Bridge Preservation Guide

March 2024

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Chapter 1 GENERAL DISCUSSION

TxDOT's bridge preservation efforts involve various maintenance or repair actions to extend the life of a bridge. This guide presents a detailed discussion of TxDOT's recommended practices and it is intended to serve as a stand-alone resource for TxDOT staff and consultants planning preservation projects, performing filed assessments, or preparing contract documents for bridge preservation projects.

1.1 General Discussion

The Federal Highway Administration (FHWA) defines preservation as "actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good or fair condition; and extend their service life". Preservation activities generally do not add capacity or structural value but do restore the overall condition of the bridge.

According to FHWA, maintenance describes work that is performed to maintain the condition of the transportation system or to respond to specific conditions or events to restore the highway system to a functional state of operation. Maintenance is a critical component of an agency's asset management plan and is comprised of both condition-based and preventive or cyclic maintenance. Condition-based maintenance encompasses work that is performed in reaction to an event, season, or overall deterioration of the bridge. This work is often driven by findings from routine inspection or to address damage resulting from impact or another event. Preventive maintenance is proactive and is performed with schedules to cost-effectively extend the useful life of bridges.

Even though bridge preservation and maintenance activities employ many similar approaches and methods, the work scope of bridge preservation projects is usually larger and more complex than routine maintenance projects. Large scale preservation work is usually not performed by TxDOT maintenance crews.

Figure.1-1- Bridge action categories (From FHWA)

Example of preservation activities are summarized in Table 2-2. When conducted, bridge condition survey reports and corresponding preservation plans must be uploaded to AssetWise by TxDOT staff.

1.2 Bridge Maintenance and Improvement Program (BMIP) VS Bridge Preventive Maintenance (BPM)

The BMIP is managed by Bridge Division and BPM is managed by Maintenance Division. Since the two programs often have overlapping work scope, it is often difficult to decide which program to submit candidate bridges. In general, BMIP project work scopes are larger and intended to address all deficiencies at a bridge whereas BPM projects may be focused to a specific deficiency or type of deficiency on a bridge. When uncertain, submit the bridges for both programs and Bridge Division will coordinate with Maintenance Division for project selection.

1.3 BMIP Project Evaluation

Typically, the routine inspection reports do not have adequate information for developing preservation plans for the bridges. Detailed field assessments or condition survey reports, with proposed repair and rehabilitation quantities, and estimated costs, are prepared and sent to Bridge Division for review during the annual program call. Project cost, traffic control, scope, and replacement costs are evaluated during the program call.

Chapter 2 BRIDGE DECKS

This discussion applies to concrete bridge decks. Steel and timber decks are not covered in this guide.

2.1 Deck Assessment Methods

Beyond visual inspection and sounding using a hammer or chain drag, advanced assessments methods may be used. However, the impacts of travel disruption and costs associated with the proposed evaluation method should be considered against the value of the evaluation technique used. For decks that have clear visual indicators of widespread advanced deterioration, advanced assessment methods, e.g., the use of ground penetrating radar (GPR), half-cell potential testing, or coring, are generally not necessary.

2.1.1 Visual Observations

Observations from a moving vehicle are not sufficient to detect small cracks on the deck. The preferred practice is to walk the bridge for a more though visual examination. Visual observation is the primary method of deck evaluation. Visual observations should include evaluation of the top and bottom surfaces.

Transverse cracks are common on bridge decks and may be attributed to various deficiencies including concrete material issues, insufficient curing or other construction issues, or environmental issues. Generally, transverse cracking alone is not cause for structural concern but unaddressed cracks in a deck can led to a reduced service life due to corrosion of reinforcement.

Map pattern cracking (consisting of closely spaced intersecting transverse and longitudinal cracks) and locations of previous repairs (asphalt or concrete patches) are indicators of more advanced deterioration. This kind of cracking may eventually lead to punch-through failure. One example of punch-through deck failure is shown in Figure 2-1.

Figure 2-1: IH35 Bridge Deck Failure

2.1.2 Sounding

Sounding with a hammer or chain drag, or electromechanical sounding is performed to help determine the extent of deck deterioration that cannot be determined. On bridges with previous deck spall repairs, sounding near the repair boundary should be performed to identify other areas of possible delamination. Delamination creates a lower pitch sound that is notably different from the higher pitch ringing of a deck without delamination. While an ASTM (D4580) standard practice exists for general guidance on sounding, very little experience is necessary for this effort and effective chain drag equipment can be constructed relatively inexpensively. A chain drag can be used to survey a large area for delamination and the boundaries of the delamination should be determined by hammer sounding.

A chain drag can be as simple as a length of chain or several lengths of chain attached to a single handle, as shown in Figure 2-2.

It should be noted that traffic noises may interfere with the sounding. Repeat sounding as necessary to ensure accurate interpretation.

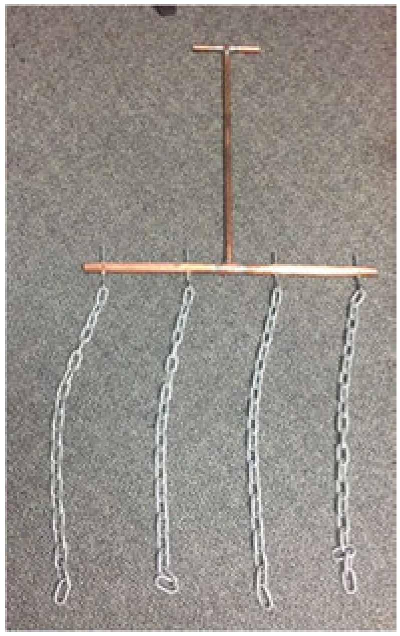


Figure 2-2. A chain drag made from copper tubing and steel chain.

2.1.3 Impact Echo Testing

Impact echo testing is a non-destructive technique that uses impact-generated sound waves to Impact Echo can detect deeper delamination that may not be detected using sounding. Impact echo is more sensitive in detecting delamination than sounding, it can detect delamination at early age whereas chain drag, or sound is good when the delamination is fully developed and audible. Disadvantage if impact echo is the data collection process is slow and it needs expertise to interpret the data. Fundamentally, impact echo testing is very similar to sounding. However, instrumentation involves transducers, impactors, computer, and an analog to or digital data acquisition system. Impact echo cannot be used on bridge decks with asphalt overlays.

2.1.4 Infrared Image Scan

Thermal imaging has been used with varying degrees of success to identify delamination. Delamination may be detected between the top and bottom of the bridge deck when temperatures are rapidly rising or falling. Due to limited window of effectiveness, this technology is not as widely used as sounding with hammer or chain drag. Additionally, thermal imaging may not be used on bridge decks with asphalt overlays. However, thermal Imaging can be used to verify the findings of chain dragging. More information on thermal imaging is available on FHWA website: https://fhwaapps.fhwa.dot.gov/ndep/DisplayTechnology.aspx?tech_id=14.

2.1.5 Half-Cell Potential

Half-Cell Potential (HCP) is a nondestructive, electrochemical evaluation method to identify the likelihood of corrosion of steel reinforcing in concrete. HCP involves identifying and exposing a reinforcing bar and measuring the voltage, potential differences, between the reference electrode and the exposed rebar.

HCP requires electrical continuity of reinforcing steel in a bridge deck. HCP cannot directly measure the degree of corrosion and is limited to use on bridge decks without overlays (the overlay could be locally removed to conduct this test) and bridge decks without epoxy coated reinforcing steel.

Table 2-1 below is a replicate of Table 2-3 of the ASTM C876 that provides interpretation of HCP measurement (relative to $Cu/CuSO_4$ electrode) to chance of rebar corrosion. A more negative measurement is indicative of a higher chance of corrosion.

Table 2-1: Interpretation of Half-Cell Potential Values as per ASTM C876

Half-cell potential (mV) relative to C_u/C_uSO_4	Likelihood of rebar being corroded
< -500	Visible evidence of corrosion
-350 to -500	95%
-200 to -350	50%
>-200	5%

2.1.6 Concrete Resistivity

Concrete resistivity is a nondestructive, electrical evaluation method to assess the risk of corrosion in reinforced concrete. Lower resistivity values indicate more free ions in the concrete which indicates higher risk of corrosion. Reinforcing steel must be located using an accurate rebar locator or cover meter prior to conducting surface resistivity test on the field. If electrodes of the resistivity meters are close enough, the electric signal will not travel deeper into concrete and interference from rebar can be negated.

Concrete resistivity is taken directly on the concrete surface and there is no need to connect to exposed reinforcement in concrete. It can be used as a fast-screening tool to identify areas with high corrosion risks. It is often agreed that the risk of corrosion is negligible when the concrete resistivity is larger than 100 kΩ-Cm.

2.1.7 Ground Penetrating Radar

Ground Penetrating Radar (GPR) is a nondestructive technique using electromagnetic pulses to identify areas with changes in electrical conductivity and dielectric properties. GPR systems utilize either a ground-coupled antenna or an air-coupled antenna. Ground-coupled antennas are typically limited to 5 mph speed while air-coupled antennas may operate at up to 65 mph speed. One good use of GPR is to determine cover depth of top rebar layer of bridge decks. GPR is used to identify regions that may be more conducive to corrosion of reinforcing. However, GPR has not been successful in identifying regions with active corrosion. Air-coupled GPR has not been proven to reliably predict deck delamination in Texas. Also, ground coupled is not good enough for detecting delamination.

2.1.8 Coring to Evaluate Chloride Content

Coring is a destructive method to extract a portion of the concrete structure to evaluate material properties and test concrete strength and rebar sizes of bridges with unknown plans primarily, for chloride contamination. Coring has been performed in the past may be warranted in some cases. It can result in increased distress and expedited deterioration due to the location of the core. Recently, coring has only confirmed the high chloride content expected based on visual distress caused by corrosion of reinforcing steel and have not otherwise changed the course of action for bridge preservation. As such, the decision to core and the coring locations must be approved by TxDOT on a case-by-case basis.

A corrosion threshold for unprotected reinforcement is generally accepted to be approximately 1.2 lbs. to 1.5 lbs. of water-soluble chloride per cubic yard of concrete (lbs./cy) while the threshold for epoxy coated reinforcing has been suggested to be approximately 4.5 lbs./cy. These thresholds are generally accepted to be the point at which corrosion initiates. Nevertheless, there have been some states (and researchers) that have found intact steel in concrete members with chloride levels above these thresholds. Note that chloride content is either tested as acid soluble or water soluble. Water soluble chloride content is what should generally be tested. The water-soluble test results provide the actual chloride content available to cause corrosion while the acid soluble test results provide the total amount of chloride including that contained in aggregates but not contributing to corrosion potential. When the chloride content at the top reinforcing steel level has reached half of the threshold value, an impervious deck overlay may be installed to slow down chloride penetration.

2.2 Deck Preservation Options

2.2.1 Penetrating Concrete Surface Treatment (Silane)

Applicable Specification: Item 483, "Penetrating Concrete Surface Treatment."

Application: Applicable for use on existing bridge decks that have minor cracking or scaling, not otherwise overlaid. This product does not provide a structural benefit. Instead, a penetrating surface treatment provides a method to seal the bridge decks against external moisture and contaminants. Silane treatment was discontinued on new decks made with High Performance Concrete (HPC) on the assumption that HPC does not require additional sealing due to its low permeability.

Surface Preparation: Surface preparation per specifications Item 483 consists of cleaning the surface with shot or abrasive blasting to remove oils and other contaminates. Concrete deck repair must be performed a minimum of 7 days prior to applying surface treatment.

Anticipated Benefits & Life: With a relatively low cost of about $10/SY (2020 cost) this is an economical method to seal a bridge deck. Many of the problems associated with silane treatments stem from poor application and an inability to properly test for proper application. The anticipated useful life of properly installed product is somewhere between 5 and 10 years.

2.2.2 Flood Coat/Gravity-Fed Sealant (Epoxy or Methacrylate)

Applicable Specifications: Item 780, "Concrete Crack Repair." See TxDOT Concrete Repair Manual for detailed discussion on materials and repair procedure not repeated here.

Application: Applicable for use on existing bridges not otherwise overlaid. This product does not provide a structural benefit. The primary use of gravity-fed sealants is to address extensive shallow bridge deck cracking (5 to 10% of deck surface), which when used correctly, will provide a barrier keeping water and chlorides from reaching the reinforcing steel. Flood coats are recommended on decks without delamination. Flood coats may be used on bridge decks with isolated delamination after partial or full depth deck repairs are performed. Repairs must be performed a minimum of 7 days prior to applying sealant.

The epoxy or methacrylate is spread over the entire bridge deck with squeegees or other hand tools and fine sand is cast onto the fresh epoxy or methacrylate prior to curing/tacking. The flood coat may be used to provide additional skid improvement, if it contains sand cover, due to its sacrificial/temporary nature.

Surface Preparation: Surface preparation consists of cleaning the surface with shot or abrasive blasting to remove oils and other contaminants.

Anticipated Benefits & Life: This treatment can be expected to seal deck cracking and delay chloride contamination and corrosion for approximately 5 years when installed properly.

2.2.3 Overlays

2.2.3.1 Asphalt Overlay and Seal Coat

Applicable Specifications: Item 316, "Seal Coat."

Application: Applicable for use on existing bridges with or without asphalt overlay. Asphalt overlays are generally not recommended in Texas. Asphalt overlays are permeable and can trap chlorides and moisture against the bridge deck. However, some asphalt concrete pavement (ACP) overlays have performed well especially with low traffic volumes. If specified on a bridge with an existing asphalt overlay, require milling of existing overlay to avoid excessive asphalt thickness. Care should be taken to properly address joints.

Surface Preparation: Surface preparation generally consists of milling any existing asphalt overlay followed by application of a membrane or tack coat.

Anticipated Benefits & Life: When installed properly, an asphalt overlay is estimated to have a 10–15-year service life depending on traffic volume.

2.2.3.2 Multi-Layer Polymer Overlay (MLPO)

Applicable Specifications: Item 439, "Bridge Deck Overlays."

Application: Applicable for use on existing bridges not otherwise overlaid. This product does not provide a structural benefit. However, the MLPO can seal extensive deck cracking, improve skid values, and delay chloride penetration. If needed, partial and full-depth bridge deck repairs are to be performed at least 7 days prior to application of MLPO. All manufacturer recommendations, including surface preparation, placement temperature, and cure time, should be strictly adhered to.

Surface Preparation: Surface preparation per specification Item 483 consists of removing any existing ACP, followed by shot blasting as recommended by the MLPO manufacturer. A minimum roughness of ICRI CSP-5 should be provided. Surface preparation is not subsidiary to the MLPO.

Figure 2-3: Application of overlay on SH 31 WB over Trinity River.

Traffic will need to be removed from the lane being treated. Do not allow traffic on the shot blasted surface prior to placement of MLPO. For short span bridges, surface preparation and MLPO placement can occur in a single day/night and traffic be returned open to original configuration. Depending on the size of the bridge deck, multiple phases may be needed. Figure 2-3 shows an example of MLPO application.

Anticipated Benefits & Life: MLPO is estimated to have a 15 – 20-year service life with proper installation. Additionally, MLPO has a quick return to service time and a thin profile which eliminates the need for approach work.

2.2.3.3 Polyester Polymer Concrete (PPC) Overlay

Applicable Specifications: 2014 TxDOT Specifications Special Specification 4106, 2024 TxDOT Specs - Item 439, "Bridge Deck Overlays"

Application: Applicable for use on existing bridge decks not otherwise overlaid. This product does not add structural value but can provide additional wearing surface and improve skid, improve ride, seal cracks, and delay chloride penetration. If needed, partial and full-depth bridge deck repairs are to be performed prior 7 days to application of PPC. All manufacturer recommendations, including surface preparation, placement temperature, and cure time, should be strictly adhered to. Pay special attention to repair near joints. It is advisable to avoid paving over the joints then saw cut into the overlay.

Surface Preparation: Surface preparation consists of cleaning the surface by shot blasting to remove oils and other contaminants. Pull-off testing is conducted on the prepared surface to verify substrate quality.
Prior to application of Polyester Polymer Concrete, a surface treatment of high molecular weight methacrylate (HMWM) primer is applied immediately after the surface is blown dry and clean with high pressure air. PPC should be applied after 15 min and within 2 hours of primer application. A mobile mixer is used to mix the polyester polymer and aggregates. The PPC is screeded with a self-propelled paving machine. Depending on the specific product used, traffic can usually be returned to the overlay after 4 hours of cure time.

Mask existing joints and deck drains before overlay placement.

Traffic will need to be removed from the lane being treated during the surface preparation, placement and curing of the PPC overlay. Preparation and PPC placement can occur in a single day/night and traffic be returned to original configuration. Depending on size of bridge deck, multiple phases can be utilized.

Anticipated Benefits & Life: At a cost of about \$10/SY - \$15/SY, PPC overlays have an estimated 20-year service life with proper installation. Notable advantages of PPC include not requiring hydro-demolition, quick return to service, and variability of overlay thickness (3/4" minimum to 3" maximum").

2.2.3.4 Concrete Overlay (CO)

Applicable Specifications: Item 439, “Bridge Deck Overlays.”

Application: Applicable for use on existing bridge decks with minor to moderate cracking and partial and full depth repairs. This product can provide structural value with or without added reinforcing steel or steel fibers. CO can restore integrity of the bridge deck, improve ride, skid values, and provide additional cover to reinforcing steel which will delay chloride penetration. A minimum thickness of 2” is desirable for concrete overlays. Steel fibers may be added in overlay concrete. Dosage rates of 40 lbs./cy have generally been used. If needed, partial and full-depth bridge deck repairs are to be performed prior to overlay. Repairs can generally occur concurrently with overlay operation with the Engineer’s approval.

CO can spall along longitudinal construction joints within wheel paths and at acute corners. Care should be taken when developing details to clearly indicate that a longitudinal phase/construction joint does not occur along a wheel path. Details should also incorporate a breakback to avoid acute corners at skews over 15 degrees.

Surface Preparation: Surface preparation per Item 483 may consist of either shot blasting or milling and hydro-demolition. A saturated surface dry (SSD) condition of the deck surface must be achieved prior to overlay. One acceptable method to achieve SSD is water blasting (minimum 4000 psi) for at least 15 minutes and then air cleaning to remove standing water.

At a minimum remove traffic from the lane(s) being treated. Positive traffic barrier is preferred due to the longer construction time. If practical, detour all traffic from the bridge during overlay application. Surface preparation operations can be performed in a single day to multiple days depending on size of bridge. Installation of screed and screed rail and additional forming can be done quickly. Placement of CO will require an additional 8-10 days of curing depending on mix design. Curing shall conform to Item 422. Contractors may place approximately 40-60 cy per day.

Anticipated Benefits & Life: At a cost of about $105/sy, CO can provide over 20 years of service with proper installation and improve structural capacity and deck performance. Note that Hydro-Demolition can cost up to $100/sy depending on depth of removal and may not be necessary.

Localized delamination between the overlay and bridge deck can be addressed by epoxy injecting along the interface to bond them together to extend service life.

2.2.3.5 Latex-Modified Concrete Overlay (LMCO)

Applicable Specifications: Item 439, "Bridge Deck Overlays."

Application: Applicable for use on existing bridge decks with moderate cracking and partial and full depth repairs. This product can provide structural value with or without added reinforcing steel or steel fibers. LMCO can restore integrity of the bridge deck, improve ride, skid values and provide additional cover to reinforcing steel which will delay chloride penetration. If needed, partial and full-depth bridge deck repairs can generally occur concurrently with overlay operation with the Engineer's approval.

LMCO can spall along longitudinal construction joints within wheel paths and at acute corners. Care should be taken when developing details to clearly indicate that a longitudinal phase/construction joint does not occur along a wheel path. Details should also incorporate a breakback to avoid acute corners at skews over 15 degrees.

Surface Preparation: At a minimum remove traffic from the lane(s) being treated. Positive traffic barrier is preferred due to the longer construction time. If practical, detour all traffic from the bridge during overlay application. Surface preparation operations can be performed in a single day to multiple days depending on size of bridge. Installation of screed and screed rail and additional forming can be done quickly. Placement of LMCO will require an additional 8-10 days of curing depending on mix design. Curing shall conform to Item 422. Contractors may place approximately 40-60 cy per day.

Anticipated Benefits & Life: At a cost of about $125/sy (plus cost for hydro-demolition), LMCO can provide up to 20+ years of service with proper installation and improve structural capacity and deck performance. Note that Hydro-Demolition can cost up to $100/sy depending on depth of removal.

Localized delamination between the overlay and bridge deck can be addressed by epoxy injecting along the interface to bond them together to extend the service life.

2.2.4 Partial-Depth and Full-Depth Deck Repairs

Applicable Specifications: Item 429, "Concrete Structure Repair" and the TxDOT Concrete Repair Manual Chapter 3, Section 4.

Application: Applicable for use on existing bridge decks with localized deteriorations or failures. Partial and Full-depth bridge deck repairs can restore structural integrity and deck performance.

Figure 2-4: Full depth deck removal and reconstruction at bridge joint due to advanced deterioration of deck at joint.

Surface Preparation: Surface preparation consists of shallow saw-cutting around the repair locations, chipping hammers up to 30 lb. for deep removal and 15lb for shallow removal around reinforcing. Remove all unsound concrete, abrasive blast cleaning of existing reinforcing and surrounding substrate prior to concrete placement. CL S concrete is the preferred replacement material, but HES CL K can be utilized for rapid removal of traffic control.

Depending on extent, location and size of the partial and full depth repairs, positive barrier may be needed. Traffic will need to be detoured from the work location throughout the duration. Traffic control can be utilized for this operation and returned to final configuration after the work shift using accelerated methods. This operation can take days to weeks depending on magnitude of repairs.

Anticipated benefits/life: At a cost of about $80/sf for partial depth and $130/sf for full-depth deck repairs (2020 cost), this can be a cost-effective method to restore structural integrity and long-term deck performance for the life of the structure if the root cause(s) has been addressed.

2.2.6 Deck Replacement

Applicable Specifications: Item 422, "Concrete Superstructures."

Application: Applicable for use on bridge decks exhibiting extensive and severe deterioration (larger than 30% deck areas require repair) with an acceptable superstructure and substructure rating. A life cycle cost analysis should be performed to determine if total structure replacement or deck replacement is the more economical option. Shear connectors can be added to non-composite steel girders to improve structural rating and overall performance. Bridge rails will also be upgraded during deck replacement projects.

Traffic will need to be removed from the structure for a complete deck replacement. Phasing can be utilized if the existing bridge width is sufficient and alternate detours are not available. Deck replacements can take a month to several months depending on size and phasing requirements. It is important to be cautious and not damage the existing superstructure during deck removal operations.

Anticipated Benefits & Life: Deck replacement costs vary widely depending on traffic control requirements, deck removal methods, and duration of contract. Proposed revisions above cover this. However, a deck replacement can be expected to significantly prolong the remaining service life of the bridge.

2.3 Deck Preservation Matrix

A deck preservation matrix is a tool that can be used in the selection of deck repair options based on deck condition rating (NBI Item 58) and element level inspection data.

For consistency, surface area deficiencies and cracking are classified by types in the deck preservation matrix. Consider deck replacement when the extent of deterioration is too widespread to be addressed with other deck preservation methods.
The types of deficiencies used in TxDOT's Deck Preservation Matrix are described below.

2.3.1 Surface Area Deficiencies

Type A: Extensive map cracking caused by material durability issues, e.g., alkali-silica reaction of concrete aggregates. Material testing may be required to confirm causes and extents of deterioration. Contact Bridge Division for evaluation and preservation options.

Type B: Spalling, delamination, and map cracking is limited to the top half of the deck depth. No deteriorated areas have extended to the soffit.

Type C: Spalling, delamination, and map cracks have extended through the entire depth of deck. The risk of punch through shear failure is high.

Type D: Localized deck soffit spalling due to rebar corrosion or construction defects. Hammer sounding in accessible areas should be performed to help identify the extent of delamination caused by chloride induced corrosion.

2.3.2 Deck Cracks

Type E: Transverse and longitudinal deck cracks caused by concrete shrinkage and thermal movements.

Type F: Significant transverse or longitudinal cracks caused by overloading, structural design, or substructure/foundation movement.

TxDOT’s recommended bridge deck preservation matrix is shown in Table 2-3.

2.4 Deck Washing

To ensure the long-term performance of bridges, concrete deck, and structural steel components, need to be periodically cleaned and washed. The major purpose of washing is to remove residual chlorides, from de-icing salts applied during the winter season, that if left, would result in the corrosion and deterioration of the bridge. The chloride contaminated runoff can damage steel bearing or bent caps if the bridge joints are leaking, especially at the interior bent and abutment locations.

Bridge washing to remove sand and other debris may be done annually in the spring, depending on the severity of the preceding winter. The areas to be cleaned shall include the approach slab, bridge deck, curb surfaces and between rail faces using manual or mechanical methods. Once cleaned, the bridge steel and concrete surfaces shall be thoroughly flushed with water. The three main types of mechanical washing are:

1. Power Washing - Low pressure water with a flow rate of more than a hundred gallons per minute.
2. Pressure Washing- Water pressure of 1,750 psi (minimum) at a flow rate of 3.5 gallons per minute used to clean surfaces.
3. High Pressure Washing – Water pressure of 5,000 psi (minimum) at a flow rate of 5 gallons, per minute typically used to clean weathering steel.

Concrete Bridges may be either power washed or pressure washed. The bridge deck may be power washed to remove debris prior to pressure washing the deck. To minimize impacts to the environment, cleaning and washing must coincide with spring high water flows.
Steel bridges over stream crossing must be manually scrapped to remove bird droppings prior to washing to avoid contaminating the stream below the bridge.
Bridge washing is not intended as a statewide preservation effort, especially not in areas where deicing is uncommon. Rather, it shall be a case by case according to district needs.
Necessary permits and approvals from Texas Commission on Environmental Quality (TCEQ) and Environmental Division must be obtained prior to bridge washing. The permits and approvals shall include the procedure for collection and disposal of the wash water.

Figure 2-5: Bridge Deck Washing (Source: FHWA Bridge Preservation Guide)

Table 2-2: Examples of Preservation Activities

Preservative/Maintenance Activity	Maintenance Activity	Bridge Component
Cyclical Maintenance Activity	Clean/Wash Bridge	Deck and/or Super/Substructure
	Clean and Flush Drains	Deck
	Clean Joints	Deck
	Deck/ Rail Sealing and Crack Sealing	Deck
	Seal Concrete	Super/Substructure
Condition-Based Maintenance Activity	Joint Seal Replacement	Deck
	Drains, Repair/Replace	Deck
	Joint Repair/Replace/Elimination	Deck
	Electrochemical Extraction (ECE)/Cathodic Protection (CP)	Deck
	Concrete Deck Repair (see halo effect below) in Conjunction with Overlays, CP Systems or ECE Treatment	Deck
	Deck Overlays (thin polymer epoxy, asphalt with waterproof membrane, rigid overlays)	Deck
	Repair/Replace Approach Slabs	Deck
	Seal/Patch/Repair Superstructure Concrete	Approach
	Protective Coat Concrete/Steel Elements	Superstructure
	Spot/Zone/Full Painting Steel Elements	Superstructure
	Steel Member Repair	Superstructure
	Fatigue Crack Mitigation (pin-and-hanger replacement, retrofit fracture critical members)	Superstructure
	Bearing Restoration (cleaning, lubrication, resetting, replacement)	Superstructure
	Movable Bridge Machinery Cleaning/Lubrication/Repair	Superstructure
	Patch/Repair Substructure Concrete	Superstructure
	Protective Coat/Concrete/Steel Substructure	Substructure/Culvert
	ECE/CP	Substructure/Culvert
	Spot/Zone/Full Painting Steel Substructure	Substructure/Culvert
	Pile Preservation (jackets/wraps/CP)	Substructure
	Channel Cleaning / Debris Removal	Substructure
	Scour Countermeasure (installation/repair)	Channel

Table 2-3: Bridge Deck Preservation Matrix

Deck Condition Rating	Surface Area Deficiency				Deck Cracking		Recommended Preservation Action	Anticipated Service Life (years)
	Type A	Type B	Type C	Type D	Type E	Type F		
>5	<10%						Contact BRG	
		<5%		<2%			Isolated partial-depth deck repair; concrete patch	10-15
							Silane treatment to bridge deck	5-10
					Spacing >4 ft.; crack width >0.01 in.		Seal individual cracks with epoxy Apply silane treatment to bridge deck	5-10
					Spacing >4 ft.; width <0.01 in.		Apply silane treatment to bridge deck	
5	10-25%						Contact BRG	
		5-25%					Partial-depth deck repair; then multi-layer polymer overlay or polyester polymer concrete overlay	10-20
							For pre-cast panel with cast-in-place partial depth deck topping, partial-depth deck repair	
			<2%				Isolated full-depth deck repair	10-20
				<10%			Chloride content < 1.5 lb. per cy (for uncoated reinforcing steel or 4 lbs per cy for epoxy coated reinforcing) – Perform isolated concrete repair. Cathodic anodes are optional.	5-15
				10% (or more)			Chloride content >1.5 lb. per cy (for uncoated reinforcing steel or 4 lbs per cy for epoxy coated reinforcing) – Perform deck replacement.	30-40
					Crack spacing < 4 ft.; width >0.01 in.		Multi-layer polymer overlay	10-20
							Polyester polymer concrete overlay	20
						Spacing > 4 ft.; width <0.01 in.	Contact BRG; Load rate deck; Retrofit with concrete inlay or overlay if necessary	
4	>25%						Contact BRG;	
		>25%					Deck Replacement	30-40
			>10%				Deck Replacement	30-40
				>10%			Deck Replacement	30-40
					Spacing <2 ft.; width >0.01 in.	width >0.01 in.	Contact BRG;	
3							Inform BRG and District	

Chapter 3 BRIDGE JOINTS

This chapter is intended to assist in the selection of proper joint repair, replacement, and installation methods for maintenance and rehabilitation projects.

3.1 Bridge Joints Functionality and Types

Bridge expansion joints are provided to accommodate movement due to temperature variations, concrete shrinkage and creep, live load deflection, and substructure, concrete pavement, or approach slab movements. They also help to ensure smooth riding surface for motorists. Expansion joints should be sized to ensure that the ends of units or spans do not close, resulting in damage to concrete or steel elements. Expansion joints are usually one of the first components of a bridge to deteriorate due to excessive movement, joint growth by accumulation of debris, wheel impact and/or snowplow damage, de-icing chemicals, or poor installation or maintenance. On newer bridges, a series of spans are usually combined to form units with construction or controlled joints thereby reducing the number of expansion joints on a bridge.

3.2 Joint Seals

One of the most used and most important bridge preservation actions is cleaning and re-sealing the expansion joints. The success of this strategy depends on surface preparation, installation, and proper application. Joint sealing can occur with or without steel or concrete repair work to the joint nosing which is discussed in Section 3.4. Table 3.1 below is intended to help with joint sealant selection. The expected service life for properly installed pourable and strip seal joint is 5 to 10 years and joint replacements may last up to 20 years. If improperly installed, the joint seals may fail before the end of the project.

Debris accumulated in joints restrict thermal movement and initiates deterioration of the joint and the structure. Routine cleaning of joints by removing debris accumulation and resealing failed joint sealants helps to extend the service life of the bridge by maintaining movement and preventing water and debris from accumulating on the superstructure and substructure elements.

Joints in pan girder bridges are difficult to clean as they are about two to three feet deep. For such bridges, it is important to clean to the top of bent caps to prevent restriction of movement and fracture of the caps.

Figures 3-1 (a) and (b) show possible consequences of failure to properly maintain bridge joints.

Figure 3-1: (a) Cap face spalling due to restricted movement; (b) Severe web corrosion due to debris build up on cap.

Table 3-1: Joint Seal Selection Guide

Joint Seal Type[1]	Movement	Application Type: Existing, Rehabilitation, or Resizing					
		Open Deck Joints	Reconstruct Deck Joints	Replace Sliding Plate Joint	Replace Finger Joints	Polymer Concrete Blockouts	Approach Slab to Concrete Pavement
Mechanically Bonded Strip Seal (S)	0-4" [2]	x	x	x [6]	x [6]		
Bonded Strip Seal (S,C,P))	0-4" [2]	x	x			x	
Pourable Seal (S,C,P)	0"-1"	x	x			x	x
Preformed Foam (closed or open cell) w/Silicone Topping, (S,C,P))	1" (Only Compression) [3]	x	x[6]			x	x
Preformed Hollow Extruded Neoprene Seal (S,C,P)	0-4"	x	x			x	
Asphalt Plug Seal	0-1" [4][5]	x				x	

Notes:
1: Bonded Joint Substrate - S (Steel), C (Concrete), P (Polymer Concrete)
2: For skews larger than 30 degrees increase the size of the seal, up to 5".
3: Seal must be installed in cooler temperature (<60 degrees), seal must stay in compression and thus must be precompressed when installed to allow joint expansion after installation.
4: Asphalt plug material must meet ASTM D 6297 (limited truck traffic, or off system overpasses)
5: The Asphalt plug joint cannot tolerate: rotation or vertical translation. Plug joints subjected to heavy truck traffic, and hard braking forces have prematurely failed in some circumstances.
6: Contact Bridge Division for more information.

Table 3-2 summarizes the applications of TxDOT approved joint sealants.

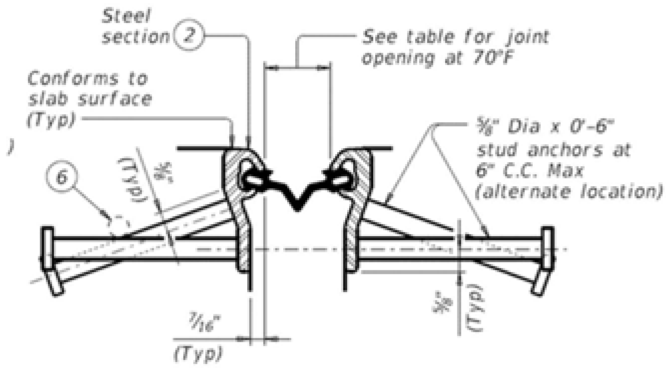
Table 3-2: Joint Sealant (TxDOT DMS 6310) Applicability

Class	Material	Joint Type	Recommendation
Class 1	Polyurethane	header-type	
Class 2	Synthetic Polymer	asphalt to concrete concrete to concrete steel or armored	
Class 3	Hot-poured rubber	asphalt to concrete concrete to concrete	Use for bridges with asphalt overlay
Class 4	Silicone	asphalt to concrete concrete to concrete steel or armored	Use for vertical joints
Class 5	Silicone or Polyurethane	asphalt to concrete concrete to concrete	Use for non-bridge joints. Not used on steel to steel.
Class 6	Solid	concrete to concrete steel or armored	
Class 7	Silicone	concrete to concrete Steel or armored header type	Use for bridge joints
Class 8	Silicone or Polyurethane	concrete to concrete	Use strictly on non-bridge joints

3.2.1 Strip Seals

Strip seals consist of a preformed gland of neoprene rigidly attached to metal facing on both sides of the joint. They are typically used for span lengths or units greater than 150 ft. The minimum opening width is typically 1¾” for 4 inches joint and 2” for 5 inches joint.

TxDOT has standard drawings for three different strip seal expansion joint systems. The SEJ-M and SEJ-S(O) systems are mechanically bonded systems, and the SEJ-B strip seal is bonded to the steel facing by an adhesive. Figures 3-1 and 3-2 show typical drawings of these joint systems. Table 3-3 summarizes the range of applicability for these joint systems.



SECTION THRU D.S. BROWN
(A2R-400 OR A2R-XTRA) JOINTS

Figure 3-2: Mechanically locked-in strip seal.

TxDOT has standard drawings for two mechanically bonded SEJ systems. The following guidance should be considered when using these systems:

- SEJ-M applicable for no overlay
- SEJ-S(O) applicable for overlay with +2 in. thickness.
- Minimum slab and overhang thickness 6.5 in.
- Available in two sizes, 4 in (total movement up to 4 in.) and 5 in. (total movement up to 5 in.)

For TxDOT practices, the bonded Sealed Expansion Joint (SEJ) is SEJ-B:

- applicable when no overlay and minimum slab and overhang thickness is 6.5in.
- Available in one size, 4in.
- Minimum opening is 0 in. and maximum opening is 4 in.

Joint Type	Size	Min. Opening	Max Opening	Min. Slab Thickness	Overlay (Y/N)
SEJ-M	4"	1.75"	4"	6.5"	N
	5"	2"	5"	6.5"	N
SEJ-S(O)	4"	1.75"	4"	6.5"	Y
	5"	2"	5"	6.5"	Y
SEJ-B	4"	0"	4"	6.5"	N

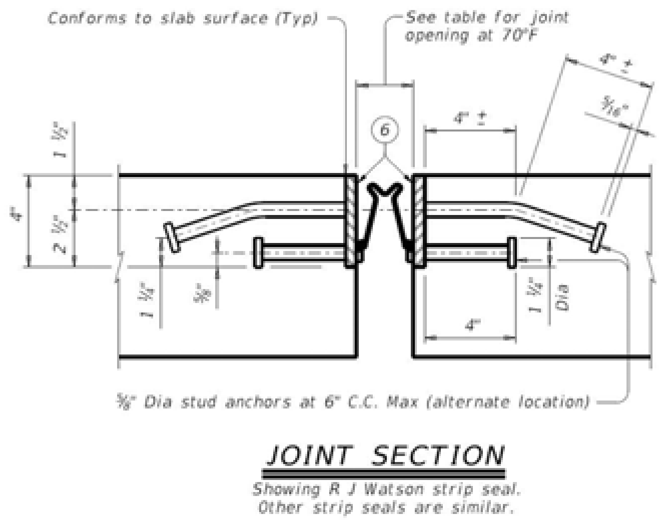


Figure 3-3: Bonded strip seal.

An example of a retrofitted strip seal joint is shown in Figure 3-4

Figure 3-4: Example of retrofitted strip seal joint.

3.2.2 Pourable Sealants

Pourable sealants consist of liquid joint sealant material placed on top of an embedded backer rod in the open joint. The most used materials are hot-poured rubber and silicone.

These materials are covered in DMS-6310, "Joint Sealants and Fillers."

3.2.2.1 Hot-Pour Rubber Joint Sealant (Class 3)

Hot-poured rubber joint sealants are compatible with both asphalt pavements and concrete but are most often used to seal joints on bridges with asphalt overlays or approaches. The joint opening should be limited to 1 1/2" and should have relatively small, expected movement (less than 3/8"). The material is supplied in solid form, but when melted and properly applied, forms a highly adhesive and flexible compound that resists cracking in winter and resists flow during the summer. Hot-poured rubber sealants require heat-resistant backer rods and may be used throughout the state.

An example of hot poured rubber seal is shown in Figure 3-5.

Figure 3-5: Example of a hot pour joint seal.

3.2.2.2 Silicone Joint Sealant (Class 7)

Silicone joint sealants are compatible with concrete, header material, and steel joint nosing and may be used on bridge expansion joints subjected to movements greater than +/- 3/8" and with joint openings up to 2 1/2”.
Other silicone sealants usually will not gain sufficient strength prior to going into tension and can debond.

Class 7 silicone is a two component self-leveling rapid setting material and can accommodate the expected movement soon after placement. Class 7 joint sealants can typically accommodate expansion movements from 50% to 100% of the joint width and compression movements up to 50% of the joint width. Extend sealant up into rail or curb 3 inches on low side or sides of deck. Since the Class 7 Sealant is self-leveling, it is recommended to use a Class 4 Silicone in vertical joints, such as in bridge railing and sidewalks, to form the seal. A Class 4 Silicone is not a rapid setting material, so it is best to install during the coolest part of the day to ensure the material goes into compression while it is setting.

Working Drawings for pourable seals can be accessed on
https://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm

An example of a poured silicone seal is shown in Figure 3-6.

Figure 3-6: Example of silicone seal.

3.2.3 Pre-formed Compression Seals

A common failure mode for most types of joint sealing systems is the inability to tolerate tension in the sealant. Pre-formed compression seals rely on keeping the joint seal in a state of compression. To accomplish this, they should be installed when the joint is at its widest, typically in the colder months. When the bridge expands in warmer months, the joint will contract, placing the sealant in compression. When installed properly, these systems can perform well, but project timelines do not always account for the ideal timing for seal installation. Additionally, the contractor should consider several different sizes of joint seal available to match the joint width at the time of installation. Figure 3-6 shows a typical pre-compressed joint seal.

Compression seals rely on a continuous preformed neoprene elastomeric rectangular-shaped section compressed into the joint opening for the total width of the bridge deck, to accomplish its waterproofing function. Compression seals are made of either pre-formed closed-cell plastic or, more commonly, hollow extruded neoprene shapes. Two providers of compression joints commonly used are Emseal/Sika (BEJS) and Watson Bowman Acme ("FS" Foam).

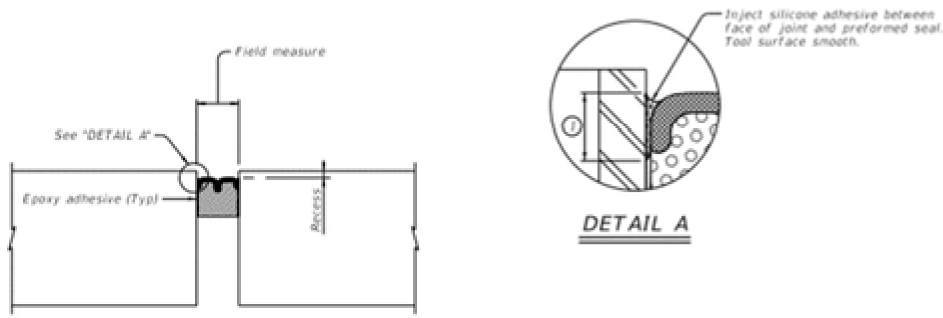


Figure 3-7: Example of pre-formed compression seal.

3.4 Joint Repair and Retrofitting Options

3.4.1 Joint Repair or Replacement

This section describes common issues and recommended solutions for repairing damaged or failed armor joint systems. While the issues and recommendations here are given for armor joints, they may be applied to other joint systems as well. Recommended repairs will vary based on the extent of damage, available funding, and the preservation strategy. Unaddressed minor issues may turn into major issues if no measure is taken.

Armored joints are also called butt joints. They are generally used to accommodate movement up to 1.5" and minor rotations. The armor (steel angles or plates embedded in concrete) act as headers to protect the concrete deck joint edges.

Loose Armor Plates

- The anchorage of armor component (steel angles or plates) to the concrete deck can become loose over time. The symptoms are cracking and delamination of concrete in the anchorage zone and the "cranking" of joint under traffic. The possible causes are poor concrete consolidation around armor anchors during construction and delamination/spalls due to corrosion of steel elements. Figure 3-8 shows a loose joint.

Figure 3-8: Loose joint due to concrete deterioration in anchorage areas.

Deformation of Steel Plates

- The steel armor plates may deform, causing a wavy shape. The symptoms are the wavy shape of the armor plates and gaps between the deck concrete and armor plates. The primary cause is corrosion of reinforcement or anchors behind the armor plate or the corrosion of the armor itself. The resulting pack rust may build up behind the plate and force the plate forward to form the wavy shape. Pack rust is a problem that cannot be easily dealt with and is often ignored until the steel components finally break free. Figure 3-9 shows a deformed joint.

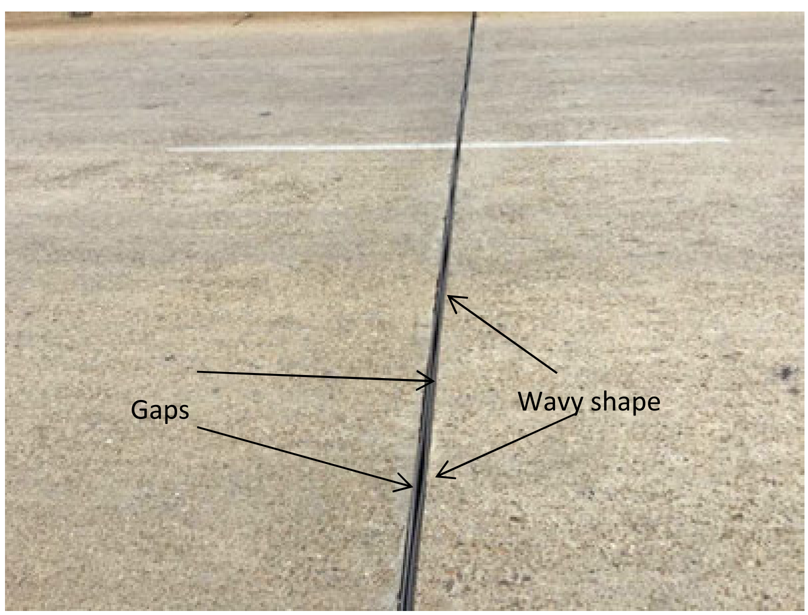


Figure 3-9: Deformed joint armors probably due to anchorage corrosion.

Broken Armor Plates

- Armor plates may be broken or missing and are often accompanied by exposed and often deteriorated or failed deck edges. The possible causes are fatigue induced fracture under traffic and/or corrosion of armor steel. Poor concrete quality under and near the armor may also be a contributing factor. A failed joint is shown in Figure 3-10.

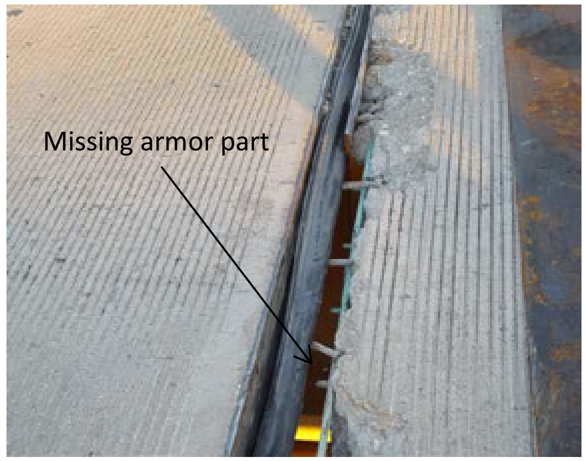


Figure 3-10: Example of a failed joint.

Closed Joints

- Joints may close over time. The possible causes are substructure movement due to foundation issues and superstructure- deck movement due to adjacent approach concrete pavement push or frozen bearings. Joint closure is not a cause for concern. However, joint closure leading to spalling on the deck, rails, or beam ends should be addressed. Where the approach pavement is pushing on the bridge abutment and spalling has resulted, joint reconstruction should be considered. Figure 3-11 shows a closed joint.

Figure 3-11: Example of closed-up armor joint.

Repair Options

Minor spalls of concrete in the anchorage zone may not require repairs if progression of damage is slow. However, widespread spalling will lead to debris accumulation, corrosion of steel elements, fatigue of steel angles/plates, and eventual failure of the joint. A full depth repair is generally not necessary for this condition and a partial-depth repair may not be very effective. High quality concrete spall repairs may be appropriate.

When spalling and delamination of concrete in the anchorage zone is widespread, a full-depth repair of concrete is warranted. Break back into the deck at least 12 inches, if feasible, and engage the deck reinforcement.

Concrete repair work shall follow appropriate procedures described in TxDOT Concrete Repair Manual. The Working Drawing available at https://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm can be used as a guide in developing repair strategies.

One option to address the issue of deformed plates before the steel components break free is to blast clean steel surface at locations where separation exists between the back of armor plate and the top of deck, blow out all debris from gap and fill the void with Class 5 or 7 silicone per DMS 6310 or Type IX epoxy per DMS 6100 with coarse sand.

When the anchorage is intact and the gap is small (less than ¼ in.), clean with high-pressure water, dry out, and then inject the gap per Type IX epoxy with DMS 6100.

It may be economical to reconstruct the joint when the steel components break free, and the joint is closed.

As broken Armor plates components often occur in the high impact area along wheel paths, weld splicing of the armor component should be performed beyond these locations. When splicing is not effective to secure the armor component, one option is to eliminate the entire armor joint fully or partially.

When eliminating the armor, remove loose concrete in the areas where the armor components are missing and apply the concrete spall repair material to restore the original shape of the joints.

An example of a partial elimination of the armor is shown in Figure 3-12.

Figure 3-12: Example of partial elimination of armor joint.

When the total repair lengths are more than 40% of the total joint length, it may be cost effective to replace the entire armor joints. A typical repair detail can be found on https://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm

3.4.2 Bridge Joint Elimination

Joint elimination can be an effective preservation strategy. Joint elimination is performed on a series of spans to reduce potential for deterioration of superstructure or substructure components due to leaking joints.

Joint elimination may require bearing replacements to accommodate the increased thermal movement. Bearing pads should be evaluated to confirm that they can accommodate the increased thermal movement.

Polymer Nosing (Header) Joints

Header joints are used on bridges with asphalt or concrete overlays. They should not be used to repair bridge ends without overlays nor on decks without thickened ends. If significant joint deterioration is present, a full depth joint repair should be performed rather than using a header material.
In the past, the recommended thickness to width ratio was 1:2, however this resulted in very wide header material applications and increases the damaged area when failure occurs. Reducing the thickness to width ratio to 1:1.5 can help minimize the impact to traffic when failure occurs. The thickness of the header material should not exceed 3".

Due to their historically poor performance and high cost, header joints are not often recommended as a preservation action on TxDOT bridges. A contributing factor to the high cost is inaccuracy of the asphalt pavement thickness in the design plans. When header joints are used, the designer should verify the pavement depth and divide payment for the joint into cubic feet of header material and linear feet of joint sealant (typically Class 7).

3.4.3 Sealing Controlled Joints

In some cases, control joints (construction joints and joints between fixed deck ends) may open due to long-term drying shrinkage, creep of concrete or cumulative irreversible movements of bridge components. The opening allows water and debris to leak through and negatively affect the deck, superstructure, or substructure.

An example of opened construction joint is shown in Figure 3-13.

Figure 3-13: Example of opened construction joint.

Summary of Recommended Practices:

The most common method to address this issue, on concrete decks without overlay, is to use hot-poured rubber to seal the opening, like sealing an expansion joint. When the joint is larger than ½'', a backer rod is required.

On bridge decks with asphalt overlay, a fabric joint underseal should be installed below the asphalt. Refer to the drawings in the appendix for more information.
Ideally, a fabric joint underseal retrofit will be timed with a project to remove and replace the overlay on the bridge. A fabric joint is not necessary if the asphalt overlay is replaced with a concrete overlay, LMC, PPC, or MLPO.

3.4.5 Other Joints

Other joints that require more detailed consideration not covered here are Sliding Plates, Finger Joints, and Modular Joints. Sliding Plate joints have been used to accommodate movement between 1” and 3”. Finger joints have been used to accommodate movement larger than 3”. Modular joints have been used to accommodate very large expansion and contraction movements.

3.5 Calculations to Determine Thermal Joint Movement

Refer to TxDOT Bridge Design Guide Chapter 5 Section 5 (https://crossroads/divisions/brg/sections/bridge-design-section.html).

3.6 Beam and Girder End Trimming

Trimming beam ends can be performed to reset joint openings that have closed due to lateral pressures against the backwall. This may not permanently address the problem of joint closure but will stop continued damage and deterioration from the beam end impacting the backwall. Consideration of need to address other contributing issues (i.e., lack of joint at approach slab to concrete pavement) should be made. Avoid trimming beams more than once if joint closure continues.

Figure 3-14: Example of Beam End Trimming.

Prestressed beams may be trimmed by saw cutting the beam ends. In these cases, the beam ends should be coated with a neat epoxy to address the potential for water infiltration to corrode the reinforcing steel exposed. In Lubbock district, the end of prestressed girders was trimmed instead of reconstructing the abutment backwall, as a bridge preservation effort. This approach resulted in less traffic.

Steel girders may be trimmed as well. Depending on the length of girder being trimmed consideration should be given to the need to adjust bearing stiffeners, diaphragms, or bearings. If trimmed, the ends of the steel girder should be re-coated.

3.7 Abutment Backwall Reconstruction

Backwall reconstruction is performed to regain expansion capabilities at the ends of a bridge. The contributing factors for abutment damage are lateral movement due to earth pressure, joint growth on the bridge or approach pavement, etc. Backwall reconstruction will significantly impact traffic as it will require removal of approach pavement of approach slab, removal of backfill, shoring (if completed in phases), removal of existing backwall, construction of formwork, and reconstruction of backfill and approach pavement.

Reconstructing the backwall alone may not address the underlying contributing factors. Engineers need to evaluate contributing factors and weigh the benefits of this work against the potential for joint closure again.

One sketch of backwall reconstruction is shown in Figure 3-15.

While reconstructing the backwall wing shafts should be installed if they are not present to prevent further rotation.

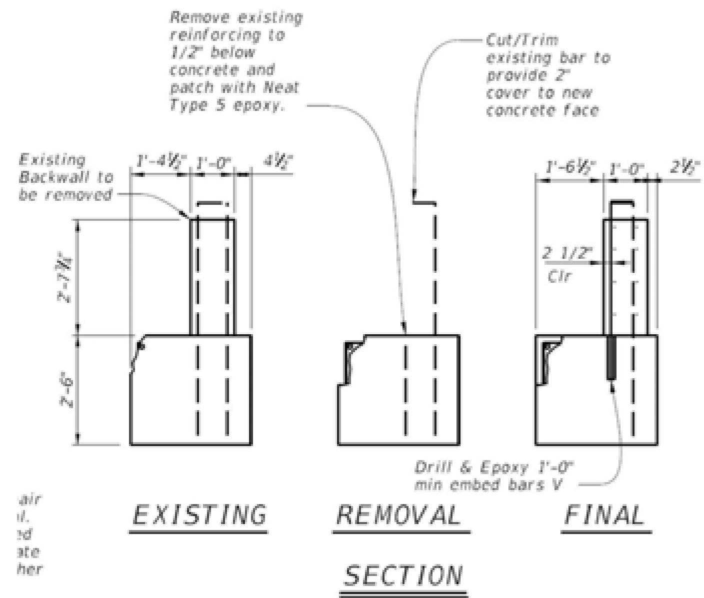


Figure 3-15: Example of backwall reconstruction.

In cases where the back of the backwall is aligned with the back of the abutment cap, additional work may be required. Additional work may consist of constructing a widened abutment cap (or ledge) on the back side of the abutment cap to support the reconstructed backwall.

Chapter 4 CONVENTIONALLY REINFORCED CONCRETE SUPERSTRUCTURES

This chapter is intended to assist in selecting appropriate preservation practices for conventionally reinforced concrete superstructures commonly used in Texas, i.e., cast-in-place slab spans, T beams, and Pan girders.

4.1 Superstructure Types

4.1.1 Cast-in-Place Slab

Simply and continuously supported cast-in-place (CIP) concrete slabs have been used in Texas to cross short spans. They are among the oldest structure types in TxDOT's inventory, and most have exceeded their design life, but are common candidates for preservation and maintenance projects.

A specific variation of cast-in-place slabs called Farm System (FS slabs) were constructed with a structural curb. Without the benefit of the structural curb, this structure type is inadequate and not recommended for widening. Rail replacements, if not carefully designed, may reduce its live load capacity. With increasing loads and traffic demands, FS slabs should be considered for replacement rather than preservation.

4.1.1.1 Cast-in-Place Slab Issues

The most common issue with cast-in-place slab spans are transverse cracks, as shown in Figure 4-1 and longitudinal cracking, especially associated with slab widenings. Reinforced concrete may crack under regular service loading and oftentimes most transverse cracking on the soffit is not a reason for structural concern. However, any kind of cracking when it extends to the deck surface, could be a concern for durability.

Figure 4-1: Transverse cracks and white efflorescence on slab soffit.

Corrosion of the steel reinforcement may sometimes cause spalling, as shown in Figure 4-2.

Figure 4-2: Slab soffit spalling due to corrosion of steel reinforcement.

4.1.1.2 Repair Options

Except for excessively large cracks (0.03” or larger), the transverse cracks are usually not repaired. It is usually difficult to raise the condition rating of a structure with deck soffit cracks and sealing minor cracks provides little benefit towards preservation. Instead, sealing or overlaying the top of the concrete slab may be utilized to mitigate infiltration of water and corrosion of reinforcing. See Chapter 2 of this guide for deck preservation options.

Address slab bottom spalling in accordance with Item 429, “Concrete Structure Repair” and following the procedures listed in TxDOT Concrete Repair Manual.

4.1.2 Concrete Beams (T Beams)

The Cast-in-place T beams were commonly used before the widespread use of prestressed concrete beams for simply supported spans prior to 1950s.

4.1.2.1 Common Issues

Besides the common corrosion issue of reinforced concrete structures, there is another specific issue common to T beams supported by steel bearings. After the onset of corrosion or loss of lead sheets permitting sliding between the bearing plate and base plate, expansion bearings frequently freeze or lock up. Since the upper plate of the steel bearing is embedded in the beam web, thermal forces result in great stress concentration and cracking of the beam ends as shown below in Figure 4-3. If not repaired in time, the beam may drop resulting in a change in elevation across the joint or a more severe condition.

Frozen steel bearing to be lubricated.

To lubricate a frozen bearing, clean and flush bearing with high pressure water or air to remove loose material. Lubricate the sliding faces with approved lubricants

Figure 4-3: Fracture of T-beam end.

Since T-beams were often designed for H15 loading, transverse flexure cracks are not uncommon on the beams.

4.1.2.2 Repair Options

Repair corrosion induced deterioration following the procedures listed in the TxDOT Concrete Repair Manual.

Epoxy injects transverse cracks on beam following the appropriate procedures described in the TxDOT Concrete Repair Manual. However, small quantities of crack injection should be avoided since it will be very expensive and inadequate to restore structural capacity.

To address the beam end cracking, existing concrete diaphragms have been used successfully to transfer the end reaction forces from the beams to the bent cap, using pedestals. The pedestals may be steel or concrete and thermal movements are permitted through elastomeric pads. An example of a set of pedestals is shown in Figure 4-4.

Figure 4-4: Pedestals used to retrofit T-beam end cracking.

4.1.3 Pan Girders

Pan girders were popular in Texas because of the associated easy and re-usable formwork and thus cheaper construction costs. Like T-beams, their use declined significantly after the introduction of prestressed concrete beams in 1960s.

4.1.3.1 Common Issues

Early designs for pan girder bridges consisted of a minimum slab thickness (at the crown) of 3.5”. These thinner slab thicknesses sometimes are problematic especially in areas of increasing traffic volume or loads, leading to wide longitudinal cracks along the crown and sometimes deck punch through failures. Later designs where the crown thickness was increased to 4 1/2" have proven to be more resilient. Deteriorated pan girder bridges with crown thickness of 3.5” should be considered for replacement rather than preservation and are generally not recommended for widening especially in areas of projected increases in traffic volume or loads.

In both designs, a common issue is open, the form releasing holes in the deck allowing runoff water or chloride laden water and debris to accumulate along the cracks and at mid-span, leading to reinforcement corrosion and concrete spalling, as shown in Figure 4-5. Minor to moderate cracks in the crown are not uncommon on this structure type and minor cracking may not require remedial action.

Figure 4-5: Longitudinal Cracks on pan girders.

4.1.3.2 Repair Options

While a structural overlay can increase capacity, pan girders with thin top slabs (3.5” at crown) should be evaluated for replacement rather than rehabilitation.

If the slab is not problematic, the form releasing holes should be plugged with cementitious materials or retrofitted with PVC pipe inserts to extend the pan girder service life.

4.2 Bearings

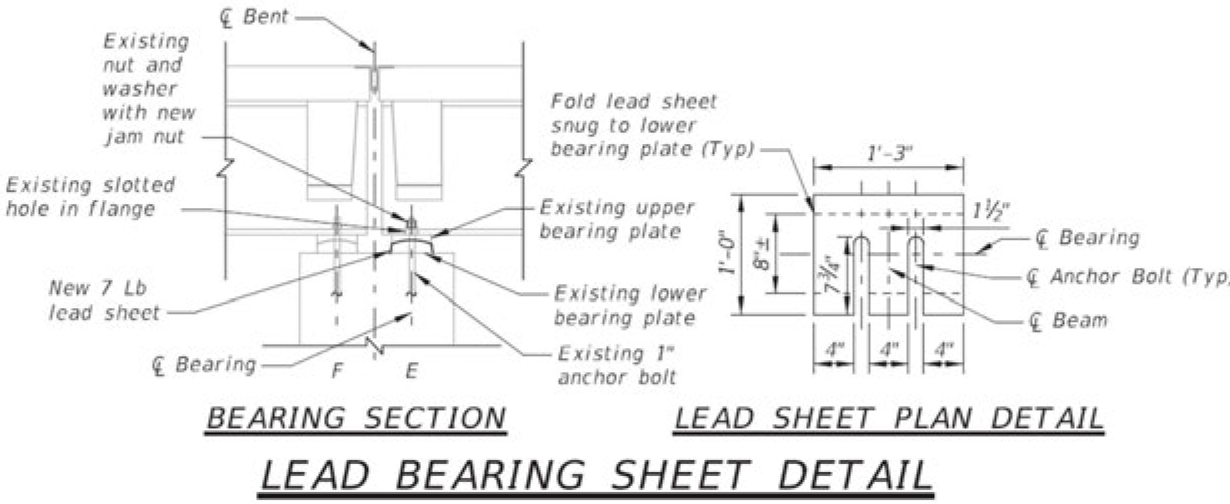
The function of bearings is to transfer load from the superstructure to the substructure. Bearings are designed to accommodate translational and rotational movements caused by component dimension variations, thermal movements, and traffic loading. Steel bearings or layers of felt paper were used under concrete superstructures in Texas before the popularity of elastomeric bearings. When the movements of bearing are hindered by debris, corrosion products, or other obstacles, damage will occur either in the substructure (bent cap and abutment) or the superstructure.

4.2.1 Lead Sheets

With repetitive relative movements between upper and lower plates, the lead sheet may slip out and let the plates sliding directly on each other, which will freeze the bearing movement, causing deterioration of structural members, as shown in Figure 4-6 below.

Figure 4-6: Lead sheet Slip.

To fix the issue, one option is to raise the girders and replace with new lead sheets. At the same time, clean and lubricate the bearing plates. An alternative to this is to trim the slipped lead sheet. If bearing replacement is considered, the replacement should be an elastomeric pad. Although shown for a steel beam condition, Figure 4-7 provides one retrofit example to replace lead sheets.



REPAIR PROCEDURE:

1. Perform lead bearing sheet replacement in phases. Close traffic lane above beams being raised. See Traffic Control Plan Narrative.
2. Loosen existing anchor bolt nuts 3/4" min above beam flange.
3. Raise beams approximately 1/2" max to facilitate lead bearing sheet replacement in accordance with Item 495.
4. Replace lead bearing sheets between bearing plates.
5. Fold lead sheets as shown in Lead Bearing Sheet Detail.
6. Break upper bearing plate free of flange and apply heavy duty corrosion inhibiting lubricant. Lubricant shall be "Bostik Never – Seez Mariner's Choice" or equivalent as approved by Engineer.
7. Lower beams until fully supported on bearings.
8. Remove jacks and tighten anchor bolt nuts to leave 1/4" gap between nut and plate. Install jam nut on top of primary nut and snug tighten.
9. Restore traffic.

Figure 4-7: Lead bearing sheet replacement detail

Chapter 5 PRESTRESSED CONCRETE SUPERSTRUCTURES

Prestressed concrete superstructures have become the most common superstructure type constructed in Texas. This superstructure type performs very well, and they are very durable. The most common preservation actions to prestressed beams consist of beam end repairs in accordance with Chapter 3 of the TxDOT Concrete Repair Manual and coatings applied to beam ends to protect the strands and slow down chloride induced corrosion.

Impacts from overheight trucks are the most common cause of significant damage to prestressed beams. Working drawings for strand splicing and concrete repairs, including carbon fiber wrapping, to address overheight impact damage have been developed and are available for use.

Chapter 6 STEEL SUPERSTRUCTURES

This chapter is intended to assist in typical steel element condition assessment and repair/rehabilitation selections for maintenance and rehabilitation projects.

6.1 Typical Steel Superstructure Issues in Texas

Generally, steel bridges requiring preservation work in Texas consist of the following superstructure types: trusses, rolled beams, box and tub girders, and plate girders. While not numerous, trusses require significant considerations given the complexity of connections and member arrangements, Nonredundant Steel Tension Members (NSTMs) details, general advanced age (most built in the 1930s and 1940s), and vulnerability to corrosion issues. Older steel rolled beam and plate girder bridges in Texas have characteristics such as lower load ratings, non-composite thin concrete decks, damage due to overheight impacts, corrosion damage at expansion and deck construction joints, older bearing types that are corroded, frozen, or rocked over, and paint failure. Newer designs for these structure types often use unpainted weathering steel for its corrosion resistance.

To ensure that all bridges reach their intended service life, especially for steel bridges, it is recommended to control roadway drainage by:

- diverting roadway drainage away from the bridge structure.
- clean troughs or reseal deck joints.
- maintain deck drainage systems.
- periodically clean and, when needed, repaint all steel within a minimum distance of 1.5 times the depth of the girder from bridge joints.
- regularly removing all dirt, debris and other deposits that trap moisture.
- regularly removing all vegetation which can prevent the natural drying of wet steel surfaces.
- maintaining covers and screens over access holes.
- removing growth of nearby vegetation that prevents the natural drying of surfaces wet by rain, spray, or other sources of moisture.

Figures 6-1 through 6-3 show the typical issues related to steel bridges.

Figure 6-1: Over-height vehicle impact damage to steel beams.

Figure 6-2: Paint failure of steel beams.

Figure 6-3: Corroded steel bearings under expansion joints.

6.2 Steel Member Repairs

Due to the complexity of repair of trusses, box girders, and tub girders the repair procedure, it is not covered in this guide. Contact Bridge Division for more information. See FHWA report on techniques for bridge strengthening here https://www.fhwa.dot.gov/bridge/steel.cfm

6.2.1 Over-height Vehicle Impacts

There are a wide variety of steel repairs that are performed and generally Item 784, "Steel Member Repair" is the Standard Specification Item that has been used to specify and pay for the work. A registered Texas professional engineer shall assess the damage and design the repair plans. The language in Item 784 mostly deals with heat straightening steel members, but it is acceptable to be used for a wide variety of repairs. As such, the plans should clearly define the work and any special processes the designer believes necessary to complete the repair. When lead is present, measures must be taken to mitigate the hazard. Per Item 6.10 "Hazardous Materials" in the 2014 Standard Specifications, dealing with hazardous materials found within the project is the responsibility of the Department to mitigate, except when cleaning and repainting steel utilizing Item 446.

6.2.2 Corrosion Related Issues

6.2.2.1 Mitigation of Pack Rust

Pack rust is a form of localized corrosion typical of steel components that often develops in crevices, resulting in rust packing between steel plates, often in built-up members. Indiana DOT has completed research at Purdue University (SPR-4121) "Pack Rust Identification and Mitigation Strategies for Steel Bridges": https://docs.lib.purdue.edu/cgi/viewcontent.cgi?article=3211&context=jtrp. Several methods have been included:

- Missouri method- Use calcium sulfonate rust penetrating sealer applied in accordance with SSPC-PA1.
- Oregon method- Remove pack rust is by mechanical cleaning and heating water-saturated pack rust between 250 and 400 °F or using 35,000 psi ultra-high pressure water blast.

For existing steel structures with details particularly vulnerable to pack rust and section loss, additional efforts beyond normal repainting may be necessary. TxDOT is currently evaluating improved methods to address pack rust and section loss. Substantial section loss could reduce the load rating and load carrying capacity of the structure and/or cause the bridge inspector to rate a component in a poor condition. Replacement of members might be necessary.

For truss bridges and bridges with complicated connection details vulnerable to corrosion, the System III-A paint system is recommended, and requires a stripe coat described in Item 446.4.7.4.4. A stripe coat provides a buildup of paint coating to better protect edges. Beyond the stripe coats, caulking can be used at plate interfaces to limit the ingress of moisture into joints vulnerable to corrosion. Approved caulking materials are located in the MPL, "Paintable Caulk for Concrete and Steel," at the following link: http://ftp.dot.state.tx.us/pub/txdot-info/cmd/mpl/pntclk.pdf. A recommended note to provide in the plans is: "Before applying the appearance coat but after application of the intermediate coat, caulk gaps or crevices greater than 1/16 inch throughout the length of the project. Use a paintable caulk meeting the requirements of DMS-8142, "Paintable Caulk for Concrete and Steel," and that is listed on the MPL, "Paintable Caulk for Concrete and Steel." Apply sealant in a manner that does not trap moisture.

6.2.2.2 Mitigation of Section Loss

Knife edging is an advanced stage of corrosion leading to thin, jagged edges on steel members. If not treated, the paint system cannot properly protect these surfaces as the edges are too small, which does not allow enough paint to cover it. Treatment of such conditions can be handled by example details such as the one shown in Figure 6-4.

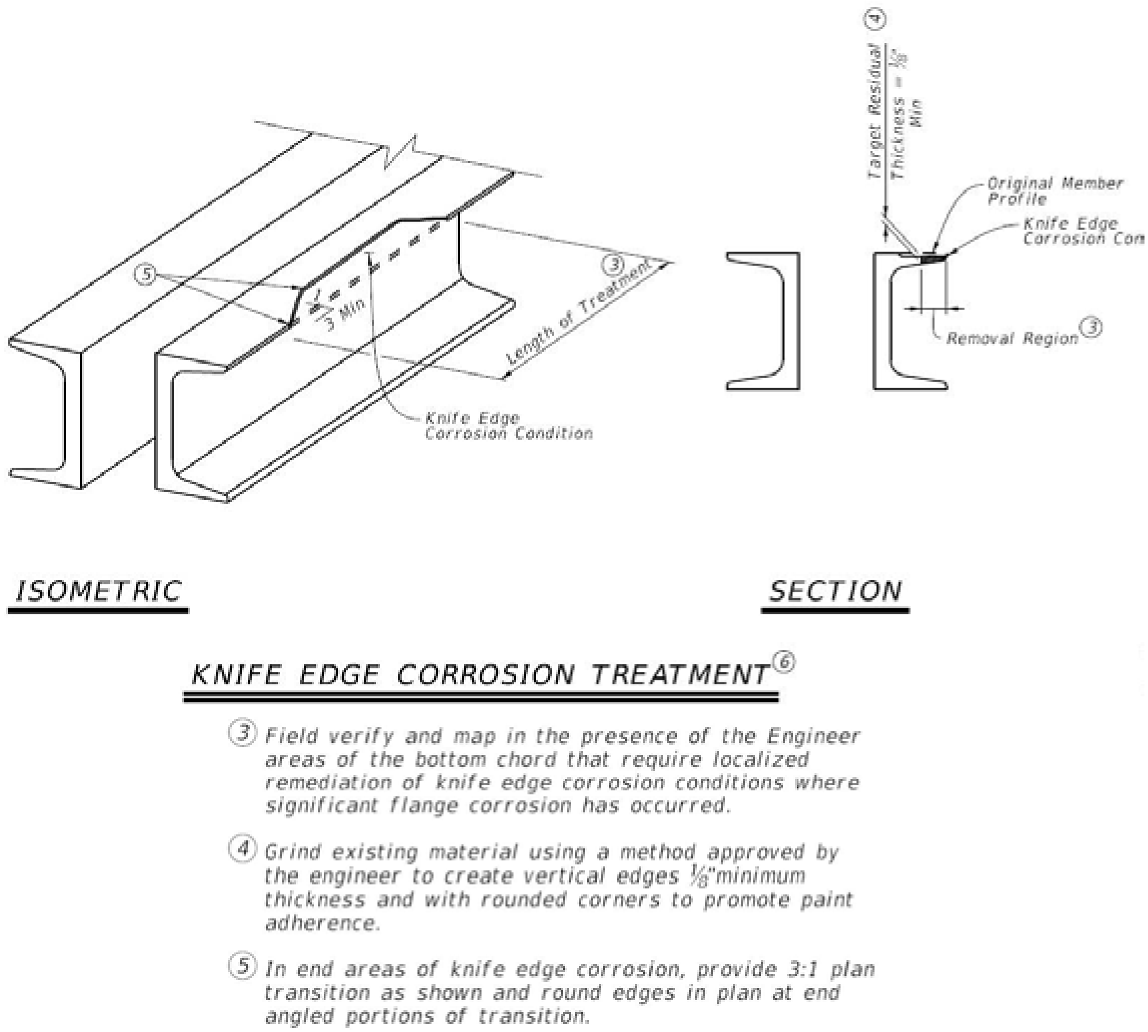


Figure 6-4: Example Detail for Knife Edge Corrosion Treatment

6.2.3 Repair of Steel Stringer (Rolled Beam and Plate Girder) Bridges

Steel stringer distress is often associated with over-height vehicle impact damage. The first two exterior beams of bridge on SB IH35 at FM 3176 were heavily damaged by an overheight vehicle, while the other interior beams had minor damaged. The southwest exterior steel plate girder and the diaphragm on span 14 and 15 of the bridge over IH 610 EB had a severe overheight impact damage. A low clearance bridge on the IH 10 WB at US 69 in Beaumont had an overheight impact that resulted in fracture of the beam web and the bottom flange. Figures 6-5 (a) through (c) show examples of overheight impact damage.

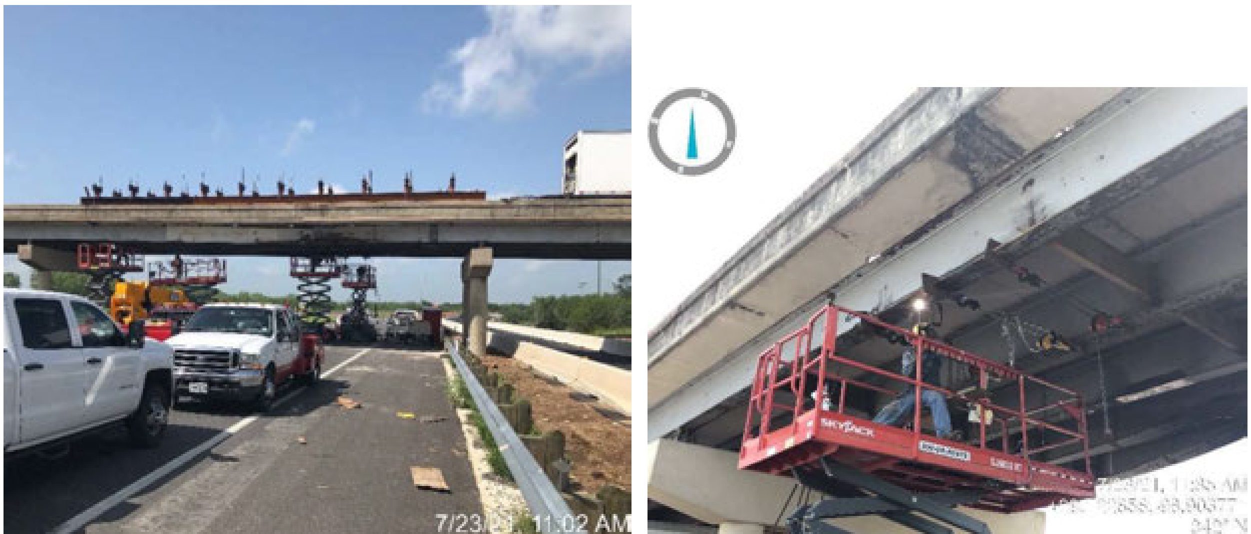
(a)

Figure 6-5: Examples of Overheight Impact Damage on: (a) SW exterior beam for IH 610 EB (b) diaphragm on span 14 and 15 of IH 610 EB (c) exterior beams of bridge on SB IH35 at FM 3176

Impact damaged steel members can often be repaired by heat straightening. However, heat straightening may not be appropriate for all impact damage and there are occasions where either half-section or full-section replacement of the damaged member is necessary. Repeated over-height impacts can introduce cracks or gouges into the member. As part of a heat straightening repair, the impact area should be ground smooth, with the grind marks running parallel to the beam. This will improve the stress flow through the member and decrease the likelihood of future fatigue induced fracture at these locations. More details are covered by NCHRP report 604, "Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance." Available at: http://www.trb.org/Publications/Blurbs/160020.aspx. Additional resources from FHWA can be found at https://www.fhwa.dot.gov/bridge/steel.cfm

It is a TxDOT practice to apply heat straightening where the impact damage is minor and to replace the beam with either a half-section or a full section when it is necessary. Where half-section or a full section beam replacement is necessary, a strong back beam may be installed to support the deck. Figure 6-6 (a) and (b) shows an example of beam replacement with provision for a strongback beam, and heat straightening repair, respectively.

All repair work must conform to Item 446, "Field Cleaning and Painting Steel." Heat straightening and steel beam replacement shall conform to Item 784, "Steel Member Repair." All welding shall be done in accordance with Item 448, "Structural Field Welding" and must be performed by a certified Welder.



(a) (b)

Figure 6-6: Examples of Overheight Impact Damage Repair: (a) Beam replacement with a Beam strongback on SW exterior beam of a bridge on SB IH35 at FM 3176 (b) Heat straightening for beams of bridge on SB IH35 at FM 3176

6.3 Steel Coatings

Projects where the major effort of work is cleaning and repainting the steel, steel repair work may be included and paint mitigation to allow the repairs to take place will be the responsibility of the contractor. However, if the contract does not include Item 446 as a major item of work, paint remediation is the responsibility of the Department. The two usual ways TxDOT handles this work is to remove the paint at the area of work either prior to letting the contract including the repair work or indicate within the plans that TxDOT will mobilize a third party to remove the hazardous coating during the contractor operations and that the contractor will need to work with the Department to allow this to happen. There are occasions, such as emergency projects and when permission is granted by TxDOT Director of District Operations, dealing with coatings containing hazardous materials to complete the work will be the responsibility of the contractor. On these occasions, the contract plans must clearly indicate where and what is to be performed within the contract. Usually, because of the complexity of steel repairs, it is a good practice to include a note in the plans requiring the Bridge Division be notified prior to beginning the work to allow coordination of having a Department Inspector be on the project to inspect the work as may be necessary.

6.3.1 Field Painting Systems

For bridge preservation, field painting for rehabilitation of steel superstructures is governed by Item 446 “Field Cleaning and Painting Steel”. There are four general paint systems for field painting in steel superstructure rehabilitation. See Table 6-1 for comparison and applications of these systems. Payment under Item 446 is usually by the Lump Sum, but if there are a mix of structures and structure sizes/types on a project, payment can also be by the Each to help account for varying levels of effort required.

Table 6-1. Comparison of Item 446 Paint Systems for Rehabilitation

System	Primer	Intermediate Coat	Topcoat	Paint Method	Note
I-A		N/A	One-Coat Overcoat	Overcoating	
I-B	Penetrating epoxy sealer	Epoxy intermediate	Polyurethane	Overcoating	
II	Organic Zinc	Not applicable	Acrylic Latex	Zone Painting Blast/Repaint	Recommended default system for non-Coastal bridges
III-A	Organic Zinc	Epoxy intermediate	Polyurethane	Zone Painting Blast Repaint	Recommended for coastal bridges and for bridges susceptible to corrosion from road salt use

6.3.2 Field Painting Methods

There are three different paint methods that are chosen based on the existing coating condition, economics, and desired life expectancy: spot or zone painting, overcoat, and blast and repaint. The life expectancy of any paint system, regardless of method will be highly dependent on proper surface preparation and application. Poor surface preparation or improper coating application will lead to early failure of the paint system.

6.3.2.1 Spot or Zone Painting

Spot painting and zone painting are both painting approaches aimed at addressing localized areas of corrosion or coating failure at locations of concern, such as bearings, diaphragms, and beam ends at expansion joints. For both approaches, the balance of the coating system should be in satisfactory or good condition. The primary purpose is to prevent the areas of concern from experiencing significant section loss. Typically, a special protection system will be specified in the plans for this type of application. Special specifications have been written for these approaches to reduce the amount of surface preparation required. An estimated surface area to be painted should be provided in the plans. For zone painting, a common practice is to specify treating the end three to five feet of the beam ends at expansion joints. To estimate surface areas for the bearings and diaphragms, it is common to use a percentage of the main beam end surface area. This percentage should be relatively high, between 25% to 40% of the gross area.

6.3.2.2 Overcoating

Overcoating is a method that may be used for structures with existing paint systems that have not advanced to high level of deterioration. The area of paint failure should be less than 10-20% of the total surface area to be a cost-effective method. The paint systems used in overcoating projects are System I-A and System I-B (for high corrosive environment). The life expectancy of this coating is approximately 10 to 20 years. One example of a candidate bridge for overcoating is shown in Figure 6-7.

Figure 6-7: Example of candidate steel superstructure for overcoating.

6.3.2.3 Blasting and Repainting

This is a method that completely removes the existing coating system and rust and applies multiple coats to all surfaces. The typical paint systems used in these projects are System II or System III-A (high corrosive environment or trusses). The life expectancy of this coating is over 20 years with a proper application (15 to 20 years in coastal environments). This method represents most painting projects because it is the most reliable long-term method that maximizes the time between maintenance efforts and associated traffic disruption. Repainting is recommended when 25% or more of the coating has failed. A candidate bridge for blasting and repainting is shown in Figure 6-8.

Figure 6-8: Example of candidate for blast and repaint.

6.3.3 Hazardous Material

Repainting older steel bridges typically involves handling hazardous materials. Handling hazardous materials increases liability and insurance costs for contractors and creates logistical, coordination, access, and scheduling issues during construction when handled by a third party.

Item 6.10 of the 2014 Standard Specifications governs the responsibility of hazardous material handling unless otherwise shown in the plans, all coatings are assumed to hazardous materials.

The removal, containment, and disposal of the existing paint and hazardous material are intrinsic parts of Item 446 "Cleaning and Painting Steel". For projects where Item 446 is a pay item, the responsibility for removal, containment, and disposal of hazardous materials falls to the contractor.

In a repainting project, lead paint removal uses abrasive media with a recycler that captures the lead paint where it is subsequently disposed of at a control facility. This equipment is very expensive, heavy and requires a lot of space and takes measurable space, as shown in Figure 6-9. For small jobs, it may be more cost effective to allow a non-recycled abrasive cleaning system. There is no current statewide Special Provision for this clause, and a one-time use example is posted here: ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/specs/2014/prov/sp446003.pdf

(a)

(b)

Figures 6-9. Recycled abrasive media recycling (a) and containment (b) on Wharton Truss.

6.3.4 Quality Control and Inspection

The performance of repainted bridges is highly contingent on the quality of inspection of the removal and painting operations. TxDOT District inspectors and Construction Engineering Inspection (CEI) representatives are generally not experienced in bridge painting. The TxDOT Bridge Division has a 3rd party indefinite deliverable paint inspection contract that can provide NACE CIP certified inspectors with years of painting and industry experience and knowledge of industry standards and practices. As projects are being developed, contact the Bridge Division to coordinate these efforts. It is also recommended to provide the following verbiage either in the General Notes to Item 446 or the plan sheets with regards to paint inspection:
"Contact the Engineer to coordinate third party paint inspection a minimum of two weeks prior to the preconstruction meeting. The Engineer shall arrange with the TxDOT Bridge Division and/or third-party inspector presence at the preconstruction meeting."

Item 446 requires Society for Protective Coatings (SSPC) QP1 and QP2 certifications for painting contractors. QP1 is "Field Application to Complex Industrial and Marine Structures" and QP2 is "Field Removal of Hazardous Coatings". For smaller projects with lower technical risk, TxDOT is open to allowing the QP7 certification "Painting Contractor Introductory Program" on a case-by-case basis. If QP7 is invoked as an option, the suggested General Note language is: "QP 7 certification, as a substitution to SSPC QP1 and SSPC QP2, is permitted for this project."

6.3.5 Paint Color and Anti-Graffiti Coatings

The default paint color per Item 446 is a concrete gray appearance coat (Federal Standard Color 595C, #35630). If another color is desired, the color of the appearance coat should be specified in the plans or via a plan note stating, "Contact Engineer for color of appearance coat". Paint color may often be dictated by an environmental process that considers community preference, or in the case of historic structures, original historic documented paint color or the input of local community representatives. Online resources for paint color include: http://www.fed-std-595.com/FS-595-Paint-Spec.html and http://www.colorserver.net/fandeck.asp?page=58. The first number in the paint color code is the level of sheen with 1 = gloss, 2 = semi-gloss, and 3 = matt. While the glossier paint finishes may be more amenable to cleaning graffiti, etc., they will also show more irregularity in brush strokes from touch up. Anti-graffiti coatings work best in semi-gloss instead of flat.

If an anti-graffiti coating is desired due to elevated risk of vandalism, a Type III anti-graffiti coating is preferred from a maintenance perspective since it only requires pressure washing (and not a graffiti remover as with Type II coatings). In these cases, it is recommended to have the following notes (depending on the paint system chosen): "For System II paint system, provide a Type III (Water-Cleanable) Anti-Graffiti Coating in accordance with DMS-8111, "Anti-Graffiti Coatings," in lieu of the acrylic latex appearance coat. Submit the proposed anti-graffiti coating to the Engineer for approval, including manufacturer certification that the anti-graffiti coating is compatible with the organic zinc primer and can be applied directly to the primer."

6.3.6 Estimating Costs

Currently, the estimated costs for easily coated surfaces (e.g., plate girders) are about $15 to $25 per SF. For hard to coat surfaces (e.g., trusses), the 2020 estimated costs are about $25 to $35 per SF.
Review recent similar project costs when estimating work.

6.4 Use of Shear Connector to Improve Loading Capacity

Modern steel bridges use shear connectors as a means of providing composite action and horizontal shear transfer for efficiency, redundancy, and enhanced load carrying capacity. Historically that was not always the case for older steel bridges in Texas. Early era rolled steel beam bridges in Texas date back to the inception of the Texas Highway Department (THD) and used simple spans until the late 1930's, and later continuous spans until the early 1960's. Slab thicknesses on these bridges were as little as 6 ½". Nearly all these earlier era steel bridges were designed to be non-composite. However, they often had details that embedded the top flange a nominal amount which could induce some unintended composite behavior. As a result of original designs with design loading as low as H-10, many of these structures have inventory load rating levels below HS-20. The addition of shear connectors is an effective method to improve this load rating.

6.4.1 Deck Replacement and New Shear Connectors

When the existing bridge deck is in poor enough condition that bridge deck replacement is necessary. It is recommended to salvage the existing steel girders and make them composite with the new deck by the addition of arc-welded shear studs. Current bridge deck design practice calls for an 8-½" thick composite deck which may be substantially thicker than the original design.

Load rating calculations should establish whether the existing steel beams, superstructure, bearings, substructure, and foundations can sustain this additional thickness. A new slightly thinner deck could be investigated as a compromise between the greater dead load, the capacity of the existing elements proposed to remain, and the benefits of providing composite action. Shear studs should be provided in accordance with the design provisions of Chapter 3, Section 13 of the TxDOT LRFD Bridge Design Manual and Chapter 3, Section 10 of the TxDOT Bridge Design Guide. Consideration should be given to whether the use of partial depth precast concrete deck panels (PCPs) is feasible for the deck replacement given the bedding width, flange width, room needed for shear stud installation, and slab thickness geometric allowance.

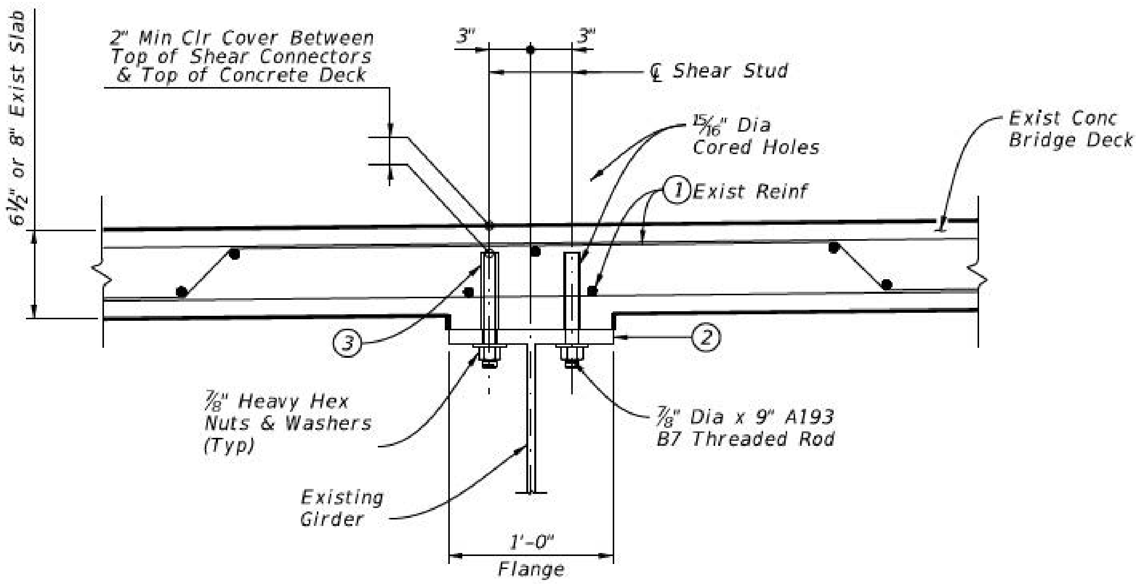
6.4.2 Post-Installed Shear Connectors

Post-installed shear connectors are another method for improving load rating of an existing steel superstructure if the condition of the bridge deck does warrant replacement. Post-installed shear connectors use drilled and epoxied threaded rods to generate composite action between an existing bridge deck and existing stringers. The primary advantage is avoiding the cost and traffic control associated with complete removal of an existing bridge deck. Post-installed shear connectors are a long-term solution and are most practical in cases where the deck width and condition are adequate for the foreseeable future. Post-installed shear connectors were studied in two research projects 4124 "Bridge Strengthening through the Use of Post-Installed Shear Connectors" and 6719 "Strengthening Continuous Steel Girders with Post-Installed Shear Connectors":

http://ctr.utexas.edu/wp-content/uploads/pubs/5_4124_01_1.pdf
http://ctr.utexas.edu/wp-content/uploads/pubs/5_4124_01_2.pdf
https://library.ctr.utexas.edu/ctr-publications/0-6719-1.pdf

The design and construction method recommended in the reports should be followed. The method can improve both the negative and positive moment load rating. The adhesive anchor shear connectors have improved fatigue performance over conventional welded studs. This permits a less dense installation and partially composite design since the strength limit state will likely be the only control. Studies from the research showed that many typical TxDOT non-composite bridges could achieve a 60% increase in load rating by this method.

This method has been used to strengthen the IH-10 Cibolo Creek Bridge (Bexar Co. CSJ 0025-02-212) and the SH 149 Sabine River Bridge (Gregg Co. CSJ 0393-01-097). Figure 6-10 is an example cross-section detail from the Sabine River Bridge plans. Figure 6-11 shows field installation on the Sabine River Bridge.



SHEAR CONNECTOR DETAIL

(Installation from below deck)

1. Do not cut reinforcing when coring holes. Locate bars and, as directed by the Engineer, shift holes to avoid steel.
2. Drill a 1-inch diameter hole through the top flange of the steel beam at the shear connector location.
3. Through the hole in the flange, drill a 15/16 – diameter hole into the concrete deck to the desired depth. Clean the hole with wire brush and compressed air, as specified by the adhesive installation procedure.

   Inject adhesive into the hole using the appropriate dispenser. Fill the hole from the top down to prevent air pockets from forming.

   Place the threaded rod into the hole using a twisting motion so the adhesive fills the threads.

   Allow the adhesive to cure in accordance with the manufacturer's recommendations.

   Tighten the nut to the torque specified by the adhesive manufacturer.

Figure 6-10: Example Post-Installed Shear Connector Detail

Figure 6-11. Installation of Post-Installed Shear Connectors.

6.5 Bearing Repair or Replacement on Steel Bridges

Older steel bridges often have fixed/bolster and rocker bearing assemblies that have corroded, frozen, or rocked over. A common rehabilitation is to repaint and reset these bearings. A more extensive rehabilitation is to completely replace these bearings usually with elastomeric bearings meeting current standards.

6.5.1 Repairing/Resetting Existing Bearings

If the corrosion condition of the bearings is not too advanced, the bearings can simply be reset under Item 499 “Adjusting Steel Shoes”. See Figure 6-12 for an example. Item 499 includes payment for the jacking effort, so Item 495 is not required. Item 499 indicates using jacks with a capacity of at least 1.5 times the shoe design load or as indicated in the plans. It is suggested to provide the calculated dead and live service load (the total original service load design is usually in the original plans), especially if the bridge will be open to traffic during these operations. In situations involving re-decking, the reduced dead load after deck removal that would require less extensive jacking and it would be advisable to reset the girders before pouring the new bridge deck. The jacking plans, provided and sealed by contractor’s engineer, must be approved by Bridge Division before execution.

Figure 6-12. Example of rocker shoes requiring resetting, but not replacement.

6.5.2 Bearings Replacement

Existing steel and elastomeric bearings may need replacement. For steel bearings, the underlying causes include exposure to corrosive influences due to leaking bridge expansion joints, and accumulated debris, excessive thermal expansion, frozen elements, and abutment movement/rotation. Figure 6-13 is an example of such a situation. It is important to identify as many root causes as possible in addition to replacing the bearings. One installation example is shown in Figure 6-14.

For older elastomeric bearing pads, the most common reason for replacement is pads slipping or "walking out" from under the beam. Like modern bearings pads, the replacement pads should be designed to prevent slip failure, while also matching the existing pad thickness as closely as possible.

Figure 6-13. Example of rocker bearing requiring replacement

(a) (b)

Figure 6-14. (a) Suspended sole bearing pad from sole plate with surface preparation and (b) Casting of bearing seat to obtain bearing at proper elevation.

An important requirement is the need to provide a shoring system to facilitate bearing replacement. The plans should indicate the calculated unfactored dead load and live load reaction at each girder line so the contractor can design a corresponding jacking and shoring system. Shoring details and lift plans should be submitted to Bridge Division for approval. Removal and disposal of the existing bearings is considered subsidiary to the other items. While existing end diaphragms are often a convenient jacking point for bearing replacement, their condition may require jacking/shoring under the girder line and in front of the abutment or bent. All beams in the bridge cross-section at a given bent/abutment location should be lifted simultaneously, and in most cases, it advised to replace all the bearings along that bearing line.

The design of elastomeric bearing replacements should follow the guidance of Chapter 5 Section 2 of the TxDOT LRFD Bridge Design Manual and Chapter 5 Section 2 of the TxDOT Bridge Design Guide. For elastomeric bearings on steel bridges, additional consideration should be given to the following factors:

- Provide sole plates between the girder and top of pad that match the existing documented girder slopes and provide convenient mechanisms for field welding to the bottom flange and anchor rods or other mechanisms for providing lateral restraint. Anchor rods should be installed using an adhesive anchor system.
- The bearing pad should be flat and tapered due to the use of sole plate, which helps with expansion and loading capacity.
- Due to the light vertical loads associated with steel girder bridges, it is recommended that the elastomeric pads be vulcanized to the bottom of the sole plate. A minimum sole plate thickness of 1 ½” at centerline bearing is recommended to protect the pad against the heat from the connection field weld.
- Continuous steel bridges have a fixed joint and the expansion length for design should be set accordingly.
- Bridge standards SGEB and SBEB can be used as an initial guide for developing details but should be customized for the bridge.
- Skewed bridges may present geometric challenges such as pad and sole plate clips, and available footprint for anchor rods, especially given the narrower substructure widths of older existing bridges.
- Sole plates need to generally be painted since most of the existing bridges are also painted (and may need repainting). For sole plates, a System IV Paint using coatings compatible with the paint system for the balance of the superstructure is recommended. Anchor rods and associated hardware should be hot-dipped galvanized.
- Payment for bearings is typically by the EACH under Item 434 “Elastomeric Bearings”.
- Payment for concrete pedestals for support shall be by Each pedestal under Item 420 (e.g., 0420-6103 CL K CONC (PEDESTAL). With small volumes of concrete, payment by EACH is a good protection against quantity errors and high potential unit bid price. The advantage of concrete pedestals is the ability to adjust height with the new bearing suspended/tied in place.

Chapter 7 CONCRETE SUBSTRUCTURE

7.1 Common Substructure Issues

It is common for concrete substructure to develop cracking, spalling, delamination, and other distresses due to construction defects, environmental conditions, use of de-icing salts, corrosion issues, impact damage, and other adverse factors. Figure 7-1 shows typical substructure distress. Repairs for substructure distress like that captured below can generally be addressed with plan notes and schematic level sketches, reference to applicable sections of the Concrete Repair Manual, and repair quantities.

Figure 7-1. Example of concrete substructure deterioration.

7.2 Vehicle Deflection Walls

Vehicle deflection walls are one of several methods of pier protection described in Section 2.2 of the TxDOT Bridge Design Manual – LRFD (Bridge Design Manual) and in the TxDOT Bent (Pier) Protection Guide. The purpose of a vehicle deflection wall is to distribute the load resulting from a vehicular impact to a larger cross section. Safety to the traveling public is achieved through end treatments, discussed in more detail in Section 7.2.2, End Treatments. Vehicle deflection walls are a retrofit option, particularly for two-column bents which lack the structural redundancy offered by bents with 3 or more columns. Other factors to be considered in implementing vehicle deflection walls are routes with a high volume of truck traffic, and locations where the annual frequency for a bridge bent or pier to be hit by a truck is greater than 0.001 as stated in Section 2.2 of the Bridge Design Manual. A typical vehicle deflection wall is shown in Figure 7-2.

Figure 7-2. Vehicle Deflection Wall along IH10 near Beaumont, TX

7.2.1 Design Considerations

The design load for vehicle deflection walls is specified in Section 2.2 of TxDOT Bridge Design Manual. Both the wall and connection to the existing columns must be designed to resist this design load. A deflection wall can be either single-sided or double-sided, depending on traffic considerations and column spacing. Generally, where the clear distance between columns is greater than 11 feet, the wall will be double-sided with a thickness equal to the column diameter. If the clear distance is less than 11 feet, the wall may be single sided, depending on the location of the bent in relation to the direction of traffic. If there is traffic on both sides of the deflection wall, the wall should be double-sided. Refer to Figure 7-3. The dimensions shown are for information only as individual deflection walls should be designed on a case-by-case basis.

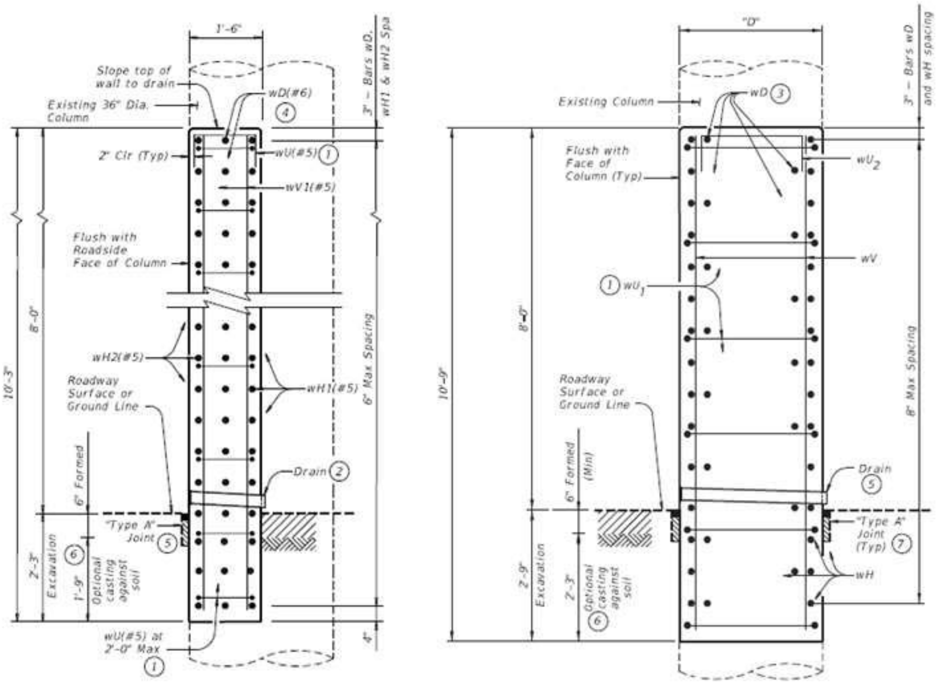


Figure 7-3. Sections showing single-sided and double-sided deflection walls.

The designer should also consider the additional loads that will be applied to the foundation because of the deflection wall.

Aesthetic form liners may also be used with a maximum 2-inch relief.

7.2.2 End Treatments

As shown in Figure 7-4, a vehicle deflection wall is often equipped with transition walls on one or both sides. The purpose of these walls is to transition from the column to an end treatment, typically metal beam guard fence. Other end treatment options include crash cushion attenuators, sand barrel systems, or transitions to concrete railing. An end treatment may be required on the upstream end of the vehicle deflection wall, depending on roadside safety design requirements and clear zone considerations. At the designer's discretion, the end treatment on the downstream end may be omitted. The district preference is to be considered when selecting crash cushion attenuators. The districts have experiences on performance of crash cushion attenuators. See Figure 7.4 for end treatment examples.

Figure 7-4. Various End Treatment Options. Top: Crash Cushion Attenuators; Left: Sand Barrel system; Right: Transition to concrete barrier

7.2.3 Construction of Vehicle Deflection Walls

The traffic control for vehicle defection wall construction shall not significantly impact the traffic.

The bid code for construction of vehicle deflection wall is 0420 6049, CL C CONC (CRASHWALL), UNIT: CY.

Chapter 8 STEEL PILING & CONCRETE ENCASEMENTS

Advanced corrosion or section loss on steel piling can be missed during routine inspections, and if unaddressed, it can lead to catastrophic consequences. The two most common methods to address deteriorated piles are painting and encasing the piles in concrete. An example of severely corroded steel piles (before and after repair) is shown in Figure 8-1 and 8-2.

8.1 Painting Steel Piles

When the deterioration of the piles is primarily due to coating failure and there is little to no measurable section loss, painting the piles may be an appropriate preservation action. Generally, a System I-A overcoat will be preferred since it does not require abrasive blasting and the piles will likely be in the main channel. However, TxDOT has found concrete encasements are often more cost effective than painting.

8.2 Pile Encasement

Concrete encasements may be considered when there is moderate to advanced section loss on the piles, or as a proactive approach instead of painting. Concrete encasements are not intended to add any structural capacity to the piles and should only be thought of as a protective measure to prevent additional deterioration. Concrete encasements are often performed by in-house maintenance personnel but may also be added to bridge maintenance or preservation contracts.

Figure 8-1. Severe steel section loss on multiple piling triggering a critical finding requiring emergency repair contract work.

Figure 8-2. Steel piling with concrete encasements installed to address severe steel section loss.

TxDOT has developed a working drawing that can be incorporated into repair plans to address significant pile deterioration. The working drawing is included in the Appendix and available at https://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm.

Chapter 9 BRIDGE RAILING RETROFITS

This chapter is intended to assist in identifying appropriate repair or retrofit strategies for bridge rails.

9.1 Background

Bridge rails retain and redirect errant vehicles and/or provide for pedestrian safety. Rail retrofits may be performed when existing rail is damaged, or when the existing rail is being upgraded to current standards, or to meet a minimum height requirement. Figure 9-1 shows a damaged rail. This rail type is no longer used for new construction.

Figure 9-1: Damaged Type PR-1 (MOD) railing.

Since the railing policy is continuously evolving, the designers are to check the current TxDOT Bridge Railing Manual (http://onlinemanuals.txdot.gov/txdotmanuals/rlg/rlg.pdf) for Texas bridge railing policy and additional specific information.

9.2 Options for Existing Railing

For bridge preservation projects, the condition of the rail must be evaluated and one of the following three options must be chosen according to TxDOT Bridge Railing Manual, Chapter 4," Treatment of Existing Railing,":

- Keep the existing rail intact if it is allowed by the current TxDOT railing policy.
- Retrofit the existing rail to meet the policy requirements.
- Replace the existing rail with new MASH compliant rail.

Figure 9-2 shows a recently retrofitted rail.

TxDOT has published a Bridge Railing Identification Guild (https://ftp.dot.state.tx.us/pub/txdot-info/library/pubs/bus/bridge/railing.pdf) to help field personnel identify the existing rails.

Figure 9-2: Retrofitted T223 Railing, Meeting MASH TL-3.

9.3 Options for Damaged Bridge Railing

It is common that railing is damaged by vehicular impact. Determine the condition of existing damaged railing using the information provided on Form 2488, “Information Sheet for Bridge Railing Upgrades, Retrofits, & Repairs,” along with photos, existing as-built plans, and routine bridge inspection records.

9.3.1 Repair

When the damaged rail is deemed as repairable. Refer to the TxDOT Concrete Repair Manual for identifying and diagnosing concrete damage categories and specifying concrete repair procedures (e.g., concrete rail or curb/anchorage). Depending on the extent of damage, it may be necessary to replace the rail, curb, and/or anchorage. Consider specifying this work in accordance with Item 776, "Metal Rail Repair", or 778, "Concrete Rail Repair" which includes both removal and replacement of damaged portions of the old rail.

9.3.2 Replacement

When the existing bridge rail warrants replacement according to TxDOT rail policy, only TxDOT approved rail is allowed. All new installations of bridge rail, on-system, and off-system bridges, with contract letting dates after December 31, 2019, must meet FHWA crash-test criteria as specified in Manual for Assessing Safety Hardware (MASH) 2016. Refer to the TxDOT Bridge

Railing Manual for more information about the MASH implementation of bridge rails. Each district may have preferred railing types.

9.4 Rail Retrofit Guidance

Always ensure that railing retrofits provide adequate anchorage. Refer to the following TxDOT Retrofit Guides, which are available on the TxDOT Bridge Standards web page.

- CC-RAIL-R (Retrofit Guide for Curb Concrete Rails)
- C-RAIL-R (Retrofit Guide for Concrete Rails)
- T131RC (Retrofit Guide for Curbed Structures (TL-3)
- RAC-R (Retrofit Guide for Box Culvert with Curbs 2’ & less)

Do not use any of the Retrofit Guides as a Standard. Either modify the Retrofit Guide or create new drawings so that the information is project-specific and relevant to a specific rail retrofit. Either way, rail retrofit sheets must be signed and sealed by a Texas Professional Engineer.

In most cases, there is no need to modify the actual rail standard. Always check with the Bridge Standards Engineer before modifying a railing standard.

9.5 Rail Retrofit Design and Specification

The type of rail that can retrofitted may be limited by the weight of the rail. For an example, replacing steel railing with concrete railing may add a significant dead load to the bridge superstructure which is transferred to the girders. Always perform a load rating calculation as change in rail weight may affect live load carrying capacity. Do not design a retrofit with a rail type that would overstress any element of the bridge superstructure or substructure, or that would lead to load posting of the bridge.

Rail retrofits may reduce the roadway/shoulder width. Typically, the “nominal face of rail” is assumed to be 1 ft. from the outside edge of the bridge slab, regardless of actual physical dimensions. Bridge rail types T66, T224, T80TT, and C66 are exceptions; their “nominal face of rail” is assumed to be 1.5 ft. from the outside edge of the bridge slab. Use nominal widths on bridge layouts and typical sections but provide actual dimensions in railing retrofit details to ensure that existing conditions accommodate the retrofit.

Bridge rail retrofits may require removal and replacement of mow strips, metal beam guard fence, Thrie-beam transitions, and/or downstream anchor terminals. If required, use standard details from the TxDOT Roadway Standards web page and coordinate with the district to ensure they would like to include those quantities are included in the plan set.

Refer to Chapter 3 of the TxDOT Bridge Railing Manual for guidance on pedestrian, bicycle, and ADA (Americans with Disabilities Act) bridge railing requirements. Coordinate with the District and accommodate District preferences whenever possible.

Chapter 10 Concrete Culverts

Concrete repairs should generally follow applicable repair procedures per Section 3.1 – Minor Spall Repair and Section 3.2 – Intermediate Spall Repair of the Concrete Repair Manual.

10.1 Culvert Slip Lining

When damage to the culvert soffit is severe and widespread, slip-lining is one option that has been used to restore structural capacity of a culvert. Slip-lining consists of installing a pipe within the deteriorated culvert followed by placing flowable fill between the new pipe and the deteriorated culvert. TxDOT Special Specification 7001 "Slip Lining Pipe or Box Culverts" can be used in combination with available working drawings for this work.

While slip-lining can restore the strength, there is a reduction in hydraulic capacity that may dictate larger scale repairs or culvert replacement as a long-term solution. Cost comparison of large-scale rehabilitation versus replacement will likely conclude that replacement is most economical especially when traffic phasing will be required for the rehabilitation work.

10.2 Culvert Undermining

When culverts become undermined, they may settle. In extreme cases, this can lead to one or more boxes breaking away from the remaining sections. Depending on the severity of the damage, one option may be to backfill the settled sections with flowable fill. All possible outlets beneath the culvert should be blocked off to prevent loss of backfill material.

When one or more boxes break away from the remaining sections, replacing those sections will likely be more cost effective than attempting a repair. Consideration should also be given to replacing the entire culvert.

(a) **(b)**

Figure 10-1: (a) Culvert Failure due to Undermining; (b) Failed boxed replaced in-kind

Chapter 11 USE OF FIBER REINFORCED POLYMER IN BRIDGE PRESERVATION

Fiber-reinforced polymer (FRP) composites are designed to resist corrosion and provide high strength-to-weight and modulus-to-weight ratios compared to steel and concrete.
Over the service life of a highway bridge, its constituent materials are continually subjected to deterioration from external loads caused by mechanical actions, and chemical and environmental factors. Aggressive chloride and humidity aggravate the effect of chloride based deicing salts on bridge structural elements, in addition to joint leakage that allows passage of deleterious substances to bent and abutment caps.

FRP has also been extended for use as containment of repair material in prestressed concrete girder with exposed strand after over height impact damage.

Use item code 0786 6001 when FRP is used as corrosion protection, and 0786 6002 when FRP is used for strengthening.

Chapter 12 CATHODIC PROTECTION

Cathodic protection (CP) is an advanced technique used to mitigate corrosion of embedded steel reinforcement in concrete, especially for structures located in marine and coastal environment. According to the extensive research conducted by numerous governmental agencies, CP is the only rehabilitation technique that has been proven to inhibit corrosion in salt-contaminated structural bridge elements, including bridge decks.

It is recommended to provide Cathodic Protection System on the substructure elements- (piles, pile caps, bent caps, etc.,) of major bridges in the coastal area of the state as a corrosion mitigation for preservation strategy. Figure 12-1 shows the installation process of cathodic protection system on the substructure of the JFK Causeway, in Corpus Christi.

CP is based on the principle of applying an external source of current to counteract the internal corrosion current produced in reinforced concrete elements. During this process, current flows from an auxiliary anode material through the electrolyte (concrete) to the surface of the reinforcing steel. Various materials in various configurations can be used as auxiliary anodes for CP, resulting in various types of CP systems. The selection of the anode material and its configuration is important to the success of the CP system.

CP system is classified according to the mode through which protection is provided:

1. Galvanic cathodic protection systems
2. Impressed current cathodic protection systems.

12.1 Galvanic Cathodic Protection System

The galvanic cathodic protection (GCP) system works on the principle of sacrificial anodes. Galvanic corrosion is achieved using an electro-chemical process which links multiple metals electrically. Its utilization of sacrificial anodes protects the embedded steel reinforcement from corrosion. The sacrificial anode is usually made of a more reactive metal that is intentionally electrically linked to the structure due to its reactivity to the corrosive environment. A major shortcoming of this method is that these sacrificial anodes must be replaced once they are corroded. However, GCP can offer protection that spans over 10-20 years. Some commonly used materials for sacrificial anodes are aluminum, zinc, and magnesium.

GCP systems are generally suited for lower resistivity soils and submerged structural element.
Recently, the GCP system was installed on the substructure of the JFK Causeway to extend the service life of the structure and as a preservation action plan.

Figure 12-1: Installation of Cathodic Protection System on the substructure of the JFK Causeway

12.2 Impressed Current Cathodic Protection System

The impressed current cathodic protection system is typically used when GCP systems are insufficient to mitigate corrosion. Impressed current systems protect reinforced concrete structures better in the most extreme corrosive environments. In this system, an active DC power source is provided which allows more current to flow to the embedded reinforcement thereby, provides a constant stream of protection from corrosion. Due to the maintenance requirements associated with this method, Impressed Currents Cathodic Protection is not typically used in Texas.

Since the design and selection of cathodic protection system is very complex, necessary approval must be obtained from the Bridge Division at the planning stage.

Chapter 13 RIPRAP REPAIR

Riprap is generally located at the ends of a bridge for drainage and erosion protection of slopes and embankments along highways. For bridges over water, riprap is also commonly used to protect channel banks from erosion and bridge abutments/piers from scour that may be caused by geomorphic factors or changes in hydraulic conditions around bridge openings.

13.1 Types of Ripraps

There are primarily two types of slope or embankment protection systems: rigid type and flexible type. Rigid bank protection consists of riprap that forms an impermeable layer which does not conform to changes in the supporting subgrade surface. Concrete riprap is the most common rigid embankment protection used in Texas. Flexible protection consists of riprap that is permeable and able to conform to changes in the supporting embankment surface (slopes and streambeds). Stone protection is the most common flexible riprap used in Texas.

13.2 Rigid Type Riprap

13.2.1 Concrete Riprap

Concrete riprap has a long history of use in Texas, and contractors are familiar with its construction. TxDOT standards of concrete riprap can be found at:
https://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/bridge/MS-CRR-19.pdf
The TxDOT standard for concrete riprap specifies 5 inches thick riprap (RR8) at stream crossings and 4 inches thick riprap (RR9) at other embankments. However, it is generally preferable to use flexible at stream crossings rather than concrete riprap.

Figure 13-1: Concrete Riprap under Bridge Abutment

13.2.2 Fully Grouted Stone Protection Riprap

This rigid type of riprap utilizes stone protection with voids filled with either concrete or grout. This creates conglomerates that provide a roughened surface for water to flow without allowing water to penetrate the voids.

Examples of fully grouted stone protection riprap are shown in Figures 13-2 and 13-3.

Figure 13-2: Grouted Stone Protection – Rough Surface

Figure 13-3: Grouted Stone Protection – Conglomerates

Because fully grouted stone protection is not considered flexible, this method is not recommended for use on Texas bridges as a scour mitigation method.

13.2.3 Field Issues of Rigid Type Riprap

Concrete riprap provides a smooth surface for water to flow, which may not help dissipate energy from runoff water on a steep slope. Fully grouted stone protection provides a rough surface, but it is generally more difficult to construct. Both forms of rigid riprap are susceptible to erosion at the toe of header banks even when a toe wall is provided. The erosion can undermine the riprap, leading to loss of supporting soil underneath the riprap. One example is shown in Figure 13-4. Rigid riprap is also susceptible to settlement cracking and separation at joints, which provides water entry points that accelerate erosion. Examples are shown in Figures 13-5 to 13-7. In addition, hydrostatic pressure can build up behind the rigid riprap, which could lead to failure of the riprap in part or in its entirety (e.g., slope stability at abutment).

It is also worth noting that voids that form under concrete riprap may not always be visible. Sounding of rigid type riprap is highly recommended to detect problems at the early age during field inspection.

Figure 13-4: Concrete Riprap – Undermining at Toe. Note that the current CRR standard required a deeper toe than shown here.

Figure 13-5: Concrete Riprap - Undermining and Settlement

Figure 13-6: Concrete Riprap - Settlement Cracks

Figure 13-7: Concrete Riprap - Loss of Supporting Soil

13.2.4 Concrete Riprap Repair and Maintenance

Undermining of concrete riprap may not always be obvious until the rigid protection system collapses partially or completely. Every effort should be made to properly inspect and detect cracks and voids as early as possible. Minor voids can become a major problem rather quickly.

Water penetration and subsequent undermining is one of the main causes of riprap slope failures. Maintain joint seals and divert surface water away from the riprap whenever possible. Shoulder drains or flumes can be installed as a mitigation method. Adequate control of surface water is key to minimizing repair needs. Avoid concentrated flows onto slopes unless water is carried using flumes. Clear and properly maintain surface drains, flumes, and other pathways.

Seal the cracks in accordance with Item 713, "Cleaning and Sealing Joints and Cracks (Concrete Pavement)" as for jointed concrete pavement.

Use Flowable Fill (Item 401) to fill voids. Precautions must be taken not to rupture the concrete riprap by the built-up hydrostatic pressure. Large and open areas can be filled using aggregate or cement stabilized sand, which can also be used if voids are encountered behind abutments.

Using stone protection riprap in lieu of concrete riprap, at a stream crossing is recommended.

13.3 Flexible Riprap Protection

Flexible protection consists of riprap that is permeable and conforms to changes in the supporting or natural surface that provides protection to abutments and channel against surface erosion and scour. It is worth to note that only Stone Protection Riprap in the CRR standard is considered as flexible riprap in any case. There are several types of flexible riprap including stone protection riprap, partially grouted stone protection riprap, interlocking articulated concrete blocks, gabions, gabions mattresses and concrete armor units. Each of the above systems have varying degrees of flexibility when exposed to channel erosion.

13.3.1 Stone Protection Riprap

Stone protection is generally installed by machine and/or hand-placed depending on the site condition (e.g., headroom under a bridge, water level and flow of a channel, and size of the stone. Etc.). Placement requirements can be found in the TxDOT Specifications in Item 432.

Stone protection riprap must be designed to resist water flowing at high velocities from displacing the stone material. The stone protection size and gradation should be designed based on the predicted velocity of flow in the channel. A stone riprap sizing spreadsheet based on HEC 23 guidance is available for download at:
https://tntoday.dot.state.tx.us/BRG/Pages/Scour%20Forms%20and%20Guidance.aspx#. The filter system must prevent finer material from escaping through voids created by large stones. One example of stone protection is shown in Figure 13-8.

Figure 13-8: Stone Protection Riprap at Bridge Abutment and Pier

13.3.2 Partially Grouted Stone Protection Riprap

Partially grouted stone protection riprap is constructed by filling some of the voids with either concrete or grout. Water is allowed to enter and escape through the voids, minimizing the risk of having hydrostatic pressure buildup behind the riprap. However, partially grouted riprap is very difficult to construct to achieve the desired flexible behavior, so it is generally not recommended on TxDOT scour remediation projects. Examples are shown in Figures 13-9 and 13-10.

Figure 13-9: Partially Grouted Riprap

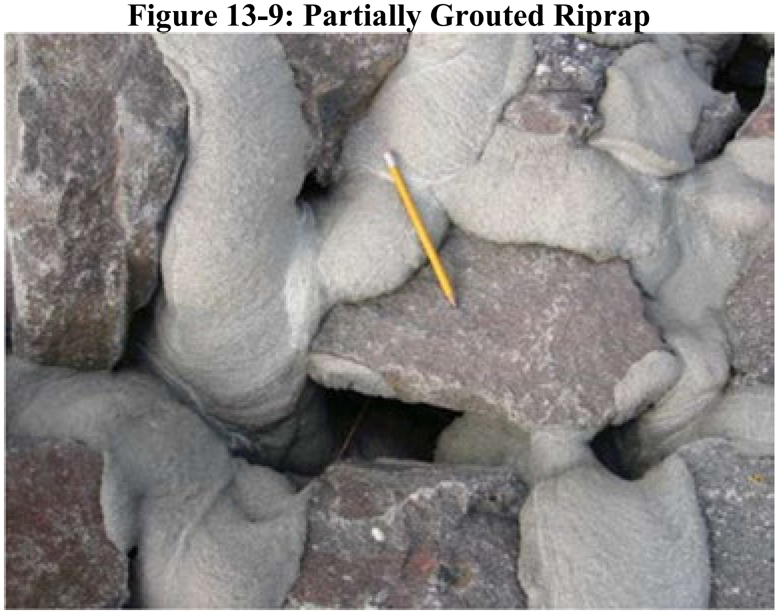


Figure 13-10: Partially Grouted Riprap – Close-up View

13.3.3 Interlocking Articulated Concrete Blocks

Interlocking articulated concrete blocks (IACB), can move without separating. Individual precast small blocks are usually placed by and/or linked with wire. IACB can conform to the natural shape of the slope when placed. Vegetation grows through the openings of the precast blocks, which contributes to the overall stability of the slope and provides a natural looking surface. Pinning IACB, top anchor, and bottom embedment of IACB can provide shear resistance against sliding of the system on a slope. Refer to the TxDOT Special Specification Item 4153. Examples are shown in Figures 13-11 and 13-12.

Figure 13-11: Interlocking Articulated Concrete Blocks

Figure 13-12: Interlocking ACBs - Vegetation Growth

13.3.4 Gabions

Gabions consist of rectangular wire mesh baskets that are filled with rock of a suitable size. Source rock can be expensive and difficult to obtain in some areas in Texas, which could limit the use of gabions. Gabions are typically used for channel bank protection, and they can be configured to fit even steep slopes ((H: V greater than 2.5:1)). The use of gabions requires a stability analysis, especially when used on a steep slope or in a retaining wall configuration. However, near vertical gabions are not typically recommended due to the difficulties for repair, design, and maintenance. Examples are shown in Figures 13-13 and 13-14.

Figure 13-13: Gabions - Channel Bank Protection

Figure 13-14: Gabions at Bridge Pier *Gabion Mattress*

Gabion Mattress

Gabion mattress consists of shallow height wire mesh baskets that are filled with rock of suitable size. Examples are shown in Figure 13-15 and 13-16.

The thickness of gabion mattress should be designed based on the hydraulic characteristics of the site.

Figure 13-15: Gabion Mattresses at Bridge Abutment

Figure 13-16: Gabion Mattress

13.3.5 Concrete Armor Units

Concrete armor units consist of three mutually perpendicular precast concrete units that are rigidly fixed at the center. The interlocking characteristic of concrete armor units usually provide greater stability against rolling or sliding compared to a rock with similar size on a steep slope. However, the strong interlocking system tends to behave more rigidly when undermining occurs. Examples are shown in Figures 13-17 and 13-18.

Figure 13-17: Concrete Armor Units

Figure 13-18: Concrete Armor Units- Vegetation Growth

13.3.6 Combination Flexible Riprap

Flexible riprap can be constructed with a combination of each of the protection systems mentioned above as shown in Figure 13-19. This provides flexibility in design and ease of adaptation in the field.

Figure 13-19: (Left to Right) Stone Protection Riprap, Articulated Concrete Armor Unit and Gabion Mattresses

13.3.7 Flexible Riprap - Field Issues

13.3.7.1 Stone Protection Riprap

Visible problems in the field include erosion at the toe where stones are carried away by the current. This can lead to rock sliding down the slope and leaving the natural slope without any protection, especially when stone protection is placed on a slope that exceeds 2.5:1. Inspect stone protection riprap and identify areas with missing stone sizes. Look for displaced stones or variations in the thickness and uniformity of the stone protection riprap. Two examples are shown in Figures 13-20 and 13-21.

Figure 13-20: Stone Protection Riprap – Erosion at Toe

Figure 13-21: Stone Protection Riprap Compromised

13.3.7.2 Stone Protection Riprap Repair and Maintenance

Construction of stone protection riprap is not complicated. It does not require specialty equipment and it can be easily adopted by maintenance forces. As such, it can also be easily inspected and repaired if necessary. However, there is no standard, and each system must be designed using guidelines offered under FHWA's Hydraulic Engineering Circular HEC-23, "Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance." Stone protection riprap must be designed to resist water flowing at high velocities from displacing the stone material and it must prevent finer material from escaping through voids created by larger stone sizes.

Small slumps or displacements of rocks should be repaired soon after they are observed. For large slumps or loss of rock from a continuous section, it is important to assess the cause(s) before repair. Review design criteria as needed. In some cases, other flexible revetment can be a viable option.

For proper design and construction of stone protection riprap, refer to TxDOT's Scour Evaluation Guide (https://ftp.txdot.gov/pub/txdot-info/library/pubs/bus/bridge/scour-guide.pdf).

After proper preparation of the subgrade, filter fabric or bedding material, should be placed in a manner that allows intimate contact with the subgrade surface. At no time should the work area be disturbed in a manner that results in loss of this contact.

Granular filter material should be tested for grain size distribution to ensure it meets the gradation specifications used in the design. Sample and test the granular material in accordance with TxDOT's Guide Schedule of Sampling and Testing.

The minimum thickness of granular filters should be the larger of 4 times the d50 of the filter stone, or 6". Increase the thickness by 50% when placing the filter underwater. A large diameter tremie pipe can be used for granular material placement around bridge piers underwater. This helps control placement location and thickness and minimizes segregation potential.

Ensure each roll of geotextile is labeled with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. Do not expose geotextile to sunlight prior to placement. Place the overlying materials as soon as practical to avoid overexposure to sunlight and to reduce damage from ultraviolet radiation. If possible, do not drive heavy machinery over geotextile fabric during construction to avoid tearing the fabric and rendering the filter inoperative. Voids, gaps, tears, or other holes should be avoided as much as possible. If they do occur, the filter must be repaired or replaced. Place geotextile filters so that the upstream strips overlap the downstream strips, and ensure the overlaps are in the direction of flow whenever possible.

13.3.7.3 Interlocking Articulated Concrete Blocks

Visible problems in the field include undermining of the subgrade material which can lead to complete loss of support for the precast blocks. Sections of the blocks can settle with partial or complete failure of the system. In the field, look for disjointed blocks and any inconsistency in the spacing between blocks. Proper subgrade preparation is an essential element of the construction of interlocking articulated concrete blocks. One example is shown in Figure 13-22.

Figure 13-22: Interlocking Articulated Concrete Blocks – Settlement

13.3.7.4 Gabions and Gabion Mattresses

Commonly encountered problems consist of erosion or undermining at the toe, which could lead to excessive settlement. In the field, look for any signs of rotation or misalignment of the gabions. Properly designed embedment with consideration of scour depth at channel is important in maintaining a functional system in case there is erosion at the toe. Gabions may be designed for accommodating some degree of settlement and lateral deflection. It is not recommended to install gabion blocks too close or wrapping around the bridge columns. This is to avoid unnecessary extra loading imposed by a deforming gabion system to bridge columns. One example is shown in Figure 13-23.

Figure 13-23: Gabions – Undermining and Settlement

13.4 Riprap Repair or Replacement

The first consideration is whether the current riprap type is suitable for the bridge sites. TxDOT has in the past replaced concrete riprap with stone protection riprap to address streambed erosion and scour. The second consideration is the short-term and long-term costs, especially when the river channel is not stable.

For repair/retrofitting, the following best practices are recommended:

- Establish mild slopes (3:1 or less) whenever possible, especially in soils susceptible to erosion.
- When possible, divert surface drainage water away from riprap.
- Stockpiling of riprap for repairs of eroding unprotected banks. Riprap should be sized for the “worst” conditions. It may be worthwhile to stockpile very large, uniformly graded rock separately, as for toe construction during emergency works.

Figure 13-24: SRR used to replace a failed CRR.

Chapter 14 PLAN PREPARATION AND WORKING DRAWINGS

14.1 Plan Preparation

PS&E plans for bridge maintenance and preservation projects can be prepared either in-house or by consultants. With either approach, the active involvement of district personnel from the very beginning of the projects is crucial to the success of the project. Design concept conference calls or design kick off meeting are essential to keep all stakeholders on the same page.

The plans shall be reviewed by district and Bridge Division at the predetermined stages, typically at 60%, 90%, and 100%. The review comments must be addressed in the next design stage to ensure the quality of the design work.

In general, PS&E for bridge maintenance and preservation plans do not require the same level of detail as plans for new construction. A simple bridge layout with repair locations identified, a quantity summary sheet, and a few simple details are often all that is required. It is highly recommended to include photographs of specific or representative repair areas to aid Contractor's in understanding the work to be performed. An example plan set can be found at https://crossroads/divisions/brg/sections/field-operations-section/bridge-preservation-resources.html (Appendix B).

14.2 Repair Working Drawings

Repair working drawings have been developed, for common repair and preservation activities. These drawings are available on the Bridge Standards Website; however, they should not be used as standard drawings. The details on these drawings must be modified by the designer to fit the specific situation and require a PE's seal and signature.

The link to the working drawings is shown on Appendix A.

Appendix A WORKING DRAWINGS

Index Sheets of Working Drawings can be found at https://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/bridge/WD-Table-22.pdf

Appendix B TYPICAL PLAN SET

An example sheet from a typical plan set is shown.

APPENDIX B

TABLE OF REPAIRS						
REPAIR NO.	ITEM	BID ITEM DESCRIPTION	UNIT	QUANTITY	REPAIR DESCRIPTION/LOCATOR	DETAILS/NOTES
D1	0429-6005	CONC STR REPAIR (DECK REP (FULL DEPTH))	SF	545	Perform full depth deck repair at the locations shown	Refer to the TxDOT Concrete Repair Manual, Chapter 3, Section 4
D2	0429-6007	CONC STR REPAIR (VERTICAL AND OVERHEAD)	SF	5	Repair minor to intermediate delaminations and spalls on overhang soffit in Span 1	Refer to the TxDOT Concrete Repair Manual, Chapter 3, Section 2
D3	0439-6013	MULTI-LAYER POLYMER OVERLAY	SY	838	Apply multi-layer polymer overlay to entire deck area.	Refer to Multi-Layer Polymer Overlay notes
	0483-6013	SHOT BLASTING	SY	838	Shot blast full deck area to prepare surface for MLPO	Refer to Multi-Layer Polymer Overlay notes
D4	0785-6008	BRIDGE JOINT REPAIR (FULL DEPTH)	LF	260	Perform full depth joint repair at Abutments 1 and 5, and Bents 2-4. Remove previous partial depth joint repairs at Bents 2-4	See Bridge Joint Repair Details
M1	0400-6001	FLOWABLE BACKFILL	CY	6	Fill voids beneath riprap and approach slab with flowable backfill	See Bridge Repair Layout for locations
R1	0451-6048	RETROFIT RAIL (ADD HSS)	LF	632	Add steel HSS section on top of existing rail.	See Type T2 Retrofit Details
R2	0545-6001	CRASH CUSH ATTEN (INSTL)	EA	4	Install crash cushion attenuator at ends of existing bridge rails	See Crash Cushion Summary Sheet (CCSS) and standards
SP1	0429-6007	CONC STR REPAIR (VERTICAL AND OVERHEAD)	SF	10	Repair minor to intermediate delaminations and spalls on end diaphragm	Refer to the TxDOT Concrete Repair Manual, Chapter 3, Section 2
SP2	4002-6001	REPLACE ELASTOMERIC BEARING PADS	EA	4	Replace elastomeric bearing pads at Abutment 1	See Bearing Pad Replacement Details
SB1	0420-6139	CL C CONC (VEH DEFL WALL) (HPC)	CY	43.1	Install Vehicle Deflection Wall at Bents 2-4	See Vehicle Deflection Wall Details
SB2	0427-6004	SILICONE RESIN PAINT FINISH	SF	615	Apply Silicone Resin Paint Finish to all exposed surfaces of Abutments, wingwalls, and as shown on interior bents.	See Interior Bent Repair Details
SB3	0786-6001	CARBON FIBER REINF POLYMER PROTECTION	SF	738	Wrap bent cap repairs in CFRP and apply protective appearance coat	See Interior Bent Repair Details
SB4	0429-6007	CONC STR REPAIR (VERTICAL AND OVERHEAD)	SF	390	Repair minor to intermediate delaminations and spalls on interior bent caps	Refer to TxDOT Concrete Repair Manual, Chapter 3, Section 2