Oregon Department of Transportation

SECTION 1.15
Repair and Strengthening

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

1.15.1 Strengthening of Bridges

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1.15 Repair and Strengthening

The terms **strengthening** and **repair** are sometimes used interchangeably to describe an action, but they are not the same. Strengthening is the addition of load capacity beyond the level provided for in the original design. Repair is the restoration of the load capacity to the level of the original design.

Whether repair or strengthening is called for, perform at least one site visit to the structure and identify/verify defects that may exist, to tailor solutions to problems found.

1.15.1 Strengthening of Bridges

Bridge strengthening is performed to increase capacity, such as, when a load rating factor for a bridge falls below 1.0. Design bridge strengthening to resist the live load given in *BDM 1.3.3.4*.

When material testing is performed, take concrete compressive cores from each concrete grade as indicated on the as-constructed plans, with at least three samples from each grade.

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See *ODOT LRFR* for additional information on load rating. For every bridge, we strive to have all traditional legal vehicles (Type 3, Type 3S2 and Type 3-3), as well as specialized hauling vehicles (SHVs) and Emergency Vehicles (EVs), with critical rating factors greater than 1.0. When any of them are less than 1.0, load posting signs are required to be installed to address the capacity issue until the bridge is shored, strengthened or replaced.

When load rating factors are below 1.0 or when the Bridge Inspection Report indicates quality issues, consider conducting material testing according to *The Manual for Bridge Evaluation Sections 5.3 and 5.4*. Additional sampling locations or tests may be required for large bridges or to address localized problems.

Material sampling of steel structures is rarely performed because of the well documented material properties, consistent material strengths and difficult to repair sampling locations. In some cases Brinell Hardness Testing may be used to increase confidence in correlating tensile properties to a known grade of steel in lieu of taking tensile test samples.

Material sampling of timber structures is discussed in the *BMDM*.

Repair all concrete sampling locations with hand patching materials from *SP 02015.20 or SP 02015.30* of the *QPL* designated for vertical and overhead application.

When a load rating factor is less than 1.0 for legal loads, a design deviation is required to support a Bridge Modernization or Retrofit category project with reduced design criteria that will not address all the rating factors less than 1.0. Consider a do nothing, or a reduced design criteria, approach only in rare cases where there are extenuating circumstances.

Legal loads are defined in *ODOT LRFR 1.5.1.2.*

Factors to be considered in the design deviation approval process may include:

- Estimated cost of repair or strengthening.
- Existing permit truck volume and potential for future increases.
- Existing girder cracking.
- Number of lanes and shoulder widths.
- Alternate routes available.
- Existing bridge detailing.

When strengthening is performed, address all longitudinal tension reinforcement deficiencies by adding reinforcement or by refined analysis.

It is not required to post bridges with longitudinal tension reinforcement deficiencies if they are in good condition without extensive cracking in the area of the deficiency. See *ODOT LRFR* for discussion on longitudinal tension.

1.15.1.1 Strengthening Methods and Details

Use of strengthening methods, not listed in the following sections, require approved design deviation (used to document use of innovative materials).

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BDM 1.15.1.1.1 thru *BDM 1.15.1.1.5* are preferred methods for strengthening girders in flexure and shear. Alternative methods are encouraged.

1.15.1.1.1 Section Enlargement

For bridge elements that require section enlargement with concrete cover less than 4 inch thick and are situated in a marine area per *BDM 1.2.4.1*, use nylon drywall anchors

C1.15.1.1.1

or stainless steel screw anchors to attain composite section.

Adding depth and width to beams can increase flexural and shear capacity.

Design interface shear reinforcement details to ensure composite section behavior between new and old concrete.

Steel dowels can be used for achieving the composite section by drilling holes into the existing concrete substrate, installing reinforcing bars or bolts, and anchoring the steel dowels with epoxy resin.

Consider high strength rebar for flexural and shear reinforcement in new concrete section, when space is limited. The high strength rebar reduces size and quantity of new rebar. See *BDM 1.5.5.15* for additional information on high strength rebar.

1.15.1.1.2 Post-Tensioning

C1.15.1.1.2

Ensure that force due to post-tensioning is not excessive to avoid unintended cracking.

Post-tensioning is an active means to restore or increase flexure, shear, and anchorage capacity of bridge structural elements.

Account for any long-term relaxation unless provision for future tightening is included.

When longitudinal post-tensioning is used as part of a strengthening system, understand that long-term relaxation of the post-tensioning system may reduce the effectiveness of the strengthening. When strengthened structural elements are located in a marine area per *BDM 1.2.4.1*, consider using corrosion-resistant high-strength bars, strands, and anchorage assemblies. Use of corrosion-resistant ducts filled with grout to encase P/T strands is also an option. See *BDM 1.5.8* for additional information.

1.15.1.1.3 Internal Shear Anchors

C1.15.1.1.3

Limit internal shear anchor (ISA) size to ¾ inch. Do not use larger sizes unless the availability of multiple contractors has been verified.

Specialty contractors are generally available for drilling 1-inch diameter holes up to 60-inches in depth. Internal shear anchors for shear strengthening can be installed either from above or below the girder. Installation from above may be easier and considered where practical.

Install internal shear anchors at a maximum 30 degree angle perpendicular to concrete surface.

Calculate the required embedment depth to develop anchor rod ultimate strength according to *ACI 318 17.4.5* with modification according to *BDM 1.5.11.2.2*. Provide adequate bar length greater than the calculated embedment length at both ends.

Calculate the shear capacity from internal shear anchors in normal weight concrete as follows:

$$V_{s_ISA} = \frac{A_s f_y d_{v_ISA}(cot\theta + cot \propto)sin \propto}{s_{ISA}}$$

where:

Effective shear depth, $d_{v_ISA} = L_{eff} sin\alpha$

Effective length, $L_{eff} = L_{anchor} - 2L_{d_ISA}$

Development length, $L_{d_ISA} = \frac{A_s f_y}{\tau_{uncr} \pi d_a}$

A_s = area of internal shear anchor (in.^{2})

f_y = minimum specified yield strength of ISA (ksi)

d_a = diameter of ISA (in^2)

τ_{uncr} = characteristic bond stress (ksi) (see *Table 1.5.11.2.2-3*)

L_{anchor} = total length of ISA (in.)

α = angle of inclination of ISA to longitudinal axis (degrees)

θ = angle of inclination of diagonal compressive stresses (degrees)

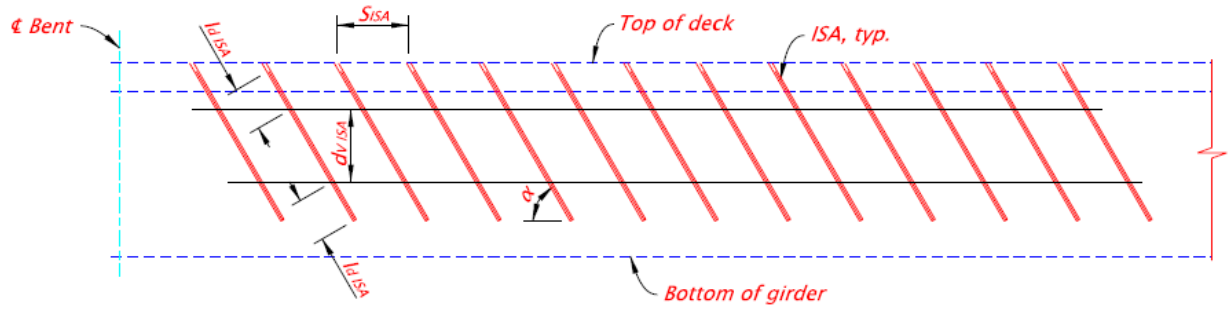
s_{ISA} = spacing of ISA (inch), minimum spacing equal to or greater than 6 x bar diameter or 6 inches.

Parameters in the above equations are shown in *Figure 1.15.1.1.3-1*.

This angle provides 96 percent of the capacity compared to 45 degree anchors and is much easier to install.

Internal shear anchors require development length at each end of the rod. The effective length of an internal anchor is the length remaining after subtracting the development length at each end of the bar.

Figure 1.15.1.1.3-1 Internal Shear Anchors



1.15.1.1.4 Bonded Strengthening Systems

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SURFACE MOUNTED SYSTEM

C SURFACE MOUNTED SYSTEM

Do not use a precured laminate for a strengthening system.

Design CFRP wet-layup strengthening according to *ACI 440.2R-08* or *AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*.

Do not use CFRP wet-layup unless the critical load rating factor is greater than 0.80.

Strengthening with CFRP wet-layup can be considered a long-term (more than 20 years) strengthening solution.

Use intermittent strips with 4 inch minimum gap on girder webs for shear strengthening.

This is to allow for inspection of the bare concrete between the strips.

Do not specify more than three plies of CFRP wet-layup.

This is to avoid crack-induced debonding failure mode.

Provide positive anchorage at the ends of CFRP wet-layup. Anchorage using CFRP wet-layup transverse to the loaded direction is not acceptable.

Proper surface preparation is critical to ensure a successful CFRP application.

NEAR SURFACE MOUNTED SYSTEM

Near surface mounted (NSM) system can be used for flexure, shear, and anchorage strengthening of bridge structural elements.

Electrical isolation details are required when using CFRP.

Design NSM-CFRP system according to *ACI 440.2R-17* or *AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements, First Edition*.

C NEAR SURFACE MOUNTED SYSTEM

NSM system includes cutting shallow grooves into existing concrete substrate and embedding reinforcement surrounded by epoxy resin inside the groove. NSM is less prone to delamination, environmental degradation and allows for effective inspection of concrete surfaces compared to the CFRP wet-layup system. Consider the following reinforcement materials for near surface mounted application:

- CFRP bar.
- High strength bar.
- Stainless steel bar.
- Titanium alloy bar.

Consider using NSM system for the following bridge elements:

- Cap beams.
- Girders.
- Deck overhang.
- Bridge deck.

Footing.

CFRP is a conductive material, that can develop galvanic corrosion, when installed in concrete elements reinforced with steel rebar.

Bond strength between CFRP bars, epoxy resin, and concrete is required to develop the tensile capacity. CFRP has linear properties up to rupture. On the other hand, metal reinforcement has a yield point with large strain before fracture and can be bent for mechanical anchorage into concrete substrate. Mechanical anchorage at terminations is added for metal reinforcement by using standard 90 degree hooks at both ends to provide anchorage.

NSM system using stainless steel and titanium alloy bars was investigated through tests for strength and long-term performance

at OSU. See *ODOT SPR 750* and *SPR 775* research reports for test information. The research studies found that the materials are suitable for strengthening bridge structural elements. However, NSM system with titanium alloy bars (NSM-TiABs) is more efficient than the system with stainless steel rebar due to higher strength.

See *AASHTO Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures* for design guidance and design examples with the following modifications:

ASTM B1009 describes material standards for NSM system with titanium alloy bars.

Consider smaller diameter bars for strengthening thin structural elements, since required tail length for end hooks is shorter. For instance, use #3 bars for deck overhang strengthening, where a tail length 5-inches.

- Reduce nominal cross-sectional area using a factor of 0.96 due to surface deformations to enhance bond.
- Use a specified yield strength of 130 ksi for design.
- Specify a maximum system length of 19 feet (excluding the hooks) for #5 bars or smaller and a maximum system length of 18 feet for #6 bars.

  The stock length for titanium bars is 20 feet total including the lengths of hook tails. Lengths up to 35 feet are possible, but not recommended. The additional length results in a much higher unit cost.

- Limit the use of #6 titanium alloy bars for members lightly reinforced concrete beams.

  This requirement is due to the required 12 inch hook tails embedded in the beam that can conflict with existing steel reinforcing.

Use NSM-TiABs for localized strengthening such as poor anchorage, specific deficient areas, etc.

Even though titanium alloy bars (TiAB) have the strength advantage over CFRP, stainless steel, and other high strength bars, the unit cost of titanium alloy bars is much higher than the others.

Concrete substrate for NSM system needs to be in good condition without significant cracking, spalling and delamination. For structural elements with extensive concrete damage, section enlargement with high strength rebar would be more appropriate.

When concrete cover repair is required, additional steps for strengthening would negate advantage of NSM construction.

1.15.1.1.5 Other Strengthening Systems

External stirrups (vertical rods) are not considered adequate for permanent strengthening.

Do not use bonded or bolted steel plates attached to the sides of concrete girders for shear strengthening.

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External stirrups can be used for temporary shear strengthening of concrete girders.

1.15.1.1.6 Strengthening Plan Details

The EOR of a bridge design project is responsible for strengthening design, except for CFRP wet-layup and CFRP-NSM systems. For CFRP strengthening systems, provide the following information on plan sheets for contractor's CFRP strengthening designers:

- Existing reinforcing details including material properties, concrete section, and relevant reinforcing details for capacity calculation.
- Locations and limits of deficiencies in structural element.
- Required total capacity from existing section and strengthening system.
- Conceptual strengthening details
- Required minimum material properties of CFRP.
- Required locations of CFRP reinforcement termination for full development.
- A reduction factor of 0.85 (ψ_f) required in the strength contribution of CFRP reinforcement.

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Include the following data of CFRP and composite materials used for the design of the CFRP strengthening system in General Notes sheet:

- Section properties.
- Ultimate and design tensile strength.
- Tensile modulus of elasticity.
- Ultimate strain.

When the CFRP strengthening system used in construction is different from design plans, document CFRP and composite material properties listed above in as-constructed drawings.

The material data are necessary for future evaluation and load rating of the bridge.

1.15.2 Repair of Bridges

All the strengthening methods in *BDM 1.15.1.1* can be used for repair as well.

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Bridge repair projects are typically limited to isolated portions of the bridge and address specific needs such as substructure issues and collision damage.

1.15.2.1 Concrete Repair

Identify known locations or bridge elements on the plan sheets that require concrete repair due to corroded reinforcing, concrete spalling, alligator cracking, surface rock pockets or construction joint rock pockets.

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Whether a structure is a few years old or a historic structure that is several decades old, the intent of a bridge rehabilitation or bridge preservation is not to rebuild bridge members but to maintain the original as built geometry and capacity. In some situations, maintaining the original aesthetic should be considered. There are situations where superstructure members will require strengthening based on deficient load rating which is covered under *BDM 1.15.1*.

Repair locations should be identified on a large scale, specifying broad areas rather than each specific location. Refer to *SP 00542* which specifies further investigation during construction. Refer to *BR 500* for details.

Alligator cracking will eventually spall and allow moisture to migrate to the reinforcing.

1.15.2.1.1 Existing Concrete Removal

Refer to *SP 00542* for existing concrete removal. The portion of *BDM 1.15.2.1.1* in single column format is commentary.

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The contractor will locate damaged concrete areas having visible spalling or delamination due to corrosion of reinforcement or by sounding with a 16-ounce masonry hammer for visibly loose, dislodged, or loosened concrete. The contract will verify locations of steel reinforcing with a rebar locator to determine location of steel reinforcing.

One of the steps for damaged concrete repair is sounding the concrete surface for delaminated concrete due to rebar corrosion. Concrete structures with large aggregate size can give a similar sounding to delaminated concrete. Avoid excessive removal to deep depths if there is no reinforcing in the removal work area. Some structures, primarily historic structures have had aggregate sizes as large as 6 inches. Locations can be verified with a rebar locator to determine if steel reinforcing is present.

The contractor will mark the extent of boundary repair area for saw-cutting perimeter to ½ inch depth. Saw-cutting perimeter provides a right-angle square edge to apply mortar. The contractor will saw cut boundary area in square or rectangular shape where applicable, avoid stepped type boundary shape.

Saw-cutting along the perimeter of repair area prevents the repair edge from having feathered surfaces that will have the potential to crack. Stepped shapes require more work for pumping grout and have the potential for more air voids in patch area.

The contractor will remove the damaged concrete within the marked boundaries with nominal 15-pound class pneumatic hammer, high pressure water jet blasting, chipping guns, manual picks, and chisels. Limit the deepest depth of removal to ½ inch past steel reinforcement that has apparent corrosion pitting or scaling.

The contractor should not remove concrete below rebar when there is no apparent corrosion to avoid debonding reinforcement resulting from hitting rebar with chipping hammer during concrete removal. The contractor should avoid chasing debonded rebar past designated repair boundary.

Contractor should limit removal depth of deep rock pockets to 2-inches. While the actual depth of the rock pockets may extend further into the member, do not repair beyond the initial 2-inches.

Some construction joints may have the rock pocket located around the entire section. The entire section should not be removed.

Refer to *BR 500* for details.

For rock pockets at construction joint, crack injection should be used for joints narrower than ¼ inch or shallower than 1½ inches. If crack injection cannot be used, remove and repair the area with 2-inch deep and 2-inch wide on each side of joint centerline and around perimeter of column.

The encased repair eliminates moisture infiltration to the column steel reinforcing. Excessive removal at construction joint locations due to soft material can be reduced by using a smaller weight chipping hammer or even a rock hammer. Excessive heat can occur during curing of crack injection material for joints wider than ¼ inch.

1.15.2.1.2 Surface Preparation

Refer to *SP 00542* for concrete surface preparation. The remainder of *BDM 1.15.2.1.2* is commentary.

C1.15.2.1.2

After removal of damaged concrete, the contractor will prepare the surface for repair patches and additional cover by abrasive-blast or waterblast. The contractor will prepare concrete surfaces according to *SP 00542*.

The contractor will provide removable watertight formwork for pumped repair mortar (pressure grouted) application per *SP 00542*.

Substrate concrete should be saturated for 24 hours by either filling forms full of water or packing a saturated foam or burlap material into the form. Forms should remain in place for three days after placement of repair mortar.

The 24-hour saturation (presoaking) ensures that the existing concrete surface will have a saturated surface dry condition during placement of repair mortar. Some structures, generally historic structures, tend to have a higher porosity concrete. Dry substrate concrete will draw moisture out of the repair mortar and increase the chances for cracking. Saturation duration is not temperature dependent.
Leaving forms in place for three days helps prevent cracking, regardless of strength levels being reached prior to that duration.

1.15.2.1.3 Anode Application

For zinc anode installation refer to *SP 00542*.

Embedded galvanic zinc anodes are used in reinforced concrete rehabilitation to prevent the corrosion of steel reinforcement from recurring at a current site or at a new site outside the removal boundary area (also referred to as the halo effect). Use anodes in the patch area to provide protection to the bars extending outside repair boundary in the existing contaminated concrete.

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See *BR 520* for details. Anodes are constructed with a sacrificial zinc core encased within a cementitious mortar shell. Incompressible material within the encasement shell allows for expansion of the zinc core. Anodes are fastened to the exposed rebar with tie wires, which causes a galvanic reaction to occur and preserve the reinforcement. Anodes are available in different sizes e.g.:

- 2.5 inch diameter x 1.1 inch height
- 1¼ inch width x 1 5/16 inch height x 4 inch length
- 1¼ inch width x 2 ¼ inch height x 4 ¼ inch length.

Do not specify galvanic anodes in repair materials with electrical resistivity greater than 15,000 ohm-cm.

Greater resistivity repair materials may be used by multiplying the spacing value with a spacing correction factor provided by anode manufacturer.

Many repair materials with polymer, fly ash, and silica fume-based cannot be used in conjunction with sacrificial anodes. Portland cement concrete is a suitable material for anodes due to its high alkalinity with PH above 10.

DESIGN FOR ANODE

Do not place anodes at every repair location. Place anodes at locations with potential for reoccurring corrosion. Bars with no signs of corrosion do not require anodes. Position anodes around outer perimeter of repair areas and at a grid pattern.

C DESIGN FOR ANODE

Possible locations for anode placement are as follows:

- Reinforcing that has signs of pitting or scaled corrosion and has resulted in a loss of rebar section
- RCDG joint/beam expansion joint location were the expansion joints fails and moisture will reactivate the corrosion process
- Concrete submitted to high chlorides due to environmental conditions

Bridge location will influence the areas that have reoccurring corrosion, whether it is in a marine environment or exposure to deicing materials, etc. Locations with exposure to chlorides tend to corrode faster.

Determine required anode spacing by calculating the steel density ratio (SDR). SDR is defined as the total steel reinforcing surface area (ft^2) within square foot of concrete repair (regardless of depth).

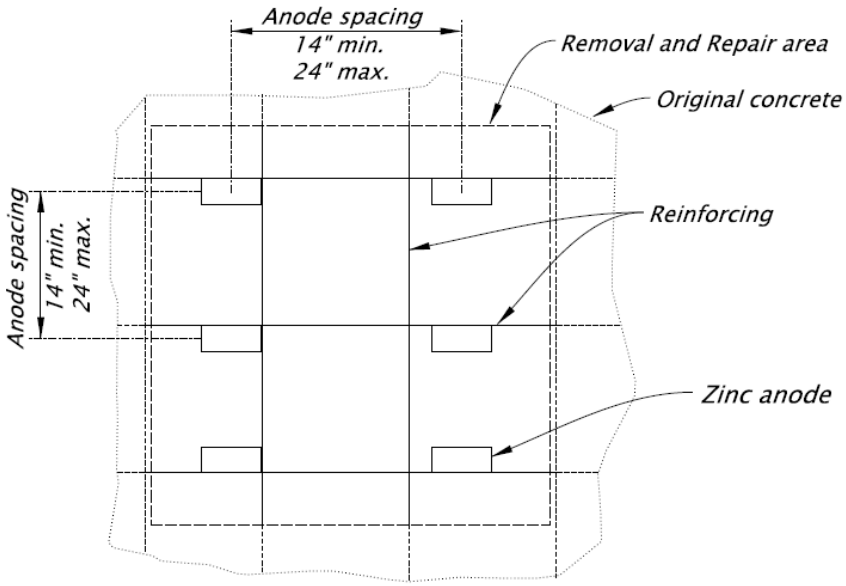
Use the steel density tables shown in *Tables 1.15.2.1.3-1* thru *1.15.2.1.3-3* to determine number of anode and spacing. The tables are based on product data from anode manufacturers listed in *SP 00542*.

When no calculations are performed, anode coverage for reinforcing without actual steel density ratio calculation can be placed in a 24-inch grid, on center. Maintain 14 inches minimum spacing and 24 inches maximum spacing (see *Figure 1.15.2.1.3-1*).

The following design examples can be used for anode puck design.

Anode puck placement calculations can be used for estimating purposes. Design is done by the manufacturer and is based largely on finding of field investigation. Approved zinc anodes are on a preapproved list in *SP 00542*.

Figure 1.15.2.1.3-1 Anode Puck General Placement



Example 1: Beam repair of stirrup

Assumptions:

18″ x 33″ Beam has #4 stirrup bars @ 9 inch on center.
Stirrup legs on one side of beam has corrosion.
Repair area is 20″ x 48″ (see *Figure 1.15.2.1.3-2*).
Corroded bars are showing signs of pitting or scaling.

Solution:

NOTE: This example assumes continuity between all the stirrups from either a horizontal bar at top or bottom of beam. If no continuity exists between each stirrup, it will require either providing continuity by wiring bars together or wiring one anode between adjacent stirrups.

Calculate steel Density Ratio per foot of area = $\pi D_{\text{bar diameter}}$/bar spacing = π x 0.5 inch / 9 inch o.c. = 0.17 required SDR.

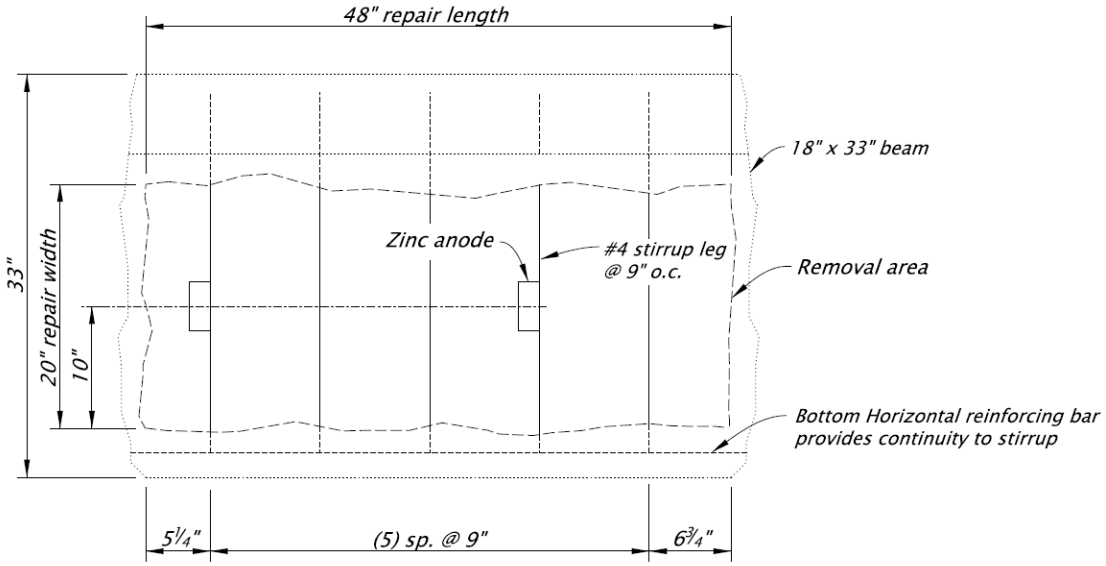
Choose a table that best describes the corrosion condition of reinforcing from *Table 1.15.2.1.3-1, 1.15.2.1.3-2, or 1.15.2.1.3-3.*

Determine anode spacing using *Table 1.15.2.1.3-2*, corroded bar, moderate corrosion.

SDR < 0.30 so maximum spacing = 28 inches.

Determine number of anodes based on maximum anode spacing: Horizontal spacing = (48 inch - 5 ¼ inch-6 ¾ inch length / 28 inch spacing) = 1.28 spaces (2 anodes) x Vertical spacing = (20 inch width / 28 inch) = 0.71 (1 anode). Requires 2 x 1 = 2 anodes.

Figure 1.15.2.1.3-2 Anode Puck Placement for Beam Repair with Stirrups (one -directional reinforcing) Elevation View



Example 2: Deck steel repair

Assumptions:

- Deck has #5 bars @ 8 inch and #5 @ 10 inch.
- Repair area is 2′-0″ x 4′-0″ (see *Figure 1.15.2.1.3-3*).
- High corrosion environment.

Solution:

Calculate steel Density Ratio / foot of area = $\Sigma(\pi D_{\text{bar diameter}}/\text{bar spacing}) = \pi \times 0.625$ inch / 8 inch o.c. + π x 0. 625 inch / 10 inch o.c. = **0.44** required SDR.

Determine anode spacing using *Table 1.15.2.1.3-3*, corroded bar, high corrosion.

SDR 0.31 – 0.60 so maximum spacing = 13 ".

Determine number of anodes based on maximum anode spacing: Horizontal spaces = (48 inch-3 ¼ inch – 4 ¾ inch length / 13 inch spacing) = 3.07 (4 anodes) x Vertical spaces = (24 inch width / 13 inch) = 1.85 (3 anodes). **Requires 4 x 3 = 12 anodes**.

Figure 1.15.2.1.3-3 Anode Puck Placement for Deck Steel Repair (two-directional reinforcing) Plan View

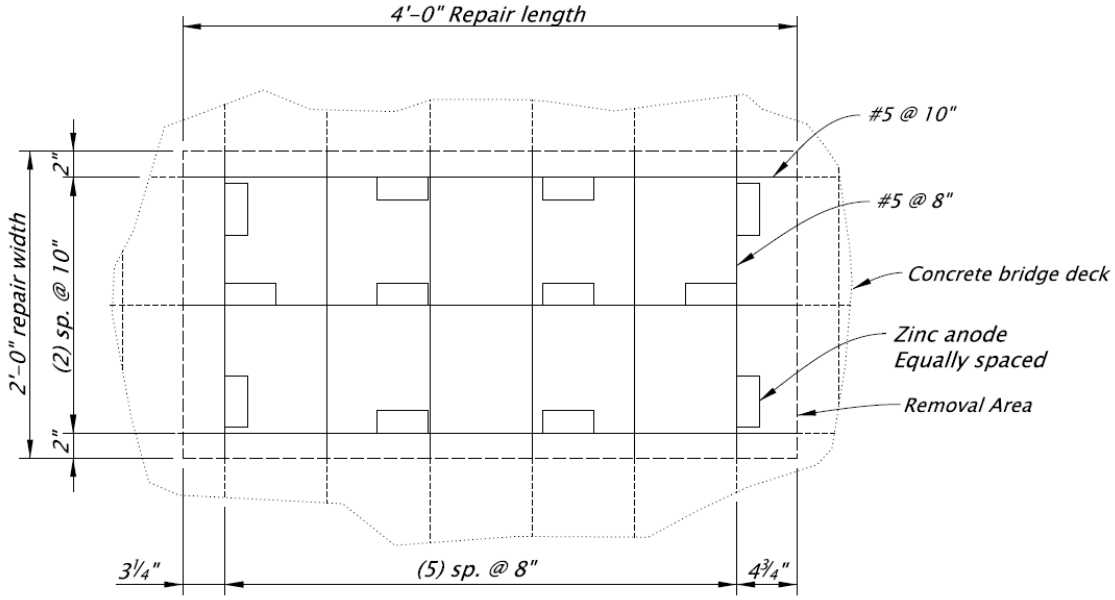


Table 1.15.2.1.3-1 Steel Density Table – Non-Corroded Bars, Low Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	30
0.31 – 0.60	28
0.61 - 0.90	26
0.91 – 1.20	22
1.20 – 1.50	20
1.51 – 2.00	17

Table 1.15.2.1.3-2 Steel Density Table – Corroded Bars, Moderate Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	28
0.31 – 0.60	19
0.61 - 0.90	15
0.91 – 1.20	13
1.20 – 1.50	11
1.51 – 1.80	10
1.81 – 2.10	9

Table 1.15.2.1.3-3 Steel Density Table – Corroded Bars, High Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	19
0.31 – 0.60	13
0.61 - 0.90	10
0.91 – 1.20	9
1.20 – 1.50	7
1.51 – 1.80	6
1.81 – 2.10	5

1.15.2.2 Expansion Joint Repair

1.15.2.2.1 Design Considerations

Evaluate conditions of existing joints before beginning design work. Conduct a minimum of one site visit and consult the District Bridge Maintenance personnel or Region Bridge Inspectors.

1.15.2.2.2 Joint Surfaces

Limit repair work when joint surfaces are in good condition.

For deteriorated joint surfaces beyond minor repair, specify one of the following materials for joint surface replacement:

- Structural polymer concrete with a minimum compressive strength of 10 ksi, a minimum tensile strength of 1 ksi, and a minimum modulus of elasticity of 2,500 ksi
- High Early Strength Concrete (HESC) with synthetic fibers
- Ultra High Performance Concrete (UHPC)

When the joint surfaces need a re-construction to adjust joint opening, consider locating removal boundaries away from the joint opening where the splice length for new and existing reinforcing bars are adequate. Specify elastomeric nosing over the new joint surfaces when HESC is used.

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C1.15.2.2.1

It is crucial to know existing joint conditions and details before joint repair design. Often times, details shown in the as-constructed plans do not reflect actual conditions of the joint.

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Minor spalls can be patched. Removing and replacing existing concrete will require work including concrete removal, existing reinforcing steel retention, new steel reinforcing placement and new concrete placement.

Structural polymer concrete and HESC have a shorter cure time. When the materials are properly mixed, traffic could return after 4 hours from a placement. Even though UHPC has a longer cure time compared to structural polymer concrete and HESC, the material has superior durability along with high compressive, tensile, and bond strengths.

Do not specify elastomeric nosing material to narrow the joint opening without structural reinforced concrete support.

Elastomeric nosing material is considered non-structural.

1.15.2.2.3 Poured Joint and Compression Joint Seals

Ensure that size of the new replacement joint seal is suitable for the current joint opening and movement range. Replace poured joint and compression joint seals with either poured joint or pre-compressed foam silicone joint (PFSJ) seal. When the joint opening is too large or too small, but the movement range is still within the seal type capacity, rebuild the joint opening to fit available joint seal sizes.

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Compression joint seal installation requires a full-width roadway closure. Poured joint and PFSJ seals can be installed in stages and have similar service life compared to compression joint seals.

1.15.2.2.4 Strip Seals

Strip seals can accommodate large joint movement and have a longer service life than other joint types. Consider repairing strip seals instead of replacing or modifying, when possible.

For older strip seals and when replacement gland is not available, use either of the following options depending on the steel edgebeam condition:

- Significantly deteriorated condition: replace the entire system with a new strip seal.
- Acceptable condition: replace existing gland with PFSJ seal.

Specify a gland replacement full width of the roadway when the following conditions are met:

- Steel edgebeams are in acceptable condition.

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- Replacement glands are available.
- The joint opening and movement range are acceptable.

PFSJ seals may be specified for replacing deteriorated strip seal gland, even if steel edgebeams are still in acceptable condition.

Request approval from Bridge Maintenance District before beginning the design work.

Possible justifications for replacing PFSJ seals include a full closure is not possible, risk reduction and construction time reduction.

Some Regions have had more success replacing strip seal glands than other Regions. Gland replacement work is difficult to accomplish and can take a long time. PFSJ seals can be installed in stages, which make the replacement easier without a full closure compared to conventional full-width gland replacement work. PFSJ can accommodate large movement and large joint opening.

For steel edgebeams with minor damage, steel edgebeams can be repaired by cutting and removing the damaged piece and splicing a new section onto the existing section by welding. Ensure that the gland is removed before the welding takes place. Specify a full-width gland replacement.

It is typical to find steel edgebeam sections damaged in wheel rut lines. See *BDM 1.9.3.1.5* for information on rut repair.

1.15.2.2.5 Asphaltic Plug Joint

C1.15.2.2.5

Specify the same joint type for Asphaltic Plug Joint (APJ) seal replacement. Include details and notes to ensure even and smooth joint surfaces on both sides of the joint opening. The condition is crucial for APJ seal to perform.

APJ seal requires significant notching into the bridge deck and approach slab to install the joint seal in the first place. When a different expansion joint seal is specified, significant surface preparation is required.

1.15.2.2.6 Precompressed Foam Silicone Joint Seal

C1.15.2.2.6

Ensure that size of the replacement seal is suitable for the current joint opening and movement range. When the joint opening is too large or too small, but the movement range is still within the seal type capacity, rebuild the joint opening to fit available joint seal sizes.

1.15.2.3 Keyway Repair

(Reserved for future use)

C1.15.2.3

1.15.2.4 Epoxy Injection

Epoxy injection is not considered a strengthening method for either flexure or shear, however, it improves corrosion protection by preventing moisture intrusion.

When large size active cracks are not desirable, address the structural issue that causes the cracking using a strengthening method.

Epoxy inject cracks with widths larger than 0.016 inches, but smaller than 0.25 inches where the bridge is:

- Located in a Snow and Ice area, defined in *BDM 1.2.4.3.*
- Located in a Coastal Area, defined in *BDM 1.2.4.2.*
- Showing signs of corrosion.

Use surface seal for cracks smaller than 0.016 inches. Specify grout for cracks larger than 0.25 inches.

C1.15.2.4

Injection of cracks smaller than 0.016 inches is difficult and is only marginally effective. Most epoxy manufacturers limit the size of cracks to be smaller than 0.25 inches due to excessive heat generated during the cure process after epoxy installation. Depending on the level of stresses in concrete and steel reinforcing and deformation of structural members, new cracks could occur next to epoxy injected cracks under subsequent loading.

1.15.3 Existing Rebar and Concrete Cover Investigation

Obtain as-constructed drawings and evaluate existing bar size, location, spacing, and cover thickness during design. Ensure that strengthening system is possible to be constructed, especially the strengthening methods that require drilling or cutting into existing concrete section.

C1.15.3

Give clear instructions to the contractor concerning how potential conflicts are to be either avoided or resolved.

It is often difficult to avoid existing deck steel or existing flexural steel. Possible solutions include but are not limited to:

- Locate existing bars and measure concrete cover using high precision rebar detector or Ground Penetrating Radar (GPR) before drilling holes or groove cutting.
- Expose the top mat of reinforcement before drilling.
- Relocate drilled hole or groove to an equivalent location, when a conflict is discovered.
- Add reinforcement, when cutting existing bar cannot be avoided.

1.15.3.1 FRP Composites

(Reserved for future use)

1.15.4 Reserved

1.15.5 Inspection and Maintenance Accessibility

Provide safe access to structures for bridge inspectors and maintenance personnel. Consider the structure type and associated inspection needs to determine appropriate access elements.

Provide access facilities that meet the *Oregon Occupational Safety and Health Code Chapter 437* rules *Division 2, General occupational safety and health Subdivision D, Walking-Working Surfaces*.

When a location on a bridge structure is intended for public access, provide appropriate pedestrian or combination railing according to *BDM 1.13.3* and *1.2.10.1*.

C1.15.5

FHWA mandates that bridges be inspected every 24 months. Inspectors require access to within 3 feet of all bridge components for visual inspection. Maintenance personnel need to access members for repair.

See *BDM 1.14.1.1* for additional information on bearing replacement access.

See *BDM 1.15.5.2* for additional information on access.

1.15.5.1 Fall Protection

ODOT policy is to use—the safest access method for any given situation. Use the hierarchy described below for evaluating and placing access and fall protection systems at a site.

1. First evaluate the feasibility of eliminating the fall hazard.

C1.15.5.1

A fall protection system can consist of a fall prevention system or a fall arrest system.

Access and fall protection should be one of many factors considered when deciding a bridge layout, but should not dictate structure arrangement (e.g., span length). Although it provides advantages to those engaged in maintenance activities it is often not feasible to eliminate, through design, situations where workers will be near the kind of drops or steep slopes described in *BDM 1.15.5.1.1*.

2. Where eliminating the fall hazard is not practical, then evaluate the feasibility of fall prevention systems.

Fall prevention systems include, but are not limited to, physical barriers, rails, walkways and covers.

Where installing fall prevention systems increases the likelihood of transient activity, consider ground access instead. In urban environments, a combination of hazard elimination and ground access for bearing inspection will frequently be the preferred alternative.

If a fall prevention system can be installed (e.g. railing or walkway) vs using mobile access equipment, the fall prevention system is the preferred method.

Temporary fall protection systems can be approved on a site-specific basis.

Anchorage points may be considered as a cost-effective alternative to fall prevention systems at accessible locations for steep slopes described in *BDM 1.15.5.1.1* (e.g. partial viaducts). Ideally this would eliminate the need to install a fall prevention system such as railing, fencing, etc..

3. Where a fall prevention system is not practical, provide a personal fall arrest system (limits falls to an acceptable force and distance).

4. Evaluate the feasibility of ground access where height from the ground to the bearings is 15 feet or less, and inspection and maintenance can be performed with a ladder.

When inspection and maintenance can be performed via ground access, evaluate the possibility of eliminating a fall protection system based on site conditions. See *BDM 1.15.5.2* for ladder access requirements.

Evaluate site conditions for when fall prevention systems on abutments is not justified (e.g. aesthetics, public access, urban proximity, frequency of access). At these sites, ladder access is a good alternative. Wingwalls do not need a fall prevention

when inspection and maintenance access is not required.

1.15.5.1.1 Fall Protection Warrants

Install fall protection under the following conditions where maintenance and inspection activities will occur during the service life of the structure:

- Along a vertical face or steep slope, defined as one upon which normal footing cannot be maintained without the use of devices due to the pitch of the surface, weather conditions, or surface material.

- When the potential vertical drop at the face of wall exceeds 6 feet, and access to the top of the feature is intended solely for the use of maintenance and operations.

C1.15.5.1.1

Consider slopes 3H:1V and steeper to be steep unless more detailed information can be documented.

1.15.5.1.2 Fall Protection Treatment Selection

Acceptable fall prevention systems include pipe rail fence (*BR192 & BR193*), chain link fencing, and roadside barrier.

Acceptable fall arrest systems include horizontal fall arrest lifelines (*BR190* & *BR191*) and anchorage systems.

C1.15.5.1.2

Fall prevention systems and fall arrest systems serve very different functions. The design loadings and inspection requirements between fall prevention and fall arrest systems are significantly different. Consider the following site factors when selecting a fall protection system:

- Location of clear zone.
- Intended access (e.g., pedestrian, maintenance only).
- Frequency of access.
- Structure type, inspection needs.

Pedestrian and roadside barrier solutions that are not listed but meet the height and design criteria requirements of *BDM 1.15.5.1.3* may be adequate for fall protection.

See *RD series drawings* for metal handrail details. They may require modifications to be suitable to use for fall prevention.

Seek concurrence from the appropriate Region bridge contact concerning the specific fall protection system proposed. Document decisions related to fall protection use and treatment type in the DCSA.

Each Region has a different process and reporting structure for bridge maintenance activities. The goal is to contact the manager in charge of performing Region bridge maintenance activities for input and concurrence.

Provide fall protection that is aesthetically appropriate for the site.

In many cases, this may involve extending the system along the full length of the wall even though portions of the wall may have less than 6 feet of vertical drop.

1.15.5.1.3 Design Criteria for Fall Protection Systems

C1.15.5.1.3

Design fall prevention systems that meets *OSHA 1926.5029(b)* using the following criteria:

- 42 inches high, measured from the top of the working level.
- Capable of resisting 200 pound force applied at the top, from any direction.

For bridge rail height requirements related to inspection and maintenance, see *BDM 1.13.6* and *BDM 1.13.7*.

This loading is based on *OSHA 1926.502(b)(4)*.

Design fall arrest systems using the following criteria:

- Design criteria in *BR190* and *BR191*
- Or an engineered system that complies with *OSHA 1910.28* and *ANSI Z359.1* standards.
- Use galvanized or stainless steel wire rope.
- Use galvanized or stainless steel cable connections and turnbuckles having a minimum ultimate strength at least as great as the cable strength.

Consider using stainless or other corrosive resistant materials (e.g., phillystran) for wire rope and connections in marine and coastal environments. Consider the environmental conditions, service life, existing structure materials, possible theft and interaction of dissimilar metals when choosing materials.

1.15.5.2 Access

Work platforms are not required on all structures. Provide work platforms only when it is appropriate for the structure type and site layout. Consider the inspection needs to determine appropriate access elements. Contact the State Bridge Operations Engineer about structure inspection needs.

Fixed ladders more than 24 feet in height require a personal fall arrest system.

Appropriate places to consider work platforms include (but are not limited to) the following:

- Wide and high bridges where the reach of the arm of an under bridge inspection truck (UBIT) is not long enough for proper inspection and maintenance of the bridge members.
- Critical areas that require close inspection such as nonredundant steel tension members, hinges, splices, hangers, expansion joints, complex bearings, utility lines, navigation lights, and areas that require frequent maintenance.
- Vertical abutments and MSE abutments needing bearing and shear lug inspection.
- Vertical abutments and MSE abutments with integral abutments needing backwall inspection.
- Between nonredundant steel tension members.

C1.15.5.2

Work platforms are considered a fall protection system and are preferred over permanent anchors and fall arrest systems, see *BDM 1.15.5*.

Work platforms is an elevated working or walking surface. An access walkway refers to access provided at the ground level.

Consider work platforms combined with other facilities such as ladders, manholes and safety cables.

Currently ODOT has access to a UBIT with an arm reach of 62 feet. See *BDM 1.5.7.6* for additional information on concrete box girder access.

FHWA has recommended maintenance walkways between all steel girders. This is a costly item and should be reviewed on a case-by-case basis. Maintenance walkways were provided on the Santiam River

Bridge (Steel Alternate) Bridge 08123D, Drawing 47448. The detailed W5x15 walkway beams are not readily available. A W8x18 alternate is recommended, as this was substituted on the John Day River Bridge, Bridge 00108D.

The following conditions do not require work platforms:

- Elastomeric bearings.
- When inspection access is from the ground.
- When access to bearing is 6 feet or less from the ground.
- When a UBIT will be used and can access all critical areas.

In certain instances, consider a work platform over UBIT access, such as in urban areas where mobility constraints make UBIT access difficult or impractical.

1.15.5.2.1 Design Criteria for Elevated Work Access

C1.15.5.2.1

Provide work platforms that meet all minimum clearances requirements under the structure and that do not infringe or reduce minimum required waterway openings.

Provide work platforms with sufficient headroom and width for inspection personnel to carry bulky equipment between walk rails without difficulty.

Provide work platforms that meet the following width and headroom requirements:

See *BR115 Slope Paving*.

- 3 foot minimum width.
- 5 foot minimum width when the height from the ground to the bearings is 15 feet or less and ground access is expected.

Width is the available space for an inspector or maintenance worker to walk as needed for inspection and maintenance activities. The ground measurement should be taken from where a ladder will be set, generally in front of the bearing.

- When bearing access is not required, there is no minimum headroom requirement.
- 3 foot minimum headroom for girder bridges, when bearing access is required.
- 5 foot minimum headroom for bridges having a solid bottom, such as a concrete box girder, when bearing access is required.

4-foot minimum headroom otherwise.

Headroom is the minimum height from the walkway surface to the bottom of superstructure.

Provide access to the work platform using one of the following, and ensure appropriate security measures are installed:

- 3 foot wide walkway along the top of the wingwall. Provide fall protection when the vertical drop exceeds 6 feet.
- Cast-in-place steel U-bar ladder steps from the ground level (under the bridge) up to the walkway. In urban environments, place the first U-bar ladder step approximately 12 feet from the ground. Access to this first step will be by portable ladder. Ensure there is an adequate bench for the ladder to seat.

See ladder bench requirements below.

When a fall arrest system is used in conjunction with a work platform, provide the following:

- Attach the cable to either the backwall or cap (approximately 4 feet above the top of walkway) or to the bottom of the girders.
- Limit the cable attachment to precast prestressed girders to the center 4 inches of the bottom flange.
- Locate the cable at least 2 feet horizontal distance away from the

vertical drop.
- Design the fall arrest system using the criteria given in *BDM 1.15.5.1.3*.
- Where potential maintenance activity is anticipated locate the cable to avoid interference with potential bearing replacement and girder jacking operations.

When a handrail is used in conjunction with a work platform, design the railing to be removable in sections to facilitate maintenance work.

For bridges in urban environments, eliminate space that can be inhabited while providing ladder access for inspection. When that is not feasible, use deterrent fencing and provide locked gates at each entrance to the walkway. Design fencing to be removable in sections to facilitate maintenance work. See *BDM 1.2.8, Bridge Security Design Considerations.*

Discuss security elements with local maintenance staff to meet site specific needs.

Discuss bridge access needs with the Region Bridge Inspector.

When the height from the ground to the bearings is 17.5 feet or less, and ground access is expected ensure the following:

- There is a 5 foot minimum bench at the top of slope to support a portable ladder. Where such a bench is not practical, provide cast-in-place steel U-bar ladder steps.
- Provide enough ladder steps so that an inspector can get within 3 feet of any bearing.
- Ladder requirements of *OSHA 1926.1053* are met.