New Jersey Turnpike Authority

P.O. Box 5042, Woodbridge, NJ 07095

March 27, 2025

Document Change Announcement

2007 Design Manual

Structural Live Load and Miscellaneous Updates

DCA2025DM-01

Subject: Revisions to

Section 3 Structures Design, Subsection 3.0 Definitions
Section 3 Structures Design, Subsection 3.2 Bridges

Description of Change:

Design Manual Section 3.2.1 has been revised to include a discussion on Service Life in the context of design. While the "expected life span" of various components was defined in Section 3.8, the intent of that language was to guide engineers as to when specific elements may need to be replaced or rehabilitated, not to serve as design guidance for new bridges. This list in Section 3.8 was being overlooked or misinterpreted with recent projects and it needed to be part of a larger discussion regarding Service Life of the Authority's bridges. The list of expected life spans of the various structural components located in Section 3.8 has been deleted, and the information from the list was used to create a table of "anticipated intervals" in Section 3.2.1.1. This DCA also clarifies the application of the TP-16 vehicular live load model, instructs the design engineer to avoid the use of center work zones, and allows the design engineer to use the "traditional method" per AASHTO LRFD BDS for desk design for deck replacement work for existing bridges.

Notice to New Jersey Turnpike Authority Staff and Design Consultants

Effective immediately, all contracts currently in the design phase shall incorporate the revisions herein. For advertised contracts awaiting the opening of bids this revision shall be incorporated via addendum. Contact your New Jersey Turnpike Authority Project Manager for instruction.

The revisions may be accessed on the Authority's webpage: https://www.njta.com/doing-business/professional-services

Recommended By:	**Approved By:**
(signature on original)	*(signature on original)*
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Distribution: Senior Staff Engineering, Law, Operations Depts., All Prequalified Consultant Firms, File

NOTE: All text herein are REVISIONS, as indicated by the tracked changes, to the latest version of the Design Manual.

Section 3 - STRUCTURES DESIGN

3.0. DEFINITIONS

Definitions as provided below supersede definitions located elsewhere within the NJTA document library and are for the purpose of this Section only. Defined terms where shown in this Section, will have only the first letter capitalized. Where capitalized terms are noted throughout the text but not below, the reader is implicitly directed to either the NJTA Procedures Manual or the NJTA Standard Specifications for the definition of those terms. Terms that are defined below but are not shown with the first letter capitalized in this Section, are provided for general information and for the purposes of enabling uniform nomenclature.

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SERVICE LIFE: The period of time that the bridge is expected to be in operation.

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3.1. PURPOSE AND INTENT

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3.2. BRIDGES

3.2.1. Design Specifications

3.2.1.1. Design

Standard Design Criteria

Except as modified within this Section, the design of all New Bridges and superstructures shall be governed by the latest edition of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design Bridge Design Specifications (LRFD BDS), with current Interims and as modified by Subsection 3.2.2 of this Manual at the time the design contract is awarded.

Project Specific Design Criteria

The Design Engineer is responsible for preparing a Project Specific Design Criteria outlining the date and version of the LRFD BDS used for the design of the Project at the time of Notice to Proceed. Project specific criteria may differ from the guidance within this section of the Manual, where warranted. Large design projects often require multi-year design durations which may span multiple interims to the LRFD BDS. Some of these interims may contain specific items that are relevant and necessary to complete the design. Where changes to the Design Criteria occur either through recommendation of the Design Engineer or at the direction of the Authority, these changes should be documented in the Project Specific Design Criteria throughout the design process.

In addition, the Design Engineer shall also include in this document all design assumptions, external referenced research, and accepted third party design guidance used to progress the design. No design guidance document is all-encompassing and the Design Engineer is ultimately responsible for interpreting guidance documents and exercising good engineering judgment in the execution of the work. Where these interpretations and judgments may affect the ability of future Design Engineers to review the Project work, they shall be published in the Project Specific Design Criteria.

The Project Specific Design Criteria shall be published and submitted as a part of the Phase A submission as described in the Procedures Manual. The Design Engineer is encouraged to consult the Authority Engineer for concurrence when changes or additions are made to this document as the project progresses. Updates made during the course of design of the Project Specific Design Criteria shall again be submitted as a part of the Phase C submission and as a part of the final calculations package submitted with the Phase D submission.

Service Life

The Design Engineer shall ensure components are designed, detailed, and constructed with general durability considerations in mind. In general, the Design Engineer shall consider a 75-year service life for Routine Bridges and a 150-year service life for Major Bridges. The minimum anticipated replacement interval of bridge components and ancillary structures shall be as follows:

Bridge Component / Ancillary Structure	**Anticipated Replacement Interval**
Routine Bridge Decks (Cast-In-Place)	50 years
Major Bridge Decks (Cast-In-Place)	75 years
Superstructure Steel	75 years
Elastomeric Bearings	35 years
HLMR Bearings	60 years
Latex Modified Concrete Overlay	20 years
Asphaltic Overlay	10 years

Joint Sealers	10 years
Deck / Concrete Seal Coats	5 years
Substructure Elements – Routine Bridges	75 years
Substructure Elements – Major Bridges	150 years
Culverts over 20 feet in Span	75 years
Concrete Culverts	50 years
Sign Structures	50 years
Light Standards	20 years

The Design Engineer shall recommend longer replacement intervals for bridge components where feasible and applicable. For any component or structure not listed above, the Design Engineer shall consult with the Authority for guidance.

Foundation Design

The Authority does not permit the use of shallow bridge foundations without prior approval of the Supervising Engineer, Structures. Driven pile or drilled shaft foundations are the Authority's preferred foundation for all bridge piers, abutments and wingwalls. Deep foundation supported bridge elements need not consider differential settlement. Refer to Section ~~-~~6 of this Manual for additional guidance.

3.2.2. Modifications to Current Codes

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3.6 Live Loads (AASHTO)~~ (Except for the design of Modular Bridge Expansion Joints)~~

3.6.1.2 Design Vehicular Live Load~~ (Except for the design of Modular Bridge Expansion Joints)~~

Turnpike Bridges

Except for the design of modular bridge expansion joints, the ~~Design~~ design vehicular live load (Strength I) to be used for all new Turnpike mainline and ramp bridges shall be TP-16 design live load unless otherwise directed by the Authority. Design live load on ~~New Bridges carrying non-Turnpike traffic or~~ U-Turns shall be HL-93 unless otherwise directed by the Authority. Design live load on New Bridges carrying non-Turnpike traffic shall be in accordance with the requirements below.

TP-16 design vehicular live load shall be in accordance with HL-93 loading except as modified below:

- The first sentence of Article 3.6.1.2.3 is changed to:
  - The design tandem shall consist of a pair of 50 kip axles spaced 4' apart.

- The first sentence of Article 3.6.1.2.4 is changed to:
    - The design lane load shall consist of a load of 0.700 klf uniformly distributed in the longitudinal direction.
- The ~~first sentence~~third bullet of Article 3.6.1.3.1~~, third bullet item,~~ is changed to:
    - ~~For negative moment between points of contraflexure, the uniform load specified above shall be considered with the tandem load above.~~For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 100 percent of the effect of two design tandems spaced a minimum of 50.0 ft between the lead axle of one tandem and the rear axle of the other tandem, combined with 100 percent of the effect of the design lane load. The two design tandems shall be placed in adjacent spans to produce maximum force effects.
- Wherever a wheel load is specified, a 25 kip load shall be used.

Refer to the NJTA Load Rating Manual for figures depicting the TP-16 live load model.

The generation of TP-16 loading is based on review of Weigh In Motion (WIM) studies performed at critical points along the Hudson County Extension and the Westerly Alignment. Results have shown that heavy and frequent traffic are common on the Turnpike roadway, which serves as an industrial / commercial / shipping artery for the region. TP-16 was derived to capture the increasingly frequent heavy trucks with tightly spaced axles~~ while also simplifying the design effort on the part of the Design Engineer. Strength based live load modelling for New Bridges has now been functionally reduced to a static lane load and a single moving load of two tightly spaced fixed axles~~.

~~Generally speaking, for positive moment regions, TP-16 generates live loads that are approximately 80% higher for spans under 60' in length and decreases in variance down to approximately 30% at spans lengths up to 200', based on parametric analysis.~~

~~For negative moment regions TP-16 generates peak live loads that are approximately 10% higher for spans under 60' in length and decreases in variance down to parity at spans lengths up to 200', based on parametric analysis.~~

Parkway Bridges

Except for the design of modular bridge expansion joints, the~~The~~ design vehicular live load to be used for all new Parkway mainline and ramp bridges, including Comprehensive Bridge Rehabilitation projects, shall be TP-16 as described above, unless otherwise directed by the Authority. ~~However, t~~The design live load to be used for all new members in Substantial Modifications to existing Parkway bridges shall be HL-93. ~~Wherever~~ For Substantial Modifications, wherever a wheel load is specified, a 20 kip load shall be used. Design live load on New Bridges carrying non-Parkway traffic shall be in accordance with the requirements below.

Bridges Carrying Non-Turnpike and Non-Parkway Traffic
Unless directed otherwise by the Authority, the design vehicular live load on New Bridges carrying non-Turnpike or non-Parkway traffic on local roads shall be as specified by the owner or governing agency. The minimum design live load shall be HL-93.

For interchange projects or other projects requiring extensive coordination with another owner or governing agency, the Design Engineer shall consult with the Authority regarding the design vehicular live load to be used for New Bridges carrying non-Turnpike or non-Parkway traffic over Authority roadways. Live load and other design criteria may be dictated by memorandum of agreement, jurisdictional agreements, or other legally binding agreements between the Authority and other parties.

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3.2.4 Superstructure Design

3.2.4.1 Stringers and Beams

1. General
   The preferred superstructure type shall be steel for both Turnpike and Parkway Bridges. Prestressed concrete superstructure types are not permitted unless approved by the Authority. This is due to poor durability of precast concrete stringer ends in the northeast region as compared to steel stringers. Prestressed Stringers traditionally feature uncoated strands which are both susceptible to corrosion and difficult to monitor when encased in concrete. In addition, prestressed concrete Stringers are not repairable after sustaining damage or decay and exhibit poor vehicular impact resistance. Furthermore, they have been subject to procurement difficulty, and raise concerns associated with repairing or replacing prestressed Stringers damaged during shipping or construction. For these reasons, the use of prestressed concrete has been discontinued. The Authority may allow exceptions on a case by case basis.

   Continuous superstructures should be used where practical and/or required for structural efficiency considerations such as achieving longer or shallower spans. Bridges with multiple simple span arrangements are no longer discouraged. Simple span arrangements, particularly where two span superstructures are considered, generally offer similar materials efficiency against continuous structures and also may offer the additional benefit of eliminating costly and time consuming field splices.

The spacing of Stringers shall be set so that future deck replacements may be made while traffic is maintained for the full number of active lanes on the bridge. The deck replacement shall be assumed to be in any single bay between Stringer centerlines, and provisions shall be made for construction barrier to protect the work area from traffic. ~~-~~In this condition, the full shoulder areas may be used for traffic and no shoulders need to be maintained through the work zone. The use of center work zones, or "cattle chutes", should be avoided. The Design Engineer may consider overbuilding a structure, if needed, to eliminate a center work zone. However, overbuilding can significantly increase the project's overall costs due to additional structure /roadway, ROW, environmental and other costs. The Design Engineer shall weigh these costs against the use of a center work zone and advise the Authority accordingly.

Refer to the provisions of Subsection 3.2.2 of this Manual for additional sizing guidelines relating to fascia beams which may be subjected to impact loading.

2. Composite Construction

~~b. Composite Construction~~

Stringers with a concrete deck slab shall normally be designed as composite structures, assuming that no temporary supports will be provided for the beam during the placement of the permanent dead load. Girders should not be made composite with the above Stringer/deck system.

Preferred shear connectors for standard steel stringer construction shall be end-welded, 7/8" diameter stud. However, 3/4" diameter studs may be advantageous to use on original steel work (pre-1970 build) to effect better penetration of the stud end weld. In addition, 1" diameter studs may be advantageous for use in precast deck panel work where reduction in the number of studs lessens the number of perforations in the new deck and speeds the end stud welding work.

For straight beam bridges, the Design Engineer shall ignore the concrete deck in computing the shear range in regions of negative flexure. The deck and its reinforcement shall not be considered effective in resisting longitudinal stress.

In continuous spans, shear connectors shall be provided through the negative moment areas at a nominal pitch not to exceed 48". The AASHTO LRFD BDS limit of 24" in Section 6.10.10.1.2 is waived.

Under no circumstances shall transverse steel members be made composite with the deck, including, but not limited to, framed-in steel Cross-Beams and end Diaphragm top members. Provisions shall be made in the design to prevent composite action at these members either by providing a physical gap between the deck and transverse members or by furnishing a primer coat only on the steel. Cross-Beams shall only be designed with a physical gap between the steel work and the deck. This provision has been added to reduce shrinkage confinement within the deck created by the shear connectors and therefore reduce early age deck cracking in these regions.

3. Curved Stringers

~~a. Curved Stringers~~

In general, fascia Stringers shall be curved in plan to match the curvature of the bridge fascia unless the mid-ordinate of the curve is so small that the curvature can be accommodated within a consistent slab overhang and the resulting appearance of the fascia is not aesthetically objectionable.

4. Intermediate Stiffeners and Connection Plates

~~b. Intermediate Stiffeners and Connection Plates~~

Transverse intermediate stiffeners for welded plate Girders and Stringers, where required, shall be placed on both sides of the web with a tight fit at the top and bottom of the stiffener. Where stiffeners are used as connection plates for Diaphragms or Cross-Frames, they shall be welded at both the tension and compression flanges. Additionally, connection plates for Diaphragms or Cross-Frames for fascia Stringers shall be bolted to the bottom flange.

5. Welded Details

~~c. Welded Details~~

Field welding of steel is strongly discouraged. Its use should be limited to regions of no or negligible tensile stress only, such as welding of bottom flanges to bearing sole plates and welding of formwork accessories to the top flange of Girders in regions of non-reversing positive flexure. The Design Engineer shall clearly designate limits of exclusion for welding to the top flange of the Girders on the Contract Plans. Field automatic end welding of shear studs is permitted in all regions of the top flange.

Fillet weld sizes as required by design shall be shown on the plans. Refer to AWS A2.4 for proper weld call out geometry and nomenclature.

6. Splices

~~d. Splices~~

Beam elements of up to 120’ have been successfully transported to the site on Authority Projects. Congested regions with tight clearances, such as the Garden State Parkway between Mileposts 120 and 163, will likely require shorter shipping lengths. The Design Engineer is responsible for determining practical shipping lengths for site delivery as a part of their constructability review. Refer to Subsection 3.5 of the Procedures Manual for guidance in preparing the Constructability Report.

For continuous spans, splices shall be placed at locations of dead load contraflexure. For simple spans, splices shall preferably be placed at the outer 1/4 span points.

When a field splice is shown on the plans, provisions for it shall be made in the design by increasing the haunch and underclearance to accommodate the splice plates and bolt heads. Additionally, splice locations should be co-located with flange thickness changes to minimize butt weld requirements.

All field splice locations shall be shown as ‘optional’. The Contractor should be given freedom to omit a splice and transport the member in fewer pieces.

Design Engineers shall locate field splice locations with care. Field splices shall preferably not be made over active lanes as the corresponding bridge construction can be time consuming and disruptive to the operation of the roadway. Erection sequencing of the Girders should also be considered to minimize the use of temporary support towers.

Splices and connections shall be designed and the details and locations shown on the plans. Field splices shall be designed and detailed with ASTM F3125 high-strength bolts. 7/8” diameter bolts are traditionally employed for field splices, however, the consideration of 1” diameter bolts is encouraged as they may provide a substantial reduction in fastener usage. Bolt strengths in excess of 120 ksi may only be used in shear applications and may not be used in direct tension.

Field splicing of flanges shall preferably be performed by matching the capacity of the smaller flange plate to the capacity of the bolted connection. The net cross-sectional area of the flange splice plates shall equal or exceed the cross-sectional area of the smaller flange.

Web splices shall be proportioned to resist the shear capacity of the lesser web plate, accounting for the eccentricity of the bolted connection on either side of the splice.

For the purposes of design, the bolted connections for splices should be proportioned to resist strength loads using only slip critical resistance with the Class B friction coefficient. Where direct shear design is required for the connection, bolts shall be clearly specified in the Plans to have threads excluded from the shear plane of the connection.

The above design criteria will somewhat increase the number of fasteners used for splices. The rationale for this is based both on cost and pragmatism. Overweight loads have been increasingly prevalent on Authority roadways. The nominal cost increase of additional fasteners is outweighed by the benefit of having a capacity matched splice detail. In addition, conversations with contractors have indicated that the cost of field splices lies almost entirely in the initial fit-up of the connection. Once a few initial fasteners are installed to locate the connection plates, the labor cost to fill the remaining holes with fasteners, is negligible as well as the cost of the hardware.

7. Diaphragms and Cross-Frames

~~e. Diaphragms and Cross-Frames~~

End Diaphragms or Cross-Frames shall be provided at all bearing lines regardless of skew. End Diaphragms or Cross-Frames and their connections shall be designed as simple spans between supporting longitudinal members for the effect of dead loads and wheel loads. They shall also be designed with provisions for future bearing replacement jacking loads including full dead and live loads. The End Diaphragms or Cross-Frames and their connections shall be designed to resist the forces listed above in appropriate combinations and shall include an impact factor for live load forces of 1.75. Detailing consideration for jack placement(s) shall be made in the End Diaphragm or Cross-Frame designs.

Diaphragms situated directly adjacent to abutment backwalls or other obstructions shall be designed with provisions for inspection of the back side of the diaphragm and future painting access. At a minimum, the following criteria shall be met for end Diaphragms:

- Open frame configurations should be considered
- At plate girder end diaphragms, reinforced access openings (port holes) shall be provided in the Diaphragm webs. The port hole shall be minimum 18" wide x 24" high and be proportioned so that no portion of the Diaphragm or connected Girders are more than 36" beyond the rim of the port hole opening.
- Plate girder end diaphragms, including flanges and stiffeners, shall provide a minimum clearance of 12" between abutment backwalls or adjacent end diaphragms.

- Where the aforementioned access opening (port hole) is not possible due to design or geometric considerations, minimum clearances of 18" and 24" shall be maintained to the abutment backwalls and adjacent end diaphragms, respectively; and a preferred 30" wide x 24" high (minimum 24" wide x 18" high) opening shall be provided between the bottoms of the Diaphragms and bearing seat areas for inspection access. This may be accomplished by providing a 'painter's notch' block-out in the abutment seat or pier cap.

Intermediate Diaphragms or Cross-Frames shall be provided at spacing not to exceed 25' along the length of any Stringer/Girder. Floor beam spacing shall not exceed 25'. In addition, intermediate Diaphragms or Cross-Frames shall be provided at or adjacent to all changes in Stringer/Girder flange thickness. Where Stringers/Girders are skewed, intermediate Diaphragms or Cross-Frames may be placed continuously or dis-continuously (staggered) along the cross section of the superstructure at skews up to and including 20 degrees. At skews over 20 degrees, intermediate Diaphragms or Cross-Frames may only be staggered Refer to Section 3.2.2 of this Manual regarding modifications to AASHTO Subsection 3.6.5, for additional intermediate Diaphragm and stiffener/connection plate requirements to resist vehicular impact.

Intermediate Diaphragms may be rolled MC18x42.7 or C15x33.9 and shall be used for Stringers/Girders of less than 36" in overall depth.

Intermediate Cross-Frames shall be used for Stringers/Girders greater than 36" in overall depth and shall be either solid rolled or fabricated "I" or "C" shape members, or shall be X type Cross-Frames with top and bottom horizontal members provided. The minimum intermediate Cross-Frame member size shall be L5x5x1/2" angles.

~~.~~This provision has been added to simplify field erection of Diaphragms and Cross-Frames across differently cambered Stringer/Girder.

8. Depth of Stringers and Girders

~~f. Depth of Stringers and Girders~~

Stringers and Girders shall generally be of uniform depth for the full length of the structure, except where changes in depth are absolutely necessary to meet underclearance requirements. Changes in depth shall not normally be made in structures with varying span lengths. Interior Stringers shall be made the same depth as the fascia Stringer(s). The fascia Stringer(s) should be the lowest projecting superstructure elements.

9. Economics of Girders and Stringer Design

~~g. Economics of Girder and Stringer Design~~

Recent research and economic analysis has shown that the material cost of fabricated and erected structural steel only represents about 20% to 30% of the overall per-pound fabricated and erected steel cost included in the Engineer's estimates. Where the Design Engineer can take measures to reduce fabrication and erection complexity, these measures should be given higher priority than attempting to achieve savings by minimizing material usage.

In the design of welded plate Stringers and Girders, consideration shall be given to minimizing fabrication cost by eliminating flange plate cutoffs. In the case of a flange plate cutoff, the fabrication cost of the butt-welded splice must be compared to the material cost of the steel being saved, and also with the consideration that the ends of the Stringers/Girders are where future corrosion is most likely to occur.

10. Flange Plate Welded Butt Splices and Thicknesses

~~h. Flange Plate Welded Butt Splices and Thicknesses~~

Where a change in thickness of a flange plate is made at a welded splice, the thicker plate will be tapered down to the thickness of the thinner plate.

Generally, the change in plate area made at a welded splice should be such that the area of the smaller plate is approximately 50% to 75% of the area of the larger plate. Small changes in plate area at a welded butt splice should be avoided, as the expense of the weldment often exceeds any savings in material. Flange plates at joints and abutments should be proportioned such that an overall thickness loss of 1/4" due to corrosion can be tolerated without reducing the live load capacity of the superstructure.

Regardless of flange plate thickness transitions, it is preferable that the width of flanges be constant within a single field section.

11. Fracture and Failure Critical Members

~~i. Fracture and Failure Critical Members~~

Fracture Critical Members (FCMs) are sometimes necessary to meet geometric needs or traverse over immovable facilities such as roads, railways, or critical utilities. Many bridges within the Authority inventory have FCMs and have served without incident for the duration of their service life. Use of Fracture Critical Members solely for the sake of material economy, however, is strongly discouraged.

Where the Design Engineer has recommended the use of FCMs, the following considerations shall be made during the design and shall be addressed as a part of the Phase A design submission:

- FCMs shall be designed to have a minimum un-factored capacity/demand ratio of 1.5 with the full design live load placed for maximum effect on the FCM. Please note that this is not to be evaluated as a load rating of 2.0, but a fully loaded total capacity divided by total demand calculation assuming Strength 1 load combination load factors.
- FCMs will require full hands-on inspection of all elements per current NBIS requirements. Design Engineers shall consider and provide inspection access facilities on all FCMs that will allow for unencumbered access for inspection staff to physically touch with bare hands all elements and surfaces of FCM components. This may be accomplished via inclusion of walking platforms, tie-off cables for harness climbing, or other methods as approved by the Authority. Closing of lanes of Authority roadways or crossed major roadways should not be required to inspect FCMs.
- Hollow shape FCMs such as framed-in closed steel box Cross-Beam pier caps shall have their interiors painted with an approved 3 coat system in the SAE-AMS-STD-595C color 27925, including members which are externally left as weathering steel patina. This has been specified to promote visibility of nascent cracks and defects within the member. Where practicable, interiors of hollow shape FCMs shall be provided with lighting.
- Failure Critical Members such as non-redundant pier columns shall be designed to have a minimum un-factored capacity/demand ratio of 1.5 with the full design live load placed for maximum effect on the Failure Critical Member. Please note that this is not a load rating of 2.0, but a fully loaded total capacity divided by total demand calculation assuming Strength 1 load combination load factors.

3.2.4.2 Deck Slabs

1. General

   Deck slabs shall be designed on the assumption that permanent stay-in-place (SIP) steel bridge deck forms shall be used with all corrugations filled with foam. Foam filled SIP forms shall be assumed to weigh 5 psf.

   The wheel load for calculating slab bending moments shall be as outlined in Subsection 3.2.2 of this Manual, except for deck joints, which may be 16 kips.

Longitudinal expansion joints shall only be provided where necessary to accommodate transverse expansion on wide structures (e.g. wider than 90') and between parallel bridges. Joints shall preferably be located at the median barrier and shall be no greater than 1" where vehicles are expected to cross over the joint. Open joints between parallel structures shall preferably be 12" to allow adequate room for seismic excitation and access for future maintenance/repair of the parapets. Where open joints between parallel structures are required, adequate clearance for inspecting the fascia of both structures shall be considered. This may be accomplished either by allowing locations for climbing down and viewing the steel work, or verifying that these areas are inspectable via under bridge inspection vehicle.

Corrective deck grinding or micro-milling of the concrete deck has become common practice for the Authority's bridge Projects to improve rideability, and shall be used on all Projects where the bridge deck is new or replaced in its entirety. Refer to Subsection 3.2.2 for additional guidance on this requirement.

Concrete decks shall be sawcut grooved in the longitudinal direction. Sawcut grooving may be chorded in the longitudinal direction where the horizontal radius of curved bridges does not permit continuous longitudinal sawcutting operations. Transverse sawcutting may be permitted in certain instances as directed by the Authority. Design Engineers shall reference the latest Specifications.

The following deck designs shall be used for all bridges that are owned and/or maintained by the Authority, as noted in Subsection 3.2.1:

a. New Bridges (Mandatory) and Deck Reconstruction (Preferred):

For all New Bridges and complete deck reconstruction of existing bridges, one course construction shall be used, consisting of a 10" reinforced HPC slab (10 1/4" with sacrificial surface). Concrete cover for the top reinforcing bars shall be 2 1/2" measured from the top of the slab, not including the 1/4" sacrificial thickness for micro-milling. Galvanized bars shall be used for the top and bottom reinforcement steel mats. The standard reinforcement for the 10" slab shall be #5 bars at 10" minimum spacing for top and bottom bars, in both directions. This design is based on the premise of the Empirical Design methodology in AASHTO LRFD BDS Section 9.7.2. The use of this design inherently requires that the conditions expressed in AASHTO LRFD BDS Section 9.7.2.4 which are not explicitly specified above (i.e. deck slab thickness, concrete cover, reinforcement size/spacing) are strictly followed with the exception that the minimum overhang provision is reduced to 3.0 times the thickness of the slab for all cases. The reinforcement spacing was selected to conform with the typical bridge parapet reinforcement spacing shown in the Authority's Standard Drawings.

Deck overhang widths detailed on bridges shall consider the installation of scuppers and drain pipes/downspouts and shall ideally be proportioned such that drainage appurtenances do not interfere with the Stringer top flanges. The prior guidance to not place drainage appurtenances outside of the fascia Stringer top flanges has been eliminated.

Deck overhangs have traditionally been designed as small as possible to reduce the expense of formwork and additional reinforcement typically required. Deck overhangs should be of adequate size to offer weather protection to the outside face of the fascia Stringers and ideally should be proportioned to be 50% of the adjacent center to center Stringer spacing. This is offered as guidance to the Design Engineer and not a strict mandate. Regardless, all deck overhangs shall be fully designed by the Design Engineer to resist TL-5 level impact loading per the provisions of AASHTO LRFD BDS Section A13.1.

b. Deck Reconstruction (Alternative):

Where the above empirical deck design is found to be impractical for full deck replacement work ~~for~~ at existing bridges, the Design Engineer shall continue to use AASHTO's empirical method with the above noted modifications and reduce the ~~sectional~~ slab thickness as necessary to carry the above design wheel loading and preserve the load carrying capacity of the superstructure. If the empirical method conditions cannot be met, the Design Engineer may utilize the traditional method per AASHTO LRFD BDS Section 9.7.3.

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3.8.4 Structural Inspection Requirements for Rehabilitation and Repair Contracts

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~~34~~ 35 Year ASL Rehabilitation: A separate hands-on bridge inspection shall be performed at the critical locations identified in the Biennial Bridge Inspection Report. If catwalks are present on the bridge, access may be available at the request of the Authority. Challenging terrain may require the use of over-the-side bridge access equipment. Bearing replacements, deck joint reconstruction, and structural steel repairs shall require hands-on inspection to collect detailed measurements and identify conflicts with jacking and repair schemes. The Engineer shall inspect/sound all areas of apparent deterioration on all Routine Bridges, and 10% of all piers for Major Bridges with a minimum of two piers fully sounded. Concrete cores and laboratory analyses may be required to confirm the properties of existing bridge decks and other concrete elements.

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