Table of Contents

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 72-month cycle.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies Chapter 42-Bridge Preservation to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in Chapter 42-Bridge Preservation. This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See Faculties Design Manual (FDM) 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* 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 ensuring some level of acceptable serviceability; however, structure preservation as explained in Chapter 42-Bridge Preservation should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation 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.

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/M_u 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.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

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. Deck Condition less than or equal 4.
    b. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck condition greater than 4.
    b. 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

As a bridge deck ages, preservation and rehabilitation techniques are necessary to maximize the life of the deck and ensure a level of acceptable serviceability. Overlays can be a useful tool to extend the service life of structures. This section discusses several overlay methods, considerations, and guidelines for deck overlays. The provided information is intended for deck-girder structures and may be applicable for slab structures. Slab structures may have different condition triggers and may warrant additional considerations.

The following criteria should be met when determining if an overlay should be used:

- The structure is capable of carrying the overlay dead load
- 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 anticipated structure life and funding constraints

Decks deteriorate at different rates depending on many factors, including deck materials, material quality, construction quality, structure geometry, exposure to deicing agents, and traffic demands. Additionally, there is a wide variance in the amount of structure preservation techniques utilized by different regions. While the deck age can be a useful parameter, it should not be the primary consideration for determining the eligibility of overlays. Recommended preservation techniques should rely heavily on quality inspection data to determine the appropriate course of action. For more information related to preservation techniques and practices, refer to Chapter 42-Bridge Preservation.

Overlays can be an effective tool to maximize the life of the deck. Figure 40.5-1 illustrates a possible preservation scenario using deck deterioration curves showing approximate deck NBI ratings at which the overlays would occur, and the benefit of performing these overlays. This scenario assumes that the underside of deck deterioration is significantly reduced due to the preservation techniques performed on the top side of the deck.

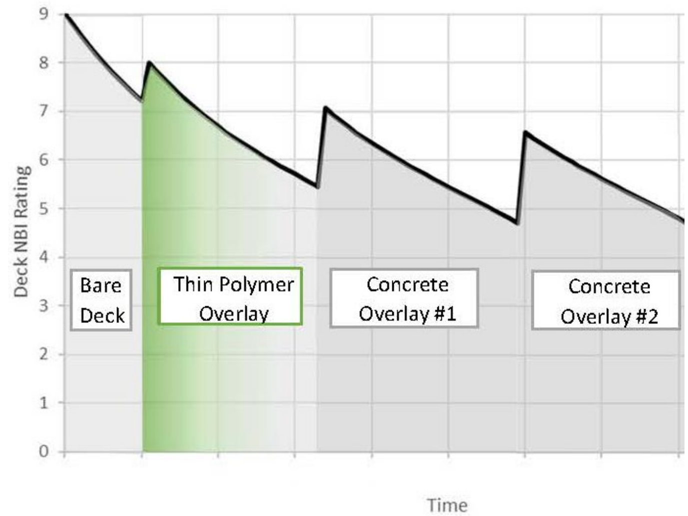

Figure 40.5-1
Deck Deterioration Curve

40.5.1 Overlay Methods

There are several commonly used overlay methods for the preservation and rehabilitation of decks. Generally, thin polymers overlays are recommended as preventative maintenance for decks with a minimal amount of deck distress. Ideally, thin polymer overlays are applied within the first couple of years to limit chloride infiltration. For decks with distress, the existing deck is typically milled and repaired with a low-slump concrete overlay as part of a more extensive bridge rehabilitation effort. For decks nearing replacement, asphaltic overlays may be a cost effective option to improve ride quality. Refer to the following sections and Table 40.5-1 and Table 40.5-2 for a list of common overlay methods and additional information.

40.5.1.1 Thin Polymer Overlay

A thin polymer overlay (TPO) is expected to extend the service life of a bridge deck for 7 to 15 years. This overlay adds minimal dead load to the existing structure while providing an impermeable surface to prevent chlorides from infiltrating the deck. It can also be used to improve or restore friction on bridge decks.

In general, thin polymer overlays are defined as 1-inch thick or less overlays consisting of a polymer binder with aggregates and can be placed either as a multi-layer, slurry, or premixed system. Typical polymer binders are either epoxy, polyester, or methacrylate based. For WisDOT applications, TPO's consist of a two-layer, two-component epoxy polymer in conjunction with natural or synthetic aggregates for a 1/4-inch minimum total thickness. For dead load purposes, use 5 psf for thin polymer overlays. Refer to the approved products list for a list of pre-qualified polymer liquid binders.

Cracks will develop in a new concrete deck throughout the first couple of years in response to vehicular and environmental loads. Initial concrete cracking should occur within the first two years of new deck construction. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. Therefore, the earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined that a thin polymer overlay should be placed in the next construction season, the thin polymer overlay should be included in the same contract as the new deck.
Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment. See 40.5.5.1 for deck sealing usage in place of thin polymer overlays.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition.

- TPO's should not be placed on Portland cement concrete patches less than 28 days old. Patch and crack repairs shall be compatible with the overlay material.

- The earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined through structure certification that a thin polymer overlay should be placed in the next construction season in lieu of future deck sealing, the thin polymer overlay should be included in the same contract as the new deck.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

Thin polymer overlays may be considered where friction needs to be restored or improved. For deck applications, a two-layer polymer overlay system shall be used throughout the deck surface (driving lanes and shoulder) for deck preservation against chloride infiltration. Additionally, the two-layer application provides deck protection against snowplow and snowmobile operations. The “Polymer Overlay” bid item is the standard two-layer polymer overlay with natural or synthetic aggregates and provides improved or “enhanced” surface friction. For situations warranting a higher skid resistance, the bid item “High Friction Surface Treatment Polymer Overlay” with calcined bauxite aggregates shall be used. See Chapter 40 Standards and the Traffic Engineering, Operations & Safety Manual TEOpS 12-5-4 for additional guidance.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1” of existing deck surface is removed along with any unsound material and asphaltic patches.

Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.

The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability. In some situations, PPC has exhibited reflective cracking from the deck below. Cracks should be sealed with methacrylate sealer as recommended by the particular PPC manufacturer.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern but should not be the primary reason for applying PPC.

Since most applications recommend a 1-inch or less overlay, PPC overlays are considered a thin polymer overlay and have similar requirements and restrictions. PPC overlays should be limited to decks in good condition that require shorter traffic disruptions for sites with high traffic volumes and lane closure restrictions. PPC is a durable product and has a relatively fast curing time (2 to 4 hours), but also has a higher cost as compared to a concrete overlay. PPC overlays should be used based on the following restrictions:

- Deck wearing surface distress should not exceed 5% of the total deck area.

- Decks should have a NBI rating of 6 or greater and be less than 20 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 2 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.

- PPC overlays should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.

- PPC shall not be used for structural repairs due to costs and performance concerns.

- PPC should not be used unless lower cost preservation treatments (e.g. thin polymer overlays) have proven ineffective. If a bridge deck has a TPO, chloride ion testing will be performed near the end of the TPO life to determine eligibility of either a TPO reapplication or a PPC overlay (if the structure also meets the condition and ADT criteria). If the average chloride concentration at 1” depth is less than 1 lb/cy, the existing TPO is considered an effective preservation treatment. The TPO should be replaced with another TPO. If the average chloride concentration at 1” depth is greater than 1 lb/cy, a PPC overlay should be considered (including a ¾”-1” milling of the deck surface to remove chlorides).

Note: PPC overlays are expensive and new to WisDOT. As a result, use of PPC overlays should be limited to preservation projects that meet the requirements outlined in Figure 40.5-2 **or as approved by the Bureau of Structures.**

Other factors which may affect BOS approval of PPC overlays include:

- Proximity to other high ADT roadways (i.e., service ramp)
- Backbone/Interstate or high priority structure
- Preservation of slab structure
- Presence of active cracking (not recommended when active cracking is present)
- Enhanced friction (not to be used as a ride correction or as a high-friction improvement)

40.5.1.4 Polymer Modified Asphaltic Overlay

A polymer modified asphaltic (PMA) overlay is expected to extend the service life of a bridge deck for 10 to 15 years. This system is a mixture of aggregate, asphalt content, and a thermoplastic polymer modifier additive, which can easily be placed as a deck overlay using conventional asphalt paving equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

The added polymer allows for the overlay to resist water and chloride infiltration. Proper mix control and placement procedures are critical in achieving this protection. Core tests have shown the permeability of this product is dependent on the aggregate. As a result, limestone aggregates should not be used.

PMA overlays can be used on more flexible structures (e.g. timber decks or timber slabs) and to minimize traffic disruptions.

Designers should contact the region to determine if a PMA overlay is a viable solution for the project. In some areas, product availability or maintaining an acceptable temperature may be problematic.

Note: PMA overlays are expensive, have a limited service life relative other overlay types, and product availability may be problematic. As a result, PMA overlays usage should be limited.

40.5.1.5 Asphaltic Overlay

An asphaltic overlay, without a waterproofing membrane, is expected to extend the service life of a bridge deck for 3 to 7 years. This system may be a viable treatment if the deck or bridge is programmed for replacement within 4 years on lightly traveled roadways and is able to provide a smooth riding surface. Without a waterproofing material, the overlay may trap moisture at the existing deck surface, which may accelerate deck deterioration.

These overlays must be watched closely for distress as the existing deck surface problems are concealed. This system is typically an asphaltic pavement with a mixture of aggregates and asphaltic materials, which can easily be placed as a deck overlay using conventional asphaltic mixing and placement equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

Note: Asphaltic overlays, without a waterproofing membrane, are not eligible for federal funds.

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

An asphaltic overlay, with a waterproofing membrane, is currently being used on a very limited basis. This system is expected to extend the service life of a bridge deck for 5 to 15 years. Experience indicates that waterproofing membranes decrease the rate of deck deterioration by preventing or slowing the migration of water and chloride ions into the concrete.

In the 1990's, waterproofing membranes were actively used with asphaltic overlays for protecting existing decks, but were phased out by 2009 when they were restricted due to performance concerns and the inability to inspect the deck. As a result, low slump concrete or PMA overlays are currently recommended when deck or bridge replacements are programed beyond 4 years, unless approved otherwise.

Note: Asphaltic overlays, with a waterproofing membrane, requires prior-approval by the Bureau of Structures. This system is currently under review for possible improvements.

40.5.1.7 Other Overlays

Several other overlay systems have been used on past projects, but are generally not used currently. Use of these systems or other systems not previously mentioned require prior-approval by the Bureau of Structures.

- Micro-silica (silica-fume) modified concrete overlay – Provides good resistance to chloride penetration due to its low permeability.

- Latex modified concrete overlay – Provides a long-lasting overlay system with minimal traffic disruptions. Several other states are currently using this overlay method with hydrodemolition deck preparations.

- Reinforced concrete overlays:

    - Thin overlays (< 4 ½”) – Uses a superplasticizer and fiber reinforcement (steel or synthetic) for additional crack control by reducing cracks and crack widths.

    - Thick overlays (≥ 4 ½”) – Uses steel reinforcements, rebar or weld wire fabric, typically for new structural decks. This overlay is intended to provide at least one layer of steel reinforcement, in each direction, for crack control. This overlay is currently recommended for PS box girder superstructures, which allows for composite details and improved means to control longitudinal reflective cracking. For most cases, steel reinforcement is not required when rehabilitation overlays exceed 4 1/2 - inches. Use of low slump Grade E concrete may not be suitable when incorporating steel reinforcements.

40.5.2 Selection Considerations

The selection of an overlay type is made considering several factors to achieve the desired extended service life. Several of these factors are provided in Table 40.5-1 and Table 40.5-2 to aid in the selection of an overlay for the preservation and rehabilitation of decks.

Overlay Type	Thin Polymer Overlay	Low Slump Concrete Overlay	Polyester Polymer Concrete Overlay (2)	Polymer Modified Asphaltic Overlay	Asphaltic Overlay (4)	Asphaltic Overlay with Membrane (2)
Overlay Life Span (years)	7 to 15	15 to 20	20 to 30	10 to 15	3 to 7	5 to 15
Traffic Impact (6)	< 1 day	7 days +/-	< 1 day	1-2 days	1-2 days	1-2 days
Overlay Costs ($/SF) (1)	$3 to $5	$4 to $7	$8 to $18	$10 to $22	$1 to $2	$5 to $8
Project Costs ($/SF) (1)	$4 to $8	$14 to $23	$10 to $30	$20 to $42	$4 to $10	$8 to $16
Overlay Minimum Thickness (Inches)	0.375	1.50	0.75	2.00	2.00	2.00
Wearing Surface Distress (delamination, spalls, or patches)	≤ 2%	≤ 25%	≤ 5%	≤ 25%	NA	≤ 25%
Deck Patch Material	Concrete (3), rapid set (2), or overlay mix	Overlay mix	Concrete (3), rapid set, or PPC	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)
Typical Surface Preparation	Shot blast	Milled and shot blast (5)	Shot blast (5)	Sand blast	Water or air blast	Sand blast (5)
Overlay Finish	Aggregates	Tined	Tined and sanded	None	None	None

(1) Estimated costs based on CY2017 and is for informational pursues only. Overlay costs includes minimum overlay thickness and overlay placement costs. Project costs includes all structure associated costs (joint repairs, deck repairs, surface preparations, minimum overlay thickness). Costs do not include traffic control costs or other costs not captured on structure costs.
(2) Requires approval
(3) Portland cement concrete patch material may require a 28-day cure prior to overlay placement.
(4) Not eligible for federal funds
(5) 1 to 3/4-inch milling recommended for decks exposed longer than 10 years and not previously milled
(6) Estimated durations based on the overlay placement time to the minimum time until traffic can to be placed on the overlay. Durations do not include time for deck repairs or staging considerations.

Table 40.5-1
Overlay Selection Considerations

Overlay Type	Advantages	Disadvantages	Notes
Thin Polymer Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction	• Requires a concrete age of at least 28 days • Requires decks with minimal defects and low chloride concentrations • Sensitive to moisture, temperature, and humidity at placement • Reflective cracking resistance concerns	
Low Slump Concrete Overlay	• Contractor familiarity and department experience • Long life span potential • Durable • Ease to accommodate grade differences and deficiencies	• Traffic disruptions • Additional dead load • High maintenance requirements • Railing height concerns • Susceptible to cracking • Specialized finishing equipment	• May require crack sealing the following year and periodically thereafter.
Polyester Polymer Concrete Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction • Long life span potential • Durable • Low maintenance requirements	• High cost • Dedicated equipment • Limited usage in Wisconsin • Sensitive to moisture, temperature, and humidity at placement	• Requires BOS Prior-Approval
Polymer Modified Asphaltic Overlay	• Minimal traffic disruptions • Ease to construct • Can be used on more flexible structures (e.g. timber decks or timber slabs)	• High cost • Susceptible to permeability • Difficult to assess top of deck condition	• Contact region for availability • Minimal research has been performed on the durability of this system in Wisconsin
Asphaltic Overlay	• Low cost • Ease to construct • Ease to accommodate grade differences and deficiencies	• Short life span • Not eligible for federal funds • Overlay permeability • Difficult to assess top of deck condition	• Deck or bridge replacement should be programmed within 4 years
Asphaltic Overlay with Membrane	• Ease to construct • Minimal traffic disruptions • Long life span potential • Can be used on more flexible structures (e.g. PS box girders)	• Susceptible to permeability • Requires a membrane • Difficult to assess top of deck condition	• Currently under review • Requires BOS Prior-Approval

Table 40.5-2

Overlay Advantages, Disadvantages, and Notes

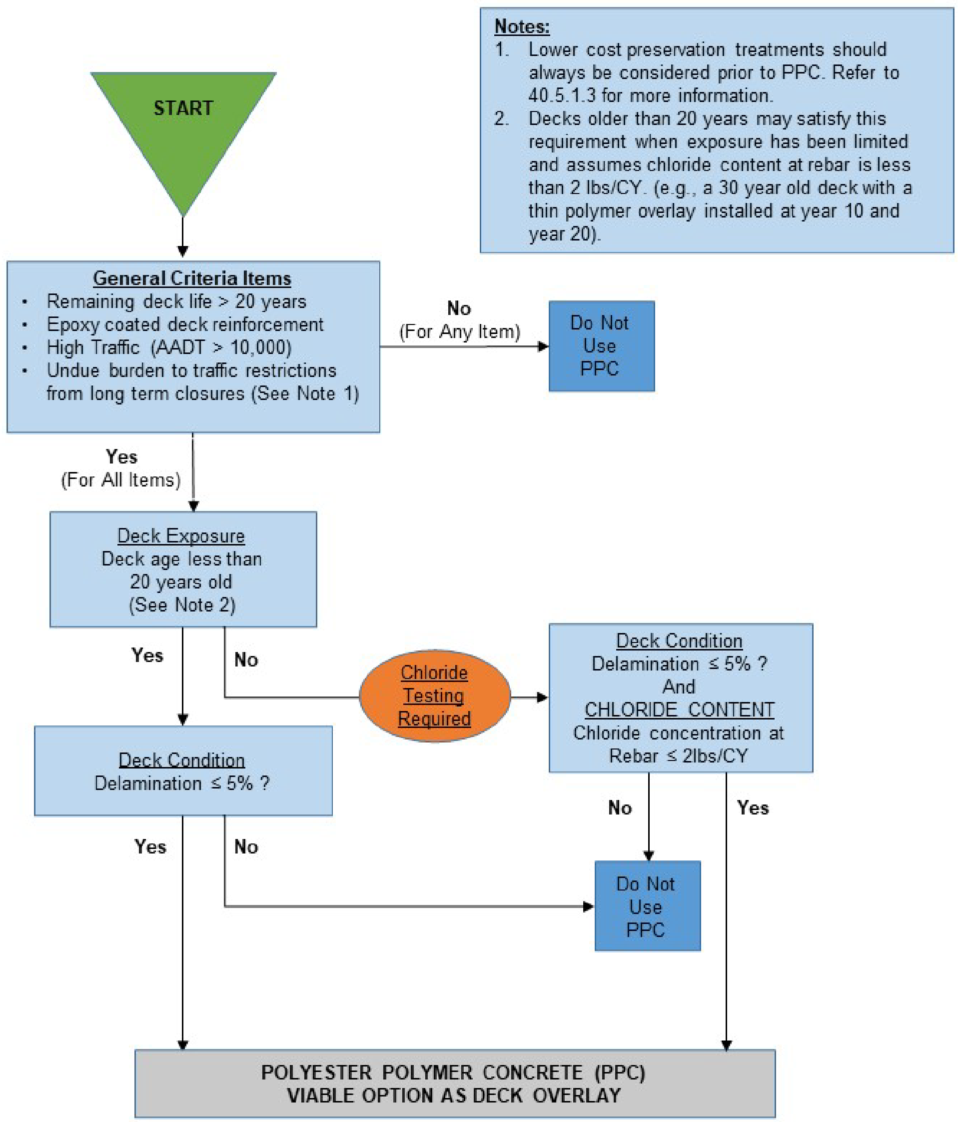

Figure 40.5-2
Polyester Polymer Concrete Overlay Usage Flowchart

40.5.3 Deck Assessment

The following are common deck assessment tools that can be used to survey existing deck conditions:

- Visual Inspections – Used to detect surface cracks, discontinuities, corrosion, and contamination.

- Audible Inspections - The two most common types of audible inspections are chain dragging and hammer sounding. Chain dragging is normally used on large concrete surface areas, such as bridge decks, while hammer sounding can be used on a number of materials in random locations. Both methods typically rely on the experience of the inspector to differentiate the relative sounds of similar materials.

- Infrared thermography - Infrared Thermography (IR) is an alternative tool for locating and mapping delaminations in bridge decks and pavements. A technique using an infrared scanner and control video camera, infrared thermography senses temperature differences between delaminated and non-delaminated areas.

- Ground penetrating radar (GPR) – GPR is a technique using electromagnetic signals, which can detect dielectric differences. This method can be used to measure concrete cover, overlay thickness, and reinforcing steel locations. This method can also be used to locate delaminations.

- Deck cores – Cores can be used to determine existing overly thicknesses, concrete cover, and concrete strength. As-built plans should only be used as a reference for existing conditions. Additionally, cores can be used to determine chloride content profiles. For asphaltic overlay, coring may be the best tool for deck assessments.

- Chloride Ion Testing - Chloride ions are the major cause of reinforcing steel corrosion in concrete. In evaluating chloride content, it is recommended that a chloride profile (chloride concentration percentage versus depth measurement below the concrete surface) be developed. This profile is important for assessing the future corrosion susceptibility of steel reinforcing and in determining the primary source of chlorides.

- Half-cell potential testing - A method used to detect whether the reinforcing steel is under active corrosion.

Visual inspections, audible inspections, and IR are the most common deck assessment tools for identifying delaminations and unsound concrete. For more information on deck assessment tools, refer to the Structure Inspection Manual – Part 5 – NDE and PDE Testing. Deck condition surveys should be placed on the structures plans. This should include the survey type and date when the survey was completed.

40.5.4 Deck Preparations

Prior to placing overlays, the existing deck surface will require deck preparations to repair the existing deck and to ensure that the overlay is properly bonded to the existing concrete. These preparations can range from sand blasting the entire deck to milling the entire deck with extensive repairs and are dependent on the existing deck conditions (distress, chloride concentration, existing overlay, proposed overlay, etc.).

The below deck preparations are typically used prior to placing overlays. Check the latest specifications for additional information.

<u>Concrete Removal</u>

Concrete deck removal usually includes the removal of unsound surface materials and the removal of a predetermined depth to remove concrete with high chloride concentrations. The following techniques can be used for large concrete removal areas:

> Mechanical scarification or milling – The removal of existing deck to predetermined depth using a milling machine and other approved operations. This process can remove concrete with high chloride contents. However, this aggressive removal process has the potential to introduce micro-cracking into the existing deck.
>
> Hydrodemolition – The removal of existing deck to a predetermined depth and the ability to selectively remove distressed areas using ultra high-pressure water-jetting (above 25,000 psi). A benefit to this process is that it does not introduce micro-cracking. WisDOT has very limited experience with this process and is usually cost prohibitive.

Generally, decks receiving a low slump concrete overlay will also include a 1-inch minimum deck removal. This assumes the existing top of deck has been exposed long enough to develop high chloride concentrations and would benefit from a milling operation. For early aged or protected (e.g. polymer overlay) decks, concrete milling may not be necessary prior to the overlay application and may be deferred to future overlay applications. Typically, only one aggressive milling operation is practical for a deck to leave sufficient cover for future overlays. Maintain ½” to 1” of rebar cover to ensure proper bonding and to protect the rebar and coating during the milling operation.

<u>Deck Repairs</u>

Care should be taken to limit damaging sound concrete and the existing reinforcement. Use of appropriate tools, hammers no more than 35 pounds and no more than 15 pounds when within one inch of the steel, is intended to limit distressed areas and avoid full-depth repairs. Additionally, saw cut depths should be carefully monitored such that the existing steel is not cut.

Cathodic protection may be warranted for decks with a high chloride content to help prevent corrosion from initiating.

The following items are associated with repairing distressed deck areas as shown in Figure 40.5-3:

Preparation Decks Type 1 – The removal of existing patches and unsound concrete only to a depth that exposes 1/2 of the peripheral area of the top or bottom bar steel in the top mat of reinforcement. Care should be taken to limit damaging sound concrete.

Preparation Decks Type 2 – The removal of existing unsound concrete below the limit of the type 1 removal described above. One inch below the bottom of the top or bottom bar steel in the top mat of reinforcement is the minimum depth of type 2 removal.

Full-Depth Deck Repair – The complete removal of existing concrete.

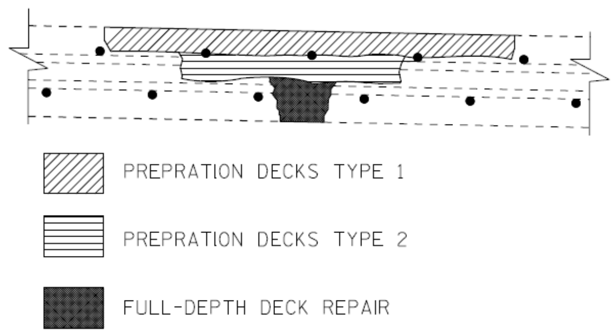

Figure 40.5-3
Deck Repairs

Deck Patches

Portland cement concrete is the preferred patch material. This material is easy to work with and very economical. When traffic impacts warrants, other materials may be considered. For concrete overlays, Type 1 and Type 2 deck patch repairs should be filled during the concrete overlay placement. Full-depth deck repairs should not be filled during the concrete overlay placement, but rather filled and curing a minimum of 24 hours before placing the concrete overlay. For other overlays, concrete repairs are usually properly cured prior to placing the overlay.

For minimal traffic impacts, a rapid-set material may be used for deck patches on asphaltic and thin polymer overlays. When repair quantities are minimal, distress areas less than 5% of the entire deck area, PPC overlays may use PPC to fill deck repairs prior overlay placement. See Table 40.5-1 for typical deck patch materials. Refer to the approved products list for a list of pre-qualified rapid setting concrete patch materials and their associated restrictions.

Surface Removal and Surface Preparation

Overlays require a properly prepared deck to achieve the desired bond strength. The following techniques are used for deck surface removal and preparations for an overlay:

Air cleaning – A preparation process to remove loose materials with compressed air. This process is intended to remove any material that may have gathered after the use of surface or concrete removal processes. This process is performed just prior to installing the overlay.

Water blasting (pressure or power washing) - A preparation process used to remove loose materials using low to high pressure water (5,000 psi to 10,000 psi). This process is beneficial as it keeps down dust and can remove loose particles.

Sand blasting – A surface removal process to remove loose material, foreign material, and loose concrete with sand material.

Shot blasting – A surface removal process to remove loose material, foreign material, and loose concrete by propelling steel shot against the concrete surface. This process also provides a roughen surface texture for improved bonding for overlays. Note: TPO's and PPC overlays provisions required a concrete surface profile meeting CSP-5 prior to overlay placement. This surface profile can be achieved using medium to medium-heavy shot blast.

40.5.5 Preservation Techniques

The following are some of the common activities being used to preserve decks and overlays:

- Deck cleaning (sweeping and power washing)
- Deck sealing/crack sealing
- Joint cleaning
- Joint repairs
- Deck patching

For additional preservation techniques and information refer to Chapter 42-Bridge Preservation.

40.5.5.1 Deck Sealing

Deck sealing has been found to be a cost-effective tool in preserving decks and overlays. In general, deck treatments should be applied as early as possible and re-applied thereafter. The frequency of deck sealing is dependent on the roadway traffic volume. Decks are to be sealed at initial construction and then resealed at the frequency shown in Table 40.5-3**.** Decks are to be resealed twice prior to applying a thin polymer overlay. Crack sealing should be considered as a potential combined treatment when deck sealing.

Roadway ADT	Deck Sealing Frequency
ADT < 2,500	4 – 5 years
2,500 <= ADT < 6,500	4 years
6,500 <= ADT < 15,000	3 years
ADT >= 15,000	2 years*

*In place of deck sealing, a thin polymer overlay is recommended within 2 years of deck construction. Use of the thin polymer overlay at this time will help minimize traffic impacts related to deck preservation work.

Table 40.5-3
Deck Sealing Frequency

Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment.

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.

- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.

- Verify the desired transverse cross slope with the Regions as they may want to use current standards.

- 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 and the bar ends are not anchored, either adjacent spans must be shored or a special analysis and removal plan are required. Reinforcement shall be anchored using Portland cement concrete.

- Asphaltic overlays 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 Portland cement concrete.

- Joints and floor drains should be modified to accommodate the overlay

- Concrete chloride thresholds – Chloride content tests measure the chloride ion concentrations at various depths. Generally, research has shown initiation of corrosion is expected when the chloride content is between 1 to 2 lbs/CY in concrete for uncoated bars and 7 to 12 lbs /CY for epoxy coated bars at the reinforcement. These limits are referred to as the threshold for corrosion. Threshold limits do not apply to stainless steel rebar.

When the chloride ion content is greater than 0.8 lbs/CY in concrete for uncoated bars and 5 lbs /CY for epoxy coated bars at the reinforcement depth, measures should be considered to limit additional chloride infiltration.

- See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

- Refer the standard details for the most current bid items.

- Overlay transitional areas should be used and coordinated when accommodating profile differences. These transitions are intended to improve ride quality and protect against snowplow damage. Ideally, transitions are placed such that the overlay thickness remains constant, which requires a tapered removal of the existing surface over a sufficient distance. For profile adjustments 1 1/2-inch or greater, transitional areas should consider a minimum taper rate of 1:250 for low-speed applications (RSD< 50 mph) and for high-speed applications up to a 1:400 taper rate. Typically, thicker profile adjustments are provided off the bridge deck and are coordinated by the roadway designer. For profile adjustments less than 1 1/2-inch, a minimum rate of 1:250 may be used regardless of the roadway design speed. For a 3/4-inch minimum PPC overlay, provide a 16-feet minimum transition length. For a 1/4-inch TPO overlay, a 3-feet minimum transition length is sufficient. See Chapter 40 Standards for additional guidance.

40.5.7 Past Bridge Deck Protective Systems

In the past, several bridge deck protective systems have been employed on the original bridge deck or while rehabilitating the existing deck as described in 17.8. The following systems have been used to protect bridge decks:

- Epoxy coated deck reinforcement – Prior to the 1980’s, uncoated (black) bars were used throughout structures, including bridge decks. Criteria for epoxy coated reinforcement was first introduced in 1981 as a deck protective system. At first, usage was limited to the top mat of deck reinforcement. By 1987, coated bars were required in the top and bottom mats for high volume roadways (ADT > 5000). By 1991, coated bars were required for all State bridges and on some local bridges (ADT > 1000). Currently, use of epoxy coated deck reinforcement is required on all bridge decks.

- Asphaltic overlay with Membranes – Use of this overlay system was largely discontinued in the 1990’s.

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.

- Thin Polymer Overlays – Use of this overlay system is currently being used.

- Polyester Polymer Concrete Overlays – Use of this overlay system currently being used limitedly.

- Additional Concrete Cover – Use of additional clear cover (> 2 ½ inches) has been used on bridges with high volume and high truck traffic.

- Stainless steel deck reinforcement – Use of stainless steel has been very limited.

- Fiber reinforce polymer (FRP) deck reinforcement - Use of FRP reinforcement has only be used for experimental purposes.

As-built plans should be reviewed for past deck protective systems to assist with the appropriate rehabilitation measures.

40.5.8 Railings and Parapets

Overlays may decrease the parapet height when the existing overlay is not milled off and replaced in-kind. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

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. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges (does not include local roadways over STN routes) eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*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 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 FDM 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. Existing concrete diaphragms, in good condition, that are full-depth to the bottom of the girder (typically located at the abutments and piers) shall not be removed for a deck replacement.

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.

For staged deck replacement projects, temporary overhangs supporting traffic operations shall be evaluated by the designer. When temporary support is determined necessary (i.e. when the existing temporary exterior girder is unable to reasonably support the temporary overhang condition), the designer should consider either reducing the overhang by modifying the traffic operations or provide a Temporary Support SPV bid item. Note: the bid item Temporary Support is intended to be used when the designer has determined there is a viable path forward through a temporary support system and is not intended to be used for typical overhang falsework stabilization, which is covered in Section 502.3.2.3 of the *Standard Specifications* and incidental to the concrete bid item. Nonetheless, the contractor is generally responsible to provide temporary support during construction.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 36”, 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 36”, 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 $0.75f_{pu}$,
- 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

36" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	76	82
6’-6”	74	80
7’-0”	69	78
7’-6”	66	76
8’-0”	65	75
8’-6”	63	69
9’-0”	62	67
9’-6”	60	65
10’-0”	59	64
10’-6”	58	63
11’-0”	51	61
11’-6”	50	60
12’-0”	49	58

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	102	112
6’-6”	100	110
7’-0”	98	108
7’-6”	96	102
8’-0”	94	100
8’-6”	88	98
9’-0”	88	96
9’-6”	84	90
10’-0”	84	88
10’-6”	82	86
11’-0”	78	85
11’-6”	76	84
12’-0”	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	130	138
6’-6”	128	134
7’-0”	124	132
7’-6”	122	130
8’-0”	120	128
8’-6”	116	124
9’-0”	112	122
9’-6”	110	118
10’-0”	108	116
10’-6”	106	112
11’-0”	102	110
11’-6”	100	108
12’-0”	98	104

70" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	150*	160*
6’-6”	146*	156*
7’-0”	144*	152*
7’-6”	140*	150*
8’-0”	138*	146*
8’-6”	134*	142*
9’-0”	132*	140*
9’-6”	128*	136
10’-0”	126*	134
10’-6”	122	132
11’-0”	118	128
11’-6”	116	126
12’-0”	114	122

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. 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 life-cycle 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 Replacement

Various types of superstructure replacements include replacing prestressed girders in-kind, replacing slabs in-kind and replacing steel girders with prestressed girders or slabs. When considering replacement of a deck on steel girders, consideration of the cost of painting the structural steel should be included in the evaluation.

Approval is required from BOS for all superstructure replacement projects. To ensure that the cost of a superstructure replacement is warranted, the substructure should be in good condition. In general, the superstructure replacement should remain the same as the original design to better ensure that substructure reuse is practical. See 40.10 for considerations regarding substructure reuse criteria.

WisDOT policy item:

Provided that the substructure meets the criteria in 40.10, the superstructure may be replaced. The superstructure shall be designed to current LRFD criteria.

Reuse of the existing substructure is contingent on the fixity of the substructure units remaining the same. If the fixity is changed, the substructure must be evaluated per the design loading of the original structure.

With the substructure needing further evaluation for increased dead load and/or change in fixity, discuss with BOS the acceptability of the evaluation results prior to continuing with final design.

40.10 Substructure Reuse and Replacement

When practical, substructure reuse may be an acceptable alternative to replacing the entire bridge. However, reuse will require early coordination with BOS, engineering judgement, and will be evaluated on a project-by-project basis. This evaluation should determine if the substructure can be reused "as-is" with or without minor surface repairs, reused with major repairs and/or strengthening, or needs to be replaced.

In general, "as-is" reuse of substructures should be reserved for in-kind superstructure replacements with little to no change in geometry, fixity, and service dead loads. Additionally, substructures should be in good condition and only require minor surface repairs. If satisfied, evaluation of the existing substructure with the load rating methodology as discussed in 45.3.2 for an existing (in-service) bridge (e.g. LFR) may be acceptable. An example of this condition would be an in-kind slab superstructure replacement with a substructure that remains in good condition. For other conditions (i.e. reuse with major repairs and/or strengthening), the substructure should be evaluated with the current load rating methodology (LRFR) as discussed in 45.3.1.1 for new bridge construction. If substructure reuse is found to be not practical due the expensive repairs and/or excessive strengthening, the substructure should be completely replaced.

<u>Approval is required from BOS for all substructure reuse projects.</u>

Normally it is acceptable to assume that the original bridge design was done correctly, however pier caps, either for multi-columned piers or open pile bents, have occasionally been under-designed. Further investigation is warranted for pier caps with nominal shear stirrups, rather than stirrups that appear to be designed for the girder configuration, etc.

See 40.15 for more information on substructure inspection.

Additional guidance regarding substructure reuse can be found in the FHWA publication *Foundation Reuse for Highway Bridges*.

40.10.1 Substructure Rehabilitation

Substructure rehabilitation work can vary significantly from minor concrete surface repairs to major repairs that includes strengthening members.

40.10.1.1 Piers

Pier caps and/or columns/shafts may show signs of distress due to spalled concrete. The spalling may be completely around some of the longitudinal bar steel, thus destroying the bond. The concrete usually remains sound under the bearing plates, possibly due to compressive forces preventing salt intrusion and/or deterioration from freeze thaw cycles.

If the bond of the structural reinforcement is <u>not</u> compromised (at least half of the bar is bonded), rehabilitation measures include:

1. Concrete Surface Repair. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required for areas larger than nominal. See 40.20 for more information on FRP.

If the bond of the structural reinforcement is compromised (at least half of the bar is not bonded), rehabilitation measures include:

1. Concrete Surface Repair, with an option for Shotcrete for larger areas. Anchors may be required if depth of repair is excessive. Replacement reinforcement (or mesh) may be required if excessive rebar loss. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

40.10.1.2 Bearings

Bearings being replaced should follow the Chapter 27 Standard Details, as well as the Chapter 40 Standard for Expansion Bearing Replacement Details. Replace lubricated bronze bearings with either laminated elastomeric bearings (preferred, if feasible) or Stainless Steel TFE bearings. If only outside bearings are replaced, the difference in friction/resistance values between adjacent girders can be ignored. In addition to the bid item for the new bearing, the STSP Removing Bearings is required.

For bearings requiring maintenance, consider the SPV Cleaning and Painting Bearings. Special Provisions Bearing Maintenance and Bearing Repair may also be worthy of consideration.

For bearings requiring jacking, provide girder reactions on the plans for informational purposes. See Standard Detail 40.10 - Concrete Bearing Block Details - for additional information.

40.11 Other Considerations

40.11.1 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.

40.11.2 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.

40.11.3 Repairs to Prestressed Concrete Girders

Repairs to prestressed concrete girders over traffic lanes should provide measures to protect against concrete patches from coming loose. After repairing the concrete section, non-structural FRP is often used to confine and protect the repair area. Other measures to positively connect concrete patches, beyond the bond strength of the two surfaces, may be considered.

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. 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.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

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. Utilize HSIS data to flag potential scour concerns (code 6000), with scour defects in condition state 4 being a significant concern.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Original pile capacities are determined from plans, or if available, the pile driving records. Reuse of steel pile sections will require checking the remaining load carrying capacity if section loss is determined to be present. Steel piling should be checked:

- Immediately below the splash zone or water line for deterioration and possible loss of section. High section loss occurs in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line.

- Below abutments where the berm soil (material beneath riprap*)* has settled below the abutment bottom and water appears to be flowing from beneath the abutment or stream water has direct access to the piling.

If there is piling section loss or undermined spread footings, capacities of existing piling and/or footings will need to be recomputed for load rating purposes.

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. Generally, timber substructures are not good candidates for substructure reuse due to their limited service life.

Bearing condition needs to be evaluated. When possible, replacement expansion bearings should be laminated elastomeric bearings. Replacing expansion devices to reduce chloride infiltration is often warranted.

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. (Refer to **LRFD [5.13]** for current AASHTO guidance)

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:

<u>Usage Restrictions</u>:

- 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 40.16-1 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.

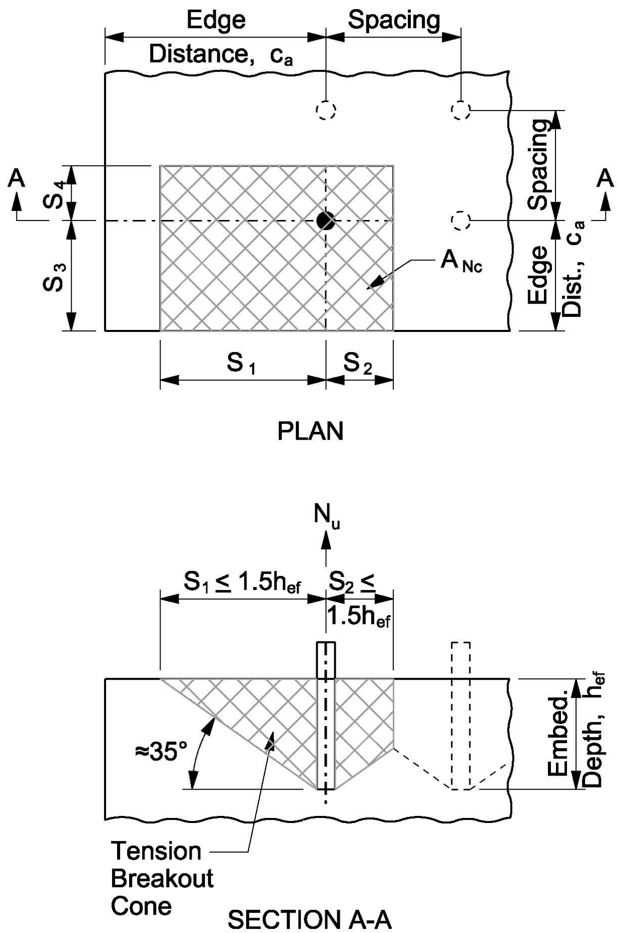

Figure 40.16-1

Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

S_i = 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 40.16-2 shows the bond failure mechanism for concrete adhesive anchors in tension.

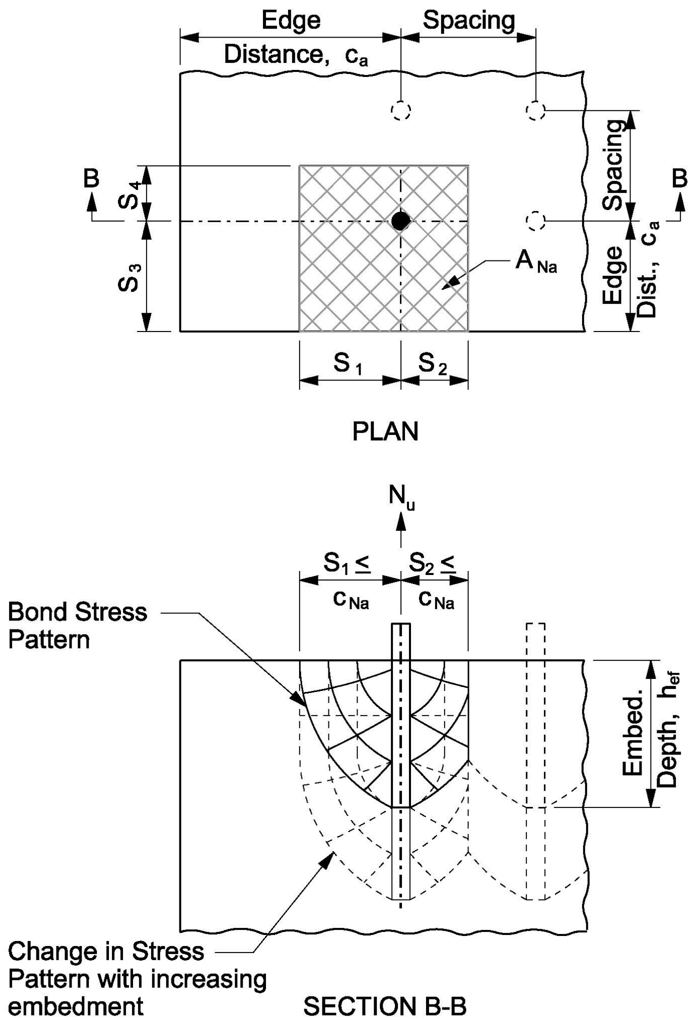

Figure 40.16-2
Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. 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 = Minimum of:

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

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	670	370	280
#5 or 5/8”	970	720	510	410
#6 or 3/4"	950	580	500	420
#7 or 7/8”	930	580	490	420
#8 or 1”	770	580	600	490

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:

ϕ_{ts} = Strength reduction factor for anchors in concrete**, ACI [17.3.3]**
= 0.65 for brittle steel as defined in 40.16.1.1
= 0.75 for ductile steel as defined in 40.16.1.1

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

= $A_{se,N}\, f_{uta}$

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

f_{ya} = Specified yield strength of anchor steel (ksi)

f_u = Specified minimum tensile strength of anchor steel (ksi)

f_{uta} = Specified tensile strength of anchor steel (ksi) and not to exceed $1.9f_{ya}$ or 125 ksi, **ACI [17.4.1.2]**

= min (f_u, 1.9 f_{ya}, 125 ksi)

ϕ_{tc} = 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

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

$$= \frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1
= $(S_1 + S_2)(S_3 + S_4)$

h_{ef} = 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.

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

= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\Psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\Psi_{cp,N}$ = 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 $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

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]**

= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (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

<u>For adhesive anchors:</u>

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

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

= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

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.

≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = 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 $c_{a,min} \geq c_{ac}$

= $\dfrac{c_{a,min}}{c_{ac}} \geq \dfrac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 or Figure 40.16-2 (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]**

= $\dfrac{A_{Na}}{4c_{Na}^2}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$

A_{Na} = Projected influence area of a single adhesive anchor, see Figure 40.16-2

= $(S_1 + S_2)(S_3 + S_4)$

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

= 1.0 if $c_{a,min} \geq c_{Na}$

= $0.7 + 0.3\dfrac{c_{a,min}}{c_{Na}}$ if $c_{a,min} < c_{Na}$

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

= $10d_a\sqrt{\dfrac{\tau_{uncr}}{1100}}$ (in)

τ_{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]**

= 1.0 if $c_{a,min} \geq c_{ac}$

$$= \frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

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

= $\tau_{cr} \pi d_a h_{ef}$

τ_{cr} = Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1

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]**:

$0.50\ \phi_{tc} N_{ba} \geq N_{ua,s}$

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. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

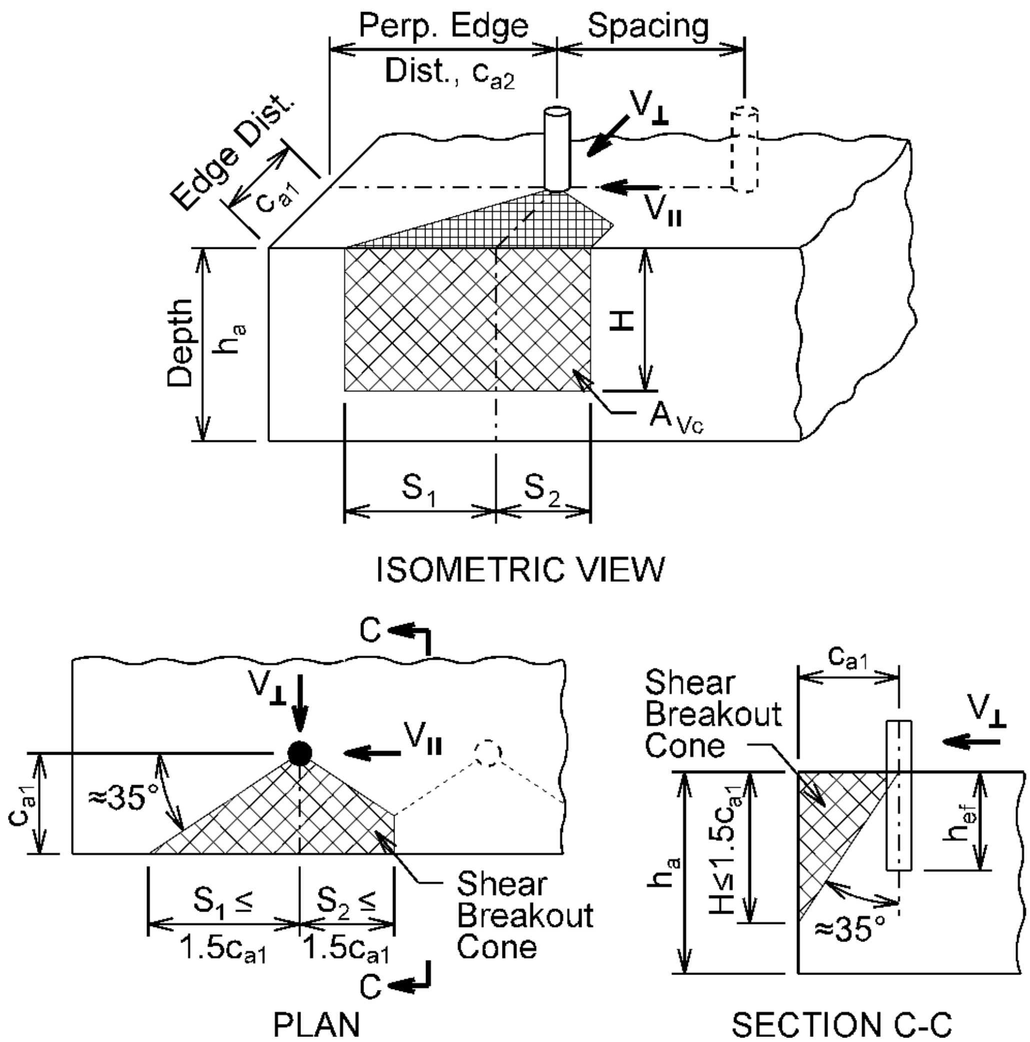

Figure 40.16-3
Concrete Breakout of Concrete Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, 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.

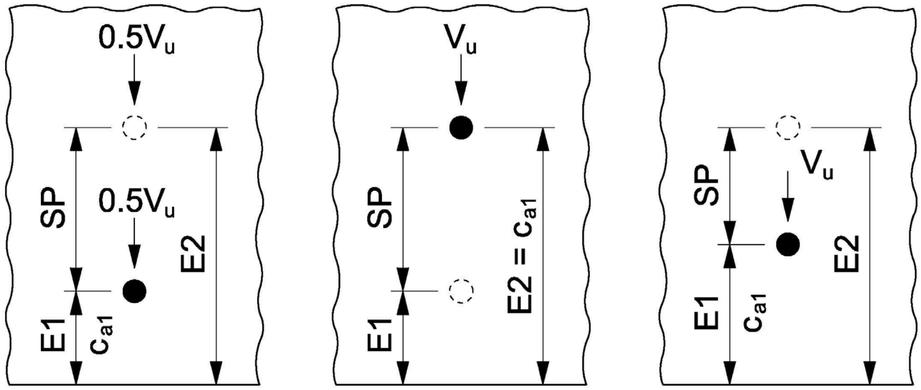

Figure 40.16-4
Concrete Anchor Shear Force Cases

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:

ϕ_{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]**
= $0.6 A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{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]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

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

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\Psi_{c,V}$ = 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 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 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 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 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

$\Psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.16-3 (in)

$\psi_{p,V}$ = 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 40.16-3
= 2.0 for shear parallel to the concrete edge, see Figure 40.16-3

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:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

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

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{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]**
= $2.0N_{cp}$

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

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

adhesive anchors: $$\frac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$$

and

$$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

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.

<u>Typical notes for bridge plans (shown in all capital letters):</u>

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”.

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

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0	6.5	#5 @ 8"	#4 @ 8.5"	#5 @ 7.5"
4-3	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-6	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-9	6.5	#5 @ 7"	#4 @ 7.5"	#5 @ 7.5"
5-0	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-3	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-6	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
5-9	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
2-3	7	#4 @ 9"	#4 @ 11"	#5 @ 6.5"
2-6	7	#4 @ 8.5"	#4 @ 11"	#5 @ 6.5"
2-9	7	#4 @ 8"	#4 @ 11"	#5 @ 6.5"
3-0	7	#4 @ 7.5"	#4 @ 11"	#5 @ 6.5"
3-3	7	#4 @ 7"	#4 @ 11"	#5 @ 6.5"
3-6	7	#4 @ 6.5"	#4 @ 11"	#5 @ 6.5"
3-9	7	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6.5"
4-0	7	#4 @ 6"	#4 @ 10"	#5 @ 6.5"
4-3	7	#5 @ 9"	#4 @ 9.5"	#5 @ 7"
4-6	7	#5 @ 8.5"	#4 @ 9"	#5 @ 7"
4-9	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
5-0	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
4-3	7	#5 @ 7.5"	#4 @ 8"	#5 @ 7"
5-6	7	#5 @ 7'	#4 @ 7"	#5 @ 7"
5-9	7	#5 @ 7"	#4 @ 7"	#5 @ 7"
6-0	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-3	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-6	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
6-9	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
7-0	7	#5 @ 6"	#4 @ 6"	#5 @ 6"
4-0	7.5	#4 @ 7"	#4 @ 10.5"	#5 @ 6"
4-3	7.5	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6"
4-6	7.5	#4 @ 6.5"	#4 @ 10"	#5 @ 6"
4-9	7.5	#4 @ 6"	#4 @ 10"	#5 @ 6"
5-0	7.5	#5 @ 9"	#4 @ 9.5"	#5 @ 6"
5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"

5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

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:

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

Where:

R_r = factored resistance computed in accordance with AASHTO LRFD Section 5

η_i = 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 ε_{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. <u>It is important to note that the FRP capacity listed on the plans will be a factored capacity</u>, 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

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