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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapter 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.

40.2 History

40.2.1 Concrete

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

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

40.2.2 Steel

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

40.2.3 General

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

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

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

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

40.3 Bridge Replacements

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

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

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

Wisconsin DOT has established minimum roadway widths for bridges to remain in place on Rural, State and County Trunk Highways. As a minimum, bridge replacement is required for all bridges less than 100 ft. long and the useable width of the bridge is less than the following:

Design ADT	Rural Arterial Typically STH	Rural Arterial Typically CTH	Town Road
0-250	22’	20’	18’
251-750	22’	20’	22’
751-2000	Traveled Way + 2’	Traveled Way + 2’	Traveled Way + 4’
2001-4000	Traveled Way + 4’	Traveled Way + 4’	Traveled Way + 4’
Over 4001	Traveled Way + 6’	Traveled Way + 6’	Traveled Way + 4’

Table 40.3-1
Bridge Clear Width Rehabilitation Requirements

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

40.4 Rehabilitation Considerations

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

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2' shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required unless it is waived for safety and public interest.

WisDOT policy item:

The Bridge may be rated using LFR or LRFR provided it was designed using ASD or LFD. However, if the rating factor (RF) for the LRFR rating falls below 1.0, use LFR to avoid possible posting issues. Bridges designed using LRFD must be rated using LRFR.

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

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

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

Options for deck rehabilitation are as follows:

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

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
   a. Deck condition equal 4 or 5 and;
   b. Wear course or wear surface less than or equal to 3.
   c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
   a. No previous work in last 10 years or;
   b. Deck Condition less than or equal 4.
   c. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
   a. Deck rehab work less than 10 years old.
   b. Deck condition greater than 4.
   c. Wear surface or wear course greater than or equal 4.

4. All Bridges

WisDOT policy item:

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

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

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

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

5. All Bridges with Roadway Work

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

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

40.5 Deck Overlays

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

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

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

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

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

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

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

40.5.1 Guidelines for Bridge Deck Overlays

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

- The structure is capable of carrying the overlay deadload;
- The deck and superstructure are structurally sound;
- The desired service life can be achieved with the considered overlay and existing structure;
- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

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

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

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1 1/2".
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

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

AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

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

- Minimum thickness is 1 1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.

- Combined distress area is less than 25%.
- May require crack sealing the following year and periodically thereafter.

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

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.
- AC overlays with a waterproofing membrane can also be used on new decks or older decks that are in good condition as preventive maintenance.

40.5.4 Special Considerations

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

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

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

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

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

Table 40.6-1

Condition Requirements for Deck Replacements

When the structure is a continuous steel girder bridge and meets criteria for deck replacement, but has an Inventory Rating less than HS10, a bridge replacement is recommended. On all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20, after the deck is replaced.

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

See the *Facilities Development Manual* for anchorage/offset requirements for temporary barrier used in staged construction.

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.

40.7 Rehabilitation Girder Sections

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

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

The 45”, 54”, and 70” girders in Chapter 40 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 – Superstructures - General

a future wearing surface of 20 pfs.

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

Haunch height = 2” or 2 ½”

Required f'_c girder at initial prestress < 6,800 psi

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"	138	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. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, the total deck should be replaced 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 sections designed to LRFD or the sections from Chapter 40 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 AASHTO 3.6.5 (400 kip loading) as a widening is considered rehabilitation. *It is intended to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

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.

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3’ or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab. Approval is required from BOS for all Superstructure replacement projects. Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading).

Evaluate the piers using current LRFD criteria, including the 400 kip impact loading. Unlike widenings, this evaluation is required since the entire superstructure is being replaced. It is unlikely that many piers could be re-used for this category of rehabilitation.

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

The superstructure shall be design to current LRFD criteria.

40.10 Replacement of Impacted Girders

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

For the parapet or railing, the designer should match the existing.

40.11 New Bridge Adjacent to Existing Bridge

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

The pier design shall be to current LRFD criteria, including the 400 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 400 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 400 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 400 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

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

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

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

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

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

40.13 Survey Report and Miscellaneous Items

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

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 special provisions.

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 for recommended railings.

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

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

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

Existing steel expansion devices shall be modified or replaced with Watertight Expansion Devices as shown in Bridge Manual Chapter 28. 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 down hill 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.

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 it's load carrying capacity, heat straightening is a viable option in many cases.

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

40.15 Substructure Inspection

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

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

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

40.15.1 Hammerhead Pier Rehabilitation

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

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

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

ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

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

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

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

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

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

2. Place wire mesh around shaft.

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

40.15.2 Bearings

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

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

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

40.16 Concrete Masonry Anchors for Rehabilitation

"Type S" anchors are either mechanical wedge or epoxy anchors for installing studs, rebar, or bolts, nuts and washers of a designated size. They are primarily used for anchoring bolts for attaching rail posts or other bolted objects and smaller size rebars or bent rebars. Type S mechanical wedge anchors are seldom used for bridge rehabilitation. The two-part epoxy is either mixed and poured into a drilled hole or pumped into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. The hole must be filled with a sufficient amount of epoxy so that the hole is completely filled with epoxy when the rebar or bolt is inserted. The drilled hole diameter should be 1/4" larger than the bar or bolt size. (7/8" for a #5 rebar). Design allowable pullout strength is to be 25% of ultimate pullout. Because of creep, shrinkage and deterioration under load and freeze-thaw cycles, "Type S" anchors should not be used in situations where the rebar experiences a constant tension stress. When "Type S" anchors are used to anchor rebars the rebar is listed in the "Bill of Bars" and paid for under the bid item "High Strength Bar Steel Reinforcement, Bridges".

"Type L" anchors are used to anchor rebars or bolts when the rebar or bolt is subject to continuous loading. "Type L" anchors of adequate length are capable of developing the tension strength of the bar or bolt for indefinite periods of time. When embedded their development length, rebars will develop their ultimate strength with a "Type L" anchor. "Type L" anchors are used for abutment and pier widenings. They can also be used to add additional reinforcement to an existing structure. They can be installed in vertical overhead positions.

"Type L" anchor systems usually consist of a highly reinforced polyester resin component and its catalyst component both enclosed in a sausage like cartridge separated by a thin plastic film. The use of a cartridge polyester resin system is not required for a "Type L" anchor but when detailing, assume that a cartridge resin system will be used. The smallest size of sausage (cartridge) available is 12" long by 0.9" diameter. A sausage cannot be opened or punctured until after it is placed in the drilled hole. The sausage is then opened and its components mixed by spinning the rebar or bolt thru it in the drilled hole. This method of mixing results in more consistency then what is obtained with a typical "Type S" anchor. The smallest size sausage is designed for a number 6 rebar and requires a hole of 1" diameter x 22" deep. Because rebars for Type L cartridge anchors are spun by a drill into the hole and thru the sausage like cartridge, they cannot be bent bars.

The manufacturer and product name of the "Type L" anchor used by the contractor must be on the department's approved product list. "Type L" anchors must undergo additional tests not required for "Type S" anchors to determine loss of strength after numerous freeze-thaw cycles and at elevated temperatures. Tests to determine creep at 600 days under continuous loadings are also required.

Masonry Anchor Size in	Embedment Depth in	Minimum Spacing in	Ultimate Shear Load, kips		Ultimate Pullout kips	
			Wedge Anchors	Epoxy Anchors	Wedge Anchors	Epoxy Anchors
#3 or 3/8”	4	4	4	4	4	7.7
	5	4	4	4	4.5	9.7
#4 or 1/2”	4	5	7.2	7.8	9.2	9.3
	5	5	7.2	7.8	10.0	12.0
#5 or 5/8”	5	7	15.2	11.2	13.2	14.0
	6	7	15.2	11.2	15.2	17.2
	7	7	15.2	11.2	16.0	20.4
#6 or 3/4”	5	8	17.2	18.0	16.0	14.1
	6	8	17.2	18.0	17.2	17.4
	7	8	18.0	18.0	20.0	20.8
	8	8	18.0	18.0	22.0	24.2
#7 or 7/8”	5	9	22.0	23.8	17.2	15.5
	6	9	22.0	23.8	20.0	19.2
	7	9	22.0	23.8	24.0	22.9
	8	9	25.2	23.8	26.0	26.6
#8 or 1”	6	10	26.0	28.6	24.0	25.2
	7	10	27.2	28.6	27.2	29.5
	8	10	30.0	28.6	30.0	33.9

Table 40.16-1

Design Table for Concrete Masonry Anchors, Type S

Minimum anchor edge distance equals one-half of "Minimum Spacing". Ultimate values in Table for Mechanical Wedge Anchors are based on 4 ksi concrete. If used in lower strength concrete, reduce values by the ratios of the concrete moduli of rupture. Ultimate values for Epoxy Anchors are for all concrete strengths greater than 2.3 ksi and vary linearly with Embedment Depth.

Design allowable pullout strength is to be 25% of ultimate specified values. On site field testing will be required on a limited number of the supplied anchors for specified pullout capacity.

Specify on Bridge Plans: CONCRETE MASONRY ANCHOR, TYPE S, NO. 5 BAR HAVING A MINIMUM PULLOUT CAPACITY OF 20 KIPS. EMBED A MINIMUM OF 7” IN CONCRETE.

BID ITEM: CONCRETE MASONRY ANCHORS, TYPE S, NO. 5 BAR. Add the following note to plan for anchored rebars: UNDER THE BID ITEM "CONCRETE MASONRY ANCHORS, TYPE S, ANCHORED REINFORCING STEEL SHALL BE PAID FOR SEPARATELY AS PROVIDED IN SECTION 505 OF THE STANDARD SPECIFICATIONS FOR BAR STEEL REINFORCEMENT".

BID ITEM: CONCRETE MASONRY ANCHORS, TYPE S, 5/8 INCH

Use the above bid item for anchoring bolts or studs. The bolt, nut, and washer or the stud as detailed on the plans is included in the bid item.

Bar Size	Cartridge Diameter in	Hole Diameter In	Grouted Length Using One 12" Cartridge In	*Basic Tens, Development Length In
#6	0.9	1.0	22.2	18
#7	1.125	1.25	18.9	24
#8	1.125	1.25	26.7	32
#8	1.375	1.625	13.8	32
#9	1.375	1.625	16.5	40
#10	1.375	1.625	20.9	51
#10	1.5625	1.75	19.5	51
#11	1.5625	1.75	24.8	63
#11	1.5625	1.875	18.0	63

*From AASHTO LRFD, 5.11.2, fc' = 3.5 ksi

Table 40.16-2
Design Table for Concrete Masonry Anchors, Type L

Specify on the Bridge Plans: CONCRETE MASONRY ANCHOR, TYPE L, NO. 9 BAR, EMBED 2'-9 IN.

To develop the ultimate bar strength a minimum embedment length equal to the development length is required. The contractor will determine the hole size and number of cartridges. Minimum hole diameter shall be as recommended by the manufacturer of the Type L anchor being used. For number 8 bars and smaller, hole diameter is usually ¼ inch greater than bar size.

BID ITEM: CONCRETE MASONRY ANCHORS, TYPE L, NO. 9 BARS

Note: For "Type L" and "Type S" masonry anchors anchoring rebars the reinforcing bar is listed in the "Bill of Bars" and its weight included in the quantities.

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 "Concrete Masonry Overlay Decks".

   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" 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 today's standard of an 0.02'/' cross slope; a cross slope of 0.01'/'/0.015'/' 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

   For all Bituminous Material Overlays:

Surface Preparation for Sheet Membrane Waterproofing	Area of Deck
Sheet Membrane Waterproofing	Area of Deck
HMA Pavement Type E-xx	Check (E-xx) with Region
Asphaltic Material PGxx-xx	Check (PGx-xx) with Region
If Asked for in Structure Survey Report	
Preparation Decks Type 1 & 2	If Blank, call Region
Concrete Masonry, Deck Patching	Use 1/2 Slab Thickness
Sawing Pavement, Deck Preparation, Curb or Joint Repair	If asked for by Region

Table 40.17-1
Quantities for Asphaltic Overlays

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 ¼” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than ¼” 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 References

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

2. *Concrete Sealers for Protection of Bridge Structures, NCHRP Report 244*, December, 1981.

3. *Durable Bridge Decks* by D. G. Manning and J. Ryell, Ontario Ministry Transportation and Communications, April, 1976.

4. *Durability of Concrete Bridge Decks, NCHRP Report 57*, May, 1979.

5. *Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads, NCHRP Report 243*, December, 1981.

6. *Strength of Concrete Bridge Decks* by D. B. Beal, Research Report 89 NY DOT, July, 1981.

7. *Latex Modified Concrete Bridge Deck Overlays - Field Performance Analysis* by A. G. Bisharu, Report No. FHWA/OH/79/004, October, 1979.

8. *Standard Practice for Concrete Highway Bridge Deck Construction by ACI Committee 345, Concrete International*, September, 1981.

9. *The Effect of Moving Traffic on Fresh Concrete During Bridge Deck Widening* by H. L. Furr and F. H. Fouad, Paper Presented 61 Annual TRB Meeting, January, 1982.

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

11. *Discussion of Control of Cracking in Concrete Structures* by D. G. Manning, Concrete International, May, 1981.

12. *Effects of Traffic-Induced Vibrations on Bridge Deck Repairs, NCHRP Report 76*, December, 1981.

40.20 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 40.20-1

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, 20 lbs/ft. Future Wearing Surface. 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.

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