Illinois Department of Transportation

Memorandum

To:	ALL BRIDGE DESIGNERS	24.3
From:	Jayme F. Schiff	Jayme F. Schiff
Subject:	Structural Services Manual Update – Section 1 & Section 2	
Date:	June 3, 2024	

All Bridge Designers (ABD) Memorandum 24.3 introduces updates to Section 1 (Introduction) and Section 2 (Repairs) of the Structural Services Manual. Section 1 updates are minor revisions and editorial changes. Section 2 updates reflect the current standards and practices of the bridge repairs unit. A summary of significant changes to submittal, design, and detailing policy for bridge repairs is provided below.

Section 2.2 Plan Preparation and Review
Updated submittal procedure to match current practice of electronic submittals.

Section 2.3 Inspections
Clarified requirements for reporting damage. Emphasis on the proper use of digital photographs.

Section 2.4 Bridge Deck Overlays
Included guidelines for the selection of overlays. This will replace selection guidelines that are no longer available.

Section 2.5 Expansion Joint Replacements / Elimination
Clarified that, with notable exceptions, the only acceptable joints for use in joint replacements are shallow strip seals, finger plate, and modular.
Included a new section (2.5.8) that allows joint elimination at abutments in their entirety by way of encasing beam ends and making them integral with approach slabs.

Section 2.7 Bridge Rails and Parapets
Included new details for rails to follow MASH 2016 requirements.
Included details for wingwall modification to match new guardrail heights.
Included details for crashwall extensions.

Section 2.9 Bearing Replacements
Included new details for PPC I beam bearing replacement.
Revised required "C" values for bearing plate design and anchor bolt material requirements and sizes.

Section 2.10 Jacking / Shoring and Cribbing
Established minimum detail requirements for jacking procedures submitted for approval.

Section 2.12 Fatigue and Fracture
Created new sub-section 2.12.2 Fracture, to differentiate from the existing Fatigue sub-section.

Section 2.13 Impact Repairs – Steel Beams
Included procedure for the replacement of damaged composite beam sections using a “zipper” detail.

Section 2.14 Impact Repairs – Concrete Beams
Included use of FRP at impacted locations.

Section 2.15 Steel Superstructure Repairs
Eliminated obsolete details and included new details that limit most repairs to the use of steel plates and angles.
Included requirements for Primary and Secondary connections.
Emphasized cleaning and painting requirements for steel repairs.

Section 2.16 Concrete Superstructure Repairs
Included procedure for the use of FRP for the repair of PPC I beams.

Section 2.17 Substructure Repairs
Updated guidelines for substructure repair selection procedure.

Section 2.18 Culvert Repairs
Included new section to include new culvert repairs that include the use of liners and slab over culvert methods.

Applicable special provisions for these repair procedures are available upon request.

While these updated sections are provided as a stand-alone document, to get it to designers as soon as possible, it will be incorporated in the update of the full Structural Services Manual to be issued later this year.

The policies found in the updated Section 2 shall be implemented on applicable projects as soon as feasible. Please direct questions and comments to Adrian Halloway, Bridge Investigations and Repairs Unit Chief, at dot.bbs.repairs@illinois.gov.

Attachments:
SSM Section 1 Update
SSM Section 2 Update

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 Highways 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).

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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 $^{3}/_{4}″\phi$, holes $^{13}/_{16}″\phi$, unless otherwise noted.	All structures with painted steel. If structures have weathering steel, galvanized steel or metallized steel see notes 5b, 5c and 5d respectively of the Design Section of the Bridge Manual.
3.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, mechanically galvanized bolts. Bolts $^{7}/_{8}″\phi$, open holes $^{15}/_{16}″\phi$, unless otherwise noted.	All structures with painted steel. If structures have weathering steel, galvanized steel or metallized steel see notes 5b, 5c and 5d respectively of the Design Section of the Bridge Manual.
4.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, mechanically galvanized bolts. Flange splice holes shall be $^{15}/_{16}″\phi$ for $^{7}/_{8}″\phi$ bolts. Web splice holes shall be $^{13}/_{16}″\phi$ for $^{3}/_{4}″\phi$ bolts.	When installing new splice plates at existing splice locations that have different size bolts.
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.”	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.”	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.	

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 ¼″ 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 Expansion Joints shall be assembled in their final relative position with the ends in place for shop inspection and acceptance.	Finger plate expansion devices. Insert the item(s) within the ( ) that apply to the project. Place note with finger plate joint 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 steel except where otherwise noted. The color of the final finish coat shall be	When painting new steel is required. Cost included with Furnishing and Erecting Structural Steel or Structural Steel Repair. Colors to match existing steel: 1. Interstate Green, Munsell No. 7.5G 4/8

	_________________, Munsell No. ________.	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.	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.	
18.	The existing structural steel coating contains lead. The Contractor shall take appropriate precautions to deal with the presence of lead on this project.	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.	

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 Structural Steel repairs or F. & E. Structural Steel.	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.	
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.	
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 with Concrete Superstructure.	To be used when new concrete area is small, and no grooving is called for (mainly at joint replacement areas).
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.	
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 damaged beam at the specified locations. The preloading system should produce a total maximum service load as shown on the center of the damaged area. Preloading shall be kept in place for at least 3 days after completion of concrete repair or until the concrete has reached an ultimate strength of 5000 psi.	Required when impact damage occurs within the middle half of the PPC I beam.
42.	The contractor’s proposed preloading system with computations sealed and signed by an Illinois Structural Engineer shall be submitted to the Bureau of Bridgers and Structures for approval. The preloading system placed shortly after bridge is closed for repairs, shall not be paid for separately but shall be included in the unit price for PPC-I Beam Repairs.	

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:

a) Structure Number
b) Route Number
c) Project Number
d) County Name
e) Marked Route
f) Feature Crossed
g) Inspection Information (including photos and sketches)
h) Description of Proposed Structural Repairs
i) Tentative Letting or Award Date
j) 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:

a) Emergency repairs to structures to correct structural issues.
b) Repairs to structures needed to prevent or lift a load posting.
c) Repairs to structures on a specific letting.
d) 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 *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 3.3.

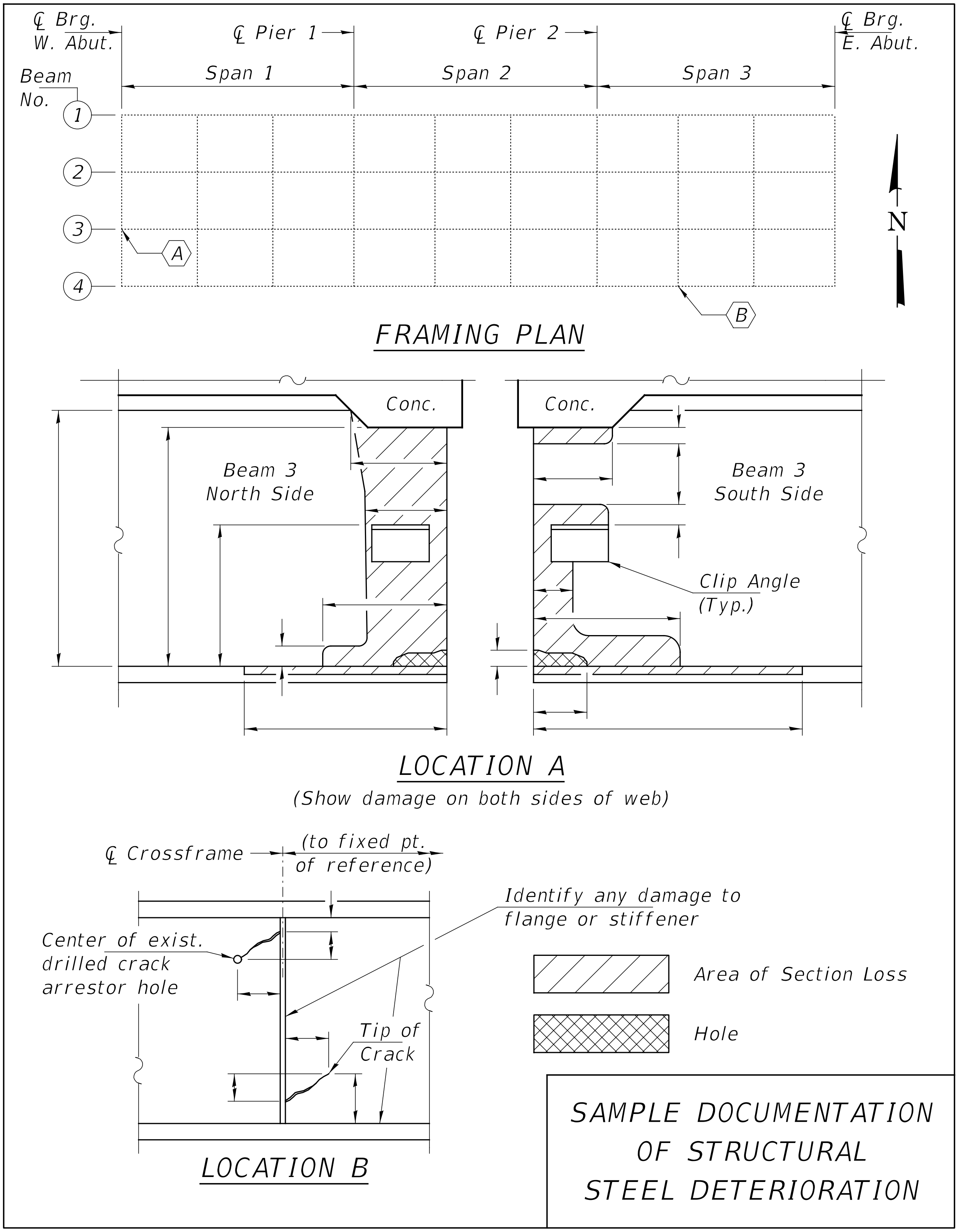
2.3.2 General Deterioration

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.


℄ Brg.
W. Abut.
℄ Pier 1
℄ Pier 2
℄ Brg.
E. Abut.
Beam
No.
Span 1
Span 2
Span 3
1
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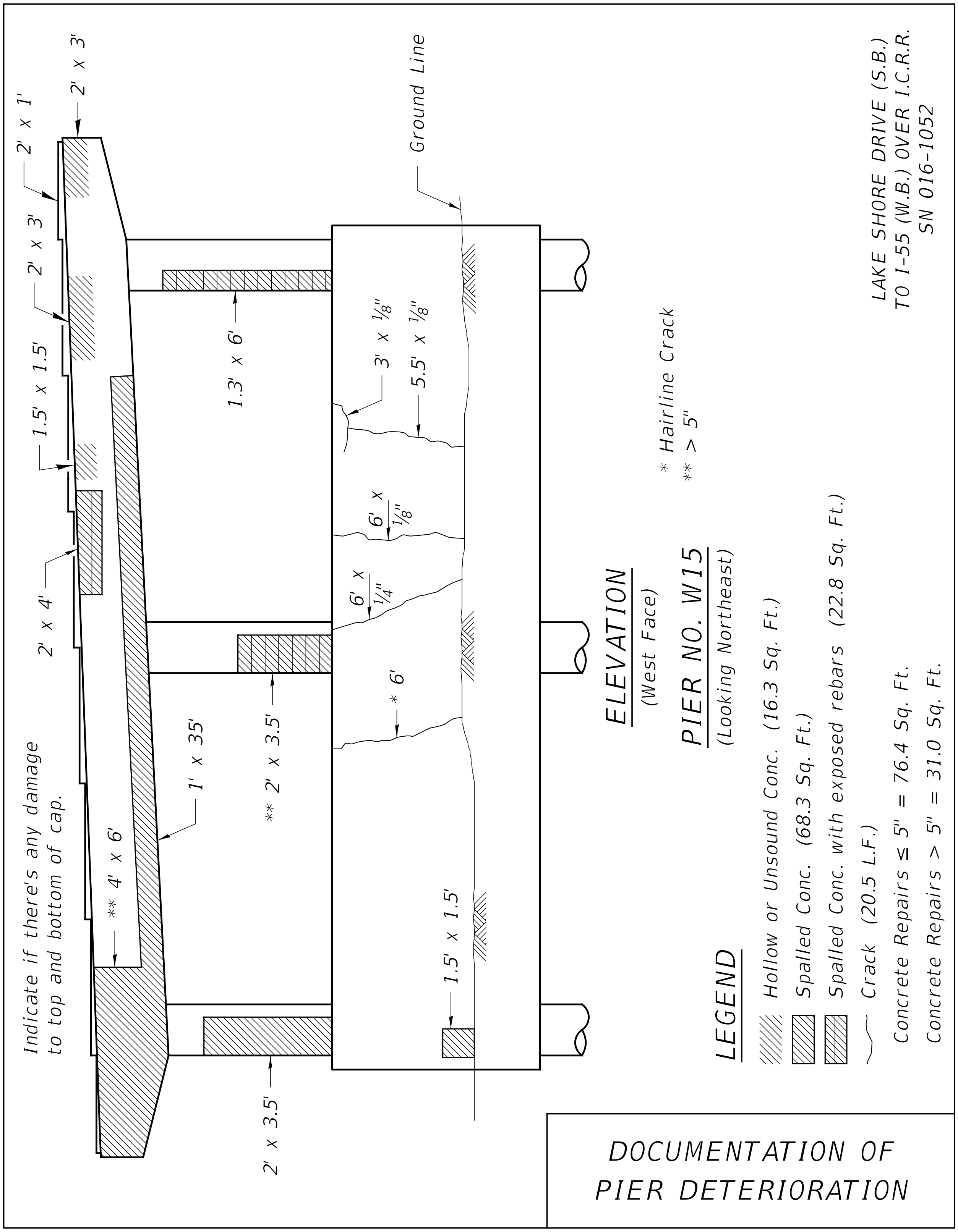
FRAMING PLAN
Conc.
Conc.
Beam 3
North Side
Beam 3
South Side
Clip Angle
(Typ.)
LOCATION A
(Show damage on both sides of web)
℄ Crossframe
(to fixed pt.
of reference)
Identify any damage to
flange or stiffener
Center of exist.
drilled crack
arrestor hole
Tip of
Crack
Area of Section Loss
Hole
LOCATION B
SAMPLE DOCUMENTATION
OF STRUCTURAL
STEEL DETERIORATION

FIGURE 2.3.2-1



DOCUMENTATION OF
PIER DETERIORATION
LAKE SHORE DRIVE (S.B.)
TO I-55 (W.B.) OVER I.C.R.R.
SN 016-1052
ELEVATION
(West Face)
PIER NO. W15
(Looking Northeast)
* Hairline Crack
** > 5"
LEGEND
Hollow or Unsound Conc. (16.3 Sq. Ft.)
Spalled Conc. (68.3 Sq. Ft.)
Spalled Conc. with exposed rebars (22.8 Sq. Ft.)
Crack (20.5 L.F.)
Concrete Repairs ≤ 5" = 76.4 Sq. Ft.
Concrete Repairs > 5" = 31.0 Sq. Ft.
Ground Line
Indicate if there's any damage
to top and bottom of cap.
2' x 3'
2' x 1'
2' x 3'
1.5' x 1.5'
2' x 4'
1' x 35'
** 4' x 6'
2' x 3.5'
1.3' x 6'
** 2' x 3.5'
3' x 1/8"
5.5' x 1/8"
6' x 1/8"
6' x 1/4"
* 6'
1.5' x 1.5'

FIGURE 2.3.2-2

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 f ound 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.

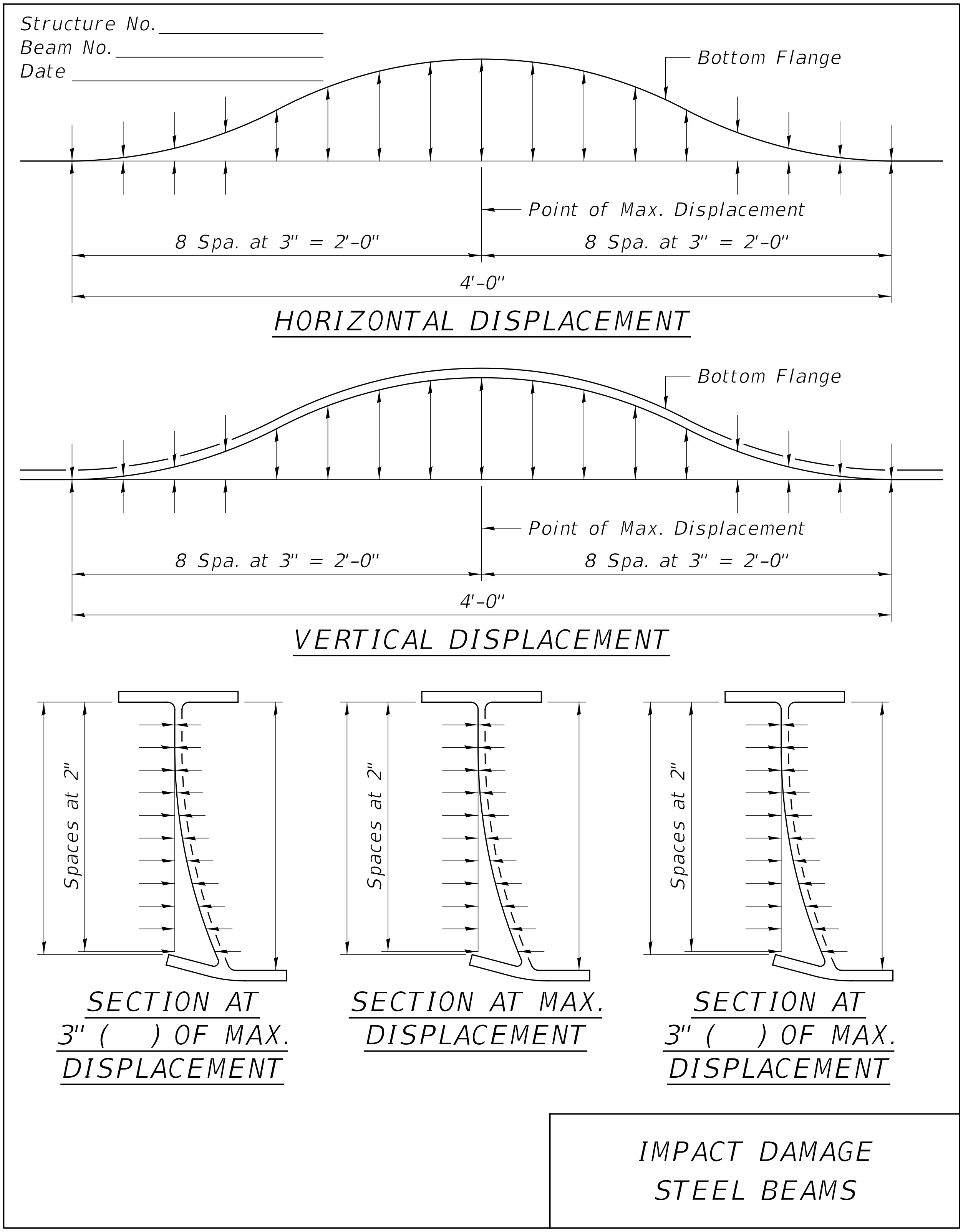
An example framing plan recording impact damage information is shown in Figure 2.3.3-2. Figure 2.3.3-3 illustrates the documentation of damage to the concrete fillet encasing the top flange of a non-composite beam. Also, a cross section of each damaged beam should be provided showing:

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.



Structure No.
Beam No.
Date
Bottom Flange
Point of Max. Displacement
8 Spa. at 3" = 2'-0"
8 Spa. at 3" = 2'-0"
4'-0"
HORIZONTAL DISPLACEMENT
Bottom Flange
Point of Max. Displacement
8 Spa. at 3" = 2'-0"
8 Spa. at 3" = 2'-0"
4'-0"
VERTICAL DISPLACEMENT
Spaces at 2"
Spaces at 2"
Spaces at 2"
SECTION AT 3" (   ) OF MAX. DISPLACEMENT
SECTION AT MAX. DISPLACEMENT
SECTION AT 3" (   ) OF MAX. DISPLACEMENT
IMPACT DAMAGE STEEL BEAMS

FIGURE 2.3.3-1

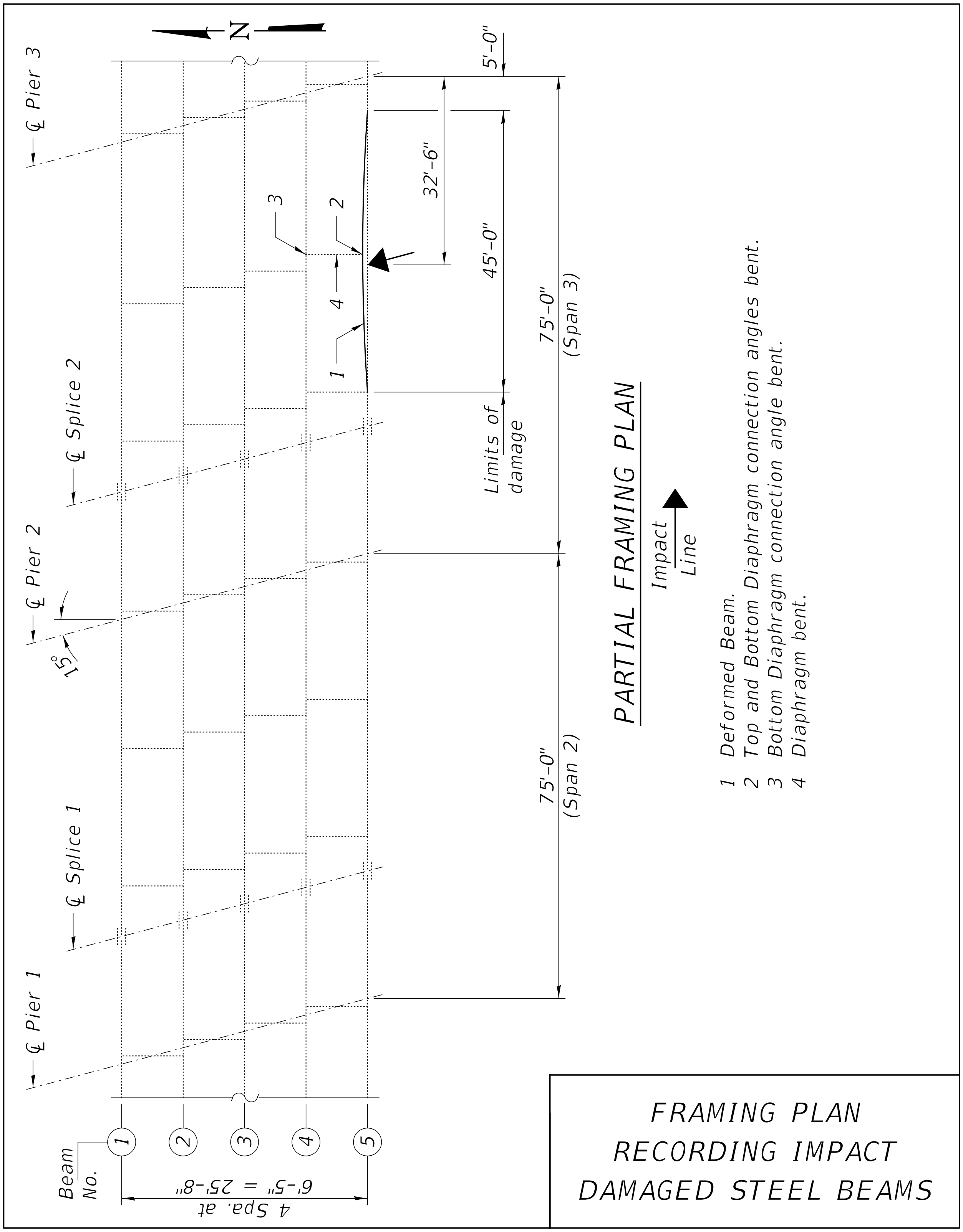

N
℄ Pier 3
℄ Splice 2
℄ Pier 2
℄ Splice 1
℄ Pier 1
15°
5'-0"
32'-6"
45'-0"
75'-0"
(Span 3)
75'-0"
(Span 2)
Limits of damage
1
2
3
4
PARTIAL FRAMING PLAN
Impact Line
1 Deformed Beam.
2 Top and Bottom Diaphragm connection angles bent.
3 Bottom Diaphragm connection angle bent.
4 Diaphragm bent.
Beam No.
1
2
3
4
5
4 Spa. at 6'-5" = 25'-8"
FRAMING PLAN
RECORDING IMPACT
DAMAGED STEEL BEAMS

FIGURE 2.3.3-2

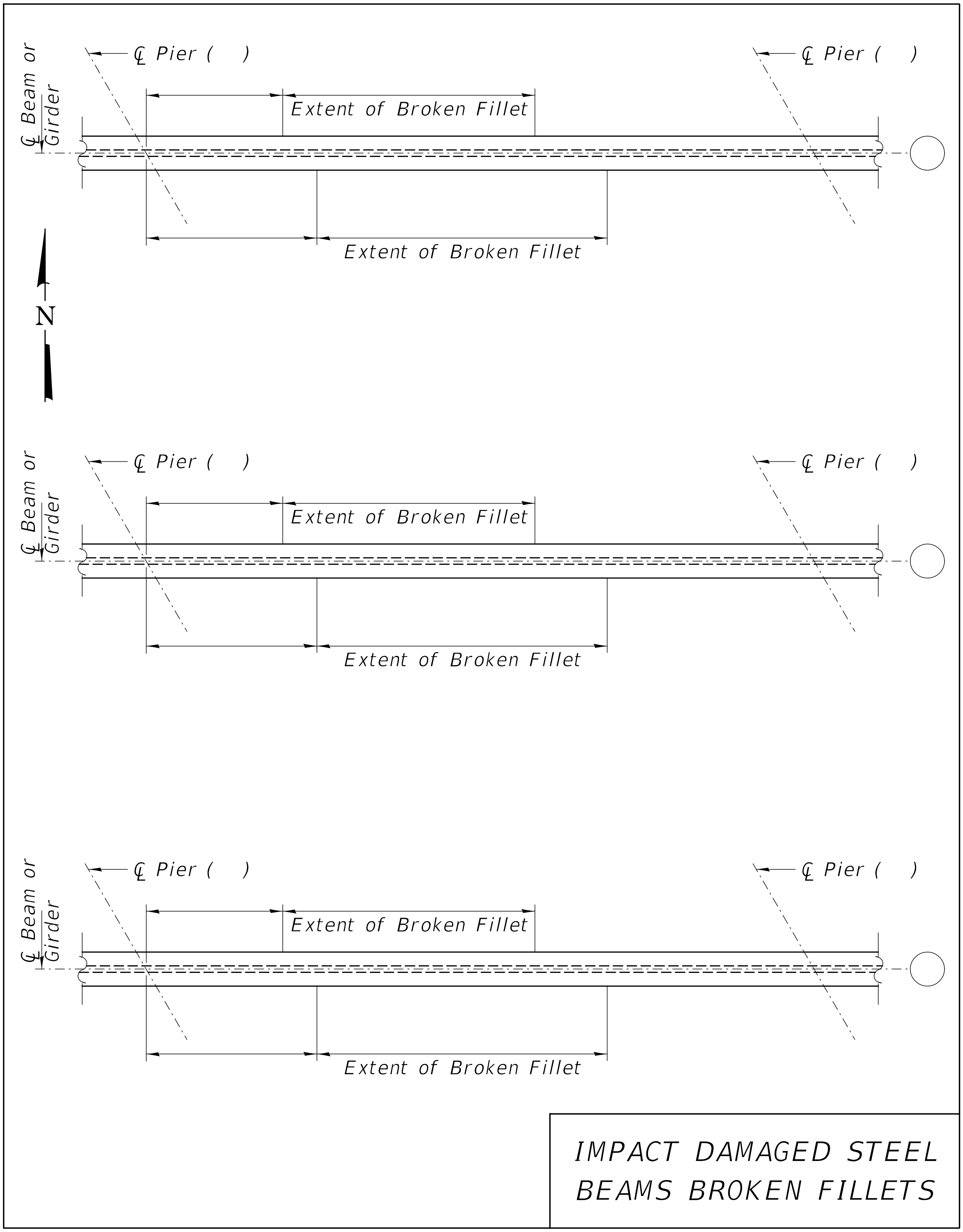

℄ Pier ( )
℄ Pier ( )
℄ Beam or Girder
Extent of Broken Fillet
Extent of Broken Fillet
N
℄ Pier ( )
℄ Pier ( )
℄ Beam or Girder
Extent of Broken Fillet
Extent of Broken Fillet
℄ Pier ( )
℄ Pier ( )
℄ Beam or Girder
Extent of Broken Fillet
Extent of Broken Fillet
IMPACT DAMAGED STEEL BEAMS BROKEN FILLETS

FIGURE 2.3.3-3

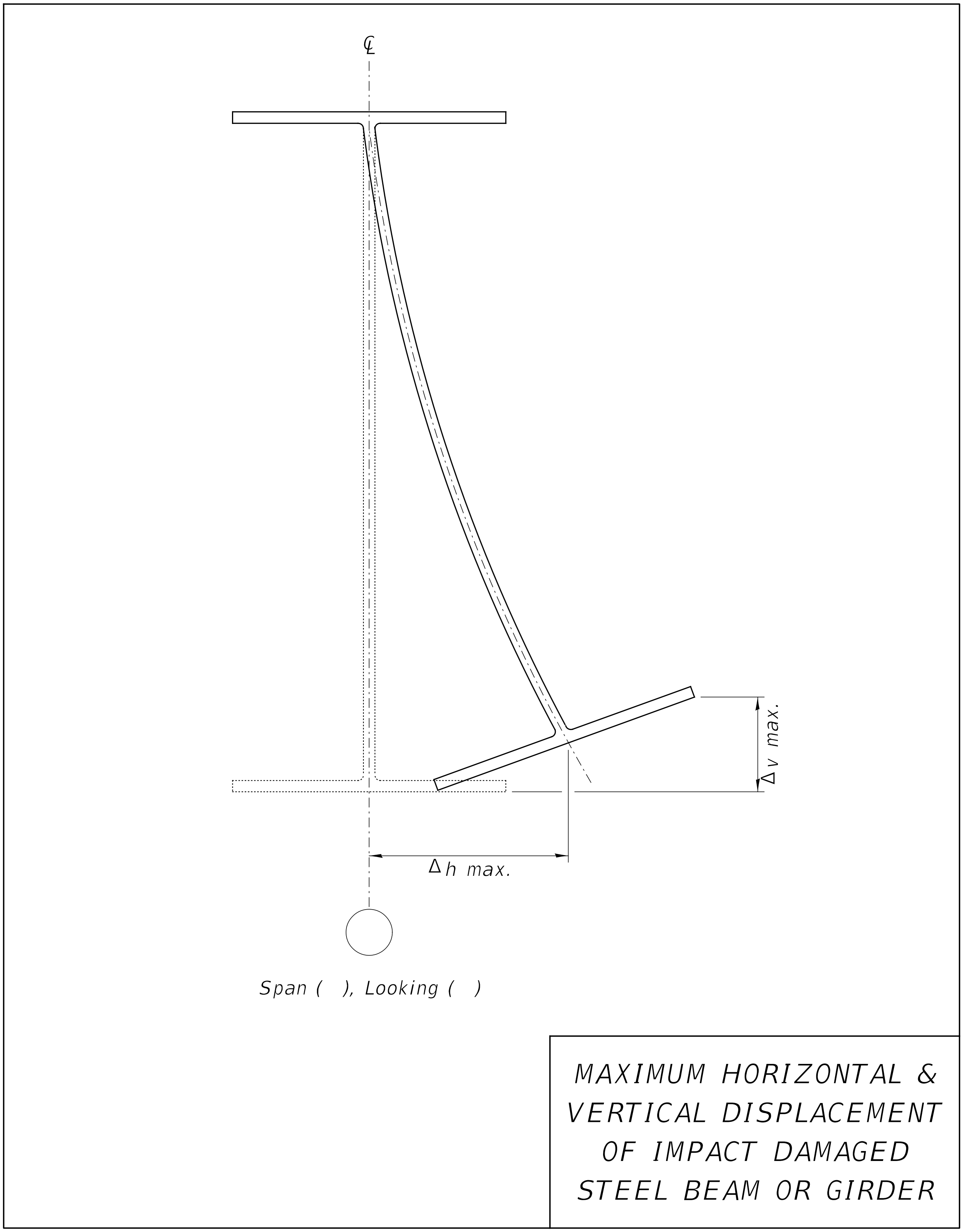

℄
Δv max.
Δh max.
Span ( ), Looking ( )
MAXIMUM HORIZONTAL & VERTICAL DISPLACEMENT OF IMPACT DAMAGED STEEL BEAM OR GIRDER

FIGURE 2.3.3-4

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.

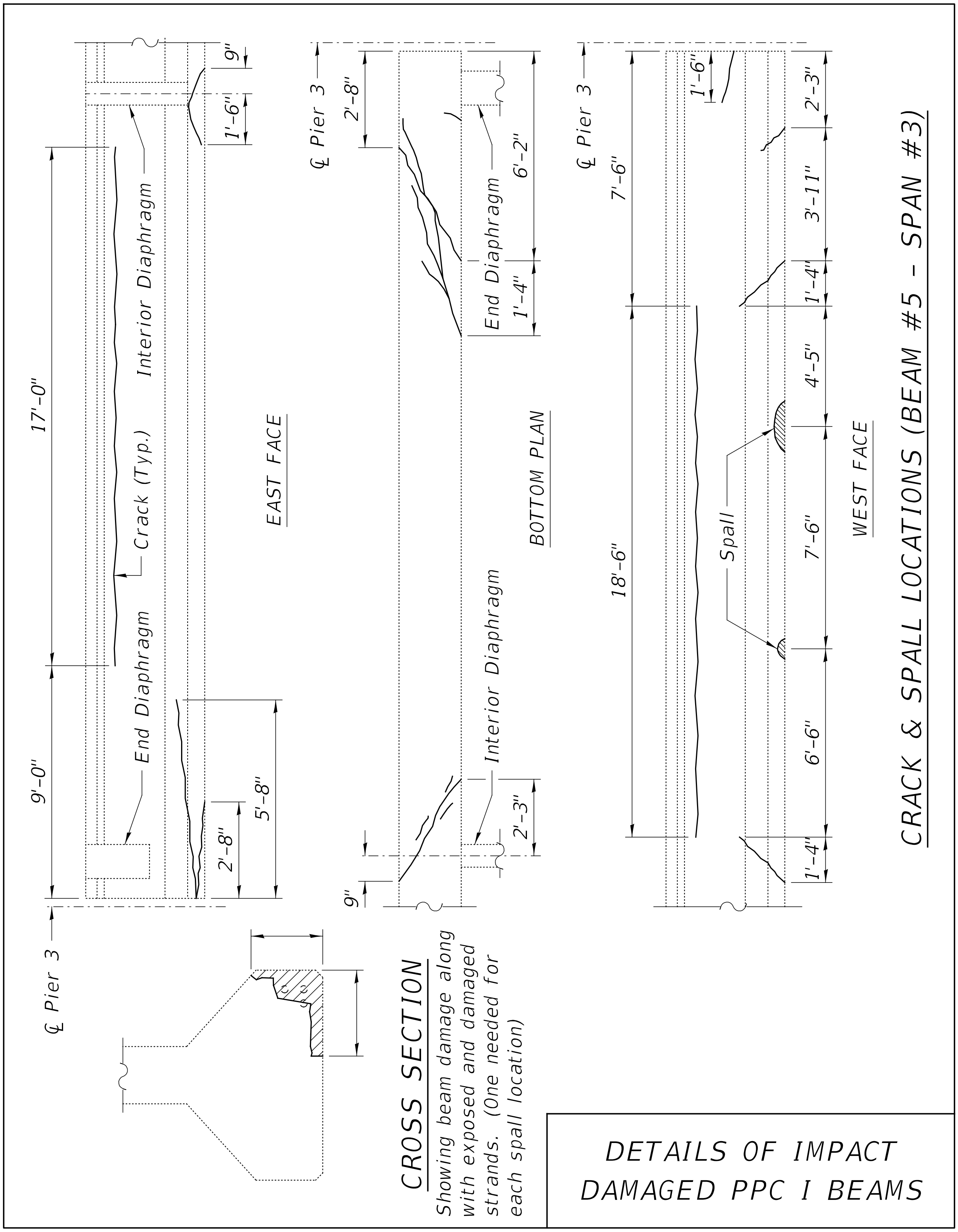

CRACK & SPALL LOCATIONS (BEAM #5 - SPAN #3)
EAST FACE
BOTTOM PLAN
WEST FACE
℄ Pier 3
Interior Diaphragm
End Diaphragm
Crack (Typ.)
Spall
17'-0"
9'-0"
9"
1'-6"
5'-8"
2'-8"
6'-2"
1'-4"
2'-3"
7'-6"
18'-6"
3'-11"
4'-5"
6'-6"
CROSS SECTION
Showing beam damage along with exposed and damaged strands. (One needed for each spall location)
DETAILS OF IMPACT DAMAGED PPC I BEAMS

FIGURE 2.3.4-1

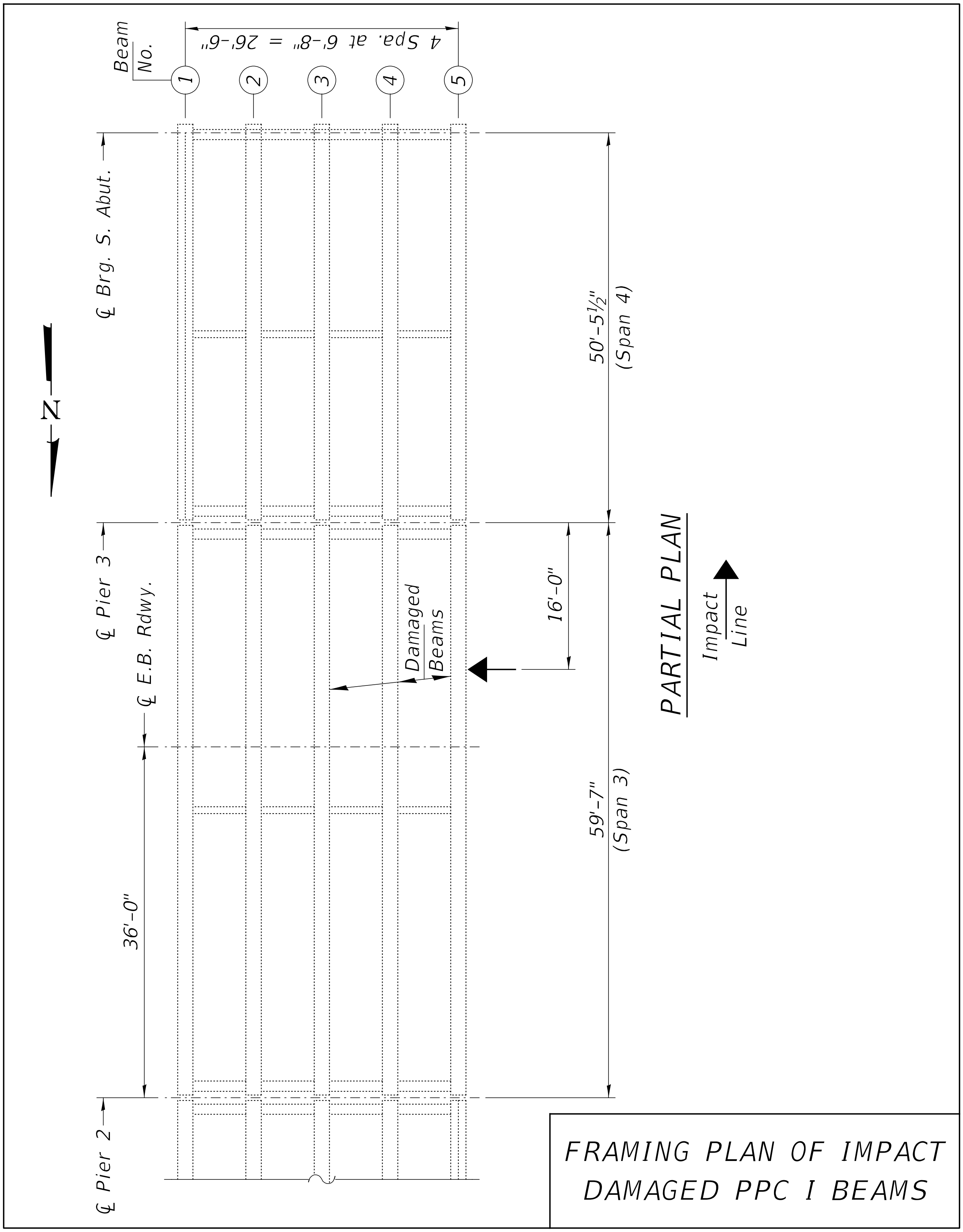

Beam No.
4 Spa. at 6'-8" = 26'-6"
1
2
3
4
5
℄ Brg. S. Abut.
N
50'-5½"
(Span 4)
℄ Pier 3
℄ E.B. Rdwy.
Damaged Beams
16'-0"
PARTIAL PLAN
Impact Line
59'-7"
(Span 3)
36'-0"
℄ Pier 2
FRAMING PLAN OF IMPACT
DAMAGED PPC I BEAMS

FIGURE 2.3.4--2

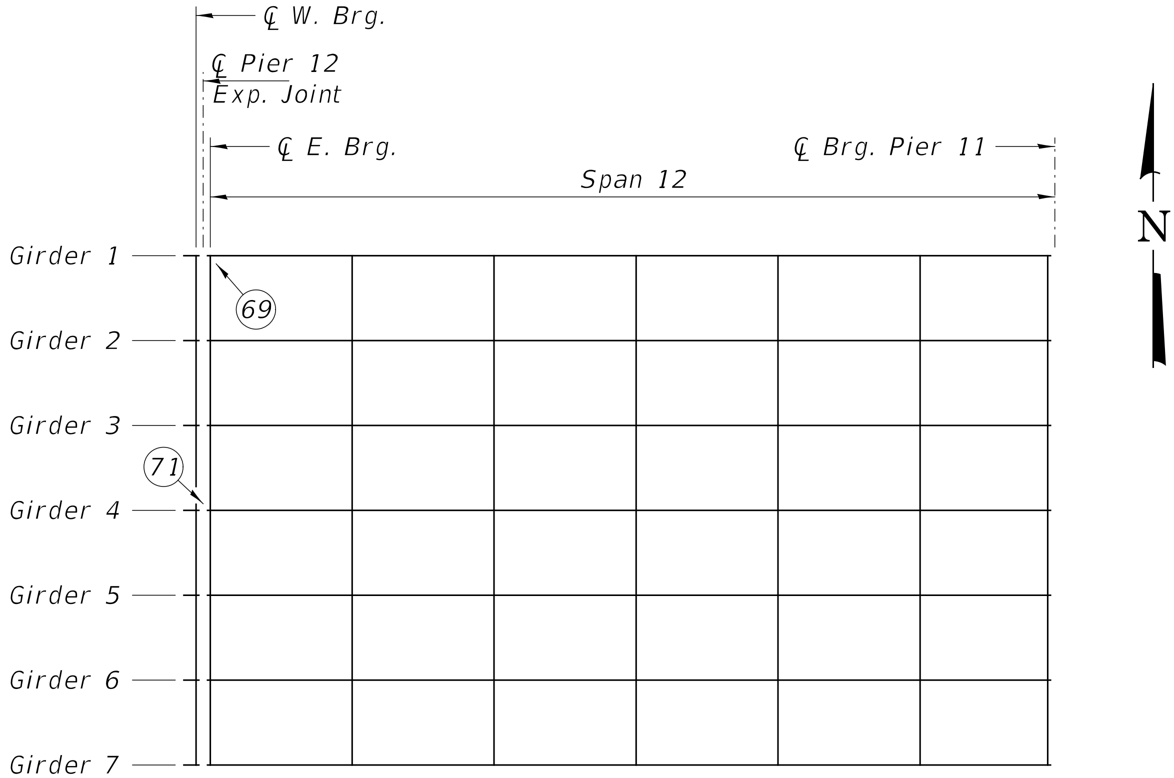
2.3.5 Inspection Photographs

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:

a) The Structure Number (SN),
b) Date of photo
c) The specific member (i.e., Beam 1)
d) The overall location (i.e., “at south abutment”)
e) The specific location (i.e., “west face", “web”, ”stiffener” etc.)
e) The type of damage (i.e., “crack”, “section loss”, “deflection” etc.)
f) 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

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

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 3.5 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 Fig. 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 GBSP 28 “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 Fig. 2.4.3-1 thru Fig. 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.

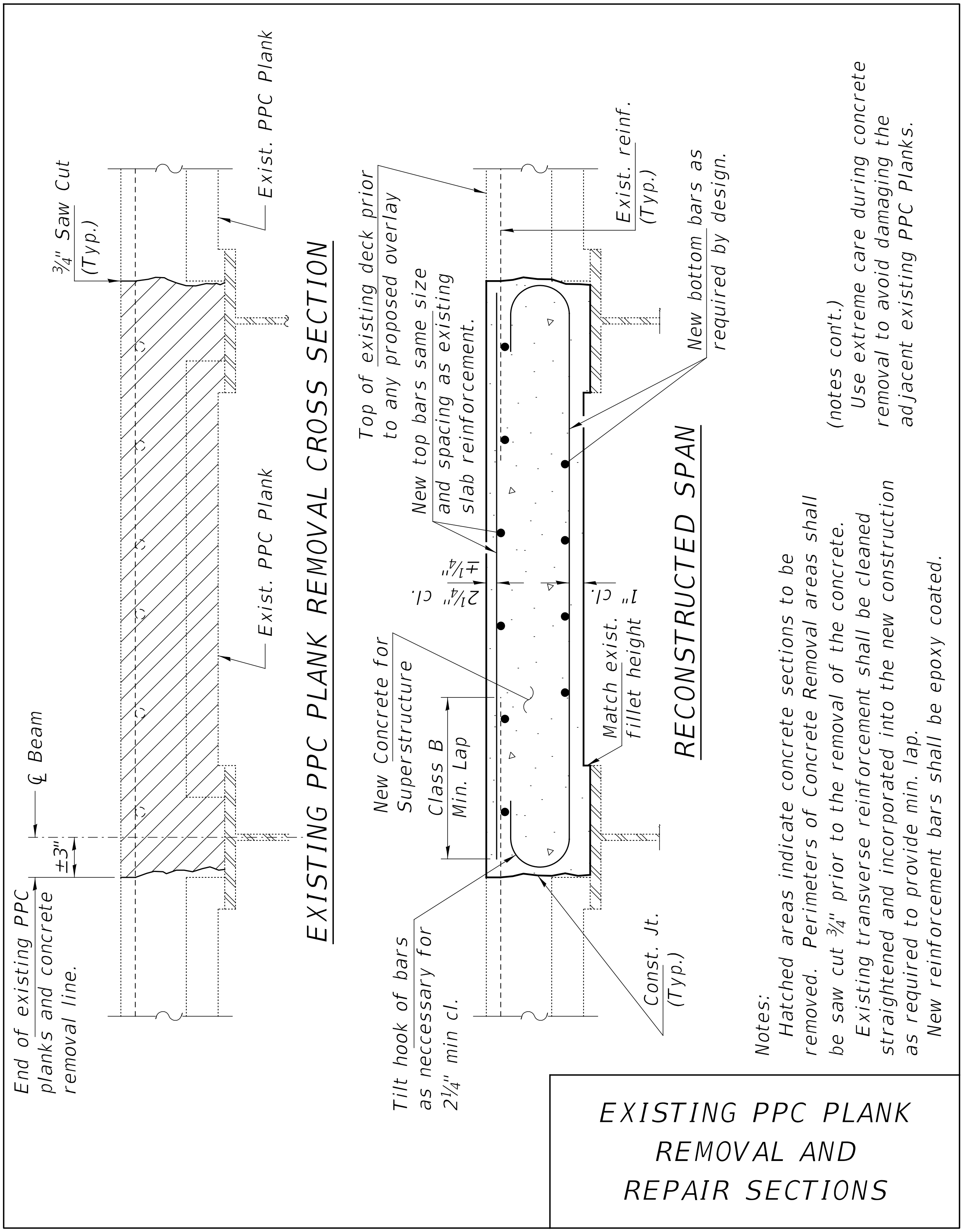

EXISTING PPC PLANK REMOVAL CROSS SECTION
¾" Saw Cut (Typ.)
Exist. PPC Plank
Exist. PPC Plank
℄ Beam
±3"
End of existing PPC planks and concrete removal line.
RECONSTRUCTED SPAN
Top of existing deck prior to any proposed overlay
Exist. reinf. (Typ.)
New top bars same size and spacing as existing slab reinforcement.
New bottom bars as required by design.
±¼"
2¼" cl.
1" cl.
New Concrete for Superstructure
Class B Min. Lap
Match exist. fillet height
Tilt hook of bars as neccessary for 2¼" min cl.
Const. Jt. (Typ.)
Notes:
Hatched areas indicate concrete sections to be removed. Perimeters of Concrete Removal areas shall be saw cut ¾" prior to the removal of the concrete.
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction as required to provide min. lap.
New reinforcement bars shall be epoxy coated.
(notes con't.)
Use extreme care during concrete removal to avoid damaging the adjacent existing PPC Planks.
EXISTING PPC PLANK REMOVAL AND REPAIR SECTIONS

FIGURE 2.4.3-1

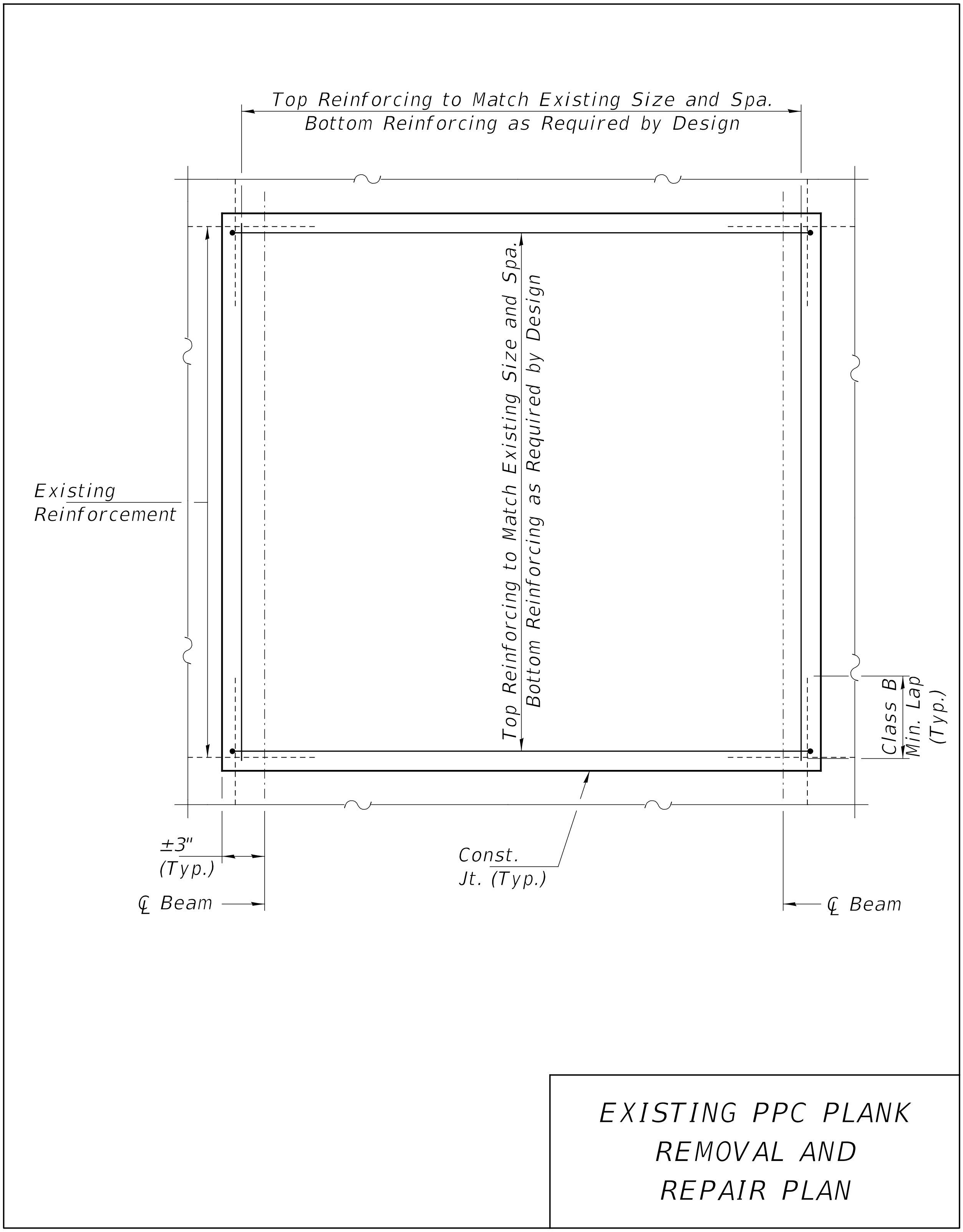

Top Reinforcing to Match Existing Size and Spa.
Bottom Reinforcing as Required by Design
Top Reinforcing to Match Existing Size and Spa.
Bottom Reinforcing as Required by Design
Existing
Reinforcement
Class B
Min. Lap
(Typ.)
±3"
(Typ.)
℄ Beam
Const.
Jt. (Typ.)
℄ Beam
EXISTING PPC PLANK
REMOVAL AND
REPAIR PLAN

FIGURE 2.4.3-2

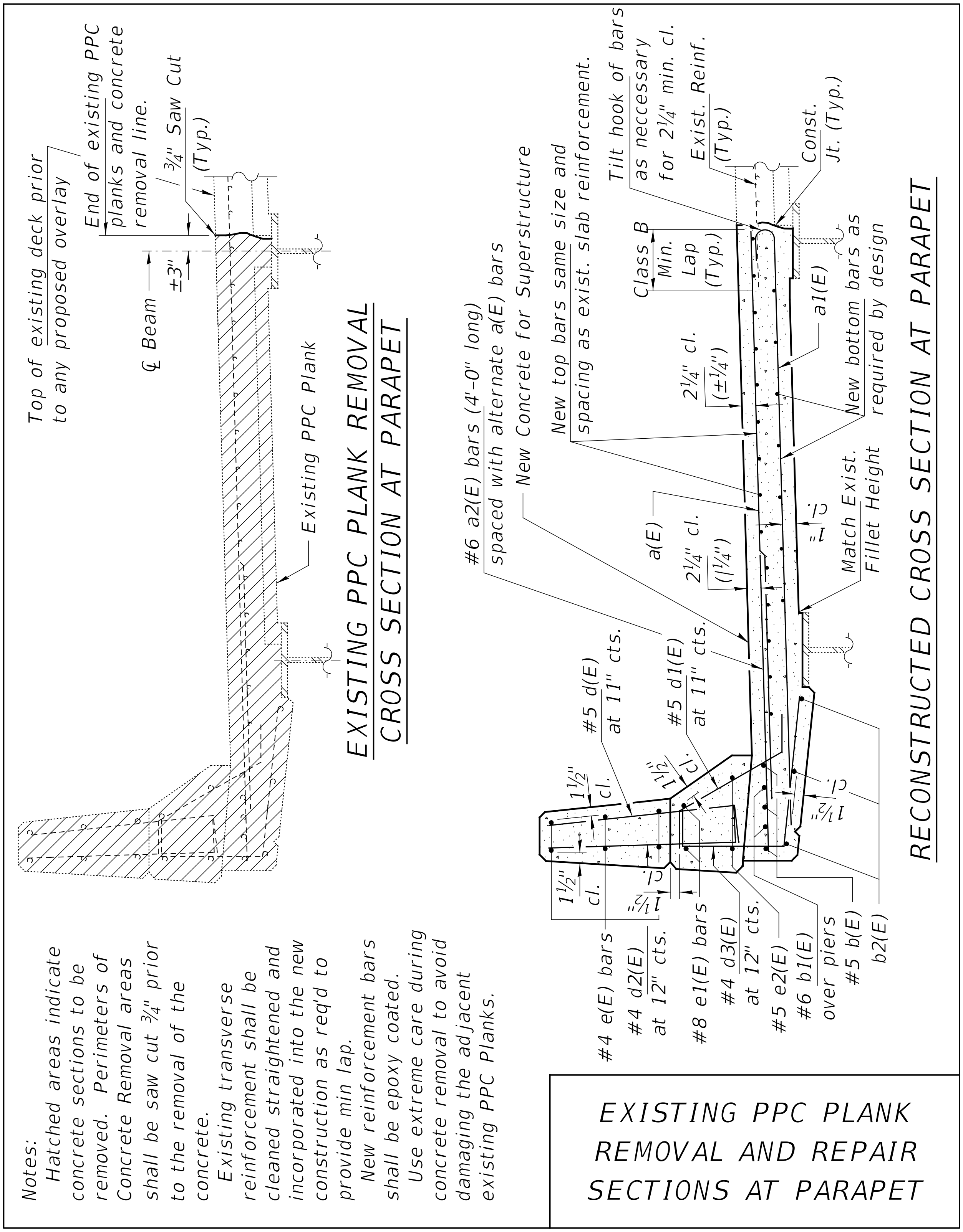

EXISTING PPC PLANK REMOVAL CROSS SECTION AT PARAPET
Top of existing deck prior to any proposed overlay
End of existing PPC planks and concrete removal line.
3/4" Saw Cut (Typ.)
℄ Beam
±3"
Existing PPC Plank
RECONSTRUCTED CROSS SECTION AT PARAPET
Tilt hook of bars as neccessary for 2 1/4" min. cl.
Exist. Reinf. (Typ.)
Const. Jt. (Typ.)
Class B Min. Lap (Typ.)
New top bars same size and spacing as exist. slab reinforcement.
New Concrete for Superstructure
#6 a2(E) bars (4'-0" long) spaced with alternate a(E) bars
a1(E)
2 1/4" cl. (±1 1/4")
New bottom bars as required by design
a(E)
2 1/4" cl. (1 1/4")
1" cl.
Match Exist. Fillet Height
#5 d(E) at 11" cts.
#5 d1(E) at 11" cts.
1 1/2" cl.
1 1/2" cl.
1 1/2" cl.
1 1/2" cl.
1 1/2" cl.
#4 e(E) bars
#4 d2(E) at 12" cts.
#8 e1(E) bars
#4 d3(E) at 12" cts.
#5 e2(E)
#6 b1(E) over piers
#5 b(E)
b2(E)
Notes:
Hatched areas indicate concrete sections to be removed. Perimeters of Concrete Removal areas shall be saw cut 3/4" prior to the removal of the concrete.
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction as req'd to provide min lap.
New reinforcement bars shall be epoxy coated.
Use extreme care during concrete removal to avoid damaging the adjacent existing PPC Planks.
EXISTING PPC PLANK REMOVAL AND REPAIR SECTIONS AT PARAPET

FIGURE 2.4.3-3

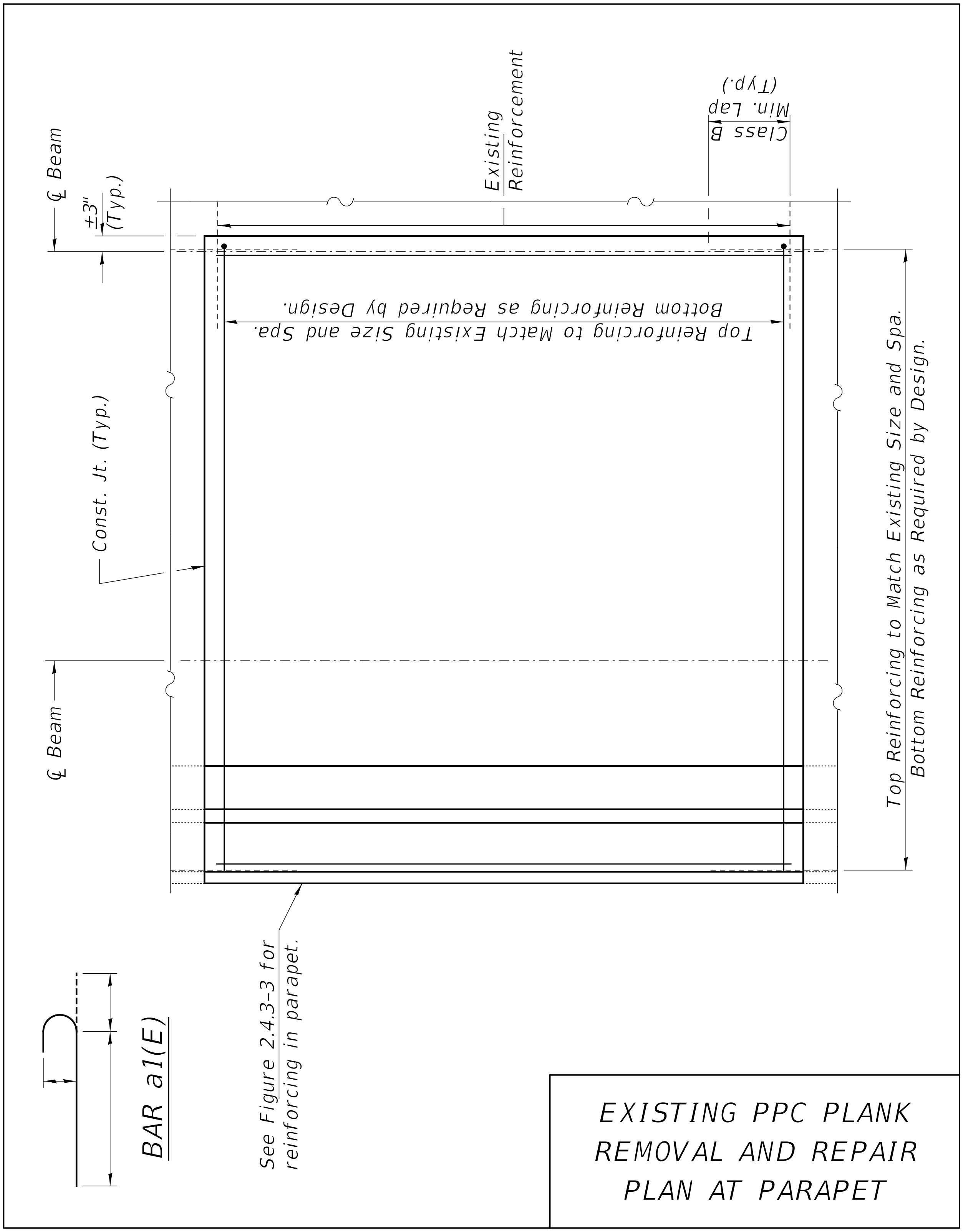

Existing Reinforcement
Class B Min. Lap (Typ.)
℄ Beam
±3" (Typ.)
Bottom Reinforcing as Required by Design.
Top Reinforcing to Match Existing Size and Spa.
Const. Jt. (Typ.)
℄ Beam
Top Reinforcing to Match Existing Size and Spa.
Bottom Reinforcing as Required by Design.
BAR a1(E)
See Figure 2.4.3-3 for reinforcing in parapet.
EXISTING PPC PLANK
REMOVAL AND REPAIR
PLAN AT PARAPET

FIGURE 2.4.3-4

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.

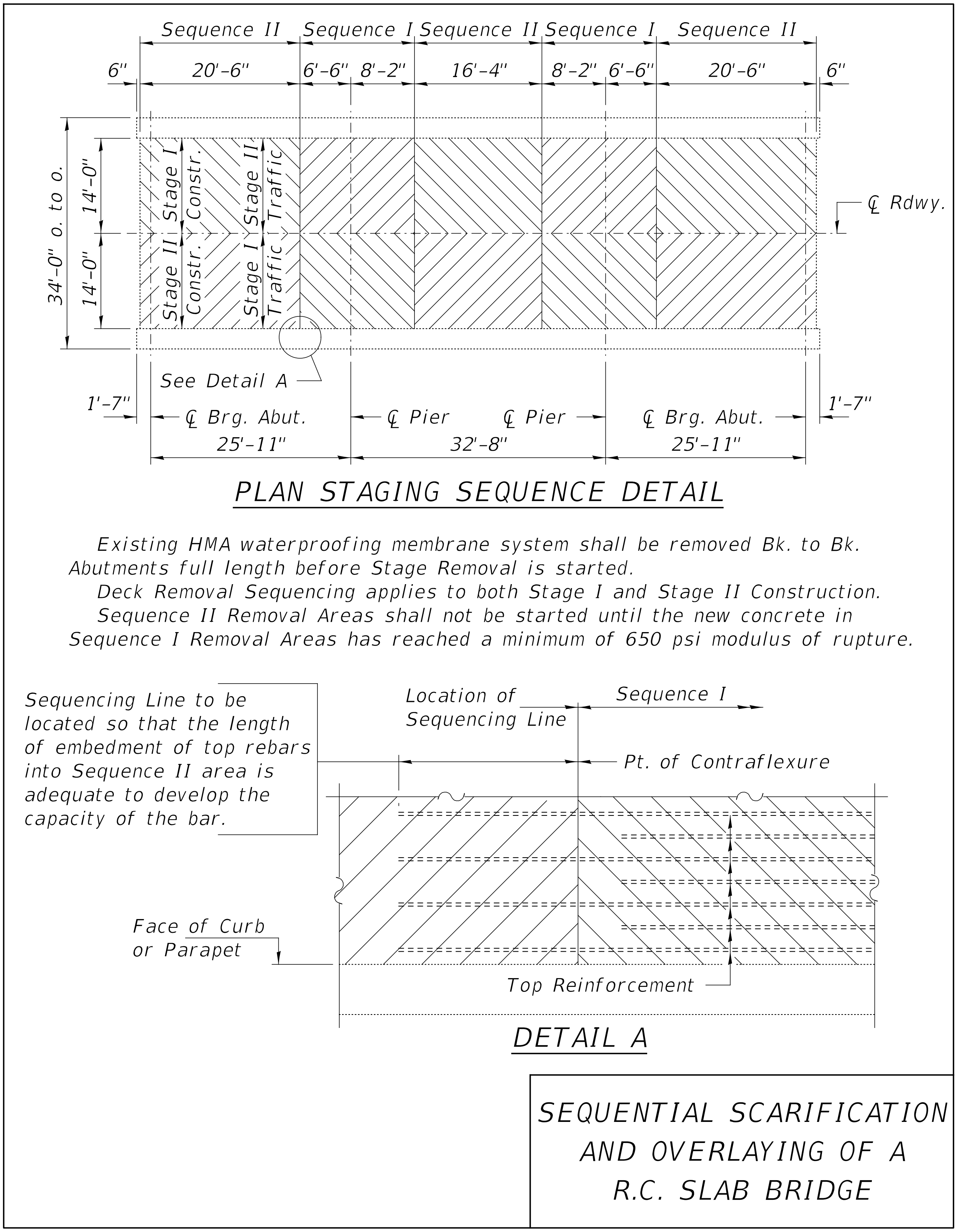

Sequence II
Sequence I
Sequence II
Sequence I
Sequence II
6"
20'-6"
6'-6"
8'-2"
16'-4"
8'-2"
6'-6"
20'-6"
6"
34'-0" o. to o.
14'-0"
14'-0"
Stage I Constr.
Stage II Traffic
Stage II Constr.
Stage I Traffic
℄ Rdwy.
See Detail A
1'-7"
℄ Brg. Abut.
℄ Pier
℄ Pier
℄ Brg. Abut.
1'-7"
25'-11"
32'-8"
25'-11"
PLAN STAGING SEQUENCE DETAIL
Existing HMA waterproofing membrane system shall be removed Bk. to Bk. Abutments full length before Stage Removal is started.
Deck Removal Sequencing applies to both Stage I and Stage II Construction.
Sequence II Removal Areas shall not be started until the new concrete in Sequence I Removal Areas has reached a minimum of 650 psi modulus of rupture.
Sequencing Line to be located so that the length of embedment of top rebars into Sequence II area is adequate to develop the capacity of the bar.
Location of Sequencing Line
Sequence I
Pt. of Contraflexure
Face of Curb or Parapet
Top Reinforcement
DETAIL A
SEQUENTIAL SCARIFICATION AND OVERLAYING OF A R.C. SLAB BRIDGE

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:

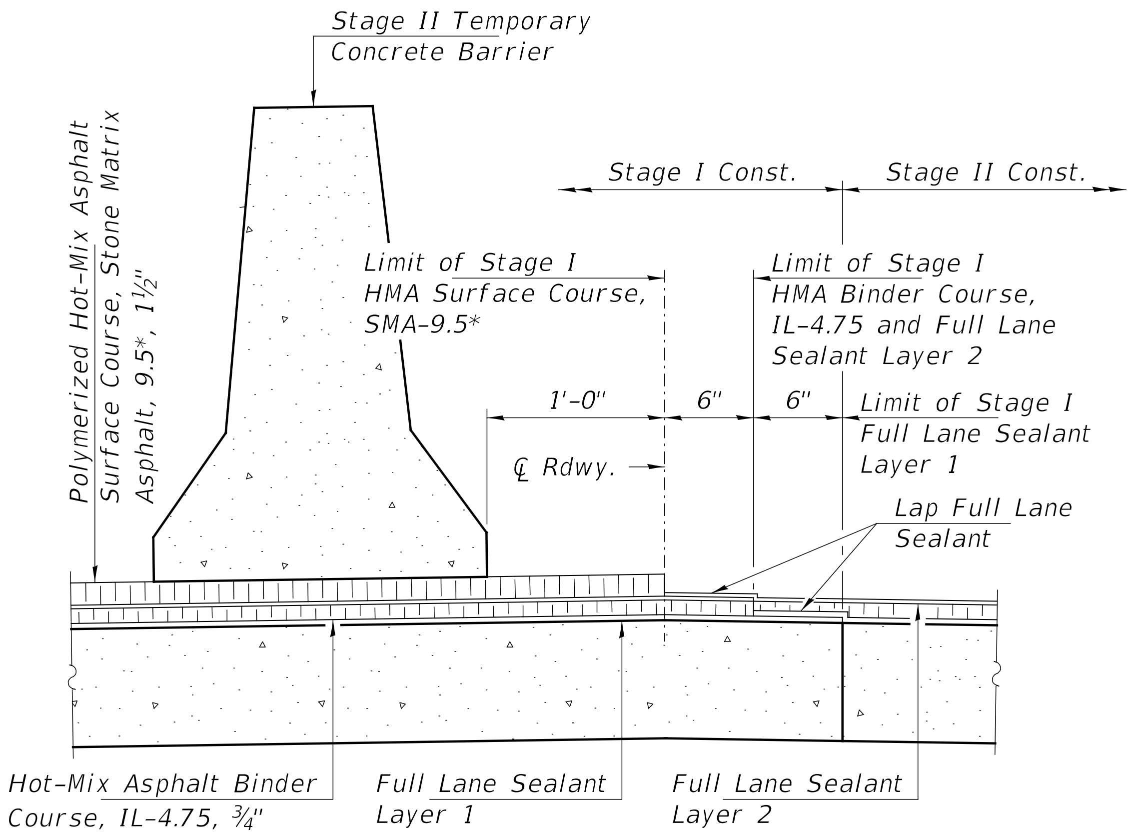
1. At least 72 hours shall have elapsed from the end of the previous pour and

2. The concrete shall have attained a minimum modulus of rupture of 650 psi or a minimum compressive strength of 3500 psi.

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

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 ABD 17.1 Bridge Deck Smoothness Grinding Policy. 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 no. 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.

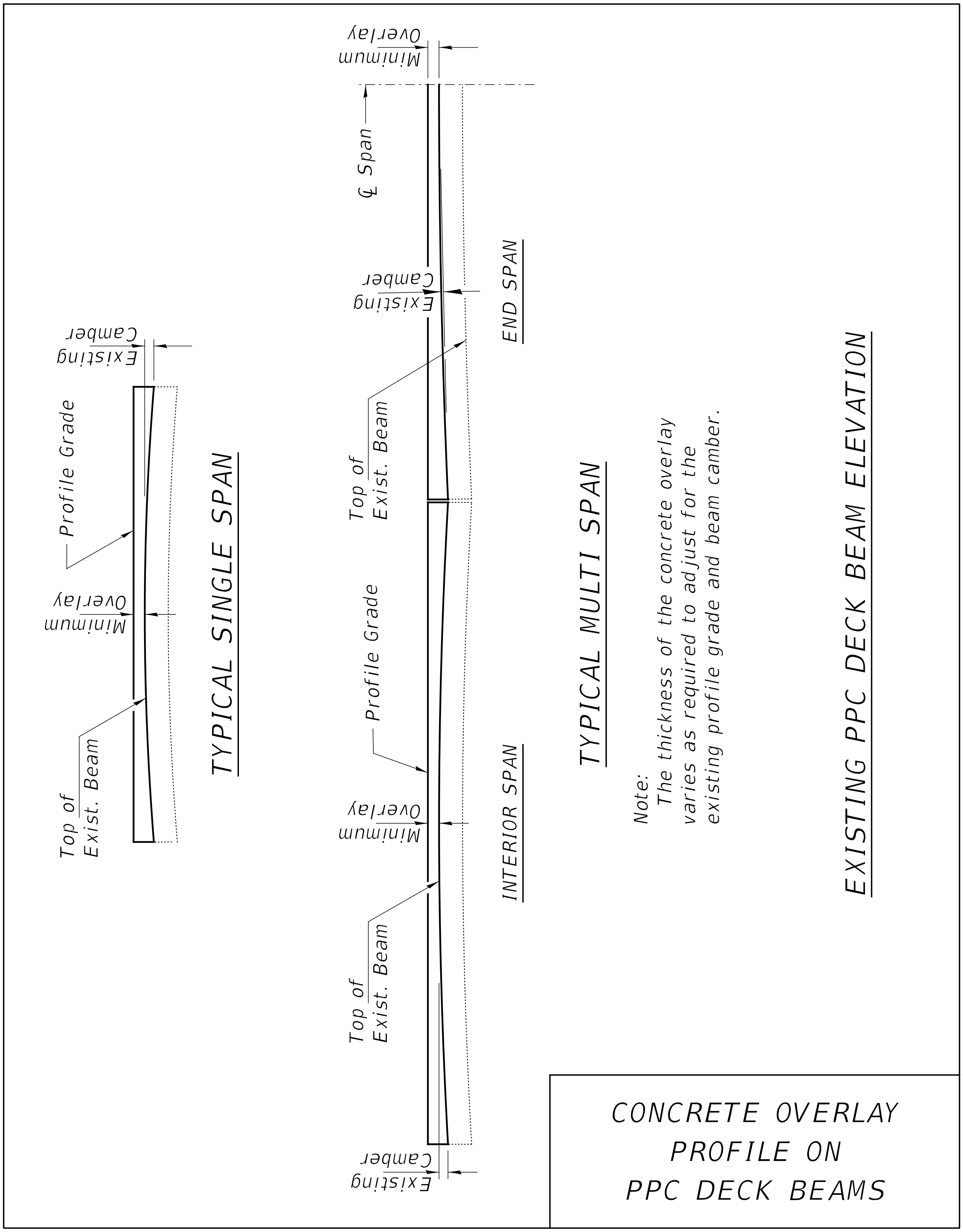

CONCRETE OVERLAY
PROFILE ON
PPC DECK BEAMS
EXISTING PPC DECK BEAM ELEVATION
TYPICAL SINGLE SPAN
TYPICAL MULTI SPAN
END SPAN
INTERIOR SPAN
Profile Grade
Top of Exist. Beam
Minimum Overlay
Existing Camber
℄ Span
Note:
The thickness of the concrete overlay
varies as required to adjust for the
existing profile grade and beam camber.

FIGURE 2.4.8-1

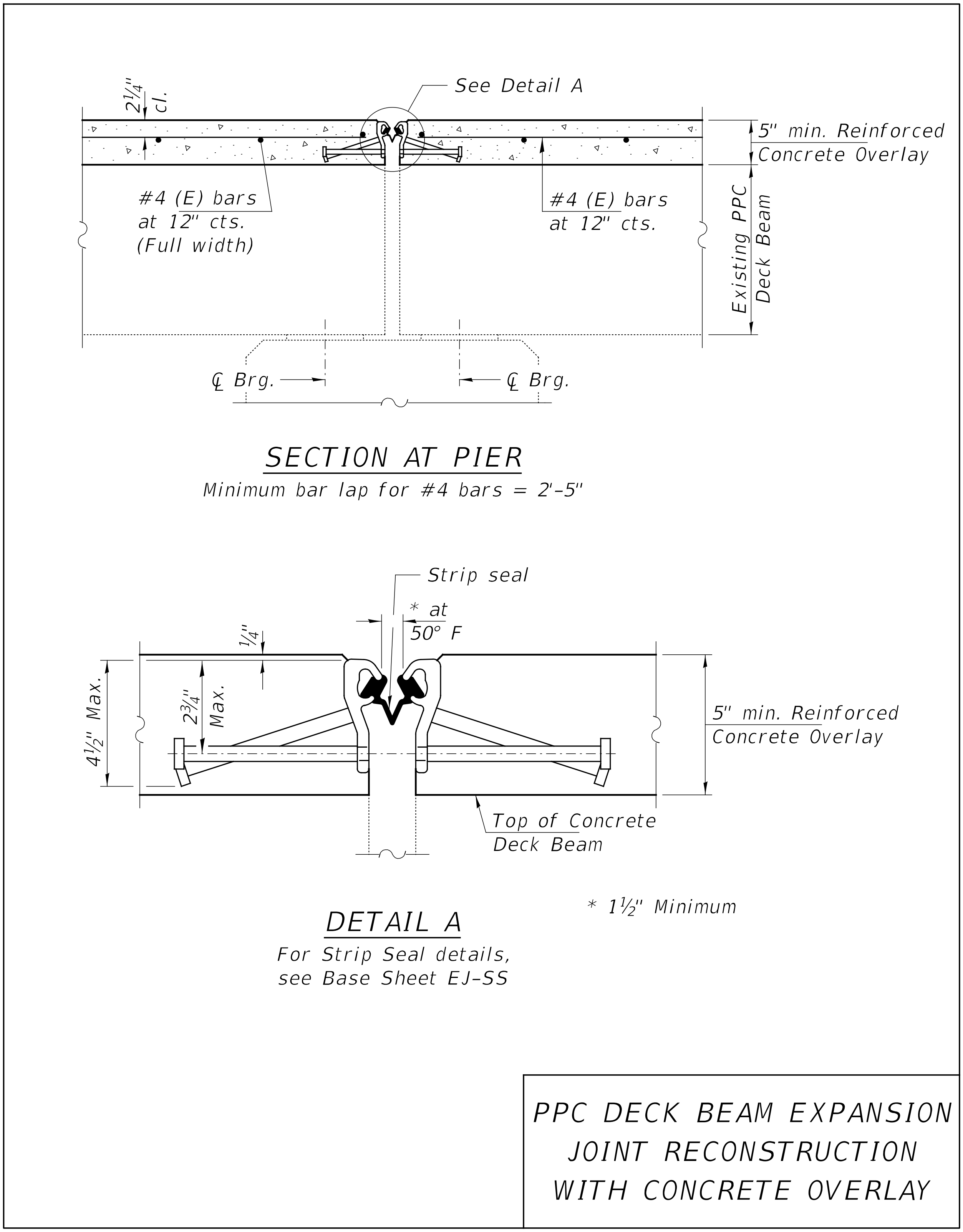

See Detail A
2¼" cl.
5" min. Reinforced Concrete Overlay
#4 (E) bars at 12" cts. (Full width)
#4 (E) bars at 12" cts.
Existing PPC Deck Beam
℄ Brg.
℄ Brg.
SECTION AT PIER
Minimum bar lap for #4 bars = 2'-5"
Strip seal
* at 50° F
¼"
2¾" Max.
4½" Max.
5" min. Reinforced Concrete Overlay
Top of Concrete Deck Beam
* 1½" Minimum
DETAIL A
For Strip Seal details, see Base Sheet EJ-SS
PPC DECK BEAM EXPANSION JOINT RECONSTRUCTION WITH CONCRETE OVERLAY

FIGURE 2.4.8-2

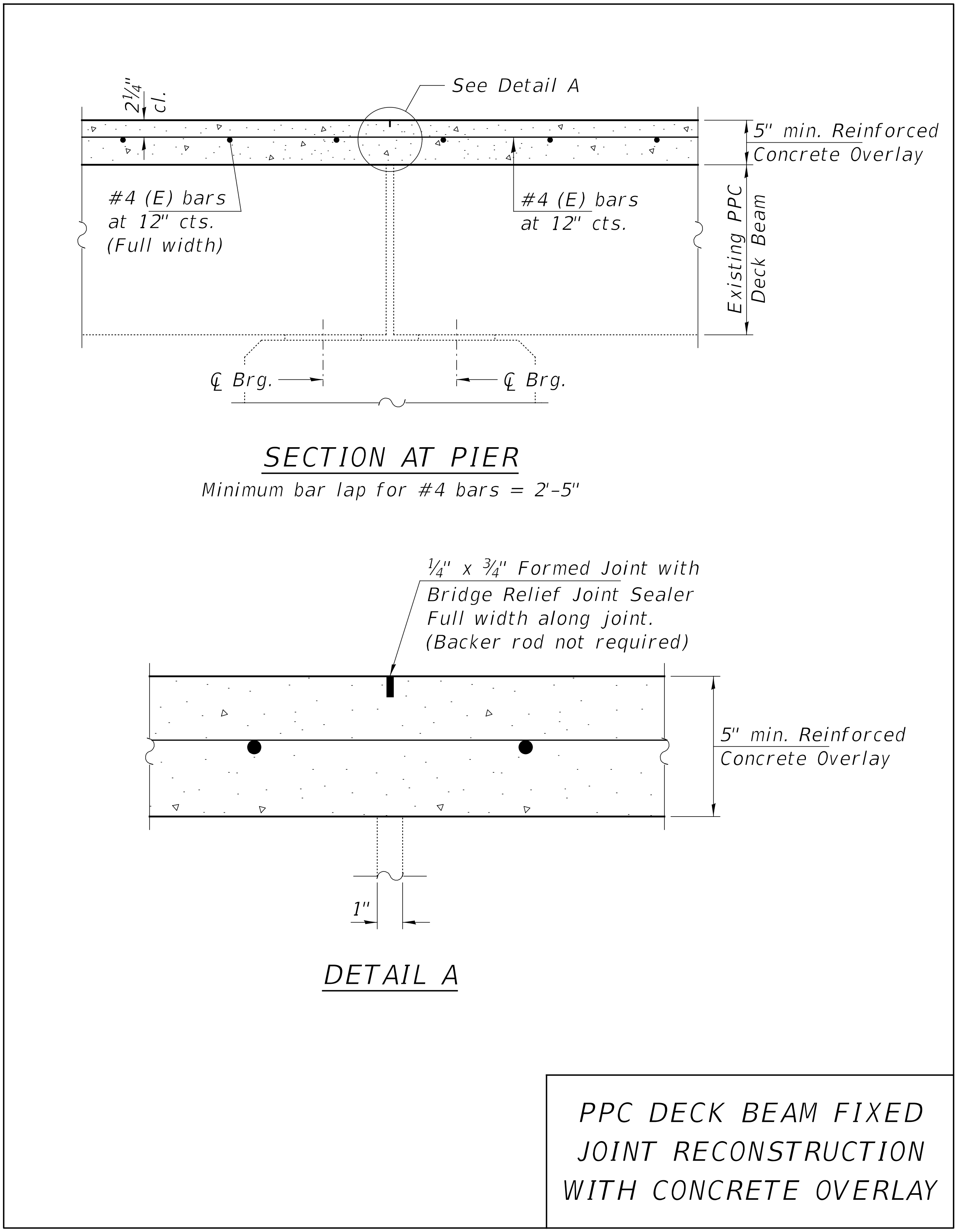

See Detail A
2¼" cl.
5" min. Reinforced Concrete Overlay
#4 (E) bars at 12" cts. (Full width)
#4 (E) bars at 12" cts.
Existing PPC Deck Beam
℄ Brg.
℄ Brg.
SECTION AT PIER
Minimum bar lap for #4 bars = 2'-5"
¼" x ¾" Formed Joint with Bridge Relief Joint Sealer Full width along joint. (Backer rod not required)
5" min. Reinforced Concrete Overlay
1"
DETAIL A
PPC DECK BEAM FIXED JOINT RECONSTRUCTION WITH CONCRETE OVERLAY

FIGURE 2.4.8-3

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 Design Section of the IDOT *Bridge Manual* (Fig. 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 Guide Bridge Special Provision (GBSP) 93 (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.

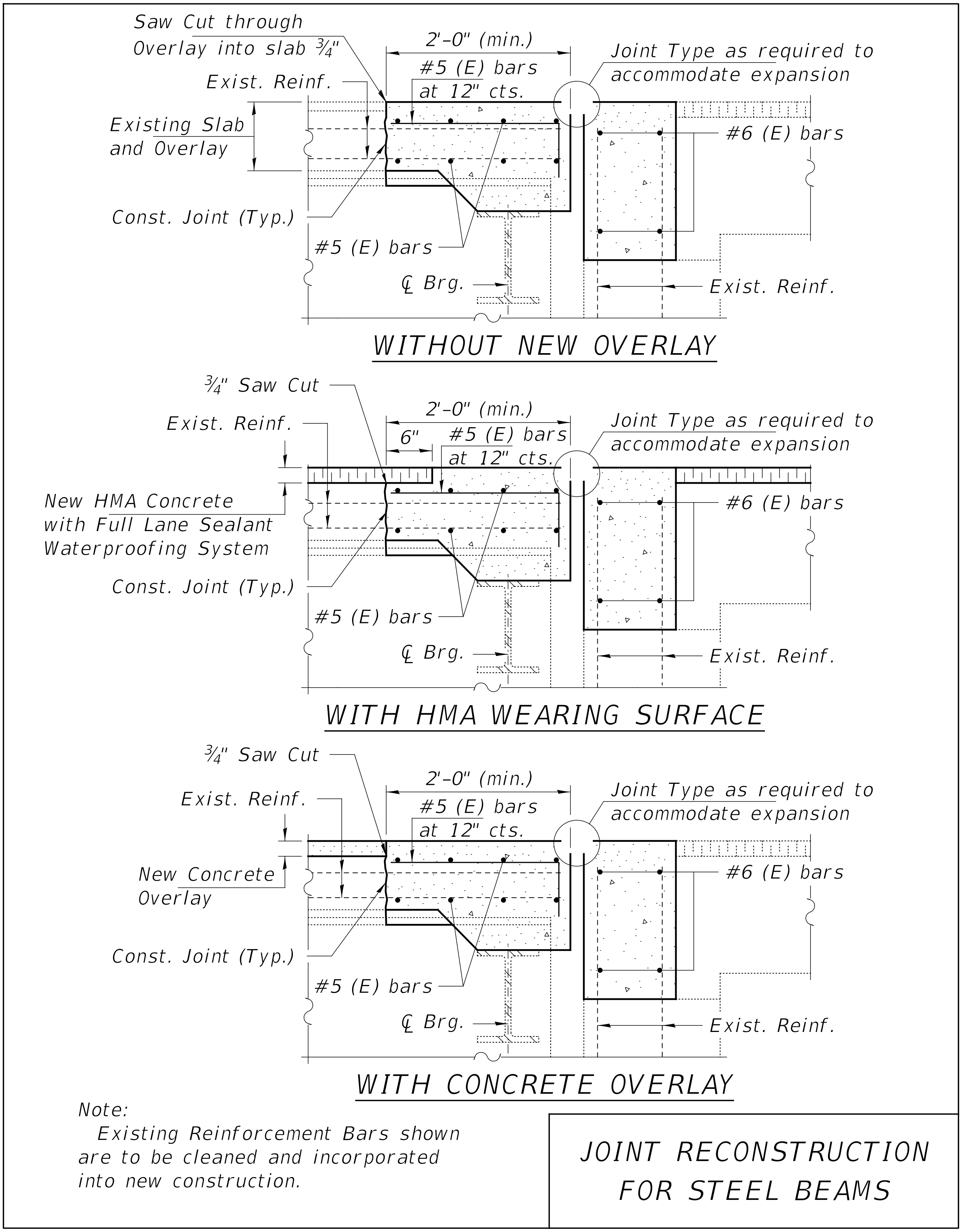
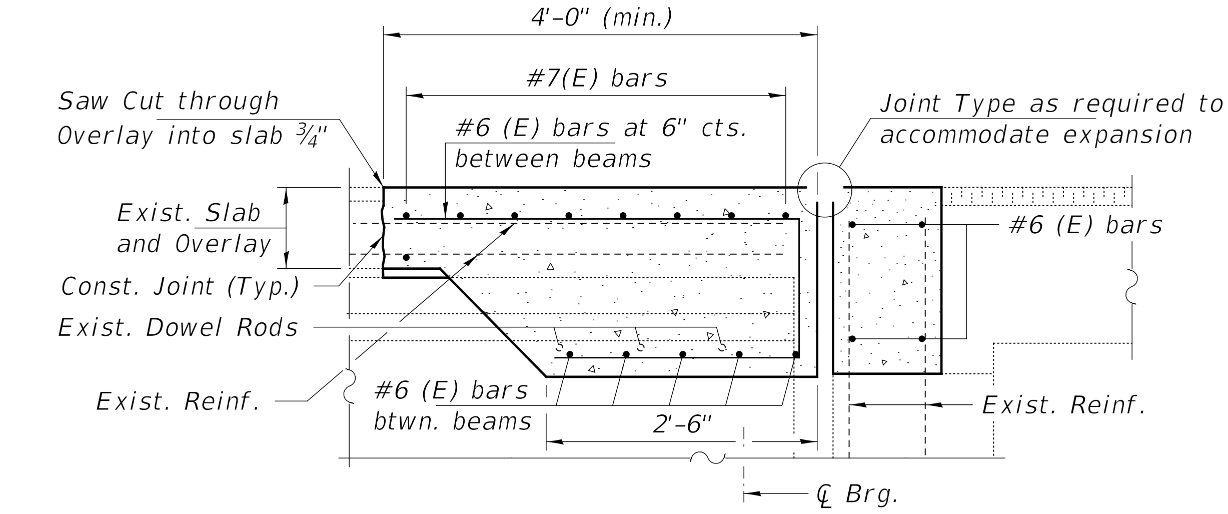
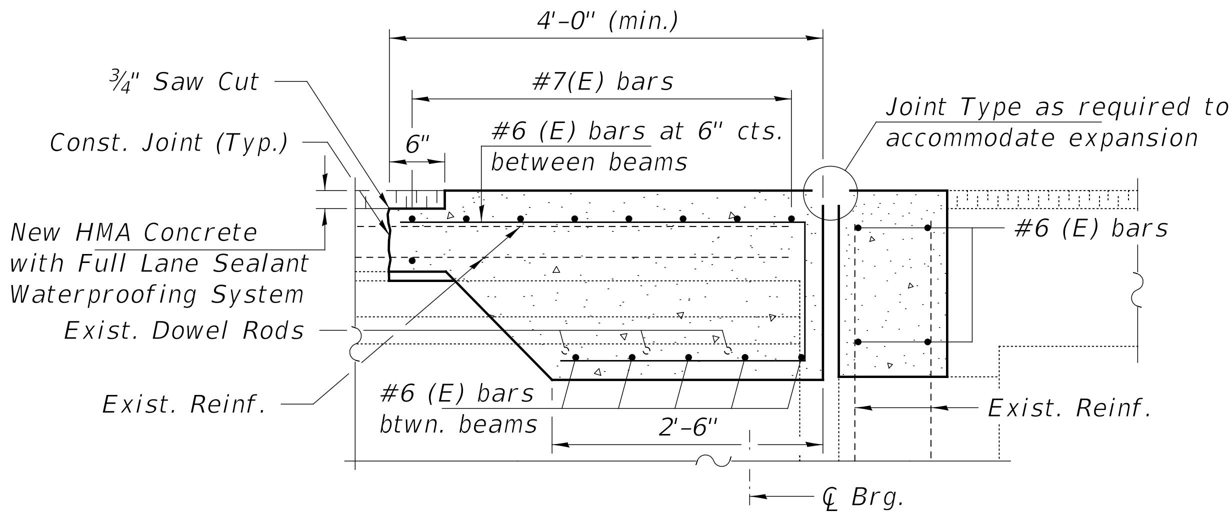

Saw Cut through Overlay into slab ¾"
2'-0" (min.)
Exist. Reinf.
#5 (E) bars at 12" cts.
Joint Type as required to accommodate expansion
Existing Slab and Overlay
#6 (E) bars
Const. Joint (Typ.)
#5 (E) bars
℄ Brg.
Exist. Reinf.
WITHOUT NEW OVERLAY
¾" Saw Cut
2'-0" (min.)
Exist. Reinf.
6"
#5 (E) bars at 12" cts.
Joint Type as required to accommodate expansion
New HMA Concrete with Full Lane Sealant Waterproofing System
#6 (E) bars
Const. Joint (Typ.)
#5 (E) bars
℄ Brg.
Exist. Reinf.
WITH HMA WEARING SURFACE
¾" Saw Cut
2'-0" (min.)
Exist. Reinf.
#5 (E) bars at 12" cts.
Joint Type as required to accommodate expansion
New Concrete Overlay
#6 (E) bars
Const. Joint (Typ.)
#5 (E) bars
℄ Brg.
Exist. Reinf.
WITH CONCRETE OVERLAY
Note:
Existing Reinforcement Bars shown are to be cleaned and incorporated into new construction.
JOINT RECONSTRUCTION FOR STEEL BEAMS

FIGURE 2.5.3-1



WITHOUT NEW OVERLAY

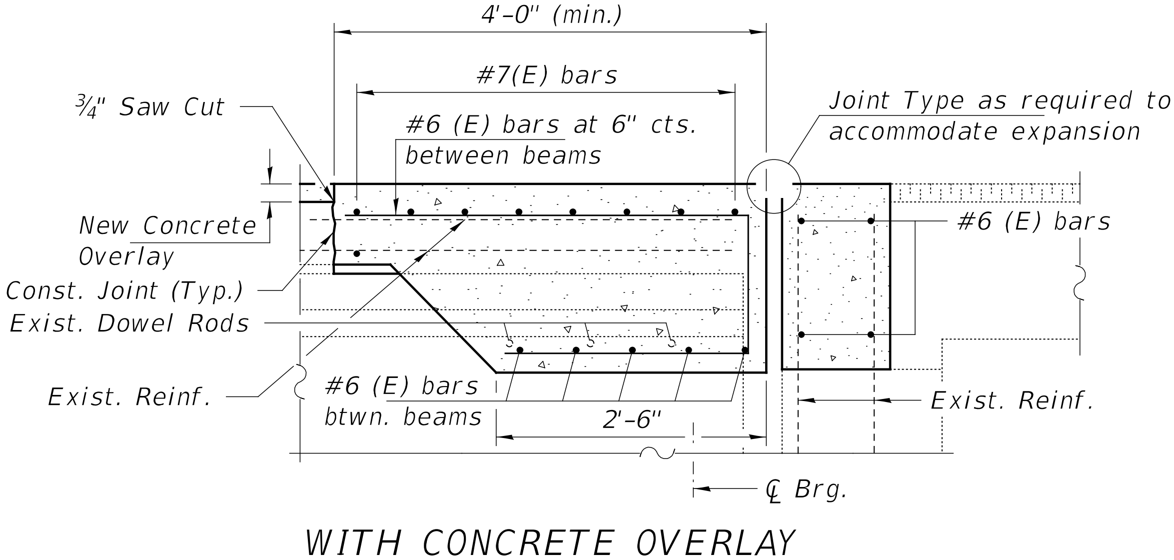


WITH HMA WEARING SURFACE

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

JOINT RECONSTRUCTION FOR PPC I BEAMS

FIGURE 2.5.3-2



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

JOINT RECONSTRUCTION
FOR PPC I BEAMS

FIGURE 2.5.3-3

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.

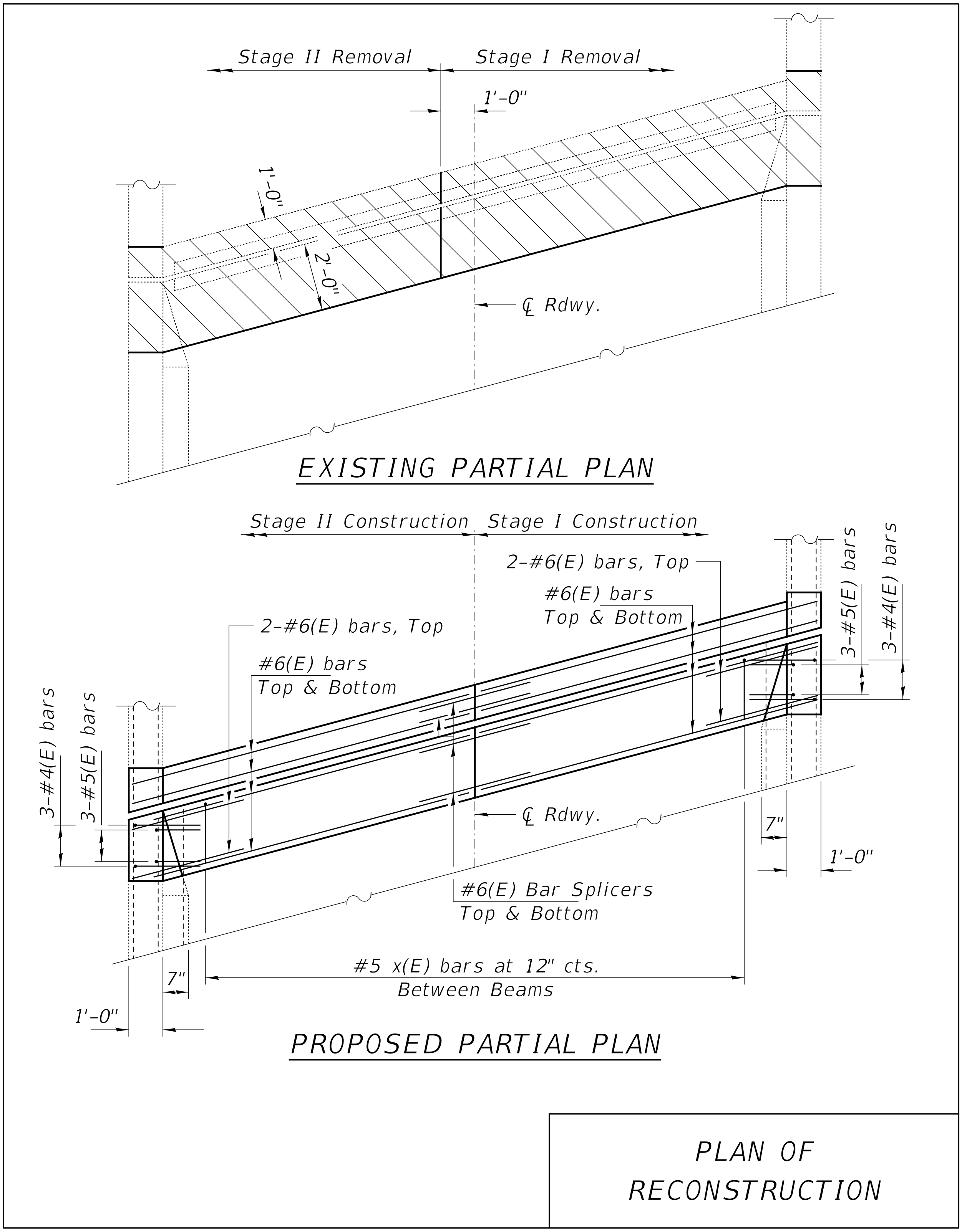

Stage II Removal
Stage I Removal
1'-0"
1'-0"
2'-0"
℄ Rdwy.
EXISTING PARTIAL PLAN
Stage II Construction
Stage I Construction
2-#6(E) bars, Top
#6(E) bars
Top & Bottom
2-#6(E) bars, Top
#6(E) bars
Top & Bottom
3-#5(E) bars
3-#4(E) bars
3-#4(E) bars
3-#5(E) bars
℄ Rdwy.
7"
1'-0"
#6(E) Bar Splicers
Top & Bottom
#5 x(E) bars at 12" cts.
Between Beams
7"
1'-0"
PROPOSED PARTIAL PLAN
PLAN OF
RECONSTRUCTION

FIGURE 2.5.4-1

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.

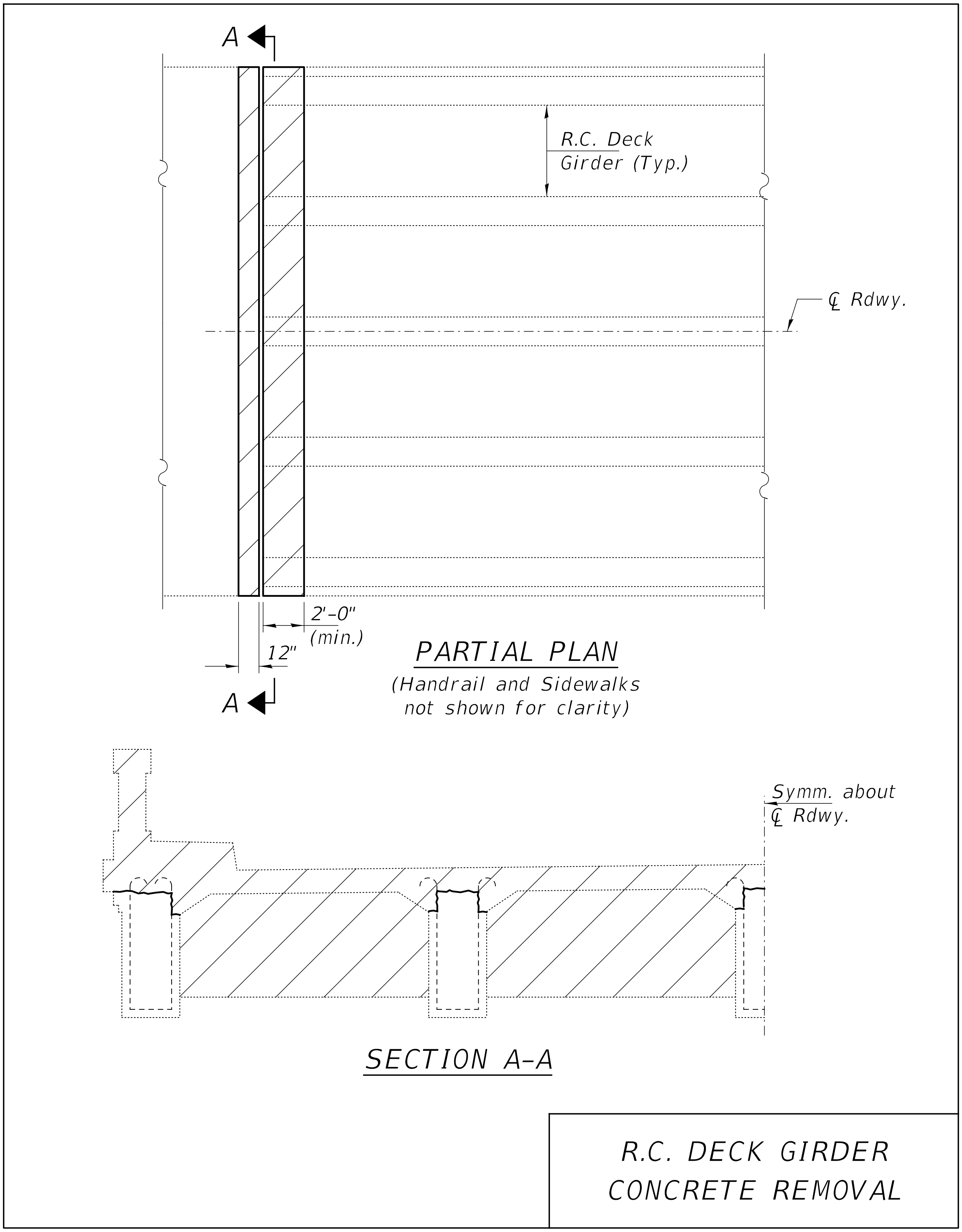

A
R.C. Deck
Girder (Typ.)
℄ Rdwy.
2'-0"
(min.)
12"
PARTIAL PLAN
(Handrail and Sidewalks
not shown for clarity)
A
Symm. about
℄ Rdwy.
SECTION A-A
R.C. DECK GIRDER
CONCRETE REMOVAL

FIGURE 2.5.5-1

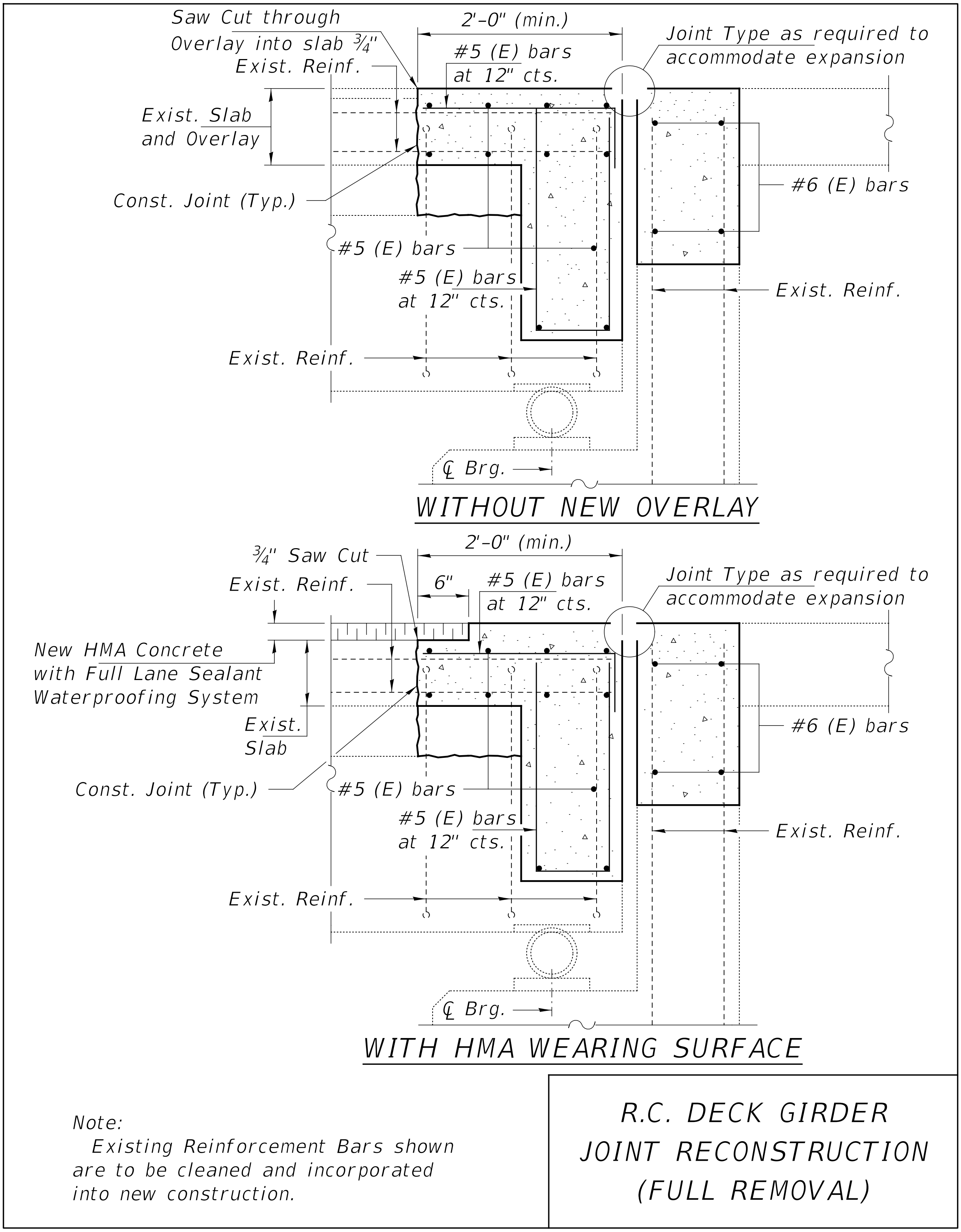
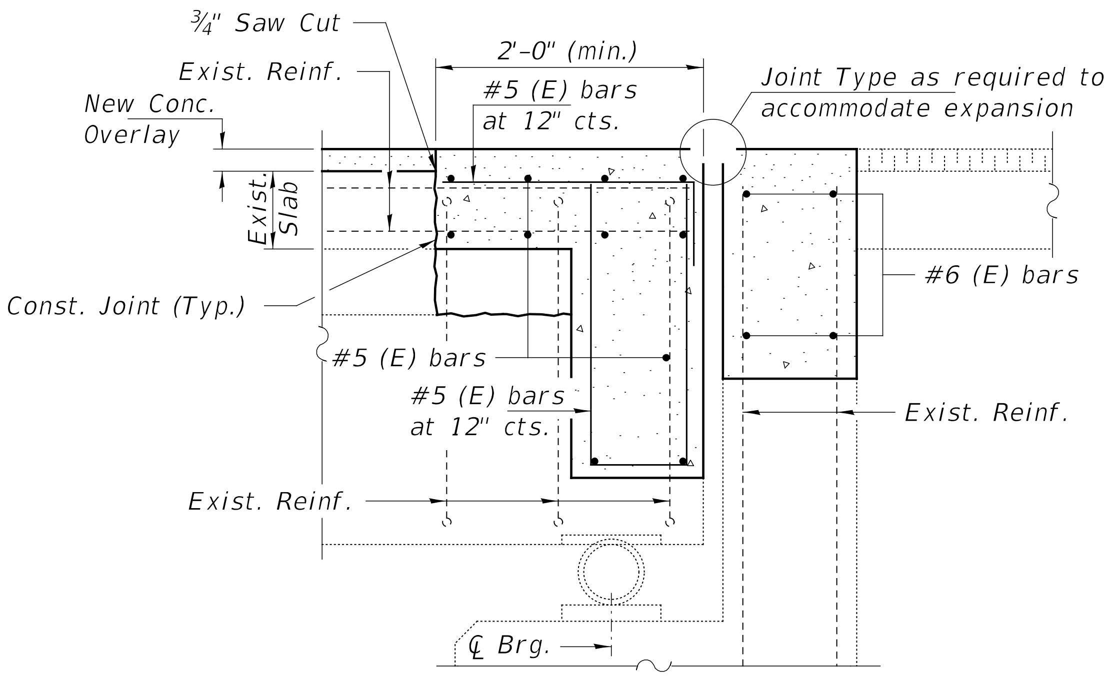

Saw Cut through
Overlay into slab ¾"
Exist. Reinf.
2'–0" (min.)
#5 (E) bars
at 12" cts.
Joint Type as required to
accommodate expansion
Exist. Slab
and Overlay
Const. Joint (Typ.)
#6 (E) bars
#5 (E) bars
#5 (E) bars
at 12" cts.
Exist. Reinf.
Exist. Reinf.
℄ Brg.
WITHOUT NEW OVERLAY
¾" Saw Cut
2'–0" (min.)
Exist. Reinf.
6"
#5 (E) bars
at 12" cts.
Joint Type as required to
accommodate expansion
New HMA Concrete
with Full Lane Sealant
Waterproofing System
Exist.
Slab
#6 (E) bars
Const. Joint (Typ.)
#5 (E) bars
#5 (E) bars
at 12" cts.
Exist. Reinf.
Exist. Reinf.
℄ Brg.
WITH HMA WEARING SURFACE
Note:
Existing Reinforcement Bars shown
are to be cleaned and incorporated
into new construction.
R.C. DECK GIRDER
JOINT RECONSTRUCTION
(FULL REMOVAL)

FIGURE 2.5.5-2



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

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.

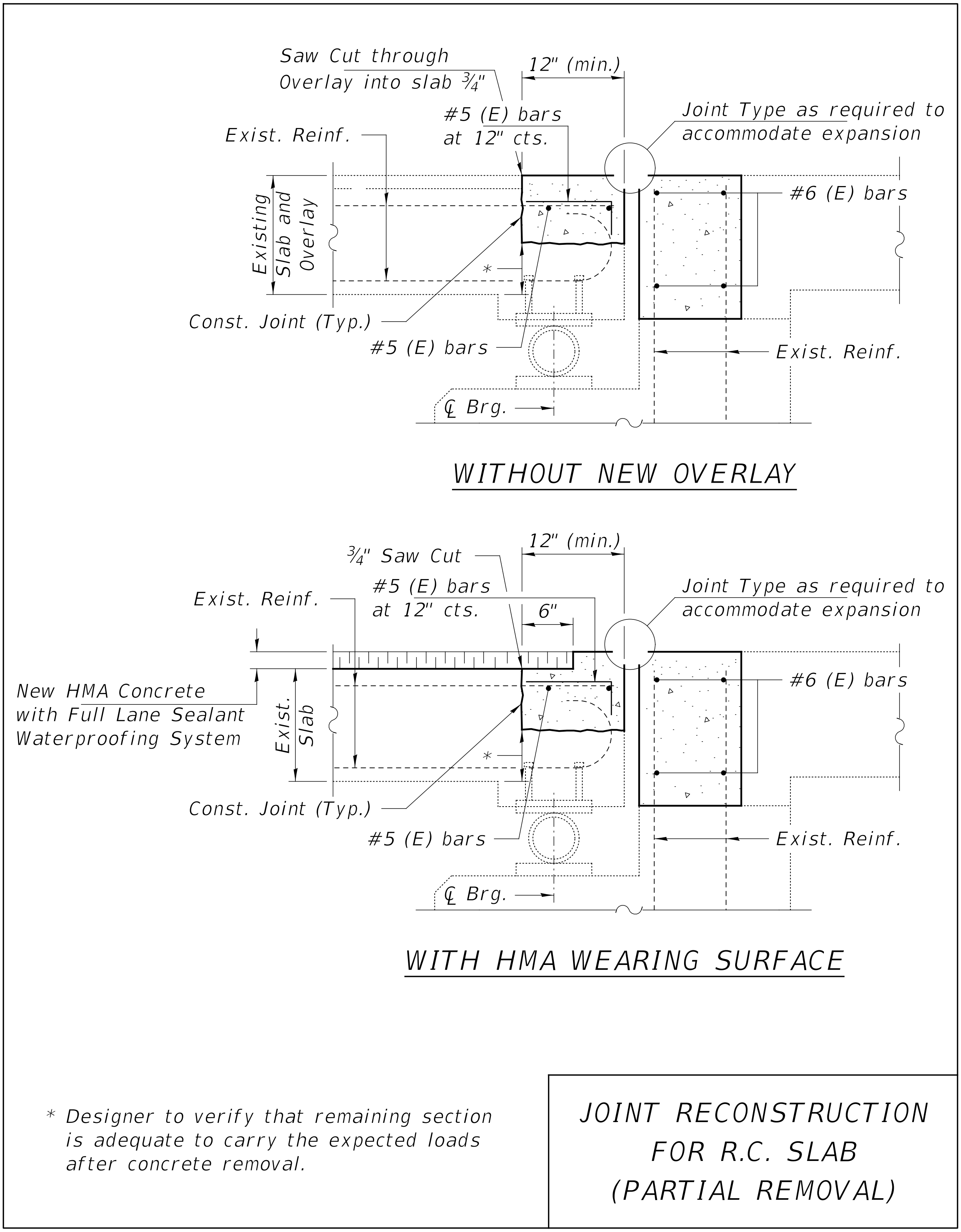

Saw Cut through
Overlay into slab ¾"
12" (min.)
#5 (E) bars
at 12" cts.
Exist. Reinf.
Joint Type as required to
accommodate expansion
Existing
Slab and
Overlay
#6 (E) bars
*
Const. Joint (Typ.)
#5 (E) bars
Exist. Reinf.
℄ Brg.
WITHOUT NEW OVERLAY
12" (min.)
¾" Saw Cut
#5 (E) bars
at 12" cts.
Exist. Reinf.
6"
Joint Type as required to
accommodate expansion
New HMA Concrete
with Full Lane Sealant
Waterproofing System
Exist.
Slab
#6 (E) bars
*
Const. Joint (Typ.)
#5 (E) bars
Exist. Reinf.
℄ Brg.
WITH HMA WEARING SURFACE
* Designer to verify that remaining section
is adequate to carry the expected loads
after concrete removal.
JOINT RECONSTRUCTION
FOR R.C. SLAB
(PARTIAL REMOVAL)

FIGURE 2.5.6-1

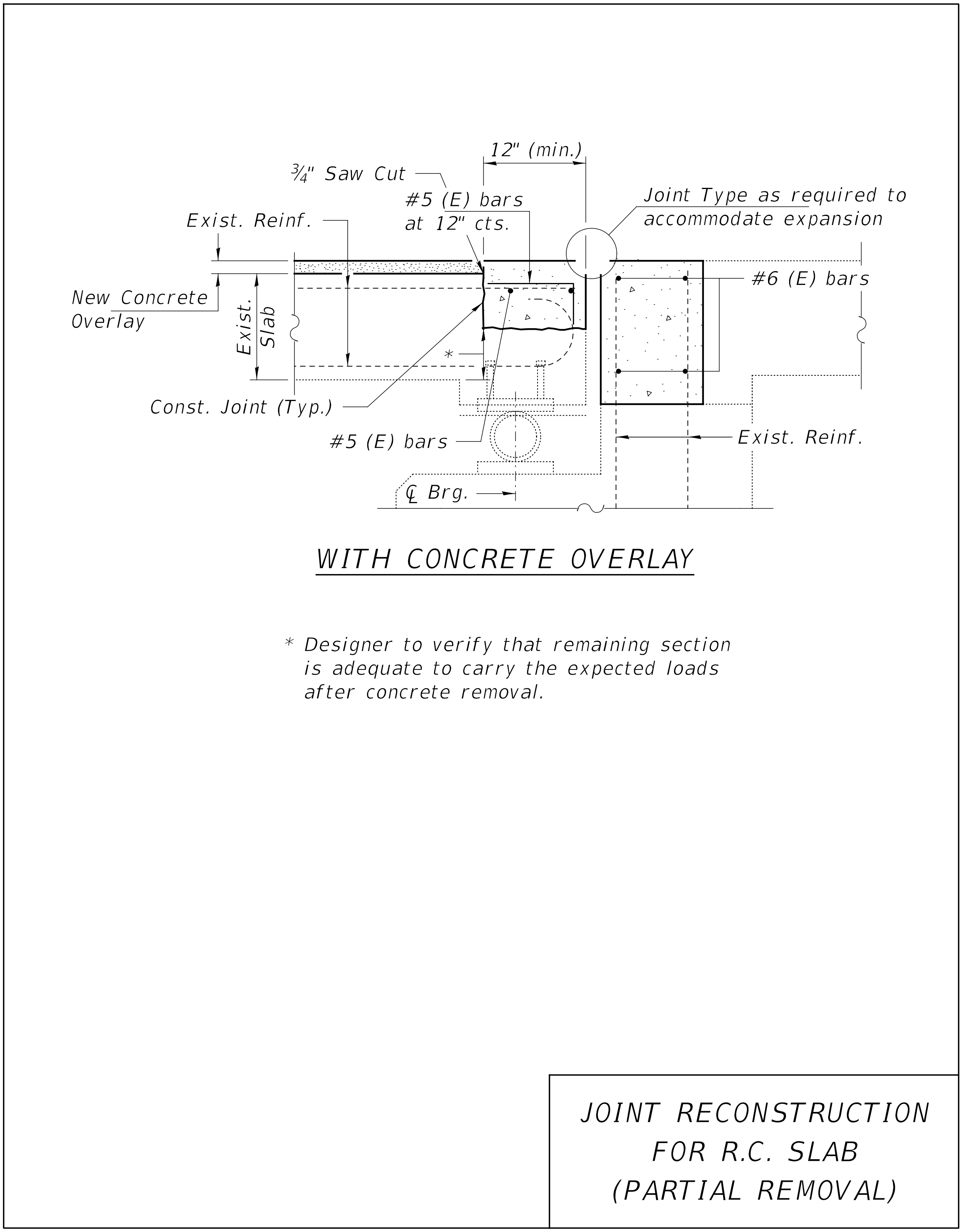

12" (min.)
¾" Saw Cut
#5 (E) bars
at 12" cts.
Joint Type as required to
accommodate expansion
Exist. Reinf.
#6 (E) bars
New Concrete
Overlay
Exist.
Slab
*
Const. Joint (Typ.)
#5 (E) bars
Exist. Reinf.
℄ Brg.
WITH CONCRETE OVERLAY
* Designer to verify that remaining section
is adequate to carry the expected loads
after concrete removal.
JOINT RECONSTRUCTION
FOR R.C. SLAB
(PARTIAL REMOVAL)

FIGURE 2.5.6-2

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 Figs. 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 Figs. 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.

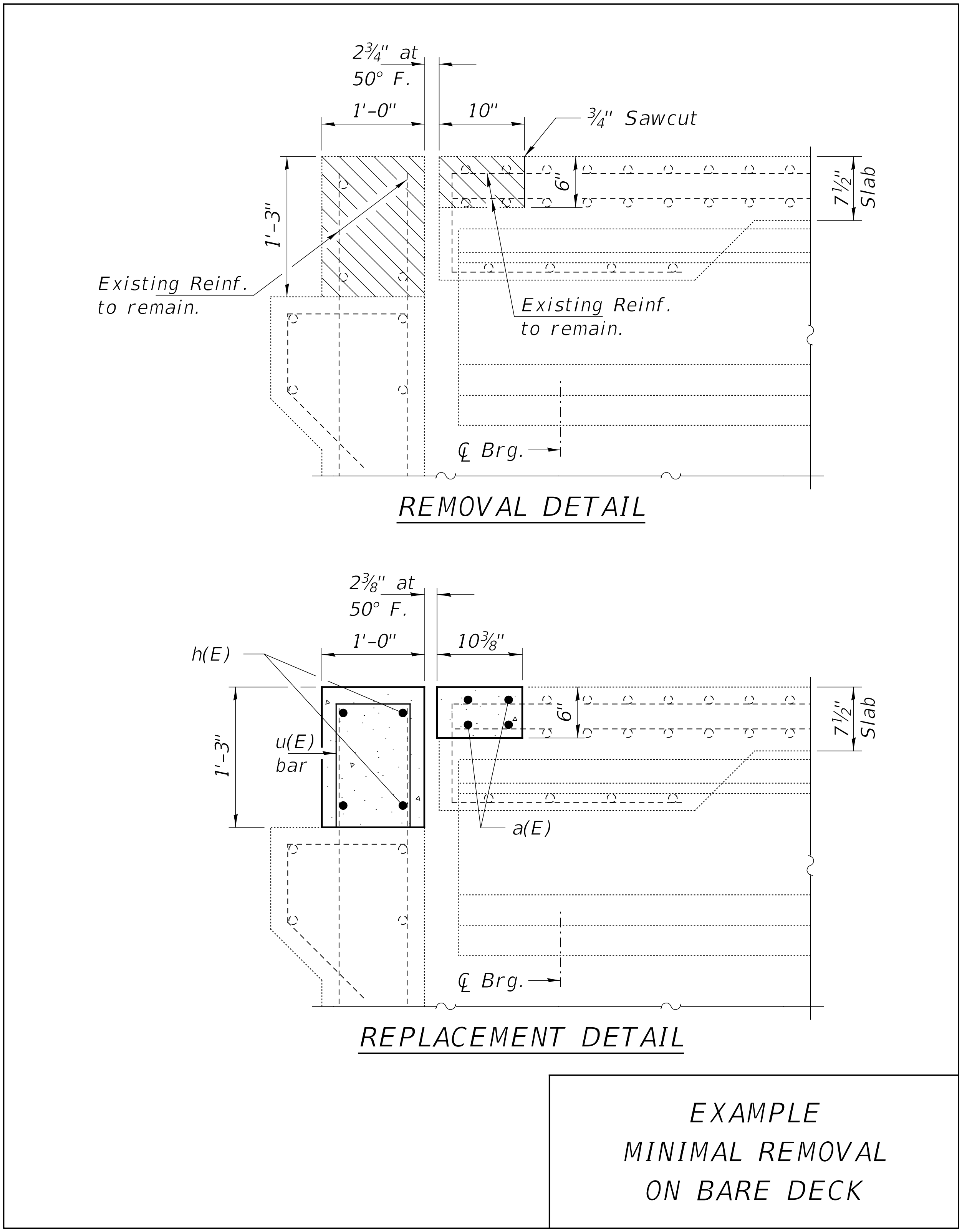
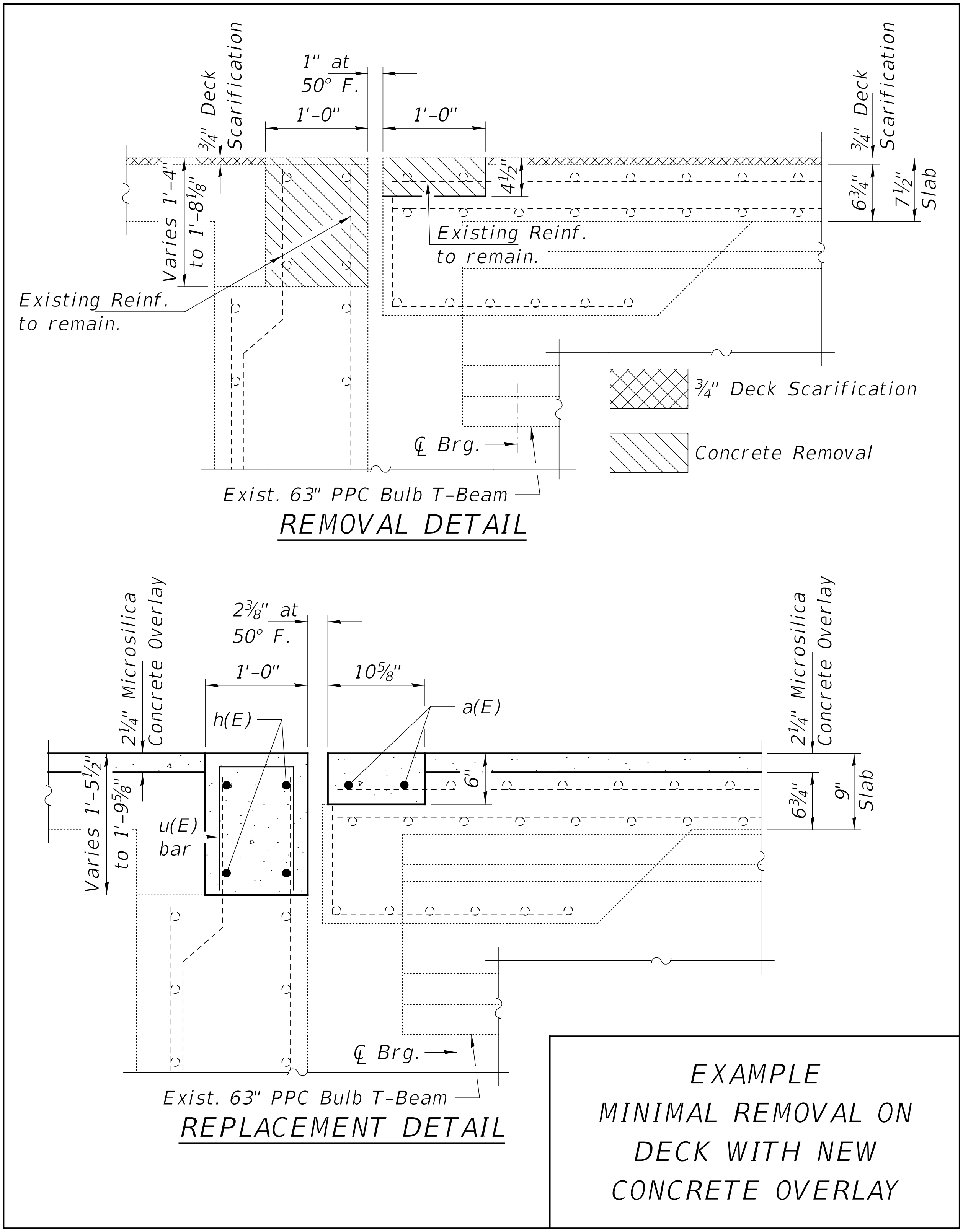

2¾" at
50° F.
1'-0"
10"
¾" Sawcut
6"
7½"
Slab
1'-3"
Existing Reinf.
to remain.
Existing Reinf.
to remain.
℄ Brg.
REMOVAL DETAIL
2⅜" at
50° F.
1'-0"
10⅜"
h(E)
6"
7½"
Slab
1'-3"
u(E)
bar
a(E)
℄ Brg.
REPLACEMENT DETAIL
EXAMPLE
MINIMAL REMOVAL
ON BARE DECK

FIGURE 2.5.7-1



1" at 50° F.
1'-0"
1'-0"
3/4" Deck Scarification
Varies 1'-4" to 1'-8 1/8"
4 1/2"
6 3/4"
7 1/2" Slab
Existing Reinf. to remain.
Existing Reinf. to remain.
3/4" Deck Scarification
Concrete Removal
℄ Brg.
Exist. 63" PPC Bulb T-Beam
REMOVAL DETAIL
2 3/8" at 50° F.
1'-0"
10 5/8"
2 1/4" Microsilica Concrete Overlay
h(E)
a(E)
u(E) bar
Varies 1'-5 1/2" to 1'-9 5/8"
6"
6 3/4"
9" Slab
℄ Brg.
Exist. 63" PPC Bulb T-Beam
REPLACEMENT DETAIL
EXAMPLE
MINIMAL REMOVAL ON DECK WITH NEW CONCRETE OVERLAY

FIGURE 2.5.7-2

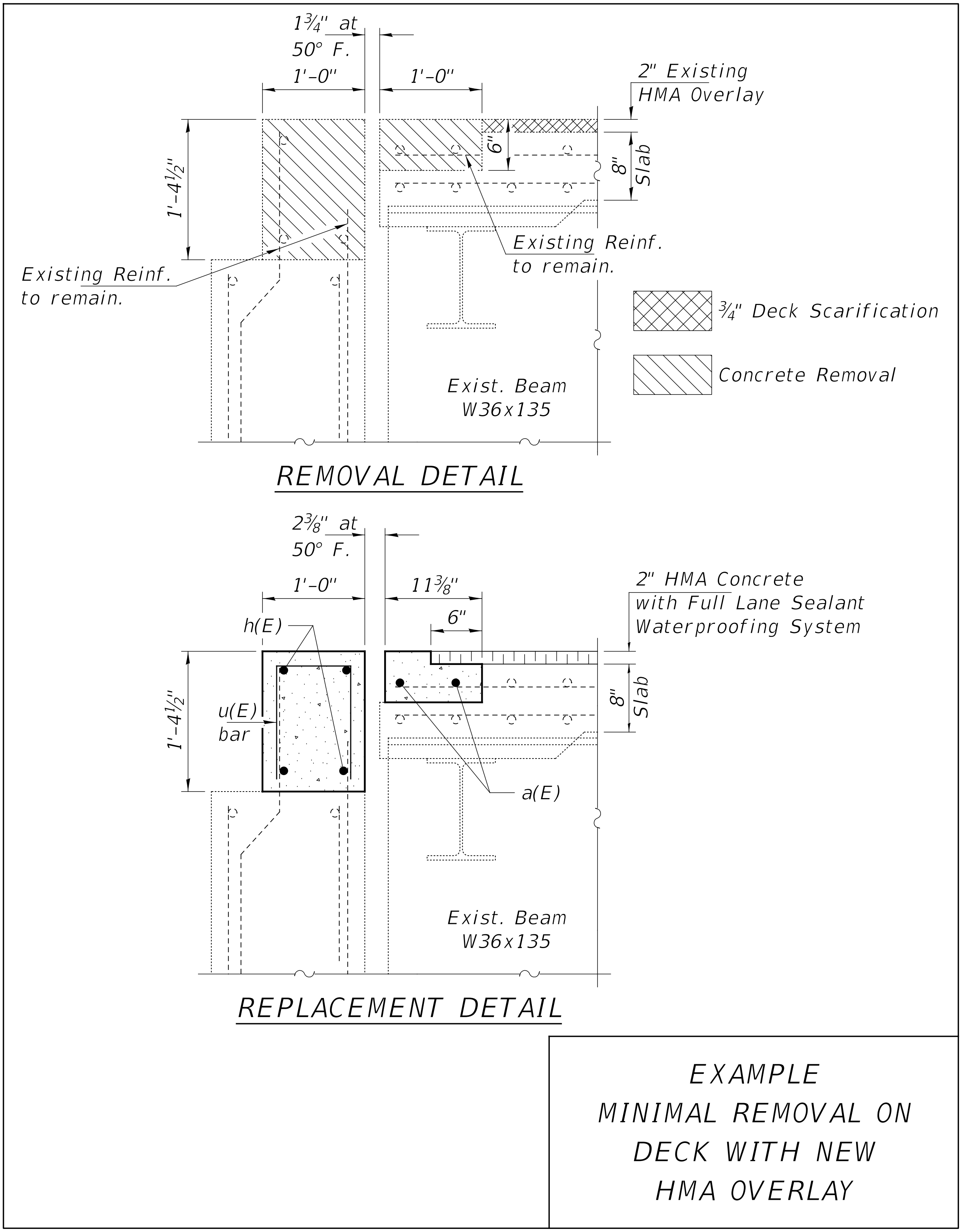

1¾" at
50° F.
1'-0"
1'-0"
2" Existing
HMA Overlay
6"
8"
Slab
1'-4½"
Existing Reinf.
to remain.
Existing Reinf.
to remain.
¾" Deck Scarification
Concrete Removal
Exist. Beam
W36x135
REMOVAL DETAIL
2⅜" at
50° F.
1'-0"
11⅜"
6"
2" HMA Concrete
with Full Lane Sealant
Waterproofing System
h(E)
u(E)
bar
1'-4½"
8"
Slab
a(E)
Exist. Beam
W36x135
REPLACEMENT DETAIL
EXAMPLE
MINIMAL REMOVAL ON
DECK WITH NEW
HMA OVERLAY

FIGURE 2.5.7-3

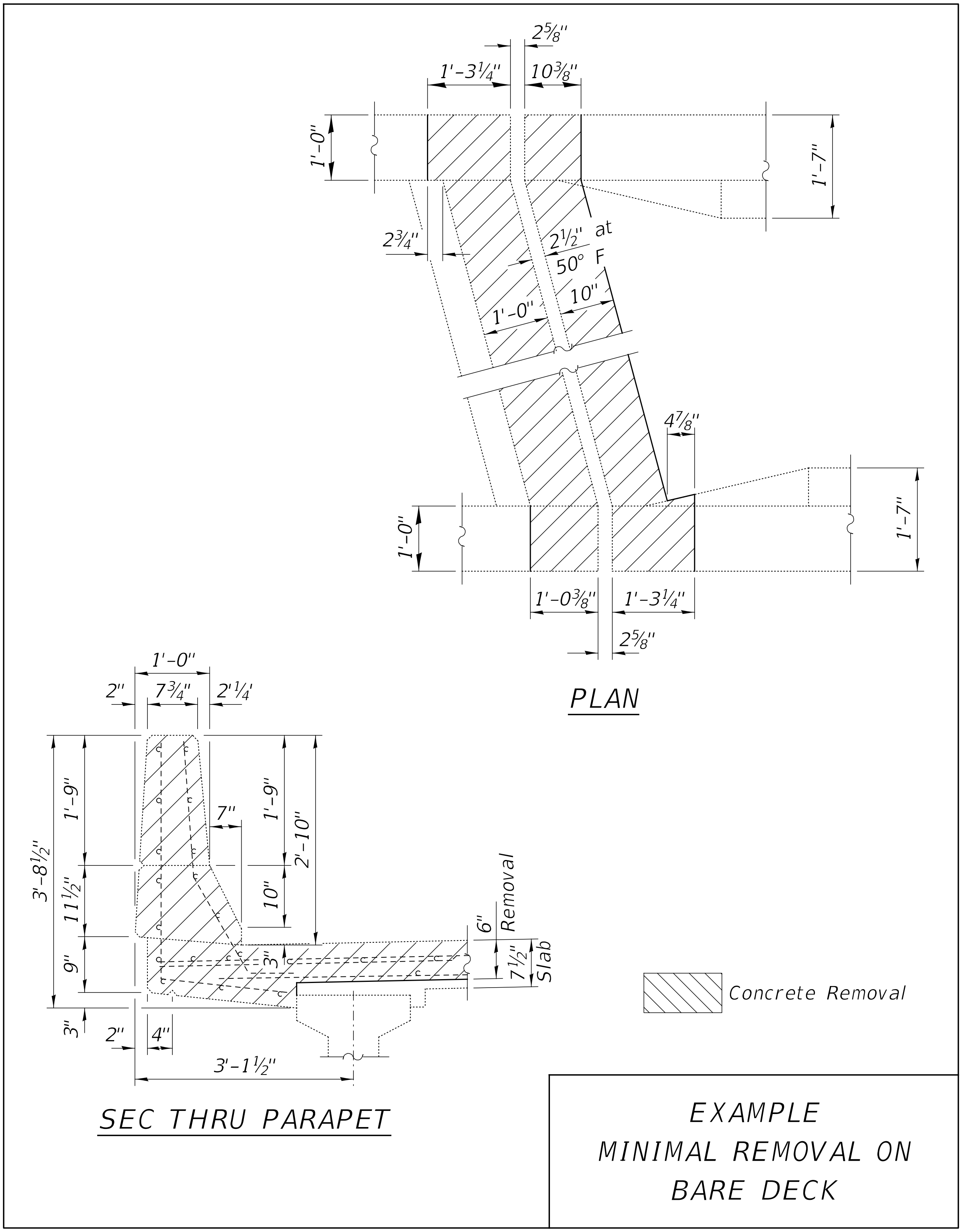

2⅝"
1'-3¼"
10⅜"
1'-0"
1'-7"
2¾"
2½" at
50° F
10"
1'-0"
4⅞"
1'-0"
1'-7"
1'-0⅜"
1'-3¼"
2⅝"
PLAN
1'-0"
2"
7¾"
2¼'
1'-9"
7"
1'-9"
2'-10"
3'-8½"
11½"
10"
6"
Removal
3"
9"
7½"
Slab
3"
2"
4"
3'-1½"
Concrete Removal
SEC THRU PARAPET
EXAMPLE
MINIMAL REMOVAL ON
BARE DECK

FIGURE 2.5.7-4

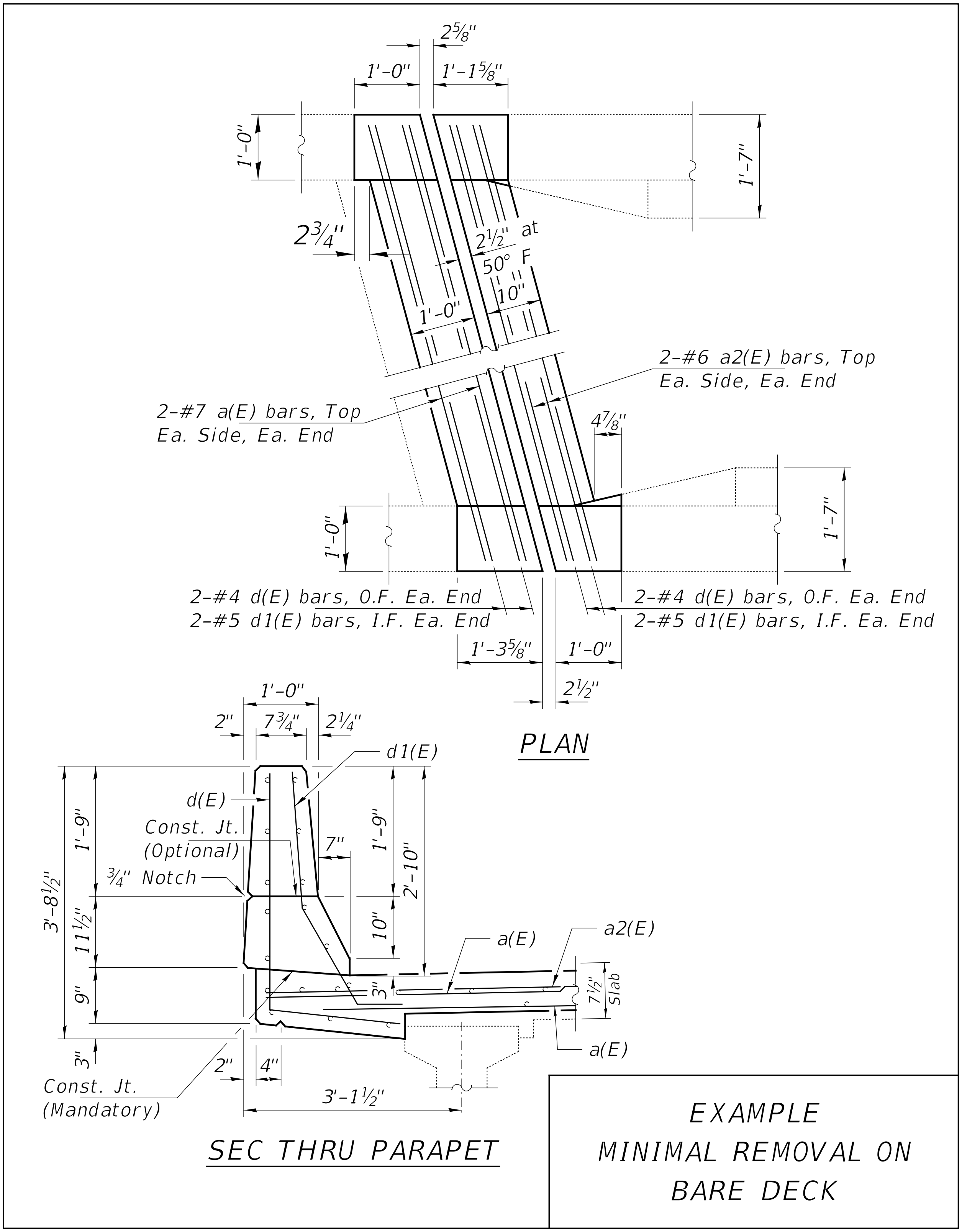

2⅝"
1'-0"
1'-1⅝"
1'-0"
1'-7"
2¾"
2½" at 50° F
10"
1'-0"
2-#6 a2(E) bars, Top Ea. Side, Ea. End
2-#7 a(E) bars, Top Ea. Side, Ea. End
4⅞"
1'-0"
1'-7"
2-#4 d(E) bars, O.F. Ea. End
2-#5 d1(E) bars, I.F. Ea. End
2-#4 d(E) bars, O.F. Ea. End
2-#5 d1(E) bars, I.F. Ea. End
1'-3⅝"
1'-0"
2½"
PLAN
1'-0"
2"
7¾"
2¼"
d1(E)
d(E)
Const. Jt. (Optional)
7"
¾" Notch
3'-8½"
1'-9"
11½"
9"
3"
1'-9"
2'-10"
10"
3"
a(E)
a2(E)
7½" Slab
a(E)
2"
4"
3'-1½"
Const. Jt. (Mandatory)
SEC THRU PARAPET
EXAMPLE
MINIMAL REMOVAL ON
BARE DECK

FIGURE 2.5.7-5

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:
    - $F_y = 30$ 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 $F_y = 40$ 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 $Q_u \leq 1.0$ 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 2.5.8-1 through 2.5.8-3 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 2.5.8-1 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 2.5.8-1 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 2.5.8-2 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 2.5.8-2 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 2.5.8-3 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 2.5.8-1 through 2.5.8-3 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.

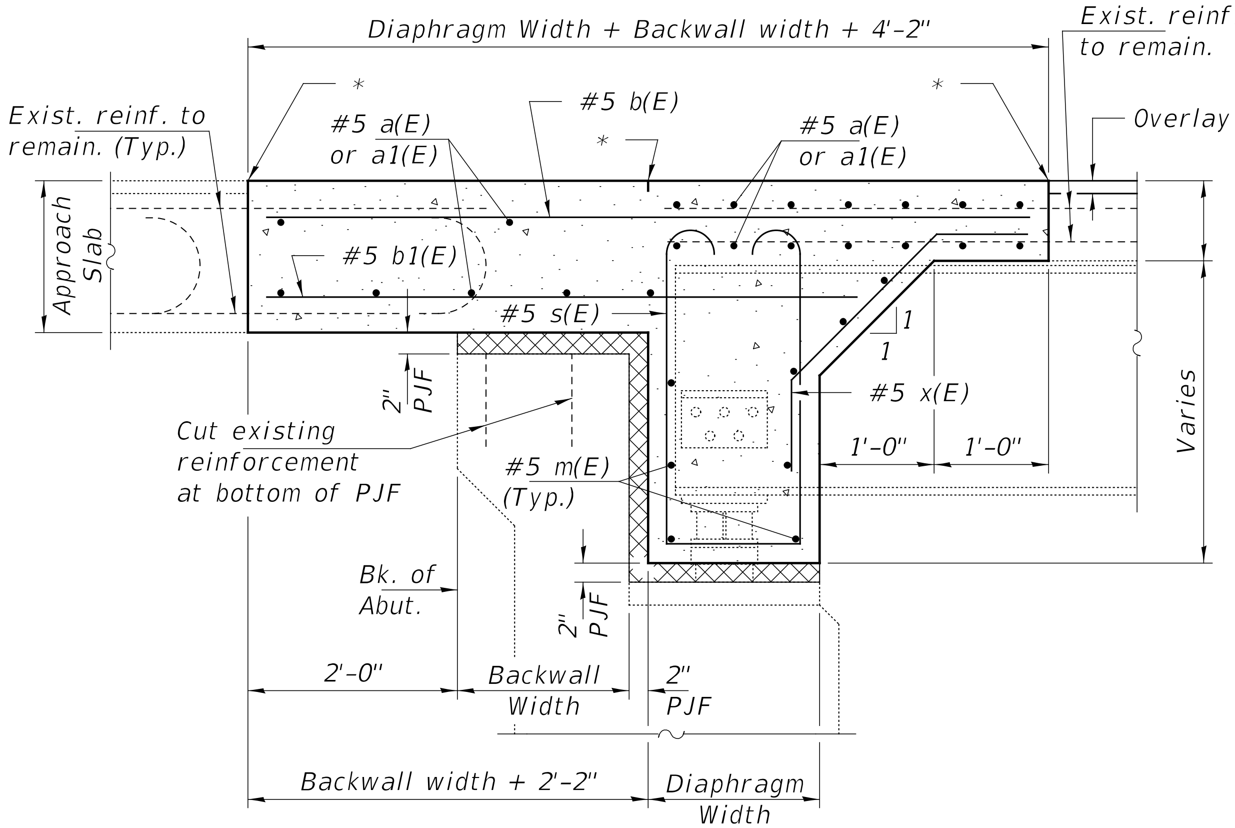
		Maximum Expansion Length (ft)				
Pile Type	Pile Size	0° skew	5° skew	10° skew	15° skew	30° ≥ skew < 45°
Steel	HP 8x36	170	150	140	130	110
	HP 10x42	180	160	150	140	120
	HP 10x57	200	190	180	170	150
	HP 12x53	220	200	190	170	160
	HP 12x63	230	210	200	190	170
	HP 12x74	250	230	220	210	190
	HP 12x84	260	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.



SECTION THRU ABUTMENT

* ¼" x ¾" Formed Joint with bridge relief joint sealer.

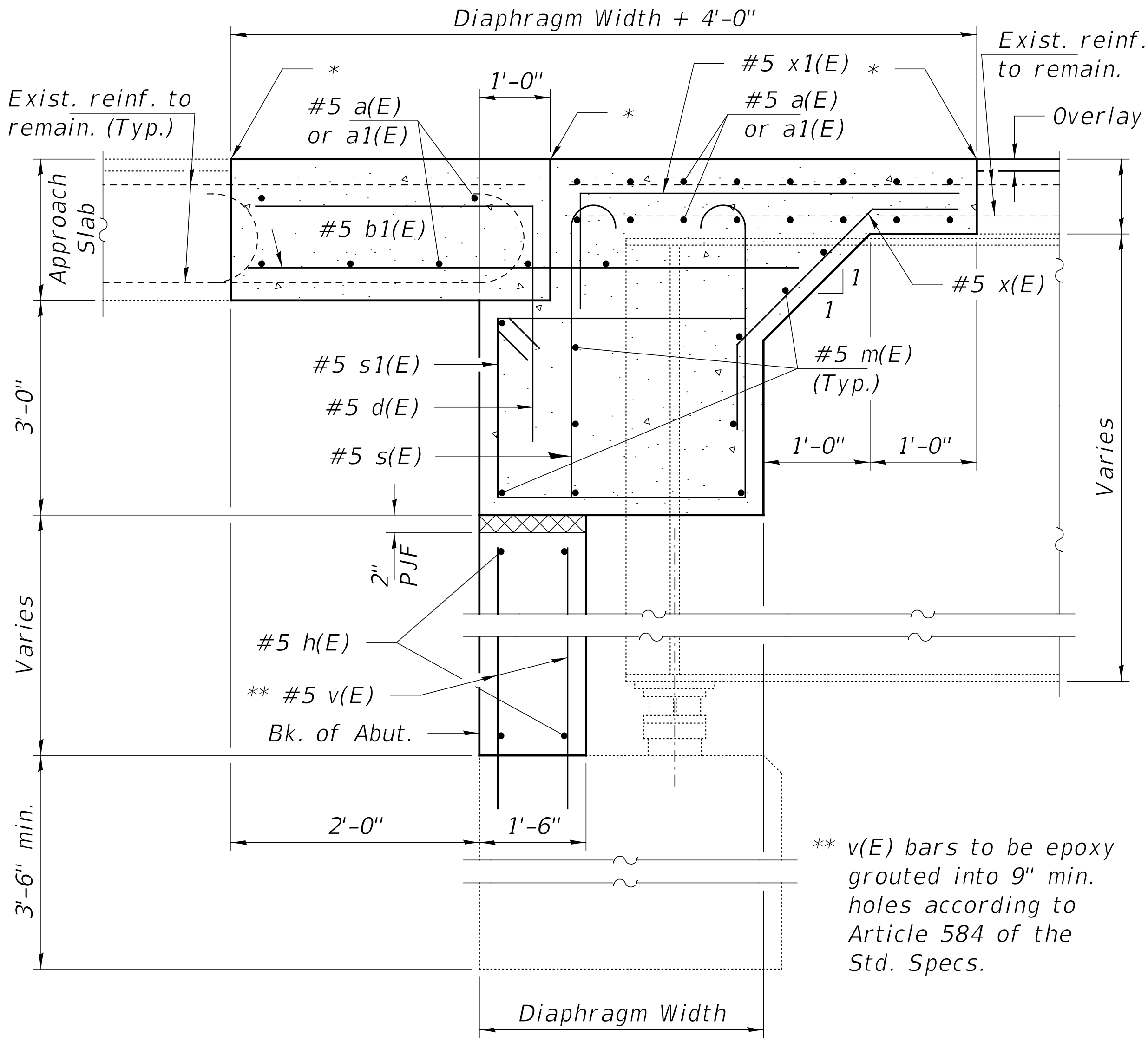
Note:

Existing end diaphragms or cross frames to be removed to facilitate placement of new reinforcement and concrete.

The m(E) bars are to be placed through holes drilled in the beam web where appropriate. (For steel beams only.)

JOINT ELIMINATION FOR STEEL OR CONCRETE BEAM PLUS BEARING DEPTH ≤ 48 INCHES

FIGURE 2.5.8-1



SECTION THRU ABUTMENT

* ¼" x ¾" Formed Joint with bridge relief joint sealer.

Note:
Existing end diaphragms or cross frames to be removed to facilitate placement of new reinforcement and concrete.
The m(E) bars are to be placed through holes drilled in the beam web where appropriate. (For steel beams only.)

JOINT ELIMINATION FOR STEEL OR CONCRETE BEAM PLUS BEARING DEPTH > 48 INCHES

FIGURE 2.5.8-2

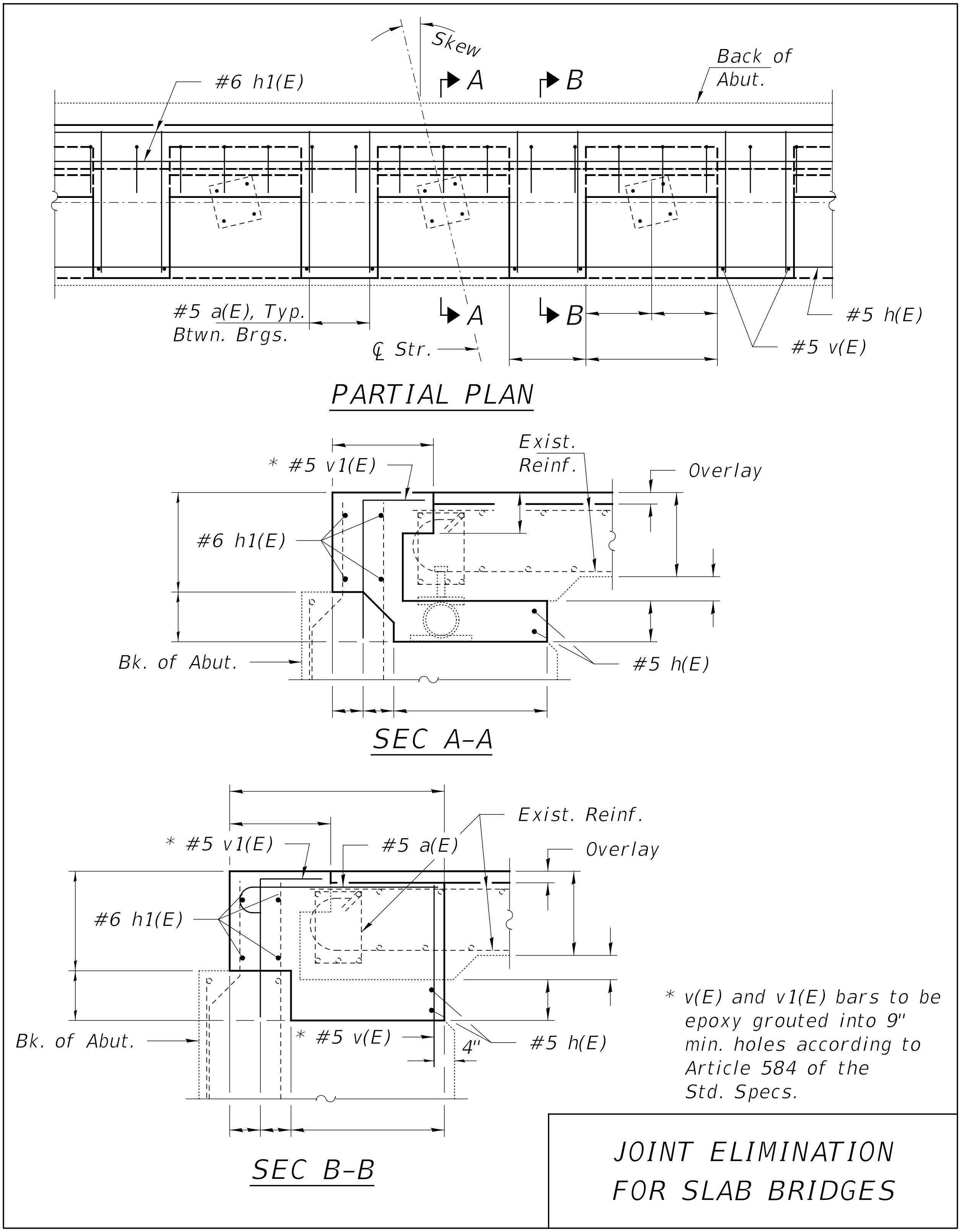

Skew
Back of Abut.
#6 h1(E)
A
B
#5 a(E), Typ. Btwn. Brgs.
℄ Str.
#5 h(E)
#5 v(E)
PARTIAL PLAN
Exist. Reinf.
* #5 v1(E)
Overlay
#6 h1(E)
Bk. of Abut.
#5 h(E)
SEC A–A
Exist. Reinf.
* #5 v1(E)
#5 a(E)
Overlay
#6 h1(E)
Bk. of Abut.
* #5 v(E)
4"
#5 h(E)
* v(E) and v1(E) bars to be epoxy grouted into 9" min. holes according to Article 584 of the Std. Specs.
SEC B–B
JOINT ELIMINATION FOR SLAB BRIDGES

FIGURE 2.5.8-3

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.

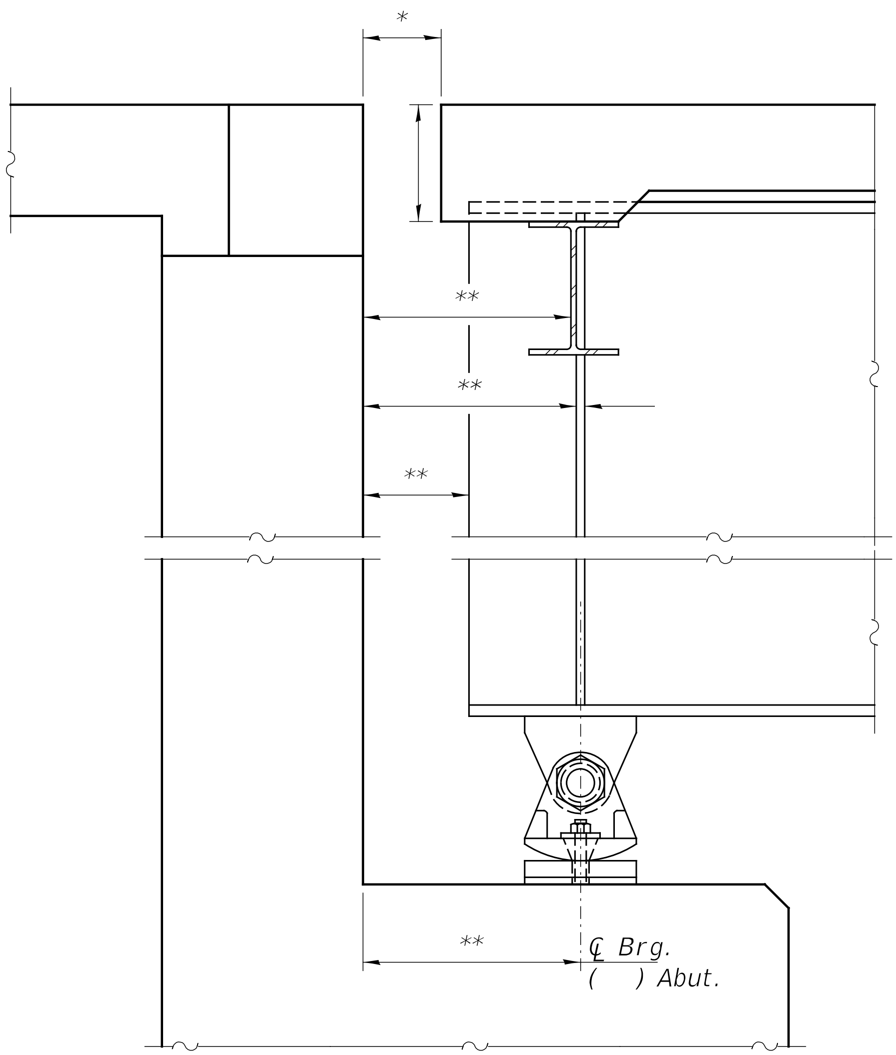
When an existing finger plate is to remain, and a new overlay will raise the profile grade, the joint must be adjusted as shown in Fig. 2.5.9-3.

Structure No. ________________

Beam No. ____________________

Date __________________________

Temperature _________________



Note:

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

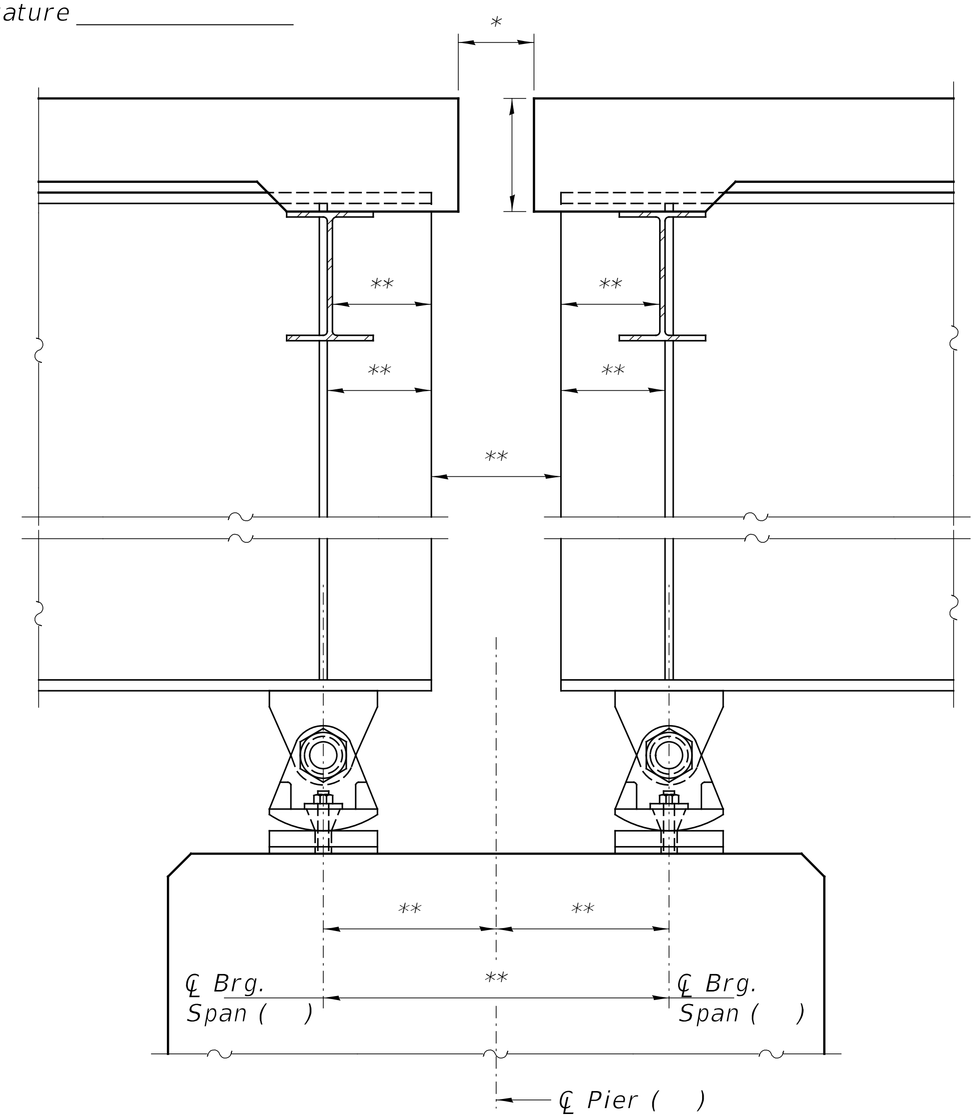
* Measurement is assumed to be taken perpendicular to centerline of joint.

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

FINGER JOINT
FIELD DIMENSIONS
(AT ABUTMENT)

FIGURE 2.5.9-1

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



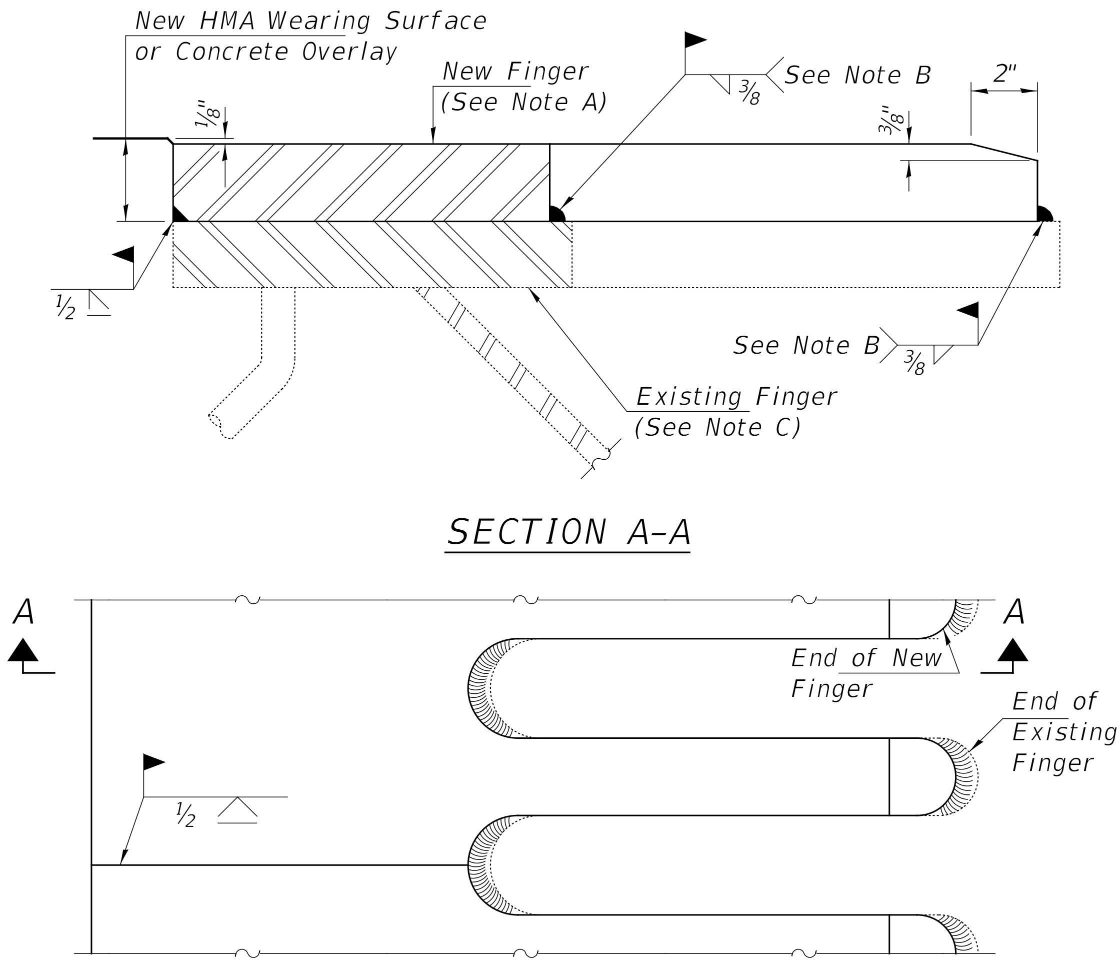
Note:
Assumed direction of measurements are noted below. The actual direction each measurement is taken shall be documented.

* 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



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

2.5.10 Modular Expansion Joints

At the district’s request modular joints can be used in lieu of finger plate joints. Designers should first consult the new structure details in the active ABD memorandum, the current Bridge Manual, and GBSP 18 Modular Expansion Joint for information on the design of these joints.

Prequalified 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 and whether they should be lowered; minimum edge beam depth and reinforcement; and desired cover for the support box reinforcement.

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, and existing structure diaphragms and/or elements may need to be reconfigured if desired. 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 me mechanically spliced to them (see Fig. 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. Designers can find manufacturer information for prequalified Modular Expansion Joint systems at the Department’s “Prequalified Structural Systems” webpage.

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 Fig. 2.5.10-2).

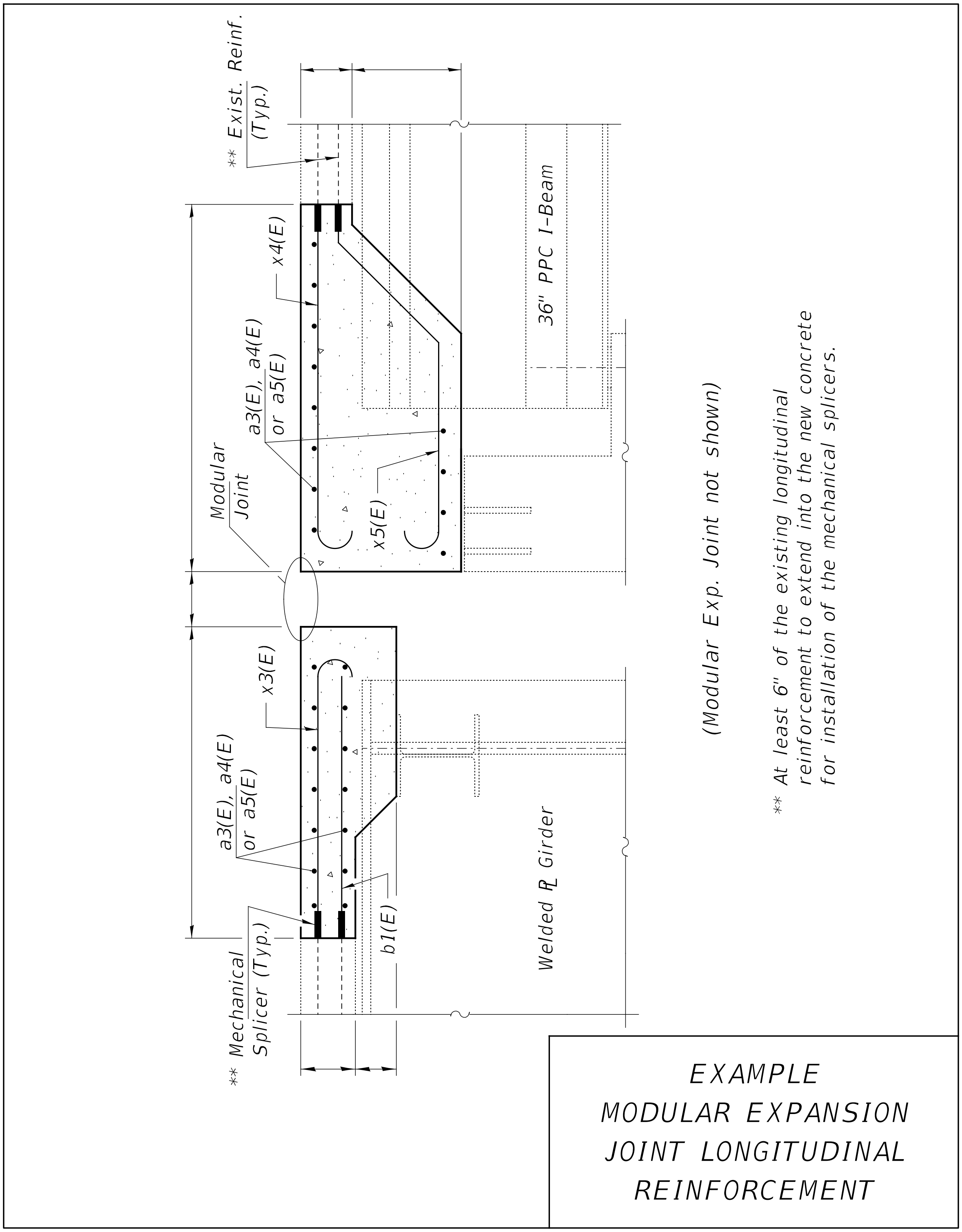


Figure 2.5.10-1

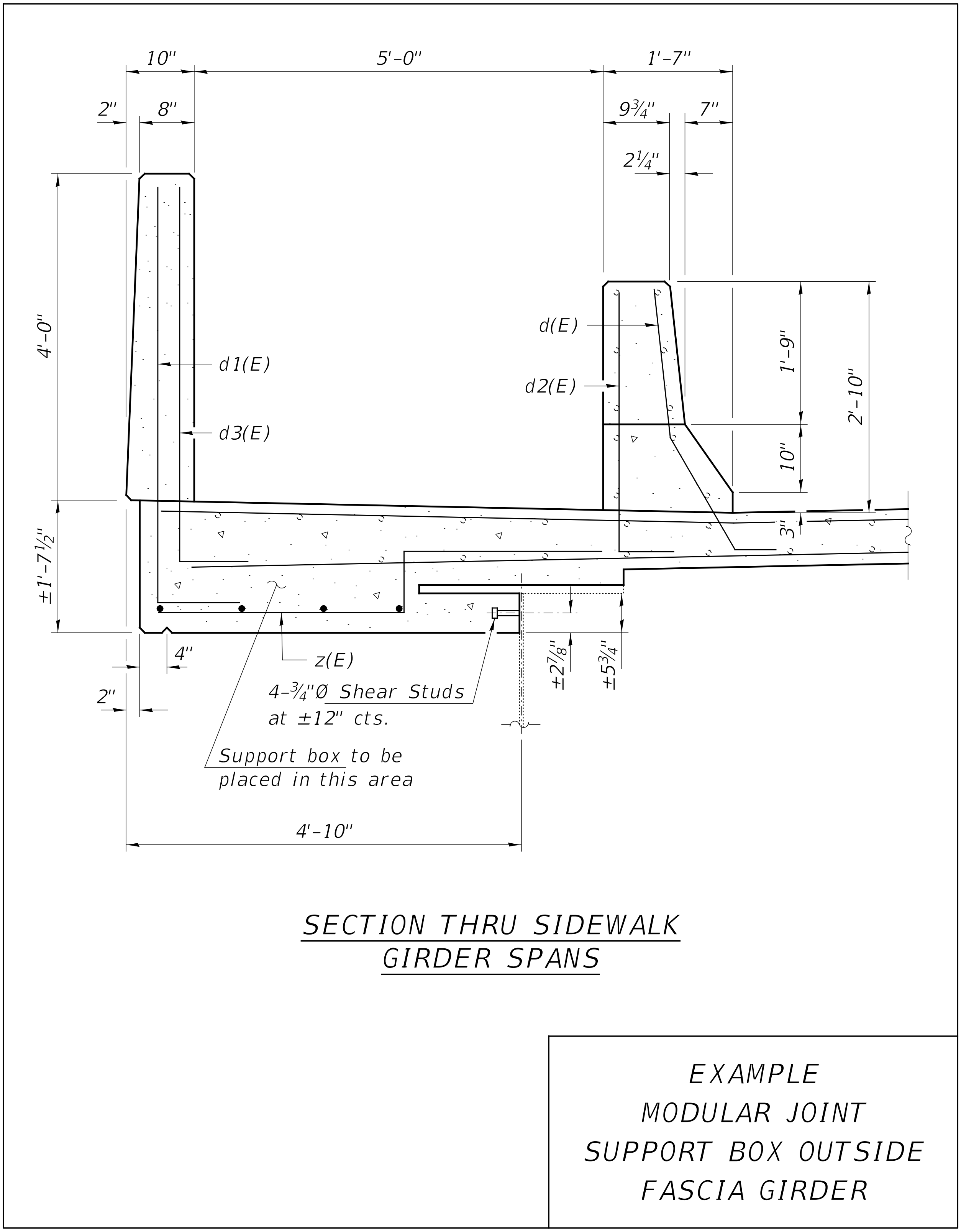


Figure 2.5.10-2

2.6 Longitudinal Joint Closure

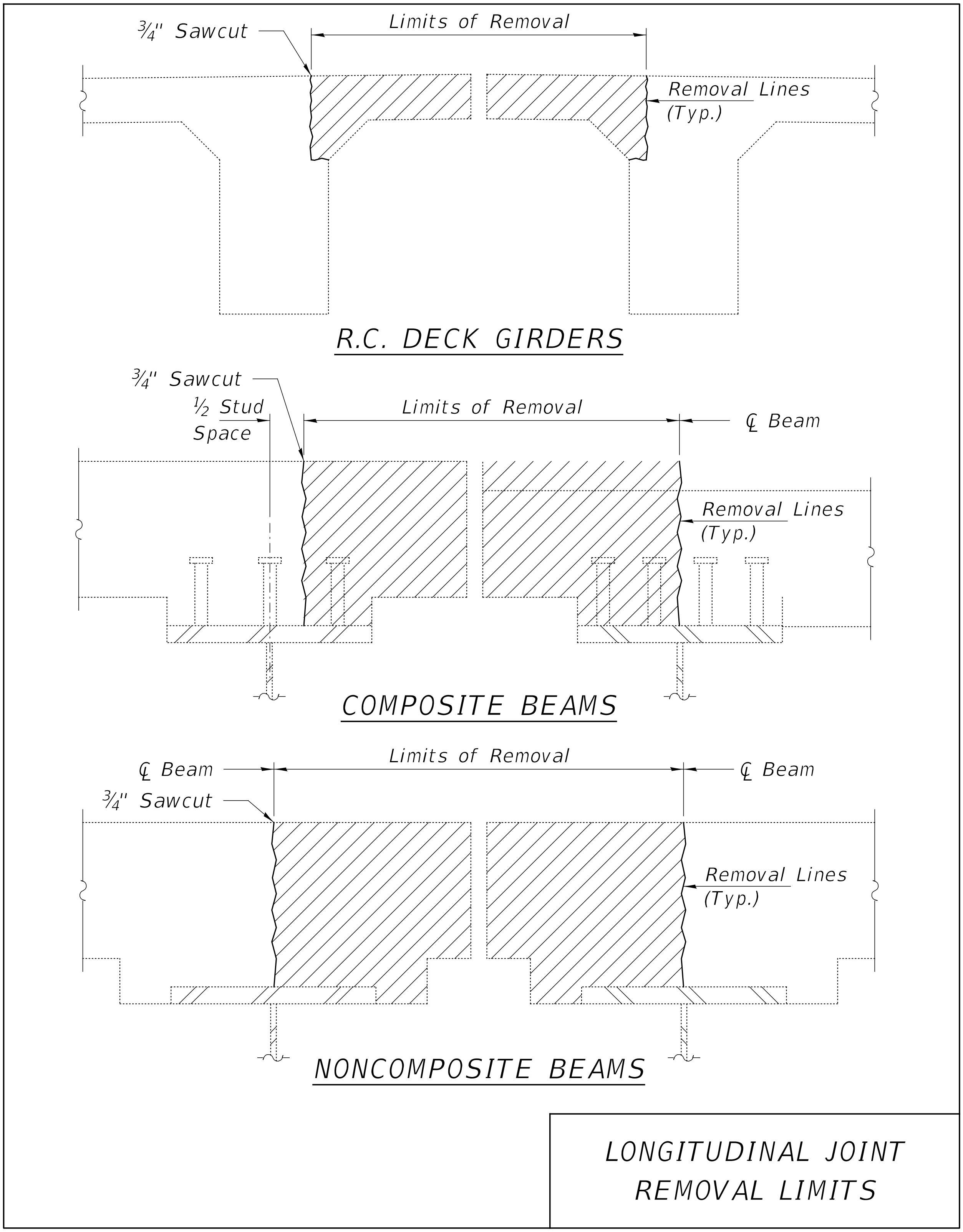
2.6.1 General

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.



¾" Sawcut
Limits of Removal
Removal Lines (Typ.)
R.C. DECK GIRDERS
¾" Sawcut
½ Stud Space
Limits of Removal
℄ Beam
Removal Lines (Typ.)
COMPOSITE BEAMS
℄ Beam
Limits of Removal
℄ Beam
¾" Sawcut
Removal Lines (Typ.)
NONCOMPOSITE BEAMS
LONGITUDINAL JOINT REMOVAL LIMITS

FIGURE 2.6.2-1

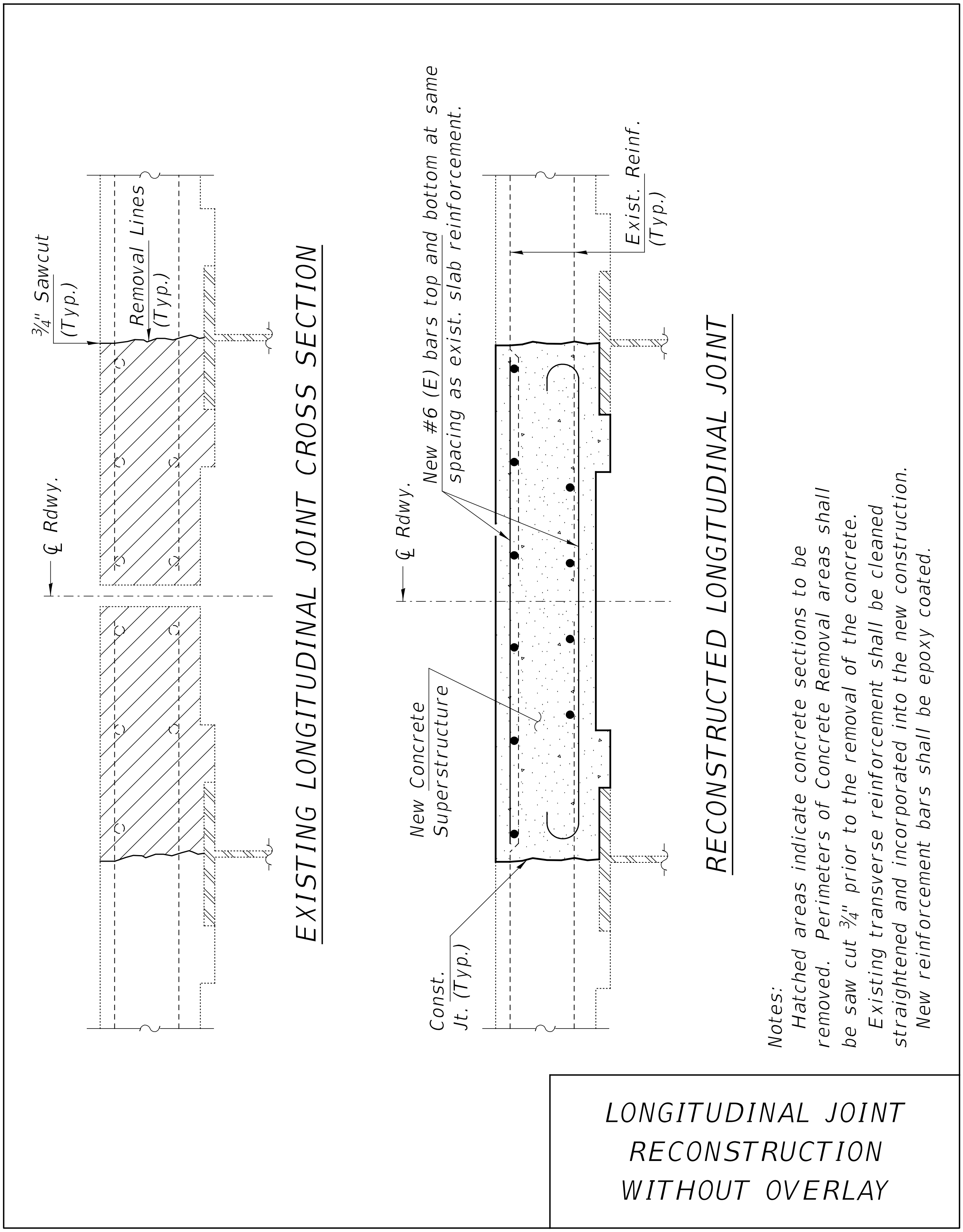
2.6.3 Reinforcement Bar Treatment

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 2.6.3-4shows 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.



3/4" Sawcut
(Typ.)
Removal Lines
(Typ.)
℄ Rdwy.
EXISTING LONGITUDINAL JOINT CROSS SECTION
New #6 (E) bars top and bottom at same spacing as exist. slab reinforcement.
Exist. Reinf.
(Typ.)
℄ Rdwy.
New Concrete Superstructure
Const. Jt. (Typ.)
RECONSTRUCTED LONGITUDINAL JOINT
Notes:
Hatched areas indicate concrete sections to be removed. Perimeters of Concrete Removal areas shall be saw cut 3/4" prior to the removal of the concrete.
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction.
New reinforcement bars shall be epoxy coated.
LONGITUDINAL JOINT RECONSTRUCTION WITHOUT OVERLAY

FIGURE 2.6.3-1

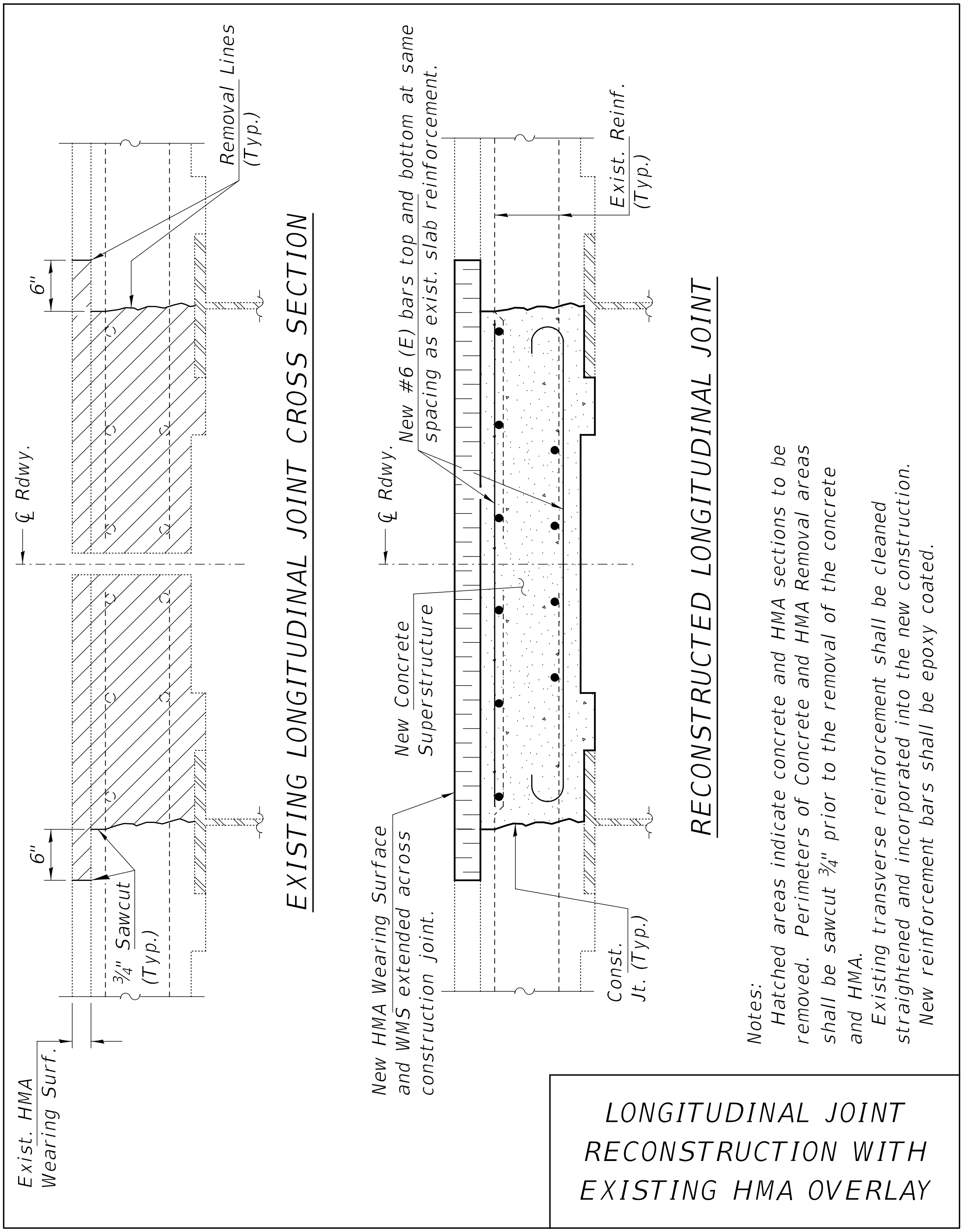

Removal Lines
(Typ.)
6"
6"
℄ Rdwy.
¾" Sawcut
(Typ.)
Exist. HMA
Wearing Surf.
EXISTING LONGITUDINAL JOINT CROSS SECTION
New #6 (E) bars top and bottom at same
spacing as exist. slab reinforcement.
Exist. Reinf.
(Typ.)
℄ Rdwy.
New Concrete
Superstructure
New HMA Wearing Surface
and WMS extended across
construction joint.
Const.
Jt. (Typ.)
RECONSTRUCTED LONGITUDINAL JOINT
Notes:
Hatched areas indicate concrete and HMA sections to be removed. Perimeters of Concrete and HMA Removal areas shall be sawcut ¾" prior to the removal of the concrete and HMA.
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction.
New reinforcement bars shall be epoxy coated.
LONGITUDINAL JOINT
RECONSTRUCTION WITH
EXISTING HMA OVERLAY

FIGURE 2.6.3-2

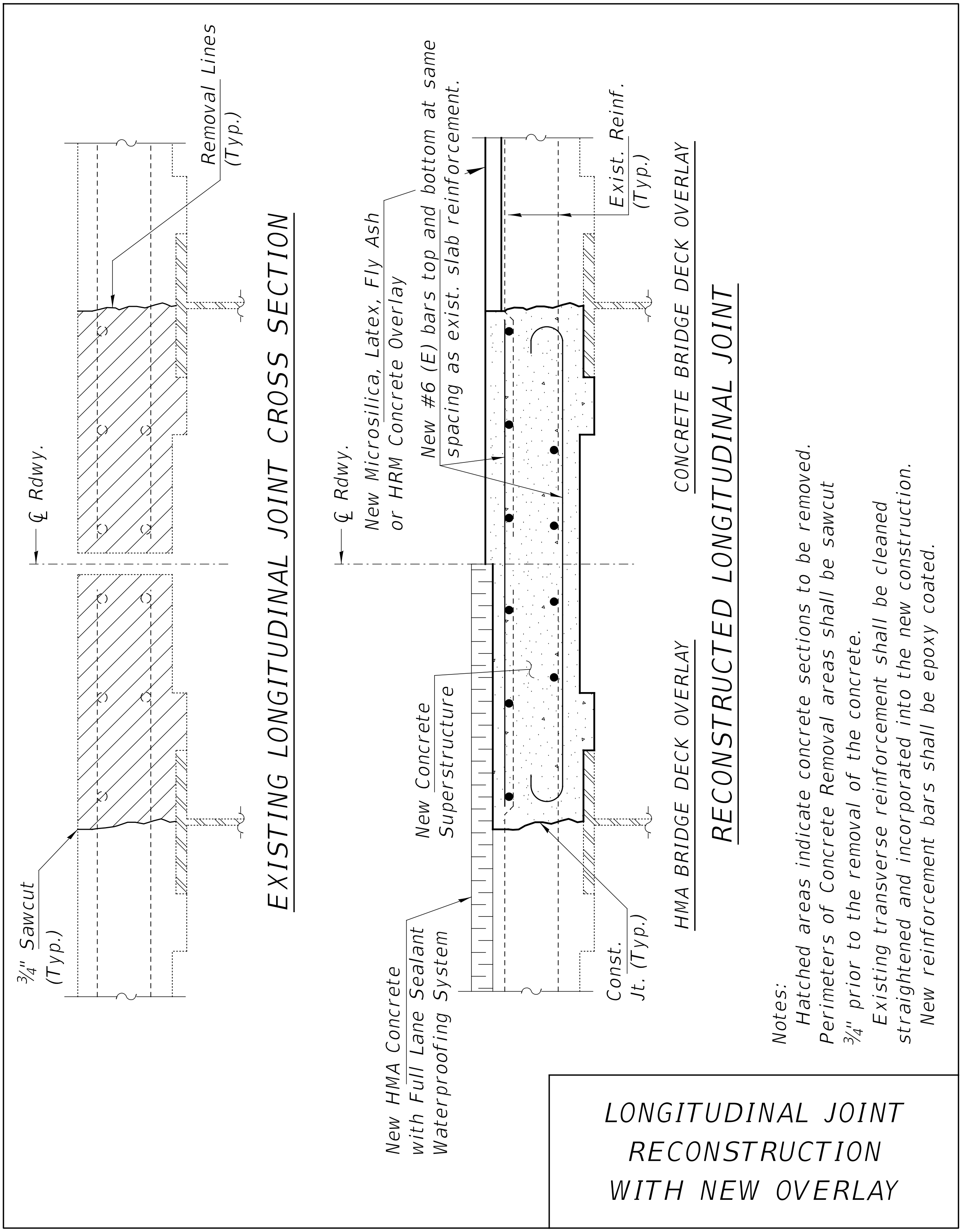

Removal Lines
(Typ.)
¾" Sawcut
(Typ.)
℄ Rdwy.
EXISTING LONGITUDINAL JOINT CROSS SECTION
New Microsilica, Latex, Fly Ash
or HRM Concrete Overlay
New #6 (E) bars top and bottom at same
spacing as exist. slab reinforcement.
Exist. Reinf.
(Typ.)
New Concrete
Superstructure
New HMA Concrete
with Full Lane Sealant
Waterproofing System
Const.
Jt. (Typ.)
HMA BRIDGE DECK OVERLAY
CONCRETE BRIDGE DECK OVERLAY
RECONSTRUCTED LONGITUDINAL JOINT
Notes:
Hatched areas indicate concrete sections to be removed.
Perimeters of Concrete Removal areas shall be sawcut
¾" prior to the removal of the concrete.
Existing transverse reinforcement shall be cleaned
straightened and incorporated into the new construction.
New reinforcement bars shall be epoxy coated.
LONGITUDINAL JOINT
RECONSTRUCTION
WITH NEW OVERLAY

FIGURE 2.6.3-3

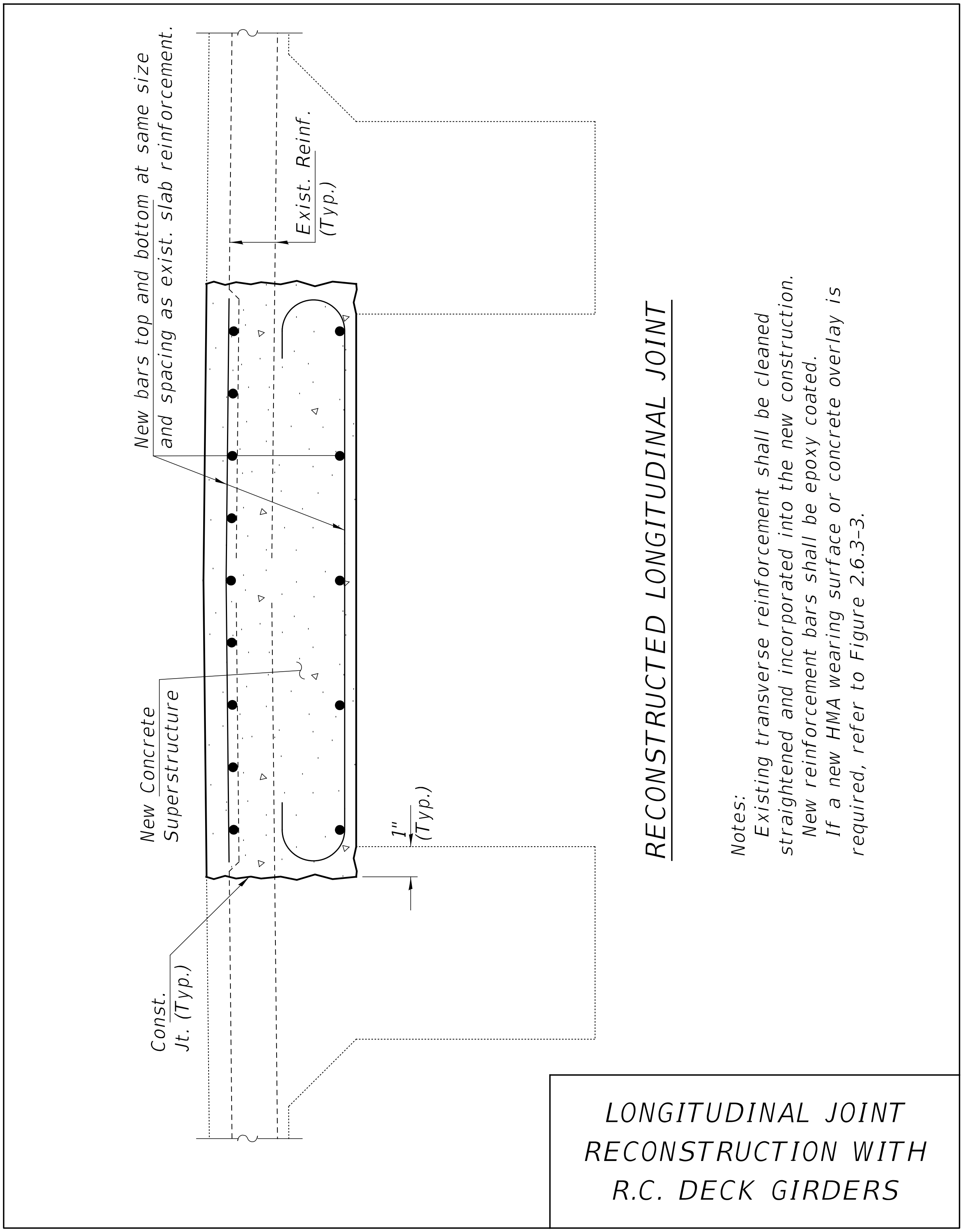

New bars top and bottom at same size and spacing as exist. slab reinforcement.
Exist. Reinf. (Typ.)
New Concrete Superstructure
1" (Typ.)
Const. Jt. (Typ.)
RECONSTRUCTED LONGITUDINAL JOINT
Notes:
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction.
New reinforcement bars shall be epoxy coated.
If a new HMA wearing surface or concrete overlay is required, refer to Figure 2.6.3-3.
LONGITUDINAL JOINT RECONSTRUCTION WITH R.C. DECK GIRDERS

Figure 2.6.3-4

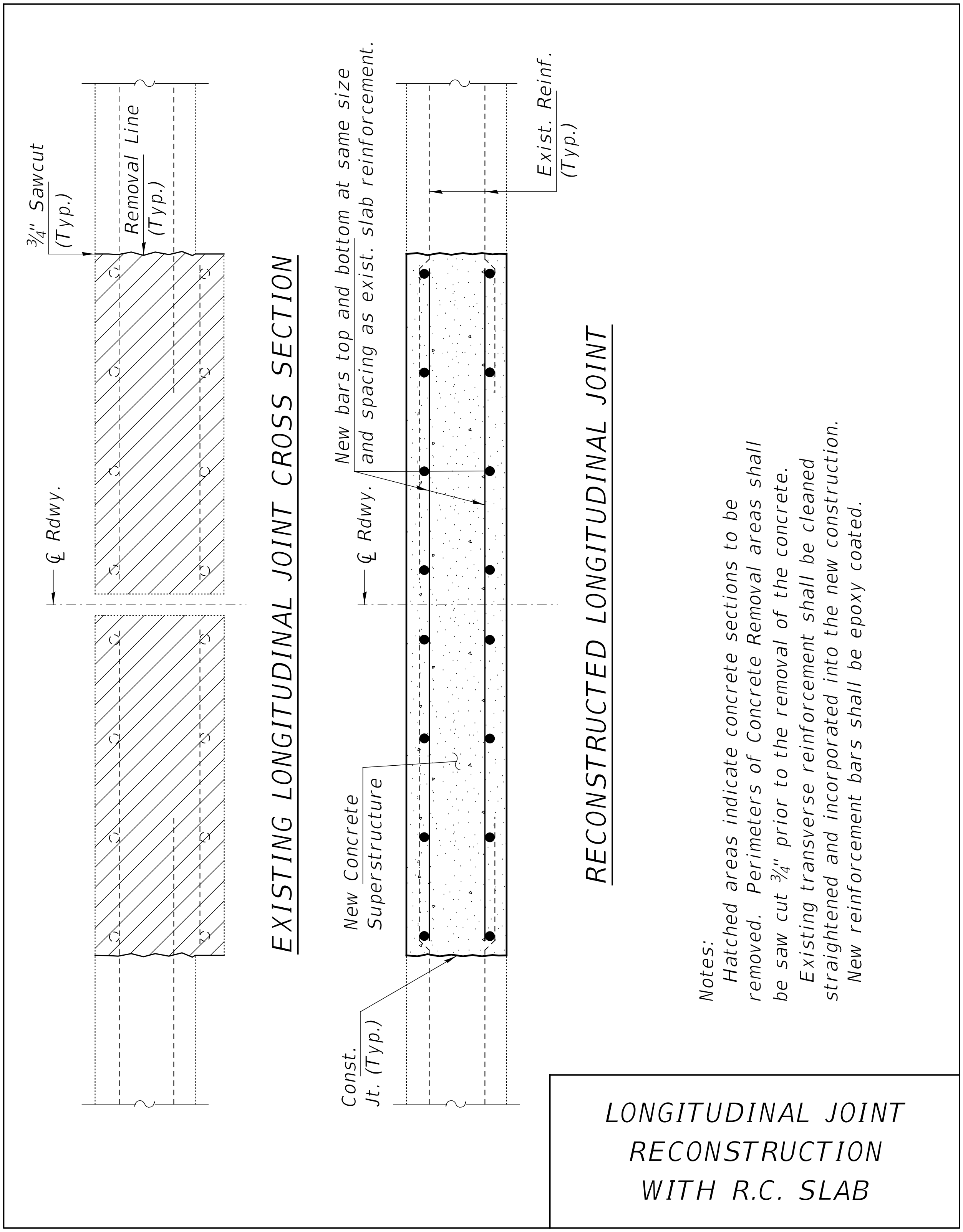

¾" Sawcut
(Typ.)
Removal Line
(Typ.)
℄ Rdwy.
EXISTING LONGITUDINAL JOINT CROSS SECTION
New bars top and bottom at same size
and spacing as exist. slab reinforcement.
Exist. Reinf.
(Typ.)
℄ Rdwy.
New Concrete
Superstructure
Const.
Jt. (Typ.)
RECONSTRUCTED LONGITUDINAL JOINT
Notes:
Hatched areas indicate concrete sections to be removed. Perimeters of Concrete Removal areas shall be saw cut ¾" prior to the removal of the concrete.
Existing transverse reinforcement shall be cleaned straightened and incorporated into the new construction.
New reinforcement bars shall be epoxy coated.
LONGITUDINAL JOINT
RECONSTRUCTION
WITH R.C. SLAB

FIGURE 2.6.3-5

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* Sec. 3.2.4 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).

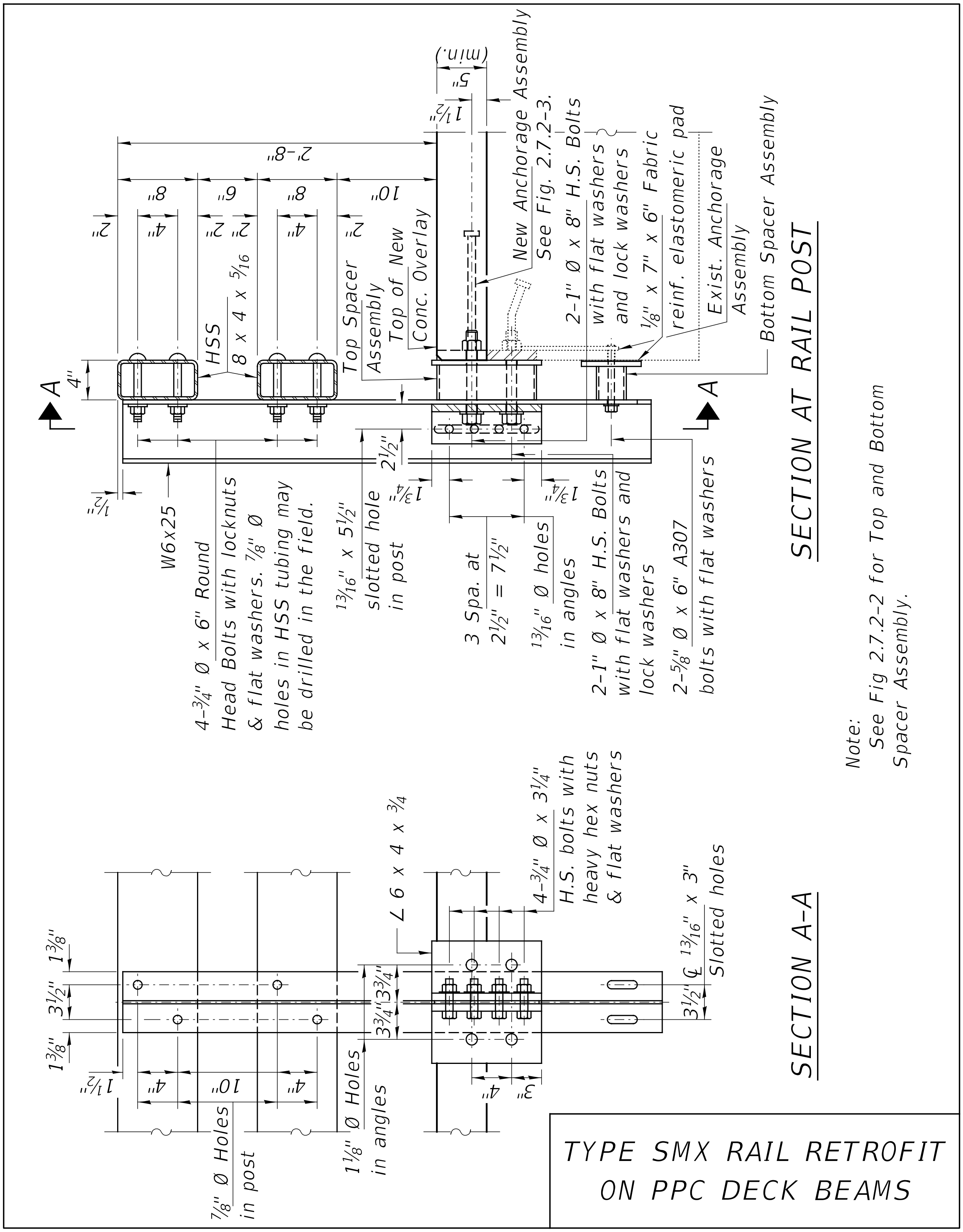

SECTION AT RAIL POST
SECTION A-A
A
A
2'-8"
10"
8"
6"
8"
2"
4"
4"
4"
1½"
5" (min.)
½"
2½"
1¾"
1¾"
New Anchorage Assembly See Fig. 2.7.2-3.
2-1" Ø x 8" H.S. Bolts with flat washers and lock washers
⅛" x 7" x 6" Fabric reinf. elastomeric pad
Exist. Anchorage Assembly
Bottom Spacer Assembly
Top Spacer Assembly
Top of New Conc. Overlay
HSS 8 x 4 x 5⁄16
W6x25
4-¾" Ø x 6" Round Head Bolts with locknuts & flat washers. ⅞" Ø holes in HSS tubing may be drilled in the field.
13⁄16" x 5½" slotted hole in post
3 Spa. at 2½" = 7½"
13⁄16" Ø holes in angles
2-1" Ø x 8" H.S. Bolts with flat washers and lock washers
2-⅝" Ø x 6" A307 bolts with flat washers
Note: See Fig 2.7.2-2 for Top and Bottom Spacer Assembly.
∠ 6 x 4 x ¾
4-¾" Ø x 3¼" H.S. bolts with heavy hex nuts & flat washers
13⁄16" x 3" Slotted holes
1⅜"
3½"
1⅜"
3¾"
3¾"
3½"
℄
10"
3"
⅞" Ø Holes in post
1⅛" Ø Holes in angles
TYPE SMX RAIL RETROFIT ON PPC DECK BEAMS

FIGURE 2.7.2-1

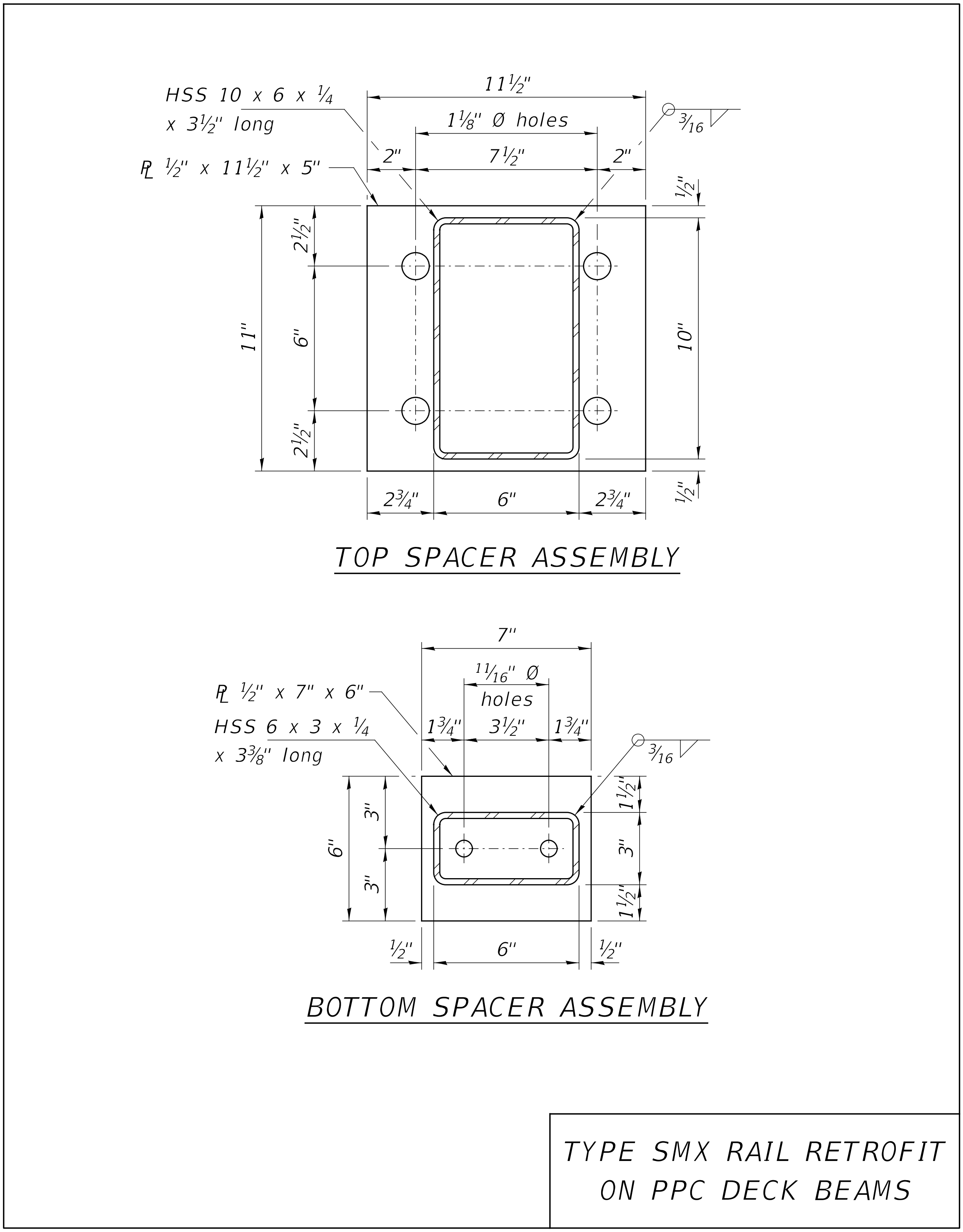

HSS 10 x 6 x 1/4
x 3 1/2" long
PL 1/2" x 11 1/2" x 5"
11 1/2"
1 1/8" Ø holes
2"
7 1/2"
2"
3/16
1/2"
2 1/2"
11"
6"
2 1/2"
10"
1/2"
2 3/4"
6"
2 3/4"
TOP SPACER ASSEMBLY
7"
11/16" Ø
holes
PL 1/2" x 7" x 6"
HSS 6 x 3 x 1/4
x 3 3/8" long
1 3/4"
3 1/2"
1 3/4"
3/16
1 1/2"
3"
3"
6"
3"
3"
1 1/2"
1/2"
6"
1/2"
BOTTOM SPACER ASSEMBLY
TYPE SMX RAIL RETROFIT
ON PPC DECK BEAMS

FIGURE 2.7.2-2

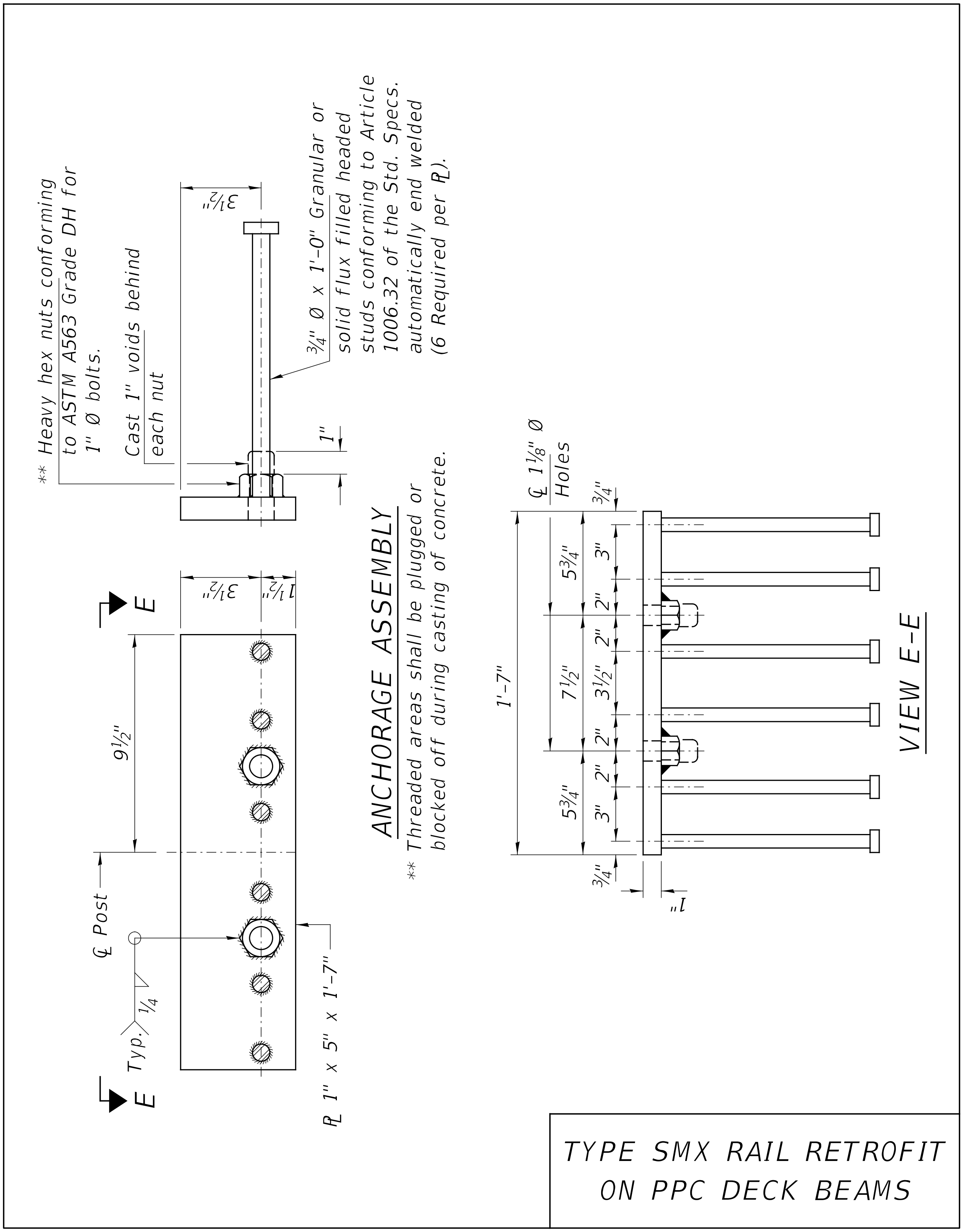

** Heavy hex nuts conforming to ASTM A563 Grade DH for 1" Ø bolts.
Cast 1" voids behind each nut
3/4" Ø x 1'-0" Granular or solid flux filled headed studs conforming to Article 1006.32 of the Std. Specs. automatically end welded (6 Required per PL).
3½"
1"
1½"
9½"
E
℄ Post
Typ.
¼
PL 1" x 5" x 1'-7"
ANCHORAGE ASSEMBLY
** Threaded areas shall be plugged or blocked off during casting of concrete.
℄ 1⅛" Ø Holes
¾"
5¾"
3"
2"
7½"
3½"
1'-7"
VIEW E-E
TYPE SMX RAIL RETROFIT
ON PPC DECK BEAMS

FIGURE 2.7.2-3

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.

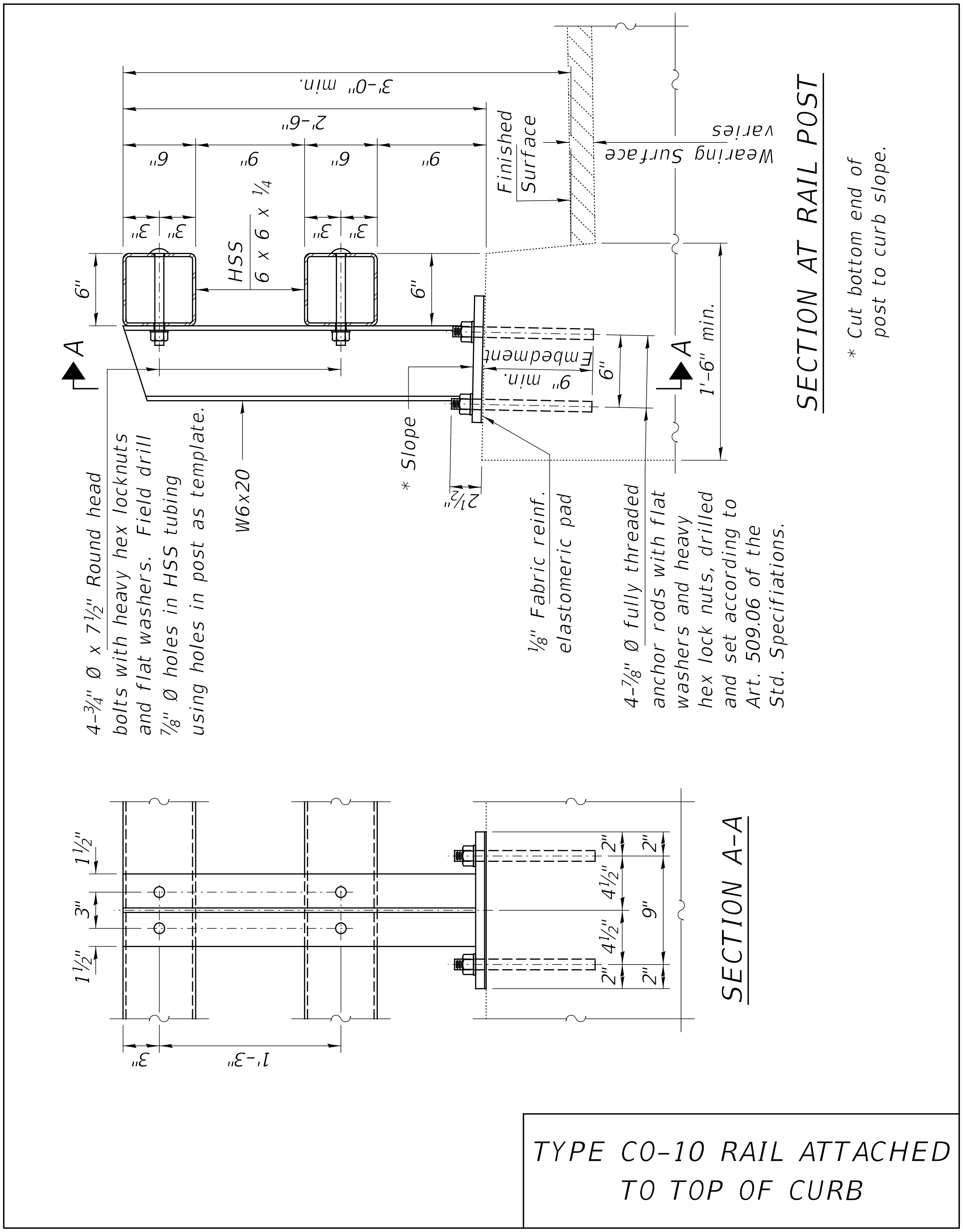
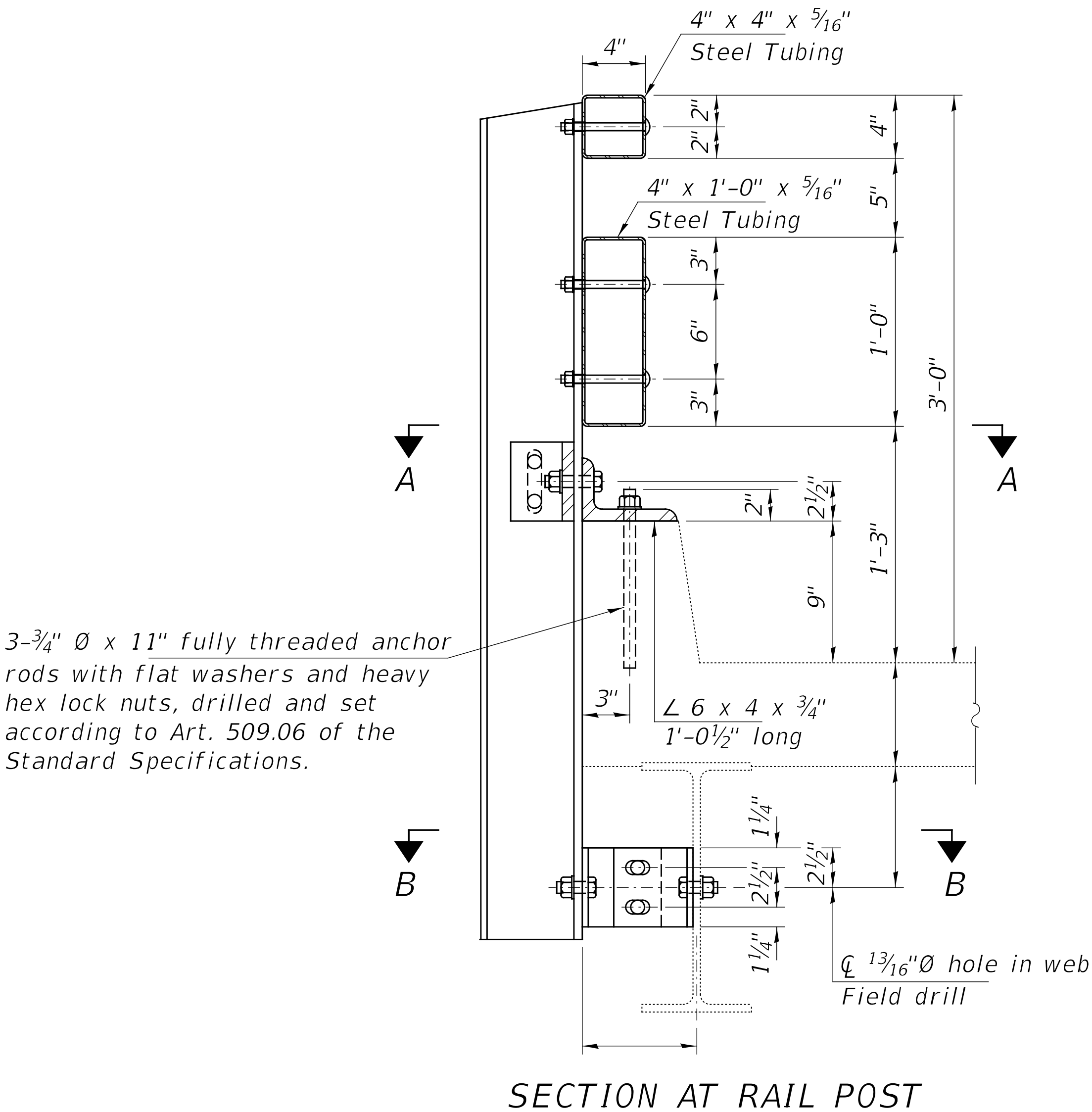

SECTION AT RAIL POST
* Cut bottom end of post to curb slope.
3'-0" min.
2'-6"
6"
9"
3"
HSS 6 x 6 x 1/4
Finished Surface
Wearing Surface varies
9" min. Embedment
1'-6" min.
* Slope
2 1/2"
W6x20
4-3/4" Ø x 7 1/2" Round head bolts with heavy hex locknuts and flat washers. Field drill 7/8" Ø holes in HSS tubing using holes in post as template.
1/8" Fabric reinf. elastomeric pad
4-7/8" Ø fully threaded anchor rods with flat washers and heavy hex lock nuts, drilled and set according to Art. 509.06 of the Std. Specifiations.
A
SECTION A-A
1 1/2"
3"
2"
4 1/2"
9"
1'-3"
TYPE CO-10 RAIL ATTACHED TO TOP OF CURB

FIGURE 2.7.3-1

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

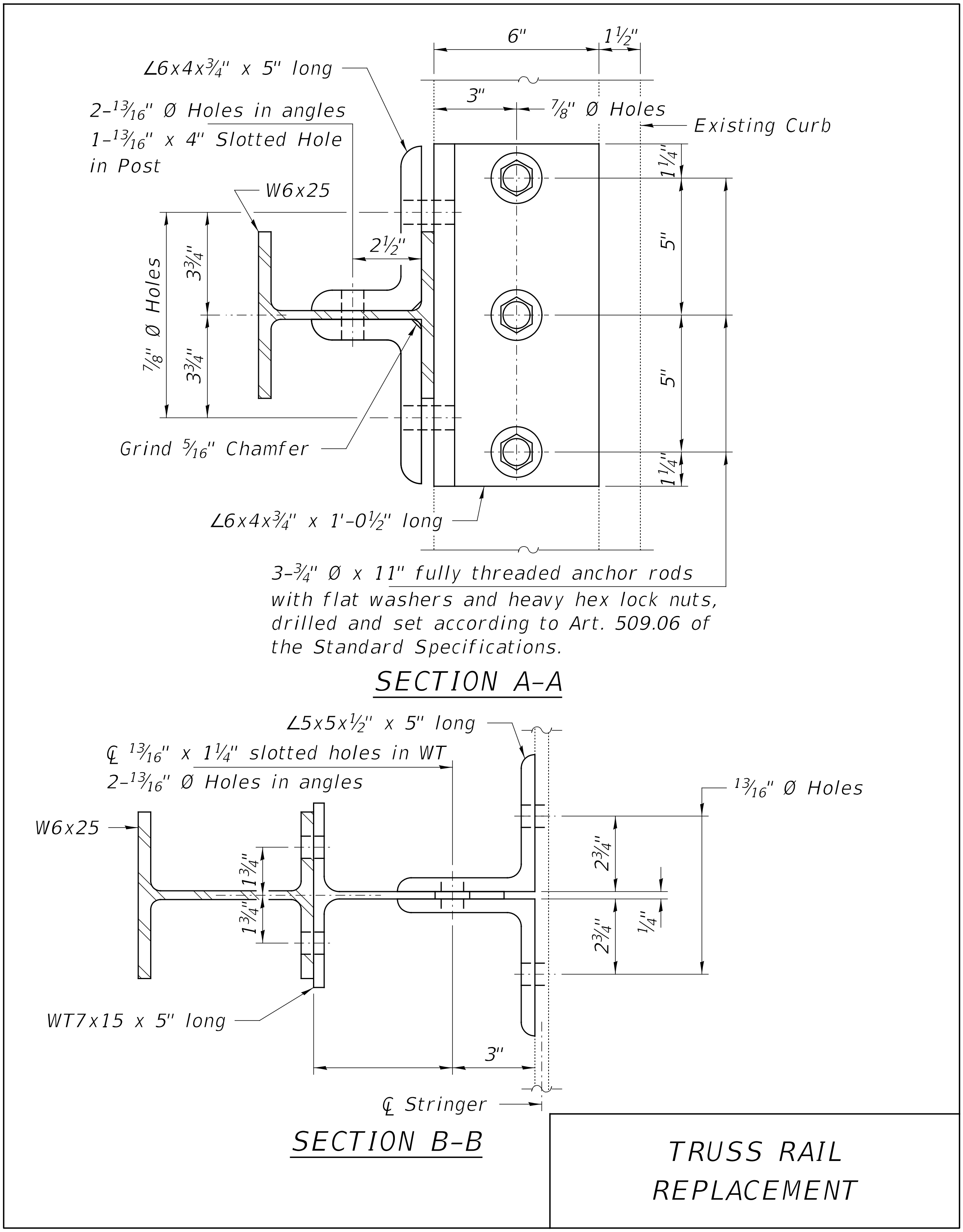

∠6x4x¾" x 5" long
2-13/16" Ø Holes in angles
1-13/16" x 4" Slotted Hole in Post
W6x25
6"
1½"
3"
⅞" Ø Holes
Existing Curb
2½"
⅞" Ø Holes
3¾"
3¾"
1¼"
5"
5"
1¼"
Grind 5/16" Chamfer
∠6x4x¾" x 1'-0½" long
3-¾" Ø x 11" fully threaded anchor rods with flat washers and heavy hex lock nuts, drilled and set according to Art. 509.06 of the Standard Specifications.
SECTION A-A
∠5x5x½" x 5" long
℄ 13/16" x 1¼" slotted holes in WT
2-13/16" Ø Holes in angles
13/16" Ø Holes
W6x25
1¾"
1¾"
2¾"
2¾"
¼"
WT7x15 x 5" long
3"
℄ Stringer
SECTION B-B
TRUSS RAIL REPLACEMENT

FIGURE 2.7.4-2

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 Fig. 2.7.5-1 or alternatively precast as shown in Figure 2.7.5-2.

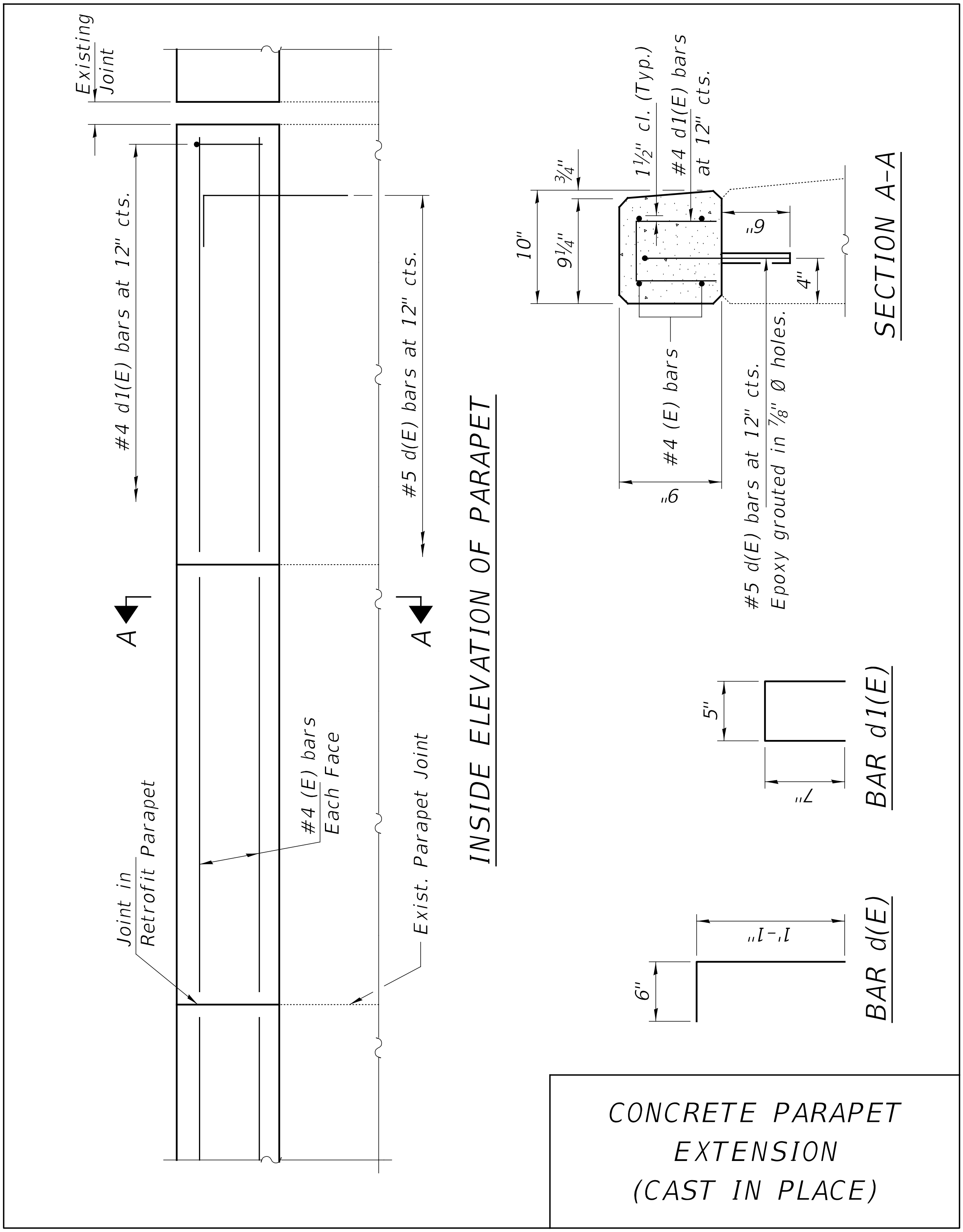
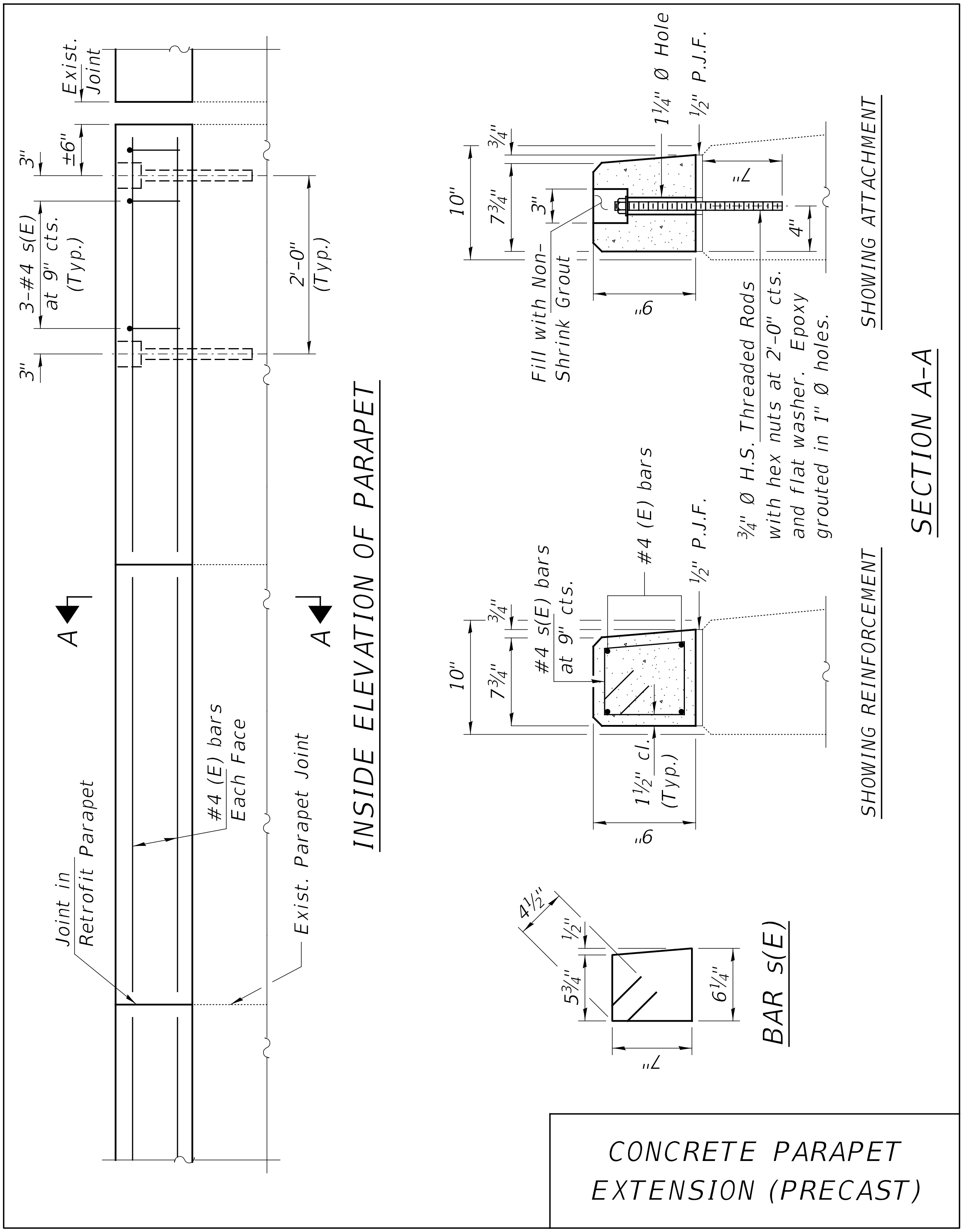

Existing Joint
#4 d1(E) bars at 12" cts.
#5 d(E) bars at 12" cts.
A
A
Joint in Retrofit Parapet
#4 (E) bars Each Face
Exist. Parapet Joint
INSIDE ELEVATION OF PARAPET
¾"
1½" cl. (Typ.)
#4 d1(E) bars at 12" cts.
10"
9¼"
6"
4"
#4 (E) bars
9"
#5 d(E) bars at 12" cts.
Epoxy grouted in ⅞" Ø holes.
SECTION A-A
5"
7"
BAR d1(E)
1'-1"
6"
BAR d(E)
CONCRETE PARAPET
EXTENSION
(CAST IN PLACE)

FIGURE 2.7.5-1



Exist. Joint
3"
±6"
3-#4 s(E) at 9" cts. (Typ.)
2'-0" (Typ.)
3"
A
A
Joint in Retrofit Parapet
#4 (E) bars Each Face
Exist. Parapet Joint
INSIDE ELEVATION OF PARAPET
10"
¾"
7¾"
3"
1¼" Ø Hole
½" P.J.F.
7"
4"
9"
Fill with Non-Shrink Grout
¾" Ø H.S. Threaded Rods with hex nuts at 2'-0" cts. and flat washer. Epoxy grouted in 1" Ø holes.
SHOWING ATTACHMENT
10"
¾"
7¾"
#4 s(E) bars at 9" cts.
#4 (E) bars
½" P.J.F.
1½" cl. (Typ.)
9"
SHOWING REINFORCEMENT
SECTION A–A
4½"
½"
5¾"
6¼"
7"
BAR s(E)
CONCRETE PARAPET EXTENSION (PRECAST)

FIGURE 2.7.5-2

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 CL of roadway are shown in Fig. 2.7.6-1 through 2.7.6-3, those in which the wingwall is stepped are shown in Fig. 2.7.6-4, and those in which the wingwall is skewed away from the CL of roadway are shown in Figs. 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.

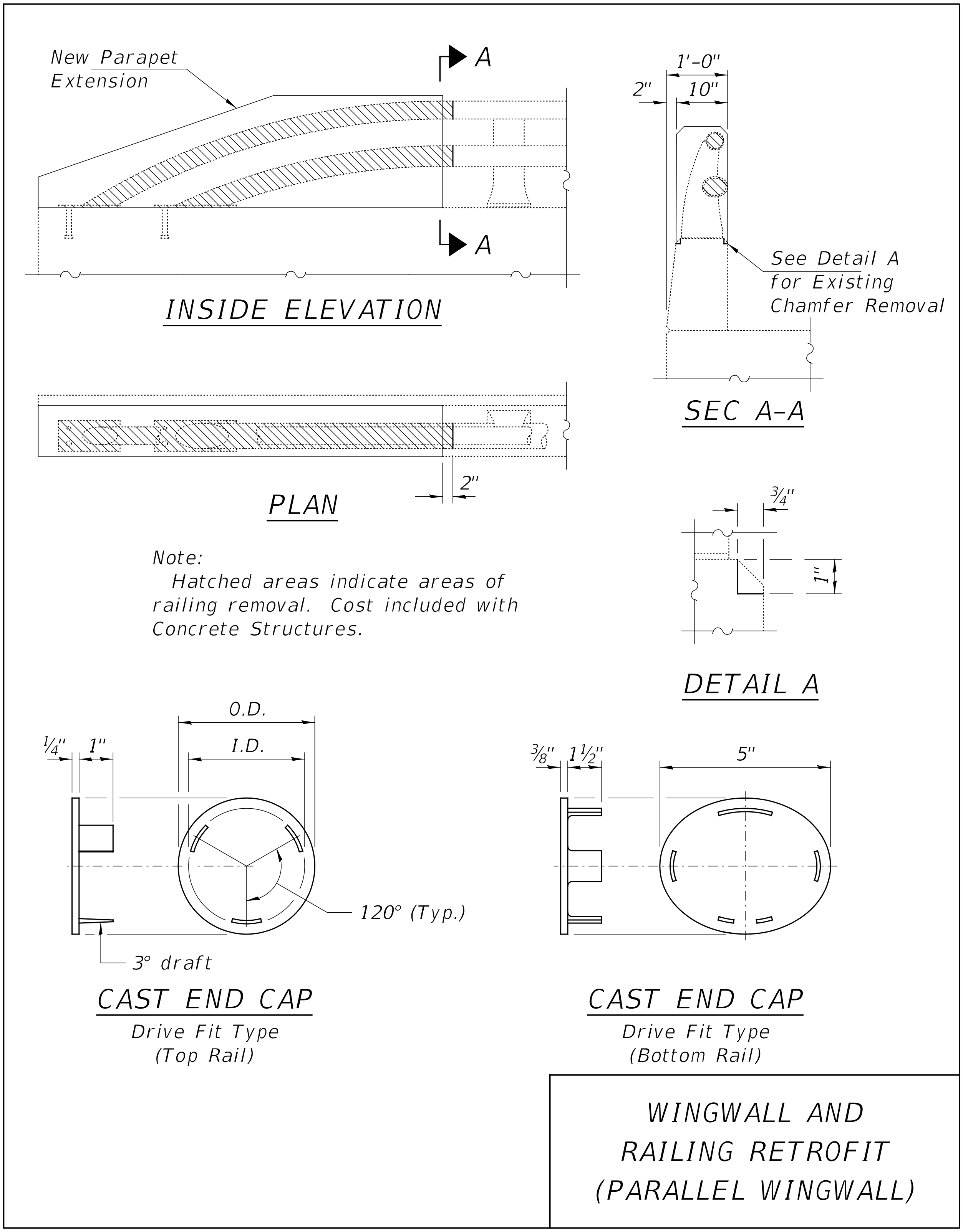

New Parapet Extension
A
A
INSIDE ELEVATION
1'-0"
2"
10"
See Detail A for Existing Chamfer Removal
SEC A–A
2"
PLAN
Note:
Hatched areas indicate areas of railing removal. Cost included with Concrete Structures.
3/4"
1"
DETAIL A
O.D.
I.D.
1/4"
1"
120° (Typ.)
3° draft
CAST END CAP
Drive Fit Type
(Top Rail)
3/8"
1 1/2"
5"
CAST END CAP
Drive Fit Type
(Bottom Rail)
WINGWALL AND
RAILING RETROFIT
(PARALLEL WINGWALL)

FIGURE 2.7.6-1

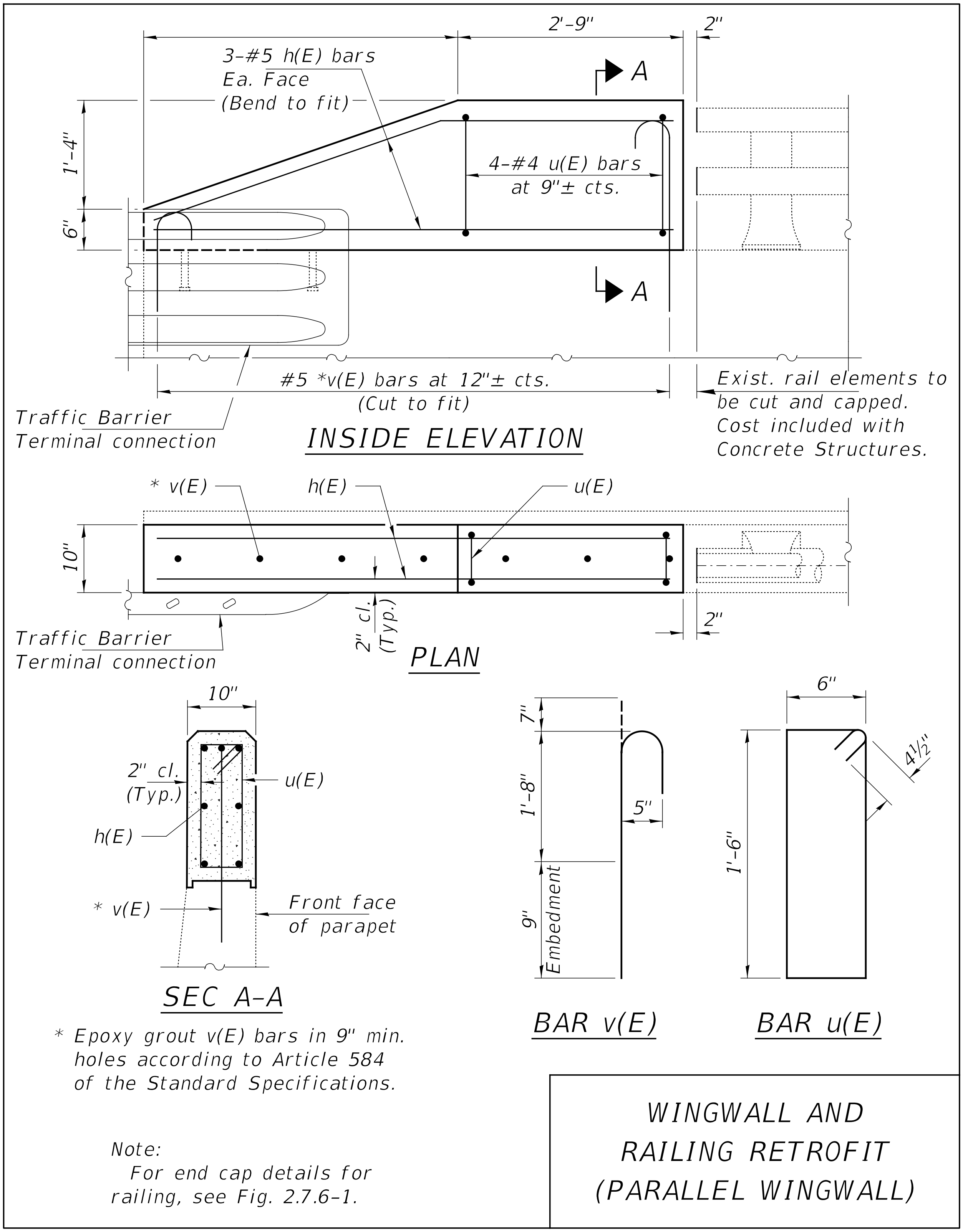

2'-9"
2"
3-#5 h(E) bars
Ea. Face
(Bend to fit)
A
A
1'-4"
6"
4-#4 u(E) bars
at 9"± cts.
#5 *v(E) bars at 12"± cts.
(Cut to fit)
Traffic Barrier
Terminal connection
Exist. rail elements to
be cut and capped.
Cost included with
Concrete Structures.
INSIDE ELEVATION
* v(E)
h(E)
u(E)
10"
2"
2" cl.
(Typ.)
Traffic Barrier
Terminal connection
PLAN
10"
2" cl.
(Typ.)
u(E)
h(E)
* v(E)
Front face
of parapet
SEC A-A
7"
1'-8"
5"
9"
Embedment
BAR v(E)
6"
4½"
1'-6"
BAR u(E)
* Epoxy grout v(E) bars in 9" min.
holes according to Article 584
of the Standard Specifications.
Note:
For end cap details for
railing, see Fig. 2.7.6-1.
WINGWALL AND
RAILING RETROFIT
(PARALLEL WINGWALL)

FIGURE 2.7.6-2

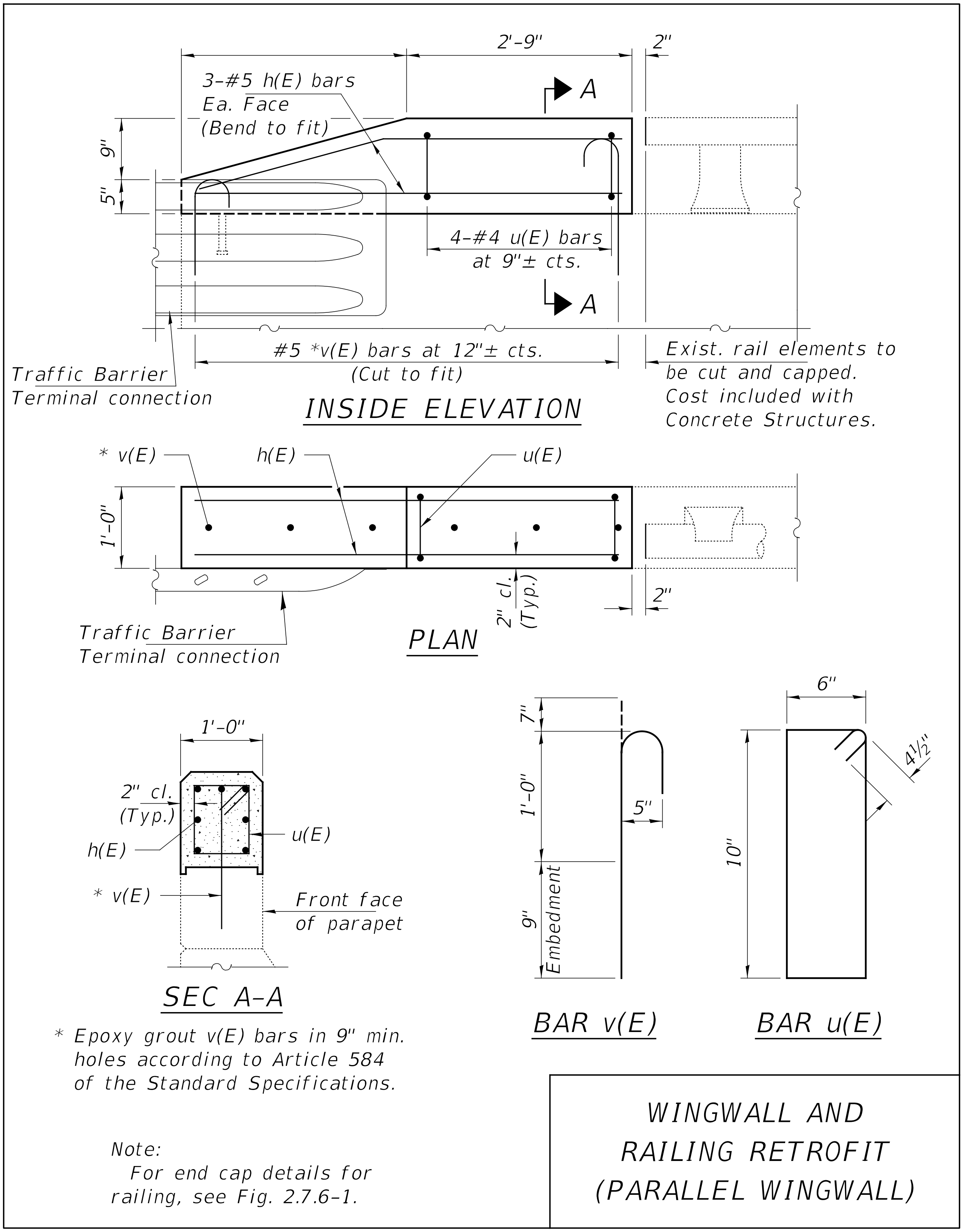
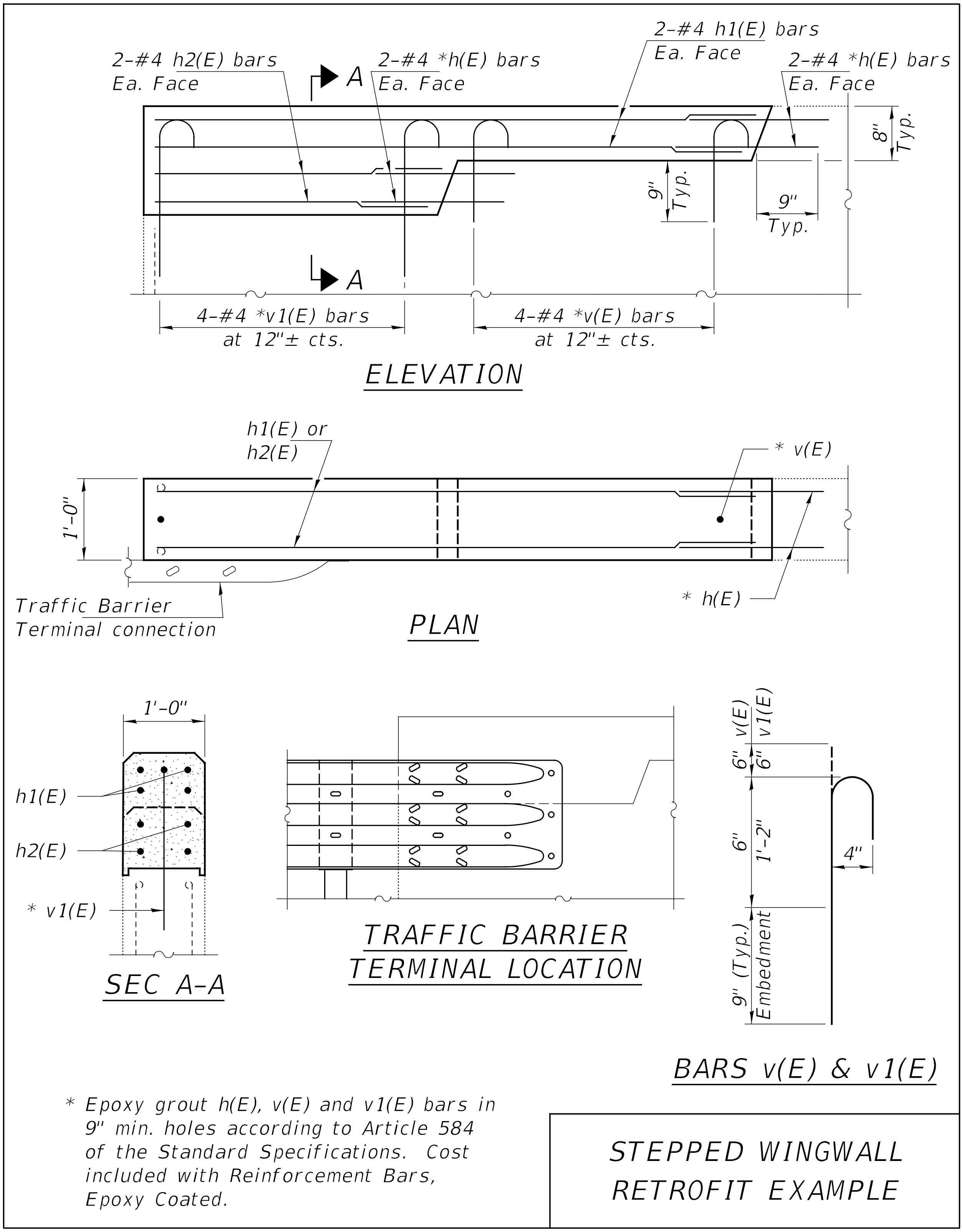

2'-9"
2"
3-#5 h(E) bars
Ea. Face
(Bend to fit)
A
9"
5"
4-#4 u(E) bars
at 9"± cts.
A
#5 *v(E) bars at 12"± cts.
(Cut to fit)
Traffic Barrier
Terminal connection
Exist. rail elements to
be cut and capped.
Cost included with
Concrete Structures.
INSIDE ELEVATION
* v(E)
h(E)
u(E)
1'-0"
2" cl.
(Typ.)
2"
Traffic Barrier
Terminal connection
PLAN
1'-0"
2" cl.
(Typ.)
h(E)
u(E)
* v(E)
Front face
of parapet
SEC A-A
7"
1'-0"
5"
9"
Embedment
BAR v(E)
6"
4½"
10"
BAR u(E)
* Epoxy grout v(E) bars in 9" min.
holes according to Article 584
of the Standard Specifications.
Note:
For end cap details for
railing, see Fig. 2.7.6-1.
WINGWALL AND
RAILING RETROFIT
(PARALLEL WINGWALL)

FIGURE 2.7.6-3



2-#4 h2(E) bars
Ea. Face
2-#4 *h(E) bars
Ea. Face
2-#4 h1(E) bars
Ea. Face
2-#4 *h(E) bars
Ea. Face
A
A
8" Typ.
9" Typ.
9" Typ.
4-#4 *v1(E) bars
at 12"± cts.
4-#4 *v(E) bars
at 12"± cts.
ELEVATION
h1(E) or
h2(E)
* v(E)
1'-0"
Traffic Barrier
Terminal connection
PLAN
* h(E)
1'-0"
h1(E)
h2(E)
* v1(E)
SEC A-A
TRAFFIC BARRIER
TERMINAL LOCATION
v(E)
v1(E)
6"
6"
6"
1'-2"
4"
9" (Typ.)
Embedment
BARS v(E) & v1(E)
* Epoxy grout h(E), v(E) and v1(E) bars in 9" min. holes according to Article 584 of the Standard Specifications. Cost included with Reinforcement Bars, Epoxy Coated.
STEPPED WINGWALL
RETROFIT EXAMPLE

FIGURE 2.7.6-4

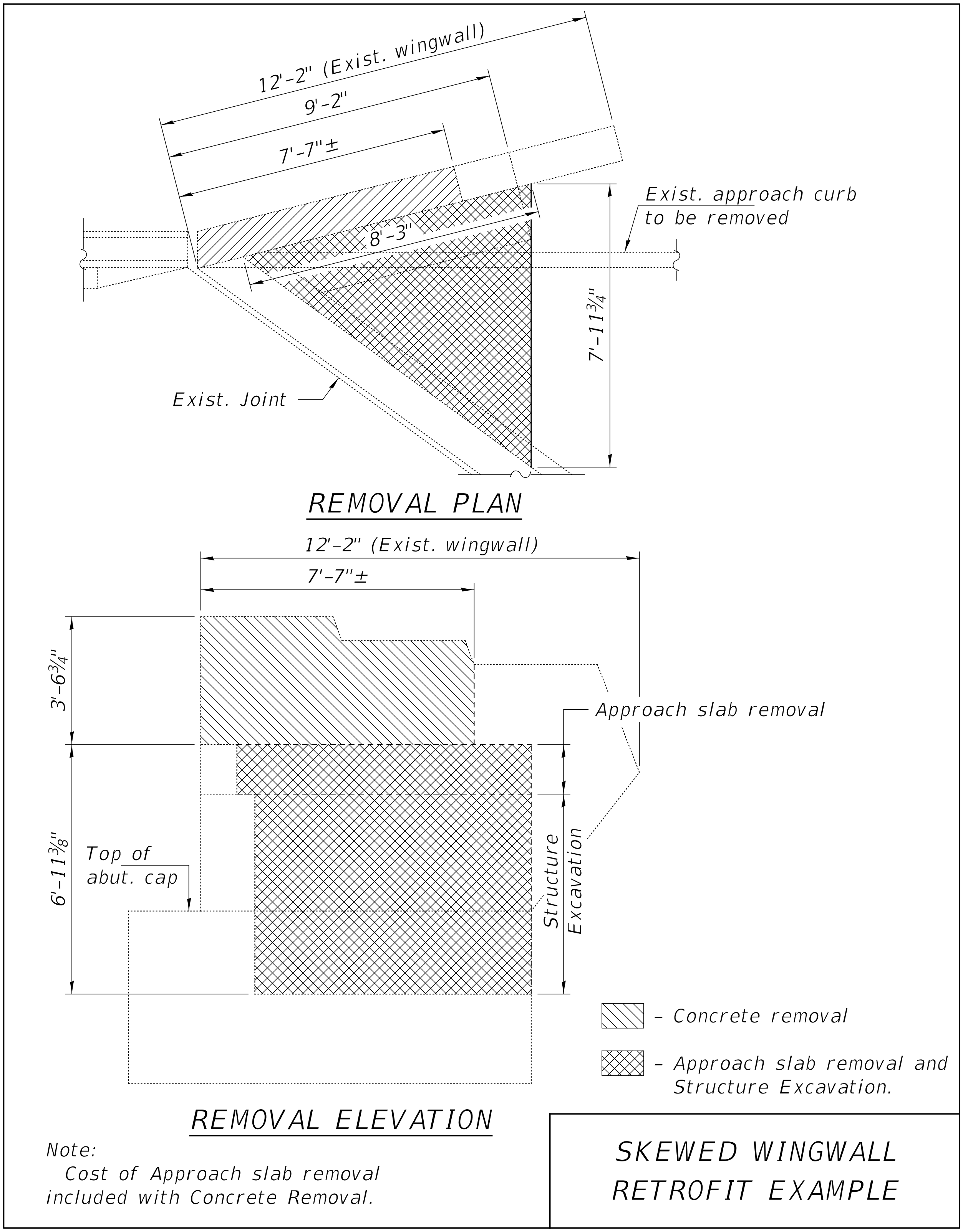

12'-2" (Exist. wingwall)
9'-2"
7'-7"±
8'-3"
7'-11¾"
Exist. approach curb to be removed
Exist. Joint
REMOVAL PLAN
12'-2" (Exist. wingwall)
7'-7"±
3'-6¾"
6'-11⅜"
Top of abut. cap
Approach slab removal
Structure Excavation
– Concrete removal
– Approach slab removal and Structure Excavation.
REMOVAL ELEVATION
Note:
Cost of Approach slab removal included with Concrete Removal.
SKEWED WINGWALL RETROFIT EXAMPLE

FIGURE 2.7.6-5

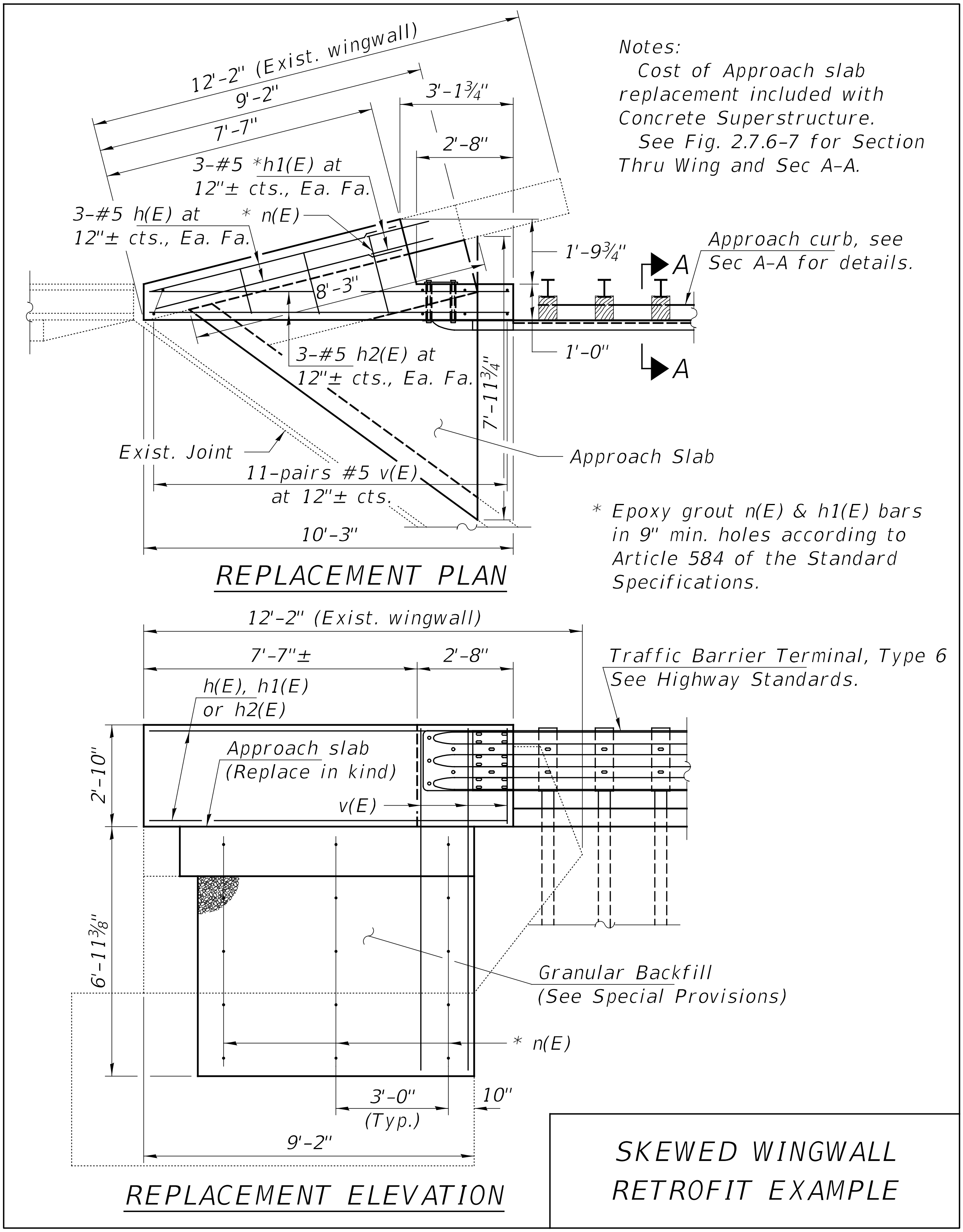
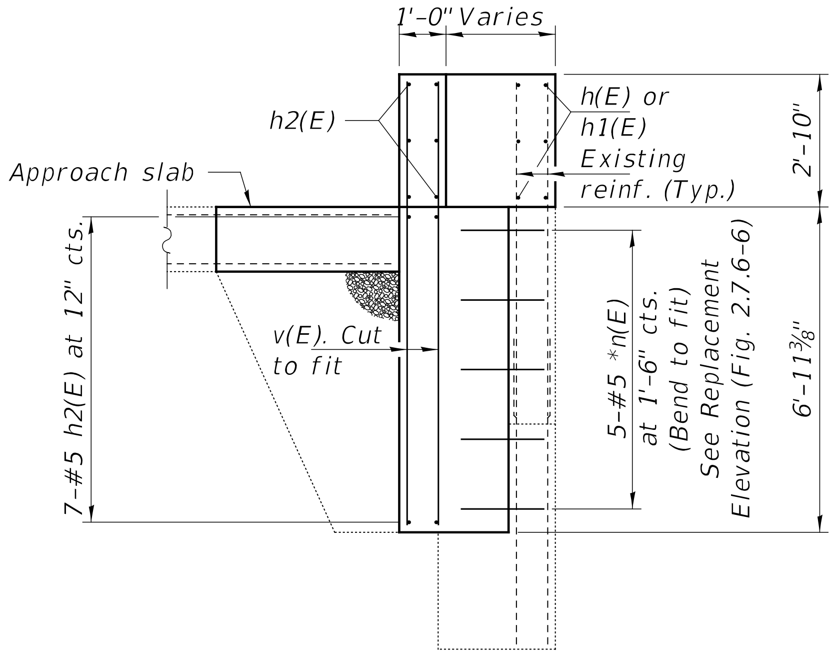
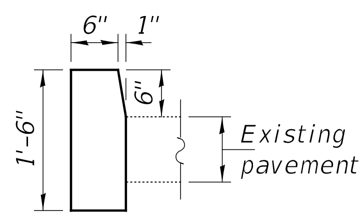

Notes:
Cost of Approach slab replacement included with Concrete Superstructure.
See Fig. 2.7.6-7 for Section Thru Wing and Sec A-A.
12'-2" (Exist. wingwall)
9'-2"
7'-7"
3'-1¾"
2'-8"
3-#5 *h1(E) at 12"± cts., Ea. Fa.
3-#5 h(E) at 12"± cts., Ea. Fa.
* n(E)
1'-9¾"
Approach curb, see Sec A-A for details.
A
8'-3"
3-#5 h2(E) at 12"± cts., Ea. Fa.
1'-0"
7'-11¾"
Exist. Joint
Approach Slab
11-pairs #5 v(E) at 12"± cts.
10'-3"
REPLACEMENT PLAN
* Epoxy grout n(E) & h1(E) bars in 9" min. holes according to Article 584 of the Standard Specifications.
12'-2" (Exist. wingwall)
7'-7"±
2'-8"
Traffic Barrier Terminal, Type 6 See Highway Standards.
h(E), h1(E) or h2(E)
Approach slab (Replace in kind)
v(E)
2'-10"
6'-11⅜"
Granular Backfill (See Special Provisions)
* n(E)
3'-0" (Typ.)
10"
9'-2"
REPLACEMENT ELEVATION
SKEWED WINGWALL RETROFIT EXAMPLE

FIGURE 2.7.6-6



SECTION THRU WING



SEC A-A

* 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

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 it 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 Fig. 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 Fig. 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.

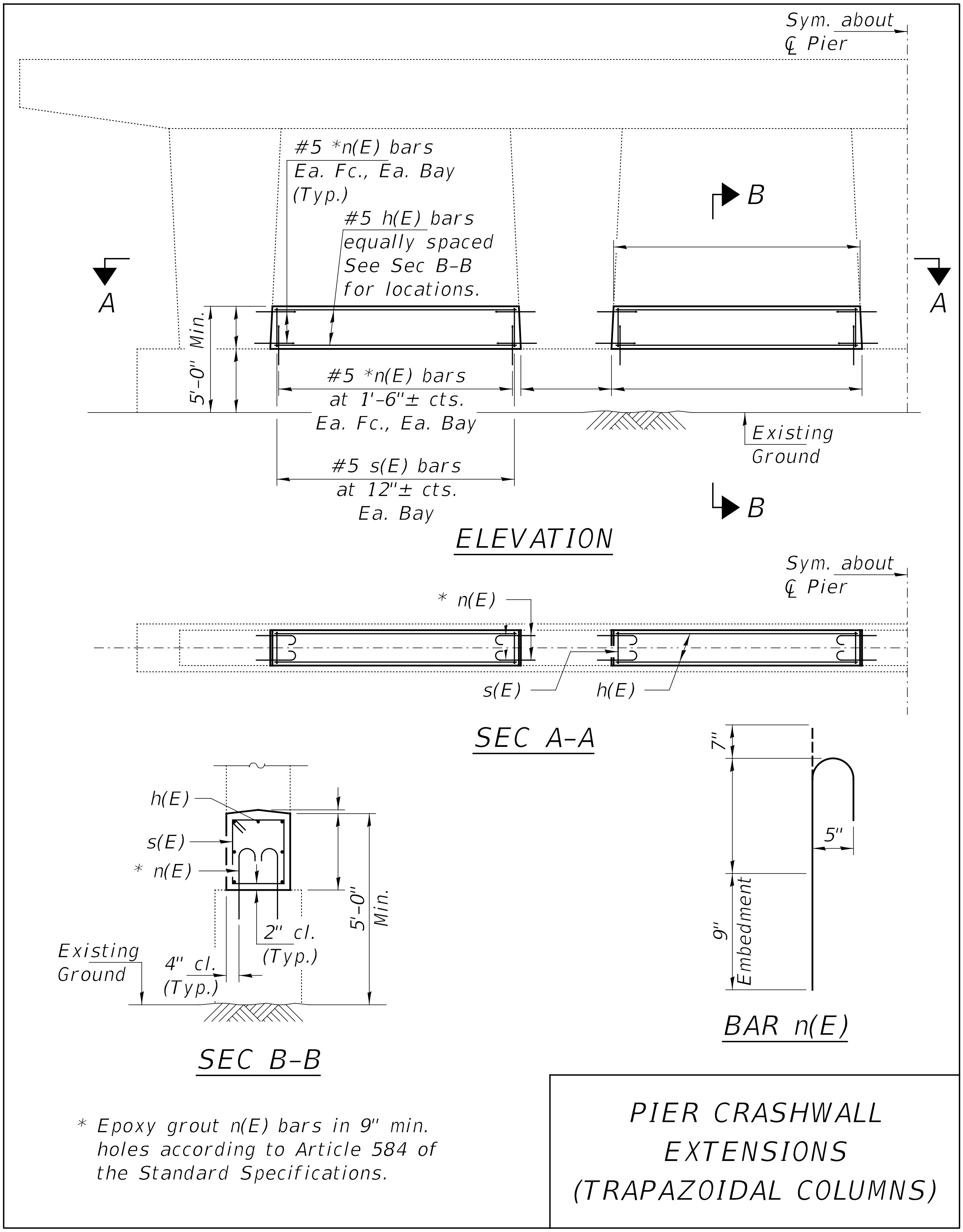

Sym. about
℄ Pier
#5 *n(E) bars
Ea. Fc., Ea. Bay
(Typ.)
#5 h(E) bars
equally spaced
See Sec B–B
for locations.
B
A
A
5'-0" Min.
#5 *n(E) bars
at 1'-6"± cts.
Ea. Fc., Ea. Bay
Existing
Ground
#5 s(E) bars
at 12"± cts.
Ea. Bay
B
ELEVATION
Sym. about
℄ Pier
* n(E)
s(E)
h(E)
SEC A–A
h(E)
s(E)
* n(E)
Existing
Ground
4" cl.
(Typ.)
2" cl.
(Typ.)
5'-0" Min.
SEC B–B
7"
5"
9" Embedment
BAR n(E)
* Epoxy grout n(E) bars in 9" min.
holes according to Article 584 of
the Standard Specifications.
PIER CRASHWALL
EXTENSIONS
(TRAPAZOIDAL COLUMNS)

FIGURE 2.7.7-1

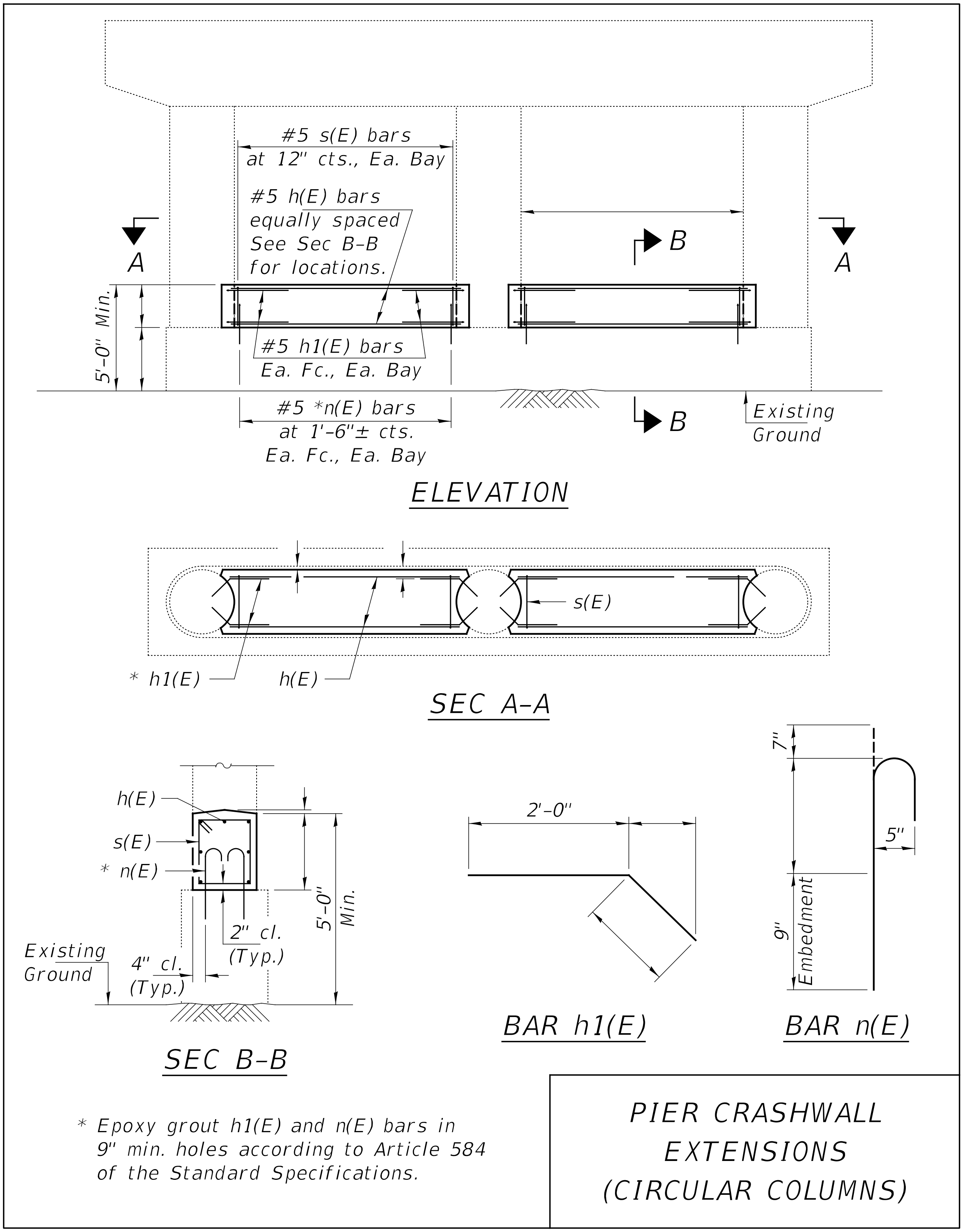

#5 s(E) bars
at 12" cts., Ea. Bay
#5 h(E) bars
equally spaced
See Sec B–B
for locations.
A
B
A
5'-0" Min.
#5 h1(E) bars
Ea. Fc., Ea. Bay
#5 *n(E) bars
at 1'-6"± cts.
Ea. Fc., Ea. Bay
B
Existing
Ground
ELEVATION
s(E)
* h1(E)
h(E)
SEC A–A
h(E)
s(E)
* n(E)
Existing
Ground
4" cl.
(Typ.)
2" cl.
(Typ.)
5'-0"
Min.
SEC B–B
2'-0"
BAR h1(E)
7"
5"
9"
Embedment
BAR n(E)
* Epoxy grout h1(E) and n(E) bars in
9" min. holes according to Article 584
of the Standard Specifications.
PIER CRASHWALL
EXTENSIONS
(CIRCULAR COLUMNS)

FIGURE 2.7.7-2

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.

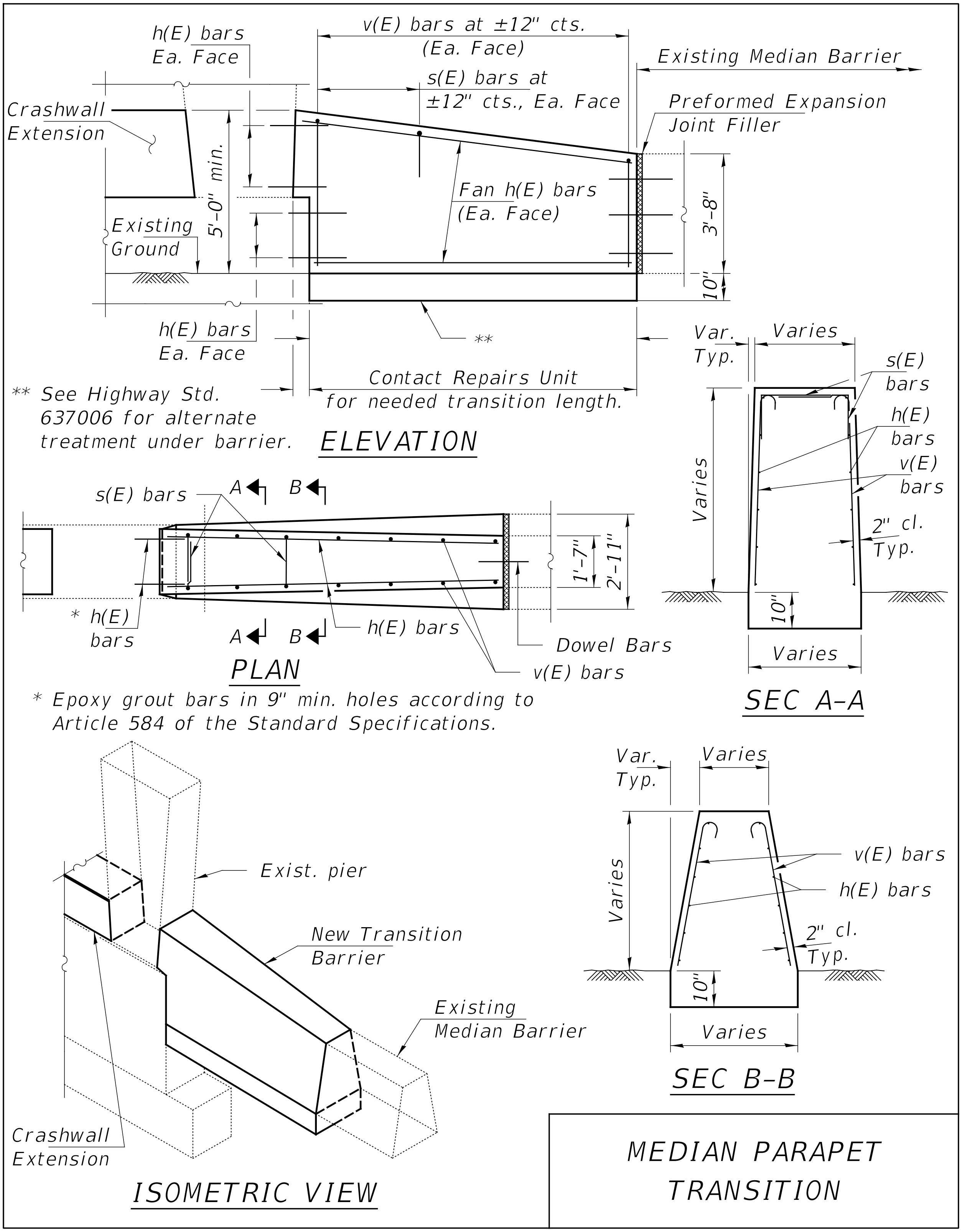

h(E) bars
Ea. Face
v(E) bars at ±12" cts.
(Ea. Face)
Existing Median Barrier
s(E) bars at
±12" cts., Ea. Face
Preformed Expansion
Joint Filler
Crashwall
Extension
5'-0" min.
Fan h(E) bars
(Ea. Face)
Existing
Ground
3'-8"
10"
h(E) bars
Ea. Face
**
Contact Repairs Unit
for needed transition length.
** See Highway Std.
637006 for alternate
treatment under barrier.
ELEVATION
Var.
Typ.
Varies
s(E)
bars
h(E)
bars
v(E)
bars
Varies
2" cl.
Typ.
10"
Varies
SEC A-A
s(E) bars
A
B
1'-7"
2'-11"
* h(E)
bars
A
B
h(E) bars
Dowel Bars
v(E) bars
PLAN
* Epoxy grout bars in 9" min. holes according to
Article 584 of the Standard Specifications.
Var.
Typ.
Varies
Exist. pier
v(E) bars
h(E) bars
Varies
New Transition
Barrier
2" cl.
Typ.
10"
Existing
Median Barrier
Varies
SEC B-B
Crashwall
Extension
ISOMETRIC VIEW
MEDIAN PARAPET
TRANSITION

FIGURE 2.7.8-1

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.

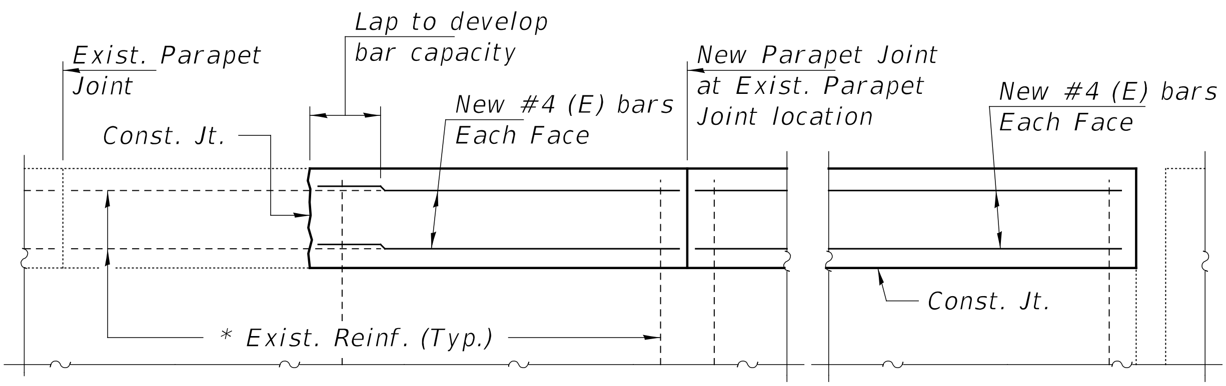
2. Salvage, clean and incorporate all existing vertical reinforcement bars into the new construction.

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.

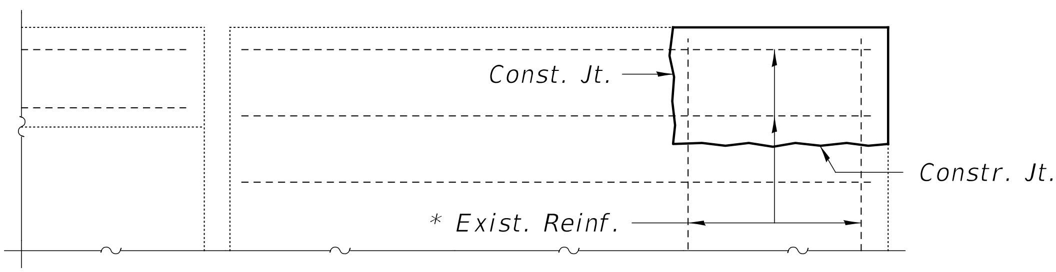
4. Except at existing parapet joints, new horizontal reinforcement bars should be lapped with existing horizontal reinforcement bars sufficiently to develop the strength of the bar.

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

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.

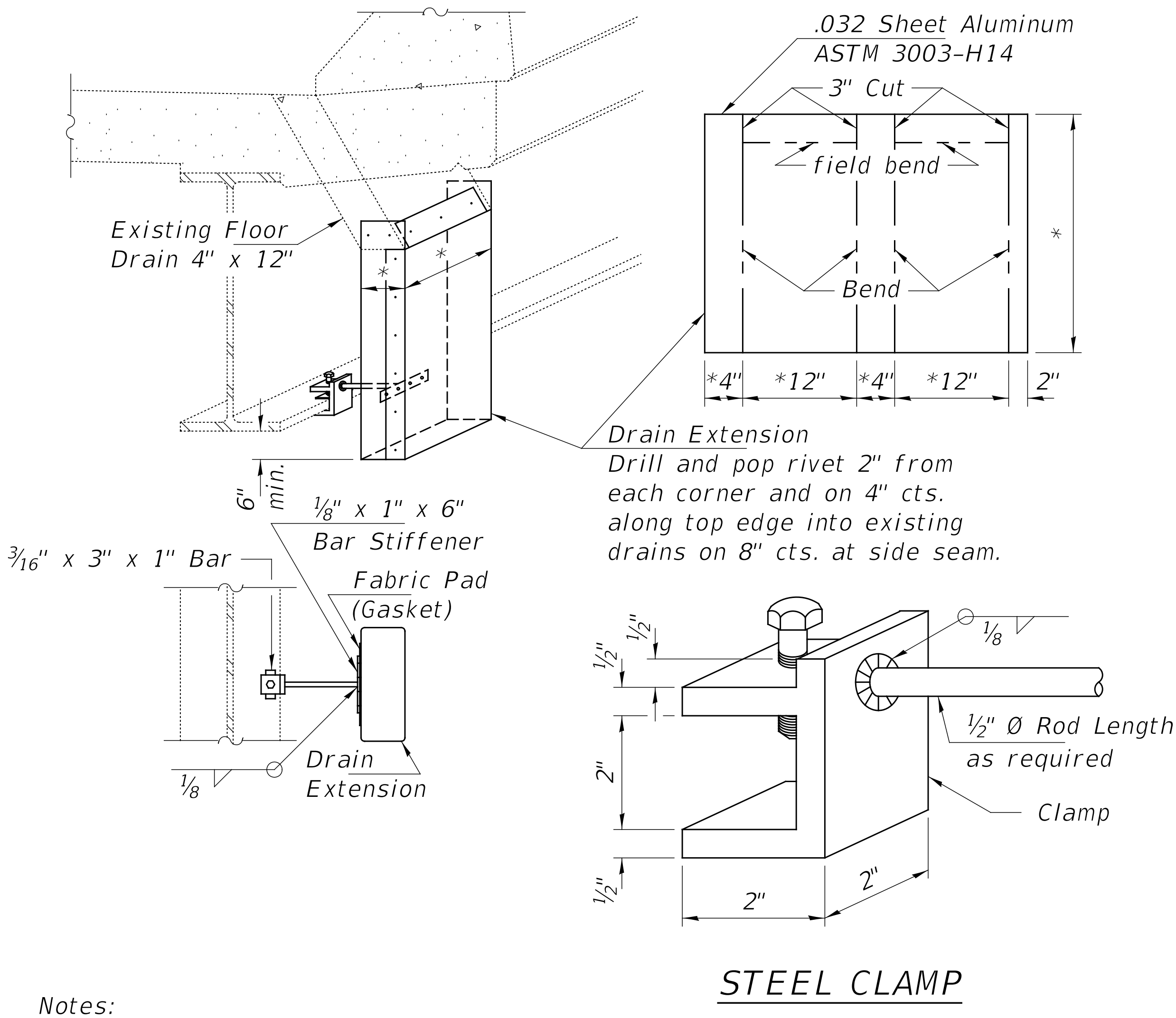
2.8.2 Drain Extensions

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.



Notes:

Pop rivet the 1/8" 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

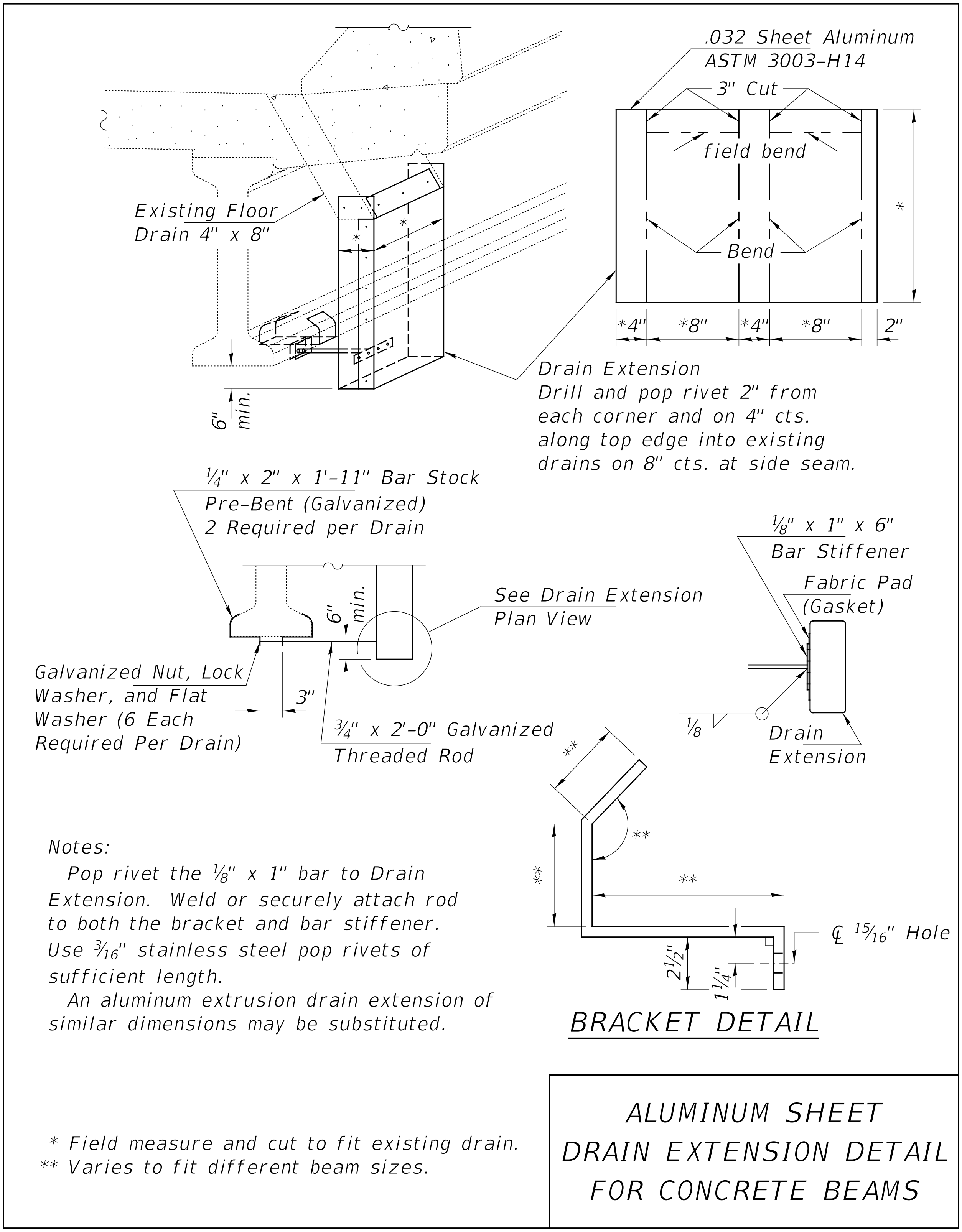

.032 Sheet Aluminum
ASTM 3003-H14
3" Cut
field bend
Bend
*
*4" *8" *4" *8" 2"
Existing Floor
Drain 4" x 8"
*
*
6" min.
Drain Extension
Drill and pop rivet 2" from each corner and on 4" cts. along top edge into existing drains on 8" cts. at side seam.
1/4" x 2" x 1'-11" Bar Stock
Pre-Bent (Galvanized)
2 Required per Drain
6" min.
See Drain Extension
Plan View
Galvanized Nut, Lock Washer, and Flat Washer (6 Each Required Per Drain)
3"
3/4" x 2'-0" Galvanized
Threaded Rod
1/8" x 1" x 6"
Bar Stiffener
Fabric Pad
(Gasket)
1/8
Drain
Extension
**
**
**
**
℄ 15/16" Hole
2 1/2"
1 1/4"
BRACKET DETAIL
Notes:
Pop rivet the 1/8" x 1" bar to Drain Extension. Weld or securely attach rod to both the bracket and bar stiffener. Use 3/16" stainless steel pop rivets of sufficient length.
An aluminum extrusion drain extension of similar dimensions may be substituted.
* Field measure and cut to fit existing drain.
** Varies to fit different beam sizes.
ALUMINUM SHEET
DRAIN EXTENSION DETAIL
FOR CONCRETE BEAMS

FIGURE 2.8.2-2

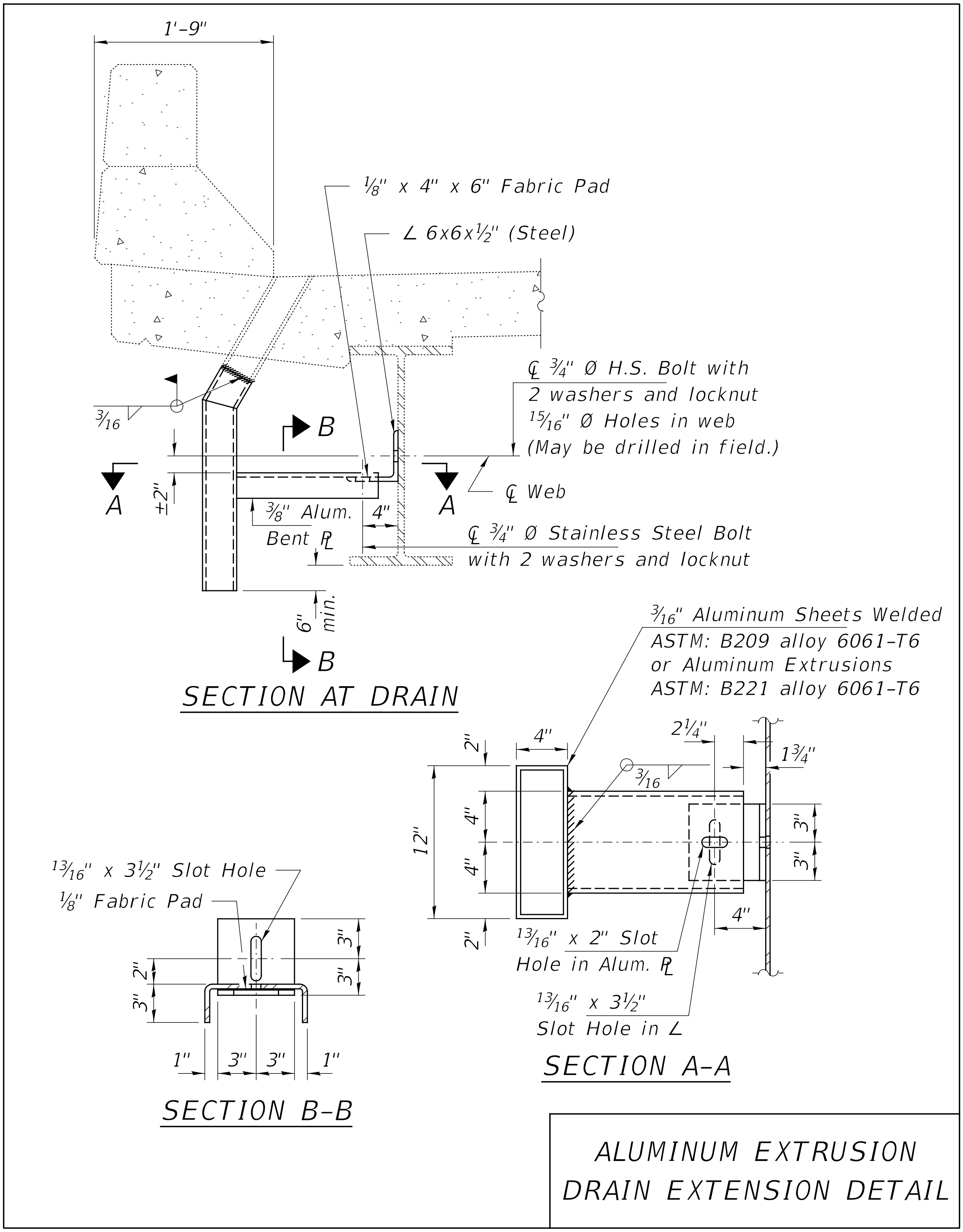
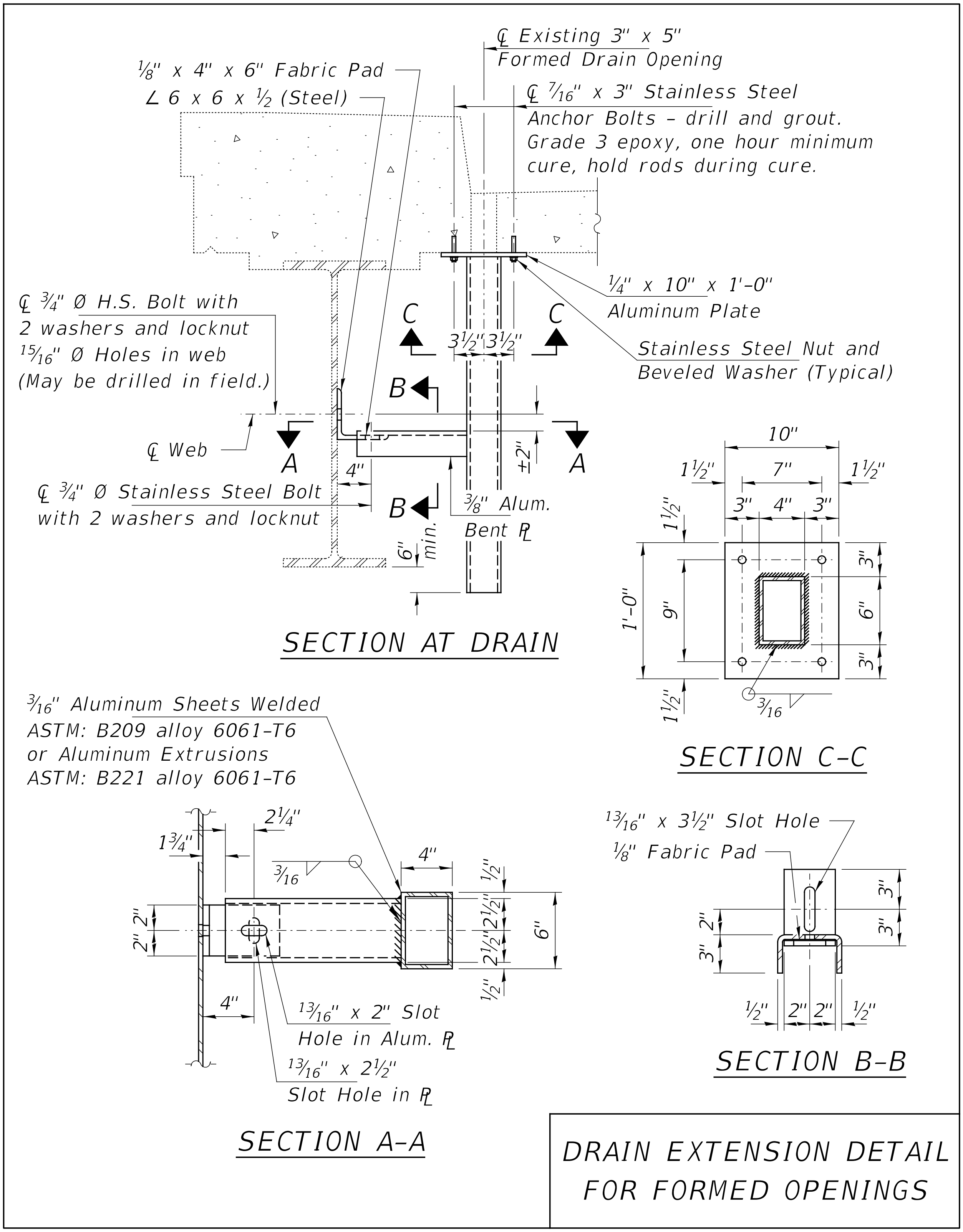

1'–9"
⅛" x 4" x 6" Fabric Pad
∠ 6x6x½" (Steel)
℄ ¾" Ø H.S. Bolt with
2 washers and locknut
15⁄16" Ø Holes in web
(May be drilled in field.)
℄ Web
3⁄16
B
A
±2"
⅜" Alum.
Bent PL
4"
A
℄ ¾" Ø Stainless Steel Bolt
with 2 washers and locknut
6" min.
B
SECTION AT DRAIN
3⁄16" Aluminum Sheets Welded
ASTM: B209 alloy 6061-T6
or Aluminum Extrusions
ASTM: B221 alloy 6061-T6
4"
2¼"
1¾"
2"
3⁄16
12"
4"
4"
3"
3"
2"
4"
13⁄16" x 2" Slot
Hole in Alum. PL
13⁄16" x 3½"
Slot Hole in ∠
SECTION A-A
13⁄16" x 3½" Slot Hole
⅛" Fabric Pad
2"
3"
3"
3"
1"
3"
3"
1"
SECTION B-B
ALUMINUM EXTRUSION
DRAIN EXTENSION DETAIL

FIGURE 2.8.2-3



℄ Existing 3" x 5"
Formed Drain Opening
⅛" x 4" x 6" Fabric Pad
∠ 6 x 6 x ½ (Steel)
℄ 7⁄16" x 3" Stainless Steel
Anchor Bolts – drill and grout.
Grade 3 epoxy, one hour minimum
cure, hold rods during cure.
¼" x 10" x 1'-0"
Aluminum Plate
℄ ¾" Ø H.S. Bolt with
2 washers and locknut
15⁄16" Ø Holes in web
(May be drilled in field.)
Stainless Steel Nut and
Beveled Washer (Typical)
℄ Web
℄ ¾" Ø Stainless Steel Bolt
with 2 washers and locknut
⅜" Alum.
Bent PL
6" min.
±2"
SECTION AT DRAIN
SECTION C-C
3⁄16" Aluminum Sheets Welded
ASTM: B209 alloy 6061-T6
or Aluminum Extrusions
ASTM: B221 alloy 6061-T6
13⁄16" x 2" Slot
Hole in Alum. PL
13⁄16" x 2½"
Slot Hole in PL
SECTION A-A
13⁄16" x 3½" Slot Hole
⅛" Fabric Pad
SECTION B-B
DRAIN EXTENSION DETAIL
FOR FORMED OPENINGS

FIGURE 2.8.2-4

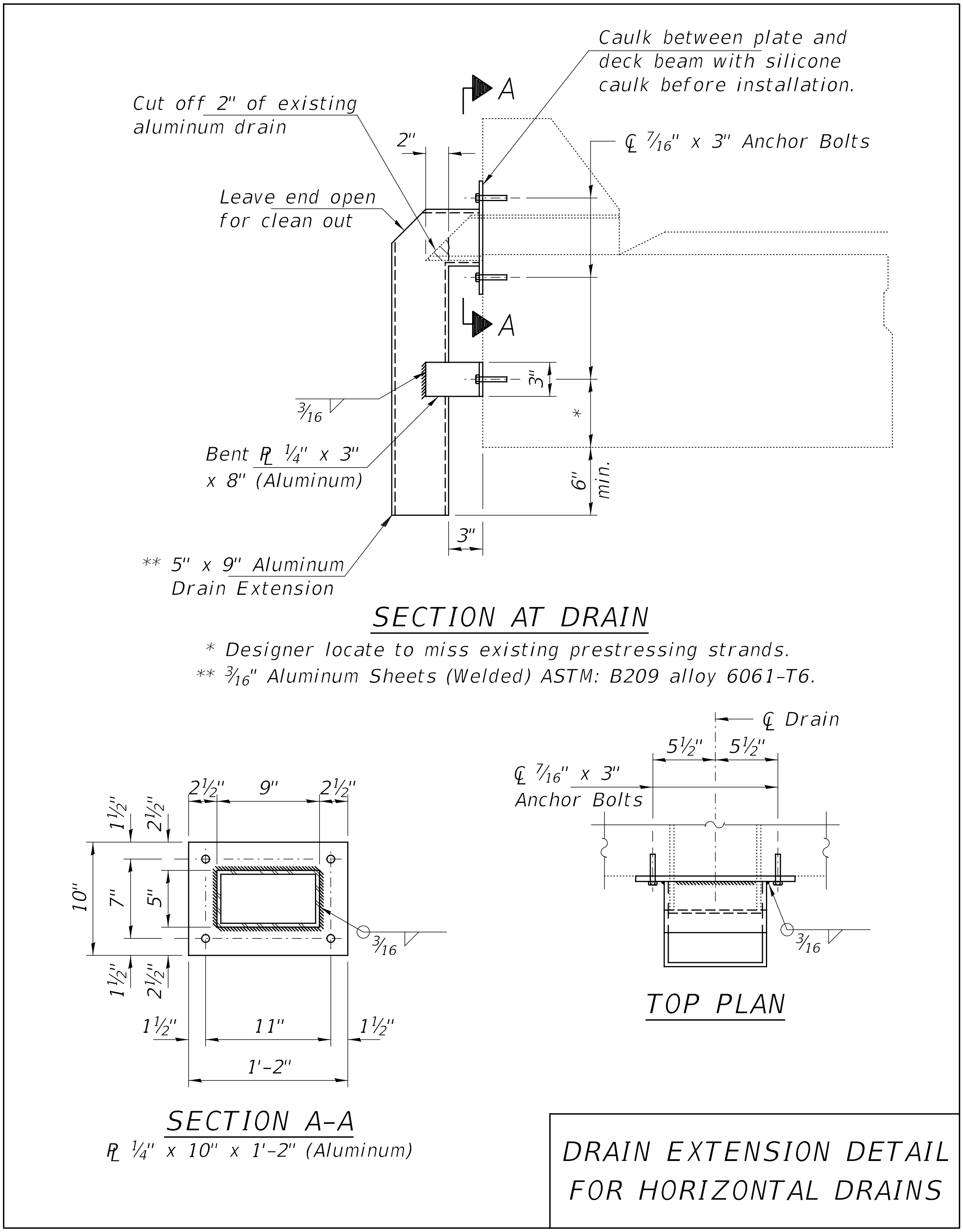

Caulk between plate and deck beam with silicone caulk before installation.
Cut off 2" of existing aluminum drain
A
2"
℄ 7/16" x 3" Anchor Bolts
Leave end open for clean out
A
3"
3/16
*
Bent PL 1/4" x 3" x 8" (Aluminum)
6" min.
3"
** 5" x 9" Aluminum Drain Extension
SECTION AT DRAIN
* Designer locate to miss existing prestressing strands.
** 3/16" Aluminum Sheets (Welded) ASTM: B209 alloy 6061-T6.
℄ Drain
5 1/2" 5 1/2"
℄ 7/16" x 3" Anchor Bolts
2 1/2" 9" 2 1/2"
1 1/2" 2 1/2"
10" 7" 5"
3/16
1 1/2" 2 1/2"
1 1/2" 11" 1 1/2"
1'-2"
3/16
TOP PLAN
SECTION A-A
PL 1/4" x 10" x 1'-2" (Aluminum)
DRAIN EXTENSION DETAIL FOR HORIZONTAL DRAINS

FIGURE 2.8.2-5

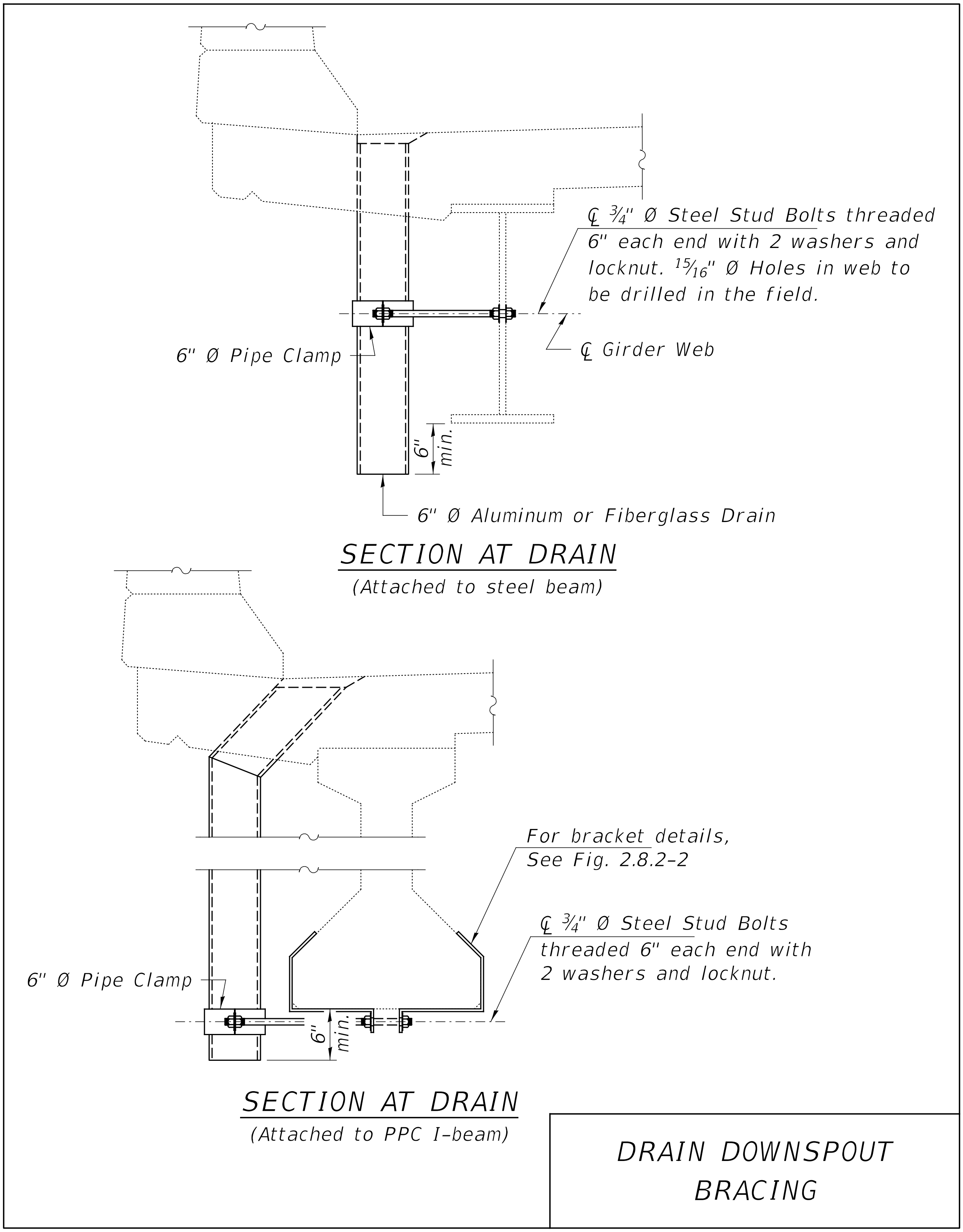

℄ 3/4" Ø Steel Stud Bolts threaded 6" each end with 2 washers and locknut. 15/16" Ø Holes in web to be drilled in the field.
℄ Girder Web
6" Ø Pipe Clamp
6" min.
6" Ø Aluminum or Fiberglass Drain
SECTION AT DRAIN
(Attached to steel beam)
For bracket details, See Fig. 2.8.2-2
℄ 3/4" Ø Steel Stud Bolts threaded 6" each end with 2 washers and locknut.
6" Ø Pipe Clamp
6" min.
SECTION AT DRAIN
(Attached to PPC I-beam)
DRAIN DOWNSPOUT BRACING

FIGURE 2.8.2-6

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.

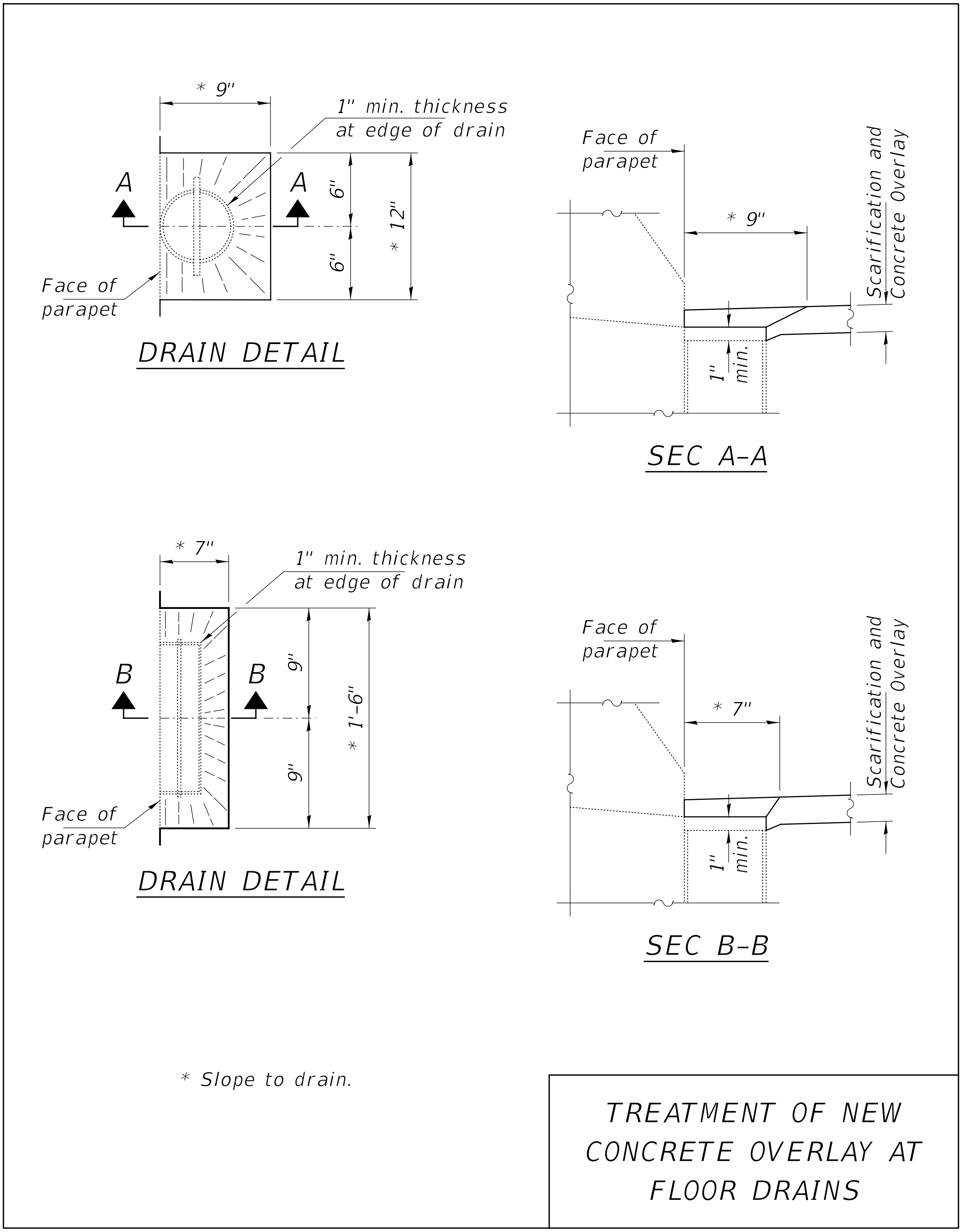

* 9"
1" min. thickness
at edge of drain
A
A
6"
6"
* 12"
Face of
parapet
DRAIN DETAIL
Face of
parapet
* 9"
Scarification and
Concrete Overlay
1"
min.
SEC A–A
* 7"
1" min. thickness
at edge of drain
B
B
9"
9"
* 1'-6"
Face of
parapet
DRAIN DETAIL
Face of
parapet
* 7"
Scarification and
Concrete Overlay
1"
min.
SEC B–B
* Slope to drain.
TREATMENT OF NEW
CONCRETE OVERLAY AT
FLOOR DRAINS

FIGURE 2.8.3-1

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.

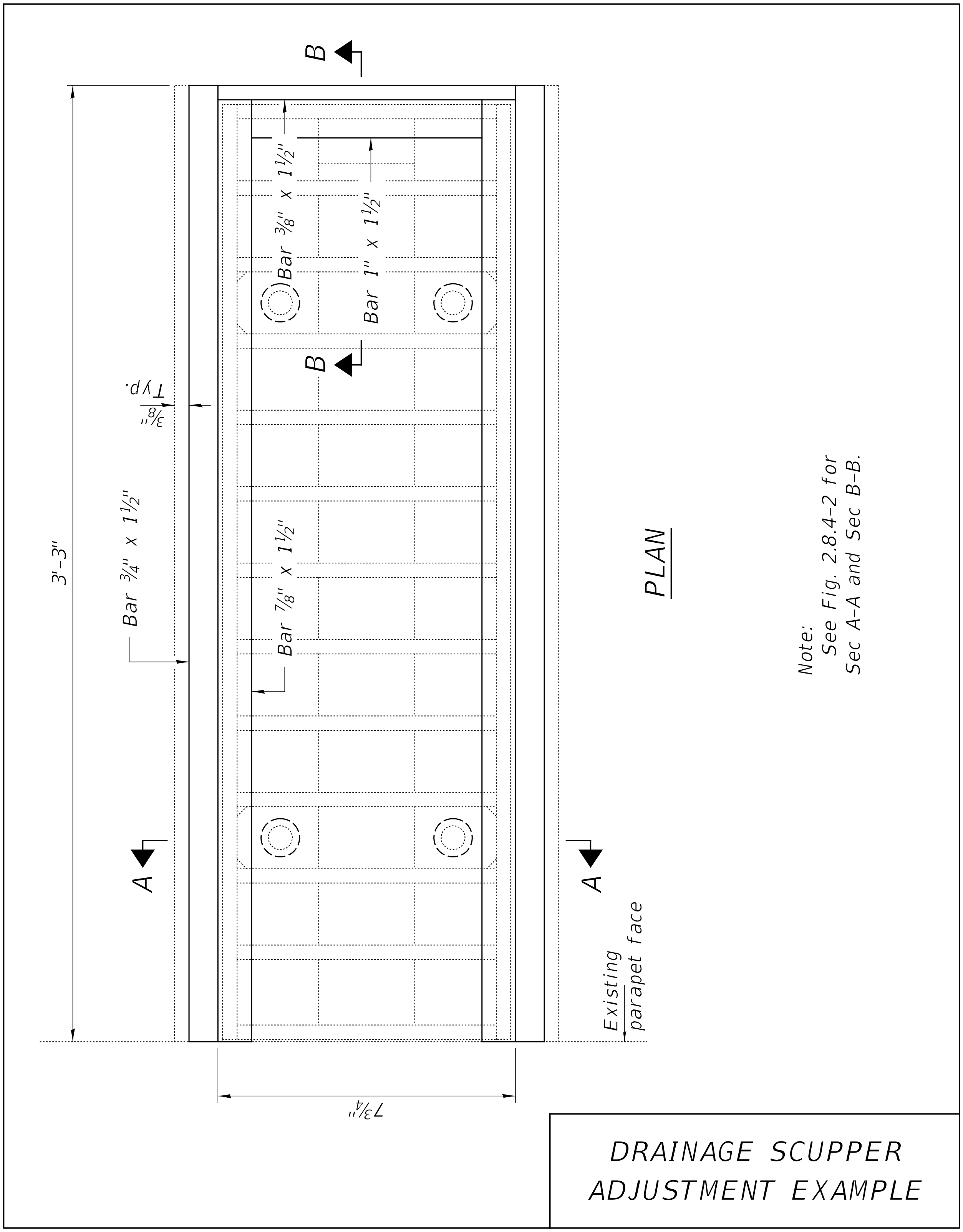

B
Bar 3/8" x 1½"
Bar 1" x 1½"
3'-3"
Bar 3/4" x 1½"
3/8" Typ.
Bar 7/8" x 1½"
A
Existing parapet face
7¾"
PLAN
Note:
See Fig. 2.8.4-2 for
Sec A-A and Sec B-B.
DRAINAGE SCUPPER
ADJUSTMENT EXAMPLE

FIGURE 2.8.4-1

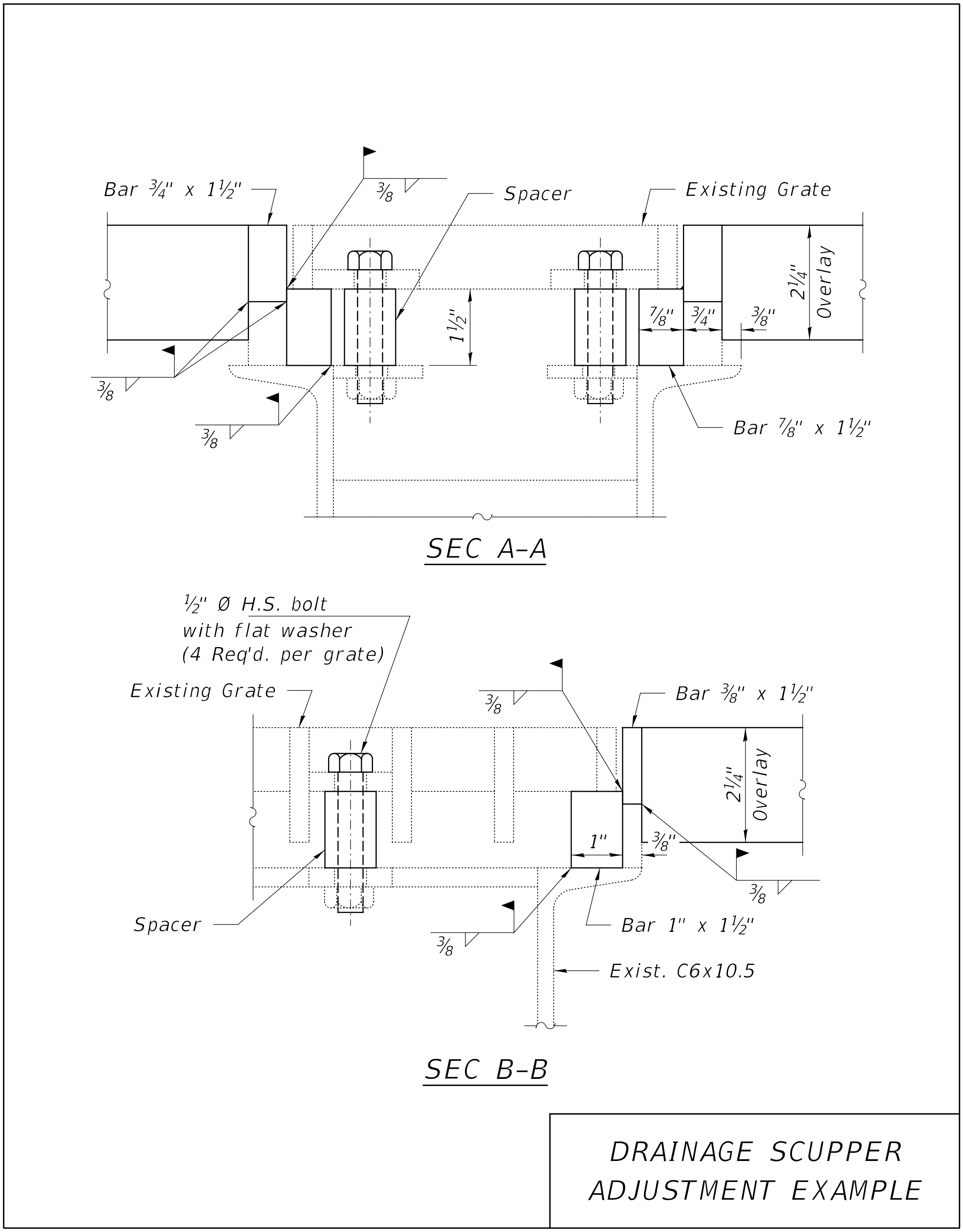

Bar 3/4" x 1 1/2"
3/8
Spacer
Existing Grate
1 1/2"
7/8"
3/4"
3/8"
2 1/4"
Overlay
3/8
3/8
Bar 7/8" x 1 1/2"
SEC A–A
1/2" Ø H.S. bolt
with flat washer
(4 Req'd. per grate)
Existing Grate
3/8
Bar 3/8" x 1 1/2"
2 1/4"
Overlay
1"
3/8"
3/8
Spacer
3/8
Bar 1" x 1 1/2"
Exist. C6x10.5
SEC B–B
DRAINAGE SCUPPER
ADJUSTMENT EXAMPLE

FIGURE 2.8.4-2

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.

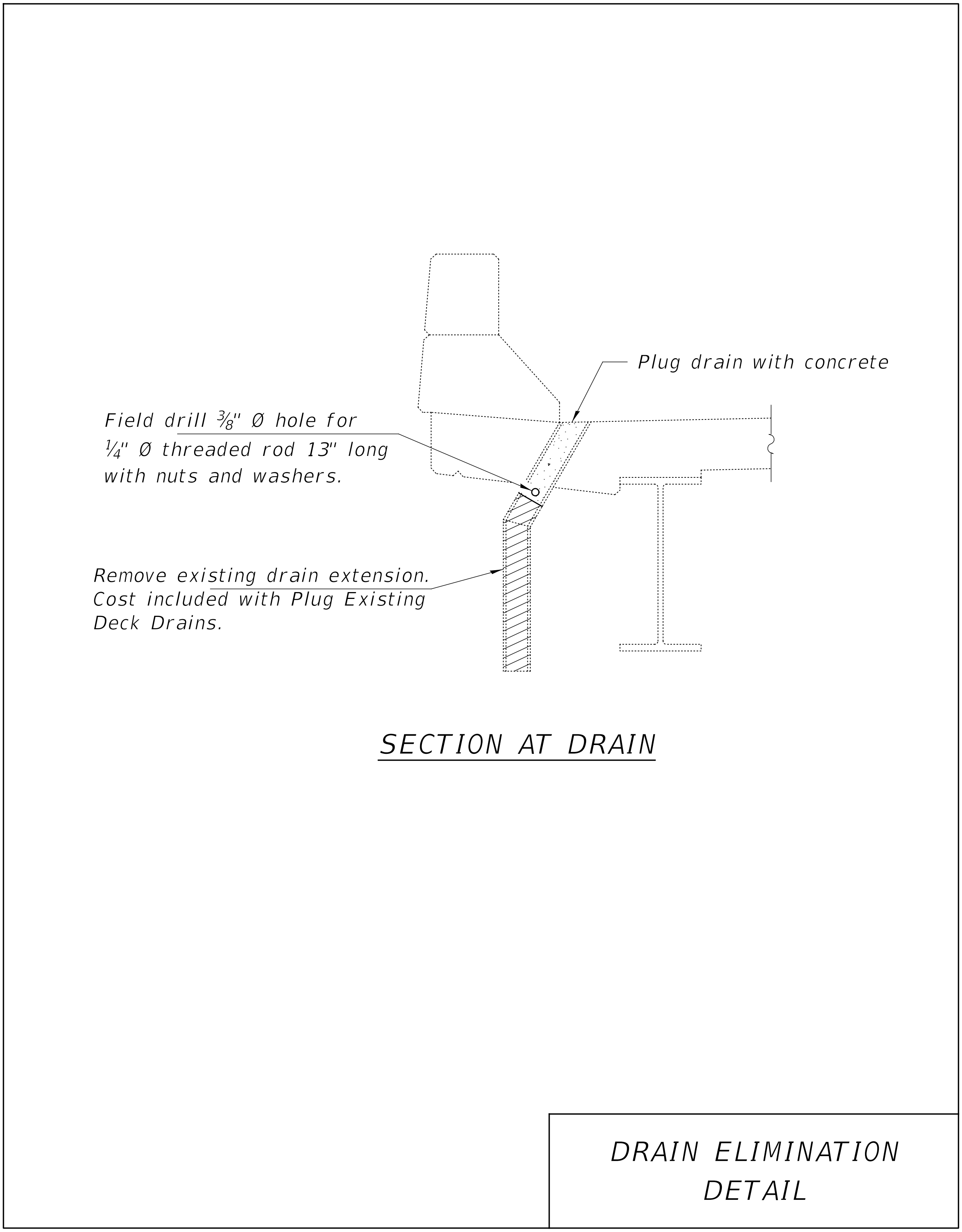
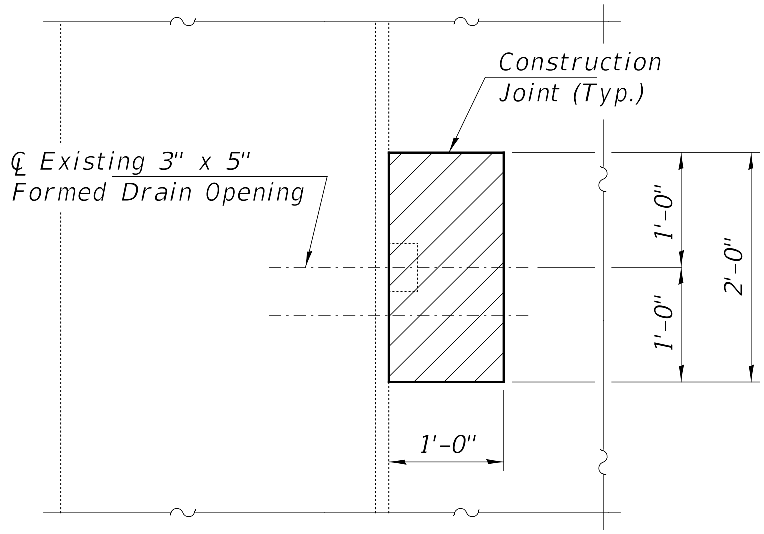
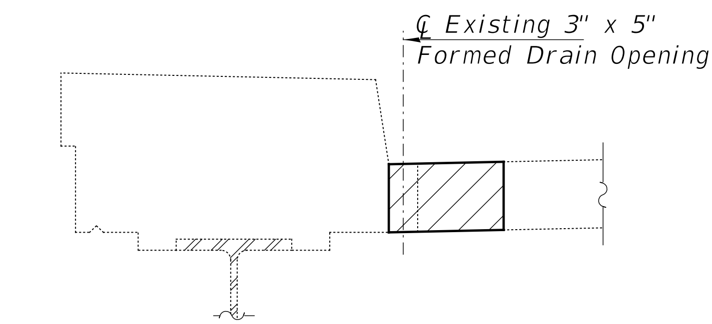

Plug drain with concrete
Field drill ⅜" Ø hole for ¼" Ø threaded rod 13" long with nuts and washers.
Remove existing drain extension. Cost included with Plug Existing Deck Drains.
SECTION AT DRAIN
DRAIN ELIMINATION DETAIL

FIGURE 2.8.5-1



PARTIAL PLAN

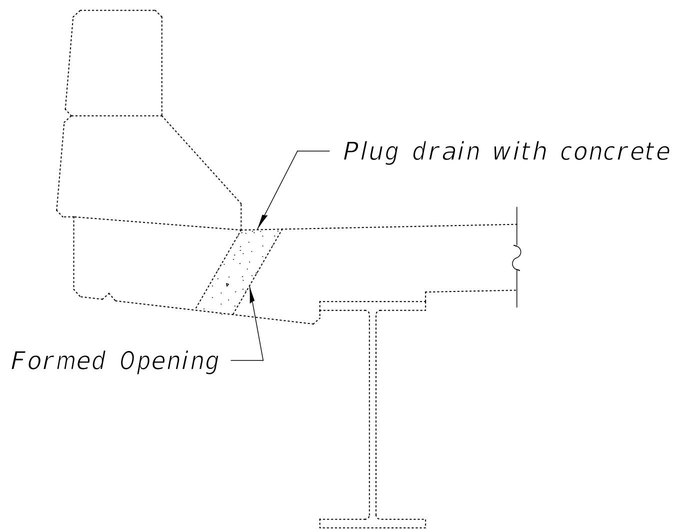


SECTION AT DRAIN

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.

DRAIN ELIMINATION DETAIL

FIGURE 2.8.5-2



SECTION AT DRAIN

DRAIN ELIMINATION DETAIL

FIGURE 2.8.5-3

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.

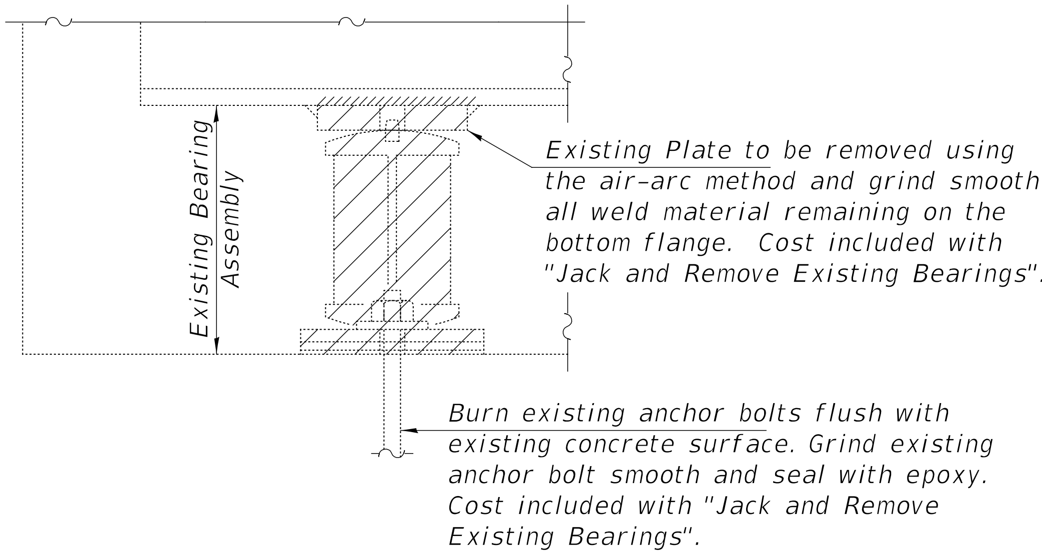
Fixed bearing design and details can be found in section 3.7.1.2 of the IDOT *Bridge Manual*.

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.



EXISTING BEARING REMOVAL DETAIL

EXISTING STEEL ROCKER BEARING REMOVAL

FIGURE 2.9.3-1

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.

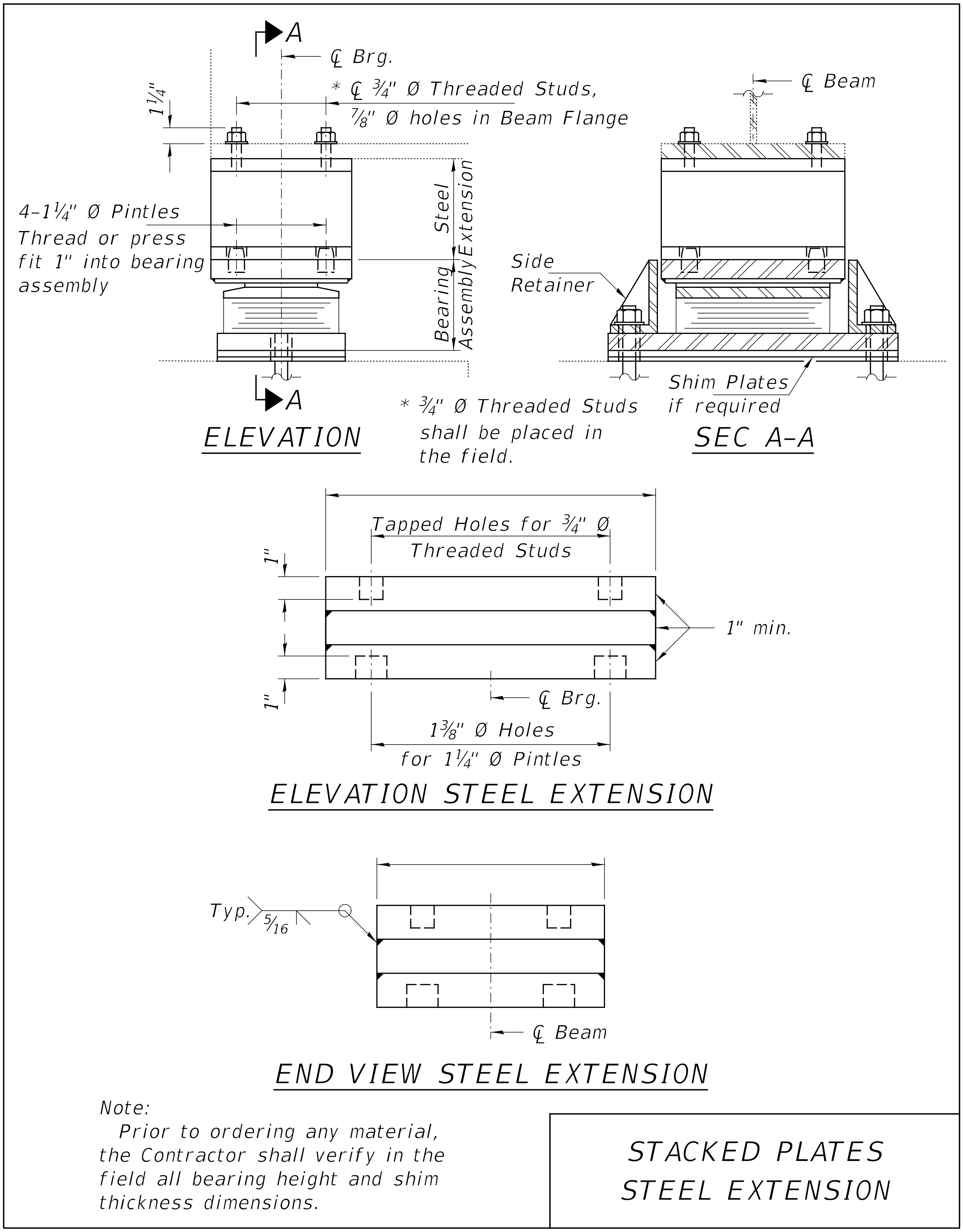

A
℄ Brg.
* ℄ 3/4" Ø Threaded Studs,
7/8" Ø holes in Beam Flange
1 1/4"
4-1 1/4" Ø Pintles
Thread or press
fit 1" into bearing
assembly
Steel
Extension
Bearing
Assembly
A
ELEVATION
* 3/4" Ø Threaded Studs
shall be placed in
the field.
℄ Beam
Side
Retainer
Shim Plates
if required
SEC A-A
Tapped Holes for 3/4" Ø
Threaded Studs
1"
1" min.
1"
℄ Brg.
1 3/8" Ø Holes
for 1 1/4" Ø Pintles
ELEVATION STEEL EXTENSION
Typ.
5/16
℄ Beam
END VIEW STEEL EXTENSION
Note:
Prior to ordering any material,
the Contractor shall verify in the
field all bearing height and shim
thickness dimensions.
STACKED PLATES
STEEL EXTENSION

FIGURE 2.9.4-1

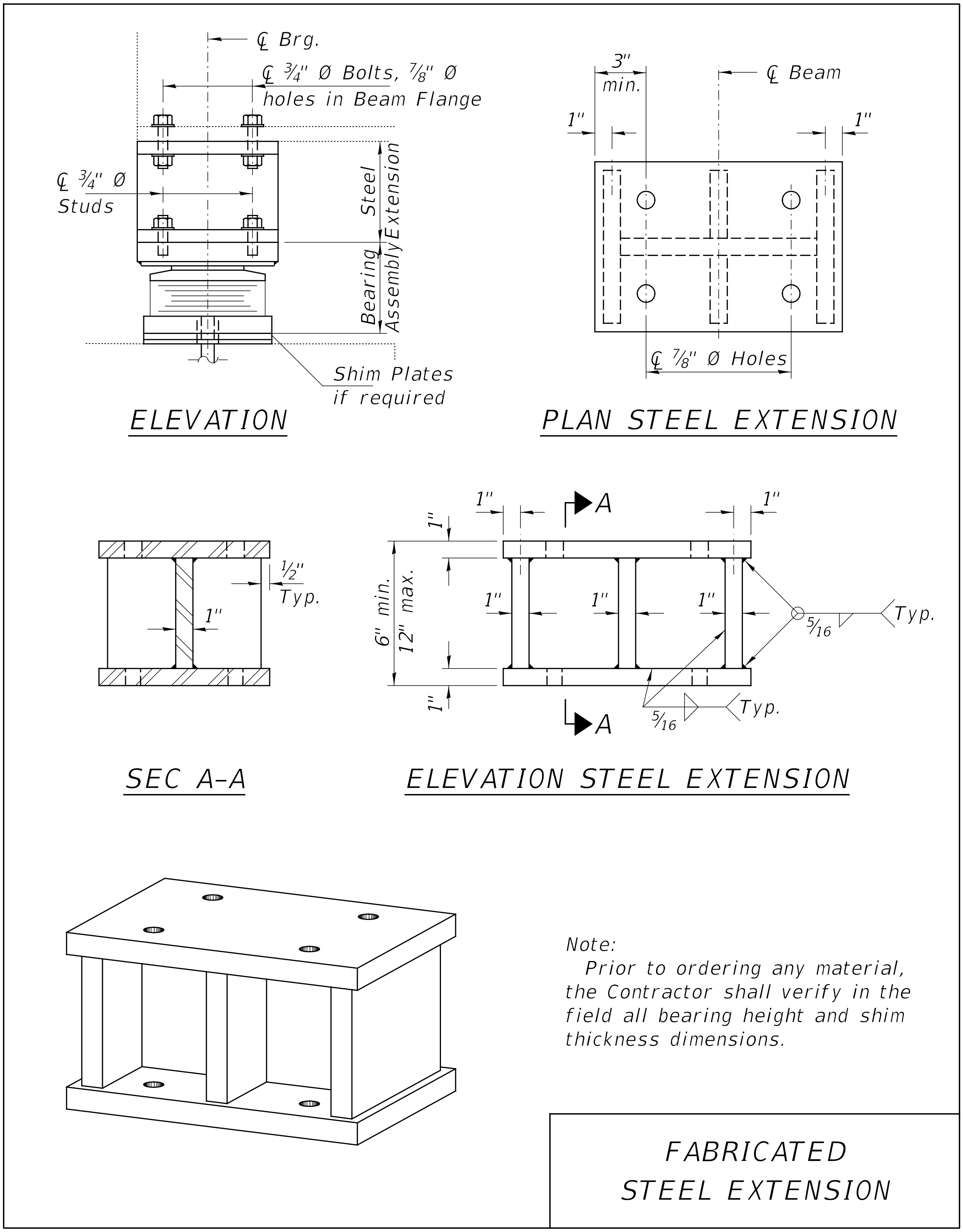

℄ Brg.
℄ 3/4" Ø Bolts, 7/8" Ø holes in Beam Flange
℄ 3/4" Ø Studs
Steel Extension
Bearing Assembly
Shim Plates if required
ELEVATION
3" min.
℄ Beam
1"
1"
℄ 7/8" Ø Holes
PLAN STEEL EXTENSION
1/2" Typ.
1"
SEC A–A
A
A
1"
1"
1"
1"
6" min.
12" max.
1"
1"
1"
5/16
Typ.
5/16
Typ.
ELEVATION STEEL EXTENSION
Note:
Prior to ordering any material, the Contractor shall verify in the field all bearing height and shim thickness dimensions.
FABRICATED STEEL EXTENSION

FIGURE 2.9.4-2

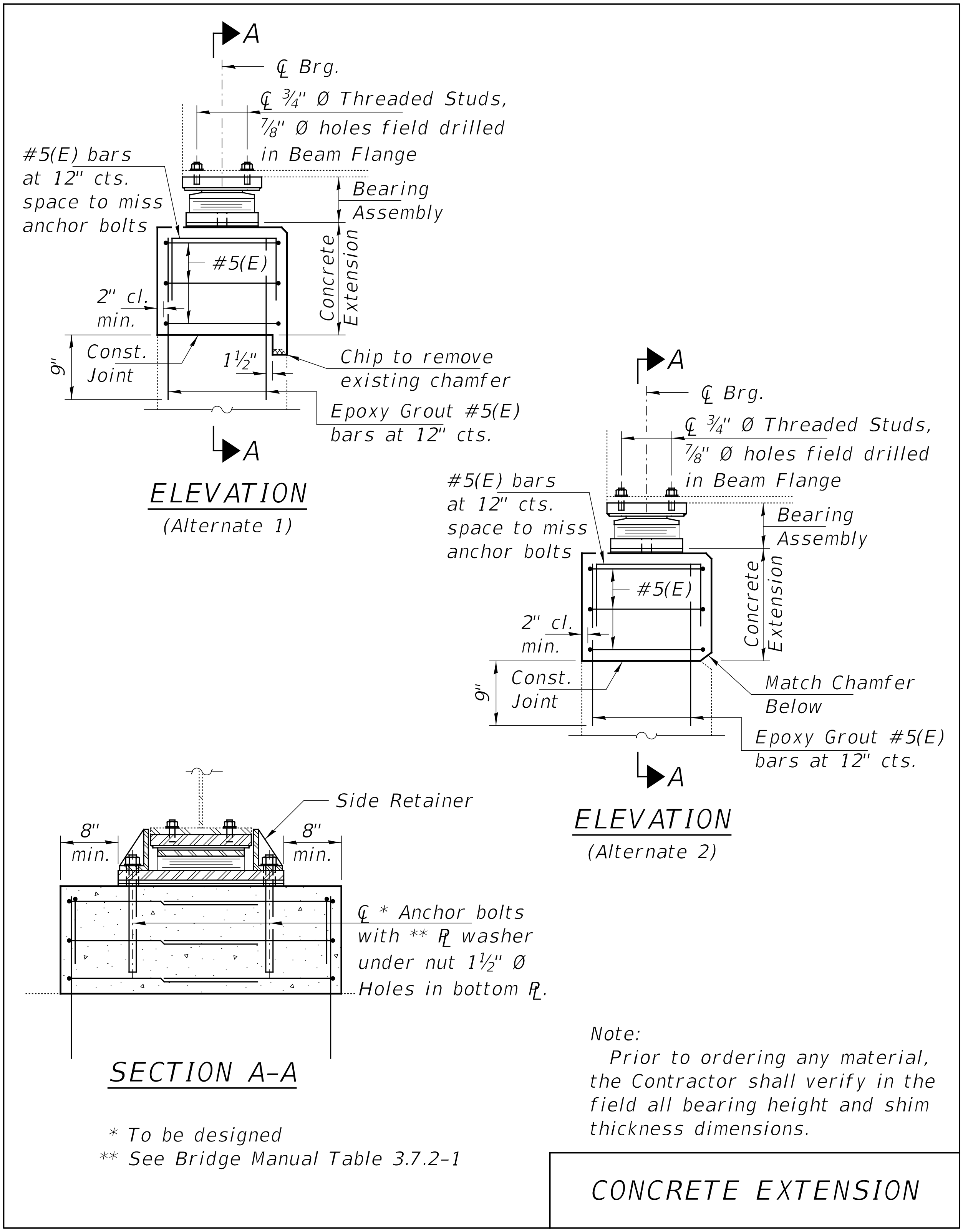

A
℄ Brg.
℄ 3/4" Ø Threaded Studs,
7/8" Ø holes field drilled
in Beam Flange
#5(E) bars
at 12" cts.
space to miss
anchor bolts
Bearing
Assembly
#5(E)
Concrete
Extension
2" cl.
min.
9"
Const.
Joint
1½"
Chip to remove
existing chamfer
Epoxy Grout #5(E)
bars at 12" cts.
ELEVATION
(Alternate 1)
Match Chamfer
Below
ELEVATION
(Alternate 2)
Side Retainer
8"
min.
℄ * Anchor bolts
with ** ℙ washer
under nut 1½" Ø
Holes in bottom ℙ.
SECTION A–A
* To be designed
** See Bridge Manual Table 3.7.2–1
Note:
Prior to ordering any material, the Contractor shall verify in the field all bearing height and shim thickness dimensions.
CONCRETE EXTENSION

FIGURE 2.9.4-3

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.

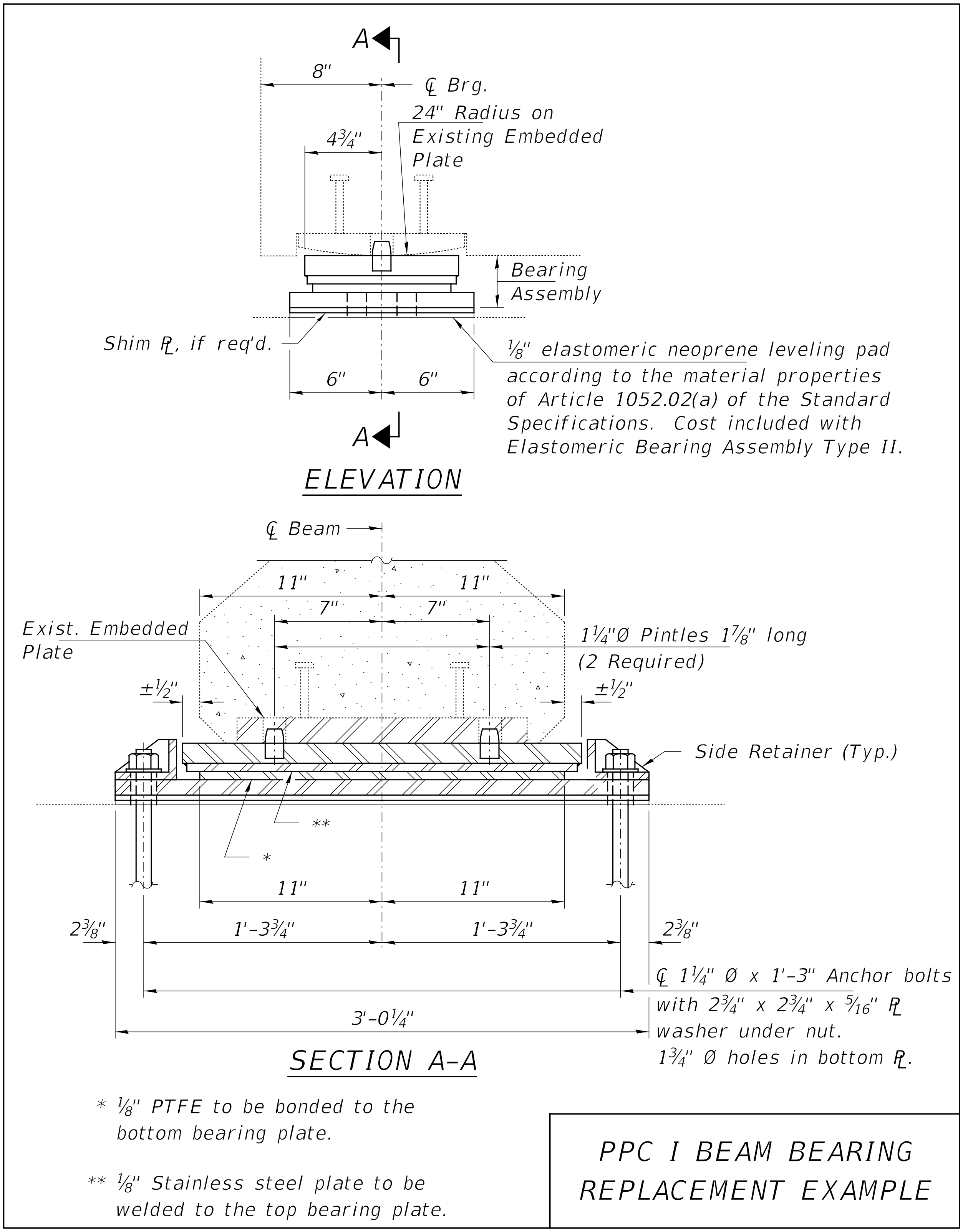

A
8"
℄ Brg.
24" Radius on Existing Embedded Plate
4¾"
Bearing Assembly
Shim ℔, if req'd.
⅛" elastomeric neoprene leveling pad according to the material properties of Article 1052.02(a) of the Standard Specifications. Cost included with Elastomeric Bearing Assembly Type II.
6"
6"
A
ELEVATION
℄ Beam
11"
11"
7"
7"
Exist. Embedded Plate
1¼"Ø Pintles 1⅞" long (2 Required)
±½"
±½"
Side Retainer (Typ.)
**
*
11"
11"
2⅜"
1'-3¾"
1'-3¾"
2⅜"
℄ 1¼" Ø x 1'-3" Anchor bolts with 2¾" x 2¾" x 5⁄16" ℔ washer under nut.
1¾" Ø holes in bottom ℔.
3'-0¼"
SECTION A-A
* ⅛" PTFE to be bonded to the bottom bearing plate.
** ⅛" Stainless steel plate to be welded to the top bearing plate.
PPC I BEAM BEARING REPLACEMENT EXAMPLE

FIGURE 2.9.5-1

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).

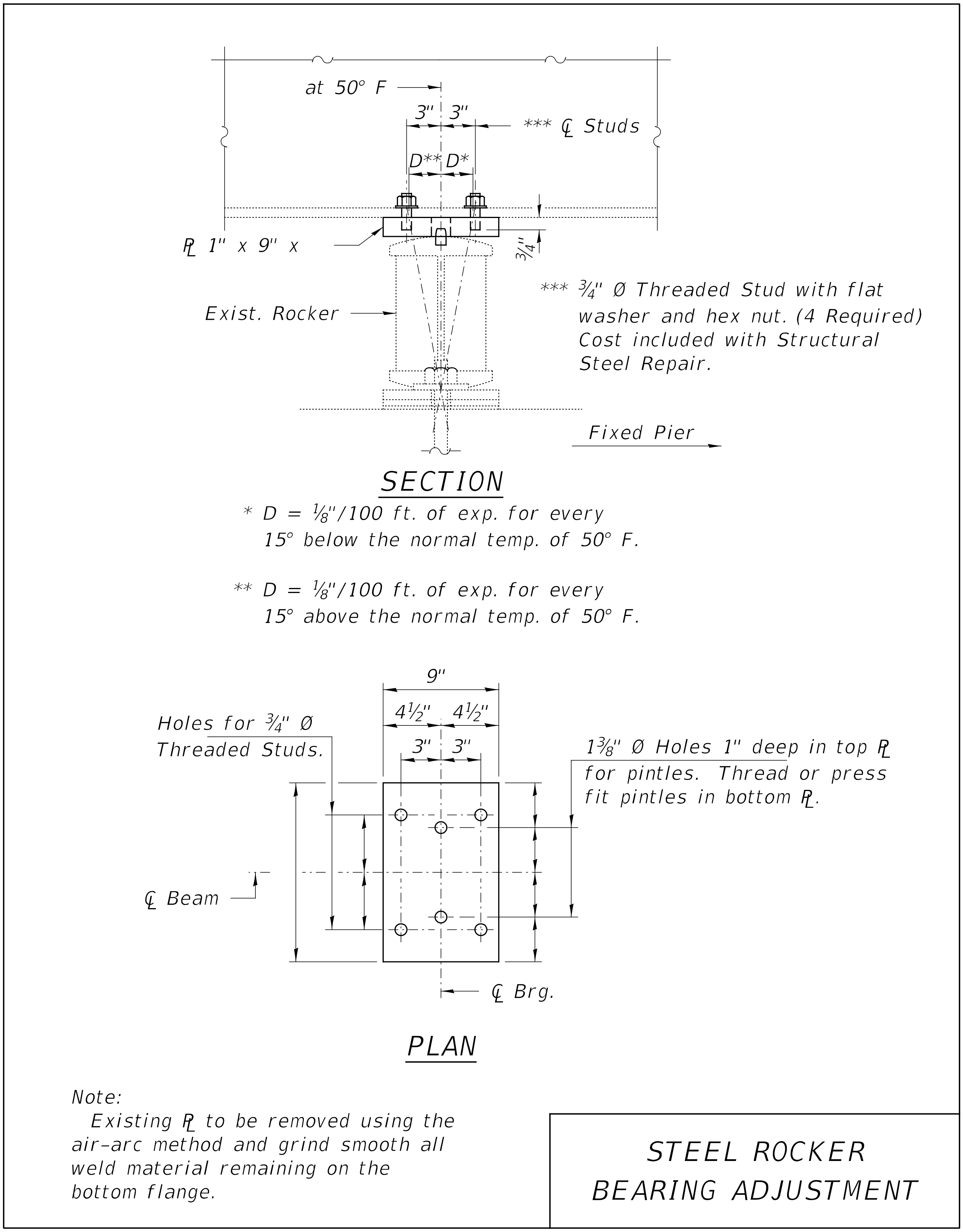

at 50° F
3" 3"
*** ℄ Studs
D** D*
PL 1" x 9" x
3/4"
*** 3/4" Ø Threaded Stud with flat washer and hex nut. (4 Required) Cost included with Structural Steel Repair.
Exist. Rocker
Fixed Pier
SECTION
* D = 1/8"/100 ft. of exp. for every 15° below the normal temp. of 50° F.
** D = 1/8"/100 ft. of exp. for every 15° above the normal temp. of 50° F.
9"
4 1/2" 4 1/2"
3" 3"
Holes for 3/4" Ø Threaded Studs.
1 3/8" Ø Holes 1" deep in top PL for pintles. Thread or press fit pintles in bottom PL.
℄ Beam
℄ Brg.
PLAN
Note:
Existing PL to be removed using the air-arc method and grind smooth all weld material remaining on the bottom flange.
STEEL ROCKER BEARING ADJUSTMENT

FIGURE 2.9.8-1

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

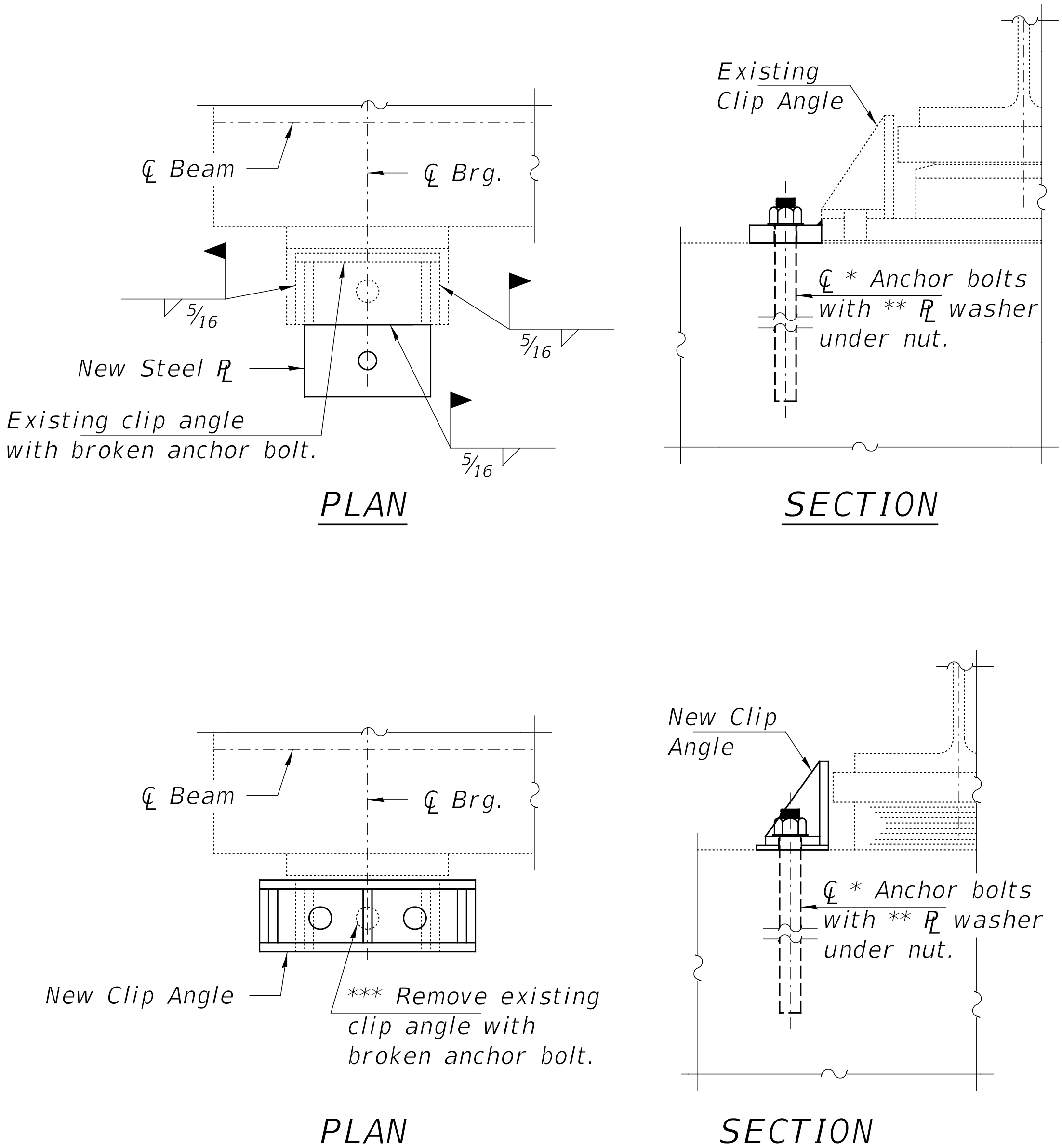
S = ((D1 + D2)/2) + 1 ¼”

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. Fig, 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.



* To be designed
** See Bridge Manual Table 3.7.2-1
*** If necessary, remove chunk of existing broken anchor bolt flush with top of concrete.

BROKEN ANCHOR
BOLT RETROFIT

FIGURE 2.9.9-1

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.

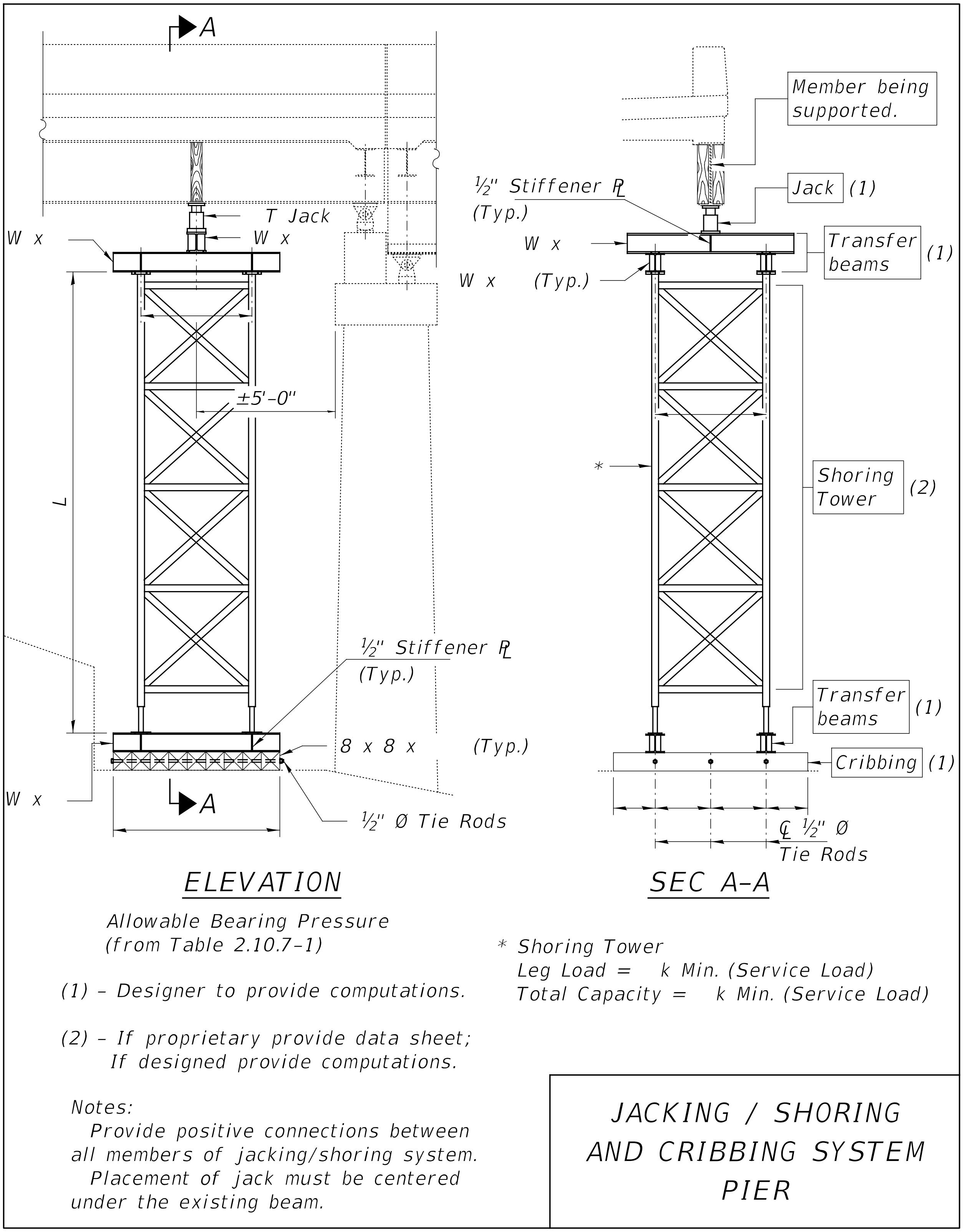


Figure 2.10.1-1

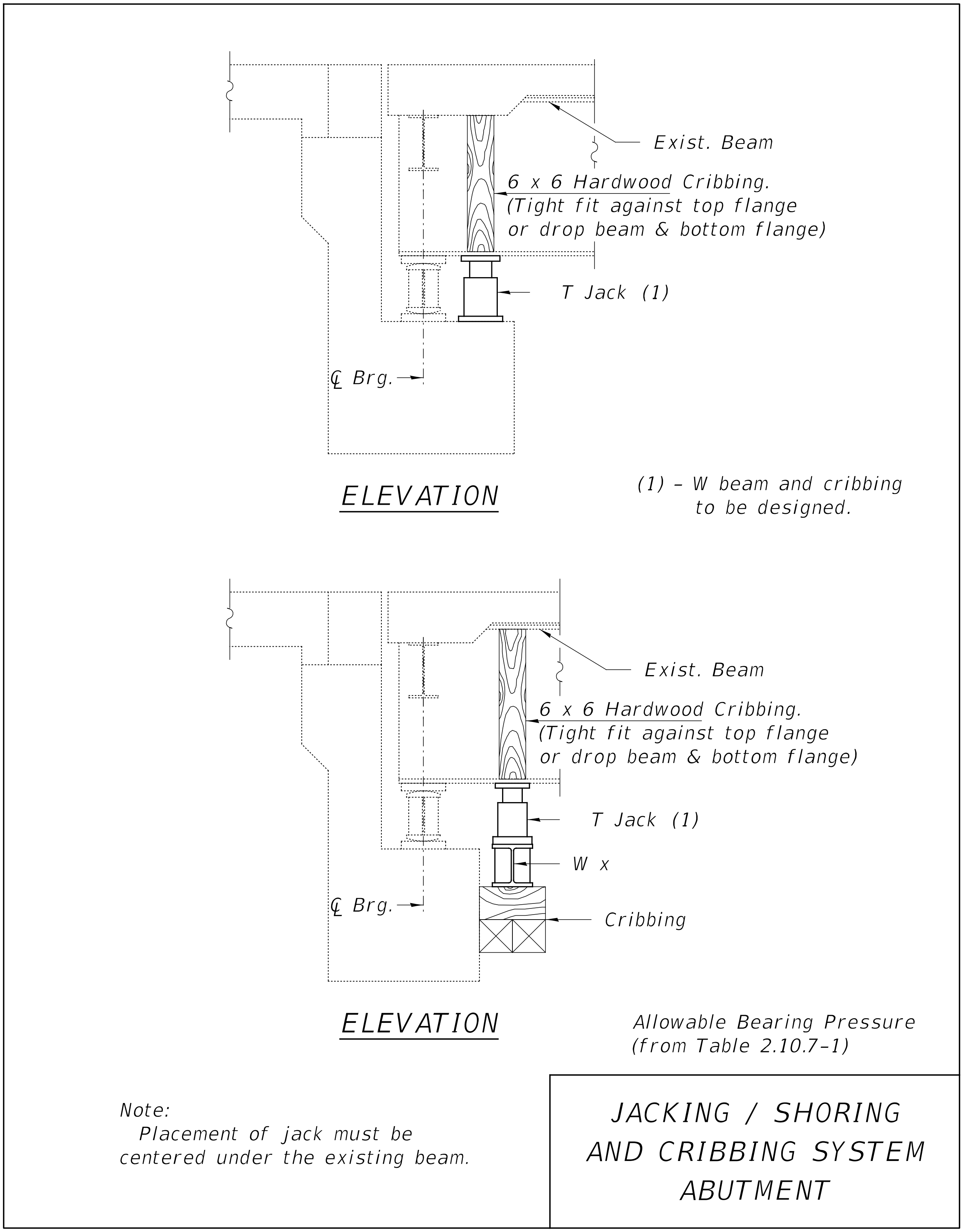


FIGURE 2.10.1-2

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 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. 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 Tons / Sq. Ft.
Conc. slopewalls & bit. Shoulders	1.0 Tons / 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

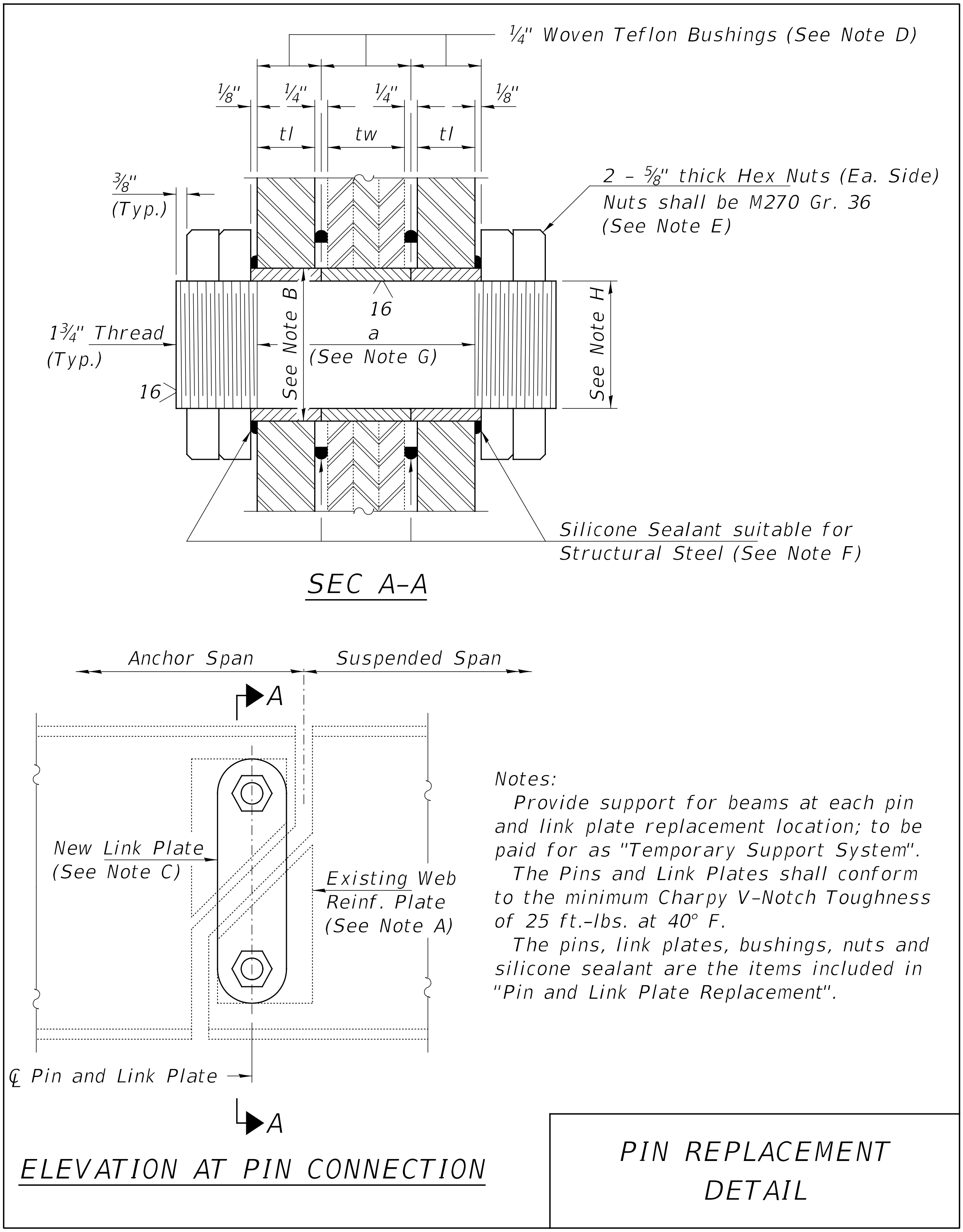


FIGURE 2.11.2-2

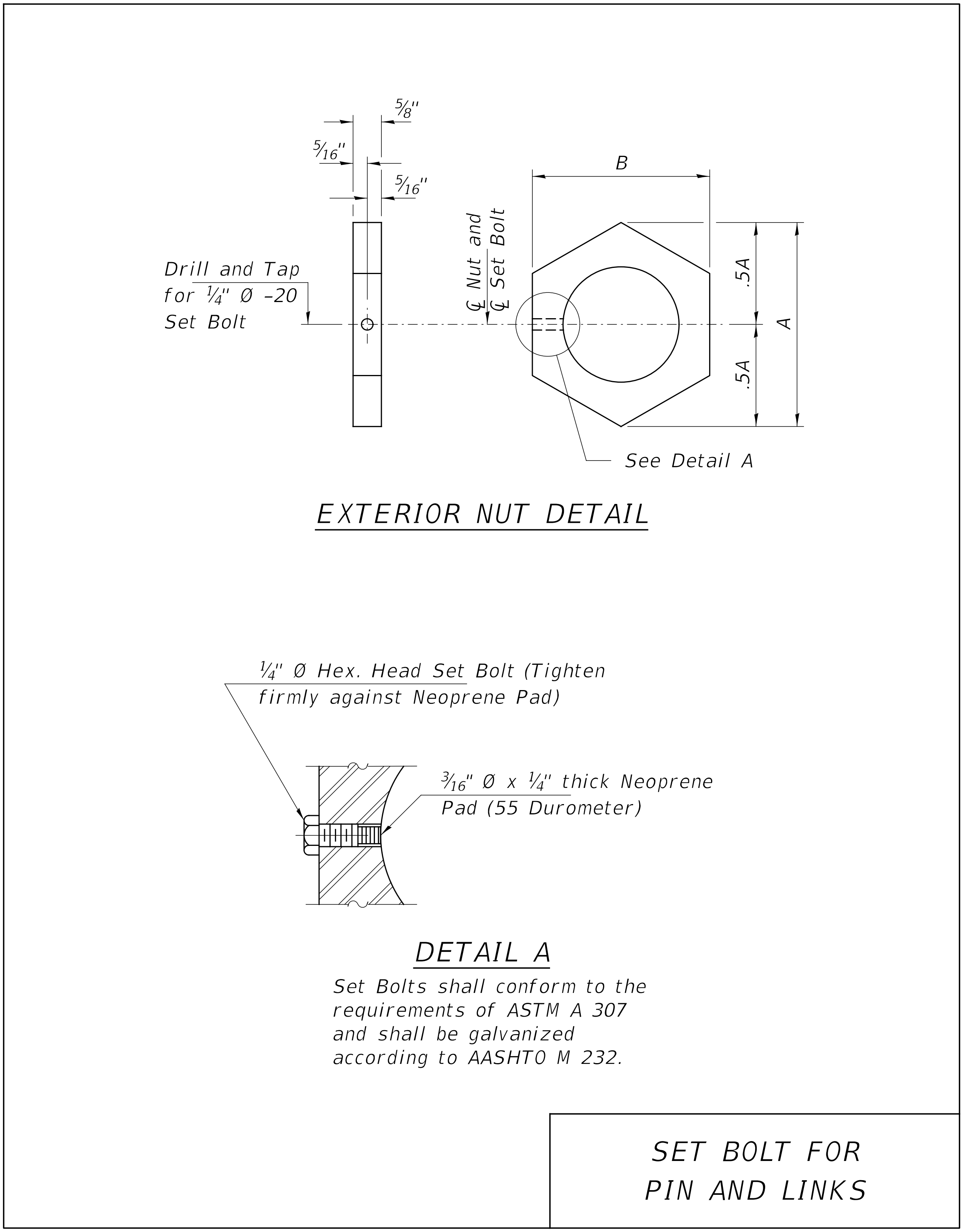


FIGURE 2.11.2-3

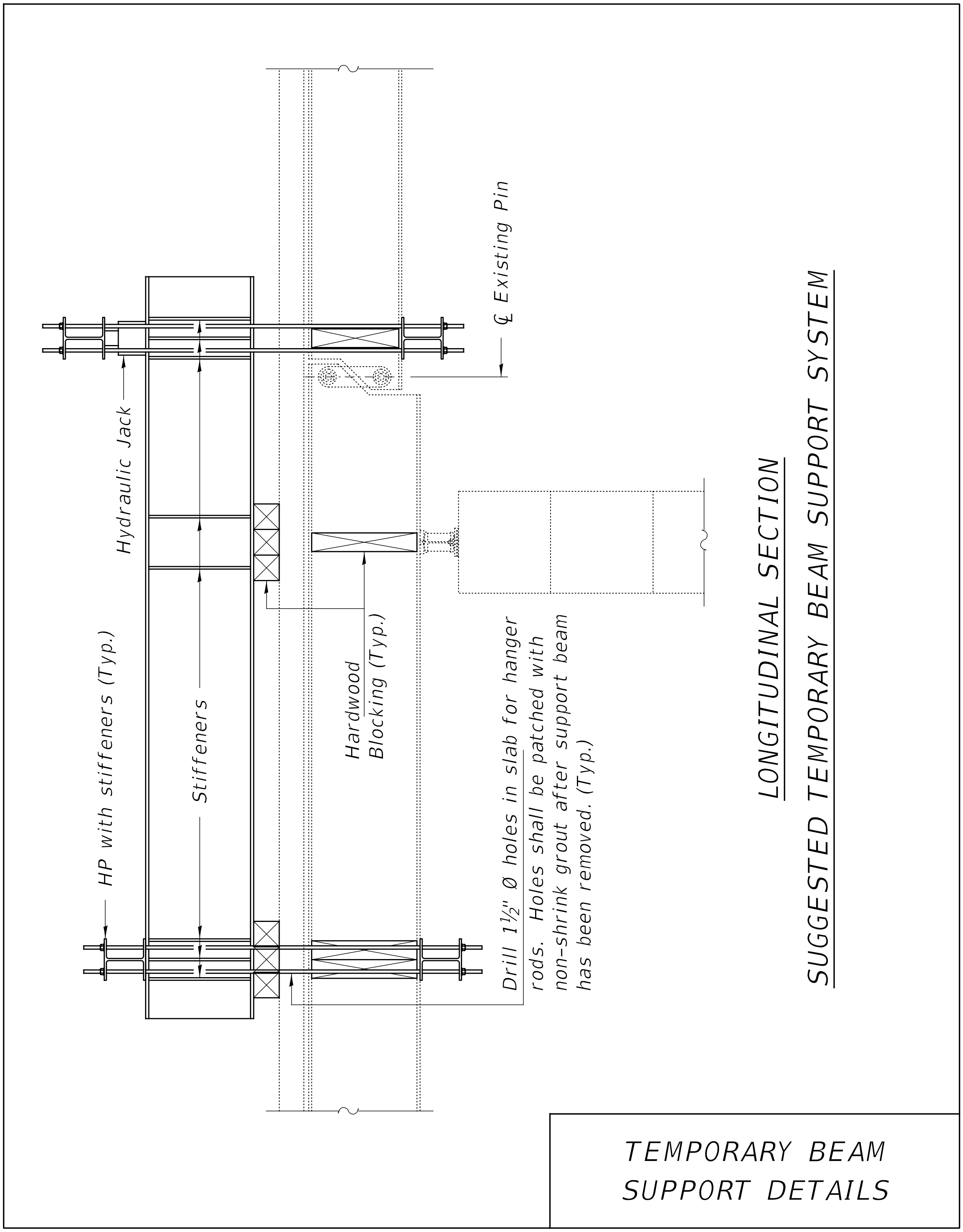


FIGURE 2.11.2-4

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.

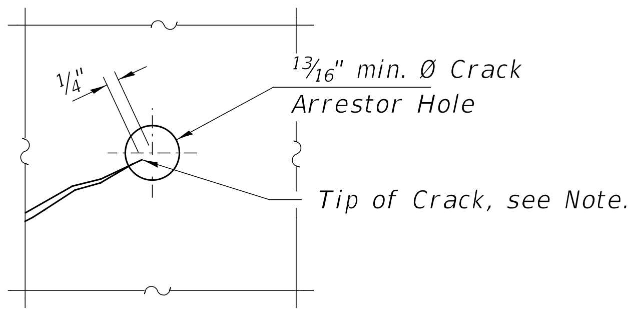
A common type of out-of-plane bending crack is found at the bottom of web stiffeners welded to the beam/girder web when there is a gap between the stiffener end and the top of the bottom flange. These cracks normally start at the tip of the stiffener and progress upwards along the stiffener to web weld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

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.

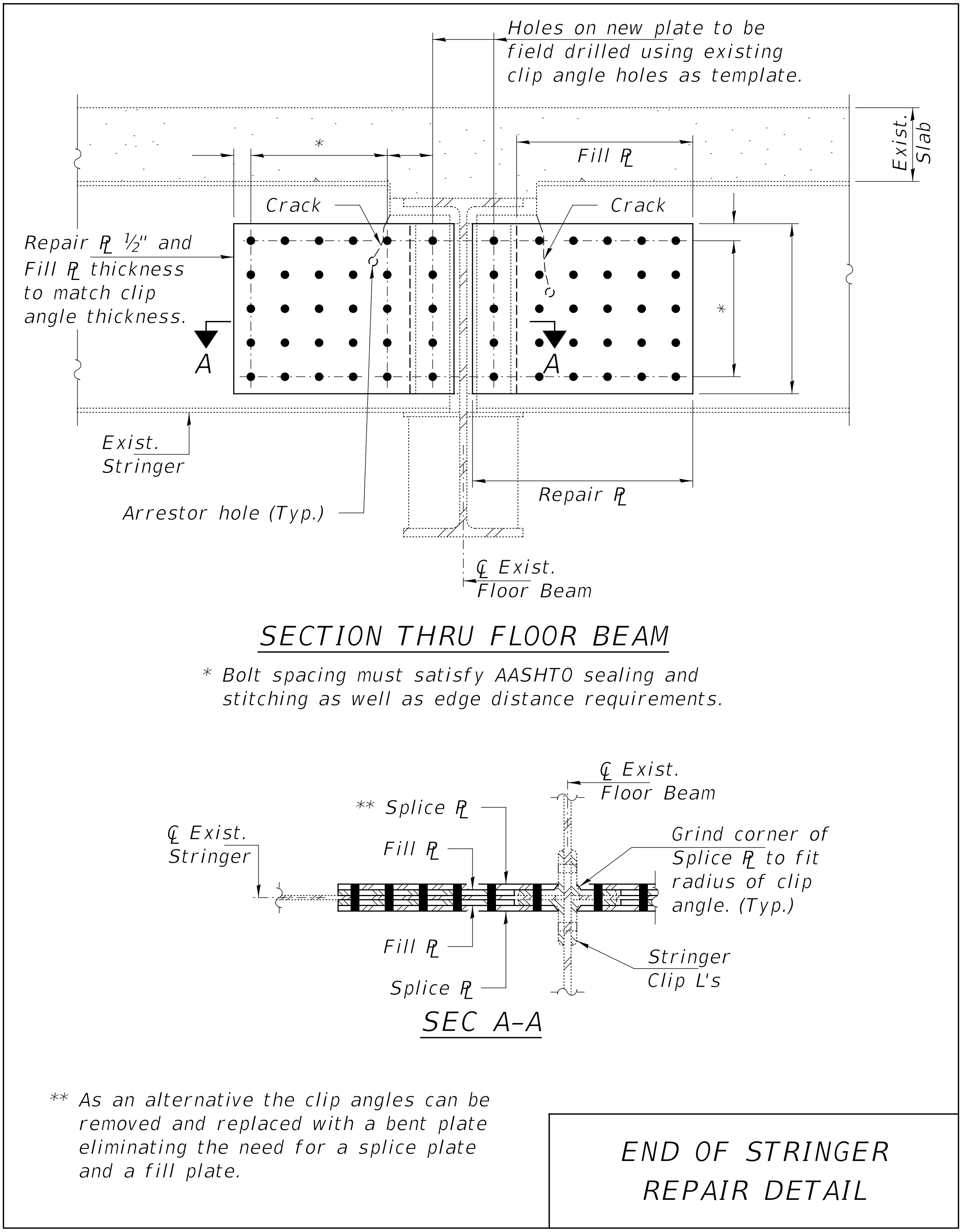


FIGURE 2.12.1.2-1

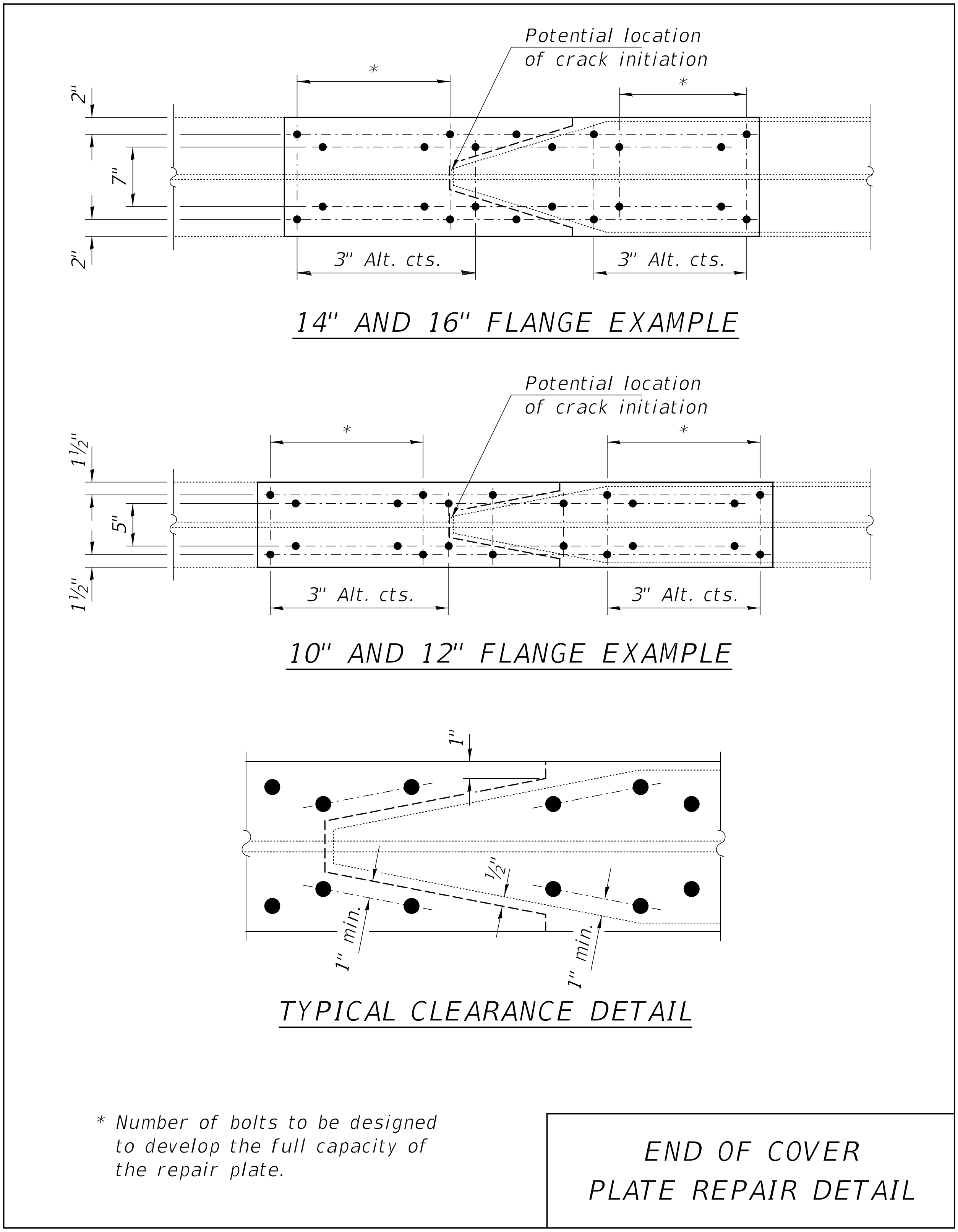


FIGURE 2.12.1.2-2

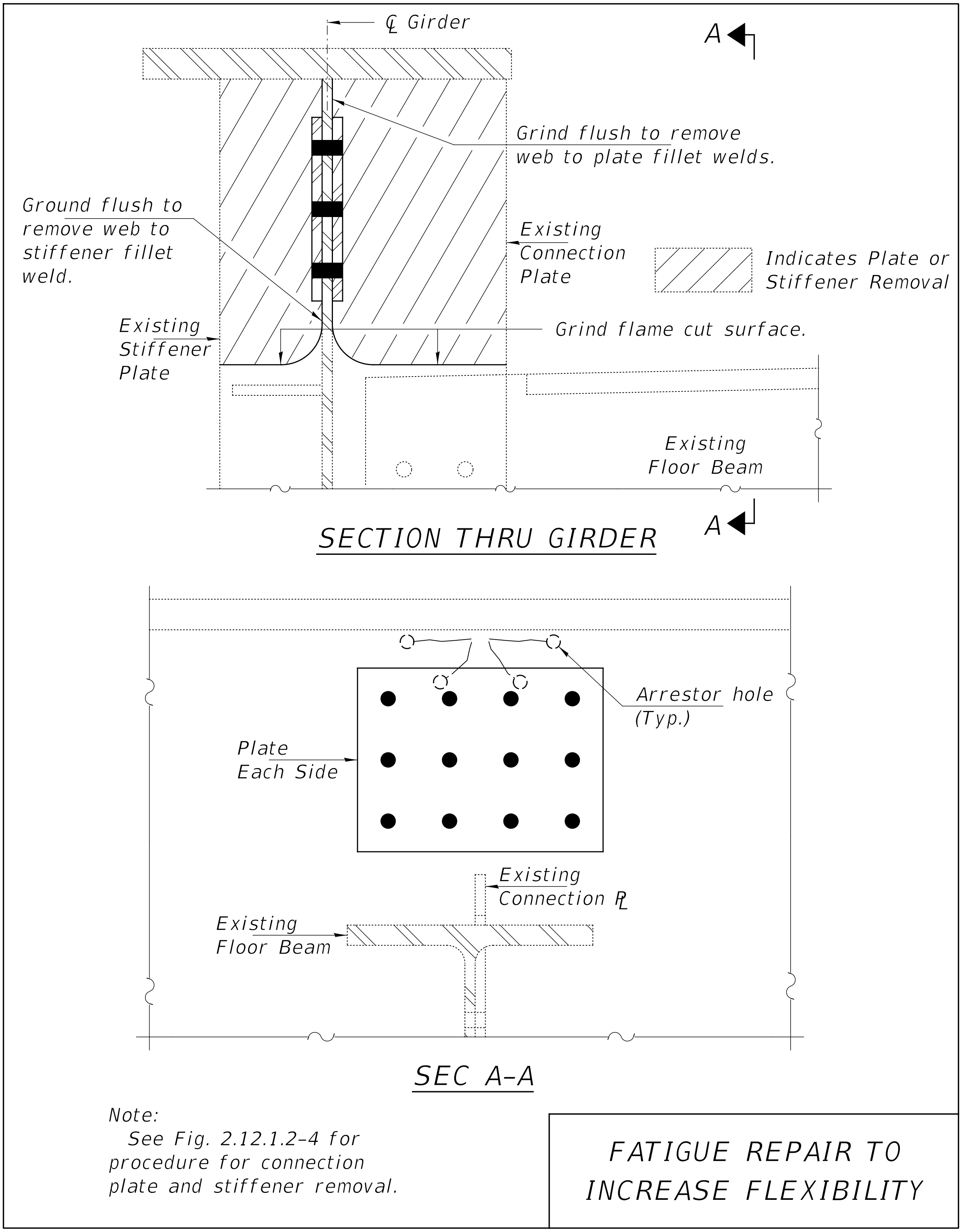


FIGURE 2.12.1.2-3

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

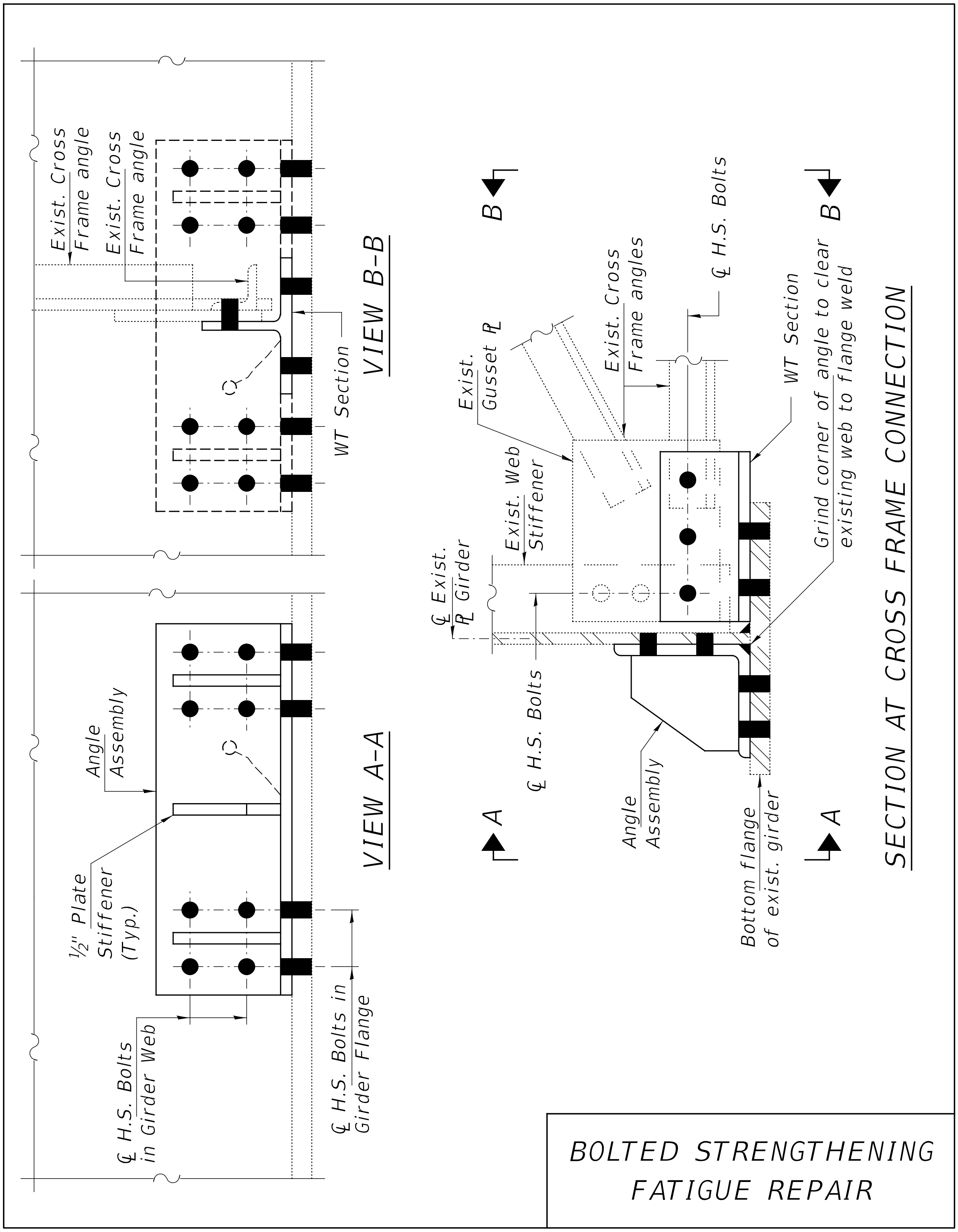


FIGURE 2.12.1.2-5

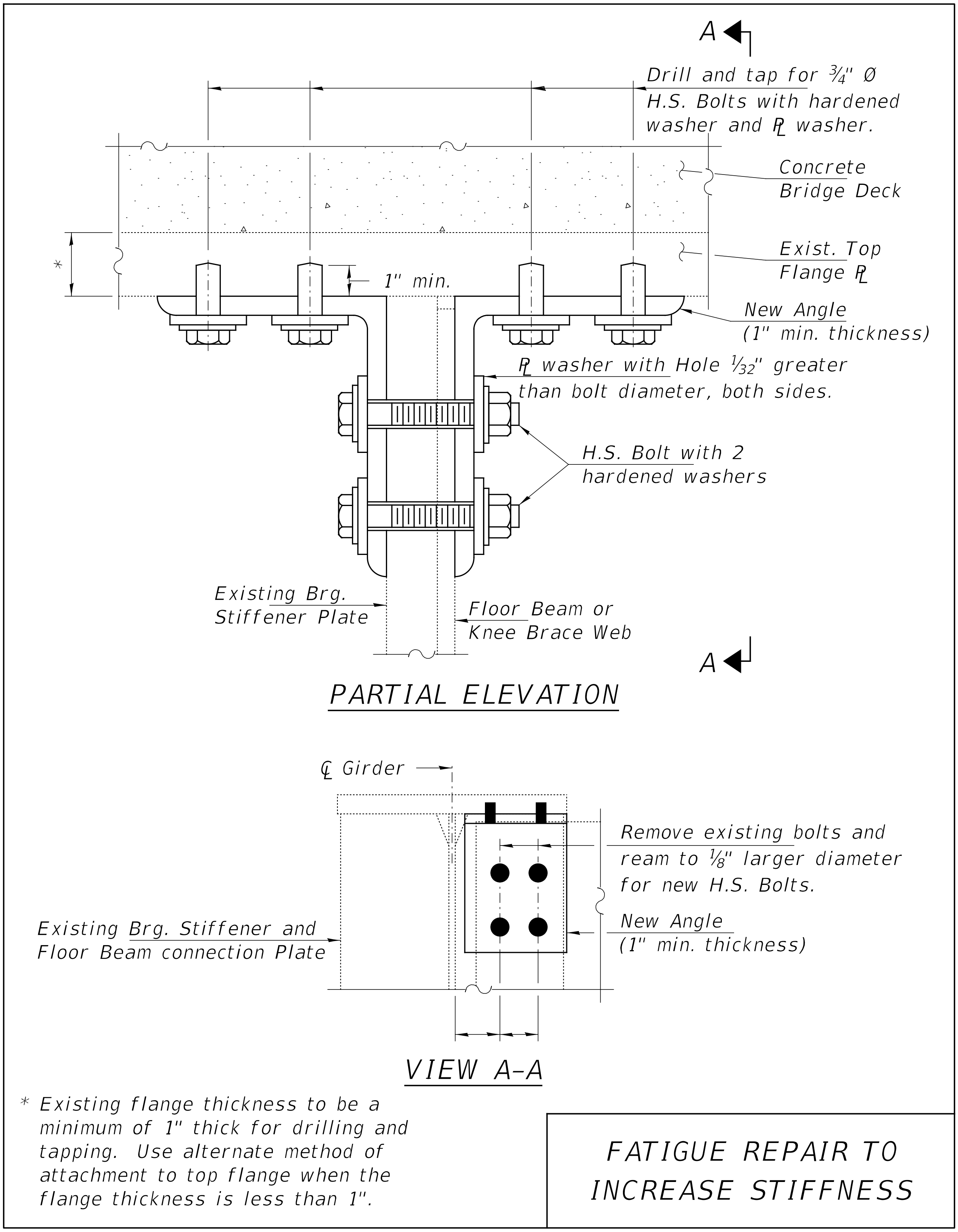


FIGURE 2.12.1.2-6

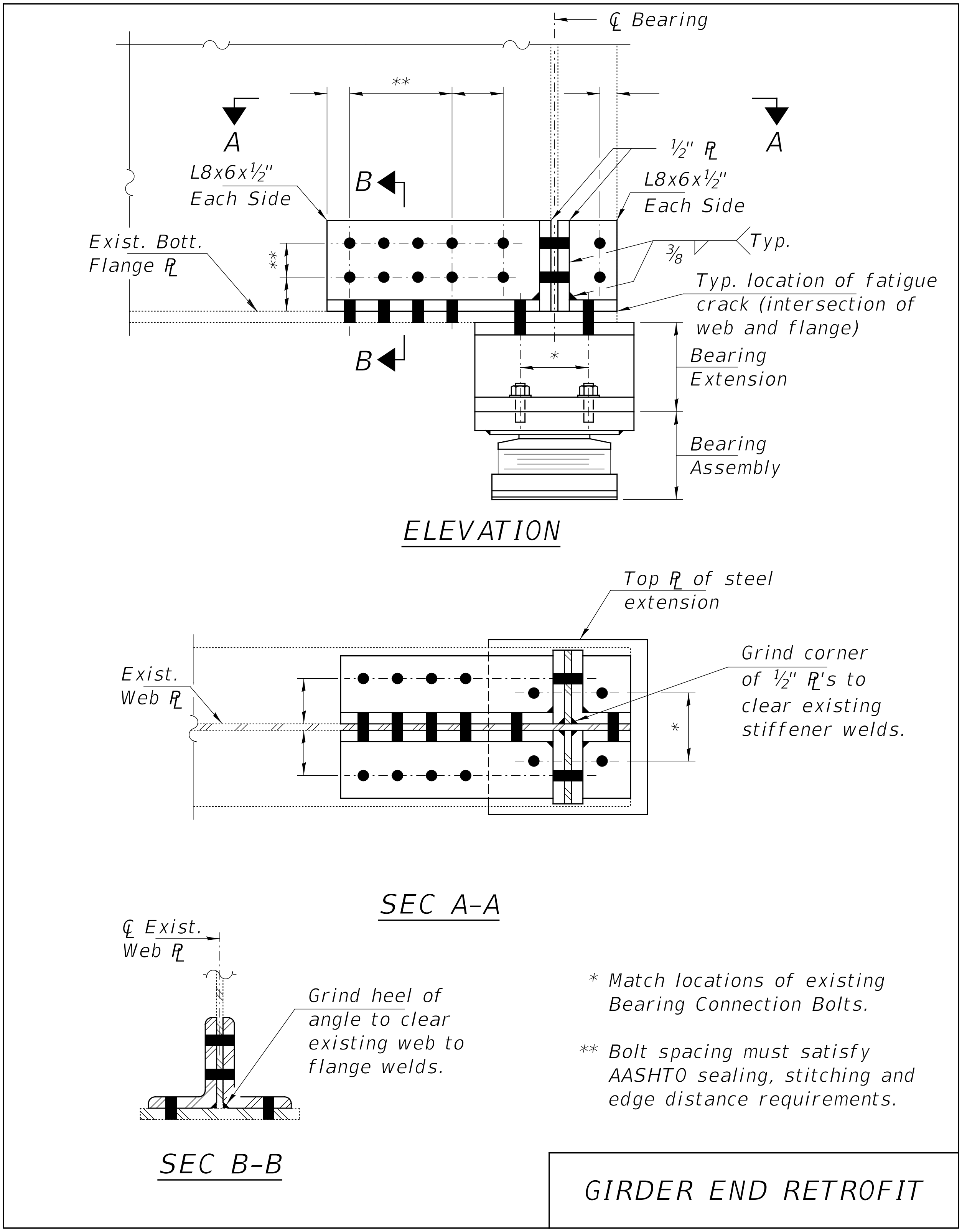


FIGURE 2.12.1.2-7

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 2.12.2.1-1 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 2.12.2.2-1 provides retrofit details and procedures for providing adequate separation between welds on a longitudinal web stiffener and a transverse stiffener.

Figure 2.12.2.2-2 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.

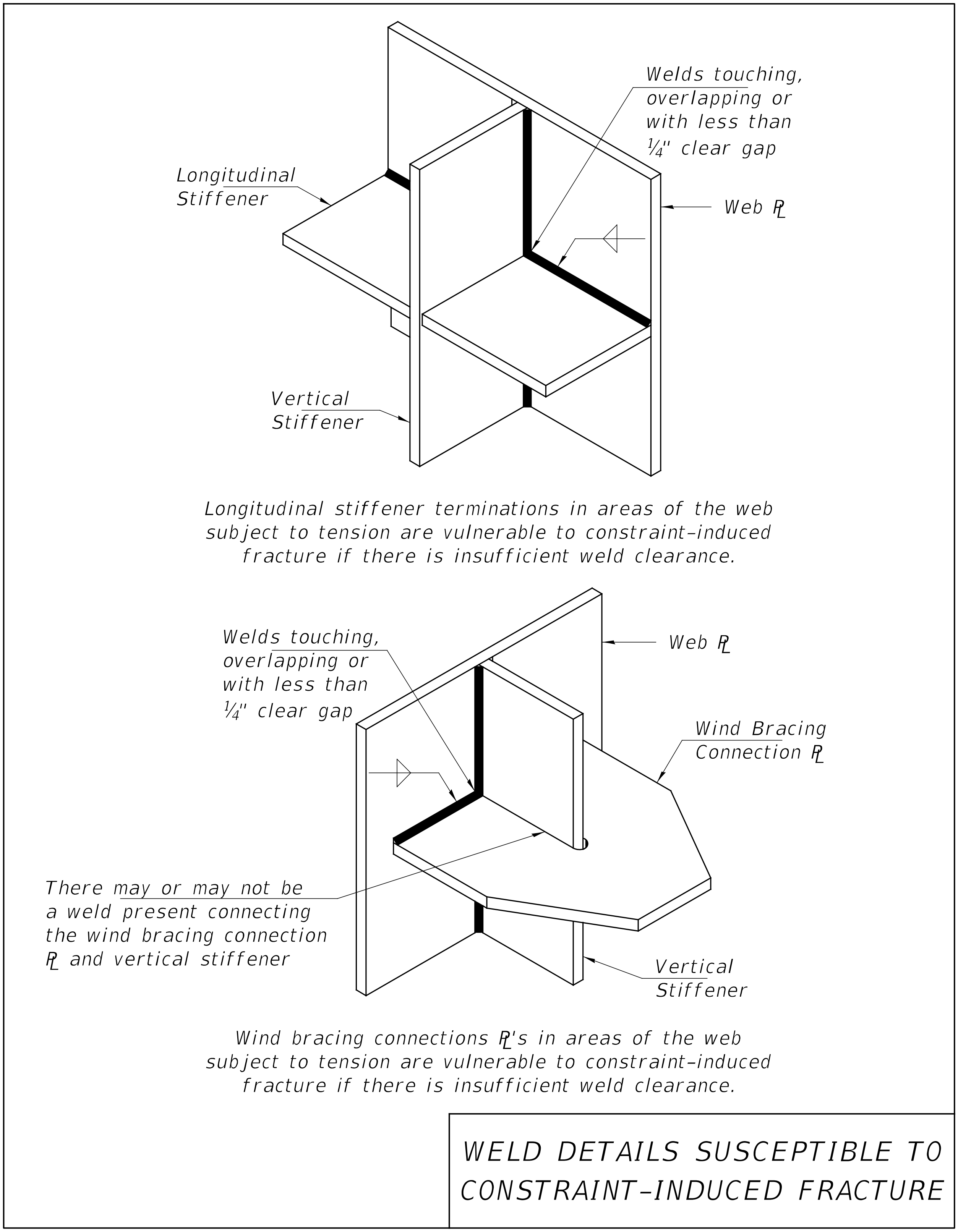
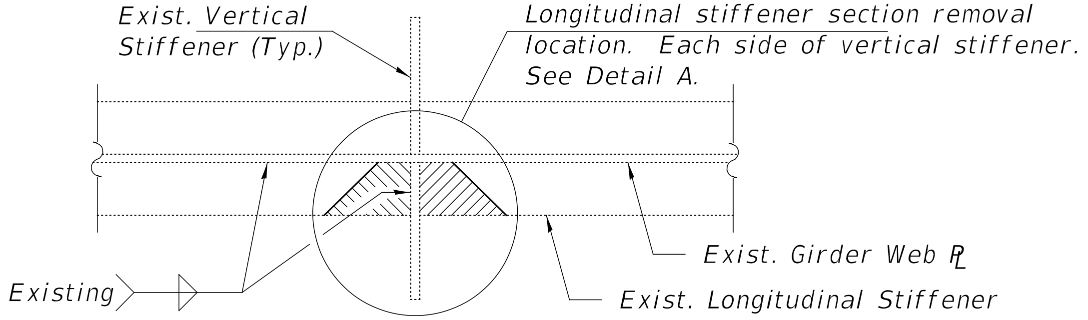
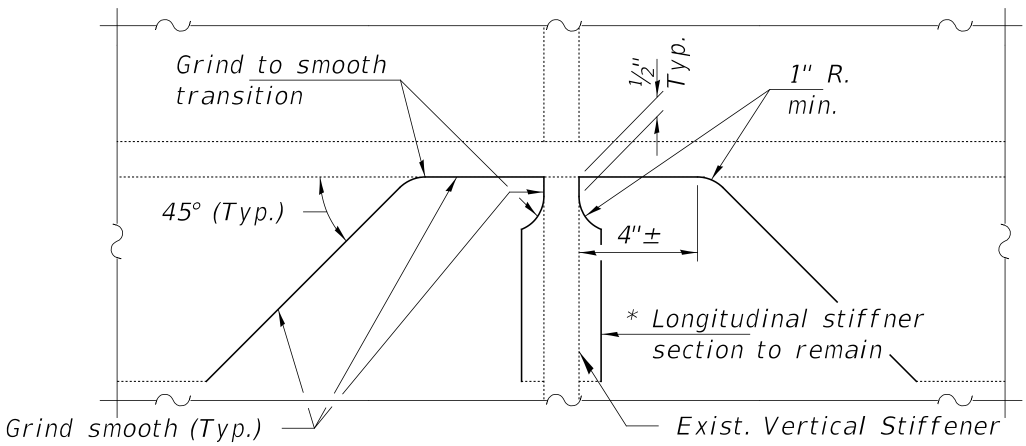


FIGURE 2.12.2.1-1



PLAN

Note:
In some cases, there may not be an existing weld connecting the longitudinal stiffener to the vertical stiffener.



DETAIL A

* 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.

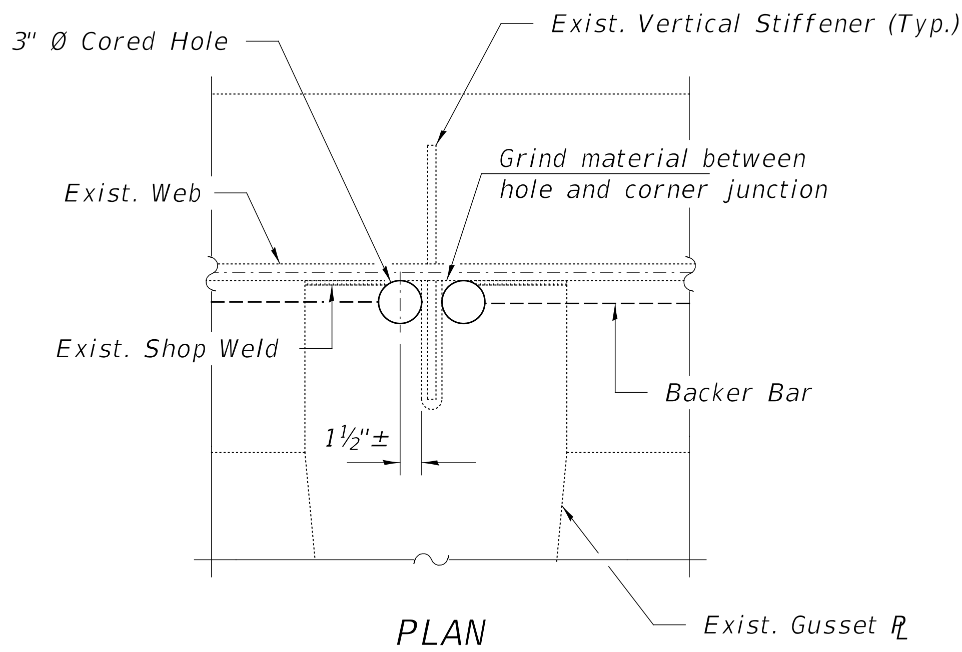
2. Web plate surface shall have a roughness average (Ra) of 250μ in. or less.

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 2.12.2.2-1



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 2.12.2.2-2

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.

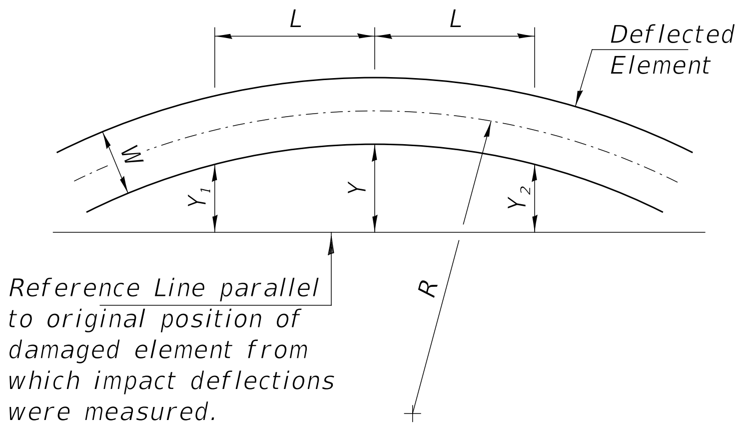
The NCHRP Report 271 guidelines for choosing a repair method are based on determining the amount of strain the impact has placed in the steel. The strain induced at any point on the member can be approximated by using the measured impact deflections adjacent to that point to determine a radius of curvature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

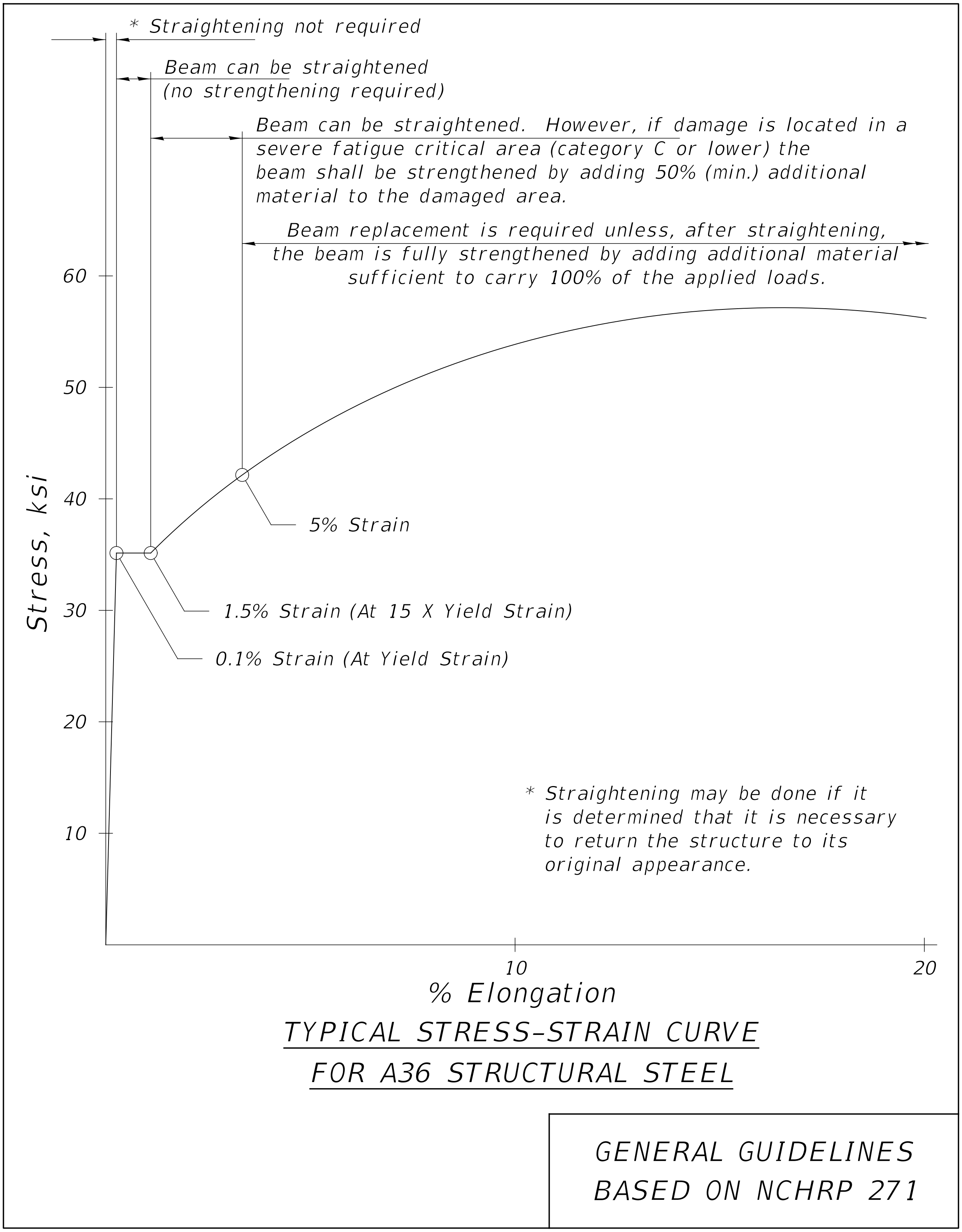


FIGURE 2.13.1-2

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.

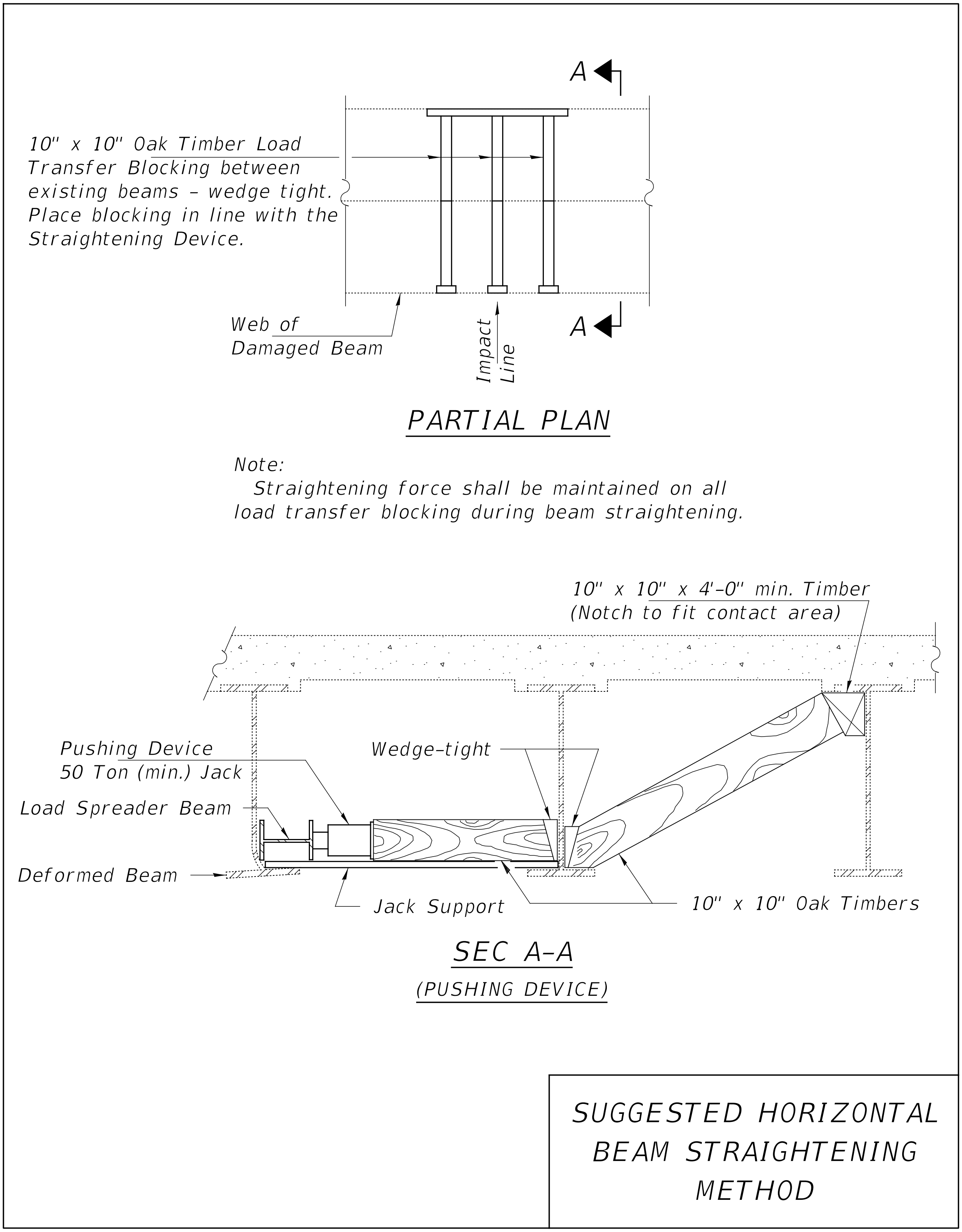


FIGURE 2.13.2-1

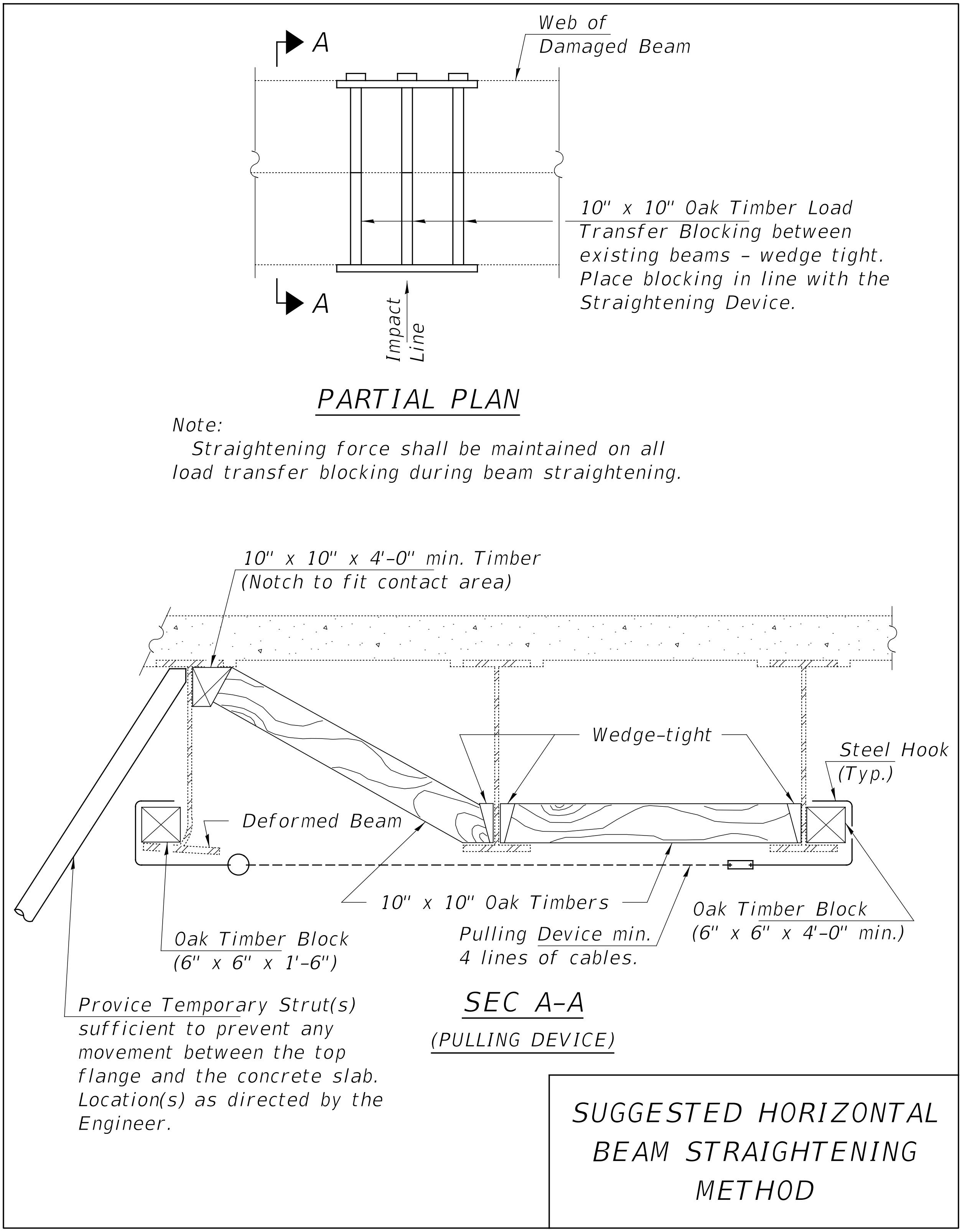


FIGURE 2.13.2-2

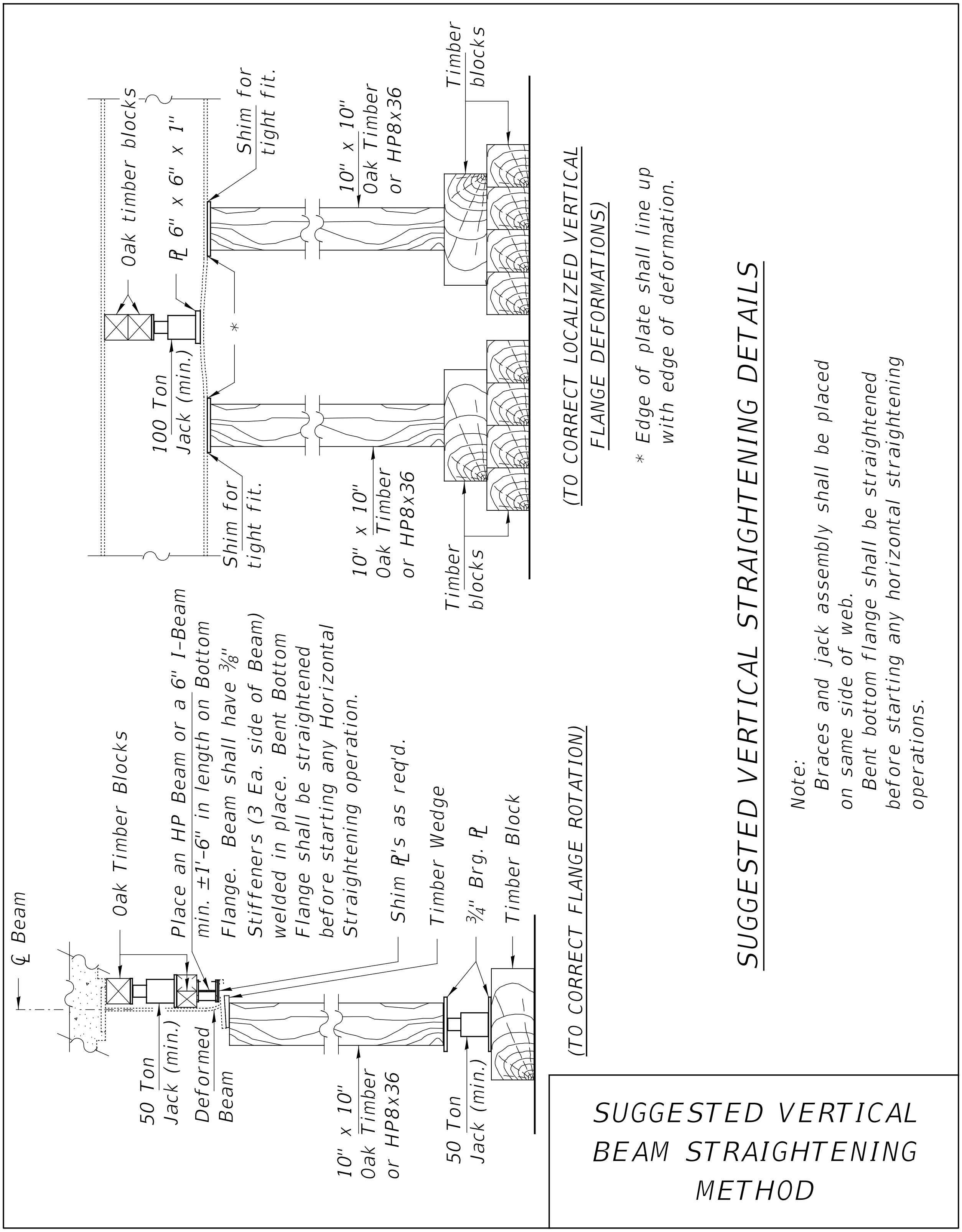


FIGURE 2.13.2-3

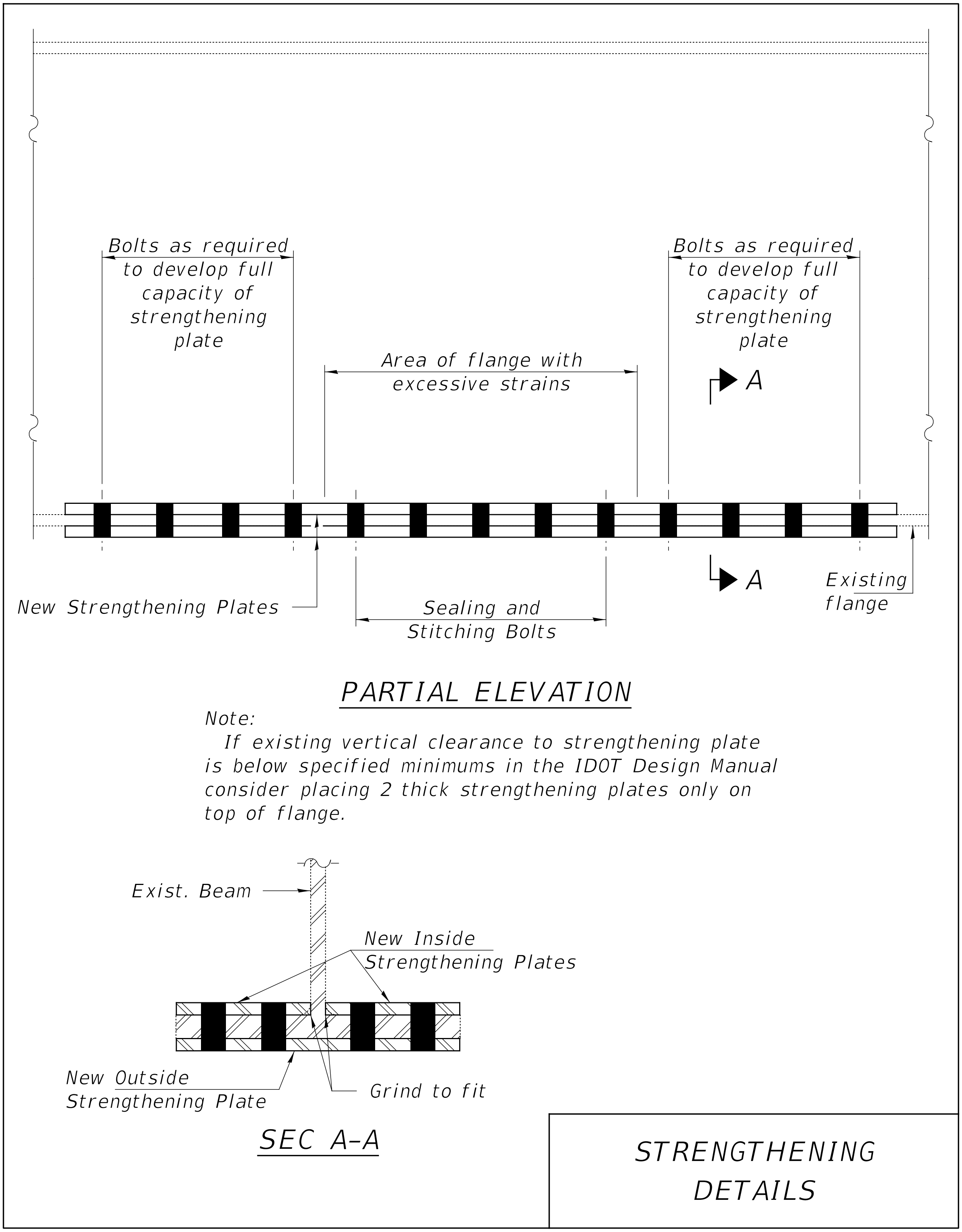


FIGURE 2.13.2-4

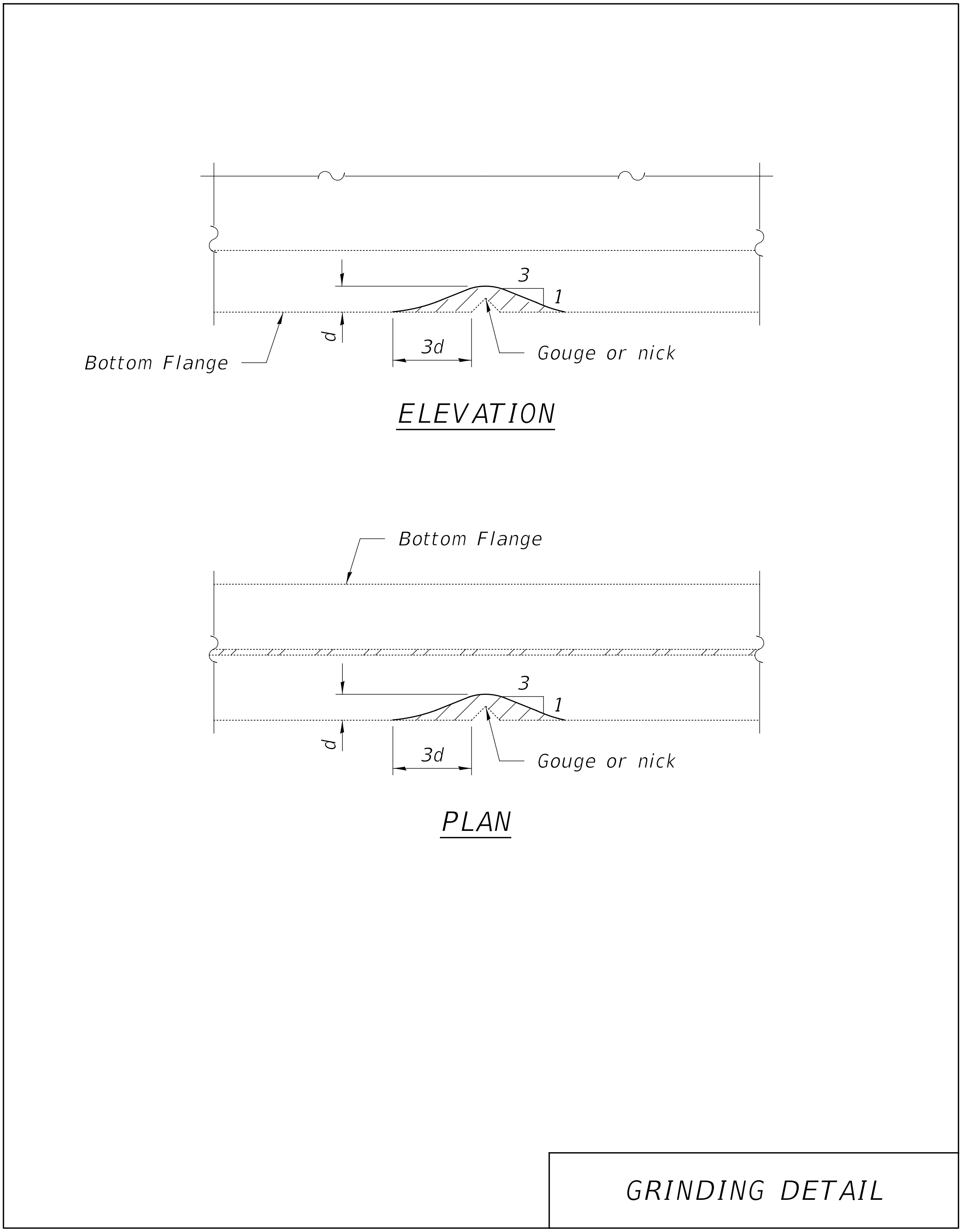


FIGURE 2.13.2-5

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.

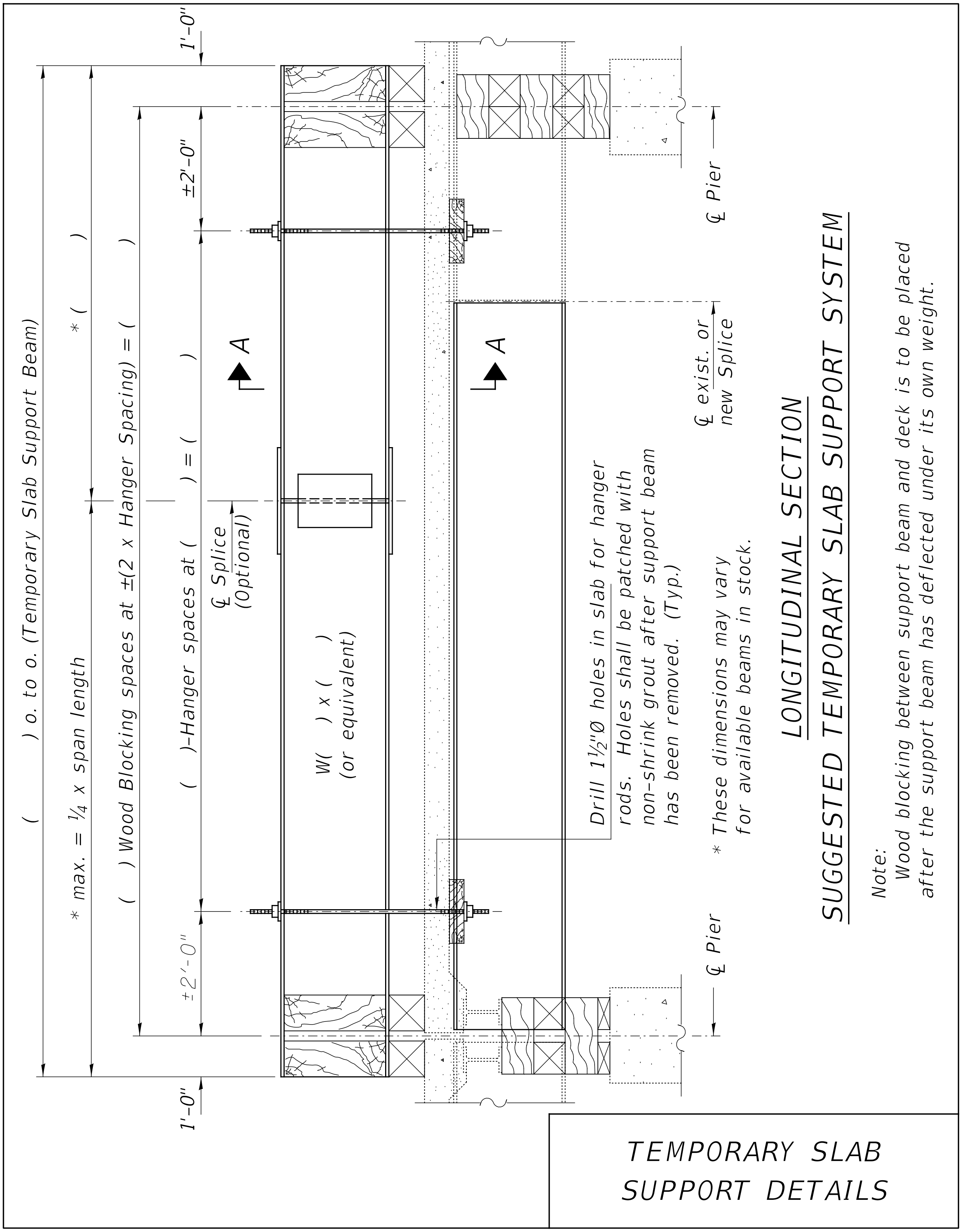


FIGURE 2.13.3-1

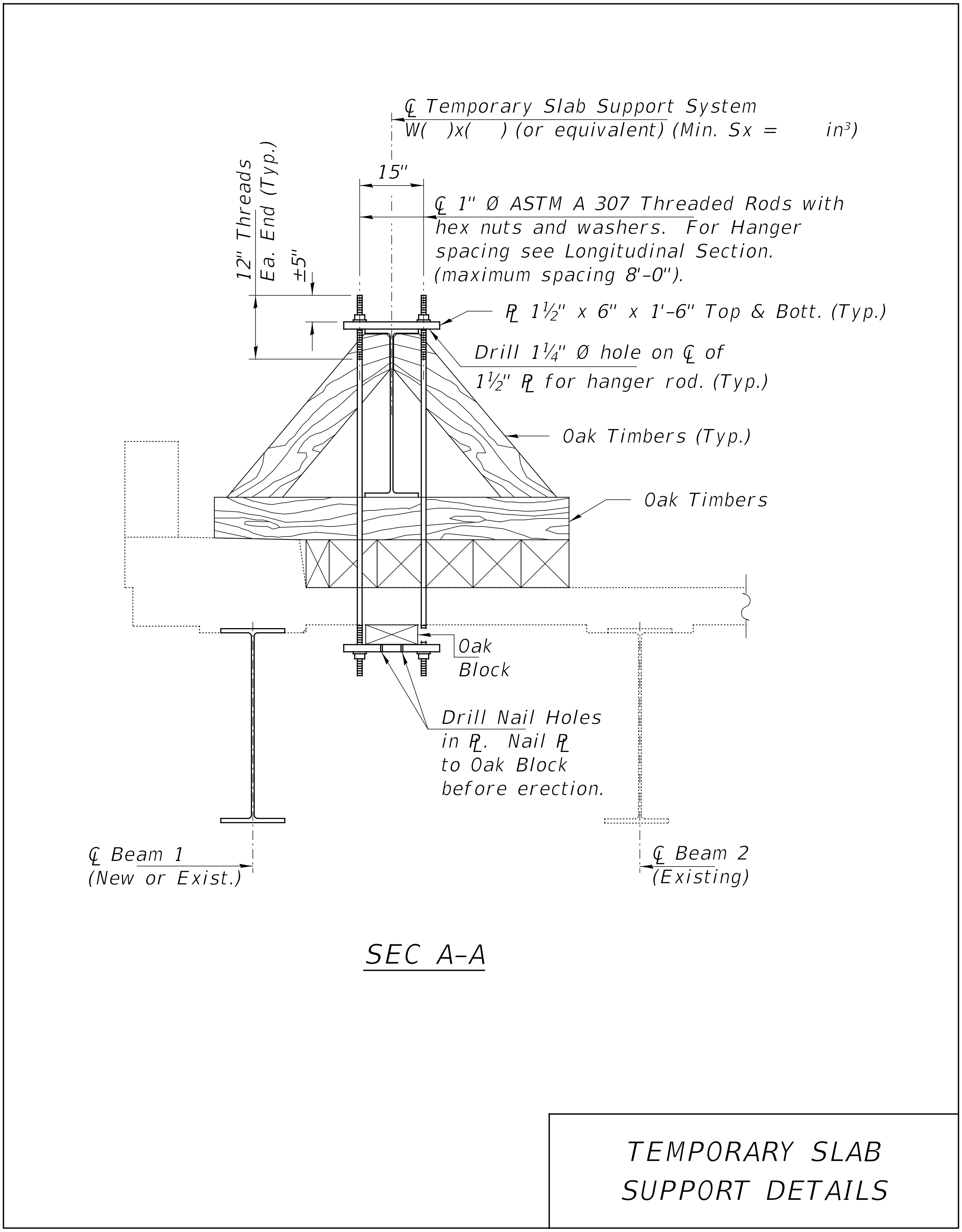


FIGURE 2.13.3-2

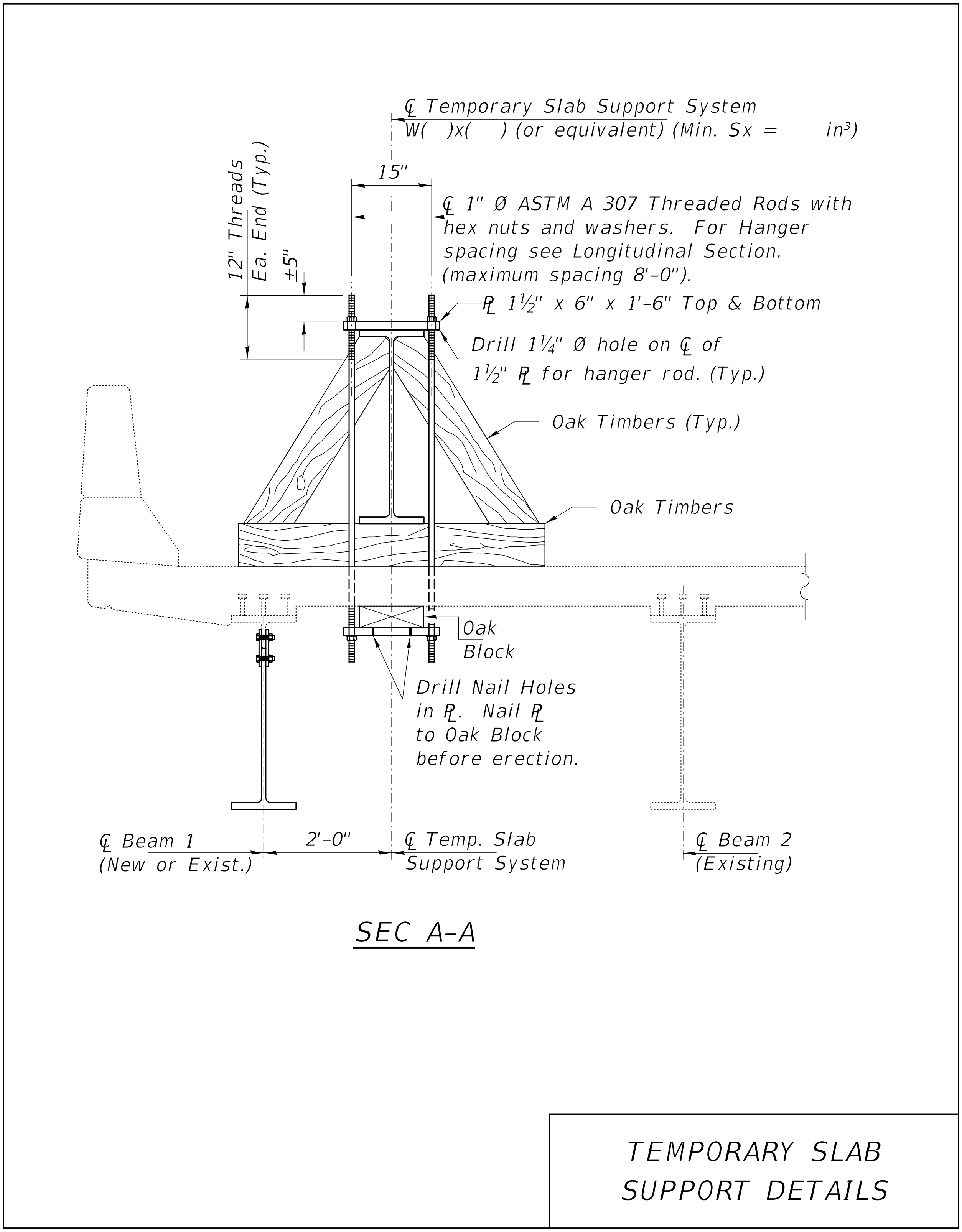


FIGURE 2.13.3-3

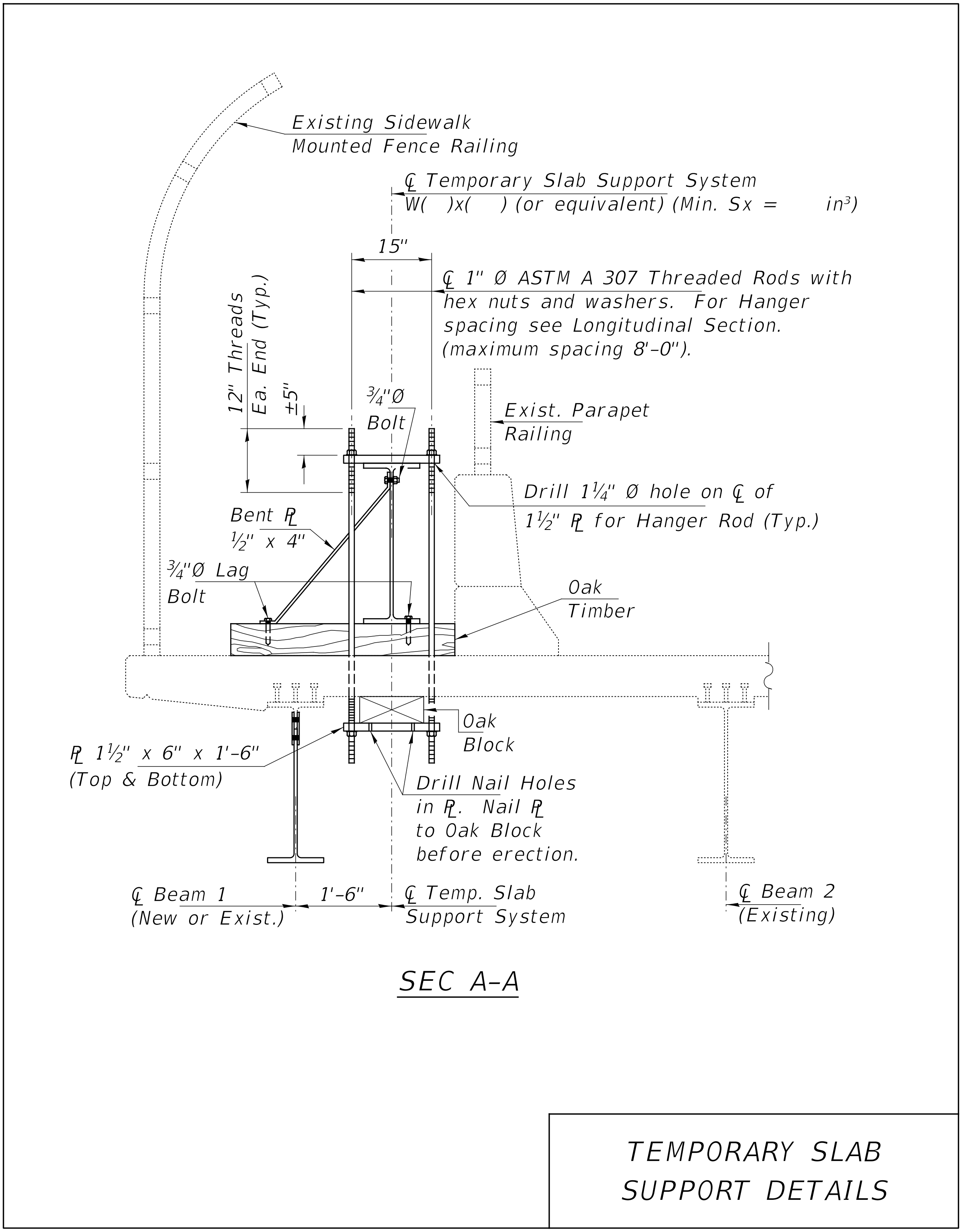
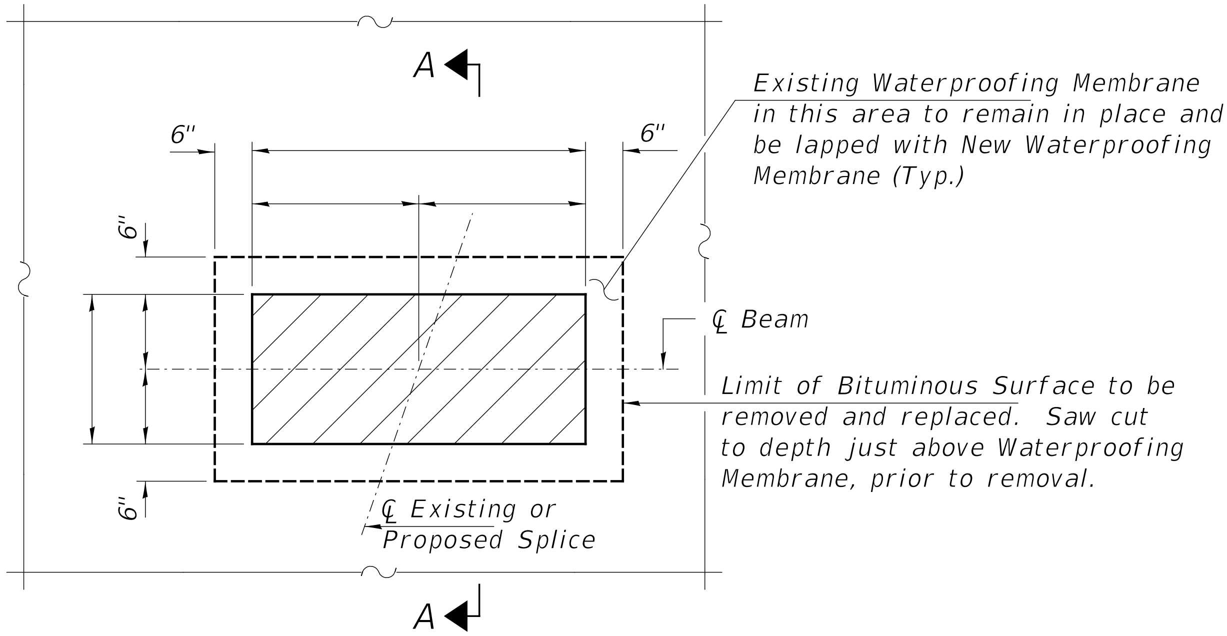


FIGURE 2.13.3-4



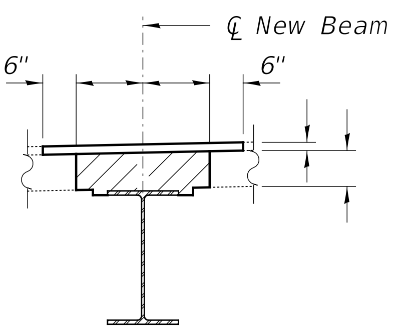
TYPICAL CONCRETE & BITUMINOUS SURFACE REMOVAL AND REPLACEMENT

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.

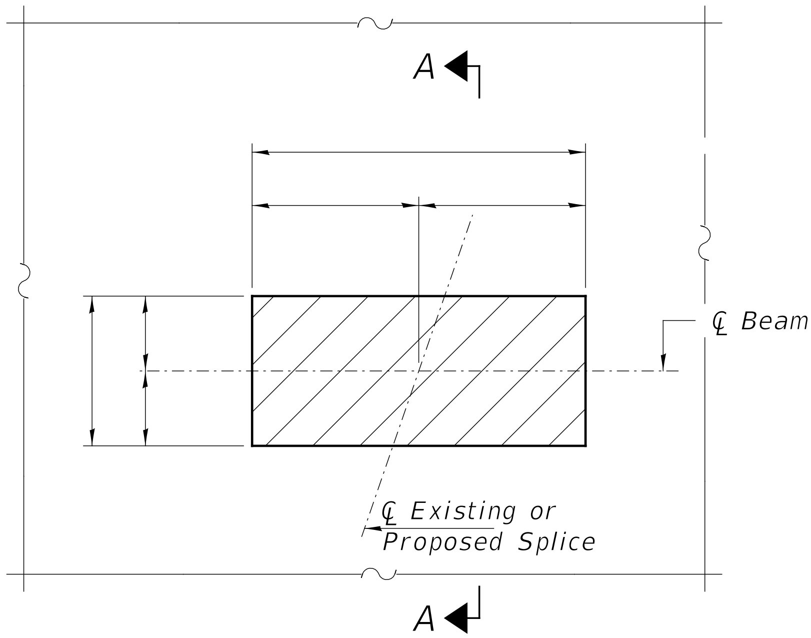
The cost of removing and replacing the existing bituminous surface, waterproofing membrane, and saw cutting shall be included in the cost of Concrete Superstructure.



SEC A–A

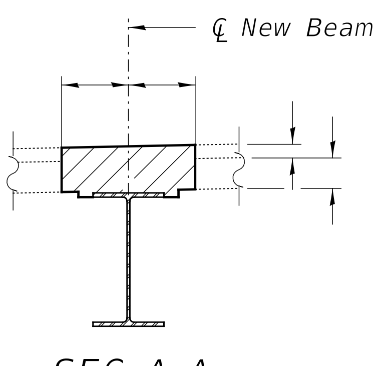
DECK SLAB AND HMA REMOVAL FOR BEAM REPLACEMENT

FIGURE 2.13.3-5



TYPICAL CONCRETE & HARD OVERLAY
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.



SEC A–A

DECK SLAB AND HARD OVERLAY REMOVAL FOR BEAM REPLACEMENT

FIGURE 2.13.3-6

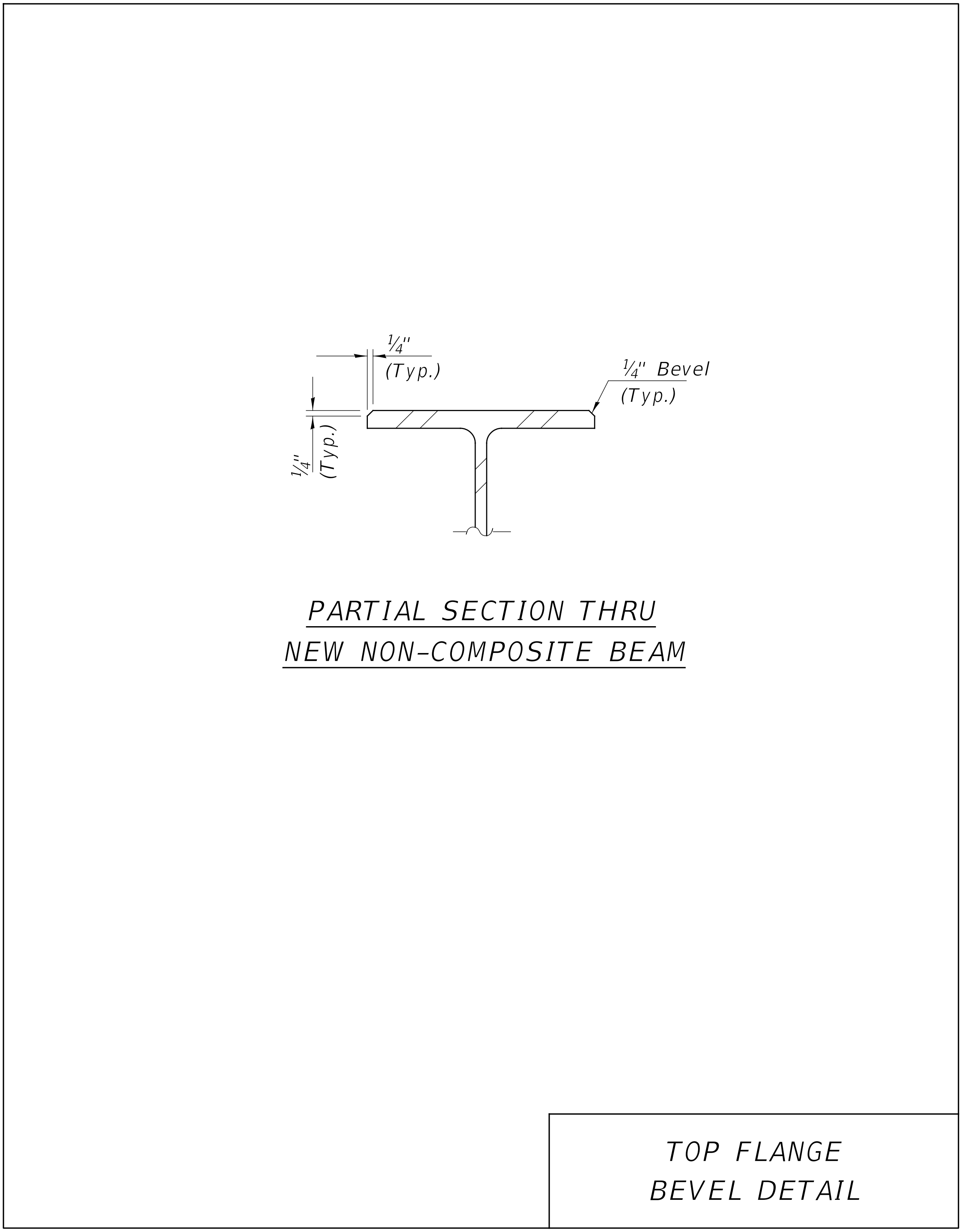
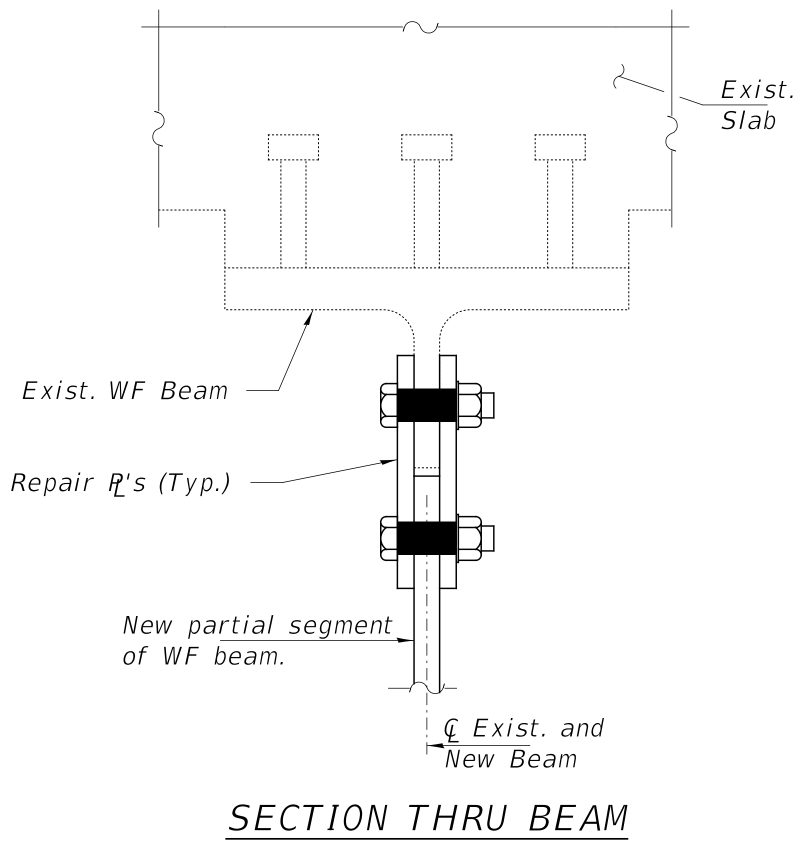


FIGURE 2.13.3-7



EXAMPLE OF ZIPPER DETAIL FOR WIDE FLANGE SECTION REPLACEMENT

FIGURE 2.13.3-8

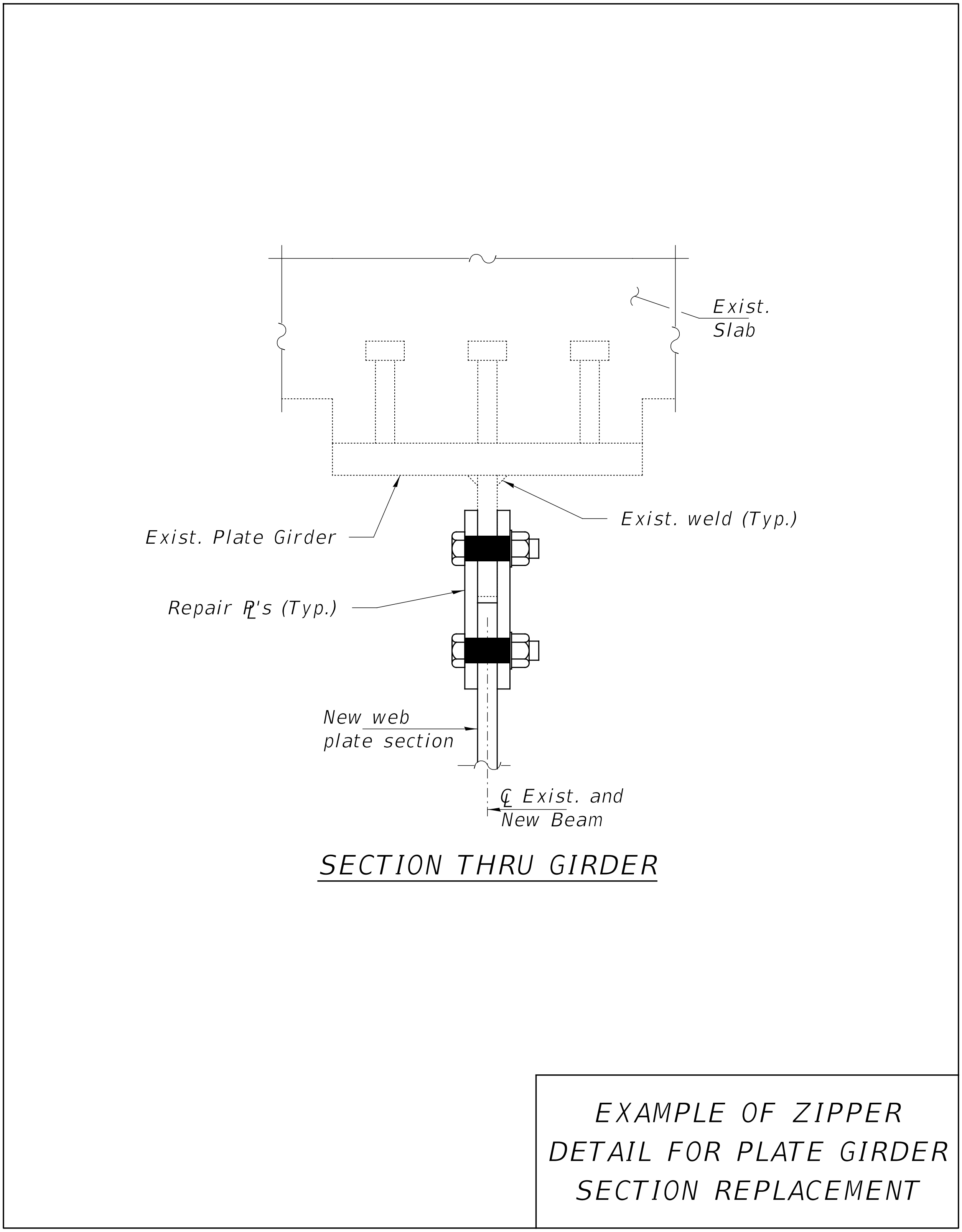


FIGURE 2.13.3-9

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.

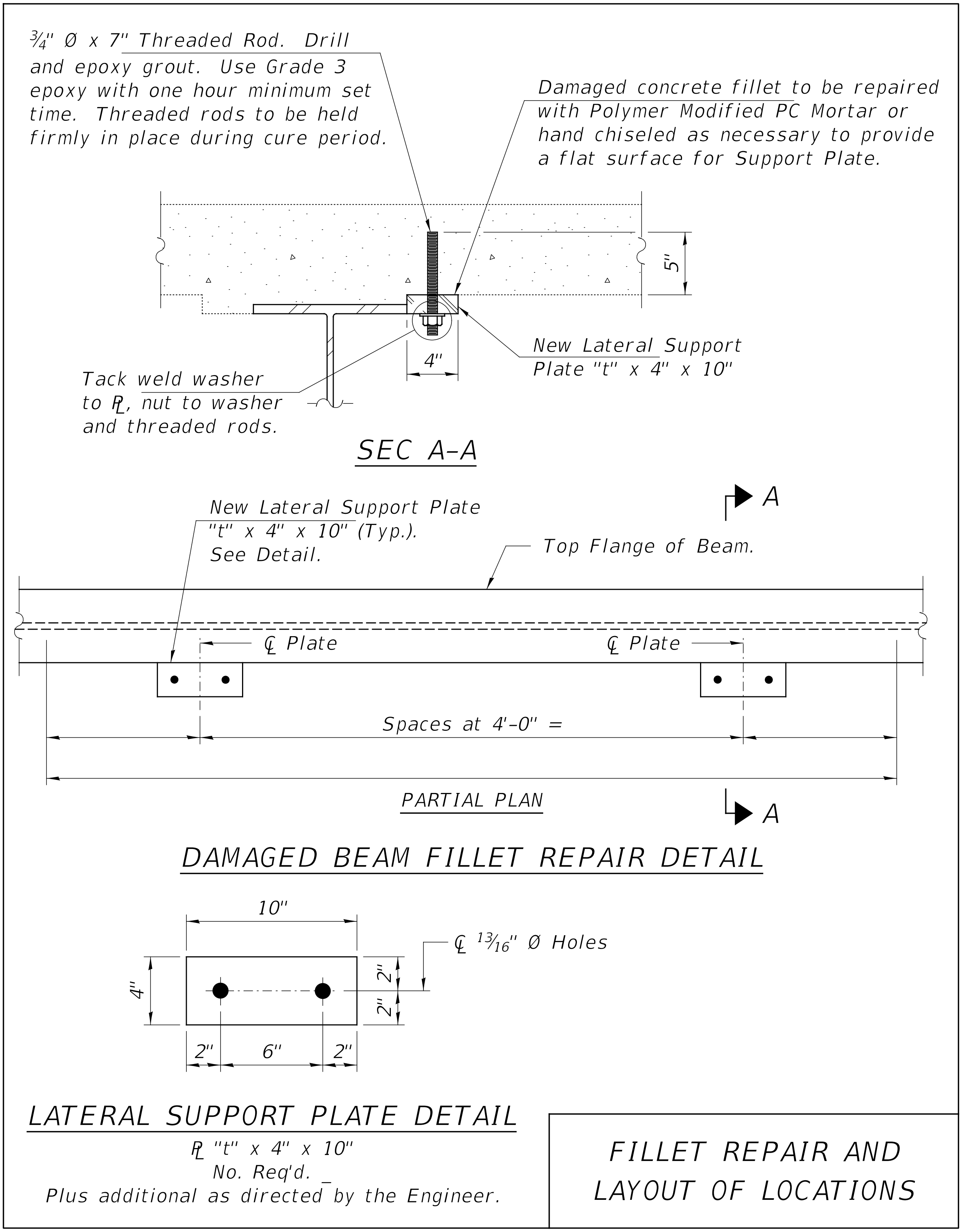


FIGURE 2.13.4-1

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.

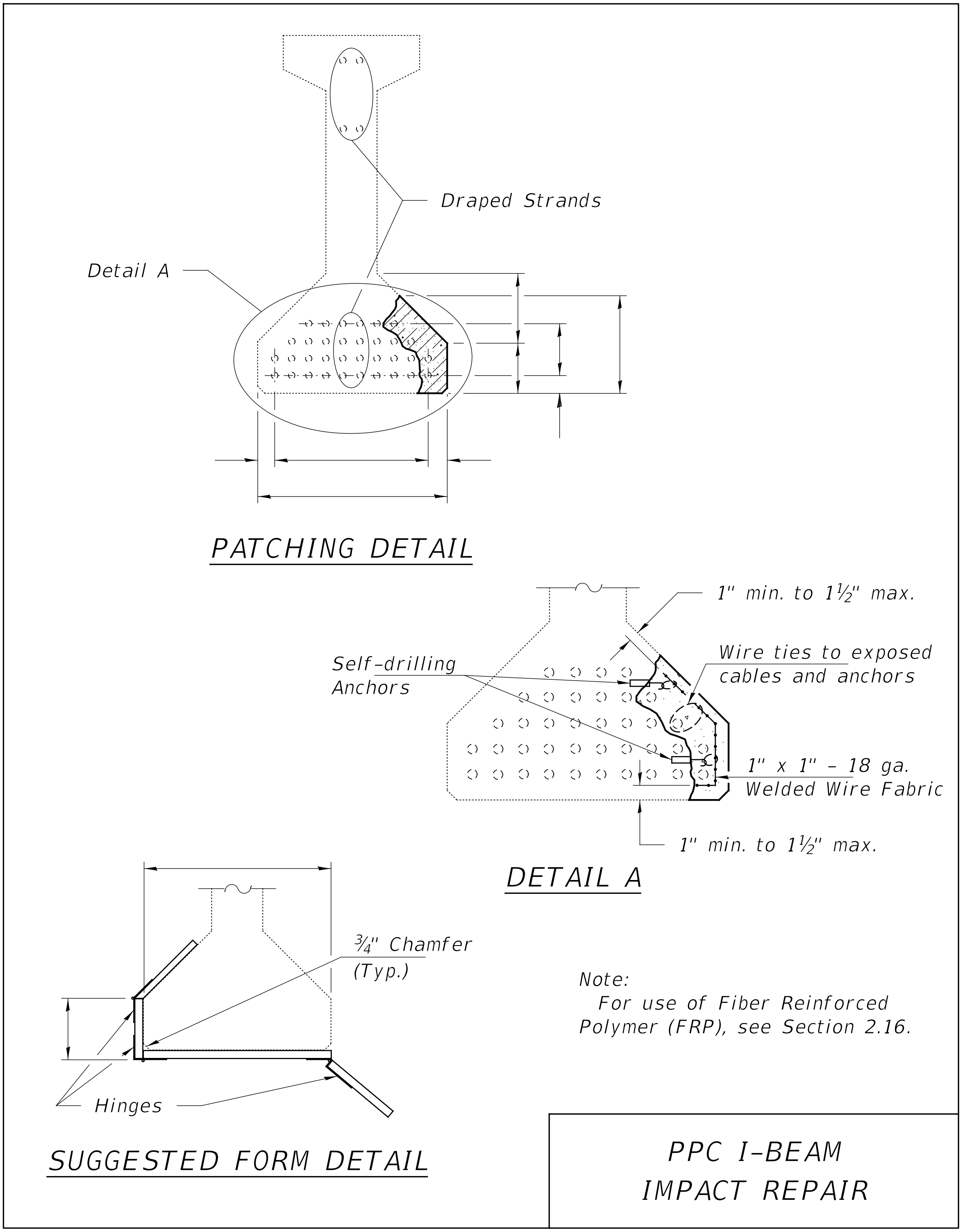


FIGURE 2.14.3-1

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 GBSP 21 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 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

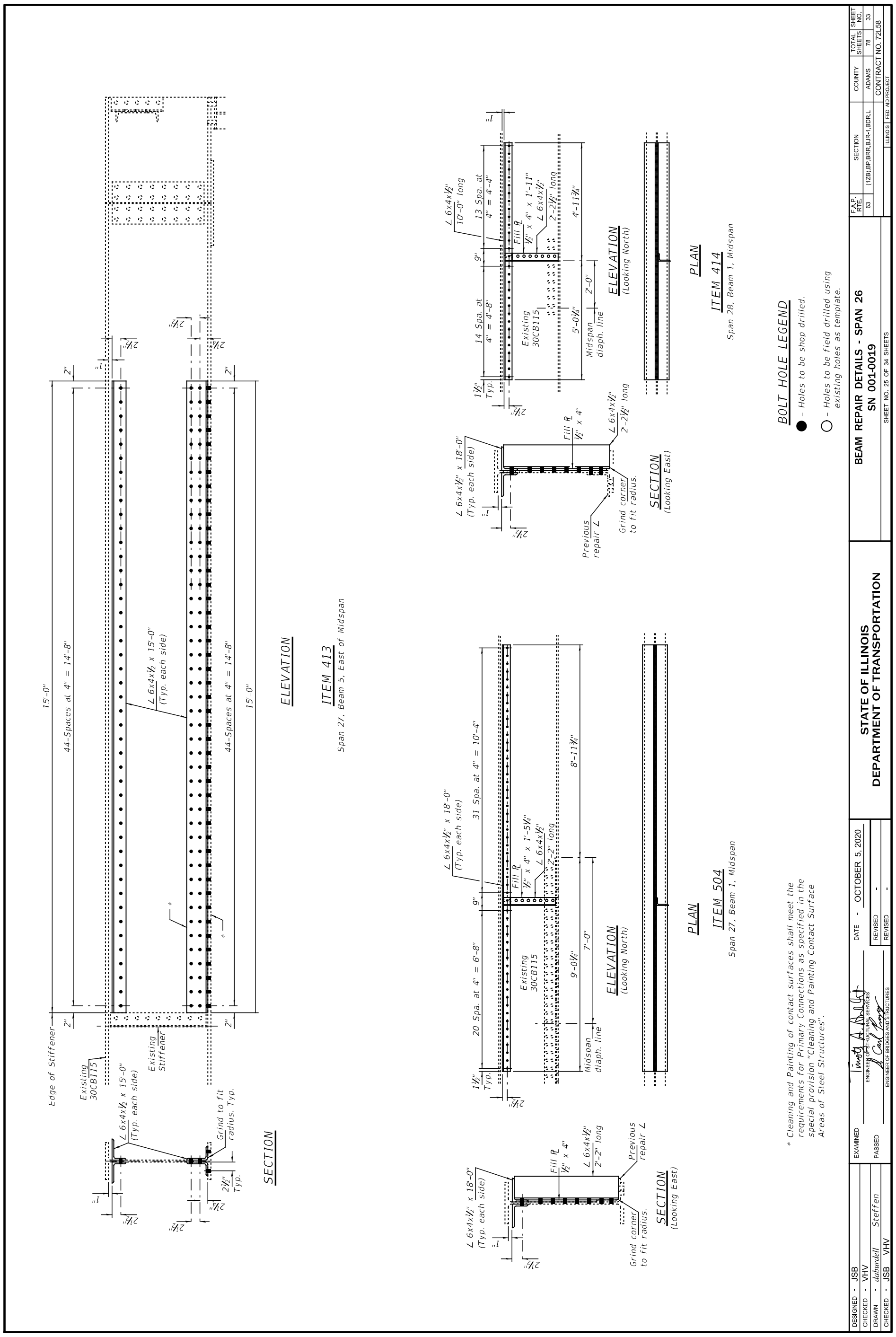
While preparing steel repairs a determination must be made as to whether the members and connections involved are either primary or secondary. This will have a significant impact on the amount of preparation work that will be required on the existing steel before the strengthening plates/angles are installed.

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 2.15.2.1-1

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.

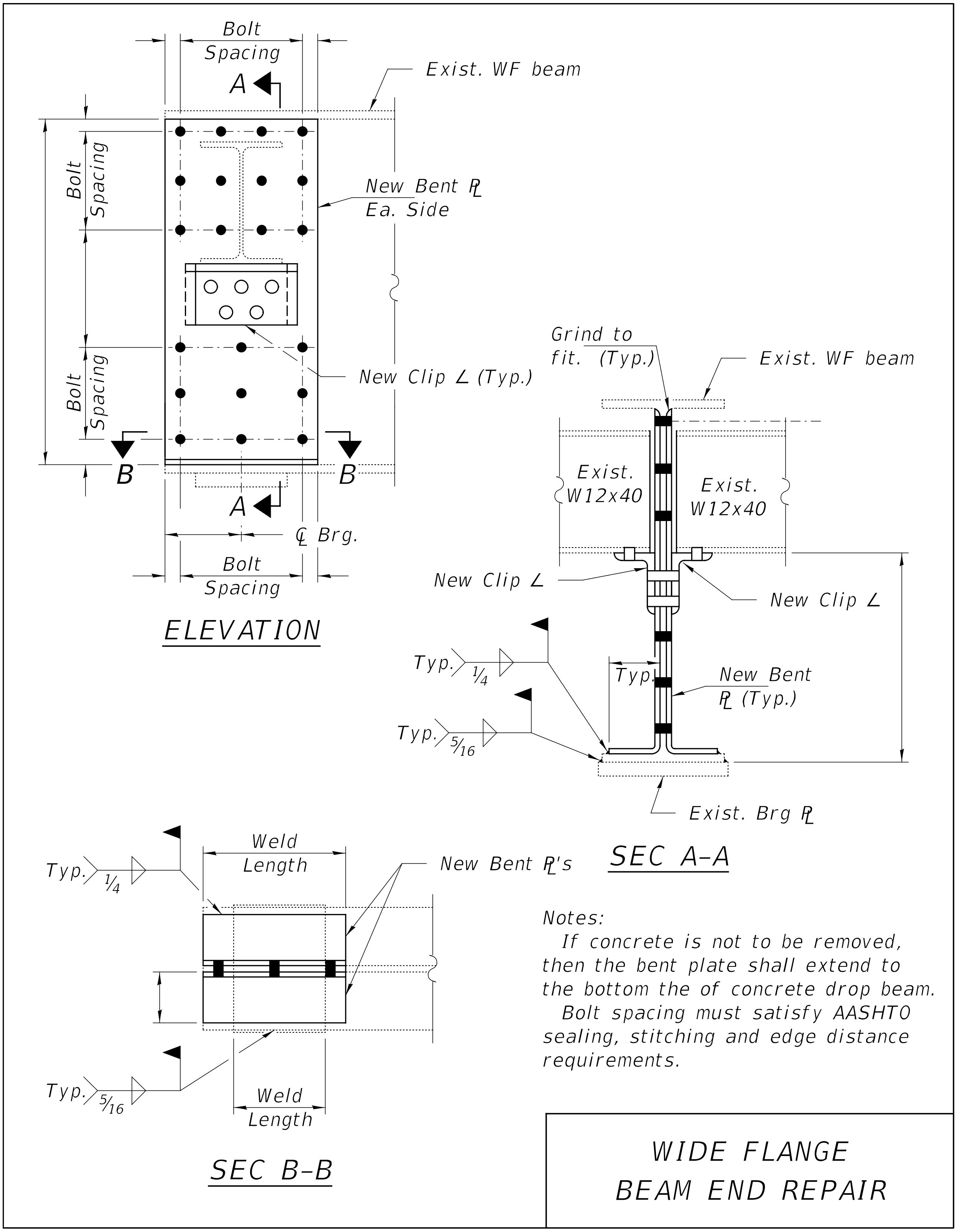
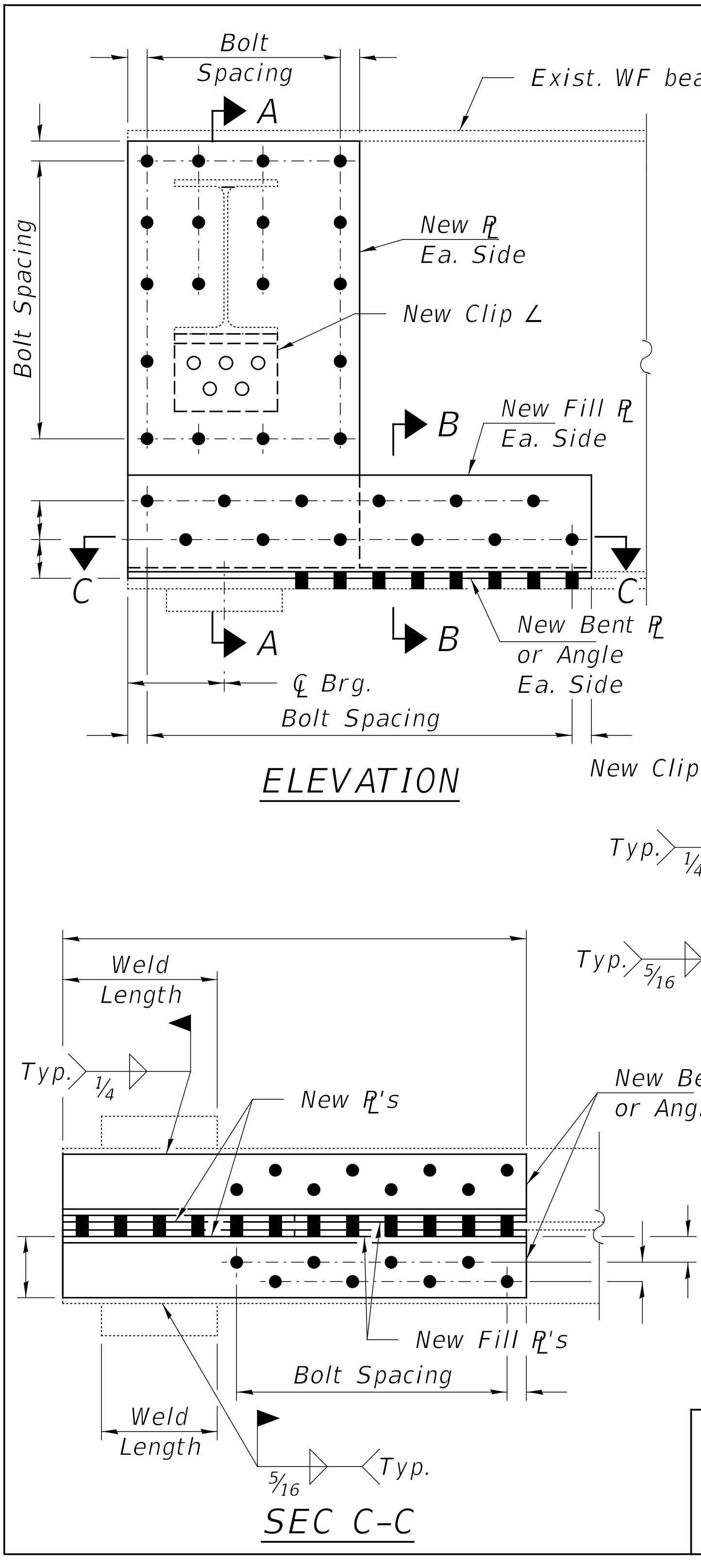


FIGURE 2.15.3-1



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

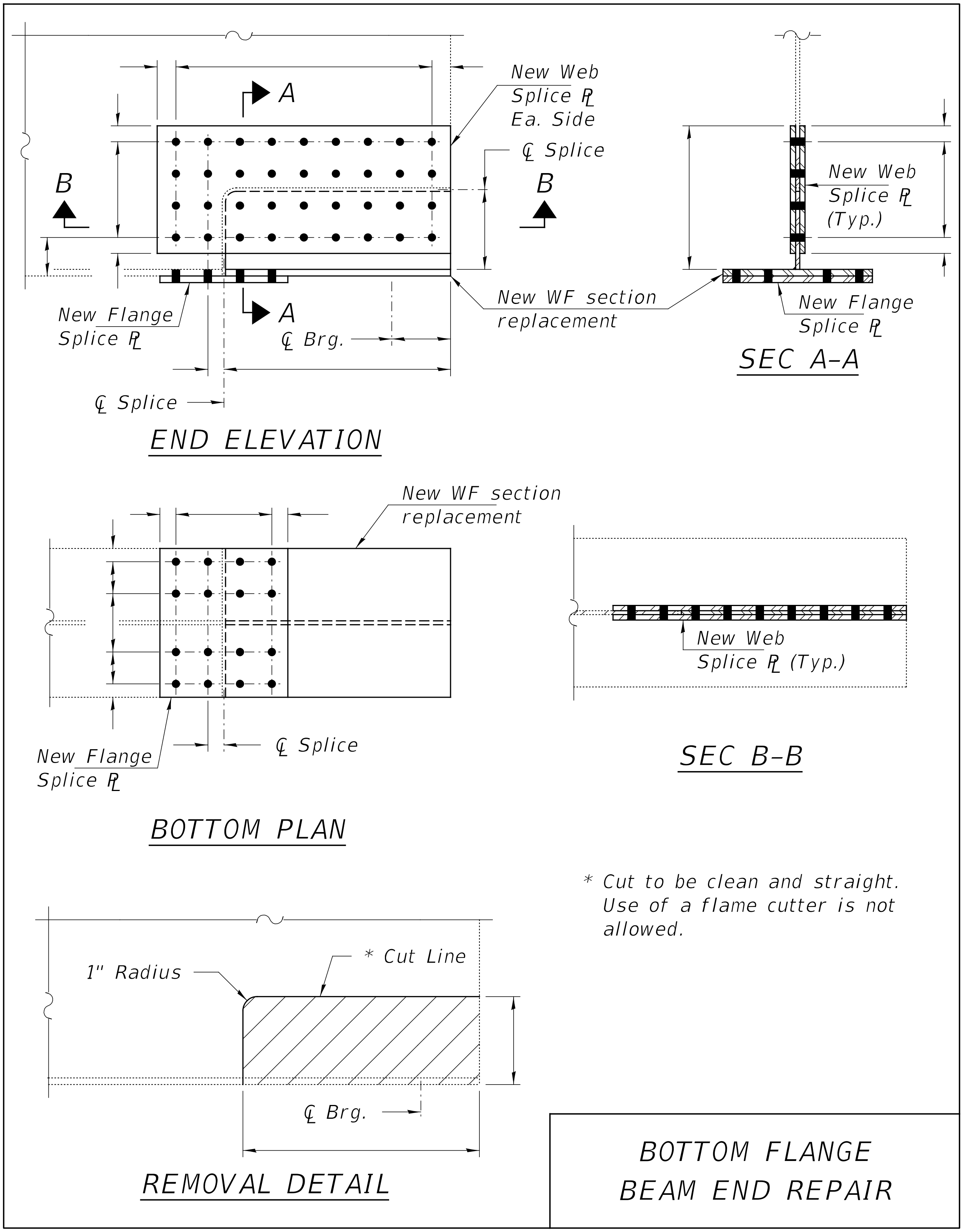
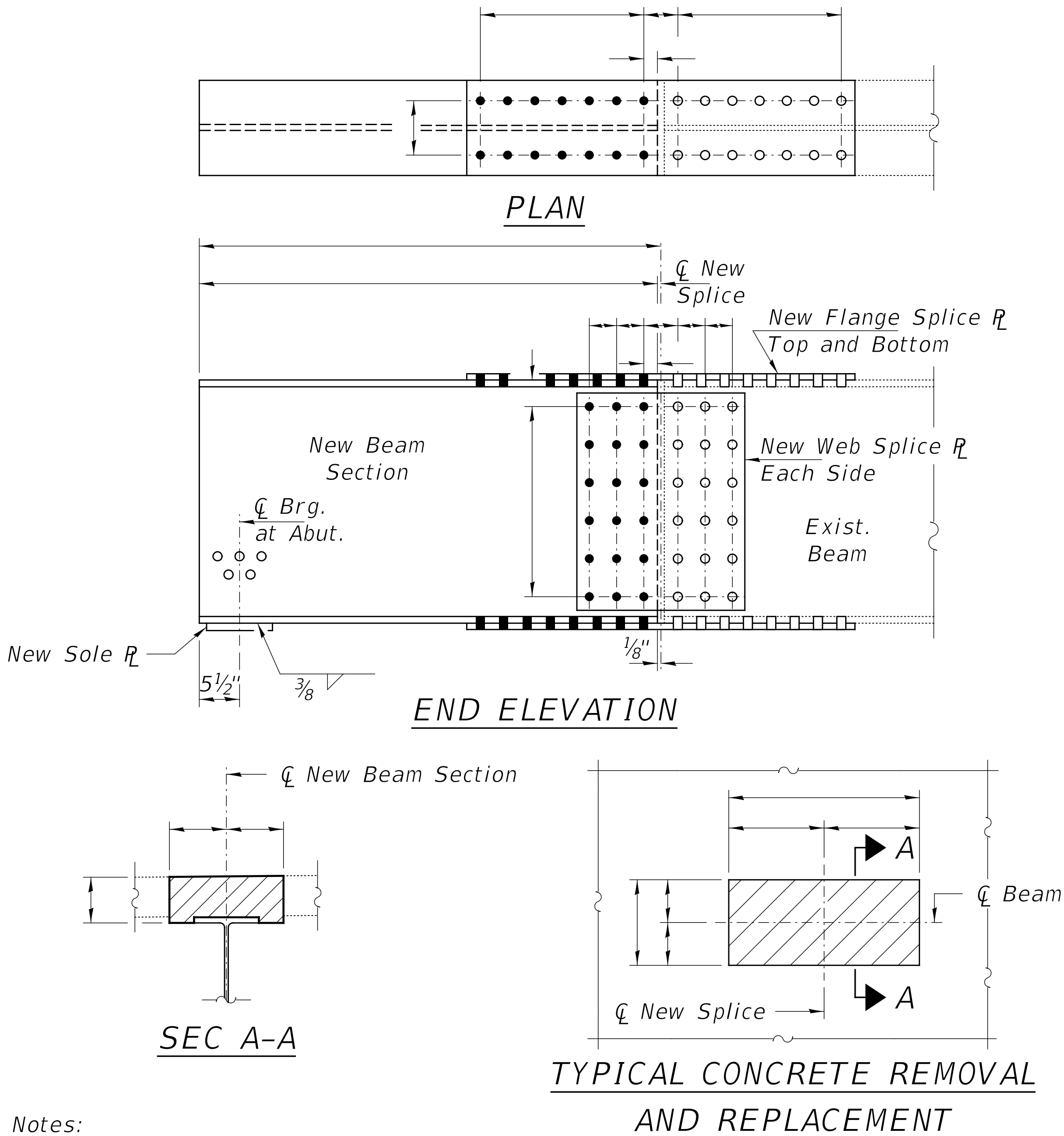


FIGURE 2.15.3-3



Notes:
Hatched areas indicate concrete sections to be removed and replaced. Perimeters of concrete removal areas shall be saw cut 3/4" 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

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.

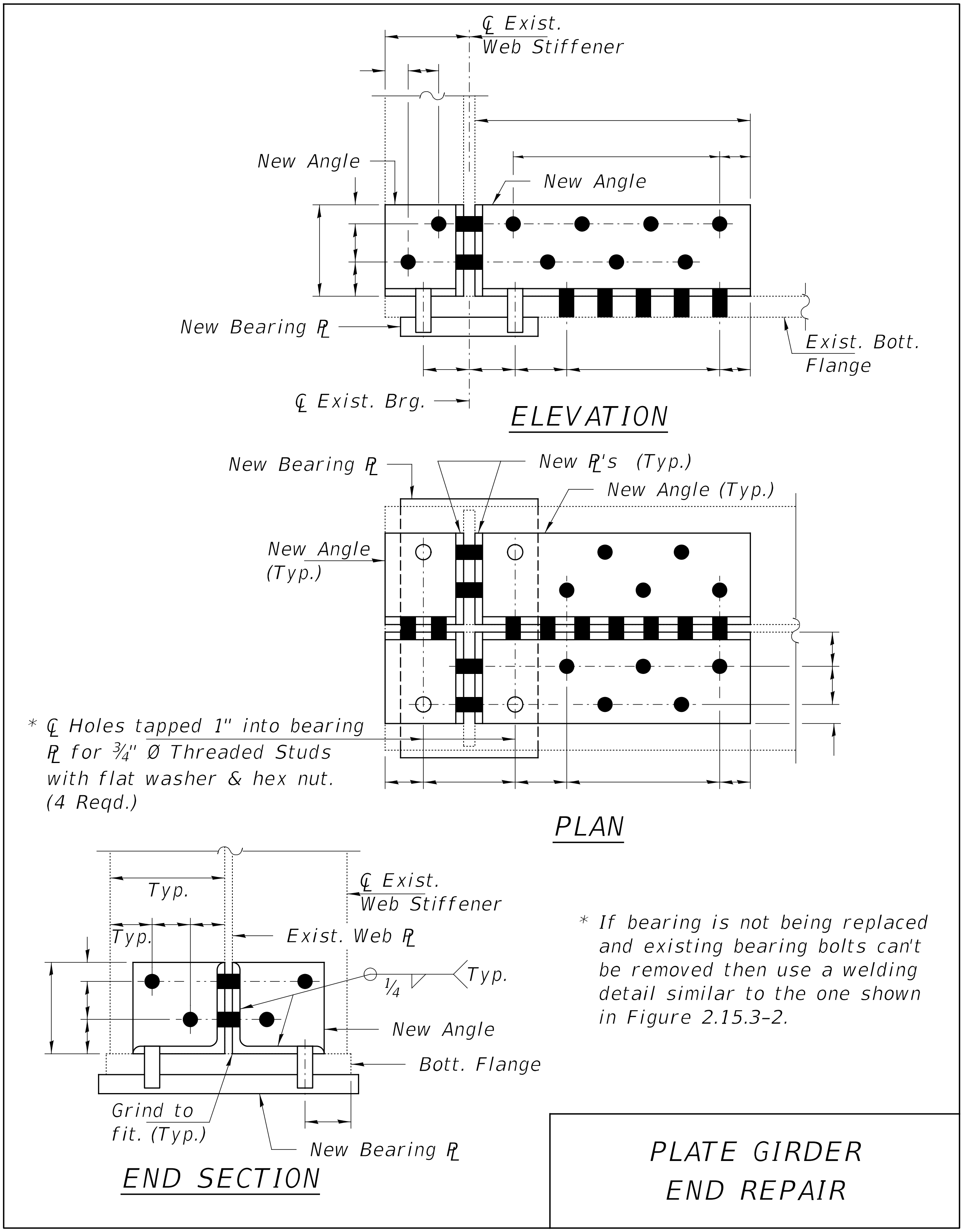


FIGURE 2.15.4-1

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.

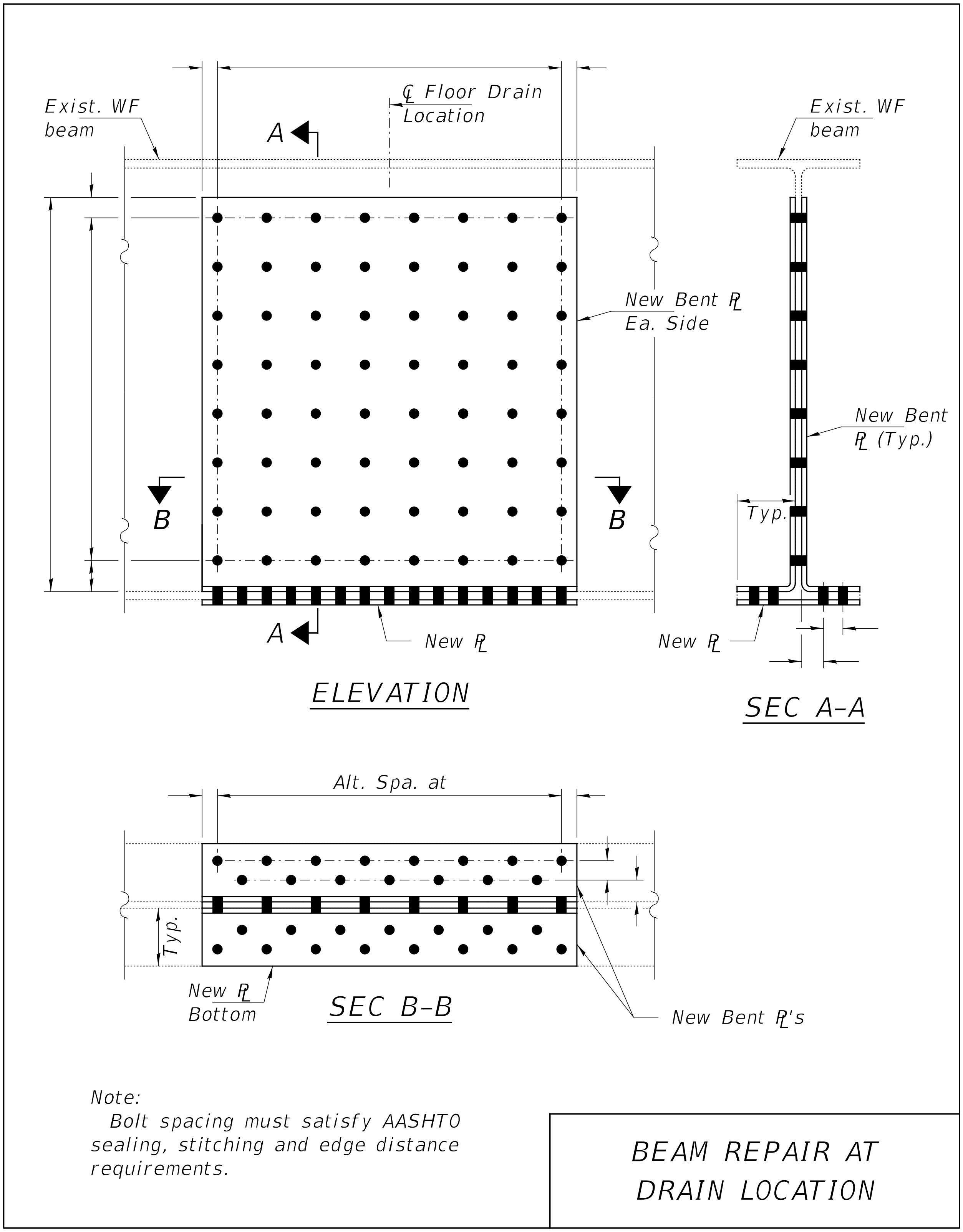


FIGURE 2.15.5-1

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.

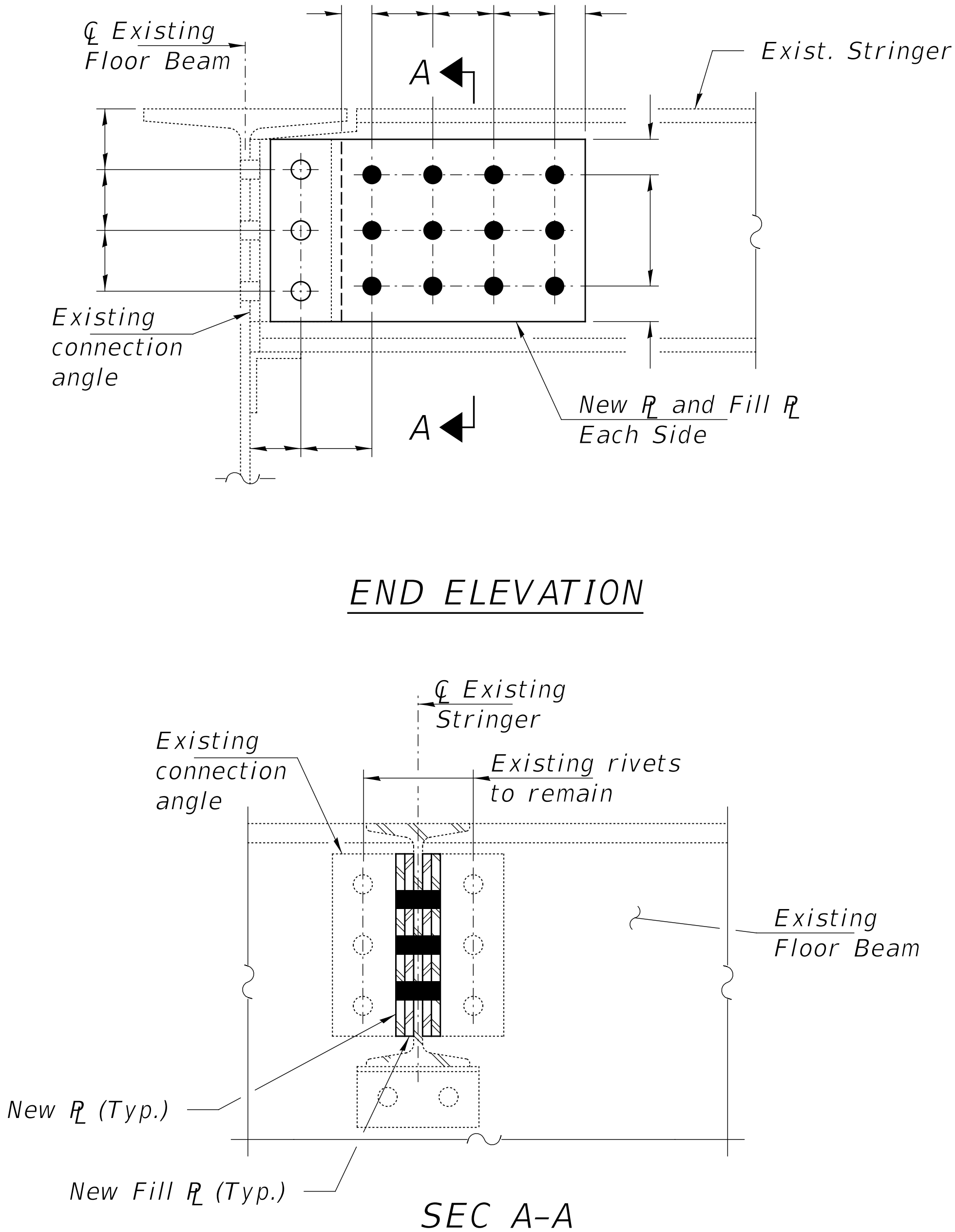
Contract plans should include a table showing the expected loads on the member needing to be supported.

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.



Notes:
Holes in the stringer web are to be drilled using the new reinforcement plate as template, except as noted.

FIXED STRINGER END REPAIR AT FLOORBEAMS

FIGURE 2.15.7-1

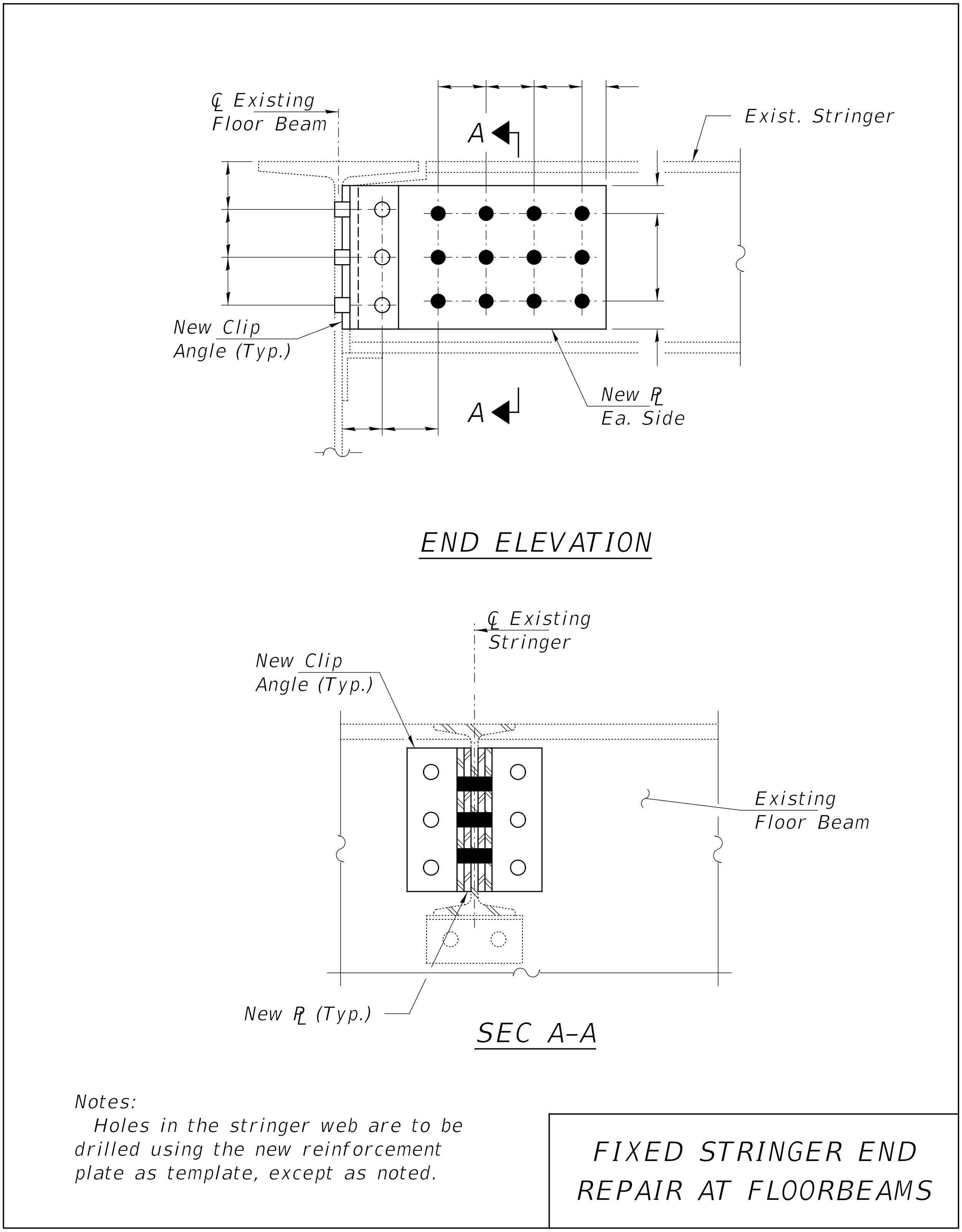


FIGURE 2.15.7-2

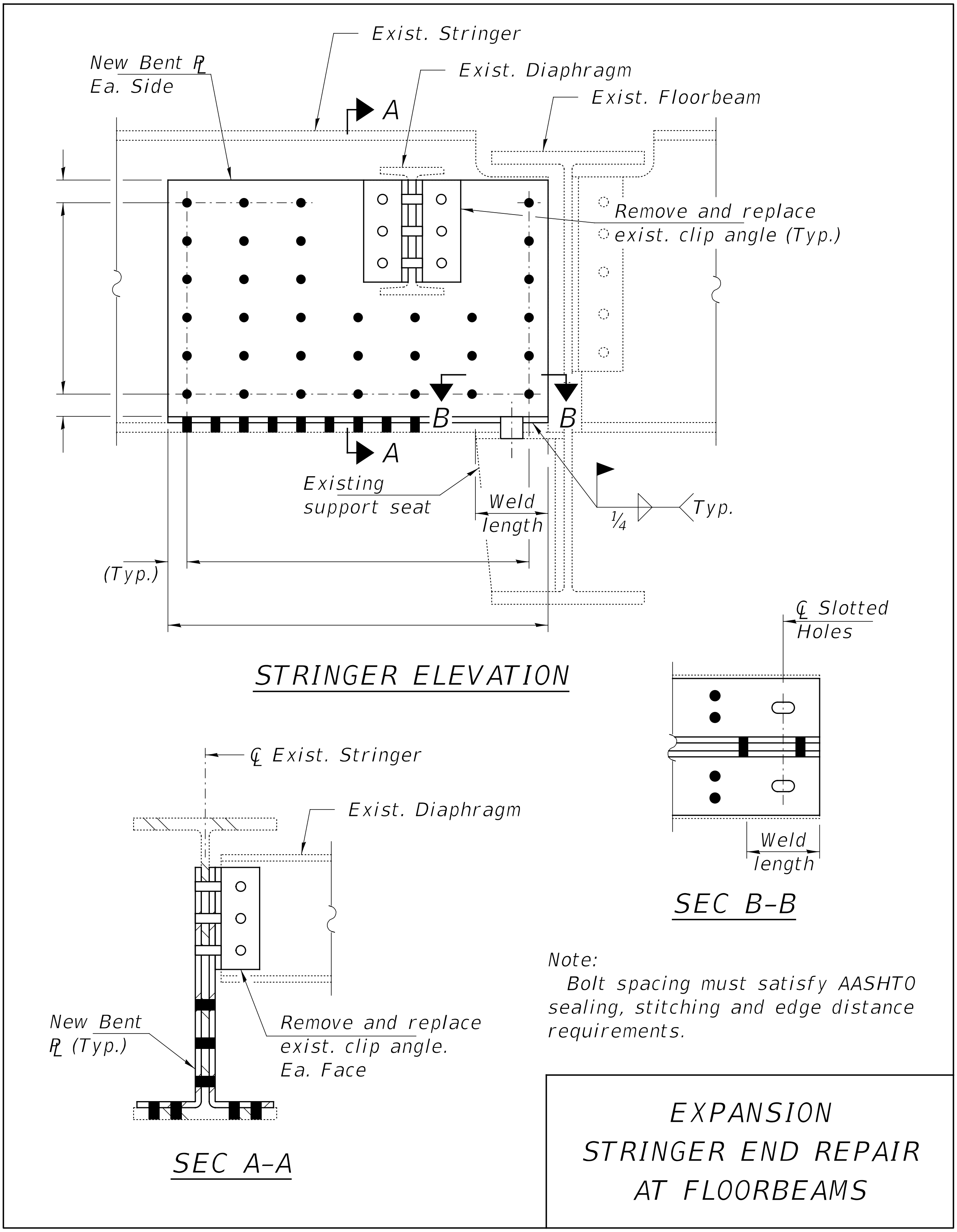


FIGURE 2.15.7-3

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.

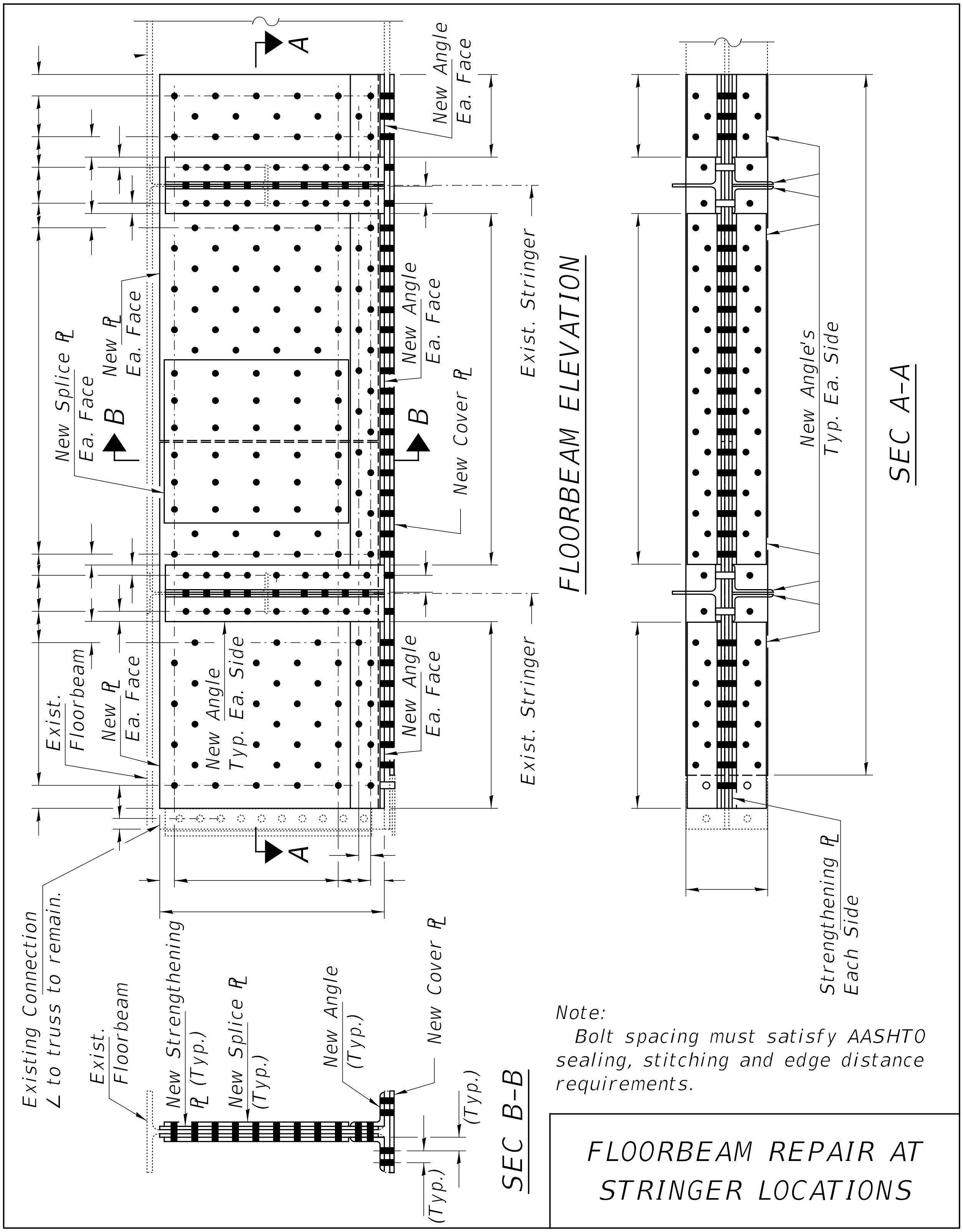


FIGURE 2.15.8-1

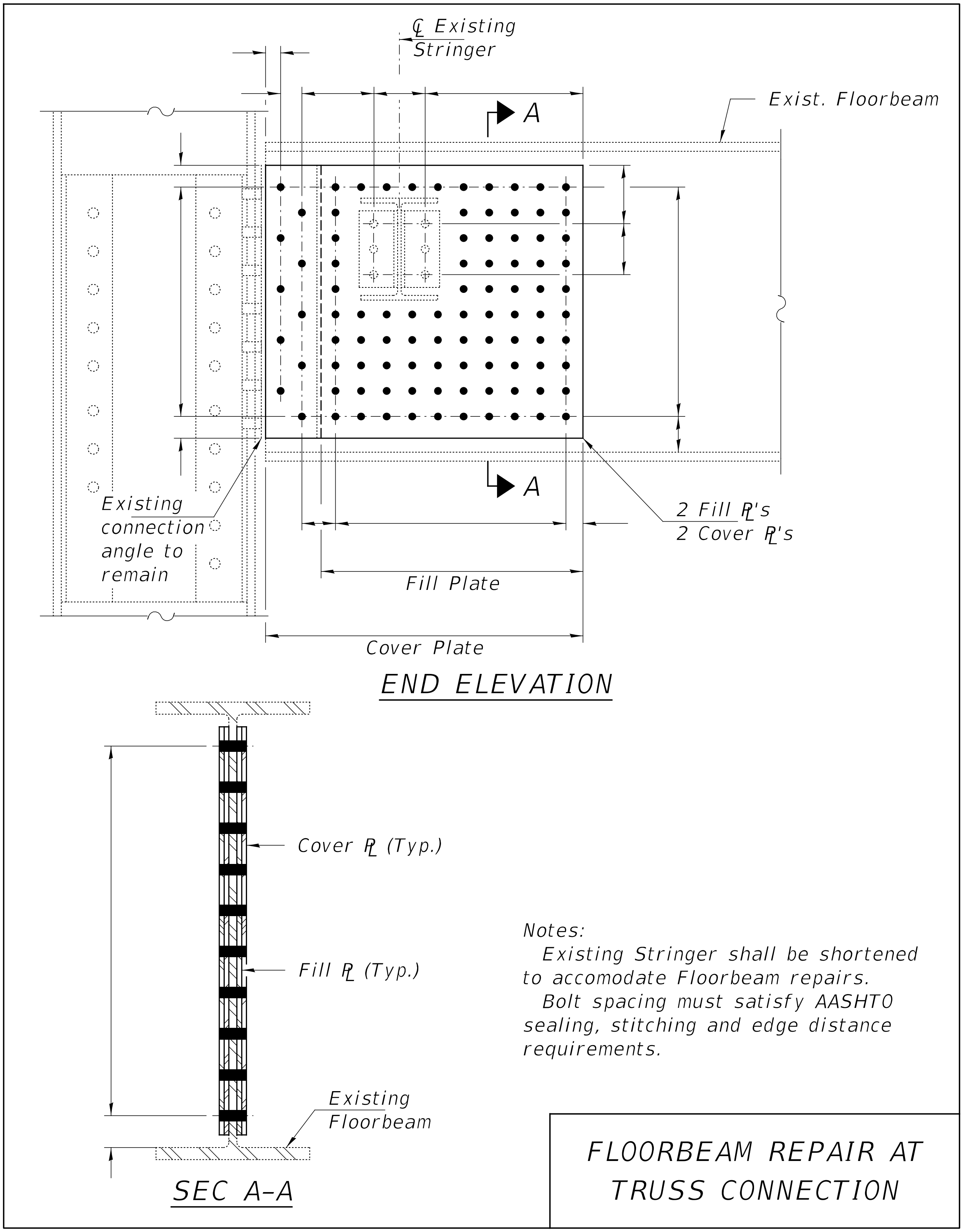


FIGURE 2.15.8-2

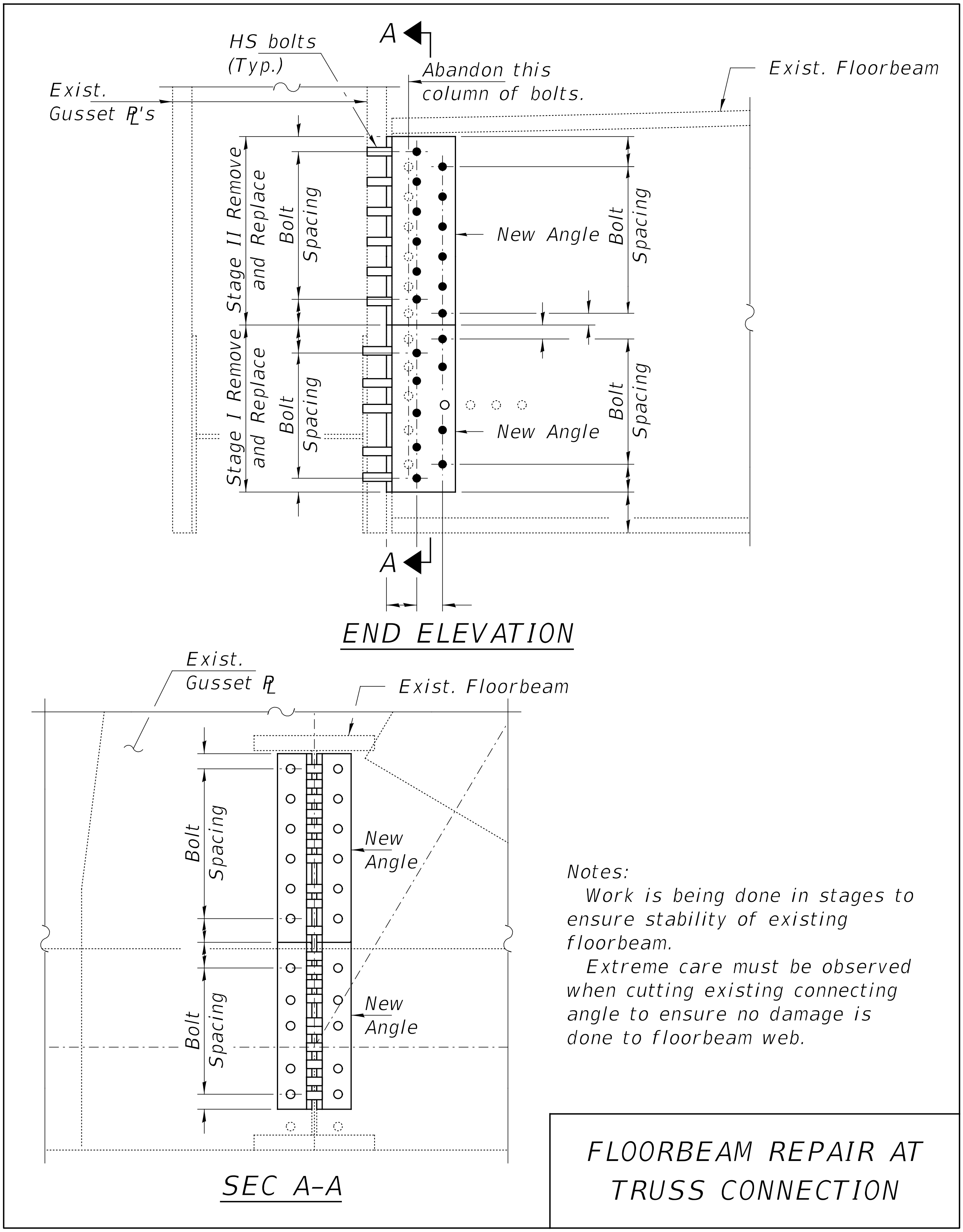


FIGURE 2.15.8-3

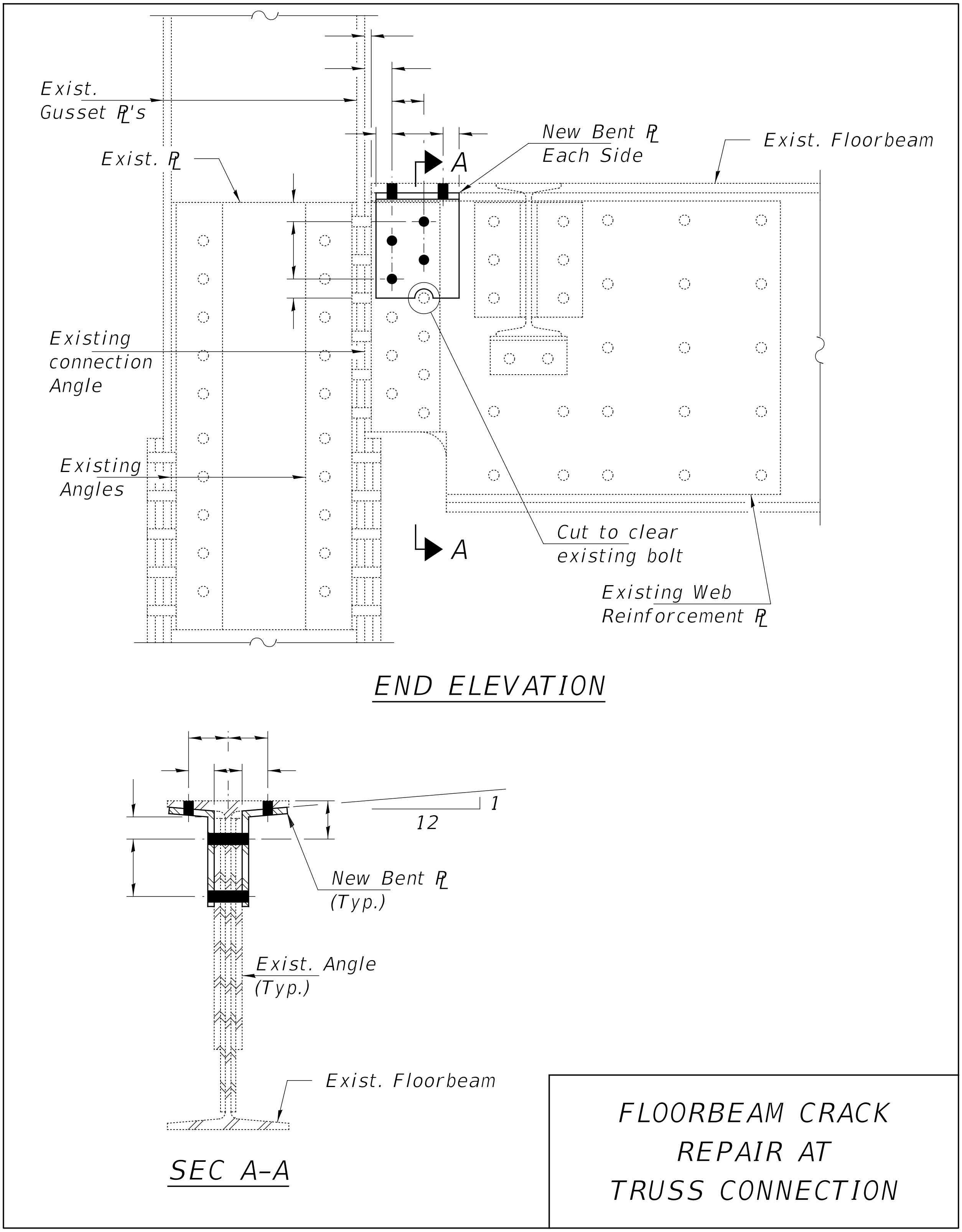


FIGURE 2.15.8-4

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:

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

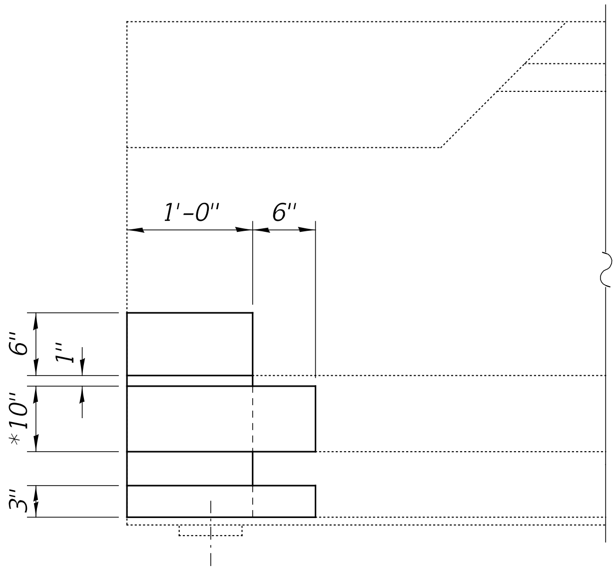
Note that major repairs typically require the installation of a preloading system as described in Section 2.14 prior to completion of the concrete repairs. If preloading is required, the FRP cannot be installed until the preloading has been removed. Minor repairs do not require a preloading.

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.

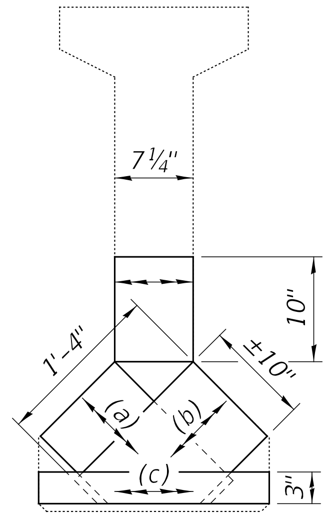
For this procedure to perform as expected it is critical that the FRP material properties shown in the special provision are satisfied, the manufacturer recommendations are carefully followed and the personnel installing the material are fully trained and experienced in this type of work. Following the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



ELEVATION



FRP TREATMENT AT END OF BEAM

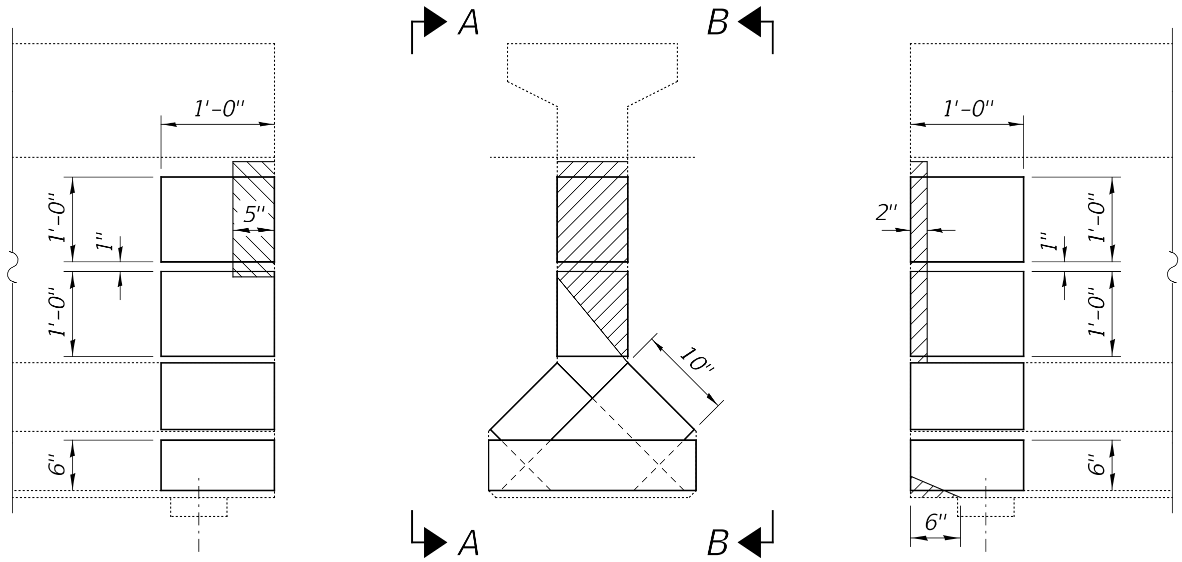
direction FRP

Note:
Place FRP in the following order (a), (b) and (c).
FRP with carbon or glass fibers allowed.

DAMAGED BEAM

PPC I-BEAM FLANGE AT BEARING REPAIR USING FRP

FIGURE 2.16.2-1



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

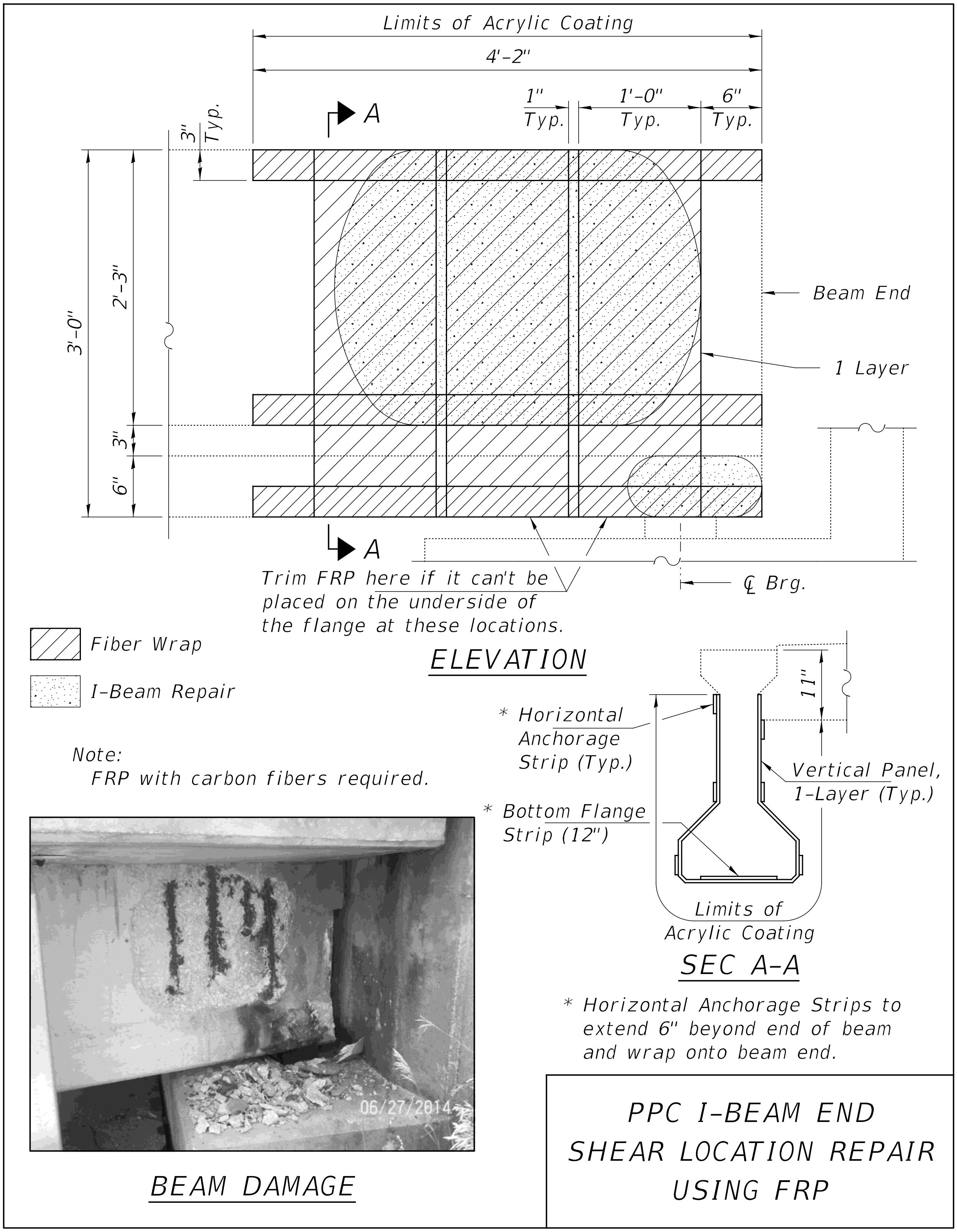


FIGURE 2.16.2-3

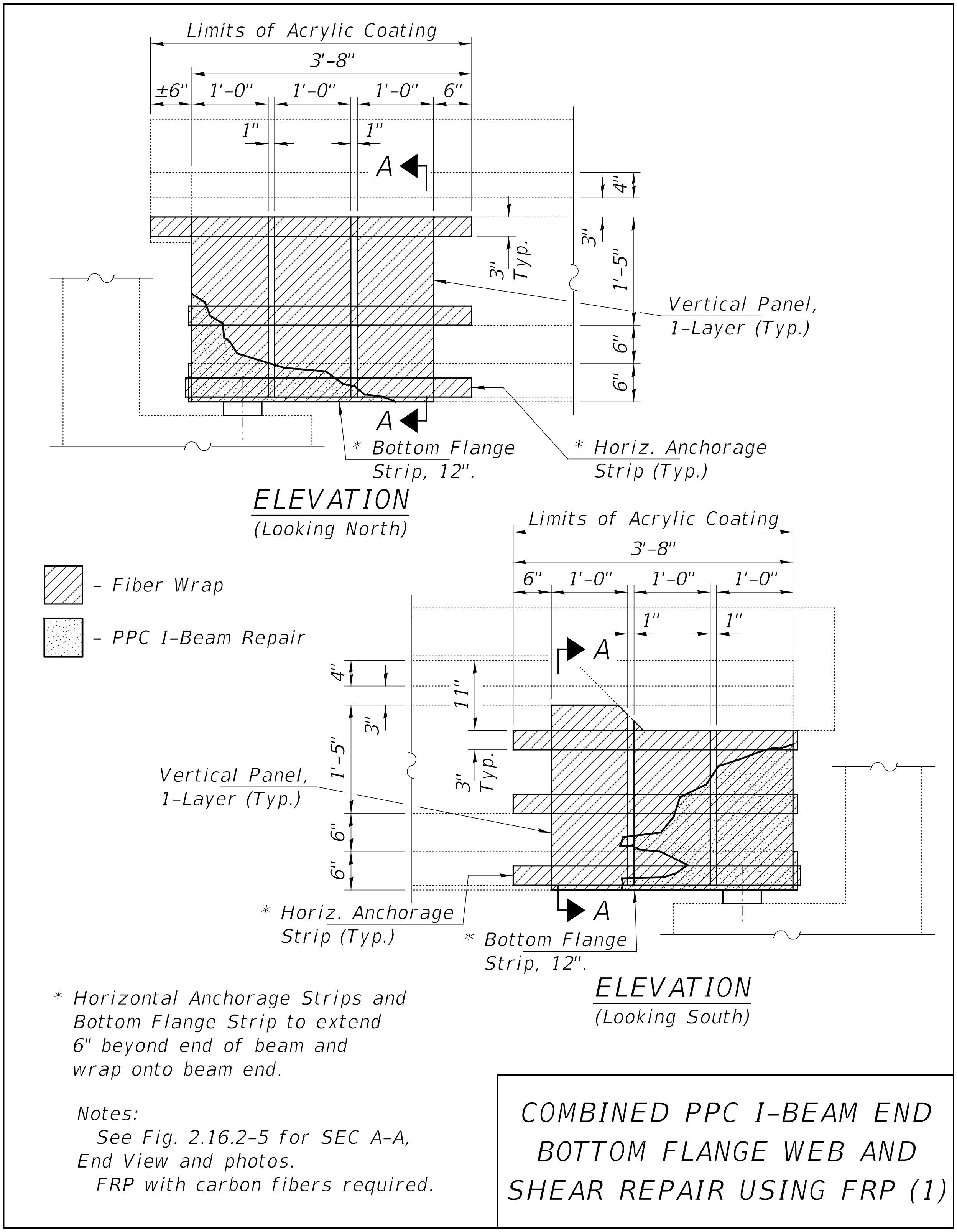
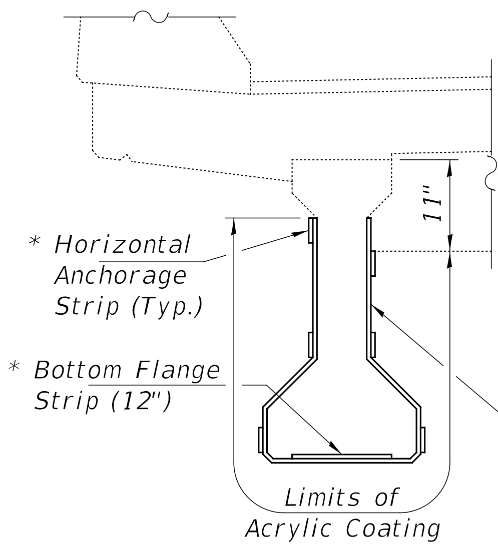
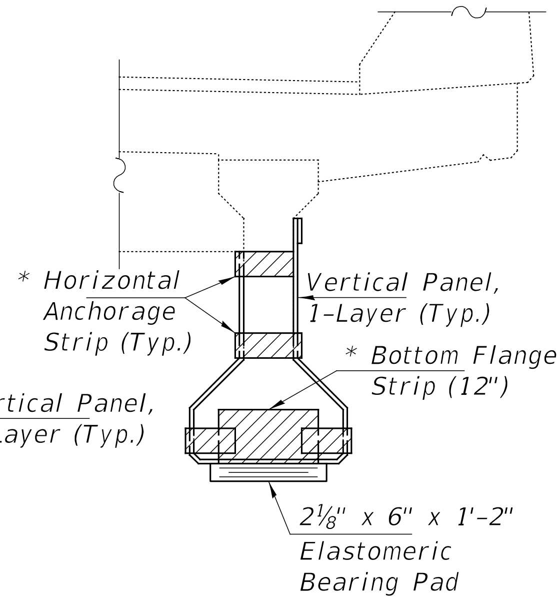


FIGURE 2.16.2-4



SEC A-A



END VIEW

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

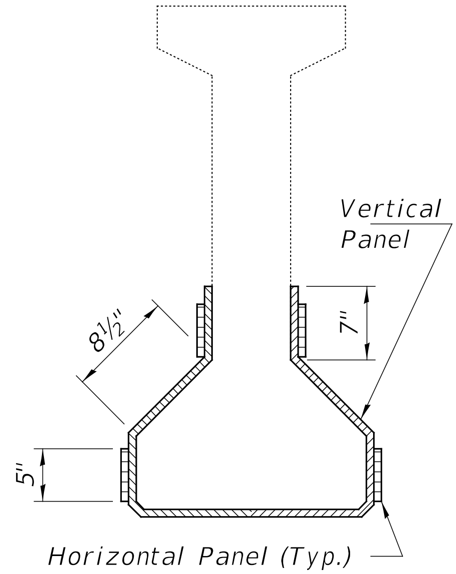
BEAM DAMAGE

BEAM REPAIR

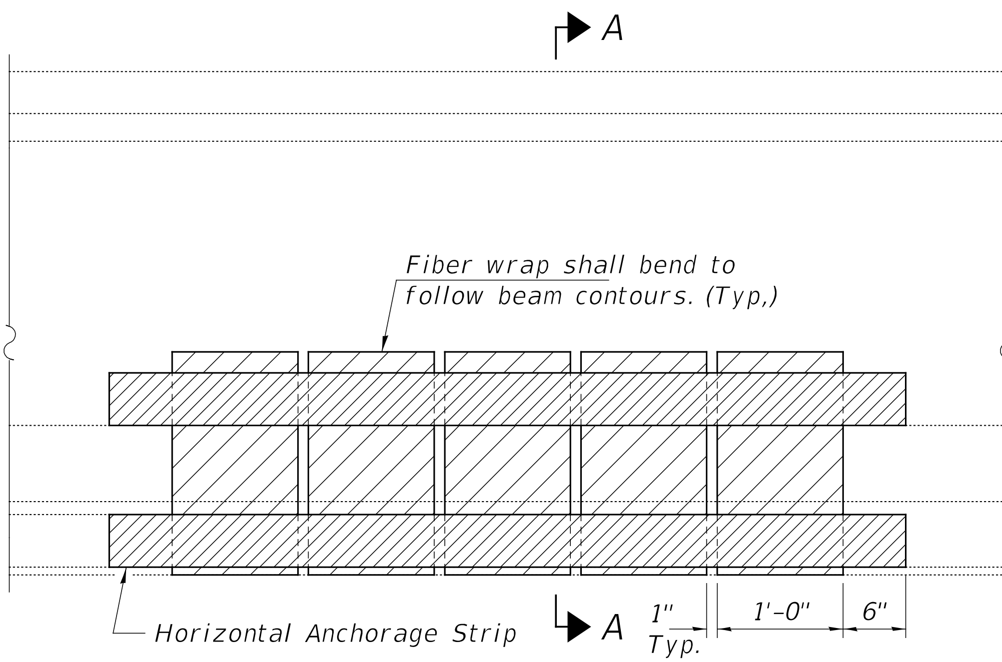
Note:
FRP with carbon fibers required.

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

FIGURE 2.16.2-5



SEC A-A



ELEVATION

Note:
FRP with carbon or glass fibers allowed.

MAJOR PPC I-BEAM REPAIR WITHIN THE SPAN USING FRP

FIGURE 2.16.2-6

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 2.16.3-1. 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.

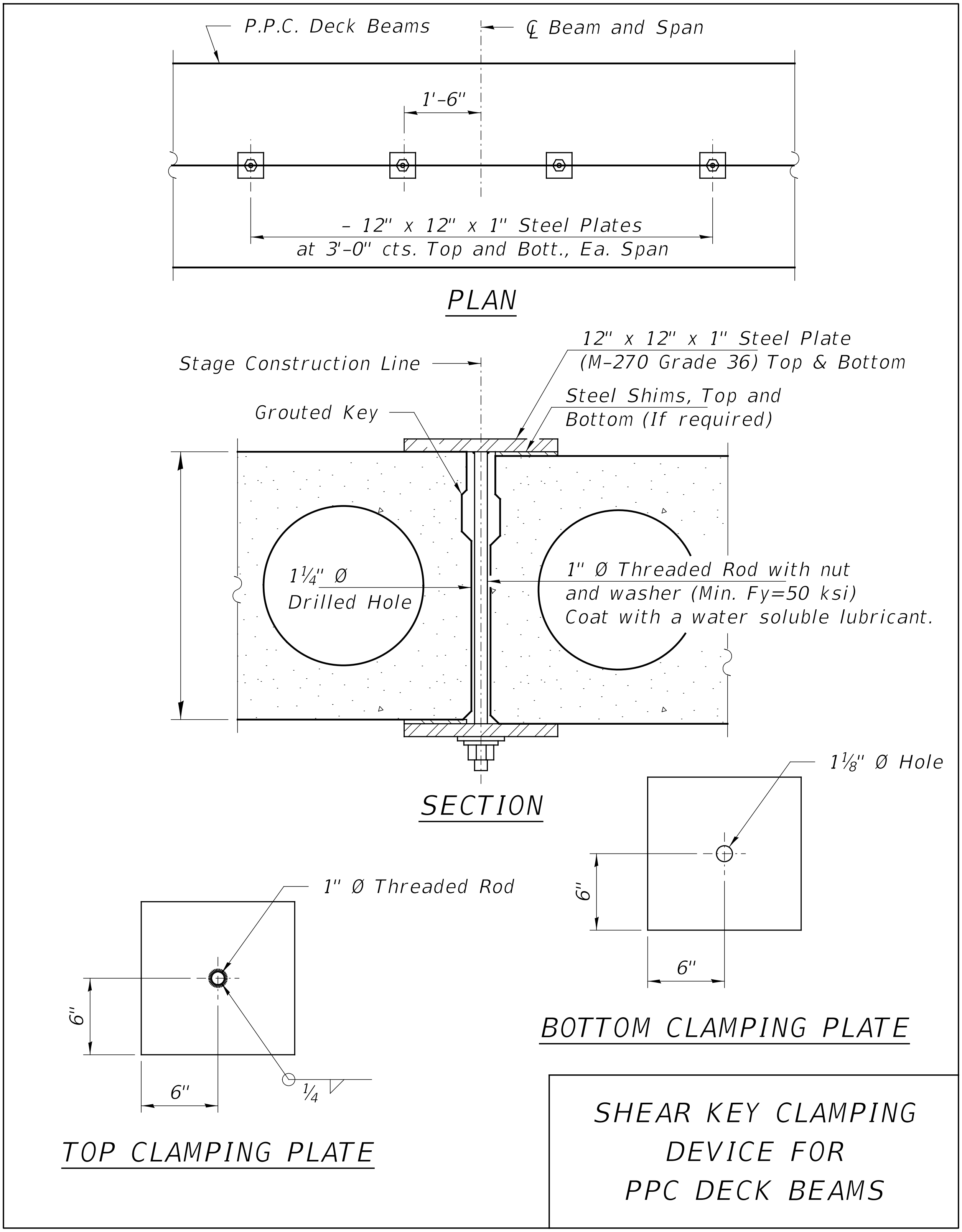


Figure 2.16.3-1

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.

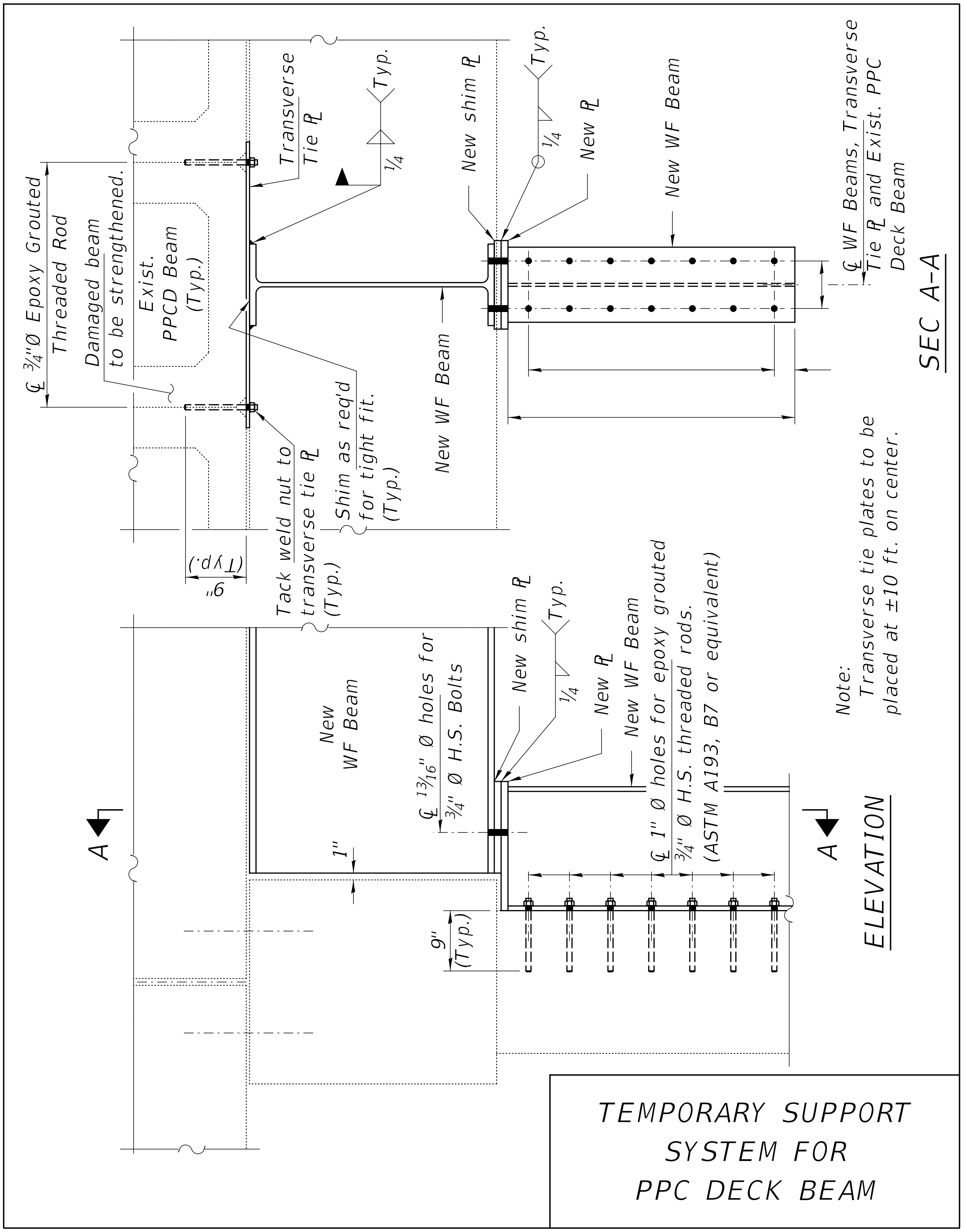


Figure 2.16.4-1

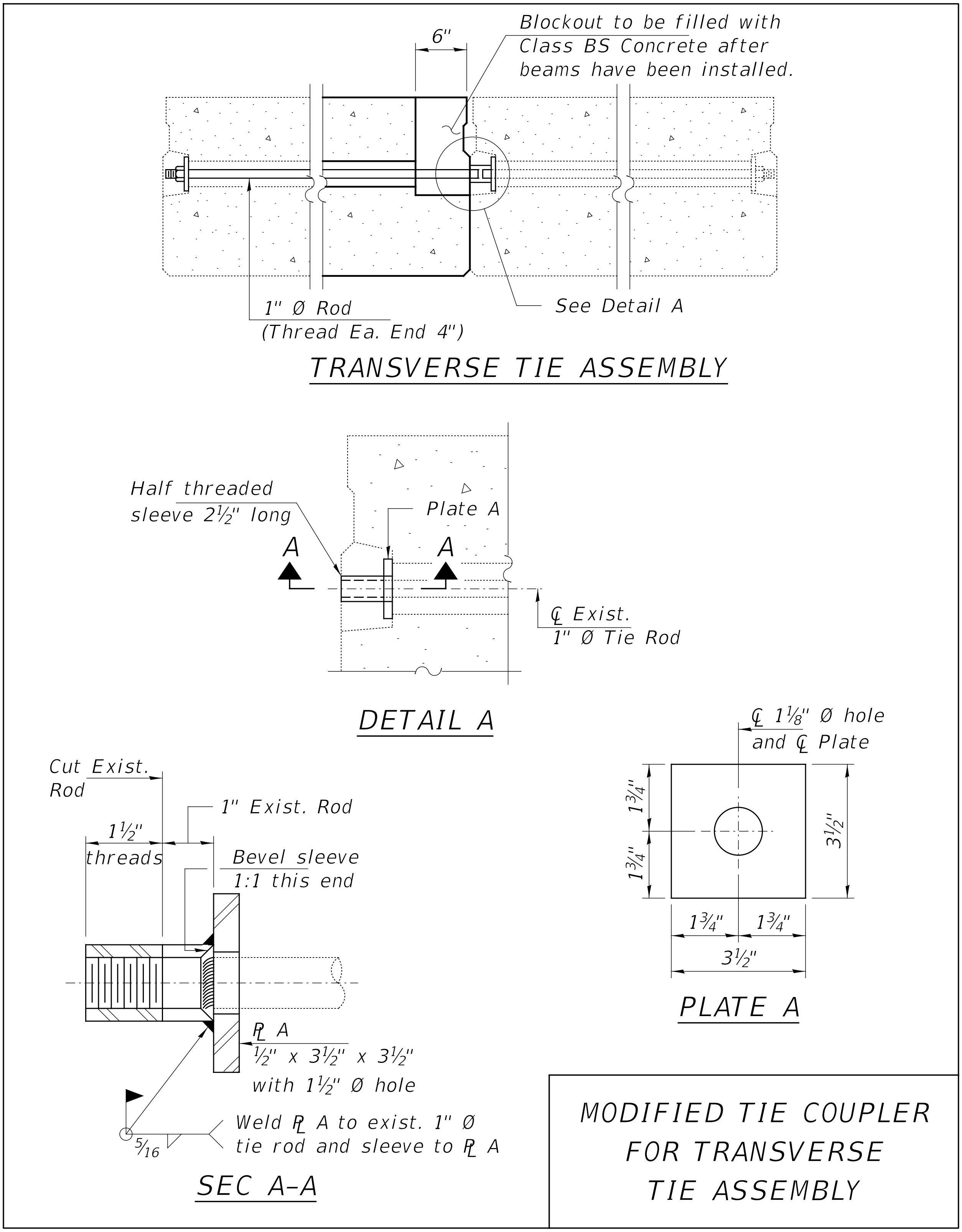


FIGURE 2.16.4-2

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.

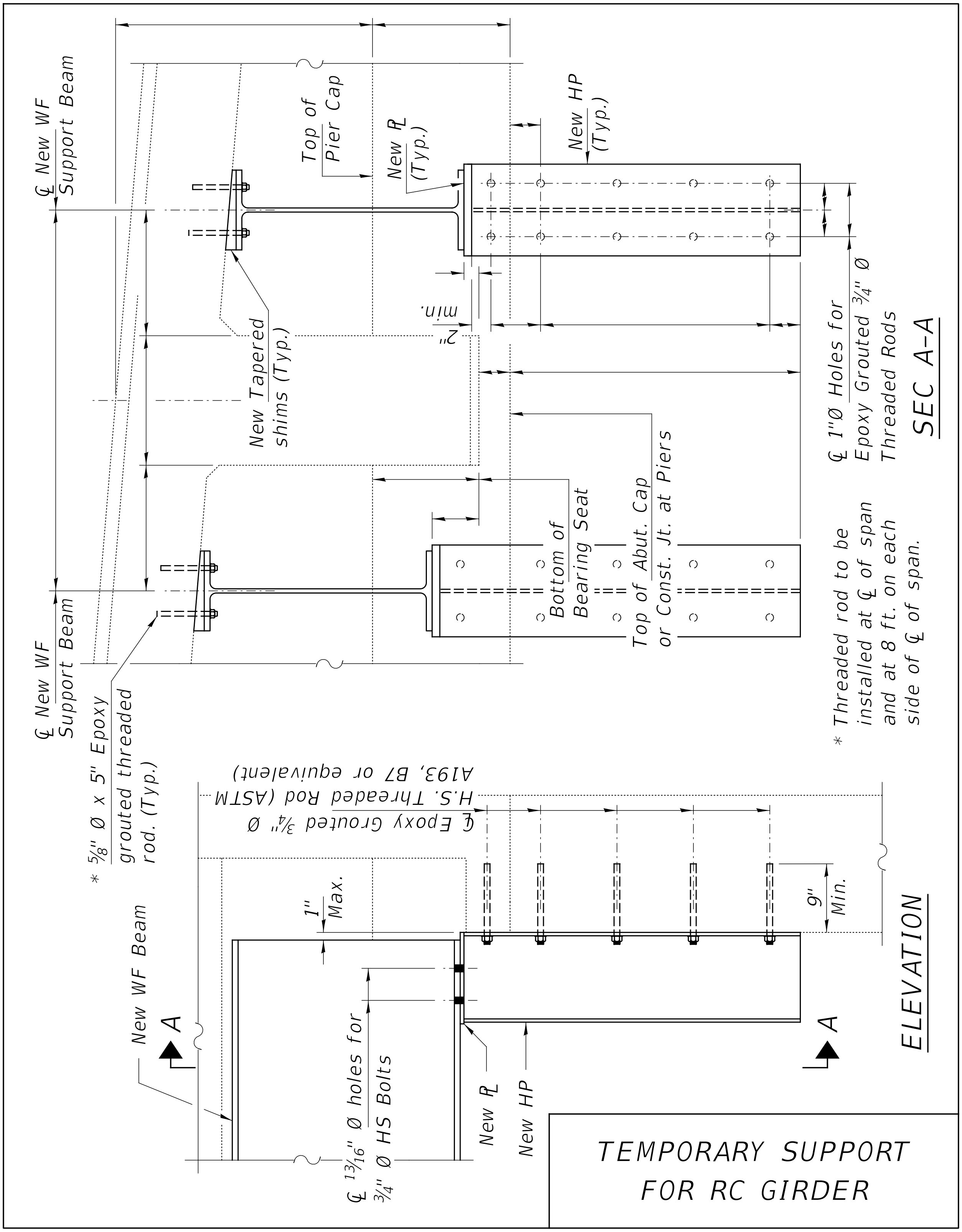


FIGURE 2.16.5-1

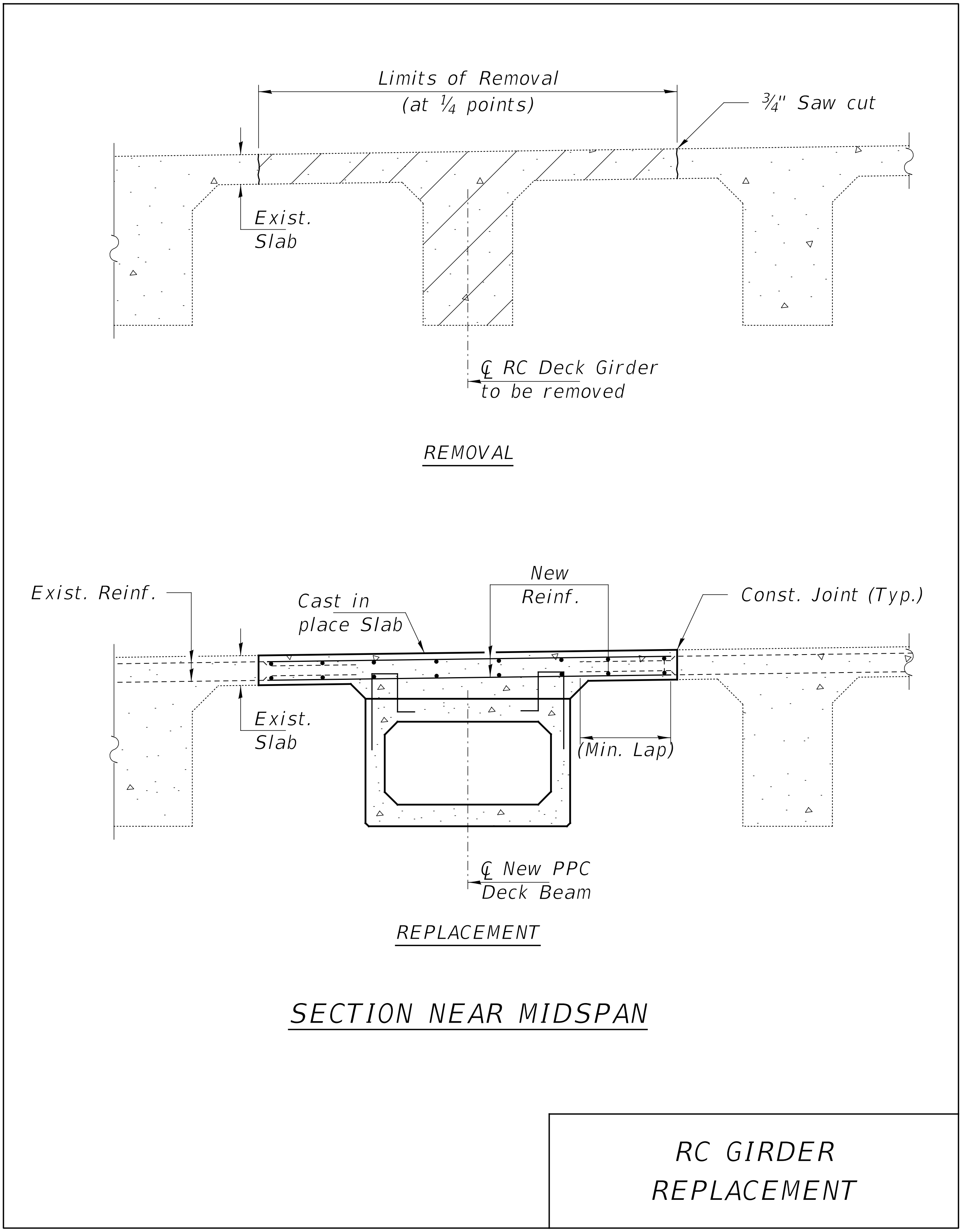


FIGURE 2.16.5-2

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.

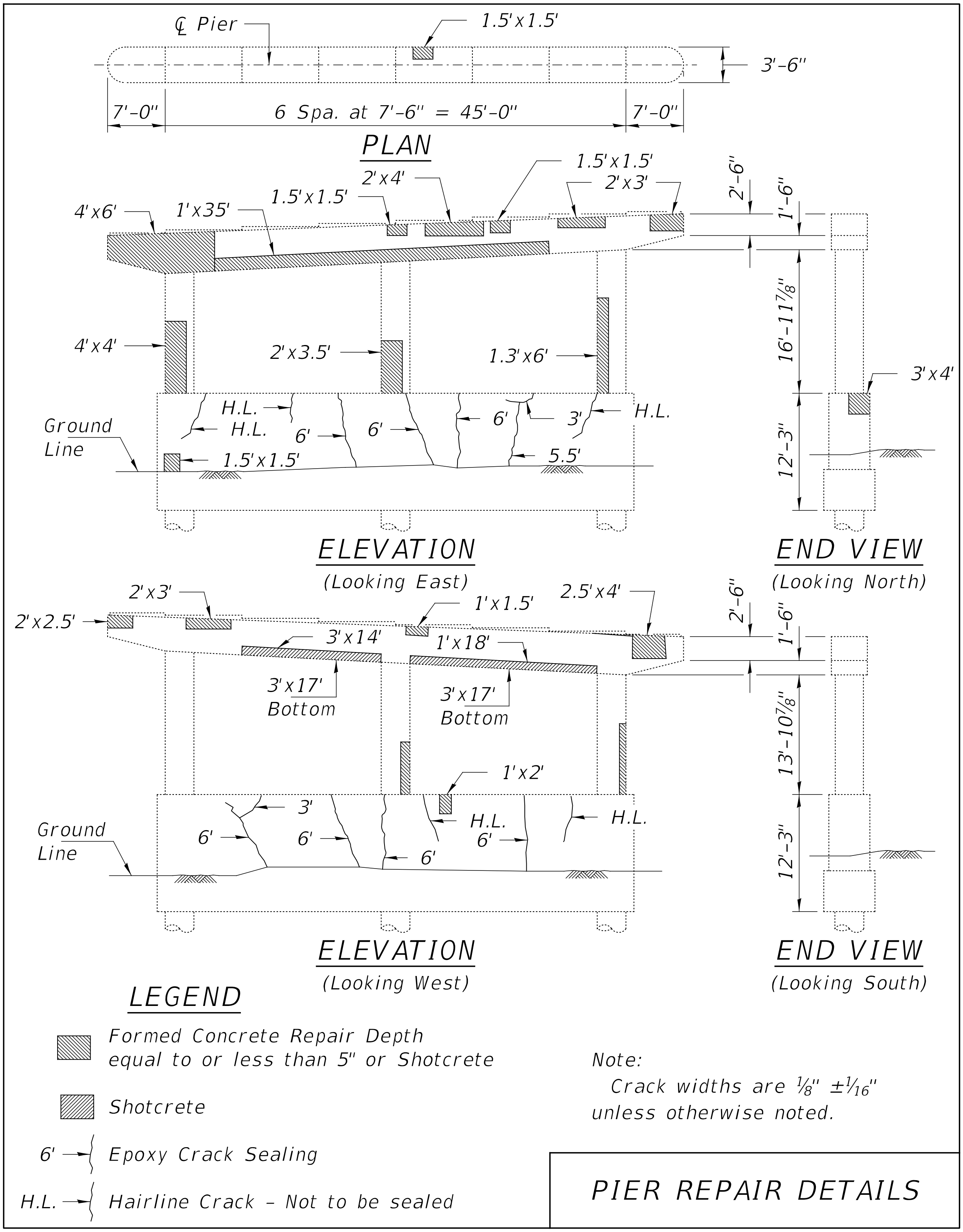


FIGURE 2.17.1-1

Substructure Repair Type	Project Quantity	Repair Location	Limits of Repair	Aesthetic Requirements / Limits / Other Comments
* Formed Concrete Repair (FCR)	Recommended method to 100 ft². Alternate with high performance shotcrete for ≥ 100 ft².	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 ft². Note: HPS should be competitive with FCR for 100 ft² to 200 ft² if all repair area is localized. HPS should be competitive regardless of locality of repairs for > 200 ft².	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 ft², but 10 ft² 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 ½". For crack > ½" 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

FIGURE 2.17.1-2

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 GBSP 53 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:

a) Formed concrete repairs with depths of repair less than or equal to 5 inches

b) 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.

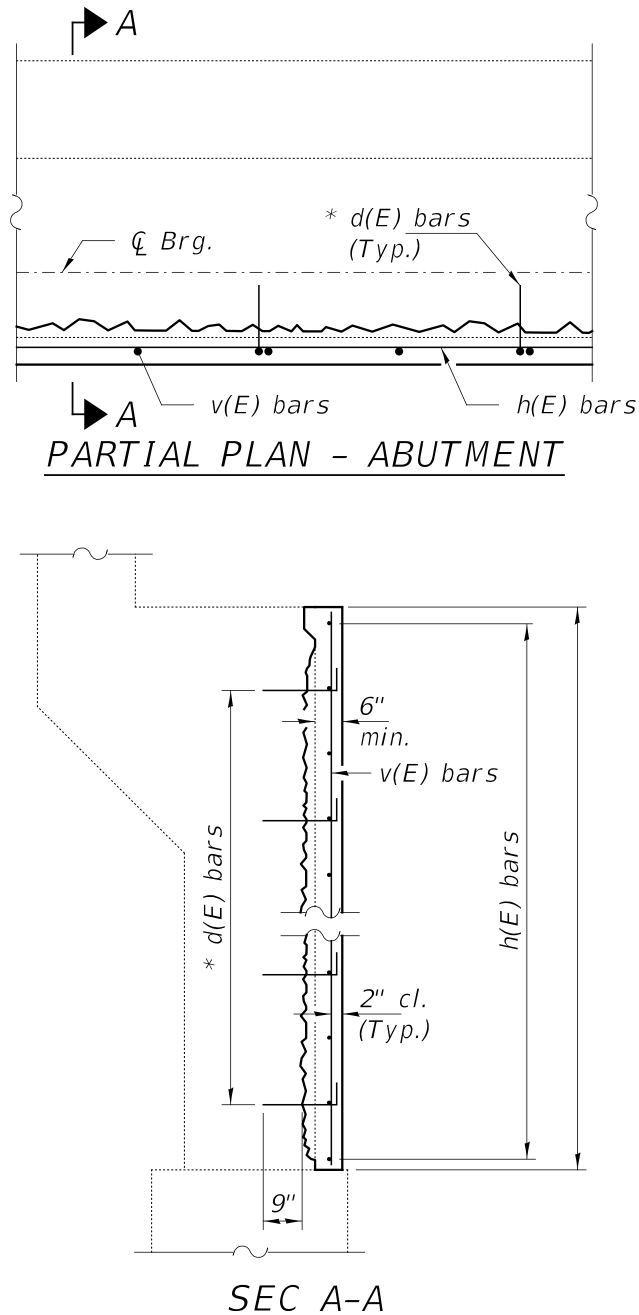
2.17.5 Pneumatically Placed Mortar

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).



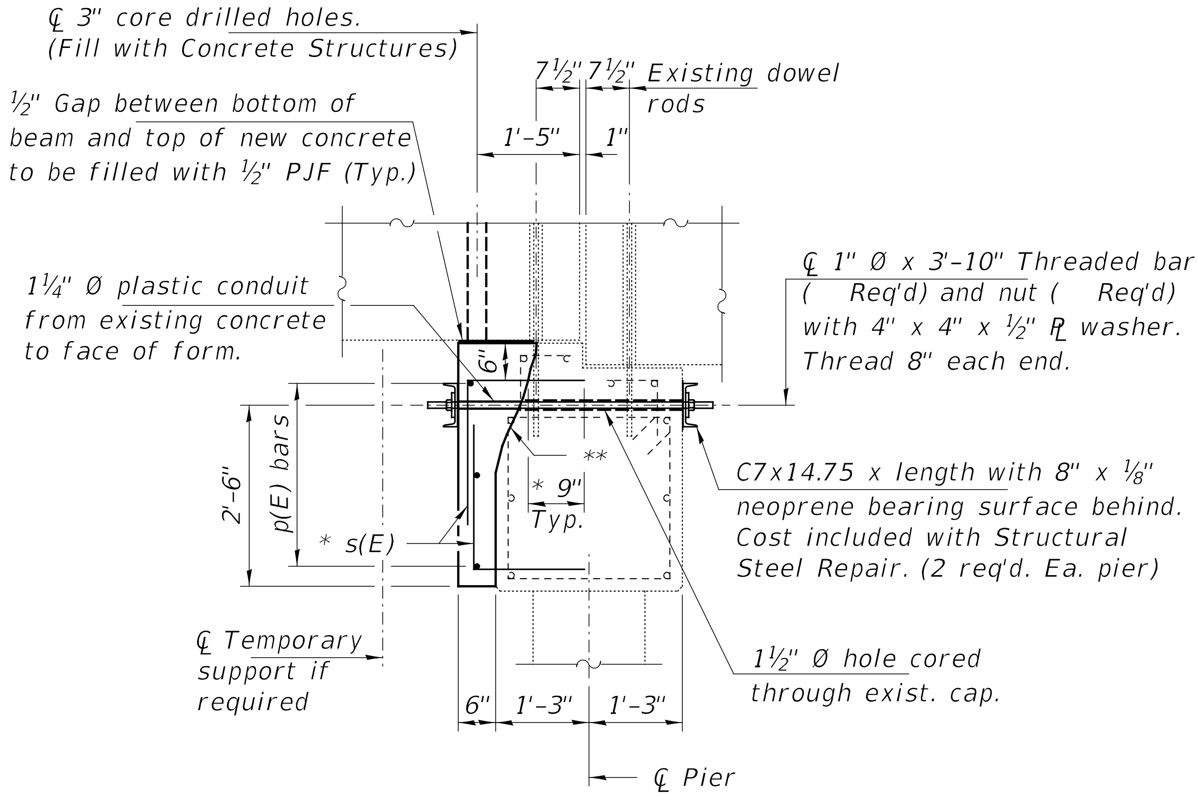
* Epoxy grout bars in 9" min. holes according to Article 584 of the Standard Specifications. Cost included with Reinforcement Bars, Epoxy Coated.

WALL ENCASEMENT

FIGURE 2.17.6-1

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.



SECTION THRU PIER

Liberally grease and wrap threaded bar in waxed paper prior to insertion.

* Drill and epoxy grout 9" min. into existing cap according to Article 584 of the Standard Specifications. Trim leg to maintain minimum clearance. Cost included with Reinforcement Bars, Epoxy Coated.

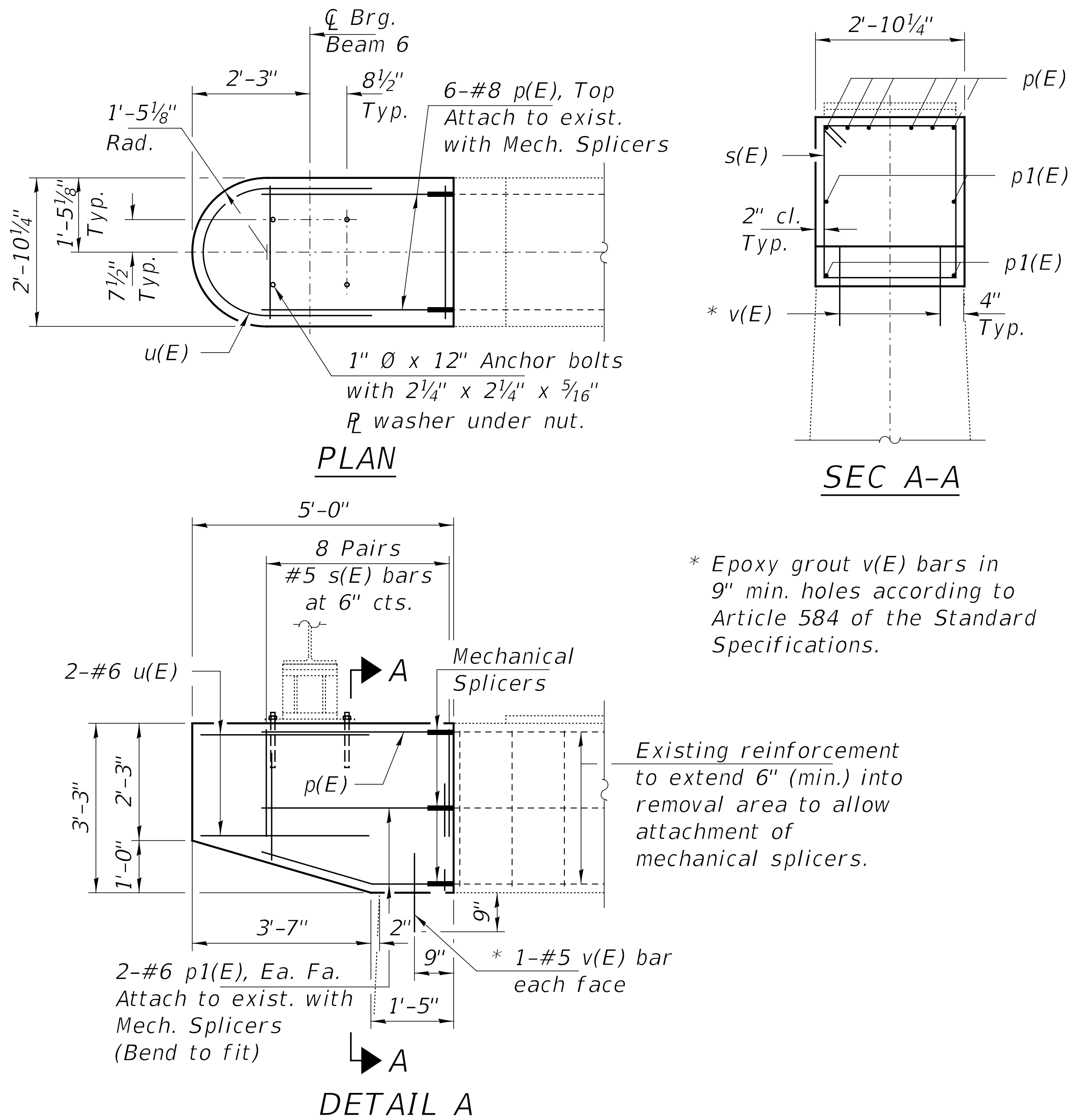
** To ensure stability of the existing beams, the removal shall not go deeper than the existing dowel rods. Cost included with Concrete Structures.

PIER CAP ENCASEMENT FOR PPC DECK BEAMS

FIGURE 2.17.7-1

2.17.8 Partial Substructure Removal and Replacement

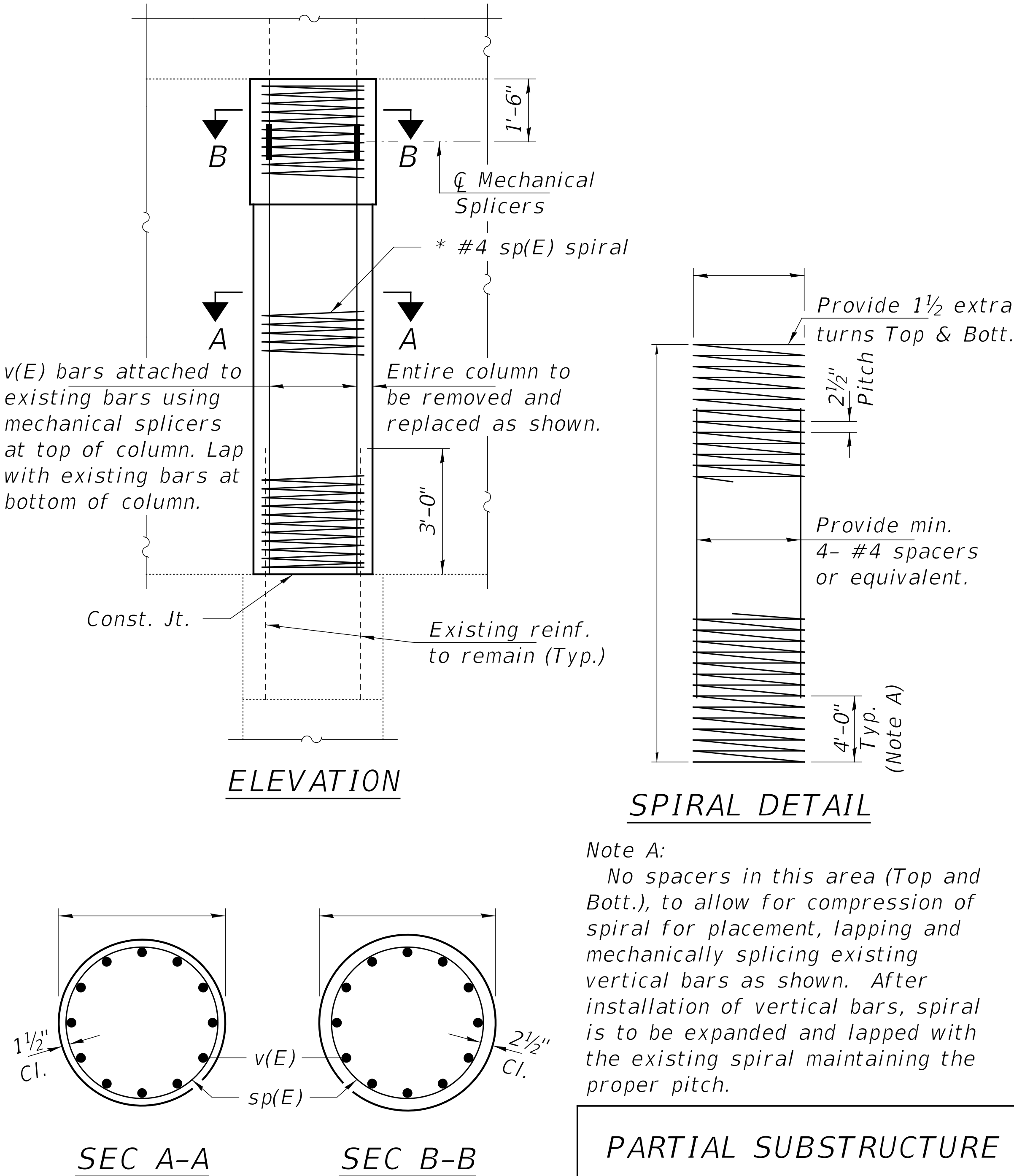
Sometimes the damage is significant enough that the full removal and replacement of a section of the substructure element is necessary. These cases normally involve the removal and replacement of a pier cap overhang, pier column or full pier cap. In all these cases the standard practice is to replace 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



Note A:
No spacers in this area (Top and Bott.), to allow for compression of spiral for placement, lapping and mechanically splicing existing vertical bars as shown. After installation of vertical bars, spiral is to be expanded and lapped with the existing spiral maintaining the proper pitch.

PARTIAL SUBSTRUCTURE REMOVAL AND REPLACEMENT (COLUMN)

FIGURE 2.17.8-2

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 GBSP 97 Folded/Formed PVC Pipe Liner, GBSP 98 Cured-in-Place Pipe Liner and GBSP 99 Spray Applied Pipe Liner.

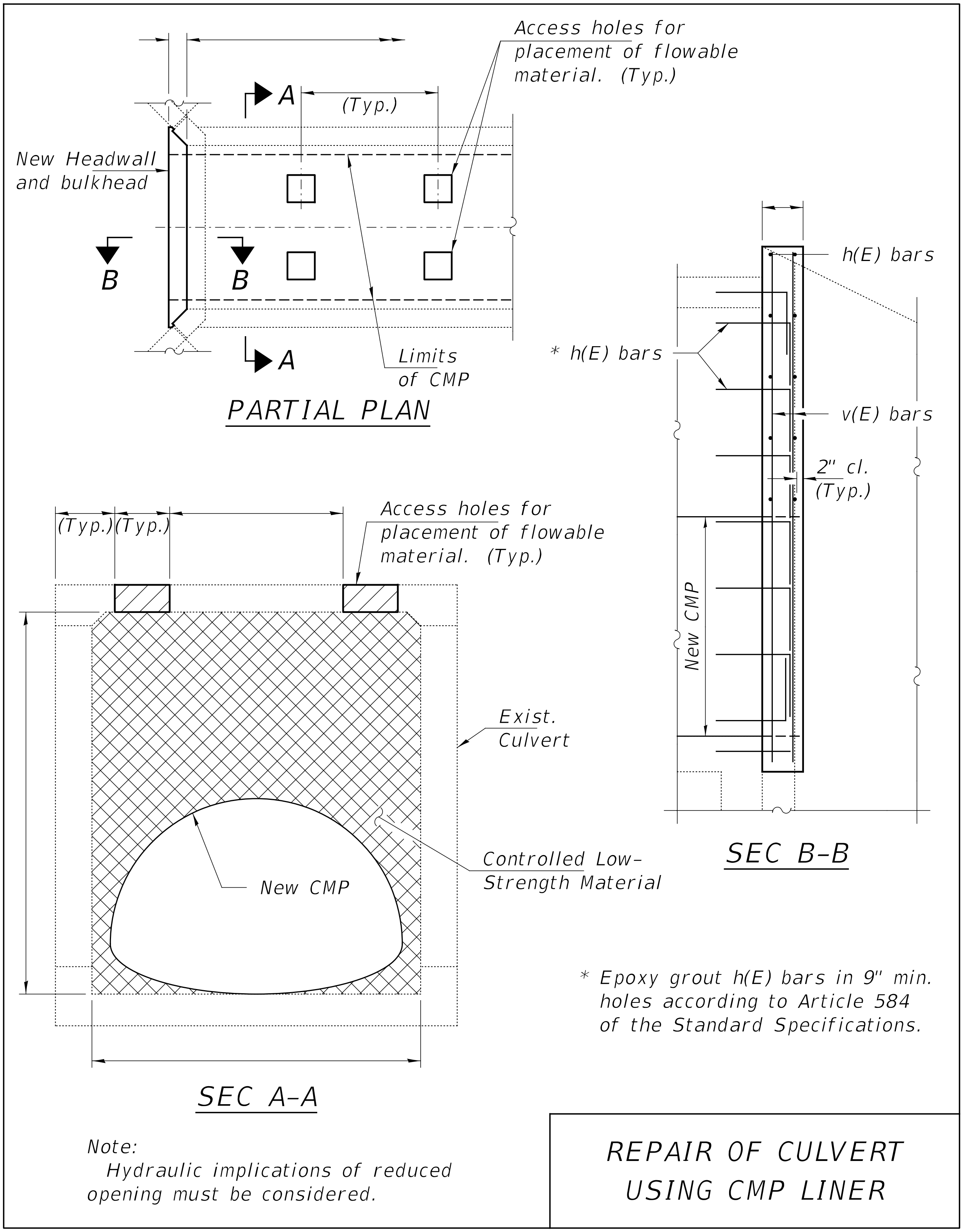


FIGURE 2.18.2-1

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 has 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:

a) Legal Load Only (LLO) maximum, regardless of the calculated strength of the new slab (no permit loads).

b) Culvert Condition rating shall continue to be based on the existing deteriorated condition, not the temporary slab condition.

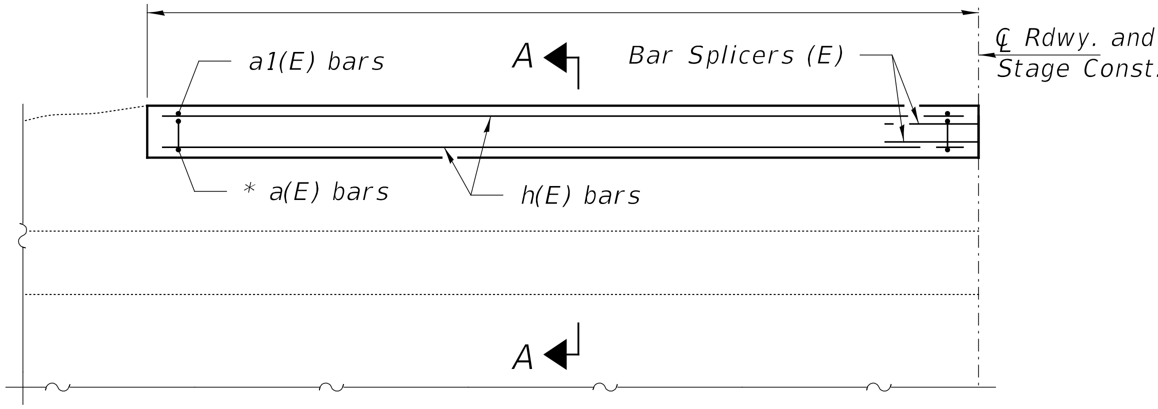
c) Bridge Status, Illinois Structure Information System Item 41, shall be changed to “5” Open, Temporary Measures in Place.

d) 6 month Special Inspections shall be performed to monitor the temporary slab, existing culvert walls and fill behind the walls / under the new slab.

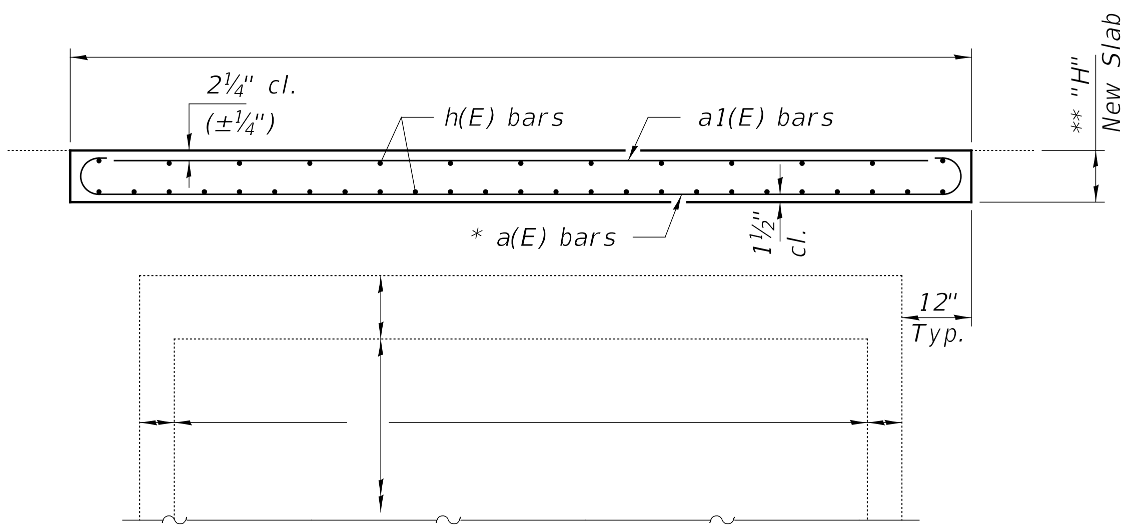
e) Place a high priority on programming the culvert for replacement.

f) 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.



HALF LONGITUDINAL SECTION



SEC A–A

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

** Remove "H" of existing overlay and replace with "H" concrete slab as shown. Removal is to be limited to the "H" shown. Slope to match roadway. Cost of removal included with Hot-Mix Asphalt Surface Removal.

H = Thickness of slab required by design.

SLAB OVER CULVERT REPAIR (TEMPORARY)

FIGURE 2.18.3-1

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.

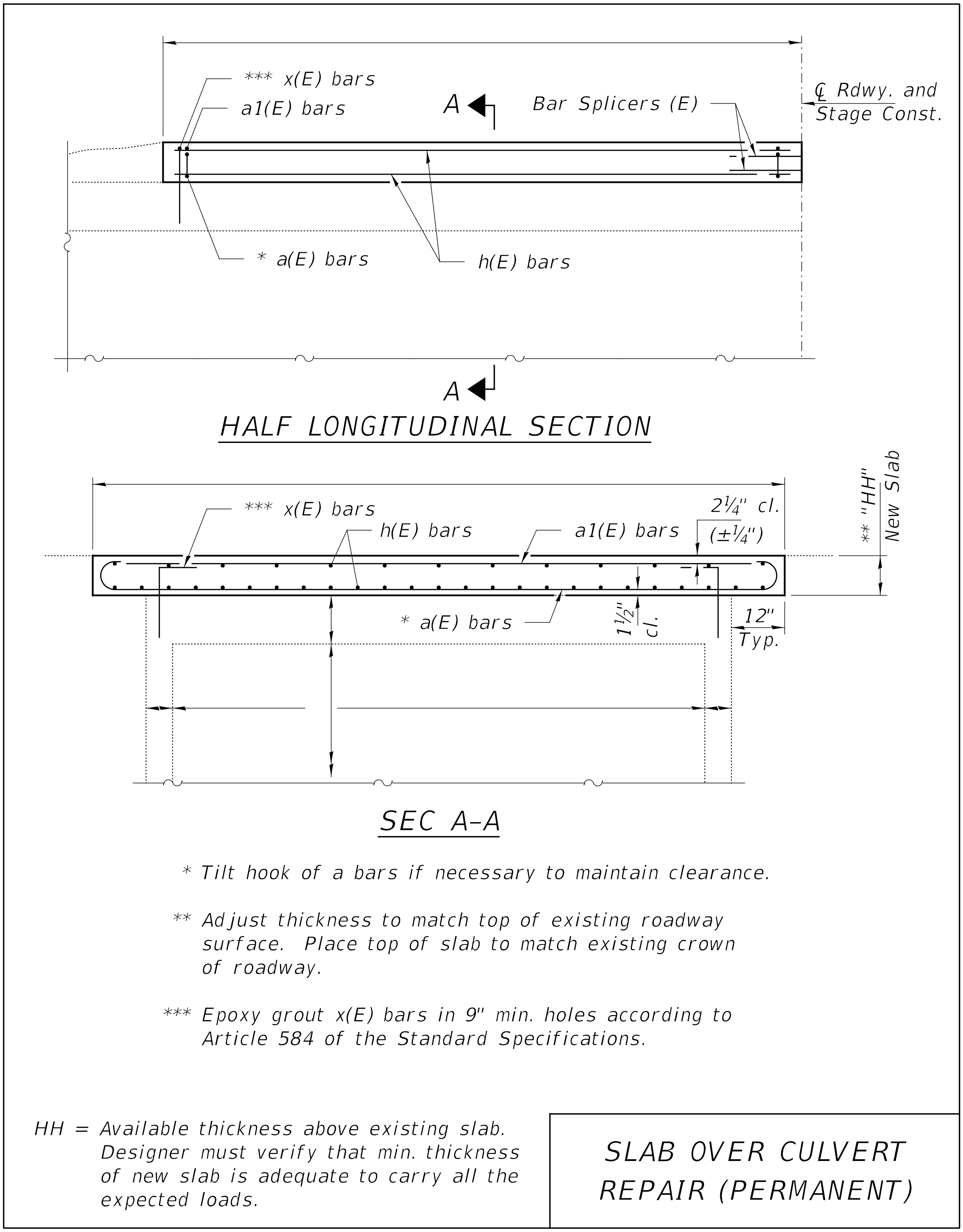


FIGURE 2.18.4-1

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.

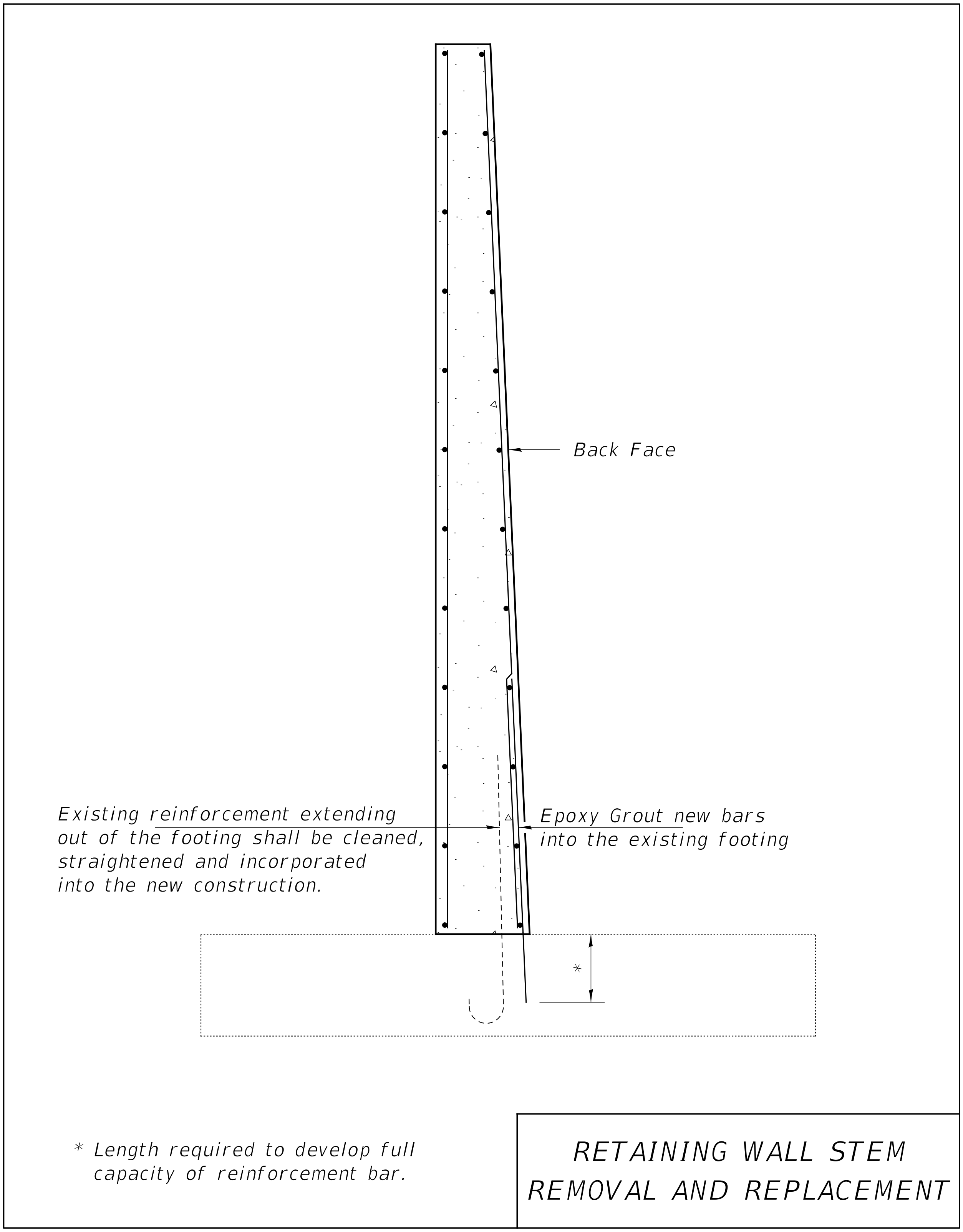


FIGURE 2.19.1-1

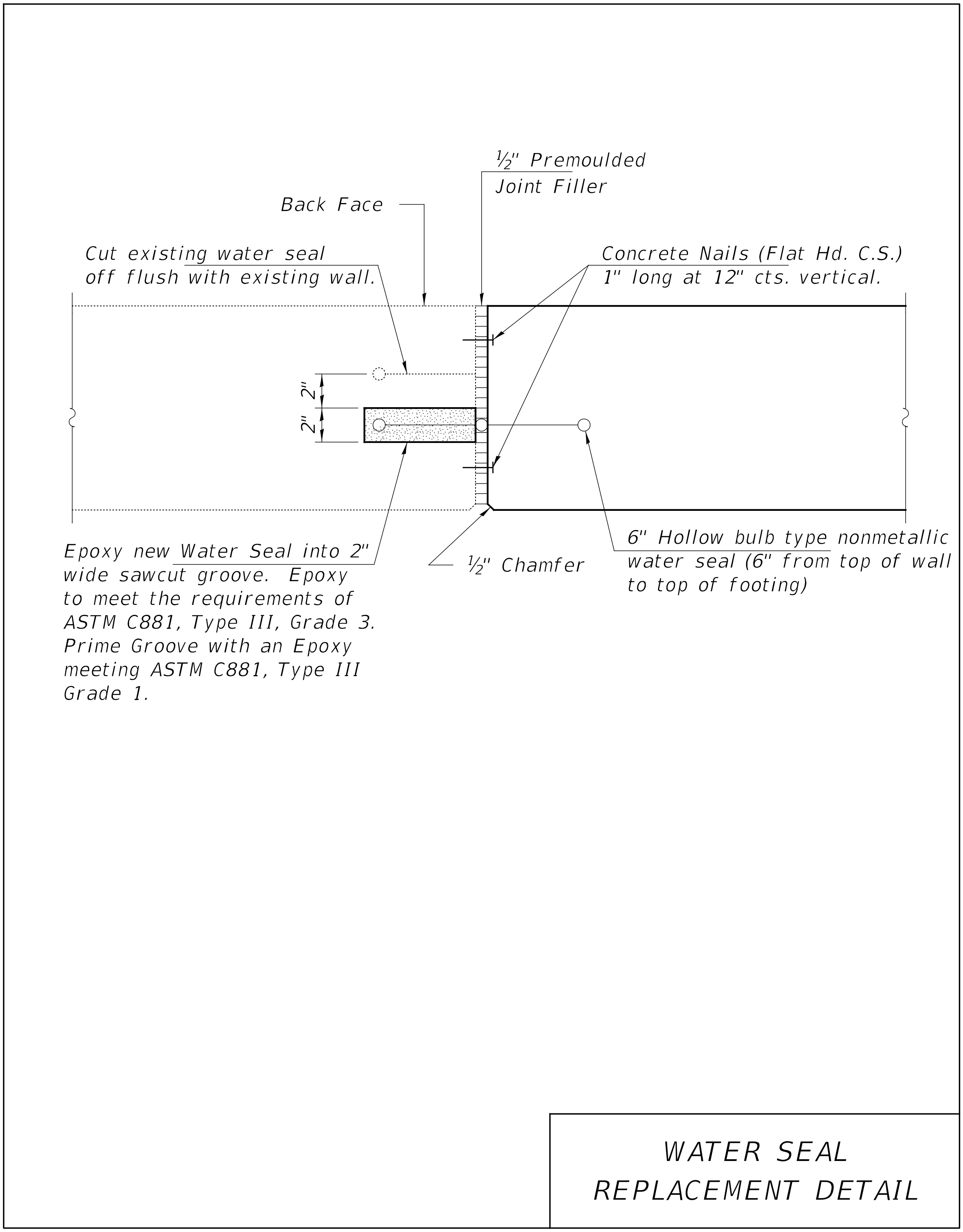


FIGURE 2.19.1-2

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.

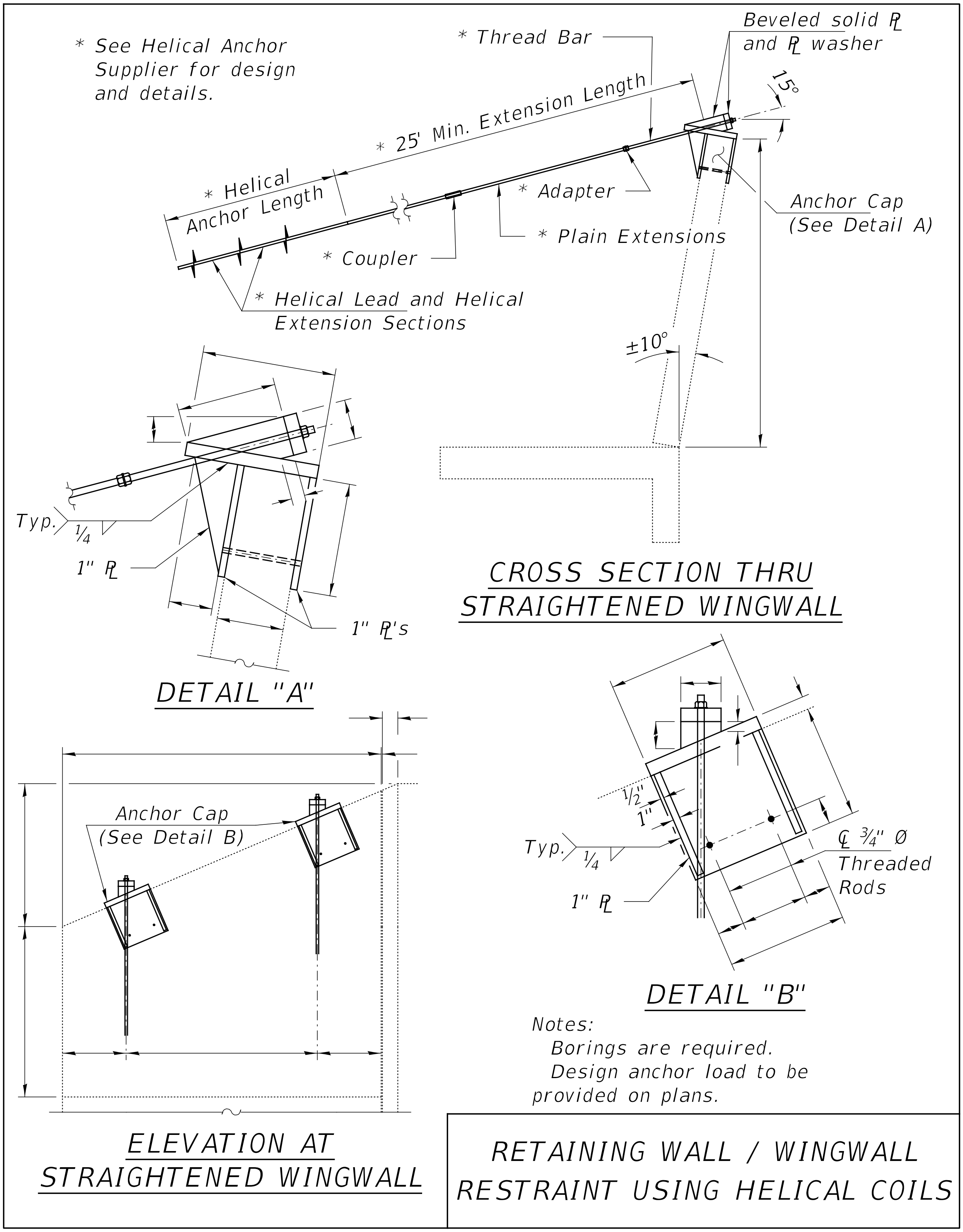


FIGURE 2.19.2-1