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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.
40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical
structures, pin and hanger systems, and pinned connections are inspected on a five-year cycle now.
40.3 Bridge Replacements

Bridge rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. In order to obtain federal funding eligibility for rehabilitation or replacement; the bridge must be Structurally Deficient or Functionally Obsolete. The Federal Sufficiency Number is a guide for federal participation which is required to be less than 50 for replacement. Also, Wisconsin DOT requires the Rate Score to be less than 75. Bridges are not eligible for replacement unless the Substructure or Superstructure Condition is 4 or less or the Inventory Rating is less than HS10 or the Alignment Appraisal is 4 or less.

A bridge becomes Structurally Deficient when the condition of the deck, superstructure or substructure is rated 4 or less; or when the inventory load capacity is less than 10 tons (89.0 kN); or when the waterway adequacy is rated a 2.

A bridge becomes Functionally Obsolete when the bridge roadway width, vertical clearance, or approach alignment is substandard (appraisal rating of 3 or less), or when the inventory load capacity is less than 15 tons; or when the waterway adequacy is rated a 3 or less.

See FDM 11-40-1, 1.5 for policies regarding necessary bridge width and structural capacity to determine eligibility for bridge rehabilitation* versus bridge replacement.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.
40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of insuring some level of acceptable serviceability. The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are sufficient to safely carry present and projected traffic. Information necessary to determine structure sufficiency includes structure inspection, inventory, traffic, maintenance, capacity and functional adequacy. The methods of reconstruction are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2’ shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required after completion of the rehabilitation work. However, if conditions exist that would prevent the completed improvement from correcting all deficiencies, WisDOT shall determine if the proposed project is eligible based on safety and the public interest. Contact the Bureau of Structures Development Section for a waiver of the sufficiency number requirement.

**WisDOT policy item:**

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/Mu reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic.
Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
   a. Deck condition equal 4 or 5 and;
   b. Wear course or wear surface less than or equal to 3.
   c. No roadway work scheduled for at least 3 years.

2. Interstate Bridge with Roadway Work
   a. No previous work in last 10 years or;
   b. Deck Condition less than or equal 4.
   c. Wear course or wear surface less than or equal to 4.

3. Rehab not needed on Interstate Bridges if:
   a. Deck rehab work less than 10 years old.
   b. Deck condition greater than 4.
   c. Wear surface or wear course greater than or equal 4.
4. All Bridges

**WisDOT policy item:**

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

b. Place overlays on all concrete superstructure bridges if eligible.

c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.
40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1-1/2” minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1-1/2” concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1” of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by the Bureau of Structures Design Section. Bridges reconstructed with overlays shall have their new Inventory and Operating Ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:
The structure is capable of carrying the overlay deadload;

- The deck and superstructure are structurally sound;

- The desired service life can be achieved with the considered overlay and existing structure;

- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1-1/2”.

- The grade change due to overlay thickness can be accommodated at minimal cost.

- Deck or bridge replacement is programmed within 7 years.

- Raising of floor drains or joints is not required.

- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

Polymer Overlay: 10 to 15 years life expectancy

- A 1/4-inch thick, two layer system comprised of a two-component polymer in conjunction with natural or synthetic aggregates. Use 5 psf for dead load, DW.

- Works well to seal decks and/or provide traction.

The minimum required concrete age is 28 days prior to application, although a longer period of time would allow more initial concrete cracking to occur which the resin would then be able to seal.
AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

Low Slump Concrete Overlay *(LSCO): 15 to 20 years life expectancy

- Minimum thickness is 1-1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.
- Combined distress area is less than 25%.
- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by the Bureau of Structures Development Section and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.

40.5.4 Special Considerations

- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.

- If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

Overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development Section to discuss solutions.
40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

<table>
<thead>
<tr>
<th>Item</th>
<th>Existing Condition</th>
<th>Condition after Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Condition</td>
<td>≤ 4</td>
<td>≥ 8</td>
</tr>
<tr>
<td>Inventory Rating</td>
<td>---</td>
<td>≥ HS15*</td>
</tr>
<tr>
<td>Superstructure Condition</td>
<td>≥ 3</td>
<td>Remove deficiencies (≥ 8 desired)</td>
</tr>
<tr>
<td>Substructure Condition</td>
<td>≥ 3</td>
<td>Remove deficiencies (≥ 8 desired)</td>
</tr>
<tr>
<td>Horizontal and Vertical Alignment Condition</td>
<td>&gt; 3</td>
<td>---</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>6 ft</td>
<td>6 ft</td>
</tr>
</tbody>
</table>

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.
WisDOT policy item:
Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* and *FDM SDD 14b7* for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don’t add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.
40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45”, 54” and 70” girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections’ draped and undraped strand patterns.

The 45”, 54”, and 70” girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at 0.75$f_{pu}$,
- A concrete haunch of 2-1/2”,
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 0.5” or 0.6” dia. strands (in accordance with the Standard Details),
- $f'_{c}$ girder = 8,000 psi,
- $f'_{c}$ slab = 4,000 psi, and
- Required $f'_{c}$ girder at initial prestress < 6,800 psi
<table>
<thead>
<tr>
<th>Girder Spacing</th>
<th>45&quot; Girder</th>
<th>54&quot; Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single Span</td>
<td>2 Equal Spans</td>
</tr>
<tr>
<td>6'-0&quot;</td>
<td>102</td>
<td>112</td>
</tr>
<tr>
<td>6'-6&quot;</td>
<td>100</td>
<td>110</td>
</tr>
<tr>
<td>7'-0&quot;</td>
<td>98</td>
<td>108</td>
</tr>
<tr>
<td>7'-6&quot;</td>
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<tr>
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<td>78</td>
<td>85</td>
</tr>
<tr>
<td>11'-6&quot;</td>
<td>76</td>
<td>84</td>
</tr>
<tr>
<td>12'-0&quot;</td>
<td>70</td>
<td>80</td>
</tr>
</tbody>
</table>

**Table 40.7-1**
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the
pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.
40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, consideration shall be given to replacing the entire deck in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the lifecycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. The girders used for widenings may be the latest Chapter 19-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet LRFD [3.6.5] (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development Section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don’t add intermediate lines of diaphragms).
40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3’ or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading). Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

Approval is required from BOS for all superstructure replacement projects. In order for a superstructure replacement to be allowed, the substructure must meet the criteria outlined below. This justifies the cost of a new superstructure by ensuring a uniform level of reliability for the entire structure.

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed \( \leq 40 \text{ mph} \). If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of “vehicle protection” is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be designed to current LRFD criteria.
40.10 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.
40.11 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.
40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.
40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.
Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6’ or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.
40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

or
2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.
The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.
40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Reuse of steel pile sections will require checking the remaining allowable load carrying capacity. Steel piling should be checked immediately below the splash zone or water line for deterioration and possible loss of section. High section loss has occurred in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line. Bearing capacities of existing footings and pilings may have to be recomputed in order to determine if superstructure loading can be safely carried.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy.

40.15.1 Hammerhead Pier Rehabilitation

Pier caps and sometimes shafts of these piers have become spalled due to leaky joints in the deck. The spalling may be completely around some of the longitudinal bar steel destroying the bond. However, experience shows that the concrete usually remains sound under the bearing plates. There is no known reason for this except that maybe the compressive forces may prevent salt intrusion or counteract freeze thaw cycles.

If the longitudinal bars are full length, the bond in the ends insures integrity even though spalling may occur over the shaft. Corrective action is required as follows:

1. Place a watertight expansion joint in the deck.
2. Consider whether bearing replacement is required.
3. Analyze the type of cap repair required.
   a. Clean off spalled concrete and place new concrete.
   b. Analyze capacity of bars still bonded to see if unbonded bars are needed. Use ultimate strength analysis.
   c. Consider repair method for serious loss of bar steel capacity.
      i. Add 6" of cover to cap. Add additional bar steel. Grout in U shaped stirrups around bars using standard anchor techniques.
ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

iii. Pour wing extension under pier caps beginning at base to take all loads in compression. This would alter pier shape.

d. Consider sloping top of pier to get better drainage.

e. Consider placing coating on pier top to resist water intrusion.

If the shaft is severely spalled, it will require a layer of concrete to be placed around it. Otherwise, patching is adequate. To add a layer of concrete:

1. Place anchor bars in existing solid concrete. Use expansion anchors or epoxy anchors as available.

2. Place wire mesh around shaft.

3. Place forms and pour concrete. 6” is minimum thickness.

40.15.2 Bearings

All steel bridge bearings should be replaced as shown in Chapter 27-Bearings. Compressive load and adhesion tests will be waived for steel laminated elastomeric bearings where these bearings are detailed to meet height requirements.

In general replace lubricated bronze bearings with Teflon bearings. If only outside bearings are replaced, the difference in friction factors can be ignored. Where lubricated bronze bearings might be used, following is the design criteria.

For the expansion bearings, two additional plates are employed over fixed bearings, a top plate and a lubricated bronze plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a lubricated bronze plate allowing expansion and contraction to occur. Laboratory testing of lubricated bronze plates indicated a maximum coefficient of friction varying from 8 to 14 percent for a loading of 200 kips. Current BOS practice for steel girder Type “A” and prestressed girder expansion bearings of employing a 10 percent maximum and a 6 percent minimum friction value for design is in accordance to laboratory test results. For Type “A” bearing details refer to Standard Details.
40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

Usage Restrictions:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.
• Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column).

• Adhesive anchors installed in the overhead or upwardly inclined position and/or under sustained tension loads shall not be used.

• The department has placed a moratorium on mechanical anchors. Usage is subject to prior-approval by the Bureau of Structures.

40.16.1.1 Adhesive Anchor Requirements

For adhesive anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive must be used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16 1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 6 times the anchor diameter. The maximum embedment depth for is 20 times the anchor diameter.

The manufacturer and product name of adhesive anchors used by the contractor must be on the Department's approved product list for “Concrete Adhesive Anchors”.

Refer to the Standard Specifications for additional requirements.

40.16.1.2 Mechanical Anchor Requirements

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 10 times the anchor diameter. The minimum member is the great of the embedment depth plus 4 inches and 3/2 of the embedment depth. Mechanical anchors are currently not allowed.

40.16.2 Concrete Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. ACI [17.4.2.9] and ACI [17.5.2.9] provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the design strength of the anchor reinforcement can be used in place of concrete breakout strength per 40.16.3 and 40.16.4. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

Per ACI [2.3], concrete anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the
requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

40.16.3 Concrete Anchor Tensile Capacity

Concrete anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.1 shows the concrete breakout failure mechanism for anchors in tension.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete anchor is determined according to this section; however, this value is only specified on the plan for mechanical anchors. The minimum pullout capacity is not specified on the plan for adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318-14 Chapter 17 is allowable, which may yield higher capacities.
Figure 40.1
Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, $A_{Nc}$, shown in Figure 40.1 is limited in each direction by $S_i$:

$$S_i = \text{Minimum of:}$$

1. 1.5 times the embedment depth ($h_{ef}$),
2. Half of the spacing to the next anchor in tension, or
3. The edge distance ($c_a$) (in).

Figure 40.2 shows the bond failure mechanism for concrete adhesive anchors in tension.
The projected influence area of a single adhesive anchor, $A_{Na}$, is shown in Figure 40.2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. $A_{Na}$ is limited in each direction by $S_i$:

$$S_i = \text{Minimum of:}$$

1. $c_{Na} = 10d_a \frac{\tau_{uncr}}{1100}$,

2. Half of the spacing to the next anchor in tension, or
3. The edge distance \( (c_a) \) (in).

<table>
<thead>
<tr>
<th>Anchor Size, ( d_a )</th>
<th>Adhesive Anchors</th>
<th>Dry Concrete</th>
<th>Water-Saturated Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min. Bond Stress, ( \tau_{uncr} ) (psi)</td>
<td>Min. Bond Stress, ( \tau_{cr} ) (psi)</td>
<td>Min. Bond Stress, ( \tau_{uncr} ) (psi)</td>
</tr>
<tr>
<td>#4 or 1/2&quot;</td>
<td>990</td>
<td>460</td>
<td>370</td>
</tr>
<tr>
<td>#5 or 5/8&quot;</td>
<td>970</td>
<td>460</td>
<td>510</td>
</tr>
<tr>
<td>#6 or 3/4&quot;</td>
<td>950</td>
<td>490</td>
<td>500</td>
</tr>
<tr>
<td>#7 or 7/8&quot;</td>
<td>930</td>
<td>490</td>
<td>490</td>
</tr>
<tr>
<td>#8 or 1&quot;</td>
<td>770</td>
<td>490</td>
<td>600</td>
</tr>
</tbody>
</table>

**Table 40.16-1**  
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for “Concrete Adhesive Anchors”. The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, \( N_u \), must be less than or equal to the factored tensile resistance, \( N_r \). For mechanical anchors:

\[
N_r = \phi_{ts} N_{sa} \leq \phi_{tc} N_{cb} \leq \phi_{tc} N_{pn}
\]

In which:

\[
\phi_{ts} = \text{Strength reduction factor for anchors in concrete, ACI [17.3.3]}
\]

\[
= 0.65 \text{ for brittle steel as defined in 40.16.1.1}
\]

\[
= 0.75 \text{ for ductile steel as defined in 40.16.1.1}
\]

\[
N_{sa} = \text{Nominal steel strength of anchor in tension, ACI [17.4.1.2]}
\]

\[
= A_{se,N} f_{uta}
\]

\[
A_{se,N} = \text{Effective cross-sectional area of anchor in tension (in}^2\text{)}
\]

\[
f_{uta} = \text{Specified tensile strength of anchor steel (psi)}
\]
\[
\begin{align*}
\leq & \quad 1.9f_{ya} \\
\leq & \quad 125 \text{ ksi}
\end{align*}
\]

\[f_{ya} = \text{Specified yield strength of anchor steel (psi)}\]

\[\phi_{tc} = \text{Strength reduction factor for anchors in concrete}\]

\[
\begin{align*}
= & \quad 0.65 \text{ for anchors without supplementary reinforcement per 40.16.2} \\
= & \quad 0.75 \text{ for anchors with supplementary reinforcement per 40.16.2}
\end{align*}
\]

\[N_{cb} = \text{Nominal concrete breakout strength in tension, ACI [17.4.2.1]}\]

\[
N_{cb} = \frac{A_{Nc}}{9(h_{ef})^2} \Psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{b}
\]

\[A_{Nc} = \text{Projected concrete failure area of a single anchor, see Figure 40.1}\]

\[
A_{Nc} = (S_1 + S_2)(S_3 + S_4)
\]

\[h_{ef} = \text{Effective embedment depth of anchor per Table 40.16-1. May be reduced per ACI [17.4.2.3] when anchor is located near three or more edges.}\]

\[\Psi_{ed,N} = \text{Modification factor for tensile strength based on proximity to edges of concrete member, ACI [17.4.2.5]}\]

\[
\begin{align*}
= & \quad 1.0 \text{ if } c_{a,min} \geq 1.5h_{ef} \\
= & \quad 0.7 + 0.3 \frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}
\end{align*}
\]

\[c_{a,min} = \text{Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.1 (in)}\]

\[\Psi_{c,N} = \text{Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, ACI [17.4.2.6]}\]

\[
\begin{align*}
= & \quad 1.0 \text{ when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels} \\
= & \quad 1.4 \text{ when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels}
\end{align*}
\]

\[\Psi_{cp,N} = \text{Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, ACI [17.4.2.7]}\]

\[
\begin{align*}
= & \quad 1.0 \text{ if } c_{a,min} \geq c_{ac}
\end{align*}
\]
\[
c_{ac} = \frac{c_{a,\text{min}}}{c_{ac}} \geq 1.5h_{ef} \frac{c_{a,\text{min}}}{c_{ac}} \text{ if } c_{a,\text{min}} < c_{ac}
\]

- \(c_{ac}\) = Critical edge distance (in)
  = 4.0h_{ef}

- \(N_b\) = Concrete breakout strength of a single anchor in tension in uncracked concrete, ACI [17.4.2.2]
  = 0.538f'_{c} (h_{ef})^{1.5} \text{ (kips)}

- \(N_{pn}\) = Nominal pullout strength of a single anchor in tension, ACI [17.4.3.1]
  = \(\psi_{c,p}N_p\)

- \(\psi_{c,p}\) = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, ACI [17.4.3.6]
  = 1.4 where analysis indicates no cracking at service load levels
  = 1.0 where analysis indicates cracking at service load levels

- \(N_p\) = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

For adhesive anchors:

\[
N_p = \phi_{ts} N_{sa} \leq \phi_{tc} N_{cb} \leq \phi_{tc} N_a
\]

In which:

- \(N_{cb}\) = Nominal concrete breakout strength in tension, ACI [17.4.2.1]
  = \(\frac{A_{nc}}{9(h_{ef})^2} \psi_{ed,N} \psi_{c,N} \psi_{c,p,N} N_b\)

- \(h_{ef}\) = Effective embedment depth of anchor. May be reduced per ACI [17.4.2.3] when anchor is located near three or more edges.
  \(\leq 20d_a\) (in)

- \(d_a\) = Outside diameter of anchor (in)

- \(\psi_{c,p,N}\) = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, ACI [17.4.2.7]
  = 1.0 if \(c_{a,\text{min}} \geq c_{ac}\)
\[
\frac{c_{a,\min}}{c_{ac}} \geq 1.5\frac{h_{ef}}{c_{ac}} \quad \text{if} \quad c_{a,\min} < c_{ac}
\]

\(c_{a,\min}\) = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.1 or Figure 40.2 (in)

\(c_{ac}\) = Critical edge distance (in)

\(= 2.0h_{ef}\)

\(N_{a}\) = Nominal bond strength of a single anchor in tension, ACI [17.4.5.1]

\[
N_{a} = \frac{A_{Na}}{4c_{Na}^2} \psi_{ed,Na} \psi_{cp,Na} N_{ba}
\]

\(A_{Na}\) = Projected influence area of a single adhesive anchor, see Figure 40.2

\[
A_{Na} = (S_1 + S_2)(S_3 + S_4)
\]

\(\psi_{ed,Na}\) = Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, ACI [17.4.5.4]

\[
\psi_{ed,Na} = 1.0 \quad \text{if} \quad c_{a,\min} \geq c_{Na}
\]

\[
\psi_{ed,Na} = 0.7 + 0.3\frac{c_{a,\min}}{c_{Na}} \quad \text{if} \quad c_{a,\min} < c_{Na}
\]

\(c_{Na}\) = Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor

\[
c_{Na} = 10d_{a} \frac{\tau_{uncr}}{1100} \quad \text{(in)}
\]

\(\tau_{uncr}\) = Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1

\(\psi_{cp,Na}\) = Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, ACI [17.4.5.5]

\[
\psi_{cp,Na} = 1.0 \quad \text{if} \quad c_{a,\min} \geq c_{ac}
\]

\[
\psi_{cp,Na} = \frac{c_{a,\min}}{c_{ac}} \quad \text{if} \quad c_{a,\min} < c_{ac}
\]

\(N_{ba}\) = Bond strength in tension of a single adhesive anchor, ACI [17.4.5.2]

\[
N_{ba} = \tau_{cr} \pi a h_{ef}
\]
τ_{cr} = \text{Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1}

\textbf{Note:} Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI 355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr}.

In addition to the checks listed above for all adhesive anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per ACI [17.3.1.2]:
\[0.55\phi_{tc}N_{ba} \geq N_{ua,s}\]

40.16.4 Concrete Anchor Shear Capacity

Concrete anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. \textbf{Figure 40.3} shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in \textbf{Figure 40.3} is limited vertically by H, and in both horizontal directions by S_i:
\[H = \text{Minimum of:}\]
\[1. \text{ The member depth (h_a) or}\]
\[2. \text{ 1.5 times the edge distance (c_{a1}) (in).}\]

\[S_i = \text{Minimum of:}\]
\[1. \text{ Half the anchor spacing (S),}\]
\[2. \text{ The perpendicular edge distance (c_{a2}), or}\]
\[3. \text{ 1.5 times the edge distance (c_{a1}) (in).}\]
If the shear is applied to more than one row of anchors as shown in Figure 40.4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.
The factored shear force on each anchor, $V_u$, must be less than or equal to the factored shear resistance, $V_r$. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} V_{sa} \leq \phi_{vc} V_{cb} \leq \phi_{vp} V_{cp}$$

In which:

- $\phi_{vs}$ = Strength reduction factor for anchors in concrete, ACI [17.3.3]
  - 0.60 for brittle steel as defined in 40.16.1.1
  - 0.65 for ductile steel as defined in 40.16.1.1
- $V_{sa}$ = Nominal steel strength of anchor in shear, ACI [17.5.1.2]
  - $0.6A_{se,V}f_{ula}$
- $A_{se,V}$ = Effective cross-sectional area of anchor in shear (in$^2$)
- $\phi_{vc}$ = Strength reduction factor for anchors in concrete, ACI [17.3.3]
  - 0.70 for anchors without supplementary reinforcement per 40.16.2
  - 0.75 for anchors with supplementary reinforcement per 40.16.2
- $V_{cb}$ = Nominal concrete breakout strength in shear, ACI [17.5.2.1]
  - $A_{vc} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{p,V} V_b$
  - $\frac{A_{vc}}{4.5(c_{a1})^2} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{p,V} V_b$

**Figure 40.4**

Concrete Anchor Shear Force Cases

Case 1:
- SP $\geq$ E1
- $c_{a1} = E1$

Case 2:
- SP $\geq$ E1
- $c_{a1} = E2$

Case 3:
- SP $< E1$
- $c_{a1} = E1$
\[ A_{VC} = \text{Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.3} \]
\[ = H(S_1 + S_2) \]

\[ c_{a1} = \text{Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.3 and Figure 40.4 (in)} \]

\[ \psi_{ed,V} = \text{Modification factor for shear strength of anchors based on proximity to edges of concrete member, ACI [17.5.2.6]} \]
\[ = 1.0 \text{ if } c_{a2} \geq 1.5c_{a1} \text{ (perpendicular shear)} \]
\[ = 0.7 + 0.3 \frac{c_{a2}}{1.5c_{a1}} \text{ if } c_{a2} < 1.5c_{a1} \text{ (perpendicular shear)} \]
\[ = 1.0 \text{ (parallel shear)} \]

\[ c_{a2} = \text{Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to } c_{a1}, \text{ see Figure 40.3 (in)} \]

\[ \psi_{c,V} = \text{Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, ACI [17.5.2.7]} \]
\[ = 1.4 \text{ for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels} \]
\[ = 1.0 \text{ for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.2 or with edge reinforcement smaller than a No. 4 bar} \]
\[ = 1.2 \text{ for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge} \]
\[ = 1.4 \text{ for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches} \]

\[ \psi_{h,V} = \text{Modification factor for shear strength of anchors located in concrete members with } h_a < 1.5c_{a1}, \text{ ACI [17.5.2.8]} \]
\[ = \frac{1.5c_{a1}}{h_a} \geq 1.0 \]

\[ h_a = \text{Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.3 (in)} \]
Ψ_{p,v} = Modification factor for shear strength of anchors based on loading direction, ACI [17.5]

= 1.0 for shear perpendicular to the concrete edge, see Figure 40.3
= 2.0 for shear parallel to the concrete edge, see Figure 40.3

\( V_b \) = Concrete breakout strength of a single anchor in shear in cracked concrete, per ACI [17.5.2.2], shall be the smaller of:

\[
7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a} \sqrt{f'_c(c_{at})^{1.5}} \text{ (lb)}
\]

Where:
\( l_e = h_{ef} \leq 8d_a \)
\( d_a \) = Outside diameter of anchor (in)
\( f'_c \) = Specified compressive strength of concrete (psi)

and

\( \phi_{vp} \)

= Strength reduction factor for anchors in concrete

= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

\( V_{cp} \)

= Nominal concrete pryout strength of a single anchor, ACI [17.5.3.1]

= 2.0\( N_{cp} \)

Note: The equation above is based on \( h_{ef} \geq 2.5 \text{ in.} \) All concrete anchors must meet this requirement.

\( N_{cp} \)

= Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors:
\[
\frac{A_{Nc}}{9(h_{ef})^2} \Psi_{edN} \Psi_{c,N} \Psi_{cpN} N_b
\]

adhesive anchors:
\[
\frac{A_{Na}}{4(c_{Na})^2} \Psi_{edNa} \Psi_{cpNa} N_{ba}
\]

and
\[
\frac{A_{Nc}}{9(h_{ef})^2} \Psi_{edN} \Psi_{c,N} \Psi_{cpN} N_b
\]
For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.5 Interaction of Tension and Shear

For anchors that are subjected to tension and shear, interaction equations must be checked per ACI [17.6].

If \( \frac{V_{ua}}{\phi V_n} \leq 0.2 \) for the governing strength in shear, then the full strength in tension is permitted:

\[ \phi N_n \geq N_{ua} \]

If \( \frac{N_{ua}}{\phi N_n} \leq 0.2 \) for the governing strength in tension, then the full strength in shear is permitted:

\[ \phi V_n \geq V_{ua} \]

If \( \frac{V_{ua}}{\phi V_n} > 0.2 \) for the governing strength in shear and \( \frac{N_{ua}}{\phi N_n} > 0.2 \) for the governing strength in tension, then:

\[ \frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \]

40.16.6 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.3.

Typical notes for bridge plans (shown in all capital letters):

Adhesive anchors located in uncracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX” IN CONCRETE. (Illustrative only, values must be calculated depending on the specific situation).

Adhesive anchors located in cracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX” IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. (Illustrative only, values must be calculated depending on the specific situation).

When using anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item “Adhesive Anchors _-Inch”.

For anchors using rebar, the rebar should be listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS Coated Structures”.

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When adhesive anchors are used as an alternative anchorage the following note should be included in the plans:

ADHESIVE ANCHORS SHALL CONFORM TO SECTION 502.2.12 OF THE STANDARD SPECIFICATION. (Note only applicable when the bid item Adhesive Anchor is not used).

It should be noted that AASHTO is considering adding specifications pertaining to concrete anchors. This chapter will be updated once that information is available.
40.17 Plan Details

1. Excavation for Structures on Overlays

There is considerable confusion on when to use or not use the bid item “Excavation for Structures” on overlay projects. In order to remove the confusion, the following note is to be added to all overlay projects that only involve removal of the paving block (or less).

Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item “(insert applicable bid item)”.

If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay, the “Excavation for Structures” bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements, show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by current standard of a 0.02 ft/ft cross slope, a cross slope of 0.01 ft/ft or 0.015 ft/ft may be the most desirable.

The designer should evaluate 3 types of repairs. “Preparation Decks Type 1” is concrete removal to the top of the bar steel. “Preparation Decks Type 2” is concrete removal below the bar steel. “Full Depth Deck Repair” is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of “Full Depth Deck Repair” on the plan sheet.

Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction, consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

The Region should provide the designer with a Rehabilitation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

Additional items:
• Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

• Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

• Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

• Coordinate asphaltic materials with the Region and roadway designers.

See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.
40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4" or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4" and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.
40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

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Table 40.19-1
Reinforcing Steel for Deck Slabs on Girders for Deck Replacements – HS20 Loading

Max. Allowable Design Stresses: $f'_c = 4000$ psi, $f_y = 60$ ksi, Top Steel 2-1/2" Clear, Bottom Steel 1-1/2" Clear, Future Wearing Surface = 20 lbs/ft. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.
40.20 Fiber Reinforced Polymer (FRP)

40.20.1 Introduction

Fiber reinforced polymer (FRP) material is a composite composed of fibers encased in a polymer matrix. The fibers provide tensile strength while the resin protects the fibers and transfers load between them. FRP can be used to repair or to retrofit bridges. Repair is often defined as returning a member to its original condition after damage or deterioration while retrofitting refers to increasing the capacity of a member beyond its original capacity.

For plan preparations, FRP repairs and retrofits are categorized as either structural strengthening or non-structural protection. Contact the Bureau of Structures Design Section for current Special Provisions and for other FRP considerations.

40.20.2 Design Guidelines

While there is no code document for the design of FRP repairs and retrofits, there are two nationally recognized design guidelines: the Guide Specification for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements (14.) hereinafter referred to as the AASHTO FRP Guide, and the Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures ACI 440.2R-08 (15.) hereinafter referred to as the ACI FRP Guide.

Note: BOS has been evaluating the design methodologies found in the AASHTO FRP Guide and ACI RFP Guide. Noticeable differences between the guides warrants further investigation, with input from industry representation. FRP repairs and retrofits shall be in accordance with the applicable Special Provisions.

40.20.3 Applicability

Not all concrete structures can be retrofitted or repaired using FRP. Most FRP research has been conducted on normal sized members, therefore many of the design equations cannot be used with exceptionally large or deep members. Additionally, members with disturbed regions (D-regions) such as deep beams and corbels are outside of the scope of many design equations.

The structure must have some amount of load carrying capacity prior to the installation of the FRP. Due to the potential for premature debonding, FRP cannot be counted on to carry service loads; it may only be used increase the ultimate capacity of the structure for strength and extreme event load cases. The unrepaired or unretrofitted structure be able to carry the service dead and live loads:

\[ R_r \geq \eta_1 [(DC + DW) + (LL + IM)] \]

Where:

\[ R_r = \text{factored resistance computed in accordance with AASHTO LRFD Section 5} \]
\( \eta_i = \text{load modifier} = 1.0 \)

- DC = force effects due to components and attachments
- DW = force effects due to wear surfaces and utilities
- LL = force effects due to live load
- IM = force effects due to dynamic load allowance

If capacity is added in flexure to accommodate increased loads, the shear capacity of the member must be checked to ensure that it is still sufficient for the new loading. For non-structural FRP applications, applicability checks may not be required.

### 40.20.4 Materials

A typical FRP system consists of a primer, fibers, resin, bonding material (either additional resin or an adhesive), and a protective coating. FRP is specified in terms of the types of fiber and resin, the number of layers, the fiber orientation and the geometry. FRP is sold as a system and all materials used should be from the same system.

#### 40.20.4.1 Fibers

The most common types of fiber used for bridge repairs are glass and carbon. Glass fibers are not as stiff or as strong as carbon, but they are much less expensive. Unless there is reason to do otherwise, it is recommended that glass fibers be used for corrosion protection and spall control. Carbon fibers should be used for strengthening and crack control.

Carbon fibers cannot be used where the FRP comes into contact with steel out of concerns for galvanic corrosion due to the highly conductive nature of carbon fibers. For applications where galvanic corrosion is a concern, glass fibers may be used, even in structural applications.

Often, FRP is requested by the region to provide column confinement. The engineer must determine if the requested confinement is true confinement where the FRP puts the column into triaxial compression to increase the capacity and ductility, or if the FRP is confining a patch from spalling off. In the case of true confinement (which is very rare in Wisconsin), carbon fibers should be used and the repair requires structural design. For spall control, glass fibers are acceptable and the repair is considered non-structural.

#### 40.20.4.2 Coatings

After the FRP has been installed and fully cured, a protective coating is applied to the entire system. A protective coating is needed to protect against ultraviolet degradation and can also provide resistance to abrasion, wear, and chemicals. In situations where the FRP is submerged in water, inert protective coatings can help prevent compounds in the FRP from leaching into the water, mitigating environmental impacts.
Protective coatings can be made from different materials depending on the desired coating characteristics. Common coating types include vinyl ester, urethane, epoxy, cementitious, and acrylic. Acrylic coatings are generally the least expensive and easiest to apply, though they may also be less durable. If no coating type is specified, it is likely that the manufacturer will provide an acrylic coating.

For shorter term repairs, acrylic coatings are sufficient, but longer repairs should consider other coating types such as epoxy. Any coating used must be compatible with the FRP system selected by the contractor.

40.20.4.3 Anchors

The bond between the FRP and the concrete is the most critical component of an FRP installation and debonding is the most common FRP failure mode. Certain FRP configurations use anchors to increase the attachment of the FRP and attempt to delay or prevent debonding. These anchors can consist of near surface mounted bars, fiber anchors, additional FRP strips, or mechanical anchors such as bolts. It is permitted to use additional U-wrap strips to anchor flexural FRP, but the use of additional longitudinal strips to anchor shear FRP is prohibited. The use of additional U-wrap strips for flexural anchorage is required in some instances.

Because neither design guide requires anchorage or provides information as to what constitutes anchorage, it is left to the discretion of the designer to determine if anchorage should be used and in what quantities. The use of anchors is highly encouraged, particularly for shear applications and in situations where there is increased potential for debonding such as reentrant corners.

Specifying anchors will add cost to the repair, but these costs may be offset by increased capacity accorded to anchored systems in shear. The additional costs can also be justified if debonding is a concern. If the designer chooses to use anchors, anchors should be shown on plans, but the design of the anchors is left to the manufacturer.

40.20.5 Flexure

Flexural FRP is applied along the tension face of the member, where it acts as additional tension reinforcement. The fibers should be oriented along the length of the member.

40.20.5.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For flexure, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

40.20.5.2 Composite Action

Composite action of the deck slab can be considered when designing flexural FRP repairs for girders, provided that the deck was designed to be composite. If composite action is
considered, composite section properties must be computed. These properties should be substituted into the design equations presented in this section. Accounting for composite action will increase the capacity provided by the FRP.

40.20.5.3 Pre-Existing Substrate Strain

Unless all loads are removed from the member receiving FRP (including self-weight), there will be strain present in the concrete when the FRP is applied. This initial or pre-existing substrate strain $\varepsilon_{bi}$ is computed through elastic analysis. All loads supported by the member during FRP installation should be considered and cracked section properties should be considered if necessary.

40.20.5.4 Deflection and Crack Control

Conduct standard LRFD serviceability checks for deflection and crack control while accounting for the contribution of the FRP. Because both the FRP and the concrete will be in the elastic zone at service levels, standard elastic analysis can be used to determine stresses and strains. Transformed section analysis can be used to transform the FRP into an equivalent area of concrete for the purposes of analysis. The condition of the member determines if the cracked or uncracked section properties should be used in computations.

40.20.6 Shear

In shear repair/retrofitting applications, the FRP acts essentially as external stirrups. The FRP wrap is applied with the fibers running transverse to the member.

40.20.6.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For shear, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

Additionally, the engineer must ensure that the amount of FRP capacity required does not exceed the maximum allowable shear reinforcement. It is important to note that the FRP capacity listed on the plans will be a factored capacity, while the maximum allowable shear reinforcement check is for an unfactored capacity. Strength reduction factors must be incorporated to make a proper comparison.

If the FRP capacity is close to the maximum allowed, the designer must take care to ensure that a design is feasible. The capacity provided by FRP depends on the number of FRP layers, with each additional layer providing a discrete increase in capacity. There may be a situation where $n$ layers does not provide enough capacity, but $n+1$ layers provides too much capacity and violates the maximum allowable shear reinforcement criteria. Changes in spacing of the wraps may help decrease the capacity provided by the FRP.
Example problems in shear can be found in the appendices of NCHRP Report 655 (16) and potential shear wrapping configurations can be found in NCHRP Report 678 (17).
40.21 References

1. *A Study of Policies for the Protection, Repair, Rehabilitation, and Replacement of Concrete Bridge Decks* by P.D. Cady, Penn. DOT.


10. *Control of Cracking in Concrete Structures* by ACI Committee 224, *Concrete International*, October, 1980.


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