R.C. Slab (Partial Removal)

2.5.7 Minimal Concrete Removal for Strip Seal Installation

If a strip seal joint is to be installed on a bare deck or at locations in which a new overlay will be placed, and when the concrete around the joint area is deemed to be in good condition, minimal concrete removal can be used. See Figures 2.5.7-1, 2.5.7-2 and 2.5.7-3 for example details. This type of concrete removal is particularly useful when the deck is supported by PPC I beams in order to minimize potential damage to the beams.

While the opening on the top section of the joint will be the one needed to accommodate the new joint, it is necessary that the existing opening in the section of concrete below, that is to remain, be sufficient to handle the expected bridge expansion. If that is not possible then a full depth removal should be used instead (See Section 2.5.4).

Treatment of the deck and parapet is shown in Figures 2.5.7-4 and 2.5.7-5. A note should be included in the plans advising the contractor to avoid damaging the beams' top flange during concrete removal operations. Depending on the skew it may be necessary to remove a larger section of the parapet than the deck.

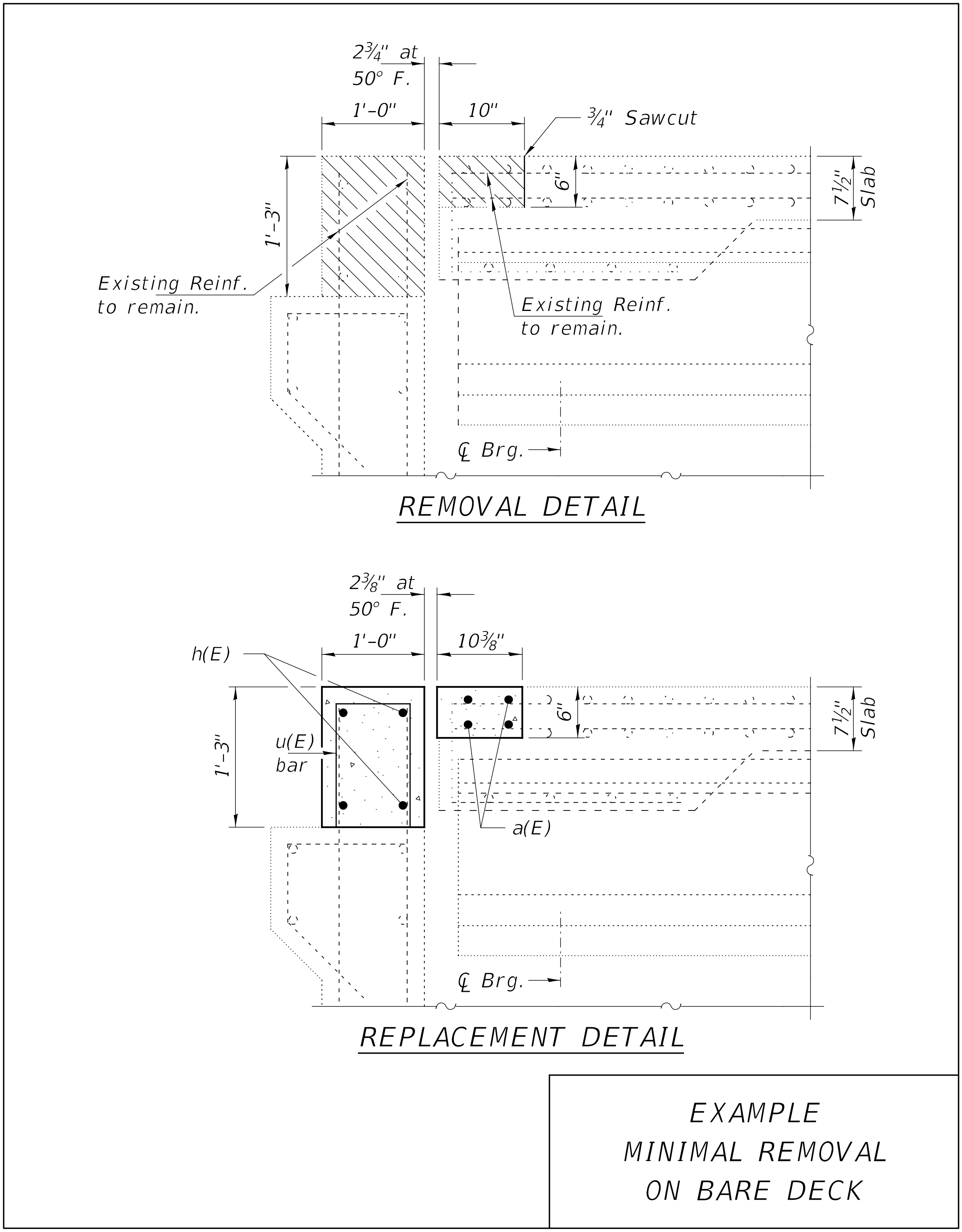


Figure 2.5.7-1: Example – Minimal Removal on Bare Deck

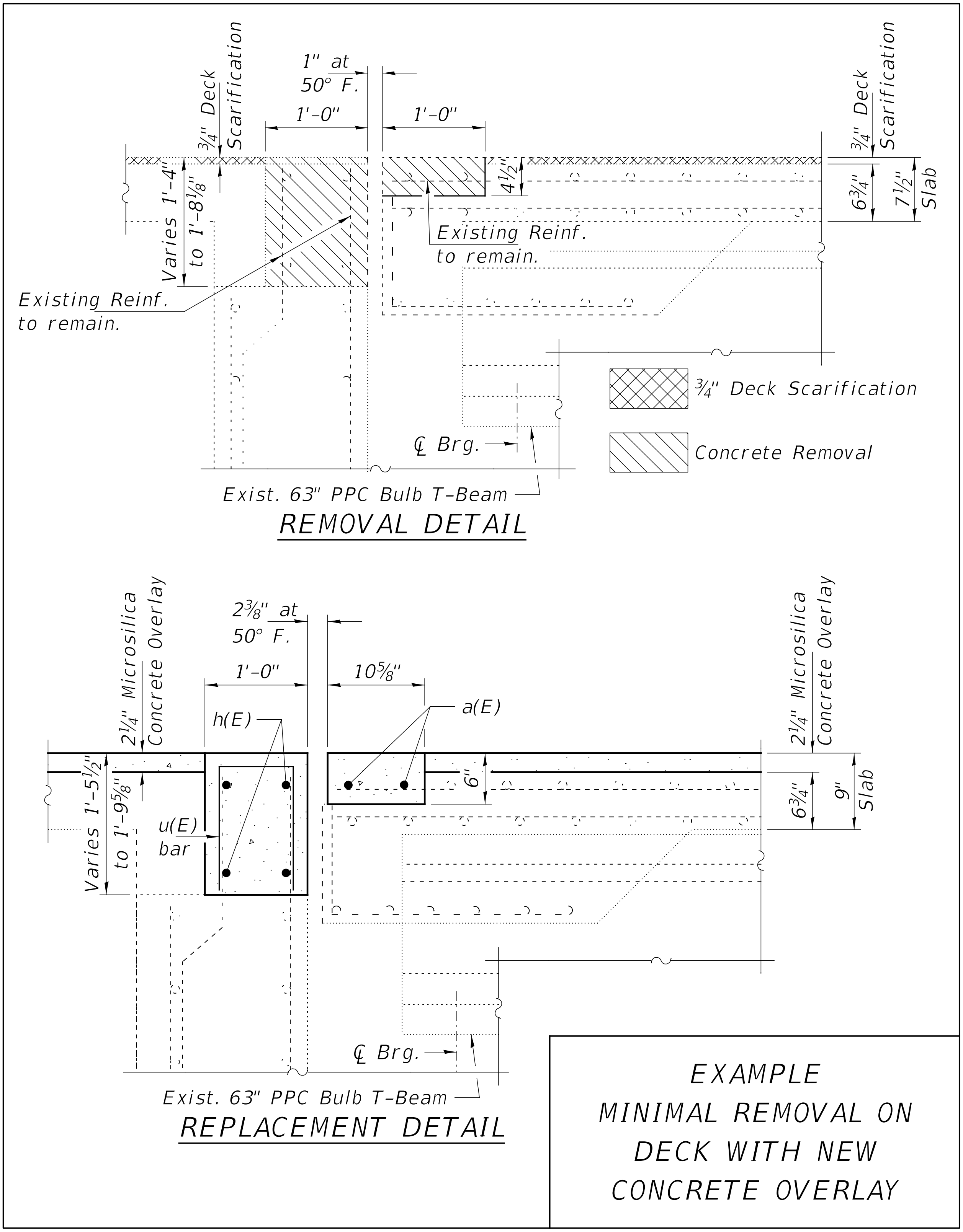


Figure 2.5.7-2: Example – Minimal Removal on Deck with New Concrete Overlay

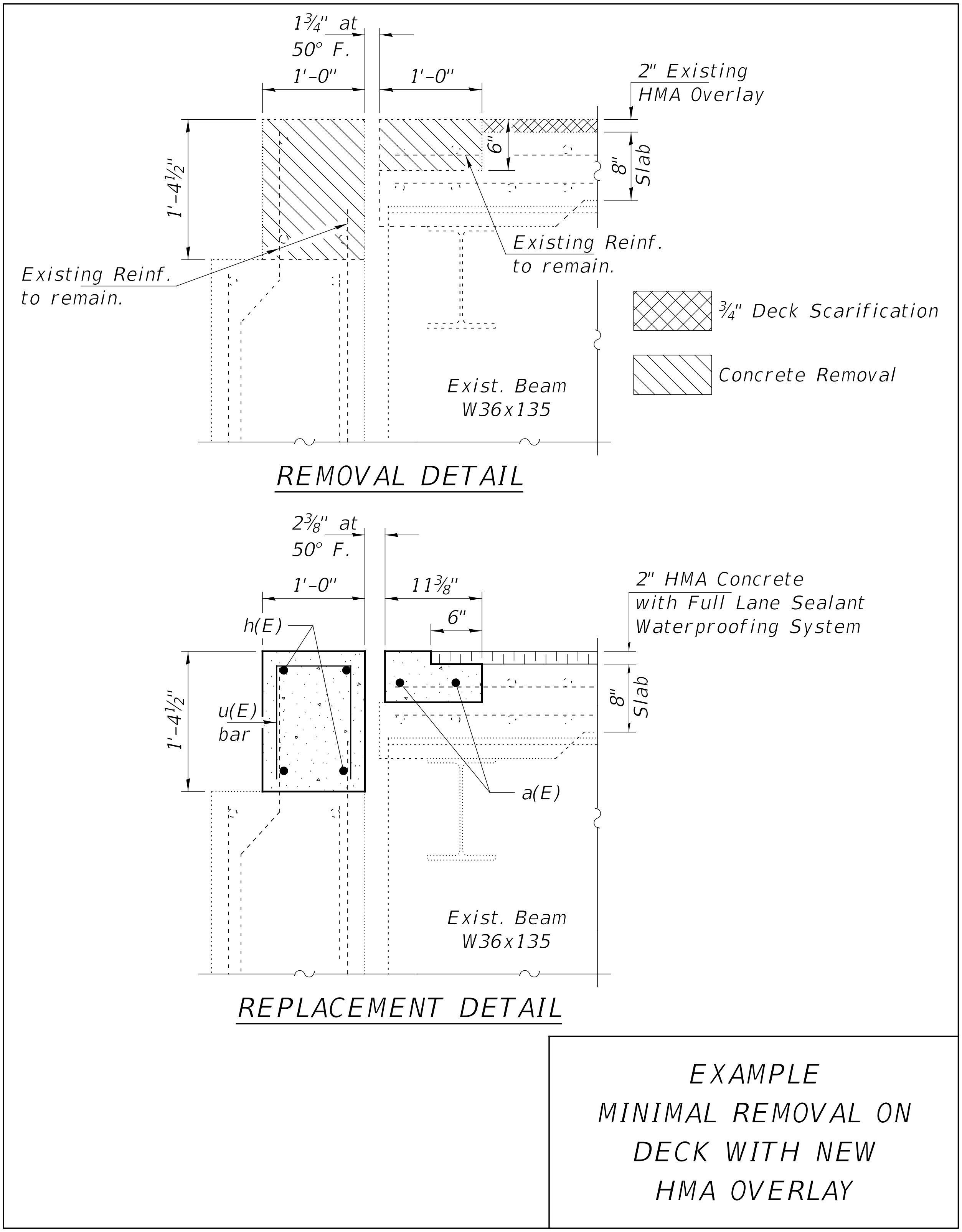


Figure 2.5.7-3: Example – Minimal Removal on Deck with New HMA Overlay

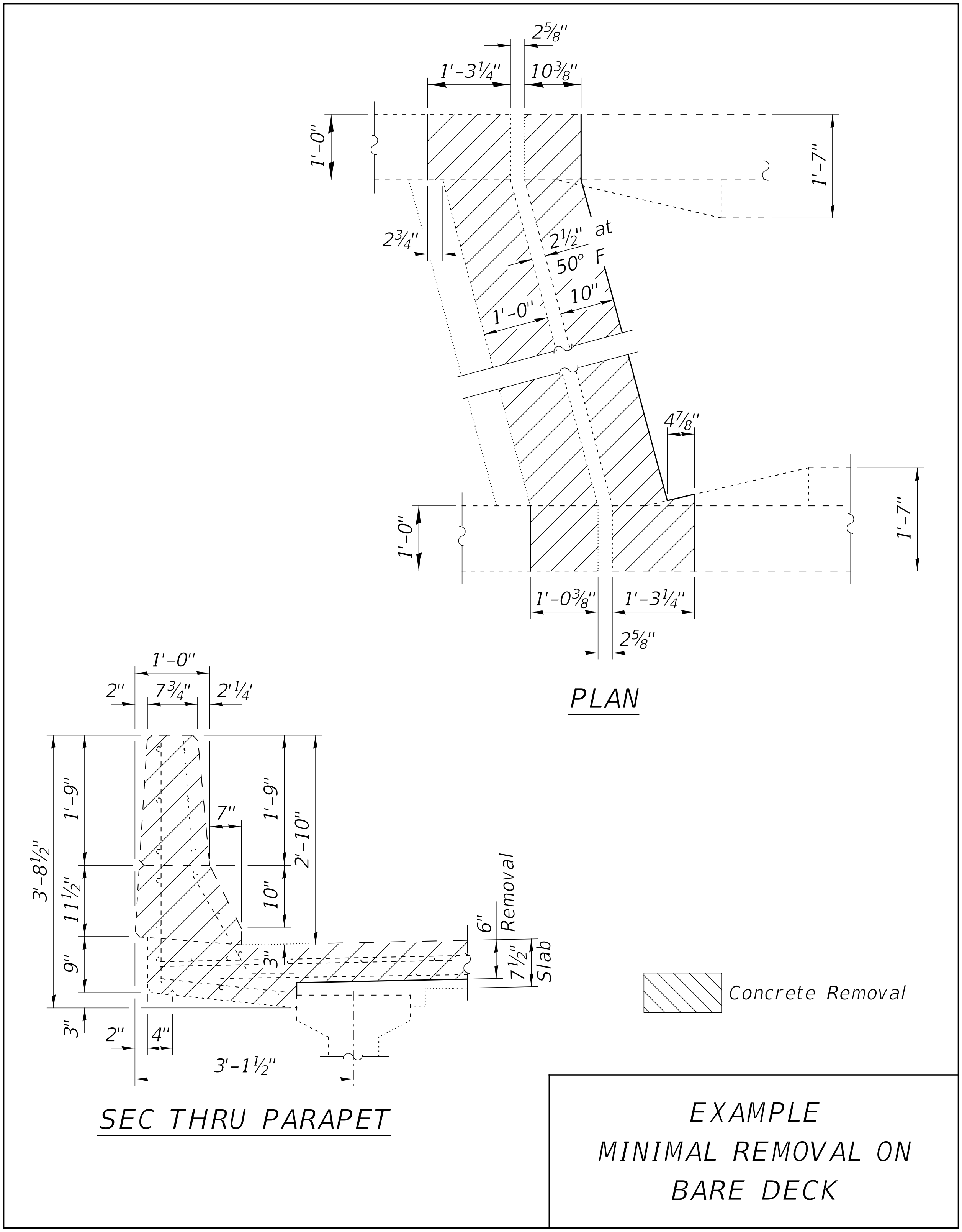


Figure 2.5.7-4: Example – Minimal Removal on Bare Deck

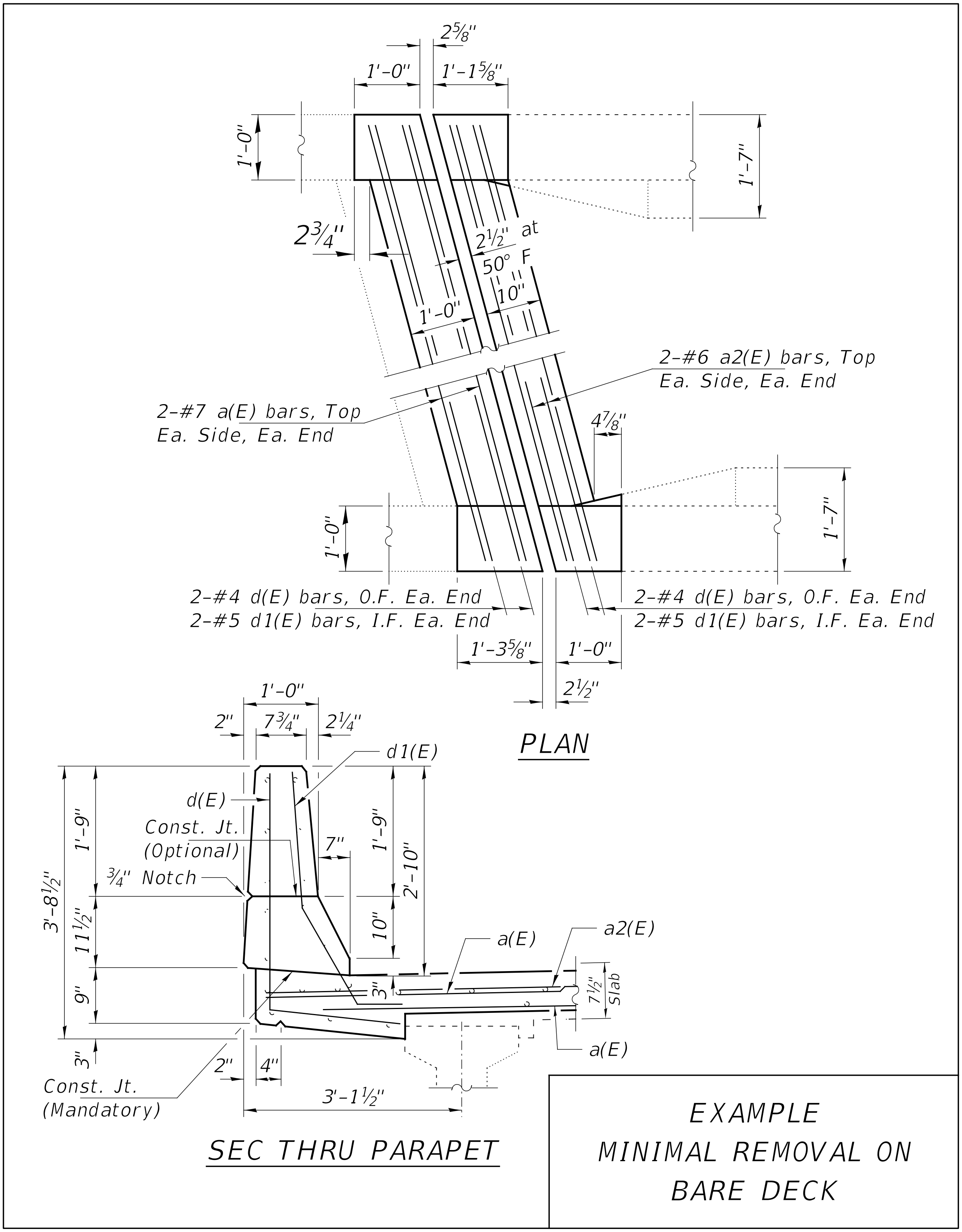


Figure 2.5.7-5: Example – Minimal Removal on Bare Deck

2.5.8 Joint Elimination

Although many new bridges are being designed and constructed without expansion joints through the use of integral abutments, the majority of bridges in the bridge inventory have expansion joints. For some bridges, it may be possible to eliminate expansion joints rather than replacing them, when those joints are leaking and no longer functioning properly. The resulting abutment configuration is typically referred to as a Jointless Stub Abutment. Eliminating expansion joints will most likely result in less maintenance, with a lower life cycle cost, for the bridge going forward.

It is possible to eliminate the expansion joint at an abutment for some structures by pouring a concrete diaphragm at the abutment that results in a monolithic connection between the superstructure and approach slab. The monolithic connection may require that expansion and contraction of the superstructure be accommodated through movement of the abutment cap and flexure of the piles (note that such movement must still be accommodated by a joint on the approach pavement away from the abutment). Therefore, before eliminating expansion joints in an existing bridge, an evaluation of the structure must be made to ensure the joint elimination will not adversely impact the bridge’s structural integrity or behavior. Parametric analyses have been performed on a range of structures with various superstructure types, pile types, number of piles, pile batters, skews, soil types and soil strengths to determine the limits of acceptable contributing expansion lengths when eliminating an expansion joint. For an abutment being considered for joint elimination, Table 2.5.8-1 is applicable to steel and concrete beam superstructures with total beam depth plus bearing height less than or equal to 48 inches and provides the maximum contributing expansion length at that abutment for joint elimination to be acceptable. Additional criteria for use of Table 2.5.8-1 are provided with the table.

The parametric analyses, as well as the details in figures discussed below, were developed for bridges only having expansion joints at the abutments. Other assumptions made for the analyses include:

- Temperature range of -20º F to 120º F
- Wearing surface allowance = 50 psf
- Staggered pile spacing with front row having 2:12 batter
- Maximum spacing between front and back row of piles = 2’-3”
- HP piles:
  - Oriented for bending about the strong axis for abutment cap deflection perpendicular to the longitudinal axis of the abutment
  - Fy = 36 ksi
- Metal shell piles:
  - Fy = 30 ksi
  - f’c = 3.5 ksi
  - Metal shell piles shall only be considered if it can be verified the piles are true metal shell piles and not a fluted or tapered cylindrical metal pile possibly joined by a compression splice
- Concrete Piles:
  - Reinforcement Fy = 40 ksi
  - f’c = 3.0 ksi
  - Main reinforcement bar clear cover = 2 inches
  - 12 inch round piles with 4-#7 bars
  - 14 inch square piles with 4-#8 bars
  - 16 inch or 18 inch square piles with 4-#9 bars
- Soil conditions:
  - Cohesive soils with Qu ≤ 1.0 tsf
- Granular soils with a loose (SPT N value ≤ 10) or medium (SPT N value ≤ 20) density. See adjustment factor for maximum contributing expansion lengths for medium density granular soils.

Figures 2.5.8-2 through 2.5.8-4 provide details for eliminating expansion joints for steel and concrete beam superstructures based on beam depth plus bearing height, and slab type superstructures.

The details in Figure 2.5.8-2 shall be used with steel and concrete beam superstructures with a total beam depth plus bearing height less than or equal to 48 inches. These details were developed with the intention of allowing rotation of the superstructure to limit pile demands, and this assumption was included in the analyses when determining the allowable contributing expansion lengths given in Table 2.5.8-1. Figure 2.5.8-2 was developed for cases where the existing bearings are steel rocker bearings. The details may also be used with existing elastomeric bearings; however, the anchor bolts shall be cut (flush with the concrete for Type I bearings and flush with the bottom bearing plates for Type II or Type III bearings), the side retainers removed and PJF or foam shall be placed around the elastomeric bearings for their full height to allow continued movement and prevent damage to the elastomeric bearings. In all cases, for ease of new concrete installation, existing diaphragms and cross bracing shall be removed.

Figure 2.5.8-3 may be used for steel and concrete beam superstructures with a total beam depth plus bearing height greater than 48 inches.

The details of Figure 2.5.8-3 can also be used for the shallower depth (less than 48") when the limitations of Table 2.5.8-1 are exceeded. This detail allows for more expansion to occur at abutments at the level of the bearing, therein better isolating the piles and allowing for longer expansion lengths.

Figure 2.5.8-4 may be used for concrete slab bridges with a total length less than or equal to 130 feet. Use of this detail is also limited to cohesive soils with Qu less than or equal to 1.0 tsf or described as soft or medium, or loose granular soils (SPT N value less than or equal to 10). Use with medium dense granular soils (SPT N value less than or equal to 20) is also acceptable with a total bridge length less than or equal to 90 feet.

The details in Figures 2.5.8-2 through 2.5.8-4 were prepared for open stub type abutments. Details similar to the details provided here for stub abutments may also be used for closed abutments with expansion bearings or fixed bearings. When the existing bearings are expansion bearings, PJF or foam shall be placed around the bearings for their full height to allow continued movement.

In addition to the details provided in the figures discussed above, contract plans developed for elimination of expansion joints should include any work required to eliminate snag points that may exist that would prevent movement and rotation of the superstructure and the approach pavement. This includes isolating the approach pavement from abutment wingwalls to allow movement of the approach pavement independently from the wingwalls. Means of accommodating movement between the bridge approach slab and roadway pavement shall be established or re-established as appropriate.

The beam end encasement details were developed for cases where beam web deterioration, if it exists, does not extend within 6 inches of the edge of the proposed encasement. If deterioration exists and does extend to within 6 inches of the edge of the proposed encasement, or beyond, the encasement shall be widened, or the beam ends shall be strengthened with steel plate repairs prior to construction of the encasement. Contract plans shall also specify cleaning corroded beam ends, to SSPC-SP3 prior to placement of encasement concrete, in areas that will not be strengthened with steel plates.

In all cases, the capacity of existing bearings to carry the additional load of the beam end encasement and approach pavement shall be verified. If existing bearing capacity is not sufficient due to the additional load or due to condition, the bearings shall be replaced.

For bridges beyond the limits described below and for bridges that also have expansion joints at the piers, the Repairs Unit shall be consulted and a special analysis of the structure to determine the effects on all components of the bridge may be required.

		Maximum Expansion Length (ft)				
Pile Type	Pile Size	0° skew	5° skew	10° skew	15° skew	30° ≥ skew < 45°
Steel	HP 8x36	170	150	140	130	110
	HP 10x42	180	160	150	140	120
	HP 10x57	200	190	180	170	150
	HP 12x53	220	200	190	170	160
	HP 12x63	230	210	200	190	170
	HP 12x74	250	230	220	210	190
	HP 12x84	260	240	230	220	200
	HP 14x73	260	240	230	220	190
	HP 14x89	280	260	250	240	220
	HP 14x102	290	270	260	250	230
	HP 14x117	300	280	270	260	240
Metal Shell	MS12x0.179	150	150	150	150	150
	MS12x0.25	190	190	190	190	190
	MS14x0.25	200	200	200	200	200
	MS14x0.312	210	210	210	210	210
Concrete	12" Round	70	70	70	70	70
	14" Square	90	90	90	90	90
	16" Square	70	70	70	70	70
	18" Square	80	80	80	80	80

Table 2.5.8-1: Maximum Expansion Length for Joint Elimination

Notes:

1. The analyses assumed 11 piles under the abutment cap. For cases with fewer than 11 piles, the maximum lengths given in the table shall be reduced by 30 feet.

2. The maximum lengths provided are applicable to cohesive soils with Qu less than or equal to 1.0 tsf and loose granular soils with SPT N values less than or equal to 10. The maximum lengths shall be multiplied by a value of 0.7 if used with medium dense granular soils with SPT N values less than or equal to 20.

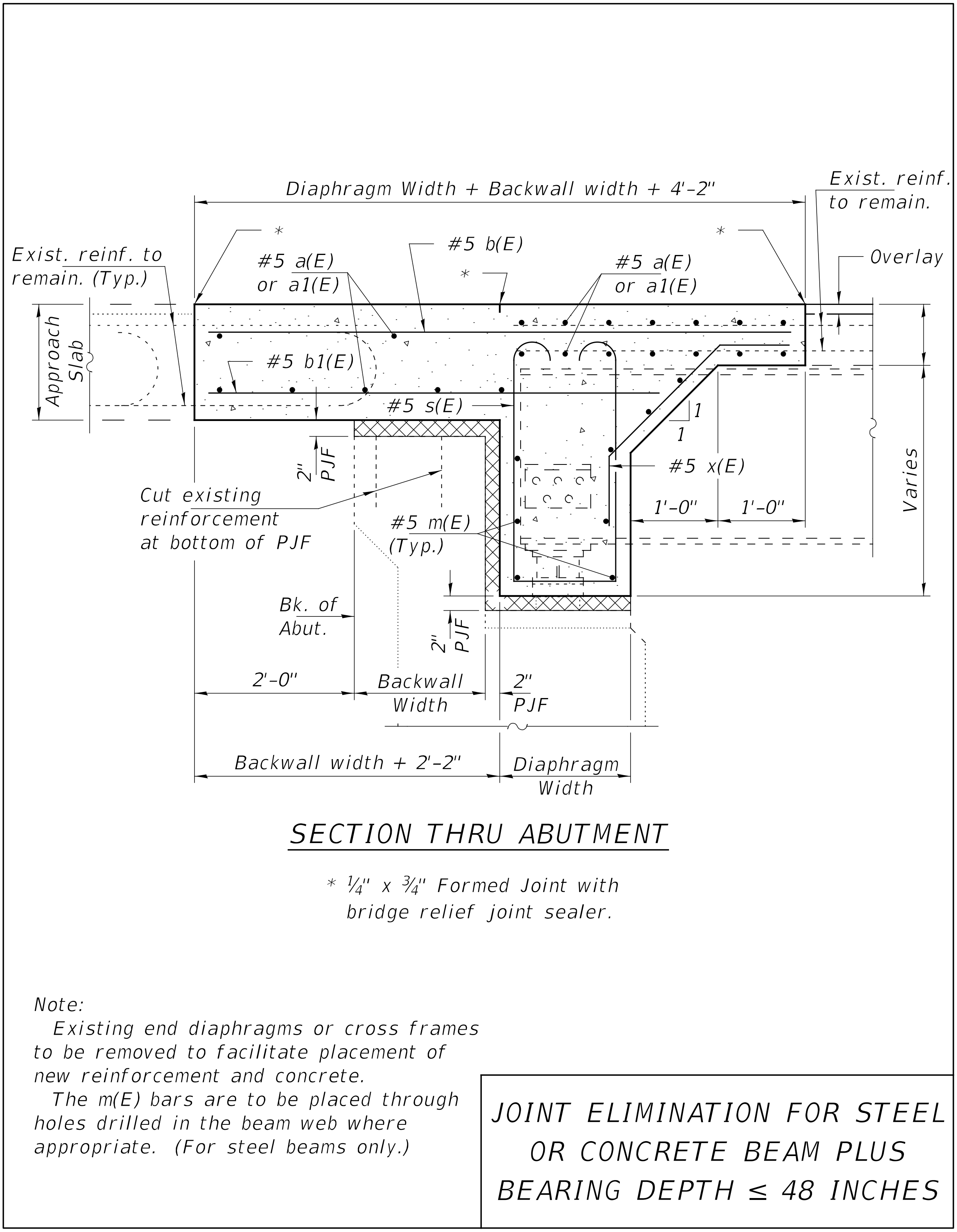


Figure 2.5.8-2: Joint Elimination for Steel or Concrete Beam Plus Bearing Depth ≤ 48”

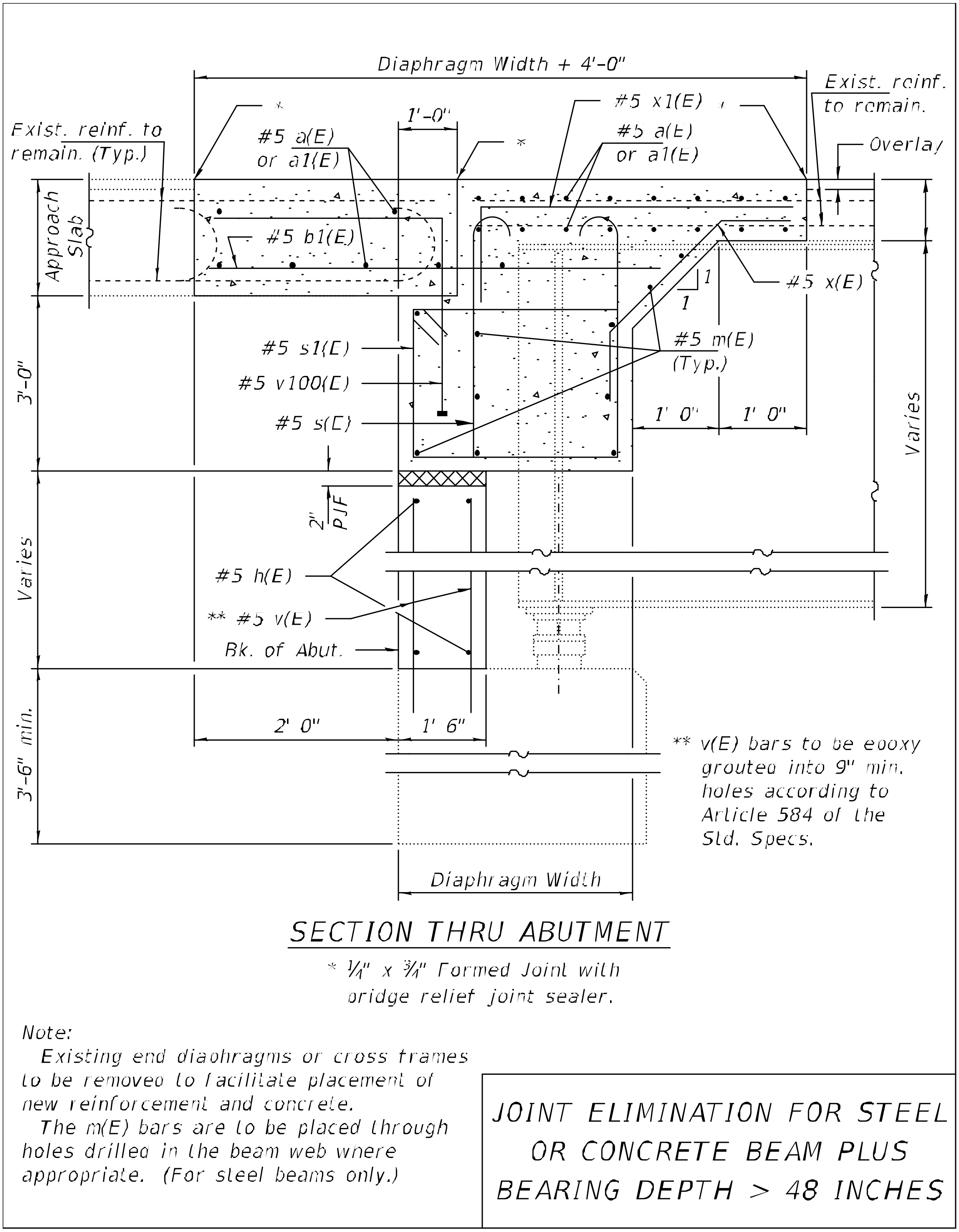


Figure 2.5.8-3: Joint Elimination for Steel or Concrete Beam Plus Bearing Depth > 48"

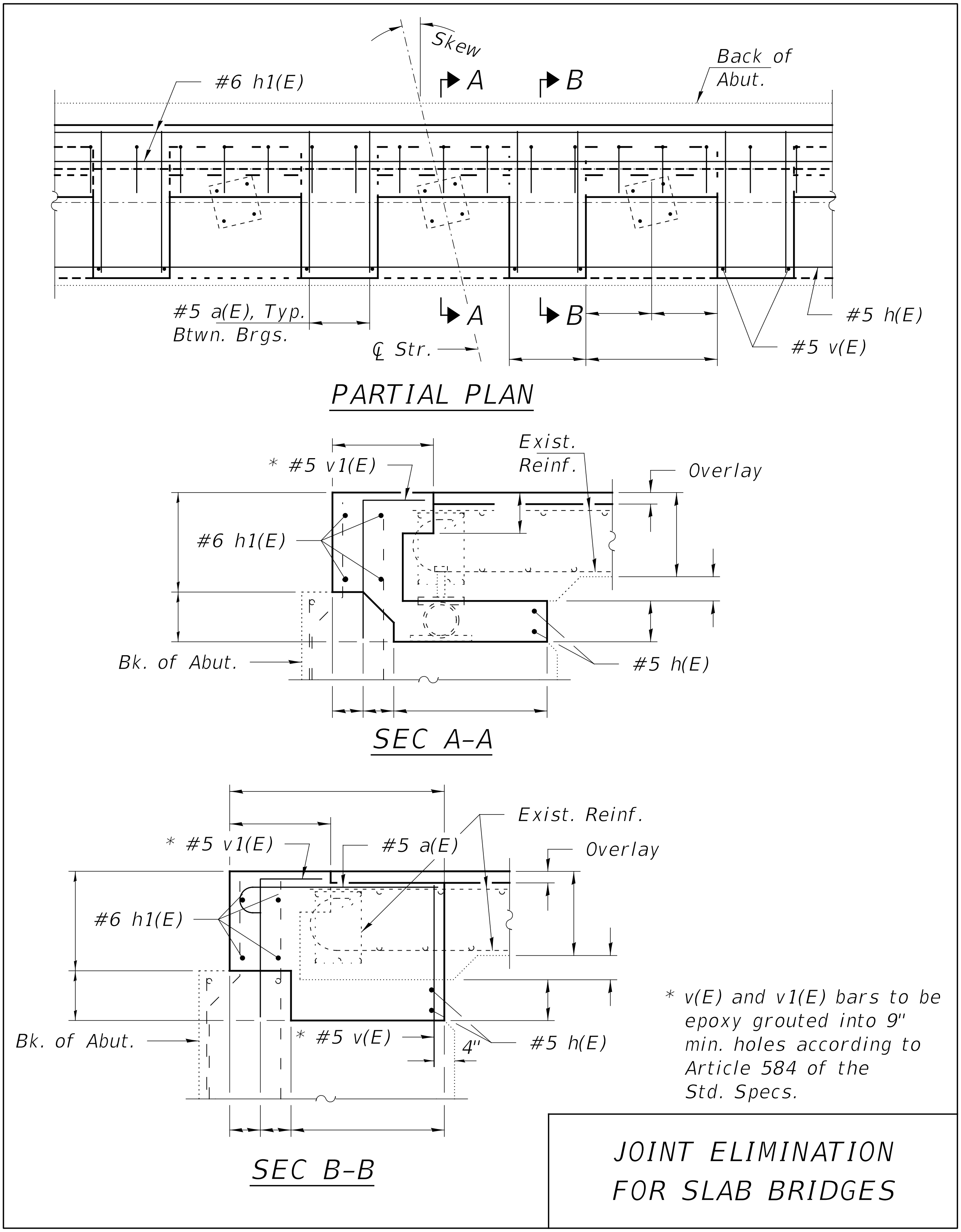


Figure 2.5.8-4: Joint Elimination for Slab Bridges

2.5.9 Steel Finger Plate Joint

When developing plans for the installation of steel finger plate joints, the information shown in the existing design plans, as-built plans (if available) and shop plans should be verified. Construction delays can occur when existing diaphragms or beam ends are found at different locations or elevations than those indicated by the existing plans. In order to avoid necessity to refabricate or redesign the joint during construction, the District should provide the engineer designing the joint with field measurements showing the location of the existing diaphragms, beam ends and bearings relative to the proposed joint opening and top of deck. Figure 2.5.9-1 and Figure 2.5.9-2 indicate typical dimensions which are critical for the accurate detailing of steel finger joint replacement plans. Note that the longitudinal dimensions are temperature dependent. Design procedures and details for new finger plates are described in Section 3.6.2.2 of the IDOT Bridge Manual.

In order to keep the troughs from clogging, the maximum possible slope should be provided for them, preferably 1”/ft. In very wide structures it may be necessary to provide more than the typical two sections sloping down and away from the Centerline of Roadway.

Because of the variations found in the field the designer should consider detailing the height of the stools ¼” shorter than needed. The gap between stool seat and top flange of diaphragm or crossframe can be filled with shim plates for proper fitting.

A note should be added to the plans as follows: Tapered shims shall be added under the stools, as required by the Engineer, to make a smooth finger joint. Cost shall be included with “Furnishing and Erecting Structural Steel”. The finger plates shall be flame cut as provided in Article 505.04(k) of the Standard Specifications.

Concrete deck removal shall consist of 3’-6” minimum on each side of the centerline of the joint to allow room for the placement of the new finger plate and reinforcement bars.

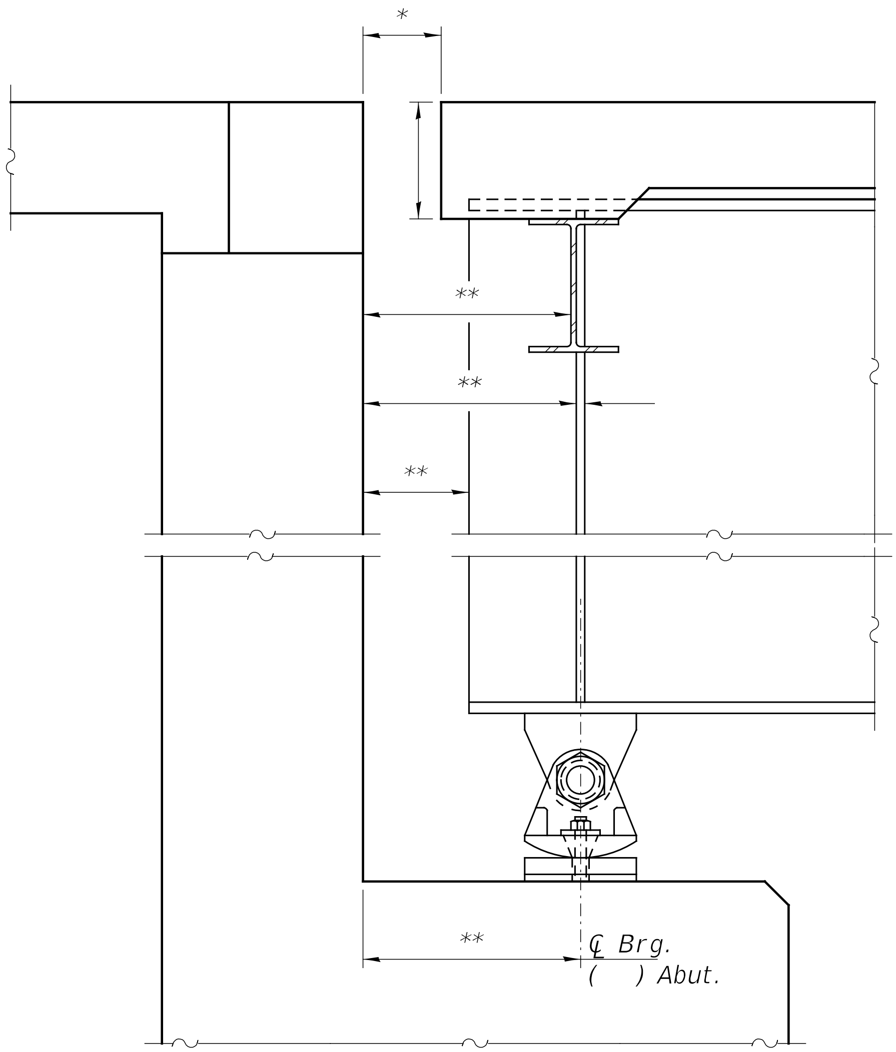
When an existing finger plate is to remain, and a new overlay will raise the profile grade, the joint must be adjusted as shown in Figure 2.5.9-3.

Structure No.________________

Beam No.____________________

Date ________________________

Temperature _________________



Note:

Assumed direction of measurements are noted below. The actual direction each measurement is taken shall be documented.

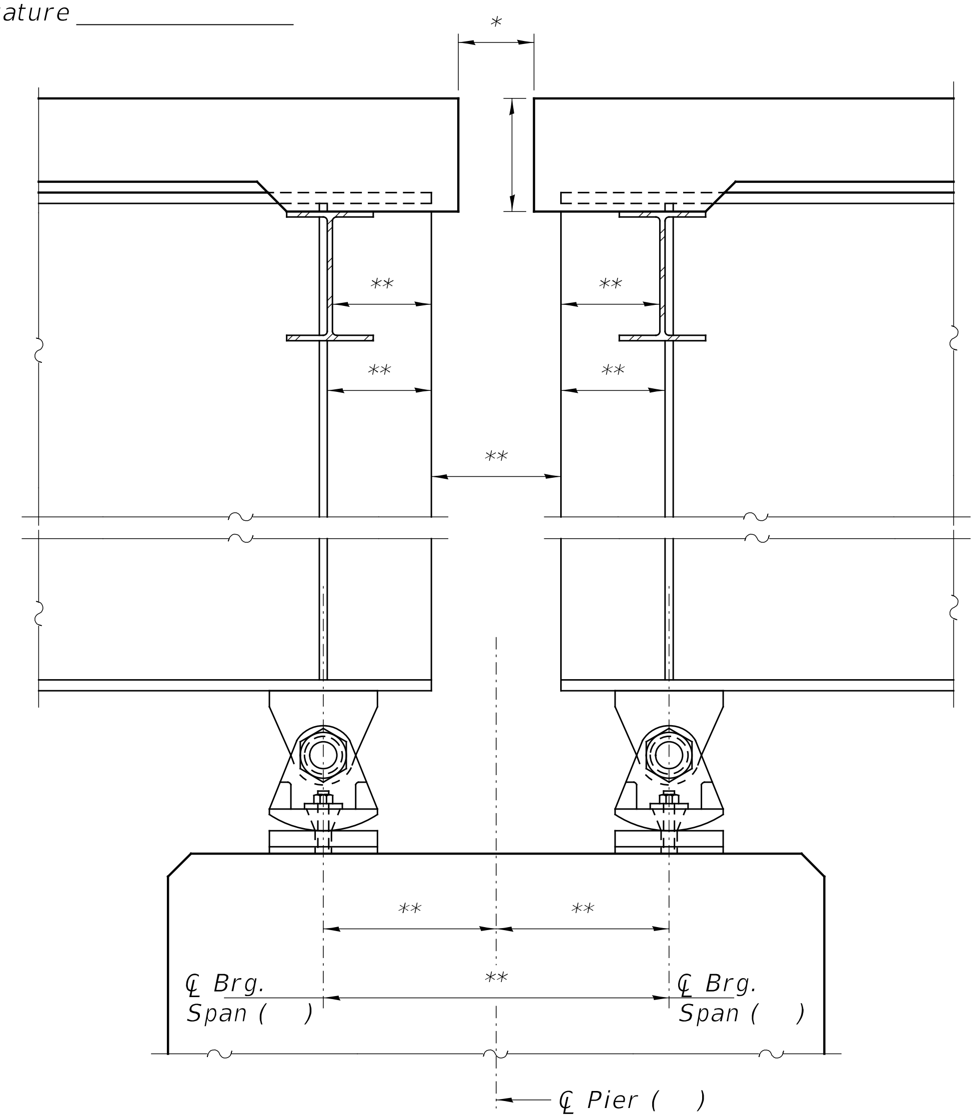
* Measurement is assumed to be taken perpendicular to centerline of joint.

** Measurements are assumed to be taken parallel to centerline of girder.

FINGER JOINT
FIELD DIMENSIONS
(AT ABUTMENT)

Figure 2.5.9-1: Finger Joint Field Dimensions (At Abutment)

Structure No.________________
Beam No.____________________
Date ________________________
Temperature _________________



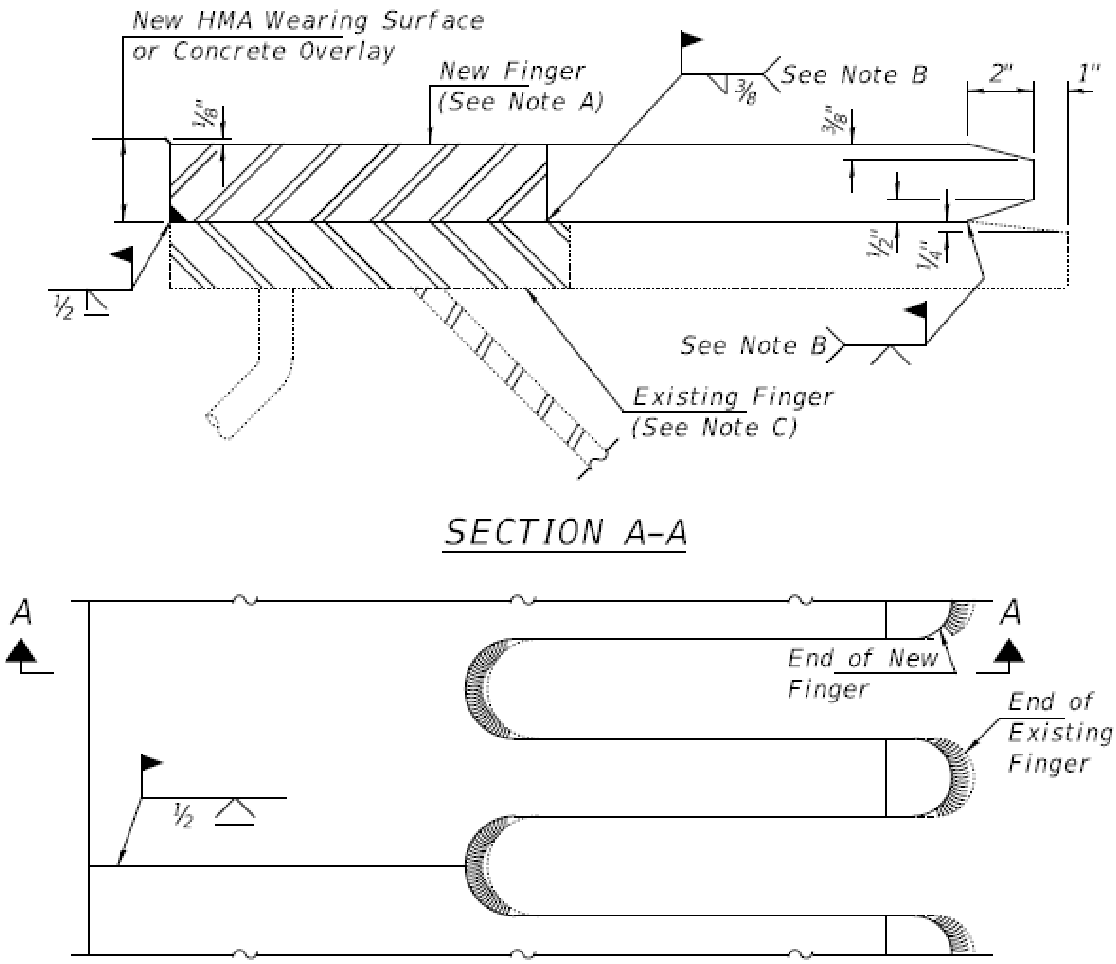
Note:
Assumed direction of measurements are noted below. The actual direction each measurement is taken shall be documented.

* Measurement is assumed to be taken perpendicular to centerline of joint.

** Measurements are assumed to be taken parallel to centerline of girder.

FINGER JOINT
FIELD DIMENSIONS
(AT PIER)

Figure 2.5.9-2: Finger Joint Field Dimensions (At Pier)



Note A:
New Finger shall be blast cleaned to SSPC SP 10 and shop painted with the inorganic zinc rich primer.

Note B:
Fillet weld sizes at ends (tips) and crotches of fingers shall vary from 3/8" near centers to 3/16" minimum near edges as new and existing plates converge.

Note C:
Existing Finger widths and exact locations must be field verified. A template shall be made to insure alignment. Remove foreign material that would prevent uniform contact between new and existing plates by method approved by the Engineer.

EXPANSION JOINT TREATMENT WITH GRADE RAISE AND EXISTING FINGER PLATES

Figure 2.5.9-3: Expansion Joint Treatment with Grade Raise and Existing Finger Plates

2.5.10 Modular Expansion Joints

At the district’s request modular joints can be used in lieu of finger plate joints. Designers should consult ABD Memorandum 22.8 Modular Expansion Joint Details and Re-Certification and the special provisions for Modular Expansion Joint for information on the design of these joints.

Manufacturers will provide the joint design details for their own joints. Designers shall provide in the plans information with respect to the expected longitudinal and transverse movement of the joint. Dimensions of the existing beam configuration similar to the ones for finger plate joints should be obtained. Critical information includes the distance from top of the final deck surface to diaphragm top flange or the cross frame top horizontal member.

Installation of modular joints on existing structures present unique challenges during construction because of having to deal with existing steel elements as well as reinforcement congestion. Unlike a new structure in which the beam ends are coped to allow for equal spacing of the support boxes, on an existing structure support boxes must be placed between the beam ends. Designers should consider potential installation issues and address them in the plans.

To avoid bending the longitudinal bars out of the way (potentially damaging the epoxy coating), longitudinal bars should be cut and new hooked bars should be mechanically spliced to them (see Figure 2.5.10-1). Because each manufacturer has a different configuration for their joints, adjustments will need to be made during construction to ensure that all the required reinforcement is installed.

Also, in order to satisfy the maximum spacing for the support boxes it is often necessary to install a box on the outside of the fascia beam. Details should be provided in the plans for the proper anchorage of this support box (see Figure 2.5.10-2).

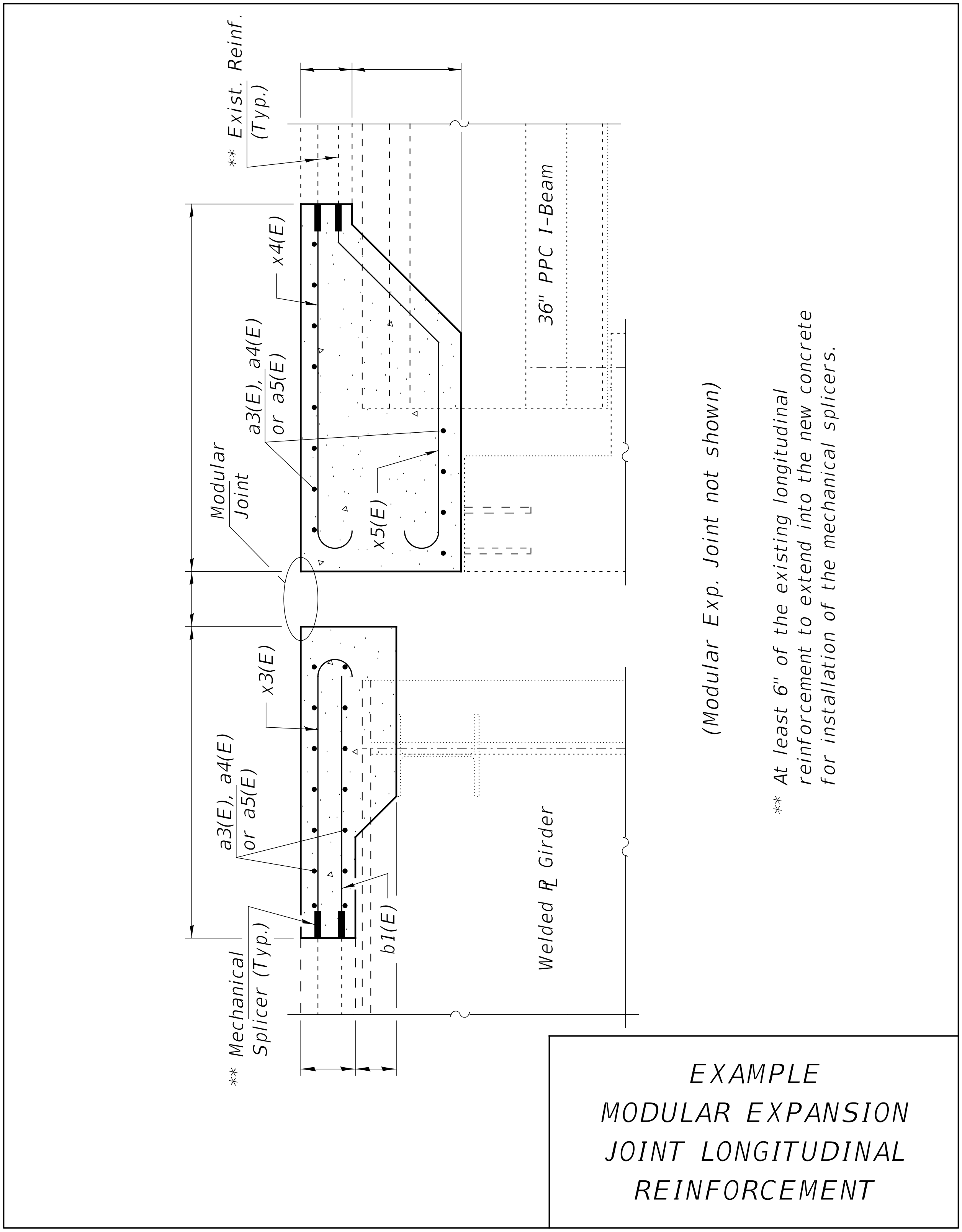


Figure 2.5.10-1: Example – Modular Expansion Joint Longitudinal Reinforcement

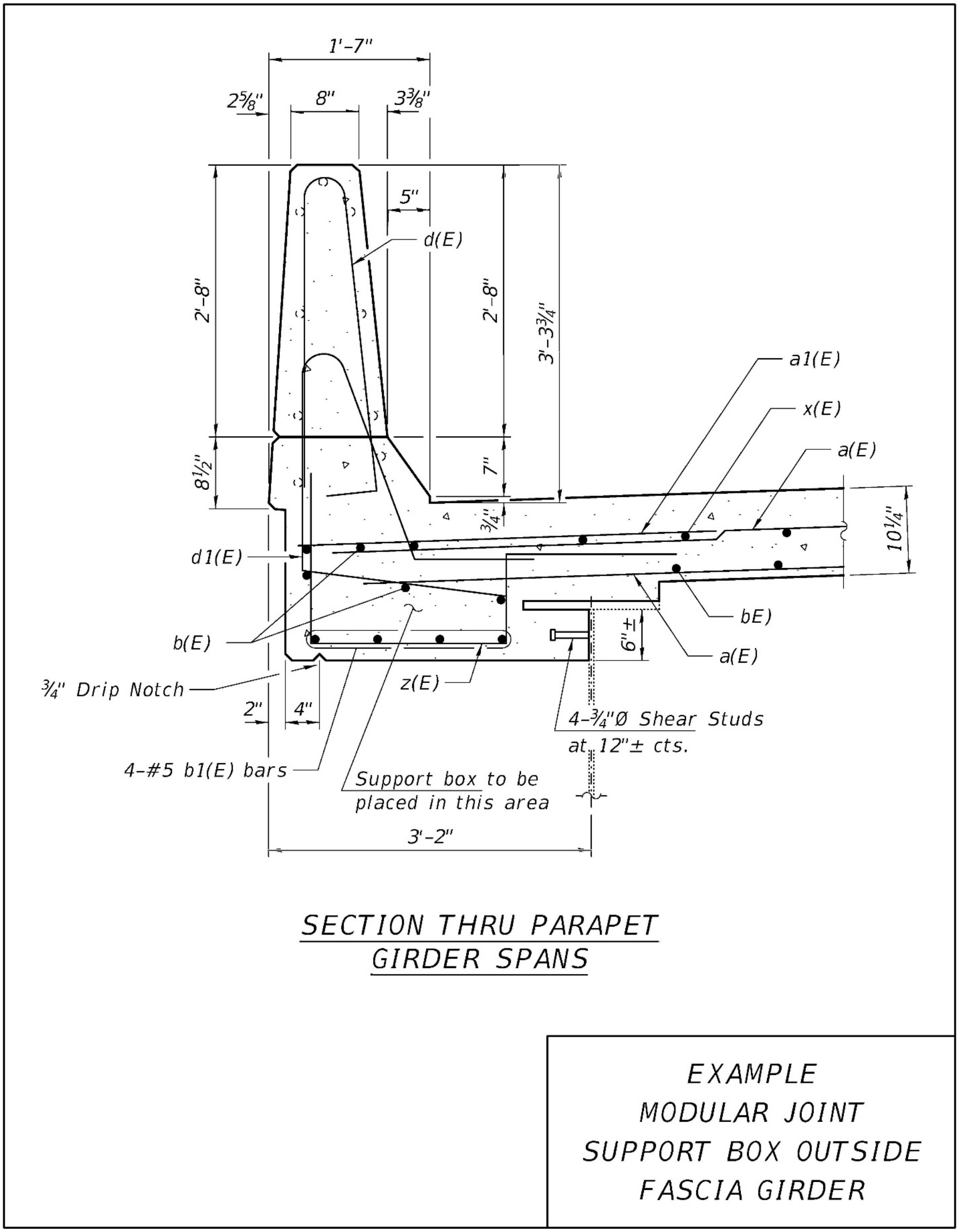


Figure 2.5.10-2: Example – Modular Joint Support Box Outside Fascia Girder

2.6 Longitudinal Joint Closure

2.6.1 General

Maintaining an effective seal against the intrusion of water, chlorides and other contaminants through a leaking longitudinal deck joint is very difficult. The beams or girders supporting the deck are often adversely affected by moisture and deicing agents passing through poorly sealed longitudinal joints. One solution to the problem of a leaking longitudinal joint is the removal of the existing joint seal along with a portion of the adjacent concrete deck and the construction of a new segment of continuous reinforced concrete deck (without a sealed longitudinal joint). This method of eliminating the leaking longitudinal joints can be applied to all concrete superstructures with out-to-out deck width of 120 feet or less. The Bureau of Bridges & Structures should be consulted when considering the closure of an existing longitudinal joint on a superstructure with an out-to-out of deck width greater than 120 feet.

2.6.2 Concrete Removal Limits

The width of the existing deck to be removed will depend on the distance between the beams or girders adjacent to the joint. The width of the removal of the existing concrete deck should be limited so that the deck removal does not extend beyond the centerline of the adjacent beam or girder. When the existing superstructure is a reinforced concrete deck girder, the removal should not extend beyond the stirrup reinforcement bars of the girder adjacent to the existing longitudinal joint. When shear stud connectors are present on the existing beams or girders, the limits of removal should be adjusted so that the remaining and new concrete will adequately encase the shear studs. Care must be exercised so as not to damage the existing studs and/or the top flange of the girders. These requirements are illustrated in Figure 2.6.2-1. When the existing superstructure is a reinforced concrete slab bridge, removal of the edge beam at the longitudinal joint will require temporary shoring and cribbing at the expansion bents if the slab was supported on bearings.

Construction joints should be specified for the joints between the existing concrete deck and the new construction. When a HMA concrete with Full Lane Sealant Waterproofing system will be placed on the deck, the waterproofing membrane should extend a minimum of 6 inches across the construction joint in the deck so it can be overlapped with the existing waterproofing membrane system.

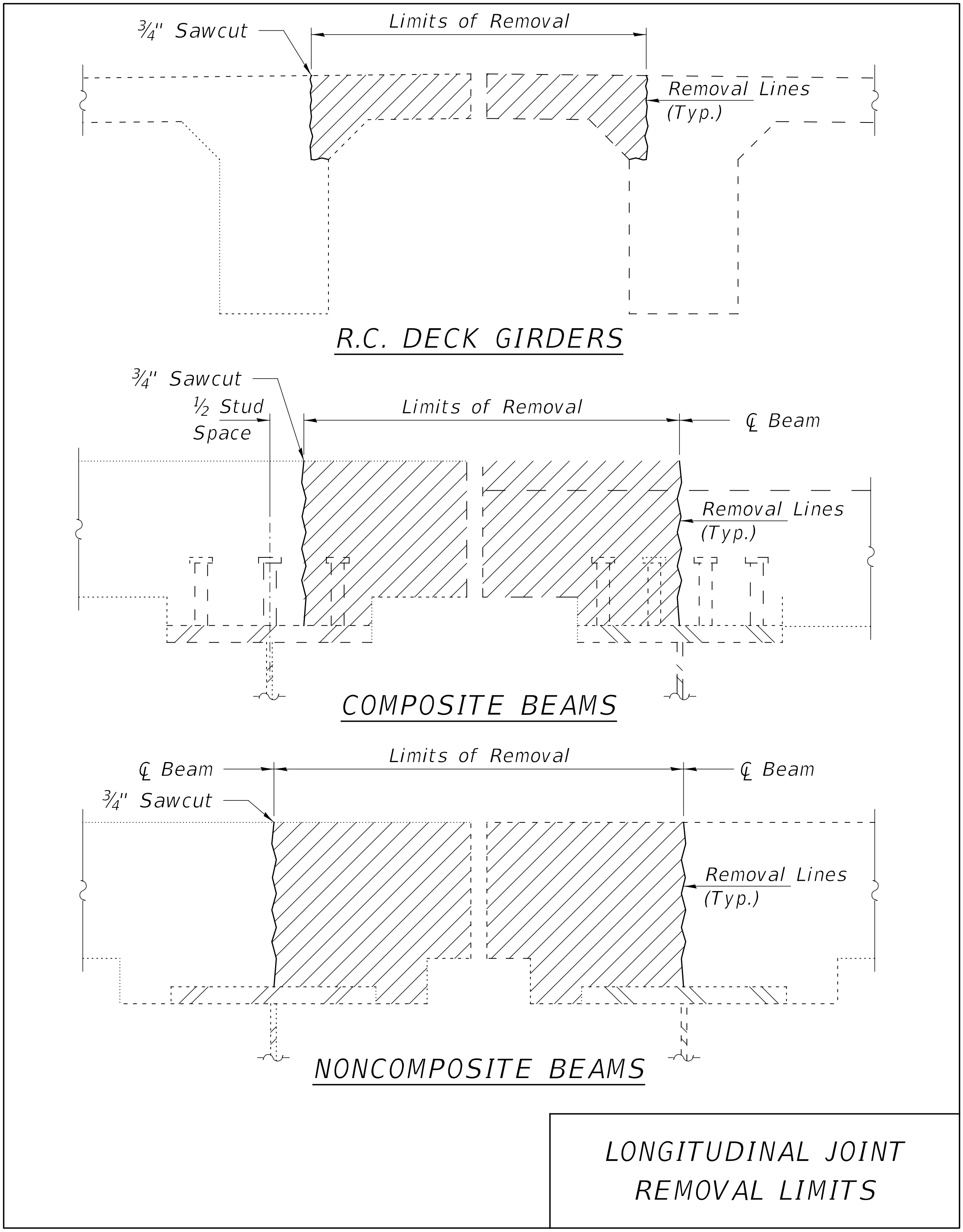


Figure 2.6.2-1: Longitudinal Joint Removal Limits

2.6.3 Reinforcement Bar Treatment

The length of existing transverse reinforcement exposed by the deck removal should be checked to determine if it can be lapped with the new transverse reinforcement bars sufficiently to develop the capacity of the reinforcement bar. When deck removal is limited and the transverse reinforcement bars cannot be lapped to develop the full reinforcement bar capacity, the new bottom transverse reinforcement bars should be hooked at each end.

All exposed existing longitudinal reinforcement bars should be removed and replaced. Existing transverse reinforcement bars should remain in place and should be cleaned and incorporated into the new construction. All new reinforcement bars should be epoxy coated. The typical features to be incorporated in a longitudinal joint closure are shown in Figure 2.6.3-1, Figure 2.6.3-2, and Figure 2.6.3-3 for a project without an existing deck overlay, with and existing HMA concrete overlay and with a proposed overlay respectively. Figure 2.6.3-4 shows a longitudinal joint closure for a reinforced concrete girder bridge without an overlay.

For a reinforced concrete slab bridge, new longitudinal reinforcement should be added to match the size and spacing of the reinforcement of the existing slab section adjacent to the edge beam being removed. See Figure 2.6.3-5.

New concrete should be paid as Concrete Superstructure.

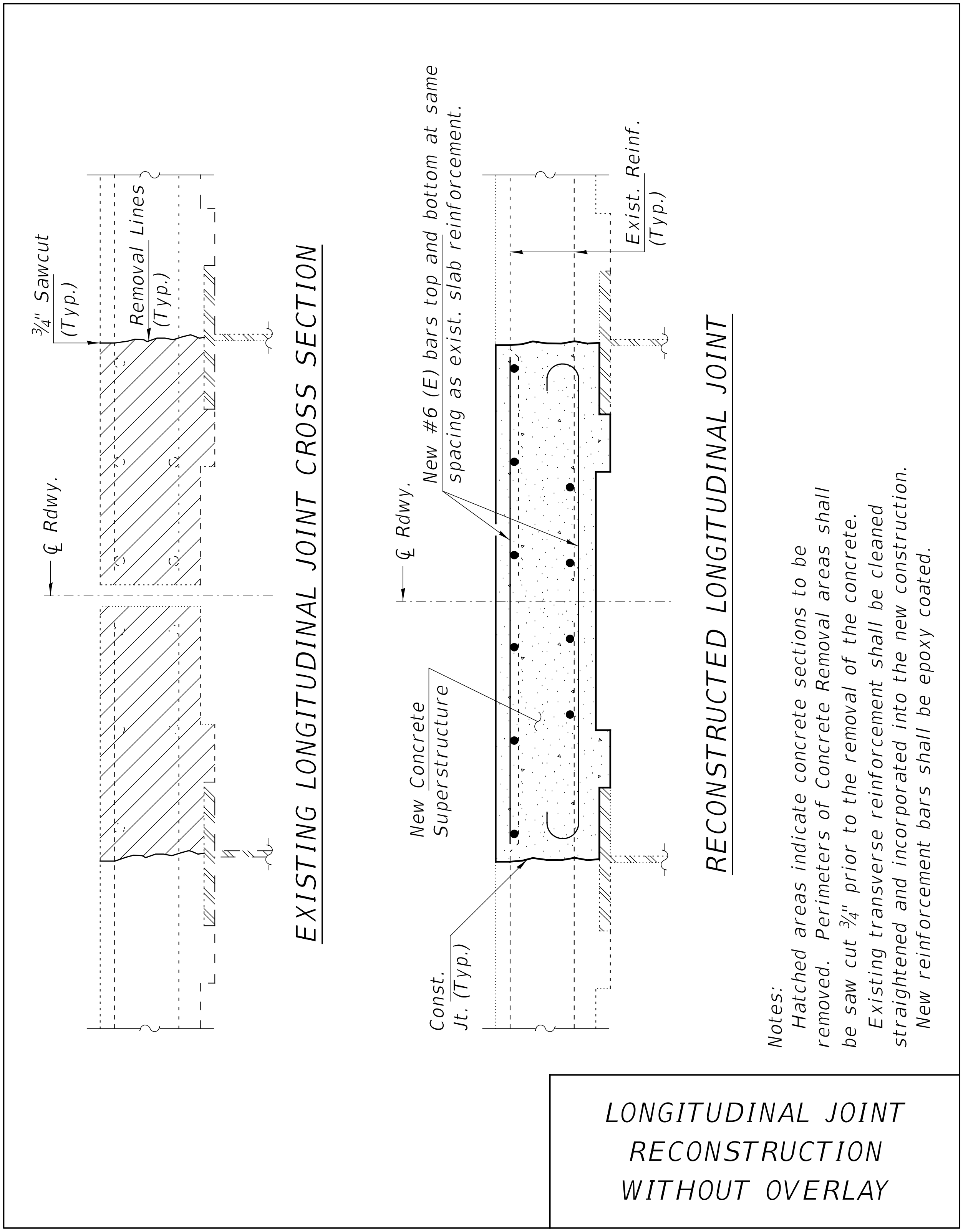


Figure 2.6.3-1: Longitudinal Joint Reconstruction Without Overlay

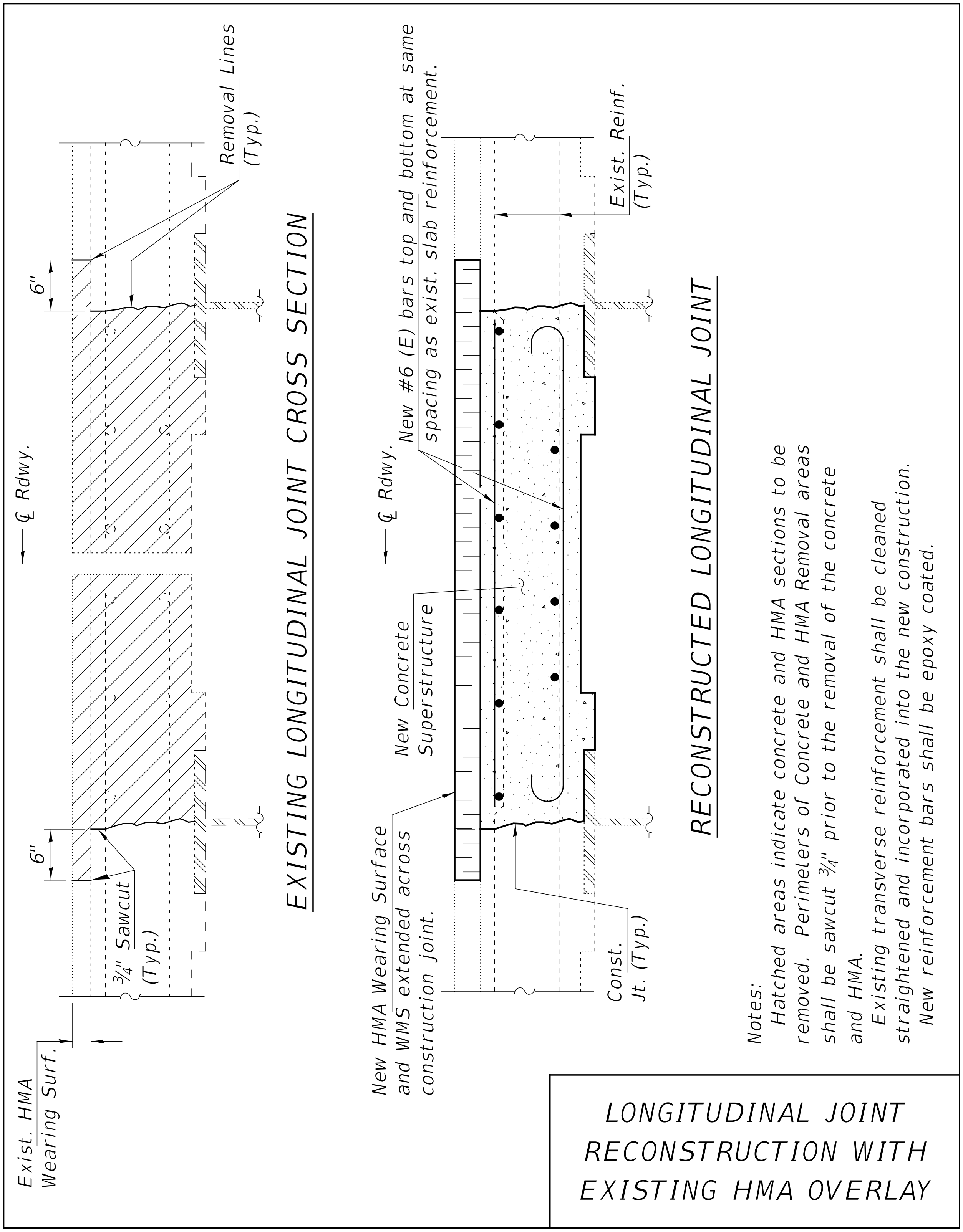


Figure 2.6.3-2: Longitudinal Joint Reconstruction with Existing HMA Overlay

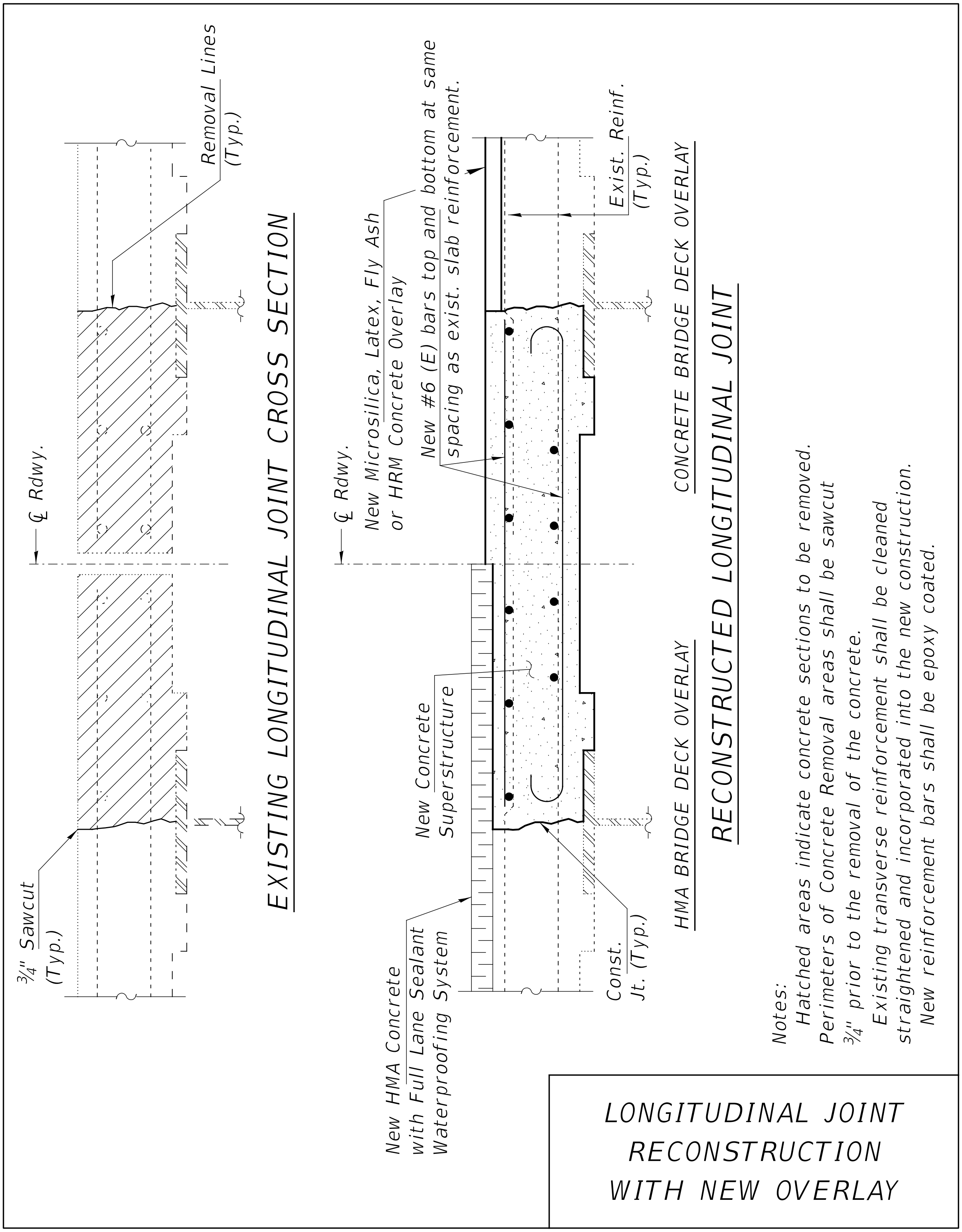


Figure 2.6.3-3: Longitudinal Joint Reconstruction with New Overlay

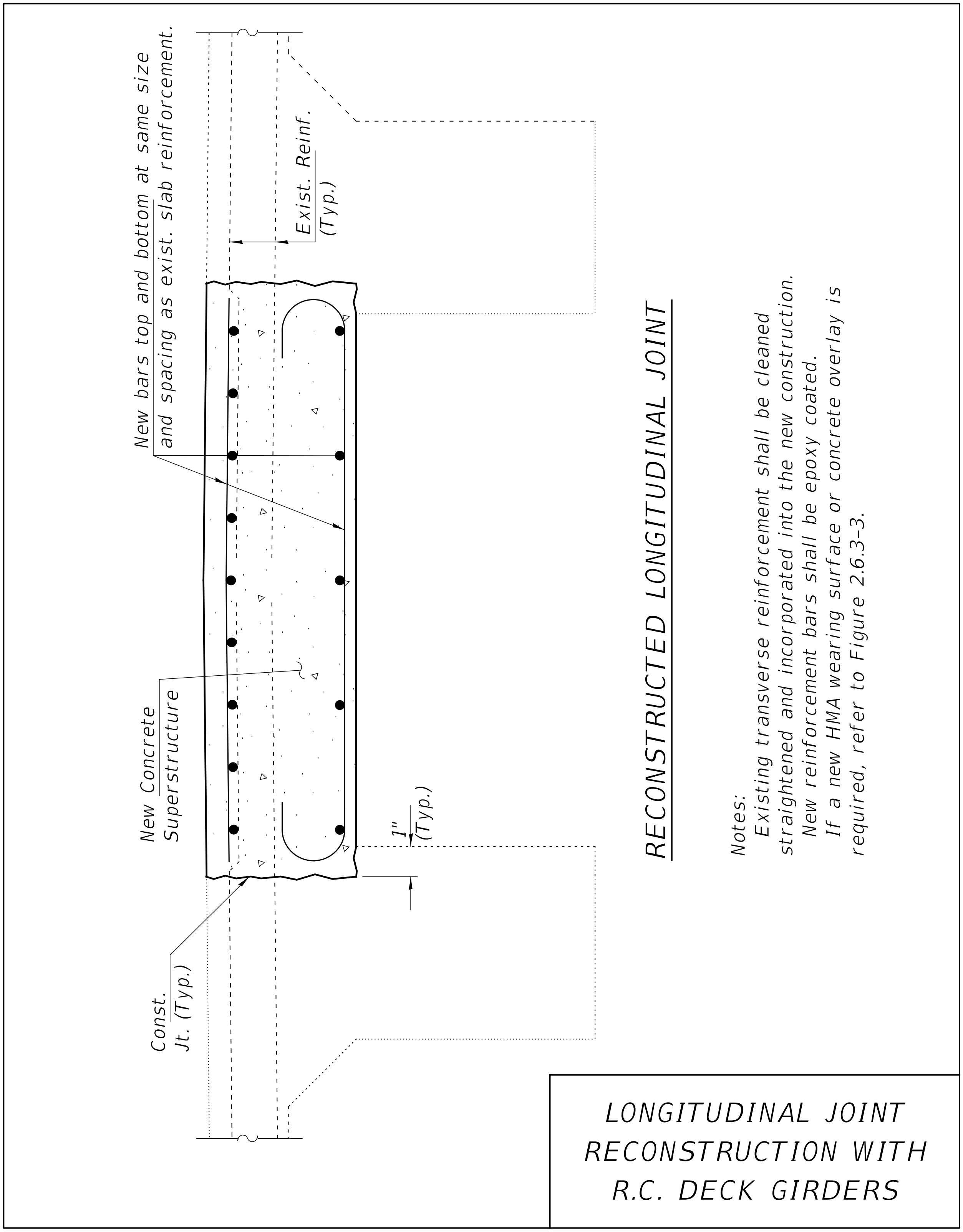


Figure 2.6.3-4: Longitudinal Joint Reconstruction with R.C. Deck Girders

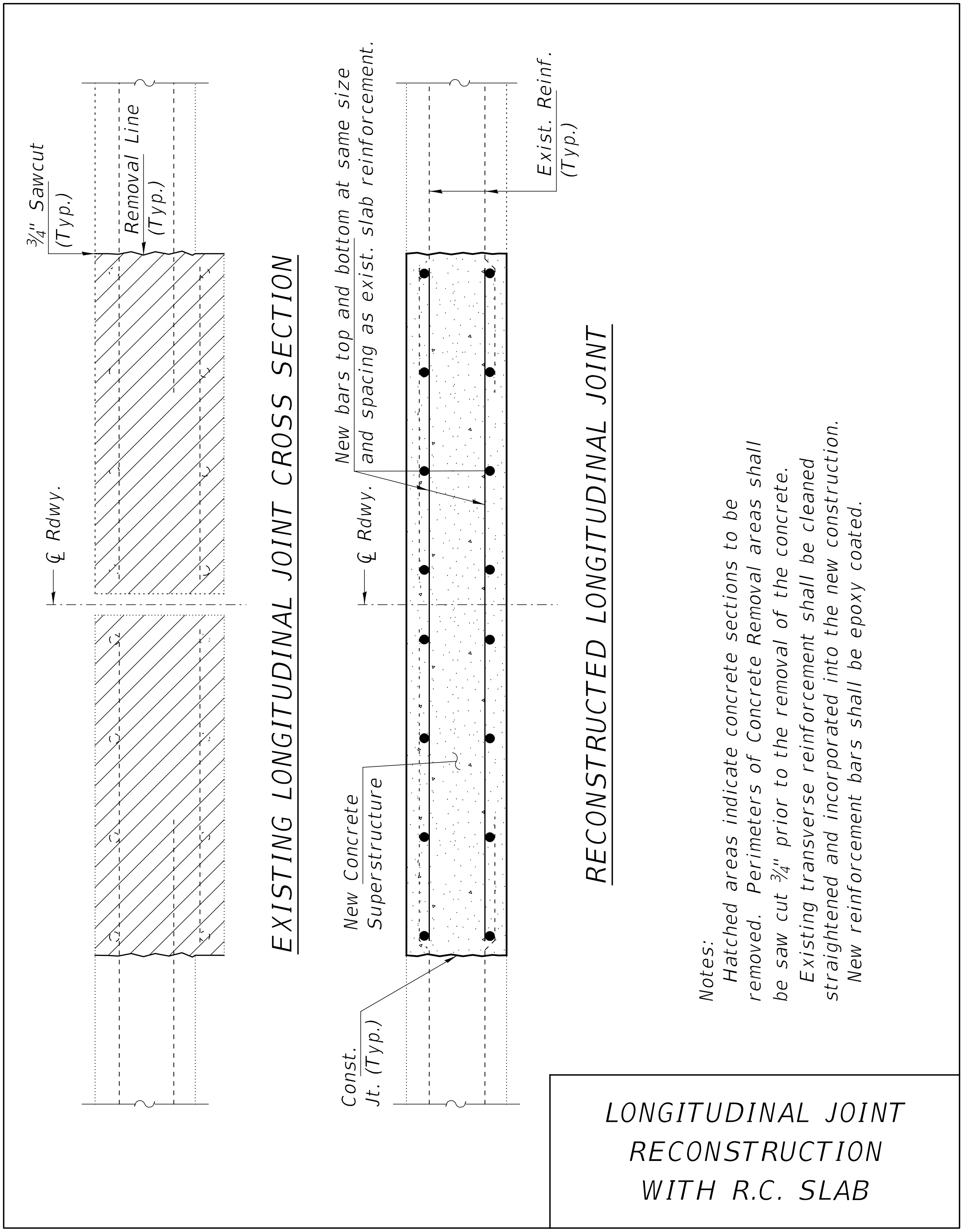


Figure 2.6.3-5: Longitudinal Joint Reconstruction with R.C. Slab

2.6.4 Diaphragms / Crossframes

Diaphragms or cross frames should be installed, when not already in place, between existing steel beams or girders adjacent to the existing longitudinal joint being closed, except that end diaphragms and diaphragms directly over interior piers are not required. When the distance between the beams adjacent to the longitudinal joint is 4 feet or less and traffic on the deck will be far enough away from the longitudinal joint so as not to cause excessive differential live load deflection in the beams during the joint reconstruction, the installation of diaphragms is not required. To ensure a minimum amount of live load deflection in beams without diaphragms adjacent to longitudinal joints, traffic should be kept at least three beam spaces away from the beams adjacent to the longitudinal joint during placement and curing of the closure slab.

The diaphragms or cross frames and their connections should be detailed to account for the installation difficulties presented by small beam or girder spacing and dimensional variations for existing members typically encountered. This typically requires that end attachment angles be field bolted to the diaphragm using slotted or oversized holes during installation and that the flanges of the diaphragms be clipped to allow them to be swung into place between the existing beams. Place proposed diaphragms midway between existing diaphragms whenever possible. If a small skew does not permit midway placement, locate the proposed diaphragms approximately one foot from the centerline of one of the existing diaphragms.

2.7 Bridge Rails and Parapets

2.7.1 Project Requirements

For structures being repaired, damaged portions of the existing bridge rail or parapet should be repaired. Also consideration should be given to installing a crash-tested replacement rail (per the *Manual for Assessing Safety Hardware* (MASH) 2016) on structures with bridge rails which do not meet the current geometric or structural requirements for vehicle induced loadings.

While considering repairs or replacement of railings, the following is recommended:

1. If the damage is small and localized the replacement should be done in-kind.
2. If the entire railing needs to be replaced, and the existing structure can accommodate the connection details and geometric constraints for a MASH 2016 crash-tested railing, then the MASH 2016 crash-tested railing should be used.
3. If the entire railing needs to be replaced, and the existing structure cannot accommodate the connection details for a MASH 2016 crash-tested railing, or there are other geometric constraints precluding the use of a MASH 2016 crash-tested railing, then the railing should be replaced in-kind.

Note that MASH 2016 crash-tested rails are a system that should be used without making any changes to them. Some of the railings have specific requirements for the strength of the concrete to which they are attached. This may preclude their use on old structures. For MASH 2016 crash-tested rails refer to IDOT *Bridge Manual* Section 2.3.6.1.7 and current railing base sheets.

If necessary, the Repairs Unit shall be contacted with questions about the type of rail that will be appropriate for a specific situation.

All replacement rails must be designed to withstand the lateral loads specified in the current edition of the AASHTO *Standard Specifications for Highway Bridges*.

2.7.2 Rail Retrofit on PPC Deck Beams

When a reinforced concrete overlay is placed on a PPC Deck Beam superstructure, the existing bridge rail is often replaced to conform with current bridge rail geometric and structural requirements. A new bridge rail, conforming with current design policies, can be attached to an overlayed PPC Deck Beam superstructure as shown in Figures 2.7.2-1 thru 2.7.2-3 (Type SMX rail, MASH 2016 conditional Test Level 3 pending crash-testing).

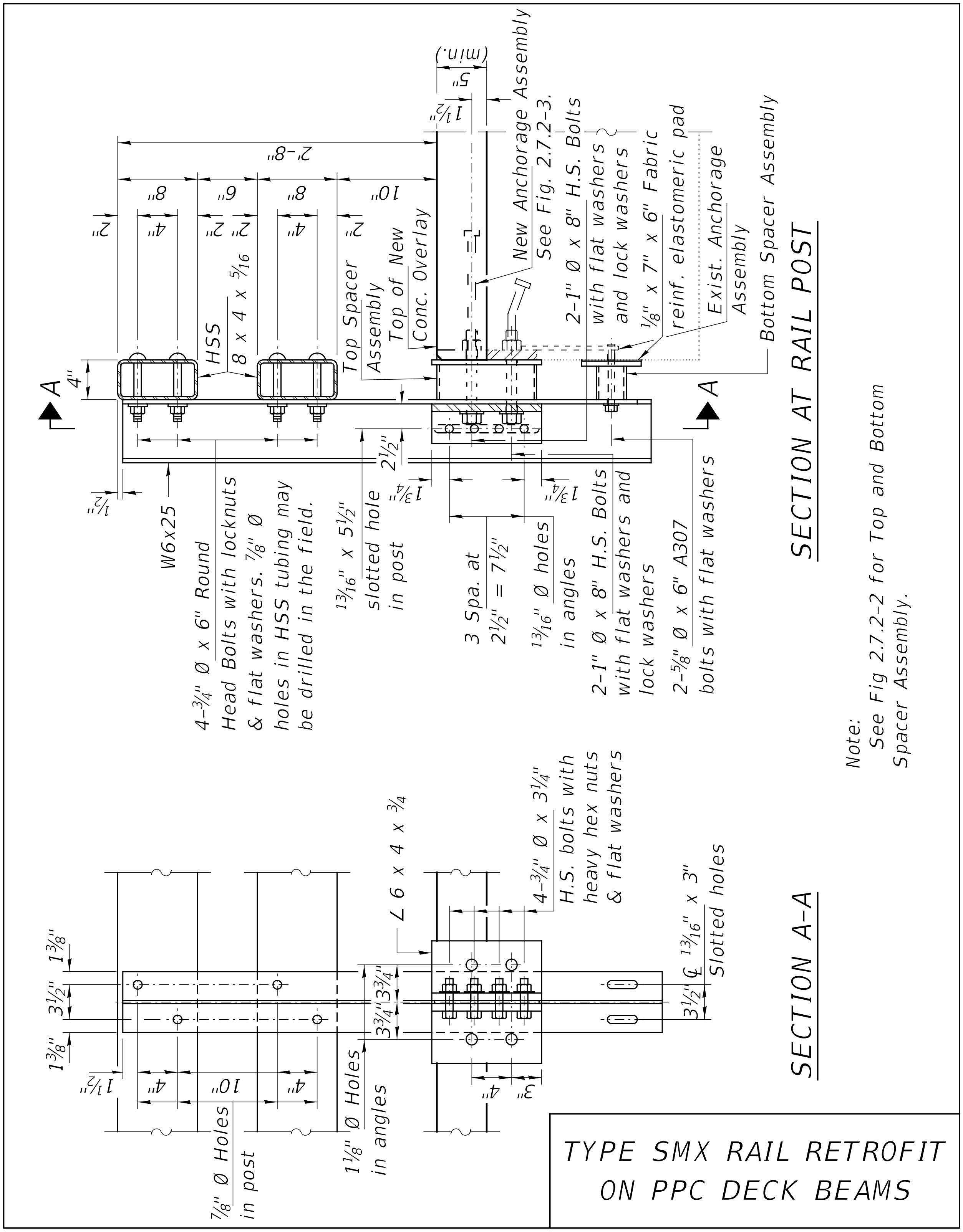


Figure 2.7.2-1: Type SMX Rail Retrofit on PPC Deck Beams

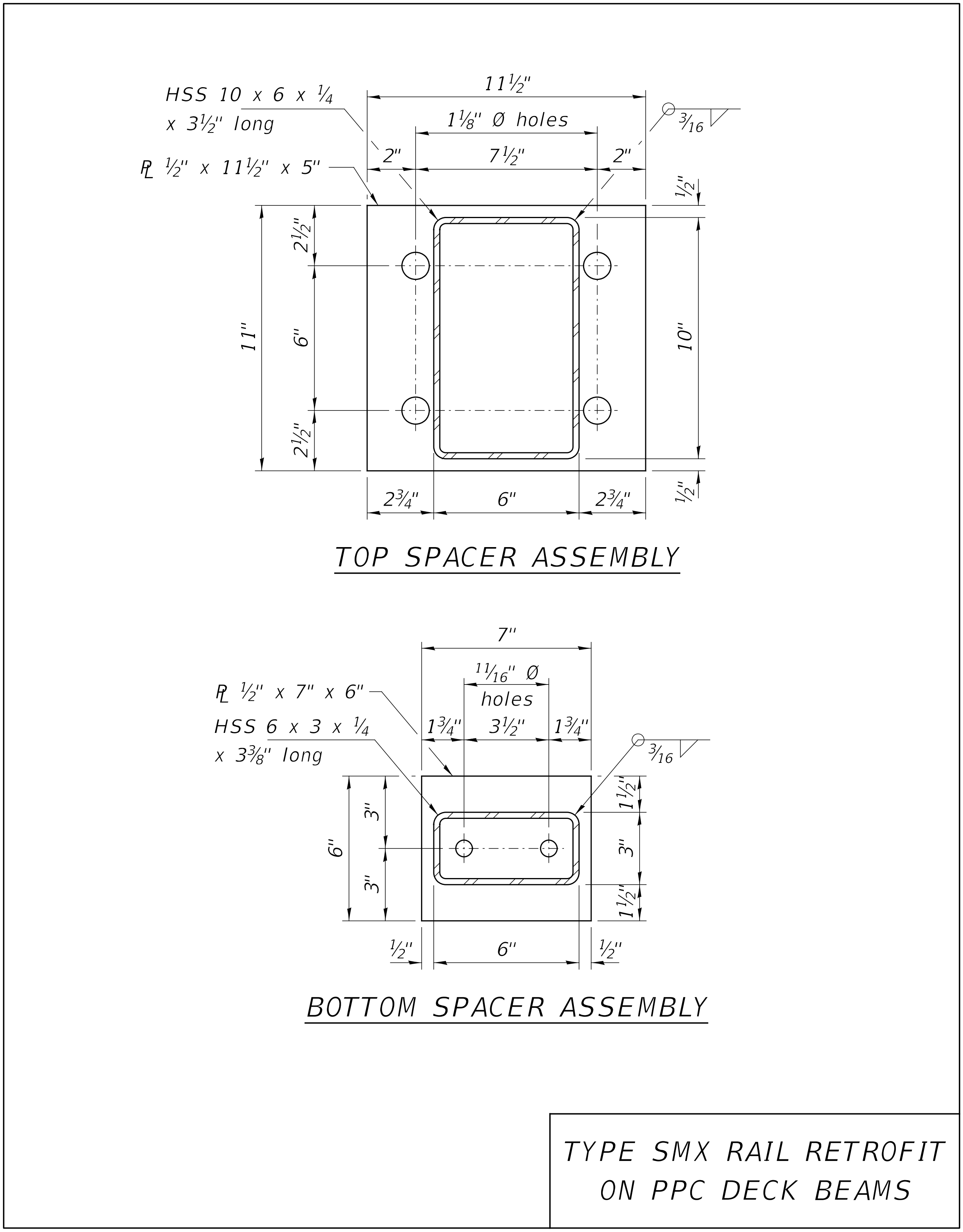


Figure 2.7.2-2: Type SMX Rail Retrofit on PPC Deck Beams

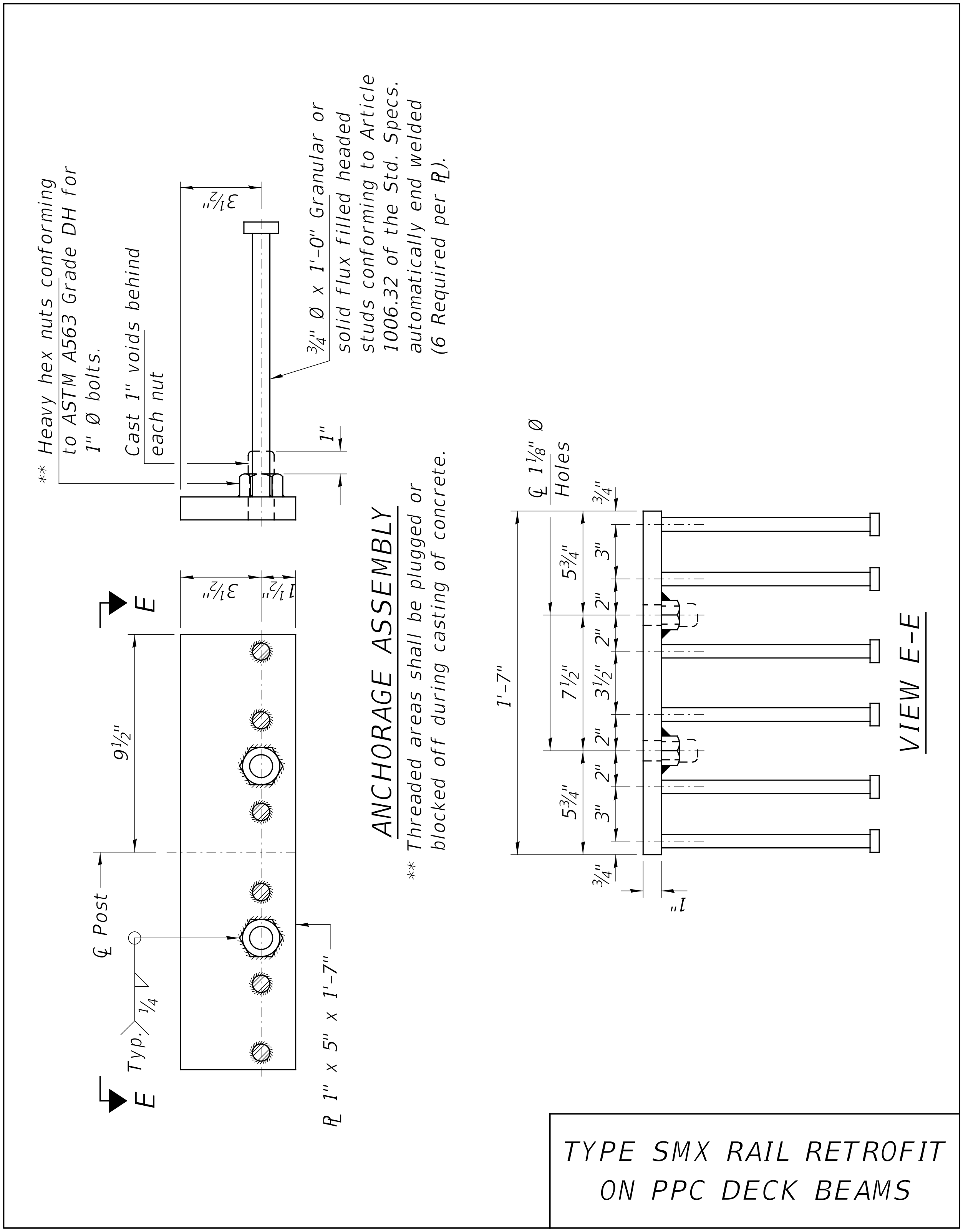


Figure 2.7.2-3: Type SMX Rail Retrofit on PPC Deck Beams

2.7.3 Steel Rail Mounted to Top of Existing Curb

When a decorative concrete rail needs to be retrofitted, the Type CO-10 rail can be used. This rail is mounted to the safety curb in front of the concrete rail. This rail meets MASH 2016 Test Level 4. Details are shown in Figure 2.7.3-1. The rail ends are designed to accept a typical guardrail terminal.

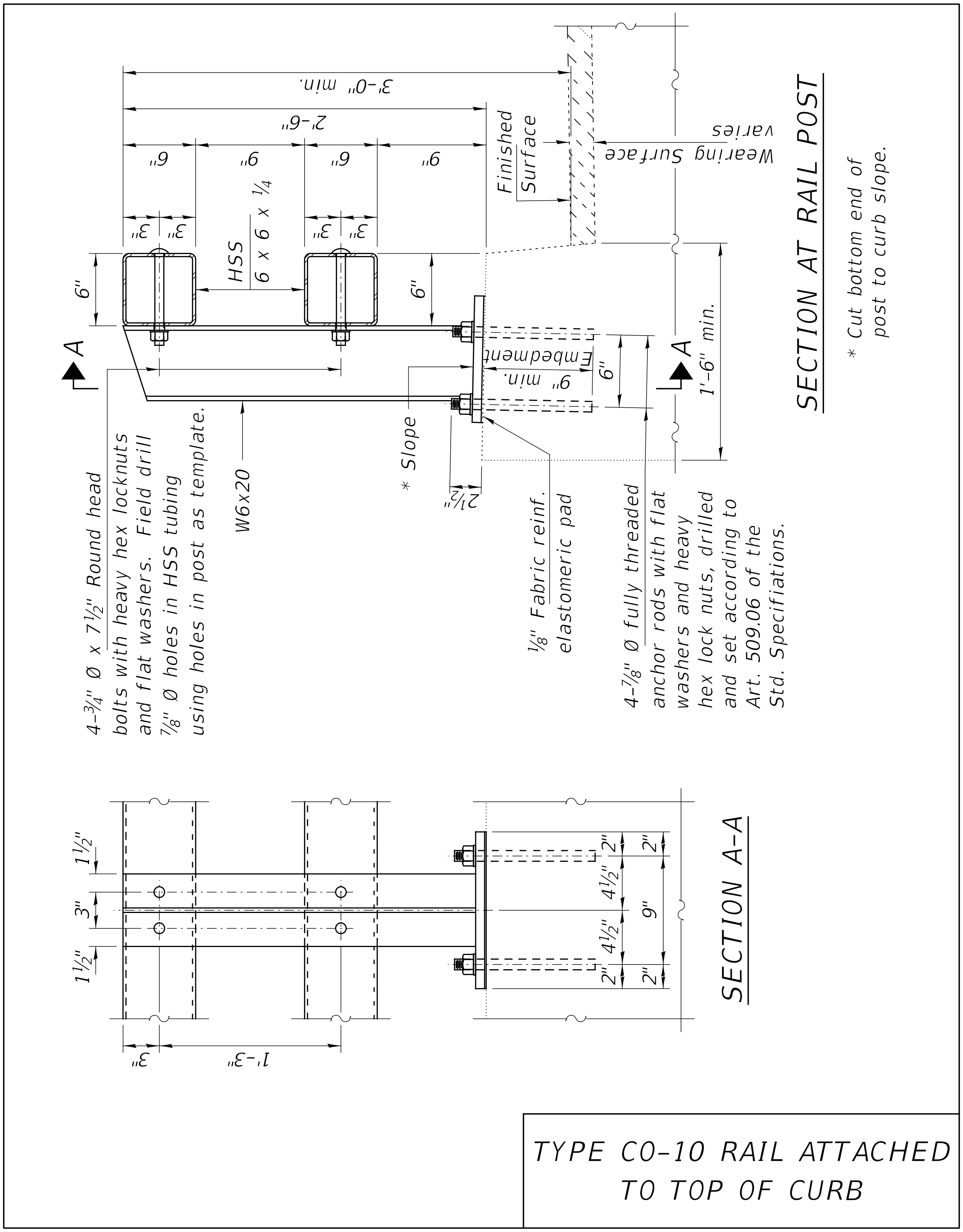
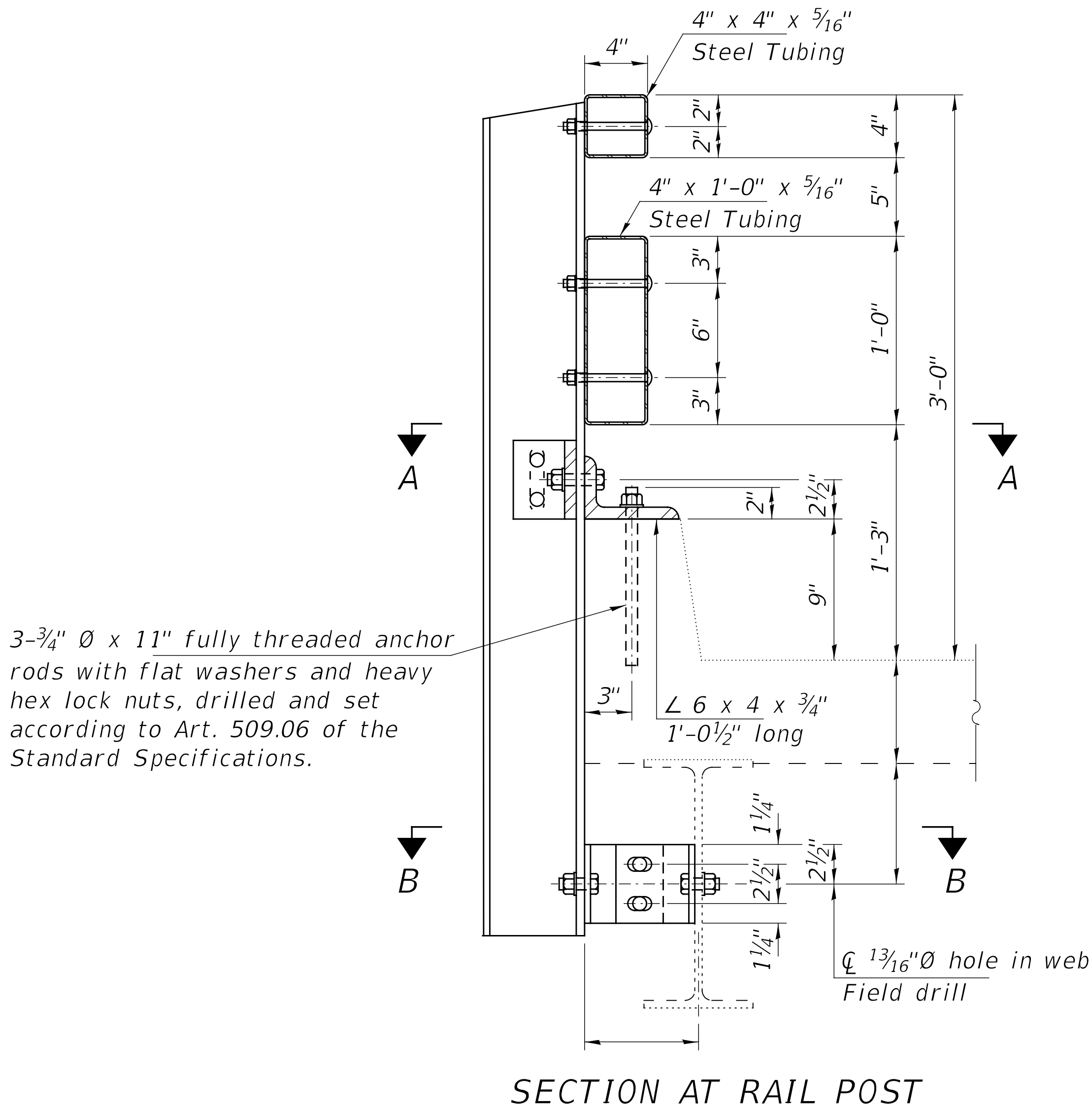


Figure 2.7.3-1: Type CO-10 Rail Attached to Top of Curb

2.7.4 Rail Replacement on Trusses

Attaching a new bridge rail to an existing truss is often done to eliminate an existing damaged or structurally deficient bridge rail. Whenever possible, the post spacing of the new bridge rail should be adjacent to the existing to avoid attaching the new rail posts to the truss members. General details of the procedure are shown in Figure 2.7.4-1 and Figure 2.7.4-2.



Notes:

For Section A-A and B-B see Figure 2.7.4-2.

Nuts for ¾" Ø threaded anchor rods connecting the angle to the concrete shall be tightened to a snug fit and given an additional ⅛ turn.

TRUSS RAIL
REPLACEMENT

Figure 2.7.4-1: Truss Rail Replacement

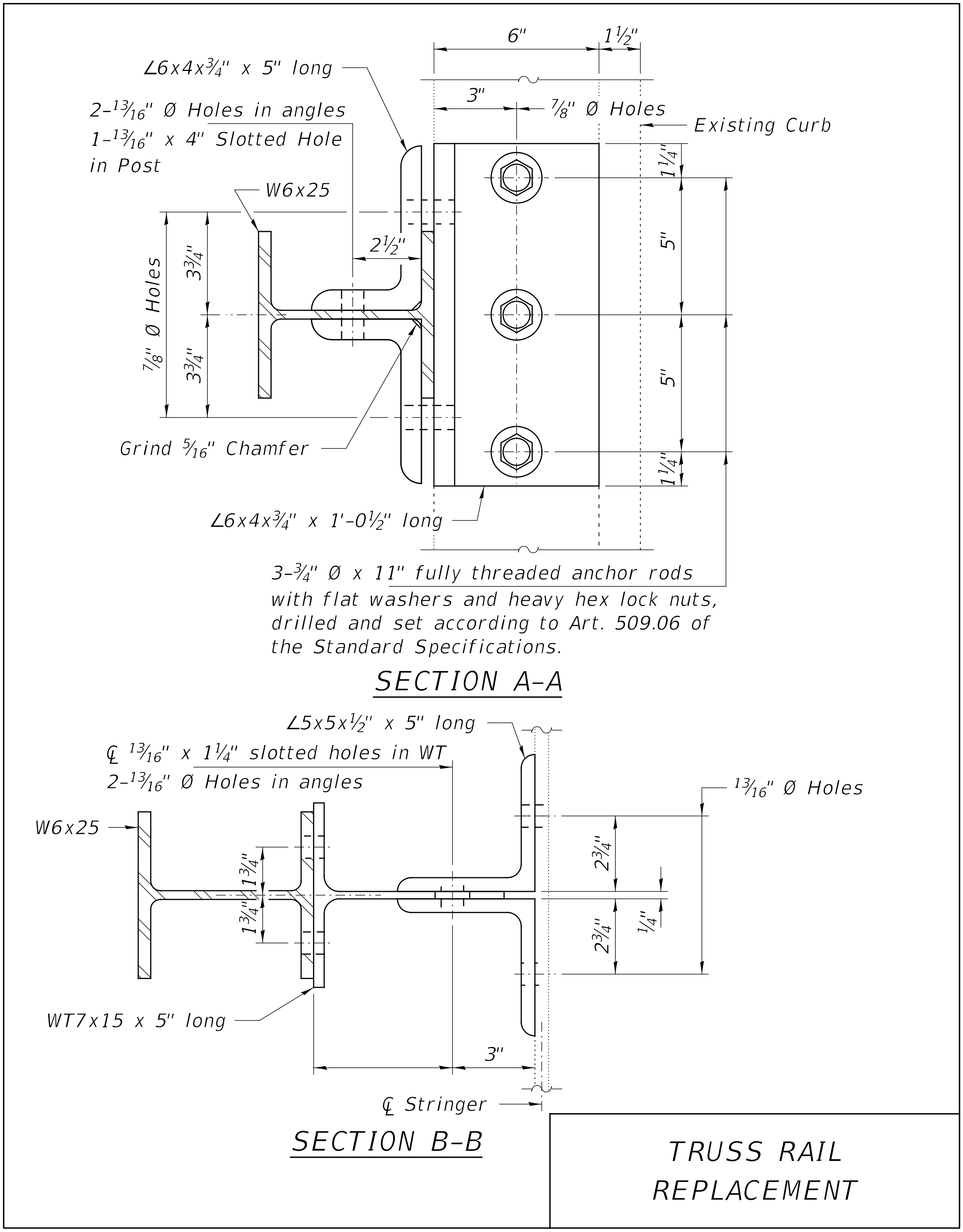


Figure 2.7.4-2: Truss Rail Replacement

2.7.5 Concrete Parapet Extensions

Existing concrete parapets are often reconstructed to meet current height design requirements or to eliminate existing aluminum rail elements. This usually involves the removal of existing aluminum rail sections from the top of an existing parapet and the addition of a section of concrete parapet to the top of the existing parapet. The section of parapet added can preferably be cast-in-place as shown in Figure 2.7.5-1 or alternatively precast as shown in Figure 2.7.5-2.

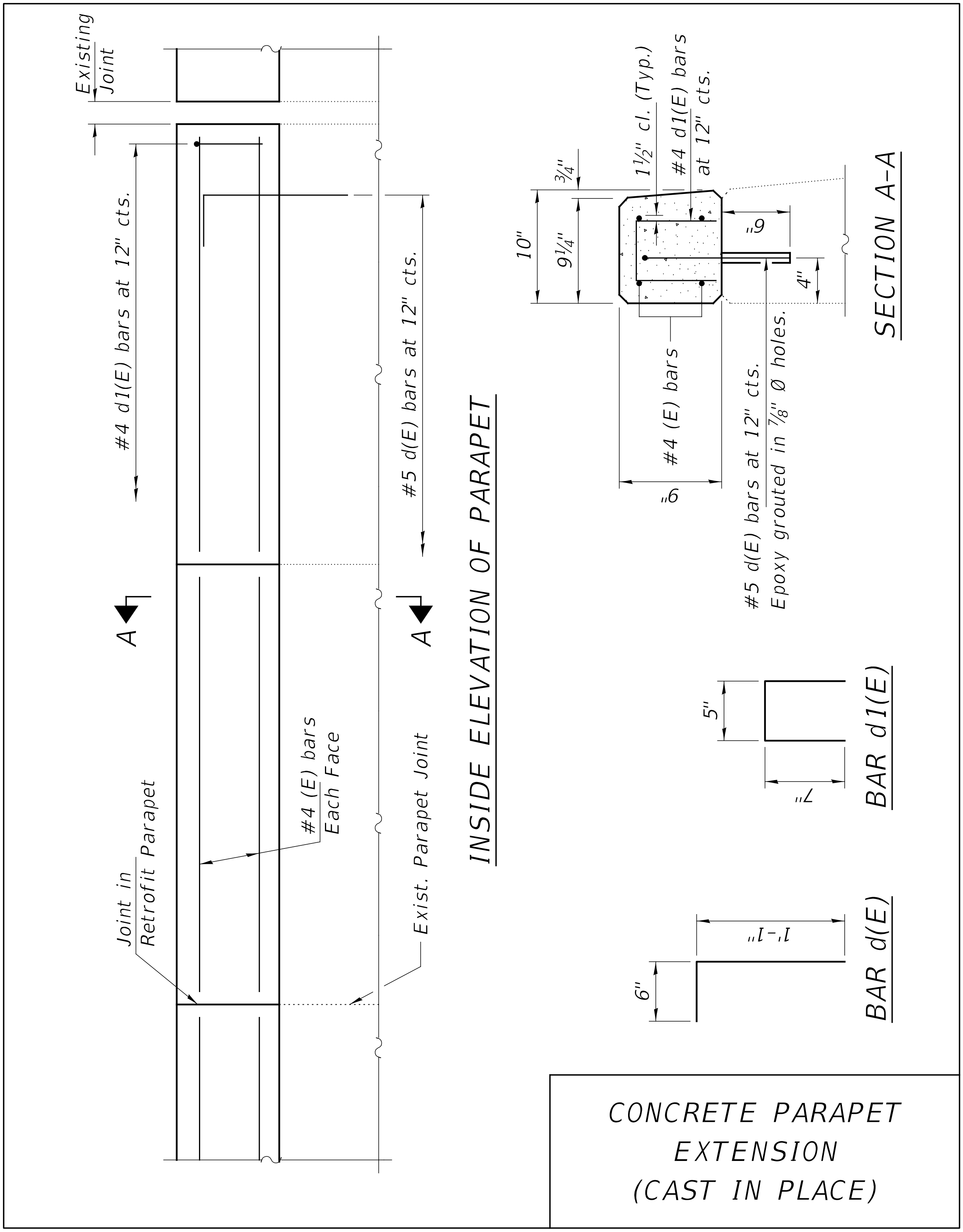


Figure 2.7.5-1: Concrete Parapet Extension (Cast in Place)

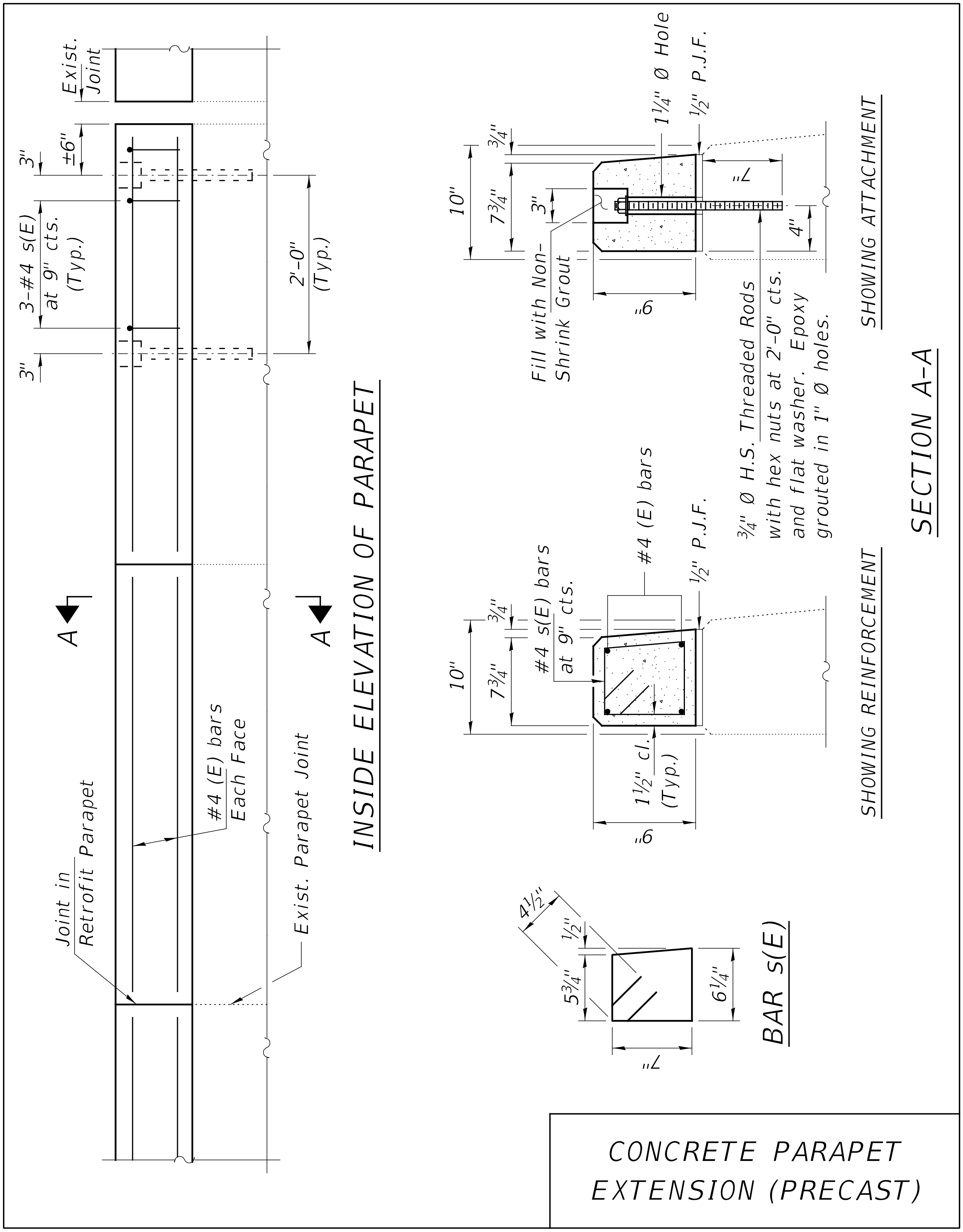


Figure 2.7.5-2: Concrete Parapet Extension (Precast)

2.7.6 Wingwall/Parapet and Railing Retrofit

In order to attach the guardrail terminal to the wingwall it is often necessary to modify the existing end of the wingwall and railing. The modifications are intended to a) provide a solid connection between the guardrail terminal and the wingwall and b) provide a smooth transition between the roadway guardrail and the bridge without any snagging points.

General details have been developed to handle these transitions on several types of wingwall configurations: those in which the wingwall is parallel to the centerline of roadway are shown in Figures 2.7.6-1 through 2.7.6-3, those in which the wingwall is stepped are shown in Figure 2.7.6-4, and those in which the wingwall is skewed away from the centerline of roadway are shown in Figures 2.7.6-5 through 2.7.6-7.

Because of the diversity of configurations found in the field the designer must adjust dimensions to match existing conditions. The designer must ensure that a vehicle “leaning” over the rail terminal will not snag a corner of the wingwall.

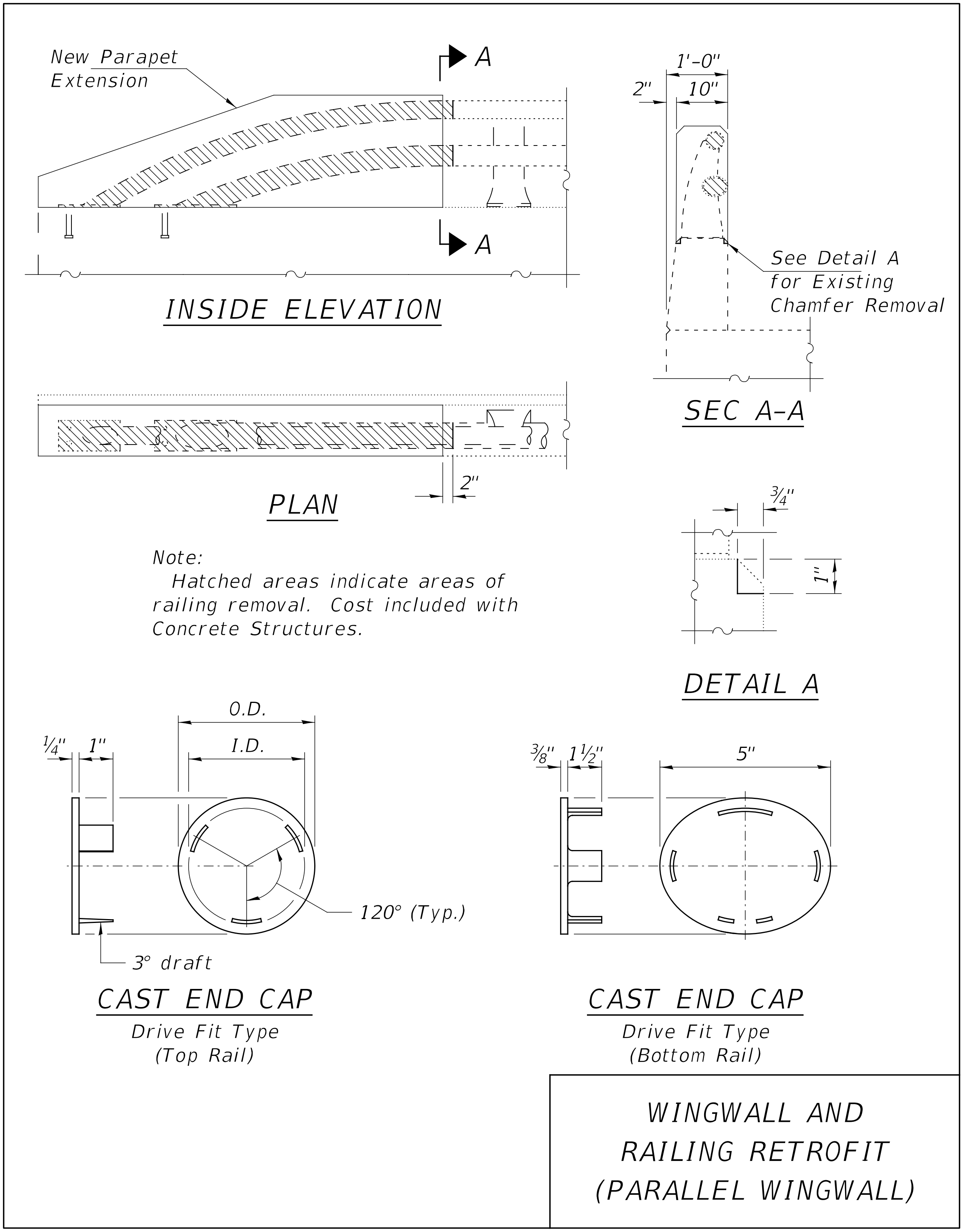


Figure 2.7.6-1: Wingwall and Railing Retrofit (Parallel Wingwall)

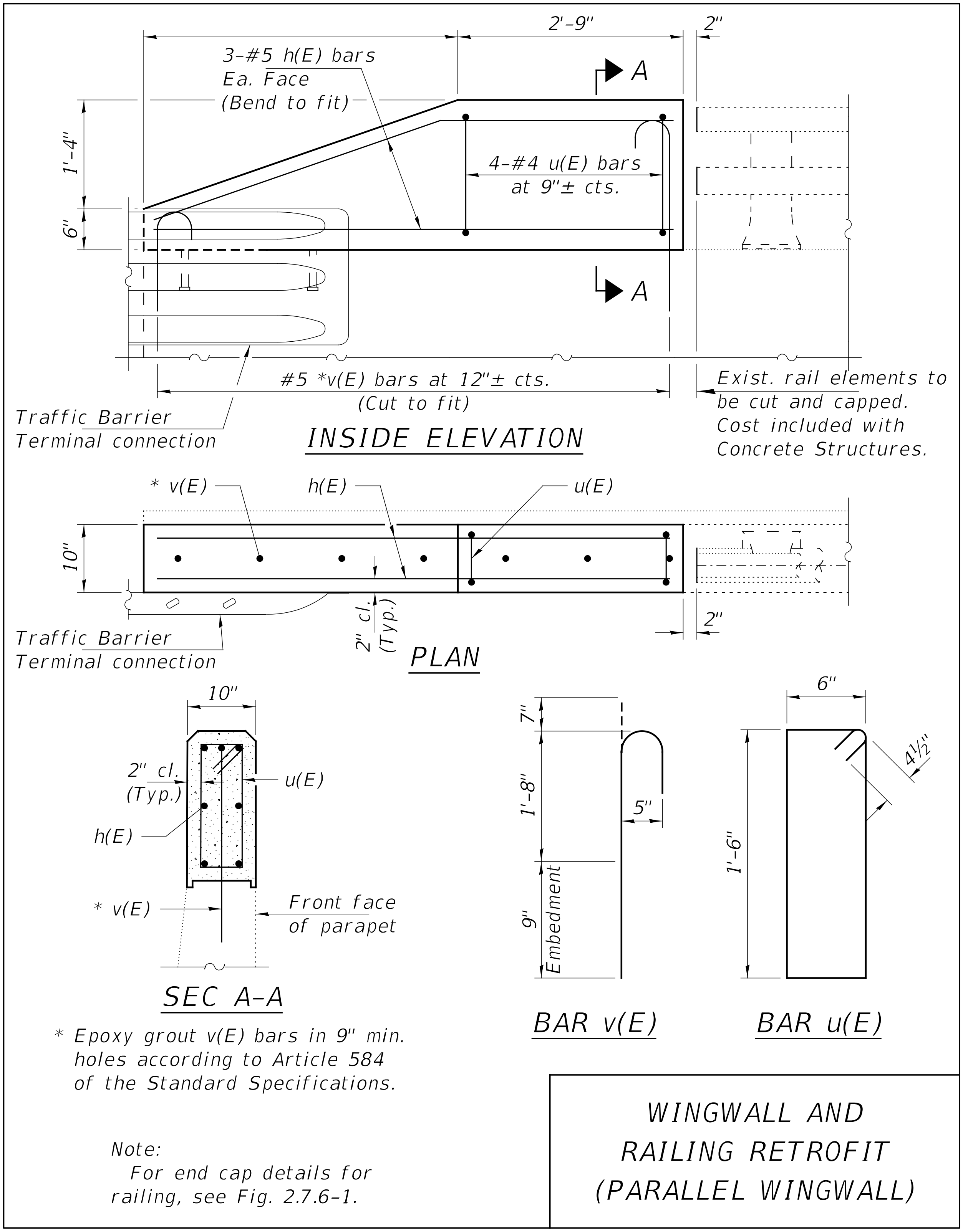


Figure 2.7.6-2: Wingwall and Railing Retrofit (Parallel Wingwall)

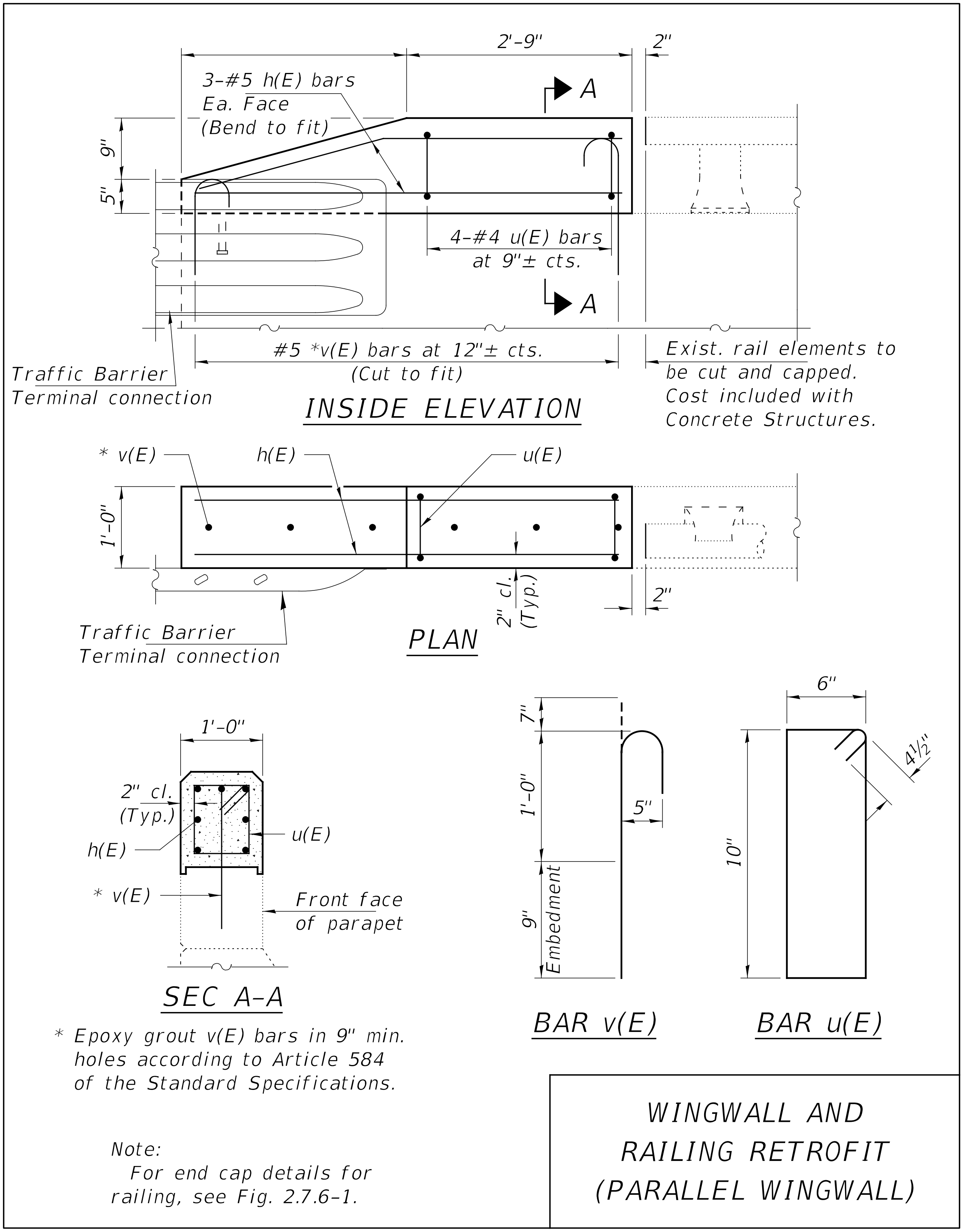


Figure 2.7.6-3: Wingwall and Railing Retrofit (Parallel Wingwall)

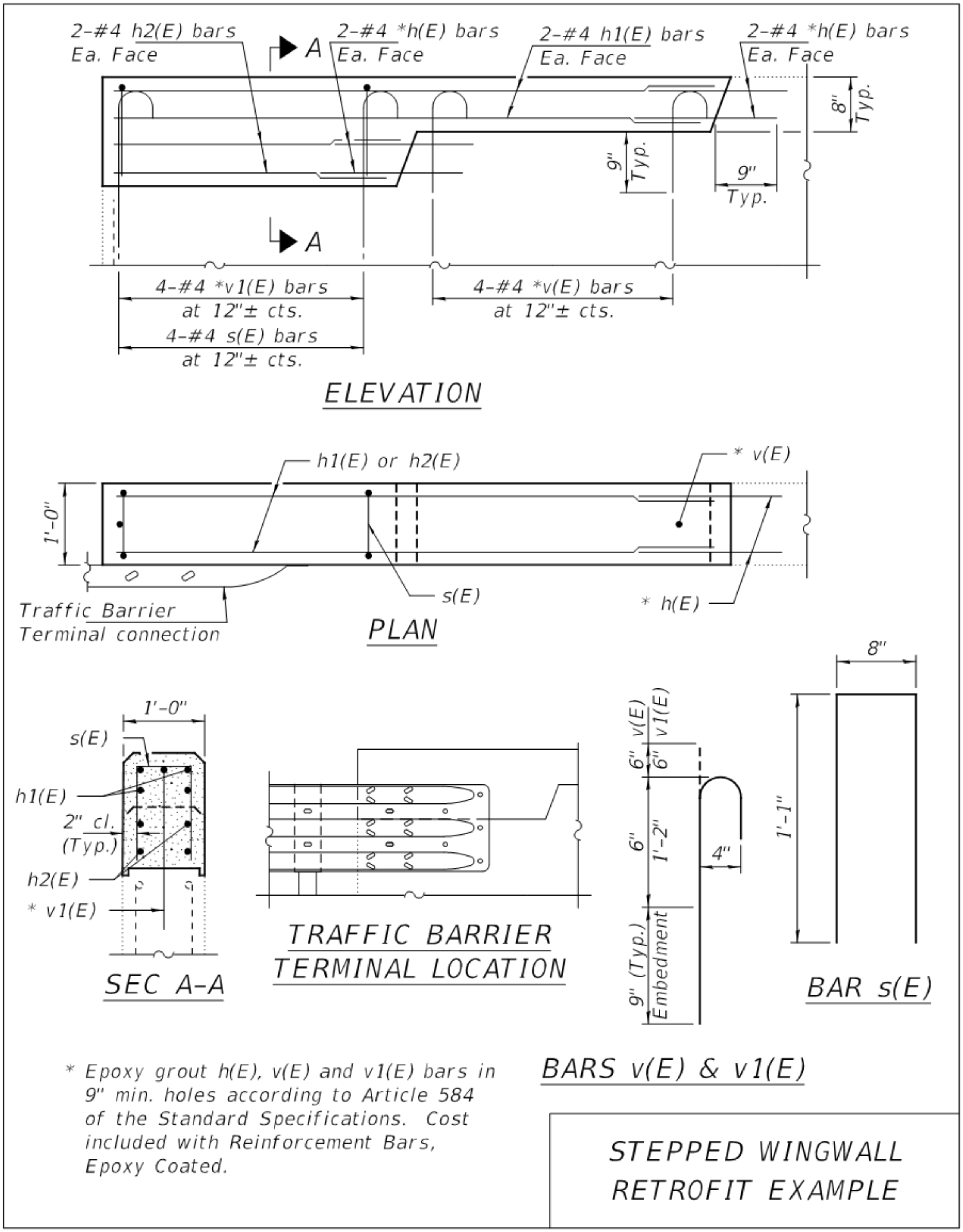


Figure 2.7.6-4: Example – Stepped Wingwall Retrofit

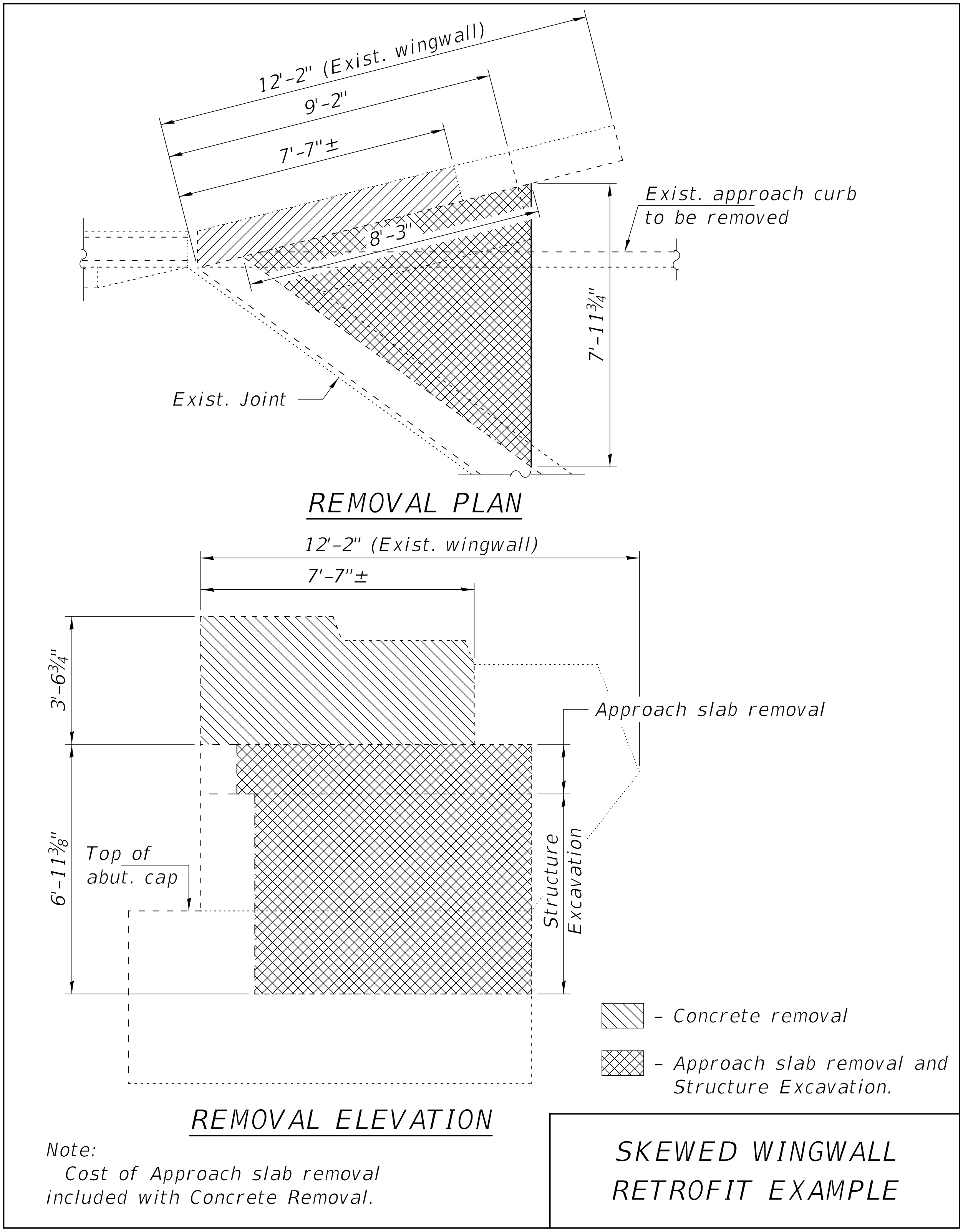


Figure 2.7.6-5: Example – Skewed Wingwall Retrofit

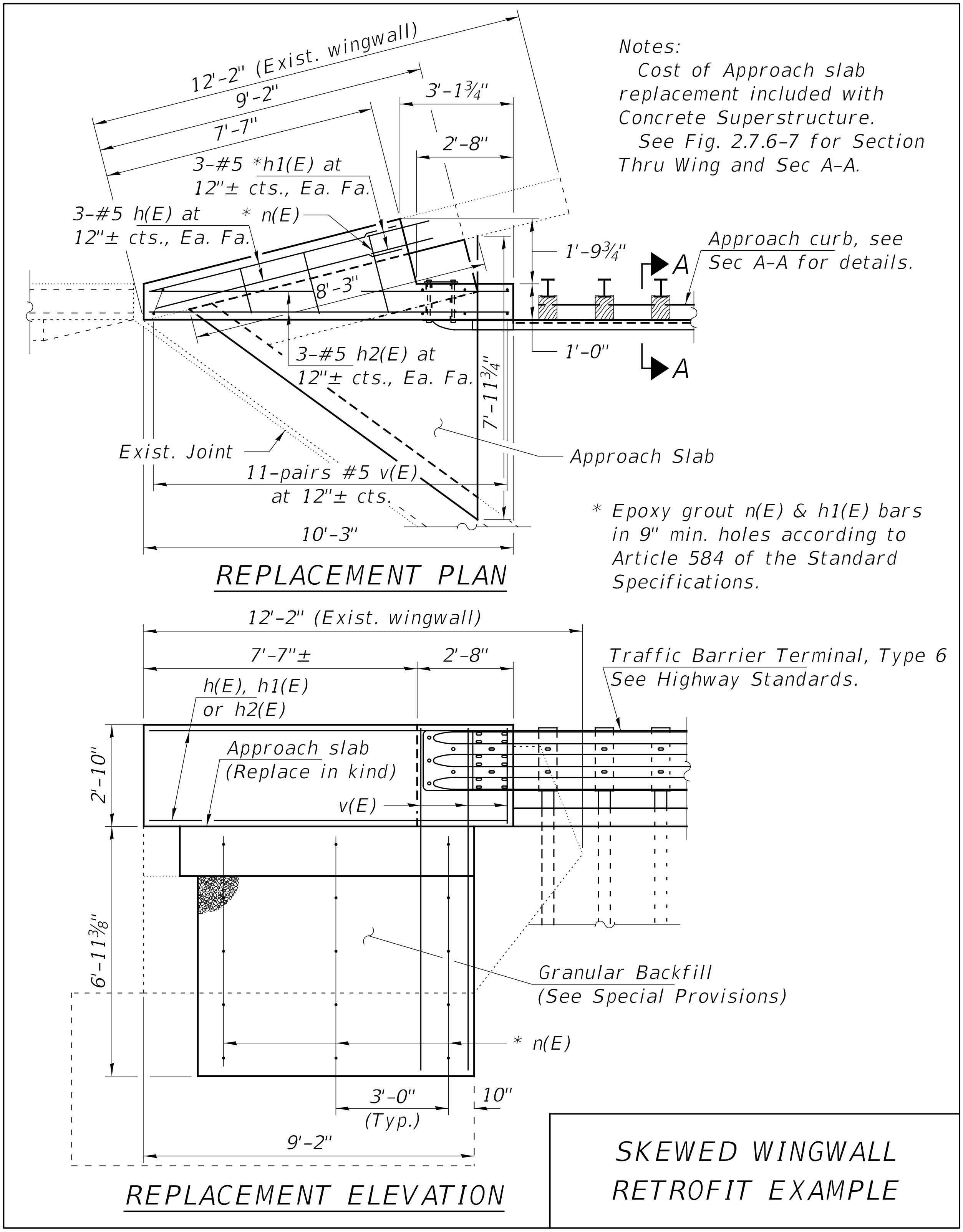
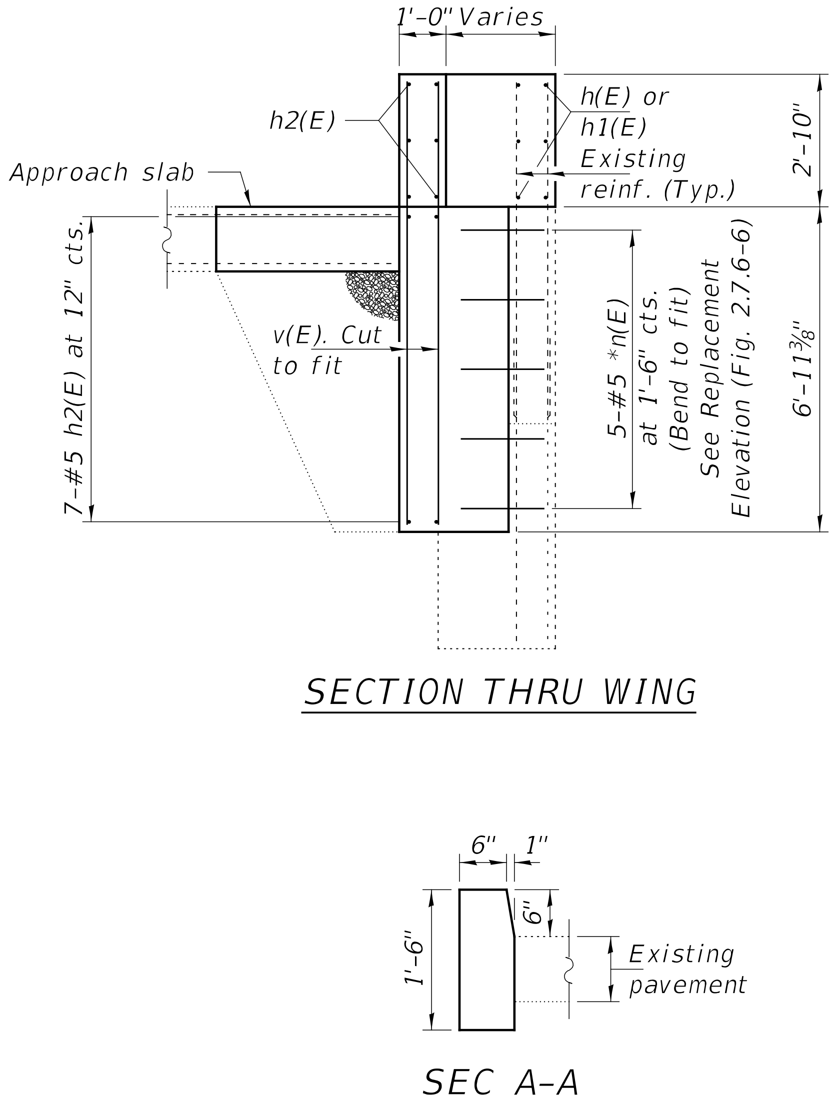


Figure 2.7.6-6: Example – Skewed Wingwall Retrofit



* Epoxy grout n(E) & h1(E) bars in 9" min. holes according to Article 584 of the Standard Specifications.

SKEWED WINGWALL
RETROFIT EXAMPLE

Figure 2.7.6-7: Example – Skewed Wingwall Retrofit

2.7.7 Crashwall Extensions

In order to eliminate any snagging points at columns on piers near traffic lanes and in accordance with AASHTO recommendations, the crashwall shall be extended to a minimum of 5 feet above the finished grade. Note that this increase in crashwall height is not intended to increase the structural capacity of the pier, so an analysis of the pier for crash loading is not required. This work should be completed while other repair work is being done on the structure or while a roadway improvement is being completed around the piers.

The District must provide existing crashwall heights at all locations being considered and extensions should be detailed to reach the desired height. See Figure 2.7.7-1 for details that can be used on a pier with trapezoidal columns. Details for a pier with circular columns are shown in Figure 2.7.7-2.

If other smaller crashwall extensions have been previously constructed, they should be removed to the original crashwall level and new extensions should be anchored into original concrete.

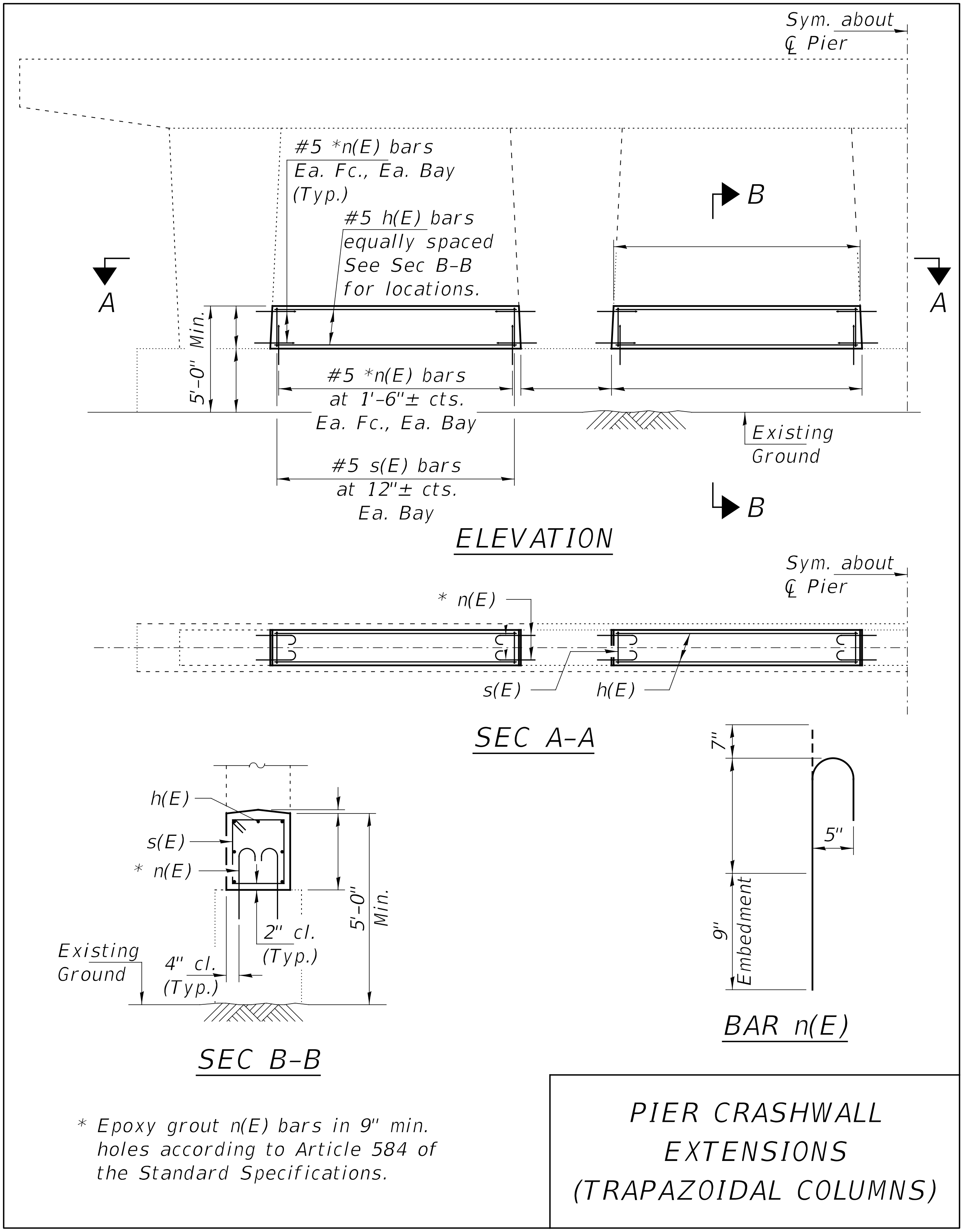


Figure 2.7.7-1: Pier Crashwall Extensions (Trapezoidal Columns)

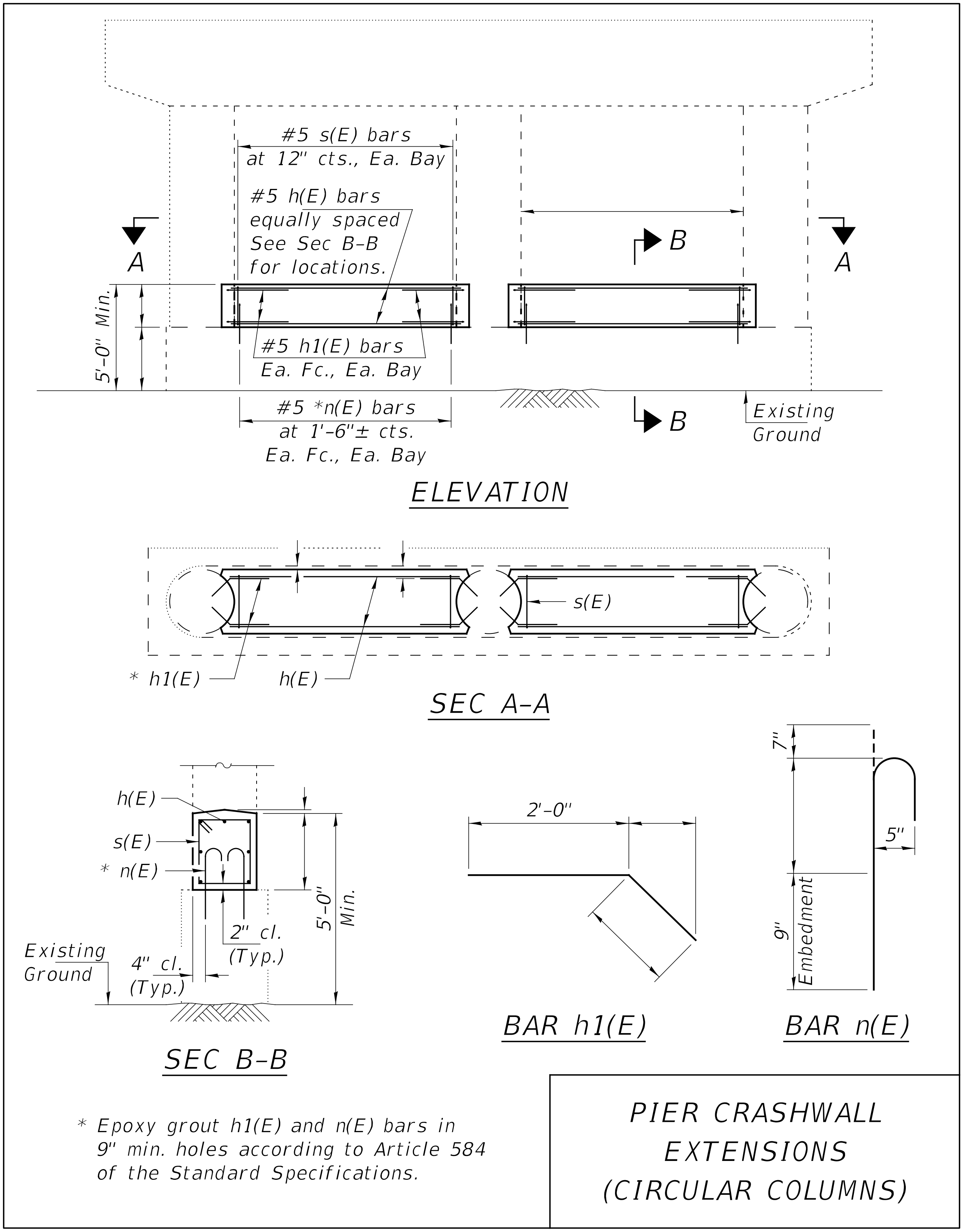


Figure 2.7.7-2: Pier Crashwall Extensions (Circular Columns)

2.7.8 Median Parapet Transition

Transition sections connecting a new median parapet to an existing pier have been developed. The details are site specific because of the need to accommodate the different shapes of the pier end to which the median parapet will be connected. The transition length is a function of the difference in width of the pier and the median parapet. This type of work is typically done in conjunction with a crash wall extension as described in Section 2.7.7. Typical details are shown in Figure 2.7.8-1.

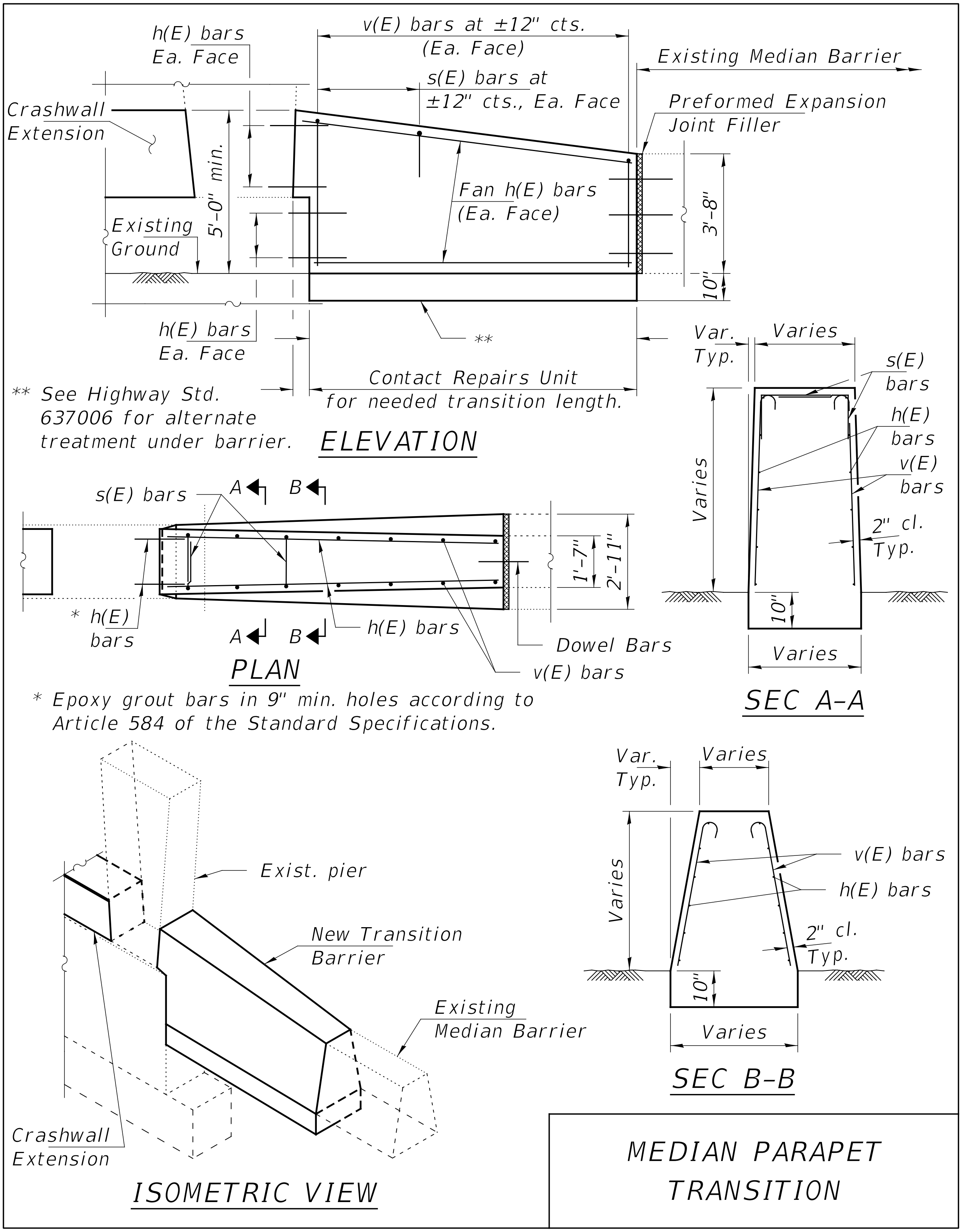


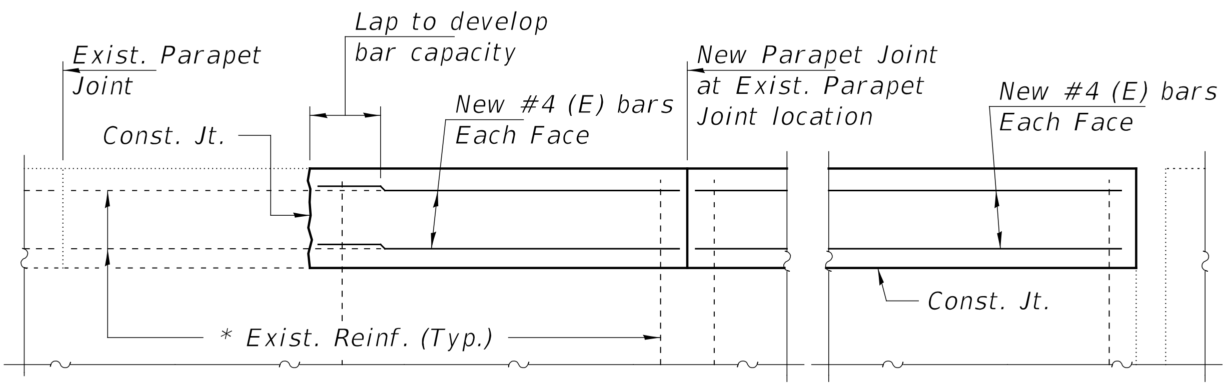
Figure 2.7.8-1: Median Parapet Transition

2.7.9 Parapet Repairs

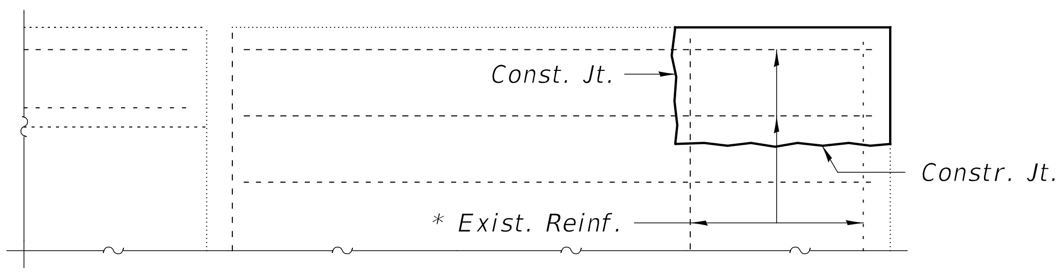
When repairing a section of parapet damaged by a vehicle impact or natural deterioration, the following procedures should be used:

1. Extend the concrete removal for the repair to an existing parapet joint whenever possible or saw cut the perimeter of the damaged area 3/4 inch deep.
2. Salvage, clean and incorporate all existing vertical reinforcement bars into the new construction.
3. Remove and replace all existing horizontal reinforcement bars in the area of concrete removal when a significant length of the bars has been exposed and there is a probability that the bars will be damaged during the concrete removal operations.
4. Except at existing parapet joints, new horizontal reinforcement bars should be lapped with existing horizontal reinforcement bars sufficiently to develop the strength of the bar.
5. Existing parapet joint spacing and features should be utilized in the repair.

See Figure 2.7.9-1 for parapet repair details.



INSIDE ELEVATION OF PARAPET



INSIDE ELEVATION OF WINGWALL

Note:
Reinforcement extending into removed area shall be cleaned, straightened and incorporated into the new construction.

PARAPET AND WINGWALL REPAIR DETAILS

Figure 2.7.9-1: Parapet and Wingwall Repair Details

2.8 Deck Drains

2.8.1 General

Many existing structures were constructed with deck drains which direct drainage onto the main load carrying members of the superstructure or onto the substructure units which can, over time, cause significant damage. Also, the number of deck drains present on many existing structures greatly exceeds the number of drains necessary to adequately control the ponding of water on the deck surface. Repair plans for major maintenance projects on the deck surface of a structure, such as overlay projects, should contain details to provide for the adjustment or elimination of existing deck drains. Also, when the low point of a vertical sag curve is located on the structure and there is no existing drainage scupper at the low point, major deck maintenance projects should provide for the installation of a drainage scupper.

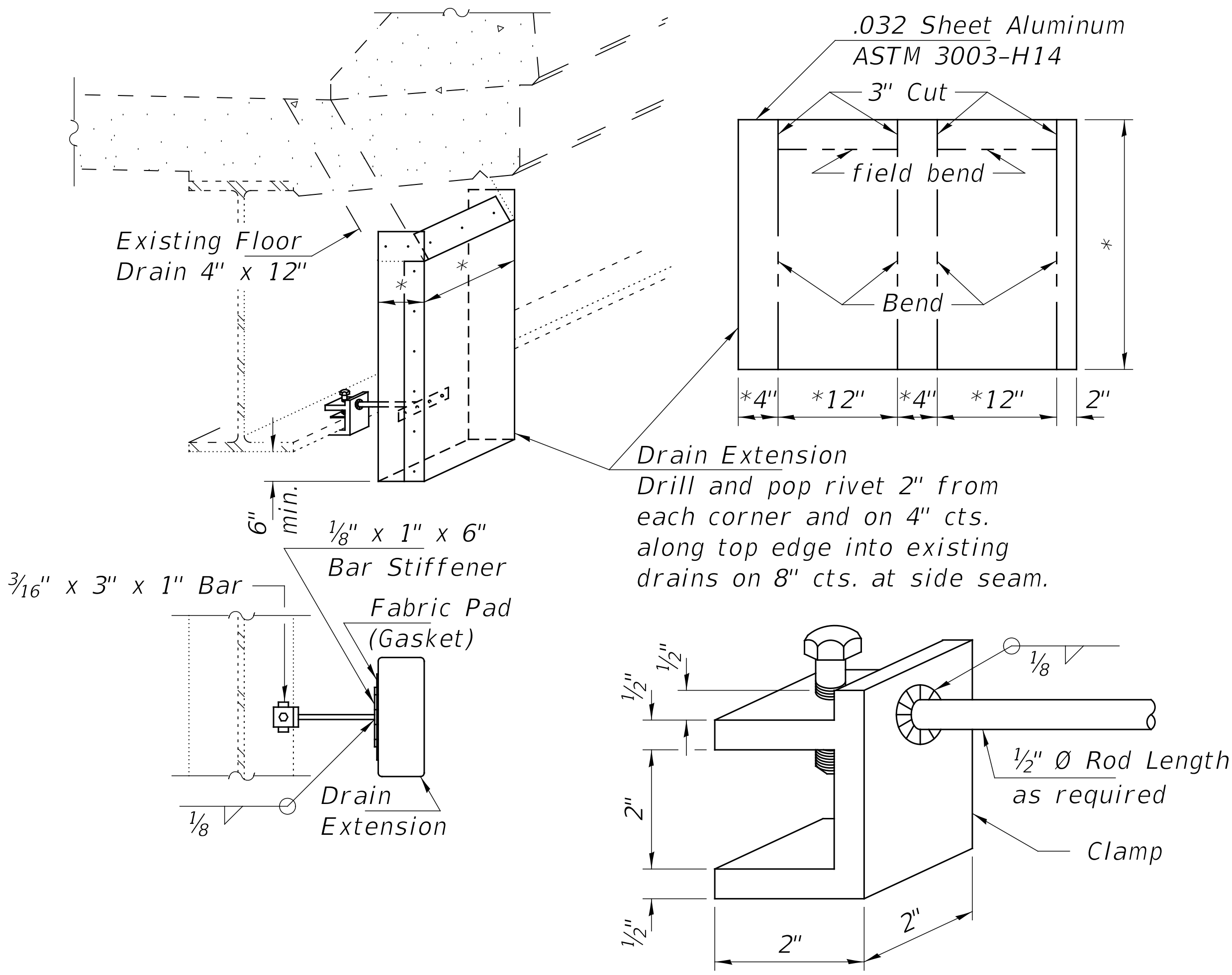
2.8.2 Drain Extensions

When existing deck drains are to remain in place, adjustments must be made to prevent the discharge of deck drainage onto the superstructure elements. Existing aluminum drains may be extended using bent aluminum sheets as shown in Figure 2.8.2-1 and Figure 2.8.2-2 or an aluminum extrusion as shown in Figure 2.8.2-3.

When the existing drain consists of a formed opening in the concrete deck, an aluminum drain can be fabricated and attached to the existing deck and beam as shown in Figure 2.8.2-4. When the existing aluminum drain extends horizontally through the curb, as is often the case with PPC deck beam superstructures, the details shown in Figure 2.8.2-5 may be utilized to prevent drainage onto the side of the fascia beam.

Deck drains should always be extended to a point at least 6 inches below the bottom of the beam or girder. Drain extensions on bridges where the extensions will be directly visible to the travelling public should be painted to match the color of the fascia beam or girder. The method of painting should be as described for deck drains in the Design Section of the IDOT Bridge Manual.

Deck drain extensions or new drains should be braced against the fascia beam to provide stability. Attachment of a typical 6” diameter drain to a steel beam/girder will require drilling holes into the web of the member in the field as shown in Figure 2.8.2-6. Because field drilling into PPC I beams is not allowed an alternate bracing detail is also shown.



STEEL CLAMP

Notes:

Pop rivet the ⅛" x 1" bar to Drain Extension. Weld or securely attach rod to both the clamp and bar stiffener. Use 3/16" stainless steel pop rivets of sufficient length.

Clamp shown in approximate dimensions. Similar commerically available may be substituted.

An aluminum extrusion drain extension of similar dimensions may be substituted.

* Field measure and cut to fit existing drain.

ALUMINUM SHEET DRAIN
EXTENSION DETAIL
FOR STEEL BEAMS

Figure 2.8.2-1: Aluminum Sheet Drain Extension Detail for Steel Beams

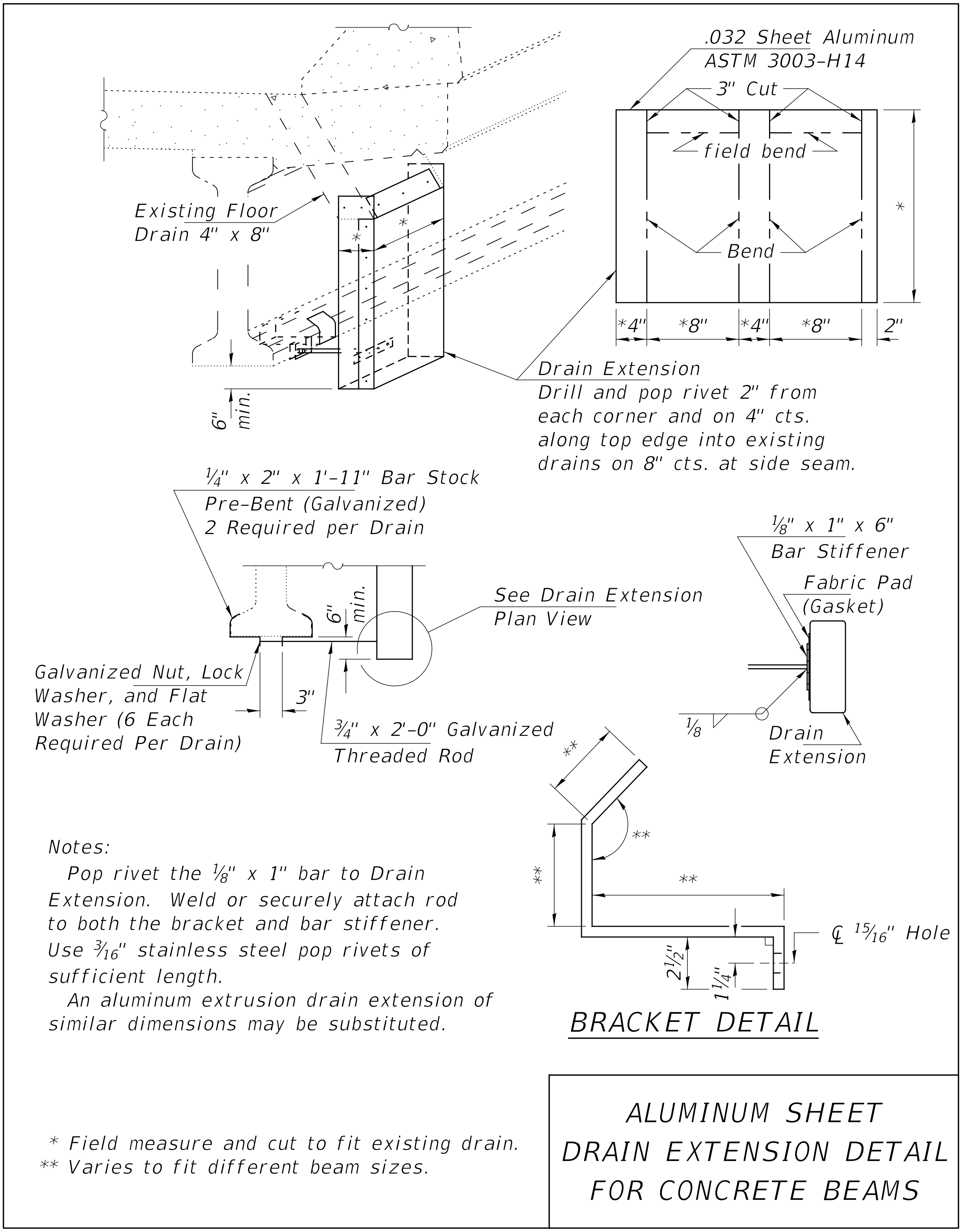


Figure 2.8.2-2: Aluminum Sheet Drain Extension Detail for Concrete Beams

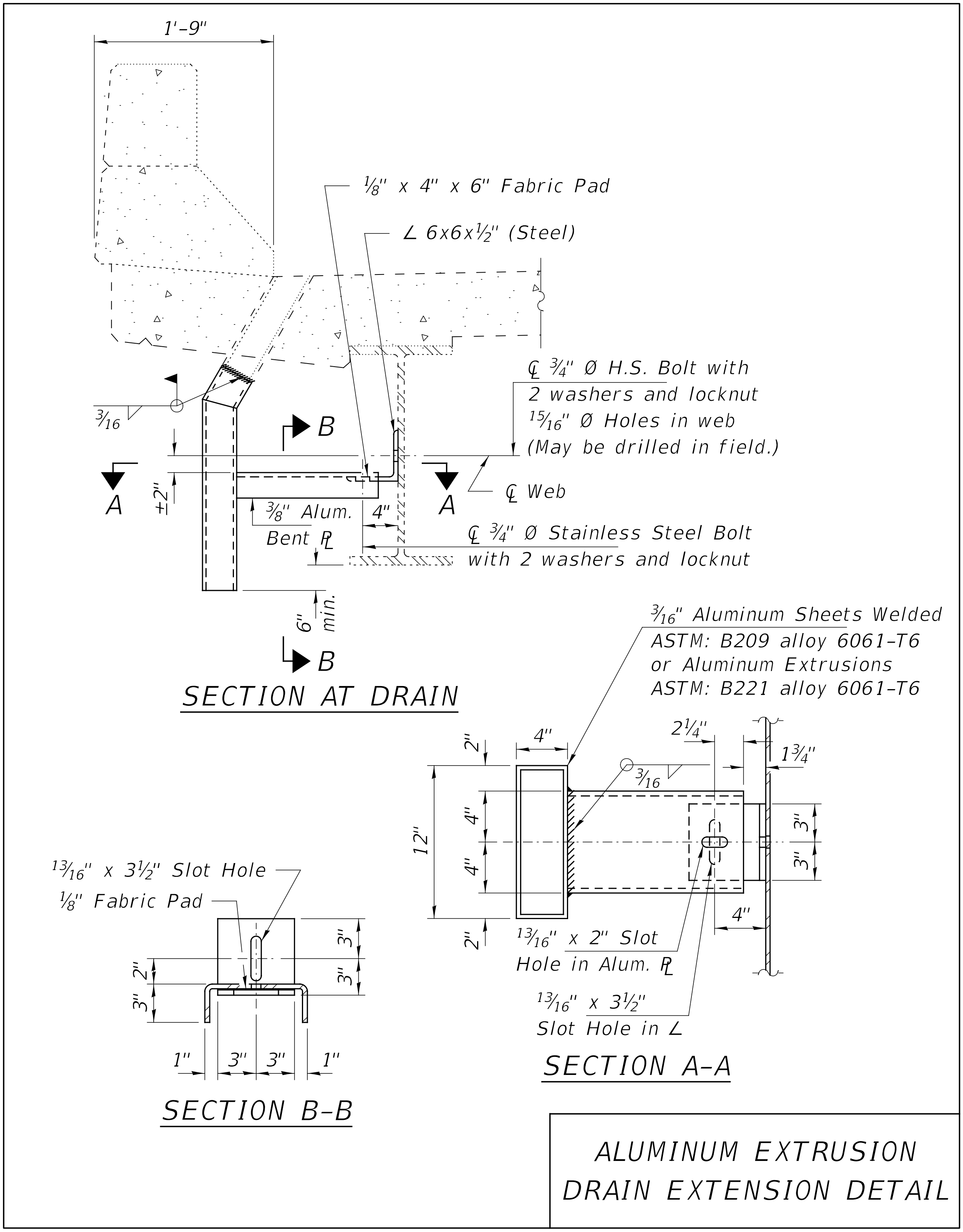


Figure 2.8.2-3: Aluminum Extrusion Drain Extension Detail

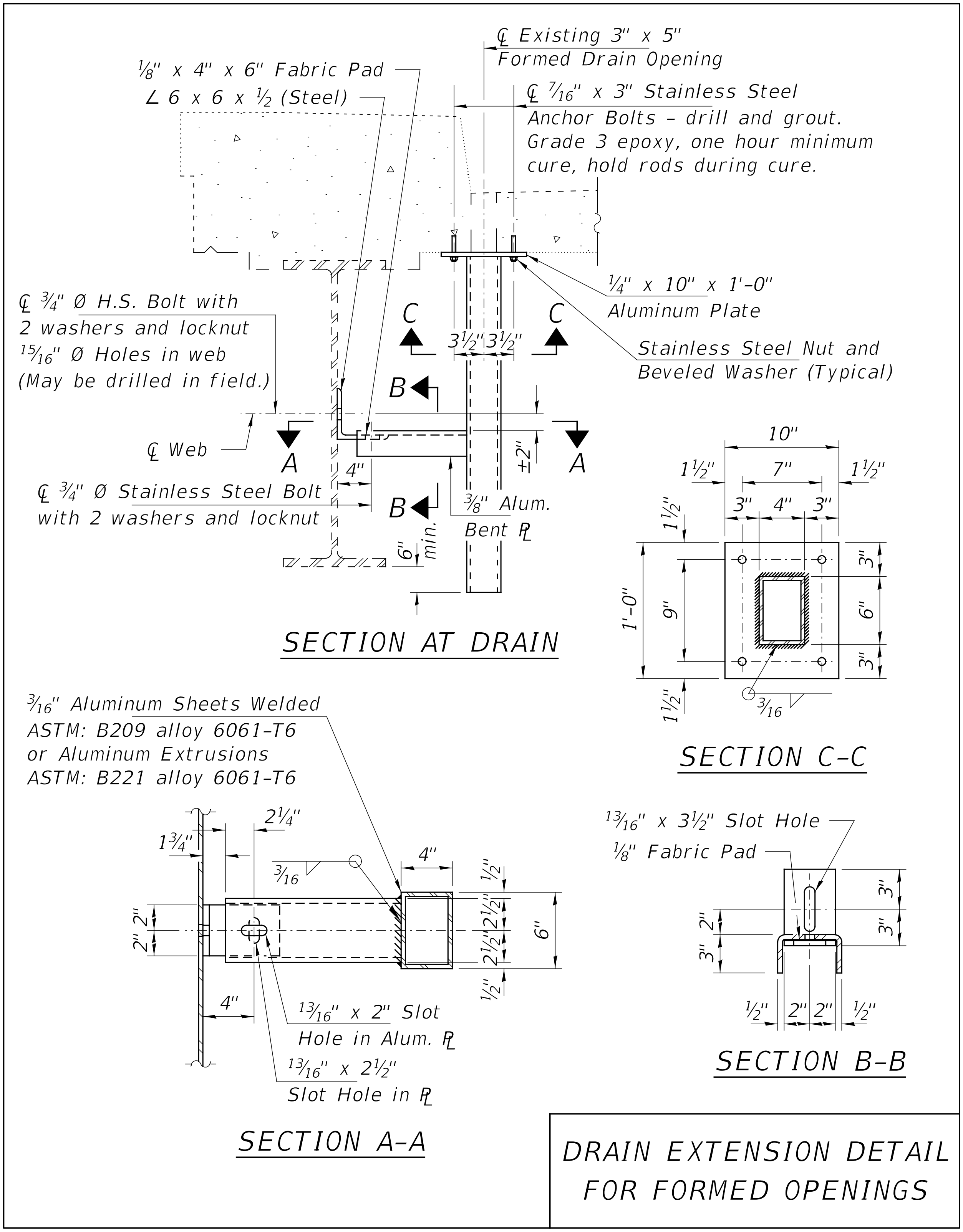


Figure 2.8.2-4: Drain Extension Detail for Formed Openings

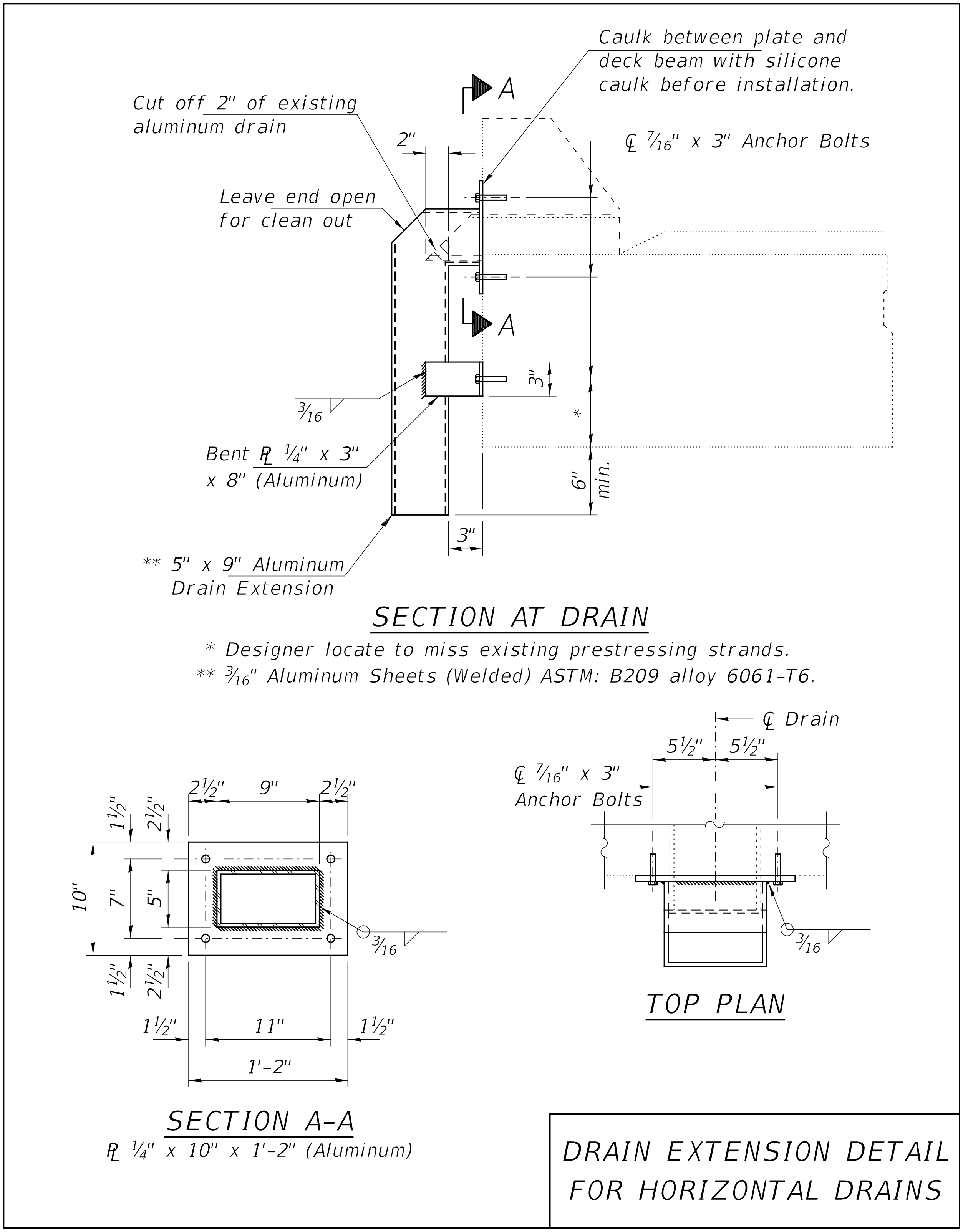


Figure 2.8.2-5: Drain Extension Detail for Horizontal Drains

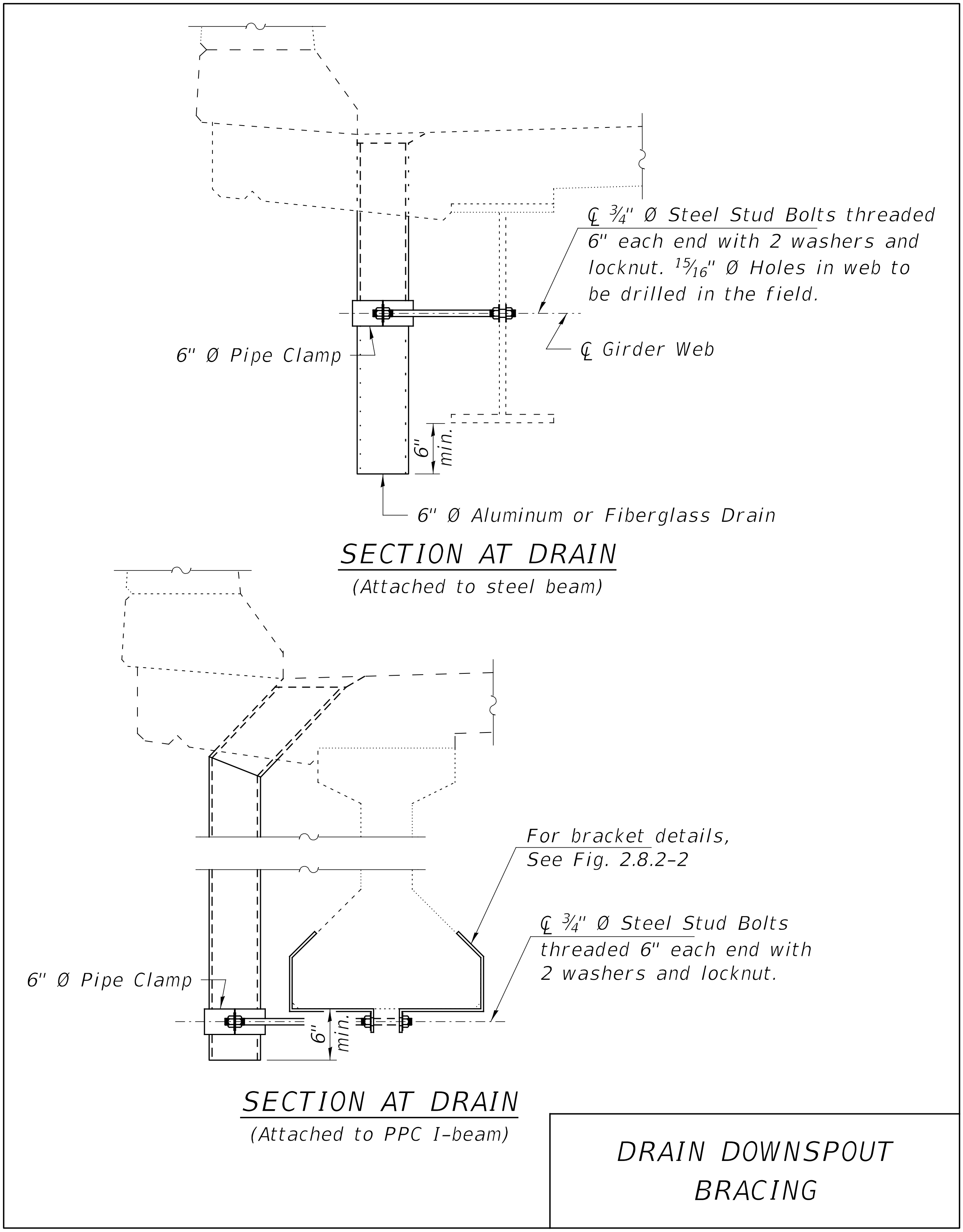


Figure 2.8.2-6: Drain Downspout Bracing

2.8.3 Treatment of New Overlays at Floor Drains

When a new overlay is installed, treatment of the floor drains is necessary in order to ensure proper drainage from the deck surface and to protect the edges of the new overlay around the drain opening.

When a new HMA or concrete overlay is installed the area around the floor drain should be tapered. For concrete overlays the minimum thickness around the drain opening should be 1”. See Figure 2.8.3-1.

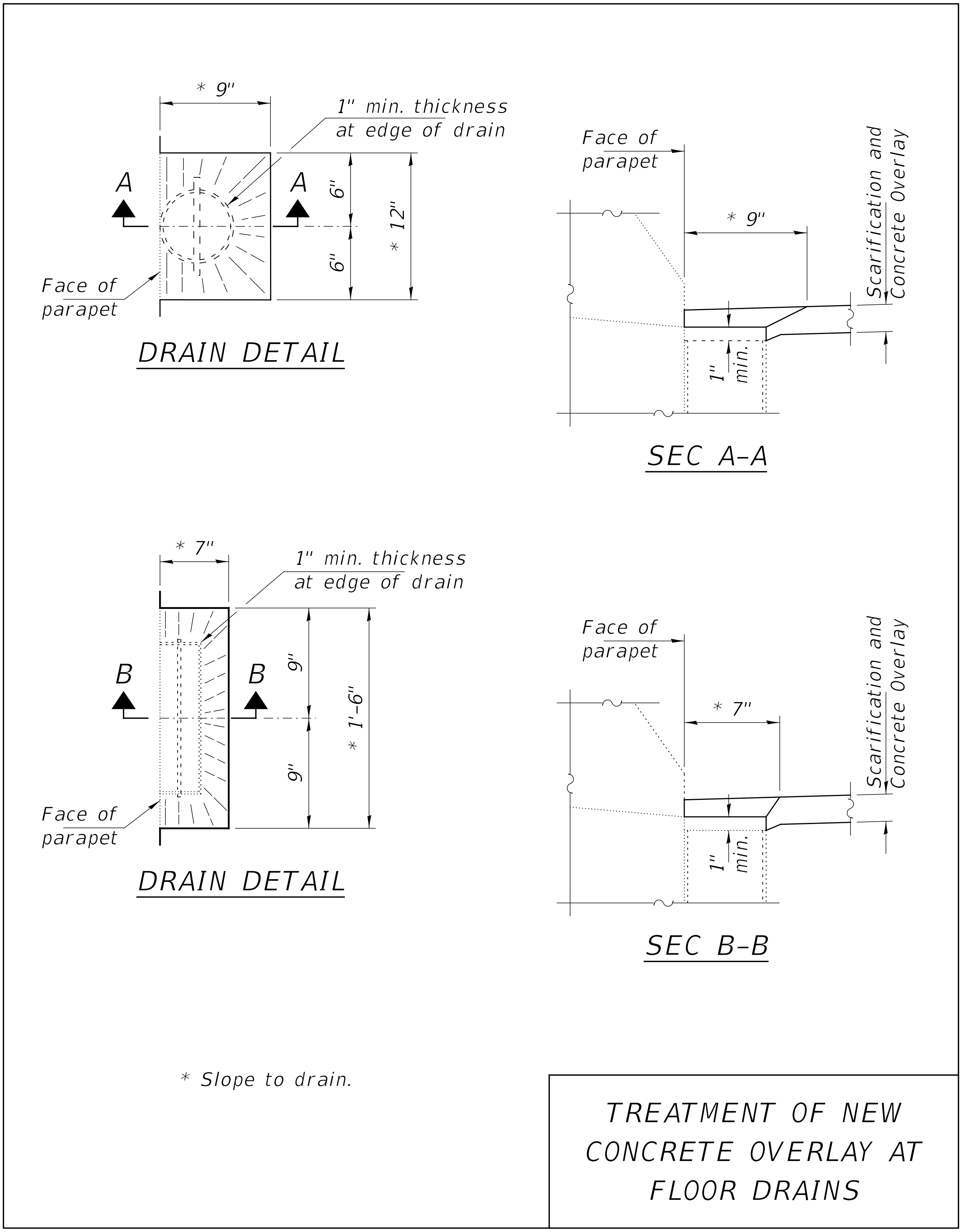


Figure 2.8.3-1: Treatment of new Concrete Overlay at Floor Drains

2.8.4 Drainage Scupper Adjustment for New Overlays

To minimize traffic hazards on bridge decks receiving a new concrete overlay, existing drainage scuppers need to be adjusted. Adjusting, rather than replacing them is recommended to avoid the associated cost of full depth deck removal. The adjustment consists of raising the level of the scupper's grate to match the new top of overlay. This is done by installing a collar between the existing scupper main body and the relocated grate. See typical detail in Figures 2.8.4-1 and 2.8.4-2.

Because of the variety of scuppers found in the field the designer must obtain accurate information about the existing scupper in order to detail a collar that will properly fit and provides a solid and secure connection. Details of the existing scupper must be included in the plans to aid in the fabrication of the collar.

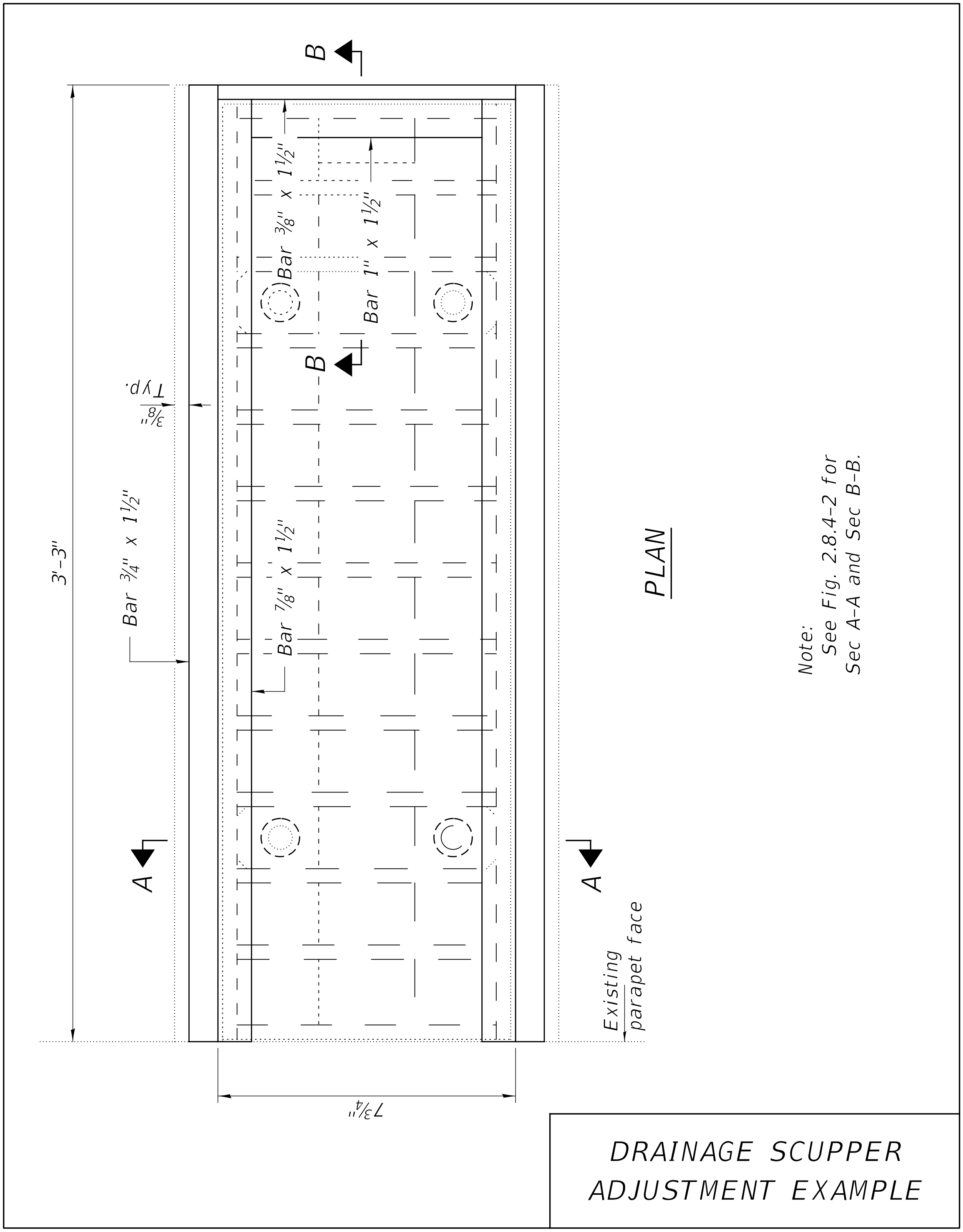


Figure 2.8.4-1: Example – Drainage Scupper Adjustment

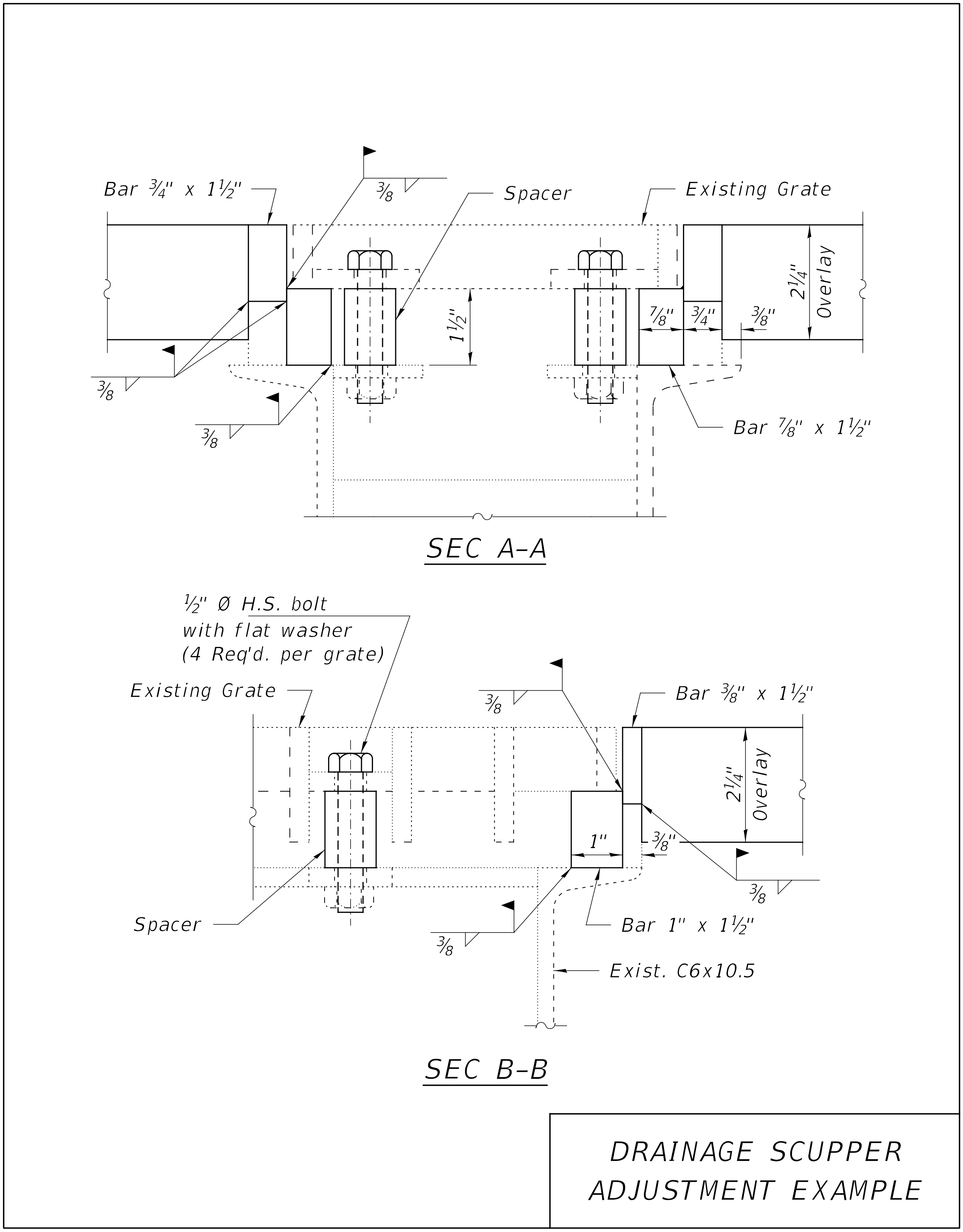


Figure 2.8.4-2: Example – Drainage Scupper Adjustment

2.8.5 Drain Elimination

Many existing structures were constructed with deck drains at 6 foot centers. The number of drains on these structures greatly exceeds the number of drains required to prevent ponding on the deck. When this situation is encountered, alternate drains may be eliminated to provide a 12 foot drain spacing. Also, existing deck drains located within 10 feet of a substructure unit should be eliminated.

Existing aluminum deck drains, which extend below the deck, can be eliminated by installing a threaded rod through the bottom of the drain and plugging the drain with concrete as shown in Figure 2.8.5-1. Existing drains consisting of a vertical formed opening in the deck slab should be eliminated by removing and replacing a 1 foot by 2 foot area of the deck as shown in Figure 2.8.5-2. Formed drain openings extending diagonally through the deck may be eliminated by coating the interior surface of the opening with a bonding agent and plugging the opening with concrete as illustrated in Figure 2.8.5-3.

The deck concrete adjacent to deck drains is often in an advanced state of deterioration relative to other areas of the deck slab. The condition of the concrete surrounding the drain should be considered when determining if a drain should be eliminated by plugging the existing drain opening or removing and replacing an area of the deck.

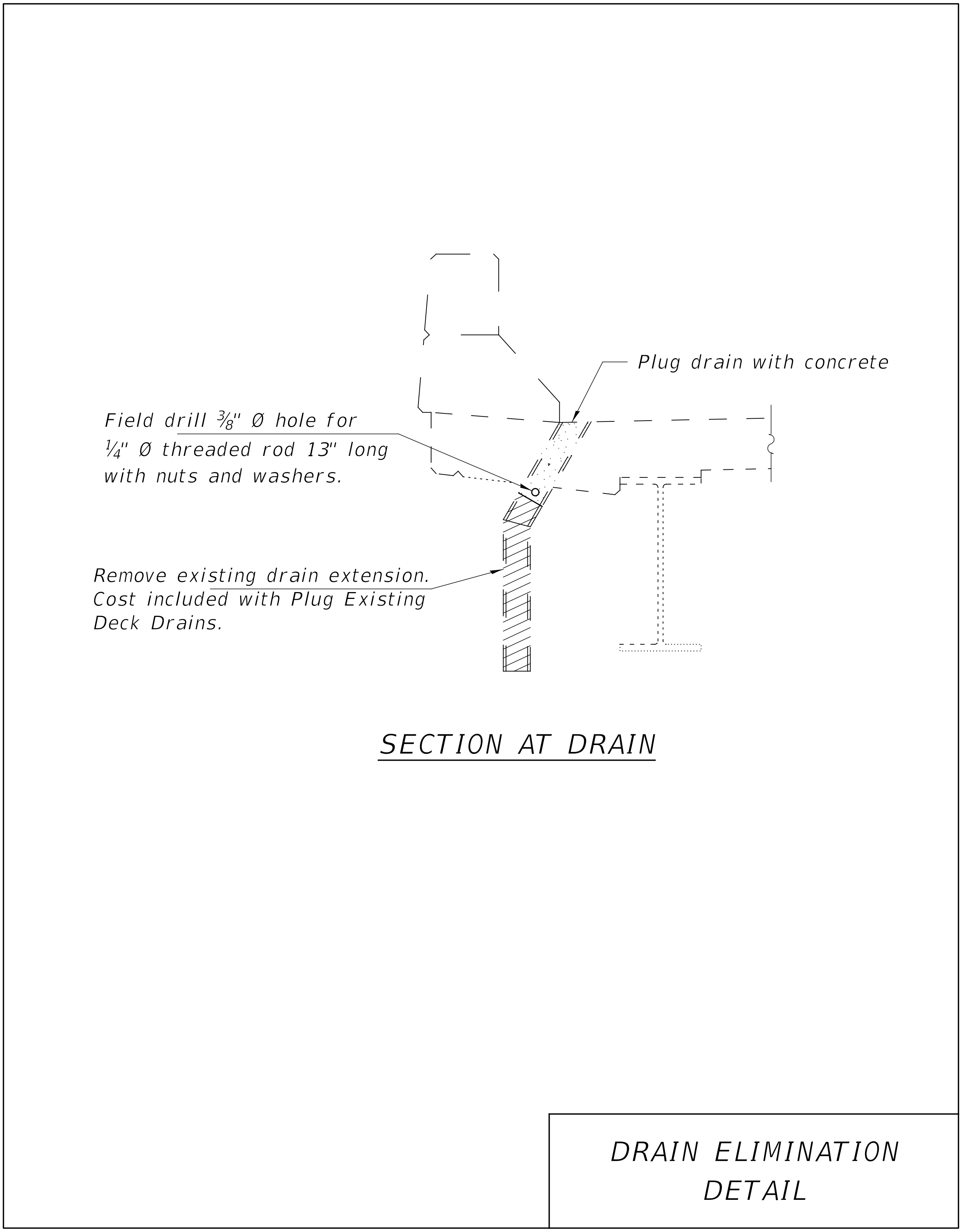


Figure 2.8.5-1: Drain Elimination Detail

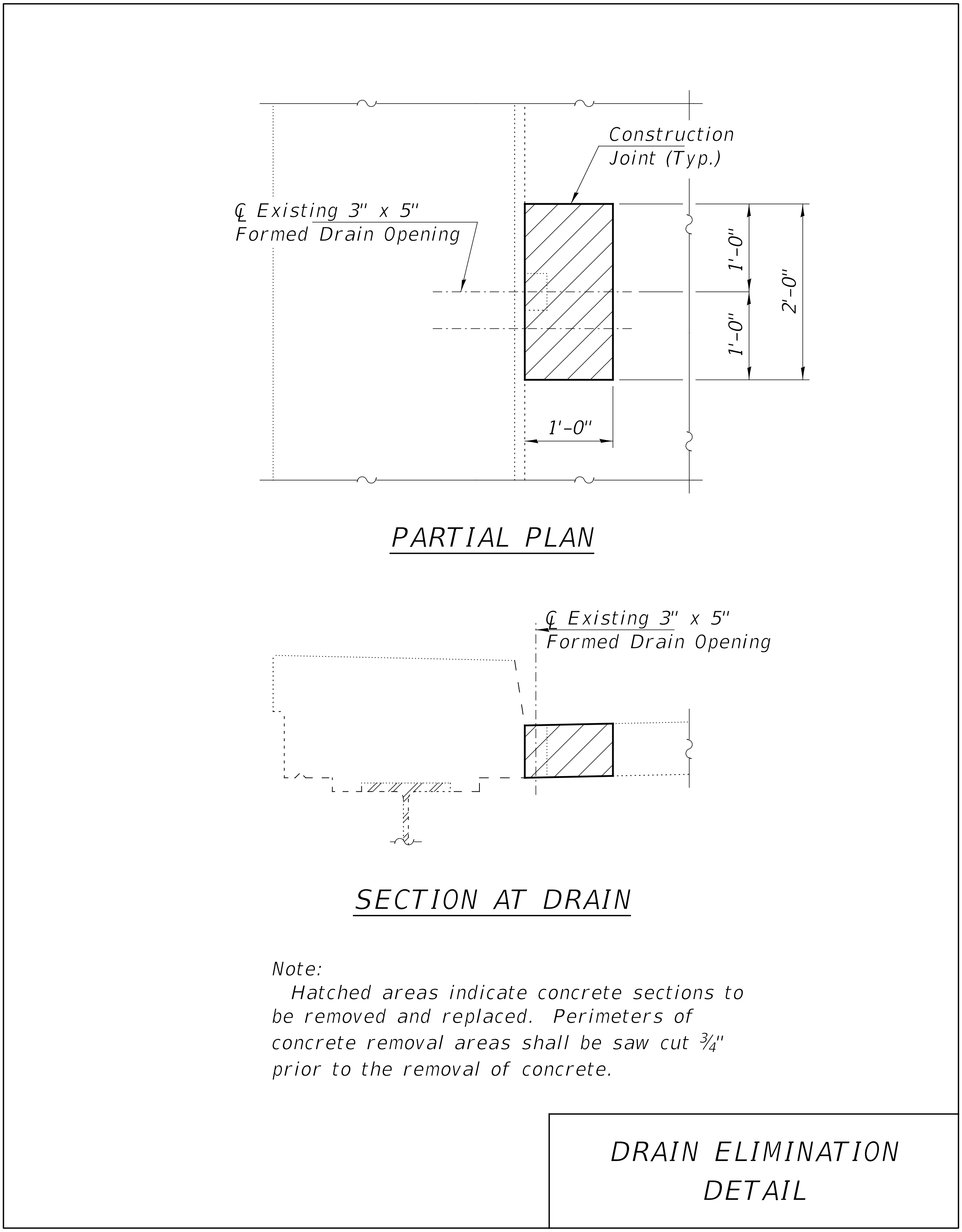


Figure 2.8.5-2: Drain Elimination Detail

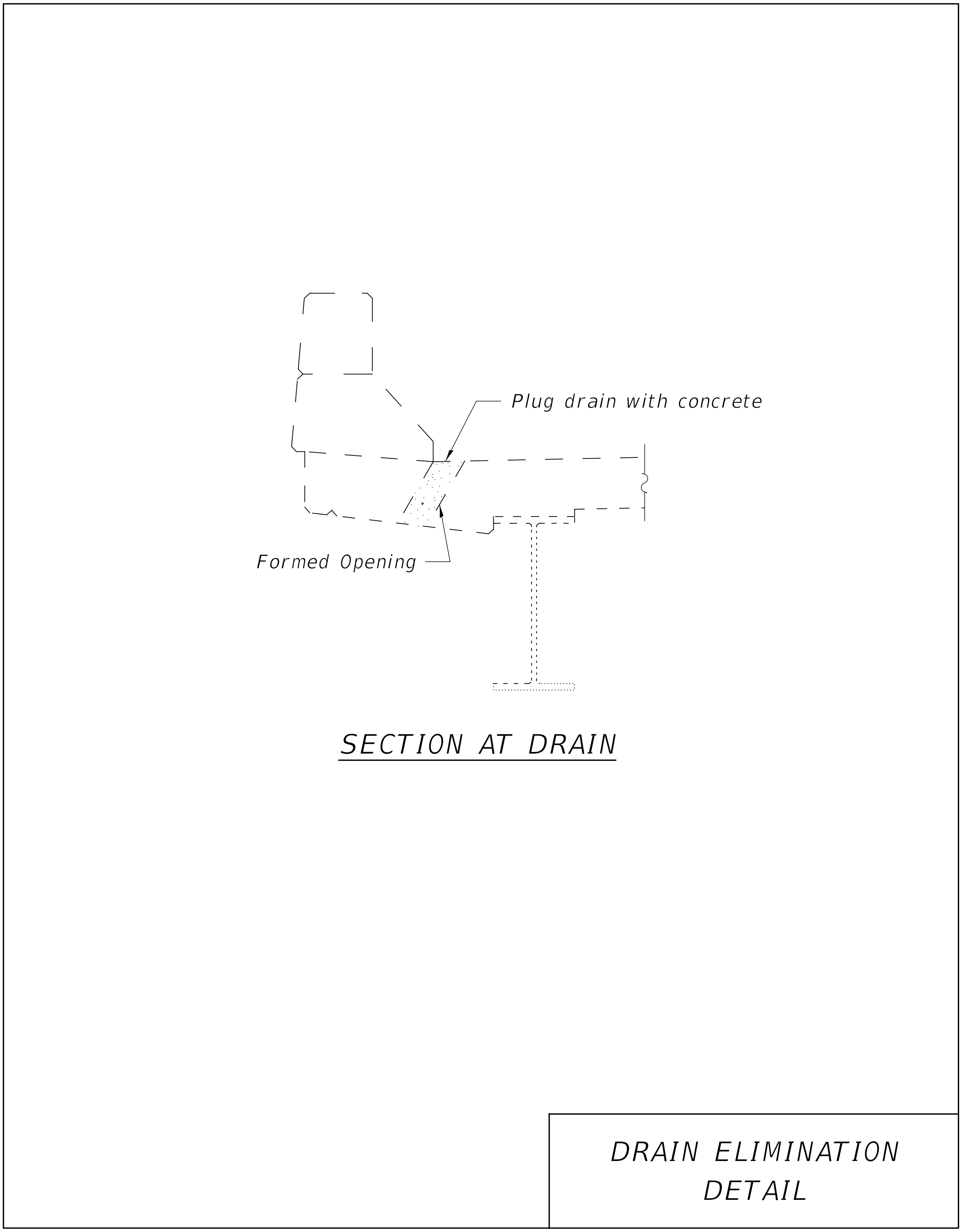


Figure 2.8.5-3: Drain Elimination Detail

2.8.6 Drain Replacement

Rather than extending an existing deck drain, a drain may be completely replaced using the details shown in the Design Section of the IDOT Bridge Manual. Installing a new drain will include the removal and replacement of any poor quality concrete in the area of the existing drain. Also, the replacement drain may be of a type which requires less maintenance or is more easily replaced when damaged or stolen.

Cost of new concrete around the removal area is included in Deck Slab Repairs.

2.9 Bearing Replacements

2.9.1 Bearing Type Selection

When existing bearings must be replaced, the replacement bearing assemblies shall be designed as specified in Section 3.7 of the IDOT *Bridge Manual*. Elastomeric bearings, with side retainers on each side, should be used when replacing existing expansion bearings except where specific design requirements necessitate the use of an alternate type of bearing, such as a High Load Multi-Rotational (HLMR) bearing. As indicated in the IDOT *Bridge Manual* it is preferred to use a Type I bearing whenever possible, even though it may require the use of a larger bearing size. In-kind steel rocker or roller bearings should be used as replacement bearings only when a limited number of bearings are to be replaced on a specific structure and the replacement bearings need to match the behavior of the existing bearings which are to remain in place in the same bearing line.

Fixed bearing design and details can be found in Section 3.7.1.2 of the IDOT *Bridge Manual*.

2.9.2 Seismic Requirements

Seismic requirements, including substructure seat width and superstructure restraint, need not be considered for maintenance contracts and Day Labor awards. However, seismic requirements should be considered as specified in the IDOT *Bridge Manual* for structures being rehabilitated or replaced. Existing seismic restraints must be repaired and if necessary, replaced in-kind when damaged.

2.9.3 Existing Bearing Removal

When removing an existing bearing assembly, the steel plate welded to the bottom flange of an existing beam or girder should be removed. The method of removal used should not damage the bottom flange and should provide for the removal of all existing weld material from the bottom flange. Figure 2.9.3-1 shows details associated with the removal of an existing bearing assembly.

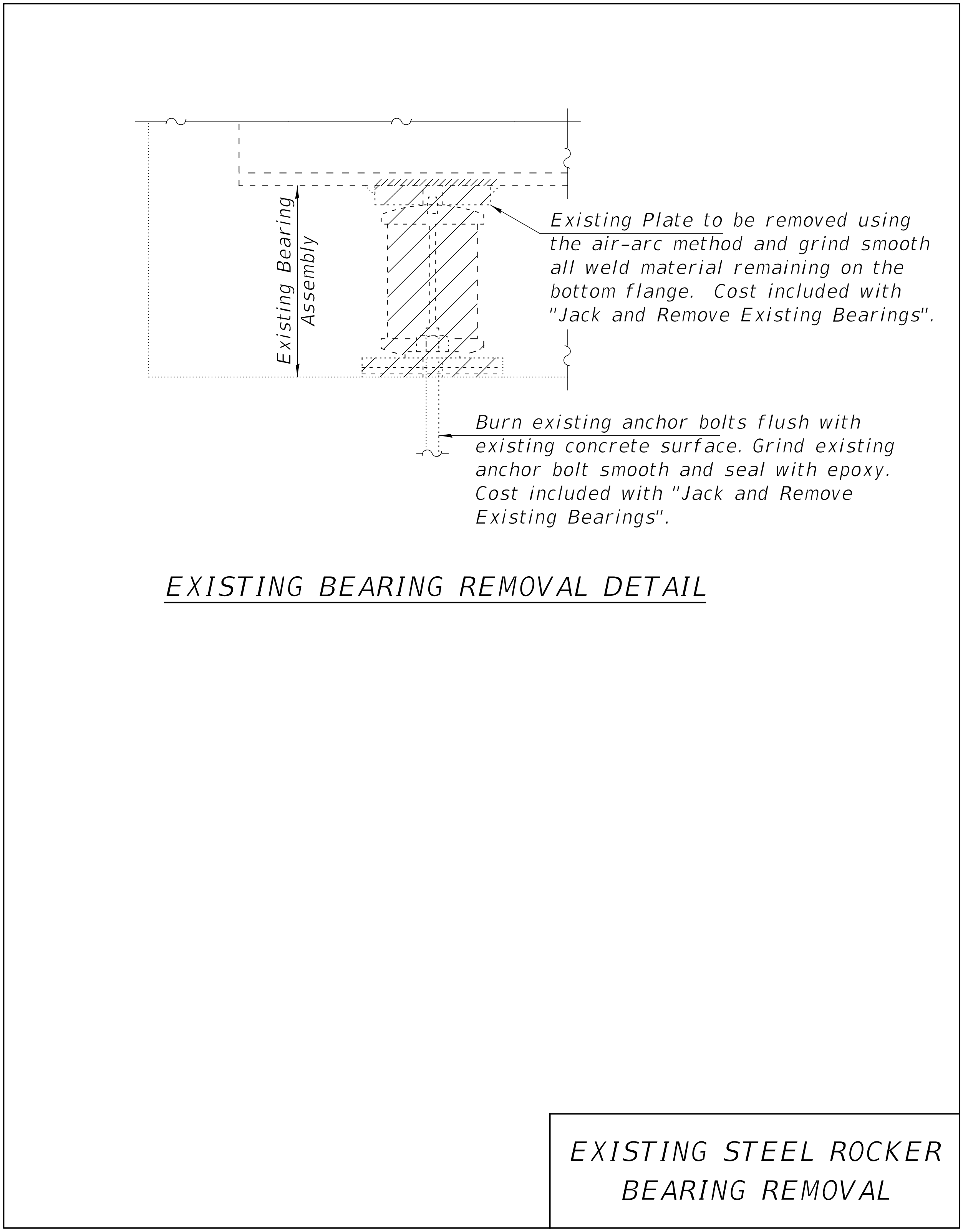


Figure 2.9.3-1: Existing Steel Rocker Bearing Removal

2.9.4 Bearing Height Adjustment

When elastomeric bearings are used to replace existing steel bearings, there is routinely a difference in the height of the existing bearing assembly versus the height of the new elastomeric bearing assembly. Differences in bearing assembly height require the addition of a steel extension to the elastomeric bearing assembly or the reconstruction of the substructure bearing seat. These adjustments are necessary to maintain the current profile grade elevations of the superstructure or to adjust the elevation of the superstructure to meet a new profile grade.

The decision to use either a steel extension or a reconstructed bearing seat will depend on the difference between the existing and proposed bearing assembly heights. Generally, steel extensions should be used to adjust the height of the elastomeric bearing assembly for extensions less than or equal to 12 inches in height. When the height of the extension is less than 6 inches, the extension should consist of steel shim plates placed between the elastomeric bearing assembly (Type II shown) and the superstructure as shown in Figure 2.9.4-1. When the required height of the extension is greater than or equal to 6 inches and less than 12 inches, an extension should be fabricated using 1-inch plates as shown in Figure 2.9.4-2. When 12 inches or more of bearing assembly height adjustment is required, the bearing seat area of the abutment or pier should be reconstructed to provide the proper elevation for the new elastomeric bearing assemblies. Figure 2.9.4-3 illustrates an acceptable method of constructing a new concrete seat for a replacement bearing assembly.

The guidelines for choosing extension types should be used with some flexibility. Although a concrete extension is preferred for extension heights of 12 inches or more, situations will occur when a fabricated steel extension may be used with a height of 12 inches or greater in order to minimize construction time and traffic disruptions. Also, fabricated steel extensions may be used for a height less than 6 inches when it is physically possible to fabricate and install the steel extension.

The same procedure can be used when a tall fixed steel bearing is being replaced with a low-profile fixed bearing.

If shim plates are required, a maximum of two plates should be used in order to minimize the number of steel plies where moisture can accumulate, and pack rust can develop.

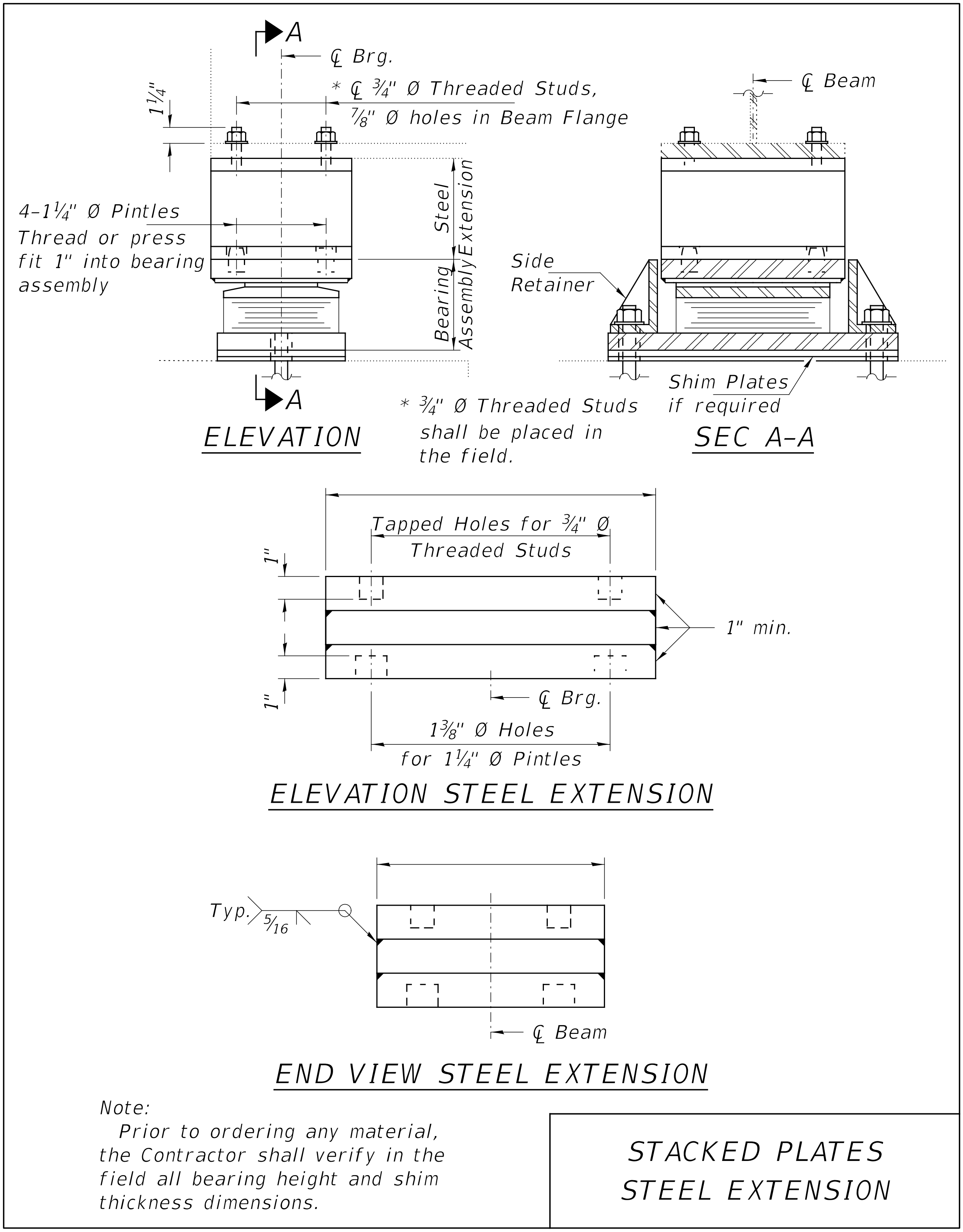


Figure 2.9.4-1: Stacked Plates Steel Extension

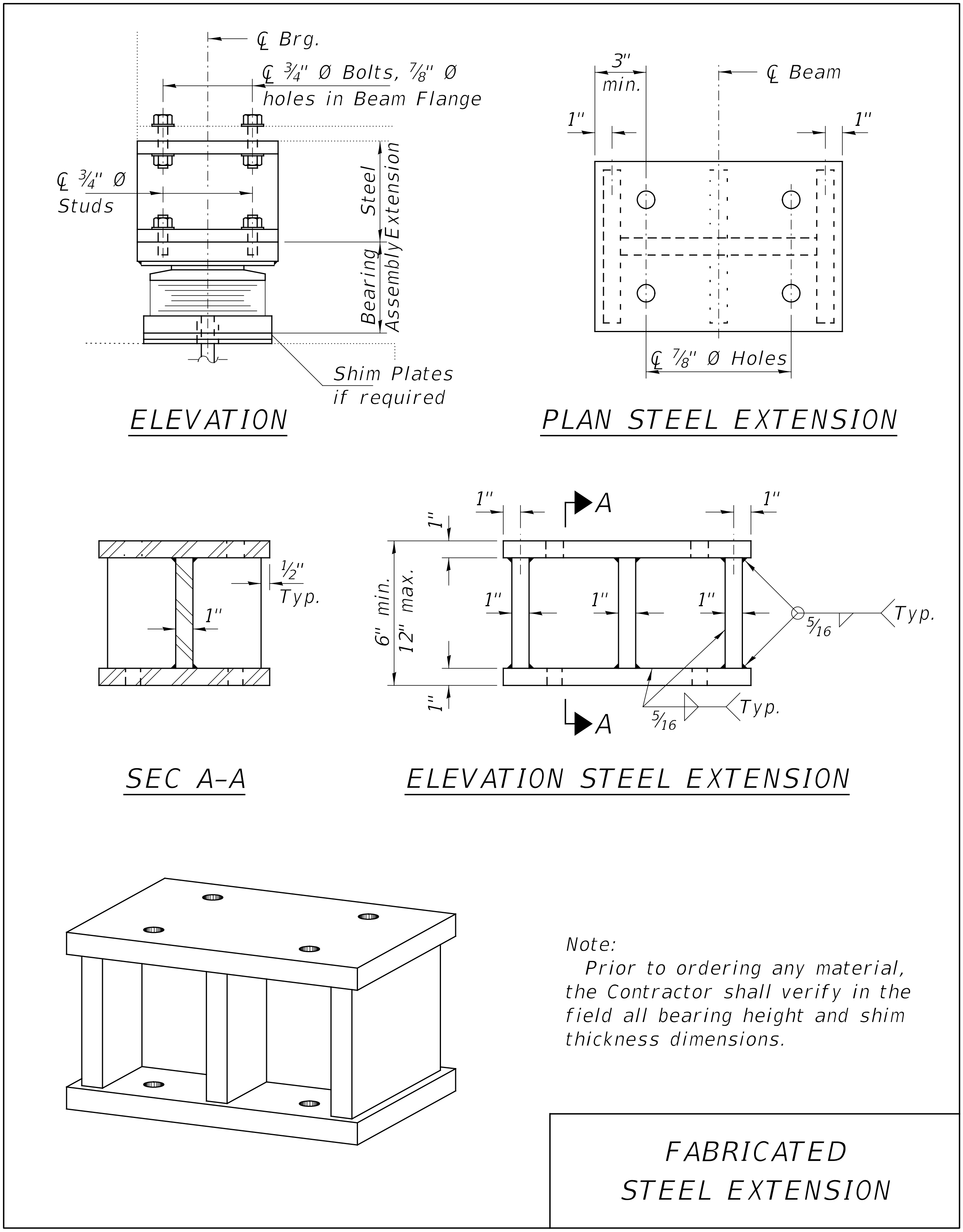


Figure 2.9.4-2: Fabricated Steel Extension

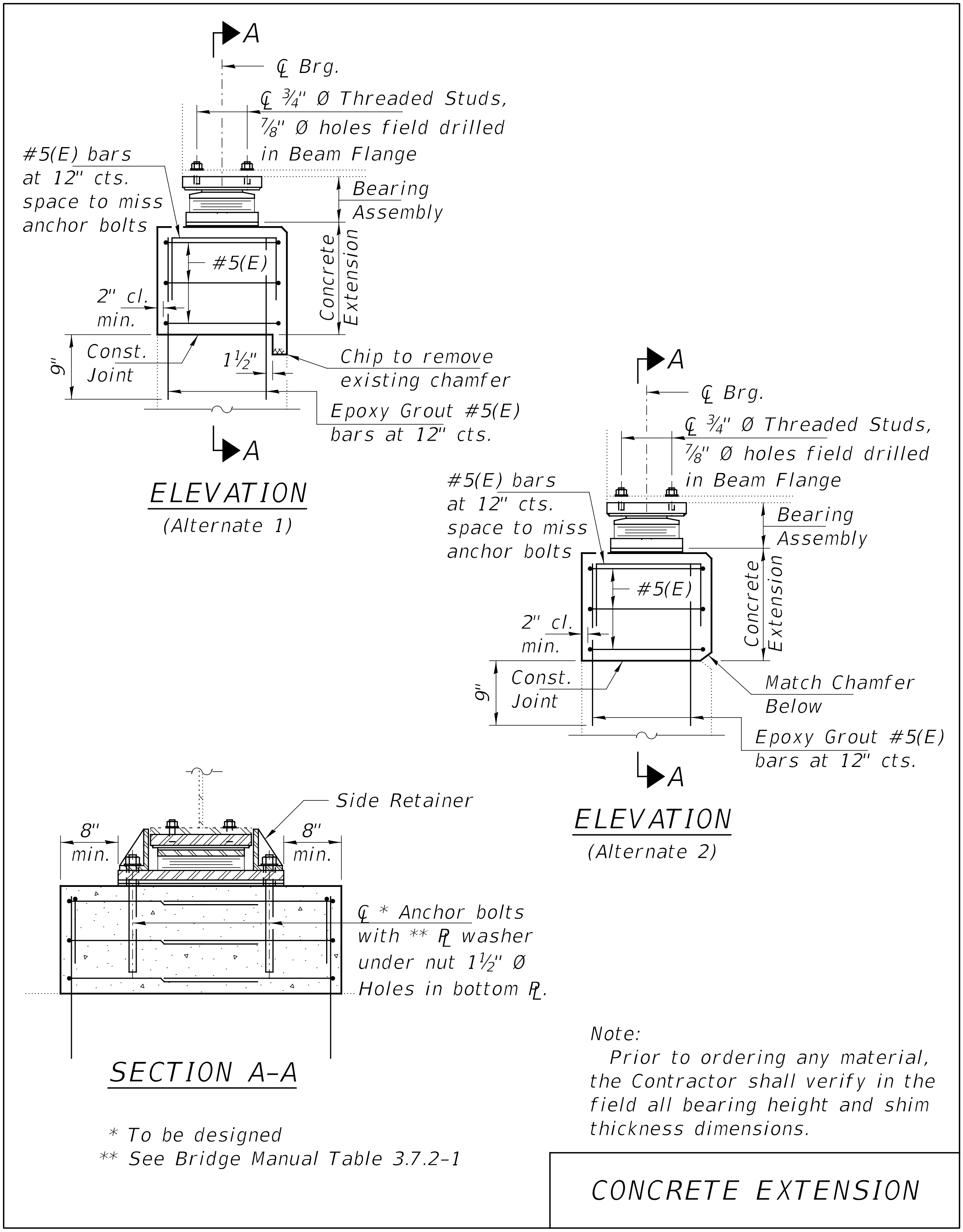


Figure 2.9.4-3: Concrete Extension

2.9.5 PPC I Beam Bearing Replacement

Because of the very limited space available between the substructure concrete and the embedded steel plate in the PPC I beam bottom flange, a typical elastomeric bearing can’t be installed. Modified Type II bearings (without elastomer) should be used as shown in Figure 2.9.5-1. The top bearing plate can slide under the beam and be lifted so the pintles can engage the embedded steel plate. The bottom bearing plate is then installed underneath the top plate followed by the placement of the side retainers and anchor bolts.

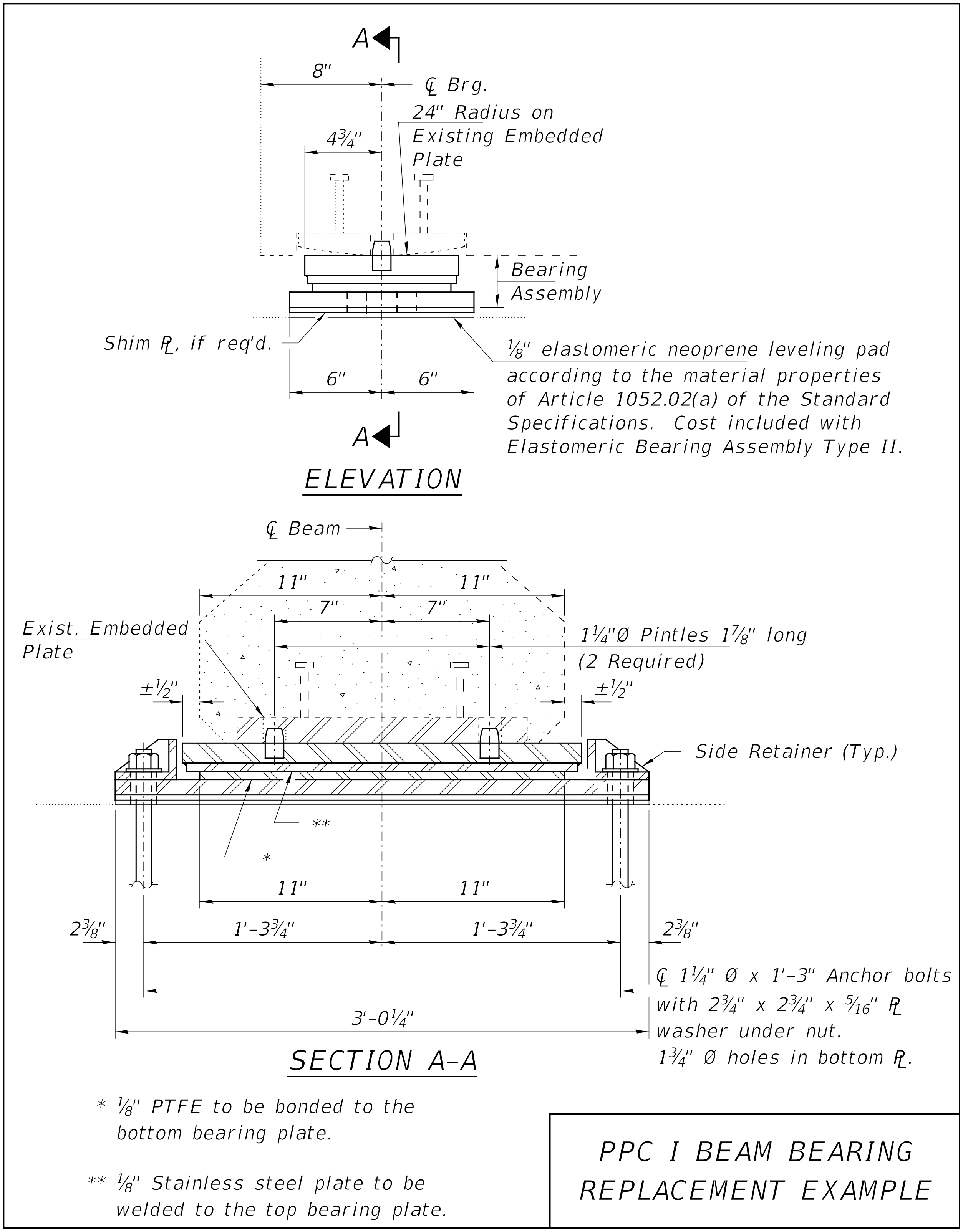


Figure 2.9.5-1: Example – PPC I-Beam Bearing Replacement

2.9.6 Reaction Table

All plans for bearing replacement should contain a table showing the girder or beam service reactions for each bearing replacement location (substructure unit). The table should include the dead load, superimposed dead load, live load, impact and total reaction per beam line as shown in the Design Section of the IDOT Bridge Manual for an “Interior Girder/Beam Reaction Table”. The Repairs Unit of the Structural Services Section of the Bureau of Bridges & Structures will provide reaction tables when plans are prepared by District personnel.

2.9.7 “C” Value for Bearing Plate Design

Almost all the structures for which bearing replacements are being done were designed using Service Load Design and HS20 loading. Using a resistance value of 0.75 Fy the “C” value to be used in the design of bearing plates for Repair projects should be as shown below:

- Fy = 36 ksi, use a “C” = 0.167
- Fy = 50 ksi, use a “C” = 0.141

2.9.8 Top Bearing Plate Adjustment

If a steel rocker expansion bearing is substantially tilted out of position with respect to its proper location for a given temperature, the top bearing plate needs to be adjusted. This consists of jacking the beam, removing the top bearing plate by the air-arc method without damaging the bottom flange and grinding smooth all weld material remaining on the bottom flange. Then a new top bearing plate should be placed in the correct position depending on the temperature at the time and bolted to the bottom flange (See Figure 2.9.8-1).

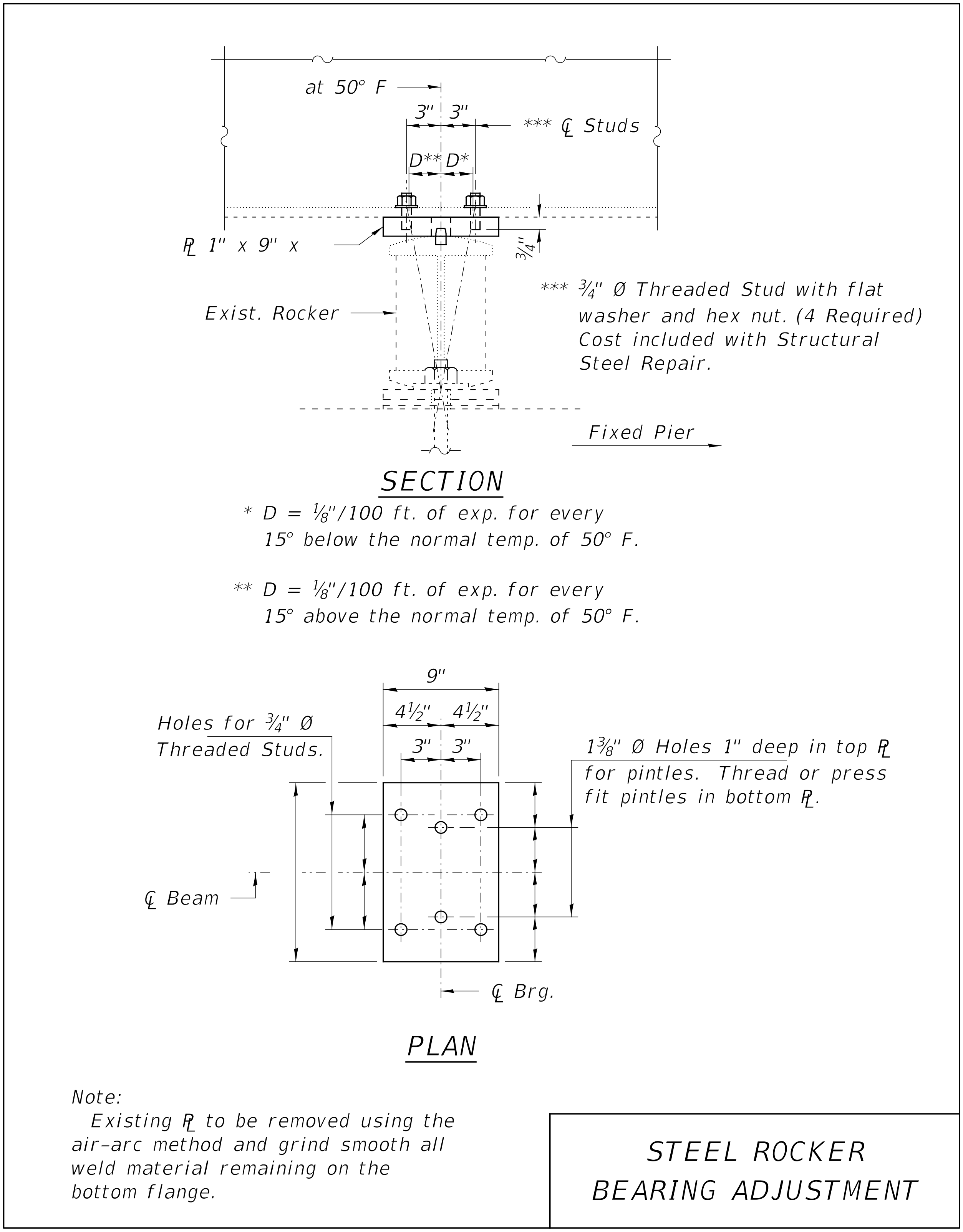


Figure 2.9.8-1: Steel Rocker Bearing Adjustment

2.9.9 Anchor Bolts

Anchor bolts needed for new bearings shall be placed such that the distance between the center of the existing anchor bolt which has been cut off and the center of the new anchor bolt is not less than “S” as shown below:

D1 = Existing bolt diameter

D2 = New bolt diameter

S = ((D1 + D2)/2) + 1 ¼”

Commonly, in order to satisfy the above space requirement, it will be necessary to adjust the length of the bottom plate of the side retainer and bottom bearing plate (if present).

When an individual anchor bolt is broken off, a new anchor bolt should be installed. Figure 2.9.9-1 shows treatment for a broken anchor bolt on a Type I bearing side retainer and for a broken anchor bolt on a bearing plate. The same anchor bolt spacing requirement shown above for new bearings should also be used when replacing a single existing broken anchor bolt. The designer must ensure that adequate space is available to place the new anchor bolt particularly on structures on a skew.

Removal and reinstallation of existing diaphragms/cross frames may be necessary for installation of new anchor bolts.

Anchor bolts are to be ASTM F 1554 Grade 55 designed for 0.2 x Dead Load. The minimum size of the anchor bolts should be 1” Φ. For ease of installation, the maximum size of anchor bolts should be limited to 1 1/2” Φ. If necessary, increase the number of bolts or the material strength.

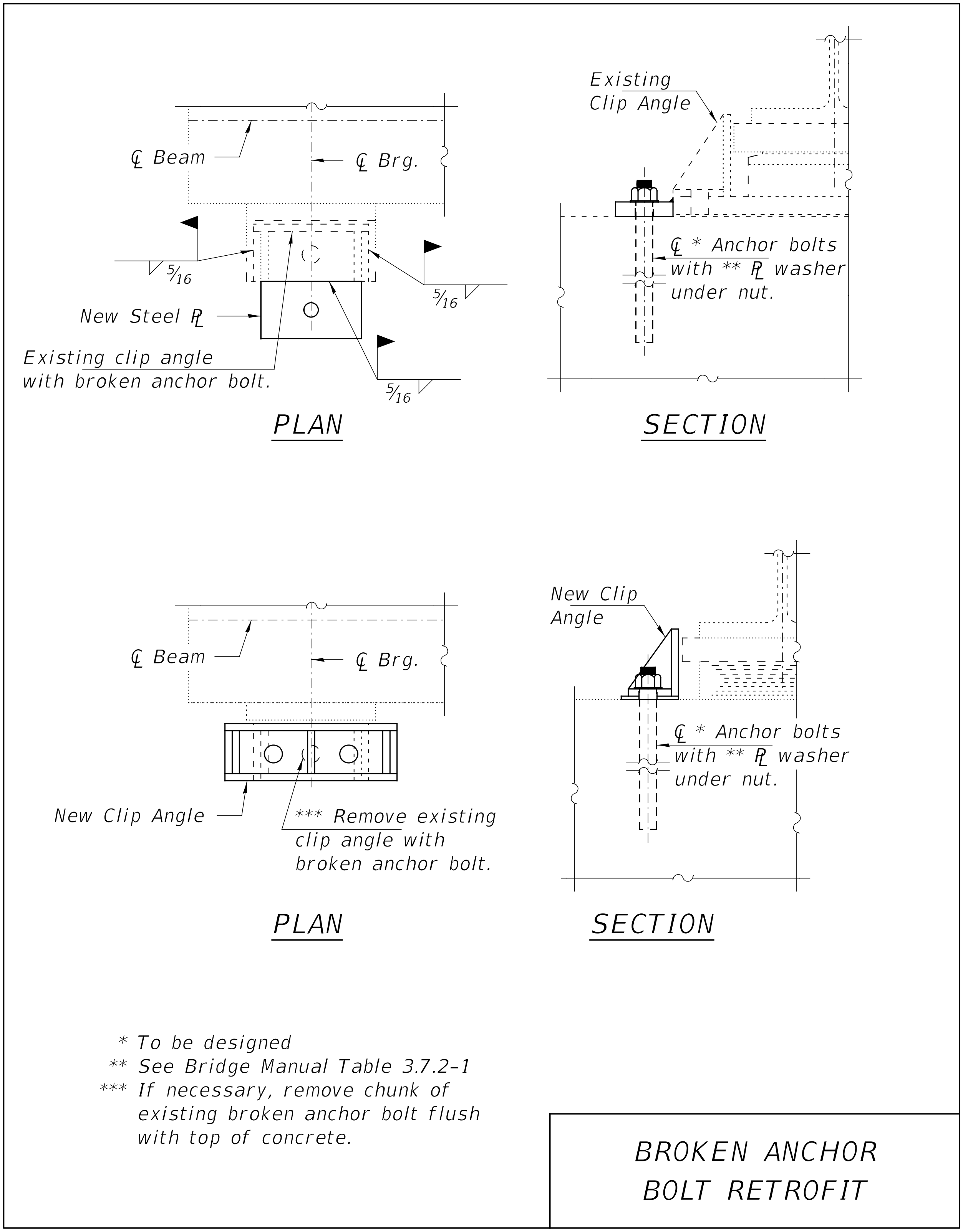


Figure 2.9.9-1: Broken Anchor Bolt Retrofit

2.10 Jacking/Shoring and Cribbing

2.10.1 General

During bearing replacements, bearing adjustment or beam end section replacement, the superstructure must be lifted and temporarily supported. This procedure is generally referred to as Jacking and Cribbing. During concrete repairs on substructure elements such as bearing caps or columns or while completing beam end repairs it may also be necessary to support specific beams. This procedure is generally referred to as Shoring and Cribbing. The purpose of these systems is to provide an alternate load path for the superstructure loads to the ground surface or in some cases to the substructure element.

Both systems require the use of a hydraulic jack to apply the required loads.

Specific details for Jacking and Cribbing or Shoring and Cribbing are not typically shown in the repair plans prepared for maintenance contracts or Day Labor awards. However, pay items and special provisions for jacking/shoring, cribbing, and bearing removal should always be included in the repair plans. See Figure 2.10.1-1 for a typical jacking/shoring and cribbing system for a beam at a pier showing the required elements needed to be submitted for review. See Figure 2.10.1-2 for a jacking/shoring and cribbing system to be used at an abutment. Specific details for the jacking/shoring and cribbing system to be used to implement the planned repairs shall be developed by the contractor's Structural Engineer and submitted to the Bureau of Bridges and Structures for review and approval prior to beginning jacking operations. Signed and sealed design calculations shall be included with the jacking/shoring and cribbing details submitted for review and approval.

Because the review of jacking/shoring systems submittals require a quick turnaround it is critical that all required information be provided to avoid the need for resubmittals and expedite the review.

If field conditions require the use of a non-conventional jacking/shoring system, it is strongly recommended that the Repairs Unit of the Structural Services Section be contacted for advice prior to development of final details.

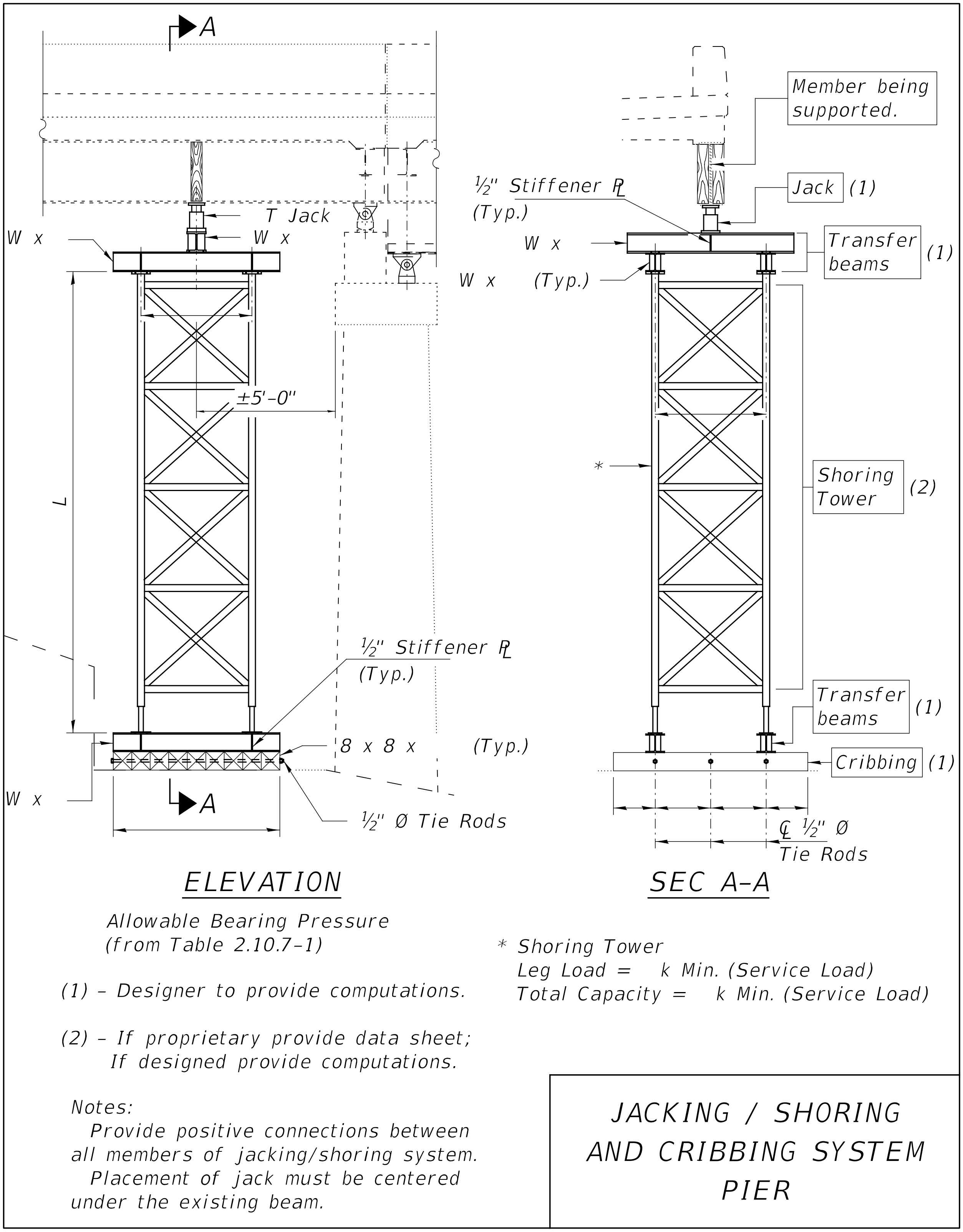


Figure 2.10.1-1: Jacking/Shoring and Cribbing System Pier

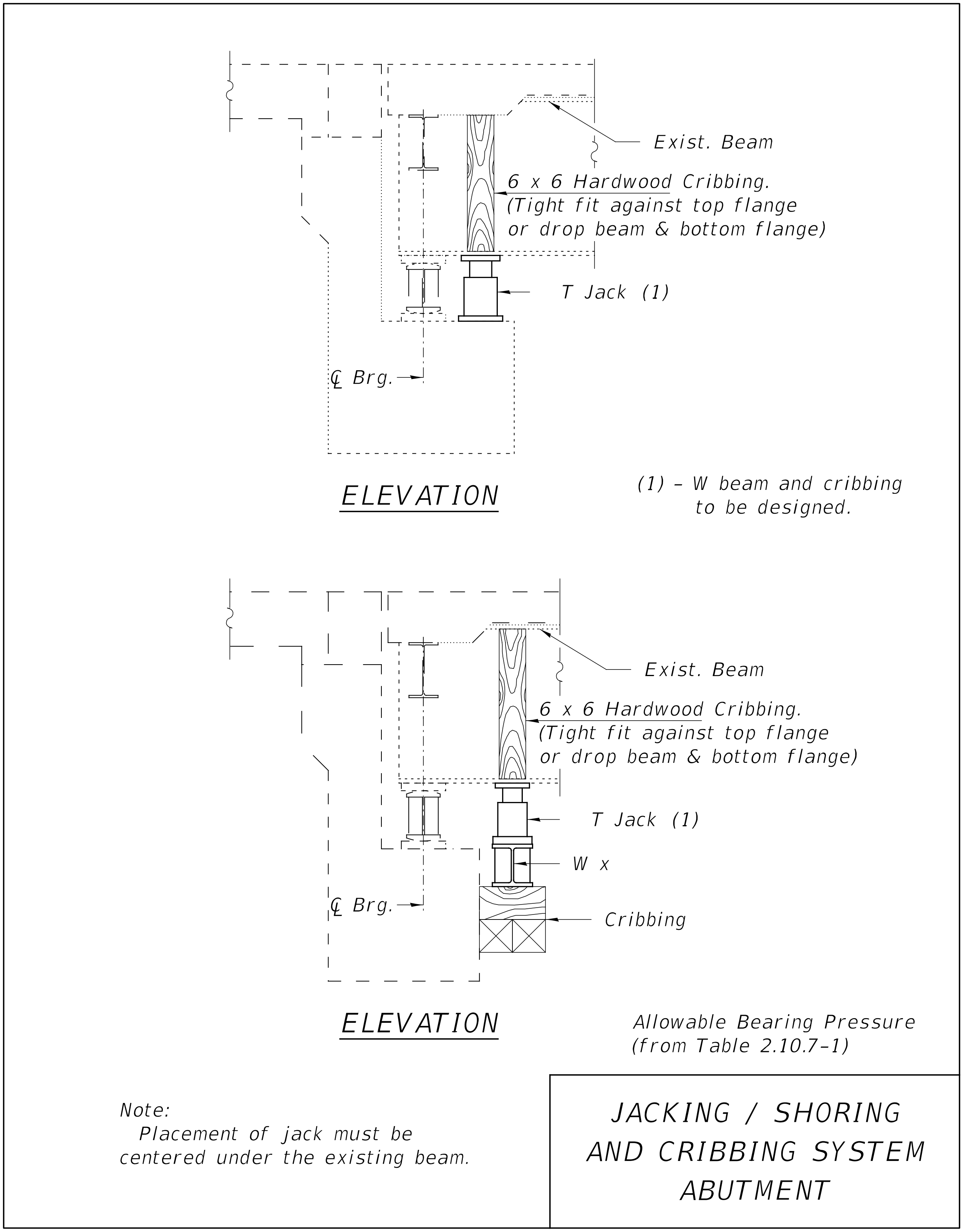


Figure 2.10.1-2: Jacking/Shoring and Cribbing System Abutment

2.10.2 Traffic Staging

The plans or special provisions should require that traffic be redirected during the jacking, shoring and cribbing operations so that vehicles will not be located directly over the shored and cribbed areas. Traffic should not be allowed directly over shored and cribbed areas of the superstructure unless there is no reasonable alternate method of routing vehicles, and the jacking, shoring and cribbing plans contain details which will provide adequate horizontal and vertical stability for the superstructure as well as being designed to carry the full live load and impact. Vehicles must not be allowed to travel over uncribbed areas of the structure supported only by hydraulic jacks. If necessary, traffic must be stopped during the short period of time during which the jacking operations are taking place.

2.10.3 Jacking Synchronization

During the jacking and cribbing operations, the lifting of the structure should be controlled so that the relative elevation between adjacent beams does not vary more than 1/8 inch from their original elevation differential. Also, the relative elevations of a continuous beam at adjacent substructure units should not vary more than 1/4 inch from their original elevation differential. If the entire superstructure is being raised a significant amount to meet new profile grade requirements, a synchronous lifting system should be used to control and equalize individual jack pressures to ensure that the superstructure is lifted uniformly without exceeding the above stated relative elevation differentials. These relative elevation restrictions and system requirements should be included in the plans or special provisions. The relative elevation restrictions should not be exceeded unless analysis indicates that greater differential elevation variances can be tolerated and will not induce stresses in the superstructure elements which will exceed the original allowable design stresses used for the structure.

2.10.4 Jacking and Cribbing Systems Capacities

As indicated in Section 2.9.6, service reactions for dead load, superimposed dead load and full live load and impact are to be included in the plans. For bearing replacements, bearing adjustment or beam end section replacement when the deck is in place during the jacking and cribbing operations and traffic is allowed in the lanes adjacent to the work area, the maximum expected load should include the full dead load and superimposed dead load plus one-half of the live load and impact reactions given in the plans. This is based on the assumption that the deck and diaphragms transfer a portion of the live load from adjacent lanes that are under traffic. If traffic cannot be kept off the portion of the structure directly above the repair work, the cribbing shall be designed to support the dead load, superimposed dead load and full live load and impact shown in the plans.

The jack capacities required should be based on the maximum expected load as described above. The jack capacity provided should be no less than 50% greater than the maximum expected loads.

Jacks can be used as cribbing if they are equipped with a locking mechanism that will allow the removal of the hydraulic pressure after the jacking has been completed.

2.10.5 Shoring and Cribbing Systems Capacities

The intent of the Shoring and Cribbing System used for substructure concrete work is to provide, at the locations shown in plans, additional support for the beams/girders while the repair work is completed, and the new concrete is cured.

The need for beam shoring is determined by the designer during the plan preparation and it is based on available inspection information for conditions in the field and engineering judgement. While field personnel may, during construction, request the installation of shoring at additional locations based on new findings, the elimination of locations requiring shoring as shown in the contract plans is not allowed, unless an obvious error has been made and the engineer-of-record has concurred.

As indicted in Section 2.9.6, service reactions for dead load, superimposed dead load and full live load and impact are to be included in the plans.

When the deck is in place and traffic is allowed in the lanes adjacent to the work area, the maximum expected load for design of the shoring and cribbing should include the full dead load and superimposed dead load plus one-half of the live load and impact reactions given in the plans. This is based, again, on the assumption that the deck and diaphragms transfer a portion of the live load from adjacent lanes that are under traffic. If traffic cannot be kept off the portion of the structure directly above the repair work, the shoring and cribbing shall be designed to support the dead load, superimposed dead load and full live load and impact shown in the plans. The jack capacity provided should be no less than 50% greater than this load.

The actual load applied shall not be larger than the dead load plus the superimposed dead load in order to avoid lifting the beam. Jacks can be used as cribbing if they are equipped with a locking mechanism that will allow the removal of the hydraulic pressure after the jacking has been completed.

2.10.6 Jack Placement

Jacking directly from the diaphragms is not allowed unless the diaphragms have been retrofitted or analyzed to carry jacking loads. Diaphragms should not be used as load carrying members in the jacking and cribbing system. The plans or special provisions should state this restriction when diaphragms are present.

When jacks are placed directly under a beam, the jack should be centered under the web and a steel plate should be placed between the top of the jack and the bottom flange of the beam. When web stiffeners bearing on the bottom flange do not exist directly over the location of the jack under a steel beam, hardwood timbers (4 x 4 or 6 x 6) should be installed tightly between the top and bottom flange to prevent flange rotation. If analysis indicates that the web does not have sufficient capacity to carry the expected loads, steel stiffeners will be required. Bolted steel angles in full contact with the bottom flange should then be installed. If bolted stiffeners are to be left in place, they should be properly painted to match the existing color of the beam. If stiffeners are to be removed, then holes in the web should be filled with H. S. bolts. Steel plates should be placed under jacks bearing directly on the existing substructure to distribute the jacking load and prevent damage to the existing concrete.

When lifting the entire superstructure as a unit, jacks should be placed in a manner and in locations that will ensure that the jacks will be equally loaded, and the load will be uniformly distributed to the foundation of the jacking system.

2.10.7 Load Distribution

Whenever the jacking system will require that lifting loads be transferred to natural ground, slopewall, roadway shoulder or roadway pavement, an adequate distribution of the load should be accomplished using steel beams, timber mats or other means approved by the Engineer. An aggregate leveling base may also be required below the mats when natural ground is used to support the jacking system.

The following maximum allowable pressures should be used to determine the area of the timber mats supporting jacking systems, unless information is available indicating that higher values may be used (note that the use of surface soil capacity methods like pocket penetrometers are not acceptable to override the limits shown below):

Supporting Material	Max. Allowable Pressure
Natural Ground (Unsaturated)	0.5 Ton / Sq. Ft.
Concrete Slopewalls & Bituminous Shoulders	1.0 Ton / Sq. Ft.
Bituminous pavements	2.0 Tons / Sq. Ft.
Concrete pavements	4.0 Tons / Sq. Ft.

Table 2.10.7-1: Maximum Allowable Bearing Pressures for Jacking System Support

2.11 Pin and Link Replacement

2.11.1 General

When pins are found to be defective or “frozen”, the pin and link assemblies must be replaced. These assemblies are considered to be Non-Redundant Steel Tension Member details for inspection and maintenance purposes, and measures should be taken to immediately correct any deficiencies associated with the pins or link plates.

2.11.2 Pin and Link Replacement

When replacing defective pins, the notes shown in Figure 2.11.2-1 and the features shown in Figure 2.11.2-2 and Figure 2.11.2-3 should be incorporated into the design of the replacement whenever possible. Also, new link plates, designed in accordance with AASHTO specifications, should always be provided when replacing defective pins.

The suspended span must be temporarily supported from below with shoring or from above using a needle beam. General details for temporarily supporting the suspended span using a needle beam are shown in Figure 2.11.2-4. Specific details for the temporary support system are typically to be designed by the contractor’s structural engineer and submitted for review and approval by the Bureau of Bridges & Structures and are not shown in the plans. However, pay items and special provisions for the temporary support system should be included with the repair plans.

A Reaction Table, similar to that required for bearing replacements, should be included with the pin and link replacement plans.

Note A:
Existing welds shall be inspected for cracks using liquid dye penetrant or magnetic particle testing. Any cracks that are found shall be identified and reported to the Bureau of Bridges and Structures for further disposition. Clean and paint before installing new link plates.

Note B:
Bore diameter for bushing in link plate, existing webs and web reinforcement plates shall correspond to bushing manufacturer's allowable tolerances for proper functioning. Hole diameter may be adjusted to allow use of stock bushings.

Note C:
Inside face of new link plates shall receive first field coat in shop. The primer shall pass the M.E.K. Rub Test before the first field coat is applied.

Note D:
Actual bushing thickness per manufacturer's specifications, ¼" is approximate. Bushings shall be a self lubricating filament wound epoxy matrix backed Duralon Bearing, metal backed Fiber Glide Bearing or equivalent. No primer or grease shall be allowed on bushings. Bushings shall be suitable for dynamic loads of 20,000 psi.

Note E:
Tighten inside nuts to bring all bushings into firm contact, then back off ¼ turn and tighten outer nuts.

Note F:
Apply ⅜" bead to face of the web reinforcing plates approximately ½" from bushing immediately before installing new link plates. Place sealant around nuts after installation. Sealant shall be suitable for prolonged exterior exposure without losing flexibility or adhesion to painted steel surfaces. Proposed products shall be subject to Department's acceptance based on documented testing or other evidence.

Note G:
Body of Pin dimension "a" shall be based on measured thickness of captured plates (including paint), plus ½". $a = 2(tl) + tw + ½"$

Note H:
Nominal Pin diameter (diameter tolerances subject to Specifications of Teflon Bushing Manufacturer and shall be approved by the Engineer). Pin shall be ASTM A276, UNS 21800 (Nitronic 60 or equal) (No step at threads) 12 threads per inch. Install prior to new link plates.

PIN REPLACEMENT DETAIL NOTES

Figure 2.11.2-1: Pin Replacement Detail Notes

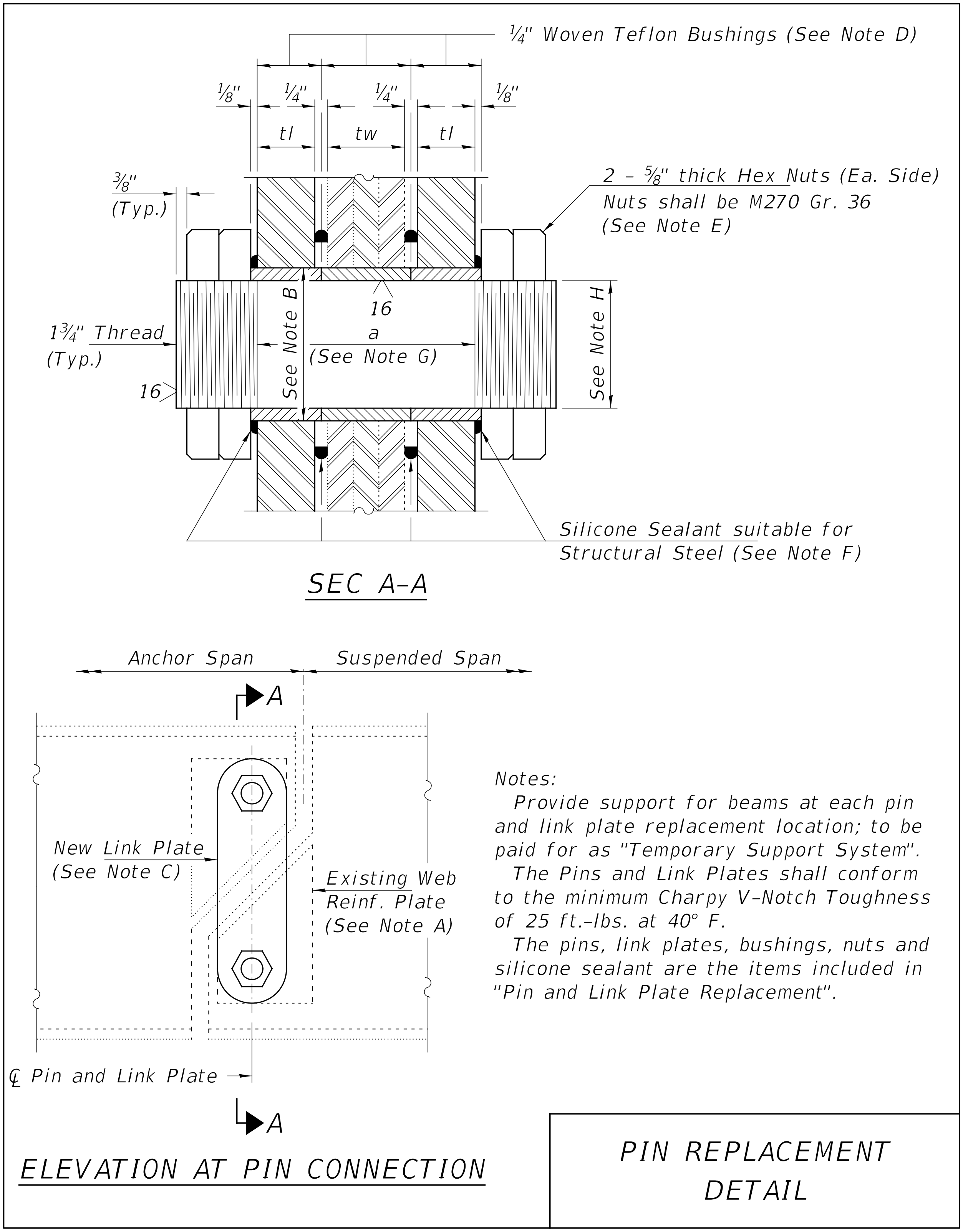
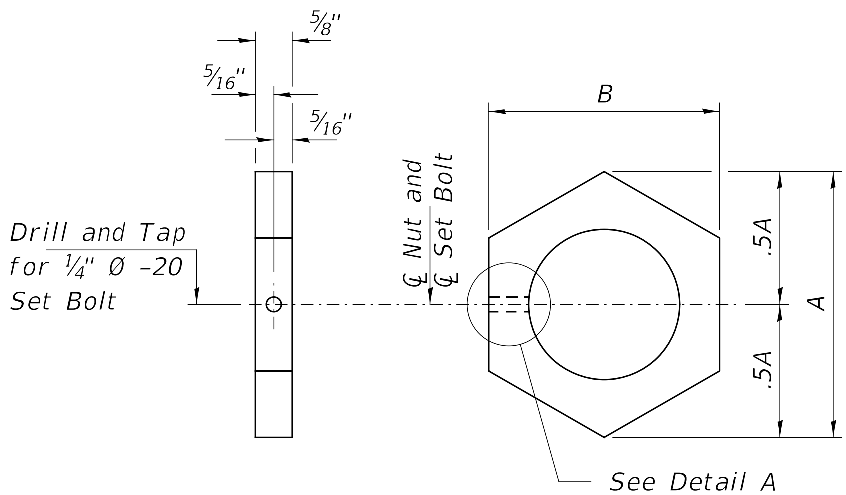
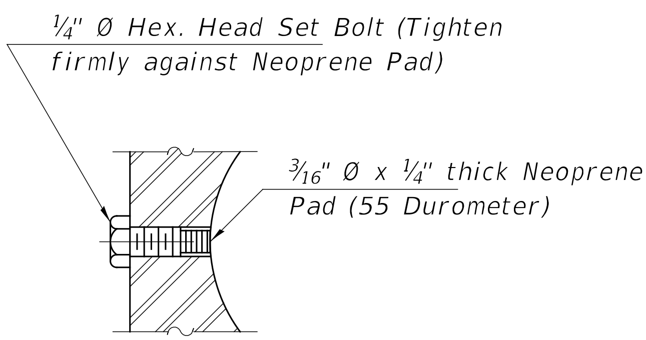


Figure 2.11.2-2: Pin Replacement Detail



EXTERIOR NUT DETAIL



DETAIL A

Set Bolts shall conform to the requirements of ASTM A 307 and shall be galvanized according to AASHTO M 232.

SET BOLT FOR
PIN AND LINKS

Figure 2.11.2-3: Set Bolt for Pin and Links

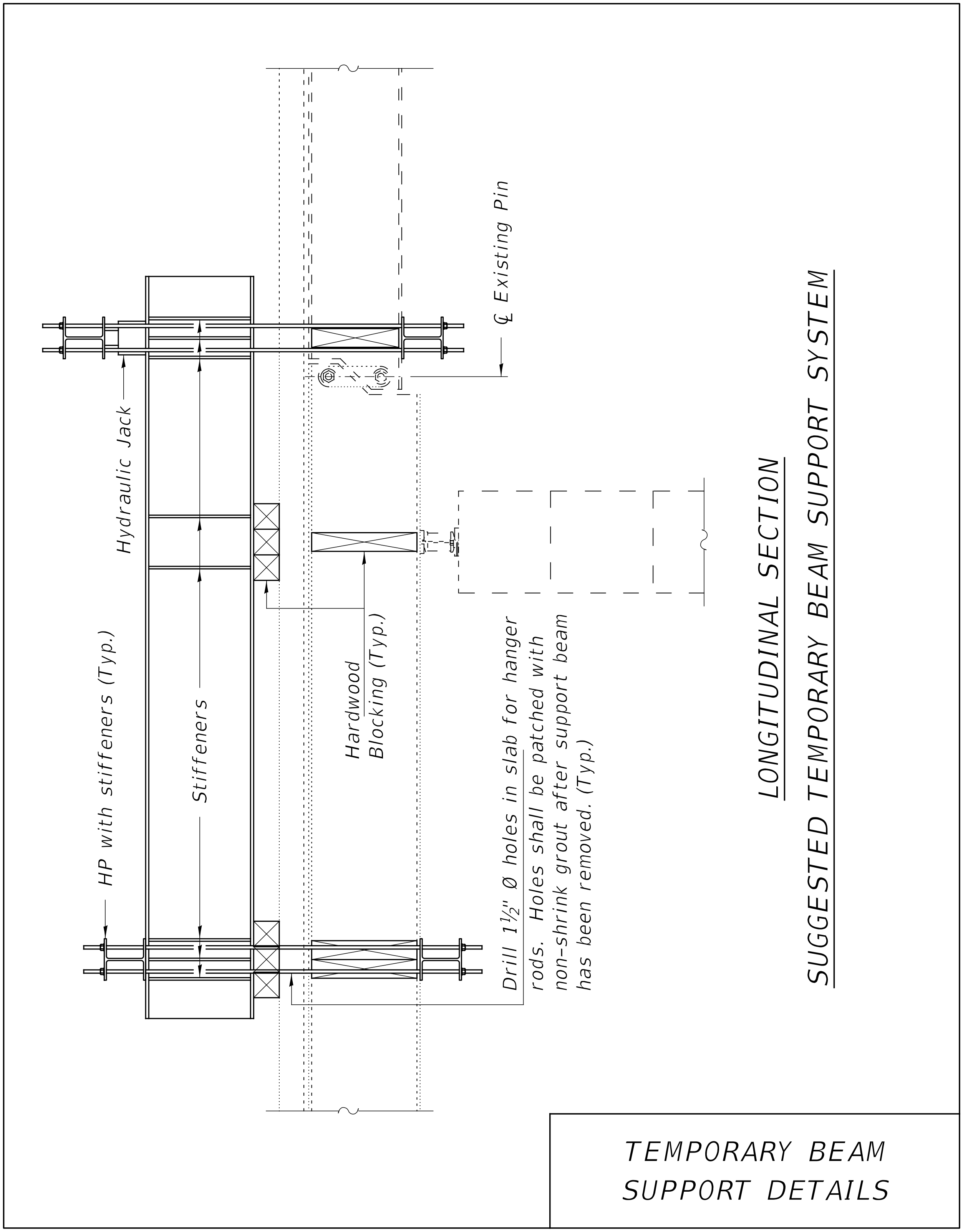


Figure 2.11.2-4: Temporary Beam Support Details

2.11.3 Pin and Link Elimination

When the major rehabilitation of a structure is planned, the elimination of the pin and link assemblies should be evaluated. The method used to eliminate the assemblies will depend on the characteristics of the individual structures and may include replacing the assemblies with bolted splices and the installation of counterweights at the simply supported ends of spans. This will eliminate a Non-Redundant Steel Tension Member detail from the structure and reduce future maintenance and inspection requirements.

2.12 Fatigue and Fracture

2.12.1 Fatigue

"Fatigue" is the general term used to indicate the eventual loss of ductility that may cause cracking in a steel member after repeated loading. Some fatigue cracks happen suddenly, their behavior is unpredictable and can have a serious impact on the ability of the member to carry the expected loads. The fatigue cracking found in steel structures is generally associated with localized deformations and fabrication operations to which a member has been subjected. Nicks, gouges and cuts present on a member due to vehicle impacts or construction operations can eventually initiate cracking in a member. Tack welds used during construction to ease erection difficulties, "plug welds" used to fill mis-drilled holes and unauthorized field welding during maintenance operations can cause fatigue cracking. Fatigue cracks can also originate from internal defects within welds. Fatigue cracks are also frequently found in the area of copes which have been cut with too small of a radius or were poorly made.

Fatigue cracks can occur due to in-plane bending or out-of-plane bending. Cracks caused by in-plane bending stresses can occur in any tensile zone of a member. They are usually related to the presence of a weld or surface defect on the member. Fatigue cracks due to in-plane bending stresses can grow quickly and should receive immediate attention. Cracks caused by out-of-plane bending are usually associated with the welded connections of a secondary member to the primary load carrying member. These types of cracks are usually in the web and grow slowly except when located in a tensile zone of the primary member.

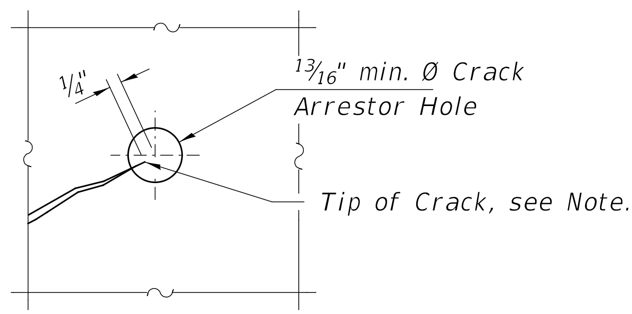
A common type of out-of-plane bending crack is found at the bottom of web stiffeners welded to the beam/girder web when there is a gap between the stiffener end and the top of the bottom flange. These cracks normally start at the tip of the stiffener and progress upwards along the stiffener to web weld or at a diagonal into the web.

A common type of fatigue cracking found in welded plate girders is related to "freezing" of articulated expansion bearings at the simply supported end of the girder. When a pin associated with the bearing "freezes", rotation of the girder over the bearing is restricted. A crack eventually forms between the bottom flange and the web plate along the flange to web weld.

2.12.1.1 Field Procedures

When cracks are found, the Bureau of Bridges & Structures should be immediately contacted. Information should be obtained and submitted describing the location and size of all cracks found so that the current structural adequacy and the procedures necessary to contain or repair the cracks can be determined.

The first step during the inspection of a fatigue crack is to locate the ends of the crack. This can be done by using dye penetrant or magnetic particle testing. Once the ends of the crack are found, 13/16 inch minimum diameter holes should be drilled as shown in Figure 2.12.1.1-1 to capture the crack ends to prevent crack growth while repair requirements are being determined or retrofit plans are being prepared. To ensure that the arresting hole captures the end of the crack, the hole should be centered 1/4 inch beyond the apparent crack end. After the arresting holes have been drilled, tests should be performed again to ensure that the drilled holes have captured the crack ends. A high strength bolt with washers should be installed and fully tightened, when possible, in the arresting hole to provide a compressive force in the area of the crack tip and to minimize distortions in the area of the crack that may encourage further growth of the crack.



CRACK ARRESTOR HOLE DETAIL

Note:
Locate crack tip using liquid dye penetrant or magnetic particle testing. Drill $^{13}/_{16}$" min. Ø Crack Arrestor hole at the crack tip. After crack arrestor hole has been drilled, dye penetrant or magnetic particle testing shall be used to verify that the drilled hole has captured the crack tip.

CRACK ARRESTOR HOLE

Figure 2.12.1.1-1: Crack Arrestor Hole

2.12.1.2 Repair Considerations

Cracks located in tensile stress areas of a primary load carrying member should be retrofitted to prevent further growth of the existing cracks and to prevent additional cracks from developing. The retrofit details should restore the load carrying capacity lost when cracking has significantly affected the section properties of the member.

Retrofits should be made using bolted details. All repair plates shall be ½ inch minimum thickness and bolts shall be high strength and ¾ inch minimum diameter. If welded details are proposed, the stresses in the primary load carrying member at the location to be welded must be considered to determine the effect that welding may have on the fatigue life of the member at that location. Also, if field butt welds are required, they must be tested using non-destructive methods and the method of testing must be specified in the plans.

Figure 2.12.1.2-1 shows details used to repair the end of a stringer where cracking has initiated due to low tensile stress repeatedly occurring in the area of a poorly coped web. Details for the repair of a beam when cracking occurs at the end of a welded coverplate are shown in Figure 2.12.1.2-2. A similar detail should be used to retrofit the ends of coverplates during bridge deck replacement projects. It is recommended to size the new retrofit plate to have an area at least equal to the flange of the beam and the number of connection bolts be chosen to develop the full capacity of the plate, rather than sizing the plate based on actual stresses or forces at the location of the plate.

When the cracking is due to out-of-plane bending, the connection of the secondary member to the primary member should be retrofitted to eliminate the cause of the cracking. This is usually achieved by retrofitting the connection details to make the connection more flexible in order to relieve out-of-plane bending stresses, or to stiffen the connection in order to prevent out-of-plane displacements. Of these two methods, the stiffening of the connection to resist out-of-plane displacement is usually the most effective in arresting and preventing crack growth. Figure 2.12.1.2-3 illustrates a method of increasing the flexibility of a floorbeam to girder connection by removing a portion of the connection plate and stiffener. Figure 2.12.1.2-4 includes notes for the procedure to remove the appropriate portions of the connection plate and stiffener. The procedure is intended to avoid damage to the girder.

The resistance of a connection to out-of-plane displacements can be increased as illustrated in Figure 2.12.1.2-5 (bottom flange) and Figure 2.12.1.2-6 (top flange).

To retrofit the cracks which form in the web near the bottom flange of girder ends located over “frozen” articulated bearings, a portion of the bottom flange and web plate must be repaired as shown in Figure 2.12.1.2-7. The defective bearing must also be replaced, in-kind, to prevent cracking from recurring. If all bearings at this bearing location are being replaced, then a new bearing type can be used.

Tack welds found in the tensile zone areas of a primary load carrying member should be removed by grinding. The grinding should be done parallel to the direction of the primary stress in the member when possible. After grinding the location shall be tested for cracks using dye penetrant or magnetic particle testing.

When plug welds are found in tensile zone areas, they should be documented and promptly reported to the Bureau of Bridges and Structures. Similar locations should be closely inspected for evidence of other plug welds. Plug welds may contain internal defects not easily detectible, may not exhibit fatigue cracking at the surface of the member, and any resulting weld failure may occur as fracture of the weld and member rather than slower fatigue crack growth. If plug welds are identified in a Non-Redundant Steel Tension Member, they should be removed. Regardless of the member classification, if a decision is made to eliminate the plug welds, details shall be prepared for their removal by coring a hole around the plug large enough to remove all of the weld material. After the coring is completed, the inside of the hole shall be tested for cracks using dye penetrant or magnetic particle testing.

If possible, high strength bolts with plate washers should be installed and fully tightened in the holes drilled to remove the plug welds.

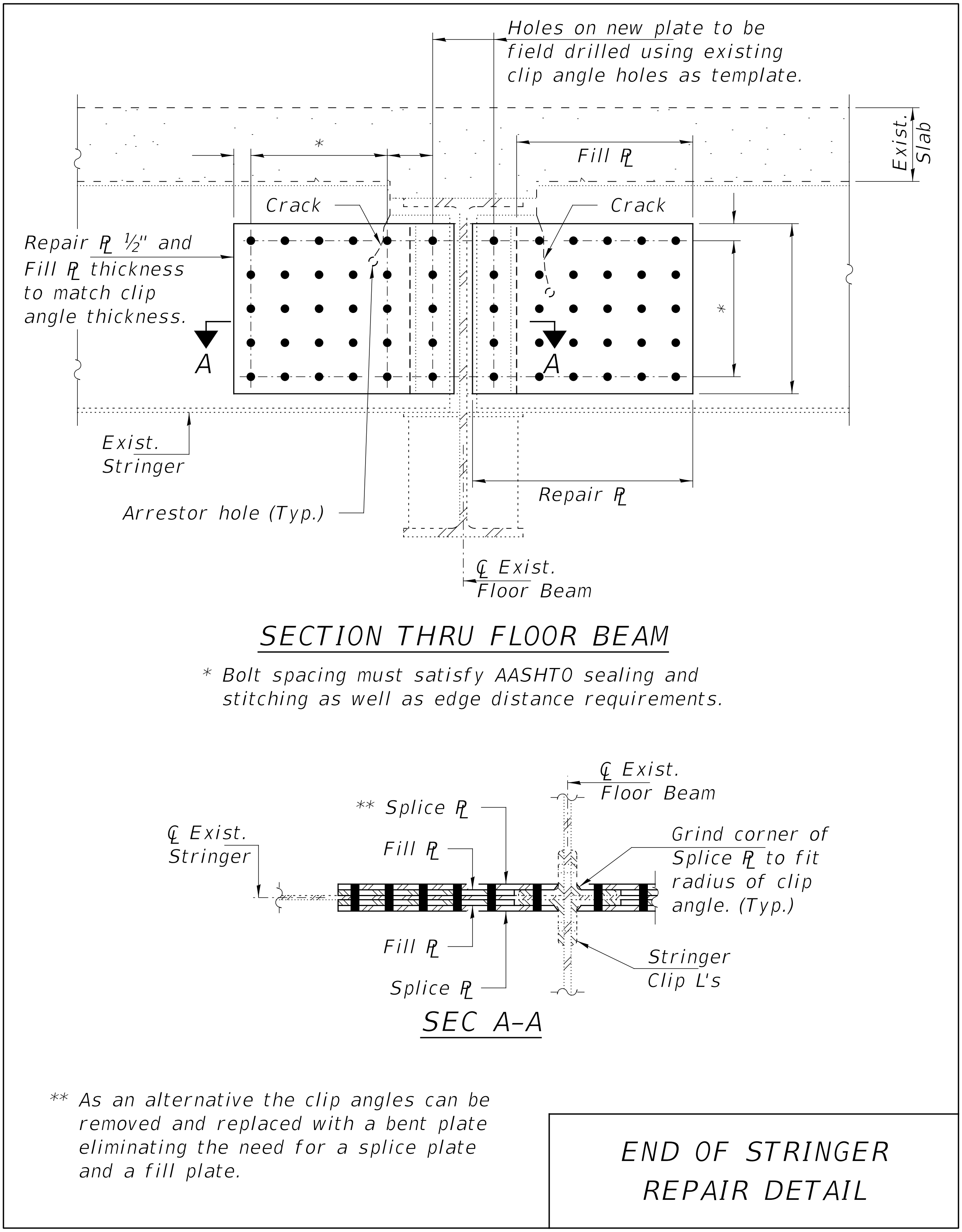


Figure 2.12.1.2-1: End of Stringer Detail

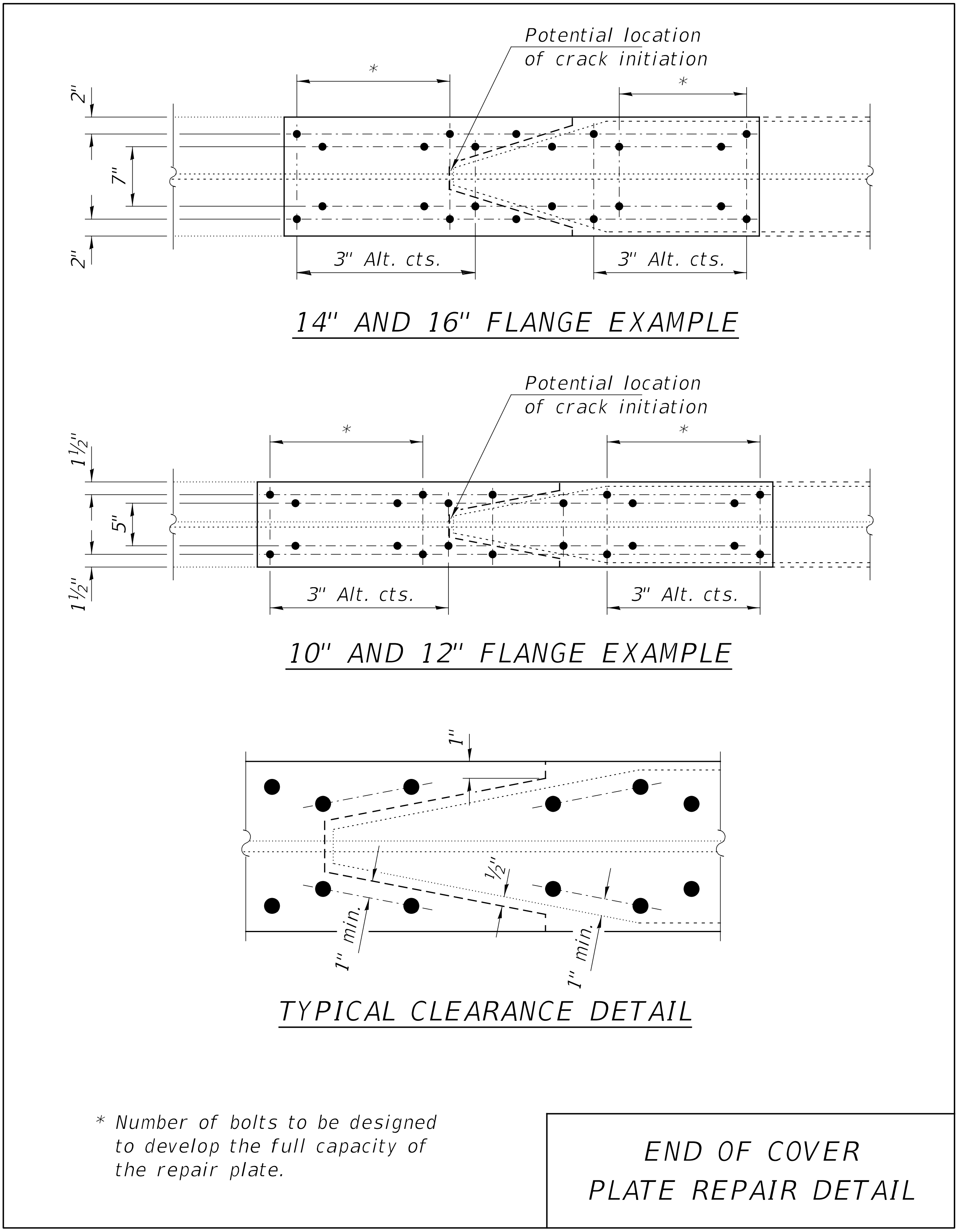


Figure 2.12.1.2-2: End of Cover Plate Repair Detail

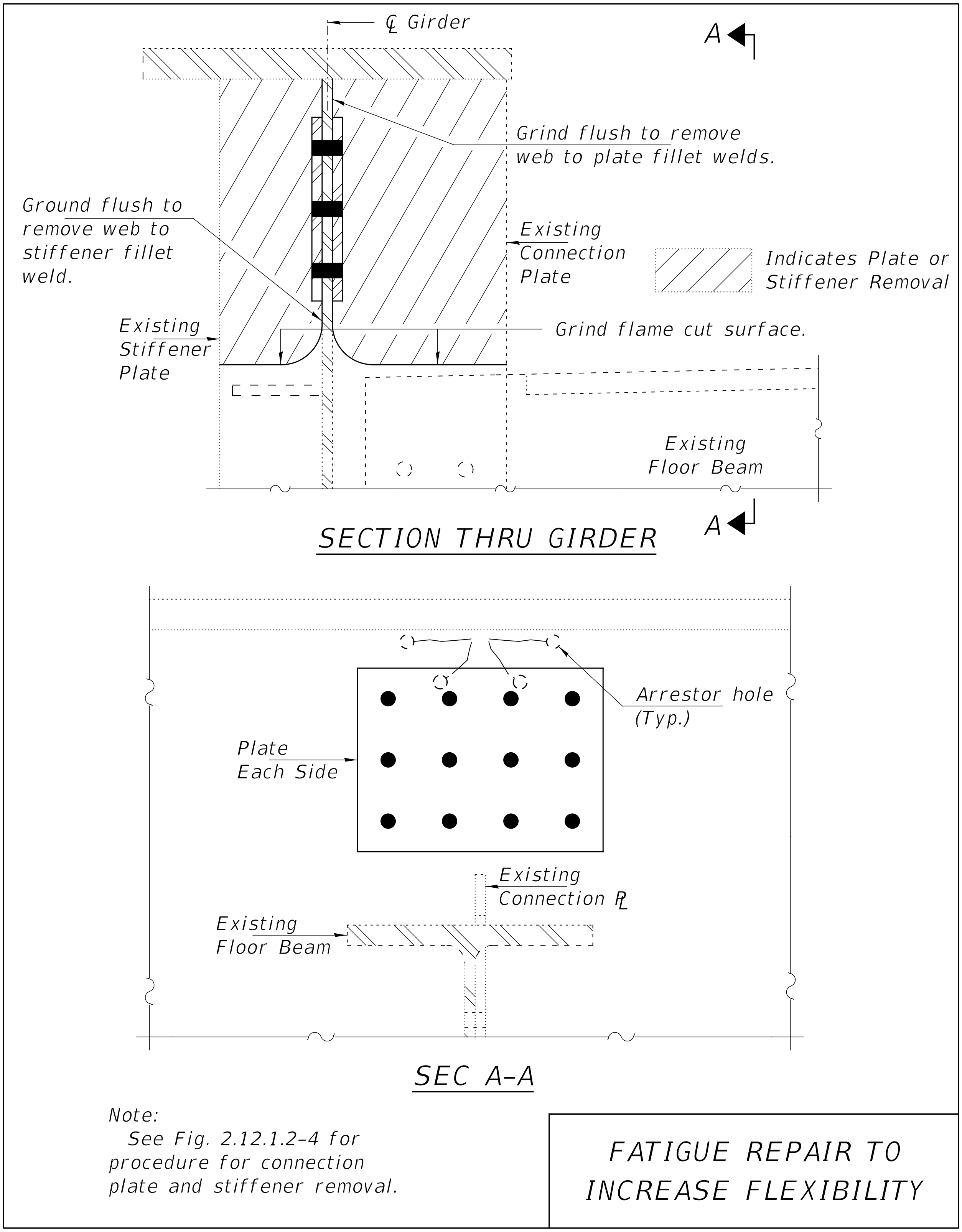


Figure 2.12.1.2-3: Fatigue Repair to Increase Flexibility

PROCEDURE FOR STIFFENER AND CONNECTING PLATE REMOVAL

1. Cut existing connecting plate and stiffener along web as shown, with a 1" R (Min.) at web. The minimum distance from cut to face of web shall be the larger of ¼" or web to plate weld size, with removal of remaining material by grinding as described below. The cut shall be made parallel to the web and flanges without angling the cut towards the web or flanges. Equipment and method of cutting shall be approved by the Engineer. Any method of removal to be used shall ensure that no damage is done to the existing web or flanges. Cutting shall be done in a manner such that the paint on the opposite face of the web is not damaged. If damage to the paint occurs due to cutting, the damaged area shall be repainted at the Contractor's expense and procedures shall be modified to prevent damage at subsequent removal locations.

2. Remove material between cut and web by grinding and grind smooth at web surfaces and cut end of connecting plate and stiffener. Web plate surfaces and cut end of connecting plate and stiffener shall have a roughness overage (Ra) of 250 µ in or less. Grinding equipment shall be approved by the Engineer. The grinding operation should not gauge the girder web plate.

3. The web and flanges surface at the modification shall be inspected using dye penetrant or magnetic particle (MT) methods. Any cracks found shall be identified and reported to the Bureau of Bridges and Structures for further disposition.

4. The exposed steel surfaces shall be cleaned and painted using an aluminum epoxy mastic primer according to Article 506.05 of the Standard Specifications.

PROCEDURE FOR REMOVAL OF CONNECTING PLATE AND STIFFENER

Figure 2.12.1.2-4: Procedure for Removal of Connecting Plate and Stiffener

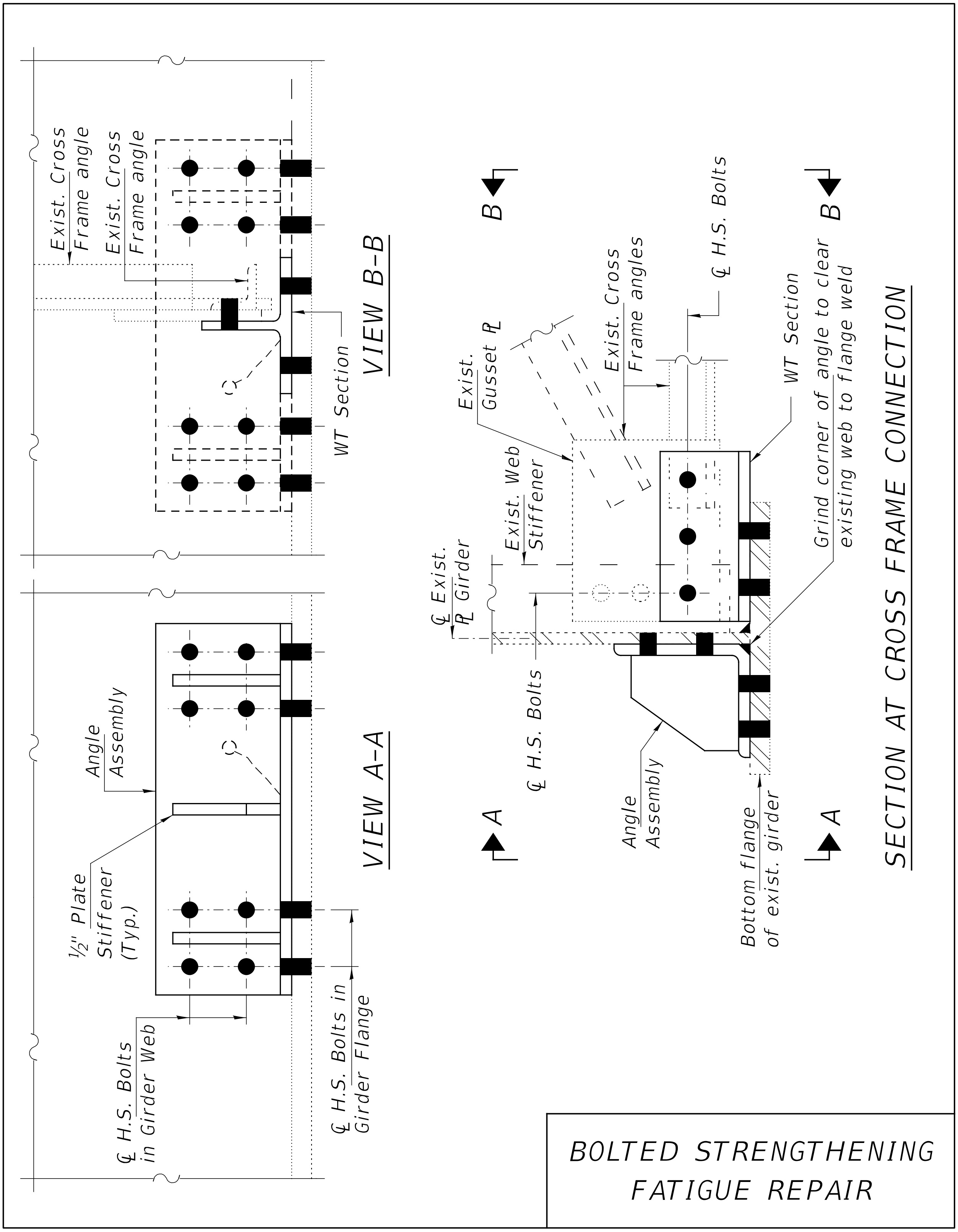


Figure 2.12.1.2-5: Bolted Strengthening Fatigue Repair

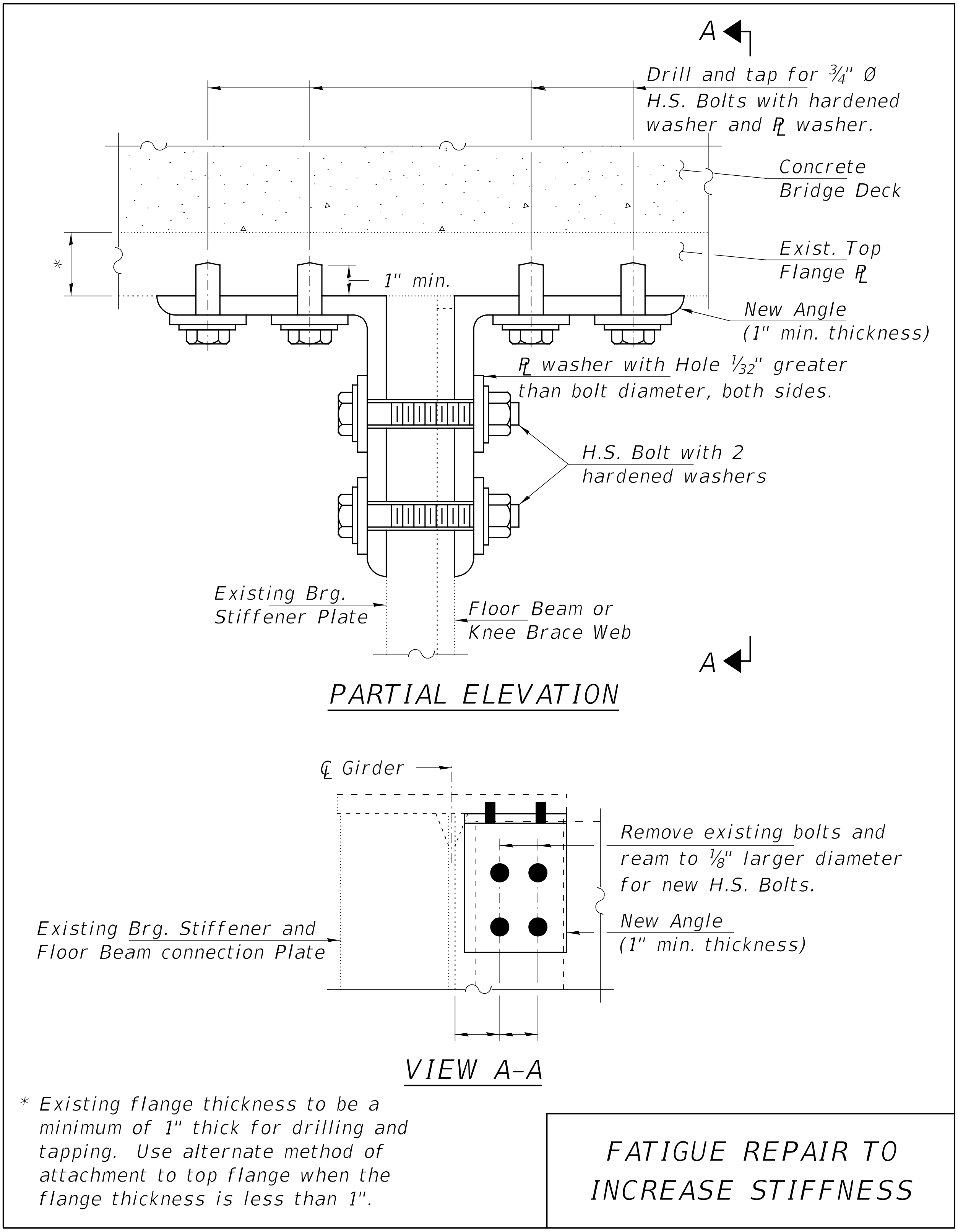


Figure 2.12.1.2-6: Fatigue Repair to Increase Stiffness

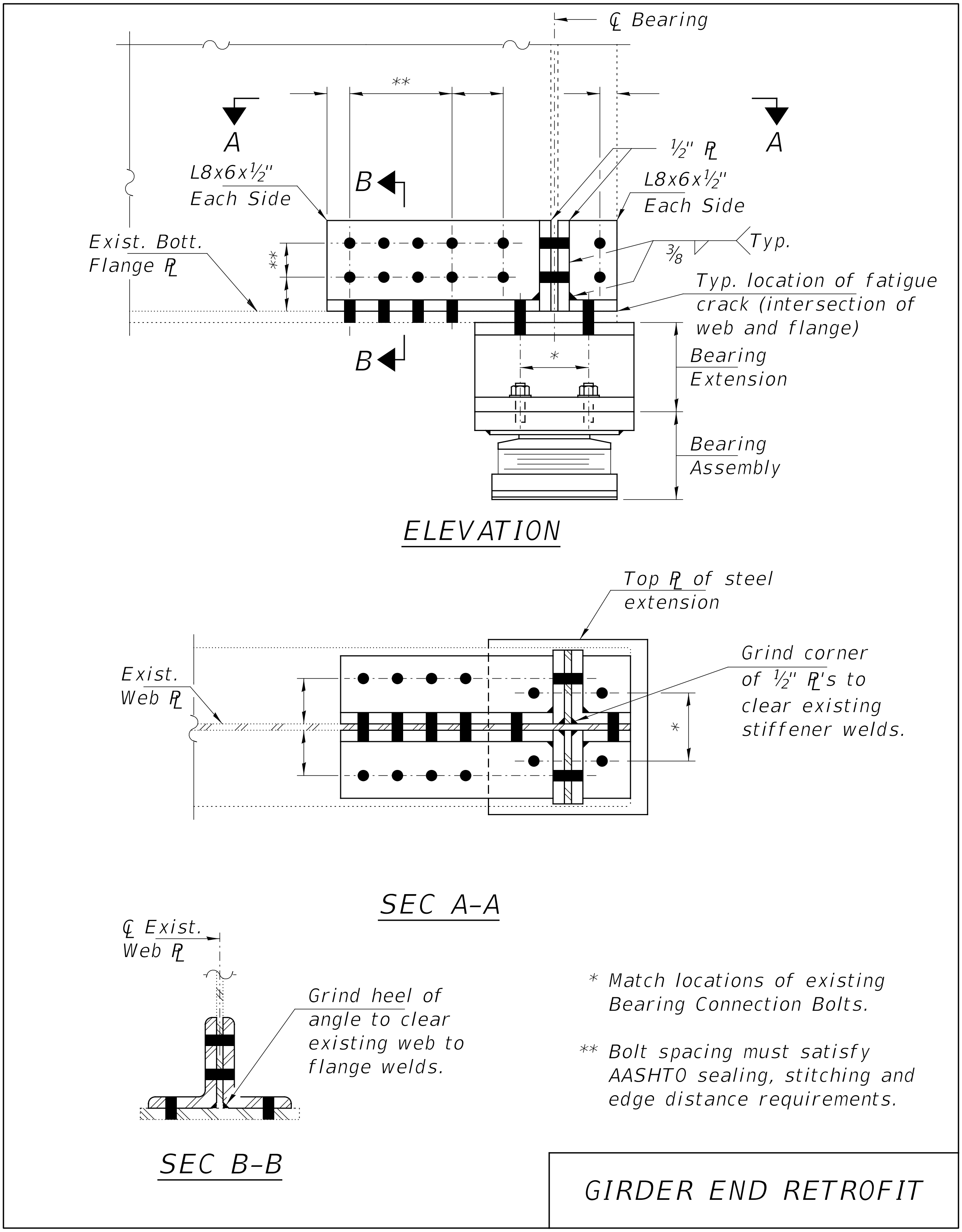


Figure 2.12.1.2-7: Girder End Retrofit

2.12.2 Fracture

2.12.2.1 General

Although uncommon, under certain conditions, brittle fracture in steel members can occur suddenly, resulting in large cracks. It can occur without warning and without perceptible fatigue crack growth, which would typically occur at a much slower rate.

Triaxial constraint is a condition that can lead to Constraint-Induced Fracture and is created by intersecting or nearly intersecting longitudinal and transverse attachments and their welds (less than ¼” gap between weld toes). Triaxial constraint prevents yielding and redistribution of local stress concentrations in these small, highly constrained areas where longitudinal and transverse attachments and their welds intersect, due to restraint of the material in multiple directions by the surrounding material that is experiencing a lower stress level. Additionally, residual stresses from the welding and cooling process alone can be near yield stress.

Steel girder webs often have welded attachments such as longitudinal stiffeners, vertical stiffeners, stiffeners used for connection of cross frames or diaphragms and connection plates for wind bracing. Details with intersecting or nearly intersecting longitudinal and transverse welds (less than ¼” gap between weld toes) can create highly constrained joints and notch-like or crack-like geometric discontinuities that are susceptible to Constraint-Induced Fracture under service loads, in regions of a girder web that are subject to tension (see Figure 2.12.2.2-1 for an illustration of these conditions). Stress concentrations in the narrow web gap between longitudinal and transverse welds occur at these notch-like or crack-like planar discontinuities as longitudinal stress flows from the heavier, longitudinally stiffened section into the area of web just past the end of the discontinuous longitudinal stiffener or notch in a wind bracing connection plate. The high stress concentrations occurring at the location of triaxial constraint can lead to sudden, brittle fracture under service conditions, when ductile behavior would otherwise be expected.

This type of detail and cracking was documented during investigation into the cracking that occurred in the girders of the Hoan Bridge in Milwaukee, Wisconsin in 2000.

Current AASHTO LRFD *Bridge Design Specifications* include provisions and detailing guidance for new bridges to ensure proper separation of welds to avoid Constraint-Induced Fracture, including minimum dimensions for weld gaps and detailing criteria for longitudinal elements to be continuous at the intersection of longitudinal and transverse welded attachments in areas of the web subject to a net tensile stress. Fracture is less likely to occur if the attachment parallel to the primary stress is continuous and the transverse attachment is discontinuous.

The National Steel Bridge Alliance (NSBA) *Guide to Evaluating Details for Susceptibility to Constraint-Induced Fracture* is a resource for more detailed information on problematic details. The guide also provides a method for evaluating and quantifying details in both new and existing bridges for susceptibility to Constraint-Induced Fracture.

Although uncommon, other weld features sometimes used in the past that can create the potential for brittle fracture include plug welds and discontinuous backing bars inside enclosed members such as box girders. If these weld features are found in tension members or portions of a member subject to tension, they should be evaluated to determine the need for retrofit.

2.12.2.2 Retrofit and Repair

Although Illinois' inventory of bridges has been reviewed to identify and retrofit those with intersecting welds, if a bridge is found to have welded attachments with intersecting or nearly intersecting longitudinal and transverse welds with less than ¼ inch gap between the weld toes, a retrofit of the intersecting members should be implemented at locations on the girders that experience net tension. For new bridges, the AASHTO LRFD *Bridge Design Specifications* state the distance between weld toes is recommended to be ¾ inch and shall not be less than ½ inch; however, for in-service bridges a gap of ¼ inch has been found sufficient to avoid Constraint-Induced Fracture and can be used as the minimum required to determine if an existing structure requires retrofit.

Figure 2.12.2.2-2 provides retrofit details and procedures for providing adequate separation between welds on a longitudinal web stiffener and a transverse stiffener.

Figure 2.12.2.2-3 show details and procedures for the retrofit of intersecting welds at a connecting plate used for lateral bracing or other attachments.

For short, welded attachments such as wind bracing complete removal of the weld and reattachment with the use of a bolted connection is also an option. Other options include removal of the lateral bracing and gusset plates if detailed analysis shows it is not needed. If the lateral bracing is needed, relocating the lateral system attachment to the bottom flange with a bolted connection is also an option that can be considered.

In the event a large Constraint-Induced Fracture is discovered in an in-service bridge, it would be considered a Critical Finding and handled as described in Section 3. Such cracking would likely require closure of the structure. Repair of such a crack may involve arresting the crack and placement of a full girder splice at the location, with sufficient bolts on both sides of the crack to carry the expected loads, or partial or complete member replacement.

When plug welds are found in tensile zone areas, they should be documented and promptly reported to the Bureau of Bridges and Structures. Similar locations should be closely inspected for evidence of other plug welds. Plug welds may contain internal defects not easily detectible, may not exhibit fatigue cracking at the surface of the member, and any resulting weld failure may occur as fracture of the weld and member rather than slower fatigue crack growth. If plug welds are identified in a Non-Redundant Steel Tension Member, they should be removed. Regardless of the member classification, if a decision is made to eliminate the plug welds, details shall be prepared for their removal by coring a hole around the plug large enough to remove all of the weld material. After the coring is completed, the inside of the hole shall be tested for cracks using dye penetrant or magnetic particle testing.

If possible, high strength bolts with plate washers should be installed and fully tightened in the holes drilled to remove the plug welds.

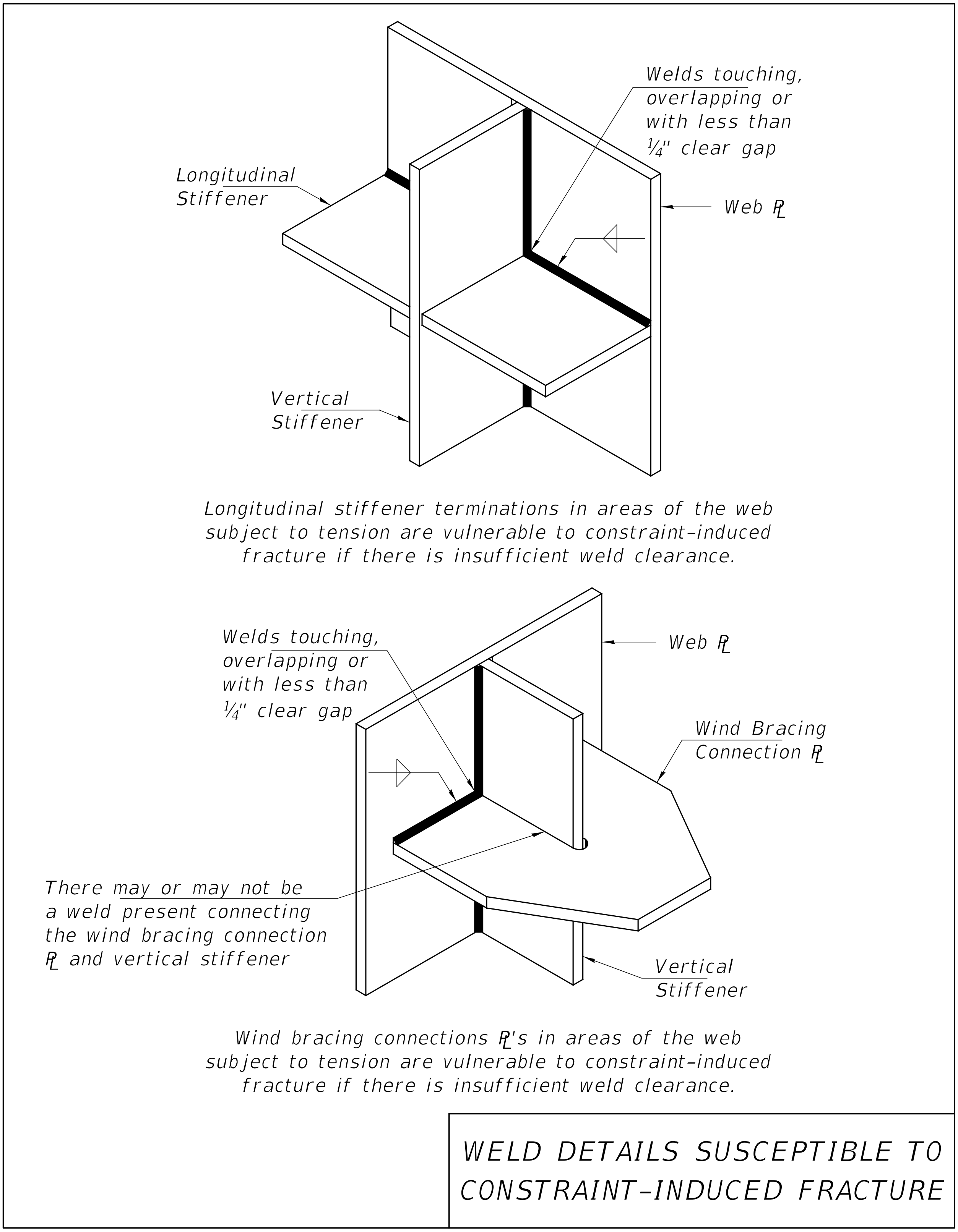
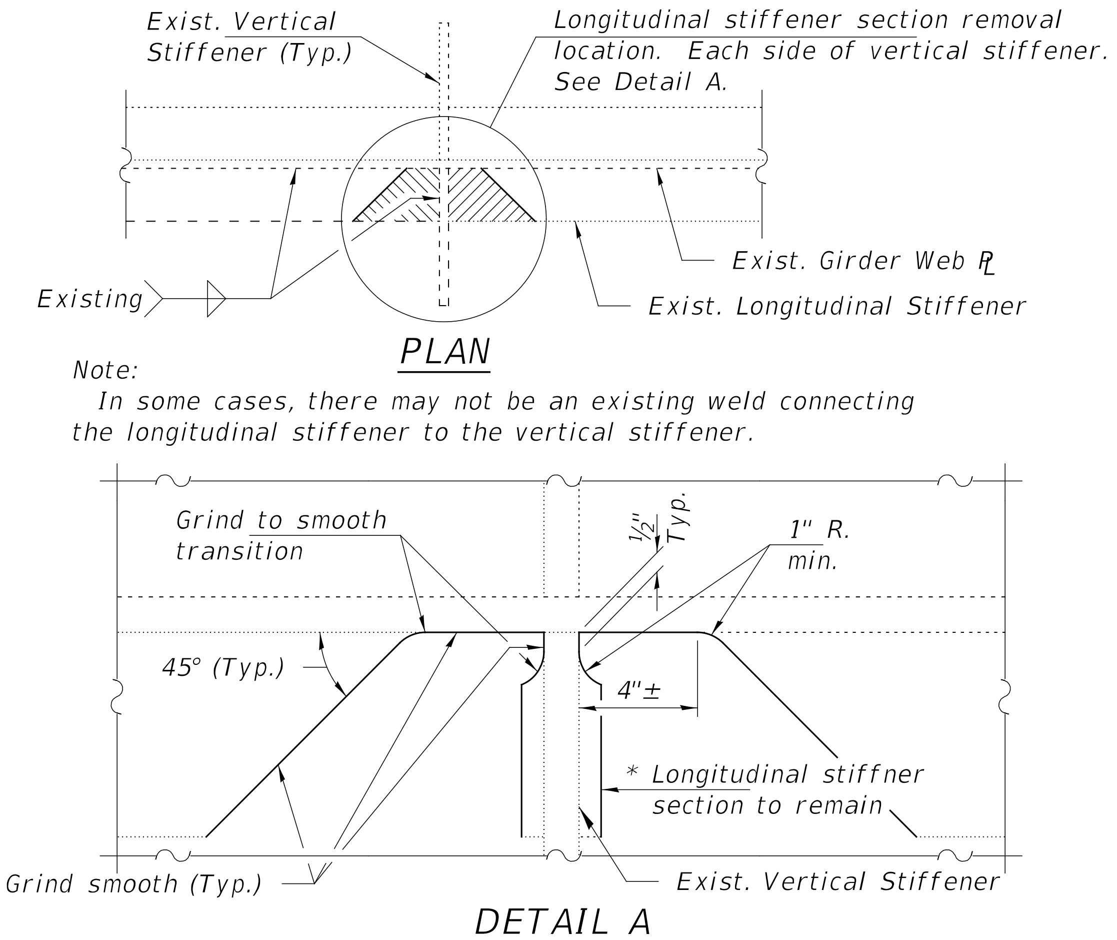


Figure 2.12.2.2-1: Weld Details Susceptible to Constraint-Induced Fracture



* Only for cases with an existing weld connecting the longitudinal stiffener to the vertical stiffener.

Procedure:

1. Remove material between longitudinal stiffener and vertical stiffener by grinding with carbide tools and die grinder. Remove all burrs from cut edge and check for irregularities.

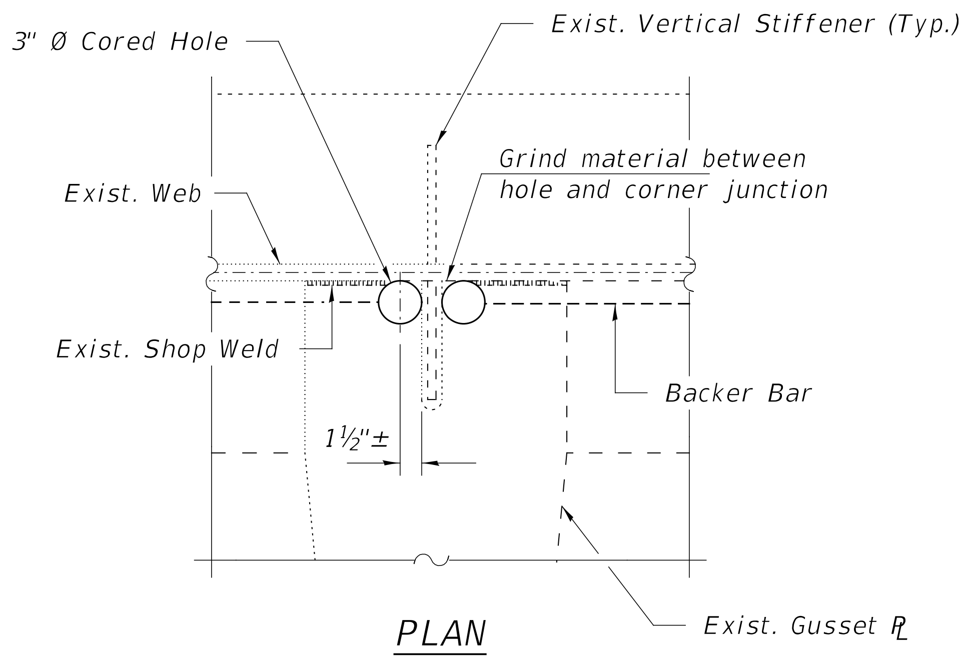
2. Web plate surface shall have a roughness average (Ra) of 250µ in. or less.

3. After burr removal, the web area shall be inspected for cracks using magnetic particle (MT) or Dye Penetrant methods. Any cracks found shall be identified and reported to the Repairs Unit of the Bureau of Bridges and Structures for further disposition.

4. The exposed steel surfaces shall be cleaned and painted using an aluminum epoxy mastic primer.

RETROFIT DETAIL FOR WELD INTERSECTION AT LONGITUDINAL STIFFENER

Figure 2.12.2.2-2: Retrofit Detail for Weld Intersection at Longitudinal Stiffener



Procedure:

1. Core 3" Ø hole thru Gusset PL and Backer Bar.

2. Remove material between core and intersecting junction by grinding with carbide tools and die grinder. Remove all burrs from cut edge and check for irregularities.

3. Web plate surface shall have a roughness average (Ra) of 250µ in. or less. Cored surfaces shall have an Ra of 250µ in. or less. Check for symmetry on each side of stiffener. Repeat step 2 and 3 if necessary for symmetry.

4. After burr removal, the web area shall be inspected for cracks using magnetic particle (MT) or Dye Penetrant methods. Any cracks found shall be identified and reported to the Repairs Unit of the Bureau of Bridges and Structures for further disposition.

5. The exposed steel surfaces shall be cleaned and painted using an aluminum epoxy mastic primer.

RETROFIT DETAIL FOR WELD INTERSECTION AT GUSSET PLATE

Figure 2.12.2.2-3: Retrofit Detail for Weld Intersection at Gusset Plate

2.13 Impact Repairs - Steel Beams

2.13.1 Determination of Repair Requirements

Based on the inspection information obtained in accordance with the procedures provided in the Repair Section for the field inspection of impact damaged steel structures, a determination can be made to:

1. Straighten a damaged beam or girder.
2. Straighten and strengthen the damaged portion of the beam or girder.
3. Replace the damaged portion of the beam or girder.

The guidelines presented in the National Cooperative Highway Research Program (NCHRP) Report 271 should be used when determining which of the above actions should be taken to repair an impact damaged steel structure.

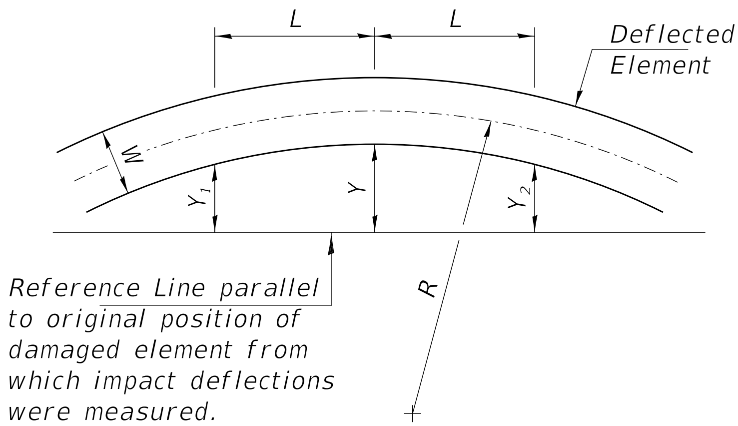
The NCHRP Report 271 guidelines for choosing a repair method are based on determining the amount of strain the impact has placed in the steel. The strain induced at any point on the member can be approximated by using the measured impact deflections adjacent to that point to determine a radius of curvature as illustrated in Figure 2.13.1-1.

Based on the approximate strain, the following NCHRP guidelines should be used to determine the repair requirements for primary tension members:

1. If the strain in the member is equal to or less than the yield point strain of the steel, straightening is not required unless necessary to restore the structure to its original appearance.
2. If the strain in the member is greater than the yield point strain but less than or equal to 15 times the yield point strain, the member can be straightened without strengthening.
3. If the strain in the member is greater than 15 times the yield point strain but less than 5 percent nominal strain, the member can be straightened without strengthening. However, if the damage is located in a highly fatigue susceptible area (lower than category C) the beam should be strengthened by adding material to the beam to provide 50 percent additional cross-sectional beam area in the damaged highly fatigue susceptible area.
4. If the strain in the member is greater than 5 percent nominal strain and straightening is attempted, the beam must be strengthened after straightening to provide additional cross sectional beam area equivalent to the existing beam (100 percent strengthening). Beams with extensive damage exceeding 5 percent nominal strain are typically replaced.

The repair requirement guidelines are illustrated graphically using a stress and strain diagram in Figure 2.13.1-2. Using approximated impact strains should be considered an aid rather than an exact solution for determining repair requirements. The age of the structure, visibility of the structure to the travelling public, roadway closure requirements during repair operations, plans for future replacement or rehabilitation of the structure and other factors peculiar to each bridge impact situation should be considered when determining the extent of member repairs or replacement required.

The guidelines given for determining repair requirements for primary members in tension or bending are based on the effect of the damage on the serviceability or fatigue life of the member. Compression members are not subject to the same guidelines and may be straightened without strengthening regardless of the impact strain unless fracture has occurred, or deformations are too large to allow the member to be adequately straightened.



CALCULATION OF DAMAGE CURVATURE

$$R \cong \frac{L^2}{Y_1 + Y_2 - 2Y}$$

L = increment lengths at which deflection measurements were taken.
Y = offset of deflected element from reference line.
R = approximate radius of curvature at location of offset Y.

MINIMUM ALLOWABLE RADII OF CURVATURE

Radius of Curvature at Yield Strain:

$$R_y = \frac{WE}{2F_y}$$

Approximate Radius of Curvature at 15 x Yield Strain:

$$R_{15y} = \frac{WE}{15 \times 2F_y}$$

Approximate Radius of Curvature at 5% Strain:

$$R_5 = \frac{W}{0.1}$$

W = the thickness of the deflected element measured in the direction of the deflection offsets measurements.
E = Modulus of Elasticity of the damaged Steel.
F_y = Yield point stress of the damaged Steel.

The information shown in this figure is a summary of the information and procedures presented in NCHRP Report 271. When the radius calculated exceeds the radius of curvature at yield, the calculated radius should be considered an approximation for use in determining repair needs. Persons using this information should obtain a copy of the report in order to familarize themselves with the assumptions made in the derivation of this procedure.

DAMAGE CALCULATIONS BASED ON NCHRP 271

Figure 2.13.1-1: Damage Calculations Based on NCHRP 271

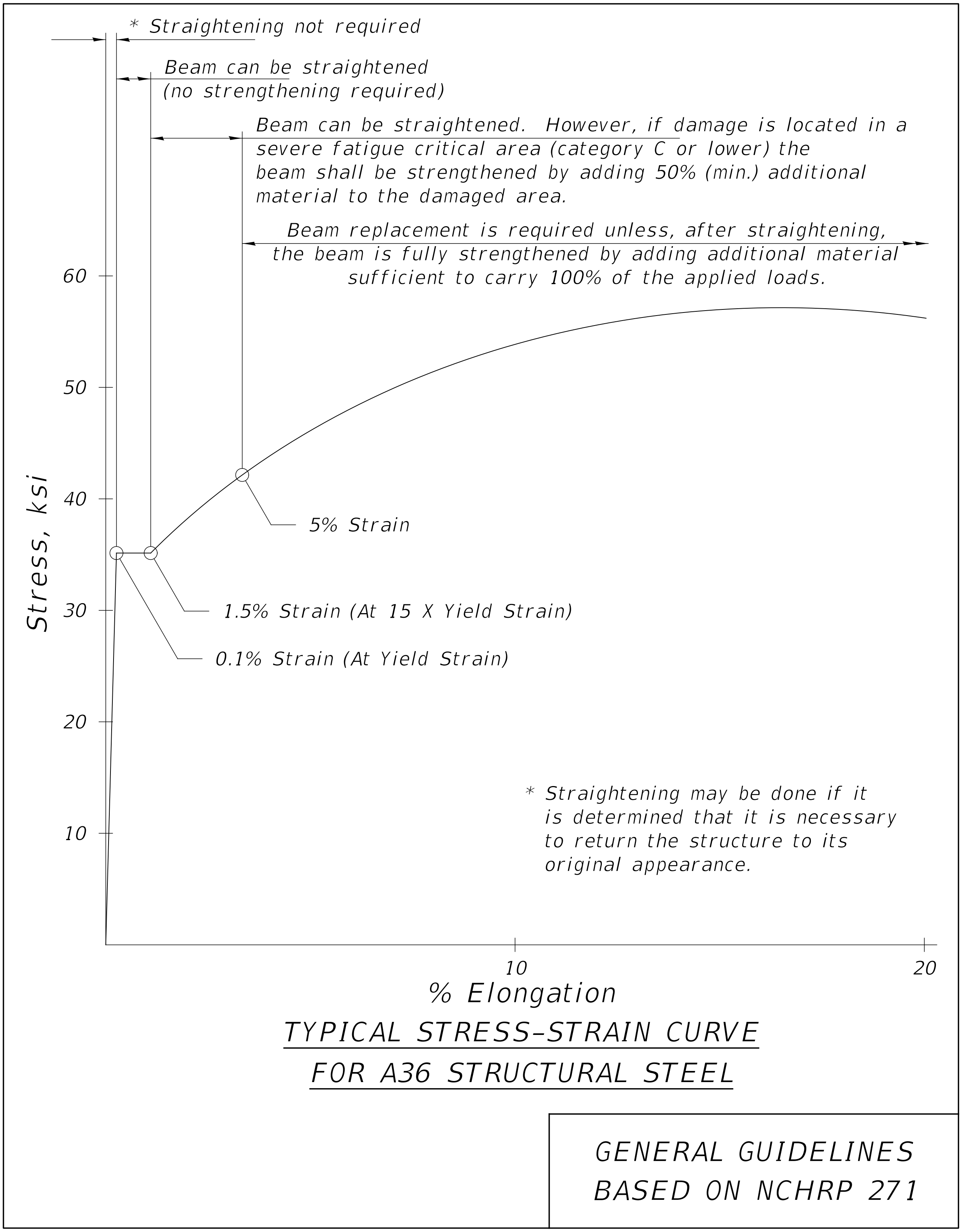


Figure 2.13.1-2: General Guidelines Based on NCHRP 271

2.13.2 Beam Straightening and Strengthening

At the present time, IDOT is straightening impacted beams using mechanical methods only. In a very few instances, because of very restrictive traffic requirements on some major highways, the use of heat-straightening has been allowed. This method requires the designer to develop elaborate heating patterns to be implemented by specialized contractors with proven vast experience on this type of work.

The mechanical straightening of damaged beams is accomplished using timber bracing, jacks, cables and winches between the beams and/or ground to push or pull the damaged beam back to its original configuration. Typical details associated with beam straightening are shown in Figure 2.13.2-1, Figure 2.13.2-2, and Figure 2.13.2-3. After straightening, the configuration of the damaged beam should meet the tolerances specified for new fabrication in the current issue of the IDOT Standard Specifications for Road and Bridge Construction.

After straightening operations are completed, welded connections in the damaged area must be inspected and welds should be nondestructively tested using magnetic particle or dye penetration testing at locations where visible evidence of damage exists. If cracks are found after straightening, further retrofitting of the member will be necessary.

When strengthening is necessary after straightening or grinding the damaged area, bolted plates should be used. To simplify the design and provide a conservative approach, it is recommended that the strengthening plates have the same capacity as the flange being strengthened. The plates should be placed continuously through the entire damaged area where the impact strains indicate that strengthening is required, with sufficient bolts to properly seal and stitch the strengthening plates to the damaged beam according to AASHTO specifications. The strengthening plates should extend beyond the above-mentioned area for a distance sufficient to install the required number of bolts in the flange and/or web to develop the capacity of the strengthening plates. These requirements are illustrated in Figure 2.13.2-4. As shown in the figure, when strengthening a bottom flange, plates can be placed on the top and bottom surface of the flange to limit the thickness of the bottom plate, thereby limiting the reduction in vertical clearance to the extent possible.

Gouges caused by the impact should be ground to eliminate sharp or sudden irregularities in the beam surface. Grinding should be done in such a way as to provide a smooth transition with a maximum slope of 3:1 between the damaged and undamaged surfaces as illustrated in Figure 2.13.2-5. Long or deep gouges should be evaluated to determine if it will be necessary to strengthen the gouged area after grinding. After grinding, the gouged area should be checked for cracks using nondestructive testing methods.

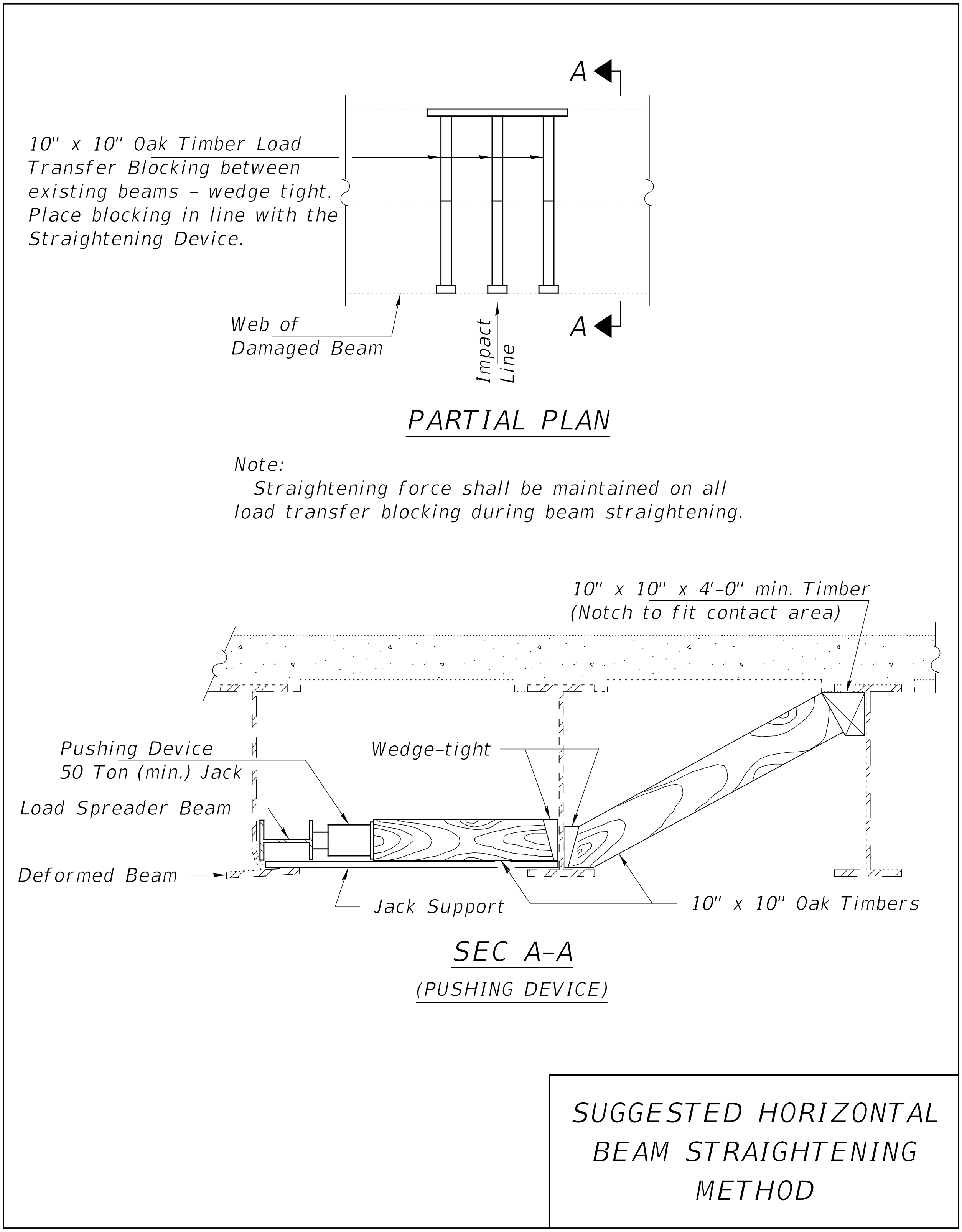
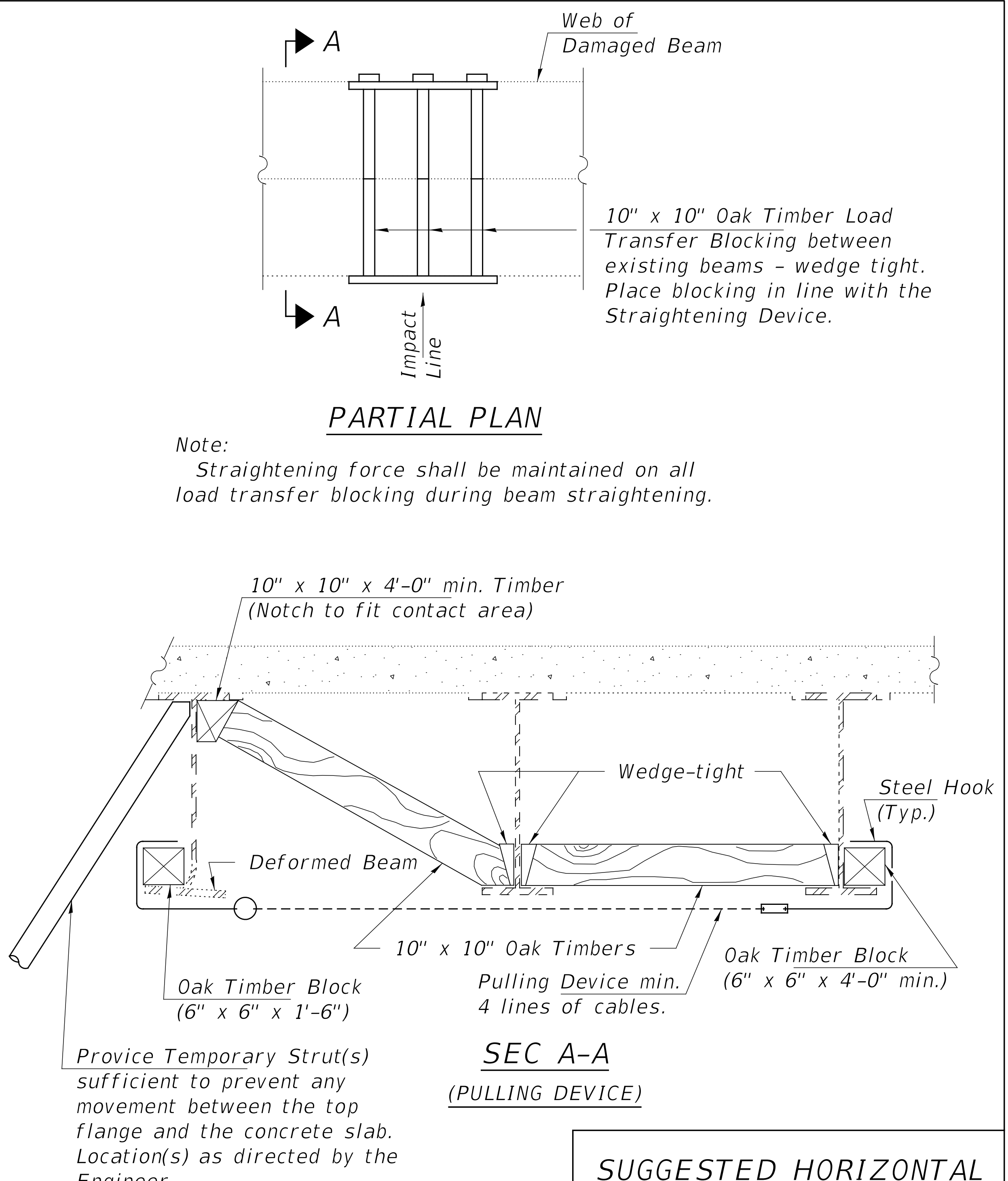


Figure 2.13.2-1: Suggested Horizontal Beam Straightening Method



SUGGESTED HORIZONTAL BEAM STRAIGHTENING METHOD

Figure 2.13.2-2: Suggested Horizontal Beam Straightening Method

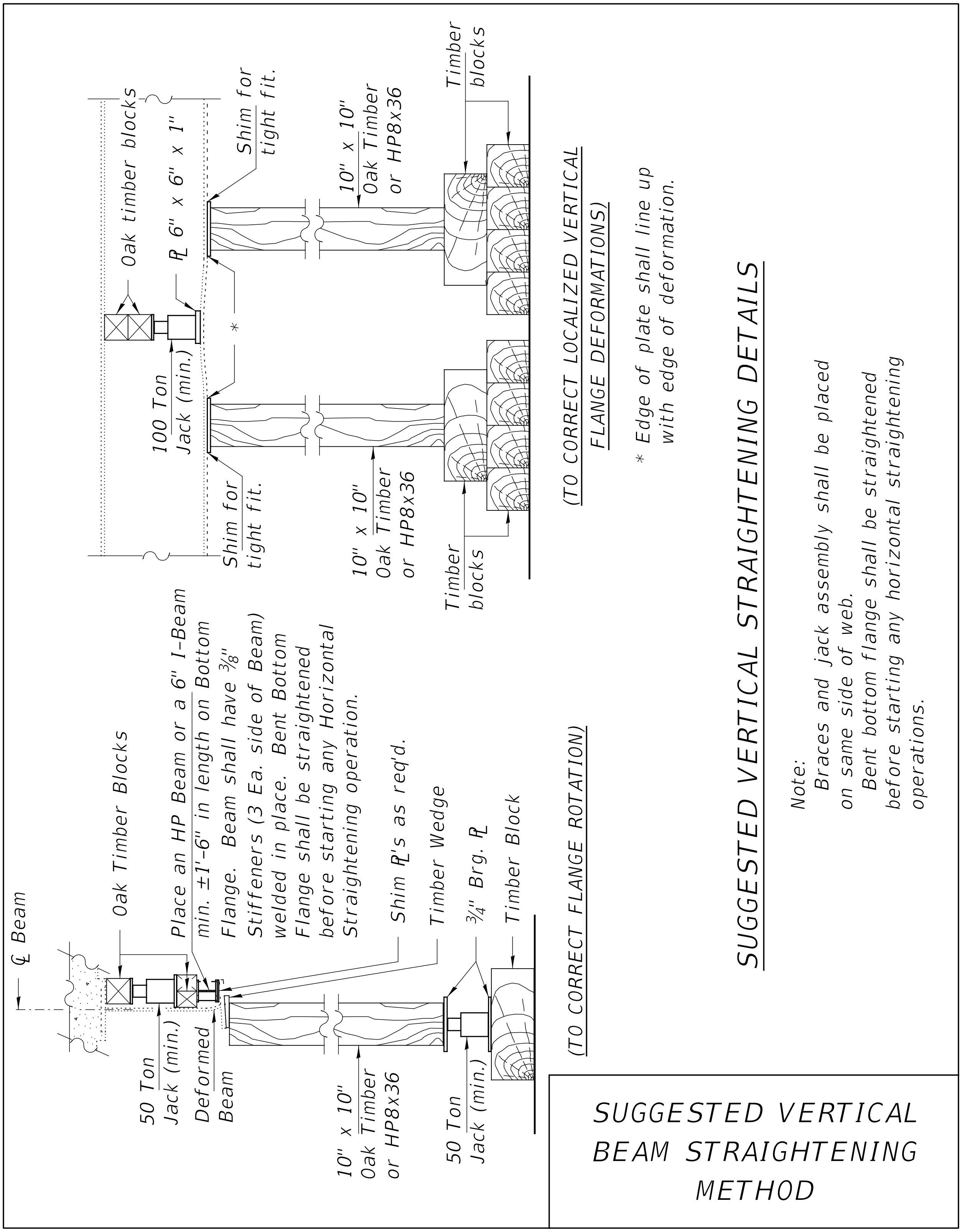


Figure 2.13.2-3: Suggested Vertical Beam Straightening Method

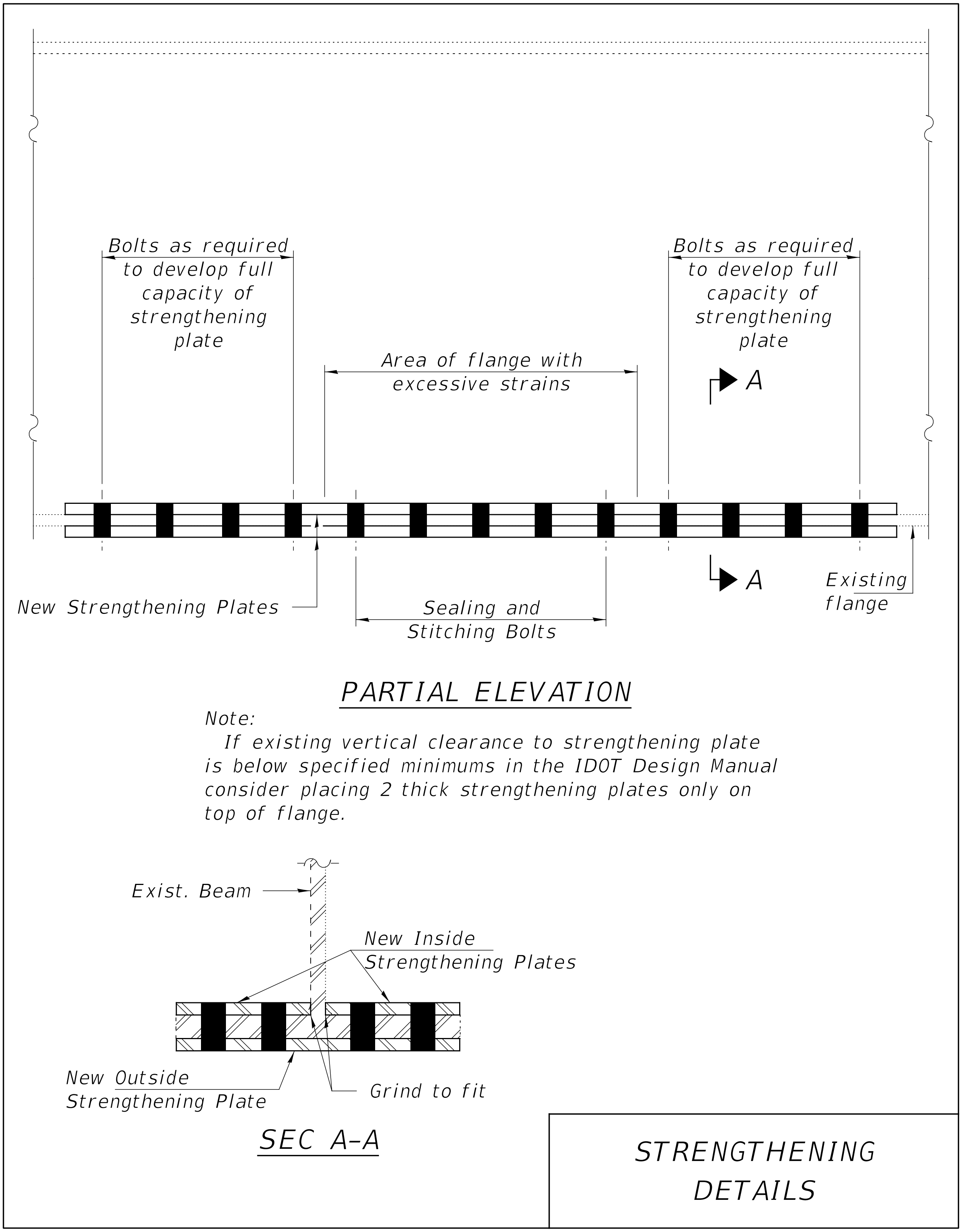


Figure 2.13.2-4: Strengthening Details

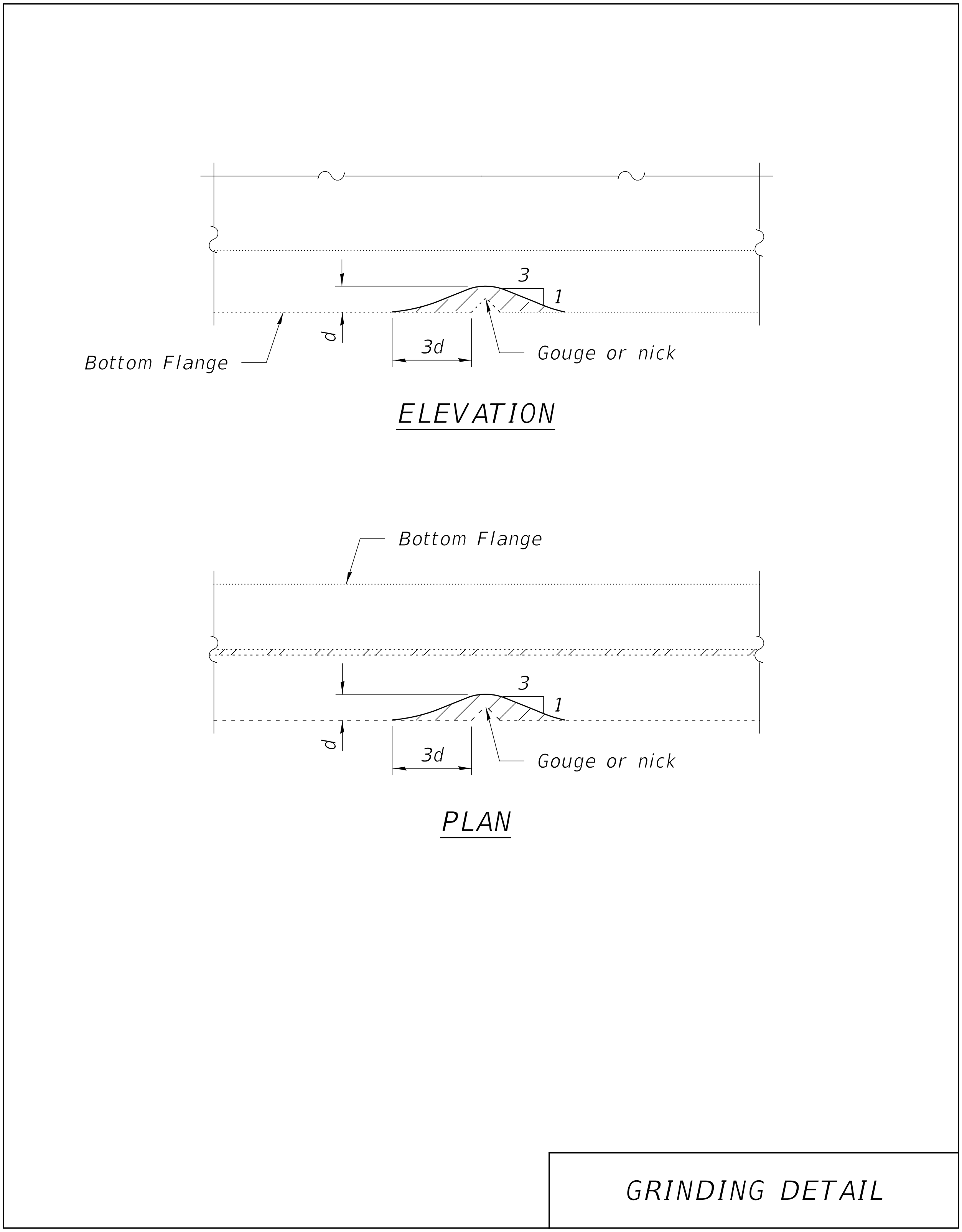


Figure 2.13.2-5: Grinding Detail

2.13.3 Beam Replacement

Beam replacements are typically done in the interior span areas of structures between points of dead load contraflexure. Beam splices may be installed at locations where field splices currently exist or at new locations. When an existing splice location is deformed and cannot be straightened for reuse, a new splice can be installed adjacent to the existing splice at a location where the existing beam can be adequately straightened for splicing with the new beam. Occasionally the replacement will extend to a point of simple support. If unavoidable, due to traffic control restraints or other reasons, beam removal and replacement may require a new beam splice at a location where sizable dead load moments occur in addition to the live load moments. The impacted beam section to be replaced may either be non-composite or composite.

Non-composite beams can be removed and replaced without removing the existing concrete deck slab above them. However, the deck must be temporarily supported and kept in its existing position during the replacement. Figure 2.13.3-1 through Figure 2.13.3-4 provide details for a method of temporarily supporting a deck slab during the replacement of a damaged non-composite or composite beam. This type of slab shoring must be submitted to the Bureau of Bridges and Structures for approval.

The removal and replacement of a portion of the existing deck slab on a non-composite beam, as shown in Figure 2.13.3-5 and Figure 2.13.3-6, is required at the locations where the top flange of the new and existing beam is to be spliced.

When inserting a new non-composite beam into the existing top flange pocket of an existing deck, the edges of the top flange should be ground as shown in Figure 2.13.3-7 to ease the installation.

The standard practice to replace damaged composite beam sections involves the removal and replacement of a section of the deck over the beam. Often the beam needing replacement is the fascia beam which will also require the removal and replacement of a section of the concrete parapet. In order to minimize construction time and traffic disturbances in some cases, IDOT has for several years used a method commonly referred to as a “zipper detail”. See Figures 2.13.3-8 and 2.13.3-9 showing an example for this “zipper detail” method.

This method involves the removal and replacement in-kind of the bottom section of the impacted beam while leaving the top section, which is attached to the concrete deck by means of shear studs, in place. In most cases the removal section extends from an existing splice to a newly created splice near a dead load contraflexure point in the same span. Cutting of the section to be removed should be precise to ensure proper fit. Normally the cuts are done by means of a guided plasma cutter. Use of flame cutting is not allowed. The new bottom section is attached to the existing beam by means of a bolted connection. The designer must determine the bolt spacing required so satisfy horizontal shear requirements. To date, these types of repairs have been completed only by the Department's Day Labor forces who have developed procedures, practices and equipment that allows them to complete the work in a very short period of time, minimizing the traffic disturbances caused by road closures. The procedures also result in accurate removal of the specified portions of the existing beam and accurate fit and connection between the new and existing sections of beam.

During beam replacements, composite or non-composite, temporary shoring and cribbing should be placed under portions of the existing beam which are to remain in place, at locations sufficient to maintain structural stability and the as-built profile of the structure. Shoring and cribbing should be placed in the spans adjacent to the new field splices. When additional shoring for the purpose of adjustment and aiding in the alignment of the splices is called for it will be labeled in the plans as “may be required”. A submittal for these supports to the Bureau of Bridges and Structures for approval is not required.

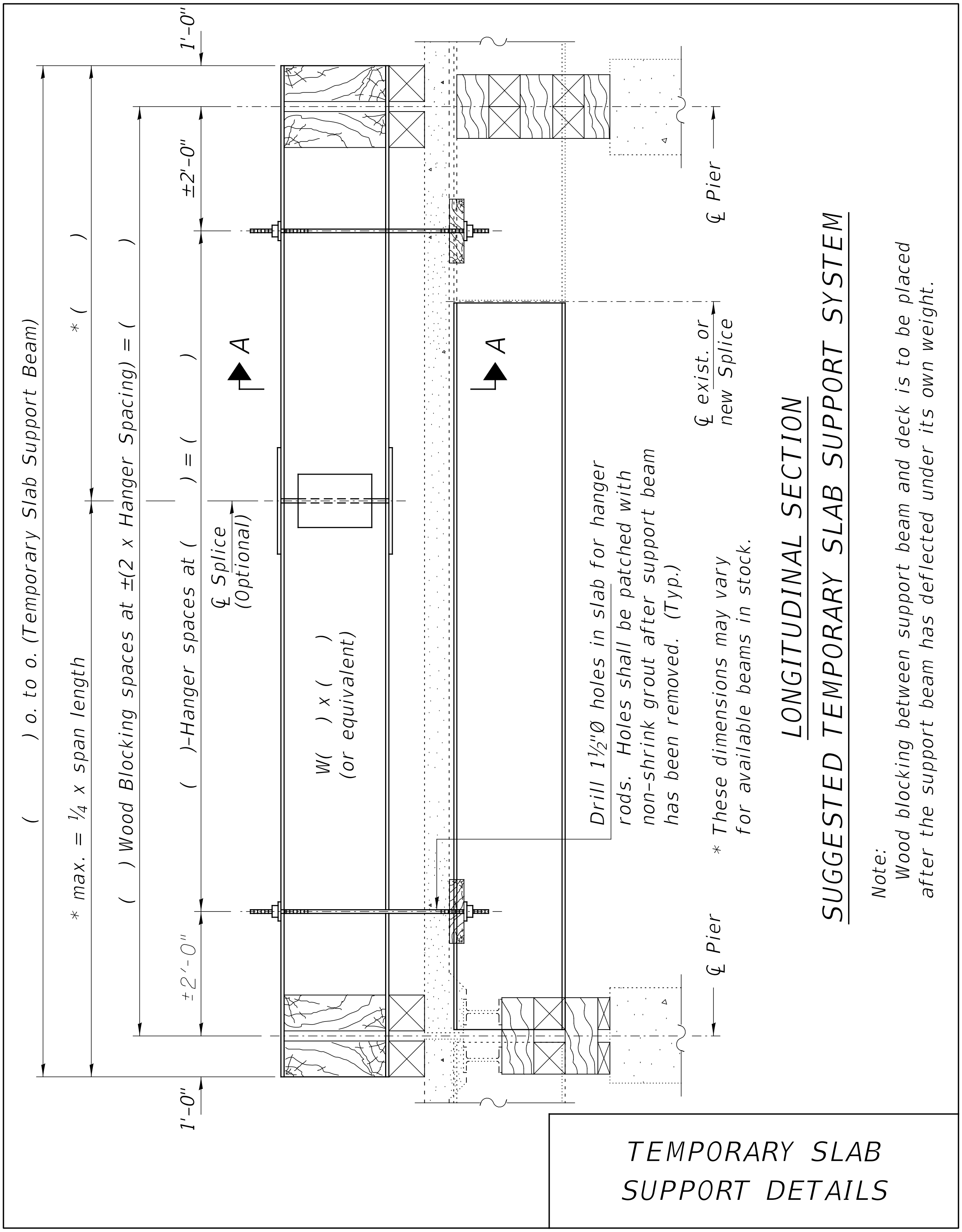


Figure 2.13.3-1: Temporary Slab Support Details

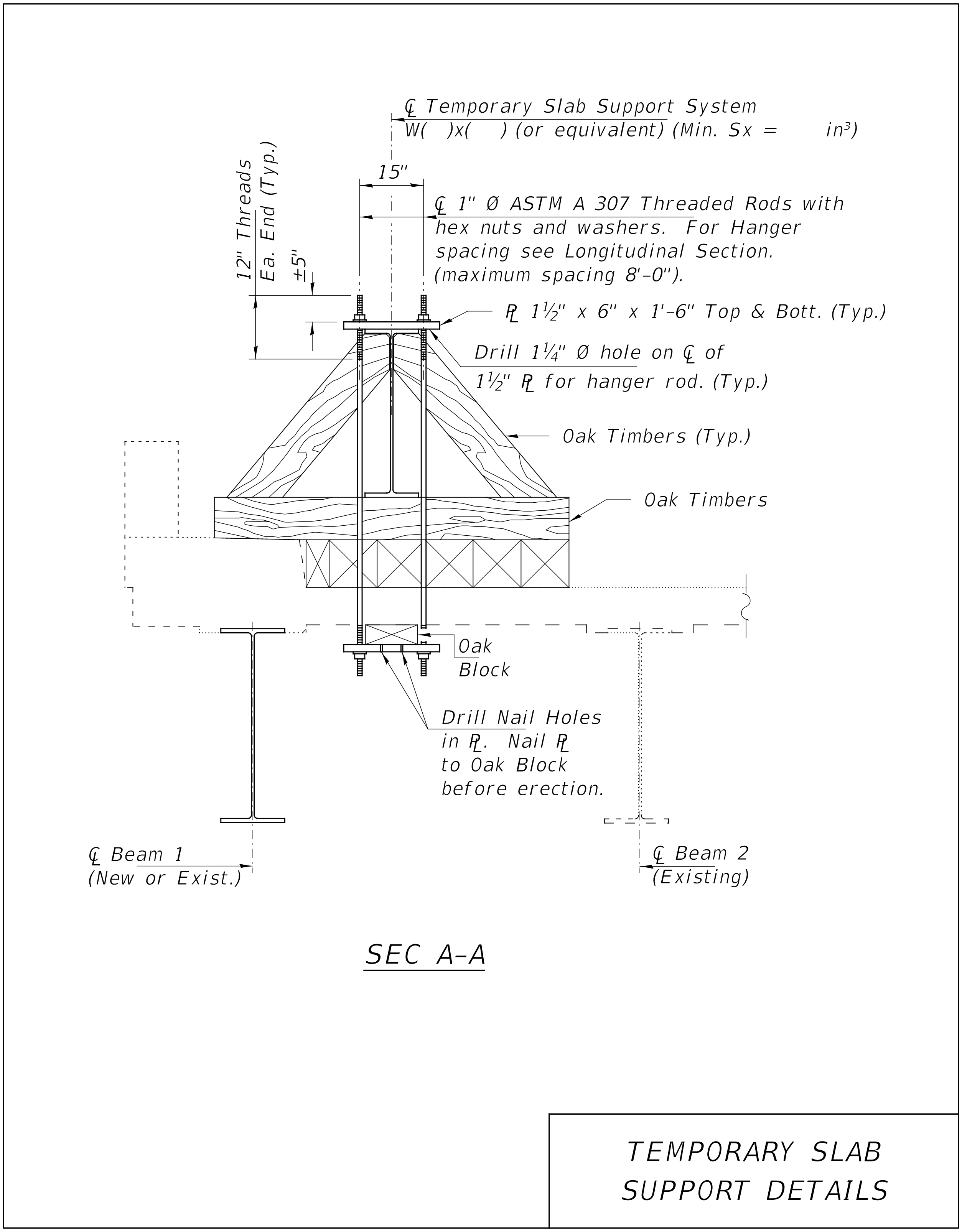


Figure 2.13.3-2: Temporary Slab Support Details

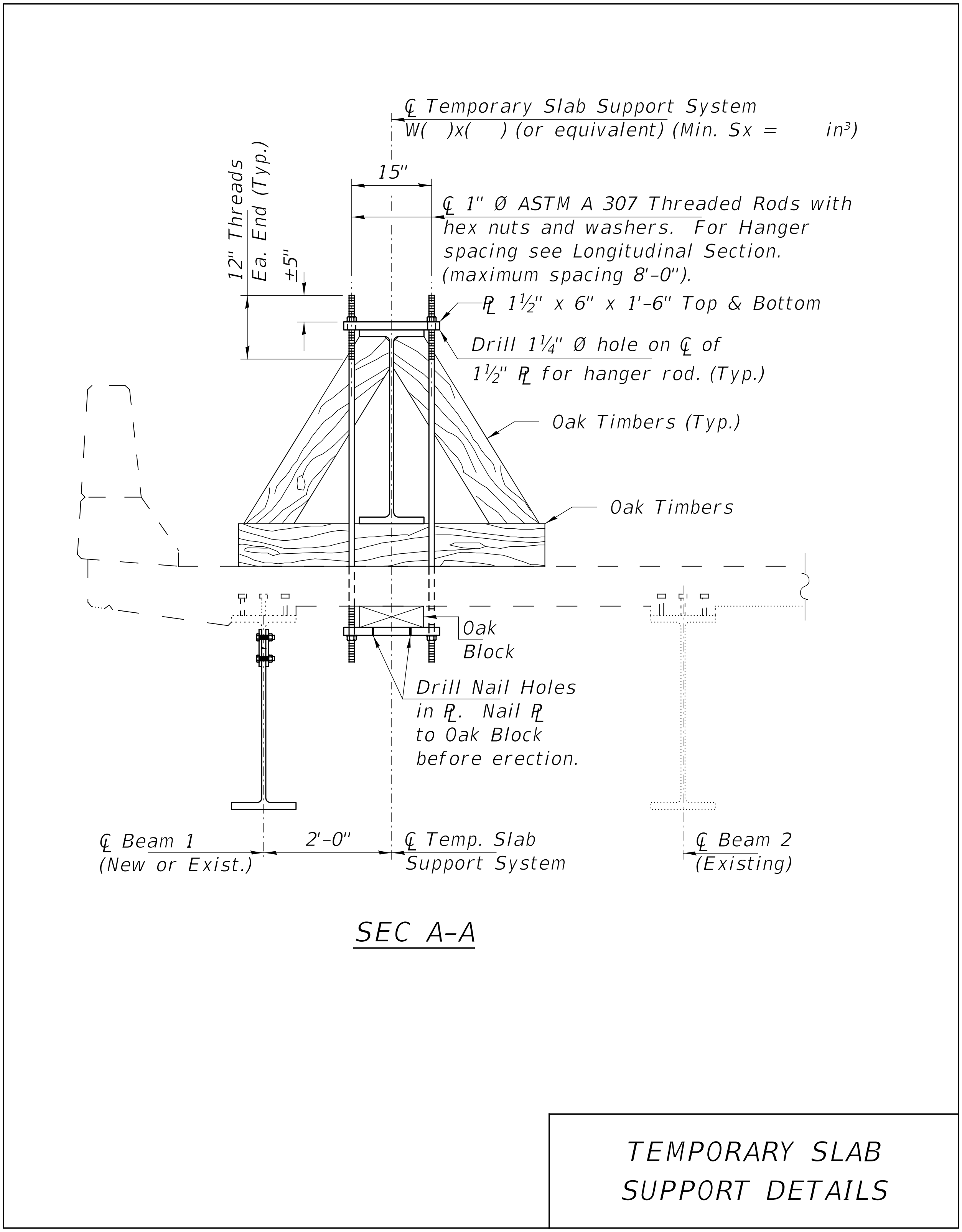


Figure 2.13.3-3: Temporary Slab Support Details

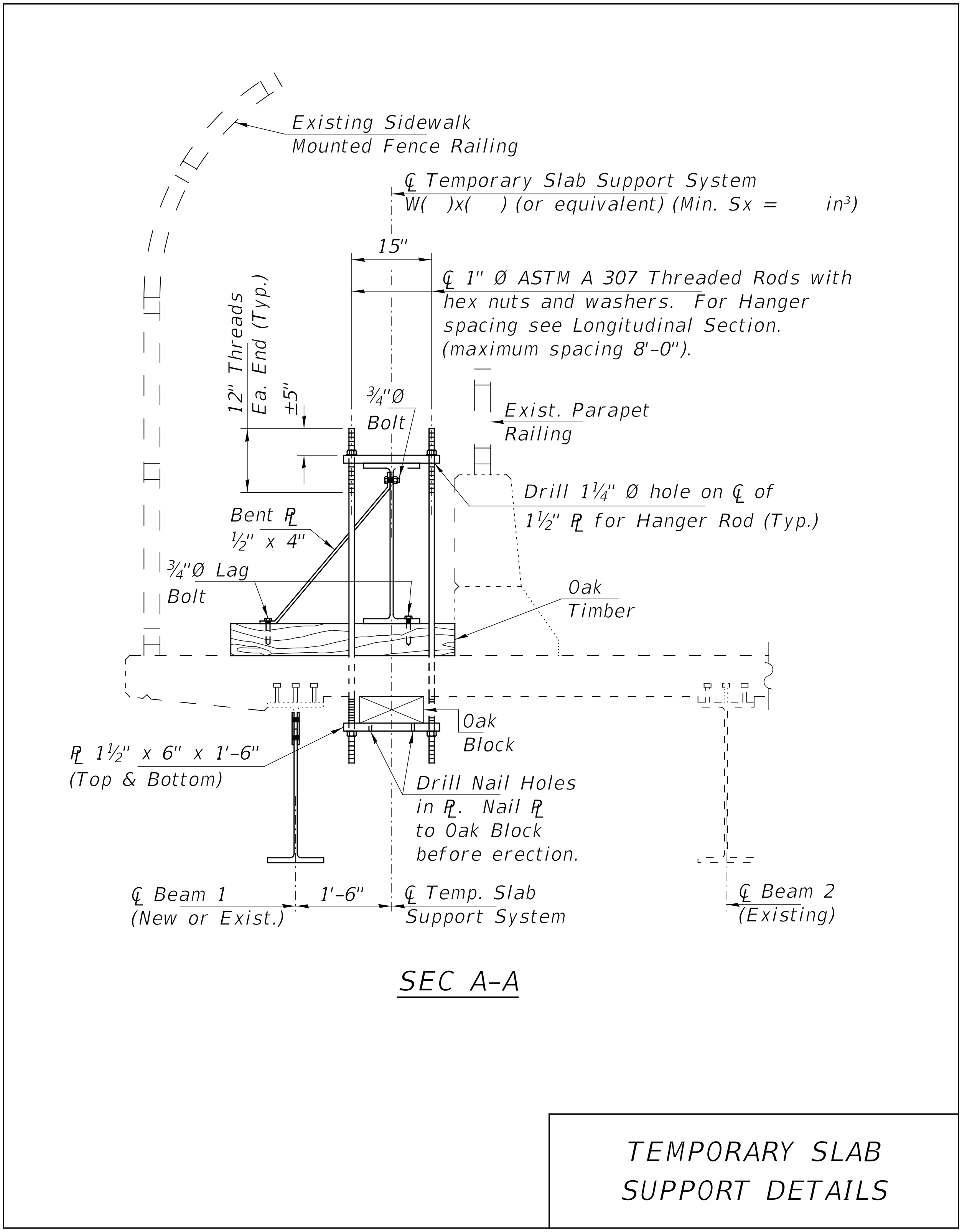
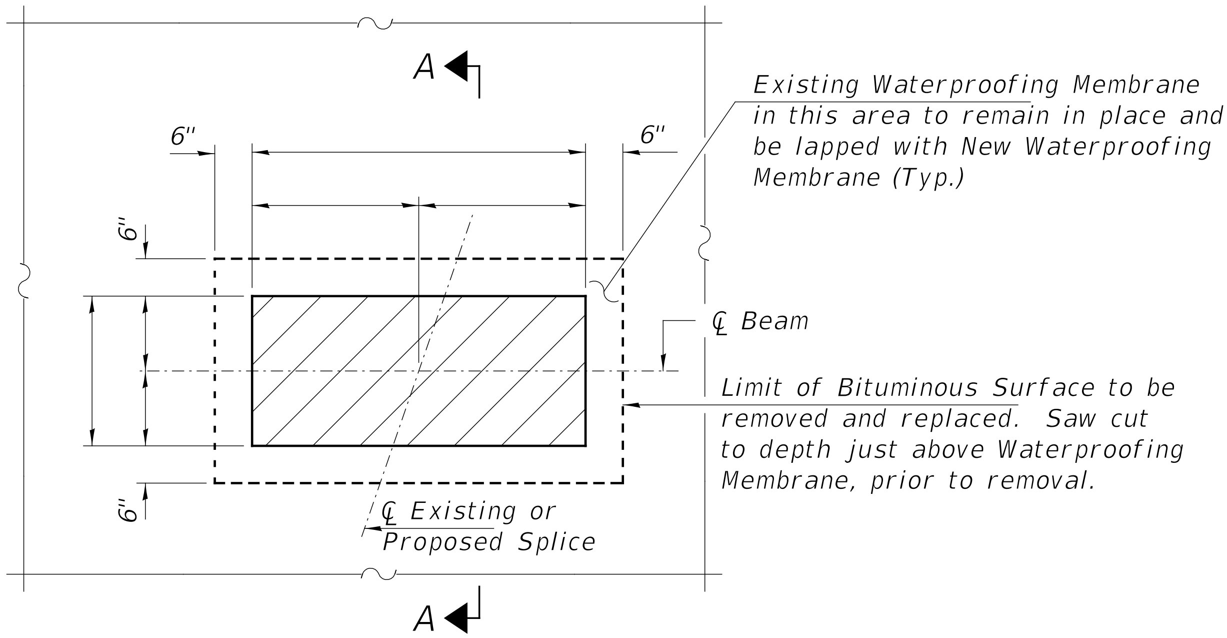


Figure 2.13.3-4: Temporary Slab Support Details



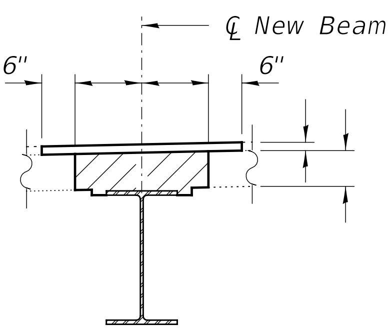
TYPICAL CONCRETE & BITUMINOUS SURFACE REMOVAL AND REPLACEMENT

Note:

Hatched areas indicate concrete sections to be removed and replaced. Perimeters of concrete removal areas shall be saw cut ¾" prior to the removal of concrete.

Reinforcement shall be cut only if required for fitting bolts. Cut reinforcement shall be spliced as directed by the Engineer. Cost shall be incuded in the cost of Concrete Removal.

The cost of removing and replacing the existing bituminous surface, waterproofing membrane, and saw cutting shall be included in the cost of Concrete Superstructure.



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DECK SLAB AND HMA REMOVAL FOR BEAM REPLACEMENT

Figure 2.13.3-5: Deck Slab and HMA Removal for Beam Replacement

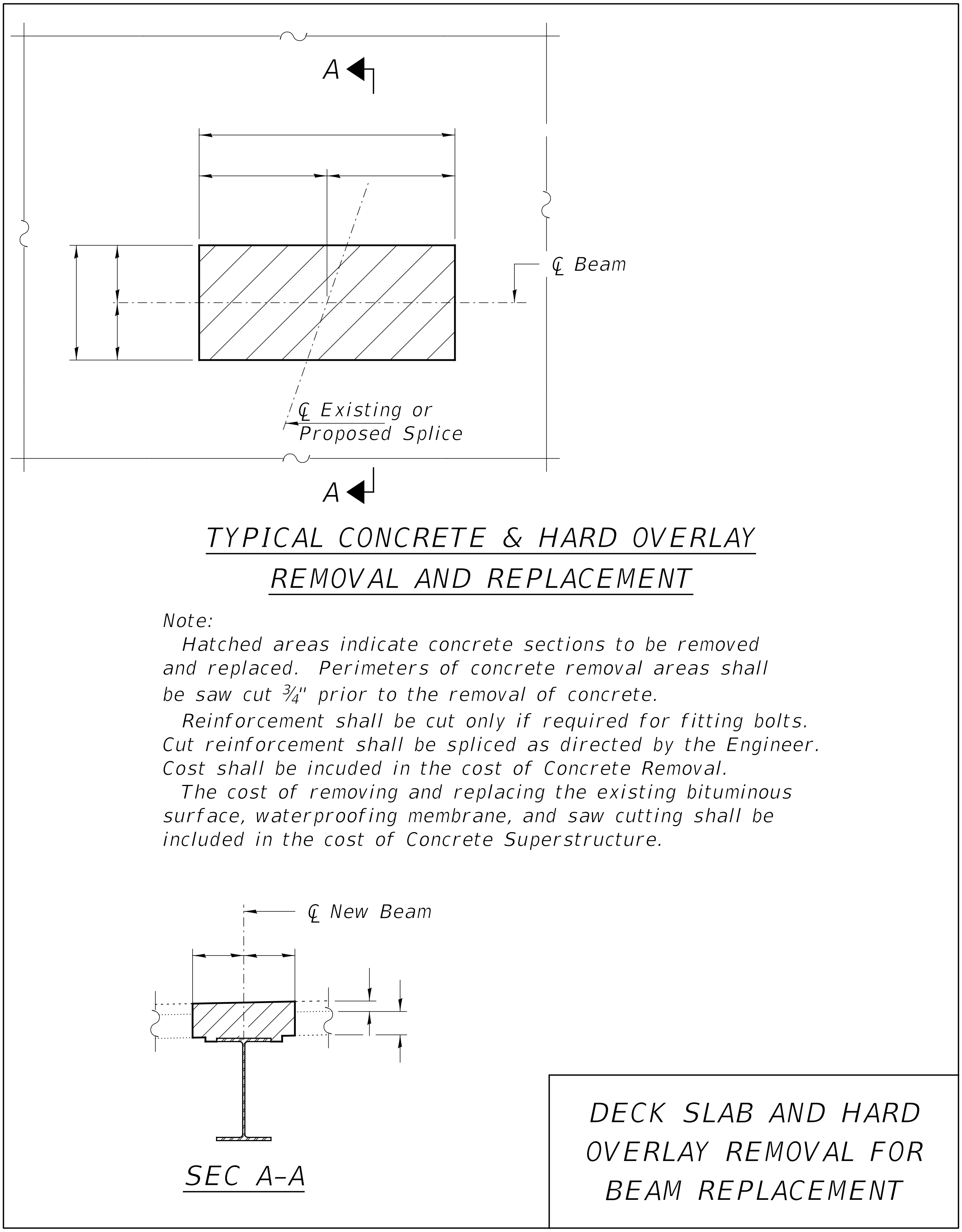


Figure 2.13.3-6: Deck Slab and Hard Overlay Removal for Beam Replacement

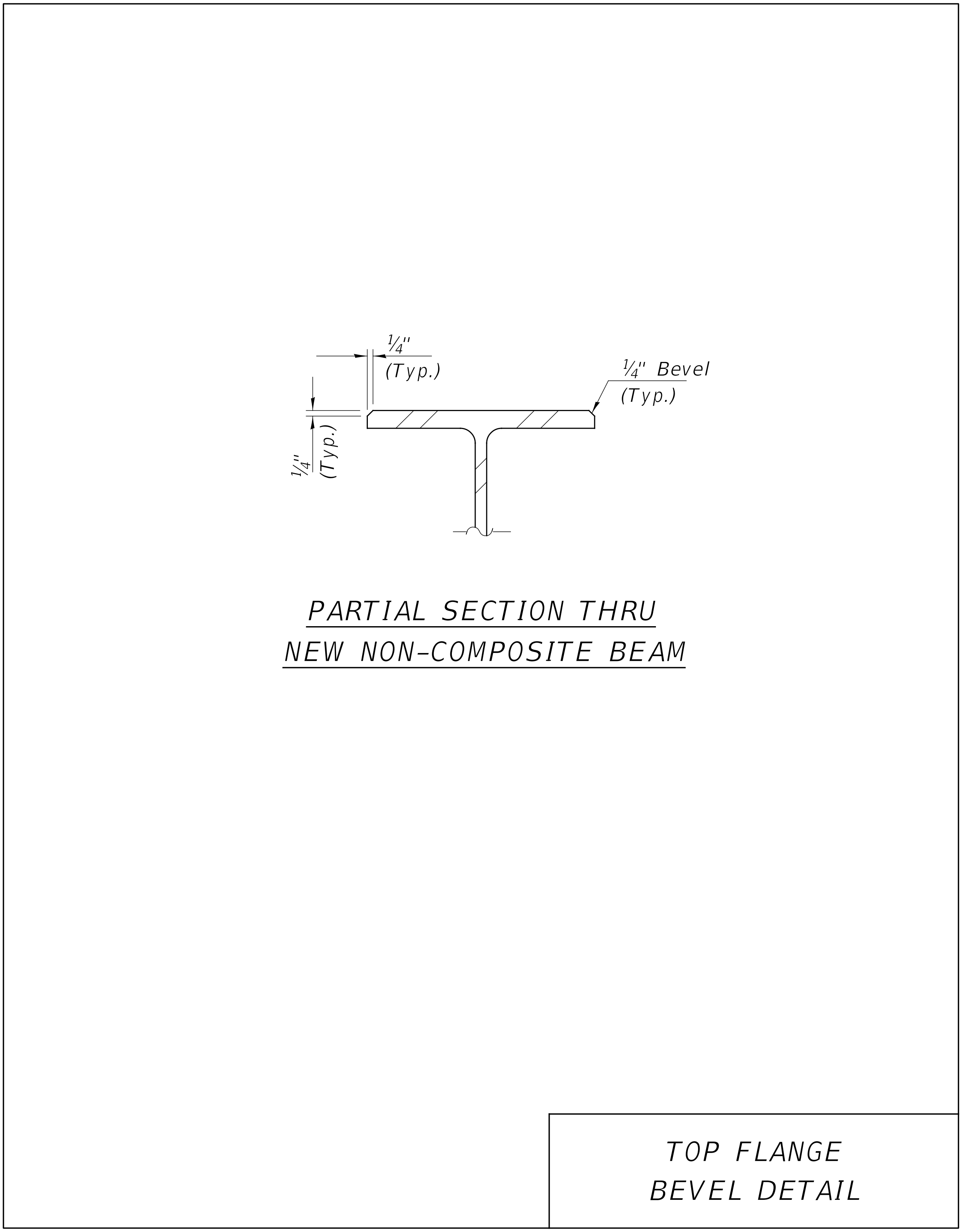


Figure 2.13.3-7: Top Flange Bevel Detail

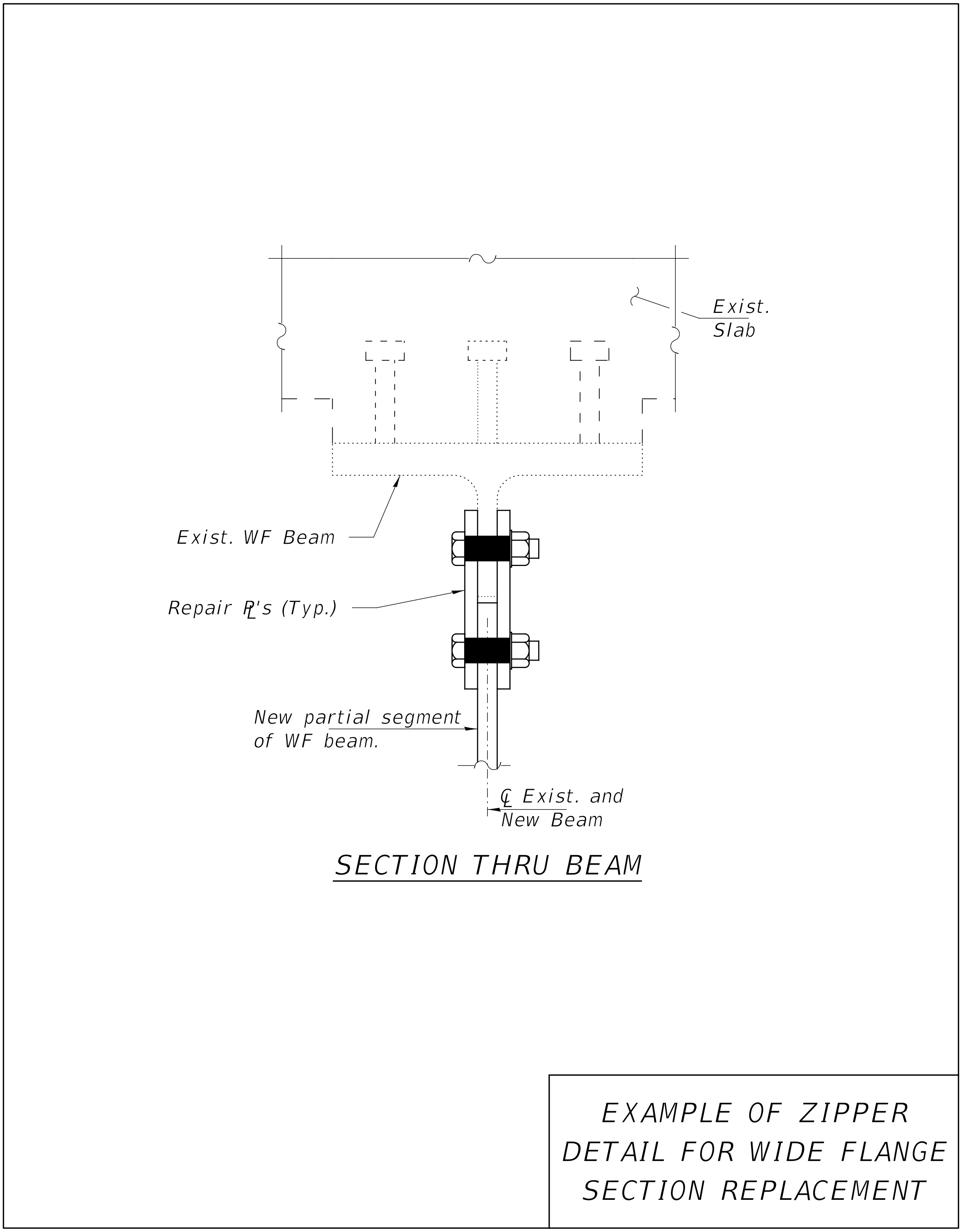


Figure 2.13.3-8: Example – Zipper Detail for Wide-Flange Section Replacement

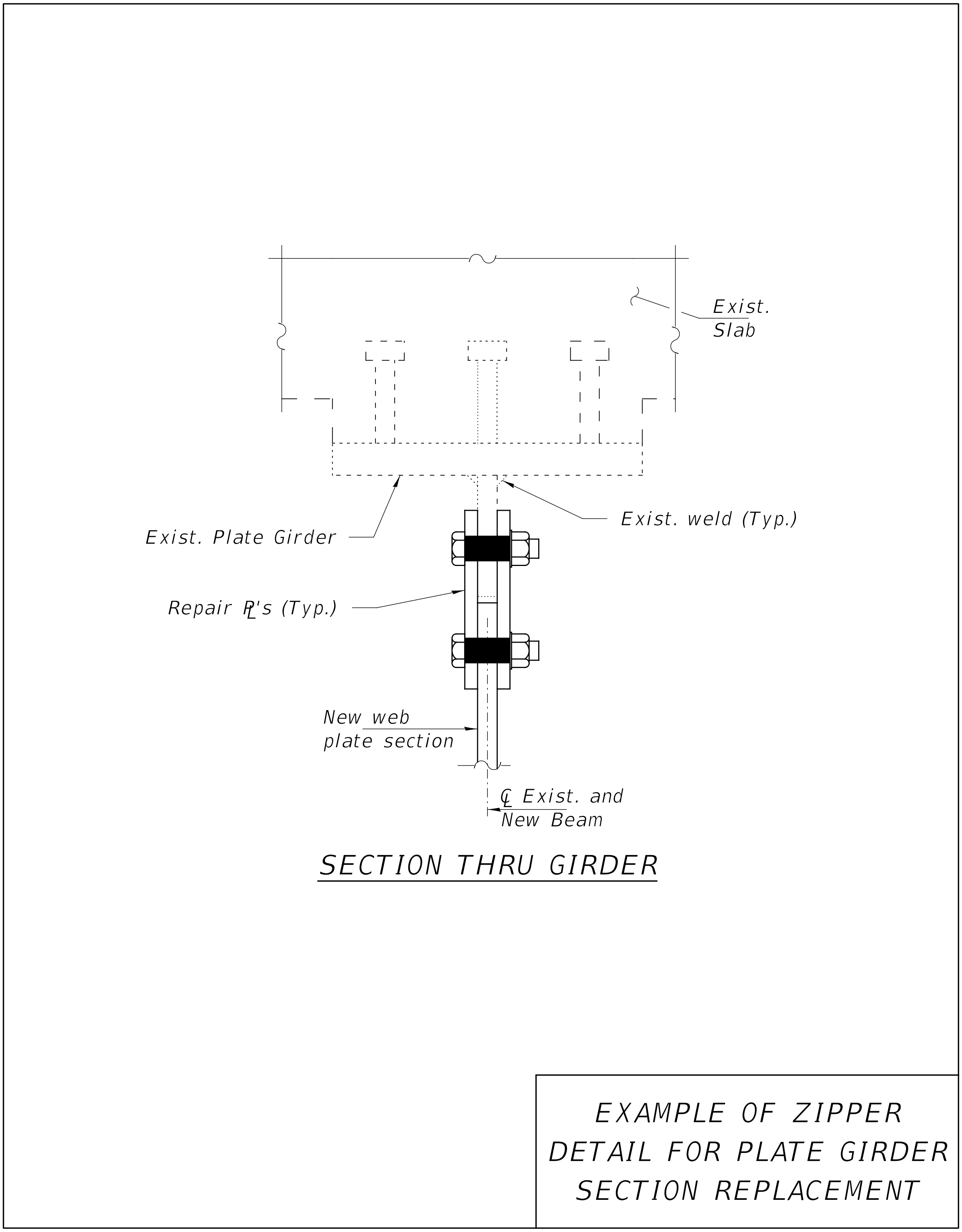


Figure 2.13.3-9: Example – Zipper Detail for Plate Girder Section Replacement

2.13.4 Top Flange to Deck Slab Repairs

After straightening or replacement, the top flange of the beam should be inspected to ensure that it is in solid contact with the deck slab. Any gaps between the top flange of the beam and the deck slab should be injected with epoxy to provide positive contact with the deck slab. Also, damaged or missing concrete fillets encasing the top flange of non-composite beams should be evaluated to determine the effect of the missing fillets on the strength of the beam's compression flange. Only when substantial lateral movement of the top flange is occurring should top flange restrainers be installed. The restrainers should be located in the area of missing concrete fillets to provide lateral restraint for the compression flange in positive moment areas as shown in Figure 2.13.4-1.

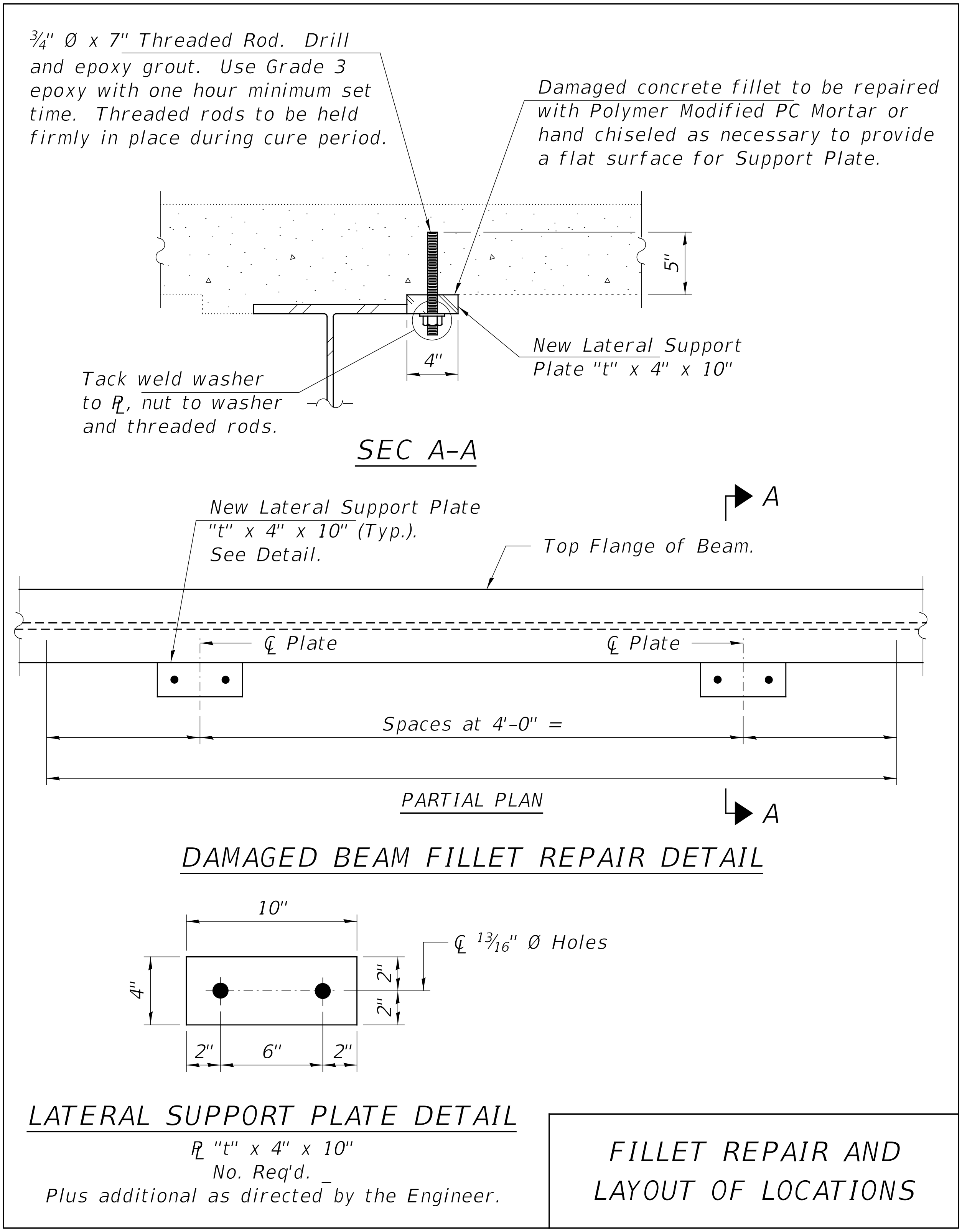


Figure 2.13.4-1: Fillet Repair and Layout of Locations

2.14 Impact Repairs - Concrete Beams

2.14.1 General

When concrete beams are struck and an area of concrete is removed from the beam by the impact, a decision must be made to repair or replace the member. Often a decision to replace a concrete member is based on the extent of the damage rather than the effect of the missing concrete on the strength of the member. Damage is usually located on the bottom of a member located over traffic. A vehicle impact on a PPC I beam will normally result in a concrete spall on the bottom flange that may expose the prestressing strands. If the strands are exposed, they should be carefully inspected. Any nicks on the strands may compromise the capacity of these highly stressed elements hence also impacting the capacity of the whole member. If the missing concrete is to be replaced, measures must be taken to ensure that the concrete patch will be securely anchored in place to protect the strands and cannot eventually loosen and spall off under traffic. Fiber Reinforced Polymer (FRP) has been found to be an effective mechanism to ensure a patch remains in place (see Section 2.16 for a description of its use).

2.14.2 Preloading

To ensure that an area of repair concrete will remain in place, anchors should be installed in the existing beam and a preload should be applied to the structure during the repair and curing period. After the repair concrete has cured, the preload should be removed from the structure to effectively place the concrete repair in compression. The preload used should approximate the effect on the member of the passage of a maximum legally loaded vehicle crossing the structure. An analysis is required to determine:

- The existing stresses in the damaged member
- The stresses in the damaged member during preloading
- The stresses in the repaired member after the preloading is removed.

If the analysis shows excessive stresses in the member during the necessary repair procedures, the member should be replaced. For the repair of precast prestressed concrete beams, the guidelines given in the National Cooperative Highway Research Program (NCHRP) Report 280 should be used.

The preload used during the repair of the damaged member can consist of loaded vehicles, stacked pig iron, concrete barriers or any load determined to approximate the effect of the maximum legally loaded vehicle on the member (provided the damaged beam is capable of carrying the preload). A detailed description of the preloading with calculations signed and sealed by a Structural Engineer must be submitted to the Bureau of Bridges and Structures for review and approval.

2.14.3 Repair Procedures

The concrete patches should be keyed into the existing member by making shallow saw cuts along the perimeter of the proposed patch and removing the damaged concrete in such a way as to provide a square (unfeathered) edge along the perimeter of the replacement concrete. Contact with the existing prestressing strands must not occur during the sawcutting operations. Patching details associated with the repair of a PPC I-beam are shown in Figure 2.14.3-1.

If the damage is located on an interior beam which is protected from the elements and analysis shows that the missing concrete does not affect the strength of the member and reinforcement and prestressing strands have not been exposed, replacement of the missing concrete may not be necessary. If reinforcement and/or prestressing strand have been exposed, then a concrete patch and FRP should be used in the repair (See Section 2.16.2).

All cracks in the members should be located and repaired using epoxy injection. Precast prestressed concrete members should be preloaded prior to and during the curing period of the epoxy crack repairs. Again, the effect of the preload on the damaged member should be analyzed prior to its application.

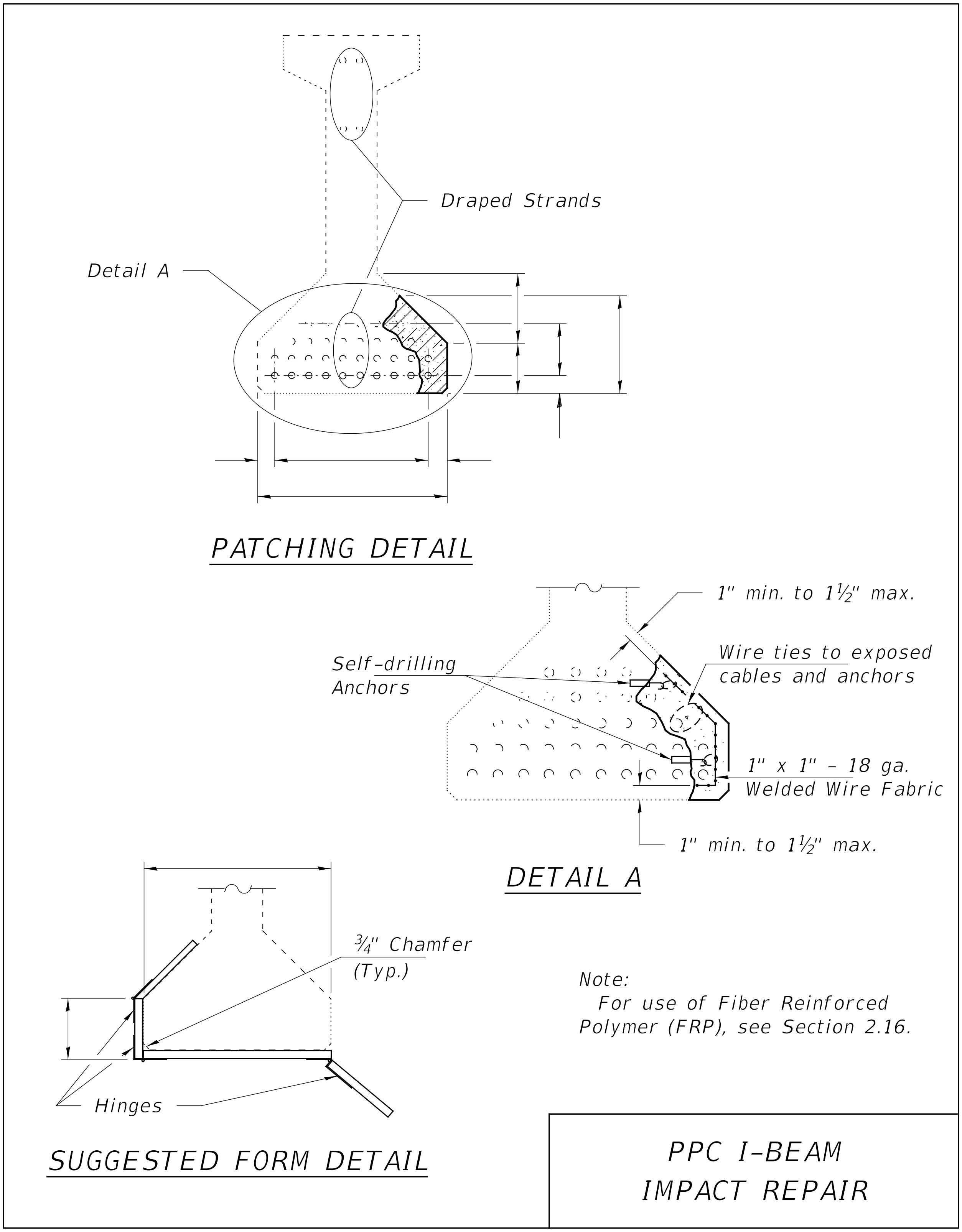


Figure 2.14.3-1: PPC I-Beam Impact Repair

2.15 Steel Superstructure Repairs

2.15.1 General

The major cause of the deterioration of steel superstructure elements is the exposure of the members to moisture and deicing agents. The types of deterioration most often encountered include:

- Section loss in beams, girders and diaphragms adjacent to unsealed or leaking expansion joints.
- Section loss in the bottom flange and lower web of beams and girders located beneath unextended deck drains.
- Section loss in stringer ends and floorbeams located near unsealed or defective expansion joints or deck relief joints.
- Section loss in truss members in the splash zone area and in areas prone to the collection of debris.

Although not as common as deterioration from exposure, steel members must occasionally be repaired to correct damage occurring to members during the construction operations associated with rehabilitation projects.

Repairs will often be required for a number of members located adjacent to different expansion joints or deck drains on the same structure. The length or extent of deterioration will vary from member to member and the type of repair required will be identical except for the length or depth dimensions of the repair. Rather than preparing an individual repair detail for each location, the number of repair details used should be minimized whenever possible by using the details developed for the location with greatest extent of deterioration at the locations with less deterioration.

Because access is one of the most critical and expensive items of any steel repair, inspectors and designers should be conservative in their decision of what members to repair. Even areas that show minor damage will, with time, become critical, consequently repairs should be considered for all locations showing deterioration.

Severely deteriorated or damaged diaphragms should be completely replaced rather than repaired. The cost of the labor involved to repair a defective diaphragm is usually enough to make the complete replacement of the diaphragm the most economical choice. Replacement diaphragms should be detailed to minimize field installation difficulties by bolting the connection angles to the diaphragm and specifying oversized holes.

Locations where complete or severe loss of section has occurred should be repaired even when the load carrying capacity of the member has not been affected. The corrosion holes or areas of severe section loss should be repaired by placing new steel plates and/or angles on both sides of the deteriorated member to prevent further deterioration. The plates and/or angles should be a minimum of 1/2 inch in thickness and the spacing of the bolts used to attach the plates and angles should meet the AASHTO requirements for stitching and sealing as well as edge distance. A good practice is to extend the length of the repair plate, when feasible, at least two bolt columns/rows beyond the deteriorated area to ensure proper load transfer, seating of the bolts and sealing of the plate edges against undamaged steel. The intent of the repairs should be not just to cover the damaged area but to provide strengthening that “bridges” the damage. For instance, repairs of damage to the bottom of the web of a beam/girder must also engage the bottom flange.

When the deterioration has resulted in a significant section loss on the area being repaired a structural sealer compatible with steel should be placed between the new repair plates/angles and the existing member to fill any irregularities or voids where moisture may accumulate, and pack rust may develop.

2.15.2 Cleaning and Painting at Steel Repairs

Cleaning and painting of existing steel is an integral part of any steel repair. While cleaning and painting of whole structures is done under “paint only” contracts not handled by the Repairs Unit, cleaning and painting of discrete areas is always included with steel repairs. Guidelines for the cleaning method and paint type to be used as well as the appropriate special provisions to be included in the contract plans are found in ABD 19.7 Cleaning and Painting Existing Steel Structures.

Prior to the installation of any steel elements used for repairs the contact areas must be cleaned and painted. This work is included in the special provisions for Cleaning and Painting Contact Surface Areas of Existing Steel Structures and normally included in the cost of Structural Steel Repairs. See Section 2.15.2.1 for cleaning method to be used on primary and secondary connections.

Additionally, on some structures, “zone painting” is also desired. ABD Memorandum 19.7 includes procedures as to how this work is to be completed. The proper pay items and special provisions depending on the presence of lead in the existing paint must be included in the contract plans.

2.15.2.1 Primary vs Secondary Connections

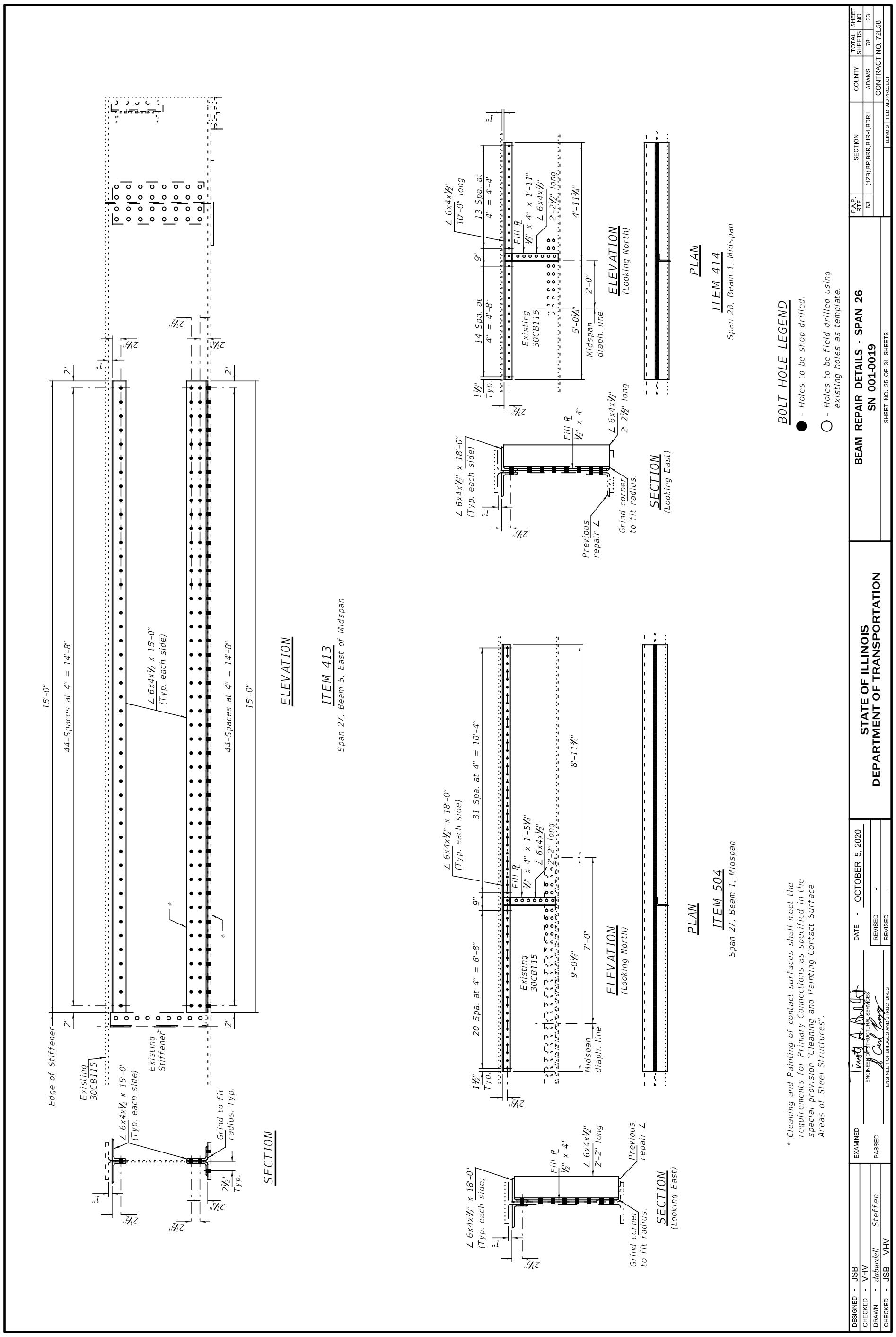
While preparing steel repairs a determination must be made as to whether the members and connections involved are either primary or secondary. This will have a significant impact on the amount of preparation work that will be required on the existing steel before the strengthening plates/angles are installed.

Primary members and connections are those generally defined to be in the load path. Examples of primary member connections are splices, cover plates, “zipper” details on beams/girders, and gusset plates on trusses. Examples of secondary members and connections are diaphragms and cross frames and their connections.

Primary members require the faying surfaces to be cleaned per SP 15 “Commercial Grade Power Tool Cleaning” to remove all rust, mill scale and existing paint, with the intent to get the existing steel to an almost “new steel” condition. On the other hand, a secondary connection need only be cleaned per SP 3 “Power Tool Cleaning” which requires only loose rust, loose mill scale, and loose, checked, alligatored and peeling paint to be removed.

While it is not hard to visualize which members are primary or secondary on a regular bridge, it is not so straightforward when dealing with curved or highly skewed structures where some of the lateral bracings are designed to carry load. The same occurs on a non-typical structure like a truss or a bascule bridge.

To avoid bidding conflicts and construction issues the designer should specify in the plans which repairs and affected members and connections are considered primary. By default, all repairs and connections not identified as such will be considered secondary. See Figure 2.15.2-1 for an example.



EXAMPLE OF PRIMARY CONNECTION
CALL OUT IN PLANS

Figure 2.15.2-1: Example – Primary Connection Call Out in Plans

2.15.3 Beam End Repairs

The typical damage on a beam end under a failed joint is normally found on the lower section of the web and the top of the bottom flange. When the damage is mostly in the area over the bearing a bent plate can be used to strengthen the member as shown in Figure 2.15.3-1. The vertical plate extends up on the web as far as feasible behind the diaphragm clip angles. When the damage on the top of the flange extends beyond the bearing area then a combination of a plate and angle will provide the most efficient repair method as shown in Figure 2.15.3-2. Previous experience indicates that is very likely that existing bearing connecting bolts will shear off while trying to loosen them. Because of this and to ensure proper connectivity between strengthening plates/angles, bottom flange and bearing welds are called for in the connection over the bearings. Additional bolts will be required beyond the bearing area to connect the bent plate/angle to the bottom flange.

If new bearings are also being installed at these locations, then the bearing connecting bolts should be used in lieu of the welds.

If the damage has severely compromised the bottom flange, then it may be necessary to remove and replace in-kind a section of the beam as shown in Figure 2.15.3-3.

In an extreme case it may be necessary to remove and replace a section of the beam end by means of a field splice as shown in Figure 2.15.3-4. Partial deck removal and replacement will be necessary. Removal, adjustment and reinstallation of diaphragms is typically needed for this work.

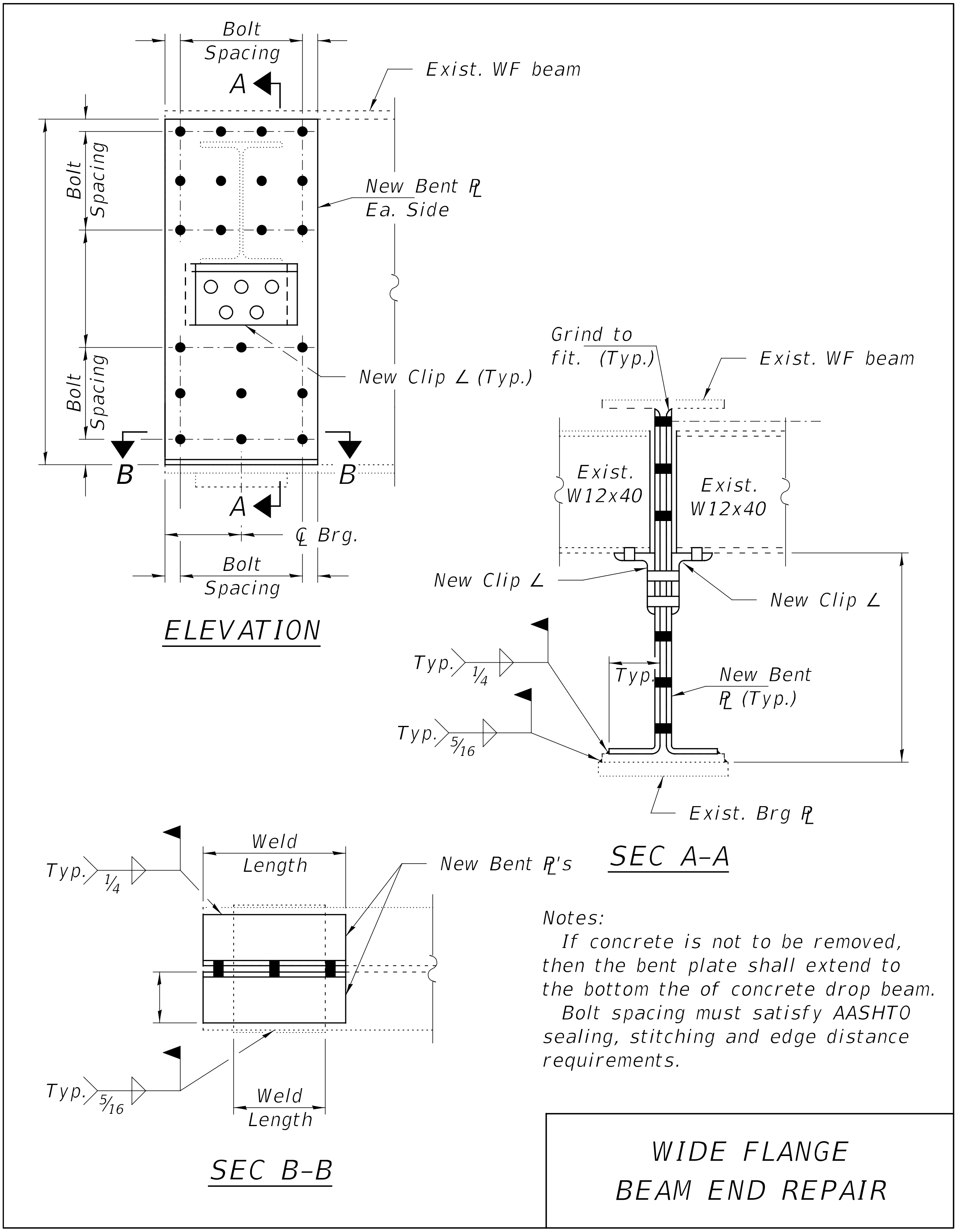
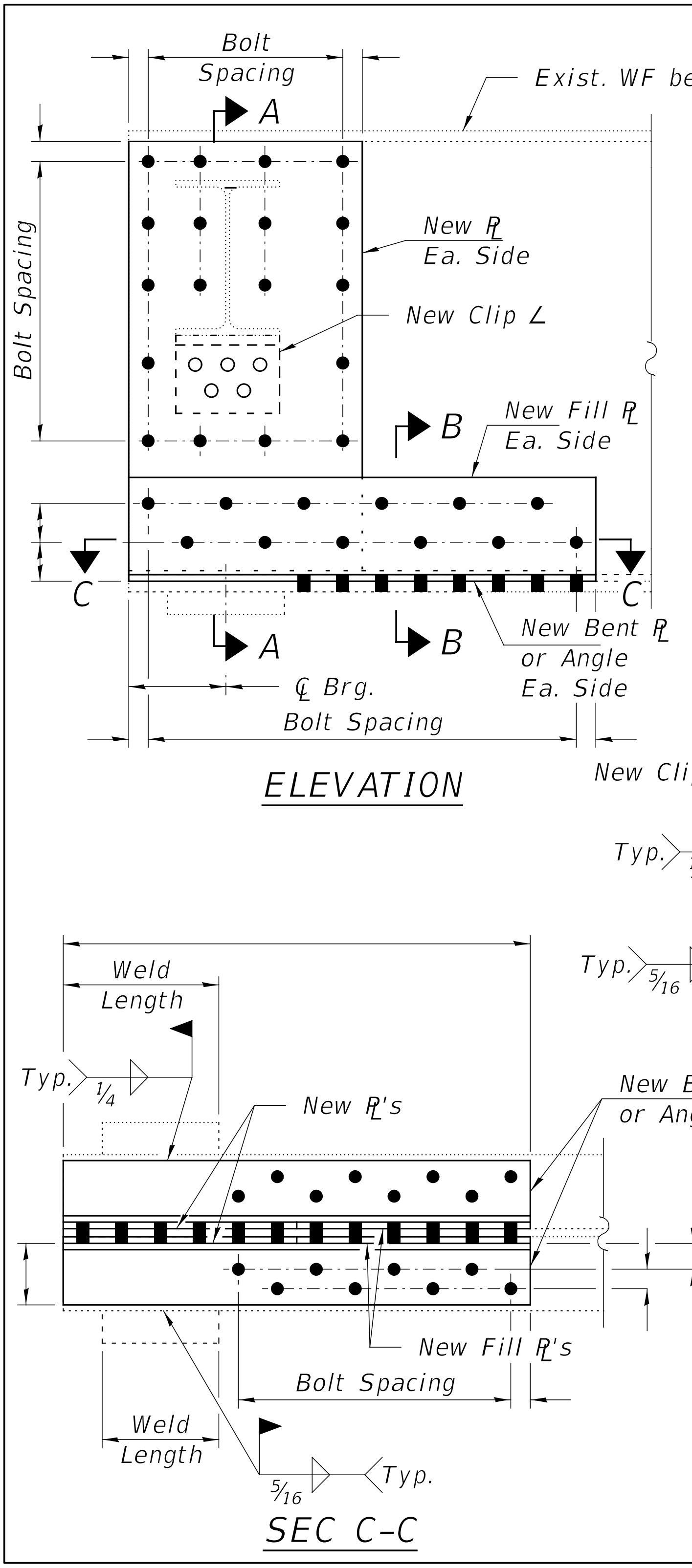


Figure 2.15.3-1: Wide-Flange Beam End Repair



Notes:

If concrete is not to be removed, then the vertical plate shall extend to the bottom of the concrete drop beam.

Bolt spacing must satisfy AASHTO sealing, stitching and edge distance requirements.

WIDE FLANGE BEAM END REPAIR

Figure 2.15.3-2: Wide-Flange Beam End Repair

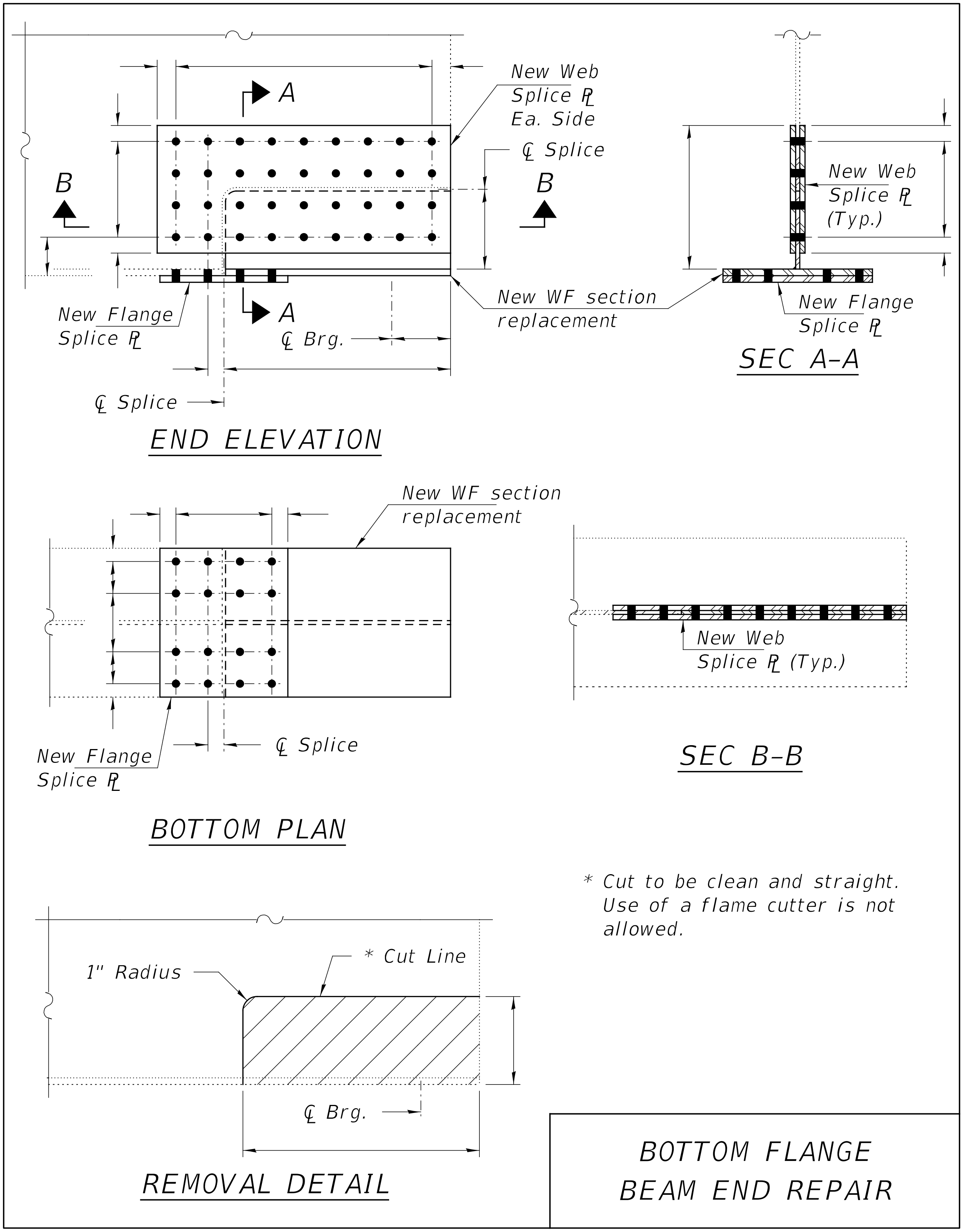
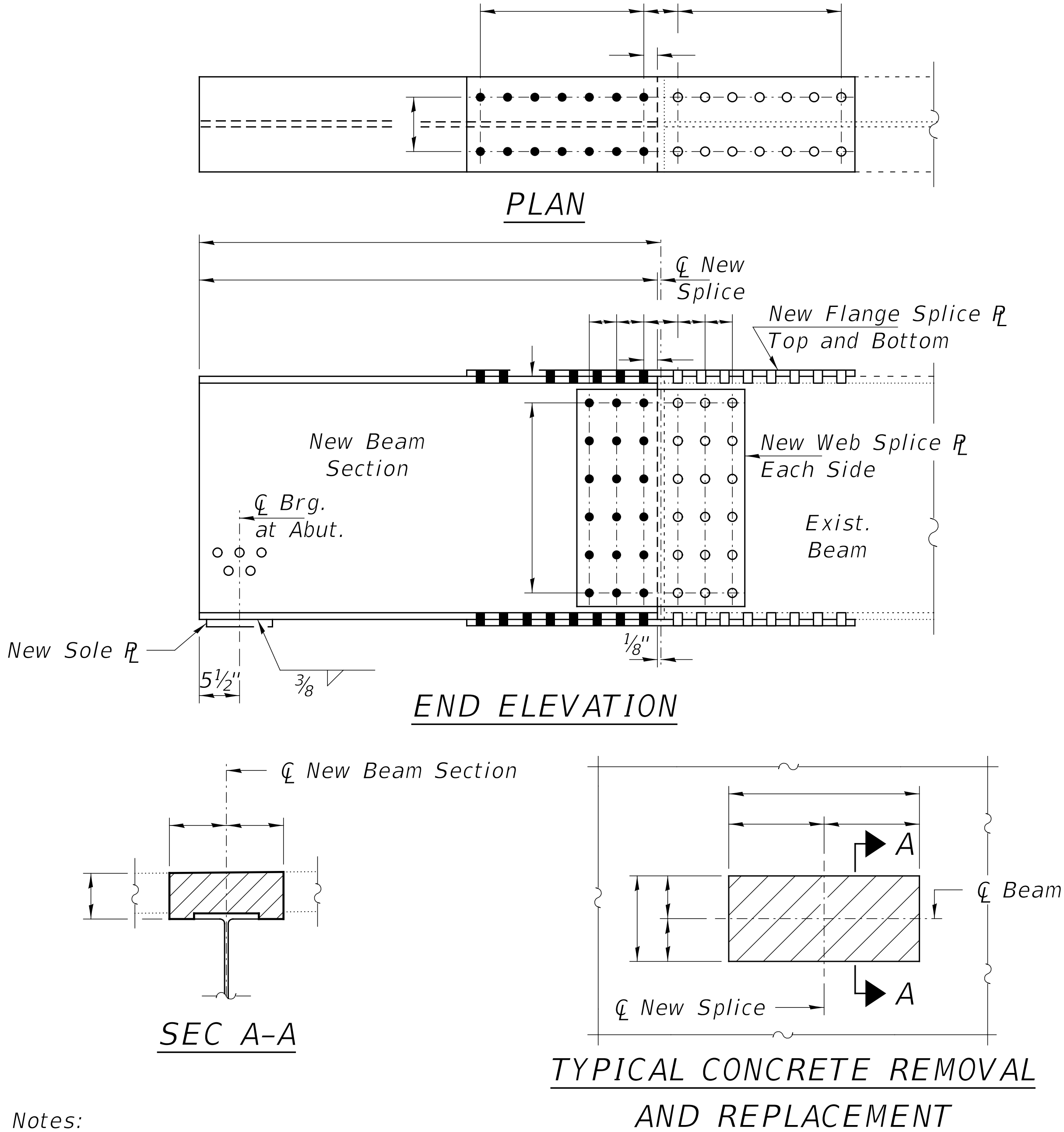


Figure 2.15.3-3: Bottom Flange Beam End Repair



Notes:
Hatched areas indicate concrete sections to be removed and replaced. Perimeters of concrete removal areas shall be saw cut ¾" prior to the removal of concrete.
Reinforcement shall be cut only if required for fitting bolts. Cut reinforcement shall be spliced as directed by the Engineer. Cost shall be included with Concrete Removal.

FULL BEAM END REPLACEMENT DETAILS

Figure 2.15.3-4: Full Beam End Replacement Details

2.15.4 Girder End Repairs

While the damage found on girder ends under a failed joint is typically similar to that found on beam ends, the repair details need to consider the existing stiffeners which will likely also be damaged. The use of “three-sided” boxes is an effective way to re-establish the connectivity of web, stiffener and bottom flange as shown in Figure 2.15.4-1. Adjustments may be necessary if the stiffeners are not orthogonal to web and/or bottom flange.

Cross frame connecting plates in the area of the repair may need to be disconnected and re-attached after the strengthening plates are installed.

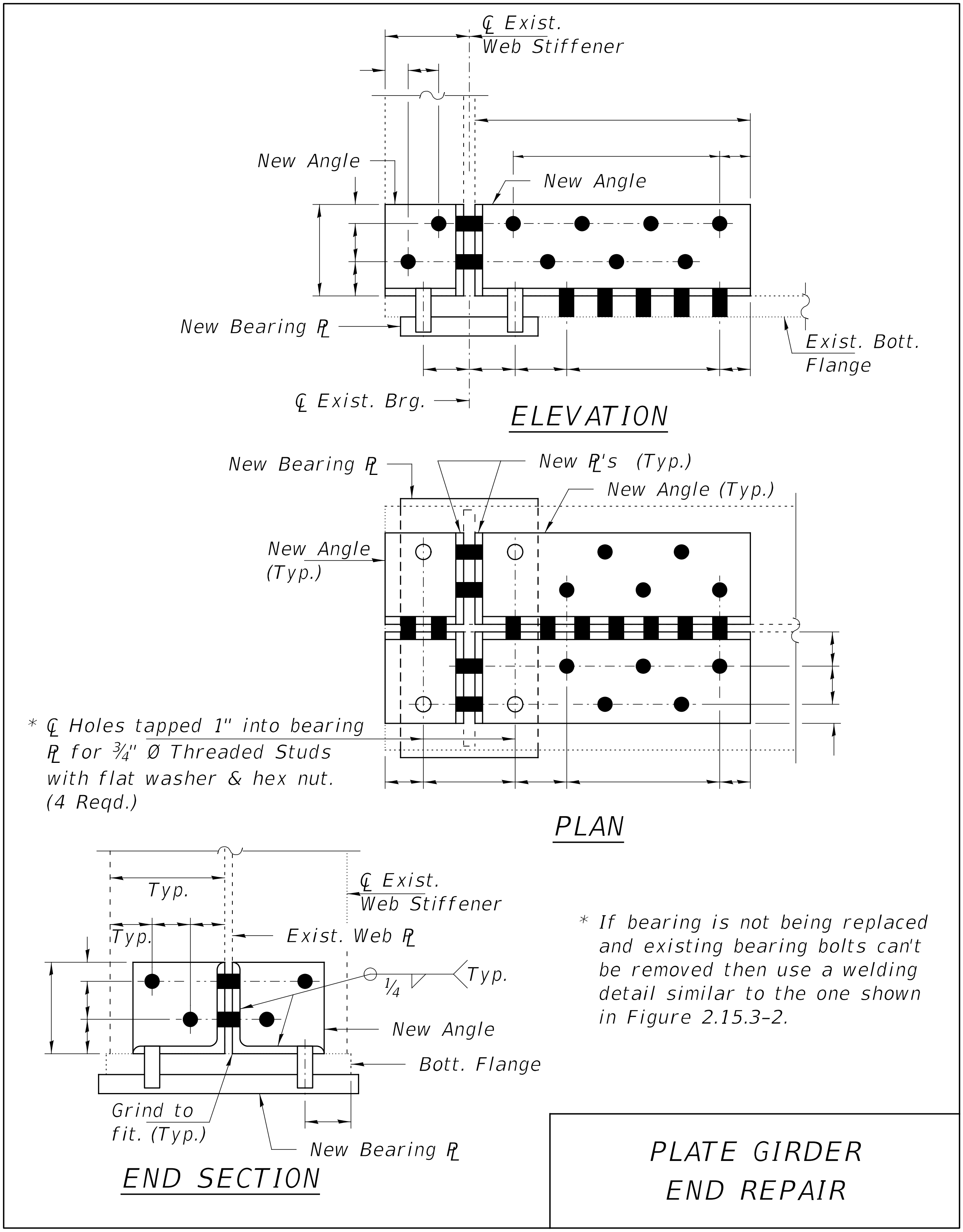


Figure 2.15.4-1: Plate Girder End Repair

2.15.5 Repairs of Beams or Girders at Drain Locations

The section loss, which occurs in the bottom flange and lower web of a member located beneath a deck drain, should be measured to determine the effect of the section loss on the load carrying capacity of the member. When the deterioration is located on a tension flange, the affected area should be ground parallel to the primary stress in the beam to remove sharp corners and deep rifts in the material. The grinding should be tapered into the undamaged areas at a 3:1 slope similar to that shown in Figure 2.13.2-5. When load carrying capacity must be restored, splice plates should be placed across the deteriorated portion of the web and bottom flange, as shown in Figure 2.15.5-1. Holes in the lower portion of the web should be repaired by placing bent plates or angles on both sides of the web to prevent further deterioration. Whenever possible, the drains adjacent to the problem area should be eliminated or extended to discharge below the member to prevent recurrence of the deterioration.

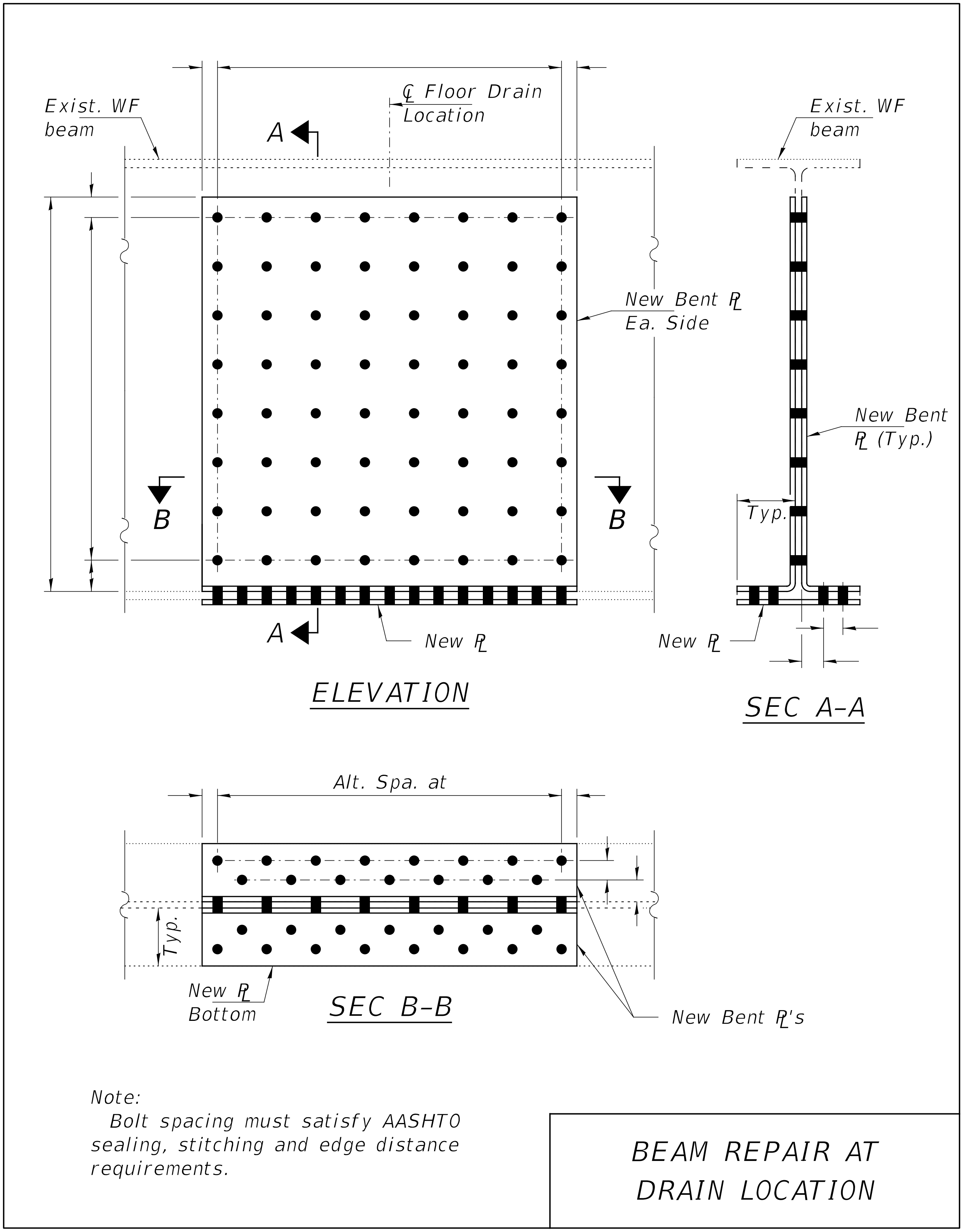


Figure 2.15.5-1: Beam Repair at Drain Location

2.15.6 Temporary Support at Repair Locations

Designers and contractors must ensure the stability of the existing connection while repairs are being completed. In order to achieve this, temporary support systems are often required to ensure that an alternate load path is provided when a connection is being disassembled. It is critical that designers and contractors have a full understanding of the loads acting on the connection and how the load transfer will affect other members.

Some of the support systems will consist merely of transferring the load to adjacent members by means of crossbeams, needle beams or bracing when doing repairs on members such as stringer ends. More complex systems involving preloading using jacks and large support beams may be required to transfer load from a floorbeam to adjacent floorbeams when repairing floorbeam to truss connections.

Pretensioned high strength steel rods will be required for replacement of tension members on a truss.

Since most repair work will be done under stage construction a careful assessment of the impact of the traffic loads on the support system must be made. In some instances, the structure may need to be temporarily closed to traffic while some of the temporary support systems are installed.

Contract plans should include a table showing the expected loads on the member needing to be supported.

2.15.7 Stringer End Repairs

Connections of stringers to floorbeams can be divided into two types. The first one is a fixed connection in which the stringer end is connected to the floor beam by means of clip angles bolted to the stringer and the floorbeam webs (alternatively the web of the stringer is bolted to a web stiffener connected to the web of the floorbeam). The second one is an expansion connection in which the stringer end rests on a built-up support attached to the floorbeam web and clip angles act as guides for the stringer movement. Typically, interior floorbeams will have a fixed and an expansion connection on opposite sides of the web using common bolts for the clip angles or stiffeners.

Section loss on a stringer end web on a fixed connection can be repaired by installing plates on both sides of the web as shown in Figure 2.15.7-1. This type of repair preserves the existing clip angles and common bolts. Alternatively, the clip angles can be removed, and new strengthening plates and angles installed as shown in Figure 2.15.7-2.

Repair of damage to the stringer end at an expansion joint must ensure proper strengthening of the bottom section to provide proper load transfer to the built-up support as shown in Figure 2.15.7-3. The bent plate should be properly attached to the web and the bottom flange of the stringer. New clip angles will need to be installed and existing bolt holes on the floorbeam web will need to be reamed to accommodate the new bolts.

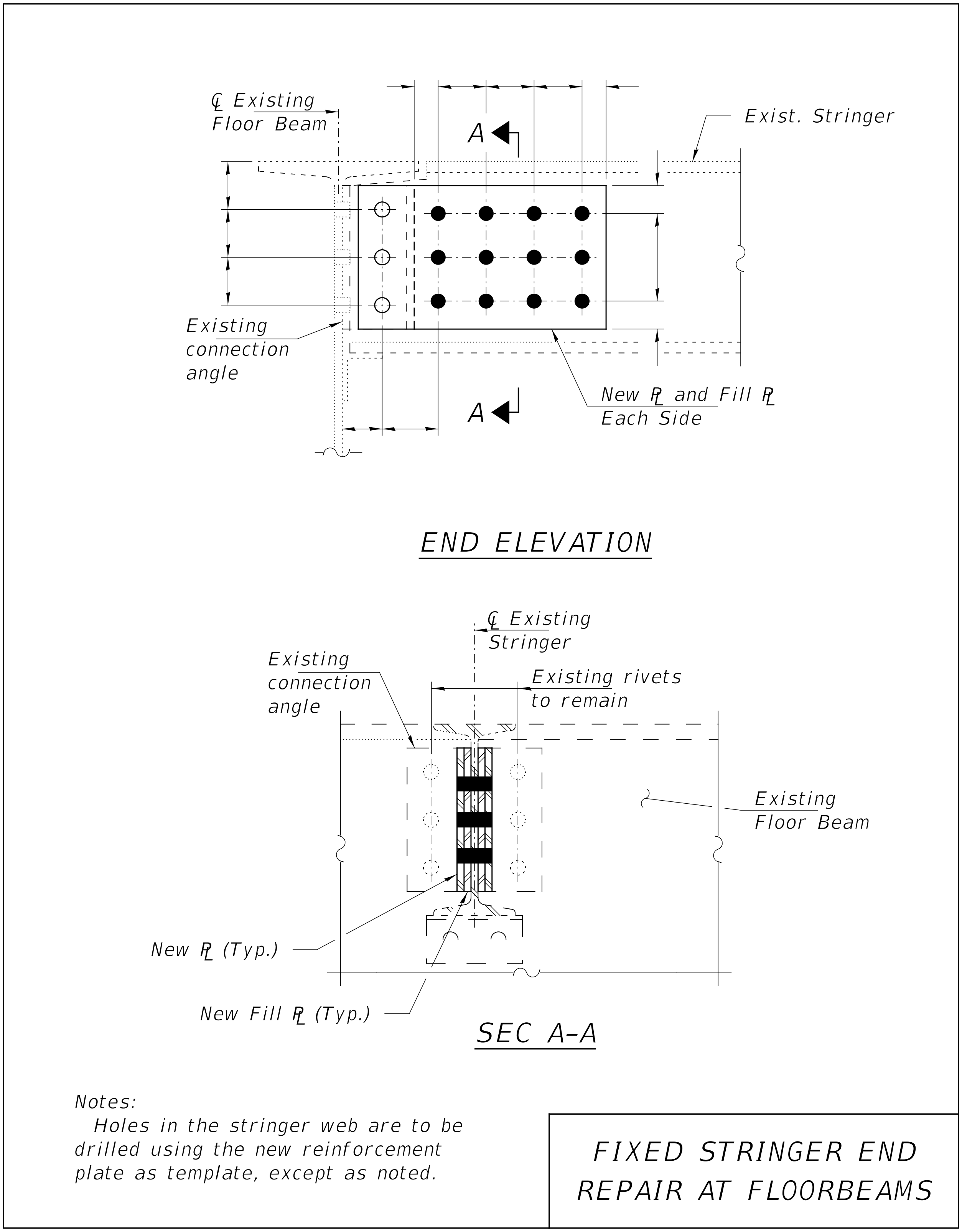


Figure 2.15.7-1: Fixed Stringer End Repair at Floorbeams

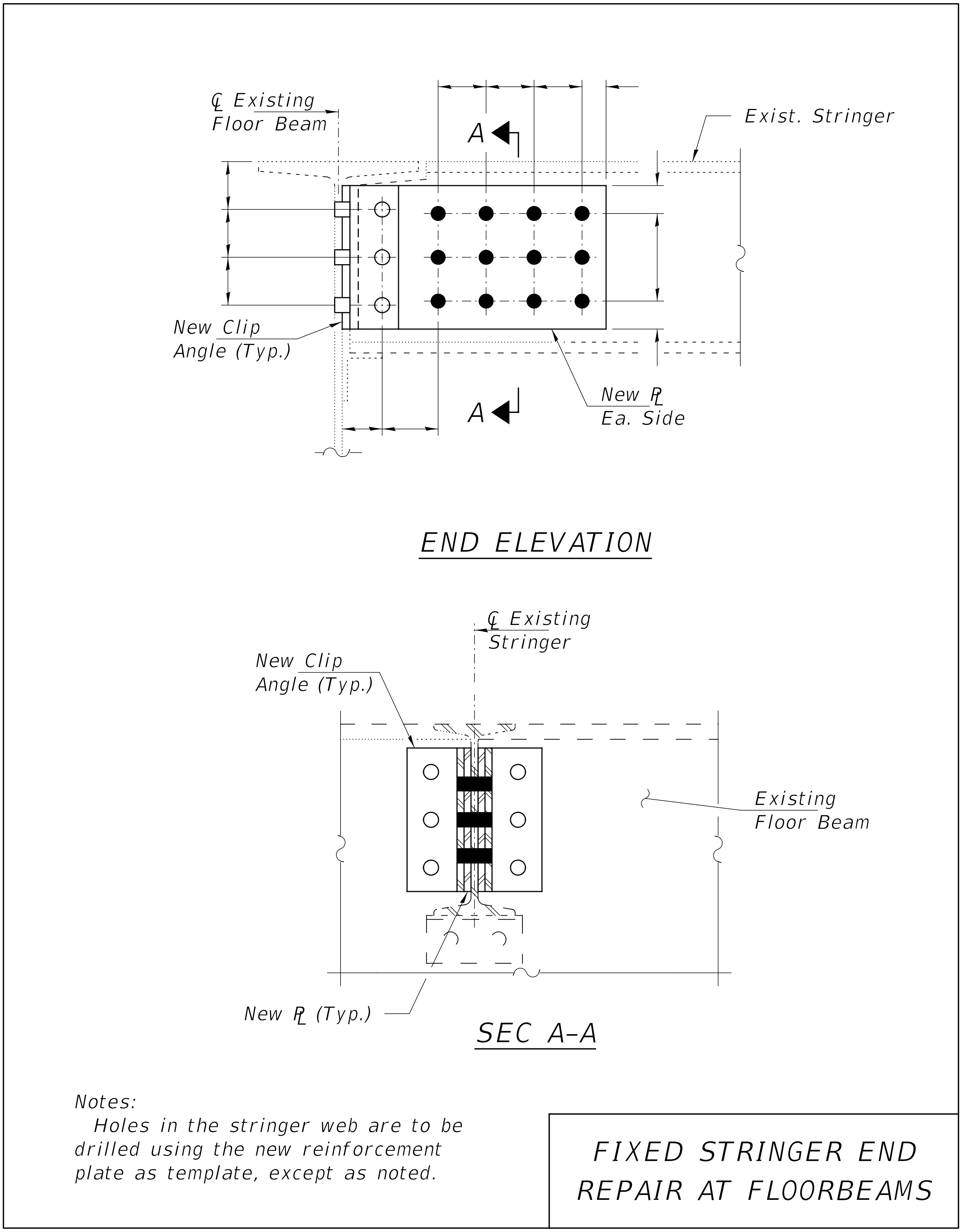


Figure 2.15.7-2: Fixed Stringer End Repair at Floorbeams

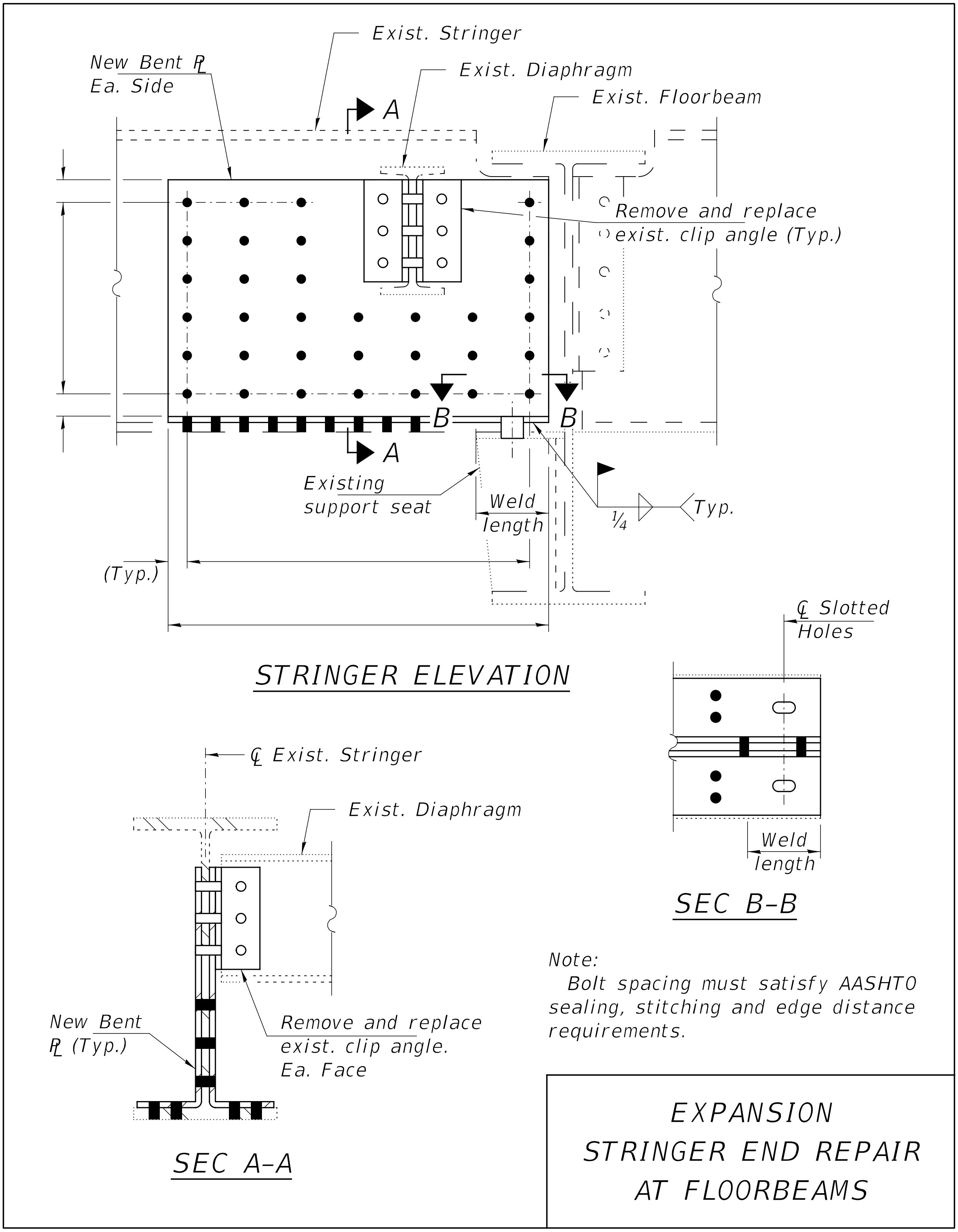


Figure 2.15.7-3: Expansion Stringer End Repair at Floorbeams

2.15.8 Floorbeam Repairs

Deterioration in the webs and flanges of floorbeams should be repaired in the manner previously described for the repair of beam and girder deterioration adjacent to deck drains. Figure 2.15.8-1 shows details used for the repair of an existing floorbeam constructed as a built-up section. The length of the floorbeam web strengthening plates will be limited by the spacing of stringers as the plate will need to slide between the stringer ends and the floorbeam web. Splices will be required to provide continuity. Because of the multiple plates/angles involved and the presence of existing stringers the designer must pay special attention to the constructability of the repairs to avoid fitting issues in the field.

Deterioration is commonly found at the locations where the floorbeams are connected to a truss lower chord. This deterioration, if significant, will compromise the carrying capacity of the floorbeam. The damage can in general be divided into two types: damage to the web of the floor beam near the connecting angle or damage to the connecting angle and bolts/rivets. The first type can be repaired by installing strengthening plates over the outstanding leg of the existing angle as shown on Figure 2.15.8-2 assuming that the size of the existing angle and location of existing bolts provide adequate edge distance for the new plate to be installed. The second type will require the removal and installation of a new angle. See Figure 2.15.8-3.

In both of these cases the floorbeam is being disconnected from the truss lower chord, hence a temporary support system is needed to transfer its load (both dead load and staged live load) to adjacent floorbeams.

When cracks develop between the top flange and web of the floorbeam in the area of the floorbeam to truss connection, angles can be installed as shown in Figure 2.15.8-4 after a crack arrestor hole has been placed at the tip of the crack.

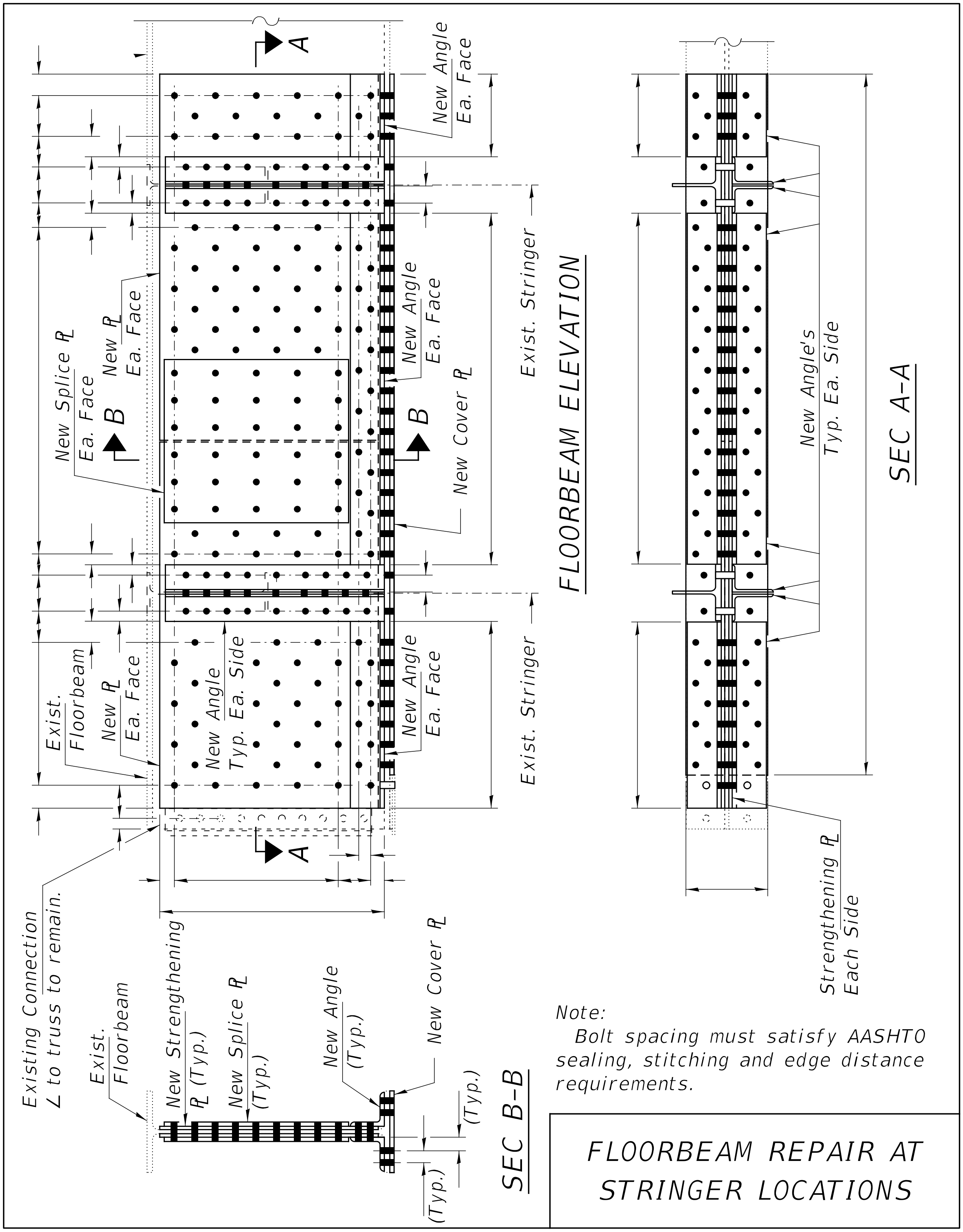


Figure 2.15.8-1: Floorbeam Repair at Stringer Locations

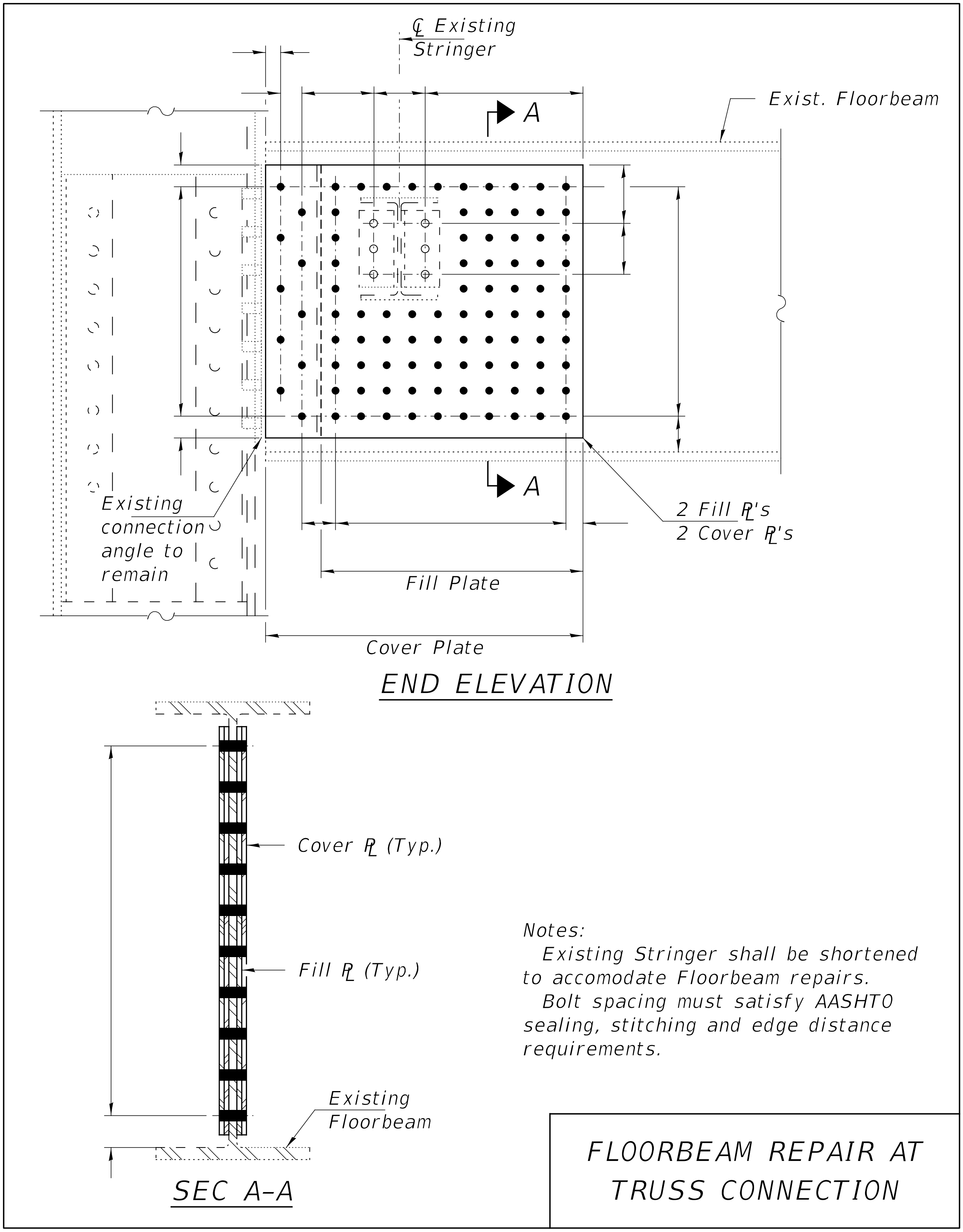


Figure 2.15.8-2: Floorbeam Repair at Truss Connection

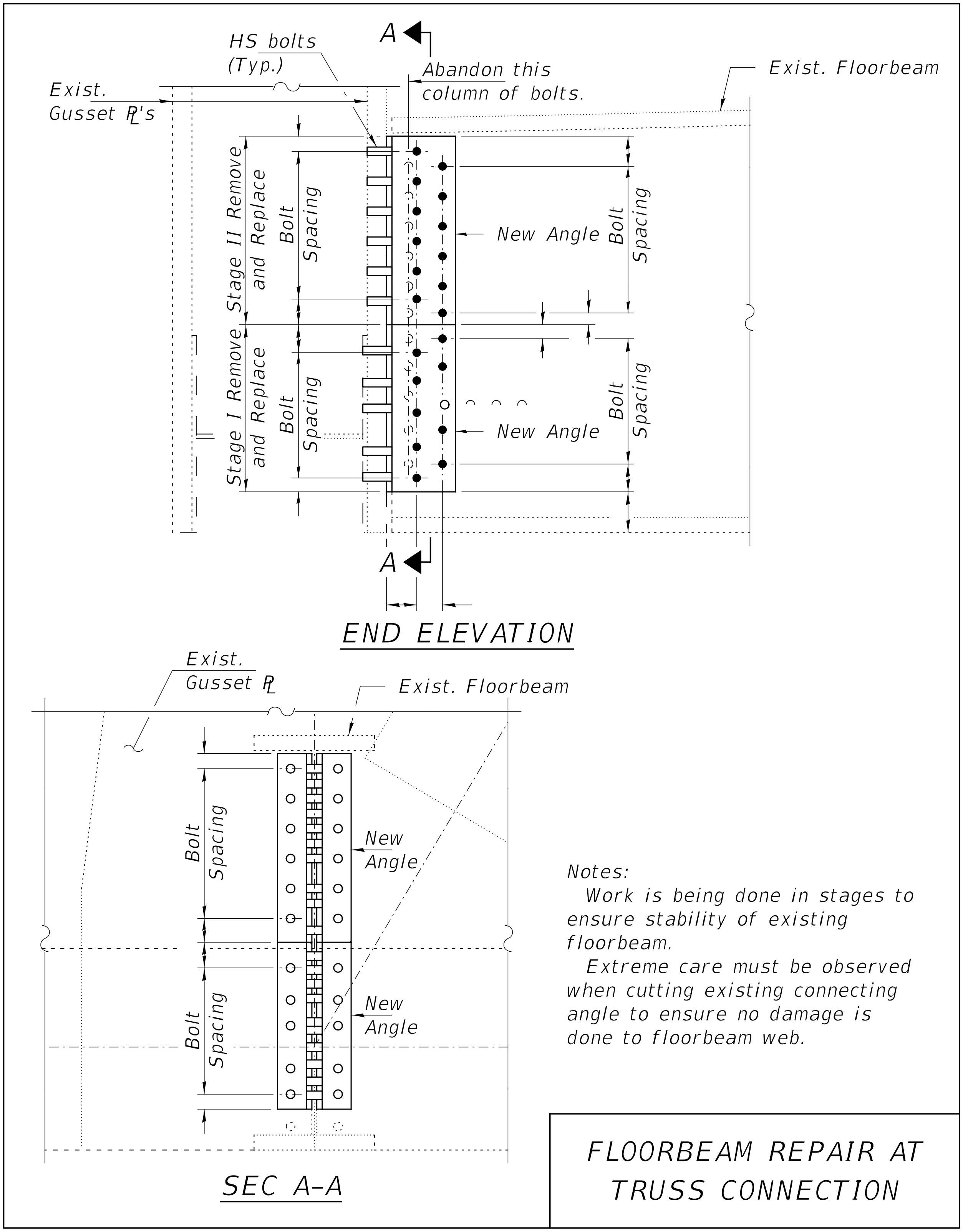


Figure 2.15.8-3: Floorbeam Repair at Truss Connection

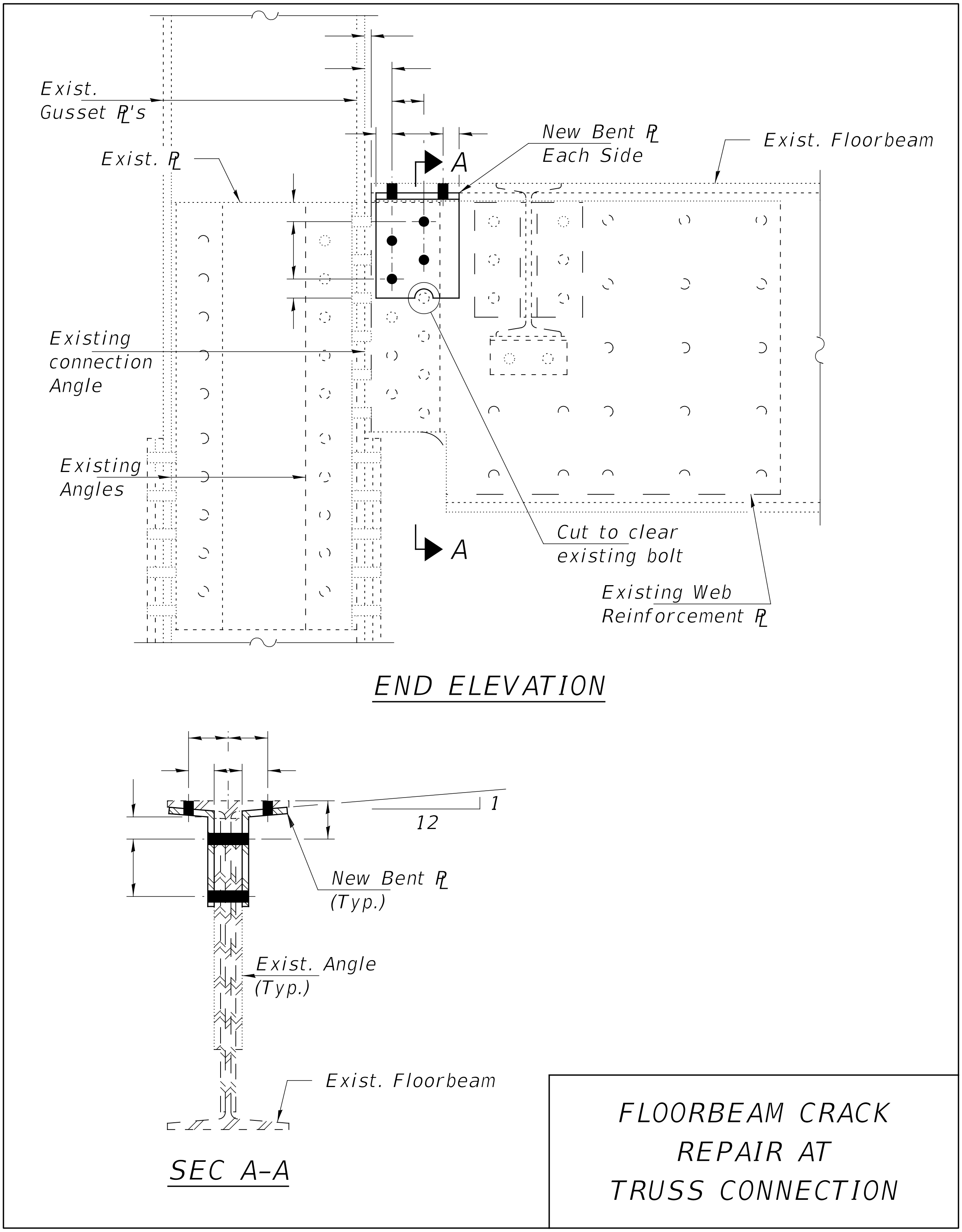


Figure 2.15.8-4: Floorbeam Crack Repair at Truss Connection

2.15.9 Construction Damage

There are many types of damage which can occur during construction. A new beam or girder can be dropped during erection, a piece of construction equipment can strike a portion of the structure which has already been constructed, or the beams can be damaged by equipment being used to remove an existing concrete deck. Damage due to impacts can be analyzed and repaired as described in Section 2.13 (Impact Repairs - Steel Beams) . This section will deal with the repair of steel beam flanges damaged by deck removal operations.

Plans for projects where portions of the existing deck will be removed should include a note instructing the contractor to mark the top surface of the existing deck to identify the location and limits of the top flanges of the beams prior to the commencement of deck removal operations. Special care is required for the removal of the concrete directly over the beams. It is not uncommon to find buried in the concrete steel stubs welded to the top flange of the fascia beams abandoned by the original contractor after the deck was poured. Removal and required grinding is to be completed as indicated in the general notes.

When a beam is damaged by deck removal operations, it is the contractor’s responsibility to repair the damage at his/her own expense. The contractor will be responsible for retaining a licensed structural engineer (State of Illinois) to analyze the effect of the damage on the structure, to make repair recommendations and to prepare repair details. The structural engineer’s analysis, recommendations and repair details will be submitted to the Bureau of Bridges & Structures for review and concurrence prior to the implementation of repair.

When the flange of a beam has been bent, nicked, gouged or cut during the removal of the existing deck, each damaged location must be analyzed to determine the effect of the damage on the load carrying capacity of the structure and on the serviceability of the structure (fatigue life). The damaged area should be ground parallel to the primary stress to remove the defects and to provide a 3:1 taper from the bottom of the defect to the undamaged steel surface. The cross sectional area remaining after grinding operations should be analyzed to determine the maximum stresses and stress range expected to occur at the damaged location if left unrepaired. Damaged locations where the stresses or stress range are determined to be unacceptable, must be strengthened by placing splice plates across the damaged area to return the damaged beam to its original load carrying capacity. The grinding details and the flange strengthening details shown on Section 2.13 (Impact Repairs - Steel Beams) can be applied to the repair of nicked, gouged or saw cut flanges.

Under no circumstance shall heat be used to straighten a bent member.

2.16 Concrete Superstructure Repairs

2.16.1 General

Concrete superstructures are exposed to the elements, and like steel superstructures, suffer deterioration damage that needs to be addressed. In order to help protect concrete bridge decks and their supporting elements, a program has been implemented to apply a concrete sealer on all concrete deck surfaces and overlays on a regular schedule. Originally the program was intended for bridges with a deck condition rating ≥ 6. As contractors have become more familiar with the sealer application and the cost per application has decreased, some districts have added bridges with a deck condition rating of 5 to the program. This sealer is currently being applied on a 3-to-4-year interval to the deck surfaces, front face and top of concrete parapets.

Deterioration of concrete support elements first become evident by spalling of the concrete surface. Loss of concrete cover reduces the protection of the reinforcement. Shallow mortar repairs do not last and when installed over traffic areas can become hazardous to the users if they become dislodged. To slow down, if not prevent, moisture from getting to the reinforcement it is prudent to at least apply epoxy to the spalled areas.

When the damage is more significant the repair methods described below should be implemented.

2.16.2 PPC I Beam Repairs

PPC I beams are very resilient and long lasting bridge elements requiring little maintenance. Nevertheless, like steel structures, their ends are susceptible to damage from exposure to water and de-icing elements under a failed joint. Areas near drains are also prone to damage. The deterioration normally starts with spalling of the concrete cover due to internal reinforcement corrosion and pack rust. This damage progresses to further expose reinforcement to moisture that increases the concrete spalling. On beam ends this eventually compromises the shear capacity of the member. If left unattended it may also damage the bearing area on the bottom flange. Near drains the spalling concrete will eventually expose highly stressed strands to deterioration.

The repair of deteriorated beam ends is among the most common repair procedures performed during maintenance contracts. Following the recommendations of a report for ICT Project R27-

156 “Repair and Strengthening of Distressed/Damaged Ends of Prestressed Beams with FRP Composites” a procedure to strengthen the beams has been developed that combines the use of structural repair of concrete and the installation of a fiber reinforced polymer (FRP) material.

In general, damage to be repaired on PPC I beam ends can be catalogued as follows:

1. Damage to bottom flanges at bearing (See Figure 2.16.2-1)
2. Damage to web above bearing (See Figure 2.16.2-2)
3. Damage to shear-critical locations (in front of bearings). See Figure 2.16.2-3

Sometimes the damage extends to more than one of the areas described above. This more severe damage should be repaired as shown Figures 2.16.2-4 and 2.16.2-5. Damage of this nature would have likely also compromised the bearing which will need to be removed and replaced. Temporary jacking and cribbing of the beam will be required.

Damage within the span to be repaired is normally divided between:

1. Minor (concrete spalled with no strands exposed).
2. Major (significant concrete spalling and strands exposed with or without damage) See Figure 2.16.2-6

Note that major repairs typically require the installation of a preloading system as described in Section 2.14 prior to completion of the concrete repairs. If preloading is required, the FRP cannot be installed until the preloading has been removed. Minor repairs do not require a preloading.

Special provisions have been developed for the concrete repairs procedures as well as the FRP material and placement requirements and are available by contacting the Repairs Unit of the Structural Services Section of the Bureau of Bridges and Structures. These special provisions need to be included with the contract plans.

For this procedure to perform as expected it is critical that the FRP material properties shown in the special provision are satisfied, the manufacturer recommendations are carefully followed and the personnel installing the material are fully trained and experienced in this type of work. Following the parameters of the report the FRP should include carbon fibers.

At drain locations or other similar locations in which the FRP is used as a containment mechanism the overall procedure is the same as on the beam ends except that both glass or carbon fibers are allowed.

Bulb T-Beams, however similar to PPC I Beams, require special consideration due to the geometry of the beams. Please contact the Repairs Unit of the Structural Services Section of the Bureau of Bridges and Structures should repairs need to be made.

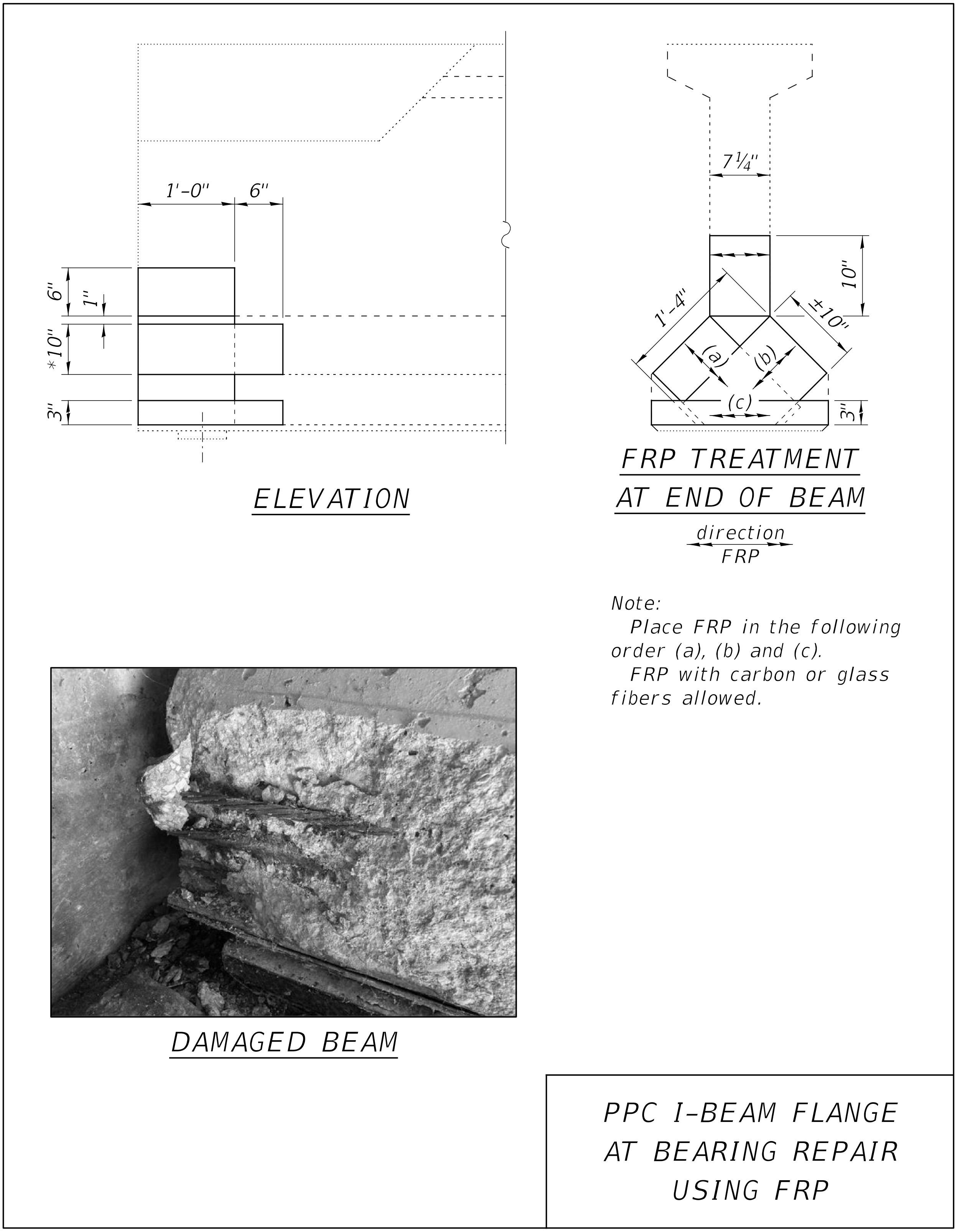
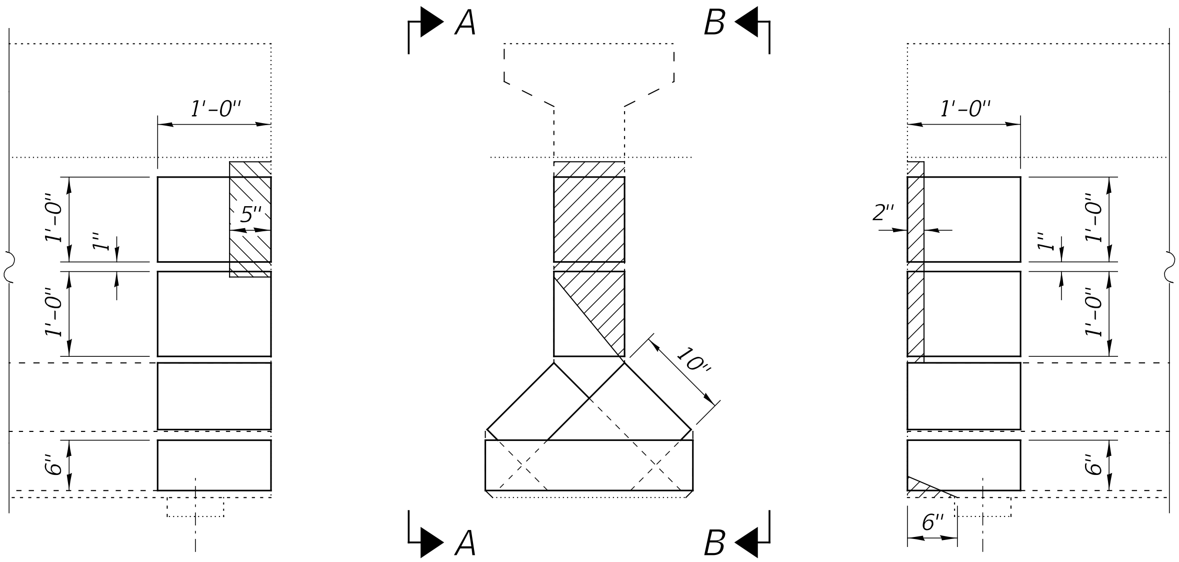


Figure 2.16.2-1: PPC I-Beam Flange at Bearing Repair Using FRP



SEC A-A BEAM END ELEVATION SEC B-B

DAMAGED BEAM

Note:
FRP with carbon or glass fibers allowed.

PPC I-BEAM WEB
OVER BEARING REPAIR
USING FRP

Figure 2.16.2-2: PPC I-Beam Web Over Bearing Repair Using FRP

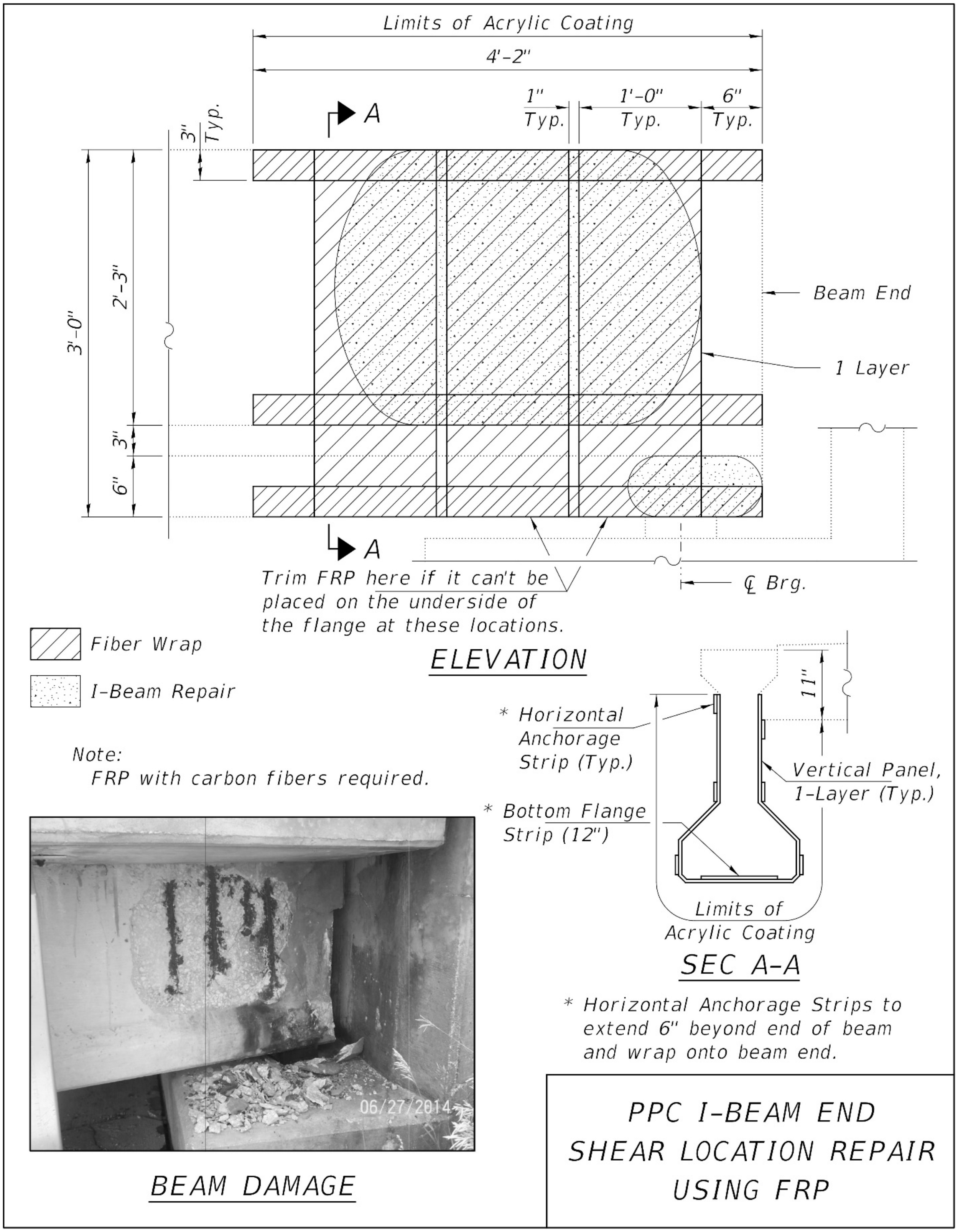


Figure 2.16.2-3: PPC I-Beam End Shear Location Repair Using FRP

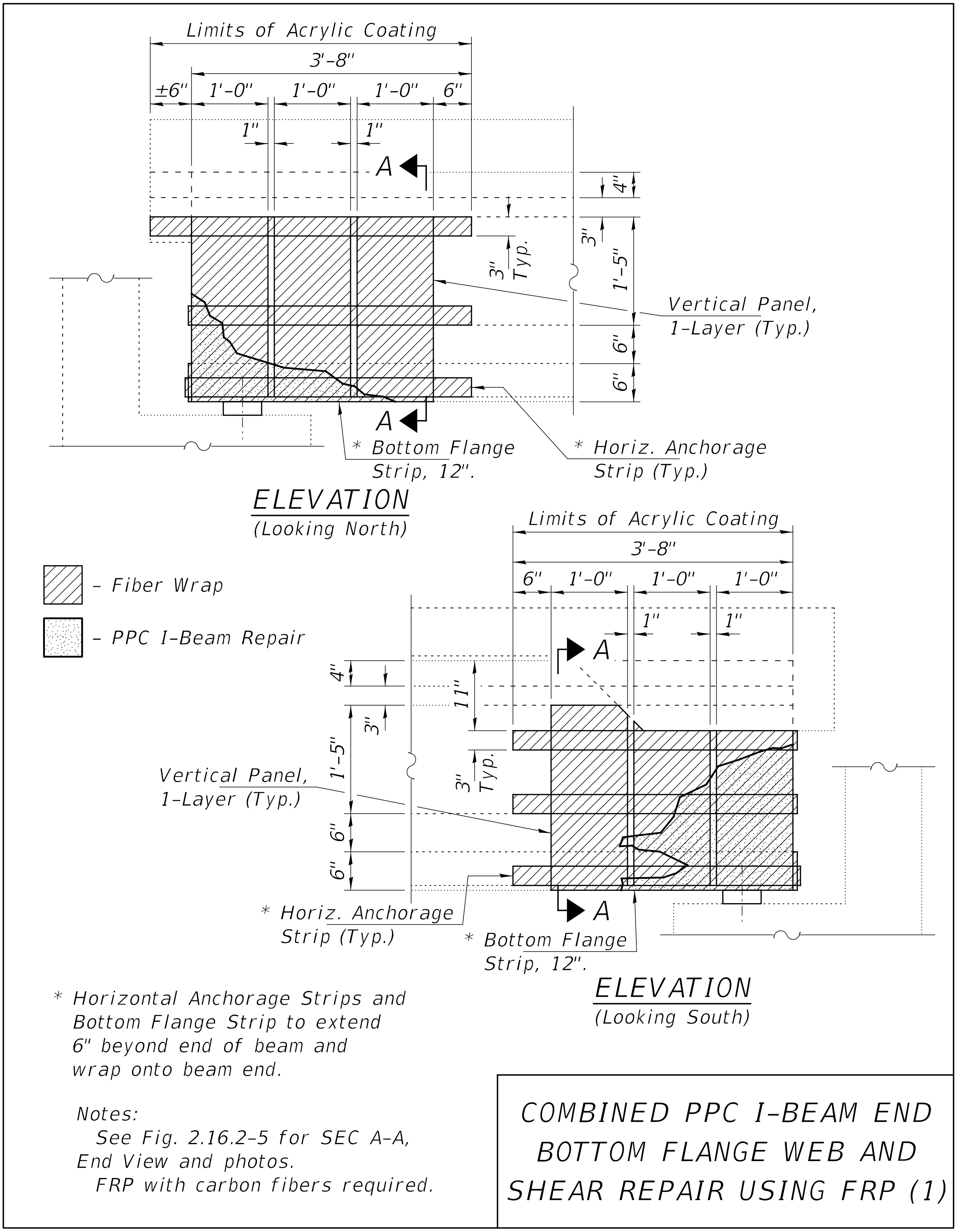
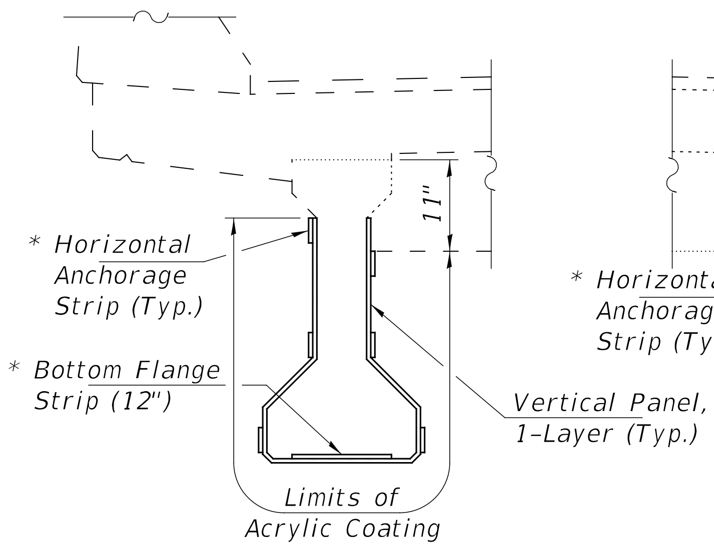
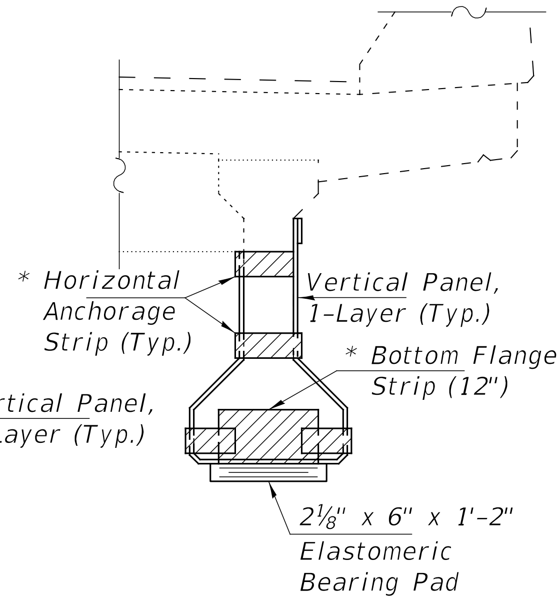


Figure 2.16.2-4: Combined PPC I-Beam End Bottom Flange Web & Shear FRP Repair 1



SEC A-A



END VIEW

* Horizontal Anchorage Strips and Bottom Flange Strip to extend 6" beyond end of beam and wrap onto beam end.

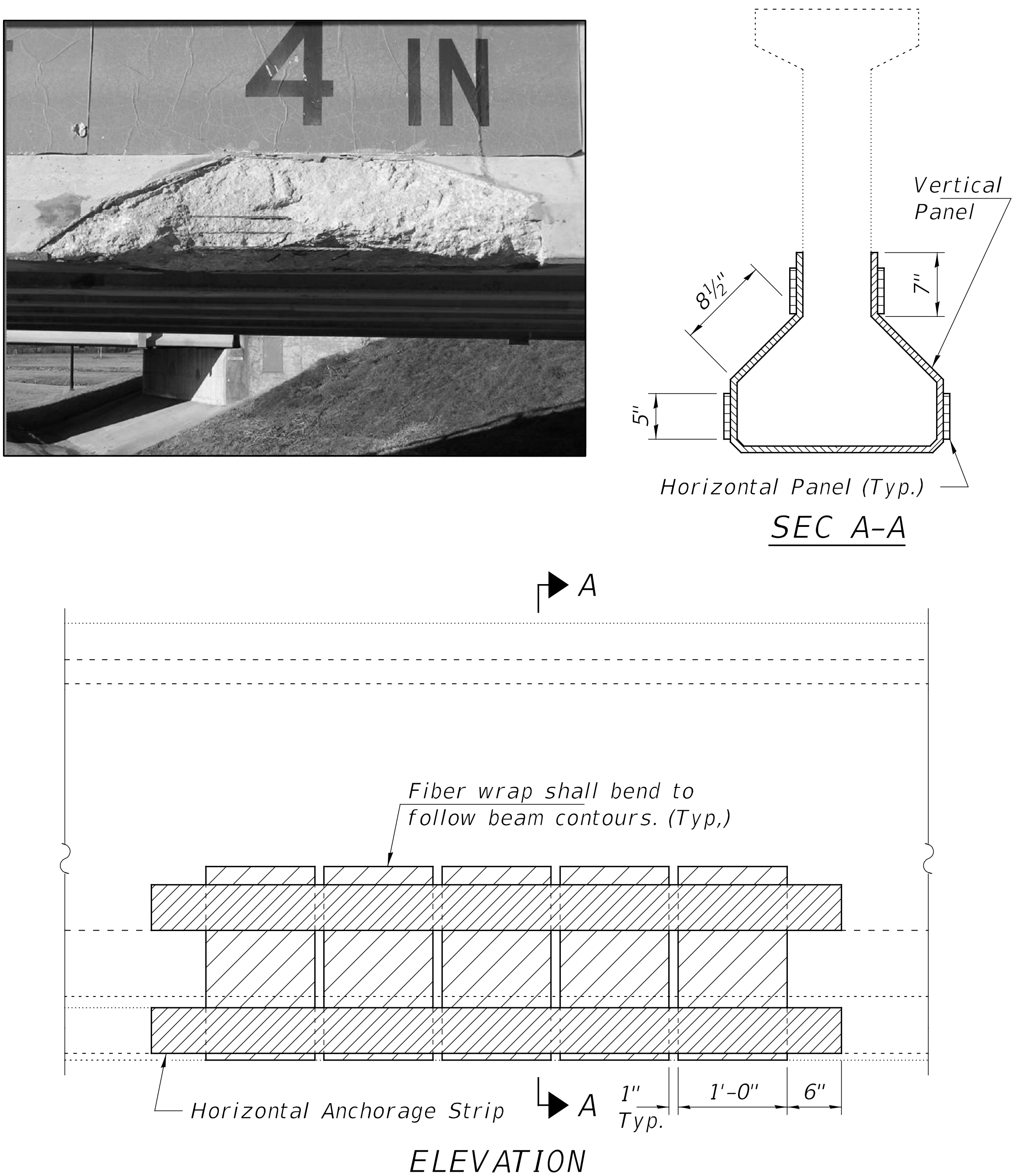
BEAM DAMAGE

BEAM REPAIR

Note:
FRP with carbon fibers required.

COMBINED PPC I-BEAM END BOTTOM FLANGE WEB AND SHEAR REPAIR USING FRP (2)

Figure 2.16.2-5: Combined PPC I-Beam End Bottom Flange Web & Shear FRP Repair 2



Note:
FRP with carbon or glass fibers allowed.

MAJOR PPC I-BEAM REPAIR WITHIN THE SPAN USING FRP

Figure 2.16.2-6: Major PPC I-Beam Repair Within the Span Using FRP

2.16.3 PPC Deck Beam Repairs - General

Among the most common repairs necessary for the maintenance of PPC deck beams are:

1. The replacement of the wearing surface
2. The repair of keyways
3. The repair of deteriorated concrete and the protection of exposed reinforcement and strands
4. The adjustment of bearing pads
5. The repair of grouted dowel rod holes
6. The repair of grouted joints at the fixed beam ends

Plans for projects including these repairs should contain a plan view of the superstructure showing the locations of all keyways, deteriorated beam areas, expansion joints and fixed joints. The locations to be repaired should be identified on the plan view. Special provisions for the repair procedures listed above can be obtained by contacting the Repairs Unit of the Structural Services Section of the Bureau of Bridges & Structures.

The existing wearing surface may be removed and replaced as described for overlays in Section 2.4.8. The repairs listed above must be included with all overlay projects on PPC deck beams when deterioration is present.

After the wearing surface and waterproofing have been removed from deck beams, any existing unplugged fabrication vent holes in the top of the beams should be located and sealed with epoxy to prevent water from entering the voids cast inside the beams. Also, during the inspection of PPC deck beams, the existing drain holes extending through the bottom of the beams into the voids inside the beam should be probed to ensure that they are not clogged, and that water is not accumulating in the voids.

2.16.3.1 Keyway Repairs

The grouted keyways between the individual beams, which combine to form the bridge superstructure, are essential for the lateral distribution of load during the application of live loads. If the keyways adjacent to a beam are severely deteriorated, that beam must carry a portion of the live load significantly greater than the amount of live load the beam was designed to carry.

When this condition exists, the Bureau of Bridges & Structures should be contacted to determine if a load restriction must be placed on the structure until repairs can be made.

Keyways are repaired by removing all deteriorated grout from the keyway and replacing it with a non-shrink grout. In areas of the keyway where the existing grout is cracked but otherwise sound, the cracks in the grout should be filled with epoxy. The grouting and crack sealing should not be done until all PPC deck beam repairs, bearing pad adjustments and dowel rod hole repairs have been completed.

When the edges of the beam forming the keyway have deteriorated to the point where they cannot provide the geometric shape necessary to contain the keyway grout, consideration must be given to beam replacement or the installation of a mechanical keyway device. A device similar to the one shown in Figure 3.5.9-2 of the IDOT Bridge Manual for use during the staged construction of a PPC deck beam bridge can be used. The device details should be modified as shown in Figure 2.15.3.1-1. When the superstructure is over a roadway or railroad, the effect of the mechanical devices on vertical clearance must be considered. Also, since the devices will interfere with the placement of waterproofing membrane and may loosen with time, they should only be installed to avoid total beam replacement on structures scheduled for replacement.

When the keyway deterioration is due to the lateral movement of the beams relative to one another, side retainers should be installed adjacent to the fascia beams to prevent further movement. Figure 3.5.7.5-1 of the IDOT Bridge Manual provides details for side retainers which can be installed adjacent to the beams when adequate bearing seat area is available.

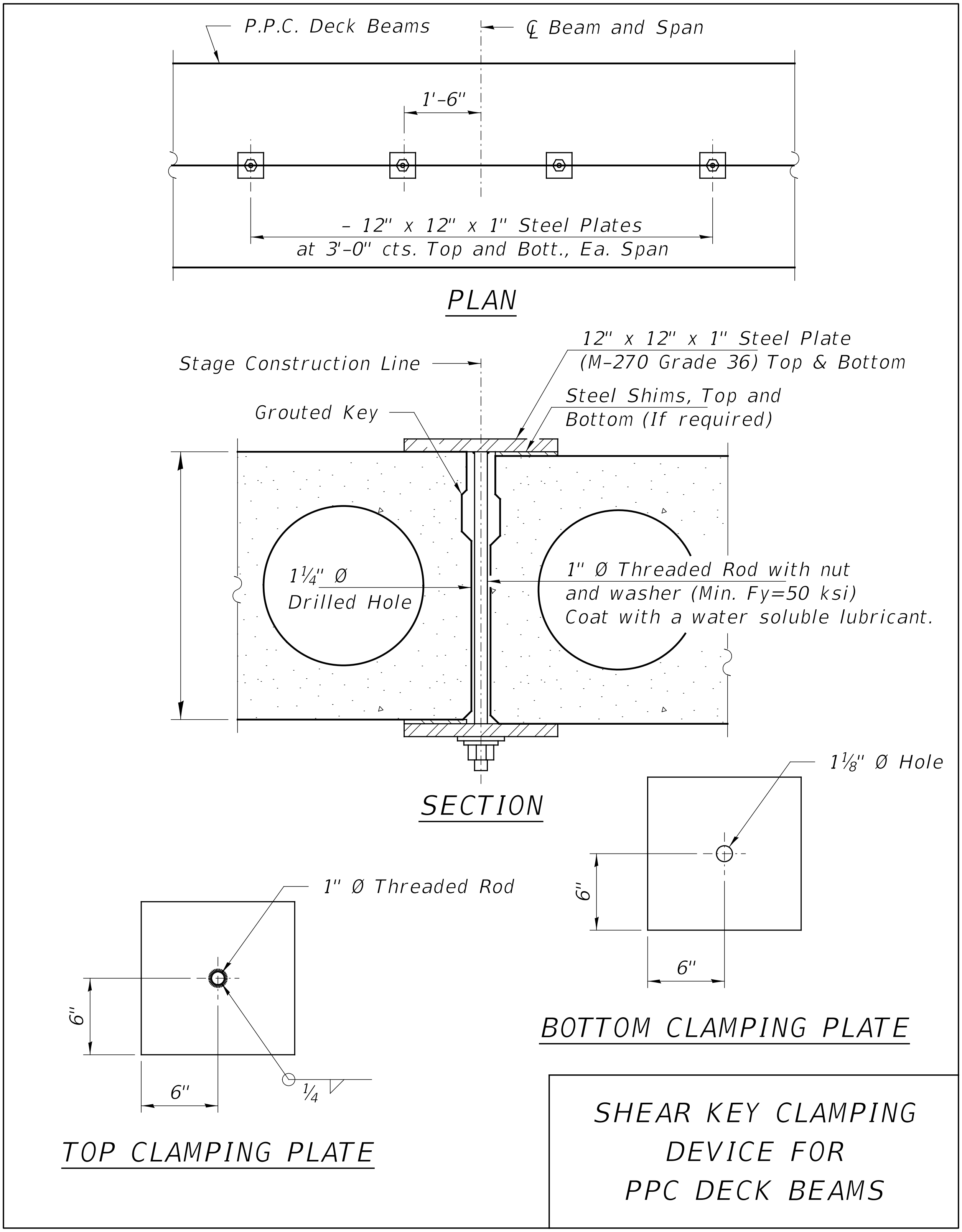


Figure 2.16.3.1-1: Shear Key Clamping Device for PPC Deck Beams

2.16.3.2 Bearing Pad Adjustment

The bearing pads located under the ends of the PPC deck beams must be inspected to identify areas where uniform bearing of the beams on the pads does not exist. These areas will be prone to rocking when live load is applied to the structure and will contribute to the accelerated deterioration of the keyways between the beams. Uniform bearing should be established by inserting steel shims under the beam ends where rocking is occurring. At fixed bearings, where the ends of the beams are held in place by dowel rods, uniform bearing can be provided by the placement of epoxy grout between the bottom of the beams and the bearing seat in lieu of furnishing shims.

2.16.3.3 Dowel Hole and Grout Joint Repairs

Deterioration of the grout in the dowel rod holes and the grout joints at the ends of the beams can occur due to the rocking of poorly supported beam ends or due to the accumulation of moisture under the overlay at the ends of the beams. After the existing overlay has been removed, all deteriorated grout should be removed from the dowel rod holes and grouted joints. Cracks in existing grout to remain in the dowel holes should be sealed with epoxy. Non-shrink grout should then be used to replace the removed deteriorated grout.

2.16.4 PPC Deck Beam Repairs

Experience has shown that Repairs to PPC deck beams are typically not effective in prolonging the life of the beams. Shallow mortar repairs on the underside of the beams has proven ineffective. Once reinforcement bars or prestressing strands become exposed in portions of the beams away from the supports, it is expected that the life of the beam is very limited and programming for beam or superstructure replacement should be initiated. When exposed reinforcement bars or prestressing strands are present or concrete deterioration extends into the voids inside the beams, the Bureau of Bridges and Structures should be contacted to determine if a load restriction must be placed on the structure or if beam replacement or temporary support is necessary.

If a temporary support is needed until the beam or the superstructure can be replaced it will need to be designed to carry its own weight, the full dead load of the PPC deck beam it is supporting and a portion of the expected live load and impact. Conservatively it can be assumed that the shear keys on the damaged beam do not transfer the load laterally and half of an axle load will need to be supported. Figure 2.16.4-1 shows details for a temporary steel support beam system. Because of the camber on the PPC deck beam and deflection of the steel beam under its own weight, steel shim packs must be provided to ensure proper contact between the damaged PPC deck beam and the steel support.

When a PPC deck beam must be replaced, the replacement beam’s load carrying capacity must not be less than the capacity of the existing beams remaining in place. The number and size of the prestressing strands placed in the replacement beam should provide the required capacity and should conform with current design and fabrication practices. Designers should also be mindful of the resulting camber of the new beams. Because the new beams have larger and stronger strands it is likely that a replacement beam will have a significantly different camber than the beam it is replacing. When that happens it will be difficult to align the transverse ties that need to be reconnected. To ensure that the replacement beam has similar camber to the existing one it may be necessary for the designer to consider an alternate arrangement and/or number of strands. When replacing an interior beam, the width dimension of the replacement beam should be reduced a maximum of 1/2 inch to facilitate the placement of the new beam between the existing beams. If two or more adjacent beams are being replaced, the width of one of the beams shall be reduced by 1”. It has been found that for fabrication purposes this is preferred over reducing the width of each replacement beam by a fraction of an inch. Whenever possible, transverse tie assemblies should be reinstalled. A modified tie coupler for use with a beam replacement is shown in Figure 2.16.4-2. It is not always possible to reinstall the transverse ties on structures with large skews and staggered tie assemblies. When transverse ties cannot be reinstalled, side retainers should be installed adjacent to the fascia beams at the ends of the beams which are not connected to the substructure with dowel rods and are not tied together by expansion joint steel.

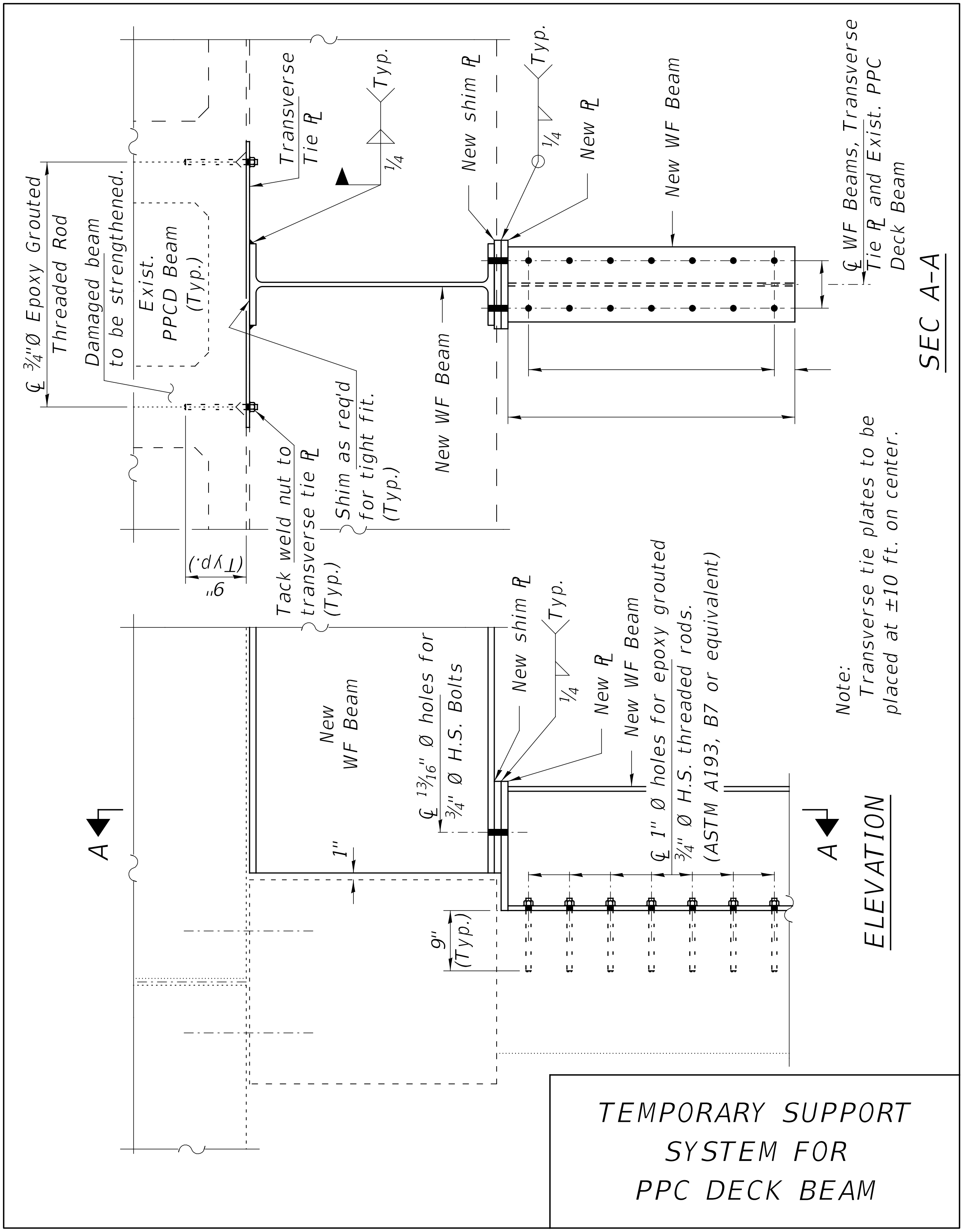


Figure 2.16.4-1: Temporary Support System for PPC Deck Beam

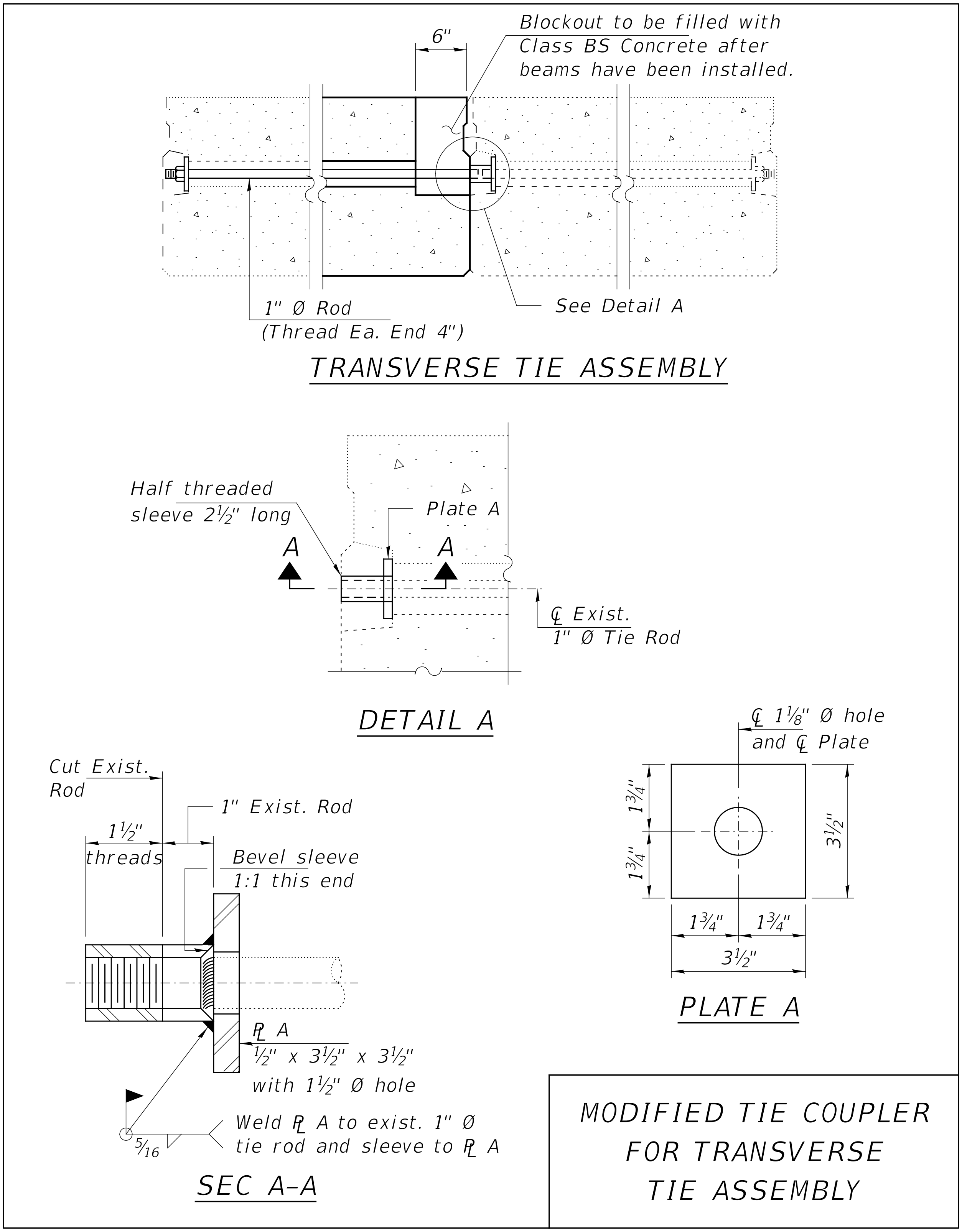


Figure 2.16.4-2: Modified Tie Coupler for Transverse Tie Assembly

2.16.5 R.C. Deck Girder Replacement

The primary cause of deterioration in R.C. deck girder ends is exposure to deicing agents. The resulting concrete spalling not only exposes the shear reinforcement at the girder end to corrosion but may also compromise the anchorage and result in the debonding of the existing main tension reinforcement in the bottom of many R.C. deck girders. Similar damage has also been found near the location of deck drains.

When the capacity of the girder has been compromised, a temporary support system can be installed to add support to the member to carry the expected loads while a more permanent solution is implemented. A support system as shown in Figure 2.16.5-1 has been used effectively on a R.C. deck girder structure in which the shear and main bottom reinforcement was exposed and showed significant section loss.

Another notable deterioration issue on this type of girder is the development of shear cracks near the quarter point running upwards from the bottom of the stem. These cracks are likely the result of insufficient bar lap (as per current standards) for the main reinforcement. In the past structural repair of concrete in combination with the installation of FRP has been found to be an effective repair procedure. The use of external post-tension rods has also been implemented to arrest the crack growth and propagation.

When deterioration requires that a girder be replaced, the girder can be replaced in-kind or a precast prestressed concrete deck beam can be used as shown in Figure 2.16.5-2. The use of the precast beam will avoid many of the problems associated with forming and temporarily supporting a cast in place R.C. deck girder.

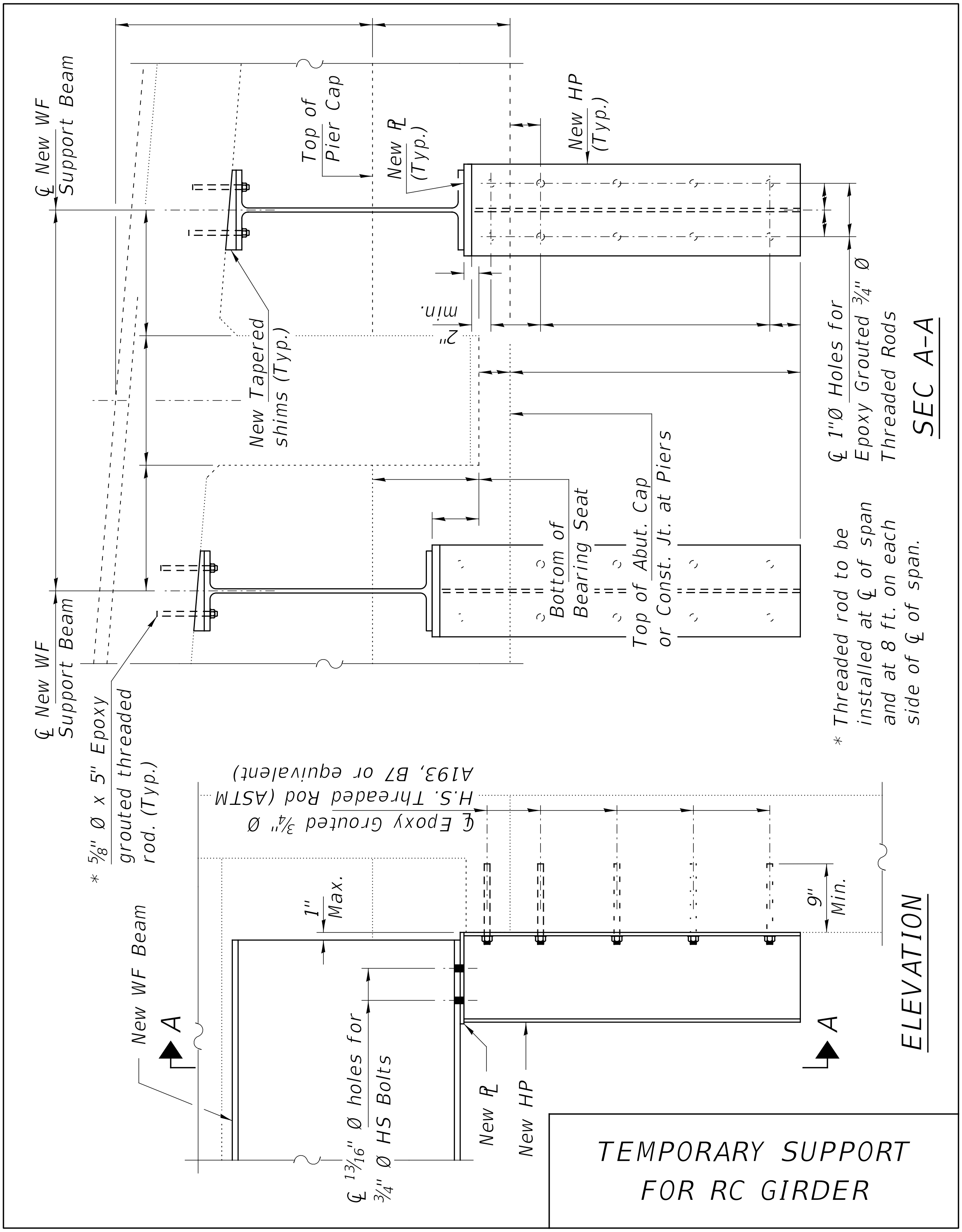


Figure 2.16.5-1: Temporary Support for R.C. Girder

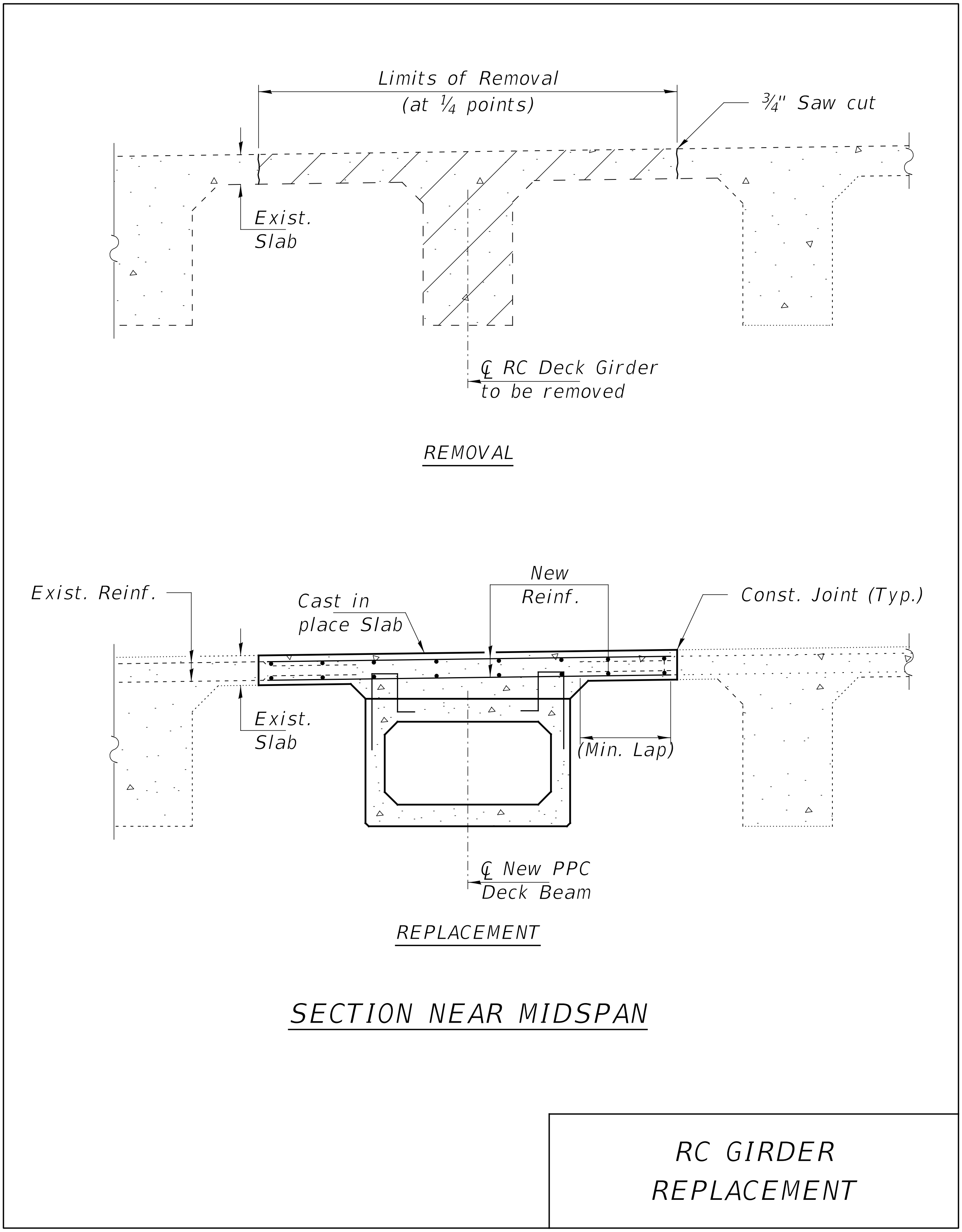


Figure 2.16.5-2: R.C. Girder Replacement

2.17 Substructure Repairs

2.17.1 General

The repair of cracks and areas of deteriorated/spalled concrete are the most common types of substructure repairs included in maintenance repair plans. The plans must contain information identifying the locations to be repaired. The information should consist of views of the top, faces and sides of the substructure showing the approximate size and location of all cracks and deteriorated concrete areas as shown in Figure 2.17.1-1 for a pier (similar details are needed for a deteriorated abutment). Of particular importance is the location of bearing plates in relation to the deteriorated area, as this may impact the need for temporary shoring and cribbing. The table in Figure 2.17.1-2 shows “Guidelines for the Selection of Substructure Repair Methods”.

When large cracks or extensive areas of deteriorated concrete are present, an investigation should be performed to determine the cause of the cracking or deterioration (failed joint, drainage too close to substructure element, insufficient concrete clear cover, insufficient load capacity for current loads etc.). The repair plans should include measures to eliminate the cause of the problems. In some cases, the corrective measures required to avoid the recurrence of the deterioration will simply call for the replacement of a leaking expansion joint or the elimination or extension of nearby deck drains.

Removal of deteriorated concrete may have an impact on the ability of the substructure element to carry the expected loads. As previously indicated in Section 2.10.5 the designer will determine the need for the installation of a shoring and cribbing system to be used while the substructure repairs are completed.

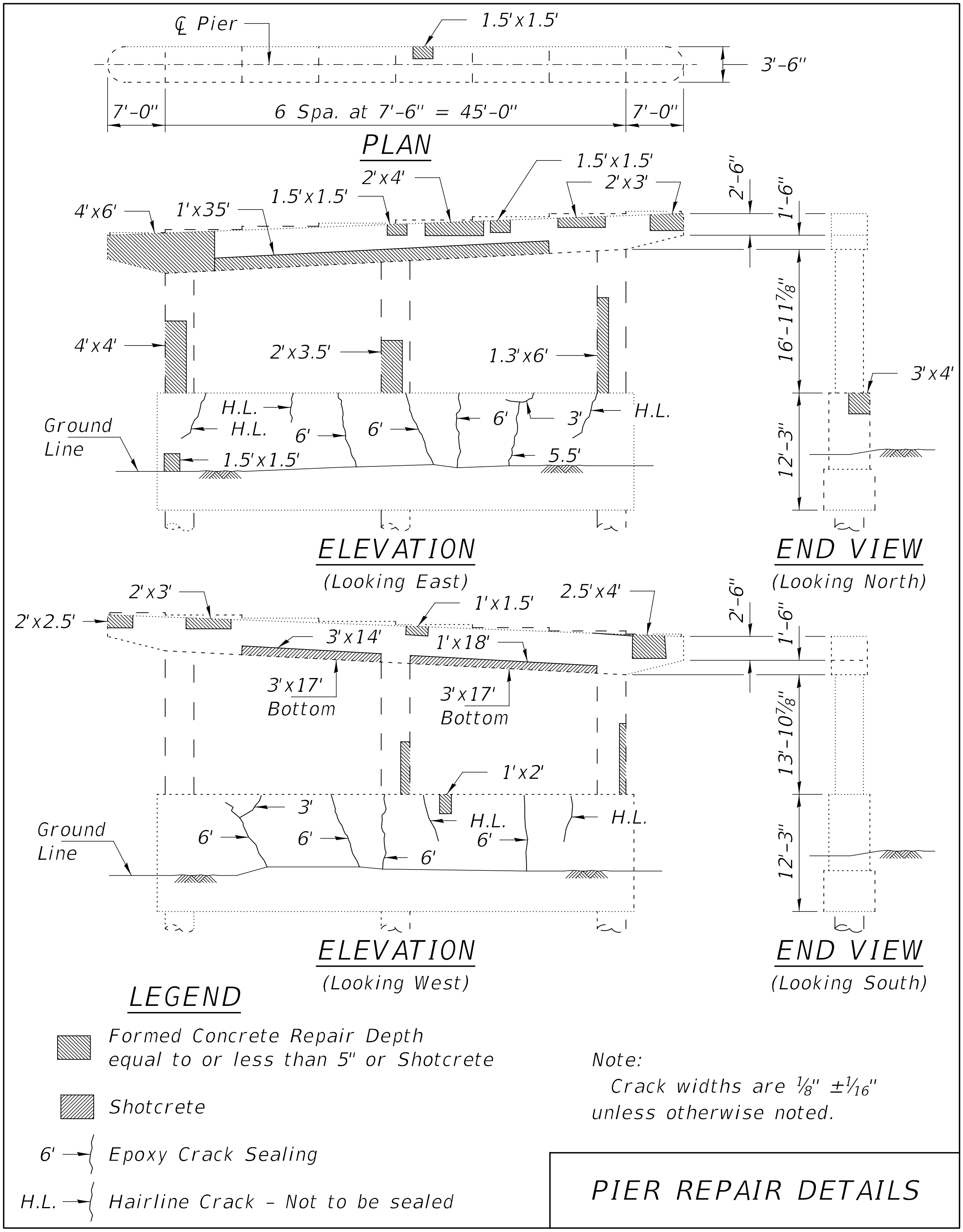


Figure 2.17.1-1: Pier Repair Details

Substructure Repair Type	Project Quantity	Repair Location	Limits of Repair	Aesthetic Requirements / Limits / Other Comments
* Formed Concrete Repair (FCR)	Recommended method to 100 ft^2. Alternate with high performance shotcrete for ≥ 100 ft^2.	Any location except overhead.	See GBSP 53 for additional requirements.	No restriction for aesthetics. Note: If repair of patch boundaries is necessary, ensure proper placement of repair material to avoid cracking and delamination. Also, ensure proper consolidation and curing of FCR to minimize honeycombing, cracking and delamination.
* High Performance Shotcrete (HPS)	Alternate to FCR for ≥ 100 ft^2. Note: HPS should be competitive with FCR for 100 ft^2 to 200 ft^2 if all repair area is localized. HPS should be competitive regardless of locality of repairs for > 200 ft^2.	Horizontal, Vertical and Overhead. Can be used in locations that are difficult to form. Must be used at locations called for in the plans.	8" max. depth, 3" maximum lift. See GBSP 53 for additional requirements. The total thickness or layer thicknesses may be exceeded for horizontal applications where the shotcrete can be applied from above in one lift, or if a manufacturer's representative is on site to assist on the placement procedure.	Acceptable for all areas for aesthetics. Note: Proper curing is extremely important.
Polymer Modified Portland Cement Mortar Repair	Recommended for < 10 ft^2, but 10 ft^2 is not a maximum limit. Can also be specified for other small quantities.	Horizontal, Vertical and Overhead (Consider potential hazard to users under structure).	Intended for very shallow repairs (2" max. depth).	Acceptable for all areas for aesthetics. Not a structural repair.
Concrete Encasement	Pier columns/stems and caps and closed abutments.	Generally intended for encapsulation to increase rebar cover (pier caps and stems).	Intended for shallow repairs. Can allow deep repair, but if repair depth > 8" then repairs should be done prior to encasement (leave rough surface).	Acceptable for all pier locations. Desirable to encase full height of pier column or pier cap faces and closed abutments. Paid as Concrete Stuctures and FRP if used.
Epoxy Crack Injection	Any Quantity	Intended for locations on structural elements exposed to water and/or salt intrusion. Generally not intended for non-structural elements (i.e. slope walls, crash walls).	Min. crack width should be more than thickness of a dime (1/16"). Crack length should be > 24" (to allow a min. of 2 zerks or nipples). Max. crack width 1/2". For crack > 1/2" use non shrink grout.	Minimize use where there is no expansion joint or other source of water intrusion. Ensure specifications are followed for stoning and grinding, zerk removal, etc. for aesthetics.

* Contractor's option under Structural Repair of Concrete unless indicated otherwise in plans.

GUIDELINES FOR THE SELECTION OF SUBSTRUCTURE REPAIR METHODS

Table 2.17.1-2: Guidelines for the Selection of Substructure Repair Methods

2.17.2 Crack Sealing

Cracks in concrete substructures, with crack openings less than or equal to 1/2 inch, are repaired by injecting an epoxy crack sealing material into the crack. Cracks located in a wall with voided areas behind the wall cannot be effectively sealed by injecting material into the cracks unless the voided area has been filled with material which will prevent the injected crack sealing material from being forced into the voided area. Hairline cracks need not be sealed but their locations should be noted in the repair plans.

Cracks with openings greater than 1/2 inch should be sealed by removing all loose material along the edges of the crack and then using a non-shrink grout to fill the crack.

2.17.3 Mortar Repairs

Shallow repairs of deteriorated concrete substructure surfaces can be made using polymer modified portland cement mortar. This type of repair should be limited to the repair of concrete surfaces when the depth of the deterioration does not extend below the existing reinforcement bars or to a depth of 2 inches for unreinforced substructure units. Polymer modified portland cement mortar repairs are considered non-structural and are intended to re-establish the geometry of the substructure element. It should not be used in overhead applications (see Section 2.17.5).

2.17.4 Formed Concrete Repairs

Formed concrete repairs are structural repairs paid for as Structural Repair of Concrete. With certain limitations as covered in the special provisions for Structural Repair of Concrete, the decision to use formed concrete repair or pneumatically placed mortar is made by the Contractor. The repairs can be categorized into the following two types:

1. Formed concrete repairs with depths of repair less than or equal to 5 inches
2. Formed concrete repairs with depths of repair greater than 5 inches

Both types of repairs use formed concrete to replace the deteriorated concrete areas.

When making a formed concrete repair, the perimeter of the deteriorated area should be saw cut to a depth of 1/2 inch after the limits of unsound concrete have been determined. The existing concrete within the saw cut area should be removed to sound concrete. Reinforcement bars that are more than 50 percent exposed shall be undercut to a depth of ¾ inch or the diameter of the reinforcement bar, whichever is greater. The existing reinforcement bars within the repair area must be cleaned and damaged or deteriorated reinforcement bars must be replaced prior to the placement of the repair concrete.

The effect of the concrete removal, required for a formed concrete repair, on the strength of the structure during the repair operations must be considered during plan development. When determined by the designer to be necessary to repair an extensive area of the substructure, such as an entire column face, the superstructure should be temporarily supported until the concrete repair has cured (See Section 2.10.5) and the shoring and cribbing shall be called for in the plans. Other criteria that would require further evaluation and/or installation of shoring and cribbing during construction are included in the special provision for Structural Repair of Concrete.

2.17.5 Pneumatically Placed Mortar

High strength pneumatically placed mortar (shotcrete) may be used in lieu of formed concrete repairs. Like formed concrete repairs, shotcrete is a structural repair paid for as Structural Repair of Concrete. This repair method option is particularly effective when large areas of concrete need repair or when repairs are needed in overhead applications. The saw cutting, concrete removal and reinforcement bar repair requirements for formed concrete repairs should also be applied to shotcrete repairs. Because of the equipment needed to complete this work, its use on small contracts with small quantities of repairs may be cost prohibitive.

Shotcrete shall be used in all overhead applications where Structural Repair of Concrete is required. Plan details shall clearly indicate areas where shotcrete is required. Shotcrete thickness is typically limited to 8 inches, with a maximum lift thickness of 3 inches. The total thickness or layer thickness may be exceeded for horizontal applications where the shotcrete can be applied from above in one lift, or if a manufacturer’s representative is onsite to assist in the placement procedure.

2.17.6 Abutment/Pier Wall Encasement

As a result of a lack of concrete cover and/or exposure to deicing chemicals from leaking expansion joints, sometimes large sections of an abutment or pier face deteriorate. In these cases, large sections of concrete are spalled off exposing corroding reinforcement. An effective repair method is to install a full encasement of the damaged face. Typically, a 6” reinforced concrete section is placed. Horizontal and vertical reinforcement is used in addition to L shape bars that are epoxy grouted into the original concrete (See Figure 2.17.6-1 for an abutment wall example).

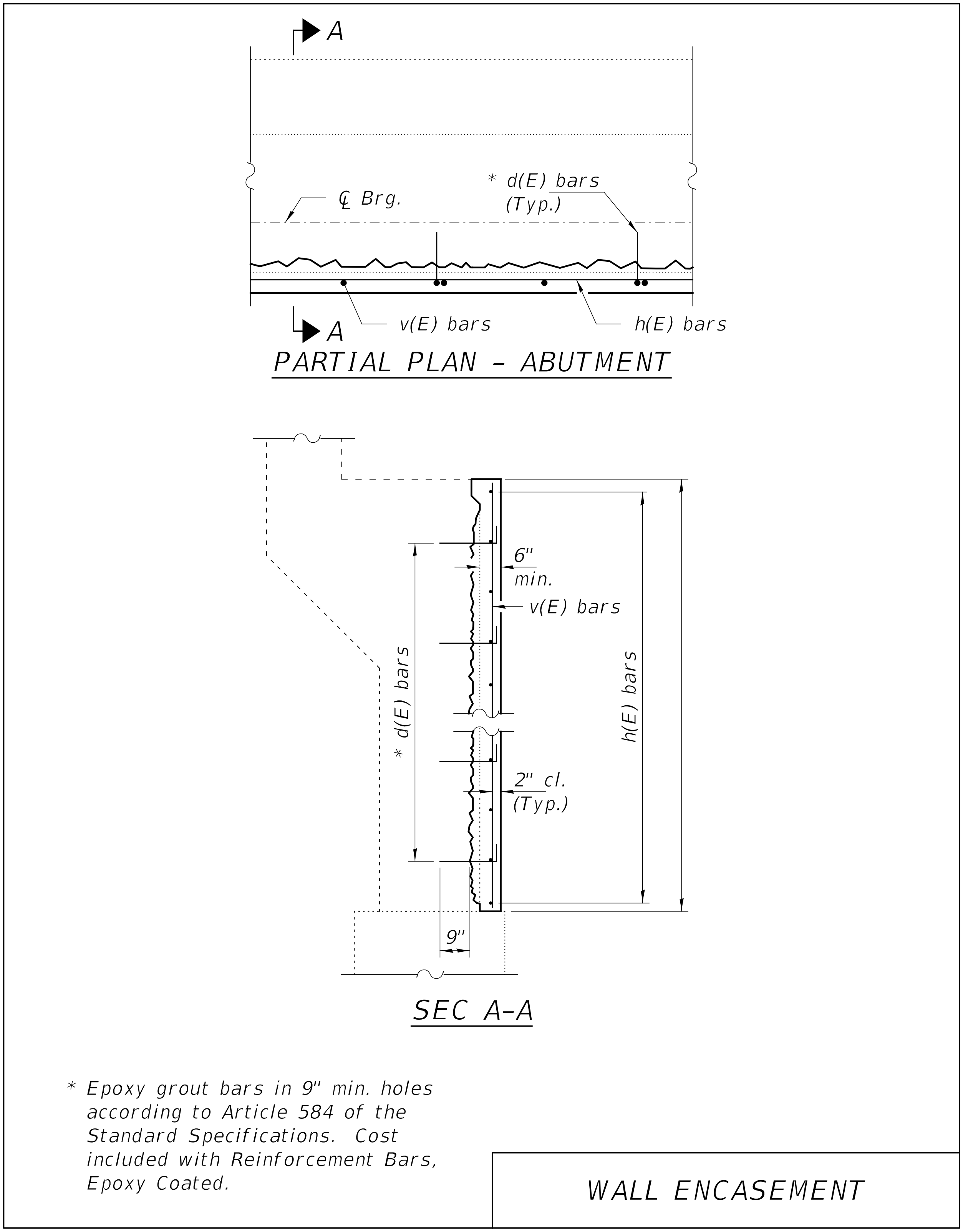


Figure 2.17.6-1: Wall Encasement

2.17.7 Pier Cap Encasement for PPC Deck Beams

Another case of lack of concrete cover is seen on the face of pier caps under PPC deck beams. To re-establish the PPC deck beam support and provide proper anchorage for the dowel rods a pier face extension can be installed as shown in Figure 2.17.7-1. Note that concrete will need to be placed from the top through holes drilled between the PPC deck beams. The contractor must ensure that proper consolidation of the concrete is achieved to avoid honeycombing and cracking. Additionally, post-tension rods and steel channels will be required for containment of the new concrete.

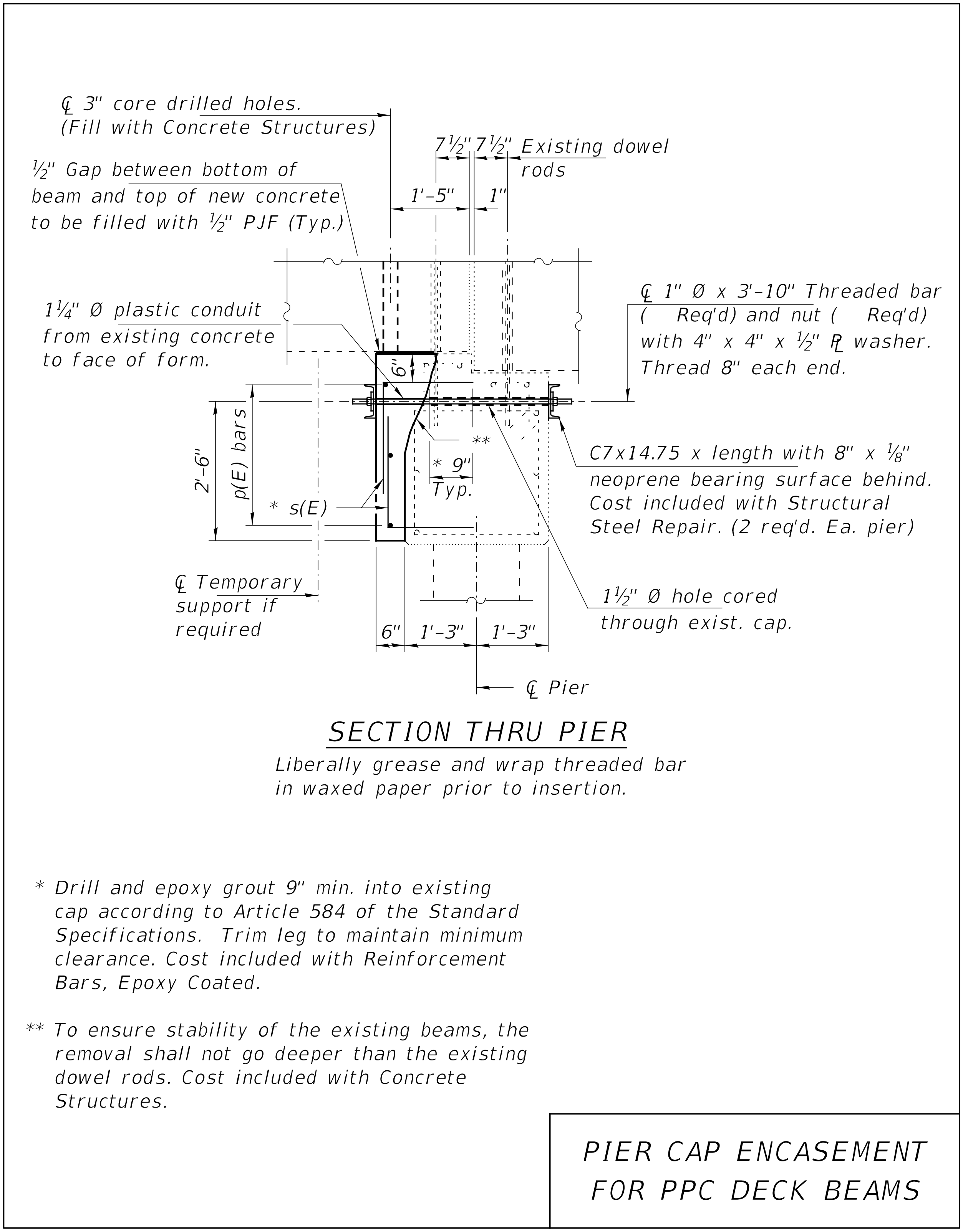
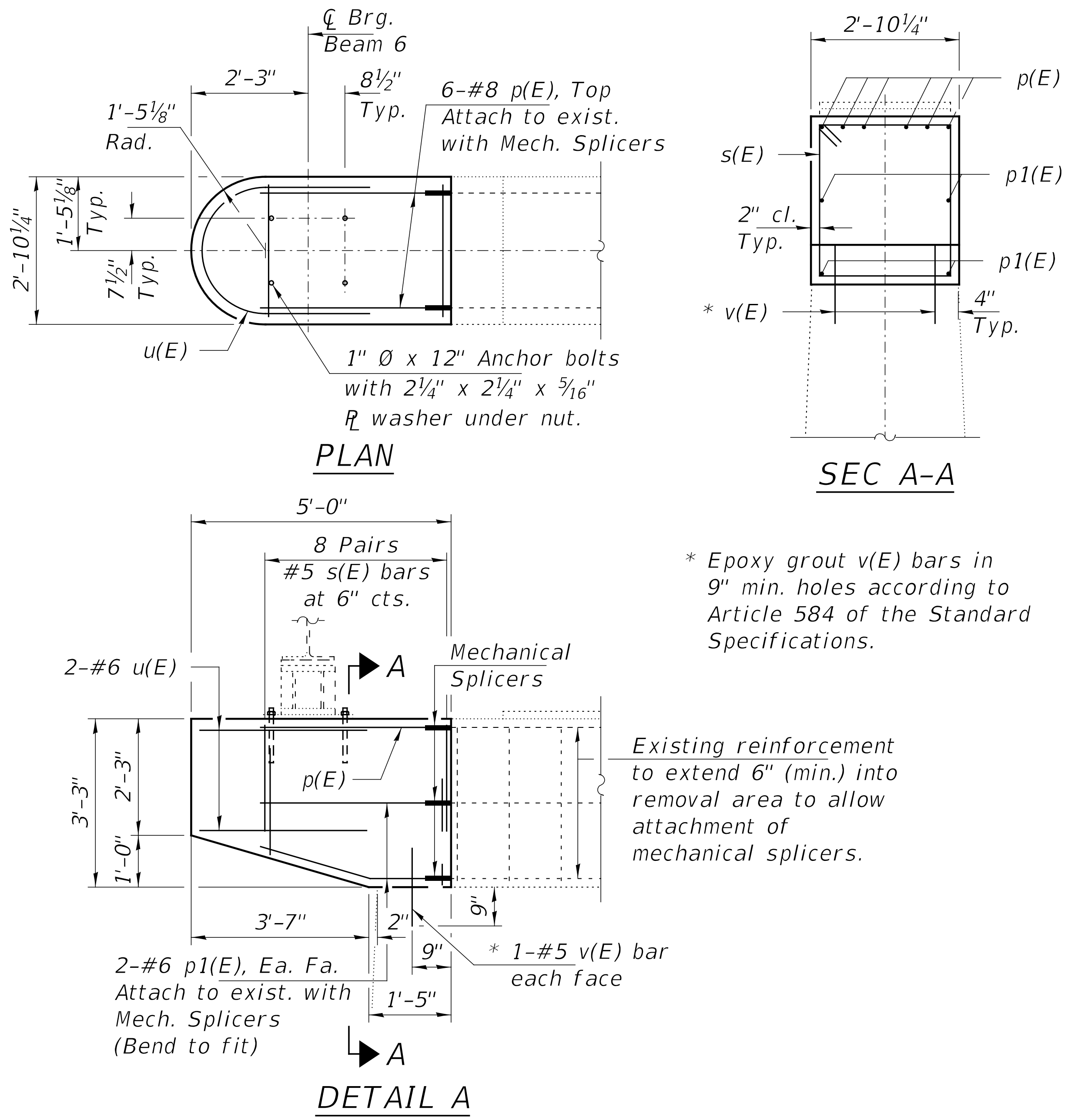


Figure 2.17.7-1: Pier Cap Encasement for PPC Deck Beams

2.17.8 Partial Substructure Removal and Replacement

Sometimes the damage is significant enough that the full removal and replacement of a section of the substructure element is necessary. These cases normally involve the removal and replacement of a pier cap overhang, pier column or full pier cap. In all these cases the standard practice is to replace the element in-kind, matching the original dimensions and reinforcement. It is critical to ensure that proper connection of the new reinforcement to the existing reinforcement is achieved, and this is normally done using mechanical splicers. The contractor must ensure proper consolidation of concrete when it is placed under an existing element. All superstructure elements in the load path of the section being removed should be temporarily supported as described in Section 2.10.5. See Figures 2.17.8-1 and 2.17.8-2 for examples of these types of substructure repairs.



Note:
Anchor bolts shall be ASTM F1554 all-thread (or an Engineer-approved alternate material) of the grade(s) and diameter(s) specified. ASTM A307 Grade C anchor bolts may be used in lieu of ASTM F1554 Grade 36 (Fy=36ksi). The corresponding specified grade of AASHTO M314 anchor bolts may be used in lieu of ASTM F1554.

EXAMPLE PARTIAL SUBSTRUCTURE REMOVAL AND REPLACEMENT (CAP)

Figure 2.17.8-1: Example – Partial Substructure Removal and Replacement (Cap)

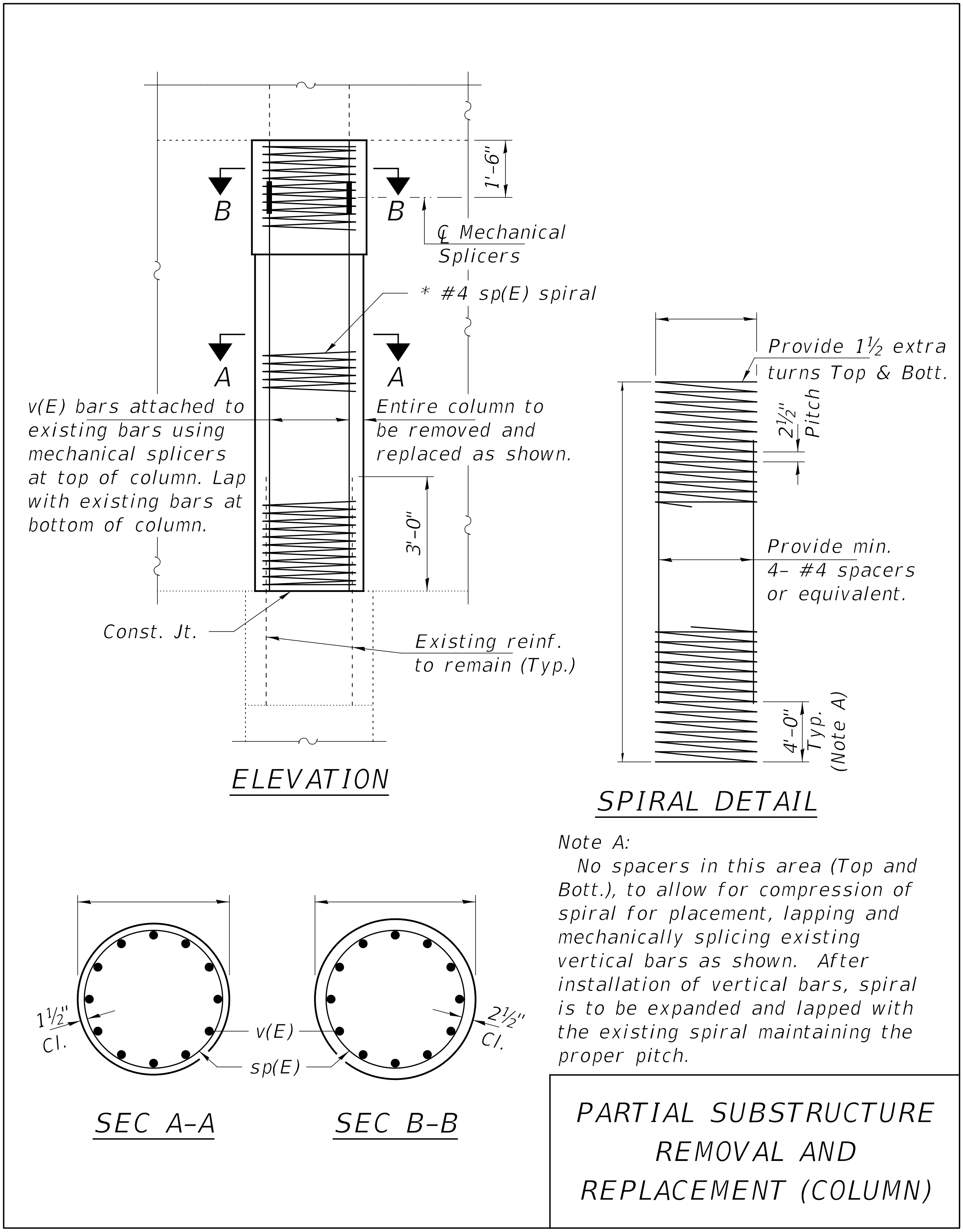


Figure 2.17.8-2: Partial Substructure Removal and Replacement (Column)

2.17.9 Abutment Wall Movement

The movement of a full substructure element (abutment or pier) presents a significant challenge, and each case is unique. A clear indication of movement is the closing of joints at the abutments. When the movement is noticed it should be reported the Bureau of Bridges and Structures so an investigation can be done to determine the cause of the movement. This type of investigation requires a significant amount of time and will likely include field visits and the use of specialized measuring and monitoring equipment. After the investigation is completed, a recommendation will be made about the best way to handle this issue. In some cases, it has been possible to arrest the movement by installing restraining systems like a “dead man” while in other more severe cases the full removal and replacement of the substructure unit has been required.

2.18 Culvert Repairs

2.18.1 General

Deterioration of culvert barrels is a common maintenance issue for the districts. Culvert replacement is expensive and the disturbance that it causes to the users is significant. Rather than replacing the deteriorated culvert, culvert liners have been used as described in Section 2.18.2 below.

2.18.2 Culvert Repair Using Liner

An effective practice for repair is the installation of a liner inside the culvert barrel and the filling of the annular area with low strength flowable fill. Before a decision is made about using this approach it is necessary to determine if the reduction in hydraulic opening will have a negative impact upstream. The liner should be designed to carry the dead and live loads as if the original culvert was non-existing. Proper bulkheads will be required at both ends. The most common liner used is Corrugated Metal Pipe (CMP). See Figure 2.18.2-1 for an example.

Other types of liners can be found in the special provisions for Folded/Formed PVC Pipe Liner, Cured-in-Place Pipe Liner and Spray Applied Pipe Liner.

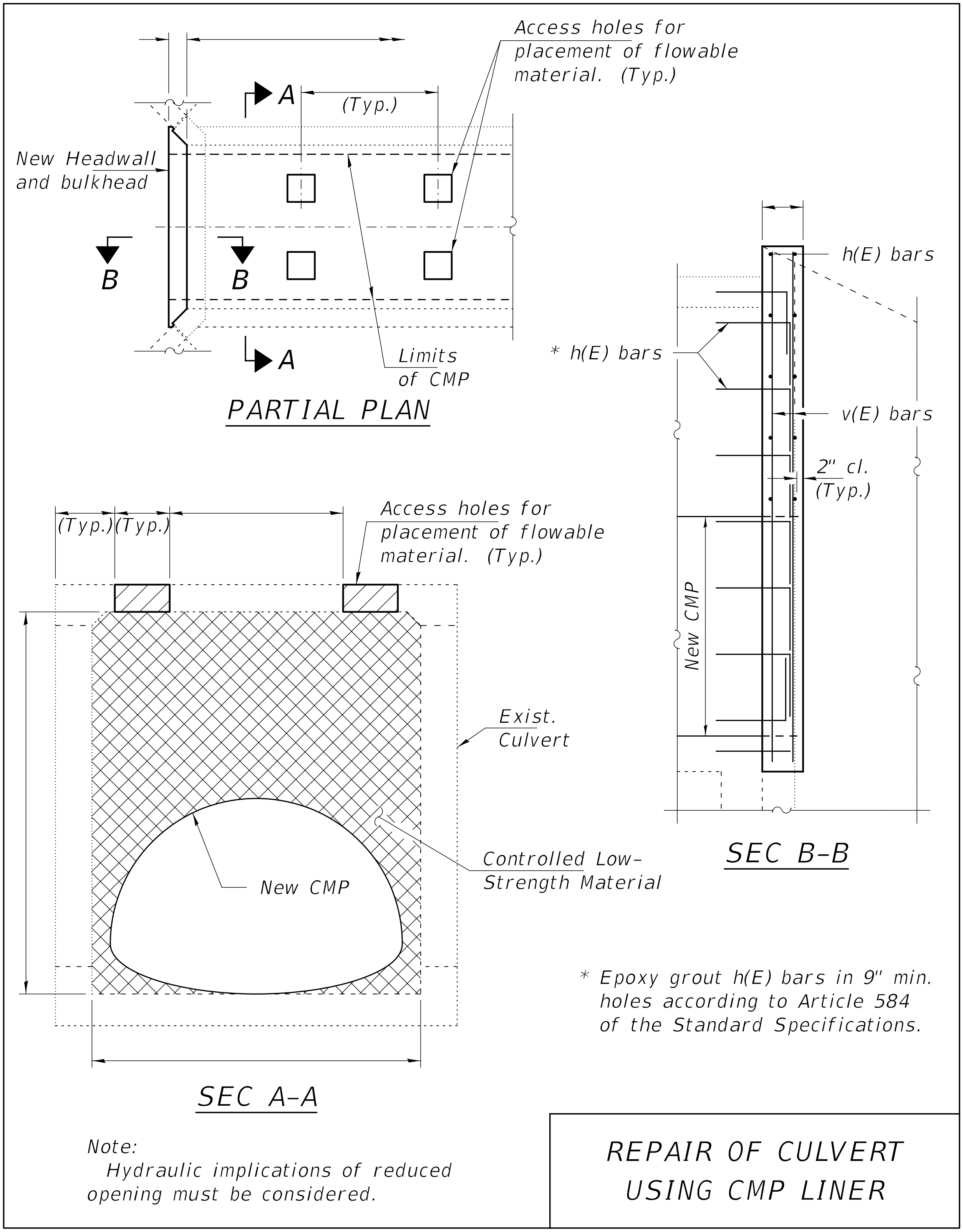


Figure 2.18.2-1: Repair of Culvert Using CMP Liner

2.18.3 Top Slab Deterioration and Temporary Repair

Deterioration of the underside of the top slab in a culvert is a common occurrence. Typically, the deterioration progresses from spalling of the concrete cover, exposure of the reinforcement steel and section loss of that reinforcement. Once the section loss has occurred the load carrying capacity of the slab is compromised, which will eventually result in the load posting of the structure. In some cases, deterioration of the concrete itself has occurred, resulting in loss of compressive strength, which can also significantly reduce the load carrying capacity of the structure.

In order to provide a remedy for this deterioration, allow time for planning and replacement and avoid having to load post the structure, a temporary repair procedure has been developed and used successfully.

The procedure involves the removal of a section of the roadway surface and the placement of a new reinforced concrete slab with the same profile grade. The slab is designed as a simply supported structure able to carry the dead load and the expected live load and impact. Because the deteriorated existing slab is used to support the new concrete until it has cured, it is critical that the removal limits be kept to a minimum and extend only as necessary to place the new slab. The designer should also verify from existing field information that the sidewalls/centerwalls are in acceptable condition. Figure 2.18.3-1 provides details for this repair.

Because this is considered a temporary repair, the following requirements must be adhered to:

1. Legal Load Only (LLO) maximum, regardless of the calculated strength of the new slab (no permit loads).
2. Culvert Condition rating shall continue to be based on the existing deteriorated condition, not the temporary slab condition.
3. Bridge Status, Illinois Structure Information System Item 41, shall be changed to “5” Open, Temporary Measures in Place.
4. 6-month Special Inspections shall be performed to monitor the temporary slab, existing culvert walls and fill behind the walls / under the new slab.
5. Place a high priority on programming the culvert for replacement.
6. Utilizing stage construction for the eventual replacement is strongly discouraged.

Note that at the discretion of the District a repair similar to the one described above can be done on culverts with slabs that do not show deterioration but have inadequate load capacity, if the District wishes to keep the structure on a schedule for replacement.

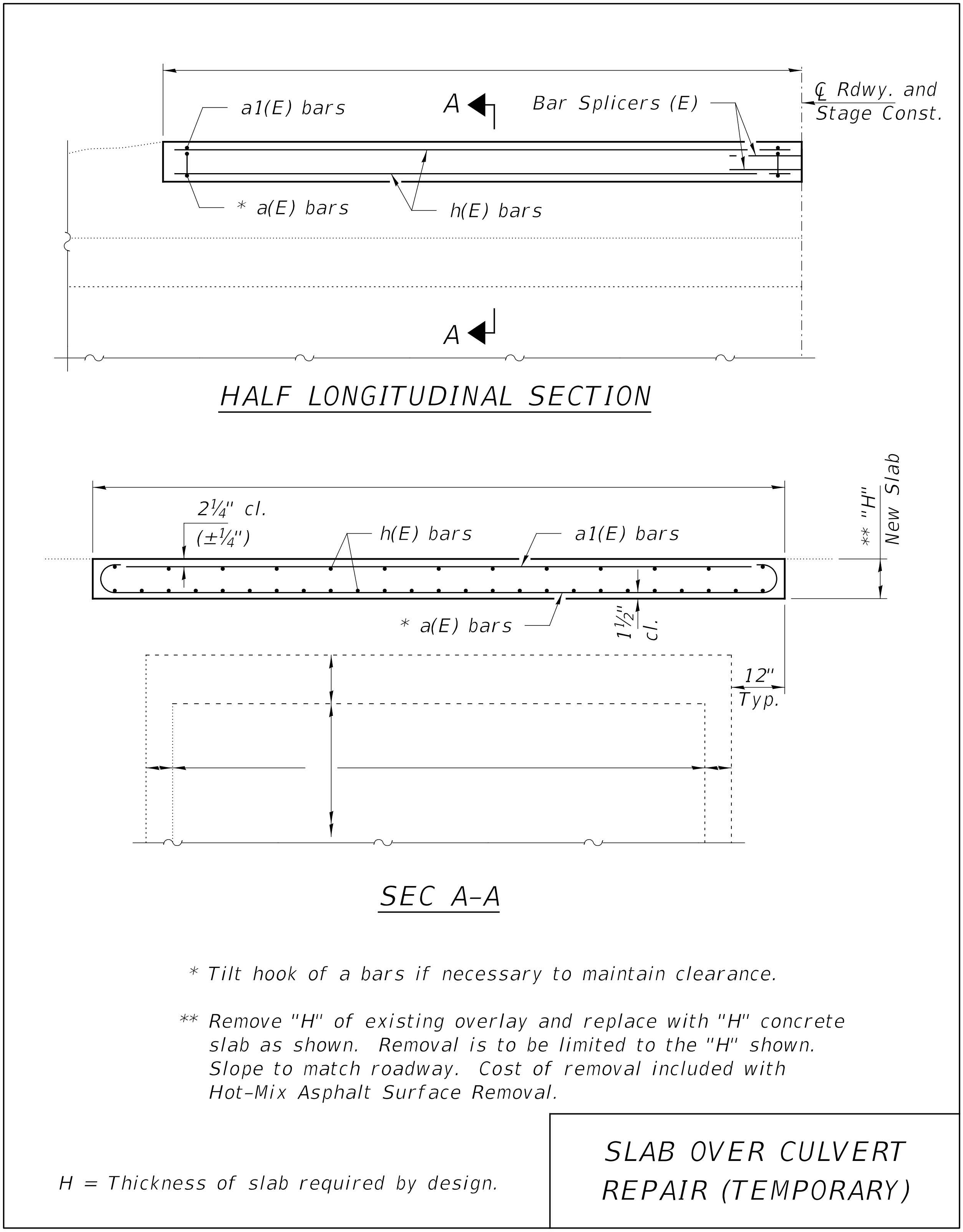
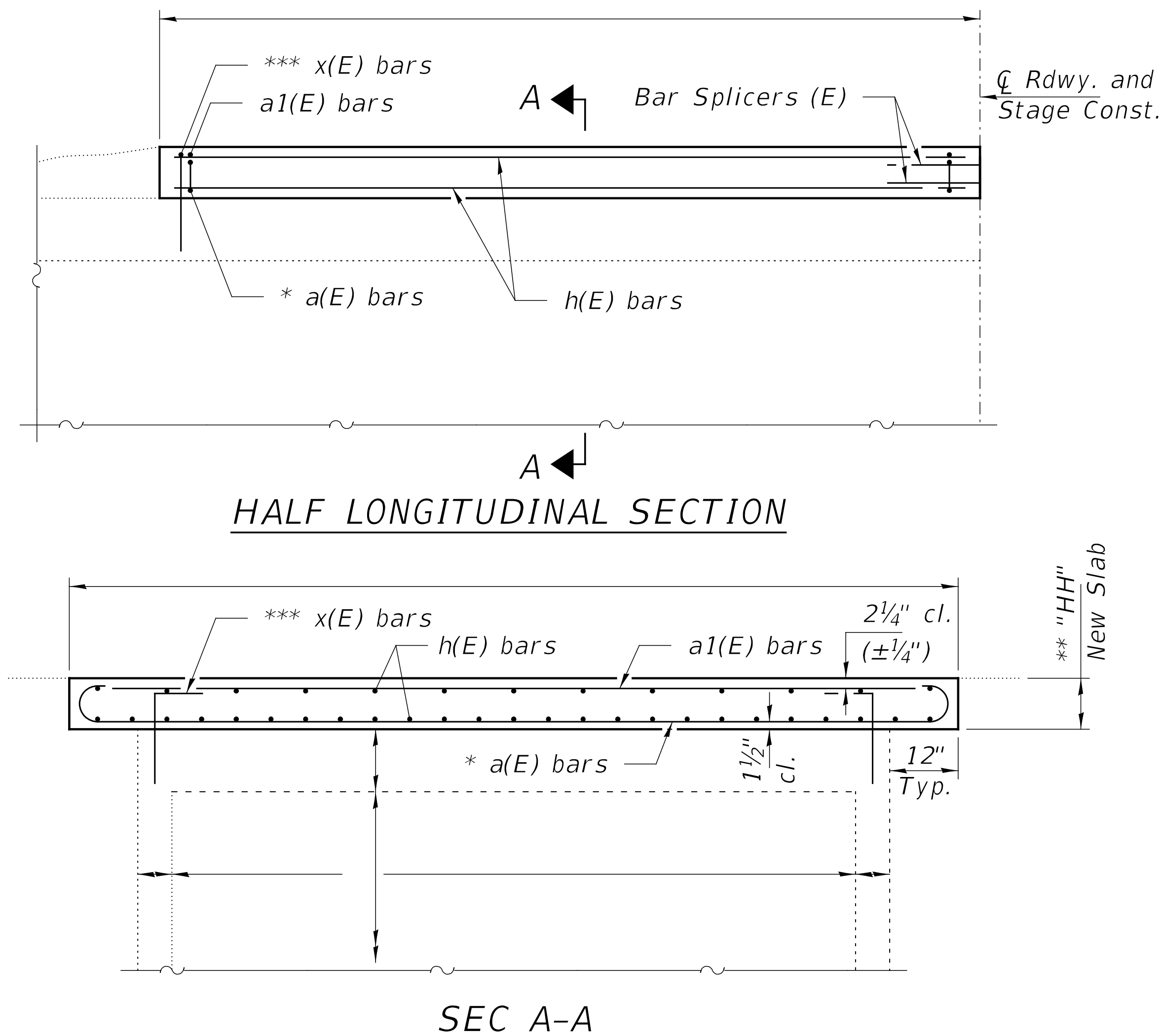


Figure 2.18.3-1: Slab Over Culvert Repair (Temporary)

2.18.4 New Top Slab for Permanent Repair

Sometimes while there is no deterioration, the capacity of the existing top slab is inadequate to carry the current expected loads and a load posting is required. To avoid this, a new slab similar to the one described in Section 2.18.3 can be used. In this case however, the new slab is attached to the existing sidewalls/center walls using anchors drilled through the existing top slab. Again, the new slab is designed to carry the dead load and (live loads + impact) and no contribution from the existing slab is assumed to carry the expected loads.

This type of repair, shown in Figure 2.18.4-1, is considered a permanent repair not subject to the requirements indicated in Section 2.18.3.



* Tilt hook of a bars if necessary to maintain clearance.

** Adjust thickness to match top of existing roadway surface. Place top of slab to match existing crown of roadway.

*** Epoxy grout x(E) bars in 9" min. holes according to Article 584 of the Standard Specifications.

HH = Available thickness above existing slab. Designer must verify that min. thickness of new slab is adequate to carry all the expected loads.

SLAB OVER CULVERT
REPAIR (PERMANENT)

Figure 2.18.4-1: Slab Over Culvert Repair (Permanent)

2.19 Retaining Wall and Wingwall Repairs

2.19.1 Retaining Wall and Wingwall Replacement

The cause of a retaining wall failure should be determined prior to the replacement of the wall. The failure may have been caused by a vehicular impact, a design or construction error, the placement of additional backfill behind the wall, the lack of sufficient drainage of ground water from behind the wall, the undermining of the footing, the slope failure of embankment material behind the wall or the general long-term deterioration of the wall. The most common of the wall failures is the loss of support due to the deterioration of the vertical reinforcement bars connecting the wall to the footing. The Bureau of Bridges & Structures should be contacted to investigate the cause of the failure.

When the entire wall, including the footing, has failed, complete replacement is necessary. However, if the footing has not failed and analysis indicates that it can carry the anticipated loads, the failed wall stem can be removed and a new wall can be attached to the existing footing using epoxy grouted reinforcement bars as shown in Figure 2.19.1-1. Accurate placement, bar lap and embedment of the new reinforcement as well as proper curing of the epoxy are essential to ensure that load from the new wall is transferred to the existing footing as intended.

The water seal in a joint between a replaced section of retaining wall and an adjacent structure should be maintained. When it is not possible to salvage the existing water seal, the details shown in Figure 2.19.1-2 can be used.

The above repair practice can also be used when it is necessary to replace a failed tall abutment backwall.

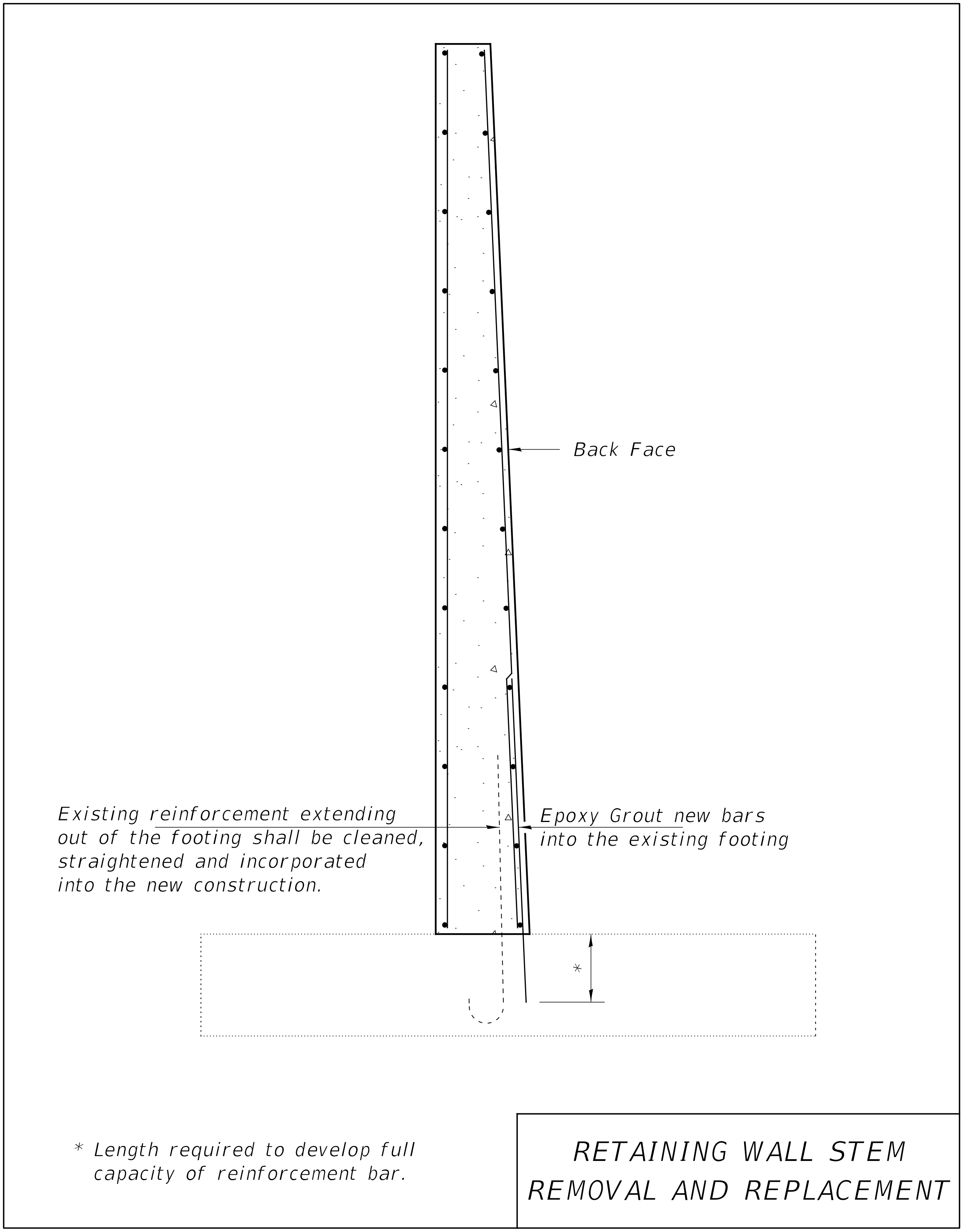


Figure 2.19.1-1: Retaining Wall Stem Removal and Replacement

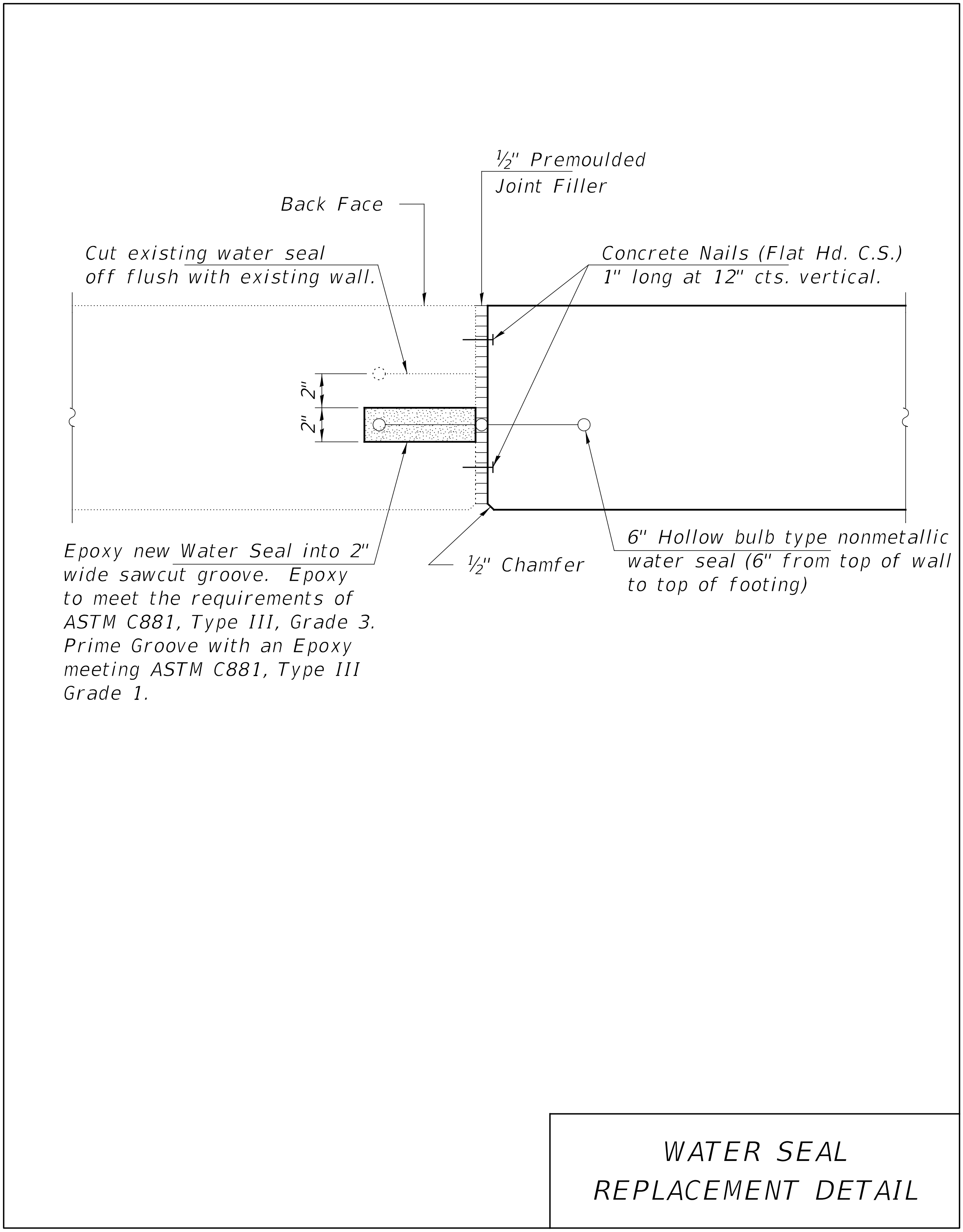


Figure 2.19.1-2: Water Seal Replacement Detail

2.19.2 Retaining Wall and Wingwall Restraint Using Helical Coils

When small movement of a retaining wall, an abutment wingwall or culvert wingwall occurs, indicating that the wall is starting to lose its support, it can be arrested using helical coils drilled into the soil and attached to the top of the wall as shown in Figure 2.19.2-1. The Designer must provide, in the contract plans, the expected loads that the helical coils are to be designed for. Because the existing wall was not originally designed to carry the load in this manner, this type of repair should be closely monitored for further movement.

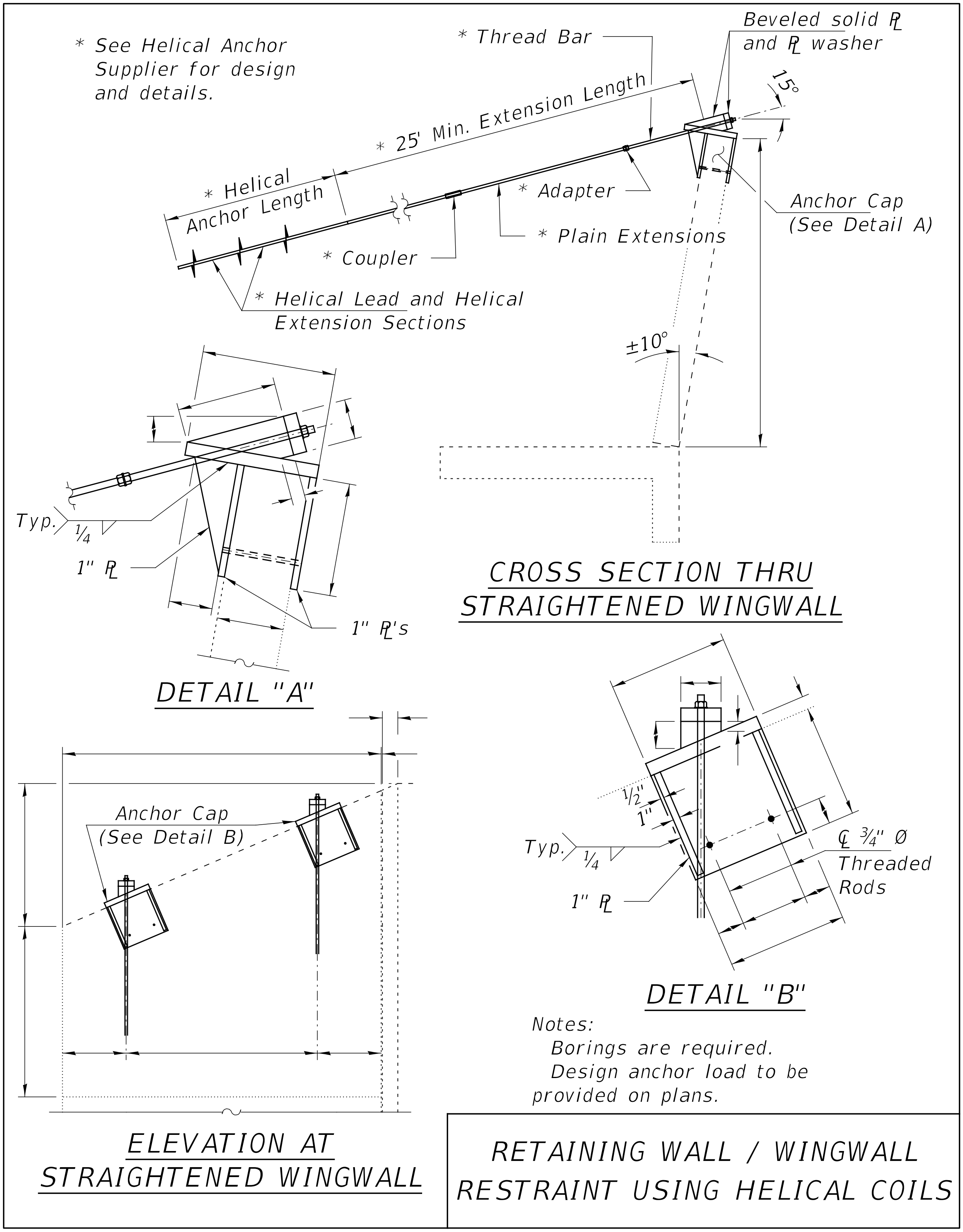


Figure 2.19.2-1: Retaining Wall / Wingwall Restraint Using Helical Coils

Section 3 Bridge Inspection

3.1 Purpose

This Section of the IDOT *Structural Services Manual* provides documentation of the official bridge inspection policies for the State of Illinois.

Inspections of structures located on public roads are necessary to provide adequate safety for the traveling public, ensure compliance with the National Bridge Inspection Standards (NBIS), and protect the large investment in Illinois bridges.

The primary purpose of this section is to provide information pertaining to bridge inventory and inspection activities. The information provided in this section summarizes IDOT inspection policies and guidelines for the effective and efficient management of the bridge inspection program. The information provided in regard to inspection types and frequencies is also applicable to structures under the jurisdiction of agencies other than IDOT, where the oversight for inspections is the responsibility of the agency having jurisdiction.

The primary function of the bridge inspections performed in accordance with the NBIS is to ensure that bridges serving public roadways in Illinois remain safe. The results of the inspections are also used as a tool to assist in determining bridge preservation, maintenance, and improvement needs.

3.2 National Bridge Inspection Standards (NBIS)

Congress implements highway policy through passage of federal statutes that govern, in part, the inspection of the Nation's bridges. The statutes are codified in the United States Code (USC) as the National Bridge Inspection Standards (NBIS) in the Code of Federal Regulations (CFR) under Title 23, Chapter I, Subchapter G, Part 650, Subpart C. To maintain full compliance with the NBIS, IDOT must adhere to all policies, procedures, and regulations established by the Federal Highway Administration (FHWA).

Bridge inspections are generally performed and recorded under the requirements of the NBIS. The NBIS are federal regulations establishing minimum requirements for inspection organizations, qualifications of personnel, frequency of inspections, inspection procedures, and the preparation and maintenance of a bridge inventory.

The FHWA administers the NBIS under the guidelines outlined in the FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (Coding Guide) which is in the process of being replaced by the Specifications for the National Bridge Inventory (SNBI). The information collected, as required by the NBIS, is reported annually to the FHWA.

3.2.1 Documents Included by Reference (23 CFR 650.317)

Additional documents are incorporated into the requirements of the NBIS by reference. These include the following AASHTO publications:

- *The Manual for Bridge Evaluation* (MBE), Third Edition (2018) and 2019 and 2020 Interim revisions, for procedures related to 23 CFR 650.305 - Definitions and 23 CFR 650.313 – Inspection Procedures.
- *Manual for Bridge Element Inspection* (MBEI), Second Edition (2019), for procedures related to 23 CFR 650.315 – Inventory.

Conformance with these manuals is required to be considered in compliance with the NBIS.

In addition to the Coding Guide and SNBI, several FHWA publications have been developed to assist the states in uniformly complying with the NBIS including, but not limited to, the following:

- *Bridge Inspector's Reference Manual* (BIRM), 2022 NBIS Edition (2023)
- *Underwater Bridge Inspection* (Publication No. FHWA-NHI-10-027)
- *Evaluating Scour at Bridges* (HEC-18), (Publication No. FHWA-HIF-12-003)

3.2.2 Applicability (23 CFR 650.303)

The NBIS apply to all structures defined as highway bridges located on all public roads, on and off Federal-aid highways, including tribally owned and federally owned bridges, private bridges that are connected to a public road on both ends of the bridge, temporary bridges, and bridges under construction with portions open to traffic. A public road is considered to be any road or street under the jurisdiction of, and maintained by, a public authority and open to public travel.

3.2.2.1 Non-IDOT Bridge Owners

Non-IDOT bridges in the inventory are delegated to a bridge owner for performance of management and duties associated with compliance with the NBIS. The owners are responsible for ensuring NBIS-related duties are performed by qualified personnel and documentation is provided to IDOT in a timely manner.

3.2.2.1.1 Non-IDOT State Agencies

- Illinois State Toll Highway Authority (ISTHA)
- Illinois Department of Natural Resources (IDNR)
- State Universities

3.2.2.1.2 Local Public Agencies

- Counties
- Townships
- Municipalities
- Park Districts
- Community Colleges

3.2.2.1.3 Other

- HOAs
- Private Companies
- Railroads

3.2.2.2 Transit Highways

The FHWA considers the NBIS to be applicable to roadways dedicated to publicly accessible transit vehicles, typically buses carrying the general public. As such, agencies with jurisdiction over these bridges are subject to the policies set forth in this manual.

3.2.2.3 Privately Owned Bridges

Although the NBIS requirements are not applicable to privately owned bridges that are not connected to a public road on both ends, the FHWA strongly encourages private bridge owners to follow the NBIS as the standard for inspecting their bridges. In instances where a privately owned bridge carries or crosses a public road, the FHWA has indicated that the agency with jurisdiction over the public road should encourage the private bridge owner to inspect the bridge in accordance with the NBIS or reroute the public road.

3.2.3 FHWA Oversight of Compliance with the NBIS

In response to a recommendation from the Office of the Inspector General (OIG) and Congressional direction, the FHWA developed a new systematic, data-driven, and risk-based oversight process for monitoring State compliance with the NBIS. The process established metrics that can be traced directly to the NBIS regulation. Annually, each metric is assessed at one of three intensity levels (Minimum, Intermediate, or In-Depth), based on risk. The FHWA then analyzes the results from the State reviews to identify nationwide bridge safety risks that may require closer review in future years.

3.2.3.1 Compliance Metrics

Metrics established by FHWA are used to assess each State’s compliance with the NBIS. Each metric is assessed individually and with equal importance. Each of the metrics is related to a specific NBIS regulation. Specific criteria are established for determining the level of compliance with each metric.

3.2.3.2 Compliance Levels

Each of the compliance metrics is annually assessed by FHWA and assigned one of four compliance levels based upon specific measures and thresholds for each compliance level identified in each metric. The degrees of compliance and resulting actions are described here:

- ***Compliant*** – Adhering to the NBIS regulation.
- ***Substantially Compliant*** – Adhering to the NBIS regulation with minor deficiencies, as set forth in each of the metric requirements. These deficiencies do not adversely affect the overall effectiveness of the program and are isolated in nature. Documented deficiencies are brought to the State's attention with the expectation that they are corrected within 12 months or less, unless the deficiencies are related to issues that would most efficiently be corrected during the next inspection. Per the Notice, an Improvement Plan describing corrective action(s) by the State is required (79 FR 27032).
- ***Non-Compliant*** – Not adhering to the NBIS regulation; in general, failing to meet one or more of the Substantial Compliance criteria for a metric. Identified deficiencies may adversely affect the overall effectiveness of the program. Failure to adhere to an approved Plan of Corrective Action (PCA) is also considered non-compliance. Metrics which remain non-compliant will invoke the penalty for non-compliance per 23 U.S.C. 144(h)(5).
- ***Conditionally Compliant*** – Taking corrective action in conformance with an FHWA-approved PCA to achieve compliance with the NBIS. Deficiencies, if not corrected, may adversely affect the overall effectiveness of the program. Metrics which are determined to be conditionally compliant will not invoke the penalty for non-compliance.

Actions taken to address findings of Substantially Compliant and Non-Compliant, respectively, are as follows:

- ***Improvement Plan (IP)*** – A written response by the State that documents the agreement for corrective action(s) to address deficiencies identified in a Substantial Compliance determination. The completion timeframe for such agreements is limited to 12 months or less, unless the deficiencies are related to issues that would most efficiently be corrected during the next inspection cycle.

- ***Plan of Corrective Action (PCA)*** – A written document prepared and submitted by the State and approved by FHWA describing the steps that are taken and timelines to take those actions in order to correct non-compliant NBIS metrics. An agreed-upon PCA for a non-compliant metric removes the possibility of a penalty based upon that metric.

3.3 Illinois Bridge Inspection Organization

3.3.1 Inspection Organization Responsibilities

Illinois complies with the NBIS program requirements for inspection and inventory data through the following responsible positions.

Statewide NBIS Program Manager (Statewide PM): The IDOT Bridge Management & Inspection Unit Chief within the Bureau of Bridges & Structures providing statewide oversight for all NBIS-related activities. The Statewide NBIS Program Manager is responsible for inspection policy and for ensuring the quality of the NBIS program.

IDOT District NBIS Program Manager: The IDOT District Bridge Maintenance Engineers providing oversight for all NBIS related activities for IDOT-maintained bridges within their designated area of responsibility.

IDOT District 1 Area NBIS Program Manager: The IDOT Area Bridge Inspection Engineers in District 1 providing oversight for all NBIS related activities for IDOT-maintained bridges within their designated area of responsibility.

Agency Designated NBIS Program Manager (Agency PM): For purposes of this section, an “Agency” is defined as any entity owning a bridge in Illinois. All entities with jurisdiction of a bridge in the NBI must designate an Agency NBIS Program Manager to ensure compliance with the NBIS and provide guidance and management of their bridge inventory. Entities may designate in-house staff or a consultant currently certified as an NBIS Program Manager. The consultant and agency must discuss and clarify the Program Manager duties such as responding to BridgeWatch® alerts, entering inspections into the ISIS, maintaining official bridge files, reporting and addressing critical findings, performing QC/QA activities, and responding to IDOT requests for additional NBIS information, and other NBIS-related activities.

The agency must complete IDOT Form BBS 3310 Agency NBIS Program Manager Designation to designate their Program Manager and submit it to the Bridge Management & Inspection Unit at DOT.BBS.BridgeMgmt@illinois.gov.

3.3.2 Illinois Structure Information System / Bridge Inspection System (ISIS / BIS)

The Illinois Structure Information System (ISIS) is the official database containing all Illinois bridge inventory and inspection information. The bridge data stored in ISIS for each structure includes an inventory record, inspection records, and information related to construction, reconstruction, highway routes, scour evaluations, microfilm, and design.

The web-based Bridge Inspection System (BIS) is utilized by bridge inspectors to record the results of bridge inspections for entry into the ISIS.

3.3.3 Specifications for Illinois Bridge Inventory Manual (SIBI)

The IDOT *Specifications for Illinois Bridge Inventory Manual* (SIBI Manual) is the official document describing ISIS, which includes the various data items it contains and coding of each item. This includes the criteria to be used by bridge inspectors when assigning condition ratings and appraisals. Use and knowledge of this manual is essential for all inspectors and others involved in the inventory and inspection of Illinois bridges. The manual is maintained by the IDOT Office of Planning and Programming, Data Management Unit.

3.3.4 Bridge Management System (BMS)

The Bridge Management System (BMS) contains element level inspection and condition state information. Element Level Inspections provide detailed quantitative condition data for each element of a bridge. This system provides IDOT with an enhanced capability of predicting future bridge programming needs. Guidance for Element Level Inspections is included in the AASHTO *Manual for Bridge Element Inspection (MBEI)* and the IDOT supplement to the MBEI.

3.4 Qualifications of Personnel

The quality of a bridge inspection program is very much dependent on the performance of the NBIS Program Manager in charge of each agency's inspection program and on the NBIS Team Leaders of the inspection teams performing the field inspections. Per the NBIS and IDOT policy, prospective applicants must meet minimum requirements in order to be considered qualified to dispense the duties of Program Manager and Team Leader. IDOT has established a certification process to review, verify and approve the education, training and experience of an individual to function as a Program Manager or Team Leader in Illinois. A list of qualified personnel is maintained on IDOT's website.

3.4.1 Certified NBIS Program Manager

The NBIS provides minimum qualification requirements to be certified as a Program Manager. IDOT Form BBS 2610 Bridge Program Manager Application must be used for the purpose of documenting and submitting Program Manager applications to the Statewide PM for review and approval.

To satisfy NBIS and IDOT requirements to function as a Program Manager, an individual must have the following qualifications and training:

- Licensed as a Professional Engineer or Structural Engineer with 6 months minimum bridge inspection experience (limited to active participation in NBIS bridge inspection) or has 10 years of bridge inspection experience.
- Successful completion of an FHWA-approved comprehensive bridge inspection training course with a score of 70 percent or greater on an end-of course assessment. NHI 130055 "Safety Inspection of In-Service Bridges" or NHI 130056 "Safety Inspection of In-Service Bridges for Professional Engineers" satisfy the comprehensive bridge inspection training requirement.
- Successful completion of a total of 18 hours of FHWA-approved bridge inspection refresher training with a score of 70 percent or greater on an end-of course assessment every 60 months. NHI 130053 "Bridge Inspection Refresher Training" is available to satisfy NBIS bridge inspection refresher training requirements.
- If the Program Manager is managing a bridge inventory with NSTMs or overseeing Team Leaders inspecting bridges with NSTMs, successful completion of FHWA-approved NSTM

inspection training is required. NHI 130078 Bridge Inspection Techniques for Nonredundant Steel Tension Members (NSTM) is available to satisfy these requirements.

- If the Program Manager is overseeing Element Level Inspections, successful completion of either the IDOT Element Level Bridge Inspection course or the FHWA 2-day Introduction to Element Level Inspections is required.

Individuals who have been certified by IDOT to function as NBIS Program Managers are also qualified to function as NBIS Team Leaders.

Personnel Type	Bridge Inspection Experience (months)	NBIS Bridge Inspection Experience (months)
Engineering Personnel with PE	6	6 (min.)
Engineering Personnel without PE or Technical Personnel	120	60 (min.)

Table 3.4.1-1: Program Manager Bridge Inspection Experience Requirements

3.4.2 Certified NBIS Team Leader

The NBIS provides minimum qualification requirements for licensed engineers and for technical personnel to be certified as Team Leaders. IDOT Form BBS 2620 Team Leader Qualifications must be used for the purpose of documenting and submitting Team Leader applications to the Statewide PM for review and approval.

To satisfy NBIS and IDOT requirements to function as a Team Leader, an individual must have at least one of the following qualifications:

- Licensed as a Professional Engineer or Structural Engineer with 6 months of bridge inspection experience (limited to active participation in bridge inspection).
- 5 years of bridge inspection experience.
- Bachelor’s degree in engineering from an ABET-accredited college or university, a passing score on the Fundamentals of Engineering Exam, and 2 years of bridge inspection experience.
- Associate’s degree in engineering or engineering technology from an ABET-accredited college or university and 4 years of bridge inspection experience.

- Individuals who have been certified by IDOT to function as NBIS Program Managers are also qualified to function as NBIS Team Leaders.

In addition to the above qualifications, each Team Leader must have the following training:

- Successful completion of an FHWA-approved comprehensive bridge inspection training course with a score of 70 percent or greater on an end-of course assessment. NHI 130055 “Safety Inspection of In-Service Bridges” or NHI 130056 “Safety Inspection of In-Service Bridges for Professional Engineers” satisfy the comprehensive bridge inspection training requirement.
- Successful completion of a total of 18 hours of FHWA-approved bridge inspection refresher training with a score of 70 percent or greater on an end-of course assessment every 60-months. NHI 130053 “Bridge Inspection Refresher Training” is available to satisfy NBIS bridge inspection refresher training requirements.
- If the Team Leader is inspecting bridges with NSTMs, successful completion of FHWA-approved NSTM inspection training is required. NHI 130078 “Bridge Inspection Techniques for Nonredundant Steel Tension Members (NSTM)” is available to satisfy these requirements.
- The NBIS requirement for comprehensive bridge inspection training also extends to divers performing Underwater Inspections. NHI 130091 “Underwater Bridge Inspection” provides an overview of diving operations that will be useful to IDOT and Agency personnel responsible for managing Underwater Inspections. This course also fulfills the NBIS bridge inspection training requirement for all divers conducting Underwater Inspections.
- If the Team Leader is overseeing Element Level Inspections, successful completion of either the IDOT Element Level Bridge Inspection course or the FHWA 2-day Introduction to Element Level Inspections is required.

3.4.3 Bridge Inspection Experience

Per the NBIS, bridge inspection experience is defined as:

> Bridge Inspection Experience: Active participation in bridge inspections in accordance with the NBIS, in either a field inspection, supervisory, or management role. Some of the experience may come from relevant bridge design, load rating, bridge construction, and bridge maintenance experience provided it develops the skills necessary to properly perform a NBIS inspection.

NBIS Bridge Inspection Experience: Active participation in bridge inspections in accordance with the NBIS.

The Bridge Inspection Experience must have been accumulated within the last 10 years.

To provide agencies with guidance and clarification of experience levels adequate to satisfy NBIS requirements, the FHWA has provided the following statement in regard to the "desired minimum bridge inspection experience level" for personnel engaged in bridge inspections:

Minimum Bridge Inspection Experience Level: The predominate amount, or more than fifty percent, shall come from NBIS bridge safety inspection experience. Other experience in bridge design, bridge maintenance, or bridge construction may be used to provide the additional required experience.

Based on the requirements of NBIS, Engineering Personnel or Technical Personnel can function as Team Leaders after the Statewide PM has evaluated their training and experience and determined that they are qualified. The values, provided in the following table for Bridge Related Experience and Performance of NBIS Inspections, represent the minimum level of experience required to satisfy criteria requirements:

Personnel Type	Bridge Inspection Experience (months)	NBIS Bridge Inspection Experience (months)
Engineering Personnel with PE	6	6 (min.)
Engineering Personnel with FE/EIT	24	12 (min.)
Engineering Personnel without FE/EIT	48	24 (min.)
Technical Personnel – Associate's Degree	48	24 (min.)
Technical Personnel	60	30 (min.)

Table 3.4.3-1: Team Leader Bridge Inspection Experience Requirements

When the experience of personnel does not meet the "Minimum Bridge Inspection Experience Level", the Statewide PM must be contacted and will coordinate the review of the experience with the responsible NBIS Program Manager and the applicant. An individual having less than the shown number of months of bridge-related experience accumulated over the course of their career through the performance of bridge inspections, bridge design, bridge maintenance, or bridge construction activities, with a portion of their accumulated bridge-related experience obtained through the performance of NBIS Inspections, meets the experience requirements to qualify as a Team Leader only if both the Statewide PM and FHWA concur that the individual's experience is acceptable. This criterion should only apply to special situations involving highly qualified individuals performing NBIS bridge inspections that require specialized knowledge or training on unusual or complex bridges. If approved, additional stipulations may be placed on the applicant.

3.4.4 Revocation and Re-instatement of IL NBIS PM and TL Certification

Per the NBIS, IDOT is responsible for all bridges > 20.0 feet on all public roads in Illinois regardless of the entity having maintenance responsibility of the bridge. The Statewide NBIS PM for Illinois is responsible for bridge inspection policies/procedures and to ensure the quality of the NBIP in Illinois. The NBIS permits the Statewide NBIS PM for Illinois to delegate any of the NBIS responsibilities to a Certified IL NBIS PM. The delegation of NBIS responsibilities does not relieve IDOT or the Statewide NBIS PM for Illinois of their previously stated responsibilities. The Statewide NBIS PM for Illinois has the authority to revoke any IL NBIS PM or TL Certification.

The qualifications for Certification as an IL NBIS PM and TL are stated in the *IDOT Structural Services Manual* Sections 3.4.1 and 3.4.2 respectively.

Actions which may result in the revocation of an individual's Certification include but are not limited to:

- Falsifying inspection records. Includes but not limited to hard copy of inspection forms and entries into the Illinois Structure Information System (ISIS)
- Interfering with BM&I Unit personnel and/or agents of the Department while coordinating and inspecting a bridge or bridges having excessive delinquencies.
- Habitual delinquencies > 30 days without prior notification to the BM&I Unit or for non-legitimate reasons.
- Uncooperative with BM&I Unit personnel; District Bureau of Local Roads & Streets (BLRS) personnel; other IDOT personnel; or agents of the Department.
- Non-responsive to BM&I Unit personnel; District BLRS personnel; other IDOT personnel; or agents of the Department. IDOT does not expect immediate response to each inquiry, but many times the BM&I Unit is attempting to resolve safety related deficiencies noted by the Department or the FHWA as part of their Annual NBIP Metrics Assessment. In some cases, the BM&I Unit is forced to repeatedly contact individuals to simply initiate resolution. These

deficiencies are higher priority and should be responded to in a timely fashion. Examples of safety related deficiencies include but are not limited to:

- Excessive delinquencies without proper notification or documentation submitted to the BM&I Unit;
- Corrections to signage noted during the Annual IDOT Load Posting & Closure Review;
- Installation of Allowable Weight Limit signs for new and revised restrictions less than legal loads only; and
- Implementation of temporary or permanent bridge closure when mandated by the Department.

- Poor-quality bridge inspections.
- Failure to perform IL NBIS PM or TL responsibilities and duties as specified in the NBIS and the IDOT Structural Services Manual Section 3.
- Revocation of PM and/or TL status in another state if it is shown the revocation was warranted.

In most cases, prior to a Certification being revoked, the following procedure will be used.

1. BM&I Unit will contact the individual via email:
    - Detailing exactly what inappropriate action(s) by the individual has (have) been identified and needs to be addressed.
    - Request an accounting from the individual detailing why the inappropriate action(s) was (were) taken instead of adhering to the NBIS and IDOT Bridge Inspection Policy.
    - Request what remediation steps will be taken by the individual to ensure compliance with the NBIS and IDOT Bridge Inspection Policy going forward.
    - Request when the remediation steps will be implemented.
    - Remind the individual if clarification/interpretation of the NBIS and/or IDOT Bridge Inspection Policy requirements are necessary, contact the BM&I Unit for assistance.

2. No Action Taken Against IL NBIS PM or TL Certification:

- If the individual provides a concise, complete response to the BM&I Unit inquiry. However, the individual will be closely monitored to ensure the remediation steps are implemented.
    - If the individual provides an incomplete and/or insufficient response to the BM&I Unit inquiry, the BMI&I Unit will respond back stating as such and requesting additional information.

3. Revoke IL NBIS PM or TL Certification:
    - If the individual does not respond within ten (10) business days to the BM&I Unit inquiry.

- If the individual has been notified and provides subsequent responses that continue to be incomplete and/or insufficient such that a resolution is not being achieved.
- If the individual has failed to implement the remediation steps previously noted and continues with the inappropriate action(s).

4. Notification of Revoked IL NBIS PM or TL Certification:
   - If the individual's IL NBIS PM or TL Certification has been revoked, the BM&I Unit will notify the following simultaneously, via email and electronic IDOT Letter, detailing the above procedure.
     - Individual having IL NBIS PM or TL Certification Revoked
     - Applicable Governing Body and/or Consultant Firm Executive Staff
     - Owner of bridge and/or agency with maintenance responsibility of the bridge

5. Re-instatement Review of Revoked IL NBIS PM or TL Certification:
   - Re-instatement will not be considered for the most egregious inappropriate actions.
   - Review only done at the request of the individual with the revoked IL NBIS PM or TL Certification.
   - Must include the individual's acknowledgement of inappropriate action(s) identified by the BM&I Unit.
   - Must include remediation steps that will (have) be (been) taken by the individual to ensure compliance with the NBIS and IDOT Bridge Inspection Policy going forward.
   - Consideration for review is at the discretion of the Statewide NBIS PM for Illinois and the FHWA Illinois Division Bridge Engineer and is based on the individual's inappropriate action(s) and/or other circumstances.

3.5 Inspection Interval

3.5.1 General

The following sections document NBIS and IDOT requirements for the required frequency of different bridge inspection types. Routine, NSTM, Underwater, Element Level, and Special Inspections (with intervals ≥ 12months) shall be completed by the end of the month in which the inspections are due. Special Inspections with intervals < 12 months must be completed by the due date.

Agency PMs must ensure the NBIS inspections do not become delinquent. For rare and unusual circumstances, such as extreme flooding, coordination with an uncooperative third party, or equipment availability, the Statewide PM, through coordination with the FHWA Division Office, may preapprove inspection delays when it is expected to cause a delinquency greater than 30 days. Notification of all bridge inspection delinquencies and delay requests must be sent by the Agency PM to the Bridge Management Group at DOT.BBS.BridgeMgmt@illinois.gov.

The email must include the inspection due date(s), reason for the delay, and date scheduled for follow-up attempt to inspect. If unable to perform the inspection by the scheduled follow-up date, notify the Bridge Management Group for further evaluation. A copy of all correspondence related to inspection delays must be provided to the Bureau of Bridges & Structures and shall be kept in the Bridge File.

3.5.2 Routine Inspection Interval

The Routine Inspection Interval will vary over the life of the bridge. The Initial Inspection of any new, rehabilitated, or structurally modified bridge must be completed within 3 months of opening to traffic. The first Routine Inspection must be within 24 months, but no sooner than 12 months, after the Initial Inspection. After the first Routine Inspection, the interval will remain at 24 months unless the bridge qualifies for a 48-month inspection interval as described in Section 3.5.2.1.

The NBIS allows bridges in good condition to be inspected at intervals greater than 24 months. The FHWA has concurred with IDOT's criteria for determining a maximum allowable interval for Routine Inspections. Based on the established criteria, the Routine Inspection interval is computer generated in ISIS for Routine Inspection Interval – Item B.IE.05.

The 48-month inspection interval represents the maximum allowable in Illinois for bridges/culverts with an NBIS Bridge Length greater than 20.0 feet and in no way precludes inspections at lesser intervals. The Agency PM may allow bridges to be inspected more frequently than policy requires.

3.5.2.1 48-Month Inspection Interval Criteria

The following conditions must be met in order to have an Inspection Interval of 48 months:

- Deck Condition – Item B.C.01 coded ≥ "6".
- Superstructure Condition – Item B.C.02 ≥ “7”.
- Substructure Condition – Item B.C.03 coded ≥ "7".
- Culvert Condition – Item B.C.04 coded ≥ "7".
- Bearing Condition – Item B.C.07 ≥ “5”
- Channel Condition - Item B.C.09 must be coded ≥ “6”.
- Channel Protection Condition – Item B.C.10 must be ≥ “6”.
- Scour Condition - Item B.C.11 must be ≥ 5.
- Inventory Load Rating Factor - Item B.LR.05 must be ≥ 1.00 and not have any weight restrictions.
- No Posting restrictions.
- No Category E or E’ fatigue details.
- Highway Minimum Vertical Clearance On - B.H.13 and Highway Minimum Vertical Highway Underclearance - Item B.H.13 must be 14’-0”, or greater (if applicable).
- Span Material - Item B.SP.04 must be coded C01-C05 or S01-S05 (concrete or steel).
- Span Type - Item B.SP.06 must be coded A01, B02-B03, F01-F02, G01-G08, S01-S02, or P01-P02.
- Scour Vulnerability Item B.AP.03 coding must be “A”, or “Blank” (for bridges not over waterways).
- The Length of Longest Span Item B.G.03 must not be greater than 100 feet.
- Estimated AADT Count Item B.H.09 must be < 30,000 and Average Daily Truck Traffic ≤ 3,000. Culverts with 2 or more feet of fill are not subject to the AADT and Truck count criteria.
- The age of the bridge must not exceed 50 years (with reference to Construction Year – Item B.W.01 where Construction Type Indicator - Item B.W.03 = “O”) unless the bridge has been reconstructed within the past 30 years (with reference to Construction Year - Item B.W.02 where Construction Type Indicator Item B.W.03 = “R”).
- All structural members are load-path redundant.

- The bridge shall not carry an interstate route, interstate ramp, or any highway over an interstate. The Functional Classification Item B.H.01 cannot be coded “1”.
- The bridge must not be on the Strategic Highway Network (STRAHNET) system as designated by STRAHNET Designation Item B.H.05 must be coded “N”.

Bridges with repair histories indicating a substantial probability of future problems should not be eligible for the 48-month NBIS Routine Inspection Interval. Such a determination may be made at the discretion of the Agency PM or the Statewide PM.

The eligibility of each bridge is automatically reviewed by the ISIS following any revision of applicable inventory and inspection data. Any formerly eligible structures which no longer meet the 48-month inspection interval criteria have the Routine Interval reduced to a maximum of 24 months. The ISIS automatically changes a 48-month inspection interval to 24 months if:

- The age of the bridge will reach 50 years from the Construction Year – Item B.W.01 before the next Routine Inspection is due: or
- The age of the bridge will reach 30 years from the Reconstruction Year – Item B.W.02 before the next Routine Inspection.

Changes to Inventory items noted above may trigger an immediate change of a 48-month interval to a 24-month interval, causing a bridge to become “instantly delinquent”. When this occurs, every effort must be made to complete the inspection within 30 days. The Delinquency Reason must be properly documented in the Bridge File, on the inspection form(s), and in the ISIS.

<u>3.5.2.2 12-Month Inspection Interval Criteria</u>

IDOT maintained bridges satisfying one or more of the following criteria are required to have a Routine Inspection interval of 12-months:

- Deck Condition Rating Item B.C.01 ≤ “3”.
- Superstructure Condition Rating Item B.C.02, Substructure Condition Rating Item B.C.03, or Culvert Condition Rating Item B.C.04 ≤ “4”.
- Bridge Bearings Condition Rating Item B.C.07 ≤ “2”.
- Scour Condition Rating Item B.C.11 ≤ “3”.
- After the Initial Inspection of Segmental Box Girders, Cable Type Superstructures, Moveable Superstructures, Truss Superstructures and Arch Type Superstructures excluding ‘Arch –

under fill without spandrel, (Span Type B.SP.06 = A02-A05, B04, L01-LX, M01-MX and T01-T03) there shall be two 12-month inspection intervals before being considered for a 24-month inspection interval.

Non-IDOT maintained bridges are currently not subject to a 12-month Routine Inspection Interval. However, IDOT may require a Special Inspection with a 12-month Inspection Interval for non-IDOT bridges in poor condition (condition ratings coded “4” or less) and based on Load Rating Inspections conducted by qualified personnel to determine the safe load carrying capacity of bridges.

3.5.2.3 Routine Inspection Interval for Bridges over Waterways

For bridges over waterways not meeting the Basic Submergence Criteria specified in Section 3.6.4, the condition of substructure elements below the waterline and the surrounding streambed must be assessed during each Routine Inspection. Evaluation of site conditions is required to establish if the normal Routine Inspection Interval is adequate to monitor the parts of the substructure underwater and the streambed. More frequent monitoring can be done with a Special Inspection requirement.

- When only a portion of the substructure units require more frequent monitoring, a Special Inspection is appropriate over a reduced Routine Inspection Interval. Special Inspections can be staggered to efficiently monitor the bridge by having the Special Inspection performed between Routine Inspections.
- When a majority of the substructure units and/or the entire streambed require more frequent monitoring, a reduced Routine Inspection Interval is appropriate over a Special Inspection requirement. See Section 3.5.3 for more guidance on monitoring requirements and recommended intervals.

3.5.2.4 In-Depth Inspection Interval

Every bridge in Illinois must receive an In-Depth Inspection at an interval not to exceed a multiple of the Routine Inspection Interval. See Table 3.5.2.4-1.

Routine Inspection Interval (Months)	In-Depth Inspection Interval (Months)
12	48
24	72
48	96

Table 3.5.2.4-1: Inspection Intervals

3.5.3 Underwater Inspection Interval

Bridges requiring Underwater Inspections as specified in Section 3.6.4 must have an Initial Inspection within 90 days of opening to traffic or from date of determined need. After the Initial Inspection has been performed, the Underwater Inspection Interval Item B.IR.03 shall be set by the Agency PM. IDOT policy is to use an interval of 60 months unless site conditions indicate a need to monitor elements below the waterline more frequently. Alternatively, the Underwater Inspection Interval could be set at 60 months and a Special Inspection could be used to monitor a specific area of concern at a shorter interval.

When only a portion of the substructure units require more frequent monitoring, a Special Inspection may be appropriate rather than reducing the Underwater Inspection Interval. Special Inspections can be staggered to efficiently monitor the bridge by having the Special Inspection performed between Underwater Inspections.

When site conditions indicate the need to monitor a majority of the substructure units and/or the entire streambed require more frequent monitoring, a reduced Underwater Inspection Interval may be appropriate over a Special Inspection requirement. See Section 3.5.3.1 for more guidance on monitoring requirements and recommended inspection intervals.

3.5.3.1 Substructure and Channel Monitoring Requirements and Options

When site conditions warrant additional monitoring, refer to Figure 3.5.3.1-1 “Substructure Condition Assessment Method Below Waterline” to determine the monitoring requirements and Table 3.5.3.1-2 “Substructure and Channel Monitoring Options” to determine the appropriate inspection type and interval.

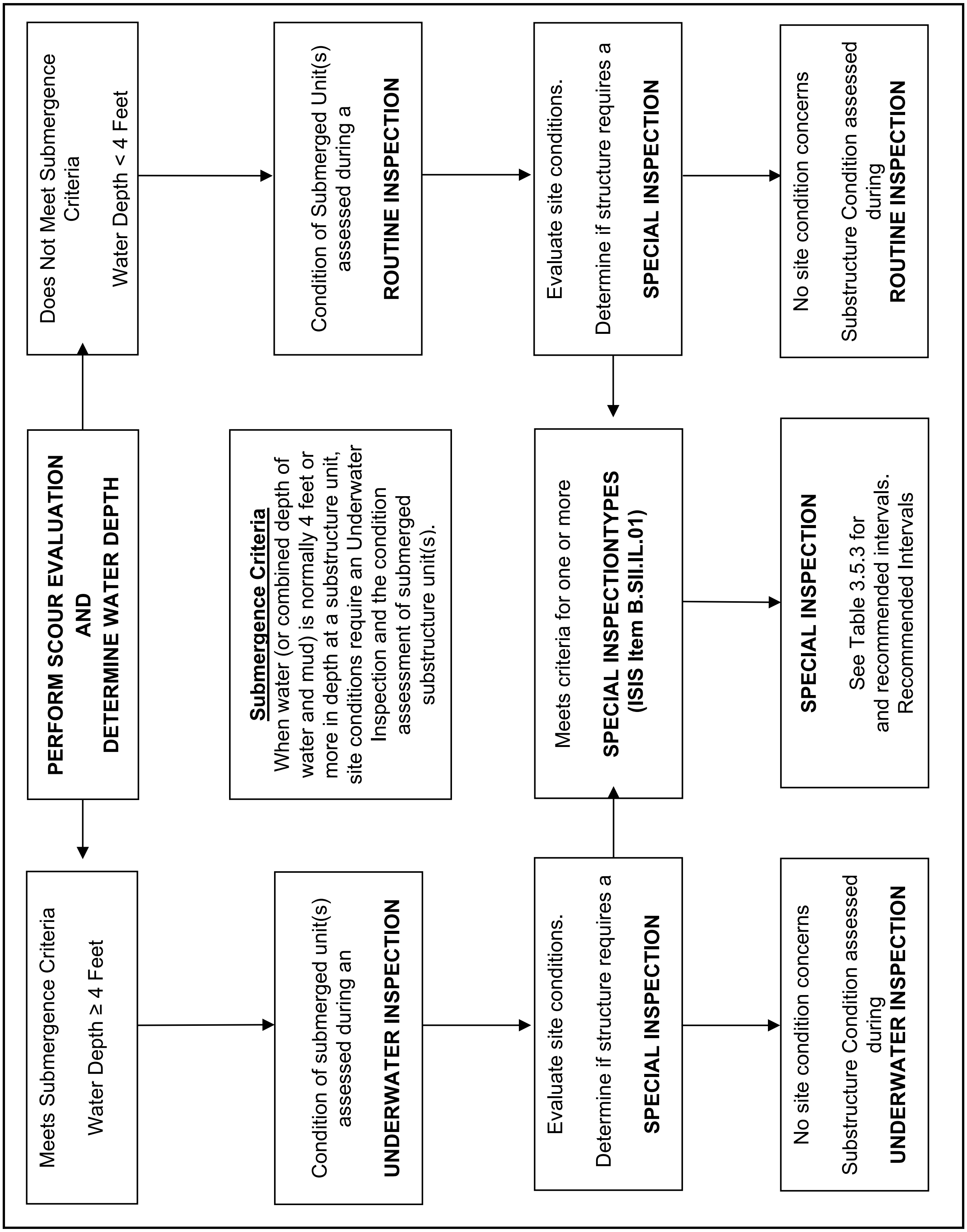


Figure 3.5.3.1-1: Substructure Condition Assessment Method Below Waterline

Description	Underwater Inspection			Routine Inspection		
	Underwater Inspection Interval	Special Inspection Type and Recommended Interval *		Routine Inspection Interval	Special Inspection Type and Recommended Interval *	
a) Known debris problem b) Erodible soils c) Channel Condition Item B.C.09 ≤ 4	60 months	G	12 months	24/48 months	G	12 months
Substructure supported by Spread Footing foundation not adequately keyed into rock or protected from streambed scour or degradation. A Scour Plan of Action (POA) required	60 months	I	24 months (redundant) 12 months (non-redundant)	24/48 months	I	24 months (redundant) 12 months (non-redundant)
Scour Critical or Susceptible as determined by a Scour Vulnerability Evaluation. A Scour Plan of Action (POA) required	60 months	K	Determined in Scour POA	24/48 months	K	Determined in Scour POA
Observed scour at one or more substructure units that has not been mitigated. A Scour Plan of Action (POA) required	60 months	L M N	Determined in Scour POA	24/48 months	L M N	Determined in Scour POA
*Agency PM shall determine the required inspection interval(s) appropriate for site conditions						

Figure 3.5.3.1-2: Substructure and Channel Monitoring Options

3.5.4 Nonredundant Steel Tension Member (NSTM) Inspection Interval

Bridges with NSTM Inspection Required Item B.IR.01 coded “Yes”, are subject to a hands-on inspection at the NSTM Inspection Interval. The intervals for NSTM Inspection are summarized as follows:

- Initial NSTM Inspections must be performed within 90 days of a new bridge opening to traffic or a bridge having undergone major rehabilitation. Bureau of Bridges & Structures suggest these inspections take place within 45 days.
- 24 months from the date of opening to traffic for all newly constructed or rehabilitated bridges. NSTM Inspections must be performed at 24-month intervals unless:
- Bridges with a NSTM Condition Rating Item B.C.14 coded ≤ “4” must be inspected at 12-month intervals.
    - Bridges with a history of fatigue cracking and/or with structural details susceptible to fatigue/fracture as specified by the BBS Bridge Management Inspection Unit.

3.5.5 Element Level Inspection Interval

All IDOT maintained bridges with an NBIS Bridge Length Item B.G.01 > 20.0 feet and non-IDOT-maintained bridges on the National Highway System (NHS) must have Element Level Inspections performed at an interval not to exceed the Routine Inspection Interval.

3.5.6 Special Inspection Interval

Special Inspections are typically required to monitor known structural defects, problematic details, or members requiring special testing.

- For known structural defects, Special Inspections are required to monitor specific locations more frequently than the Routine Inspection Interval and may allow the bridge to have a less restrictive load posting.
- For problematic details or members requiring special testing, such as ultrasonic testing of multi-beam pins and links, Special Inspections are required at specific locations more or less frequently than the Routine Inspection Interval.

3.6 Inspection Procedures

3.6.1 General

There are various types of bridge inspections required by the NBIS and IDOT policy:

- Hands-On Inspection: Inspection within arm's length of the component/element using visual techniques supplemented by non-destructive testing.
- Element Level Inspection: Inspection of the individual elements of a bridge as defined by the AASHTO MBEI and the Illinois supplement to the MBEI. Element quantities are assigned for Condition States on a scale from 1 (no defects) to 4 (severely deficient).
- Initial Inspection: The initial inspection of a new bridge or a bridge having undergone rehabilitation to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data.
- Routine Inspection: A regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements. Major components are assigned condition ratings from 9 (new) to 0 (failure).
- In-Depth Inspection: A close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using Routine Inspection procedures. This typically requires hands-on inspection at some locations.
- Underwater Inspection: Inspection of the underwater portion of a bridge substructure and the surrounding channel which cannot be properly assessed by wading and/or probing. This may require an underwater diving inspection or other specialized techniques.
- NSTM Inspection: A hands-on inspection of NSTMs including visual and other non-destructive evaluation methods.
- Special Inspection: An inspection scheduled at the discretion of the Bureau of Bridges and Structures or the Agency PM, used to monitor a particular known or suspected deficiency. Often used to avoid imposing weight restrictions or closure of the bridge.
- Load Rating Inspection: A scheduled inspection performed to investigate damage or deterioration in order to evaluate potential reductions in the live load carrying capacity. Typically initiated due to a drop in a major component Condition Rating for the Deck Item B.C.01, Superstructure Item B.C.02, Substructure Item B.C.01, or Culvert Item B.C.04, but they can be requested for other reasons.

- Damage Inspection: An unscheduled inspection performed to assess structural damage resulting from environmental factors or human actions.
- Complex Bridge Inspection: This is an in-depth inspection, requiring hands-on inspection procedures, to assess the unique or unusual characteristics of the bridge according to a pre-established, written Complex Bridge Inspection Plan. This is conducted as part of the Routine, Underwater, NSTM, and/or Element Level inspection of a complex bridge.

The various inspection types are performed at intervals based on major component condition, structure type, redundancy, site conditions, load capacity evaluation, and scour vulnerability evaluation.

Once a bridge inspection is completed, the NBIS Team Leader is required to enter the inspection into the ISIS. The Agency PM is then required to “approve” the information in the ISIS to finalize the inspection. ***IDOT policy is SI&A data must be entered/approved into the ISIS within 30 calendar days for intervals ≥ 12-months and within 10 calendar days for intervals < 12-months.*** The 30-day and 10-day allowances only apply to entering/approving the data into the ISIS and shall not be used to extend the inspection intervals.

Bridge inspections must be performed and overseen by qualified personnel as per the NBIS and/or IDOT policy. A certified NBIS Program Manager or Team Leader must be present during the entirety of all Element Level, Initial, Routine, In-Depth, Underwater, NSTM, Special, Load Rating, and Complex Bridge Inspections.

Inspection procedures may vary depending on the characteristics of the bridge, the inspection type, and the extent of bridge deterioration. Detailed inspection procedures, including guidance for taking and recording field measurements, are provided in FHWA *Bridge Inspector’s Reference Manual (BIRM)*, AASHTO *The Manual for Bridge Evaluation (MBE)*, and this manual. Inspection procedures are provided for commonly encountered bridge types and elements.

3.6.1.1 IDOT Bridge Inspection Forms

The current applicable IDOT form must be used to document each bridge inspection. Traditionally, the form is signed/dated by the Team Leader performing or leading the inspection and then signed/dated by the Agency PM. The original document with “wet” signatures must be kept in the Bridge File and an electronic copy of the “wet” signature form must be uploaded to the ISIS. For situations where the Team Leader and Program Manager are the same person, the form must be signed/dated in both places. The web-based ISIS has a workflow designed to eliminate the need for “wet” signatures. First, the Team Leader creates a new inspection in the ISIS and “submits” when ready for review. Next, the Program Manager reviews and “approves” in the ISIS, finalizing the inspection. The actions of “Submitted” by the Team Leader and “Approved” by the Program Manager are equivalent to electronic signatures.

For situations where the Program Manager and the Team Leader are employed by separate entities, such as a government agency and a consultant, the Program Manager still has ultimate responsibility for the quality of the inspection(s). The Program Manager may delegate Quality Control of the Team Leader’s work to others, and it is recommended documentation of this be required for quality assurance purposes. This policy is in no way intended to discourage additional QC/QA measures by the Program Manager within an inspection program but to clarify ultimate responsibility for that program.

3.6.2 Initial Inspection – Applicable to New and Rehabilitated Bridges

The Initial Inspection shall include each inspection type applicable to the bridge. The Agency PM is responsible for ensuring:

- The inspection is performed in accordance with the NBIS and IDOT policy
- The inspection information is entered into the ISIS in a timely fashion.
- IDOT Form BBS 3400 Routine Inspection Report must be used to record the Initial Inspection. Additional forms will be required based on the applicable inspection types for each bridge.

3.6.3 Routine Inspection

The primary objective of a Routine Inspection is to ensure public safety. These inspections are also utilized to determine maintenance, preservation, and repair needs. Routine Inspection must be performed on all bridges on public roadways with an NBIS Bridge Length Item B.G.01 > 20.0 feet. Routine Inspections are performed using visual methods supplemented by various NDT methods and by taking sufficient measurements to determine:

- Assignment, verification, and updating of the various Condition Ratings and appraisal items pertaining to the physical condition and the functionality of the bridge.
- Identification and documentation of deterioration and other deficiencies via description of each defect type, size and location on the bridge component.

- Verification and subsequent update of the bridge inventory data. During Routine Inspections, inspectors are required to verify inventory data items. Bridge inventory data is updated using Form 3320 Bridge Inventory Report.
- Documentation of required maintenance work.

IDOT Form BBS 3400 Routine Inspection Report shall be used to record the Routine Inspection. Structure Inventory and Appraisal (SI&A) data items are coded in accordance with the IDOT SIBI Manual.

The Inspector's Appraisal's section of IDOT Form BBS 3400 contains space for comments next to each Condition Rating item. A concise description of all notable deficiencies must be included for all Condition Ratings of "6 (Satisfactory)" or less and are strongly recommended for Condition Ratings > "6." Photographs of notable deficiencies are required for Condition Ratings of "6 (Satisfactory)" or less. In general, photographs are required for each inspection per the AASHTO MBEI. These include, but are not limited to:

- Roadway on top of the bridge
- Corners of the bridge showing traffic barrier terminals and guardrail
- Upstream view from the bridge
- Downstream view from the bridge
- Elevation view of the bridge
- Deck (Above and below)
- Superstructure
- Substructure
- Culvert
- Channel
- Channel Protection

When the Condition Rating for Superstructure Condition Item B.C.02, Substructure Condition Item B.C.03, or Culvert Condition Item B.C.04 is judged to be ≤ "4 (Poor)", the Condition Rating for Deck Condition Item B.C.01 is judged to be ≤ "3 (Serious)", or the Condition Rating for Bridge Bearings Condition Item B.C.07 is judged to be ≤ "2 (Critical)", a Load Rating Inspection is required to evaluate load carrying capacity. Such a Condition Rating is indicative of the primary load carrying members having deteriorated to a point where the load carrying capacity has been reduced. The Bureau of Bridges & Structures monitors the ISIS to determine when Condition Ratings are lowered to levels requiring a Load Rating Inspection and will schedule an inspection and perform a load rating based on the findings to determine the revised load carrying capacity of the bridge.

When Superstructure Condition Item B.C.02 for a steel superstructure is dropped to ≤ “3” due to section loss of the web over the bearing, the Agency PM responsible for the structure shall ensure hardwood blocking is installed between beam flanges at the identified bearing locations to resist buckling or crushing of the web.

If an inspection reveals an imminent danger to the traveling public is likely, the inspector must immediately take the appropriate action to protect the traveling public, such as lane restrictions or complete closure of the bridge, prior to notifying the Agency PM responsible for the bridge and the Bureau of Bridges & Structures.

When the Bureau of Bridges & Structures is unable to provide a Load Rating Inspection, the agency having maintenance responsibility for the bridge will be notified to retain the services of a NBIS Team Leader to perform a Load Rating Inspection and an Illinois Licensed Structural Engineer to perform a Load Rating to evaluate the load carrying capacity of the bridge. The Structural Engineer’s Load Rating documentation, analysis, models, IDOT Form BBS 2795 Structure Load Rating Summary, and recommendation **must** be submitted to the Bureau of Bridges & Structures for review, concurrence, and approval.

When the normal water depth at all substructure units for a bridge is less than 4 feet, typically referred to as “shallow water conditions,” and the Underwater Inspection Required Item B.IR.03 is coded “N”, an Underwater Inspection is not required. **Supporting documentation including, but not limited to, channel cross sections, photographs, and/or plans must be included in the bridge file to confirm an Underwater Inspection is not required.** Condition of substructure units and the surrounding streambed below the waterline are typically determined by wading and probing during the Routine Inspection as required for the proper coding of Substructure Condition Item B.C.03.

If site conditions at the bridge indicate there is a known deficiency or concern at one or more substructure units, the Agency PM must determine how best to monitor:

- During subsequent Routine Inspections at the normal interval; or
- By reducing the Routine Inspection Interval; or
- By initiating a Special Inspection to address the deficiency; or
- By closing the bridge or restricting traffic.

Information obtained during Special Inspections for substructure deficiencies will be used for the coding of Substructure Condition Item B.C.03.

3.6.3.1 Element Level Inspection

The primary purpose of an Element Level Inspection is to provide a basis for a statewide Bridge Management System. Element Level inspections quantify the physical condition of individually identified elements of a bridge, such as decks, beams, beam ends, expansion joints, bearings, piers, abutments, etc. The Condition States assigned to the elements also indicate the level of action, if any, necessary to bring the element up to a “like new” condition.

Element Level Inspections are required on all IDOT-maintained bridges with a length > 20.0 feet. Element Level Inspections are required on non-IDOT bridges with an AASHTO bridge length > 20.0 feet on the National Highway System (NHS). However, IDOT encourages non-IDOT agencies to perform Element Level Inspections on their respective bridge inventories.

Element Level Inspections are typically performed in conjunction with the Routine Inspection. However, there is not a 12-month reduced interval for Element Level Inspections.

The Agency PM and Team Leader for all Element Level Inspections must have successfully completed either the IDOT Element Level Bridge Inspection course or the FHWA 2-day Introduction to Element Level Inspections.

IDOT Form BBS 3410 Element Level Inspection Report must be used to report Element Level Inspections. A concise description of notable deficiencies (type, size, and location) must be included in the comment fields for any quantity in Condition State “2”, “3”, or “4”.

3.6.3.2 In-Depth Inspection

The NBIS defines an In-Depth Inspection as a close-up, detailed inspection of one or more bridge members above or below the water using visual or non-destructive evaluation techniques to identify any deficiencies not readily detectable using routine inspection procedures. Hands-on inspection may be necessary at some locations. Traffic control and special equipment such as under-bridge inspection equipment, manlifts, scaffolding, boats, and barges, should be provided for access, as needed. In addition, coordination with third parties, such as railroads, may be required.

In-Depth Inspections are to ensure areas of potential concern have received the proper level of inspection. Areas of emphasis include areas under expansion joints, web stiffeners, cross frame connections, lateral bracing connections, fixed bearings, accessible vaulted spans and all other areas of the bridge not normally visible during a Routine Inspection must be inspected. It is imperative the activities, procedures and findings be completely and carefully documented.

3.6.3.2.1 Inspection of Accessible Vaulted Spans

If vault doors and/or access panels are present, the portion of the bridge enclosed by the vault must be inspected as part of In-Depth Inspections.

3.6.3.2.2 Channel Cross Sections

Channel cross-sections, along the upstream and downstream fascias, shall be taken along the entire bridge length for comparison with initial baseline cross-sections at an interval equal to the In-Depth Inspection Interval as defined in Section 3.5.2.4 and after significant flood events. Additional cross-sections should be taken when conditions indicate significant changes from original construction or previous inspections.

Channel cross-sections, along the upstream and downstream fascias, shall be taken along the entire bridge length using the following guidelines. Additional cross-sections may be appropriate after significant flood events.

1. All vertical measurements must be taken from a reference datum line on the bridge that is not likely to change over time. Examples are top of parapet/rail; top of curb/deck; and top or bottom of abutment/pier cap.
2. All channel cross-sections should be taken and plotted with the orientation looking downstream at both fascias of the bridge.
3. Substructure Units should be labeled per the existing plans, if applicable, for consistency.
4. Vertical measurements must be taken at all substructure units. At abutments, measure where ground intersects the exposed face. At Piers/Intermediate Bents, measure at the centerline of the substructure unit.
5. Vertical measurements should be taken at regular longitudinal intervals in each span. The longitudinal interval can be a predetermined percentage of span length or at fixed points on the structure, such as side mounted rail posts.
6. Vertical measurements must be taken at: the beginning and end of a slope; beginning, low points, and end of a scour hole; edges of water; low streambed elevation and any other location(s) with a substantial change in elevation.
7. Vertical measurements should be taken, at a minimum, at the midpoint of a significant ‘flat’ area and the location(s) labeled.
8. Vertical measurements should be taken to the nearest one-half foot unless a scour hole is being measured. Scour hole measurements should be taken to the nearest tenth of a foot.
9. Horizontal measurements should be taken to the nearest foot.
10. If debris piles are present, suggest taking measurements at the ends of the debris pile and across the top of the debris pile to document the extent of the decreased structure opening.
11. If there is a set of twin bridges in close proximity, such as an interstate, and no appreciable difference in streambed elevation between the adjacent fascias, the measurements obtained from one fascia can be used for both bridges.
12. Additional channel cross-sections should be taken when conditions indicate significant changes from original construction or previous inspections. An elevation ‘grid’ may be established for tracking local scour around individual substructure units. Whether accomplished during Routine Inspection or by a Special Inspection, the condition of substructure units below the waterline and the streambed adjacent to those units must be determined to verify existing conditions do not compromise the safety of the bridge. This need for inspection applies to all bridges over water, including those which may have been designed

to structurally accommodate an established scour depth determined by analysis. See Figure 3.5.3.1-1.

13. Inputting the vertical measurements obtained through the above steps into an Excel Spreadsheet is one way to produce a neat and legible Stream Channel Cross-Section. This also allows additional information to be easily added/graphed in the future.

A copy of the most recent channel cross-section data/graphs/sketches, preferably in pdf format, must be maintained in each Bridge File and uploaded to the ISIS.

3.6.4 Underwater Inspection

When the normal water depth is ≥ 4 feet or the combined depth with mud makes it unsafe to wade at any substructure unit of a bridge, site conditions meet the “Basic Submergence Criteria” definition and an Underwater Inspection is required. The Underwater Inspection typically requires the use of a boat and sonar depth finder to properly map the streambed or underwater diving to determine conditions of the substructure units.

The procedures used for determining the need for an Underwater Inspection are the same for new structures which have been designed for anticipated scour and for older structures which were designed prior to the development of procedures for estimating scour. Figure 3.5.3.1-1 illustrates the use of the various inspection types referred to in this Section.

IDOT Form BBS 3420 Underwater Inspection Report shall be used to record Underwater Inspections. For bridges requiring an Underwater Inspection, an Underwater Inspection Plan is required to document the inspection of each substructure unit. IDOT Form BBS UIP Underwater Inspection Plan must be used to develop the Underwater Inspection Plan. A typical Underwater Inspection Plan may include, but is not limited to, the following:

- A plan view of the bridge displaying each substructure unit requiring an Underwater Inspection.
- Detailed Underwater inspection procedure(s) for each substructure unit.
- All equipment required to properly complete the Underwater Inspection.
- Training/certification requirements for the dive team.
- Detailed description of what deliverables will be produced from the Underwater Inspection.

Underwater Inspection procedures may vary depending on weather, stream level, stream current, and debris collection. The Underwater Inspection Plan must include a detailed history of such variances for determining the optimal time of year when appropriate site conditions exist. The Underwater Inspection Procedures and Plans must be updated at an interval not to exceed sixty (60) months.

The objectives of an Underwater Inspection are to ensure public safety by determining the Condition Rating and to provide a history of the channel conditions under and adjacent to the bridge. Conditions such as channel opening width, depth at substructure elements, channel cross-sections, water flow velocity, channel constriction and skew must be noted and compared to historical records. Local scour conditions at or near substructure units can be monitored by establishing a streambed elevation “grid.” Over time, vertical changes, due to either degradation or aggradation processes, or horizontal alignment changes, due to lateral migration of the channel, could result in foundation undermining, bridge overtopping, or partial/total collapse of the bridge.

For bridges/culverts located in low-flow or no-flow conditions, such as lakes, ponds, or lagoons, or in controlled pool conditions above locks and dams, channel cross sections are still required to be taken at the bridge/culvert.

3.6.4.1 Assessing Substructure Conditions Below Waterline

Bridges requiring an Underwater Inspection are identified in the ISIS by having the Underwater Inspection Interval Item B.IR.03 coded with the appropriate value, and all Underwater Inspection Substructure Units Item B.IE.IL.08.

If site conditions at the bridge indicate there is a known deficiency or concern at one or more substructure units, the responsible Agency PM must determine how best to monitor:

- During subsequent Underwater Inspections at the normal interval; or
- By reducing the Underwater Inspection Interval; or
- By initiating a Special Inspection to address the deficiency or concern; or
- By closing the bridge or restricting traffic.

If the Agency PM elects to initiate monitoring with a Special Inspection, the appropriate Special Inspection Type Item B.SII.IL.01 and Special Inspection Interval Item B.IE.05 must be determined and entered in the ISIS. See Figure 3.5.3.1-2 for guidance on Special Inspection and Substructure and Channel Monitoring Options.

3.6.4.2 Procedures for Underwater Inspections

Procedures utilized for an Underwater Inspection may include, but are not limited to, the following:

- Visual observation of the channel at, and adjacent to, all substructure units.
- Manual probing of the channel bottom and/or substructure footing at, and adjacent to, the substructure units to identify areas of concern or to establish channel bottom elevations.
- Probing of the substructure walls below the waterline for damage.
- Recording of the channel bottom elevations with depth finding sonar or other means at, and adjacent to, the substructure units, as well as at channel cross sections upstream and downstream from the bridge.
- Visual observation of the substructure units to detect possible abnormalities in elevation, plumbness, or rotation.
- Observation of conditions below waterline by qualified personnel including properly trained certified divers.

When the water depth is too great or site conditions prohibit an inspector from adequately determining the condition of any of the substructure units by visual means, probing, or sonar, a diving inspection is required.

Procedures used for accomplishing the Underwater Inspection are recorded on IDOT Form BBS 3420 and in ISIS as Underwater Inspection Method Item B.EI.IL.07. As bridge deficiencies are discovered or suspected, more thorough evaluation techniques should be employed. These may include surveying measurements to determine the elevation, plumbness, and possible rotation of substructure units or obtaining soil borings.

3.6.4.3 Substructure Condition Rating Based on Deficiencies Below Waterline

The findings from the Routine, Underwater, and, if applicable, Special Inspections should be taken into account when determining the Substructure Condition Item B.C.03 coding. Condition of the substructure unit(s) below the waterline must be included during the evaluation of the substructure as part of each Routine Inspection. Inspectors are encouraged to take advantage of low water conditions during Routine Inspections to observe elements below the waterline, especially those elements normally receiving an Underwater Inspection.

The rating for Substructure Condition Item B.C.03 cannot be greater than the rating for Underwater Condition Item B.C.15, but the Underwater Condition Item B.C.15 rating can be greater than the Substructure Condition Item B.C.03 rating.

3.6.4.4 Determining Safe Conditions for Standard Diving Inspections

- With IDOT approval, Underwater Acoustic Imaging may be performed at unsafe bridge sites to preliminarily evaluate the bridge prior to the Underwater Inspection Interval date. The approved types of Acoustic Imaging equipment are Sector Scanning Sonar, Mechanical Scanning Sonar, and Multibeam Sonar. The Underwater Inspection Note Item B.IE.11 must include a comment stating Acoustic Imaging was done without diving until diving could be rescheduled due to unsafe diving conditions. The Dive Team shall reschedule the dive as soon as conditions are safe.
- FHWA does not accept sonar evaluations for Underwater Inspections as a substitute for diving, even in situations where the Underwater Inspection cannot be safely performed by divers. However, FHWA recognizes safety of the diving inspectors and adherence to policies are necessary when conditions are unfavorable. Full documentation of the conditions and reasons why sonar inspection was employed must be included in the Underwater Inspection Report.

A typical diving inspection team consists of the following:

- IDOT defines the Underwater Bridge Inspection Dive Team as those actively participating in the underwater diving inspection.
- Individuals solely responsible for safety of personnel and equipment operation are not considered part of the Underwater Bridge Inspection Dive Team.
- Dive Team Size: 3- or 4-person team and must include an Illinois Certified NBIS Team Leader having an Illinois Structural Engineer license on site for 100% of the inspection.
- Standard dive equipment (does not include hot-water suits or hyperbaric chamber)
- Commercial scuba or surface-supplied air in accordance with OSHA Dive Standards
- All dive team members are required to be certified divers and to have taken NHI 130091 Underwater Bridge Inspection.
- Appropriately sized boat (typically less than 25 feet in length)

3.6.4.4.1 Procedures for Underwater Diving Inspections

The following procedures must be performed during an Underwater Diving Inspection:

- If over navigable water, proper notification and inspection schedules must be sent to the Bureau of Bridges & Structures and the United States Coast Guard in advance of the inspection.
- Visual and tactile inspection of 100% of the underwater portions of the substructure units.
- Provide measurements of pitting and damage of each substructure unit.
- Provide a complete report with topside photographs, diagrams, and underwater photographs necessary to adequately describe conditions and deficiencies found at the water surface and below, and anything above the waterline that would be hidden from an inspector not diving.
- Reports will include scope of work, description of the bridge, method of investigation, existing site conditions, substructure conditions with elevation views of each unit detailing any damage or deterioration, streambed elevations, channel probing information at each unit, channel elevation grid at 25-foot intervals out to 50 feet from each unit, streambed cross-sections at both fascias and at 100 feet upstream and downstream, IDOT Form BBS 3420 Underwater Inspection Report, and additional information as required by the Bureau of Bridges & Structures.
- The reports shall be signed and sealed by an Illinois Licensed Structural Engineer.

For non-IDOT maintained/inspected bridges, the Agency PM will determine the requirements of the Underwater Diving Inspection for each bridge and is responsible for coordination with the waterway authority and local law enforcement. However, IDOT recommends the above requirements be incorporated into the Underwater Diving Inspection.

3.6.5 Nonredundant Steel Tension Member (NSTM) Inspection

Per the NBIS, a Nonredundant Steel Tension Member (NSTM) is defined as:

> A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.

NSTM Inspections must be performed on bridges having NSTMs inventoried in the ISIS. NSTM inspections consist of a 100% hands-on (within arm’s length) investigation for the entire length of all NSTMs present on the bridge. Various nondestructive testing techniques are also employed to supplement the hands-on visual inspection.

A NSTM Inspection Plan must be developed which includes, but is not limited to, the following:

- IDOT Form BBS 3430 Nonredundant Steel Tension Member Inspection Report
- NSTM Inventory Sketch identifying all NSTM types including brief descriptions and locations.
- Maximum Inspection Interval of 24 months for rating of NSTM Condition Item B.C.14 ≥ "5"
- Maximum Inspection interval of 12 months for rating of NSTM Condition Item B.C.14 ≤ "4"
- Description of NSTM inspection procedures including:
  - Specific inspection requirements including cleaning of critical areas prior to inspection
  - Specific measurements to be taken
  - NDT methods to be used
  - Access equipment required to see all sides of all NSTMs and all welds attaching the members
- Where the fracture toughness of the steel is not documented, testing may be needed to determine the threat of brittle fracture at low temperatures. (Consult with the Bureau of Bridges & Structures before taking samples for testing.)
  - Maintenance of traffic plan
  - Detour/bypass information
  - Additional lighting and magnification needs
  - Critical Findings procedures
  - Emergency contact information

The NBIS requires an NSTM Inspection Plan be included in the Bridge File for each bridge having NSTMs. It is the responsibility of the Agency PM to ensure the NSTM Inventory and Inspection Plan is developed, submitted to the Bridge Management and Inspection Unit, included in the Bridge File, updated at an interval not to exceed twenty-four (24) months, and uploaded to the ISIS.

As part of the development and/or review of the NSTM Inspection Plan, the Agency PM shall verify any active FHWA Technical Advisories applicable to the bridge, or its components, are included. All FHWA Technical Advisories are located at the FWHA website.

When documenting and recording cracks or other deterioration during NSTM Inspections, several items of information should be noted:

- The precise location of the crack or deterioration with respect to the member must be recorded, as well as the exact location of the member with respect to the entire bridge. Any noticeable lengthening, opening and closing, and visible distortion of the crack when exposed to live load must be documented.
- The members must be labeled in the field using paint or other permanent markings. The ends of all cracks must be marked, and the date identified. It is important to compare the new markings with previous markings, if any.
- Detailed sketches and photographs of cracking and other deterioration must be provided showing the orientation, length, width, depth and end of crack. A close-up view should be provided, as well as a general view of the member from the same perspective to show context.
- The inspection team must document dimensions and details of the member containing the crack. The general condition at the location of the problematic detail must be noted, including section loss, dirt/debris, traffic impact, and steel type if available from existing plans or shop drawings.

For bridges with superstructures containing NSTMs, the rating for “Superstructure Condition Item B.C.02” shall not be > the rating for NSTM Inspection Condition Item B.C.14 for the superstructure NSTM, though it may be lower.

For bridges with substructures containing NSTMs, the rating for Substructure Condition Item B.C.03 shall not be greater than the rating for NSTM Inspection Condition Item B.C.14 for the substructure NSTM, though it may be lower.

3.6.5.1 Identifying Bridges with NSTMs

NSTMs have either all or part of their cross section subject to tension and do not have load path redundancy.

When identifying bridges with NSTMs, only load path redundancy is to be considered. While the presence of structural redundancy and internal redundancy may reduce the consequences of the failure of a NSTM, these types of redundancy are not to be considered when identifying NSTMs.

The FHWA *Bridge Inspector's Reference Manual* (BIRM) defines redundancy as follows:

> Load Path Redundancy: Bridge designs that have three or more main load-carrying members or load paths between supports are considered load path redundant.
>
> Structural Redundancy: System redundancy is a redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse (23 CFR 650.305). System redundancy cannot be determined in the field or from bridge plans. A refined analysis, with methodology and evaluation criteria reviewed by FHWA, is necessary to determine whether a bridge exhibits system redundancy.
>
> Internal Redundancy: Internal Redundancy exists when a bridge member contains three or more elements that are mechanically fastened to each other so that multiple independent load paths are formed.

The Bureau of Bridges & Structures ultimately determines if a bridge has NSTMs and will coordinate the NSTM Inventory and NSTM Inspection Plan with the Agency PM. The Bureau of Bridges & Structures is responsible for the NSTM Inventory entry into the ISIS.

3.6.5.1.1 Types of NSTM Bridges in Illinois

Common bridge types with NSTMs include, but are not limited to, the following:

- One- or two-girder systems (I-girder or box girder)
- Suspension systems with eyebar components
- Steel abutment/pier caps
- Deck/Through truss bridges
- Steel tied arches
- Pin and hanger connections on two- or three-girder systems
- Bascule/Lift bridges with two primary girders or trusses
- Floorbeams with spacing exceeding 15 feet or supporting other longitudinal members
- Ends of flared primary members attached to webs of other longitudinal members.
- Transfer beams/girders
- Steel columns/piles

3.6.5.1.2 NSTMs - Common Problematic Details

Problematic structural details, which may result in cracking, may exist on a variety of bridge superstructures and substructures. Problematic details may include, but are not limited to, the following:

- Triaxial constraint
- Welds – Intersecting; field welds on patch/splice plates; intermittent; tack; poor quality; plug
- Welded Cover plates
- Suspended spans
- Details prone to out-of-plane bending (small web gaps, diaphragm connection plates that are not attached to flanges)
- Improperly installed or spliced back-up bars
- Ship lap girders
- Dapped beam/girder ends.

Information on problematic details and procedures for proper inspection can be found in the following publications:

- FHWA *Bridge Inspector's Reference Manual (BIRM)*
- AASHTO *The Manual for Bridge Evaluation (MBE)*
- AASHTO Design Specification – LRFD Manual (for identification of fatigue categories)
- NHI-130078 "Bridge Inspection Techniques for Nonredundant Steel Tension Members (NSTM)"

Fatigue-prone details must be given special attention as part of the NSTM Inspection and may require non-destructive testing.

<u>3.6.5.2 Recording NSTM Inspections</u>

All NSTM Inspections must be reported using IDOT Form BBS 3430 NSTM Inspection Report. New and rehabilitated NSTM bridges shall receive an initial NSTM Inspection within 90 days of opening to traffic to establish a baseline of the as-built conditions.

3.6.5.3 Identifying Misclassified Bridges

The Agency PM and Team Leader are responsible for identifying bridges that have not been classified as having NSTMs. If a bridge is identified as having NSTMs, the Agency PM must contact the Bridge Management & Inspection Unit at DOT.BBS.BridgeMgmt@illinois.gov for review and approval. Upon IDOT's confirmation of the bridge having NSTMs, IDOT Form BBS 3432 NSTM Inventory and Inspection Plan must be completed and submitted to the Bridge Management Group at DOT.BBS.BridgeMgmt@illinois.gov for review, approval, and entry into the ISIS.

3.6.5.4 NSTM Inspection – Gusset Plates

While gusset plate assemblies are not a specific NSTM type, each gusset plate assembly must have a Hands-On Inspection as part of a NSTM Inspection. Detailed thickness measurements must be taken to document and assess any section loss on the gusset plates. Initiation or growth of cracks and development or increase in any gusset plate distortion must also be documented. The inspection record for each gusset plate must document all significant changes to previously identified areas of concern as well as any new areas identified with each new inspection.

The Agency PM shall determine if new inspection findings are significant enough to require structural analysis and load rating. Consultation with the Bureau of Bridges & Structures is required to determine the need for further analysis.

Copies of the inspection report, including a summary of all field measurements and photographs shall be kept in the Bridge File and uploaded to the ISIS. It is imperative that complete and accurate records are kept to chronicle the initiation and progression of gusset plate deterioration and the steps taken to repair or mitigate those findings.

All repairs, retrofits and/or member replacements completed since the last NSTM Inspection must be documented in the Bridge File and uploaded to the ISIS. The condition of the work performed and a general assessment of its effectiveness in arresting or mitigating previous damage or deterioration must be noted.

3.6.5.5 NSTM Inspection – Non-Destructive Testing

Often, visual inspection alone is not sufficient to determine the Condition Rating of a NSTM. Non-destructive Testing (NDT) must be performed to fully ascertain the NSTM Condition Rating. The use of NDT methods is vital in determining the section remaining, the presence of cracking, and the extent of cracking. The NDT methods most frequently used by IDOT inspection personnel are ultrasonic testing, dye penetrant testing, and magnetic particle testing. Ultrasonic testing is used for the various measurements, including remaining thickness of plates, and for detection of defects/flaws in pins. Dye penetrant and magnetic particle testing are used for detecting the presence and limits of cracks. Personnel must have the proper training in the use and limitations of the various methods of NDT.

3.6.6 Special Inspection

Special Inspections are performed to monitor a specific structural feature, deficiency, or condition more frequently than at the Routine, NSTM, or Underwater Inspections.

Special Inspections are initiated for a variety of reasons at the discretion of the responsible Agency PM or the Bureau of Bridges & Structures (BBS). The BBS initiates Special Inspections through the Structures Ratings and Permits Unit for IDOT maintained bridges or the Local Bridge Unit for non-IDOT maintained bridges.

Procedures used during Special Inspections should be adopted in accordance with the specific deficiency or condition to be monitored, such as monitoring the propagation of cracks in steel or concrete members or the additional section loss to steel, concrete, timber, and other materials. The Special Inspection Type Item B.SII.IL.01 and the Special Inspection Interval Item B.IE.05 are determined by the initiator and are inventoried in the ISIS. Failure to comply with the inspection frequency and/or the established procedure for the required Special Inspection may result in posting, reduced posting, and/or partial or complete closure of the bridge.

3.6.6.1 Performing and Recording Special Inspections

IDOT Form BBS 3440 Special Inspection Report must be used to record Special Inspections.

It is imperative the personnel performing Special Inspections must compare the conditions noted in the field to the initial conditions to determine the proper coding of Special Inspection Condition Status Item B.SII.IL.08.

Special Inspection Remarks Item B.SII.IL.05 are required for all Special Inspections.

For Special Inspections initiated by the BBS, the Agency PM must notify the BBS when the Special Inspection Condition Status Item B.SII.IL.08 is coded:

- 0 – Worsening Condition Indicative of Imminent Structural Failure (closure required until follow-up inspection by BBS staff)
- 1 - Progression of Deterioration or Worsening of Condition noted (immediate follow-up inspection by BBS staff or District Bridge Maintenance Engineer required)
- 3 - Corrected Condition Noted (Special Inspection no longer required after verification of adequacy of corrected condition by appropriate IDOT personnel)

The inspector must document the change in condition by providing adequate comments on the form, proper photographs, and updated sketches.

Personnel performing Special Inspections must be fully instructed on the specific members and locations required to be inspected and what signs could manifest indicating a worsening condition. A worsening of the condition could result in posting, reduced posting, and/or partial or complete closure of the bridge. The BBS strongly recommends consecutive Special Inspections be performed by the same inspection personnel to ensure sufficient site-specific knowledge.

3.6.6.2 Special Inspections for Multi-Girder Pin-and-Link Assemblies

Multi-girder/beam bridges with pin-and-link or pin-only assemblies that have been retrofitted with stainless steel pins and Teflon bushings are required to have a Special Inspection with an Inspection Interval not to exceed 48 months. Multi-girder/beam bridges with pin-and-link or pin-only assemblies that have not been retrofitted as described above have a Special Inspection with an Inspection Interval not to exceed 24 months. Ultrasonic testing of the pins and hands-on inspection of the beam ends are required at each inspection. The link assemblies shall be visually observed to verify they are functioning as intended. The link plates shall be measured as required to verify any section loss or cracks. Any defect found in the pins or link plates must be reported to the Bureau of Bridges & Structures immediately for evaluation.

If a defect is found in a pin or a crack is found in a link plate, the Inspection Interval of the Special Inspection shall be changed by the Bureau of Bridges & Structures as noted below:

- If the assembly appears not to be functioning properly or section loss in the link plates is between 5% and 10%, the Inspection Interval shall be 24 months or less.
- If the loss is greater than 10%, the Interval shall not be greater than 12 months, and the Bureau of Bridges & Structures must be notified to perform a structural evaluation.

3.6.6.3 Special Inspections for Dapped Beams

Steel dapped beams in multi-girder/beam bridges are required to have a Special Inspection with an Inspection Interval equal to the interval of the Routine Inspection. This requirement is also strongly recommended for dapped concrete girders/beams. Each dapped girder/beam end must have a Hands-On Inspection as part of the Special Inspection. Detailed thickness measurements must be taken to document and assess any section loss for steel members. Initiation or growth of cracks and new development or increase in any section loss must also be documented. The inspection record for each dapped girder/beam end must document all significant changes to previously identified areas of concern as well as any new areas identified with each new inspection.

The Agency PM shall determine if new inspection findings are significant enough to require structural analysis and load rating. Consultation with the Bureau of Bridges & Structures is required to determine the need for further analysis.

3.6.7 Damage Inspection

Damage Inspections are performed on an emergency basis to assess a bridge for damage resulting from environmental factors or human actions causing a sudden change in the structural capacity or stability of a bridge. The inspection is typically performed by staff of the Agency PM, or an Illinois Licensed Structural Engineer as directed by the Agency PM.

The primary objective of a Damage Inspection is to determine the need for immediate load restrictions or partial/complete closure of the bridge to traffic and/or to assess the need for temporary measures to remain open. The secondary objective is to assess the level of effort necessary to repair the damage. The inspection team performs in-depth hands-on inspection of the damaged members, determines the extent of section loss, takes measurements for misalignment of members, and checks for any loss of foundation support. Traffic control and special equipment are often necessary to accomplish these inspections.

Photographs, detailed sketches, and detailed descriptions documenting all notable deficiencies potentially impacting the load-carrying capacity and/or stability of the bridge must be submitted to the Bureau of Bridges & Structures as soon as possible for evaluation.

3.6.8 Load Rating Inspection

The primary objectives of a Load Rating Inspection are to document all areas of section loss greater than 10% and any other deficiencies affecting the load-carrying capacity of the bridge and to identify members/elements requiring additional monitoring through Special Inspections. Load Rating Inspections are required when a Routine Inspection results in the following:

- Superstructure, Substructure, or Culvert Condition Rating Item B.C.02, 60, or 62 drops to “4 - Poor”, or less, and subsequent drops in Condition Rating.
- Deck Condition Rating Item B.C.01 drops to “3 - Serious”, or less, and subsequent drops in Condition Rating.
- Bridge Bearings Condition Rating Item B.C.07 drops to “2 – Critical”, or less.
- Or if 10 years have elapsed since the Last Rating Date Item B.LR.03 and the Superstructure, Substructure, or Culvert Condition Rating Item B.C.02, B.C.03, or B.C.04 ≤ “4 - Poor” or the Deck Condition Rating Item B.C.01 ≤ “3 - Serious”

An Illinois Certified NBIS Program Manager or Team Leader must be present during the entirety of all Load Rating Inspections. and must be trained to collect and document the information necessary to conduct a load rating analysis. Detailed information in the form of photographs and sketches displaying field measurements of deteriorated areas and the precise locations are required to determine the load carrying capacity. Also measured/verified during the Load Rating Inspection are permanent loads such as wearing surface, railing attachments, and other sources of dead loads. Traffic control, under bridge inspection equipment, and specialized testing equipment are essential to accomplish a Load Rating Inspection.

3.6.8.1 Load Rating Inspection for IDOT-Maintained Bridges/Culverts

Load Rating Inspections are performed under the direction of the Bureau of Bridges & Structures, Structure Ratings and Permits Unit by IDOT District Bureau of Operations personnel or a consulting firm.

The Structure Ratings and Permits Unit monitors major component Condition Rating reductions for IDOT maintained bridges using the information in the ISIS and schedules load rating inspections for the applicable bridges. In addition, Districts Operations can request a Load Rating Inspection by contacting the Structure Ratings and Permits Unit

The findings of the Load Rating Inspection are reviewed by the Structure Ratings and Permits Unit, and a structural analysis is done to determine the load-carrying capacity. After completing the analysis, the Bureau of Bridges & Structures will provide the Region/District with documentation of the Load Rating Inspection and results of the structural rating. The documentation provided to the Region/District will include field data and photographs, revisions to major component Condition Ratings, updated Inventory and Operating Rating Factors, and, if necessary, required weight restrictions and/or Special Inspection requirements.

3.6.8.2 Load Rating Inspection for Non-IDOT Bridges/Culverts

Load Rating Inspections are performed by the Bureau of Bridges & Structures Local Bridge Unit of the Bureau of Bridges & Structures and consultants under the direction of the agency having maintenance responsibility.

Using the information in the ISIS, the Local Bridge Unit monitors major component Condition Rating reductions for Local Public Agency maintained bridges and automatically schedules Load Rating Inspections for the applicable bridges to be done by in-house forces. In addition, Local Public Agencies can request a Load Rating Inspection by submitting IDOT BLR Form 06510 Local Agency Load Rating Request to the Local Bridge Unit at DOT.LocalBridgeUnitt@Illinois.gov.

If the Local Bridge Unit is unable to provide a Load Rating Inspection, the Local Public Agency will be notified to retain the services of a qualified consultant firm to perform the Load Rating Inspection and subsequent Load Rating. The documentation for the Load Rating Inspection and Load Rating must be submitted to the Local Bridge Unit for review and concurrence with the Load Rating, required weight restrictions, and/or Special Inspection requirements. IDOT Form

BBS 2795 Structure Load Rating Summary shall be included with the Load Rating calculations and submitted to the Bureau of Bridges & Structures for review and approval.

3.6.9 Complex Bridge Inspection

Complex Bridge Inspections are performed on bridges with Complex Features. The NBIS defines a Complex Feature as:

> Bridge component(s) or member(s) with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures. This includes mechanical and electrical elements of moveable spans and cable-related members of suspension and cable-stayed superstructures.

Complex bridges must receive a heightened degree of investigation and evaluation to preserve continued service to the travelling public and to protect the owner's investment. The inspection of complex bridges presents many unique challenges for an experienced bridge inspection team.

IDOT has determined the following types of bridges are considered Complex Bridges based on the NBIS definition of Complex Feature and <u>must</u> be inspected under the provisions and policies of this section:

- Suspension – Cables and Anchorages
- Cable-Stayed – Cables and Anchorages
- Moveable (Bascule, Swing, and Lift) – Mechanical and Electrical Elements

The NBIS requires each Complex Bridge to have a detailed Complex Bridge Inspection Plan. The plan must be kept in the official Bridge File, be uploaded to the ISIS, and be available for the inspection team prior to and during each inspection.

<u>3.6.9.1 General Complex Bridge Inspection Procedures</u>

A comprehensive Complex Bridge Inspection Plan must provide a detailed outline for all aspects of the inspection process. The following sections provide guidance for development of a Complex Bridge Inspection Plan. There may be additional information required for specific bridges.

3.6.9.1.1 Team Members

The Agency PM with direct oversight of the Complex Bridge must select a qualified and experienced bridge inspection team, trained for the specific tasks required for the Complex Bridge. The on-site Illinois Certified NBIS Team Leader is responsible for ensuring the quality and completeness of the inspection and is expected to provide guidance to the remainder of the inspection team. Inspection team members are required to be familiarized with a specific bridge's previously noted deficiencies and concerns. It is preferable if the inspection team members have previous experience inspecting the specific bridge or a bridge of similar type.

Traffic Control: Complex Bridge Inspections typically require an extensive Traffic Control Plan which may include, but is not limited to:

- Time restrictions imposed by the owner
- Detailed timing and sequence of lane restrictions/closures
- Traffic control devices (message boards, impact attenuators, signage, barrels, cones, etc.)
- Coordination with 3rd parties (railroads, utility providers, others)
- Movement and temporary storage of inspection access equipment

The Team Leader overseeing the inspection must initially develop the Traffic Control Plan with the Agency Traffic Operations, Maintenance, and Construction personnel for inclusion in the Complex Bridge Inspection Plan. Prior to new inspections, follow-up coordination is required to ensure the traffic control does not interfere with other transportation related activities in the area.

Typically, the agency having maintenance/inspection responsibility already has a Traffic Control Plan developed due to previous maintenance or repair activities. However, if a consultant is performing the inspection, coordination with the agency's appropriate personnel (operations, traffic, construction, or other) is required. The agency contacts must be provided for reference.

3.6.9.1.2 Navigable Waterways

For Complex Bridges crossing navigable waterways, the Agency PM must coordinate the inspection with the United States Coast Guard (USCG) prior to new inspections. The USCG contact information must be included.

3.6.9.1.3 Inspection Access Equipment

Complex Bridge Inspections typically require the use of a variety of access equipment which may include, but is not limited to:

- Under-bridge inspection crane (Snooper, towable inspection platform)
- Manlift or Bucket Truck
- Ladders
- Scaffolding
- Rope access equipment
- Boats
- Barges

Coordination of inspection access equipment and availability of operators through the use of in-house equipment or third-party vendors must be included. Access to enclosed restricted areas, such as gated portions of the undersides of the bridges, and confined spaces, such as box girders, tower entrances, anchorage pits, mechanical/electrical rooms, etc., must be detailed, documented, and the appropriate contact information provided for access.

3.6.9.1.4 Inspection Equipment

A Complex Bridge Inspection requires extensive use of inspection tools, testing devices, and other equipment in order to complete a thorough inspection of the bridge. The tools and equipment required may include, but are not limited to:

- Hand tools (hammers; scrapers; wire brushes; drills, grinders, etc.)
- Cameras
- Sonar depth finders
- NDT equipment
- Drones may be considered for specific cases. A proposal outlining the intended scope of the drone inspection must be submitted for approval prior to their use.

3.6.9.1.5 Documentation

A Complex Bridge Inspection requires detailed and comprehensive documentation when compared to the typical Routine Inspection, and the findings must be compiled into a formal Inspection Report. The report must contain, but is not limited to, the following:

- Overall description of bridge
- IDOT forms (signed/dated)
- Detailed inspection notes
- Organized narrative
- Photographs with descriptions and locations
- Table or listing of all notable deficiencies
- Recommendations for general maintenance, prevention activities, and required repairs

The notable defects should be prioritized by importance and severity. Any defect of a primary load carrying member with > 10% section loss likely has reduced load carrying capacity and must be reported immediately to the Agency PM and the Statewide PM.

3.6.9.2 General Inspection Requirements for Suspension Bridges

A suspension bridge is typically comprised of a steel superstructure of beams, girders, and floorbeams supporting a riding surface, all suspended by cables or wires from two large multi-wire cables stretched over high towers and anchored in large concrete structures. The general inspection requirements for a suspension bridge are as follows:

3.6.9.2.1 Deck

The deck, including any wearing surface should be inspected for signs of distress using appropriate procedures, such as chain drag, infrared thermography, ground-penetrating radar, etc. Deck elevations should periodically be surveyed and compared to original as-built plans and previous inspection reports for any marked changes. An inspection of the traffic barriers, joints, and deck drainage system should be included.

3.6.9.2.2 Cable

The main cables of a suspension bridge are usually comprised of hundreds of individual wire strands, combined together to form the main cable. The cable is usually protected from the elements by a system of wire wrapping and a barrier coating of paint. The cables can also be wrapped in special waterproof fabric material. Inspection of the cables is typically restricted to a visual, hands-on procedure, concentrating on identification of any signs of potential problems with the protection system. Any signs of tearing or gouges that could allow intrusion of water should be documented. Areas surrounding the cable bands, areas around tower saddles, and entrances to the anchorages should be inspected thoroughly. The cable inspection should also document the condition of the safety handhold cable and its connections to the main cables.

A schedule shall be documented in the Complex Bridge Inspection Plan and followed for an In-Depth Inspection involving the use of Non-Destructive Evaluation (NDE) techniques and/or intrusive inspection, where a portion of the cable wrapping is removed and the wire strands are wedged apart allowing for both a visual and borescope inspection of individual wires.

3.6.9.2.3 Cable Anchorage

Consult the Confined Space Entry Plan before entering the anchorage structure. The inspection should note the overall condition of the anchorage house. The inspector should document the temperature and humidity both inside and outside the anchorage and should also document any sources of water intrusion into the anchorage building. The inspector should carefully inspect the condition of the cables as they pass through the anchorage walls and into the splaying (cable spreading) structure noting any signs of rust or deterioration.

3.6.9.2.4 Hangers

Hangers or suspenders should be thoroughly inspected for any signs of wear or fraying. Note any signs of rust, pitting, or section loss of the wires. Connections to the cable, typically at cable bands should be carefully inspected for signs of rust staining or water intrusion. Hanger cable connections to the superstructure, usually through lead filled sockets and mechanically fastened brackets should also be thoroughly inspected for signs of rust, pitting, section loss, or distortion.

3.6.9.2.5 Towers

Typically comprised of massive steel legs and bracing members, towers can rise hundreds of feet above the deck surface. A detailed plan for access and inspection of the tower members should be part of the Complex Bridge Inspection Plan. This may require hiring an experienced inspection team, trained in rope and free climbing techniques in order to gain access to the external components of the tower. A typical suspension bridge tower will have an internal system of ladders or stairs that gain access to the top of the tower. The inspection should include a complete documentation of the condition of all tower members, including the competence of the access ladder or stair structures. Proper operation of the tower aviation lighting and condition of their supports should also be noted.

3.6.9.2.6 Miscellaneous

The inspection report should include a complete inspection of the structure's expansion joints, noting the opening and air temperature and any out-of-tolerance movements. Superstructure wind-tongue structures should be inspected, noting any areas of deterioration or section loss, and any defects in the operation of the system. Proper operation of navigation lighting and condition of their supports should also be noted.

<u>3.6.9.3 General Inspection Requirements for Cable-Stayed Bridges</u>

A cable-stayed bridge is typically comprised of a steel or concrete box or girder superstructure supporting a deck structure, all supported by a series of fanned or splayed high strength multi-strand cables, spanning from anchorages near the top of tall concrete or steel towers to the superstructure near deck level. The general inspection requirements for a cable-stayed bridge are as follows:

3.6.9.3.1 Deck

The deck of a cable-stayed bridge is typically concrete, either cast in place or comprised of precast concrete panels with a bituminous or concrete overlay wearing surface. In most designs, the deck of a cable-stayed bridge is always in compression. The inspection should note any distressed areas in the deck, including any signs of compressive force damage or any notable transverse cracks that could indicate a loss of deck compression and a potential sag in the superstructure. The deck, including any wearing surface should be inspected for signs of distress using appropriate procedures, such as chain drag, infrared thermography, ground-penetrating radar, etc. A deck survey should be completed periodically to document elevations against original as-built plans and past inspection surveys. The inspection should include an inspection of the deck drainage system.

3.6.9.3.2 Cable

A typical cable-stayed bridge cable is comprised of a series of multi-wire, high strength steel strands inside a steel or plastic pipe encasement. The encasement pipe will usually have a secondary protection system such as a fabric wrap and/or paint. The cable protection system should be visually inspected for any signs of distress or areas of possible water intrusion. Areas near cable anchorages in the towers, areas where the cable typically passes through the deck structure, and areas near the connection to the superstructure should be inspected thoroughly. Any cable damping system should be inspected to ensure that it continues to provide proper damping of wind-induced cable vibrations. A schedule shall be developed and followed for an In-Depth Inspection involving the use of Non-Destructive Evaluation (NDE) techniques and/or intrusive inspection of the cables.

3.6.9.3.3 Cable Anchorages

If possible, the cable anchorages at the superstructure connections and at the towers should be inspected for signs of moisture intrusion. Signs of trapped water and any leaching should be noted. Anchorage protective end caps can usually be removed for a close inspection of the strand ends and wedge condition.

3.6.9.3.4 Tower

Cable-stayed bridge towers are usually massive, hollow, concrete structures. The inspection should include a visual assessment of the outside of the towers, noting any cracks, rust staining, leaching, or significant deterioration. The Confined Space Entry Plan shall be followed before entering the inside of the tower. A system of ladders, stairs, or elevators is available for access to the top of the towers, including access to cable anchorages near the top of the tower. Proper operation of tower aviation lighting and condition of their supports should be noted also. The access door should be inspected for proper security locks.

3.6.9.3.5 Miscellaneous

The inspection report should include a complete inspection of the structure's expansion joints, noting the opening and air temperature and any out-of-tolerance movements. Proper operation of navigation lighting and condition of their supports should also be noted.

3.6.9.4 General Inspection Requirements for Movable Bridges

A movable bridge presents unique challenges. Dynamic loadings are induced into structural members of the bridge as the bridge is operated. Inspectors should observe the operation of the bridge in order to identify any unusual vibrations or noises or irregular movement of the structure. The bridge operators and bridge mechanics should be interviewed to discuss any operational issues identified.

In addition to structural aspects of the bridge, movable bridge owners usually employ bridge mechanics and electricians to oversee the routine inspection, maintenance, and repair to the mechanical and electrical elements of the bridge. These employees are usually responsible for the inspection and maintenance of the power supply, wiring, motors, control panels, trunnions, rack castings, gears, and counterweights. Although the NBIS inspection is the prime responsibility of the Illinois Certified NBIS Team Leader, he/she should discuss operation of the bridge with the mechanic and electrician. Mechanical / electrical issues can lead to improper operation of the bridge and induce unusual forces into members, potentially causing severe buckling or torsion. The general inspection requirements for a moveable bridge are as follows:

3.6.9.4.1 Deck

Most movable bridges have an open steel grid deck system in order to reduce the overall dead load of the span. The steel grid shall be inspected for potential member breakage, weld failures, and deterioration of connection details to the steel superstructure. The open grid design allows deicing salts and water to fall directly on the steel superstructure members below the deck.

3.6.9.4.2 Superstructure

The superstructure inspection shall follow the Complex Bridge Inspection Plan for the bridge.

3.6.9.4.3 Mechanical Systems

The inspection of the mechanical systems of the bridge shall include any operational reports from the bridge operators and mechanics. In addition, the inspector shall observe the operation of the span, noting any unusual sounds or vibrations. The lift system shall periodically be scheduled for a span balancing procedure to ensure efficient and safe operation of the structure.

3.6.9.4.4 Electrical Systems

The inspection of the electrical systems of the bridge shall include any operational reports from the bridge operators and electricians. In addition, the inspector shall observe the operation of the span, noting the proper function of traffic gates and signals.

3.6.10 Critical Findings

Per the NBIS, a Critical Finding is defined as a structural or safety related deficiency that requires immediate action to ensure public safety.

3.6.10.1 Automatic Critical Finding Per NBIS

The instances below are considered Critical Findings, regardless of the circumstances:

- Bridges requiring partial/complete closure of a bridge lasting > 24 hours.
- Deck Condition Item B.C.01, Superstructure Condition Item B.C.02, Substructure Condition ItemB.C.03, or Culvert Condition Item B.C.04 coded ≤ “2 - Critical”
- Channel Condition Item B.C.09 or Channel Protection Condition Item B.C.10 coded ≤ “2 – Critical”
- Scour Vulnerability Item B.AP.03 coded ≤ “D - Critical”
- Nonredundant Steel Tension Member Appraisal Rating Item B.C.14 coded ≤ “3 - Serious”
- Special Inspection Condition Status Item B.SII.IL.08 coded “0”
- Bridges requiring immediate load restriction or posting, or immediate repair work to a bridge, including shoring, to remain open

3.6.10.2 Potential Critical Finding

Potential Critical Findings include bridges sustaining damage from extreme events including, but not limited to, the following:

- Vehicle impact
- Barge impact
- Fire
- Flood
- Earthquake

When a potential Critical Finding is identified, the safety of the traveling public is the priority. The Illinois Certified NBIS Program Manager or Team Leader on-site must take the necessary steps to ensure the safety of the travelling public. If the identified deficiency or defect seriously reduces the load carrying capacity of the bridge, traffic must be restricted to isolate the area in question, or the bridge must be closed within 24-hours pending further evaluation by the Bureau of Bridges & Structures. Simple shoring, such as timber blocking, must be installed within 72 hours. All relevant information regarding the deficiency or defect must be provided to the Agency PM. As the immediate risk to public safety has been addressed, additional follow-up actions may be required, such as a Load Rating Inspection, structural analysis to determine if a weight restriction below legal loads is required, or the development of repair plans.

Once the Agency PM has reviewed, the information must be submitted to the Statewide PM to determine if the Critical Finding is warranted. The Agency PM completes IDOT Form BBS CF 1 Critical Finding Report with all supporting documents, signs/dates the form, and submits it to DOT.BBSBridgeMgmt@Illinois.gov.

For LPA bridges, also send to DOT.BBSBridgeMgmt@Illinois.gov.

The Bureau of Bridges & Structures will coordinate with the Agency PM to determine if there is a need for a Damage Inspection or Load Rating Inspection, follow-up structural analysis, Special Inspection, and to develop a plan of action to mitigate the deficiency.

3.6.10.3 Critical Finding Reporting Requirements

All Critical Findings reported to, or generated by, the BBS are submitted to the FHWA Illinois Division. The Agency PM must submit IDOT Form BBS CF 1 Critical Finding Report to the BBS as soon as possible to meet the timeframes in Section 3.6.10.4. The Agency PM must provide sufficient detailed information on the form; sign as Agency PM and submit it to the Bridge Maintenance & Inspection Unit for review and signature by the Statewide PM. The Statewide PM will upload the Critical Finding documentation to the ISIS. The Agency PM can download the information for the Bridge File.

If the Statewide PM determines the deficiency or defect does not constitute a Critical Finding, the Agency PM will be notified by email with a detailed explanation of the determination and any recommendations for follow-up action.

3.6.10.4 FHWA Notification of a Critical Finding

Per NBIS regulation §650.313(q)(2)(i), the Bureau of Bridges & Structures - Bridge Management & Inspection Unit will report Critical Findings to the Illinois Division Office of the FHWA within 24 hours for bridges on the NHS that require full or partial closure and/or have an NSTM that is to be rated in serious or worse condition, as defined in the NBIS (see §650.315) by the NSTM Inspection item, coded three (3) or less. All other critical findings shall be reported to FHWA within 30 days.

IDOT will provide monthly status reports of open Critical Findings to the FWHA in accordance with 23 CFR 650.313 (q)(2)(ii).

3.6.11 Inspection of Bridges/Culverts under Construction

When a bridge/culvert, or any portion of a highway bridge, is open to the traveling public, it is subject to the requirements of the NBIS and must be inspected.

3.6.11.1 Bridge/Culvert Replacement, Repair, or Rehabilitation – Staged Construction

For staged construction projects, the portion of the bridge open to the traveling public must be inspected at regular intervals to ensure safety. Routine, Underwater, NSTM, Element Level, and Special Inspections are to be completed in accordance with the NBIS. The NBIS Initial Inspection is required within 90 days of the bridge being opened to each stage of traffic. However, the Bureau of Bridges & Structures considers 30 days to be more than sufficient time to perform the NBIS Initial Inspection.

3.6.11.2 Bridge/Culvert Replacement – New Bridge/Culvert on New Alignment

When an existing bridge is being replaced with a new bridge located on a new alignment, the existing bridge is to be inspected in accordance with the NBIS until the bridge is closed. The NBIS inspection for the new bridge must be completed within 90 days of opening to traffic.

3.6.11.3 Bridge Replacement/Rehabilitation – Closed to Traffic During Construction

For an existing bridge that is closed to public traffic during total replacement or rehabilitation work, an NBIS inspection is to be completed, and SI&A data must be updated, within 90 days of completion of the work (all lanes open to public travel).

3.6.11.4 Temporary Structures – Open to Traffic for > 24 Months

If a temporary bridge is being utilized to carry the traveling public for more than 24 months during the construction of the permanent bridge, the temporary bridge must be inventoried in the ISIS and inspected in accordance with the NBIS. Generally, the bridge being rehabilitated or replaced remains in the inventory and appropriate SI&A data are to be coded. The existing bridge will be “deleted” with Status “D” when it is removed.

3.6.12 Inspection of Closed Bridges

Bridges closed to the travelling public are not subject to the requirements of the NBIS. However, IDOT policy is to continue to monitor all closed bridges.

The annual IDOT Local Posting and Closure Reviews by District Operations in District 1, and by the Bureau of Local Roads and Streets for Districts 2 thru 9, satisfy the IDOT requirement to monitor closed bridges maintained by Local Public Agencies. However, IDOT encourages bridge owners, including IDOT districts, to monitor all closed bridges as well. The monitoring is conducted to verify the closure is per the MUTCD and IL-MUTCD and is not being violated. Any potential or suspected hazards from falling debris must be noted and reported immediately to the Agency PM.

For bridges suspected of continued use by trespassers, additional measures may be warranted to prevent access to the bridge such as a more permanent means of closure or removal of the deck/superstructure.

A closed bridge in the inventory may pose undue liability to the agency having maintenance responsibility. IDOT recommends scheduling the bridge for demolition if the bridge continues to be a source of use by trespassers, concern for partial/total collapse, or is a hazard to the public.

Once bridge closure has been mandated by IDOT and implemented in the field, the bridge cannot be reopened without approval from the Department.

3.7 Non-Destructive Testing

3.7.1 General

Non-Destructive Testing (NDT) is often an essential tool used during the inspection of steel bridge members to determine the remaining thickness of various deteriorated elements, presence and/or extent of defects when visual inspection suggests the existence of a defect or is not sufficient to verify the internal integrity of structural members, such as pins. The most frequently used NDT methods by IDOT Inspection personnel are:

- D-Meter Thickness Measurement (DM) – Used to measure steel thickness
- Dye Penetrant Testing (PT) – Used to determine presence and extent of cracks
- Magnetic Particle Testing (MT) – Used to determine presence and extent of cracks
- Ultrasonic Testing (UT) – Used to confirm internal integrity of bridge pins

Inspection personnel responsible for the inspection of steel bridges must be familiar with the use and limitations of the various NDT methods. To ensure knowledge of the proper application of the testing procedures, limitations, and interpretation of results, inspection personnel require additional training by certified instructors and extensive hands-on experience.

3.8 Inspection of Pins, Links, and Hangers

3.8.1 Inspection of Pin & Link Assemblies

3.8.1.1 General

The information provided in this section provides an expedient method for evaluating in-place double-pin and link or single-pin moment release joints on multi-beam/girder and 2-girder bridges. Region/District inspectors are trained by IDOT employees previously certified by outside agencies in the use of ultrasonic testing (UT) equipment and evaluation methods. UT and visual inspection (VT) shall be utilized for a Preliminary Pin Inspection (PPI), based on pass-fail criteria related to the detection of defects or discontinuities within the length of the pin. This procedure, although based on industry practices, is an IDOT method developed to provide basic condition information, and neither intends nor purports to identify all irregularities within a pin.

3.8.1.2 Inspector Qualifications

Bridge Inspection personnel utilizing UT for the inspection of pins must be adequately trained. IDOT District Inspection personnel performing a PPI to detect significant defects/discontinuities are trained by IDOT Central Office Bridge Inspection personnel. At a minimum, the training provides:

- Familiarization with operating principles of UT;
- Calibration methods, using American Society for Testing Materials/American Society for Nondestructive Testing (ASTM/ASNT) standard UT reference blocks; and an IDOT UT reference standard for preliminary pin evaluation.
- Initial VT and subsequent UT scanning methods for various situations and configurations, including assessments and reporting of results.
- IDOT pin scanning methods, result interpretation, testing documentation and recommendations for follow-up inspection or immediate action.
- Potential problems with access and interpretation, and alternative tests or methods.
- Field exercise or classroom mock-up.
- Additional discussion, questions and answers, etc.

The training includes extensive hands-on and interactive participation, encouraging participants to critically examine the methodology and propose modifications and improvements. Upon the successful conclusion of the training, inspectors will be considered qualified to conduct preliminary pin and link inspections. During a PPI, if results indicate the presence of anomalies or are inconclusive, a Supplemental Pin Inspection (SPI) must be scheduled for additional nondestructive testing (NDT) to be performed by IDOT District or BBS personnel or outside testing agency personnel qualified to at least ASNT UT Level II criteria.

3.8.1.3 Link Inspections

All accessible surfaces of link plates shall be visually inspected (VT) for corrosion; paint deterioration; distortion due to misalignment or seizure; and cracking, paying particular attention especially at the critical sections near the pins. Any confirmed or suspected cracks shall be immediately reported to the Agency PM, to schedule further NDT and/or the implementation of temporary measures such as shoring lane restrictions; or weight restriction. Other defects shall be documented for future reference or for directing follow-up NDT.

UT or DM may be used during link inspections to verify remaining material thickness, but they are not typically used for crack evaluation and detection in link plates. Magnetic particle inspection (MT) or dye penetrant testing (PT) may also be used to assess possible cracks if the inspector is equipped and trained for their use.

The inspector should attempt to determine if the pins are “frozen” within the girder webs and/or link plates or are free to rotate due to temperature changes or the passage of vehicles across the bridge. A continuous layer of paint or corrosion across the interface with no interruption caused by rotation may indicate seizure. Single pins in moment release joints may undergo sufficient motion under live load to disclose the absence of seizure. Dual pins in hanger links usually rotate gradually due to temperature changes and do not show perceptible motion under live load, so seizure is difficult to assess.

3.8.1.4 Pin Inspections

If pins are found to have no defects, based on the criteria herein, no additional NDT is required. If anomalies or non-conforming pins are detected, the Agency PM must schedule for a SPI to provide additional UT evaluation by IDOT District, or BBS personnel or outside testing agency personnel qualified to at least an ASNT UT Level II. Based on the results of the SPI, recommendations can be made for replacement or subsequent inspection criteria for suspect pins. If the PPI indicates the actual (or potential) failure of one or more pins, the Agency PM, and the Statewide PM must be notified immediately and temporary measures, such as shoring and/or lane restrictions, weight restrictions, or complete bridge closure may be required until further inspections, pin replacement or supports can be implemented.

An actual or suspected pin failure is considered a potential Critical Finding. Once steps have been taken to ensure the safety of the traveling public, a “Critical Finding Report” must be submitted to the Bureau of Bridges & Structures Bridge Maintenance & Inspection Unit as specified in Section 3.6.10.

3.8.1.4.1 *Preliminary Pin Inspections (PPI)*

The objective of a PPI is to detect the presence of significant defects or discontinuities within the length of a pin. A method for a Preliminary Pin Inspection is as follows:

- If issues arise in evaluating the far end of long and/or small diameter pins, the 3.5 MHz transducer may be more accurate with its smaller beam spread. The greater beam spread of the 2.25 MHz transducer may be preferred for pins with cotter pin holes near the contact end, and to access areas behind threads or step-down diameter reductions at ends.
- Calibrate the equipment before beginning work, using the transducer and IDOT test specimen. For pins up to 10” (250 mm) long, adjust the display so horizontal screen increments represent 1” (25mm). For longer pins, adjust screen increments to represent convenient units (for example: 1 ¼”, 1 ½”, 2”), while keeping the pin’s far end back signal on the screen.
- At the first pin to be tested, verify there are no unexpected reflectors (not including threads, cotter pin holes, etc.) are present between the transducer and far end. If any appear, go to another pin until a satisfactory pin is found. Adjust the gain so the back reflection of the pin’s far-end is 50% of the screen height (use this value as the reference level on IDOT Form 2760 Preliminary Pin and Link Inspection Journal) and record the transducer and dB reading on the inspection form. For inspection scanning of that pin and all similar pins, increase that gain by 10 dB.
- Distance calibration should be periodically verified by checking the IDOT test specimen after each two hours of work; at each new work location, measuring the pins at one deck joint; for significant changes in pin lengths or condition; or if the pin’s temperature rises or falls more than about 30°F, which could affect couplant behavior. The IDOT test specimen should be at approximately the same temperature as the tested pins.
- Scan one end of each pin, using a circular motion and positioning the transducer so near-end threads do not degrade the image while evaluating the far end of the pin. Compare results to

the pin geometry, so diameter changes, cotter pin holes or far-end threads are not mistaken for defects. If the “shelf” or “shoulder” (change in diameter of the pin) is more than ¼ inch, scanning should be done from both ends.

- Indications, especially in bearing contact areas, producing an image more than 20% screen height at reference level (50% of screen height at end of pin) requires the pin to be scanned from the opposite end as well. The results shall be recorded for future reference, indicating the location (distance from each end); gain; signal height and transducer position to maximize the signal for each reflector. The transducer’s positions when the defect initially appears and finally disappears or is minimized on the screen shall also be noted. “Transducer position” shall be described by its circumferential location, based on a theoretical clock face with 12 at the top of the pin, and radial distance from the center of the pin. For consistency, pins requiring additional inspections shall have the 12 o’clock position center punch marked on both ends near the periphery, and “clockwise” shall be considered independently for each end.
- Scanning from both ends may provide sufficient data to characterize the type, size, orientation and criticality of defects. If any reflector produces a signal more than 50% of screen height at reference level, or if a pin’s far end response is completely lost while scanning from either end, the results for both ends must be immediately reported to the Designated Certified NBIS Agency Program Manager and the Statewide NBIS Program Manager. Large internal discontinuities may prevent most of the pulse from reaching the far end or redirect the reflected signal, resulting in loss of far end back reflection.

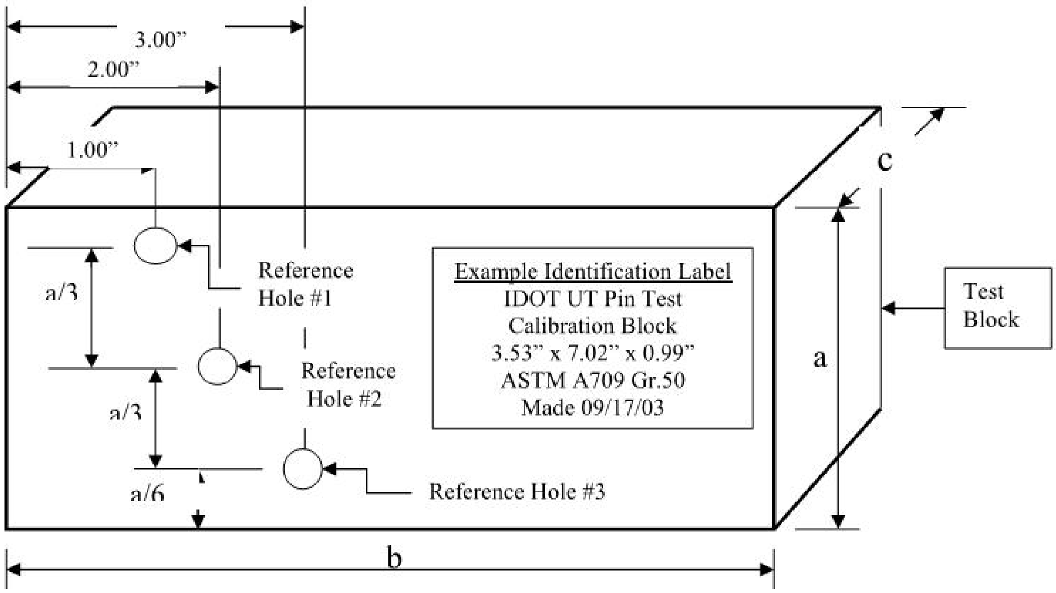


FIGURE 1: IDOT UT Pin Test Calibration Block

Based on pins inspected and material available, select a, b & c as follows:

a (height): nearest ½" from 3" to 6"

b (length): nearest whole inch from 6" to 12"

c (thickness): nearest ¼" from ¾" to 1¼"

Accurately determine all dimensions (± 0.01") and place label on specimen.

Reference Holes (RH) #1, #2 and #3 are 1/16" (0.0625") diameter, through-thickness of test block, and perpendicular to face within ± 0.5 degrees.

Material for test block must be ultrasonically "clean", and either low alloy, carbon steel or stainless steel may be used. All surfaces are milled and polished, and all corners are perpendicular (within 0. 5°).

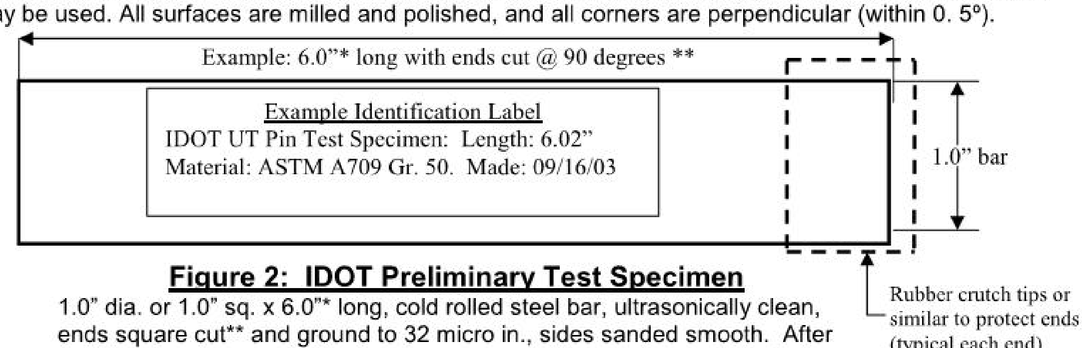


Figure 2: IDOT Preliminary Test Specimen

1.0" dia. or 1.0" sq. x 6.0"* long, cold rolled steel bar, ultrasonically clean, ends square cut** and ground to 32 micro in., sides sanded smooth. After finishing, bar is coated with clear lacquer to avoid corrosion and labeled.

Notes: *Test specimen length of 6" is arbitrary. Convenient lengths representative of typical pin lengths may be used. Even inch increments (7", 8", 9", 10") are recommended for ease of calibration.

** Ends of specimen cut square to within 0. 5 degrees, length tolerance ± 1/8"

Figure 3.8.1.4.1-1: Diagram "A" – IDOT Test Block & Specimen

3.8.1.4.2 Supplemental Pin Inspections (SPI)

The equipment and procedures used to conduct a Supplemental Pin Inspection (SPI) are the same as used for performing the PPI. The SPI is, however, performed by personnel qualified to be at ASNT Level II.

3.8.1.5 Inspection Records

Permanent inspection records must be maintained for pins and links. Forms for documenting the condition of pins and links during PPIs and SPIs are provided by IDOT Form BBS 2760, “Preliminary Pin and Link Inspection Journal” and IDOT Form BBS 2780 Supplemental Pin / Link Inspection Journal. Pins and links with no structurally significant anomalies detected by the PPI only require stating “No Defects Noted”, along with the item’s location, the pin end(s) scanned, and any pertinent remarks to assist future inspections. Note that “location” for pins and links includes span and girder/beam number, distance to nearest identified support, pin location (upper or lower) and link position, such as “east side”. Other comments regarding field conditions; paint condition; possible seizure; excessive rust staining; etc. or recommended maintenance actions must be reported to the Agency PM.

A copy of the permanent inspection records for each pin and/or link with reportable anomalies, indicating all pertinent field observations and recommendations for additional investigations, shall accompany the preliminary inspection report submitted to the Agency PM. The permanent record documents the initiation and subsequent growth of defects and will be maintained in the Bridge File even after the item is replaced. If reported indications are proven to be non-significant or “spurious”, caused by technique or other causes, but not actual internal defects, the record shall include possible explanations and verification testing results.

Pins requiring additional investigation, due to a reflector exceeding 20% of screen height during the PPI, shall have the location of all reflectors exceeding 20% of screen height at the reference level documented, as described in Item 6 of the “Preliminary Pin Inspection Method” in Section 3.8.1.4.1.

Additional investigation required for links to quantify potential structural problems noted during the PPI may employ UT, MT, PT, DM or radiographic testing (RT), as directed by the Agency PM. The additional test results shall be recorded on a separate form for each link and included in the link’s permanent inspection record for future reference, even if the indications prove to be non-significant or “spurious.” The record shall include possible explanations and verification of the testing results. Additional information, test reports, photos or other pertinent information shall be attached to the permanent record and stored in the Bridge File.

If paint, rust and/or other foreign material is removed from links, nuts and non-stainless-steel pins for VT, MT, UT or PT, bare steel shall be protected after the inspection is complete. UT couplant, PT residue and moisture shall be removed and then a zinc-rich primer or other owner-approved coating may be spray-applied. For pins to be inspected on a regular interval, the exposed ends may be coated with grease.

Copies of each bridge’s overall evaluation, together with copies of the permanent records for each pin and/or link and additional testing recommended to evaluate anomalies, shall be signed by the Team Leader and submitted to the Agency PM, as soon as possible after the inspection. Reports, including summaries, photos and additional investigations of PPI-revealed deficiencies, along with inspection personnel’s suggestions or comments applicable to the particular location and similar installations, shall be compiled by the Team Leader and submitted to the Agency PM.

Inspector suggestions and comments may include initial and follow-up investigation methods, preventative maintenance actions, inspection documentation, employee safety considerations, and methods to improve inspection efficiency.

3.8.2 Inspection of Cantilever Truss Suspended Span Pins & Hanger Members

3.8.2.1 General

Continuous cantilever trusses typically have suspended simple span trusses in the main span, which are supported by large steel pins and hanger members. The pins can be up to 30” in length and 12” in diameter. Hangers are typically built-up members or rolled sections with additional reinforcement plates at the pin locations. The pins and hangers in suspended truss spans are typically considered NSTMs.

3.8.2.2 Pin Inspections

In general, suspended truss pins and hangers are to be inspected by the same criteria as beam/girder pin and links. A PPI shall be completed at each location. A detailed access and pin preparation plan may be required. Suspended truss pins may have center bore holes with steel threaded rods and end cap plates requiring removal before a PPI can be completed. Consult previous inspection records and procedures or discuss potential limitations of cap or retention plate removal before the inspection. In some instances, test pins, fabricated to the same specifications as the existing pins, may be available for calibration of UT equipment.

As with pin and link inspections, the objective of the PPI is to detect the presence of significant defects or discontinuities within the length of the pin. If anomalies or non-conforming pins are detected, Agency PM must schedule a Supplemental Pin Inspection (SPI).

Any actual or suspected pin failure is considered a potential Critical Finding. Once steps have been taken to ensure the safety of the traveling public, a Form BBS CF 1 Critical Finding Report must be submitted to the Bridge Management and Inspection Unit as specified in Section 3.6.10.

3.8.2.3 Hanger Inspections

Suspended span truss hangers are typically large vertical members consisting of built-up members or rolled sections with additional reinforced plates at the pin locations. The entire hanger, a NSTM coded in the ISIS, must be hands-on inspected for the full length of the member. Any suspected cracks, distortions, or other potential defects must be recorded on the appropriate inspection form. Further investigation may require specialized NDT methods by qualified personnel.

3.8.3 Inspection of Pins and Eyebars in Eyebar Trusses

3.8.3.1 General

Eyebar trusses are no longer used in modern bridge construction, but several remain in the Illinois bridge inventory. These are typically located on low traffic volume routes. Eyebar trusses are typically light, relatively short, simple-span trusses but can reach major bridge lengths and span major waterways.

3.8.3.2 Pin Inspections

Eyebar truss pins are to be inspected by the same criteria as beam/girder pins. A PPI shall be performed on each pin and the results recorded on the appropriate inspection forms. Eyebar truss pins are considered NSTMs and must be inspected during each scheduled NSTM Inspection.

In many instances, excessive pack rust between eyebars at pin locations may have totally seized any movement of the pins. Excessive pack rust should be carefully removed, if possible, without causing any damage to the eyebars. In addition, the inspector should verify that there is sufficient pin length beyond the outside eyebars or that the end caps, retainer nuts, or cotter pins are functioning properly. Excessive pack rust can exert enough force to push eyebars off their pins in extreme cases.

3.8.3.3 Eyebar Inspections

Eyebars are to be inspected by the same criteria as links. Most eyebars are typically cast, forged, or cut from rolled steel plate material and are rectangular in cross-section and transition to a wider section with an opening to receive a steel pin. The “eyes” in round eyebars are generally flattened by forging. The “eye” can also be formed by creating a loop that is forged or welded to close the loop. The entire eyebar is a NSTM and must be hands-on inspected for the full length of the member.

As with links and hangers, the eyebar shall be visually inspected for any cracks, gouges, section loss, or distortions. Particular attention shall be paid to the forged or formed “eye” areas. Any anomalies or potential defects shall be reported immediately to the District/Area or Agency PM. Any major flaws or suspected fractures shall be immediately reported to the District/Area or Agency PM and the Statewide PM after taking all necessary procedures to safeguard public safety. IDOT Form BBS CF 1 Critical Finding Report must be completed and submitted to the Bureau of Bridges & Structures as specified in Section 3.6.10.

3.9 Bridge/Culvert Scour Vulnerability Evaluation and Scour Plan of Action (POA)

3.9.1 General

In 1988, the FHWA initiated the National Scour Evaluation Program. The NBIS specifies all bridges over waterways must be assessed for scour susceptibility and to determine the required protection in the form of countermeasures to ensure the stability of the bridge. The quantification of scour susceptibility is by the Scour Vulnerability Item B.AP.03. The NBIS further specifies the agency having maintenance/inspection responsibility for bridges determined to be scour critical or scour susceptible must prepare, implement; and periodically update a Scour POA to monitor known and potential deficiencies.

The following definitions are applicable to terms used in this section:

> Scour: Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.
>
> Scour Critical Bridge: A bridge with Scour Vulnerability Item B.AP.03 coded “C” or “D”.
>
> Scour Susceptible Bridge: A bridge with Scour Vulnerability Item B.AP.03 coded “B” or “E”.

All bridges in the inventory over waterways with an NBIS Bridge Length Item B.G.01 greater than 20 feet must be evaluated for scour. A Scour POA is required for bridges with a Scour Vulnerability Item B.AP.03 coding of “B”, “C”, “D”, “E”, or “U”. The Scour POA is maintained in each Bridge File and must be uploaded to the ISIS. See Section 3.9.4 regarding monitoring scour critical and scour susceptible bridges.

Bridges over waterways have the potential for severe scour. Heavy rainfall events producing stream flows beyond design levels, buildup of debris against one or more substructure units; pressure flow; or other factors. Once the actual scour progresses beyond the limits of the Scour Vulnerability Item B.AP.03, the bridge must be closed until the stability of the bridge can be re-evaluated. This will very likely require a new Scour Evaluation Study and structural analysis.

3.9.2 Scour Evaluation and Stability Assessment (SES) of New/Rehabilitated Bridges over Waterways

New bridges/culverts and rehabilitation projects reusing the foundations must have a SES, determination of estimated scour depths, and a Scour Vulnerability Item B.AP.03 rating completed during the preliminary planning phase.

- For IDOT maintained bridges, the Bureau of Bridges & Structures must review and approve the SES for each bridge.
- For other state, agency, and LPA bridges, the designer must provide the SES, estimated scour depths, and Scour Vulnerability Item B.AP.03 rating when submitting IDOT Form BLR 10210 Preliminary Bridge Design and Hydraulics Report.

See the IDOT *Bridge Manual* for design guidance.

3.9.3 Scour Evaluation and Stability Assessment of Existing Bridges Over Waterways

When scour has occurred exceeding the assessed or calculated scour depths, a SES is required to re-evaluate these characteristics. The study must determine:

- A revised calculated estimated scour depth
- If additional scour is likely to occur, due to rainfall events, before scour countermeasures can be implemented
- If the bridge is stable for the observed scour and for additional scour in the interim

A SES is also required for existing bridges not previously having a Scour Vulnerability Evaluation and requires the following information:

- Information relative to the current rainfall event (estimated flood frequency)
- Historic site conditions including streambed and channel stability as well as previously observed scour
- Soil borings or other data indicating the type and properties of the material present at, and below, the elevation of the streambed
- Hydraulic data (current and historic flood events)
- Structural information related to the substructure units of the bridge (existing plans, as built plans, pile driving records, etc.)

The Bureau of Bridges & Structures and the District/Agency will coordinate to determine if the study and stability analysis are required and agency having maintenance/inspection responsibility is to provide. The completed study and stability analysis along with IDOT Form BBS 2678 Scour Vulnerability Evaluation Report must be submitted to the Bureau of Bridges & Structures for review and approval. If approved, the Scour Vulnerability Item B.AP.03 coding will be entered into the ISIS by the Bridge Management & Inspection Unit.

If the study determines the bridge is not stable for the depth of scour observed or the potential future scour, the bridge must be closed immediately and can be reopened once engineered scour countermeasures are designed, installed per plans/specifications, and functioning as intended. The bridge will require future monitoring through a Scour POA and IDOT's scour monitoring program BridgeWatch®.

The Agency PM must contact the Bureau of Bridges & Structures immediately for consultation and to determine the proper course of action. If further analysis is required before the stability of the bridge can be determined, the bridge must remain closed or must have designed temporary countermeasures installed. The Agency PM and the Statewide PM will determine the appropriate temporary measures on a case-by-case basis.

3.9.4 Reporting Field-Observed Scour

3.9.4.1 Scour Critical Bridges

Observed scour meeting either of the criteria below must be reported to the Agency PM and the Statewide PM immediately for further direction:

- Observed scour ≥ 25% of the as-built overburden above the top of footing has occurred; or
- Observed scour exposing the top of a spread footing or > six (6) feet at a pile supported footing or pile bent substructure unit.

The IDOT Form BBS 2678 Scour Vulnerability Evaluation Report must be completed immediately and submitted to the BBS for entry into the ISIS. The Scour Vulnerability Item B.AP.03 must be coded "D".

3.9.4.2 Scour Susceptible Bridges

For bridges with Scour Vulnerability Item B.AP.03 coding of “C” or “E”, observed scour meeting one of the criteria below must be reported to the Agency PM and the Statewide PM as soon as possible for further direction:

- Observed scour ≥ 50% of the as-built overburden above the top of footing has occurred; or
- Observed scour exposing the top of a spread footing or > six (6) feet at a pile supported footing or pile bent substructure unit; or
- Scour countermeasures have been damaged and are partially or completely ineffective.

IDOT Form BBS 2678 Scour Vulnerability Evaluation Report must be completed immediately and submitted to the BBS for entry into the ISIS. The Scour Vulnerability Item B.AP.03 must be coded “D”.

3.9.4.3 Non-Scour Critical/Susceptible Bridges

For bridges with Scour Vulnerability Item B.AP.03 coding of “A”, observed scour meeting one of the criteria defined in Section 3.9.4 must be reported to Agency PM and the Statewide PM for further direction.

The IDOT Form BBS 2678 Scour Vulnerability Evaluation Report must be completed immediately and submitted to the BBS for entry into the ISIS. The Scour Vulnerability Item B.AP.03 must be coded “D”.

3.9.5 Revising the Coding of Scour Vulnerability – Item B.AP.03

- In cases where Scour Vulnerability coding does not align with inspection information or implemented site conditions, a revision of the coding is required:In order to change the Scour Vulnerability Item B.AP.03 coding from “Critical” (“C”, “D”, or “E”) to “B”, the scour countermeasures must be engineered per HEC-23, installed according to specifications, and functioning as intended. The Bureau of Bridges & Structures must review and approve the scour countermeasure design and installation.
- In order to change the Scour Vulnerability Item B.AP.03 coding from “B” to “A”, the previously installed scour countermeasure (engineered per HEC-23) must have performed adequately through two documented design storm events with little to no damage or the engineered

countermeasures are in place functioning as intended for at least 15 years and documentation (channel cross sections, photographs, and/or documented flood events) is provided. The Statewide PM must concur.
- When scour is observed during an inspection which meets or exceeds the scour limits based in the current Scour Vulnerability Item B.AP.03 coding, the Agency PM must contact the Statewide PM immediately for further direction. Scour Vulnerability Item B.AP.03 should be changed to “D”.
- When the Substructure Condition Rating Item B.C.03 is coded ≤ 5 due to scour, the Scour Vulnerability Evaluation must be re-evaluated.

3.9.6 Scour Plan of Action (POA)

Bridges with Scour Vulnerability Item B.AP.03 coded “C”, “D” are considered Scour Critical. Those with Scour Vulnerability Item B.AP.03 coded “B” or “E” are considered Scour Susceptible. Each scour critical or scour susceptible bridge must have a Scour POA. Bridges with unknown foundations require a Scour POA. The Scour POA must be submitted to the BBS for review and approval.

IDOT Form BBS 2680 Scour Critical Bridge Plan of Action (POA) must be used, or the basis for developing a Scour POA. While IDOT Form BBS 2620 is applicable to most bridges, Major River Bridges or certain unique bridges may require additional activities, documentation, and requirements not found in the standard form.

The Scour POA must include, but is not limited to:

- Contact information for the Agency PM and backup contact.
- Description of bridge including each substructure element and foundation type.
- Description of substructure elements covered by the Scour POA.
- Summary of the coding of Scour Vulnerability Item B.AP.03 for each substructure unit.
- Information related to the type of countermeasures in place.
- Detailed guidance regarding the type and frequency of inspections required to ensure adequate monitoring at the bridge site including monitoring after BridgeWatch® notifications.
- Identification of fixed scour monitoring devices or reference points installed at the site.
- Reference data for use by inspection personnel when assessing site conditions.
- Guidance for reporting site deficiencies.

- Guidance for initiating or recommending bridge closure. The following are examples of when the bridge / roadway shall be closed; other site-specific items will be included in the POA:
  - Bridges experiencing pressure flow which is water to the bottom of the superstructure.
  - Bridge approach roadways overtopping.
- Detour route information.
- Guidance for reopening the roadway.
  - Information related to proposed engineered scour countermeasures to mitigate existing or potential scour concerns.

The Scour POA must be maintained within the Bridge File and uploaded to the ISIS. The Scour POA must be updated after changes in conditions are observed, changes in contact information, changes to Agency PM, or other reason. At a minimum, the Scour POA must be updated at an interval not to exceed sixty (60) months or as dictated by findings from a Scour Monitoring Inspection.

Channel cross-sections must be taken, at a minimum, along the upstream and downstream fascia after each rainfall event as defined by a BridgeWatch® "Alert" for:

- Scour Critical Bridges (Scour Vulnerability Item B.AP.03 coded "C", "D")
- Scour Susceptible Bridges (Scour Vulnerability Item B.AP.03 coded "B" or "E")
- Bridges with Scour Vulnerability Item B.AP.03 coded "A" requiring an Underwater Inspection

Additional cross-sections may be taken as needed or as required in the Scour POA.

3.9.7 Bridge Scour Monitoring System - BridgeWatch®

IDOT utilizes a third-party bridge scour monitoring system to monitor scour critical bridges, scour susceptible bridges, and select bridges that are stable for scour. The system notifies the agency having maintenance/inspection responsibility when a forecasted rainfall event meets or exceeds certain thresholds. BridgeWatch® monitors the National Oceanic and Atmospheric Administration (NOAA); National Weather Service (NWS); NEXRAD rainfall accumulation products, and the United States Geological Survey (USGS) network of stream flow gages to determine when Warnings and/or Alerts are necessary.

Bridges with Scour Vulnerability Item B.AP.03 coded B, C, D, E, and U must be monitored by BridgeWatch®. Bridges with Scour Vulnerability Item B.AP.03 coded A and requiring underwater inspection must be monitor by BridgeWatch®.A Warning is provided as advance notification of a potential Alert, and no immediate action is required. An Alert is a notification to users that a scour critical or scour susceptible bridge has experienced a forecasted rainfall event exceeding the thresholds in Table 3.9.7-1 and the Scour POA. The bridge must be inspected in accordance with the Scour POA to determine if the rainfall event has adversely impacted the structure.

Table 3.9.7-1 details the Warning and Alert rainfall events for the various Scour Vulnerability Item B.AP.03 coding levels and is based on a specific rainfall event having occurred.

Scour Vulnerability Item B.AP.03	Rainfall Event			
	10-Year	25-Year	50-Year	100-Year
B, C, D, E, or U	Warning	Alert	Alert	Alert
A	---	Warning	Alert	Alert

Table 3.9.7-1: BridgeWatch® Scour Monitoring Schedule

BridgeWatch® Warnings and Alerts are based on the Scour Vulnerability Item B.AP.03 coding for the bridge and the corresponding rainfall event. If a Warning or Alert threshold is exceeded, notification is sent via e-mail and text message to cell phone numbers provided by the users. Upon notification, a Certified NBIS Program Manager or Team Leader will assess the current condition of the bridge and determine if any immediate action is required. The following information will then be documented for the site conditions:

- Date
- Time
- Water level
- Bridge condition
- Debris accumulation
- Approach roadway site condition
- Photographs

The information must be entered into the BridgeWatch® System to clear the Warning/Alert. Each Agency PM is responsible for determining who receives notifications when a Warning and Alert occurs.

The Bridge Management & Inspection Unit maintains and updates users' contact information in BridgeWatch®. The Agency PM should notify the Bridge Management & Inspection Unit when critical information changes to ensure up-to-date records in BridgeWatch®. The Bridge Management & Inspection Unit must be contacted to add, modify, and delete the user contact information.

Table 3.9.7-2 details the required response time to inspect after the BridgeWatch® Alert for various codings of Scour Vulnerability Item B.AP.03.

Scour Vulnerability Item B.AP.03	Response Time to BridgeWatch® Alert (Hours)
C, D, or U	4
E	24
B	48
A	96

Table 3.9.7-2: BridgeWatch® Alert Response Time

If a given bridge experiences multiple storm events and does not produce scour concerns, the specified storm event triggering a warning/alert can be modified with approval of the Statewide PM.

If it is determined BridgeWatch® has not adequately estimated the predicted rainfall event or a stream gage has generated data inconsistent with observed or actual rainfall for a given rainfall event, the Statewide PM must be contacted in order to calibrate the BridgeWatch® or to contact USGS to verify the stream gage is functioning properly.

3.9.8 Countermeasures

FHWA *Bridge Scour and Stream Instability Countermeasures* (HEC 23 (Vol 1 / Vol 2)) is required for the selection and design of scour countermeasures. This publication provides the following definition:

> Countermeasure: Measure intended to prevent, delay or reduce the severity of hydraulic problems.

Since monitoring is included within the definition of countermeasures, a properly developed and implemented Scour POA may be considered a countermeasure, but Scour Vulnerability Item B.AP.03 cannot be raised to “B” unless there are engineered scour countermeasures installed and functioning as intended.

When considering scour countermeasures and the effect on the coding of Scour Vulnerability Item B.AP.03, only those utilizing HEC-23 to select and design the countermeasures will be considered. Only engineered scour countermeasures installed per the specifications and functioning as intended are considered to have a positive effect when coding Scour Vulnerability Item B.AP.03.

3.10 Inspection Safety

During bridge inspection, the safety of the inspection team and traveling public is of paramount importance. All members of the bridge inspection team must be familiar with the safety policies and guidelines set forth by the Occupational Safety and Health Administration (OSHA) and the agency having maintenance responsibility. The FHWA *Bridge Inspector's Reference Manual* (BIRM) is recommended for bridge inspection safety precautions.

IDOT safety policies and rules are included in Departmental Order 5-1: Employee Safety Code. All IDOT personnel engaged in bridge inspection activities must read and be familiar with the Employee Safety Code, paying particular attention to Chapters 1 & 8.

3.10.1 Bridge Inspection Safety Training

All bridge inspection personnel must complete the appropriate training based on the type of inspection to be performed and be familiar with and follow applicable OSHA, Illinois Department of Labor, and other Federal, State, and local laws pertinent to bridge inspection and roadway safety. All personnel must consult the safety policies of the entity having maintenance responsibility before conducting any bridge inspection activities.

3.10.1.1 Personal Protection Equipment (PPE)

All bridge inspection personnel must be equipped with the appropriate personal protection equipment based on the inspection type. Refer to the BIRM for guidance.

3.11 Quality Control and Quality Assurance

3.11.1 General

The NBIS (23 CFR 650.313(p)) states quality control and quality assurance must:

- Assure systematic Quality Control (QC) and Quality Assurance (QA) procedures identified in Section 3.11.2 - Documents Included by Reference (23 CFR 650.317) are used to maintain a high degree of accuracy and consistency in the inspection program.
- Document the extent, interval, and responsible party for the review of inspection teams in the field, inspection reports, NBI data, and computations, including scour appraisal and load ratings. QC and QA reviews are to be performed by personnel other than the individual who completed the original report or calculations.
- Perform QC and QA reviews and document the results of the QC and QA process, including the tracking and completion of actions identified in the procedures.
- QC findings will be documented in ISIS as part of the inspection report approval process. Inspection reports shall not be approved unless all QC findings are resolved.
- QA findings are documented in the annual QA reports as described in Section 3.11.3.2. Findings from QA reviews performed by IDOT Districts and LPAs shall be documented on IDOT Form BBS 2790 Bridge Inspection Procedures Review.
- QA reviews shall not be closed out until all findings have been addressed.

In conjunction with these statements regarding QC/QA, the NBIS provides the following definitions:

> Quality Control: Procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level.
>
> Quality Assurance: The use of sampling and other measures to assure the adequacy of Quality Control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.

Established and documented Quality Control and Quality Assurance (QC/QA) procedures are essential for ensuring the Illinois Bridge Inspection Program is compliant with the NBIS regulations. In addition, IDOT's QC/QA procedures ensure bridge inspections performed in Illinois are done in an appropriate, consistent, and uniform manner ensuring bridge inspection staff are adequately trained and experienced to readily identify conditions adversely affecting the safety of the traveling public. Through the QC/QA procedures, the agency having maintenance responsibility has the ability to obtain accurate bridge inspection information required for determining load capacity and bridge maintenance, preservation, repair, rehabilitation and replacement needs.

3.11.2 Quality Control (QC)

The Quality Control (QC) procedures established by IDOT are to define, monitor, and document the certification and performance of personnel responsible for the management of inspection programs, the performance of field inspections, inspection procedures, and bridge load rating. IDOT has QC procedures in place for the following elements to ensure compliance with the NBIS.

3.11.2.1 Bridge Inspection Organization Responsibilities (23 CFR 650.307)

3.11.2.1.1 Bridge Inspection Organization

NBIS Reference: 23 CFR 650.307 – Bridge inspection organization responsibilities

Compliance Criteria

- An organization is in place to perform or cause to be performed the proper inspection and evaluation of all highway bridges that are fully or partially located within the State's boundaries, except those owned by Federal agencies or Tribal governments.
- Organization roles and functions are clearly defined, documented, and carried out for each of the following aspects of the NBIS:
    - statewide bridge inspection policies and procedures
    - registry of nationally certified bridge inspectors
    - criteria for inspection intervals for all inspection types (Required by June 6, 2024)
    - roles and responsibilities of personnel
    - bridge inspection reports and files
    - quality control and quality assurance
    - bridge inventory data
    - load rating
    - posting and other restrictions
    - managing activities and corrective actions taken in response to critical findings
    - scour appraisals and scour plans of action

  - other requirements of these standards
- For those functions that are delegated, the State has documented the roles and functions of all individuals, agencies, and other entities involved.
- The State has the necessary authority to take actions needed to ensure NBIS compliance.
- A Program Manager (PM) is assigned the responsibility for the NBIS.
- Each bridge that crosses a border between a State, Federal agency, or Tribal government, has a joint written agreement that determines the NBIS responsibilities of each agency, including the designated lead State for reporting NBI data.

3.11.2.2 Qualifications of Personnel (23 CFR 650.309)

3.11.2.2.1 Program Manager

NBIS Reference: 23 CFR 650.309(a) – Program manager
23 CFR 650.313(p) – Quality control and quality assurance

Compliance Criteria

- The Program Manager (PM) has the following qualifications:
  - Is a registered Professional Engineer or has 10 years of bridge inspection experience.
  - Has successfully completed FHWA-approved comprehensive bridge inspection training.
  - Has completed periodic bridge inspection refresher training according to State policy. (Completion of a cumulative total of 18 hours of FHWA-approved bridge inspection refresher training over each 60-month period.)
- The PM maintains documentation supporting the satisfaction of the above requirements.

3.11.2.2.2 Team Leader(s)

NBIS Reference: 23 CFR 650.307(e)(2) – Registry of nationally certified bridge inspectors
23 CFR 650.309(b)(c) – Team leader
23 CFR 650.313(p) – Quality control and quality assurance

Compliance Criteria

- Each Team Leader (TL) has at least one of the following qualifications:
  - PE registration (and six months of bridge inspection experience).
  - Five years of bridge inspection experience.

  - Bachelor's degree in engineering from ABET-accredited college or university, and a passing score on the Fundamentals of Engineering Exam, and two years of bridge inspection experience.
  - Associate's degree in engineering or engineering technology from ABET-accredited college or university, and four years of bridge inspection experience.
- In addition to the above qualifications, each TL has all the following training:
  - Successful completion of FHWA-approved comprehensive bridge inspection training.
  - Completion of periodic bridge inspection refresher training according to State policy. (Completion of a cumulative total of 18 hours of FHWA-approved bridge inspection refresher training over each 60-month period.)
- A registry of nationally certified bridge inspectors (NCBI) that are performing the duties of TL in the State is maintained.
- In addition to the above qualifications, each TL performing NSTM inspections has successfully completed FHWA-approved NSTM inspection training.
- Each TL has provided documentation to the Program Manager that the above requirements (as applicable) are satisfied.

3.11.2.2.3 Underwater Bridge Inspection Diver

NBIS Reference: 23 CFR 650.309(e) – Underwater Bridge Inspection Diver

<u>Compliance Criteria</u>

- Underwater bridge inspection divers have successfully completed one of the following training courses:
  - FHWA-approved underwater bridge inspection training
  - FHWA-approved comprehensive bridge inspection training (if completed prior to June 6, 2022)

3.11.2.2.4 Damage, Special, and Service Inspection Types

NBIS Reference: 23 CFR 650.309(f) – Damage and special inspection types
23 CFR 650.309(g) – Service inspection type

<u>Compliance Criteria</u>

- State has established documented personnel qualifications for:
  - Damage and Special inspection types

  - Service inspection type, when routine inspection intervals exceed 48 months
- Personnel performing these inspection types meet the State's qualifications.

3.11.2.3 Inspection Interval (23 CFR 650.311)

3.11.2.3.1 Routine

NBIS Reference: 23 CFR 650.311(a) – Routine inspection intervals
23 CFR 650.311(e) – Bridge inspection interval tolerance
23 CFR 650.313(b) – Initial inspection

Compliance Criteria

- Each bridge designated for inspection at regular intervals of 24 months is inspected within that interval, plus the acceptable tolerance of up to 3 months.
- Each bridge designated for inspection at extended intervals of up to 48 months is inspected within that interval, plus the acceptable tolerance of up to 3 months, and adheres to FHWA-approved extended interval criteria.
- Exceptions to the inspection interval tolerance due to rare and unusual circumstances are approved by FHWA in advance of the inspection due date plus acceptable tolerance.
- Each bridge designated for inspection at reduced intervals of less than 24 months is inspected within that interval, plus the acceptable tolerance of up to 2 months.
- An initial inspection is performed for each new, replaced, rehabilitated, and temporary bridge within 3 months of the bridge opening to traffic.

3.11.2.3.2 Underwater

NBIS Reference: 23 CFR 650.311(b) – Underwater inspection intervals
23 CFR 650.311(e) – Bridge inspection interval tolerance

Compliance Criteria

- Each bridge designated for underwater inspection at regular intervals not to exceed (NTE) 60 months is inspected within that interval, plus the acceptable tolerance of up to 3 months.
- Exceptions to the inspection interval tolerance due to rare and unusual circumstances are approved by FHWA in advance of the inspection due date plus acceptable tolerance.
- Each bridge designated for underwater inspection at reduced intervals of 24 to 59 months is inspected within that interval, plus the acceptable tolerance of up to 3 months.

- Each bridge designated for underwater inspection at reduced intervals of less than 24 months is inspected within that interval, plus the acceptable tolerance of up to 2 months.

3.11.2.3.3 Nonredundant Steel Tension Member (NSTM)

NBIS Reference: 23 CFR 650.311(c) – NSTM inspection intervals
23 CFR 650.311(e) – Bridge inspection interval tolerance
23 CFR 650.311(f) – Next Inspection

<u>Compliance Criteria</u>

- Each bridge with NSTMs designated for inspection at regular intervals not to exceed (NTE) 24 months is inspected within that interval, plus the acceptable tolerance of up to 3 months.
- Each bridge with NSTMs designated for inspection at extended intervals of up to 48 months is inspected within that interval, plus the acceptable tolerance of up to 3 months, and adheres to approved extended interval criteria.
- Exceptions to the inspection interval tolerance due to rare and unusual circumstances are approved by FHWA in advance of the inspection due date plus the acceptable tolerance.
- Each bridge with NSTMs designated for inspection at reduced intervals of less than 24 months is inspected within that interval, plus the acceptable tolerance of up to 2 months.

3.11.2.3.4 Special, In-Depth, and Service

NBIS Reference: 23 CFR 650.311(a)(3) – Service inspection
23 CFR 650.311(d) – In-depth and special inspections

<u>Compliance Criteria</u>

- Each bridge designated for a Special, In-Depth, and/or Service inspection is inspected within the coded interval, plus the acceptable tolerance.
- Exceptions to the inspection interval tolerance due to rare and unusual circumstances are approved by FHWA in advance of the inspection due date plus acceptable tolerance.

3.11.2.3.5 Interval Criteria

NBIS Reference: 23 CFR 650.311 – Inspection interval

Compliance Criteria

- Criteria are established to determine level of inspection and interval for all the following inspection types where appropriate:
  - Routine inspections – for less than 24-month intervals
  - Underwater inspections – for less than 60-month intervals
  - NSTM inspections – for less than 24-month intervals
  - Damage inspections
  - In-depth inspections
  - Special inspections
- All coded bridge inspection intervals as reported to the NBI are consistent with established criteria.
- For each bridge that received an in-depth or special inspection, the interval and level of inspection are consistent with established criteria.
- Each bridge coded in the NBI for reduced interval less than 24 months for routine or NSTM, or less than 60 months for underwater, is inspected within the coded interval plus the acceptable tolerance.

3.11.2.4 Inspection Procedures (23 CFR 650.313)

3.11.2.4.1 Quality Inspections

NBIS Reference: 23 CFR 650.313(a) through (j) – Inspection procedures

Compliance Criteria

- Each bridge is inspected to determine condition, identify deficiencies, and document results in an inspection report in accordance with the MBE, as measured by the following criteria:
  - Deck, Superstructure, Substructure, and/or Culvert component condition codes have documented justification and are accurate within the acceptable tolerance,
  - All notable bridge deficiencies are identified and include supporting photographic documentation. Notable bridge deficiencies are those leading to an NBI component condition rating of 5 or less, or those requiring some kind of immediate action.
  - Condition codes are supported by narrative that appropriately justifies and documents the component/element condition rating.
  - All notable deficiencies are quantified in element CS3 or CS4 and properly documented. Non-NHS bridges are not required to use national elements but must document to the MBE standard.

  - Inspection reports document the use of special equipment or techniques, and/or traffic control as necessary for inspections in circumstances where their use provided the only practical means of accessing and/or determining the component/element condition.
- A qualified team leader is at the bridge and actively participates at all times during each initial, routine, in- depth, NSTM, underwater, and special inspection.

3.11.2.4.2 Load Rating

NBIS Reference: 23 CFR 650.313(k) – Load rating

<u>Compliance Criteria</u>

- All bridges are rated for their safe load carrying capacity in accordance with the incorporated articles of Sections 6 and 8 of the MBE, for all legal vehicles and State routine permit loads.
- Load ratings were completed within a reasonable timeframe (no later than 3 months) after the initial inspection and if a change in condition that warranted re-rating was identified.
- Load ratings are performed by, or under the direct supervision of, a licensed structural engineer.
- All bridges are analyzed for routine and special permit loads which cross those bridges.
- Procedures for completion of new and updated bridge load ratings are developed and fully documented.

3.11.2.4.3 Post or Restrict

NBIS Reference: 23 CFR 650.313(l) – Load posting
23 CFR 650.313(m) – Closed bridges

<u>Compliance Criteria</u>

- Bridges are posted or restricted in accordance with the incorporated articles of Section 6 of the MBE or in accordance with State law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor.
- Posting deficiencies are resolved within 30 days. Posting deficiencies are resolved within the timeframe established by the procedures but no later than 30 days.
- Any bridge with a gross live load capacity less than three tons is closed.
- Procedures to ensure that bridges are posted in a timely manner, but no more than 30 days, have been developed and fully documented.
- Criteria for closing bridges have been developed and fully documented.

- All bridges that meet the criteria for closure are appropriately closed.

3.11.2.4.4 Bridge Files

NBIS Reference: 23 CFR 650.313(n) – Bridge files

Compliance Criteria

- Bridge files are prepared and maintained in accordance with Section 2.2 of the MBE. The bridge file contains the following specific bridge information per the MBE:
  - General file information
  - Field inspection information
  - Critical findings and actions taken
  - Waterway information – channel cross-sections, soundings, stream profiles
  - Significant correspondence
  - Bridge-specific inspection procedures or requirements (i.e., NSTM, underwater, in-depth, complex feature) and reference to general inspection procedures where they are used
  - Load rating documentation, including load testing results
  - Posting documentation
  - Scour appraisal
  - Scour Plan of Action (POA) for scour critical bridges and those with unknown foundations

3.11.2.4.5 Nonredundant Steel Tension Member (NSTM)

NBIS Reference: 23 CFR 650.313(f) – NSTM inspection
23 CFR 650.313(g) – NSTM, underwater, in-depth, and complex feature inspections

Compliance Criteria

- All bridges with NSTMs have been identified.
- Each bridge with NSTMs has documented inspection procedures in the bridge file developed in accordance with Section 4.2 of the MBE. The procedures, whether general or bridge specific:
  - Require a qualified inspection team leader with NSTM credentials be present during the entire NSTM inspection.
  - Identify for each bridge the location of each NSTM and require each NSTM location to be also identified in the inspection report.

- Require hands-on inspection of the entire NSTM or member component, plus identify additional specialized inspection methods and procedures needed to fully ascertain condition for any of the members, including NDE and NDT.
- Specify the interval between inspections.

- Inspections of NSTMs are performed in accordance with the procedures developed.
- All NBI data related to NSTMs is properly reported.
- A State that chooses to demonstrate a member is not considered an NSTM due to system or internal redundancy has developed appropriate formal written policy and procedures that have been approved by FHWA and include:
  - Identification of the nationally recognized method used to determine system or internal redundancy.
  - Baseline condition of the bridge(s) to which the policy is being applied.
  - Description of design and construction details on the member(s) that may affect the system or internal redundancy.
  - Routine inspection requirements for bridges with system or internally redundant members.
  - Special inspection requirements for the members with system or internal redundancy.
  - Evaluation criteria for when members should be reviewed to ensure they still have system and internal redundancy.
- Bridge members demonstrated not to require NSTM inspection due to system or internal redundancy are inspected in accordance with the approved procedures for routine and special inspections.

3.11.2.4.6 Underwater

NBIS Reference: 23 CFR 650.313(e) – Underwater inspection
23 CFR 650.313(g) – NSTM, underwater, in-depth, and complex feature inspections

Compliance Criteria

- Each bridge requiring an underwater inspection has documented inspection procedures developed in accordance with Section 4.2 of the MBE. The procedures, whether general or bridge specific:
  - Require a qualified inspection team leader be present during the entire underwater inspection.
  - Ensure an appropriate level of pre-inspection preparation occurs, including review of the bridge inspection file.

  - Identify for each bridge the specific underwater elements to be inspected, including any underwater scour countermeasures present.
  - Require that the underwater inspection:
    - Is performed in accordance with methods and techniques contained in the MBE, as well as State requirements.
    - Evaluates the waterway under and adjacent to the bridge, taking into account its unique characteristics.
  - Specify the interval between inspections.
  - Ensure the underwater inspection follows the scour POA requirements as it pertains to directions for underwater inspections.
- Underwater inspections are performed in accordance with the procedures developed.
- Underwater inspections are recorded as such in the bridge inventory, with NBI items updated accordingly.

3.11.2.4.7 Scour

NBIS Reference: 23 CFR 650.313(o) – Scour

<u>Compliance Criteria</u>

- All bridges over water have a scour appraisal and the process and results are documented in the bridge file.
- Scour appraisal procedures are consistent with HEC-18 and HEC-20.
- All bridges determined to be scour critical or with unknown foundations have a scour plan of action (POA) which has been prepared and documented consistent with HEC-18 and HEC-23 for deployment of scour countermeasures for known and potential deficiencies, and which addresses safety concerns. The POA addresses a schedule for repairing or installing physical and/or hydraulic scour countermeasures, and/or the use of monitoring as a scour countermeasure.
- Scour countermeasures identified in the POA are being deployed for each bridge determined to be scour critical or having unknown foundations.
- For bridges with changed scour conditions, re-appraisals have been performed.

3.11.2.4.8 Complex Feature

NBIS Reference: 23 CFR 650.313(g) – NSTM, underwater, in-depth, and complex feature inspections

Compliance Criteria

- All bridges with complex features have been identified and are so coded in the NBI.
- Each bridge with complex features has documented inspection procedures in the bridge file developed in accordance with Section 4.2 of the MBE. Procedures are bridge specific and:
    - Require a qualified inspection team leader be present during the complex feature inspection.
    - Ensure an appropriate level of pre-inspection preparation occurs, including review of the bridge inspection file.
    - Identify for each bridge the specific complex feature(s) to be inspected.
    - Identify any additional qualifications, or specialized training or experience, required of the inspection team leader or specific bridge inspectors (i.e., mechanical or electrical systems inspectors) for the complex feature being inspected.
    - Identify additional specialized inspection methods and procedures appropriate for the specific complex feature.
    - Specify the interval between inspections.
- Inspections of complex features are performed in accordance with the procedures developed.

3.11.2.4.9 In-Depth

NBIS Reference: 23 CFR 650.313(g) – NSTM, underwater, in-depth, and complex feature inspections

Compliance Criteria

- All bridges requiring in-depth inspections have been identified and are so coded in the NBI.
- Each bridge requiring in-depth inspections has documented inspection procedures in the bridge file developed in accordance with Section 4.2 of the MBE. The procedures, whether general or specific:
    - Require a qualified inspection team leader be present during the entire in-depth inspection.
    - Identify each bridge member requiring an in-depth inspection and document each location.
    - Detail advanced inspection methods or techniques to be used to fully ascertain the existence of or extent of a deficiency not readily detectable using routine inspection procedures.
    - Specify the interval between inspections.
    - Specify any additional qualifications, or specialized training, required of the inspection team leader or specific bridge inspectors (divers, riggers, NDE certified, etc.).
    - Specify needed special access equipment or traffic control.

- In-depth inspections are performed in accordance with the documented procedures.
- Inspection reports document actual members inspected, and inspection methods utilized during the inspection. Findings are documented to the level of detail needed, which may be significantly more detailed than a normal routine inspection.

3.11.2.4.10 Inspection Procedures – Quality Control (QC) and Quality Assurance (QA)

NBIS Reference: 23 CFR 650.307(e)(6) – Quality control and quality assurance activities
23 CFR 650.313(p) – Quality control and quality assurance

Compliance Criteria

- Systematic, documented QC and QA procedures identified in Section 1.4 of the MBE are used to maintain a high degree of accuracy and consistency in the inspection program. These include:
  - Periodic field review of inspection teams
  - Periodic bridge inspection refresher training for program managers and team leaders
  - QC/QA measures for inventory data
  - Independent review of inspection reports and computations
- The extent, interval, and responsible parties for the QC and QA activities are documented.
- QA measures include the overall review of the inspection and rating program – including inspection teams in the field, inspection reports, NBI data, and computations including scour appraisals and load ratings – to ascertain that the results meet or exceed the standards established by the PM.
- QC and QA reviews are performed by personnel other than the individual who completed the original report or calculations.
- The findings of the QC and QA reviews are addressed.
- The procedures for QC and QA include organizational tracking to verify completion of actions identified in the procedures.
- The results, findings, corrective action recommendations, and resulting corrective actions that address the findings from QC and QA reviews are documented. (Assessed in PY25)

3.11.2.4.11 Critical Findings

NBIS Reference: 23 CFR 650.313(q) – Critical findings

Compliance Criteria

- Procedures are in place and documented to address critical findings in a timely manner:
  - The procedures define critical findings considering the location and redundancy of the member affected and the extent and consequence of a deficiency.
  - The procedures provide timeframes for addressing critical findings.
- FHWA is notified of all critical findings and the actions taken to resolve them:
  - FHWA is notified within 24 hours of the discovery of all critical findings on the NHS that meet either of these criteria:
    - Full or partial closure of any bridge
    - An NSTM to be rated in serious or worse condition
  - FHWA is notified monthly, in a written report, of all critical findings and the actions planned, undertaken, or completed to resolve the critical findings.
  - The monthly report contains the following information:
    - Owner
    - NBI Structure Number
    - Date of finding
    - Description and photos (if available) of critical finding
    - Description of completed, temporary and/or planned corrective actions to address critical finding
    - Status of corrective actions: Active/Completed
    - Estimated date of completion if corrective actions are active
    - Date of completion if corrective actions are completed

3.11.2.5 Inventory (23 CFR 650.315)

3.11.2.5.1 Bridge Data Quality

NBIS Reference: 23 CFR 650.303 - Applicability
23 CFR 650.305 - Definitions
23 CFR 650.307(e)(7) – Bridge inspection organization responsibilities
23 CFR 650.315(a) - Inventory
23 CFR 650.317 – Incorporation by reference

Compliance Criteria

- An inventory of all bridges subject to the NBIS is prepared and maintained by the State. The inventory includes:
  - Private bridges that are connected to a public road on both ends of the bridge.
  - Temporary bridges open to traffic greater than 24 months.
  - Bridges under construction with portions open to traffic.
- Inventory (NBI) data are collected, updated, and retained by the State, and reported in accordance with the Coding Guide.
- Inventory data include element level bridge inspection data for bridges on the NHS, reported in accordance with the SNBI.

3.11.2.5.2 Timely Updating of Data

NBIS Reference: 23 CFR 650.315 – Inventory

Compliance Criteria

- Inventory data, as defined in 23 CFR 650.305, are updated by the State and submitted to FHWA on an annual basis or whenever requested, using FHWA established procedures.
- Changes to the NBI data are entered into the State's inventory within 3 months for the following events:
  - For all inspection types (initial, routine, in-depth, NSTM, underwater, complex features, and special inspections), within 3 months after the field portion of the inspection is completed.
  - For modifications to existing bridges that alter previously recorded NBI data and for newly constructed bridges, within 3 months after the bridge is opened to traffic.
  - For changes in load restriction or closure status of the bridge, within 3 months after the change is implemented.
- A process has been established and documented that ensures and can verify the time constraint requirements are fulfilled.

3.11.2.6 Review of Bridge Inspection Reports

To ensure the completeness and quality of a Team Leader's inspection reports, all reports must be reviewed and approved by the Agency PM. At a minimum, the review must verify the Condition Ratings are appropriate, including notes with sufficient detail (defect type, size, and location) to justify the condition ratings, the documentation is complete and accurate, and the number and content of photographs are sufficient.

Agency PMs should monitor the number of bridges inspected per day by each inspector to ensure that each inspection is of high quality. The Bridge Management & Inspection Unit monitors the number of bridges inspected per day during the annual QA reviews and will discuss with each agency. IDOT considers four (4) to eight (8) inspections per day to be realistic. There may be instances where ten (10) or more can be done, but this should not be the norm.

3.11.2.7 Inspector Performance

Agency PMs must conduct in-depth reviews of the field procedures of all Certified Illinois NBIS Team Leaders under their supervision to ensure inspections are being performed in accordance with the NBIS and are done in an appropriate, consistent, and uniform manner. If an agency only has an NBIS Program Manager, it is required by the NBIS regulation to coordinate with other adjacent agencies or a consultant in order to have proper QC of each agency's inspections.

At least once every twenty-four (24) months, Agency PMs are required to accompany each Certified Illinois NBIS Team Leader to observe the performance of NBIS bridge inspections on at least three (3) bridges, preferably over the course of a thirty (30)-day period. The Agency PMs are required to complete IDOT Form BBS 2790 Bridge Inspection Procedures Review for each Illinois Certified NBIS Team Leader. The forms must be kept in the inspector's files. If the Agency PM or Team Leader desires, the forms can be sent to the BBS at DOT.BBS.BridgeMgmt@Illinois.gov and kept with the inspector's records.

Due to the complexity and duration of the major bridge inspections the Statewide PM, the Bridge Inspection Group Leader, or the Bridge Inspection Group Technician must accompany each Illinois Certified NBIS Team Leader to observe the performance of NBIS bridge inspections on at least two (2) bridges over a 12-month period.

Agency PMs must choose inspections of different bridge types, verifying major component Condition Ratings, and include an Element Level, Underwater, NSTM, and Special Inspection, if applicable. The Agency PM must increase the number of bridge inspections monitored, or reduce the period of time between field reviews, as needed to address concerns with Certified Illinois NBIS Team Leader performance. If the Agency PM determines a Certified NBIS Team Leader is not capable of performing inspections at an appropriate level to ensure adequate inspection procedures, the individual must be reported to the Statewide PM for final resolution up to and including revocation of certification, and the individual should not be used as a Certified Illinois NBIS Team Leader in the interim.

3.11.2.8 Illinois Certified NBIS Program Manager/Team Leader - Supporting Documentation

The Bridge Management and Inspection Unit maintains the official records of all Illinois Certified NBIS Program Managers and Team Leaders in the State of Illinois. This includes any records of performance deficiencies for Illinois Certified NBIS Program Managers and Team Leaders.

Agency Program Managers and Team Leaders must also maintain a file containing certifications and all supporting documents.

In addition, the file must contain records of the Bridge Inspection Performance Reviews as noted in Section 3.9.3.2. This documentation should be located in a central location and be readily accessible by the Agency PM.

3.11.2.9 Review of Bridge Files

The NBIS requires the preparation and maintenance of Bridge Files in accordance with Section 2.2 of the MBE. A Bridge File is a cumulative record with information representing a complete history of a bridge, including details of any damage and all strengthening and repairs made to the bridge. The bridge file should report data on the capacity of the bridge. The computations substantiating reduced load-carrying capacity, if applicable is kept at the Central BBS Structural Ratings and Permitting Unit. The Bridge File shall contain, as applicable:

- NBI data
- Design plans and specifications
- Hydraulic report
- Geotechnical report
- Construction records
- As-built plans
- Maintenance and repair records
- All NBIS bridge inspection records
- Bridge specific inspection procedures as applicable
- All channel cross sections, if applicable
- Scour Vulnerability Report
- Border Bridge Agreements
- Scour POA, as applicable
- Load rating/load posting information

- NSTMs, if applicable
- Critical Findings, as applicable
- Photographs/ sketches
- Bridge File Checklist
- Any other pertinent information.

Bridge Owners shall maintain a complete, accurate, and current record of each bridge under their jurisdiction. Complete information, in good usable form, is vital to management of the Illinois bridge inventory and for NBIS compliance. Furthermore, such information allows the owner to properly program bridges for maintenance, preservation, repair, rehabilitation, or replacement.

A separate Bridge File must be maintained for each Structure Number in the bridge inventory. IDOT Form BFC Bridge File Checklist must be completed, updated as required, and stored with each Bridge File. The Bridge Files for all bridges maintained by an Agency must be located in a central location, maintained, and readily accessible by the Agency PM and Certified NBIS Team Leader.

It is not practical or necessary to physically store all required documents in the Bridge File. However, the actual location of each item must be referenced on the Bridge File Checklist.

3.11.2.10 Inventory Data Verification

An extremely vital part of NBIS inspections is the verification and updating of the inventory data. When the Certified Illinois NBIS Program Managers and Team Leaders sign/date the inspection form, or submit/approve in the ISIS, they are indicating the bridge's inventory data has been verified and updated as necessary. Existing plans can be used for many of the static inventory items such as structure length, deck structure width, design load, etc. Inventory data changes are done by the following IDOT District Bureaus:

- IDOT-maintained bridges
  - Bureau of Operations – Bridge Maintenance

- Non-IDOT-maintained bridges
  - Bureau of Local Roads and Streets for LPAs
  - BBS for IDNR, ISTHA, other

For rehabilitated, replaced, and new bridges, IDOT Form BBS 3320 Bridge Inventory Report must be reviewed to verify consistency with the information provided on the as-built bridge plans or design plans.

3.11.2.11 Bridge Inspection Refresher Training

All Certified Illinois NBIS Program Managers and Team Leaders must successfully complete eighteen (18) hours of FHWA-approved and nationally recognized bridge inspection refresher training over the course of sixty (60) months.

The following classes satisfy the requirement for bridge inspection refresher training:

- NHI 130053 Bridge Inspection Refresher Training
- Illinois Bridge Inspection Refresher Training (When FHWA approved)

The content of the IDOT-developed IL Bridge Inspection Refresher Training will be reviewed and updated as necessary before each offering. The training is reviewed by the Bridge Management & Inspection Unit and the consultant under contract to deliver the training to determine if additional materials are necessary or new areas of emphasis have been identified related to the annual quality assurance review of bridge inventory data. The Bridge Management Group will annually update the IDOT Inspection Refresher Training to incorporate annual data quality findings identified during their quarterly SI&A data QA reviews. Annual IDOT Inspection Refresher Training updates will be coordinated, at a minimum, through the FHWA IL Division Bridge staff to ensure the content and implementation meets the intended requirements of the National Bridge Inspection Program oversight process.

There are currently no refresher training requirements for Nonredundant Steel Tension Member or Underwater inspections.

3.11.3 Quality Assurance (QA) – Bridge Inventory / Bridge Files / Inspection

QA measures are required to ensure established Quality Control (QC) procedures are effective and being adhered to in order to ensure bridge inspections are done in accordance with the NBIS. QA Reviews are performed annually to ensure the quality of bridge inspections and bridge load ratings. The Statewide PM conducts or oversees the QA Review and may delegate these duties as necessary.

The bridges reviewed vary in regard to Main Span Material Item B.SP.01 and Main Span Type Item B.SP.06. An effort is made to include bridges with Condition Ratings ranging between “8” (Very Good) to “2” (Critical) for Deck Condition Item B.C.01, Superstructure Condition Item B.C.02, Substructure Condition Item B.C.03, or Culvert Condition Item B.C.04. When applicable, the QA Review will include a bridge with Scour Vulnerability Item B.AP.03 coded ”C” (Scour Critical) or “B” (Scour Countermeasures). When applicable, the QA Review will include bridges requiring Underwater, NSTM, or Special Inspections.

The Agency PM must be present during the QA Review.

3.11.3.1 IDOT NBIS Reviews

IDOT performs annual NBIS QA Reviews on two (2) IDOT Districts and seventeen (17) County Highway Departments. Other non-IDOT agencies, such as IDNR and ISTHA, are also subject to IDOT QA Reviews. IDOT Districts have an NBIS QA Review every five (5) years whereas County Highway Departments are reviewed every six (6) years. The interval between NBIS QA Reviews may be reduced in response to agency personnel changes or to address issues identified by the BBS.

3.11.3.1.1 Bridge Inventory and File Review - Virtual

NBIS QA Reviews include, at a minimum, an extensive review of the agency’s Bridge Inventory, inspection personnel, inspection procedures, and the Bridge Files for seven (7) bridges. The agency being reviewed is contacted and instructed to upload all applicable files to the ISIS if not already done. IDOT reviews the Bridge Inventory for each inspection type and tabulates each, including numbers having various intervals, delinquencies in the last three (3) years, and how agency is progressing with NBIS Metric IP/PCA initiatives. The Bridge Files are reviewed for completeness and the two (2) most recent Routine Inspections along with the most recent of other inspection types associated with the sample. The inventory and files are checked for adherence to the NBIS. The final step is to have a virtual meeting with agency inspection personnel to review the findings, inquire about procedures, and ultimately determine compliance with all applicable metrics.

3.11.3.1.2 Bridge Inspection Review - Field Visit (at IDOT's Discretion)

Based on the findings of the inventory and Bridge File review, IDOT will decide if a field review of the sample is necessary. Personnel conducting the field review perform an independent inspection and compare the results to the agency's inspection reports. IDOT Form S-107 Master Structure Report may be utilized for reference by NBIS QA Review personnel during the field review of each bridge to verify various inventory data items and note any errors or omissions. Conditions affecting the load carrying capacity of major components will be noted during the NBIS QA Field Review and follow-up must occur to ensure the noted conditions have been taken into account in the load rating. The NBIS QA Review personnel may perform a Load Rating Inspection while on-site to document the deficiencies.

3.11.3.2 NBIS QA Review Report

A final NBIS QA Review Report noting commendable practices, areas needing correction, specific review findings, NBIS metric determination for applicable metrics, and actions required to address deficiencies noted during the review will be provided to the Agency PM for review and comment. Once review is completed and parties agree, the Statewide PM and Agency PM will sign and date.

IDOT will provide the agency with an electronic copy of each final NBIS QA Review Report. In addition, each report will be posted on the Bridge Management & Inspection Unit website.

IDOT will assemble a NBIS QA Review Summary Report summarizing the findings of each agency NBIS QA Review and will post it to the Bridge Management & Inspections Unit website.

3.12 Inspection of Non-NBIS Bridges/Culverts

3.12.1 General

IDOT is responsible for establishing policies and procedures to ensure the safety of bridges carrying public traffic, including those not subject to the NBIS. The NBIS requires bridges on public roadways, with an NBIS Bridge Length Item B.G.01 > 20.0 feet, to be included in the National Bridge Inventory. Bridges not meeting those conditions are "Non-NBIS Bridges."

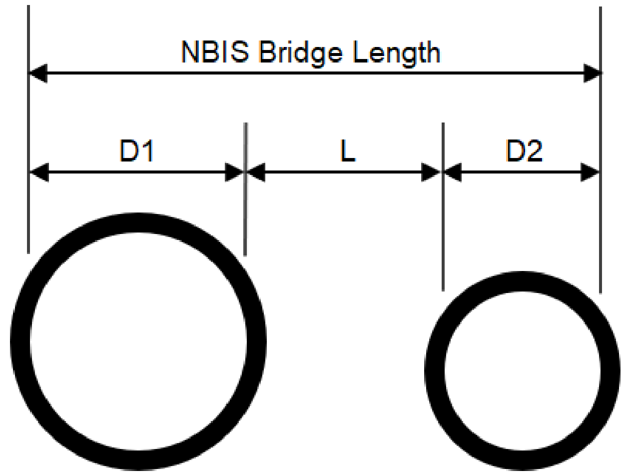
In order to track the condition of Non-NBIS Bridges which could affect the safety of the travelling public, SI&A information is maintained for select bridges in the ISIS. This section provides information related to the Non-NBIS Bridges to be tracked in the bridge inventory and inspected.

3.12.2 Small Bridge/Culvert Inspection Program

In the early 2000's, IDOT developed the Small Bridge Inspection Program (SBIP) to address the safety of bridges not meeting the NBIS Bridge Length Item B.G.01 requirement of the NBIS. IDOT defines a "Small Bridge" as:

> Small Bridge: A bridge on a public roadway with an NBIS Bridge Length Item B.G.01 ≥ 6.0 feet and ≤ 20.0 feet, including supports, erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads. This includes multiple pipes where the clear distance between openings is < ½ of the diameter of the smaller contiguous opening and at least one of the pipes has an inside diameter ≥ 60 inches.

Figure 3.12.2-1 provides guidance for measuring the NBIS Bridge Length for multiple-pipe culverts.



When determining the NBIS Bridge Length Item B.G.01 for multiple pipe installations, the value used for the distance between pipes cannot be greater than half of the smaller opening on either side (L ≤ ½ X D2).

Figure 3.12.2-1: NBIS Bridge Length Item B.G.01 for Multi-Pipe Culverts

The SBIP is only required for IDOT-maintained bridges/culverts carrying public traffic with an NBIS Bridge Length Item B.G.01 ≥ 6.0 feet and ≤ 20.0 feet. IDOT strongly recommends non-IDOT agencies should inventory and inspect their bridges/culverts with an NBIS Bridge Length – Item B.G.01 ≥ 6.0 feet and ≤ 20.0 feet, per this section, to ensure the safety of the traveling public.

3.12.2.1 Inventory and Appraisal

IDOT maintained bridges/culverts in the SBIP must be inventoried in the ISIS. The SIBI Manual must be used for guidance when entering SI&A information. The bridges/culverts in the SBIP must be inventoried in the same manner as bridges/culverts with an NBIS Bridge Length Item B.G.01 > 20.0 feet.

Records related to bridges in the SBIP must be maintained similar to those for bridges/culverts with NBIS Bridge Length Item B.G.01 > 20.0 feet and uploaded to the ISIS. The Bureau of Bridges & Structures maintains the official version of bridge plans for IDOT maintained bridges/culverts, including small bridges/culverts, and routinely updates the archive by obtaining plan information prior to the award of contract for the construction of new bridges. When a small bridge/culvert is constructed as part of a roadway project rather than a stand-alone bridge contract, the Bureau of Bridges & Structures relies on the Regions/Districts to identify the small bridges/culverts included in the roadway contract. Regions/Districts must review the contract plans for roadway projects to identify bridges/culverts meeting the definition of a small bridge, and information must be forwarded to the Bureau of Bridges & Structures for inclusion in bridge plan archives.

3.12.2.2 Routine Inspection of Small Bridges/Culverts

Routine Inspections must be performed for IDOT-maintained small bridges/culverts to ensure highway safety. Since many of the small bridges/culverts are culverts which are typically buried and protected from direct contact loading and de-icing agents, extended inspection intervals may be appropriate for small bridges/culverts with Culvert Condition Rating Item B.C.04 ≥ 6 - Satisfactory. Small bridges/culverts not classified as culverts have inspection intervals consistent with bridges having an AASHTO Bridge Length Item B.G.01 > 20.0 feet. Inspection intervals for small bridges are provided in Tables 3.12.2.2-1 and 3.12.2.2-2.

A Buried Structure is defined as follows:

> A culvert having ≥ 2.0 feet of fill including pavement and subgrade material over the culvert located within the travel way and shoulders of the roadway.

Main Structure Type Item B.SP.06 coded 19 – Culverts or 91 – Culvert - Rigid Frame and Structure Fill Depth Item B.G.IL.01 coded: ≥ 2.0 feet	
Culvert Condition Item B.C.04	Maximum Inspection Interval (Months)
7 - Good, 8 – Very Good. or 9 - New	72
5 - Fair or 6 - Satisfactory	48
4 - Poor	24
3 Serious	12
2 Critical	12*

Table 3.12.2.2-1: Routine Inspection Interval for Buried Structures

** Special Inspection may be required at a more frequent interval.*

Main Structure Type Item B.SP.06 not coded 19 - Culvert or 91 – Culvert – Rigid Frame OR Main Structure Type Item B.SP.06 not coded 19 - Culvert or 91 – Culvert – Rigid Frame and Structure Fill Depth Item B.G.IL.01 coded < 2.0-Feet	
Lowest Condition Rating of: Superstructure Condition Item B.C.02 Substructure Condition Item B.C.03 or Culvert Condition Item B.C.04	Maximum Inspection Interval (Months)
5 - Fair, 6 - Satisfactory, 7 - Good, 8 – Very Good, or 9 - New	48
4 - Poor	24
3 - Serious	12
2 - Critical	12*

Table 3.12.2.2-2: Routine Inspection Interval for Non-Buried Structures

** A Special Inspection may be required at a more frequent interval.*

3.12.3 Ancillary Bridges

There are many bridges/culverts maintained by various agencies with an NBIS Bridge Length Item B.G.01 < 6.0 feet. These are referred to as Ancillary Bridges/Culverts and are defined as follows:

> Ancillary Bridge: A bridge on a public roadway with an NBIS Bridge Length Item B.G.01 < 6.0 feet. This may also include multiple pipes where the clear distance between openings > ½ of the diameter of the smaller contiguous opening.

IDOT does not have policies and procedures for the inspection of Ancillary Bridges/Culverts. Agencies are responsible for the maintenance of the Ancillary Bridges/Culverts under their respective jurisdictions. The policies and procedures employed for addressing Ancillary Bridge issues must adequately address safety to the traveling public, and each owner must evaluate how to apply available resources to the task of inspecting, maintaining, repairing, and replacing these assets.

3.12.4 Bridges Not Carrying Public Roadways

It is not uncommon for a bridge not carrying a public roadway to be located over a public roadway. Examples are bridges carrying a railroad or pedestrian-only traffic over a public roadway. Since these bridges do not carry public roadways, the NBIS is not applicable. However, all bridges crossing public roadways must be included in the Illinois bridge inventory record to document the vertical and horizontal clearances provided by the underpass for the movement of oversize permit loads and NBI reporting requirements.

- NBIS Inspections are not required for bridges not carrying public traffic. However, the agency having maintenance responsibility of the bridge is responsible for ensuring the safety of the bridge and of the traveling public under the bridge. The following guidance is provided to address bridges not carrying public roadways, but crossing a public roadway:

3.12.4.1 Bridges Not Carrying Public Roadway Over Public Roadway – Both Maintained by Same Agency

- The bridge must be inventoried in the Illinois Bridge Inventory and inspected per NBIS with an interval per Table 3.12.2.2-2. A hands-on inspection is not typically required unless deemed necessary by the agency having maintenance/inspection responsibility.

Several agencies, IDOT included, own and maintain bridges carrying private railroad traffic over public roadways. These bridges must be in the Illinois Bridge Inventory and must be inspected in the same manner as bridges subject to the NBIS. Coordination with the railroad is required for obtaining right of entry and for scheduling flaggers.

3.12.4.2 Bridges Not Carrying Public Roads Over Public Roadway – Maintained by Different Agencies

The agency with maintenance responsibility of the bridge not carrying public roadway is responsible for having policies and procedures in place to ensure the safety of the bridge, the traffic crossing the bridge, and the traffic under the bridge. If, during the course of normal operations, the agency with maintenance responsibility for a public roadway under the bridge notes a safety concern, the agency with maintenance responsibility for the bridge must be notified immediately and necessary measures must be taken to protect the traveling public under the bridge until the agency having maintenance responsibility for the bridge has taken action.

3.12.4.3 Bridges Not Carrying Public Roadway Over Private Roadway

If a situation should be presented in which an agency has maintenance responsibility for a bridge not carrying a public roadway crosses a private roadway, the bridge should be included in the Illinois Bridge Inventory and inspections should be conducted per the NBIS with an interval per Table 3.12.2.2-2.

3.13 Bridge/Culvert Inventory

3.13.1 The National Bridge Inventory (NBI)

Illinois National Bridge Inventory data, Element Level Inspection data are submitted to the FHWA on an annual basis by the Office of Planning and Programming (OP&P) with the assistance of the BBS. The data becomes part of the National Bridge Inventory (NBI) which is defined by the FHWA as:

> National Bridge Inventory: An aggregation of State transportation department, Federal agency and Tribal government bridge and associated highway data maintained by the Federal Highway Administration (FHWA). The NBIS requires each State transportation department, Federal agency, and Tribal government to prepare and maintain a bridge inventory, which must be submitted to FHWA in accordance with these specifications on an annual basis or whenever requested. (23 CFR 650.315)

3.13.2 Recording Bridge Inventory Information

Directions for recording structure inventory information are provided in the IDOT *Specifications for Illinois Bridge Inventory Manual* (SIBI Manual).

Each bridge carrying or crossing a public roadway and having an NBIS Bridge Length Item B.G.01 > 20.0 feet, including temporary bridges open to traffic greater than 24 months, must be included in the Illinois bridge inventory and be inspected in accordance with NBIS regulations. Bridges not carrying public traffic but crossing over a public road, such as pedestrian bridges or railroad bridges, must also be included in the Illinois bridge inventory. Bridges are typically added to the Illinois bridge inventory during the planning phase and then completed once the bridge is opened to traffic.

Structure Inventory and Appraisal (SI&A) data must be entered into the ISIS within 3 months of changes occurring.

In general, the bridge inventory data is entered by Region/District personnel. Data entry for some items, such as load rating, NSTM inventory, most Special Inspection inventory, scour critical input, and hydraulics, are the responsibility of the Bureau of Bridges & Structures.

The agency with maintenance/inspection responsibility of the bridge is responsible for submitting the initial inventory information and subsequent changes to IDOT. For bridges not carrying a public roadway, but passing over a public road, the agency with maintenance responsibility of the roadway under the bridge is responsible for submitting the initial inventory information and subsequent changes to IDOT.

3.13.3 Updating Structure Inventory & Appraisal (SI&A) Information

A general review of bridge inventory items is required during each Routine Inspection with errors/omissions promptly reported to appropriate parties to ensure the bridge data in the ISIS is accurate and up-to-date. It is recommended the inspector utilize IDOT Form BBS 3320 Bridge Inventory Report during the Routine Inspection and record any errors or omitted information on the report for follow-up and correction.

Section 4 Load Rating

4.1 General

4.1.1 Purpose and Scope

The purpose of this section is to supplement the AASHTO MBE with IDOT policies, procedures, and guidelines for load rating bridges. The goals of load rating bridges are to ensure public safety, prevent structural damage, and assist in the evaluation of programming needs. By determining the safe load capacity of bridges, agencies are able to:

- Comply with federal regulations and state statutes
- Evaluate the as-built condition of a bridge
- Evaluate the effects of bridge damage and deterioration
- Establish appropriate bridge weight restrictions
- Evaluate the movement of vehicles that do not conform to the Illinois Vehicle Code
- Evaluate the feasibility of proposed structural modifications

4.1.2 Load Rating Regulations

4.1.2.1 Federal Regulations

Federal regulations governing load ratings are included in the Code of Federal Regulations (CFR) under the NBIS. The NBIS requires all structures defined as highway bridges on public roads, regardless of jurisdiction or ownership, be load rated and posted in accordance with the AASHTO MBE. The NBIS defines bridge as follows:

> Bridge: A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it includes multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Further, the NBIS distinguishes between publicly owned bridges and private bridges.

> Private bridge: A bridge open to public travel and not owned by a public authority as defined in 23 U.S.C. 101.

A private bridge is subject to NBIS requirements if both ends of the bridge are connected to a public roadway.

The NBIS also requires each agency prepare and maintain an inventory of all bridges subject to the NBIS, which includes load rating and load posting data. IDOT maintains this inventory in a database referred to as the Illinois Structure Information System (ISIS). ISIS data is annually provided to the FHWA for updating the NBI.

4.1.2.2 State Statutes

State statutes governing load ratings in Illinois are included in the Illinois Compiled Statutes (ILCS) under the Illinois Vehicle Code, the Illinois Highway Code, and the Structural Engineering Practice Act of 1989 (SE Act).

The Illinois Vehicle Code and Illinois Highway Code assign responsibility to IDOT for the load rating and load posting of bridges on state and PA highways, without regard to highway jurisdiction or bridge ownership. For bridges on tollways, load ratings and load postings are the responsibility of the bridge owner, but coordination with IDOT is still required.

The Illinois Vehicle Code defines the size and weight limitations of vehicles allowed to have unrestricted access to Illinois highways. The Illinois Vehicle Code grants authority to highway and bridge owners to issue permits to vehicles exceeding these limitations.

The SE Act requires the individual with overall responsibility for the load rating inspection, load rating analysis, and load posting recommendations to be licensed in the State of Illinois as a Structural Engineer.

4.1.3 Structures Requiring Load Ratings

Load ratings are required for the following structures on public roads:

- All structures subject to the NBIS requirements.
- Non-NBIS structures under IDOT jurisdiction enrolled in the Small Bridge Inspection Program (SBIP). This includes structures between 6 feet and 20 feet and multiple pipe installations where at least one pipe has an inside diameter of 60 inches or greater.

Additional information on the Small Bridge Inspection Program can be found in Section 3.

Load ratings are encouraged for Non-NBIS structures enrolled in a PA’s Small Bridge Inspection Program or equivalent inspection program.

4.1.4 Qualifications and Responsibilities of Load Rating Personnel

All individuals involved with the bridge rating shall be knowledgeable in bridge design and familiar with standard load rating procedures, AASHTOWare BrR software, IDOT policies regarding load ratings, and the requirements of the AASHTO MBE.

Bridge ratings shall be prepared under the direction of an Illinois Licensed Structural Engineer acting as the Engineer of Record (EOR). At a minimum, the load rating team shall consist of an Engineer of Record (EOR), a load rater and a load rating checker. The EOR may assume the task of either the load rater or the load rating checker.

The load rater is responsible for gathering all the required materials, performing the necessary calculations, using judgement as needed to adjust the load rating results and filling out the load rating documentation. The load checker is responsible for confirming the assumptions used for the load rating, checking calculations and ensuring the load rating results are reasonable.

4.1.5 Documentation and Deliverables

The primary documentation of a load rating is IDOT Form BBS 2795, “Structure Load Rating Summary” (SLRS). The purpose of the SLRS is to provide documentation a load rating was performed, with a brief summary of its findings. The SLRS shall be sealed by an Illinois Licensed

Structural Engineer and submitted to IDOT BBS where it will be placed in the structure's load rating file.

When load ratings of bridges on state and PA highways (excluding bridges owned by border states and federal agencies) are not completed by IDOT, the SLRS, load rating calculations, AASHTOWare BrR and/or other applicable software files, as-built bridge plans, and shop drawings, when applicable, shall be submitted to the BBS. Documentation of IDOT's concurrence with the load rating will be returned to the owner for inclusion in their bridge file. IDOT will update ISIS based on information on the SLRS.

For tollway bridge load ratings, a SLRS, sealed by an Illinois Licensed Structural Engineer, must be submitted to the BBS. IDOT concurrence is not required and the SLRS will serve as documentation a load rating was completed and the basis for updating ISIS.

All ratings, supporting calculations and AASHTOWare BrR files shall be in US Customary Units. Existing structures with plans in metric units shall be soft converted to US Customary Units.

4.1.6 Load Rating Software

Load ratings of bridges on state and PA highways, regardless of jurisdiction, should be completed, when possible, using AASHTOWare Bridge Rating™ (BrR) software in order to be compatible with the ITAP system and other software used by IDOT. If a load rating cannot be performed using AASHTOWare BrR, the load rater shall contact BBS to determine an acceptable alternative.

AASHTOWare BrR license requests, appropriate software version, setup, default settings, and modeling guidance may be found at:

https://idot.illinois.gov/doing-business/procurements/engineering-architectural-professional-services/consultant-resources/bridges-and-structures/bridge-load-ratings.html.

4.1.7 Quality Control and Quality Assurance

Quality Control (QC) and Quality Assurance (QA) are established procedures used to ensure load ratings are accurate and comprehensively evaluate the safe load capacity of bridges.

QC procedures must ensure:

Load ratings are completed using an independent load rater and checker.

- Staff performing and checking the load ratings are appropriately qualified.
- Load rating forms are complete and have been sealed by an Illinois Licensed Structural Engineer.
- Records are maintained documenting the load rating calculations, assumptions made, specifications used, and identify the staff performing the load rating.

QA procedures must ensure:

- All established QC procedures have been followed.
- Load rating conclusions are reasonable given the bridge type and condition.
- The load rating has been sealed by an Illinois Licensed Structural Engineer and documented using the appropriate IDOT forms.

4.2 Load Rating Requirements

Load ratings shall be completed using US Customary Units regardless of the units used in the original bridge plans. See Section 4.2.4.1 for IDOT reinforcement bar conversion standards.

4.2.1 Evaluation Methods

The AASHTO MBE provides both analytical and empirical methods for load rating bridges. The analytical methods include:

- Allowable Stress Rating (ASR)
- Load Factor Rating (LFR)
- Load and Resistance Factor Rating (LRFR)

The empirical methods include:

- Load ratings based on load testing
- Approximate load ratings based on rational criteria

Load ratings shall be completed using analytical methods when possible. Empirical methods should only be used when no plans are available for reinforced concrete bridges or masonry bridges, load testing has been performed, or an analytical load rating results in a less severe restriction than preferred given the condition of the structure. The BBS must concur with the use of empirical methods.

The evaluation method used depends on several factors including:

- Member(s) or bridge type being evaluated
- AASHTO specification used to design the bridge initially
- Bridge material type
- Availability of original design and rehabilitation plans and specifications

If existing plans cannot be located, the BBS shall be contacted for guidance on the load rating. Historic IDOT manuals, standards, and manufacturer’s details may be used to augment empirical evaluations. A load rating inspection may be needed to verify the required measurements prior to these aids being used.

4.2.2 Rating Factors

A rating factor is a numerical representation of a bridge's capacity to safely carry a specific transient load having a defined magnitude and dimensional configuration. The general load rating equation for an individual load carrying member can be expressed as:

$$RF = \frac{(C - DL)}{(LL + IM)}$$

Where:

RF	=	Rating Factor
C	=	Capacity of load carrying member
DL	=	Load effect of all permanent loads
LL	=	Load effect of transient load
IM	=	Impact or dynamic load allowance, when applicable

An inventory load rating is comparable to a level of stress or reliability used during the design of a bridge to determine the proportions of load carrying members. An operating load rating is representative of the maximum load effect that can occasionally be tolerated by a bridge without causing appreciable damage.

See AASHTO MBE for more detailed information on calculating rating factors.

To determine a bridge's rating factor for a specific transient load, a rating factor must be calculated for each load effect of each applicable load carrying member in the bridge. The smallest of these factors is the bridge's rating factor for that specific transient load, and the corresponding load carrying member is the controlling member. This process is repeated for several different transient loads.

4.2.3 Rating Factor Variables

Accurately defining the variables necessary to calculate rating factors is essential for completing a quality load rating. This information can be obtained from sources including but not limited to:

- Original design and rehabilitation plans and specifications
- As-built plans and shop drawings
- Construction records
- Load rating inspection documentation

- Routine Inspection reports
- Special Inspection reports

4.2.3.1 Load Rating Inspection

Before a load rating is started, the load rating team reviews existing documentation and determines if a Load Rating Inspection (LRI) is necessary. This inspection may be performed by the load rating team or delegated to a separate inspection team. At a minimum, the LRI must:

- Confirm the condition and dimensions of the primary load carrying members
- Confirm the bridge geometric dimensions
- Confirm the presence, magnitude, and location of permanent loads with particular attention paid to utilities, attachments, and thickness of wearing surface
- Document any deterioration or repairs made to primary load carrying members

If necessary, the load rating team will discuss specific information to be collected or verified and how the information is to be documented. Section 3 contains information on recommended procedures for documenting LRI findings.

Additional information on LRIs can be found in Section 3.

4.2.3.2 Load Carrying Member Properties

Section and material properties of load carrying members are required to calculate the capacities of individual members and to determine load effects throughout the bridge.

4.2.3.2.1 Section Properties

Section properties are typically calculated using dimensions given in existing plans when available. Information taken from existing plans and the condition of the load carrying members may need to be field verified. If load carrying members are damaged or deteriorated, the section properties shall be modified accordingly. Unsound material is excluded from section property calculations.

If existing plans are not available, an LRI may be required to determine the section properties. Although devices are available for estimating the location and size of reinforcement bars, IDOT has not found these to be sufficiently reliable and has not adopted this technology for determining the section properties of reinforced concrete members. If section properties cannot be calculated or estimated with reasonable confidence, empirical methods may be used.

4.2.3.2.2 Material Properties

Material properties are based on information provided in the existing plans and specifications when available. If existing plans are not available, the following historical material properties shall be used:

		Allowable Stress Rating	
Year of Construction	Minimum Yield Strength, F_y (ksi)	Inventory ($0.55\ F_y$) (ksi)	Operating & Posting ($0.75\ F_y$) (ksi)
Before 1905	26.0	14.3	19.5
1905 - 1929	30.0	16.5	22.5
1930 - 1961	33.0	18.1	24.7
After 1961	36.0	20.0	27.0

Table 4.2.3.2.2-1: Structural Steel Material Properties

		Allowable Stress Rating	
Year of Construction	Minimum Yield Strength, f_y (ksi)	Inventory (ksi)	Operating & Posting (ksi)
Before 1905	26.0	14.3	19.5
1905 – 1920	32.0	17.6	24.0
1921 – 1944	33.0	18.1	24.7
1945 – 1979	40.0	20.0	28.0
After 1979	60.0	24.0	36.0

Table 4.2.3.2.2-2: Reinforcing Steel Material Properties

Year of Construction	Tensile Strength, f_{pu} – LRFD f'_s – Std. Spec's (ksi)
Before 1963	228.0
1963 - 1976	248.0
After 1976	270.0

Table 4.2.3.2.2-3: Prestressing Strand Material Properties

	Year of Construction	Compressive Strength, f'_c (f_c) (ksi)
Cast in Place Concrete	Before 1930	3.0 (1.2)
	1930 to date	3.5 (1.4)
Precast Concrete	All years	4.5 (1.8)
Precast Prestressed Concrete	All years	5.0 (2.0)

Table 4.2.3.2.2-4: Concrete Material Properties

	Member Type	Allowable Inventory Stresses Normal Load Duration *(See Note 2)* (ksi)	
Members subject to bending and shear, dry service conditions	Stringer, Floor Beam, Pile Cap	Bending F_b = 1.15	Shear F_y = 0.155
Members subject to axial loading	Truss Member	Compression F_c = *(See Note 1)*	Tension F_t = *(See Note 1)*
Members subject to axial compression and bending, wet service conditions	Piling	Compression F_c = 1.25	Bending F_b = 1.95

Table 4.2.3.2.2-5: Timber Material Properties

Notes:

7. Truss member properties are dependent on member sizes. Stresses shall be the lesser of the Select Structural values found in the NDS for Douglas Fir-Larch, Red Oak, or Southern Pine.
8. Operating/Posting allowable stress is typically 133% of the inventory allowable stress, except for timber piles, which is 116.5% of the inventory allowable stress. This is to account for the eccentric loading at piers, and to a lesser extent at abutments, induced by the simple span configurations of many PA bridges.

Condition	Allowable Bending Stresses, F_b Normal Load Duration & Dry Service Conditions	
	Inventory (ksi)	Operating & Posting (ksi)
Good	1.650	2.195
Fair	1.485	1.975
Poor	1.320	1.755

Table 4.2.3.2.2-6: Timber Deck Material Properties

The AASHTO MBE shall be referenced for material properties not included in these tables.

It is reasonable to expect the original material properties of sound structural steel or iron have not changed. However, the strength of some sound materials, such as concrete or timber, may have changed over time. Material testing may justify using higher or lower strengths than originally expected.

4.2.4 Load Carrying Member Policies and Guidelines

Experience with load rating different types of load carrying members has led IDOT to develop various policies and guidelines, which are included in the following sections. The policies recognize it is not necessary to load rate every load carrying member on a bridge. However, it is the responsibility of the load rater to review the bridge's structural system and make the final determination if a load carrying member can be safely excluded from a load rating.

In general, policies and guidelines stated in the IDOT *Bridge Manual* for bridge design are also applicable for load ratings.

4.2.4.1 Metric Reinforcement Bars

When converting reinforcement bars from metric to English units, the following table shall be used:

Metric Bar	English Substitution
10	3
13	4
15	5
16	5
19	6
20	19 *(See Note 1)*
22	7
25	8
29	9
30	29 *(See Note 1)*
32	10
35	10 *(See Note 1)*
36	11

Table 4.2.4.1-1: Reinforcement Bar Metric to English Conversion

Notes:

1. Use the spacing that equates to the same area of steel per foot as the metric bar size and spacing

4.2.4.2 Decks

4.2.4.2.1 Concrete and Metal Decks

Concrete decks and metal decks supported by primary load carrying members typically do not need to be load rated. A deteriorated concrete deck or metal deck is usually a maintenance issue that requires patching or shielding to protect areas below the deck.

When concrete decks and metal decks are not maintained and have deteriorated to the point where weight restrictions need to be considered, the BBS shall be contacted for guidance.

The following are IDOT policies regarding concrete and metal deck load ratings:

Concrete Cracks: When a concrete deck is load rated, the Crack Control Parameter (ASR and LFR) and Exposure Factor (LRFR) are typically 130 k/in and 0.75, respectively, for the top reinforcement in the deck based on severe exposure condition.

Decks with Overlays: When a deck is load rated, the structural thickness of the deck is the thickness remaining after scarification. The overlay thickness shall not be included when calculating the section properties of the deck unless it meets the requirements given in Section 4.2.4.3 for Concrete Deck-Slabs with Concrete Overlays. The weight of the overlay shall be accounted for accordingly.

4.2.4.2.2 Timber Decks

Timber plank decks supported by primary load carrying members must be included in a load rating.

Special attention is given to:

- Deteriorated timber planks
- Decks without running boards
- Decks with narrow running boards (Running boards are not expected to distribute wheel loads unless they are a minimum width of 2'-6" for each wheel path)

The following are IDOT policies regarding timber deck load ratings:

Load Rating Guidelines: Timber deck load ratings follow the procedures in the USDA Forest Service *Timber Bridges Design, Construction, Inspection, and Maintenance.*

Wheel Load Distribution: The distribution of wheel loads shall be determined based on loading and effective tire width. BBS practice is to distribute the wheel load to a single deck plank, except where running boards are present. When running boards are present, the wheel load is distributed vertically through the running boards at a 45° angle.

4.2.4.3 Superstructures

The following are IDOT policies regarding superstructure load rating:

Concrete Deck-Slabs with Concrete Overlays: When a superstructure with a scarified deck-slab and hard concrete overlay is judged to act composite with the original deck, the actual scarified deck and overlay thicknesses will be used as tributary for dead loads. The deck structural thickness will be the original, pre-scarification structural thickness.

A concrete overlay considered to act composite with the original deck-slab is expected to have the following characteristics:

- Scarification was performed with hydro-scarification. This is the case for scarification on state highway system bridges since 2003 inclusive. Prior to 2003, hydro-scarification would be indicated as a separate pay item on the overlay plans.
- The concrete overlay is in good condition, including physical structure and adherence to the original slab.

Future Wearing Surface: Future Wearing Surface shall not be included in any load ratings. Only as-built or as-inspected overlays and permanent loads shall be included in load ratings.

Exterior and Interior Beams: Both the exterior and interior beams of multi-beam superstructures shall be evaluated in a load rating. However, if the analysis results in the exterior beam governing and a restriction is required, the analysis can be refined by limiting the application of the vehicular live load(s) to the actual striped lanes. In some cases, the exterior beam can be neglected or omitted from the load rating. Examples of this would be physical traffic control devices installed to deter traffic or an approach roadway width that is less than the width of the bridge roadway.

Vehicle Lanes/Live Load Distribution: When performing load ratings, the number of traffic lanes to be loaded, and the transverse placement of wheel lines, shall be in conformance with the current AASHTO Standard Specifications or AASHTO LRFD Specifications and the following:

- Roadway widths from 18 to 24 feet shall have two traffic lanes, each equal to one half the roadway width.
- Roadway widths less than 18 feet shall have one traffic lane only.
- Timber decks with running boards shall have one traffic lane for every pair of running boards.

However, the following alternatives may be used with approval from the BBS:

- When lane striping is present: The number of traffic lanes and transverse placement of wheel lines may be determined by placing truck loads only within the striped lanes. When load rating a structure based on the existing striped lanes, the transverse positioning of the truck shall include placing the wheel load anywhere within the lane, including on the lane stripe.
- When lane striping is not present: If ADT is less than 2000 and ADTT is less than 400, one traffic lane may be used. PA owned structures with very low ADT, regardless of the presence of lane striping, may be rated for one lane.
- The BBS does not allow these alternatives for the Design Vehicle at the Inventory Level. An Inventory Level load rating is used to evaluate the bridge using the design loading and design specifications.

For the purposes of load ratings, the exterior beam LLDF shall be computed as follows:

- ASR or LFR: Use the Lever Rule exclusively.
- LRFR: Use AASHTO MBE 6A4.5.6. If the inventory load rating falls below 1.0, then the provisions found in IDOT Bridge Manual Article 3.1.12.1 may be used to reduce the LLDF to the exterior beam.

Approach Spans: An approach span supported by a vaulted abutment shall be evaluated in the load rating except when supported by backfill.

4.2.4.3.1 Cast-in-Place Concrete Superstructures

Cast-in-place concrete is used in a variety of superstructure systems. The slabs, stringers, beams, and girders in these superstructure systems are primary load carrying members and shall be evaluated in the load rating.

For slab bridge type structures, the rating shall all types of slab sections if the cross-section varies across the width of the bridge.

Concrete bridges constructed after the early 1940's generally used air entrainment in the concrete mix to improve the long-term freeze/thaw durability of the bridge. Bridges constructed prior to this date did not use air entraining agents and generally can exhibit significant concrete deterioration due to internal damage caused by freeze/thaw cycles. Cores were taken on many state owned concrete bridges built prior to the 1940's. The BBS should be contacted to verify if core reports and strengths are available. For more information see IDOT All Deputy Directors of Highways memos "Deterioration of Reinforced Concrete Structures" and "Inspection and Coring of Reinforced Concrete Structures" in the Appendix.

4.2.4.3.2 Precast Concrete Superstructures

Precast concrete beams are commonly referred to as "Channel Beams", "Nelsen Beams", "Midwest Beams", or "Precast Concrete Bridge Slabs". They are primary load-carrying members and shall be evaluated in the load rating.

IDOT uses the same policies regarding live load distribution factors for precast concrete superstructure load ratings as precast prestressed concrete box beams given in Section 4.2.4.3.4

4.2.4.3.3 Precast Prestressed Concrete I Beam Superstructures

Precast prestressed concrete I beams (PPCI) is used in a variety of superstructure systems. These systems are primary load carrying members and shall be evaluated in the load rating. When present, diaphragms between the beams are included in the analysis model but are typically not evaluated in the load rating.

The following are IDOT policies regarding PPCI superstructure load ratings:

> Beam Deterioration: Policies for load rating PPC box beams given in Section 4.2.4.3.4 are also applicable to PPCI superstructures. Deterioration in PPC box beams is more prevalent due to the tendency for shear key leakage. PPCI superstructures can also be exposed to deicing agents, especially below deck drains, scuppers, and leaking expansion joints.

PPCI superstructures occasionally receive impact damage that results in exposed prestressing strands. If the exposed strands are undamaged and the PPCI beam receives a structural concrete repair, as described in Section 2.14, the strands can be considered fully effective.

Shear: The 1994 & 2003 IDOT *Precast Prestressed Concrete Design Manual* directed engineers to neglect prestressing force effects when designing shear reinforcement in the negative moment region of continuous spans. Although not allowed by design, many beams were fabricated with non-conforming shear stirrup spacing, which were then reanalyzed using prestressing force effects to justify use in construction. For all precast prestressed concrete beams, use the contribution of prestressing force effects, even if neglected in the original design. In addition, the method for design of shear reinforcement presented in the 1979 Interim AASHTO *Standard Specifications for Highway Bridges* is not allowed for load rating.

4.2.4.3.4 Precast Prestressed Concrete Box Beams (Deck Beams)

PPC box beams (deck beams) are primary load carrying members and shall be included in the load rating.

The following are IDOT policies regarding PPC box beam load ratings:

Live Load Distribution Factors: For LFR, there are two primary methods used to calculate LLDF's for PPC box beams in Illinois: AASHTO *Standard Specifications for Highway Bridges 13th Edition* (AASHTO 1983) and *17th Edition* (AASHTO 2002). AASHTO 2002 is similar to current design practice and is generally more conservative for wider bridges (greater than or equal to 30 feet), while AASHTO 1983 is more conservative for narrower bridges (less than 30 feet). Thus, the preference is to use AASHTO 2002 for state bridges and AASHTO 1983 for PA bridges as described below:

- For state owned bridges being evaluated using LFR, the LLDF shall be calculated using AASHTO 2002, Section 3.23.4.
- For PA bridges being evaluated using LFR, the LLDF shall be calculated using AASHTO 1983, Section 3.23.4. AASHTO 2002 shall only be used if specified on the existing bridge plans.

Shear Keyways: The grouted shear keyways between individual PPC deck beams allow for the lateral distribution of load during application of live loads. If it is determined during inspection that a keyway has become ineffective, live load can no longer be distributed across the longitudinal joint between beams. The live load distribution factors for all beams in the span(s) with the ineffective keyway(s) must be re-calculated accordingly based on groups of beams that act together between ineffective keyways. However, when a PPC deck beam superstructure has a reinforced concrete overlay that is in good condition or has hairline reflective cracks, live load distribution to adjacent deck beams can be considered unaffected by ineffective shear keyways due to the overlay's ability to distribute live load between beams.

Beam Deterioration: Some PPC box beams have exhibited rapid deterioration rates. IDOT sponsored the University of Illinois at Urbana-Champaign to research this issue. The main finding of the research was once corrosion has initiated within a prestressing strand; the rate of corrosion is sufficient to expect the strand has lost the ability to carry load. In addition, if a strand is in the vicinity of corroding conventional reinforcement (stirrups, welded wire fabric, etc.), that strand is expected to be corroded as well.

Special attention is given to:

- Areas of delaminated concrete
- Areas of inadequate concrete cover
- Areas of longitudinal cracking
- Areas of transverse cracking
- Areas with spalls, exposed stirrups, or exposed prestressing strands

The following guidelines shall be used for estimating strand loss in PPC box beams.

Prestressed strands incorporated in PPC Deck Beams shall be disregarded during analysis for load carrying capacity as specified below. If deterioration is located within the center half of the span, disregard the entire length of prestressed strand. If deterioration is located within the end quarters of the span, disregard the prestressed strand from the beam end up to the extent of deterioration. This can be accomplished by debonding the strands up to this location.

LONGITUDINAL CRACKS

The intent is to disregard all strands that could intersect the crack and be exposed to air and moisture.

1. Cracks observed in the middle area of the beam underside, with or without rust stains or other discoloration of the concrete adjacent to the cracks:

   Disregard all strands from all rows of strands that may be located adjacent to the cracks.

2. Cracks observed along the edges of the beam underside, with or without rust stains or other discoloration of the concrete adjacent to the cracks:

   Disregard at least the strands located adjacent to the edge of the beam in the bottom row of strands. When the crack is extensive in length and its location varies in distance from the beam edge, disregard additional interior strands from all rows of strands that may be intersected by the crack.

3. Cracks observed along the outside face of the exterior beam, with or without rust stains or other discoloration of the concrete adjacent to the cracks:

   Disregard strands in the outside face of the beam adjacent to or intersected by the cracks.

4. Two longitudinal cracks observed crossing or meeting:

   Disregard all strands in all rows of strands located between the cracks and one strand from all rows of strands located adjacent to the outer edge of the cracks.

DETERIORATION

1. Exposed strands observed with sound concrete adjacent to and above the exposed strands:

   Disregard exposed strands only.

2. Exposed strands observed with unsound concrete adjacent to and above the exposed strands:

   Disregard exposed strands and all strands located in rows above and immediately adjacent to the area of unsound concrete.

3. Exposed welded mesh or reinforcement bars observed on the bottom corners and/or extending up the side face.

   Disregard the strands located at the edge of the beam in the bottom row directly above the exposed reinforcement. If the concrete is found to be unsound adjacent to the exposed reinforcement bars, disregard all strands in all rows located above the area of unsound concrete.

4. Exposed wire mesh or full width reinforcement stirrup bars observed on bottom of beam:

   Judge whether the wire mesh or reinforcement bars are in contact with the strands.

   - If in contact, disregard all strands in the lower row directly above the exposed wire mesh or stirrups.
   - If not in contact but the concrete adjacent to the exposed wire mesh or stirrups is found to be unsound, disregard all strands located above the area of unsound concrete.
   - If not in contact and concrete adjacent to the exposed wire mesh or stirrups is sound, do not disregard strands during analysis.

5. Areas of delaminated concrete observed:

   Note: Patches are considered delaminated.

   Remove <u>all</u> delaminated concrete to determine the depth of concrete deterioration.

   - If reinforcement stirrup bars, wire mesh or strands are exposed, treat as in “1” through “4” above.
   - If no reinforcement, mesh or strands are exposed but there are indications that the exposed concrete is unsound within the affected area, disregard all strands located in the rows of strands above the area.
   - If no reinforcement, mesh or strands are exposed in the affected area and concrete in the area is found to be sound, do <u>not</u> disregard strands in analysis.

6. Wet or stained areas observed on bottom or side of beams:

   Closely inspect the wet or stained area to determine the soundness of the concrete.

   - If close inspection indicates the concrete is unsound or delaminated, treat as in “5” above.
   - If close inspection confirms the concrete is sound, do not disregard strands in analysis.

   Note: Wet and/or rust stained areas should be watched closely. These areas will be the next areas to experience significant deterioration.

4.2.4.3.5 Steel Superstructures

Steel is used in a variety of superstructure systems. The stringers, beams, girders, floorbeams, diaphragms, and crossframes on curved bridges are primary load carrying members and shall be evaluated in the load rating. Diaphragms and crossframes on straight bridges are included in the analysis model but are typically not evaluated in the load rating.

Special attention is given to:

- Fracture critical members
- Members with noted section loss

The following are IDOT policies regarding steel superstructure load ratings:

Plastic Capacity: The use of plastic capacity in steel members is not allowed when existing plans cannot be located or for steels with F_y greater than 70ksi.

Moment Redistribution: Moment redistribution in continuous steel members shall not be used.

Beam Splices: Steel beam splices are typically not evaluated in the load rating.

Bearing Stiffeners: Bearing stiffeners are included in the analysis model but are typically not evaluated in the load rating except when evaluating the bearing capacity of deteriorated beam ends.

Fatigue and Serviceability: Load ratings of steel members generally do not evaluate fatigue, overload, and live load deflections.

Cover Plates: The length used for analysis of cover plates is shorter than the actual end-to-end cover plate length. The Theoretical End of Cover Plate location is reduced 1.5 times the full width of the cover plate as shown in Figure 4.2.4.3.5-1.

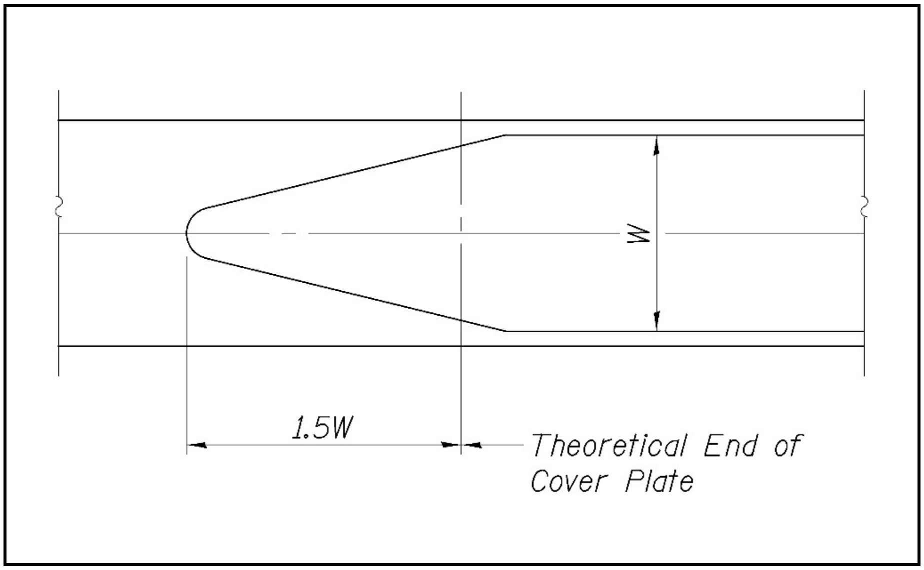


Figure 4.2.4.3.5-1: Theoretical End of Cover Plate

Section Loss: When the steel superstructure condition rating drops due to section loss caused by corrosion, the condition rating may be raised back to the previous rating if:

- Legal, Routine Permit, and Emergency Vehicle load rating operating rating factors are all greater than or equal to 1.00 (considering section loss)
- Corroded area is cleaned and painted
- This may only be done with the approval of the BBS.

4.2.4.3.6 Trusses

Unless a structural analysis has identified a specific truss member as a non-load carrying member, all elements of steel trusses (including gusset plates and other connection elements) are considered primary load carrying members and shall be evaluated in the load rating. Tension members and floor beams of steel trusses are fracture critical members and may be given special attention.

The following are IDOT policies regarding steel truss load ratings:

Dead Load Forces: IDOT uses a minimum dead load factor of 0.90 when the dead load acts to reduce the overall loading (i.e. the dead load and live load are acting in opposite directions). The maximum dead load factors shall be as defined in the AASHTO MBE.

Live Load Force Effects: Live load force effects may be determined by either the maximum envelope method or concurrent force method. The maximum envelope method should be used for the individual member rating checks. However, using the maximum envelope method for the overall gusset plate shear rating checks may be unconservative and the concurrent force method should therefore be used.

Both maximum enveloped force effects and concurrent force effects can be attained using the AASHTOWare BrR software.

Additional policies specific to gusset plates:

Shear Reduction Factor, Ω: For the shear reduction factor, Ω, the BBS recommends using Ω = 0.88 for typical cases. This factor can vary from 0.74 to 1.00 and a more refined analysis may provide more favorable results. The BBS must approve the use of a factor other than 0.88.

Slip Critical Connections: Slip critical considerations need not be considered in the gusset plate load rating.

Partial Shear Planes: Based on research conducted on behalf of IDOT, the partial shear plane checks within the 2014 AASHTO MBE Interims may be overly conservative in some instances. Therefore, if partial shear plane checks control a load rating, a more refined analysis shall be performed.

Load Distribution between Gusset Plates: There are typically two gusset plates at each panel point node location; an inside gusset plate and an outside gusset plate. In some instances, there are multiple gusset plates on each side. Generally, half the loads shall be distributed to the inside gusset plate(s) and half to the outside gusset plate(s). However, for gusset plates with resulting substandard load ratings, load redistribution between inside and outside gusset plates at a given node may be based on the remaining capacity of each gusset plate(s).

The BBS uses a maximum redistribution of 30% of the load carried by the gusset plate(s) on one side of the member to the gusset plate(s) on the other side of the member. Under the design assumption of the gusset plates(s) on one side carrying 50% of the load, the maximum redistribution would result in the gusset plate(s) on the stronger side carrying a maximum of 65% of the total load and the gusset plate(s) on the weaker side carrying a minimum of 35% of the total load. If other load redistribution techniques are used, it must be with the approval of the BBS.

<u>Section Loss</u>: Load rating of the gusset plates for section loss shall be based on the AASHTO MBE procedures. However, when the AASHTO MBE procedures conclude the gusset plate is the controlling member, a more refined analysis is used. Procedures for the refined section loss calculations are outlined in the IDOT *Gusset Plate Evaluation Guide* (see “Refined Analysis Load Ratings” policy). If this refined section loss method is utilized, it must be done with approval from the BBS.

If there is no recorded corrosion of the gusset plates, splice plates, or fasteners, the load rating need not consider section loss. If there is recorded corrosion of the gusset plates, splices plates, and/or the fasteners, the load rating shall reduce the areas and capacities of the affected members to account for the greater of:

- The actual section loss based on the detailed measurements
- An assumed minimum section loss of 10%

If the member section loss is not proportional between the inside plate(s) and the outside plate(s), the load distribution between the plates must be verified (see “Load Distribution between Gusset Plates” policy).

<u>Refined Analysis Load Ratings</u>: The *Gusset Plate Evaluation Guide* prepared for IDOT by Wiss, Janney, Elstner Associates, Inc. provides examples to assist load raters with the refined analysis of gusset plates. This guide is available on the IDOT website under the “Bridges & Structures” and “Load Ratings” tabs.

Other refined analysis approaches, including the Truncated Whitmore Section method, are also acceptable.

4.2.4.3.7 Cables and Hangers

Cables and hangers, along with their anchorage and end connections, are structural elements typically found in suspension and tied arch bridges. They are primary load carrying members and shall be evaluated in the load rating. Unless designed to provide load path redundancy, these are fracture critical members and may be given special attention.

4.2.4.3.8 Pin and Link Connections

Pin and link plates as part of a pin and link connection are primary load carrying members and shall be evaluated in the load rating.

4.2.4.3.9 Arches

Arches utilize various types of material and construction techniques. The arch is a primary load carrying member and shall be evaluated in the load rating.

Although some arches may not be considered a culvert, the FHWA *Culvert Inspection Manual* provides information relating the observed field conditions of arches to structural deficiencies. This information can be useful in identifying conditions indicative of reduced load carrying capacity.

4.2.4.3.10 Precast Concrete Three-Sided Structures

Precast concrete three-sided structures are produced under several different proprietary names. They are primary load carrying members and shall be evaluated in the load rating. The initial load rating is typically provided by the manufacturer. In order to effectively evaluate these members, future load ratings must utilize software programs that take into account the soil structure interaction.

4.2.4.4 Substructures

Substructures are generally excluded from the load rating. However, the condition and configuration must be reviewed to confirm the primary load carrying members of the substructure can be safely excluded from the load rating.

4.2.4.4.1 Abutment and Pier Caps

Abutment and pier caps can be made of steel, concrete, or timber. These are primary load carrying members and shall be evaluated in the load rating when the following conditions are observed:

- Pile deterioration affecting the ability to adequately support the cap
- Deteriorated areas of timber caps (identified by sounding or coring)
- Deteriorated areas of concrete cap with exposed reinforcement bars
- Concrete caps with flexure or diagonal shear cracks (width greater than 0.012 inches)
- Steel caps determined to be fracture critical members
- Deteriorated areas of steel caps with measurable section loss
- Loss of bearing area

4.2.4.4.2 Cantilever Abutment and Pier Caps

Cantilever abutment and pier caps are typically made of reinforced concrete. These are primary load carrying members and shall be evaluated in the load rating when the following conditions are observed:

- Exposed tension or shear reinforcement
- Flexure or diagonal shear cracks (width greater than 0.012 inches)
- Loss of bearing area

4.2.4.4.3 Piles

Typical pile types in Illinois include timber, precast concrete, metal shell, and steel H-piles. When piles are not exposed, they are excluded from the load rating. When piles are exposed, they shall be evaluated in the load rating when the following conditions are observed:

- Scour has exposed the pile(s) under a footing
- Section loss due to damage or deterioration
- Deterioration of timber piles is a major factor in the load posting and closure of PA bridges.

4.2.4.4.4 Scour

Scour is the leading cause of bridge failure. The loss of material supporting the bridge foundation can affect the stability of the bridge as well as the safe load capacity of the foundation. The effects of scour must be accounted for in the load rating.

4.2.4.5 Culverts

Individual elements of a culvert barrel are the primary load carrying members and shall be evaluated in the load rating. Wingwalls and headwalls are not evaluated in the load rating.

4.2.4.5.1 Concrete Box Culverts/Rigid Frames

Reinforced concrete box culverts can be cast-in-place or precast. The top slab and sidewalls are the primary load carrying members and shall be evaluated in the load rating. Bottom slabs are typically excluded from the load rating. If existing plans cannot be located, the load rating shall be based on empirical methods. Empirical methods may also be used to evaluate non-NBIS culverts, even when existing plans are available. When following the LRFR method, the AASHTO MBE special live load factors for reinforced concrete box culverts shall be used.

The following are IDOT policies regarding all concrete box culvert load ratings:

Fill Heights: Load ratings shall consider both maximum and minimum fill heights, within the travel way, as shown on existing plans or as measured in the field.

Shear: Shear shall be evaluated for the top slab when required by the AASHTO Standard Specifications (LFR) or the AASHTO LRFD Specifications (LRFR).

Skew: Vehicles shall be assumed to travel perpendicular to the culvert sidewalls. For the rare case where vehicles travel parallel to the culvert sidewalls i.e. the culvert runs parallel with the centerline of roadway, the rater shall contact the IDOT Rating Unit for direction.

Barrel Walls: When the top slab is simply supported, the exterior and interior walls are typically excluded from the load rating. If the condition rating of the culvert is “4” (Poor) or less and the walls have horizontal cracks or show signs of distortion, the load rating of the culvert shall be based on the lower of:

- Rational evaluation method considering condition of sidewalls. See Table 4.3.3.3-1.
- Analytical top slab load rating

For rigid frames (i.e. moment connection between slab and wall) the walls shall be evaluated in the load rating regardless of condition.

Negative Moment Reinforcement: If the existing plans of a multi-cell box culvert do not detail negative moment reinforcement or if a construction joint with no reinforcement passing through it is detailed directly over the interior wall(s), the culvert shall be evaluated as multiple single cells.

Skewed reinforcement: Culverts with large skews often have reinforcement placed parallel to the centerline of the roadway or headwalls as shown in IDOT Culvert Manual Figure 3.5.2-1 rather than the more typical perpendicular to the sidewall configuration. The AASHTOWare BrR program currently cannot account for skewed reinforcement and assumes placement perpendicular to the sidewalls. This requires the rater to enter a perpendicular rebar spacing that results in an equivalent strength to a skewed bar. The adjustment is made as follows:

For culverts with the reinforcement spacing detailed perpendicular to the bars, the spacing shall be multiplied by sec^2(skew). For example, #8 bars at 7” cts placed parallel to a 25^o skew; specify #8 bars at 8.52” cts. For culverts with the reinforcement spacing detailed parallel to the sidewalls, the spacing shall be multiplied by sec(skew). For example, #7 bars at 6” cts parallel to the walls for a culvert on a 40^o skew; specify #7 bars at 7.83” cts. See Figure 4.2.4.5.1-1 and Figure 4.2.4.5.1-2.

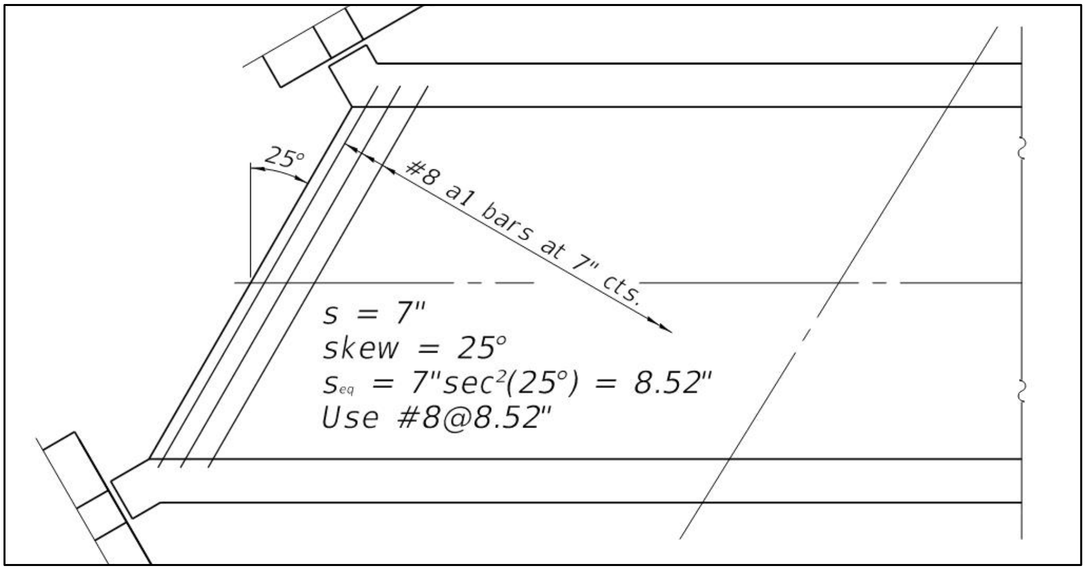


Figure 4.2.4.5.1-1: Spacing detailed perpendicular to bars

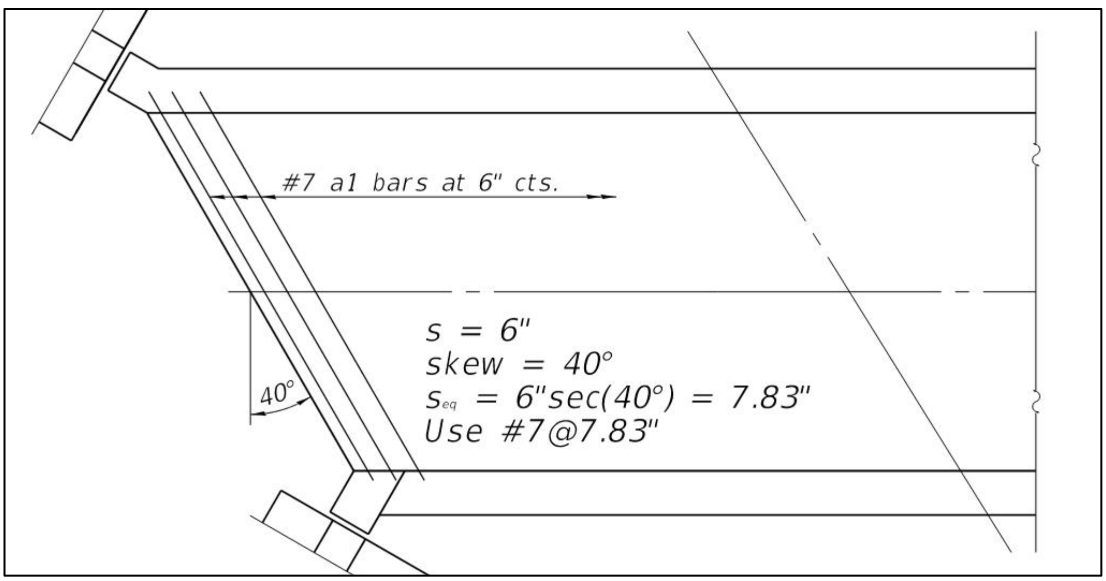


Figure 4.2.4.5.1-2: Spacing detailed parallel to the sidewalls

Precast Boxes: The as-built shop drawings, if available, shall be used when preparing the load rating for an in-service culvert. If the shop drawings are not available, the rater may either use an empirical method, load testing or the reinforcement details from the ASTM or AASHTO specification shown on the original contract plans. When no original contract plans are available, the empirical method or load testing shall be used.

For new precast boxes, the initial load rating performed during the design phase shall be based on the dimensions and reinforcement details of the precast box culvert specification noted on the contract plans or the referenced IDOT Standard Specification. The current specification is ASTM C1577. If the reinforcement or other details are changed during the shop drawing approval process, the initial rating shall be updated and forwarded to IDOT BBS. A note shall be added to the SLRS describing the source of information used to determine dimensions and reinforcement used for the rating.

4.2.4.5.2 Concrete Pipe Culverts

Regardless of the shape of the concrete pipe culvert, the pipe walls are the primary load carrying members and must be evaluated in the load rating. These load ratings are typically based on empirical methods.

4.2.4.5.3 Metal Pipe Culverts

The corrugated metal walls are the primary load carrying members in a metal pipe culvert and shall be evaluated in the load rating. The National Corrugated Steel Pipe Association (NCSPA) *Design Data Sheet No. 19*: *Load Rating and Structural Evaluation of In-Service, Corrugated Steel Structures* and the AASHTO Standard Specifications are the basis for load rating of metal pipe culverts.

The deflected shape of the culvert, distress at the joints of adjoining plates, and section loss must be included in the load rating. Both minimum and maximum fill depths must be considered. Impact shall be included for fill heights less than 3 feet. Rating factors can be controlled by either the structural capacity of the metal walls or the minimum cover.

4.2.4.5.4 Masonry Culverts

Masonry culverts are typically box-shaped or arch-shaped. The sides of boxes, arches, and arch supports are the primary load carrying members and shall be evaluated in the load rating.

4.3 Design and Legal Load Ratings

A design load rating is the evaluation of a bridge's safe load capacity relative to the design live load defined as HS20 (ASR or LFR) or HL93 (LRFR) using dimensions and properties of the bridge in its present as-built or as-inspected condition. This evaluation is completed at both the inventory and operating levels. Regardless of what load or specification the original bridge was designed to, the design load rating shall be based on the HS20 or HL93 vehicle to provide a consistent basis of comparison across the nation.

A legal load rating is the evaluation of a bridge's safe load capacity relative to the Illinois Statutory Loads defined in the Illinois Vehicle Code. A legal load rating is only required when the design inventory load rating factor is below 1.0. This evaluation is generally completed at the operating level as further discussed in Section 4.3.5 . The purpose of a legal load rating is to determine if a weight restriction, to either Legal Loads Only or below legal loads, is required.

4.3.1 Assignment of Responsibility

IDOT's responsibility for load ratings is granted by 625 ILCS 5/15-317.

4.3.1.1 State Bridges

State bridges are those under the jurisdiction of IDOT, and include structures covered by the IDOT Small Bridge Inspection Program.

Design and legal load ratings for state bridges are typically completed by IDOT staff. When load ratings are completed by consulting engineers, IDOT must concur with the load rating and any subsequent weight restrictions.

4.3.1.2 Public Agency Bridges

Public Agency (PA) bridges include those under the jurisdiction of counties, townships, road districts, municipalities, park districts, forest preserve districts, and conservation districts.

Design and legal load ratings for all new construction, rehabilitations, and other work affecting the safe load capacity of a PA bridge must be completed by the PA or their designated representative.

IDOT typically assists PAs in completing design and legal load ratings for damaged or deteriorated bridges. The LBU monitors bridge condition ratings in ISIS and automatically schedules a load rating for bridges meeting the requirements of Section 4.3.2.2.2 . PAs may also request load rating assistance from IDOT by submitting IDOT Form BLRS 06510, "Local Agency Load Rating Request". If a rapid response is critical, the LBU shall be contacted directly to notify them of the situation.

Due to limited resources, IDOT is typically not able to assist PAs with load rating major bridges requiring under bridge inspection equipment or man-lifts to inspect, or bridges serving high traffic volume roadways requiring traffic control to safely inspect. In these cases, the load rating must be completed by the PA or their designated representative.

IDOT must concur with load ratings completed by PAs or their designated representative and any subsequent weight restrictions. Maintaining documentation of IDOT concurrence with weight restrictions is especially important for effective enforcement of load postings.

All PA load ratings shall be submitted to the LBU at dot.localbridgeunit@illinois.gov. This submittal shall include:

- AASHTOWare BrR model (.xml file) or model from another program if AASHTOWare BrR is not capable of load rating the specific structure type
- Design Plans
- Shop Drawings, if available
- Rehab Plans, if applicable
- PDF of Analysis Output
- Inspection Sketches and Photographs, if model includes deterioration
- BBS 2795: Structure Load Rating Summary (SLRS)

4.3.1.3 Other Bridges

Other bridges include those which are not under IDOT or PA jurisdiction. Border state bridges are those with maintenance responsibility assigned to the adjacent state. These bridges may or may not carry state or PA highways.

Design and legal load ratings for bridges not under IDOT or PA jurisdiction must be completed by the bridge owner or their designated representative. Except for bridges owned by border states or federal agencies, IDOT must concur with the load rating and any subsequent weight restrictions if the bridge serves a state or PA highway. Maintaining documentation of IDOT concurrence with weight restrictions is especially important for effective enforcement of load postings. Load ratings and weight restrictions for bridges not serving state or PA highways do not require IDOT concurrence.

Regardless of the owner or the highway system, all load ratings and any weight restrictions must be submitted to IDOT for inclusion in ISIS.

4.3.2 Load Rating Frequency

4.3.2.1 Initial Load Rating

All new bridges and new structures in the NBI or covered by the IDOT Small Bridge Inspection Program require an initial analytical load rating. For practical reasons, the initial load rating is typically performed as part of the design process. The variables used in the initial load rating must then be verified with the as-built bridge. If variations are noted affecting the safe load capacity of the bridge, a new load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated. For PA structures, the initial load rating should be completed in conjunction with the Final Design Plans. An as-built load rating submittal is required, if there were changes during construction that affect the load carrying capacity of the structure. An as-built load rating shall be submitted to the LBU prior to the structure being open to traffic.

4.3.2.2 Revised Load Ratings

When the safe load capacity of a bridge has changed, a load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated.

4.3.2.2.1 Modified Permanent Loads

A load rating is required when a new feature is added to a bridge that increases or redistributes the existing permanent loads on a bridge. Common examples of this are the addition of a new or modified wearing surface or the attachment of new utilities to a bridge.

When a project proposes to modify existing permanent loads, a preliminary evaluation should be completed early in the project development phase to determine the project’s effects on the safe load capacity of the bridge. The preliminary evaluation findings may affect the project's scope of work. After construction, the as-built improvements must be reviewed to verify the variables used in the preliminary evaluation are accurate. A final load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated.

Existing permanent loads on a bridge are sometimes unintentionally modified as part of routine roadway maintenance. A typical example is when an oil and chip treatment is placed on a roadway and the bridge is not omitted. When this happens, the existing load rating must be reviewed to determine if the modifications warrant a new load rating.

Load ratings must be performed and submitted to the BBS for concurrence for resurfacing projects down to a zero-inch increase in surface thickness to verify the safe load capacity of the bridge. The condition of the bridge must be considered.

4.3.2.2.2 Structural Deterioration

IDOT policies for load rating deteriorated bridges are as follows:

- Perform load rating inspection when "Deck Condition" (SNBI Item B.C.01) rating drops to "3" or less, and for subsequent drops in the condition rating.
- Perform load rating inspection when "Superstructure Condition" (SNBI Item B.C.02), "Substructure Condition" (SNBI Item B.C.03), or "Culvert Condition" (SNBI Item B.C.04) rating drops to "4" or less, and for subsequent drops in the condition rating.
- Perform load rating inspection when "Bridge Bearings Condition" (SNBI Item B.C.07) rating drops to a "2" or less, and for subsequent drops in the condition rating.
- After an initial load rating inspection of precast prestressed concrete box beams (deck beams) exposed to de-icing chemicals, continue performing load rating inspections at 2-year intervals for condition ratings of "4" and 1-year intervals for condition ratings of "3" or less.
- After an initial load rating inspection for all other structure types, continue performing load rating inspections at an interval determined by the BBS based on bridge type and anticipated deterioration rate. This interval shall not exceed 10 years.

Although IDOT has established procedures to automatically initiate load ratings based on condition ratings, the BBS must be contacted immediately if an inspection finds a condition that may significantly affect a bridge's safe load capacity.

4.3.2.2.3 Structural Damage

When structural damage is identified, a preliminary evaluation may be required to determine the effect of the damage on a bridge's safe load capacity. The preliminary evaluation will help determine a scope of work to repair the damage and identify temporary measures to be implemented until the repairs are completed.

After repairs are made, photographs and/or construction plans of the as-built improvements must be submitted to the BBS to verify the variables used in the preliminary evaluation are still accurate. A final load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated.

4.3.2.2.4 Bridge Rehabilitation

A new load rating is required when a bridge is rehabilitated. Depending on the scope of work, a rehabilitation could include changes to permanent loads, capacity of load carrying members, or allowable travel way.

When a project proposes to rehabilitate a bridge, a preliminary evaluation should be completed early in the project development phase to determine the project's effect on the bridge's safe load capacity. This preliminary evaluation is critical in defining the project's scope of work. After construction, the as-built improvements must be reviewed to verify the variables used in the preliminary evaluation are accurate and the load rating shall be updated as required. A final load rating must be documented, submitted to the BBS if applicable, and ISIS information updated.

Although pavement milling/scarification reduces the overall dead load, it may also reduce the overall capacity of the bridge. The capacity of concrete slab bridges and composite decks on beams or stringers must be reduced to account for the loss of structural thickness caused by milling/scarification.

Funding for some rehabilitation projects is contingent on reestablishing a minimum level of the bridge's safe load capacity. Therefore, it is imperative to have IDOT concurrence with the project scope and preliminary evaluation before proceeding with a rehabilitation project.

4.3.2.2.5 Temporary Measures

Temporary measures can be utilized to keep a bridge in service until permanent measures can be installed. These measures may involve temporarily increasing a load carrying member's capacity, providing alternative load paths within the bridge, or altering the allowable travel way. A special inspection shall be assigned while temporary measures are in place.

A preliminary evaluation should be completed to ensure the proposed temporary measures adequately restore the bridge's safe load capacity. After implementation, the as-built improvements shall be reviewed to verify the variables used in the preliminary evaluation are accurate and the load rating shall be updated as required. A final load rating must be documented, submitted to the BBS if applicable, and ISIS information updated.

Both the design load rating and legal load rating shall include the contribution of the temporary measures. Except for the case of altering the allowable travel way, all structures utilizing temporary measures shall be restricted to legal loads only.

Temporary measures in place more than five years shall be considered a permanent part of the bridge. The load ratings after this point shall consider the temporary measures as permanent measures, except when timber is used. When timber is used as a temporary measure, any positive contribution of the timber shall be ignored after five years. Thus, the load rating will ignore any bracing, stability, or transfer of load previously assumed.

4.3.3 Evaluation Methods

The analytical method to use for design and legal load ratings depends on the AASHTO specification used to design the original bridge. IDOT policies for this are given in Table 4.3.3-1: Rating Method for Existing Bridges and Table 4.3.3-2: Rating Method for New or Rehabilitated Bridges .

Original Design Specifications	Acceptable Rating Method
Allowable Stress Design (ASD)	Load Factor Rating (LFR)
	Load & Resistance Factor Rating (LRFR)
	Allowable Stress Rating (ASR) (*See Note 1*)
Load Factor Design (LFD)	Load Factor Rating (LFR)
	Load & Resistance Factor Rating (LRFR)
Load & Resistance Factor Design (LRFD)	Load & Resistance Factor Rating (LRFR)

Table 4.3.3-1: Rating Method for Existing Bridges

Notes:

2. ASR can only be used for timber and/or masonry bridges. Non-NHS bridges with design and legal load ratings using ASR completed prior to 1994 are allowed to remain in ISIS.

Construction Type	Design Specification	Acceptable Rating Method
New	LFD	LFR
	LRFD	LRFR
Rehabilitation	LFD	LFR
	LRFD	LRFR

Table 4.3.3-2: Rating Method for New or Rehabilitated Bridges

IDOT's policy regarding acceptable analytical rating methods is based on a FHWA Policy Memorandum "Bridge Load Ratings for the National Bridge Inventory" dated November 5, 1993 and the subsequent follow-up memoranda dated December 22, 1993 and October 30, 2006 which may be found at:

https://www.fhwa.dot.gov/legsregs/directives/policy

4.3.3.1 ASR and LFR Methods

Design and legal load ratings based on the ASR and LFR methods shall follow the provisions in the AASHTO MBE and AASHTO Standard Specifications.

4.3.3.2 LRFR Method

Design and legal load ratings based on the LRFR method shall follow the provisions in the AASHTO MBE and AASHTO LRFD Specifications except as modified herein.

IDOT provides the following condition and system factors for use with design and legal load ratings:

Structural Condition of Members	φ_c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

Table 4.3.3.2-1: Condition Factor: φ_c

Superstructure Type	φ_s
Welded Members in Two-Girder/Truss/ Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing > 12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

Table 4.3.3.2-2: System Factor φ_s for Flexural and Axial Effects

The system factors in Table 4.3.3.2-2: System Factor φs for Flexural and Axial Effects shall only be applied when checking flexural and axial effects at the strength limit state of typical spans and geometries. A constant value of φ_s = 1.0 shall be applied when checking shear at the strength limit state or when evaluating flexure and shear of timber members.

4.3.3.3 Rational Evaluation Method

When the use of empirical methods is justified and approved by IDOT, the design and legal load rating can be assigned using the rational evaluation method. This method is based on the premise that the inventory rating, operating rating, and load posting are proportional to each other and can be assigned based on condition ratings. When using this method, the “Load Rating Method” (SNBI Item B.LR.04) shall be coded as “EJ” (Field evaluation and documented engineering judgement).

Using this methodology, Table 4.3.3.3-1 gives **suggested** rating factors and load postings for a given superstructure, substructure, or culvert condition rating:

Condition Rating	Load Posting (tons)			Rating Factor	
	Single Unit	3 or 4 Axle Combinations	5 or More Axle Combinations	Design IRF	Design & Legal ORF
Good to Excellent No signs of structural deterioration or distress.	No Restriction			1.00	1.67
Fair Initial evidence of structural deterioration or distress.	No Restriction			0.80	1.33
Poor Some structural deterioration or distress. *(See Note 1)*	No Restriction			0.70	1.17
	LLO			0.60	1.00
Serious Advanced structural deterioration or distress evident. *(See Notes 1 and 2)*	14	21	28	0.42	0.70
	8	12	15	0.23	0.39
Critical Severe structural deterioration or distress evident.	5			0.14	0.24

Table 4.3.3.3-1: Rational Evaluation Method Rating Factors and Load Postings

Notes:

1. The upper and lower values relate to the seriousness of the deterioration or distress (i.e. is it closer to Poor / Fair (upper) or Critical / Serious(lower)). Rating factors and load postings may be linearly interpolated between condition categories.

2. The load rater may alternatively require a single load posting based on the Single Unit load posting. Weight restrictions on township roads are typically single load postings.

4.3.3.4 Load Testing Method

Design and legal load ratings based on load testing shall follow the provisions of the AASHTO MBE.

IDOT collaborated with the Illinois Center for Transportation and the University of Illinois at Urbana-Champaign on a research project for load testing of concrete bridges, ICT R27-205: Development of Bridge Load Testing Program for Load Rating of Concrete Bridges. The final report for this project can be viewed at: https://doi.org/10.36501/0197-9191/24-025. See the Appendix for the Manual for Using Wireless Instrumentation for Field Testing of Concrete Bridges.

4.3.4 Transient Loads

Transient loads may include pedestrian loading if there is expectation of high pedestrian usage. A reduced pedestrian loading may be appropriate.

4.3.4.1 Design Load Rating

The transient load used for a design load rating following either the ASR or LFR evaluation method is the HS20 loading defined in the AASHTO Standard Specifications. The transient load used for a design load rating following the LRFR evaluation method is the HL93 loading defined in the AASHTO LRFD Specifications. For all three evaluation methods, the appropriate impact load must also be included. Design load rating factors are reported to FHWA each year.

4.3.4.2 Legal Load Rating

The transient loads used for a legal load rating consist of several suites of vehicles. The transient loads are the same regardless of the evaluation method. For all three evaluation methods and each suite of vehicles, the appropriate impact load must also be included.

4.3.4.2.1 Illinois Legal Vehicles

The Illinois Legal Vehicles are shown in Table 4.3.4.2-1 and Table 4.3.4.2-2. The Illinois Legal Vehicles are notional loads, not actual vehicles. This means they produce load effects enveloping the load effects produced by the AASHTO Legal Loads and the Illinois Statutory Loads. This includes the Specialized Hauling Vehicles (SHVs) defined in the AASHTO MBE.

The IL-PD6-40 vehicle is only applicable in the negative moment region over interior supports. The IL-PD6-200' is only applicable for span lengths or influence lengths greater than 200 feet.

For span lengths or influence lengths greater than 200 feet, the transient load used for a legal load rating shall consist of a truck train composed of two trucks, spaced 30 feet apart head to tail, using each of the Illinois Legal Vehicles shown in Table 4.3.4.2.1-1: Illinois Legal Vehicles (Single Unit Vehicles) and Table 4.3.4.2.1-2: Illinois Legal Vehicles (Combination Unit Vehicles), in all lanes. The IL-PD6-40 and IL-PD6-200' are considered truck train configurations themselves, thus, do not require additional spacing at 30 feet. The Illinois Routine Permit Vehicles shown in Table 4.3.4.2.2-1: Illinois Routine Permit Vehicles and Emergency Vehicles shown in Table 4.3.4.2.3-1: Emergency Vehicles shall also be evaluated. However, truck train configurations are not required for these vehicles.

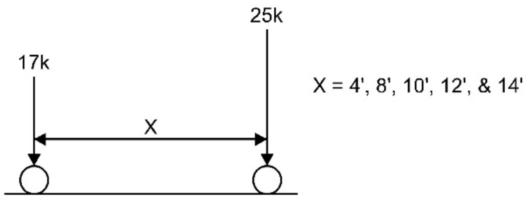
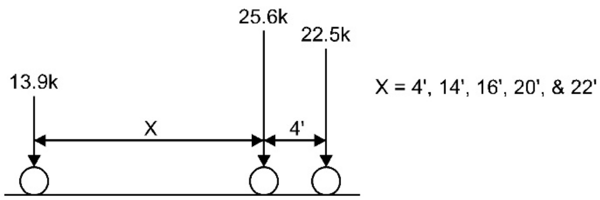
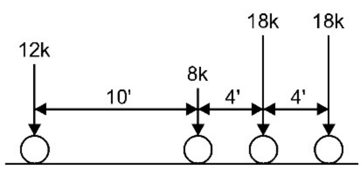
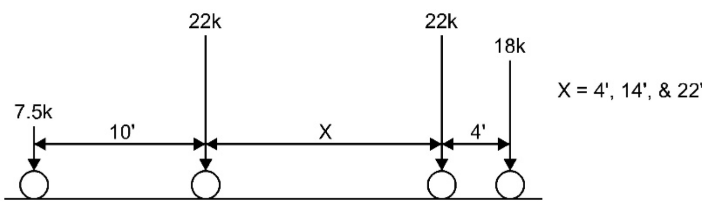
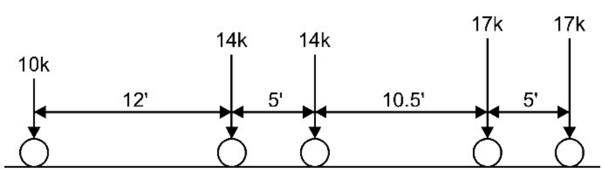
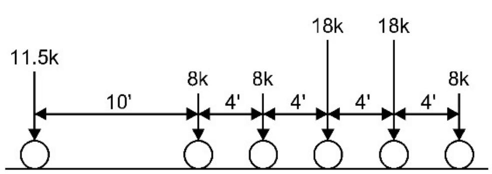
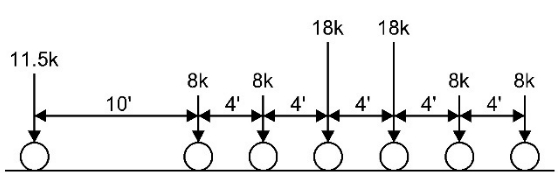
Vehicle Configuration	Name & GVW
	IL-PS2-21 GVW = 21 Tons
	IL-PS3-31 GVW = 31 Tons
	IL-PS4-28 GVW = 28 Tons
	IL-PS4-34.75 GVW = 34.75 Tons
	IL-PS5-36 GVW = 36 Tons
	IL-PS6-35.75 GVW = 35.75 Tons
	IL-PS7-39.75 GVW = 39.75 Tons

Table 4.3.4.2.1-1: Illinois Legal Vehicles (Single Unit Vehicles)

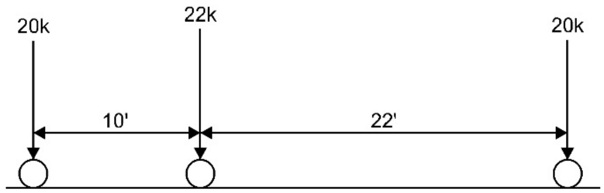
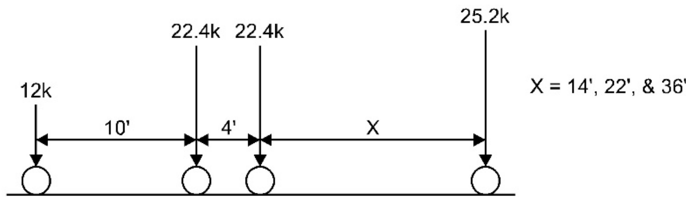
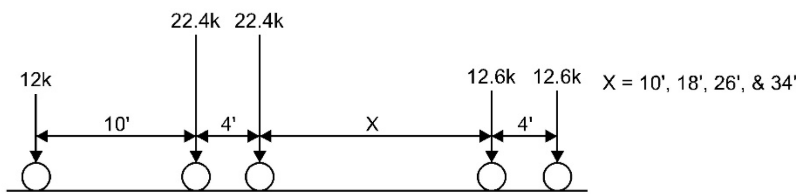
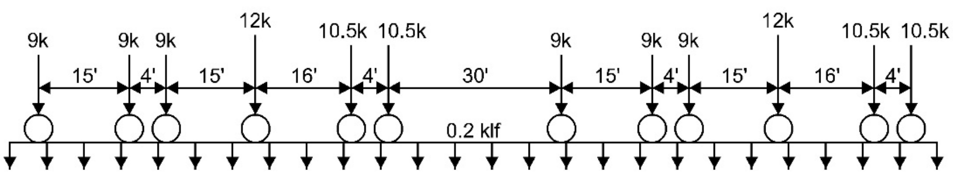
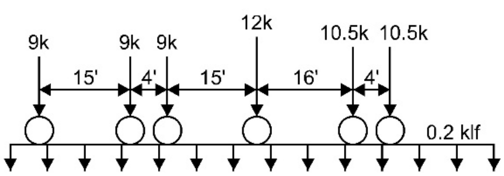
Vehicle Configuration	Name & GVW
	IL-PC3-31 GVW = 31 Tons
	IL-PC4-41 GVW = 41 Tons
	IL-PC5-41 GVW = 41 Tons
	IL-PD6-40 GVW = 40 Tons *(See Note 1)*
	IL-PD6-200' GVW = 40 Tons *(See Note 2)*

Table 4.3.4.2.1-2: Illinois Legal Vehicles (Combination Unit Vehicles)

Notes:

1. Only used for negative moments and interior reactions.
2. Used for all force effects when span length or influence length exceeds 200 feet.

4.3.4.2.2 Illinois Routine Permit Vehicles

The Illinois Routine Permit Vehicles are shown in Table 4.3.4.2.2-1: Illinois Routine Permit Vehicles. The Illinois Routine Permit Vehicles are notional loads, not actual vehicles. This means they produce load effects enveloping the load effects produced by limited continuous operation overweight permits allowed by the ILCS.

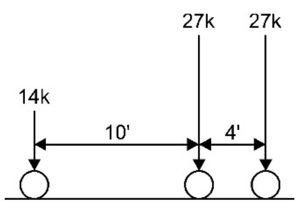
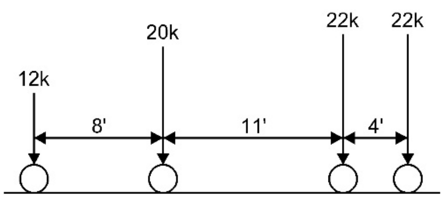
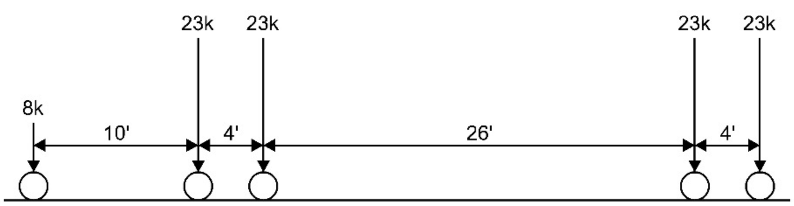
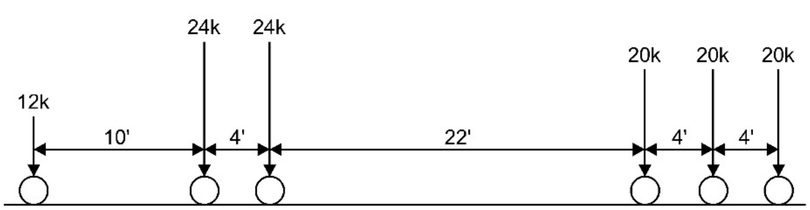
Vehicle Configuration	Name & GVW
	IL-RS3-34 GVW = 34 Tons
	IL-RS4-38 GVW = 38 Tons
	IL-RC5-50 GVW = 50 Tons
	IL-RC6-60 GVW = 60 Tons

Table 4.3.4.2.2-1: Illinois Routine Permit Vehicles

4.3.4.2.3 Emergency Vehicles

Emergency Vehicles are shown in Figure 4.3.4.2.3-1: Emergency Vehicles. The Emergency Vehicles are notional loads, not actual vehicles. Emergency Vehicles must be evaluated as part of every legal load rating.

In general, Emergency Vehicles shall initially be evaluated similar to other legal load rating transient loads. However, this may result in an overly conservative legal load rating. If the rating factor for an Emergency Vehicle is below 1.00, the following modifications may be used:

- Emergency Vehicles need only to be considered in a single lane of one direction of the bridge with a multiple presence factor equal to 1.0.
- A live load factor of 1.3 may be utilized for both LFR and LRFR evaluations.
- Subject to approval by IDOT BBS Rating Unit, the recommendations of NCHRP 20-07 / Task 410 Load Rating for the Fast Act Emergency Vehicles EV-2 and EV-3 may be used.

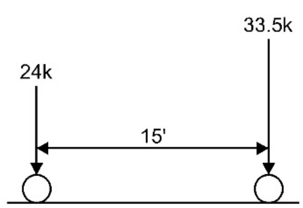
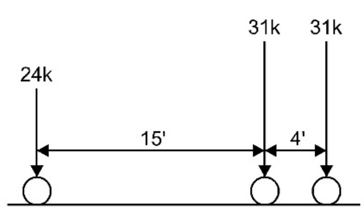
Vehicle Configuration	Name & GVW
	EV2 GVW = 28.75 Tons
	EV3 GVW = 43 Tons

Table 4.3.4.2.3-1: Emergency Vehicles

4.3.5 Weight Restrictions

Load ratings can be determined at both the inventory and operating levels, which represent a range of safe load capacities for the bridge. Bridge weight restrictions are based on the operating level. The BBS must be contacted for concurrence when the load rater feels a situation may warrant more conservative weight restrictions.

4.3.5.1 Implementing Weight Restrictions

After calculating a rating factor for each of the Illinois Legal, Routine Permit Vehicles and Emergency Vehicles, the load rater will choose one of the following courses of action for the bridge:

- No weight restriction
- Restrict to Illinois Statutory Loads (Legal Loads Only)
- Restrict below Illinois Statutory Loads (Load Posting)
- Close the bridge
- When choosing a course of action, the load rater should consider the enforceability of the weight restrictions. If there is concern significant disregard of the load posting will occur, the load rater may recommend closure of the bridge in the interest of public safety.

Occasionally, emergency situations will require a bridge owner to use judgment and temporarily load post or close a damaged or deteriorated bridge until an inspection and load rating can be completed by the appropriate personnel. These emergency load postings or closures may have limited legal enforceability, but owners are encouraged to implement and maintain such measures for the safety of the traveling public. While these temporary measures may be implemented immediately, the BBS must be notified as soon as practical for concurrence with their continued use.

4.3.5.1.1 No Weight Restrictions

When operating rating factors are greater than or equal to 1.00 for **all** Illinois Legal, Routine Permit Vehicles and Emergency Vehicles, the bridge can safely carry all legal loads and all routine permit vehicles. However, if weight restrictions are not required but the operating rating factor from the design load rating is less than 1.00, the SLRS must include commentary documenting the findings of the analysis.

4.3.5.1.2 Weight Restricted to Illinois Statutory Loads (Legal Loads Only)

When operating rating factors are greater than or equal to 1.00 for **all** Illinois Legal Vehicles and greater than or equal to 0.84 for both Emergency Vehicles but less than 1.00 for **any** Illinois Routine Permit Vehicle, the bridge can safely carry all legal loads but not all routine permit vehicles. Thus, the bridge must be restricted to Legal Loads Only. If an agency allows routine permit vehicles to travel on roadways under its jurisdiction, Legal Loads Only bridges shall be posted using the appropriate signage per the MUTCD and IMUTCD.

4.3.5.1.3 Weight Restricted Below Illinois Statutory Loads (Load Posting)

When operating rating factors are less than 1.00 for **any** Illinois Legal Vehicle or 0.84 for either Emergency Vehicles, the bridge cannot safely carry all legal load configurations and the bridge must be load posted unless temporary measures are used to mitigate the load posting. The use of temporary measures must be documented on the SLRS.

The allowable weight limits used to load post bridges are based on the allowable GVW. The allowable GVW is determined by multiplying the operating rating factor for a given Illinois Legal Vehicle or Emergency Vehicle by the vehicle's GVW.

The Illinois Legal Vehicles are divided into three posting groups as shown in Table 4.3.5.1.3-1: Combination Posting Method Groups. A maximum allowable GVW limit is determined for each posting group.

Posting Group	Illinois Legal Vehicles
Single Unit	IL PS2-21
	IL-PS3-31
	IL-PS4-28
	IL-PS4-34.75
	IL-PS5-36
	IL-PS6-35.75
	IL-PS7-39.75
Combination Unit - 3 or 4 Axles	IL-PC3-31
	IL-PC4-41
Combination Unit - 5 or More Axles	IL-PC5-41
	IL-PD6-40
	IL-PD6-200

Table 4.3.5.1.3-1: Combination Posting Method Groups

The maximum allowable GVW limits for each posting group is determined as follows:

<u>Single Unit</u>: Determine the allowable GVW for all Illinois Legal Vehicles in the “Single Unit” posting group with an operating rating factor less than 1.00. The maximum allowable GVW for the “Single Unit” posting group is the smallest of these values.

<u>Combination Unit - 3 or 4 Axles:</u> Determine the allowable GVW for all Illinois Legal Vehicles in the “Combination Unit – 3 or 4 Axles” posting group with an operating rating factor less than 1.00. The maximum allowable GVW for the “Combination Unit – 3 or 4” posting group is the smallest of these values.

<u>Combination Unit - 5 or More Axles:</u> Determine the allowable GVW for all Illinois Legal Vehicles in the “Combination Unit – 5 or More Axles” posting group with an operating rating factor less than 1.00. The maximum allowable GVW for the “Combination Unit – 5 or More Axles” posting group is the smallest of these values.

The maximum allowable GVW for Emergency Vehicles with operating rating factors less than 0.84 is the smallest allowable GVW of the EV2 and EV3 vehicles. However, in almost all cases where the Emergency Vehicle operating rating factors fall below 0.84, the “Single Unit” posting group will have a maximum allowable GVW limit that controls over the Emergency Vehicles. In the rare cases when the Emergency Vehicle maximum allowable GVW limit does control over the "Single Unit" posting group, the "Single Unit" posting value shall be lowered to match the Emergency Vehicle maximum allowable GVW limit.

Consideration must be given to practical limitations when determining allowable weight limits. The minimum "Combination Unit - 3 or 4 Axle" load posting is 12 Tons and the minimum "Combination Unit - 5 or More Axle" load posting is 14 Tons. The AASHTO MBE requires a bridge to have a minimum allowable weight limit of 3 Tons to remain open. However, IDOT's policy more conservatively limits this to 10 Tons on the state system and generally 5 Tons on the PA system.

A load posting may restrict the number of heavy vehicles allowed to simultaneously use a bridge. This restriction is referred to as a "One Truck at a Time" (OTAT) load posting. IDOT discourages OTAT load postings and restricts its use to bridges carrying roadways with low traffic volumes and adequate sight distance for opposing vehicles to easily identify one another and reduce speed.

After a weight restriction has been determined, ISIS must be updated accordingly.

<u>4.3.5.2 Posted Weight Restriction Signage</u>

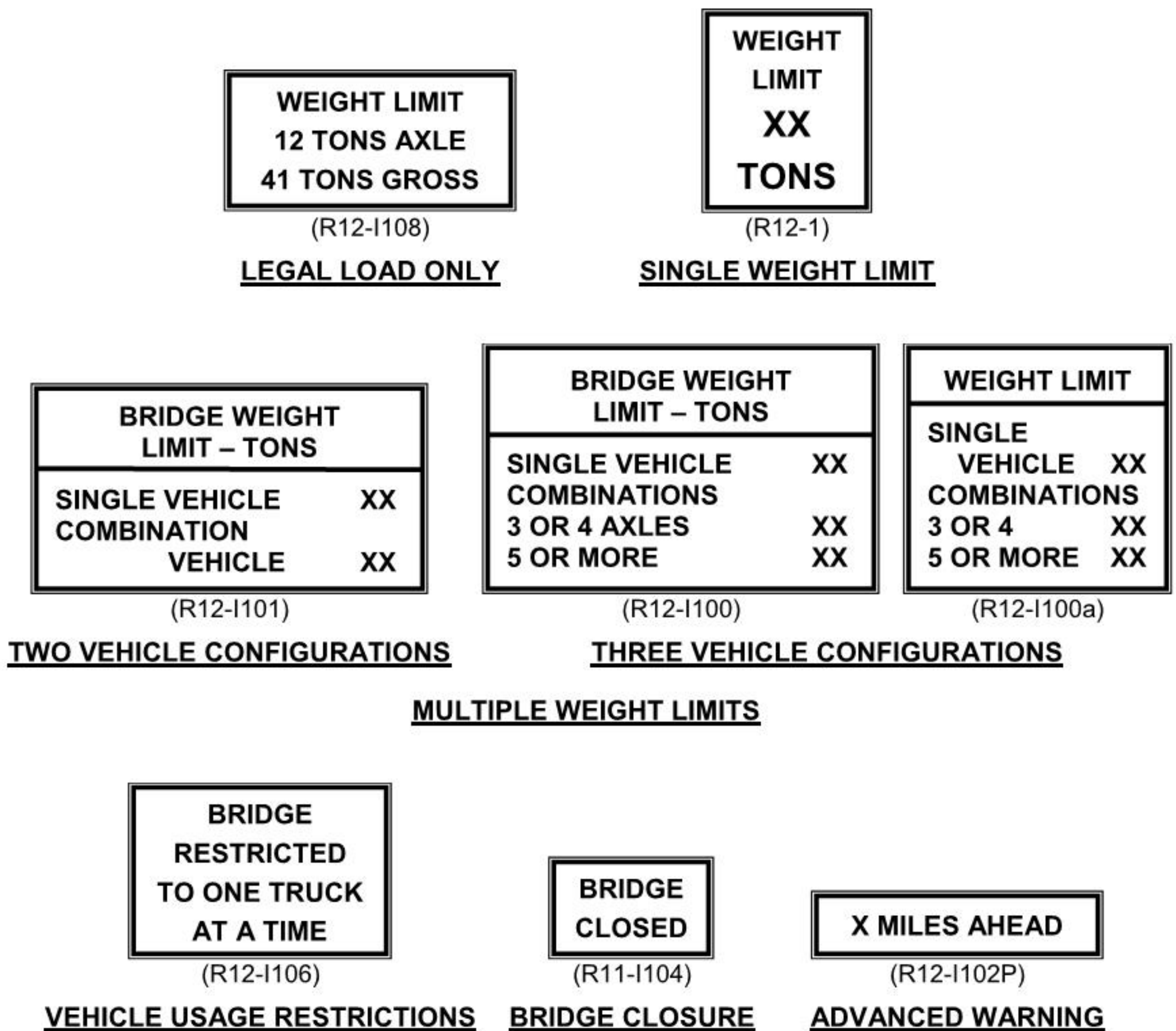
When the decision is made to implement a load posting or closure, it is the responsibility of the agency with jurisdiction of the roadway carried by the bridge to erect the necessary signage and barricades. This must be completed within 30 days of notification for load postings and within 24 hours of notification for closures.

To ensure violations of the weight restrictions can be successfully enforced, the signage must comply with the MUTCD and IMUTCD. The provisions in the IMUTCD shall supersede the MUTCD when in conflict. In addition, owners of state or PA highways must have documentation of IDOT concurrence with the weight restriction and the posted weight limits <u>must match the allowable weight limits given in that documentation</u>.

The agency with roadway jurisdiction must notify appropriate school districts, emergency service providers, and other affected agencies of the weight restrictions as appropriate. School buses and emergency vehicles are not exempt from posted weight limits.

The agency with roadway jurisdiction must monitor signage and barricades for repair or replacement needs. The frequency of monitoring should be appropriate for the classification of the roadway carried by the bridge. Although IDOT standard procedures require routine NBIS safety inspections to review signage and barricades in the vicinity of the bridge, additional inspections should be performed more often when reasonable.

See Section 3 for additional posting and closure requirements. See the IDOT BLRS Manual for additional PA bridge posting and closure requirements.



Note: Sign designations in parentheses are from the MUTCD, IMUTCD, and Illinois Standard Highway Signs Book.

Figure 4.3.5.2-1: Typical Load Posting Signs

4.3.5.3 Removing and Altering Weight Restrictions

Once a weight restriction has been established for a bridge, it cannot be removed or altered without approval from IDOT. Modifying weight restrictions requires rehabilitation of the bridge or the installation of temporary measures. The load rating procedures for these courses of action are included in Section 4.3.2.2.4 and Section 4.3.2.2.5, respectively.

IDOT **must approve** the installed improvements, final load rating, and proposed modified weight restrictions before the current weight restrictions can be removed or altered.

4.4 Overweight Permit Evaluation

The purpose of this section is to provide policies, procedures, and guidelines for evaluating bridges for overweight permits in Illinois. General information on issuing overweight permits is included for context purposes only. The detailed policies and procedures governing the issuing of overweight permits by an agency should be documented by the bridge owner.

The two types of overweight permits are a routine permit and a special permit. Routine permit vehicles may mix in the traffic stream and move at normal speeds without any movement restrictions but may be restricted to specific routes. Special permit vehicles are usually heavier and have more restrictions than routine permit vehicles. Each of these permit types has unique bridge evaluation requirements.

4.4.1 Assignment of Responsibility

The Illinois Vehicle Code assigns the responsibility of issuing permits to the highway owner. When a bridge and the highway it carries are under separate jurisdiction, a permit must also be obtained from the bridge owner. A separate permit must be obtained from each individual highway and bridge owner along every route the permit load plans to utilize.

Information governing IDOT's issuance of permits is included in a document titled "Oversize and Overweight Permit Movements on State Highways". For IDOT owned highways and bridges, the IDOT Bureau of Operations processes the permit load applications and the BBS completes the required bridge evaluation.

Non-IDOT bridge owners are responsible for processing the applications for permit loads to cross their bridges. Each agency should have policies and procedures in place to ensure permit load applications are properly evaluated. Overweight permit evaluations must be performed with the oversight of an Illinois Licensed Structural Engineer.

4.4.2 Routine Permit Evaluation

The load rating associated with the evaluation of routine permits is completed as part of a legal load rating. A routine permit authorizes the carrier to use any state route unless posted or restricted.

Routine permit applications are evaluated by comparing the proposed permit load to the Illinois Routine Permit Vehicles. To issue a permit, the proposed permit load must have an axle configuration similar to one of the Illinois Routine Permit Vehicles and axle weights equal to or less than those of that vehicle.

4.4.3 Special Permit Evaluation

A special permit application evaluation requires a special permit load rating comparing the safe load capacity of each bridge along a proposed permitted route relative to the proposed permit load. Special permit load ratings should follow the LFR method unless the bridge material or type precludes its use. ASR is used to evaluate timber and masonry load carrying members. LRFR is used on a selective basis when a less conservative, yet reliable load rating is required.

To issue a permit, the special permit load rating must result in an operating rating factor greater than or equal to 1.00 for that load.

The need to complete a load rating inspection is left to the discretion of the load rating team. This determination is typically based on the size of the permit load, the condition of the bridge, availability of as-built plans, inspection sketches and photographs, and whether a load rating inspection was completed for previous load ratings.

4.4.3.1 Transient Load Configurations

The proposed permit load is the transient load configuration used to evaluate a special permit. In order to complete a load rating, the following vehicle information must be provided:

- Gross Vehicle Weight
- Number of axles (utilized to carry the gross vehicle weight)
- Weight on each axle (steering axle to trailing axle)
- Axle spacing (distance center to center of axles)
- Non-standard gauge information (distance center to center of wheels along the width of vehicle, standard gauge = 6 feet)
- The load rating of more complex permit loads may also require the configuration of the individual wheel loads on each axle.

4.4.3.2 Permit Restrictions

When a load rating follows standard procedures and determines a bridge does not have the safe load capacity required to issue a permit, special restrictions may be placed on the permit loads movement reducing the load effects on load carrying members. The load rating may be evaluated again with a restriction or a combination of restrictions in place. If the safe load capacity of the bridge is then sufficient, the permit may be issued with specific restrictions.

Some permits may require a police escort or other means of supervision to ensure compliance with permit restrictions.

4.4.3.2.1 One Lane Loaded

When a permit restriction specifies one lane loaded, the live load distribution factors given in either the AASHTO Standard Specifications or the AASHTO LRFD Specifications can be adjusted accordingly. This permit restriction is commonly specified as, “One lane loaded, at least 300’ between nearest vehicle.”

4.4.3.2.2 Load Position

A permit restriction may specify a permit load place its axles in a specific position relative to load carrying members. This may include specifying the permit load straddle the longitudinal centerline of the bridge or use methods such as crabbing (i.e. staggering front and rear axles in separate lanes) to distribute the load over more beam lines. Coordination with law enforcement may be required.

4.4.3.2.3 Reduced Impact Loading

A permit restriction may specify a permit load reduce speed to limit the effects of impact. It is generally acceptable to eliminate the effects of impact from a load rating when the permit load speed is 5 mph or less. It is generally acceptable to reduce the effects of impact to 10% for speeds up to 45 mph when there is a smooth riding surface at the approaches, along the bridge deck, and at expansion joints.

4.4.3.3 Temporary Measures

When a load rating determines a bridge does not have the safe load capacity required to issue a permit, the permit applicant may choose to investigate the use of temporary measures. Temporary measures may include systems that spread the load to additional superstructure members, provide alternative load paths for the structural system, strengthen existing load carrying members, or span partially or completely over the existing bridge.

The development and use of temporary measures are the responsibility of the permit applicant. An Illinois Licensed Structural Engineer must prepare and seal the plans and specifications for the proposed measures. The owner must approve the temporary measures before they are implemented, and a permit must be issued.

4.5 Construction Load Rating

A construction load rating is the evaluation of a bridge's safe load capacity relative to a specific special construction load along with other transient loads that may be allowed to access the bridge simultaneously. The load rating results are used to determine if a special construction load can be approved to access the bridge.

Approving special construction loads is the responsibility of the bridge owner.

4.5.1 Project Development Phase

Material Transfer Device: A Material Transfer Device (MTD) is used on hot-mix asphalt paving projects to aid in transferring material from the trucks to the paver. The axle spacing and weights are typically significantly higher than legal loads. For this reason, bridges within the paving contract limits must be evaluated for an MTD. A typical empty MTD configuration is Axle 1 = 38,500 pounds, Axle 2 = 40,500 pounds, spaced 14'-4". For more information see IDOT All Regional Engineers memo; "Special Provision for Material Transfer Devices".

BCR, TSL, and Final Plans: The Engineer of Record shall evaluate whether the existing bridge or any portion of the existing bridge planned for reuse is structurally adequate. It is the engineer's responsibility to notify the owner if the condition of the existing bridge warrants interim repairs, select member replacement, structural support, load restrictions, or bridge closure. For more information see IDOT All Deputy Directors of Highways memo, "Consultant Plan Responsibility for Existing Bridges".

4.5.2 Project Implementation Phase

Demolition Plan: The contractor shall submit a demolition plan to the owner for approval, including the proposed methods of demolition and the location(s) and type(s) of equipment to be used for the removal of existing bridge structures or bridge decks (except for box culverts), when such work will be adjacent to or over an active roadway, railroad, or waterway designated on the plans as "Public Waters". The demolition plan shall include an assessment of the bridge condition and an evaluation of the capacity and stability of the bridge during demolition. The plan shall be sealed by an Illinois Licensed Structural Engineer.

Special Construction Loads: The configuration and magnitude of a special construction load relies on the contractor's means and methods. The contractor is responsible for completing a load rating to ensure the bridge can safely support the anticipated loads.

For IDOT projects, the contractor must follow the procedures of IDOT Construction Memorandum No. 06-39, "Transportation or Operation of Heavy Equipment on Pavement or Bridges Within the Contract Limits" and the IDOT *Standard Specifications for Road and Bridge Construction* for obtaining permission for special construction loads within the limits of the construction section. IDOT Construction Memorandum No. 06-39 provides a means for the contractor to submit some common special construction loads for the BBS to complete the load rating. The contractor must allow IDOT a minimum of ten working days to complete the load rating. For other special construction loads specific to a contractor's means and methods, the contractor must retain the services of an Illinois Licensed Structural Engineer to complete the load ratings. The contractor must submit the sealed load ratings along with supporting calculations to IDOT for review and approval.

For non-IDOT projects, the contractor must follow the policies and procedures of the bridge owner for obtaining permission for special construction loads within the limits of the construction section. Construction load ratings must be completed and sealed by an Illinois Licensed Structural Engineer. When a special construction load is unusual compared to those typically employed during project implementation, the Engineer of Record for the bridge being constructed should be given the opportunity to review and comment on the evaluation.

Outside of the construction section, the contractor must obtain a special permit from the highway owner.

Structural Assessment Reports: Structural Assessment Reports (SARs) are required on a contract-by-contract basis when Guide Bridge Special Provision 67, "Structural Assessment Reports for Contractor's Means and Methods" is included with the contract. Two primary criteria are as follows:

1. For any portion of the bridge that is not reused, the effects of the applied loads shall not exceed their capacity at operating level.
2. For any portion of the bridge that is reused, the effects of the applied loads shall not exceed their capacity at inventory level.

4.5.3 Evaluation Methods

Construction load ratings may utilize the ASR, LFR, or LRFR evaluation method. No preference is placed on any method. IDOT construction load ratings follow the LFR method unless the bridge material or type precludes its use. ASR is used to load rate timber and masonry load carrying members. LRFR is used on a selective basis when a less conservative, yet reliable load rating is required.

Section 5 Tunnel Inspection

5.1 Purpose and Scope

The highway bill Moving Ahead for Progress in the 21st Century Act (MAP-21) required the Federal Highway Administration (FHWA) to establish national standards for tunnel inspections. The National Tunnel Inspection Standards (NTIS) (23 CFR 650 Subpart E – National Tunnel Inspection Standards) contains the regulatory requirements for the tunnel inventory and inspection program. The NTIS can be obtained from the Federal Registrar using the link below:

https://www.govinfo.gov/content/pkg/CFR-2019-title23-vol1/xml/CFR-2019-title23-vol1-part650.xml

This section of the IDOT *Structural Services Manual* provides documentation of the official tunnel inspection policies for the State of Illinois.

A tunnel is defined as: "an enclosed roadway for motor vehicle traffic with vehicle access limited to portals, regardless of type of structure or method of construction, that requires, based on the owner's determination, special design considerations to include lighting, ventilations, fire protection systems, and emergency egress capacity. The term "tunnel" does not include bridges or culverts inspected under the National Bridge Inspection Standards (23 CFR 650 Subpart C – National Bridge Inspection Standards)."

5.1.1 National Tunnel Inspection Standards

The FHWA administers the NTIS under the guidelines outlined in the FHWA *Specifications for the National Tunnel Inventory* (SNTI) and the FHWA *Tunnel Operations, Maintenance, Inspection and Evaluation Manual* (TOMIE). The inventory and inspection information collected, as required by the NTIS, is reported to the FHWA and recorded in the National Tunnel Inventory (NTI).

The SNTI contains information on conducting inspections and for submitting the inventory and inspection data to the FHWA. The TOMIE provides uniform and consistent guidance on the operation, maintenance, inspection, and evaluation of tunnels.

The NTIS, SNTI, TOMIE, and NTI were developed to ensure tunnels continue to provide safe, reliable, and efficient levels of service for the traveling public.

To maintain full compliance with the NTIS, the Illinois Department of Transportation (IDOT) will adhere to all policies, procedures, and regulations established by the FHWA. The Statewide NBIS Program Manager or the District One Bridge Maintenance Engineer will act as the Statewide NTIS Program Manager overseeing the NTIS Program for all tunnels in the state.

The primary purpose of this section is to provide information pertaining to tunnel inventory and inspection activities. The information provided in this section summarizes IDOT inspection policies and guidelines for the effective and efficient management of the tunnel inspection program. The information provided regarding inspection types and frequencies is also applicable to structures under the jurisdiction of agencies other than IDOT, where the oversight for inspections is the responsibility of the agency having jurisdiction.

The primary function of the tunnel inspections performed in accordance with the NTIS is to ensure tunnels serving roadways in Illinois remain safe for all users of the highway system. The results of the inspections are also used as a tool to assist in determining maintenance and improvement needs.

5.1.2 Illinois Tunnel Inspection Organization

Illinois complies with the NTIS program requirements for inspection and inventory data through the following responsible positions.

<u>Statewide NTIS Program Manager</u>: The IDOT Bridge Management & Inspection Unit Chief within the Bureau of Bridges & Structures or the District One Bridge Maintenance Engineer, acting as the Statewide NBIS Program Manager, shall provide statewide oversight for all NTIS related activities. The Statewide NTIS Program Manager is responsible for inspection policy and to ensure the quality of the NTIS program.

<u>District NTIS Program Manager</u>: A IDOT District Bridge Maintenance Engineer may be designated as a District NTIS Program Manager as needed. A District NTIS Program Manager is responsible for providing oversight for all NTIS related activities within their designated area of responsibility which may include tunnels in other Districts.

<u>IDOT District 1 Area NTIS Program Manager</u>: The IDOT Area Bridge Inspection Engineers in District 1 providing oversight for all NTIS related activities for IDOT-maintained tunnels within their designated area of responsibility.

<u>Agency Designated NTIS Program Manager</u>: All public agencies with jurisdiction of a tunnel in the National Tunnel Inventory (NTI) shall designate a Program Manager to ensure compliance with the NTIS and provide guidance and management of their tunnel inventory. Public agencies may designate in-house staff or a consultant who is an approved Program Manager. The consultant and agency should discuss and clarify the program manager duties such as submitting inspection reports to IDOT, maintaining official tunnel files, responding to IDOT requests for additional NTIS information, and other NTIS related activities.

<u>NTIS Team Leader</u>: The team leader is the person on-site who is responsible for the entire inspection team of initial, routine, complex, or in-depth inspections. This person is responsible for inspection planning, preparation, performance, and reporting: including coordination of field work. The team leader is responsible for evaluating the deficiencies, reporting inspection data, and ensuring inspection reports are complete, accurate, and legible. The team leader shall provide recommendations for the repair of defective items and communicate to the responsible program manager when critical findings are discovered.

5.1.3 Illinois Structure Information System

The Illinois Structure Information System (ISIS) is the official database containing all Illinois bridge and tunnel inventory and inspection information. The data stored in ISIS for each tunnel includes an inventory record, current and previous inspection records, and information related to construction, reconstruction, highway routes, microfilm, and design.

5.1.4 Bridge Inspection System

The web-based Bridge Inspection System (BIS) is utilized by tunnel inspectors to record the results of the tunnel inspections.

5.2 Tunnel Inventory

5.2.1 Reporting Inventory Information

Each tunnel involved with a public road, shall be included in the tunnel inventory. In accordance with NTIS, tunnels carrying a public roadway shall be inspected and reported to FHWA. Directions for recording tunnel inventory information is provided in the FHWA *Specifications for the National Tunnel Inventory* (SNTI).

https://www.fhwa.dot.gov/bridge/inspection/tunnel/snti/hif15006.pdf

Tunnel inventory information, or changes to the tunnel affecting the inventory, shall be recorded in ISIS, within 90 days of opening to traffic or within 90 days of completion of inspection, completion of rehabilitation/repair work, or a change in status of the tunnel. In general, the inventory data is entered by Region/District personnel. Data entry for some items, such as load rating, is the responsibility of the Bureau of Bridges & Structures.

5.2.2 Updating Inventory Information

A general review of inventory items shall be a part of each Routine Inspection, with any needed corrections promptly reported to appropriate personnel, so the tunnel data in ISIS can be kept as accurate as possible. This general review of inventory information during the Routine Inspection does not necessarily require the inspector to take physical measurements but should include an effort to identify obvious errors in existing inventory information. The Statewide Program Manager will review the Federal Submittal for all tunnels in advance of the submittal.

5.2.3 The National Tunnel Inventory

ISIS data is submitted to FHWA annually. The data becomes part of the National Tunnel Inventory (NTI) which is defined by FHWA as:

National Tunnel Inventory: The NTI is a collection of information (database) covering over 500 of the Nation's tunnels located along public roads, including Interstate Highways, U.S. highways, State, County or Local roads. It presents a State-by-State summary analysis of the number, location, and general condition of highway tunnels within each State.

5.3 Tunnel Inspections

5.3.1 General

There are various types of tunnel inspections derived directly from the NTIS and other FHWA referenced publications and technical documents as follows:

Initial Inspection: The first inspection of a new tunnel or a tunnel that has undergone rehabilitation to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline tunnel conditions.

Routine Inspection: A regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the tunnel, to identify any changes from initial or previously recorded conditions, and to ensure the structure continues to satisfy present service requirements.

Special Inspection: An inspection scheduled at the discretion of the Statewide NTIS Program Manager or the responsible Program Manager, used to monitor a particular known or suspected deficiency, typically at a more frequent interval than the routine inspection allows.

Damage Inspection: An unscheduled inspection performed to assess structural damage resulting from environmental factors or human actions.

Complex Inspection: When a tunnel is complex, the Statewide NTIS Program Manager shall determine whether special procedures, increased training, or additional qualifications and experience are necessary to lead the inspection. In accordance with the NTIS, the type of construction, functional systems, history of performance, and the physical and operating condition of the tunnel shall be considered when determining the inspection requirements for complex tunnels.

In-depth Inspection: A close-up inspection of one, several, or all tunnel structural elements or functional systems to identify any deficiencies not readily detectable using routine inspection procedures; hand-on inspection may be necessary at some locations. In-depth inspections may occur more or less frequently than routine inspections, as outlined in the tunnel-specific inspection procedures.

Load Rating Inspection: A scheduled inspection performed to investigate damage or deterioration to evaluate potential reductions in the live load carrying capacity. The load rating analysis shall be prepared and reported in accordance with Section 4. Tunnels shall be posted or restricted, as appropriate, after conducting the load rating in accordance with the NTIS.

These various inspections are performed at intervals controlled by such things as structural condition, structure type and details, site conditions, and load capacity evaluation. SI&A data shall be entered into ISIS within 90 days of inspection for all tunnels regardless of jurisdiction.

NTIS related inspections shall be coordinated and performed by qualified personnel as stated in the NTIS requirements. The individuals with overall responsibility for controlling the quality of the NTIS inspection program in a specific area, designated as Program Managers, and the individuals leading field inspection teams, designated as Team Leaders, shall meet the qualifications specified in Section 5.6.2. An approved Team Leader shall be present during Initial, Routine, In-Depth, Damage, Load Rating, and Complex Tunnel Inspections.

NTIS requires all Routine Inspections, Special Inspections, In-Depth Inspections, and Complex Tunnel Inspections be performed within the inspection interval specified in Section 5.4. Program Managers shall ensure their NTIS inspections do not become delinquent.

Inspection procedures vary greatly depending on the characteristics of the tunnel, the inspection type, and the extent of deterioration. Detailed inspection procedures including guidance for taking and recording field measurements are provided in FHWA *Tunnel Operations, Maintenance, Inspection and Evaluation Manual* (TOMIE), AASHTO *The Manual for Bridge Evaluation*, and in this manual inspection procedures are provided for commonly encountered tunnel types and elements.

When taken as a whole, the element level data collected during the tunnel inspection will provide information on the overall safety and reliability of the structural, civil, and functional systems. The structural elements contained in the NTI database include tunnel liners, roof girders, columns and piles, cross passageways, interior walls, portals, ceiling slabs, ceiling girders, hangers and anchorages, ceiling panels, invert slabs, slabs on grade, invert girders, joints, and gaskets. The civil elements included in the NTI database are roadway wearing surfaces, traffic barriers, and pedestrian railings. The functional systems contained in the NTI database include the mechanical, electrical and lighting, fire and life safety, security, systems, sign, and protective systems.

If possible, tunnels should be observed during the passage of heavy vehicular loads to assess the presence of excessive vibration, deflection, or noise. If detected, further investigations shall be made to determine their cause. Complex tunnels require specific inspection procedures that are documented in an inspection plan for each Complex Tunnel as specified in Section 5.3.7.

5.3.1.1 Inspection Forms

The applicable IDOT forms shall be used to document each inspection. The latest IDOT inspection forms, available on the date of the inspection, shall be used. If the inspections are entered into BIS by the Team Leader and approved by the Program Manager no hard copy signature will be required. If not entered into BIS by the Team Leaders, the form shall be signed.

For situations where the Program Manager and the Team Leader are employed by separate entities, the Program Manager has ultimate responsibility for the quality of the inspections. The Program Manager may delegate quality control of the Team Leader’s work to others but shall require documentation of this for quality assurance purposes. This policy is in no way intended to discourage additional QC/QA measures within an inspection program but to clarify ultimate responsibility for that program.

5.3.2 Initial Inspection

An initial inspection shall be performed on rehabilitated highway tunnels within 90 days of the completion of all significant repair activities. If the tunnel was closed to complete the rehabilitation, the inspection shall be completed prior to reopening the tunnel to traffic. On new tunnels, the initial inspection shall be conducted after the completion of construction activities and the testing of functional systems but prior to opening the tunnel to traffic.

At a minimum, the initial inspection shall consist of a sufficient number of observations and measurements to determine the physical and functional condition of the tunnel. These inspections are intended to be comprehensive covering the structural, civil, mechanical, electrical and lighting, fire and life safety, security, signs, and protective systems. The results are to be recorded in accordance with the instructions contained in the SNTI.

The initial tunnel inspection establishes the baseline conditions of the tunnel; and it is used to field verify the initial tunnel inventory data. The baseline results can be used to evaluate changes over time to the tunnel systems and to help identify trends. The tunnel owner is responsible for performing the inspection in accordance with NTIS requirements and reporting the information to IDOT, allowing sufficient time for data entry by IDOT. Agencies should coordinate the timing of submittal of inspection data with IDOT.

Documents, including photographs, drawings (design, as-built, and shop), foundation information, pile driving records, and field changes shall be included in the Tunnel File as applicable. See Section 5.6.3.4 for Tunnel File content requirements.

5.3.3 Routine Inspection

Although commonly used to also determine tunnel maintenance and repair needs, the primary focus of the Routine Inspection is public safety.

Following the initial inspection, routine inspections are conducted within the intervals specified in the NTIS. (See Section 5.4) Routine inspections are regularly scheduled inspections that help to ensure continued safe, reliable, and efficient service. These inspections are similar in scope to the initial inspection. Routine tunnel inspections record the changes to the tunnel over time and can be used to help identify trends and predict future life expectancy of components.

Inspectors should understand how defects impact the function and capacity of tunnel systems. Tunnel inspectors shall be able to recognize the common deficiencies that impact the structural, civil, and functional systems. The observations and measurements used to carry out the inspection should be comprehensive. For each NTI tunnel element, the SNTI defines the general extent of deficiencies for each of the four condition states: good, fair, poor, and severe.

Inspection procedures are the written documentation of policies, methods, considerations, criteria, directions, and other conditions for planning and conducting tunnel inspections. Written procedures are used to enhance the overall effectiveness of the tunnel inspection program and to formalize the inspection process. Procedures should be developed to ensure adequate planning and scheduling takes place prior to conducting the inspection. The written inspection procedures shall also capture inspections for functional systems as appropriate for the tunnel.

Written procedures shall describe the requirements for:

- Inspection documentation, forms, and reports
- Record keeping and documentation requirements.
- Planning and scheduling to include unique structural or functional system characteristics.
- Inspection “best practices” and inspection techniques.
- Requirements for functional system testing, direct observation of critical system checks, and testing documentation.
- General, tunnel-specific, and specialized instructions.
- Specialized procedures, training, and experience for complex tunnels.
- Components to disassemble or clean.
- Measurements and survey control.
- Use of current technology and practices.
- Addressing critical findings and reporting to FHWA within 24 hours.
- Maintenance and protection of traffic during the inspection.
- Parking and staging areas during the inspection.
- Contact information for delegated maintenance / inspection responsibility
- Quality assurance and quality control implementation plans

Examples of tunnel specific procedures include identification of:

- Interval of Non-destructive Testing (NDT) or In-Depth inspections
- Tunnel elements, especially for Agency Defined Elements

Examples of specialized inspection procedures:

- Level of effort required to inspect certain unique tunnel components
- Certain details about accessing portions of the tunnel and/or tunnel equipment

At a minimum, routine inspections consist of a sufficient number of observations and measurements that are used to determine the physical and functional condition of the tunnel. These inspections are intended to be comprehensive covering the structural, civil, mechanical, electrical and lighting, fire and life safety, security, signs, and protective systems. The results are to be recorded in accordance with the instructions contained in the SNTI. The inspection shall accomplish the following functions:

- Verification and updating element ratings assigned to various items pertaining to the physical condition and the functionality of the tunnel.
- Identification and documentation of potential problems that may affect tunnel safety.
- Correction of inaccuracies in the tunnel inventory data. During Routine Inspections, inspectors are required to verify inventory data items.
- Determination of need for the Bureau of Bridges & Structures, or an Illinois Licensed Structural Engineer retained by an agency, to evaluate load carrying capacity or repair needs.
- Documentation of required maintenance work.

IDOT Form BBS-TIR, "Tunnel Inspection Report" contains “Comments” fields for all elements inspected during the Tunnel Inspection. A concise description of deficiencies is required to be included in the comment fields for elements with a Condition State of 2, 3 or 4, and encouraged for Condition State 1. The BBS-TIR for each Routine Inspection is kept in the official Tunnel File at the headquarters of the District or the Agency responsible for inspection. If entered directly into BIS, no hard copy of the inspection is required.

If a tunnel includes any members which meet the definition of “Non-Redundant Steel Tension Member (NSTM)” as stated in Section 3.6.5, that member shall receive a hands-on inspection during each routine inspection. Routine Inspection interval shall not exceed 24 months for any tunnel with Nonredundant Steel Tension Members. The inspection of those members shall be documented in the inspection plan for the tunnel, similar to the Nonredundant Steel Tension Members Inspection plan for bridges.

Tunnels with any significant quantity of critical structural elements (i.e. tunnel liners, roof girders, columns and piles, cross passageways, interior walls, portals, ceiling slabs, ceiling girders, hangers and anchorages, ceiling panels, invert slabs, invert girders) deemed to be in severe condition, a subsequent Load Rating Inspection as specified in Section 5.3.6 may be required to evaluate load carrying capacity. Such a condition rating is an indication that a main structural member may have deteriorated to a point where its load carrying capacity has been reduced.

After approving any tunnel inspection with any significant quantity of critical structural elements deemed to be in severe condition, the responsible Program Manager shall advise the Statewide NTIS Program Manager that a Load Rating Inspection will be required. The Statewide NTIS Program Manager will schedule an inspection and perform analysis as needed to determine the revised load carrying capacity of the deteriorated structure.

If an inspection reveals an imminent danger to the travelling public is likely, the inspector shall immediately take necessary action to protect the travelling public prior to notifying the Program Manager responsible for the structure and the Bureau of Bridges & Structures.

When the Bureau of Bridges & Structures cannot provide a Load Rating Inspection for an Agency within 90 days, the responsible Agency will be notified to retain the services of an Illinois Licensed Structural Engineer to evaluate the load carrying capacity of the affected tunnel. If the inspection findings are deemed urgent and require an immediate load rating, the Bureau of Bridges & Structures will require the evaluation of the load carrying capacity be expedited. The Structural Engineer’s recommendation shall be submitted to the Bureau of Bridges & Structures for concurrence and approval.

5.3.4 Special Inspections

A special inspection is typically performed after an initial, routine, damage, or in-depth inspection when significant deficiencies have been discovered and need to be monitored. Special inspections are scheduled based on the needs of the tunnel facility, inspection findings, and established written procedures. These types of inspections continue, but perhaps at adjusted intervals or durations, until the deficiency is repaired, the component is removed from service, or further study determines the conditions are no longer deteriorating at accelerated levels. For example, a light fixture built of dissimilar metals and installed over traffic might have problems with excessive corrosion. As such, this light fixture may be monitored on a regular basis to ensure it remains securely anchored and safe until repairs can be made.

Special Inspections are performed to monitor a specific structural feature, deficiency or condition that must be monitored more frequently than the Routine Inspection Interval. Special Inspections may be initiated by structural damage or deterioration, conditions affecting the stability of the structure, or for other reasons at the discretion of the responsible District or Agency Program Manager or the Bureau of Bridges & Structures. Some examples of concerns that may be cause for a Special Inspection are damage/deterioration to main load carrying members, existing or structural details with histories of poor performance. Procedures used during these inspections should be adopted in accordance with the specific deficiency or condition to be monitored.

Special Inspections for public agency tunnels are typically initiated by or after consultation with the Statewide NTIS Program Manager. Failure to comply with the inspection frequency and/or the established procedure for the required Special Inspection may result in posting, reduced posting, or closure of the tunnel.

See Section 3 and IDOT *Structure Information and Procedure Manual* (SIP Manual) for information regarding recording and performing Special Inspections.

5.3.5 Damage Inspections

Damage inspections are performed in response to natural disasters or human activities that damage the tunnel. Damage may occur by motor vehicle impact, fire, flood, earthquake, vandalism, or explosions. When severe damage occurs, the tunnel should remain closed until a damage inspection has been completed. Structural analysis and follow-up emergency repairs may be needed. Structural materials may need further evaluation as identified in the Tunnel Operations, Maintenance, Inspection, and Evaluation Manual for (TOMIE).

Safety is of paramount importance after an incident. Devices such as breathing apparatus, protective clothing, and specialized equipment may be necessary. Inspection work shall be coordinated with emergency responders. It is important the tunnel owner develop detailed plans and conduct training exercises with tunnel facility personnel in advance of these events.

Damage Inspections are performed on an emergency basis. The inspection shall be performed by District staff, Agency staff, Bureau of Bridges & Structures staff, or an Illinois Licensed Structural Engineer who is an Illinois certified Team Leader or Program Manager, for tunnels.

The scope of a Damage Inspection shall be enough to determine the need for emergency load restrictions or closure of the tunnel to traffic, and to assess the level of effort necessary to repair the damage. The level of effort required for a Damage Inspection may vary significantly, depending upon the extent of the damage. If major damage has occurred, the inspection team must evaluate the damaged members, determine the extent of section loss, take measurements for misalignment of members, and check for any loss of foundation support. It may be desirable to make on-site calculations to establish emergency load restrictions.

Photos, sketches, and detailed description documenting defects that potentially impact the load carrying capacity of the tunnel shall be transmitted from the field immediately to the Bureau of Bridges & Structures.

Any calculations or analysis performed as a result of a Damage Inspection shall be sealed by an Illinois Licensed Structural Engineer and submitted to the Bureau of Bridges & Structures for review and concurrence. A Damage Inspection may be supplemented by a timely Load Rating Inspection to document verification of field measurements and calculations. In addition, a more refined analysis may be warranted to establish or adjust interim load restrictions or required follow-up procedures.

5.3.5.1 Damage Inspections for State Maintained Tunnels

Damage Inspections are typically conducted by the Bureau of Bridges & Structures for state owned structures. However, the inspection may be performed by District personnel, at the direction of the Statewide NTIS Program Manager, for the measurement and recording of existing conditions. These inspections include measurement of main structural members to obtain detailed documentation of member size, section loss, structural defects, and member deflections/distortions.

Traffic control and special equipment are often necessary to accomplish these inspections. These elements of the inspection are described in the tunnel specific inspection procedures.

The results of these inspections, for state owned structures, are reviewed by the Structural Ratings and Permits Unit in the Bureau of Bridges & Structures, which performs an analysis to determine the load carrying capacity of each inspected structure. After completing the analysis, the Bureau of Bridges & Structures will provide the Region/District with documentation of the inspection and the results of the structural rating. The documentation provided to the Region/District will include field data and photographs, recommended revisions to condition ratings, updated Inventory and Operating Ratings, and recommendations for load restrictions and/or Special Inspections.

5.3.5.2 Damage Inspections for Public Agency Structures

For public agency tunnels, Damage Inspections should be performed by trained agency staff, or consultants.

Public agencies also have the option of employing the services of a qualified Illinois Licensed Structural Engineer to perform Damage Inspections, calculate the load ratings and provide posting recommendations. The Structure Load Rating Summary sheet (Form BBS 2795) shall be included with the Load Rating and submitted to the Bureau of Bridges & Structures for review and approval.

5.3.6 Load Rating Inspections

Load Rating Inspections are performed to confirm and document the variables affecting the safe live load carrying capacity of the tunnel. A Load Rating Inspection shall be scheduled whenever a ‘critical’ structural element is assigned a significant portion of the element into Condition State 4. Such a condition rating is an indication that a main structural member may have deteriorated to a point where its load carrying capacity as been reduced.

A critical structural element will be any element which, if deteriorated, will impact the load carrying capacity of the tunnel. These elements shall include but not be limited to: tunnel liners, roof girders, columns and piles, interior walls, portals, ceiling slabs, ceiling girders, hangers and anchorages, ceiling panels, invert slabs, and invert girders.

Expansion joints in the driving surface, slabs on grade and cross passageways are not considered as critical structural elements.

Load Rating Inspection personnel are required to be Illinois Tunnel Team Leaders and shall be qualified by training and/or experience to collect and document the information necessary to conduct a quality load rating analysis.

5.3.6.1 Load Rating Inspections for State Maintained Structures

Load Rating Inspections are typically conducted by the Bureau of Bridges & Structures, Structural Ratings and Permits Unit for state owned structures. The inspections may be performed by District personnel with guidance from the Statewide NTIS Program Manager, for the measurement and recording of existing conditions. These inspections include measurement of main structural members to obtain detailed documentation of member size, section loss, structural defects, and member deflections/distortions. Traffic control and special equipment are often necessary to accomplish these inspections.

The results of these inspections are reviewed by the Structural Ratings and Permits Unit in the Bureau of Bridges & Structures, which performs an analysis to determine the load carrying capacity of each inspected structure. After completing the analysis, the Bureau of Bridges & Structures will provide the Region/District with documentation of the inspection and the results of the structural rating. The documentation provided to the Region/District will include field data and photographs, recommended revisions to element level condition ratings, updated Inventory and Operating Ratings, and recommendations for load restrictions and/or Special Inspections.

5.3.6.2 Load Rating Inspections for Public Agency Maintained Structures

Load Rating Inspections are performed by the Bureau of Bridges & Structures Local Bridge Unit of the Bureau of Bridges & Structures. The inspections may also be performed by District personnel, public agency staff or consultants with guidance from the Statewide NTIS Program Manager,

If the Local Bridge Unit is unable to provide a Load Rating Inspection within a reasonable time frame or due to the complexity of the bridge, the Local Public Agency is notified to retain the services of a qualified consultant firm to perform the Load Rating Inspection and subsequent Load Rating. The documentation for the Load Rating Inspection and Load Rating must be submitted to the Local Bridge Unit for review and concurrence with the Load Rating, required weight restrictions, and/or Special Inspection requirements. IDOT Form BBS 2795 Structure Load Rating Summary shall be included with the Load Rating calculations and submitted to the Bureau of Bridges & Structures Local Bridge Unit for review and approval.

5.3.7 Complex Tunnel Inspections

A Complex tunnel is defined as a tunnel characterized by advanced or unique structural elements, or advanced functional systems.

- Extensive electrical systems such as switchgear, electrical distribution panels, computer and control systems/rooms, etc.
- Mechanical and ventilation systems such as pumps, fans, air monitors, etc.
- Fire suppression systems - such as sprinklers, hydrants, standpipes, fire alarm boxes, heat and smoke sensors, etc.
- Emergency Egress facilities – such as exit lighting/signage, exit corridors/passageways, refuge rooms, etc.
- Communication, monitoring, and security systems such as cameras, access restricted areas, phones, etc.
- Difficult inspection access methods
- Constructed of complex material or design methods.

The Statewide NTIS Program Manager is responsible for determining whether a tunnel is complex and for deciding what special procedures, increased training, or additional qualifications and experience are necessary to lead the inspection. When determining the inspection requirements for complex tunnels, the type of construction, functional systems, history of performance, and the physical and operating condition of the tunnel shall be taken into account in accordance with the NTIS.

<u>5.3.7.1 General Complex Tunnel Inspection Procedures</u>

A comprehensive Complex Tunnel Inspection Plan shall provide a detailed outline for conducting all aspects of the inspection process. The following is a brief outline for a detailed plan.

<u>Team Members</u>: The Program Manager shall select an experienced inspection team, trained for the specific tasks required for the unique structure. The Team Leader shall provide experience and guidance to the remainder of the inspection team. It is preferable that members of the team have previous experience inspecting the subject tunnel type and are familiar with any past issues and concerns. The Team Leader shall assign specific tasks for each member in order to conduct an efficient and thorough inspection.

Discipline Specific Specialists – When complex civil/structural, mechanical, or electrical systems need to be inspected, the team leader should assign discipline specific specialists with suitable training and experience to help conduct these inspections. Ideally, these specialist individuals should be licensed engineers.

Traffic Control: The inspection of a Complex Tunnel may require an extensive Traffic Control Plan. The responsible Program Manager / Team Leader shall coordinate the Complex Tunnel Inspection Plan with IDOT or Agency Traffic Operations, Maintenance, and Construction personnel to ensure the inspection operation, lane closures, or lane width restrictions do not interfere with other activities in the area. The plan should detail timing and sequence of operations, including lane closure sequences, signage, flagman responsibilities, and coordination with movement of inspection access equipment.

Access Equipment: A Complex Tunnel Inspection will typically require use of a wide variety of access equipment, including bucket trucks, manlifts, and ladders. The Complex Tunnel Inspection Plan should coordinate availability of equipment and operators for the entire inspection. Access to enclosed areas should be documented, including box girder access doors, tower entrance doors, etc.

Documentation: A Complex Tunnel Inspection will typically require a greater extent of documentation and shall therefore be compiled into an Inspection Report. The report shall contain all inspection notes, photos, and findings in an organized narrative. It should contain an overall description of the structure, a table or listing of significant findings, and recommendations for both general maintenance and any needed repairs. The findings should be prioritized by importance and severity. Any defect of a structural member that could potentially affect the load carrying capacity of the structure shall be reported immediately to the Statewide NTIS Program Manager Tunnel. See Section 5.3.8 for details and requirements for a Critical Finding.

5.3.8 Critical Findings

A Critical Finding is defined as a structural or safety related deficiency that may pose an imminent threat to the safety of the traveling public.

5.3.8.1 Identifying a Potential Critical Finding

Any of the following will be considered a potential critical finding:

- Damage from significant vehicle impacts, fire, severe flooding or other natural disaster
- Significant quantity of structural elements (i.e. tunnel liners, roof girders, columns and piles, cross passageways, interior walls, portals, ceiling slabs, ceiling girders, hangers and anchorages, ceiling panels, invert slabs, and invert girders.) deemed to be in severe condition
- Failure of mechanical, electrical, ventilation or other functional systems that may endanger the public
- Any unforeseen event the inspector considers to be a threat to the safety of the traveling public

When a potential critical finding is identified, the safety of the traveling public shall be the initial focus. The Team Leader shall take all necessary steps to ensure the tunnel is secured. If the Team Leader determines the identified defect may seriously reduce a tunnel's load carrying capacity, the Team Leader shall, if feasible, isolate the defect from traffic by closing lanes. If the defect is extensive enough where lane closures may be inadequate, the tunnel shall immediately be closed to traffic until further analysis can be performed.

5.3.8.2 Critical Finding Determination

When a potential critical finding is identified, information regarding the conditions that contributed to a potential critical finding shall be immediately provided by the Team Leader to the responsible District or Agency Designated NTIS Program Manager.

The responsible Program Manager shall provide sufficient, detailed information to allow the Statewide NTIS Program Manager to make an initial determination of the severity of the finding. The Statewide NTIS Program Manager will then work with the responsible Program Manager to determine if there is a need for a Damage Inspection, Load Rating Inspection, follow-up structural analysis, Special Inspection, submission of IDOT Form BBS CF 1, "Critical Finding Report" and to develop a plan of action to mitigate the deficiency.

If the Statewide NTIS Program Manager determines submission of a "Critical Finding Report" is not required, the responsible Program Manager shall follow the initial plan of action, unless conditions change or additional deficiencies are subsequently found.

5.3.8.3 Critical Finding Reporting

If the Statewide NTIS Program Manager determines the deficiency will require FHWA notification, the responsible Program Manager shall submit IDOT Form BBS CF 1, "Critical Finding Report" within 24 hours. The responsible Program Manager shall provide in sufficient detail all required information on the form and submit it to the Statewide NTIS Program Manager, retaining a copy for the Tunnel File. This shall include basic structure inventory and location information, a description of the deficiency, the immediate steps taken to ensure public safety, and a summary of the initial plan of action to mitigate the finding.

5.3.8.4 Submitting a Critical Finding to FHWA

The Statewide NTIS Program Manager shall report each Critical Finding to the Illinois Division Office of the FHWA within 24 hours of receiving the "Critical Findings Report" with updates on a regular basis until it is resolved. A complete file, containing all pertinent data, will be retained by the Bureau of Bridges & Structures for each Critical Finding.

The Statewide NTIS Program Manager shall send an annual report to the FHWA summarizing each critical finding identified during the cycle year inspection and update any existing unresolved critical findings from previous inspections. The report shall indicate the status and resolution of each critical finding. An annual report shall be filled regardless of whether or not a new critical finding has occurred.

5.3.9 Inspection of Structures under Construction

When a highway tunnel is open to public travel, it shall be inspected per the NTIS.

5.3.9.1 Existing Structure Rehabilitation

For an existing tunnel closed to public traffic during rehabilitation work, an NTIS inspection is to be completed prior to reopening the tunnel and the tunnel inventory data is to be updated within 90 days of completion of the work (all lanes open to public travel).

For an existing tunnel open to public traffic during rehabilitation work, regularly scheduled NTIS inspections are to be performed. If an NTIS inspection cannot be conducted due to reasonable circumstances such as a hazardous project site or conditions unfavorable to complete an inspection, then those circumstances shall be documented, in a written notice to the Statewide NTIS Program Manager for concurrence by the FHWA. The inspection is to be rescheduled at the earliest date possible. Once all risks have been mitigated, an NTIS inspection is to be completed and updated SI&A data is to be input within 90 days.

5.3.10 Inspection of Closed Structures

For a tunnel to be considered closed, it must be closed to traffic travelling through the tunnel, as well as traffic of any variety traveling over the tunnel. NTIS requirements do not extend to tunnels closed to traffic, however, all closed tunnels shall be inspected according to the NTIS unless the closure is permanent and approved by the Statewide NTIS Program Manager.

The following “Tunnel Status” (SNTI Item L4) code represent closed tunnels in the inventory:

Tunnel Status	Description
K	Tunnel Closed to all traffic

5.3.11 In-depth Inspections

In-depth inspections are close-up, hand-on inspections conducted on one, several, or all of the elements or functional systems. These inspections are used to identify deficiencies not readily detectable during initial, routine, or damage inspections. In-depth inspections may involve testing of tunnel system, components, and materials. More extensive disassembly and cleaning of equipment parts may occur. This type of inspection may be used to support a structural analysis or a functional system evaluation where more information is needed. In-depth inspections are scheduled based on the needs of the tunnel facility, inspection findings, and established written procedures.

5.4 Tunnel Inspection Intervals

5.4.1 General

The following sections document IDOT policy for the required frequency of different types of tunnel inspections.

Every tunnel shall receive periodic routine inspections. Those inspections shall be completed within 2 months (before or after) of the tunnel's assigned Routine Inspection Date (RID). In the case of a Special Inspection with an interval of twelve months or less, the inspection shall be completed by the due date.

5.4.2 Routine Tunnel Inspection Interval

The Routine Tunnel Inspection Interval may vary over the life of the structure. After the first routine inspection, a RID will be assigned by the Statewide NTIS Program Manager and the inspection interval will remain at 24 months until the tunnel requires a 12-month inspection interval as described in Section 5.4.2.1.

Each tunnel has an established NTIS Routine Inspection Date (RID) for determining timing of future inspections. Subsequent routine inspections are conducted within 2 months before or after the established RID, typically on 24-month intervals. The Statewide NTIS Program Manager can approve a modification to the RID with adequate justification, after notification to the FHWA Division office.

The NTIS recognizes certain tunnels in poor condition should be inspected at lesser intervals

5.4.2.1 Routine Inspection 12-Month Interval Criteria

Tunnels with the following characteristic shall receive a Routine Inspection at 12-month maximum intervals.

Tunnels with a significant quantity (i.e. greater than 50%) of structural elements (i.e. tunnel liners, roof girders, columns and piles, cross passageways, interior walls, portals, ceiling slabs, ceiling girders, hangers and anchorages, ceiling panels, invert slabs, and invert girders) or essential functional systems deemed to be in Condition State 3 or 4. The Statewide NTIS Program Manager shall make the final determination of when the inspection interval shall be reduced to 12-months, based on a review of the routine inspection reports.

5.4.3 In-depth Inspection Interval

In-depth inspections are scheduled based on the needs of the tunnel facility, inspection findings, and established written procedures. See Section 3 for more information. Intervals are determined by the Statewide NTIS Program Manager.

5.4.4 Special Inspection Interval

Special inspections shall be performed to monitor specific details, deficiencies, or conditions. Intervals shall be selected at the discretion of the Statewide Tunnel Program Manager.

5.5 Tunnel Load Rating, Posting or Restrictions

A load rating is required for all tunnels that have a structurally supported roadway system to carry vehicles within the tunnel bore or that are subjected to live load force effects from a roadway located above the tunnel. At-grade roadways in tunnels are exempt from load rating, however any roadway over the tunnel shall be evaluated for live load carrying capacity. The NTIS requires all tunnels to be load rated in accordance with Sections 6 or 8, AASHTO Manual for Bridge Evaluation (MBE). References in the MBE to the AASHTO LRFD Bridge Design Specification should be understood to instead refer to AASHTO *LRFD Road Tunnel Design and Construction Guide Specifications* 1st Edition, 2017, LRFDTUN-1. In addition, pertinent information related to tunnel load rating may be found in the FHWA's *Reference Guide for Load Rating of Tunnel Structures Publication No. FHWA-HIF-19-010,* Infrastructure Office of Bridges and Structures.

Load rating vehicles, procedures, documentation, reporting and subsequent postings, if required, shall be in accordance with applicable portions of Section 4 except as noted.

5.5.1 Assignment of Responsibility

5.5.1.1 State Tunnels

State tunnels are those under the jurisdiction of IDOT. Design and legal load ratings for state tunnels are typically completed by IDOT staff. When load ratings are completed by consulting engineers, IDOT must concur with the load rating and any subsequent weight restrictions.

5.5.1.2 Non-State Tunnels.

Non-state tunnels include those under the jurisdiction of counties, townships, road districts, municipalities, park districts, forest preserve districts, tollways, conservation districts, private owners and border states.

Design and legal load ratings for all new construction, rehabilitations, and other work affecting the safe load capacity of a non-state tunnel must be completed by the tunnel owner or their designated representative.

IDOT typically will not assist non-state tunnel owners in completing design and legal load ratings for damaged or deteriorated tunnels. When a Load Rating Inspection is required, the non-state tunnel owner shall have the inspection completed by the owner or their designated representative.

Except for tunnels owned by border states or federal agencies, IDOT must concur with load ratings and any subsequent weight restrictions submitted by the tunnel owner. Load ratings and weight restrictions for tunnels not serving state or local highways do not require IDOT concurrence.

Regardless of the owner or the highway system, all load ratings and any weight restrictions must be submitted to IDOT for inclusion in ISIS.

All non-state-owned tunnel load ratings shall be submitted to the LBU at dot.localbridgeunit@illinois.gov. This submittal shall include:

- AASHTOWare BrR model (.xml file) or model from another program if AASHTOWare BrR is not capable of load rating the specific structure type
- Design Plans
- Shop Drawings, if available
- Rehab Plans, if applicable
- PDF of Analysis Output
- Inspection Sketches and Photographs, if model includes deterioration
- BBS 2795: Structure Load Rating Summary (SLRS)

5.5.2 Load Rating Frequency

5.5.2.1 Initial Load Rating

All new tunnels in the NTI require an initial analytical load rating. For practical reasons, the initial load rating is typically performed as part of the design process. The variables used in the initial load rating must then be verified with the as-built tunnel. If variations are noted affecting the safe load capacity of the tunnel, a new load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated.

For non-state tunnels, the initial load rating should be completed in conjunction with the Final Design Plans. if there were changes during construction that affects the load carrying capacity of the structure, an as-built load rating submittal is required. An as-built load rating shall be submitted to the LBU prior to the structure being open to traffic.

5.5.2.2 Revised Load Ratings

When the safe load capacity of a tunnel has changed, a load rating must be completed, documented, submitted to the BBS if applicable, and ISIS information updated.

5.5.3 Weight Restrictions

Weight restrictions, when required, shall be in accordance with Section 4.3.5 except the word “tunnel” shall be substituted for “bridge”.

5.5.4 Overweight Permit Evaluation

Overweight permit evaluation, when required, shall be in accordance with Section 4.4 except the word “tunnel” shall be substituted for “bridge”.

5.5.5 Construction Load Rating

Construction load rating, when required, shall be in accordance with Section 4.5 except the word “tunnel” shall be substituted for “bridge”.

5.6 Quality Control and Quality Assurance

5.6.1 General

The National Tunnel Inspection Standards (NTIS) states agencies shall "Use systematic quality control and quality assurance procedures to maintain a high degree of accuracy and consistency in the inspection program. Include periodic field review of inspection teams, data quality checks, and independent review of inspection reports and computations." Quality in the inspection and the resulting documentation is an important aspect that must be considered to ensure the safety of the traveling public in tunnels. The definitions for QC and QA are as follows:

QC refers to quality related activities associated with the creation of project deliverables, i.e. the inspection results. QC is used to verify deliverables are of acceptable quality, complete and correct. Examples of QC activities include peer reviews of deliverable products from the inspection documentation and verifying the findings in the field are as recorded by the inspector.

QA refers to the process used to create a quality product or deliverable, (or in this case inspection) and may be performed by a manager, client, or even a third-party reviewer. Examples of QA for tunnel inspections include the establishment of guidelines or checklists that would lead to quality inspections, creating minimum inspector qualifications, establishing condition state requirements that yield consistency between inspection periods, requiring equipment be calibrated, having testing personnel submit certifications on the equipment being used, developing standard inspection forms with data fields included, and following standard processes/procedures for performing inspections.

Both QC and QA shall be performed to ensure the inspections conducted and deliverables produced meet the tunnel owner's quality requirements, while ultimately ensuring the public's safety.

IDOT recognizes established and documented Quality Control and Quality Assurance (QC/QA) procedures are essential for ensuring tunnel inspections are performed in an appropriate, consistent, and uniform manner by all Program Managers and Team Leaders employed by various agencies having tunnel inspection responsibilities throughout the state. Through the application of QC/QA procedures, agencies enhance their ability to obtain accurate inspection information required for determining load capacity and tunnel maintenance, repair, rehabilitation and replacement needs. Of utmost importance is the role QC/QA plays for ensuring tunnel inspection staff are adequately trained and experienced to readily identify conditions that adversely affect public safety.

5.6.2 Personnel Qualifications

The quality of an agency's bridge inspection program is very much dependent on the performance of the Program Manager in charge of the agency's inspection program, and on the Team Leaders leading the inspection teams. These individuals shall be qualified to perform their duties. IDOT has established procedures for reviewing, verifying and approving the acceptability of the education, training and experience of an individual to function as a Program Manager or Team Leader. IDOT Form BBS 2611, "Tunnel Program Manager Qualifications" and IDOT Form BBS 2621, "Tunnel Team Leader Qualifications" shall be used for documenting the qualifications of Program Managers and Team Leaders.

5.6.2.1 Statewide NTIS Program Manager / Program Manager (Tunnel)

The Statewide NTIS Program Manager is the individual in charge of the tunnel inspection program for a State with one or more tunnels in their jurisdiction. This person shall be capable of leading the tunnel inspection organization and ensuring the requirements of the NTIS are fulfilled. The Statewide NTIS Program Manager may delegate duties and responsibilities to qualified delegates, i.e. program managers, who take charge of a particular subset of tunnels; however, the Statewide NTIS Program Manager remains responsible for ensuring compliance. The delegated program managers shall remain in close communication with the Statewide NTIS Program Manager on all issues concerning compliance with the NTIS.

On behalf of the tunnel inspection organization, the program manager develops written procedures, schedules inspections, procures inspection and safety equipment, coordinates with tunnel facility staff, and advises the team leader as necessary. Ideally, the program manager has a general understanding of all aspects of tunnel engineering including design, construction, operation, maintenance, inspection, evaluation, load rating, and rehabilitation. Good judgment is essential for this position to respond appropriately to safety and structural concerns within the tunnel.

Refer to the NTIS for the complete requirements of this position. The program manager must be a registered professional engineer or have at least 10 years of tunnel or bridge inspection experience. This individual must also be a Nationally Certified Tunnel Inspector, which requires comprehensive training, end-of-course assessment, and periodic refresher training. Delegate program managers must be familiar with the requirements of the SNTI and TOMIE Manual, and will be included in the IDOT registry of Nationally Certified Tunnel Inspectors.

5.6.2.2 Team Leader (Tunnel)

The team leader is the person on-site in charge of the inspection team. This person is responsible for inspection planning, preparing, performing and reporting to include coordinating the field work. The team leader is responsible for evaluating the deficiencies, quality checking of the inspection data, and making sure the inspection reports are complete, accurate, and legible. The team leader shall also conduct safety briefings as needed. The team leader should be able to provide recommendations for the repair of defective items and must initiate appropriate actions when critical findings are discovered.

Refer to the NTIS for the complete requirements. A team leader must be a nationally certified tunnel inspector which requires comprehensive training, end-of-course assessment, and periodic refresher training. Additionally, the team leader is expected to meet at least one of the following:

- Licensed Professional Engineer and at least 6 months of tunnel or bridge inspection experience.
- 5 years of tunnel or bridge inspection experience.
- Appropriate combination of education and experience as described in the NTIS.

5.6.2.3 Discipline Specific Specialists

Discipline-specific specialists shall be qualified through education, training, experience, certification, and licensure as required by industry standards and norms for the systems being evaluated.

Specialists must demonstrate mastery of requirements set by trade organizations and code-writing bodies, including the National Fire Protection Association (NFPA), the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE), the National Electrical Code, the American National Standards Institute (ANSI), and the American Association of State Highway and Transportation Officials (AASHTO).

Additionally, specialists shall be proficient in using inspection tools and software, and they must possess strong analytical and communication skills to effectively report findings and collaborate with engineering teams.

5.6.2.4 Tunnel Inspection Training

Federal regulations require all personnel, including registered (note the term is “licensed” in Illinois) professional or structural engineers, managing tunnel inspection programs or directing inspections in the field, to have successfully completed the Federal Highway Administration (FHWA) Tunnel Safety Inspection (FHWA-NHI-130110). The course is primarily directed at members of Federal, State, local (Authority or Commission) and Tribal highway agency employees, who are involved with tunnel design, inspection and maintenance, as well as consultants involved in inspecting tunnels or in tunnel inspection management and leadership positions.

Prior to taking this course, individuals shall be a Certified Illinois NBIS Team Leader or Program Manager.

5.6.2.5 Tunnel Inspection Refresher Training

All Program Managers and Team Leaders shall receive 18 hours of NTIS tunnel inspection refresher training at intervals not to exceed 60 months. However, each Program Manager shall evaluate the performance and experience level of each Team Leader performing field inspections within their designated area of responsibility and, if necessary, establish an interval for tunnel inspection refresher training that is less than the maximum allowed to maintain inspection quality.

The “Tunnel Safety Inspection Refresher Training” (FHWA-NHI-130125) class is available through the NHI and satisfies NTIS tunnel inspection refresher training requirements.

5.6.3 Quality Control

Quality control and quality assurance programs are used to promote accuracy, ensure consistency, facilitate improvement, and help maintain a high level of reliability. Periodic field reviews of inspection team and their work, quality checks on data, and independent reviews of the inspection results shall also be part of the program. The use of checklists is recommended practice. Quality control refers to observations, monitoring, and performance testing to maintain the quality of the tunnel inspections and load ratings; these practices are usually performed continuously by the teams performing the work. Quality assurance is associated with a systematic approach to improve the overall program effectiveness, verify the accuracy of the quality control procedures, and ensure established standards are met; these procedures are performed independent of the inspection and load rating teams performing the work.

The quality control (QC) procedures established by IDOT are intended to define, monitor, and document the qualifications and performance of personnel engaged in the management of inspection programs, the performance of field inspections, and the load rating of tunnels.

Factors related to the maintenance of effective QC procedures are:

- Review of tunnel inspection reports
- Inspector's performance
- Personnel qualifications
- NTIS data verification
- Tunnel inspection refresher training
- Identifying special skills, training or equipment

5.6.3.1 Review of Tunnel Inspection Reports

To ensure the quality of a Team Leader's inspection reports, all reports shall be reviewed by the Tunnel Program Manager prior to final acceptance of the reports and subsequent data being entered into ISIS. Any inconsistencies or potential errors will result in rejection of the report with a required resubmission of a corrected report with updated NTI and element data within a reasonable timeframe. The depth of review shall be determined by the Tunnel Program Manager based on the experience level of the Tunnel Team Leader as well as the type, age and typical exposure of the tunnel. At a minimum, the review shall verify the element condition states are appropriate and the documentation meets the requirements of Section 5.3.3.

5.6.3.2 Inspector's Performance

The District NTIS Program Manager or Agency Designated NTIS Program Manager shall conduct in-depth reviews of the field procedures of all Team Leaders functioning under their supervision to ensure inspections are being performed in an appropriate, consistent, and uniform manner.

At least once every 48 months, the responsible program manager shall accompany each Team Leader to observe the performance of NTIS tunnel inspections on at least one (1) tunnel inspection. The program manager shall document the results of their observations for each Team Leader.

5.6.3.3 Personnel Documentation

The Bridge Management and Inspection Unit maintains documentation of the qualification approvals issued for all Program Managers and Team Leaders employed by IDOT and by other agencies. The Bridge Management and Inspection Unit also maintains documentation of performance deficiencies reported by the Statewide NTIS Program Manager for resolution.

Approved Tunnel Program Managers and Approved Tunnel Team Leaders shall be documented in the BIS Program.

The Statewide Tunnel Program Manager shall maintain a file containing documentation of education, professional registrations, training, and certifications received from each Program Manager, Team Leader and Nationally Certified Tunnel Inspector (NCTI) functioning under their direction.

In addition, the file shall contain documentation of Tunnel Inspection Performance Reviews performed to monitor the tunnel inspection procedures used by the Team Leader. The documentation for all inspection personnel functioning in a District or Agency should be located in a central location and readily accessible by the Statewide NTIS Program Manager.

5.6.3.4 Tunnel File

The available records for each tunnel facility shall be kept in the official Tunnel File. Important records that are normally part of the Tunnel File include the construction plans, shop drawings, working drawings, as-built drawings, specifications, cost-estimates, correspondence, photographs, material certifications, material test data, load test data, specific inspection procedures, and specific emergency response procedures. The history of the operating, inventory, maintenance, inspection, and repair records shall also be maintained. Also included are accident records, posting, and permit loads.

A separate file shall be maintained for each tunnel. The documentation for all tunnels in a District or Agency should be located in a central location that is readily accessible by the Team Leader.

Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual Section 4.3.3 provides the following guidance for the contents of a Tunnel File:

- Construction Plans
- Correspondence
- Photographs
- Maintenance, Inspection, and Repair records
- Accident Reports
- etc.

It is recognized that it is not practical or necessary to physically store all required items in the file. However, the actual location of each item may be referenced on the File Checklist. This can include separate file locations, plan stacks, electronic files, databases, and data in document storage systems, where applicable.

5.6.3.5 NTIS Data Verification

5.6.3.5.1 Recognizing Errors, Omissions, or Changes in the Field

The Tunnel Inventory data shall be reviewed in the field during each inspection and any errors brought to the attention of the Tunnel Statewide NTIS Program Manager.

5.6.3.5.2 Resolution of Inspection Errors or Omissions

During the review of inspection reports or the monitoring of field tunnel inspection procedures, the District or Agency Assigned NTIS Program Manager must note data errors or omissions and provide personnel responsible for their occurrence with directions that will eliminate similar incidents in the future. For minor reporting deficiencies, the initial resolution of the findings may be in the form of verbal instructions by the Program Manager to the Team Leader with a note to the Team Leader's file, if considered necessary by the Program Manager.

For major deficiencies, such as those resulting in the assignment of inappropriately high element condition state ratings to deteriorated tunnel elements or the omission of data concerning critical structural or functional deficiencies from an inspection report, the Program Manager must provide documentation to the Team Leader's file describing the inspection report deficiency and the measures taken to both correct the inspection deficiency and to prevent the reoccurrence of the inspection deficiency.

If the Team Leader continues to provide inspection reports or to perform field inspections in a manner that does not address a previously noted major deficiency, the Program Manager shall no longer utilize the Team Leader for NTIS tunnel inspection purposes, and the situation must be reported to the Statewide NTIS Program Manager for final resolution.

5.6.3.6 Identifying Special Skills, Training, and Equipment

Inspection teams performing Complex Tunnel or Functional System Inspections shall have skills, training, and equipment suitable for the inspection being performed. Discipline specific specialists are to be used when complex civil/structural, mechanical, or electrical systems require inspection. The team leader, with approval from the responsible program manager, shall assign discipline specific specialists with suitable training and experience to conduct these inspections. These specialist individuals should be licensed professional and structural engineers and shall have the education, training and experience put forth in the FHWA TOMIE Manual Sections 4.3.3 and 4.4.4.

5.6.4 Quality Assurance

Quality Assurance (QA) measures are required to ensure established Quality Control (QC) procedures are being followed and are effective for ensuring tunnel safety on all public roadways. Quality Assurance reviews of all agencies with highway tunnels are performed to assure the quality of tunnel inspections and tunnel load ratings. The Statewide NTIS Program Manager will oversee the Quality Assurance review which includes review of the tunnel inspection procedures and inspection reports prior to final approval.

5.6.4.1 Quality Assurance Reviews

Due to the limited number of tunnels in the state of Illinois, IDOT will conduct quality assurance reviews on a four - year basis to determine the adequacy and effectiveness of the quality control procedures utilized for tunnel inspections and load ratings. Depending on the cycle, this may mean no tunnel inspections are due every year. In these years, the tunnel will receive a cursory inspection to verify ratings. Reviews shall be performed on both the state tunnels and public agency tunnels in subsequent reviews. The intervals between these reviews may be reduced in response to agency personnel changes or to address findings from previous quality assurance reviews.

5.6.4.1.1 Office Reviews

Quality Assurance reviews include a review of the office files and procedures used to document inspections, to track personnel performance and qualifications, and to schedule follow-up inspections or repairs to address NTIS tunnel inspection findings. The procedures used to establish inspection schedules and to select Team Leaders and inspection personnel will be included in the Quality Assurance review. During the office review, the District and Agency Designated Program Managers and member of their staff included in the review process, will be requested to provide information describing the procedures employed within their designated area of responsibility to ensure tunnel safety and compliance with NTIS.

5.6.4.1.2 Field Reviews

Quality Assurance reviews include field observations of at least one tunnel within the Agency's jurisdiction.

The Statewide NTIS Program Manager or a delegated substitute shall be present during the Quality Assurance field review. Personnel conducting the Quality Assurance field review will refer to the Tunnel Files maintained by the Program Manager to compare the inspection information contained in the files to the conditions observed during the Quality Assurance field review. The most recent NTI data shall be obtained and comprehensively reviewed for accuracy. The Quality Assurance staff shall note inventory data errors or omissions as well as observed element level condition state-related discrepancies. Conditions that could affect load ratings will be noted during the Quality Assurance field review. Follow-up must occur to ensure the noted conditions have been considered in the current load rating. Any inventory data errors or omissions and element level discrepancies must be corrected and updated in ISIS in a timely manner prior to the next data submission to FHWA.

5.6.4.2 Final Quality Assurance Review Report

A “Final Quality Assurance Review Report - Tunnel” noting commendable practices, review findings, and the measures to be taken to address Quality Control deficiencies will be provided to the Program Manager. When requested and noted in the “Final Quality Assurance Review Report”, the reviewed Program Manager shall provide the Statewide NTIS Program Manager with documentation verifying corrective measures described in the report have been implemented.

IDOT will provide FHWA with a copy of every “Final Quality Assurance Review Report - Tunnel”.

Section 6 Appendix

6.1 ADD 20070810 ”Deterioration of Reinforced Concrete Structures”

Illinois Department of Transportation

Memorandum

To:	ALL DEPUTY DIRECTORS OF HIGHWAYS
From:	Ralph E. Anderson
Subject:	Deterioration of Reinforced Concrete Structures
Date:	August 10, 2007

When reinforced concrete structures are inspected, it is not always possible for personnel to directly observe the top surface of the structure. This condition is routinely encountered during the inspection of:

1) Reinforced Concrete (RC) Slab Bridges that have non-structural wearing surfaces placed on the superstructure
2) RC Box Culverts that support embankment materials and pavement

When assessing the extent to which the superstructure of a RC Slab Bridge or the top slab of a RC Box Culvert is deteriorated, inspection staff must base their assessment on a visual examination of the underside of the slabs. While visual inspection of the underside of slabs is typically sufficient for structures functioning within their expected service life, older concrete structures that have been exposed to the elements and deicing agents for extended periods may have deterioration present in the upper portions of their slab that cannot be identified visibly from below.

Conditions encountered during two recent projects for the removal and replacement of small RC Slab Bridges illustrated situations where visual inspection of the underside of the slab was not sufficient for assessing the condition of the superstructures. Both bridges had functioned far beyond what would be considered their expected service life, one having been built in 1917 (90 years old) and the other in 1928 (79 years old). The top surface of the concrete slabs were not visible for inspection on either bridge, and the underside of both bridges exhibited deterioration that required inspection staff to assess the superstructures as being in poor condition. The contract plans for both structures called for staged removal, with a portion of the existing superstructure carrying traffic during the first stage of construction for the new structure. While performing the first stage of structure removal, severe concrete deterioration was discovered in the top portion of the concrete slabs, to the extent that both bridges were immediately closed to all traffic for the duration of the bridge replacement operations. These experiences illustrate the need for bridge inspections and project scoping to include procedures that thoroughly assess the ability of aging or deteriorated concrete structures to carry traffic.

Subsequent to evaluating the conditions encountered during the removal of the two RC Slab Bridges previously described, the Bureau of Bridges and Structures (BBS) requested that concrete cores be taken for a sampling of older RC Slab Bridges and RC Box Culverts, with 4 structures being tested under the general direction of each Region/District Bridge Maintenance Engineer. The structures chosen for sampling were constructed prior to 1940 (at least 67 years old), which is the accepted/established date for the routine use of air entrained concrete in bridge construction. Based on the testing of the concrete cores, approximately 20% of the structures exhibited concrete deterioration that would 1) not be conducive to Staged Removal & Construction and, 2) could require a load restriction to be placed on the bridge.

To provide the BBS with information needed for load-carrying capacity analysis and to provide programming/planning personnel with information necessary for project scoping, Regions/Districts shall establish programs to obtain and test concrete cores from older and deteriorated RC Slab Bridges and RC Box Culverts.

Concrete cores should be obtained as soon as possible from all RC Slab Bridges and RC Box Culverts that have Superstructure or Culvert condition ratings of “4” or less (poor or worse condition), with priority given to structures that may be removed in stages and utilized to carry traffic during a construction project. Subsequent to the testing of the structures in poor or worse condition, all RC Slab Bridges and RC Box Culverts constructed prior to 1940 with Superstructure or Culvert condition ratings greater than “4” should also have concrete cores obtained, as resources become available.

Prior to utilizing staged removal and construction to replace a RC Slab Bridge or a RC Box Culvert, a staging feasibility evaluation shall be performed. When a Bridge Condition Report (BCR) is prepared, the staging feasibility evaluation, which includes testing of concrete cores obtained from the structure, is required to be included in the BCR. When a BCR is not required during the Phase I portion of the project, a staging feasibility evaluation, which includes coring and testing, shall be performed by the Region/District. If it is determined that a structure cannot be utilized for staged construction, other alternatives should be considered, such as the establishment of a detour route or the construction of a temporary span over the deteriorated bridge.

A sufficient number of concrete cores should be obtained from each identified structure, with at least one core being obtained within the limits of each traffic lane and within each span of the bridge. If possible and without causing undue disruptions of traffic, concrete cores should also be taken in the proximity of the centerline of the roadway. In all cases the concrete cores should be located at the midspan point (halfway between the substructure elements or walls supporting the slab). To facilitate testing, the concrete cores should have a nominal diameter of 4-inches. The information submitted should include a plan view of the structure indicating the location and numbering of the cores, photos of the cores and the tested compressive strength of each core. If it is not possible to obtain a solid core and a test cannot be performed, a description of the remaining concrete and/or a reason that a test could not be performed should be included. Some examples: the core may have cracked during retrieval; the core was all aggregate; the core was aggregate for the top

4 inches and the remaining core wasn't sufficient for testing. All information should be forwarded to Tim Armbrecht of this office for compilation and analysis.

Regions/Districts should establish screening procedures for use in establishing priorities for obtaining concrete cores among the structures to be tested. The BBS has discussed possible screening procedures with statewide personnel who have used hammer drills and/or coring-bits to assess the condition of concrete, but a best practice has not been identified. We request that Regions/Districts employ the tools/equipment available to them for the purpose of screening and that they document the effectiveness of their screening procedures. We anticipate that best practices for screening procedures will be established after Department personnel share their experiences at future Bridge Maintenance Engineer meetings.

Based on the results of the testing that has been accomplished to date, we anticipate that Regions/Districts will encounter, on some structures, deterioration that has advanced to a state where the concrete has fragmented or returned to an aggregate-like material. In these cases, intact cores will not be obtainable. When this condition is encountered, the BBS should be informed so that a load-carrying capacity assessment can be made and load restrictions established.

Since the two bridge projects described earlier in this memorandum were both addressing the replacement of existing Small Bridges, it is readily apparent that the inclusion of Small Bridge inventory and inspection information in the Illinois Structure Information System (ISIS) is an essential element for identifying and tracking bridges that shall be included in concrete coring and testing programs. In order for all affected structures to be readily identified, all Regions/Districts shall expedite the inclusion of inventory and inspection information for Small Bridges within the ISIS. Two memoranda from Ralph E. Anderson to "ALL DEPUTY DIRECTORS OF HIGHWAYS" were issued in regard to the establishment of the Small Bridge Inspection Program, with the first being distributed on April 8, 2005 and the second on July 2, 2007. These memoranda provide the guidance necessary for establishing records within the ISIS for Small Bridges.

If you have any questions, please contact Tim Armbrecht at (217) 782-6266.

JAM/TAA/2007.8

cc- Milt Sees
Dick Smith
Priscilla A. Tobias
Joseph Hill
Charles J. Ingersoll
Brian McPartlin (Illinois State Toll Highway Authority)
Sam Flood (Illinois Department of Natural Resources)
Norman R. Stoner (FHWA)

6.2 ADD 20080318 “Inspection and Coring of Reinforced Concrete Structures”

Illinois Department of Transportation

Memorandum

To:	ALL DEPUTY DIRECTORS OF HIGHWAYS
From:	Ralph E. Anderson Ralph E. Anderson
Subject:	Inspection and Coring of Reinforced Concrete Structures
Date:	March 18, 2008

This memorandum is a supplement to our August 10, 2007 memorandum "Deterioration of Reinforced Concrete Structures" to ALL Deputy Directors of Highways, in which procedures for evaluating older reinforced concrete slab and culvert structures were detailed. Since the distribution of that memorandum, it has become clear that the effort required to perform the tasks outlined in that memorandum will quickly overwhelm the resources of the Districts and the Bureau of Bridges and Structures. As a result, the intent of this memorandum is to outline new procedures and criteria that will prioritize the investigation of these structures based on recent study and evaluation. It is also anticipated that the new criteria may reduce the number of structures requiring cores to be taken. Note that these items apply to RC Slab Bridges and RC Box Culverts only.

Concrete cores should continue to be obtained as soon as possible from all RC Slab Bridges and RC Box Culverts that have Superstructure or Culvert condition ratings of "4" or less (poor or worse condition). However, if a "poor or worse" culvert condition rating is not due to the condition of the top slab of the culvert, cores are not required. In order to expediently complete the effort of obtaining the necessary cores and submitting reports to this office for those structures currently rated "4" or less, we are recommending a target date of April 30, 2009.

If staged removal of an RC slab bridge or RC Box Culvert is anticipated, cores must be taken as part of preparing a Bridge Condition Report (BCR). When a BCR is not required during the Phase I portion of the project, a staging feasibility evaluation, which includes coring and testing, shall be performed by the Region/District. If it is determined that a structure cannot be utilized for staged construction, other alternatives should be considered, such as the establishment of a detour route or the construction of a temporary span over the deteriorated bridge.

Personnel from this office have taken the opportunity to study a large number of core samples taken by District personnel. Based on this investigation, it appears that there may be a correlation between the visual condition of slabs and the concrete strength of cores taken from the slabs. As a result, for structures with condition ratings of "5" or greater, the Districts should be able to identify bridges for which cores should be taken, and those that would not require that a core be taken.

The primary visual quality of the slab that indicates that there is a high probability of a significant reduction in concrete strength is evidence of leaching, efflorescence and/or moisture in and around multiple closely-spaced cracks that run longitudinally along the soffit of the slab. If these conditions are detected during a routine NBIS inspection, a core should be scheduled as soon as possible to verify the strength of the concrete, and the results forwarded to this office. It is anticipated that engineering judgment will be necessary to determine if the conditions warrant that a core be taken. If there are any questions about the necessity of taking a core, please do not hesitate to contact this office. If the core results indicate that there is a significant reduction in concrete strength, the superstructure or culvert condition rating should be lowered to a “4” or less, which will initiate a damage inspection by this office, and a subsequent load rating.

The August 10, 2007 memorandum to All Deputy Directors of Highways indicated that “all RC Slab Bridges and RC Box Culverts constructed prior to 1940 with Superstructure or Culvert condition ratings greater than “4” should also have concrete cores obtained”. Based on the coring information that has been obtained to date, the need for obtaining cores is no longer to be based on the age of the structure. The need for taking cores is to be determined solely on visual observation of the underside of the slab, the condition rating assigned to the Superstructure or Culvert; and anticipated Staged Construction, as previously described in this memorandum.

After cores are taken from a structure, a core data form shall be filled out. A blank form and a completed example, which illustrates the information required by this office, are attached for your use. As discussed in the previous memorandum, at least one 4-inch diameter core should be obtained within each traffic lane for each span of the bridge, preferably along the centerline of a “wheel-path” at the midspan point. After cores have been taken, steps should be taken to perform full depth repairs on the holes in order to minimize future exposure of the slab to moisture, deicing agents and other destructive elements. All information should be forwarded to Mr. Steven Negangard of this office for review.

It is imperative that a detailed description of the core be provided in the event that it is not possible to obtain a solid core and a test cannot be performed. If the deterioration has advanced to a state where the concrete has fragmented or returned to an aggregate-like material, this is sufficient evidence that the slab has little or no compressive strength. In this case a load restriction will likely be required. Immediately notify this office in these cases and submit a core data form as soon as possible so that a load-carrying capacity assessment can be made and load restrictions established.

If you have any questions, please contact Carl Puzey at (217) 782-2125.

TAA/kktRCslabsADD-20080318

cc:	Milt Sees	Christine Reed	Dick Smith
	Joseph Hill	Charles J. Ingersoll	Brian McPartlin (ISTHA)
	Sam Flood (IDNR)	Norman R. Stoner (FHWA)	

BRIDGE CORE

Core Number ________________ **Element Cored** ________________

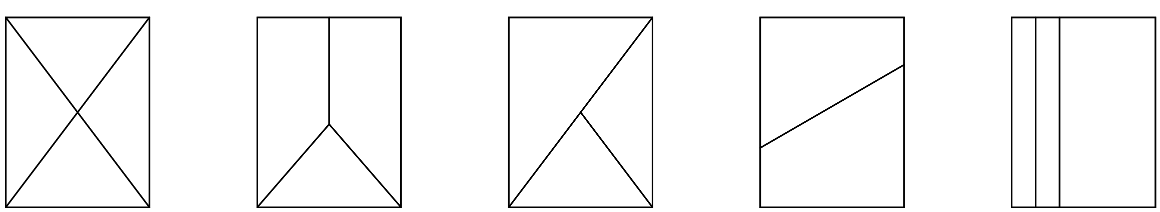
Length and Description

Segment	Length	Description
A		
B		
C		
D		
E		
F		
G		
H		
I		
J		
K		
L		
M		

Test Length __________ **Correction Factor** __________ **Diameter** __________ **Area** ____________

Factored Compressive Strength ________ **psi** **Actual Maximum Load** ________ **pds.**

Type of Fracture ____________________

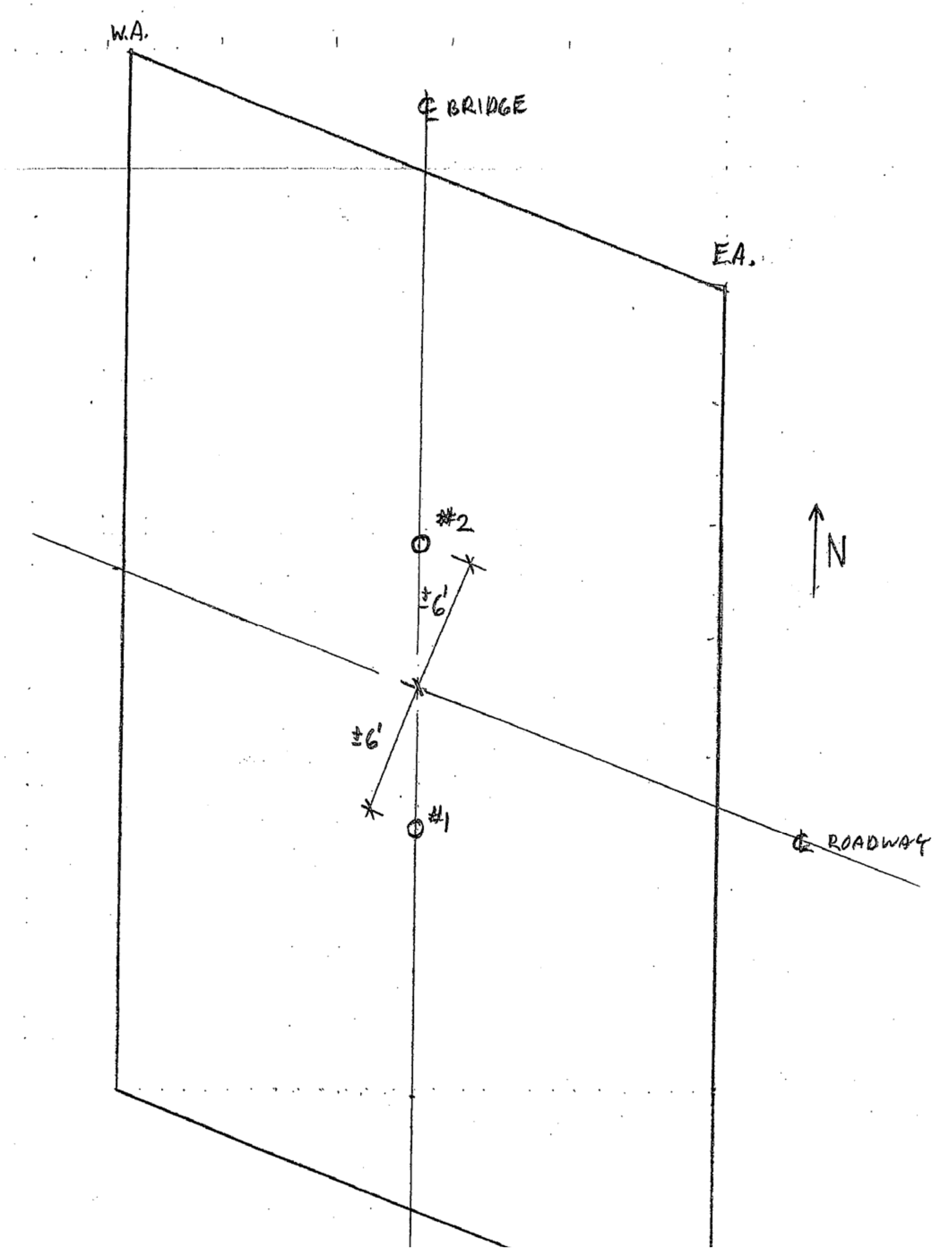


Cone Cone & Split Cone & Shear Shear Columnar

Age of Specimen __________ **Defects in Specimen** ______________________________

Coring report
Structure Number 090-0040

DISTRICT 4
10-10-2007
BY: Mark Eckhoff


W.A.
℄ BRIDGE
E.A.
#2
±6'
±6'
#1
℄ ROADWAY
N

BRIDGE CORE

Core Number 1 **Element Cored** slab

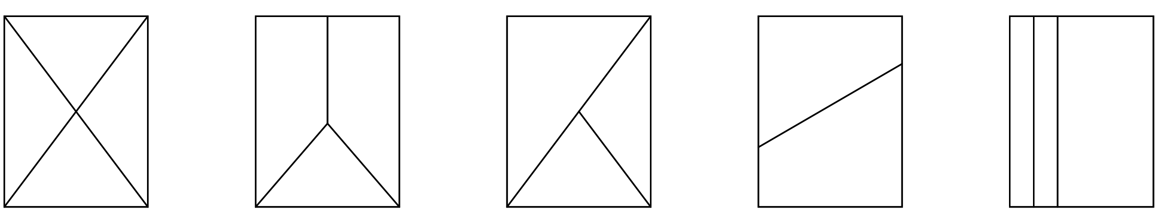
Length and Description

Segment	Length	Description
A	5”	Bituminous overlay
B	1.5”	Bituminous overlay
C	7.5”	Concrete pavement-sound
D	9”	Top of concrete slab- gravel
E	6”	Slab- sound- test
F	6”	Slab- concrete chunks
G	1”	Bottom of slab- sound- rebar not rusted
H		
I		
J		
K		
L		
M		

Test Length 4” **Correction Factor** .87 **Diameter** 3.95” **Area** 12.25 sq. in.

Factored Compressive Strength 5945 psi **Actual Maximum Load** 83730 pds.

Type of Fracture Cone & split



Cone **Cone & Split** **Cone & Shear** **Shear** **Columnar**

Age of Specimen 1927 **Defects in Specimen** HORIZ. CRACKS

Printed BBS (draft)

BRIDGE CORE

Core Number 2 **Element Cored** SLAB

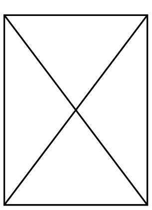
Length and Description

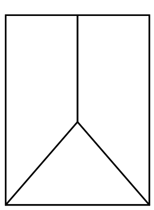
Segment	Length	Description
A	5.5”	Bituminous overlay
B	8.5”	Pavement-sound
C	10”	Top of slab-concrete chunks
D	8.5”	Slab- sound- test
E	2.5”	Bottom of slab- sound- rebar not rusted
F		
G		
H		
I		
J		
K		
L		
M		

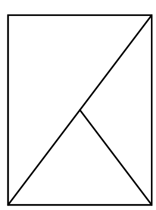
Test Length 7.75” **Correction Factor** 1.0 **Diameter** 3.95” **Area** 12.25 sq. in.

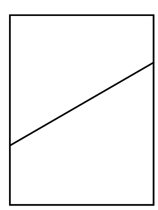
Factored Compressive Strength 3651 psi **Actual Maximum Load** 33190 pds.

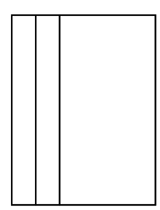
Type of Fracture Cone & split


Cone


Cone & Split


Cone & Shear


Shear


Columnar

Age of Specimen 1927 **Defects in Specimen** none

Printed BBS (draft)

6.3 Manual for Using Wireless Instrumentation for Field Testing of Concrete Bridges

Manual for Using Wireless Instrumentation for Field Testing of Concrete Bridges

Prepared By
Bassem Andrawes
Isaias Andres Colombani
Md Abdul Hamid Mirdad
Alexander Jai-syn Chen
University of Illinois Urbana-Champaign

Based on the findings of

ICT PROJECT R27-205

Development of Bridge Load Testing Program for Load Rating of Concrete Bridges

TABLE OF CONTENTS

CHAPTER 1: HARDWARE

BDI STS4

The Bridge Diagnostics Inc. (BDI) 4th generation Structural Testing System (STS4) functions with various components that communicate via a wireless link to transmit the data to the computer. The system is built for rugged outdoor use to minimize environmental effects and prevent damage to the equipment during field tests. The system's main components are the wireless base station (WBS), which provides a wireless interface to the data acquisition system (DAQ) and the computer; the STS4 nodes, which transmit data to the computer and provide power to the connecting sensors; the sensors used for measuring the desired load effects of the bridge; and the computer, which is used to store and visualize the field data.

WIRELESS BASE STATION

The wireless base station (WBS) presented in **Figure 1** serves as the connection between the instruments and the laptop. It consists of a red control unit and a white dual-band wireless Access Point (AP) from which four antennas protrude. The control unit has a battery life of approximately 16 hours and is rechargeable. The control unit also functions on an external (wired) 24 VDC power supply in case of long-term monitoring applications. The WBS has ethernet ports for when a wired interface is preferred. In the most ideal conditions, the WBS has a range of 6.4 km (4 mi). However, this range is reliant on the line-of-site and is reduced with physical obstructions between the components. The antennas are detachable for easy storage but are not freely interchangeable. They are labeled, with two antennas for 2.4 GHz and two for 5.0 GHz. Pressing the blue power button on the red control unit briefly causes it to show the battery status without turning on. Pressing and holding the button will turn it on. This can be verified by checking if the light continues to blink. When the test is completed, press and hold the button to power it off.

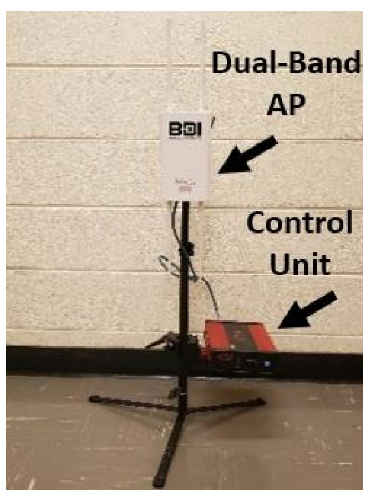


Figure 1. Base station

NODE

The nodes allow the various instruments to send information to the base station. Each node can connect up to four different sensors of any type as presented in **Figure 2**. A maximum of 128 nodes can be connected using a single WBS for the test. These nodes are equipped with a rechargeable internal battery that can supply power to the sensors for over 40 hours when used at the maximum sampling rate of 1,000 samples per second (S/s). The STS4 nodes have an internal memory of 8 GB for backup data storage. They also have temperature sensor inputs for each channel. The node can operate via a wireless connection to the WBS or ethernet cable. The nodes are lightweight and compact for easy transport during field tests. Each node also has two antennas. These can be removed for easier storage. Press the power button briefly to check the battery status. Press and hold the power button to turn on the node. A light will continuously blink to indicate that the node is on. Press and hold again to power off.

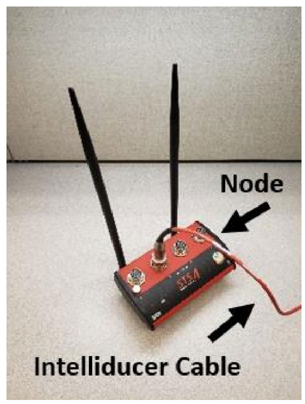


Figure 2. Node and a sensor cable

LINEAR VARIABLE DIFFERENTIAL TRANSFORMER (LVDT)

The LVDTs (**Figure 3**) measure displacement and are useful for providing global information about the bridge. They cannot be attached to the bridge directly and need a frame to attach to instead. They are typically placed at the midspan, because that is where deflection will be the greatest. To minimize error, keep the LVDT perpendicular to the concrete surface. The LVDTs have a range of ±25.4 mm (±1.0 in.). Their total length is 0.3 m (10.8 in.), and they have a supply voltage of 6 to 18 VDC.

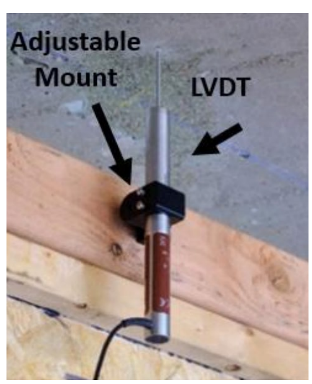


Figure 3. LVDT in the field

STRAIN TRANSDUCER (ST350)

The strain transducers (ST350) measure strain and are useful for providing local information about the bridge. The ST350s (**Figure 4**) are encased in a sealed aluminum shell making them ideal for outdoor use. They have an effective gauge length of 76.2 mm (3 in.) with the option to add an extension for installations up to 0.6 m (24 in.). The ST350s can be installed with the typical adhesive mounting technique by applying the adhesive to the reusable mounting tabs. They can also be installed using anchors because they have 6.4 mm (0.25 in.) mounting holes at both ends. To facilitate the installation of the extensions and ensure that the extensions are properly aligned, a jig was purchased to align the extensions to the ST350. This should be done prior to installing the ST350s to the concrete.

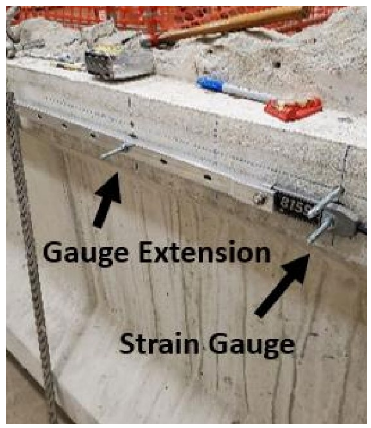


Figure 4. Strain gauge with gauge extension

The recommended gauge extension length used for the test is a function of the member type, member depth and span length. **Table 1** provides guidelines for choosing the appropriate gauge length based on these parameters.

Table 1. Ext. Multiplier for recommended gauge extension length

RECOMMENDED LOWER AND UPPER GAGE LIMITS		
STRUCTURE TYPE	LOWER LIMIT	UPPER LIMIT
SLABS & RECTANGULAR BEAMS	1.0 X DEPTH OF MEMBER	LENGTH OF SPAN / 20
T-BEAMS	1.5 X DEPTH OF MEMBER	LENGTH OF SPAN / 20

After obtaining the appropriate gauge length, the measurements must be reduced to account for the extension effects on the readings. The new reading can be determined from **Equation 1**, where the amplification factor and the ext. multiplier can be found using **Table 2**.

$$Adjustment\ Factor = \frac{Amplification\ Factor}{Ext.Multiplier} \tag{1}$$

Table 2. Adjustment factor

GAGE EXTENSION LENGTH	EXT. MULTIPLIER	AMPLIFICATION FACTOR
6 in	2	1.1
9 in	3	1.1
12 in	4	1.1
15 in	5	1.1
18 in	6	1.1
21 in	7	1.1
24 in	8	1.1

To minimize the force on the ST350s, BDI recommends keeping the maximum strain below ±1,000 με. There are also maximum strain ranges which depend on the extension and properties of the concrete. These maximum values are provided to ensure accuracy for the given gauge length. Greater strain values can be measured. However, BDI suggests that special attention be given to the gain settings on the DAQ being used when exceeding these ranges. These maximum strain ranges are provided in **Table 3**.

Table 3. Maximum strain gauge range for concrete and gauge length

EXT. MULT.	ACTUAL GAGE LENGTH WITH EXTENSION	MAXIMUM STRAIN RANGE	APPROX. CONC. STRESS FOR F'C = 3,000 PSI (20.7 MPA)	APPROX. CONC. STRESS FOR F'C = 4,000 PSI (27.6 MPA)	APPROX. CONC. STRESS FOR F'C = 5,000 PSI (34.5 MPA)	APPROX. STEEL RE-BAR STRESS
1	3 in (76.2 mm)	±1000 με	3.1 ksi (21.4 MPa)	3.6 ksi (24.8 MPa)	4.0 ksi (27.6 MPa)	30 ksi (207 MPa)
2	6 in (152.4 mm)	±500 με	1.6 ksi (11.0 MPa)	1.8 ksi (12.4 MPa)	2.0 ksi (13.8 MPa)	15 ksi (103 MPa)
3	9 in (228.6 mm)	±330 με	1.0 ksi (6.9 MPa)	1.2 ksi (8.3 MPa)	1.3 ksi (9.0 MPa)	9.9 ksi (68.3 MPa)
4	12 in (304.8 mm)	±250 με	780 psi (5.3 MPa)	900 psi (6.2 MPa)	1.0 ksi (6.9 MPa)	7.5 ksi (51.7 MPa)
5	15 in (381.0 mm)	±200 με	625 psi (4.3 MPa)	720 psi (5.0 MPa)	800 psi (5.5 MPa)	6.0 ksi (41.4 MPa)
6	18 in (457.2 mm)	±160 με	500 psi (3.4 MPa)	575 psi (4.0 MPa)	650 psi (4.5 MPa)	4.8 ksi (33.1 MPa)
7	21 in (533.4 mm)	±140 με	440 psi (3.0 MPa)	500 psi (3.4 MPa)	560 psi (3.9 MPa)	4.2 ksi (29.0 MPa)
8	24 in (609.6 mm)	±125 με	390 psi (2.7 MPa)	450 psi (3.1 MPa)	500 psi (3.4 MPa)	3.8 ksi (26.2 MPa)

TILTMETER

The tiltmeters (**Figure 5**) measure rotational information. They are typically attached at the ends of the bridge span to monitor the rotation near the supports, with a range of ±0.5° to ±60°. The tiltmeters can be attached to concrete directly by applying adhesives to the reusable mounting tabs. Remember to always keep the serial plate oriented facing up. There are mounting holes on all faces of the tiltmeter. For example, when installing upside down, use the mounting holes on the top of the tiltmeter instead of flipping the tiltmeter upside down.

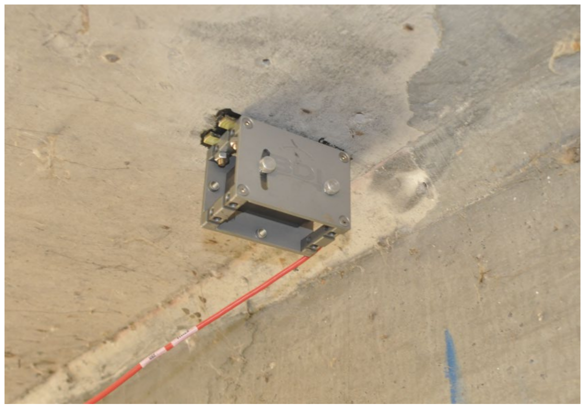


Figure 5. Tiltmeter

LAPTOP

The STS4 system is not complete without a laptop computer for data visualization and processing. BDI provides some recommendations for computer requirements. The computer used for the testing was equipped with an Intel Core i7 processor and 16 GB RAM. The computer has 512 GB SSD internal storage capacity. It also has up to 13 hours of battery run time. Windows 10 Pro was installed and recommended by BDI for using their software applications. The computer will run BDI’s STS-LIVE software, which is used for visualizing the sensor readings during the test. The computer was also equipped with the BDI’s STS-VIEW software for post-processing the results.

CHAPTER 2: SOFTWARE

STS-LIVE

The STS-LIVE software is provided with the STS4 package for visualizing the sensor data during the field tests. It gives control over all the connected devices, sensors, calibration files, and more, and is necessary for setting up the load tests and configuring the devices. Upon opening the STS-LIVE software, the user will see the Settings tab where the project name, description, file directory, and sampling rate can be selected (**Figure 6**). Recommended sampling rates depend on the type of test being conducted. For typical diagnostic tests where the truck moves at a crawling speed along the bridge span, the sampling rate should be between 50 and 20 S/s. For static load tests common in proof load testing, a 10 S/s rate is recommended. Under this same tab, there is a sub-tab where the user can set autozero on or off, which is typically turned "on." The final sub-tab allows the user to lock access to the calibration files for the sensors. The password to unlock or lock the sensor edit feature is "BDI."

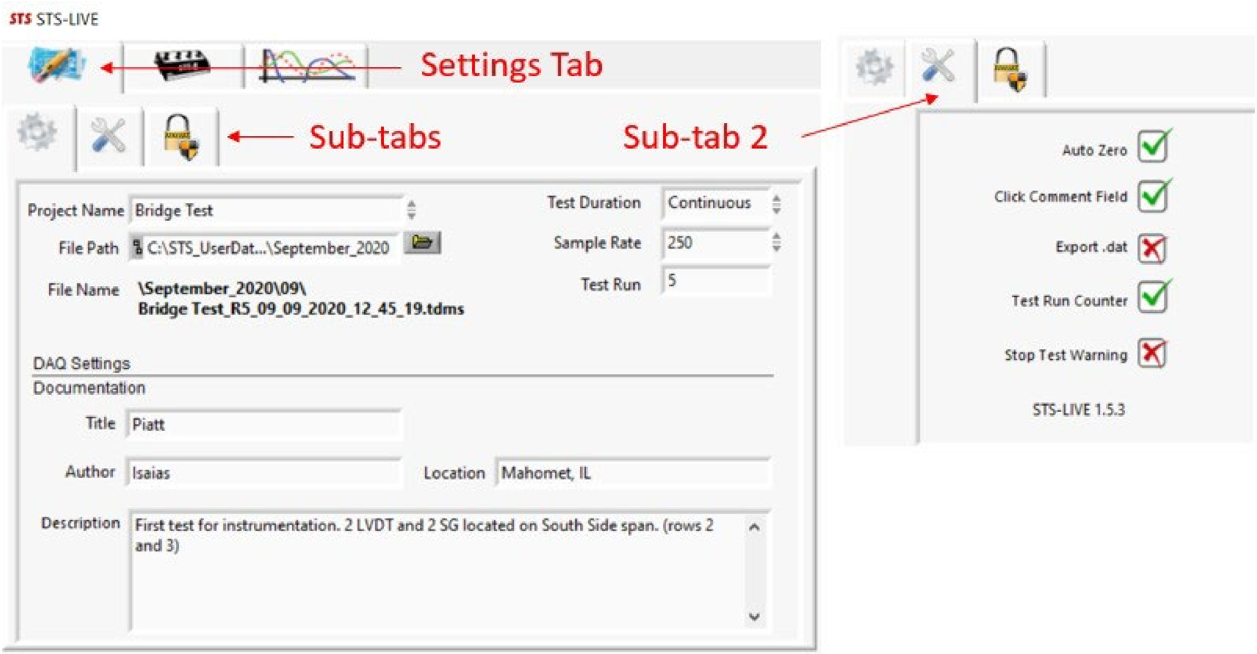


Figure 6. Settings tab

The second tab is the Hardware tab shown in **Figure 7** where the WBS, STS4 nodes, and sensors can be modified for the test. The connected STS4 nodes will appear in the window with their battery power and signal strength along with some other information. The signal strength is given by the RSSI in decibels (dB), which should be no greater than $\pm$80 dB to provide a good connection. If the RSSI is high, then the WBS should be relocated. The signal strength will update automatically in real-time in STS-LIVE as the WBS is moved closer or further from the computer.

Every node will show the sensors connected to its corresponding data channel in the drop-down list. When a sensor is selected, the window will display the maximum and minimum readings for each sensor, units of the measurements, and other information that is relevant to the test.

The Hardware tab is also where the user has access to the calibration factors and sensor limits. These parameters can be adjusted, but it is not recommended for most cases since the sensors are already calibrated.

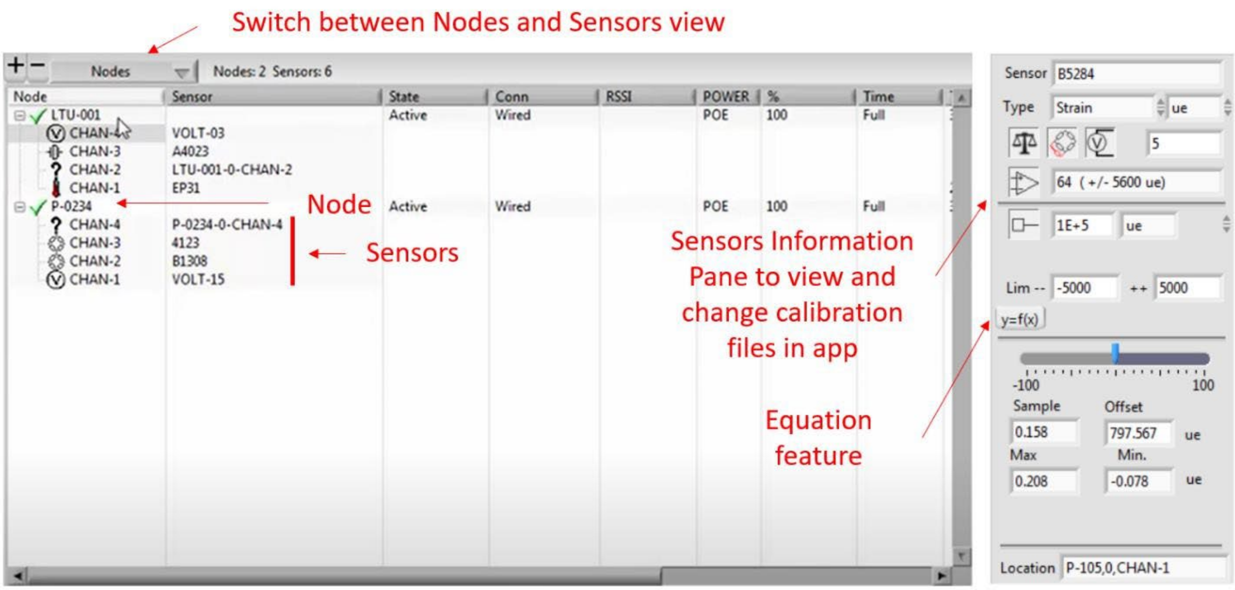


Figure 7. Hardware Tab

One useful feature in the sensor information pane is the equation feature. This is useful when converting the readings for the ST350s when they have extensions on them. An important consideration when using this feature is that the values which are recorded and saved into the TDMS file are the values after the operation is performed. This means that the raw readings from the sensors are not saved into the file for post-processing.

When setting up the hardware for the test, the computer must be connected to the appropriate IPN to connect to the WBS. Video tutorials on how to set up and use the STS-LIVE can be found at https://bditest.com/applications-tutorials/. Once the settings for the IPN are changed, the WBS can be turned on and the STS network will be available within minutes. The WBS will appear first in the Hardware tab with a checkmark to indicate that the connection has been established. It is important to first check the signal strength under the RSSI column to ensure that the signal strength is sufficient. This number should be no more than ± 80. The closer the number is to zero, the better the connection. After connecting the sensors to the nodes for the test, the nodes can be turned on. These should connect to the WBS and appear in the window when they have been successfully connected to the system.

The final tab in STS-LIVE is the Data Display tab where the data can be visualized during the load tests. The display window shows the sensor values for any sensors used. Multiple sensors can be shown on the same plot. The sensors can also be grouped to show only specific sensors during the load test. The display can be customized by the user with different visualization tools. Note that the more sensors being visualized at once, the more CPU is required. This window also has a “comment” feature in the bottom left-hand corner of the screen next to the start button. This comment feature can be used to place ticker marks (or time stamps) in the data during the load test. This is useful for marking the truck load position during the load tests, with each tick mark corresponding to a different truck position. When the test is completed, the clicker data with the time stamp will be saved in the TDMS file for post-processing.

Data is only recorded once the start button is pressed (**Figure 8**). A preview data button can be used to preview the measurements without storing the data on the hard drive. There are also controls for setting the nodes into sleep and standby modes to save battery during long field tests.

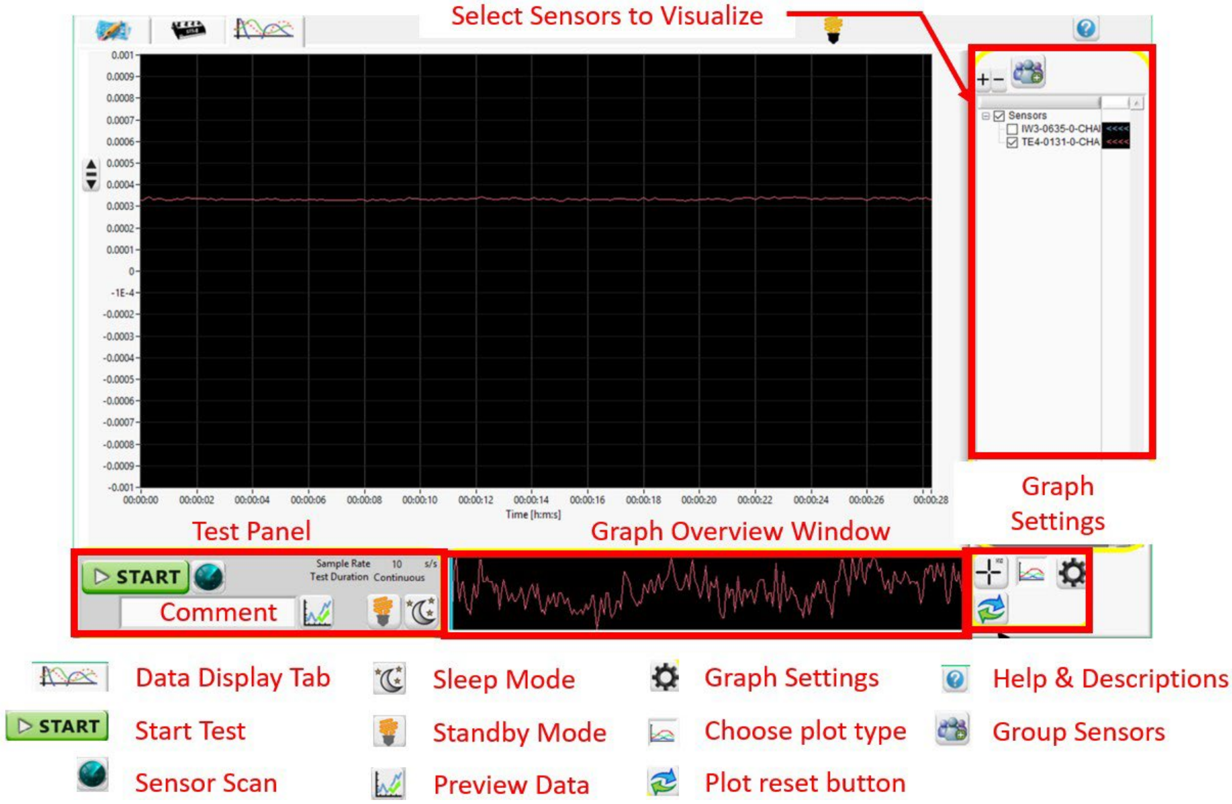


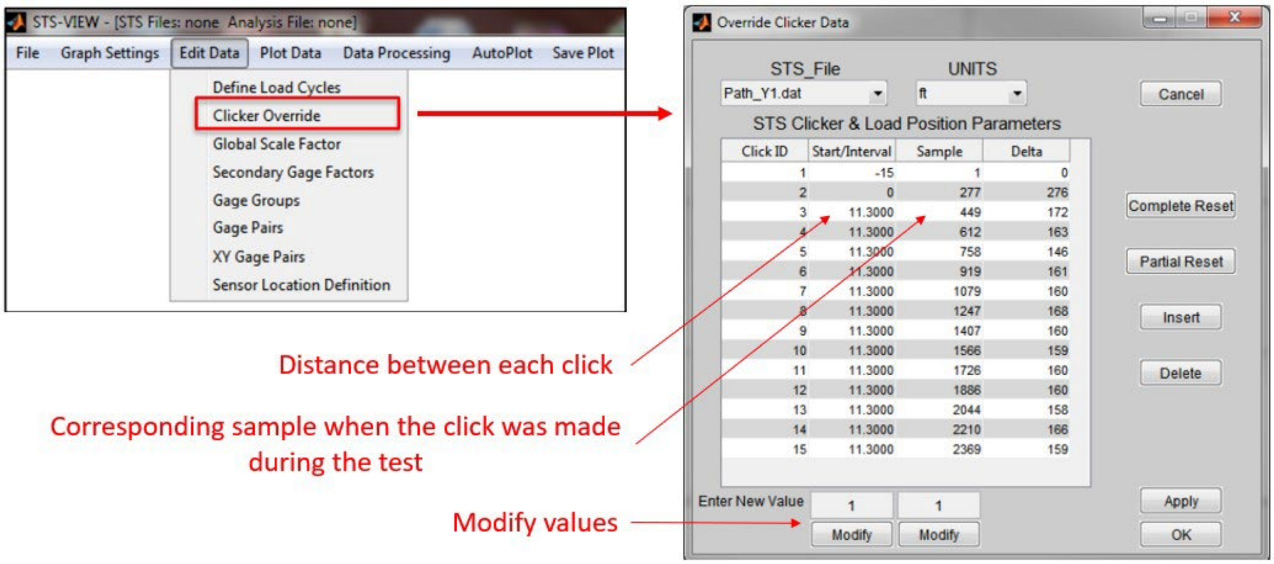
Figure 8. Data display tab

It is recommended that the instrumentation be turned on and left connected for at least 30 minutes before starting the load test. This will allow the random noise in the sensors to stabilize beforehand. For typical load tests, the start button is pressed before any loading is placed on the bridge to give the sensors a reference point for zero load. If the Autozero was checked in the Settings tab prior, the sensors will then automatically zero. The truck(s) can then start driving across the bridge to apply the loading. Markings can be placed on the roadway to represent the quarter spans of the bridge so that the comment feature can be used to track the position of the truck(s). Once the truck(s) have completely crossed the bridge, the test can be stopped after a few seconds to save the data to the TDMS file. The process is repeated for every load path. The results can then be visualized in Microsoft Excel with the TDMS file or in BDI's STS-VIEW software.

STS-VIEW

The software used for post-processing the results was BDI's STS-VIEW software. The software runs on MATLAB and allows the user to easily visualize the TDMS file data (MathWorks 2021). The data can be uploaded from the files menu tab and the user can upload multiple TDMS files at once. The clicker data taken during the test is automatically saved into this file with the time stamp corresponding to when the comment was clicked during the test. In the event that these markings do not give the maximum or minimum value that the user wants to process, new time stamp data points or modifications to the clicker data can be made to manually change where to inspect the data. This can be found under the clicker overrides option in the Edit Data tab shown in **Figure 9**. If position data is of interest, the positions of the truck between clicks can be input into the data to get a measurement versus truck position plot. The

Sample column is where the user can modify the position of the click or add new clicks to the data. The desired sample number can be calculated by finding the time where the desired point is to be taken and multiplying this time by the sample rate. The clicker override feature was used before in this study to obtain the peak magnitudes from the data and plot the distribution of the peak values across the slab after the field tests. It will often occur that the critical section of the bridge is not exactly at the position predicted by the analysis. For this reason, the clicker modification window is important for the analysis and post-processing.



Click ID	Start/Interval	Sample	Delta
1	-15	1	0
2	0	277	276
3	11.3000	449	172
4	11.3000	612	163
5	11.3000	758	146
6	11.3000	919	161
7	11.3000	1079	160
8	11.3000	1247	168
9	11.3000	1407	160
10	11.3000	1566	159
11	11.3000	1726	160
12	11.3000	1886	160
13	11.3000	2044	158
14	11.3000	2210	166
15	11.3000	2369	159

Figure 9. Clicker override feature

The software is useful for quickly obtaining the cross-sectional distribution of the sensor readings on the bridge from the load tests. This can be done by using the gauge groups window under the edit data tab as shown in **Figure 10**. The sensors should be organized such that they are in the order they appear on the cross section of the bridge. The group can then be saved and plotted based on the truck position specified in the clicker override window. The procedure for obtaining the group sensor distribution is shown in **Figure 10**.

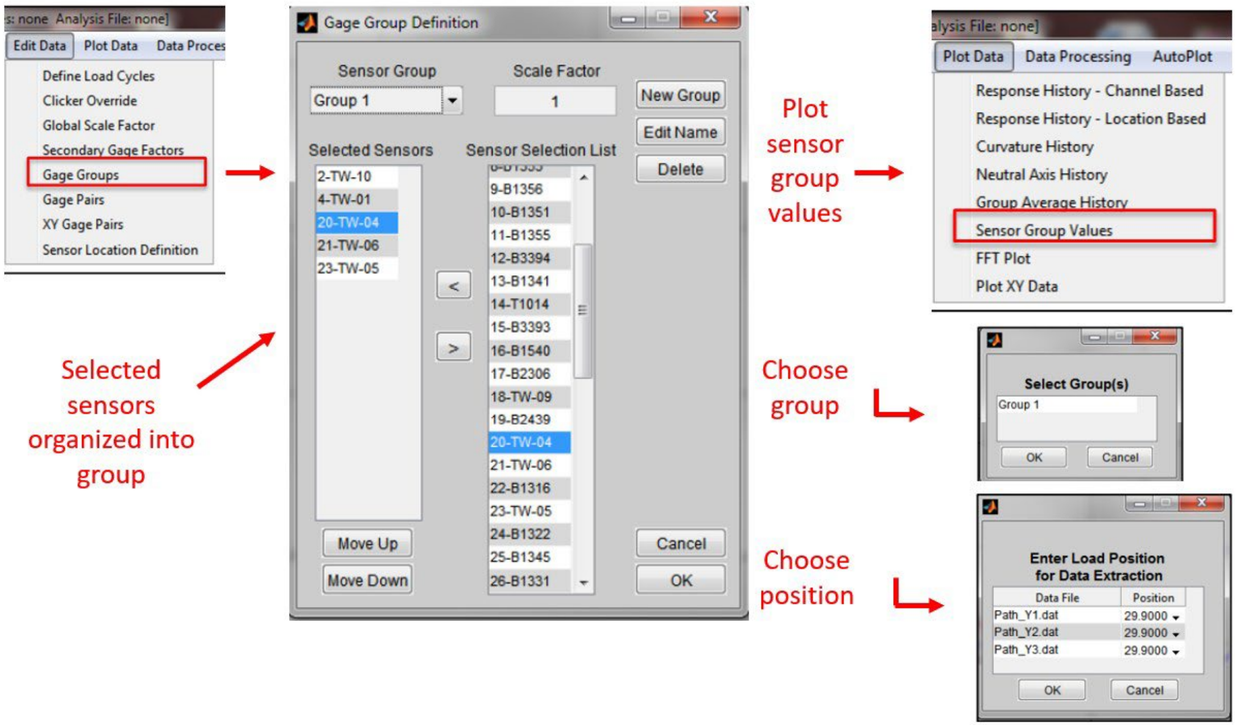


Figure 10. Gauge groups window and sensor group values plot

The sensor results can be plotted versus time by using the response history channel-based option in the Plot Data drop-down menu. When the user has input the position data in the clicker override window, a plot of the sensor response versus truck position can also be generated. This is useful for model calibration and for understanding the position of the axle loads that produced the responses. Note that multiple datasets can be displayed on a single plot for either option, which makes it easy to compare responses based on multiple load paths or compare repeat passes of the truck(s) on the bridge.

TDMS PLUG-IN

It may be desirable to use Microsoft Excel to plot and analyze the data. National Instruments provides software called “TDM Excel Add-In” to open .tdms files in Microsoft Excel. The download can be accessed through the following link: https://www.ni.com/en/support/downloads/tools-network/download.tdm-excel-add-in-for-microsoft-excel.html#378046. Once downloaded, right-click on the .tdms file, select “Open with” and then “excel add-in importer.”

CHAPTER 3: DATA INTERPRETATION

The safety of the bridge is dependent on many influencing factors. A trained professional should be present throughout the test and should rely on their experience to interpret the data as the test proceeds. Testing should terminate if there is any reason to believe that the structural integrity of the bridge has been impacted by the diagnostic or proof load test. The following figures from the Coles County bridge proof load test are included to illustrate concepts and should not be repurposed to justify any other bridges' safety.

General considerations when determining bridge safety include the stiffness of the bridge, residual displacements and/or rotations, peak strains, and crack development and growths. These factors are interdependent and thus should be considered simultaneously. The stiffness of the bridge should be monitored by plotting the demand on the bridge against the displacements from the LVDTs for each stage of the proof load test. A substantial decrease in the slope, as seen between the final few data points in **Figure 11**, suggests a loss of stiffness in the bridge. The residual deformation should also be tracked and judged together with the stiffness. Some residuals are to be expected throughout testing as gaps within the bridge close under loading. However, a substantial consistent increase in the residuals between any subsequent stages is unusual. A sudden growth can be seen in the final runs of Stages 3 and 4 in **Figure 12**. Additionally, the peak strains from the strain gauges at each stage should be compared with the predicted concrete cracking limit. If any strains approach the limit, the test should be stopped. The cracks underneath the bridge deck should also be monitored. Photographs of the cracks should be taken at the start of the test for comparison. It is also recommended to mark the ends of the cracks with a permanent marker, so any crack propagation is easily identifiable. During the Coles County test, these factors were weighed together and the team decided to not proceed to Stage 5.

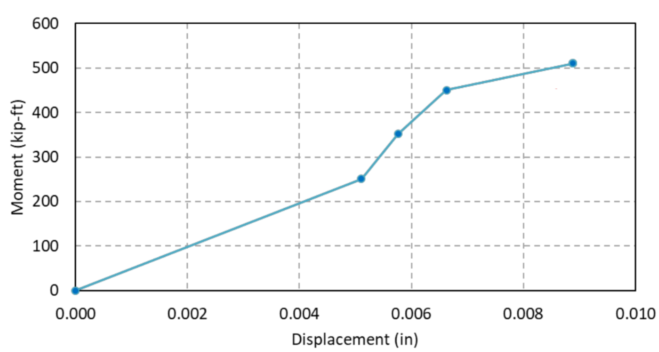


Figure 11. Coles County Proof Test Moment-Displacement Graph

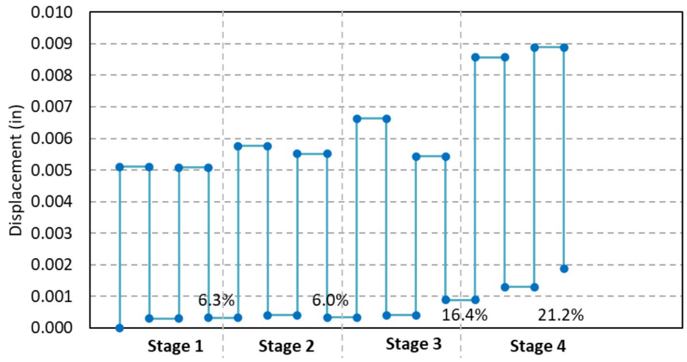


Figure 12. Coles County Proof Test Displacements