SECTION 33
PRESERVATION AND REHABILITATION OF STRUCTURES

33.1 GENERAL REQUIREMENTS

The provisions of this section apply to structure preservation and rehabilitation projects, as defined herein.

33.1.1 Definitions of Preservation and Rehabilitation

Preservation and rehabilitation projects can be categorized into two primary groups based on the general scope of the work performed and the expected improvement to structure condition and structure lifecycle.

33.1.1.1 Bridge Preservation

The FHWA defines bridge 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 condition and extend their life” (2011). Preservation includes bridge maintenance activities (both preventive and reactive), as well as major preservation work.

Bridge maintenance projects are typically narrow in scope and restore the structure to its original condition by addressing existing deficiencies. These projects have minor costs and require minimal new design work. Example work types are crack sealing, concrete patching, debris clearing, and joint repair.

Preservation involves the repair and protection of a bridge element against future deterioration, thereby extending the service life of a bridge without significantly increasing load-carrying capacity or improving geometrics.

Preservation projects typically cost less than 30 percent of the cost of a new replacement bridge.

33.1.1.2 Bridge Rehabilitation

Bridge rehabilitation involves a significant investment in a bridge to improve its condition, geometrics, or load-carrying capacity to a minimum standard. This work is expected to provide a long-term benefit and reduce the need for additional investments. Projects that cost more than 30 percent of the cost of a new bridge are generally considered rehabilitation projects. Deck replacements, bridge widenings and superstructure replacement projects are considered rehabilitation projects regardless of estimated costs.

Bridge replacement should be considered if the cost of rehabilitation approaches or exceeds 70 percent of the cost of a new replacement bridge. The final determination on rehabilitation vs. replacement should be based on many factors, as discussed in the following sections.

33.1.2 Rehabilitation vs. Replacement Selection Guidelines

The following factors should be considered when deciding between rehabilitation and replacement for a structure. It should be noted that these are not absolute criteria for investment decisions. Because each project is unique, all circumstances and constraints should be considered during evaluation.

33.1.2.1 Cost

In conjunction with the CDOT Project Manager, the Designer shall coordinate the development of an appropriate comparison of the total project cost estimates for both rehabilitation and replacement options. Comparison of total project costs (including any anticipated costs associated with phasing, realignment, detours, environmental concerns, right-of-way acquisition, etc.) is necessary to determine the most cost-effective alternative. Rehabilitation and replacement costs should be estimated after all other factors have been investigated because the other factors may affect or determine the scope of the rehabilitation or replacement project.

As the estimated cost of the rehabilitation project approaches 70 percent of the cost of the replacement project, replacement becomes the more cost-effective choice in terms of life-cycle costs. This threshold is based on life-cycle cost models of rehabilitation and replacement for various bridges and is consistent with thresholds adopted by other state agencies.

As an alternative to using the above threshold, a refined life-cycle cost analysis may be performed. In this case, estimated life-cycle costs for rehabilitation and replacement options should be compared directly; applying the 70 percent factor when dealing with life-cycle costs is not appropriate. For more information about bridge life-cycle cost analysis, see NCHRP Report 483, “Bridge Life-Cycle Cost Analysis.”

33.1.2.2 Safety

Accident history should be considered for the existing structure. Accident potential should be considered for both existing and potential replacement structures. If the accident history or potential of the existing structure is determined to be unacceptable, the safety problem must be addressed either through rehabilitation or replacement. Rehabilitation costs associated with safety improvements shall be included in the rehabilitation estimate for comparison to replacement cost.

33.1.2.3 Structure Type

Certain bridges will be inherently predisposed to either rehabilitation or replacement based on their type and location. Structure types that are difficult or costly to rehabilitate may be stronger candidates for replacement. Special consideration should be given to the replacement of non-redundant bridges because they present increased maintenance costs and risk.

Historical significance may be a factor in favor of rehabilitation. For historical bridges that will be kept in the system, the Secretary of the Interior's Standards for Rehabilitation shall be consulted and close coordination with the Environmental group will be required throughout the project.

33.1.2.4 Bridge Standards

Existing vertical clearance, horizontal clearance, lane width, and shoulder width should be considered. If the existing features are nonstandard, consideration should be given to improving them through rehabilitation or replacement. Substandard geometry that cannot be reasonably addressed through rehabilitation is a factor in favor of replacement.

33.1.2.5 Hydraulic Performance

The hydraulic history of the bridge should also be reviewed. If the existing features are nonstandard, consideration should be given to improving them through rehabilitation or by replacing the bridge. Up and downstream impacts should be considered because the hydraulic implications of rehabilitation or replacement can push the decision in either direction.

Scour critical bridges for which there are no feasible countermeasures to mitigate the scour problems are stronger candidates for replacement.

33.1.2.6 Traffic Control

In some cases, practical solutions for temporary traffic control may drive the rehabilitation vs. replacement decision. For example, if project specifics prohibit temporary traffic configurations that could accommodate bridge replacement, rehabilitation may be the reasonable decision.

33.1.2.7 Environmental

Environmental impacts should be estimated for rehabilitation and replacement option and considered in the rehabilitation vs replacement decision.

33.1.3 Required Inspection and Testing

During the project scoping phase and before developing preliminary cost estimates, the Designer should conduct a field visit after general drawings have been developed to verify the deficiencies noted on the Structure Inspection and Inventory Report (SIA) and to document any additional issues that should be addressed or might require further testing and analysis. The Designer shall review the recommended maintenance activities and expand on them, if necessary. The Designer should verify and address the fundamental issue that caused the structure to be targeted for rehabilitation or replacement.

Chloride testing is required during the scoping phase for any project with new full-depth overlay replacements, deck widening or deck rehabilitation/repair. A minimum of 5 cores, but not less than 1 per 3,000 square feet of bridge deck, are required to be taken and tested. The cores shall be evenly distributed over the travel lanes. At a minimum, the chloride content at the level of the top mat of reinforcing must be determined. This requirement can be waived for bridge decks less than 20 years old that have been continuously protected throughout the life of the bridge deck by a thin bonded epoxy overlay, a polyester concrete overlay, or a functioning waterproofing membrane and asphalt wearing surface. This exception is granted under the assumption that these decks have not been critically contaminated with chlorides.

On partial-depth resurfacing projects, the thickness of the existing asphalt mat should be verified to prevent damage to the waterproofing membrane if applicable and to prevent damage to the bridge deck from the milling operations. One method of verifying the existing asphalt thickness is by drilling or coring into the asphalt mat down to the deck surface and measuring the asphalt thickness with a probe. The plan-view location of each measurement and the asphalt thickness should be recorded in an organized format and submitted to the Engineer prior to milling. Figure 33-1 shows an example asphalt thickness verification detail prior to milling and after the resurfacing.

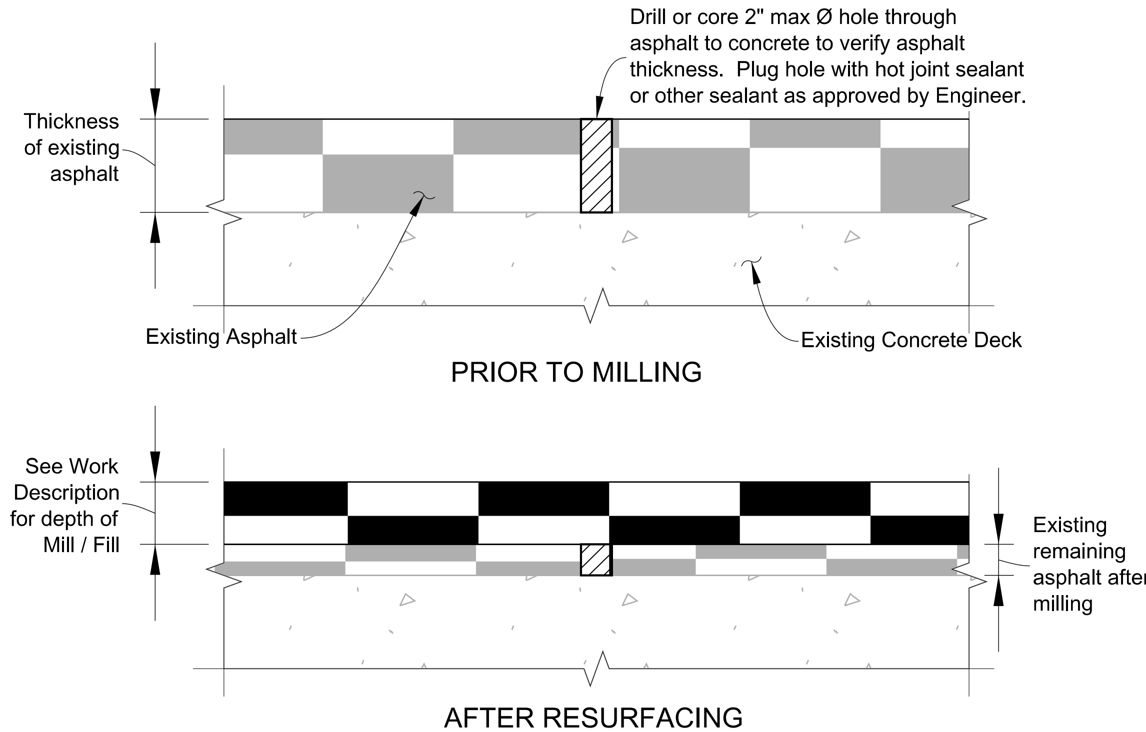

Figure 33-1: Asphalt Thickness Verification Detail

33.2 CODE AND PERFORMANCE REQUIREMENTS

The following provisions apply to all preservation and rehabilitation projects.

33.2.1 Existing Structure Evaluation and Preservation Projects

This section defines the acceptable design methodologies, codes, and minimum performance requirements to be used for both preservation projects (as defined in Section 33.1.1.1) and when evaluating an existing structure to determine if repair or rehabilitation measures are necessary. This includes existing structures that are being evaluated for scour criticality or increased dead load and structures with measured corrosion, section loss, or other damage in superstructure or substructure elements. Permanent load increases of 3 percent or less over what the bridge was originally designed for may not require analysis or rating, at the Designer's discretion.

33.2.1.1 Code Requirements

Structures designed per AASHTO LRFD shall be evaluated using AASHTO LRFD.

Structures designed by LFD or ASD methods may be evaluated with either the AASHTO Standard Specifications or AASHTO LRFD.

It is appropriate and acceptable to analyze older structures with the AASHTO Standard Specifications. However, in some cases, an LRFD analysis may yield more favorable results due to more refined methods of live load distribution or structural capacity. The intent of this provision is to not preclude the use of LRFD in these situations. A structure found to meet the minimum performance criteria when checked with either code should be considered acceptable.

When projects in this category require the design of a new element or retrofit, it is preferred to use AASHTO LRFD, when practical.

If existing caissons meet the current S-Standard requirements and the anchor bolts are in good condition, i.e. minimal corrosion, no loss of capacity, they may be reused for new sign structures.

33.2.1.2 Required Documentation and Minimum Performance Criteria

For existing structure evaluations, a rating summary sheet shall be completed for the element(s) under investigation using the applicable design code. Super- and substructure ratings shall be completed and documented in accordance with the CDOT Bridge Rating Manual and the Technical Rating Memorandum dated February 10, 2017. Additionally, for applicable substructure load combinations beyond the standard rating equations, performance ratios shall be reported separately.

Acceptable performance objectives for existing structure evaluations are as follows:

- Operating rating factor ≥ 1.0
- White color code
- Performance ratios for other load combinations ≥ 1.0

If all the above criteria are met, generally, no action needs to be taken or scour critical designation applied. When scour is involved, the operating rating factors and performance ratios typically refer to substructure elements affected by the scour, e.g. pile or caisson capacity.

If any of the above criteria are not met, it is not necessarily cause for action. The ratings of the element(s) under investigation shall be compared to the overall load rating of the bridge. In some cases, the overall bridge rating will not be controlled by the elements that required special investigation. If the overall rating is controlled by a substructure element, repairs are typically desired before making posting decisions. In all cases, existing structure evaluation results that do not meet the above criteria shall be discussed with Staff Bridge to determine the appropriate course of action. If a substructure is determined to be scour critical, refer to Section 33.13 for more information.

33.2.2 Rehabilitation Projects

This section defines the acceptable design methodologies, codes, and minimum performance requirements to be used for rehabilitation projects, as defined in Section 33.1.1.2. Because rehabilitation projects represent a substantial investment in an existing structure, they are subject to more stringent performance criteria to help ensure that they meet service life extension goals commensurate with their level of investment.

33.2.2.1 Code Requirements

All structures for rehabilitation projects shall be evaluated and/or designed using AASHTO LRFD regardless of original design code unless previous documentation in the Structure Selection Report and approval by Unit Leader (per Section 1.3 of this BDM).

33.2.2.2 Required Documentation

A Load and Resistance Factor Rating (LRFR) summary sheet shall be completed for the super- and substructure, as required. Super- and substructure ratings shall be completed and documented in accordance with the CDOT Bridge Rating Manual and the Technical Rating Memorandum dated 02/10/2017. For applicable substructure load combinations beyond the standard rating equations, performance ratios shall be reported.

For rehabilitation projects where no additional load is transferred to the substructure, and the substructure is otherwise performing adequately and has an NBI rating of 6 or greater, no analysis or rating of the existing substructure is required. Permanent load increases of 3 percent or less over what the original bridge was designed for may not require analysis or rating, at the Designer’s discretion.

Note that changes in superstructure continuity or boundary conditions can alter the distribution of forces and impose additional load on some substructure units. Such changes in load distribution shall be considered when determining if a substructure rating is required for a rehabilitation project.

33.2.2.3 Minimum Performance Criteria – Excluding Deck Replacements and Existing Portions of Bridge Widenings

For rehabilitation projects, excluding deck replacements, the inventory rating factor and all performance ratios shall be 1.0 or greater.

33.2.2.4 Minimum Performance Criteria for Deck Replacements

For deck replacement projects, the inventory rating factor shall be 0.9 or greater. The reduced minimum inventory rating accounts for the fact that some of the service life of the structure has already been realized. The new deck shall meet all AASHTO requirements.

For deck replacement projects where additional load is transferred to the substructure, the inventory rating of the substructure shall be 0.9 or greater. For load combinations not including live load, the performance ratio shall be 1.0 or greater.

33.2.2.5 Minimum Performance Criteria for Existing Portions of Bridge Widenings

Acceptable performance objectives for the existing portion of a widened structure are as follows:

- Operating rating factor ≥ 1.0
- No required posting
- White color code

- Performance ratios for other load combinations ≥ 1.0
- Superstructure, substructure, and deck condition ratings of 6 or greater

If the existing portion does not meet these performance objectives, the structure should be evaluated for strengthening and/or repair to the same load-carrying capacity as the widened portion. For the evaluation, the following should be considered, as appropriate:

- Cost of strengthening or repairing the existing structure
- Physical condition, operating characteristics, and remaining service life of the structure
- Other site-specific conditions
- Width of widening
- Traffic accommodation during construction

The final decision on whether the existing portion requires rehabilitation, and what it should include, shall be coordinated with the Region and Unit Leader.

33.3 REHABILITATION

33.3.1 General Requirements

The rehabilitated structure shall have a fair or good NBI condition rating after rehabilitation.

Rehabilitation projects should seek to eliminate functional obsolescence if reasonable. For example, if widening a bridge, the width should be increased enough to accommodate standard roadway geometry, where feasible.

If a structure is functionally obsolete for reasons that cannot be easily addressed through rehabilitation, structure replacement should be considered rather than making further investments in a functionally obsolete structure through a rehabilitation project. The ability to address functional obsolescence during structural rehabilitation is highly project specific.

33.3.2 Added Service Life

The following are target service life extensions for various types of rehabilitation and preservation:

- Estimated deck service life
  - Terminal decks (condition rating 3 or less) with minor patching and bituminous overlay: 2 to 5 years
  - Deck to remain in place with protective measures: 20 years for deck
- Membrane waterproofing and bituminous overlay. The life of the bituminous overlay may be 10 to 12 years. The membrane may need to be replaced each time the overlay is replaced if it has been damaged or is otherwise performing poorly.

- Polyester concrete overlays, cathodic protection, and rehabilitation of other deck types: 15 to 25 years depending on traffic volume and prior condition of deck
- New concrete deck with epoxy-coated reinforcement: 50 years

- Expansion joint end dams
  - Same as deck – periodic replacement of glands or trough should be expected
- Beam end repairs and/or rehabilitation
  - Minimum: Same as deck
  - Desirable: 50 years
- Repair and/or rehabilitation of other superstructure types and their elements
  - Minimum: Same as deck
  - Desirable: 50 years
- Bearings
  - Same as the existing girders
- New superstructure
  - Minimum: 50 years
  - Desirable: 75 years
- Substructure rehabilitation
  - Same as superstructure
- Retaining walls
  - Minimum: 25 years
  - Desirable: 50 years
- Culverts
  - Minimum: 15 years
  - Desirable: 50 years
- Bridge widening
  - Minimum: 50 years
  - Desirable: 75 years
- Sign structures
  - Minimum: 25 years
  - Desirable: 50 years
- Ground-mounted sound barriers
  - Minimum: 15 years
  - Desirable: 40 years
- Structure-mounted sound barriers
  - Same as deck

- Temporary bridges
  - 3 to 5 years

33.3.3 Acceptable Methods

Many systems and products can be effectively used for rehabilitation, including, but not limited to, the following.

33.3.3.1 Micropiles

Micropiles are commonly used for a range of retrofit or rehabilitation purposes, including:

- Arresting or preventing structure movement
- Increasing load-bearing capacity of existing foundations
- Repairing or replacing deteriorating or inadequate foundations
- Adding scour protection to existing structures

Micropiles are well suited to projects with the following constraints:

- Restrictions on footing enlargements
- Low overhead clearances
- Difficult access

33.3.3.2 External Post-tensioning

External post-tensioning (PT) may be considered for retrofit of all girder or other structural elements, including concrete and steel. Active strengthening systems, such as external PT, introduce external forces to the structural elements that would offset part or all the effects of external loads. Active systems are usually engaged in load sharing immediately after installation and can provide increased strength and instantaneously improve the service performance, such as reducing tensile stresses (or cracking) and deflections.

An advantage of external PT is that it needs to engage the structure only at end anchorages and at points of tendon deviation. For this reason, external PT can be added to existing structures with relative ease. Both steel and concrete box girders can usually accommodate the necessary anchorages and tendon deviations from inside the box. Monostrands require relatively small anchorage forces on a per tendon basis, thereby allowing simplified anchorage and deviation details on the retrofitted structure.

33.3.3.3 Carbon-fiber Reinforced Polymer

Passive strengthening systems, such as Carbon-fiber Reinforced Polymer (CFRP), do not introduce forces to the structure or its components. Passive systems contribute to load sharing and the overall resistance of the member when it deforms under external loads. As such, the effectiveness and load sharing of passive systems significantly affect their axial and bending stiffness.

CFRP features include a slim profile, high strength to weight ratio, chemical resistance, and ease of application. These attributes can lead to long-lasting, inexpensive, and rapid restorations that can be implemented in the field with minimal disturbance to traffic flow. Lastly, the structure's original configuration, including vertical and horizontal clearances, is maintained.

ACI 440.2R-08, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures," provides guidance for the design and construction requirements of CFRP retrofits.

33.3.3.4 Ultra-high Performance Concrete

Ultra-high performance concrete (UHPC) exhibits high early strength, develops a strong bond to existing concrete surfaces, and has enhanced durability. These characteristics make it an acceptable candidate for repair and rehabilitation work such as concrete patching, closure pours, and toppings.

33.3.4 Timber Structures

Due to the difficulty of finding new girders, durability and challenges of crash worthiness, widening of timber structures is generally not recommended.

33.3.5 Concrete for Repairs

Concrete Class DT shall be used only for complete toppings.

The current CDOT Project Special Provision – Revision of Section 601 Concrete Class DR – allows the use of either pre-packaged concrete patching material (bagged mix) or plant batched mix, giving the Contractor the ability to select the most economical and practical choice for the project. However, the Designer should be aware that certain circumstances may necessitate the use of a bagged mix only.

Patch repairs on bridge decks present logistical complications. Because of traffic control implications, deck repairs are often performed at night when batch plants are not operating. In this case, a bagged patching mix must be used. Additionally, total patch volume is commonly much less than the smallest volume able to be batched (2 cubic yards), resulting in waste.

Projects involving night-time lane closures may also benefit from the use of a bagged patching mix because of the reduced cure time compared to Class DR. A bagged mix can accommodate traffic loading in as little as 3 hours, where Class DR requires 6 hours. This time constraint is especially restrictive when replacing expansion devices and end dams because these projects require the completion of time intensive tasks during the closure, thereby limiting the time available for concrete curing. If a night-time closure cannot accommodate the required cure time before reopening to traffic, temporary bridge decks must be used. Temporary bridge decks may require the placement of extensive asphalt ramps and have experienced other difficulties in the field. For these reasons, a bagged mix is typically preferable for deck patching and placement of new expansion joint end dams.

The current policy of allowing either a bagged mix or Class DR may be revised in the future if either option proves to have superior durability.

33.4 BRIDGE WIDENING

Bridge widening represents a substantial investment in an existing structure and presents many unique challenges and opportunities for improvement. See Section 33.2.2 for required design code and performance objectives for the new and existing portions of widened bridges.

33.4.1 General Widening Requirements

The new portion of a widened structure shall comply with the following requirements:

- The bridge should be widened sufficiently to accommodate standard lane and shoulder widths, where feasible.
- Longitudinal deck joints are not permitted due to durability concerns.
- Fatigue-prone details should not be perpetuated.
- Mixing steel and concrete girders in the same span should be avoided due to thermal movement incompatibility.

33.4.2 Design Considerations

33.4.2.1 Differential Superstructure Stiffness

Live load distribution factors given in AASHTO 4.6.2.2 for beam-slab bridges are conditional upon the beams having approximately the same stiffness. Widening a bridge with a girder shape different from the existing girders may require a more refined analysis to determine accurate live load distribution and to verify the design loads for the deck between the new and existing girders.

Generally, the Designer should attempt to limit the amount of differential deflection between the widened and original portions of the superstructure, where feasible. The Designer shall account for additional forces and stresses due to any differential deflection anticipated along the widening interface.

33.4.2.2 Differential Superstructure Creep and Shrinkage

Newly placed prestressed concrete will shorten due to long-term creep and shrinkage. When connected to an existing concrete structure that has already experienced most of its creep and shrinkage, the existing structure will restrain the shortening of the new structure to some degree. This restraint causes forces along the widening interface that shall be considered in design.

Similarly, differential strains of the superstructures can result in force effects at the interface between the existing and new substructures. Isolating the existing and new substructures is a potential strategy to mitigate this issue.

33.4.2.3 Differential Foundation Stiffness

When a structure widening includes widening the substructure and foundation elements, the compatibility of the new and existing foundation systems should be considered. If the new and existing foundations have substantially different stiffness, a differential deflection or settlement can be expected. This effect should be considered and minimized, particularly as it relates to imposed deformation and stresses on the superstructure.

The effect of initial settlement of the new foundation elements relative to the existing foundation should also be considered. This phenomenon can be expected even where the widened foundation is of similar type and stiffness to the existing foundation. Isolating the existing and new substructures is a potential strategy to mitigate this issue.

33.4.2.4 Closure Pours

Closure pours shall be used between the existing and new portions of deck when the dead load deflection due to deck placement is greater than 0.25 in.

The width of closure pours should be a function of the amount of differential deflection expected and a minimum of 24 in. for conventional concrete. The width of the closure pour may be less than 24 in. if UHPC is used in conjunction with a wearing surface to smooth out any abrupt differences in elevation on either side of the closure.

33.4.2.5 Galvanic Anodes

When a bridge widening includes exposing and lapping onto existing uncoated reinforcing steel in the deck or any other element that may be contaminated with chlorides, consideration shall be given to the use of galvanic anodes along the widening interface.

If the concrete of the existing bridge deck is sufficiently contaminated with chlorides and galvanic anodes are not used, corrosion along the existing-new concrete boundary can initiate or accelerate. See Section 33.5.1 for more information.

33.5 BRIDGE DECK REPAIR AND REHABILITATION

33.5.1 Chloride Induced Corrosion

Infiltration of chloride ions into concrete is the most common cause of corrosion initiation in reinforcing steel. Bridge decks in Colorado are primarily exposed to chloride ions through the application of deicing salts, such as magnesium chloride.

Once the concentration of chloride ions at the level of reinforcing reaches a critical threshold, the protective passive film surrounding the reinforcing breaks down and corrosion initiates. While the subsequent rate of corrosion depends on many parameters, including several environmental factors, some level of corrosion will be observed until the concentration of chloride ions is reduced to below the threshold through remedial measures.

Several options are available for repair and rehabilitation of chloride contaminated concrete structures, including, but not limited to:

1. Do nothing.
2. Remove spalled and delaminated concrete and replace with patching material.
3. Remove all chloride contaminated concrete and replace with patching material (this includes sound but chloride contaminated concrete).

4. Use electrochemical chloride extraction (ECE) to remove chloride from the surface of the reinforcing bars.

5. Install a barrier system.

6. Install cathodic protection to protect the steel from further corrosion.

Repair and rehabilitation options involving concrete patching introduce additional complications. The process of patching unsound and/or chloride contaminated areas of existing decks requires placing new chloride-free concrete adjacent to existing concrete. If the existing concrete has a sufficiently high chloride concentration level, the patching process will lead to the formation of incipient anodes just outside the patched area. The difference in electric potential between the steel in the chloride-free and chloride contaminated sections drives corrosion at the incipient anodes, accelerating deterioration of the adjacent concrete. Rapid deterioration of the concrete surrounding the patch necessitates future repairs, creating a compounding maintenance and service issue. This phenomenon is commonly referred to as the halo effect.

Installing a barrier system (i.e., waterproofing membrane and wearing surface) on a deck that is chloride contaminated but not yet showing signs of distress may be ineffective. If the chloride concentration is at or near the threshold, corrosion of reinforcing will continue, resulting in deck deterioration. The damage occurring in the deck may become apparent only after significant damage has occurred under the overlay. In this scenario, the expected service life of the barrier system will likely not be realized.

For these reasons, projects that will include deck repair, patching, or installation of new waterproofing membrane and overlay should first identify the chloride contamination of the deck before determining viable rehabilitation methods. See Section 33.1.3 for requirements on coring and chloride testing of existing bridge decks.

33.5.2 Susceptibility Index

The first step in selecting a corrosion control system is to identify if local systems will suffice. If not, appropriate global systems must be identified. To determine the appropriateness of a local or global system, the distribution of chloride ions needs to be determined. NCHRP Report 558, "Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements," proposed a quantitative method for determining viable corrosion control alternatives that includes calculating a Susceptibility Index (SI) for the structure.

Chloride testing results are required to calculate the SI of the structure.

The distribution of chloride ions at the steel depth should be used to quantify both the susceptibility of the concrete element to corrosion in areas that are not currently damaged and the future susceptibility to corrosion-induced damage. If sufficient chloride ions are present to initiate corrosion, then corrosion-induced damage in the near future is expected, and only aggressive corrosion mitigation techniques, such as cathodic protection and electrochemical chloride extraction, can be used to control the corrosion process. However, if the chloride ion concentration distribution at the steel depth is low and future corrosion is not expected to initiate, less expensive corrosion control systems—such as sealers, membranes, and/or corrosion inhibitors—can be used to either control or stop the rate of corrosion. Therefore, an index that provides a good representation of the distribution of chloride ions at the steel depth is useful in selecting a corrosion control system.

The SI shall be calculated as follows:

$$SI = \left[\left(\sum_{1}^{n}(Cl_{th} - X_i)\right) \Big/ (n \times Cl_{th}) \times 10\right]$$

Where

Cl_{th} = Chloride concentration threshold

X_i = Chloride concentration at the *i*th location at the depth of reinforcing

n = number of locations where measurements were made

The chloride concentration threshold depends on many factors but may be assumed to be 1.2 lbs/CY of concrete (or 0.03 percent chloride by weight), for uncoated reinforcing, if no better information is available.

The SI is a scaled ratio of the average moment from the threshold normalized by the threshold. An SI of 10 means that no chloride ions exist at reinforcing depth for any test location. The SI is 0 if the chloride concentration at every location is equal to the threshold. A negative SI indicates that corrosion has initiated at most tested locations and that deterioration of the deck, even in currently sound areas, is expected.

33.5.3 Selection of Corrosion Control Alternatives

Once the SI of a structure has been calculated, corrosion control alternatives can be evaluated and selected. A lower SI, which corresponds to higher levels of chloride contamination, requires a more aggressive corrosion control system.

Most corrosion control systems, including those described in the following sections, are intended for use with uncoated (black) reinforcing. For concrete elements with epoxy-coated reinforcing, the Designer shall select a compatible corrosion control system. Any damage to the epoxy coating in the repair area should be repaired. NCHRP Web Document 50, “Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement,” provides guidance on the repair and rehabilitation of concrete with epoxy-coated reinforcing.

Selection of corrosion control systems must also consider the desired service life of the rehabilitated element to avoid unnecessary expenditures. For example, structures programed for replacement within the next 10 years may not be good candidates for a cathodic protection system that could be expected to last up to 25 years.

Figure 33-2 shows the optimal corrosion control systems for a given SI. See Section 33.5.3.1 through Section 33.5.3.7 for more information.

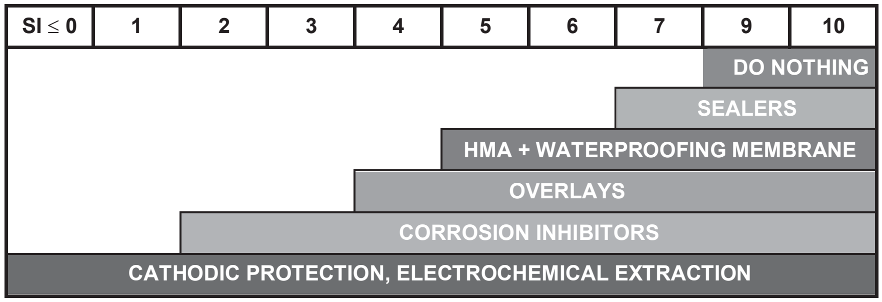

Figure 33-2: Optimal Corrosion Control Based on Susceptibility Index

The control systems shown in Figure 33-2 are intended to be used in conjunction with the removal and patching of spalled and delaminated concrete. Consideration should also be given to removing and patching all chloride contaminated concrete, in addition to spalled and delaminated concrete. There is a risk of corrosion initiating or continuing in the original concrete if contaminated concrete is left in place. For example, if a polyester concrete overlay is installed over sound but chloride contaminated concrete, corrosion may still occur, resulting in deterioration of the original concrete. This may compromise the newly placed overlay, resulting in a reduced effective service life and necessitating future repairs.

33.5.3.1 Do Nothing

SI values greater than or equal to 8.0 indicate that a corrosion control system is not necessary.

33.5.3.2 Sealers

For the purposes of Figure 33-2, a sealer is defined as any coating that is “breathable,” that is, capable of limiting the flow of moisture into the concrete but still allowing the flow of moisture out of the concrete. CDOT commonly uses an alkyl-alkoxy silane sealer. Sealers are an acceptable form of corrosion control for decks with SI values of 7.0 or greater.

33.5.3.3 Membranes

Membranes are differentiated from sealers in that they restrict the movement of moisture in either direction and do not allow chloride intrusion. The membrane category includes asphalt wearing surfaces over a waterproofing membrane and thin-bonded epoxy overlays. As shown in Figure 33-2, membrane type corrosion control systems can be used as the primary form of protection when the SI is 5.0 or greater. For decks with an SI less than 5.0, a membrane may be used in conjunction with more aggressive corrosion control systems.

33.5.3.4 Overlays

Overlays include both cementitious and non-cementitious wearing surfaces installed on the deck surface. Polyester concrete overlays fall into this category. Asphalt wearing surfaces are not considered overlays (in terms of corrosion protection) because they do not serve as barriers to moisture and chloride ions. Overlays limit corrosion by reducing the rate of chloride and water diffusion into the deck and by increasing the depth to which chlorides must diffuse to reach the reinforcing. The result is an increased time to initiation of corrosion. Overlays also serve as a wearing surface.

As shown in Figure 33-2, overlays can be considered the primary form of corrosion protection when the SI is 4.0 or greater or can be used in conjunction with more aggressive corrosion control systems for lower SI values.

33.5.3.5 Corrosion Inhibitors

Corrosion inhibitors include any material that chemically slows or stops the corrosion process. Inhibitor systems can be surface applied or admixed with repair concrete. Deck repairs on structures with an SI less than 4.0 should include corrosion inhibitors or a more aggressive corrosion control system. Corrosion inhibitors are not commonly used in Colorado.

33.5.3.6 Electrochemical Chloride Extraction

Electrochemical chloride extraction (ECE) is a short-term treatment of the bridge deck that lowers the chloride levels in the bridge deck to an acceptable level. Removing chloride ions increases the alkalinity at the surface of the reinforcing, which re-passivates the reinforcing and prevents future corrosion from initiating. ECE is not commonly used in Colorado.

33.5.3.7 Cathodic Protection

Cathodic protection systems include galvanic systems and impressed current systems and can be used in conjunction with other corrosion control systems.

Cathodic protection is a rehabilitation technique that has been proven to stop corrosion in chloride contaminated bridge decks (Sohanghpurwala, 2006). However, it is appropriate for use only on structures with SI values less than 2.0 and is most cost-effective for structures where a service life extension of greater than 15 years is desired.

One acceptable form of cathodic protection is the application of galvanic anodes in the patch area. The galvanic anodes corrode sacrificially themselves, reducing the corrosion in the reinforcing itself. The size and spacing of anodes should be selected to provide the desired service life of the repair. When no better information is available, CDOT has commonly specified 100 gram anodes at 18 in. to 24 in. spacing along the interface.

33.5.3.8 Complete Topping Replacement

Rehabilitation options that involve removing and replacing the top layer of concrete in its entirety may be more cost-effective than patching each damaged area individually. This type of repair can be performed using hydrodemolition or standard methods of concrete removal. When the depth of replacement is selected such that all chloride contaminated concrete is removed, this type of repair also serves as a method of corrosion control. The cause of the corrosion (chlorides) has been removed and, therefore, no other corrosion control system is necessary. However, because of the relatively high cost of this type of repair, it is discouraged for decks that require minimal patching or have an SI of 5.0 or greater.

33.6 CONCRETE REHABILITATION – EXCLUDING BRIDGE DECKS

Other concrete elements besides bridge decks can be exposed to chlorides throughout their service life. This includes abutments, piers, and walls within the splash zone, as well as elements exposed to chlorides due to leaking expansion joints.

Concrete repairs required on elements within the splash zone or due to damage caused by leaking expansion joints should include galvanic anodes at the patch interface to mitigate the halo effect and protect the surrounding concrete from accelerated corrosion. Depending on the element and its risk to continued exposure to chlorides, the addition of a membrane or sealer may also be appropriate.

33.7 DECK REPLACEMENT

Deck replacement projects can be a cost-effective means of extending the service life of a bridge when a deck has deteriorated beyond what can be reasonably repaired but the remainder of the structure is otherwise performing well and has no underlying deficiencies. They also present opportunities to strengthen the superstructure, upgrade bridge rail, and move or eliminate expansion joints. However, due to their cost, these projects should be considered carefully to ensure that completed structures do not result in the continuation of substandard conditions (such as insufficient clearances or roadway geometry) that would need to be addressed during the anticipated life of the new deck.

Deck replacement projects should implement the following improvements, where feasible:

- Make the new deck composite with the girders to increase capacity.
- Eliminate any existing longitudinal deck joints.
- Provide a deck with 8 in. minimum thickness.
- Eliminate expansion joints at abutments and/or throughout the structure. See Section 33.8 for more information on expansion joint removal.

If the weight of the proposed deck and attachments causes the load rating of the girders or substructure to fall below the minimum acceptable rating as defined in Section 33.2.2.4, the following measures may be considered to reduce dead load:

- Specify a lighter wearing surface (either a ¾ in. minimum polyester concrete or ⅜ in. thin-bonded epoxy overlay) in combination with waiving the minimum rating requirement for a future 3 in. overlay.
- Use a lighter bridge rail (e.g., use a Type 10 MASH instead of Type 9).
- Use a voided sidewalk.
- Reduce deck thickness, with the approval of Unit Leader in coordination with the State Bridge Engineer. CDOT allows a minimum deck thickness

of 7.0 in. Reducing the deck thickness should be considered only after all other strategies for reducing weight have been exhausted. Deck thickness should be reduced only by the minimum amount needed to meet the minimum rating requirement.

33.8 EXPANSION JOINT ELIMINATION

Preservation and rehabilitation projects present opportunities to either eliminate or relocate existing expansion joints. Removing existing expansion joints reduces future inspection and maintenance needs, eliminates the possibility of future joint failure, and can improve ride quality.

Expansion device elimination should be considered for all preservation and rehabilitation projects. Changes in the structural behavior of the structure must also be considered, which may result in necessary modifications to other elements.

33.8.1 Expansion Joints at Abutments

For expansion joints at abutments, moving the joint to the end of the approach slab should be considered. This solution may require modification or replacement of the approach slab to resist the imposed forces and movements.

Figure 33-3 depicts one option for moving an expansion joint at a seat-type abutment to the end of a new approach slab.

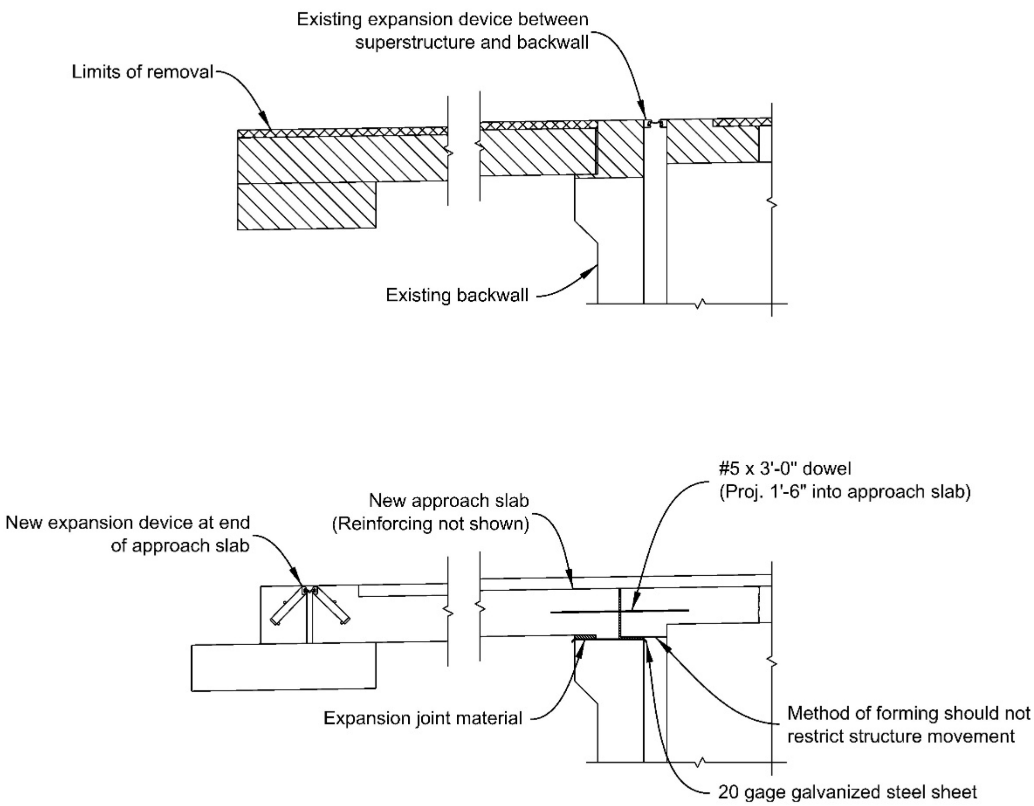

Figure 33-3: Expansion Joint Relocation

33.8.2 Expansion Joints at Piers

Eliminating an expansion joint at an interior pier requires that some degree of continuity be established, either complete continuity of the deck and girders or continuity of the deck only. Establishing continuity can alter the structural behavior of the bridge, including thermal movement demands from a new bridge center of stiffness location. External longitudinal force distribution may also be affected. As a result, the bridge may require modification or replacement of bearings to mitigate the behavior change. All structural consequences related to the elimination of expansion joints at piers must be carefully considered and resolved.

This type of joint replacement should be considered for existing multi-simple span bridges. In some cases, it will be possible to eliminate some but not all expansion joints. This is still considered an improvement over not eliminating any joints.

CDOT has accomplished this type of joint removal successfully in the past. Details for any proposed joint elimination shall be coordinated with Staff Bridge.

33.9 BEARING REPLACEMENT

Bearing replacements should address the root cause of the existing bearing deficiency. Fixing the root cause of an issue may not be possible given the cost of necessary modifications and funding constraints.

For neoprene pads, replacement bearings shall meet current design standards, or as close to current standards as practical, without requiring excessive modifications to bearing seats or other structural elements. Current seismic connection force requirements should be met, where practical.

The contract plans shall show:

- Jacking locations and design forces
- Any structural modifications required prior to jacking
- Phasing or traffic restrictions
- Extents of required removals
- Details for the new bearing devices
- Other special requirements

33.9.1 Structure Jacking Requirements

The Designer is responsible for determining suitable jacking locations for the structure. Structures are typically jacked from the diaphragms between girders at supports or from the girders directly in front of the bearing device either from the support seat or next to the support seat. See Section 14.5.6 of this BDM for typical jack clearance requirements.

The Designer shall verify that the structure can be jacked to the necessary height without overloading any structural components, including, but not limited to, girders, diaphragms, deck, and substructure.

To avoid overloading structure components, modifications may be required prior to jacking, such as adding bearing stiffeners to steel I-girders if jacking under the girder in front of the bearing device.

For situations where a jacking height of ¼ in. or less is required and all girders at a support will be jacked simultaneously, 1.3 times the permanent load reaction at the adjacent bearing may be assumed as the design jacking force. Otherwise, a refined jacking analysis is required to determine the design jacking force. The unfactored jacking force resulting from a refined analysis shall be increased by a minimum load factor of 1.3 to obtain the design jacking force. ***AASHTO 3.4.3.1***

Refined jacking analyses shall account for the stiffness contributions of the deck, diaphragms, and other structural elements, as appropriate.

Overload traffic shall not be permitted on the structure during jacking operations. Normal traffic shall not be permitted on the bridge during jacking operations unless:

- Overnight closures are not permitted, and
- Prior approval is obtained from Unit Leader in coordination with Fabrication/Construction Unit).

If traffic is permitted on the structure during jacking operations:

- Traffic should be shifted away from the jacking locations, where possible,
- Locking jacks should be used as a fail-safe in the event of jack failure, and
- The jacking load shall include factored and service dead and live load reactions, including impact, consistent with the permitted traffic positioning during jacking operations.

33.10 BRIDGE RAIL REPLACEMENT

Substandard bridge rail and guardrail transitions should be replaced when feasible with TL-4 MASH compliant rail and TL-3 transitions respectively. The approximate test level of the existing bridge rail should be provided to the Region for their safety and replacement considerations. Retrofit or rehabilitation options should be provided as well. If overhang strength is a concern, replacement with Type 9 bridge rail will spread out loads greater than Type 10 MASH bridge rail. A 6” bridge deck with #4s at 6” was tested to NCHRP 350 TL-3 levels with no damage.

See Sections 2.4.1 and 13 of this BDM for additional information.

33.11 FATIGUE

33.11.1 Load Induced Fatigue

For rehabilitation projects involving steel superstructures, all superstructure components shall be checked for the remaining fatigue life. When feasible, the remaining fatigue life shall be at least the desired service life of the type of rehabilitation being considered.

33.11.2 Distortion Induced Fatigue

Unlike load induced fatigue, distortion induced fatigue is not equilibrium based but instead arises from stiffness incompatibility and differential deflection of adjacent members.

Distortion induced fatigue cracking is prevalent in steel bridges built before 1985. Bridges built during this period commonly did not connect diaphragm connection plates to the girder flange out of perceived fatigue concerns. This practice results in a length of unbraced web from the girder flange to the termination of the connection plate, known as the web gap. When adjacent girders undergo differential deflection due to live load, forces are induced in the connecting diaphragms, producing distortion and potentially large stresses in the web gap. Because these stresses are cyclical, fatigue cracking can occur. Figure 33-4 and Figure 33-5 depict this behavior.

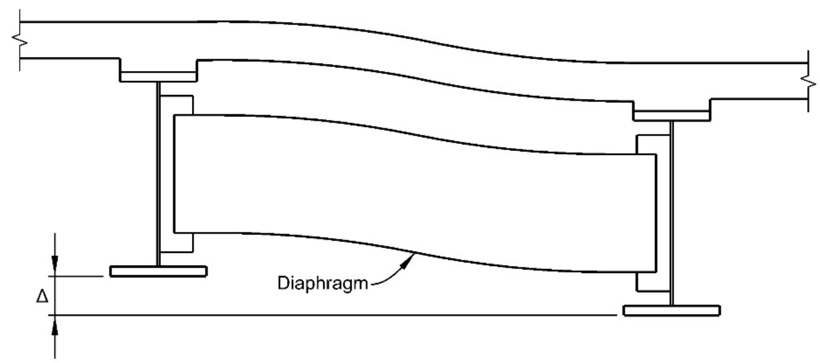

Figure 33-4: Differential Deflection

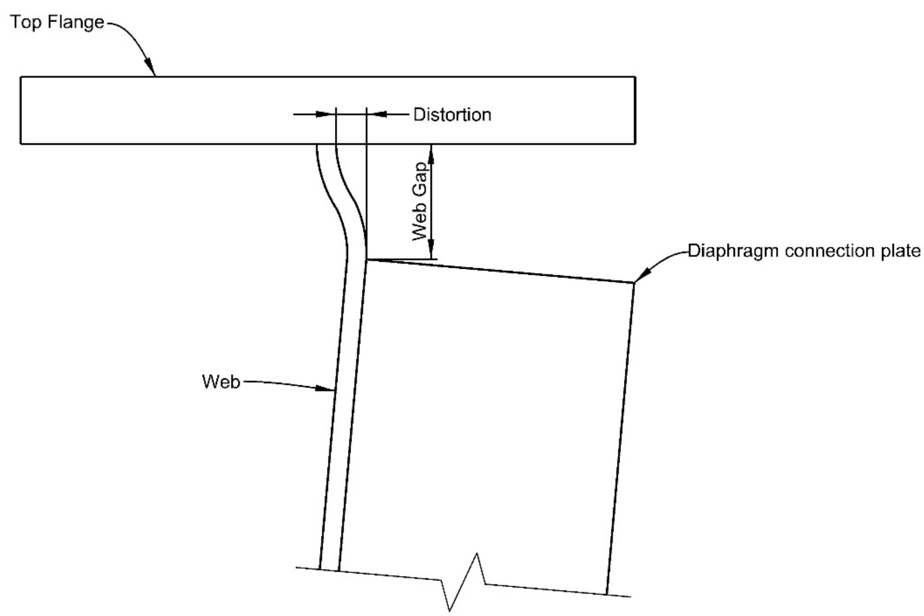

Figure 33-5: Web Gap Distortion

The magnitude of web gap distortion is proportional to the degree of differential deflection between the adjacent girders. For this reason, bridges with skewed supports and perpendicular diaphragms are particularly susceptible to distortion induced fatigue cracking.

The length of the web gap has a significant impact on the magnitude of fatigue stresses in the web gap. A longer web gap is more flexible and may be able to distort without resulting in large stresses, while a shorter web gap may be sufficiently rigid to reduce web gap distortion, which can also reduce fatigue stress magnitudes. Web gaps of approximately 2 to 4 in. in length generally produce the largest magnitude fatigue stresses.

Steel girder bridges built before 1985 and detailed with unstiffened web gaps are considered high risk for development of fatigue cracks. This includes bridges where girder connection plates attach to floor beams, diaphragms, or crossframes. Any preservation or rehabilitation project on a high-risk bridge shall determine if distortion induced fatigue cracking has occurred and develop a repair and retrofit plan to address any discovered deficiencies.

Superstructures that exhibit distortion induced fatigue cracking should be repaired and retrofitted according to the guidance in the FHWA *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges*. A stiffening type retrofit is preferred because it produces similar behavior to that resulting from current design and detail methodologies.

The complexities of distortion induced fatigue may require refined structural models if accurate out-of-plane stress ranges in the web gap region need to be determined.

33.12 CULVERTS

For roadway widening projects that require extending an existing box culvert, consideration should be given to replacing the existing culvert in lieu of extending it if the existing portion is in poor condition and/or would require extensive repair during the predicted service life of the extended portion.

33.13 SCOUR CRITICAL STRUCTURES

33.13.1 Evaluation of Existing Structures for Criticality

Refer to Section 33.2.1 for code requirements and minimum performance criteria when determining if a structure is scour critical.

33.13.2 Rehabilitation of Scour Critical Structures

Once a structure has been assessed as scour critical, the processes and procedures outlined in FHWA Hydraulic Engineering Circular (HEC) numbers 18, 20, and 23 shall be followed, including development of a Plan of Action.

Depending on project specifics, the ideal corrective actions may be structural, hydraulic, or biotechnical countermeasures, a monitoring program, or a combination thereof. Acceptable scour countermeasures are shown in Table 2.1 of HEC 23. Scour countermeasures that are not acceptable for new structures may be acceptable for existing structures.

33.13.3 Structural Countermeasure Requirements

Structural scour countermeasures shall be designed to meet all requirements of AASHTO LRFD, where practical. Example structural scour countermeasures include foundation and substructure strengthening and independent structures that reduce or eliminate scour of the bridge.

33.14 PAINTING OF STEEL STRUCTURES

The corrosion of structural steel bridge members is an ongoing concern that must be addressed to prolong service life. Not only does corrosion change bridge aesthetics, it can seriously jeopardize the structural integrity of the entire structure. Painting is an efficient and economical method to provide corrosion protection to existing steel bridge members.

Maintenance painting is important for all bridges but is of particular concern for bridges more than 100 ft. long. For smaller bridges (less than 100 ft.), the proportionally higher cost of environmental controls for cleaning may outweigh the benefits of painting. Packaging multiple bridges into one contract for structures less than 100 ft. may be appropriate. For larger bridges (longer than 500 ft.) or complex bridges, paint preservation should be prioritized due to the high replacement cost of the bridge.

Bridge painting is weather sensitive. The air temperature must be warm and the humidity must be low. Therefore, work/letting needs to be scheduled when there is low probability of unsuitable weather conditions. Typically, May through September is the ideal time to accomplish bridge painting. If a painting project occurs outside this range, a controlled environment is required.

When possible, painting projects should be coordinated with roadway projects. The necessary time for a Professional Engineer to design and analyze a containment system should be included in the project schedule between the notice to proceed and the physical start of work. Also, consider the necessary time required for the industrial hygienist/certified professional to develop/review the lead safety plan and other submittals.

When repainting existing bridges over high ADT roadways where roadway restrictions must be minimized, use of a rapid deployment strategy should be considered. Rapid deployment is a viable option primarily designed for use on highway overpasses where the structural steel is easily accessible from the roadway below using a mobile work platform. This mobile work unit includes a containment device, dust collector, and blast equipment. Rapid deployment methodologies may be specified using Special Provisions. For field painting activities, use a two-coat system with an organic primer.

33.14.1 Zinc Rich Paint Systems

For a properly shop-installed zinc rich paint system, Table 33-1 identifies typical painting activities and frequencies to establish painting guidelines to maintain and preserve the life of steel bridges. Widespread use of these zinc rich paint systems began in the 1980s. Environmental factors (e.g., under a leaking deck joint, within “splash zone”) will have a detrimental effect on the life of the paint system, which will require an increased frequency of painting activities.

Leaking deck joints and other bridge deficiencies that may affect paint system performance should be corrected before completing any new painting activities.

Consideration must also be given to bridges that are on a program to be improved, rehabilitated, or replaced. Bridges on a program must be evaluated to determine if a painting activity is still warranted. The high cost of containment and mobilization require that a cost/feasibility estimate be completed to determine the most economic work scope for any given structure. For example, use of spot/zone painting vs. a full re-paint for any given structure or entire component replacement must be evaluated. This work scope should include aesthetic considerations for the visible portions of the bridge, such as fascia beams.

While a study of preliminary costs will likely conclude that an overcoat system is the most economical alternative, a life-cycle cost analysis will often show full paint removal and application of a high durability coating system to be more cost-effective than an overcoat option, particularly for bridges exposed to significant deicing salt application.

Table 33-1: Maintenance Painting Frequencies

Painting Activity	Frequency
Spot/Zone Painting	10–18 years
Full Re-paint	30–40 years

33.15 BRIDGE PREVENTATIVE MAINTENANCE

33.15.1 Program Objectives

Bridge Preventative Maintenance (BPM) seeks to extend the service life of structures through targeted improvements. Structures in good condition are the top priority of BPM funds because these bridges are near the top of their deterioration curve, and, therefore, see the greatest extension in service life per dollar spent. BPM projects typically cost less than 30 percent of the cost of a new bridge.

See Section 33.2 for code requirements and minimum performance criteria when design is required for a BPM project.

The primary BPM goals are to:

- Seal bare concrete decks.
- Add a waterproofing membrane to bridge decks that currently have an asphalt overlay but no membrane.
- Replace membranes on bridges where the existing membrane is nearing the end of its service life (approximately 30 years) or otherwise shows signs of deterioration.
- Replace leaking or otherwise non-functioning expansion joints.

- Replace functioning expansion joints at the end of their predicted service life, when convenient.

Examples of BPM actions include but are not limited to:

- Bridge rinsing
- Sealing deck joints
- Facilitating drainage
- Sealing concrete
- Painting steel
- Removing channel debris
- Protecting against scour
- Lubricating bearings

BPM projects also present an opportunity to perform other miscellaneous repair activities, such as bridge rail and substructure repair. The Designer should coordinate with CDOT to determine what additional activities to include in the project.

33.15.2 Bridge Preventative Maintenance Resources

33.15.2.1 Staff Bridge Worksheets for BPM

As of this writing, CDOT is in the process of developing standard worksheets for BPM work, including:

- General Information
- Summary of Quantities
- Deck Repair Details – HMA Overlay
- Deck Repair Details – Polyester Concrete Overlay
- Bridge Expansion Device (0–4 Inch) at Approach Slabs
- Taper Details for Polyester Overlay at Beginning/End of Structure and Bridge Drains

These worksheets can be obtained from Staff Bridge upon request.

33.15.2.2 Expansion Joint Replacement

The preferred type of replacement expansion device depends on the type of joint that is being replaced. A 0 to 4 in. joint is the preferred replacement joint type, when feasible. Table 33-2, the BPM joint replacement matrix, shows preferred and acceptable replacement types based on existing joint type.

Expansion joint elimination should be considered for all bridges requiring joint replacements. See Section 33.8 for more information.

Table 33-2: BPM Joint Replacement Matrix

BPM Joint Replacement Matrix												
		Existing Joint Types										
Replacement Joint Type	Replacement Item Number	Modular Expansion Device	New Style Strip Seal/Bridge Expansion Device (0-4 Inch)	Old Style Strip Seal	Bridge Expansion Joint (Asphaltic Plug)	Compression Joint Sealer	Pre- compressed Foam Joint	Sliding Plate/Finger Joint	Premolded Rubber Transflex Joint	Asphalt Over Non-expansion Joint	Pourable Joint Seal at Abutment or Pier	Roadway Pressure Relief Joint
Bridge Expansion Device (0-___ Inch)	518-010XX	X										
Bridge Expansion Device (0-4 Inch)[1]	518-01004	X[6]	X	X	X	X	X	X	X		X	
Bridge Expansion Device (Gland) (0-4 Inches)[2]	518-01060		X	X								
Bridge Expansion Joint (Asphaltic Plug)[3]	518-01001				X	X	X			X		
Bridge Compression Joint Sealer	518-00000				X	X	X					
Joint Sealant[4]	408-01100										X	
Sawing and Sealing Bridge Joint	518-03000									X	X	
Roadway Compression Joint Sealer[5]	518-00010											X
None										X		

These are general recommendations, final determination of replacement joint type shall be discussed with Staff Bridge unit leader.

X	= Preferred joint type
X	= Acceptable joint type

[1]This is CDOT's default joint. It has the longest service life and should be considered strongly for any location where there is potential leaking onto pier caps or abutment seats.

[2]The gland manufacturer must be the same as the manufacturer of the rails.

[3]To be used for rotational movement only. Translational movement of joint should be limited to ½". Proper seating of the bridging plate is critical to ensure it doesn't rock.

[4]To be used for rotational movement only. Translational movement of joint should be limited to ½".

[5]Parallel saw-cuts are critical on both sides of joint for proper placement.

[6]Some modular joints can be replaced with 0-4 Inch joints with an oversized gland.

33.15.2.3 Overlay and Wearing Surface Guidance

See Section 33.1.3 for bridge deck chloride testing requirements for projects that include installation of a new overlay. Chloride testing results may impact the selection of the wearing surface type or necessitate deck corrosion mitigation measures before installing the new wearing surface. See Section 33.5 for more information.

The following types of deck protection systems are permissible for use on preservation and rehabilitation projects:

- 3 in. HMA/SMA wearing surface over a waterproofing membrane
- ¾ in. polyester concrete overlay
- ⅜ in. thin-bonded epoxy overlay

BPM projects with an asphalt approach roadway can be combined with roadway surface treatment projects to realize a substantially lower unit cost for asphalt. For this reason, the preferred deck protection system for these bridges is a waterproofing membrane with a 3 in. asphalt wearing surface.

A ¾ in. polyester concrete overlay should be considered for BPM projects where the approach roadway is concrete or where other factors prevent reasonable inclusion in a surface treatment project. However, the additional height of the overlay requires that a taper detail be implemented to avoid modifying the existing expansion devices and end dams.

A ⅜ in thin-bonded epoxy overlay is not a preferred option for long-term structure use (≥ 10 years) due to a high life cycle cost. However, if modification of expansion devices and end dams cannot be avoided or if it is cost prohibitive to do so, a ⅜ in. thin-bonded epoxy overlay should be considered. Thin-bonded overlays are placed directly on the existing bridge deck without requiring modification of expansion joints and end dams.

When changing asphalt thickness, the maximum permanent grade changes shall be in accordance with the CDOT Roadway Design Manual.

33.15.3 Bridge Preventative Maintenance Project Delivery

33.15.3.1 Standalone Projects

Standalone BPM projects are maintenance projects that are independent of any other project work and specifically scoped for preventative maintenance work. Examples of these projects are Expansion Joint projects, Polymer Concrete Overlay projects and critical culvert projects. They are generally run by a regional RE or project engineer.

33.15.3.2 Overlay Projects

Bridge Maintenance work is often added on to CDOT overlay projects to take advantage of project mobilization and lane closures. For any overlay project, a letter shall be provided to the RE outlining all structures within the overlay limits and any restrictions for milling work and limitations on overlay thicknesses. Overhead limitations such as sign structures or bridges over the highway are also good to list with their clearance limitations. Vertical clearances should be verified after any overlay projects. The letter shall also include a listing of any essential repairs on the structures and all funded preventative maintenance work. Lastly, the letter shall include a listing of the current bridge rail or guardrails associated with the structures along with their assumed MASH compliance levels and recommendations for replacement or rehabilitation. Bridge funding is typically not available for rail replacement except for essential repair findings. This letter shall be delivered prior to FIR and preferably at the Scoping meeting.

33.15.3.3 Maintenance Projects

A third type of project delivery for Bridge Maintenance work would be maintenance projects going to advertisement similar to standalone projects or work done directly by maintenance forces. Plan and specification requirements would be similar to other project delivery methods.

33.15.3.4 Process Flow Charts

The process flow charts for rehabilitation and preservation projects are very similar to new bridge projects but are usually simplified due to the removal of some of the reports and survey requirements. Widening projects will be almost identical to new structures. Structure selection reports are still required for widenings but will be shorter. Rehabilitation work is simplified since survey, hydraulics and other specialty information is generally not required. Overlay recommendation/funded work letters are not required for standalone or M-projects.

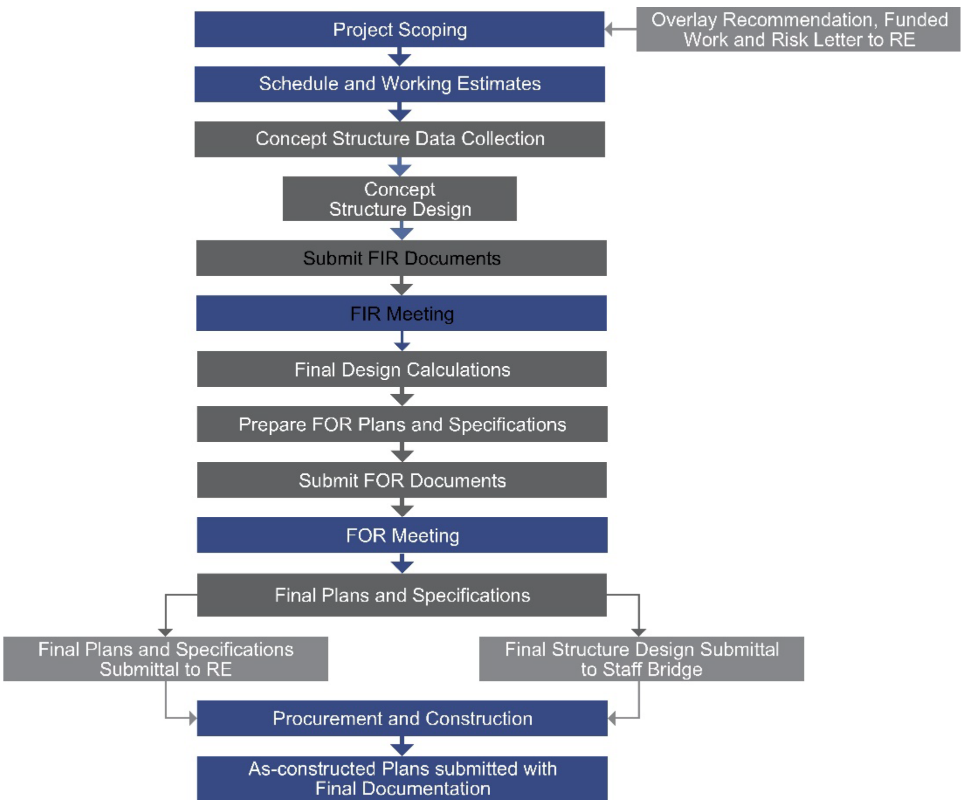

Figure 33-6: Structure Process Diagram (Overlay)

33.16 REFERENCES

The following references may be considered for further guidance:

ACI 222R-01: Protection of Metals in Concrete Against Corrosion.

ACI 440.2R-08: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.

Dexter, R.J. and J.M. Ocel. 2013. Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges, Report No. FHWA-IF-13-020. March.

Federal Highway Administration (FHWA). 2018. Bridge Preservation Guide. FHWA Publication Number: FHWA-HIF-18-022.

FHWA. 2009. Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance-Third Edition. Hydraulic Engineering Circular No. 23. FHWA Publication No. FHWA-NHI-09-111. September.

FHWA. 2001. Long-Term Effectiveness of Cathodic Protection Systems on Highway Structures. Publication No. FHWA-RD-01-096.

Hawk, H. 2003. Bridge Life-Cycle Cost Analysis. Washington, DC: Transportation Research Board. National Research Council, NCHRP Report 483.

Sohanghpurwala, A.A. 2006. Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements. Washington, DC: Transportation Research Board. NCHRP Report 558. doi:10.17226/13934

Sohanghpurwala, A.A., W.T. Scannell, and W.H. Hartt. 2002. Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement. NCHRP Web Document 50.