STRUCTURES - PLANNING AND DESIGN

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I. DESIGN SPECIFICATIONS

Except as modified below, the design of all highway bridges shall be governed by the latest edition of the LRFD Bridge Design Specifications of the American Association of State Highway and Transportation Officials and all current Interim Specifications issued by the Association (referred to as the AASHTO LRFD Specification).

Structural foundations shall be designed by the Service Load method of the latest edition of the AASHTO Standard Specifications for Highway Bridges, with all current interims.

Highway bridges utilizing curved steel girders shall be governed by Part II (Load Factor Design Criteria) of the Guide Specifications for Horizontally Curved Highway Bridges published by AASHTO, with current interims, and all superstructure and substructure elements of these bridges shall be designed in accordance with the Load Factor method of the latest edition of the AASHTO Standard Specifications for Highway Bridges, with current interims.

Existing bridge members shall be evaluated in accordance with the AASHTO Manual for Condition Evaluation of Bridges, 2nd Edition.

Bridges constructed to carry railways shall conform to the latest edition of the Manual for Railway Engineering published by the American Railway Engineering and Maintenance-of-Way Association (AREMA), subject to the requirements of the railroad concerned.

II. MODIFICATIONS TO AASHTO LRFD SPECIFICATION

The following modifications to the AASHTO LRFD Specifications shall apply:

1.3.5 Operational Importance

The Operational Importance strength limit state shall classify all Turnpike mainline and ramp bridges as "important", therefore: $\eta_I = 1.05$. Local roads over the Turnpike shall be designed in accordance with the requirements of the appropriate governing agency.

2.3.3 Clearances

The minimum vertical clearance over the full width of roadways and shoulders of any roadway carrying Turnpike traffic shall be 15 feet to the lowest projection of any structure.

2.5.2.4 Rideability

A ½ inch of the concrete deck slab thickness shall be considered as dead load only and shall not be considered effective in carrying secondary dead loads, live load or impact.

2.5.2.6.2 Criteria for Deflection

The following principles shall apply to deflections:

- When investigating the maximum absolute deflection, all design lanes should be loaded and all supporting components should be assumed to deflect equally
- The live load portion of Load Combination Service I of Table 3.4.1-1 should be used, including the dynamic load allowance, IM
- The live load shall be taken from Article 3.6.1.3.1
- The provisions of Article 3.6.1.1.2 shall apply.

Deflections of all Turnpike bridges shall conform to the following:

- Vehicular load, general......................Span / 1000
- Vehicular and/or pedestrian loads......Span / 1000
- Vehicular load on cantilever arms .....Span / 400
- Vehicular and/or pedestrian loads
  on cantilever arms ....Span / 400

The following additional criteria shall also be followed:

- The span length shall be defined as the distance between supports.
- For the design of continuous beams, the total deflection range shall be limited to Span / 800 to limit the "reversal" or "upward" deflection due to live loading in other spans.

The deflections of bridges carrying State Highways shall be checked in accordance with the current policy of the New Jersey Department of Transportation.

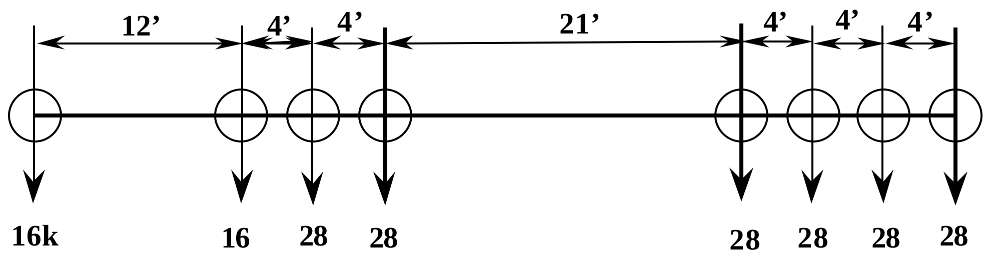
3.5.1 Dead Loads: DC, DW and EV

The dead load for bridges with unsurfaced decks shall include 25-psf additional dead load, applied over the full width of deck between curbs to provide for a future wearing surface. No additional dead load shall be applied to bridges constructed with a wearing surface. The dead load for bridges with reinforced concrete deck slabs supported by stringers shall also include 13-psf applied over the deck slab soffit between stringers to provide for the weight of corrugated metal deck forms and the additional concrete in the form corrugations.

3.6 Live Loads

The highway design live load to be used for all new bridges and for new members in modifications to existing bridges shall be HL-93. Wherever a wheel load is specified, a 20 Kip wheel shall be used.

In addition to the HL-93 analysis, a strength II limit state calculation shall be made for the following NJDOT permit vehicle configuration:



LRFD Permit Vehicle, NJDOT (200 Kips)

The permit vehicle shall occupy one design lane with all remaining design lanes occupied by the HL-93 design vehicle.

Where the loading on existing members is altered by dead load or geometric modifications, the capacity of the member to carry HS-20 live loadings shall be checked in accordance with AASHTO’s Manual for Condition Evaluation of Highway Bridges using Load Factor Design criteria. Members that are computed to be under strength shall either be strengthened, if practical, or replaced.

3.11 Earth Pressure: EH, ES, LS and DD

Earth pressure on structures retaining compacted fills shall be assumed to be the forces from a backfill of cohesionless soil with a unit weight of 120 pounds per cubic foot and an angle of internal friction of 33 degrees 15 minutes. Where structures retain cut soil faces, the value of the unit weight, cohesion and angle of internal friction shall be determined from borings, but in no case shall the forces used for design be less than those given above for fills. If a structure is prevented from deflecting freely at the top, the computed earth pressure forces shall be based on at-rest earth pressure in accordance with AASHTO LRFD Section 3.11.5.7, but in no case shall this value be less than 50 percent greater than the active earth pressure. This condition may occur in concrete rigid frame bridges, culverts and U-type abutments.

The earth pressures due to the retained soil shall be computed using Rankine’s theory if the surface of the soil is plane and the surcharge can be assumed to be a uniform loading over the soil surface. A trial wedge analysis shall be performed if these conditions are not met. An analysis using Coulomb's theory with an appropriate value for wall friction may be used to compute earth pressures on an existing structure in order to evaluate the effects of changes in the loading.

3.15 Earthquake Effects: EQ

This section shall not be considered in the design of new structures or the rehabilitation of existing structures. Refer to Section VI of this Chapter for seismic design and retrofit criteria.

4.6.1.2 Structures Curved in Plan

Horizontally curved superstructures not meeting the requirements of AASHTO LRFD Article 4.1.2.1 shall be designed in accordance with Part II (Load Factor Design Criteria) of the Guide Specifications for Horizontally Curved Highway Bridges published by AASHTO, with current interims using an HS-25 design live load vehicle.

4.6.2.2 Beam-Slab Bridges

Replace the 7th paragraph of Article 4.6.2.2.1-Application (Where bridges meet the conditions specified…) with the following:

The dead load considered as supported by the outside roadway stringer or beam shall be that portion of the floor slab from the fascia to the centerline between the outside stringer and the first interior stringer. Curbs, parapets, railings, sidewalks, and safetywalks, if placed after the slab has cured, shall be divided between the outside three roadway stringers in the ratio of 50 percent to the outside stinger, 35 percent to the first interior stringer and 15 percent to the second interior stringer. Where there is an open joint in the median, the dead load of the median barrier or raised median shall be distributed in the same manner as for fascia stringers. Where the deck slab is continuous through the median, the dead load of median dividers or barriers shall be apportioned between the stringers assuming the slab to act as a simple span between stringers. Wearing surface shall be considered to be carried by the stringer or beam carrying the slab on which it is laid.

5.4 Material Properties

The average annual ambient relative humidity shall be taken as 70%.

5.4.4 Prestressing Steel

The following is added:

Low relaxation strands shall be used and accounted for in the design of prestressed concrete beams.

5.9.4.2.2 Tension Stresses

Article 5.9.4.2.2 shall be replaced by the following:

For service load combinations that involve traffic loading, tension stresses in members with bonded or unbonded prestressing tendons should be investigated using Load Combination Service III and the tension in the precompressed tensile zone shall be zero. Stress Limit limitations stated in AASHTO LRFD Bridge Design Specification Table 5.9.4.2.2-1 shall be accounted for in the permit vehicle check.

5.12.3 Concrete Cover

Article 5.12.3 shall be replaced by the following:

The minimum clear cover for all reinforcement shall be two inches except as given below:

1. Concrete permanently in contact with earth: 3 inches
2. Concrete exposed to salt or brackish water:

Piers and abutments:	4 inches
Walls and culverts:	3 inches

3. Concrete in piers and abutments exposed to flowing water other than the above: 3 inches

4. Concrete deck slabs:

Top Reinforcement:	See Section IV-B
Bottom Reinforcement:	1 inch

In piled foundations, reinforcement or supports for reinforcement shall be positioned a minimum of three inches clear from the piles.

6.4.1 Structural Steels

AASHTO M270, Grade 50W (ASTM A709, Grade 50W) shall be used for all exposed structural steel unless other wise directed. Preliminary analyses shall investigate the possible use of AASHTO M270, Grade HPS70W in hybrid girders as outlined in Subsection IV.

6.7.2 Dead Load Camber

In computing cambers, the weight of the concrete deck slab shall include the permanent metal deck forms and the concrete contained in the forms.

In determining cambers in bridges containing overlays, the weight of the overlay shall be taken as a superimposed dead load in computing deflections of the steel section acting compositely with the first course concrete slab.

Steelwork shall be cambered to compensate for the weight of utilities. The utility dead load shall be taken by the steel section only.

These instructions shall apply unless it is known that the construction method will be such as to make them inappropriate.

III. MATERIALS

A. Structural Steel

Structural steel shall comply with AASHTO M270, Grade 50W (ASTM A709, Grade 50W) unless otherwise approved by the Authority. Hybrid designs incorporating Grade HPS70W, in bottom flanges and negative moment top flanges with conventional Grade 50W Steel in positive moment top flanges and all webs, should be evaluated on a case by case basis to economize structural steel designs.

All structural steel within a distance of 1½ times the depth of girder from a bridge joint shall be painted. No other steel requires painting, but the exterior surfaces of fascia stringers, including the underside of the bottom flange, shall be blast cleaned in accordance with SSPC 10.

B. Concrete

Concrete for use in deck slabs for new bridges, widenings and major deck reconstruction shall be High Performance Concrete (HPC) with a minimum compressive strength (f'c) of 4,400 psi unless otherwise directed by the Authority. The concrete strength used for deck slab design using HPC shall be 4,000 psi.

Concrete for use in precast prestressed slabs, box beams and girders shall have a minimum compressive strength at 28 days of 5,000 psi. Higher strengths may be used, if required by the design, where this can be justified on grounds of economy. The required minimum concrete compressive strength at the time of application of the prestress force (f'_{ci}) shall be computed and shown on the plans to the nearest 100 psi and shall not normally be less than 0.8 (f'_c).

Concrete Class A, with a minimum compressive strength at 28 days (f'_c) of 4,500 psi, shall be used for median barriers, barrier parapets, cast-in-place concrete bearing piles and all precast concrete except prestressed precast concrete.

Concrete Class B, with a minimum compressive strength (f'_c) of 4,000 psi, shall be used for deck slab rehabilitations, approach slabs, safetywalks, sidewalks, culverts and for all pier elements.

Concrete Class C, with a minimum compressive strength (f'_c) of 3,500 psi, shall be used for all cast-in-place walls, abutments and footings.

The value of the concrete strength (f'c) to be used for the design of reinforced concrete using Class A, B or C concrete shall be 500 psi less than the specified minimum compressive strength except for deck slabs using high performance concrete.

C. Reinforcement Steel

Reinforcement steel shall conform to the requirements of ASTM Designation A615, Grade 60.

IV. SUPERSTRUCTURE DESIGN

The LRFD bridge design philosophy dictates that bridges are designed for specified limit states to achieve constructability, safety and serviceability goals, while still achieving desired economy, aesthetics and inspectability. The four main limit states are as follows:

- Service Limit State
- Fatigue and Fracture Limit State
- Strength Limit State
- Extreme Event Limit States

In general terms, bridges designed to the LRFD Specifications are not supposed to be specifically weaker or stronger than those designed using the AASHTO Standard Specifications for Highway Bridges, but should provide more uniform levels of safety. The strength limit states were calibrated based upon theories of structural reliability. The other limit states, however, were calibrated based on past and current practices.

The Service Limit State limits stresses, deformations and crack widths under normal service conditions.

The Fatigue and Fracture Limit State limits the allowable fatigue stress range for anticipated stress cycles to control crack initiation and propagation and to prevent fracture during the design life of the bridge.

The Strength Limit State ensures the bridge has the appropriate strength and stability required to maintain its structural integrity under the load combinations that it is expected to experience during its design life.

The Extreme Event Limit States ensure that the bridge is proportioned to resist collapse due to unique occurrences such as earthquake, vessel collision, flood etc.

The following basic equation is the basis of LRFD methodology and must be satisfied for each Limit State:

$\eta\Sigma\gamma_I Q_I \leq \phi R_n = R_r$ Where $\eta = \eta_D\eta_R\eta_I \geq 0.95$ (LRFD Eq. 1.3.2.1-1)

η = A factor relating to ductility, redundancy and operational importance.
η_R = A factor relating to redundancy
η_D = A factor relating to ductility
η_I = A factor relating to operational importance
γ_I = Load factor: A statistically based multiplier
ϕ = Resistance Factor: A statistically based multiplier
Q_I = Force Effect
R_n = Nominal Resistance
R_r = Factored Resistance: ϕR_n

As stated in the LRFD commentary C1.3.2.1, “Ductility, redundancy and operational importance are significant aspects affecting the margin of safety of bridges. Whereas the first two directly relate to physical strength, the last concerns the consequences of the bridge being out of service.” As a result, a structural system designed using LRFD should:

- Be proportioned to ensure the development of significant inelastic deformations prior to failure for ductility.
- Use multiple-load-path and continuous structures for redundancy.
- Establish the importance of the structure with regards to its social, survival and security requirements.

A. Stringers and Beams

1. General

Continuous superstructures should be used whenever foundation conditions and structure layout permit. The most important considerations in this choice are the reduction in the number of deck joints, economy and the possible reduction in structure depth. The effects of differential settlement shall be included in the design. When the settlement cannot be reliably predicted, consideration should be given to the use of pile foundations.

The spacing of stringers shall be set so that future deck replacements may be made while traffic is maintained for the full number of 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 be maintained through the construction zone.

2. Composite Construction

Steel or precast prestressed concrete beams with a concrete deck slab shall normally be designed as composite structures, assuming that no temporary supports will be provided for the beams or girder during the placement of the permanent dead load.

Shear connectors for steel stringers shall be end-welded studs, $^{7}/_{8}$” diameter. Stud heights are dependent on concrete haunch dimensions. Studs shall extend at least 2 inches above the bottom mat of deck slab reinforcement, but the stud head shall be at least 3 inches below the top of slab. Whenever possible, use of the same height stud on any one bridge is preferred.

In continuous spans, the positive moment areas shall be designed with composite sections as in simple spans. The negative moment areas, however, shall be designed as non-composite sections. Shear connectors shall still be provided through the negative moment areas at a nominal pitch not to exceed 24 inches.

Additional shear studs shall be placed in the region of the points of dead load contraflexure as specified in AASHTO LRFD Article 6.7.4.3.

3. Curved Stringers

   In general, outer 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 the normal slab overhang and the resulting appearance of the fascia is not aesthetically objectionable.

4. Intermediate Stiffeners and Connection Plates

   Transverse intermediate stiffeners for welded plate girders preferably shall be single plates fastened to one side of the web plate only. They shall not be placed on the exposed face of exterior beams. Vertical connection plates shall be rigidly connected to the web and both tension and compression flanges. Ordinarily, the attachment shall be by fillet welding along each edge of the connection plate.

5. Welded Details

   Certain miscellaneous details - supports for screed rails and reinforcement, steel deck forms, connection plates, gussets, etc. - shall normally not be welded to members or parts subject to tensile stress. At locations where welding cannot be avoided, the maximum live load stress range at the point of attachment shall be checked in accordance with AASHTO LRFD Section 6.6. The plans shall show clearly the flange areas where no welding is permitted.

   Fillet weld sizes as required by design shall be shown on the plans.

6. Splices

   For span lengths between 120 ft and 150 ft, an optional field splice shall be permitted, preferably located between the $^1/_3$ and outer ¼ points of the span (near dead load contraflexure points for continuous spans). For spans in excess of 150 ft, optional field splices shall be located between each of the $^1/_3$ and outer ¼ points.

   When an optional 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 consistent with flange thickness transitions to minimize butt weld requirements.

   The Contractor should be given freedom to omit a splice and transport the member in fewer pieces. The Contractor may submit alternative designs or locations for the splices, at no extra cost to the Authority, subject to the approval of the engineer, however, the Contractor should not be permitted to introduce a field splice unless absolutely necessary.

Splices and connections shall be designed and the details and locations shown on the plans. Field splices shall be designed and detailed with ASTM A325 high-strength bolts.

7. End Diaphragms

   End diaphragms and their connections shall be designed for the effect of wheel loads which they may be required to support, for the effect of transverse movement due to thermal, wind or seismic forces and for jacking loads required for future bearing replacement. The diaphragms and their connections shall be designed to resist the forces listed above in appropriate combinations.

8. Depth of Stringers

   Stringers, beams 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 or where a change in depth is desirable to enhance the appearance of the structure. Changes in depth shall not normally be made in structures with varying spans. Interior stringers shall be made the same depth as the fascia stringer.

9. Economics of Beam and Stringer Design

   In the design of welded plate girders, consideration shall be given to minimizing fabrication cost by reducing the number of intermediate stiffeners and eliminating flange plate cutoffs. The use of a fully-stiffened web of the minimum permissible thickness is not normally the most economical solution, and greater economy can be achieved with a thicker web and a lesser number of stiffeners. For shallow beams, the use of an unstiffened web may be economical. In the case of a flange plate cutoff, the cost of the butt-welded splice must be carefully assessed in relation to the weight of steel saved.

10. Flange Plate Welded Butt Splices and Thicknesses

   A welded butt splice shall not normally be made in a plate that exceeds two inches in thickness. Where a change in thickness of a flange plate is made at a splice, the thicker plate will be tapered down to the thickness of the thinner plate. In this case, the thickness of the thinner plate shall not exceed two inches. The Engineer shall ensure that plates of a thickness greater than two inches can be obtained in the lengths shown without a splice being required.

   The change in plate area made at a welded splice shall normally not exceed 50 percent of the area of the smaller plate. Small changes in plate area at a welded butt splice should be avoided, as the expense of the weld will probably exceed the savings in material.

B. Deck Slabs

1. General

Deck slabs shall be designed on the assumption that permanent steel bridge deck forms shall be used. The wheel load for calculating slab bending moments shall be 20,000 pounds. The maximum allowable compressive stress due to bending in the concrete shall be 1,200 psi, and the maximum allowable tension stress in the reinforcement steel shall be 24,000 psi.

The following deck designs shall be used for all bridge construction by the New Jersey Turnpike Authority, including roads over the Turnpike that do not carry Turnpike traffic:

(a) New Bridges, Widenings and Major Deck Reconstruction:

One course construction shall be used, consisting of a reinforced HPC slab. Concrete cover for the top reinforcing bars shall be 2½" minimum measured from the top of the slab. Epoxy-coated bars shall be used for the top and bottom reinforcing bar mats. The design shall include 25-psf additional dead load to allow for a future wearing surface and 13-psf for permanent metal deck forms where corrugations match the bar spacing. The design of these decks shall be governed by AASHTO's LRFD Specifications using the applicable load and resistance factors specified in Chapters 3 and 9 of that manual using the "traditional method". The designs shall be in accordance with Table IV-B-1.2 with the specified slab thickness for each effective slab span held as a minimum.

On a case by case basis, the Authority may request the designer to provide a two-course deck slab. When two-course construction is specified, it shall consist of a Class B reinforced concrete base slab and a latex modified concrete (LMC) overlay, 1¼" thick. Concrete cover for the top reinforcing bars shall be 2½" minimum measured from the top of the LMC overlay. Epoxy-coated bars shall be used for the top and bottom reinforcing bar mats. No allowance need be made for a future wearing surface; however, the design shall include 13-psf for permanent metal deck forms where corrugations match the bar spacing. The design of these decks shall be governed by AASHTO's LRFD Specifications using the applicable load and resistance factors specified in Chapters 3 and 9 of that manual using the "traditional method". The designs shall be in accordance with Table IV-B-1.1 with the specified slab thickness for each effective slab span held as a minimum.

(b) Deck Repair / Replacements:

(1) Bridges with existing surfacing

The new deck shall be surfaced to match the existing construction. However, the surfacing shall incorporate membrane waterproofing on the new concrete deck prior to reapplying the overlay surfacing in the case of an asphalt concrete overlay.

Concrete cover for the top reinforcing bars shall be 1½" minimum. Epoxy-coated reinforcing bars shall be used for both the top and bottom reinforcing bar mats. No allowance shall be made for future wearing surface in the design of these slabs. The design loading shall include 13-psf for permanent metal deck forms where corrugations match the bar spacing. The design shall be allowable stress designs in accordance with AASHTO's Standard Specifications using the Working Stress Design method and Tables IV-B-1.3, 1.4 and 1.5, provided that the thickness of surfacing does not exceed six inches. Where this thickness is exceeded, special designs shall be made.

(2) Bridges with Bare Concrete Decks

The new deck slabs shall be one-course Class B concrete slabs. No LMC or other overlay shall be used.

Concrete cover for the top reinforcing bars shall be 2½" minimum. Epoxy-coated reinforcing bars shall be used for both the top and bottom reinforcing bar mats. The design shall include 25-psf additional dead load to allow for a future wearing surface and 13-psf to allow for permanent metal deck forms where corrugations match the bar spacing. The design shall be in accordance with AASHTO's Standard Specifications using the Working Stress Design method and Table IV-B-1.6.

(c) State Highways over the Turnpike

The current design standards of the New Jersey Department of Transportation shall be followed.

2. Reinforcement

For deck slab designs, the Authority's standard shall be epoxy coated reinforcing steel. On a case by case basis, however, the Authority may consider other corrosion resistant reinforcing steel to replace epoxy coated reinforcement or to use as alternative bid item.

Main reinforcement shall be straight continuous bars and the same reinforcement shall be used in the top and bottom of the slab. Longitudinal distribution reinforcement, computed in accordance with the AASHTO LRFD Specification Article 9.7.3.2 for new bridges and AASHTO Standard Specification Article 3.24.10 for deck repair areas, shall be No. 5 bars in the bottom of the slab spaced uniformly between stringers. Longitudinal top reinforcement shall be No. 5 bars at 15 inches, spaced uniformly over the full width of the deck.

Additional longitudinal reinforcement shall be provided in the negative moment region of continuous spans. The additional reinforcement shall not all be terminated at the same location and shall be developed beyond the dead load point of contraflexure.

The quantity of $^7/_8$" diameter plain bars used to support the deck reinforcement on the stringers will be included in the quantity to be measured for payment under the item "Reinforcement Steel" in the Proposal. However, the quantity of weld metal, angles and any other supports for reinforcement steel - such as chairs, bolsters, etc. - will not be measured for payment.

TABLE IV-B-1.1: NEW BRIDGES & MAJOR RECONSTRUCTIONS

SLAB SPAN (S)	1st COURSE SLAB THICKNESS (Actual w/o 1 1/4" LMC)	TRANSVERSE REINFORCEMENT STEEL (TOP & BOTTOM)	LONGITUDINAL REINFORCEMENT STEEL (BOTTOM)
4'- 0" to 6'- 6"	7 1/4"	# 5 @ 7"	# 5 @ 10"
6'- 6 1/2" to 7'- 6"	7 1/4"	# 5 @ 5 1/2"	# 5 @ 8"
7'- 6 1/2" to 8'- 3"	7 1/2"	# 5 @ 5 1/2"	# 5 @ 8"
8'- 3 1/2" to 8'- 9"	7 3/4"	# 5 @ 5 1/2"	# 5 @ 8"
8'- 9 1/2" to 9'- 3"	8"	# 5 @ 5 1/2"	# 5 @ 8"
9'- 3 1/2" to 9'- 9"	8 1/4"	# 5 @ 5 1/2"	# 5 @ 8"
9'- 9 1/2" to 10'- 6"	8 1/2"	# 6 @ 7"	# 5 @ 7"
10'- 6 1/2" to 11'- 3"	8 3/4"	# 6 @ 6"	# 5 @ 6"
11'- 3 1/2" to 12'- 0"	9"	# 6 @ 6"	# 5 @ 6"

(S) is defined as the spacing of beams, centerline to centerline

TABLE IV-B-1.2: NEW BRIDGES & MAJOR RECONSTRUCTIONS (ALT.)

SLAB SPAN (S)	DECK SLAB THICKNESS (Actual)	TRANSVERSE REINFORCEMENT STEEL (TOP & BOTTOM)	LONGITUDINAL REINFORCEMENT STEEL (BOTTOM)
4'- 0" to 5'- 3"	8 1/2"	# 5 @ 8"	# 5 @ 11"
5'- 3 1/2" to 6'- 1"	8 1/2"	# 5 @ 7"	# 5 @ 10"
6'- 1 1/2" to 6'- 10"	8 1/2"	# 5 @ 6"	# 5 @ 9"
6'- 10 1/2" to 7'- 6"	8 1/2"	# 5 @ 5 1/2"	# 5 @ 8"
7'- 6 1/2" to 8'- 3"	8 3/4"	# 5 @ 5 1/2"	# 5 @ 8"
8'- 3 1/2" to 8'- 9"	9"	# 5 @ 5 1/2"	# 5 @ 8"
8'- 9 1/2" to 9'- 3"	9 1/4"	# 6 @ 7"	# 5 @ 7"
9'- 3 1/2" to 9'- 9"	9 1/2"	# 6 @ 7"	# 5 @ 7"
9'- 9 1/2" to 10'- 6"	9 3/4"	# 6 @ 6"	# 5 @ 6"
10'- 6 1/2" to 11'- 3"	10"	# 6 @ 6"	# 5 @ 6"
11'- 3 1/2" to 12'- 0"	10 1/4"	# 6 @ 6"	# 5 @ 6"

(S) is defined as the spacing of beams, centerline to centerline

TABLE IV-B-1.3: RECONSTRUCTION OF SURFACED DECKS (SURFACING 2 INCHES THICK OR LESS)

REINFORCING STEEL		MAXIMUM ALLOWABLE EFFECTIVE SLAB SPAN - FEET						
TRANSVERSE TOP & BOT	LONGITUDINAL BOTTOM	8.00" SLAB	8.50" SLAB	9.00" SLAB	9.50" SLAB	10.00" SLAB	10.50" SLAB	11.00" SLAB
# 5 @ 9.00"	# 5 @ 13.43"	3.62	4.03	4.44	4.84	5.22	5.60	5.96
# 5 @ 8.50"	# 5 @ 12.69"	3.90	4.33	4.76	5.17	5.57	5.96	6.34
# 5 @ 8.00"	# 5 @ 11.94"	4.21	4.67	5.11	5.54	5.96	6.36	6.76
# 5 @ 7.50"	# 5 @ 11.19"	4.56	5.04	5.50	5.95	6.39	6.81	7.22
# 5 @ 7.00"	# 5 @ 10.45"	4.95	5.45	5.94	6.41	6.87	7.31	7.74
# 5 @ 6.50"	# 5 @ 9.70"	5.39	5.92	6.44	6.93	7.42	7.88	8.33
# 5 @ 6.00"	# 5 @ 8.96"	5.90	6.46	7.00	7.53	8.03	8.52	8.99
# 5 @ 5.50"	# 5 @ 8.21"	6.48	7.08	7.66	8.21	8.75	9.26	9.76
# 5 @ 5.00"	# 5 @ 7.46"	7.16	7.80	8.42	9.01	9.58	10.12	10.64
# 6 @ 9.00"	# 5 @ 9.46"	5.47	6.01	6.53	7.04	7.53	8.00	8.46
# 6 @ 8.50"	# 5 @ 8.94"	5.83	6.39	6.94	7.46	7.97	8.46	8.93
# 6 @ 8.00"	# 5 @ 8.41"	6.23	6.82	7.38	7.93	8.46	8.97	9.45
# 6 @ 7.50"	# 5 @ 7.89"	6.67	7.29	7.88	8.45	9.00	9.53	10.04
# 6 @ 7.00"	# 5 @ 7.36"	7.13	7.81	8.44	9.03	9.61	10.16	10.68
# 6 @ 6.50"	# 5 @ 6.84"	7.46	8.41	9.06	9.69	10.29	10.86	11.41
# 6 @ 6.00"	# 5 @ 6.31"	7.83	9.00	9.77	10.43	11.06	11.66	12.24
# 6 @ 5.50"	# 5 @ 5.78"	8.24	9.46	10.59	11.29	11.95	12.58	13.19
# 6 @ 5.00"	# 5 @ 5.26"	8.73	10.00	11.24	12.28	12.98	13.65	14.28

Effective Slab Span as established in accordance with Section 8.1.1 of AASHTO Standard Specifications

TABLE IV-B-1.4: RECONSTRUCTION OF SURFACED DECKS (SURFACING UP TO 4 INCHES THICK)

REINFORCING STEEL		MAXIMUM ALLOWABLE EFFECTIVE SLAB SPAN - FEET						
TRANSVERSE TOP & BOT	LONGITUDINAL BOTTOM	8.00" SLAB	8.50" SLAB	9.00" SLAB	9.50" SLAB	10.00" SLAB	10.50" SLAB	11.00" SLAB
# 5 @ 9.00"	# 5 @ 13.43"	3.58	3.98	4.38	4.77	5.14	5.51	5.86
# 5 @ 8.50"	# 5 @ 12.69"	3.85	4.28	4.69	5.09	5.48	5.86	6.23
# 5 @ 8.00"	# 5 @ 11.94"	4.16	4.60	5.03	5.45	5.86	6.25	6.63
# 5 @ 7.50"	# 5 @ 11.19"	4.49	4.96	5.41	5.85	6.27	6.69	7.08
# 5 @ 7.00"	# 5 @ 10.45"	4.87	5.36	5.84	6.30	6.74	7.17	7.58
# 5 @ 6.50"	# 5 @ 9.70"	5.30	5.82	6.32	6.80	7.27	7.72	8.15
# 5 @ 6.00"	# 5 @ 8.96"	5.79	6.34	6.87	7.38	7.87	8.34	8.79
# 5 @ 5.50"	# 5 @ 8.21"	6.36	6.94	7.50	8.03	8.55	9.05	9.53
# 5 @ 5.00"	# 5 @ 7.46"	7.02	7.64	8.23	8.80	9.35	9.88	10.38
# 6 @ 9.00"	# 5 @ 9.46"	5.38	5.90	6.41	6.90	7.38	7.83	8.28
# 6 @ 8.50"	# 5 @ 8.94"	5.73	6.28	6.80	7.31	7.80	8.28	8.73
# 6 @ 8.00"	# 5 @ 8.41"	6.12	6.68	7.23	7.76	8.27	8.77	9.24
# 6 @ 7.50"	# 5 @ 7.89"	6.54	7.14	7.71	8.26	8.80	9.31	9.80
# 6 @ 7.00"	# 5 @ 7.36"	6.99	7.65	8.25	8.82	9.38	9.91	10.42
# 6 @ 6.50"	# 5 @ 6.84"	7.30	8.22	8.85	9.45	10.03	10.59	11.12
# 6 @ 6.00"	# 5 @ 6.31"	7.66	8.79	9.53	10.17	10.78	11.36	11.91
# 6 @ 5.50"	# 5 @ 5.78"	8.06	9.23	10.32	10.99	11.63	12.24	12.82
# 6 @ 5.00"	# 5 @ 5.26"	8.53	9.75	10.94	11.94	12.61	13.25	13.87

Effective Slab Span as established in accordance with Section 8.1.1 of AASHTO Standard Specifications

TABLE IV-B-1.5: RECONSTRUCTION OF SURFACED DECKS (SURFACING UP TO 6 INCHES THICK)

REINFORCING STEEL		MAXIMUM ALLOWABLE EFFECTIVE SLAB SPAN - FEET						
TRANSVERSE TOP & BOT	LONGITUDINAL BOTTOM	8.00" SLAB	8.50" SLAB	9.00" SLAB	9.50" SLAB	10.00" SLAB	10.50" SLAB	11.00" SLAB
# 5 @ 9.00"	# 5 @ 13.43"	3.54	3.93	4.32	4.70	5.07	5.42	5.77
# 5 @ 8.50"	# 5 @ 12.69"	3.80	4.22	4.62	5.02	5.40	5.77	6.12
# 5 @ 8.00"	# 5 @ 11.94"	4.10	4.54	4.96	5.37	5.76	6.15	6.52
# 5 @ 7.50"	# 5 @ 11.19"	4.43	4.89	5.33	5.75	6.17	6.57	6.95
# 5 @ 7.00"	# 5 @ 10.45"	4.80	5.28	5.74	6.19	6.62	7.04	7.44
# 5 @ 6.50"	# 5 @ 9.70"	5.22	5.72	6.21	6.68	7.13	7.57	7.99
# 5 @ 6.00"	# 5 @ 8.96"	5.70	6.23	6.74	7.23	7.71	8.17	8.61
# 5 @ 5.50"	# 5 @ 8.21"	6.25	6.81	7.35	7.87	8.37	8.85	9.32
# 5 @ 5.00"	# 5 @ 7.46"	6.88	7.48	8.06	8.61	9.14	9.65	10.14
# 6 @ 9.00"	# 5 @ 9.46"	5.30	5.81	6.30	6.78	7.24	7.68	8.11
# 6 @ 8.50"	# 5 @ 8.94"	5.63	6.17	6.68	7.17	7.65	8.11	8.55
# 6 @ 8.00"	# 5 @ 8.41"	6.01	6.56	7.09	7.61	8.10	8.58	9.04
# 6 @ 7.50"	# 5 @ 7.89"	6.42	7.00	7.56	8.09	8.61	9.10	9.58
# 6 @ 7.00"	# 5 @ 7.36"	6.86	7.49	8.07	8.63	9.17	9.68	10.18
# 6 @ 6.50"	# 5 @ 6.84"	7.16	8.05	8.65	9.24	9.80	10.34	10.85
# 6 @ 6.00"	# 5 @ 6.31"	7.50	8.59	9.31	9.93	10.51	11.08	11.61
# 6 @ 5.50"	# 5 @ 5.78"	7.89	9.02	10.07	10.71	11.33	11.92	12.48
# 6 @ 5.00"	# 5 @ 5.26"	8.34	9.52	10.67	11.63	12.28	12.90	13.49

Effective Slab Span as established in accordance with Section 8.1.1 of AASHTO Standard Specifications

TABLE IV-B-1.6: RECONSTRUCTION OF BARE CONCRETE DECKS

REINFORCING STEEL		**MAXIMUM ALLOWABLE EFFECTIVE SLAB SPAN - FEET**						
TRANSVERSE TOP & BOT	LONGITUDINAL BOTTOM	8.00" SLAB	8.50" SLAB	9.00" SLAB	9.50" SLAB	10.00" SLAB	10.50" SLAB	11.00" SLAB
# 5 @ 9.00"	# 5 @ 13.43"	2.75	3.18	3.60	4.01	4.41	4.80	5.18
# 5 @ 8.50"	# 5 @ 12.69"	2.99	3.44	3.87	4.30	4.72	5.13	5.53
# 5 @ 8.00"	# 5 @ 11.94"	3.25	3.72	4.18	4.63	5.07	5.49	5.91
# 5 @ 7.50"	# 5 @ 11.19"	3.55	4.04	4.53	5.00	5.45	5.90	6.33
# 5 @ 7.00"	# 5 @ 10.45"	3.88	4.40	4.91	5.41	5.89	6.35	6.80
# 5 @ 6.50"	# 5 @ 9.70"	4.26	4.81	5.35	5.87	6.38	6.87	7.34
# 5 @ 6.00"	# 5 @ 8.96"	4.50	5.27	5.84	6.40	6.93	7.45	7.95
# 5 @ 5.50"	# 5 @ 8.21"	4.77	5.81	6.42	7.01	7.57	8.12	8.65
# 5 @ 5.00"	# 5 @ 7.46"	5.08	6.17	7.09	7.72	8.32	8.90	9.46
# 6 @ 9.00"	# 5 @ 9.46"	4.12	4.88	5.42	5.96	6.47	6.97	7.45
# 6 @ 8.50"	# 5 @ 8.94"	4.29	5.21	5.78	6.33	6.87	7.39	7.88
# 6 @ 8.00"	# 5 @ 8.41"	4.47	5.49	6.17	6.75	7.31	7.84	8.36
# 6 @ 7.50"	# 5 @ 7.89"	4.66	5.72	6.61	7.21	7.79	8.35	8.89
# 6 @ 7.00"	# 5 @ 7.36"	4.88	5.97	7.06	7.73	8.34	8.92	9.49
# 6 @ 6.50"	# 5 @ 6.84"	5.13	6.25	7.38	8.31	8.95	9.57	10.16
# 6 @ 6.00"	# 5 @ 6.31"	5.41	6.57	7.74	8.89	9.65	10.29	10.91
# 6 @ 5.50"	# 5 @ 5.78"	5.72	6.94	8.15	9.34	10.45	11.13	11.78
# 6 @ 5.00"	# 5 @ 5.26"	6.09	7.36	8.62	9.87	11.09	12.10	12.79

Effective Slab Span as established in accordance with Section 8.1.1 of AASHTO Standard Specifications

3. Concrete Haunch

Where concrete slabs are supported on steel or precast concrete stringers, girders or other beams, the bottom of the concrete slab shall be positioned above the top of the supporting beam so as to provide a concrete haunch. The haunch shall be made deep enough to ensure that the concrete slab can be constructed to the nominal depth shown on the plans and with its top surface at the required profile, without any decrease in slab depth over the beam due to construction tolerances, variation in beam depth, variation in camber, deflection of the beams or other causes. The dimension from the top surface of the slab to the top of the beam shall not be less than the nominal slab plus one-inch. The top of the beam shall normally be set so as to provide the minimum haunch depth over the thickest flange plate, except that for continuous girders, the haunch may be reduced over the interior support where the variability of the elevation of the top of the beam may be expected to be less. Where field splices in the stringers are shown on the plans, or permitted by the Specifications, the haunch shall be a minimum depth of one-inch over the splice plate. Bolt heads may project into the haunch, but one-inch minimum of clear cover shall be maintained between the main steel reinforcement and the bolts.

4. Permanent Steel Bridge Deck Forms

The main reinforcement in the slab shall be so arranged as to be parallel to the direction of the corrugations in the permanent steel bridge deck forms, and the spacing of the bars should be made the same as a commonly-available corrugation pitch of the forms. The main reinforcement shall be positioned over the centerline of a depressed corrugation in the form, and the top surface of the raised corrugation of the form shall be set one-inch minimum clear of the longitudinal (distribution) reinforcement. The concrete cover over the main bars shall be not less than one-inch clear in any direction to the surface of the form. Where it is impracticable to arrange the main reinforcement parallel to the slab corrugations, the level of the steel bridge deck forms shall be dropped so as to provide one-inch of clear cover to the main reinforcement over the top of the corrugations. Where forms are dropped the appropriate dead load allowance for additional concrete in both design and details must be included.

5. Slab Corners

The reinforcing of the acute corners of skewed slabs shall be given special consideration. In these areas, it may be necessary to place the main reinforcement in a fanned arrangement extending into the corner and dropping the permanent steel bridge deck forms as necessary to provide the required cover for the reinforcement as shown in Design Standard Drawing DS-17.

An alternative method for accommodating acute deck slab corners, however, would be to add 7 additional No. 5 bars splayed equally beneath the top layer of reinforcement. These bars would be placed to maintain 2” minimum clearance between bars and would extend sufficiently to overlap the fully developed sections of the main transverse reinforcement. If chosen, this method shall be sufficiently detailed on the plans.

C. Bearings

1. Standard Drawing Types:

Previous standard bearing designs have been made and are detailed on Standard Drawing No. BR-1. These, or similar, designs have been used on Turnpike structures for years and have proved satisfactory in service through that time. However, in light of advances in seismic design and evaluation requirements, these bearings are no longer recommended and shall only be maintained as Standard Drawings for informational purposes regarding past construction techniques and dimensions for future maintenance and repair work.

2. New Designs, Widenings and Retrofits:

Bearings shall be designed in accordance with the appropriate provisions of the AASHTO LRFD Specification. Seismic loadings resisted by bearings and their connections shall be evaluated in accordance with Division IA of the AASHTO Standard Specifications. Elastomeric Bearings are preferred where their use is practical. Pot, disc, sliding, seismic isolation and other alternative bearing types will be evaluated on a case by case basis. Their intended use must be approved by the Authority prior to the Phase C plan submission.

3. Provisions for Substructure Movement

Settlement of fill under and behind abutments is frequently accompanied by horizontal movement of the abutment top, and small rotations of tall piers will result in appreciable displacement of the bearings. In these circumstances, and others where movements or settlements may take place, provisions shall be made in the design for resetting the bearings. The end diaphragm shall be positioned and designed to provide for jacking the end of the span. Sufficient expansion capacity shall be provided in the bearings to accommodate the substructure movement, and so minimize the need to reset them.

4. Provisions for Bearing Replacement

All bearing designs and details shall provide a means for ready removal of the bearing for the purpose of inspection, maintenance and replacement. As an example, the bearing may be placed between steel plates so that removal does not entail the demolition of reinforced concrete substructures. Substructures shall be designed to furnish space for jacks or other devices for temporarily supporting the superstructure. Superstructures shall be designed to accommodate the loads imposed by these devices.

V. SUBSTRUCTURE DESIGN (including Retaining Walls)

A. Piers

1. Restrictions on the use of frame piers

These restrictions apply to all piers adjacent to the Turnpike or Turnpike ramps unless they are positioned more than 30 feet clear from the outer edge of shoulder.

(a) Single shaft piers shall have a minimum horizontal cross-section area of 30 square feet.

(b) Frame piers shall have a minimum of three columns.

2. Footings

Frame piers shall generally be designed with a continuous footing supporting all the shafts, except that piers founded on rock or piles may have individual footings for each shaft. The footing width of piers founded on soil shall be at least one-third of the height (from bottom of footing to top of cap beam) and for piers founded on rock or unyielding soil shall be at least one-fourth the height. Soils with a bearing capacity of at least three tons per square foot may be considered unyielding. If piles are used, the distance between the outer rows shall be not less than one-fourth the height of the pier.

3. Temperature and Shrinkage

Frame piers shall be designed for the combined effects of temperature change and concrete shrinkage, unless a placing sequence is specified for the cap beam that will reduce or eliminate the shrinkage stresses. Open joints in cap beams and footings shall only be used at a corresponding discontinuity in the superstructure.

B. Earth Retaining Structures

1. Foundation Design

Foundations for earth retaining structures shall be designed to satisfy the following conditions:

(a) The ratio of the righting moment of the vertical forces to the overturning moment of the horizontal forces shall exceed 2.0 for soil bearing foundations and 1.75 for pile bearing foundations. The moments are to be computed about the toe or front pile row, respectively.

(b) For soil bearing structures, the ratio of the total horizontal force to the total vertical force shall not exceed the coefficient of friction between the footing and the underlying soil divided by 1.5. The final design value of the coefficient of friction for the soil should be estimated as part of the geotechnical investigation, but may be taken as 0.45 for granular soils as a preliminary estimate. The value for other soils may be lower.

(c) For structures founded on soil, the resultant of all the forces acting on the structure above its base shall intersect the base within the middle third. For structures founded on rock, the resultant shall intersect the base between the quarter points. No uplift shall be permitted on piles except under seismic loadings as provided for by Division IA of the AASHTO Standard Specifications.

(d) The loads on the rear piles of abutments and retaining walls shall be computed with the horizontal forces from the soil reduced to 75 percent of the maximum values. The normal allowable load shall not be exceeded for this condition.

(e) Careful consideration shall be given to the position of the water table, and the stability of the structure against sliding and overturning shall be checked with the water table at its highest normal level, with the factors of safety specified in (a) and (b) above. Where appropriate, the stability shall also be checked for exceptional flood conditions, in which case the factors of safety may be divided by 1.25. In these cases, both the horizontal earth pressures and the vertical dead loads should be reduced for material below the water table. Possible draw down conditions should be considered and forces from the unbalanced hydrostatic head should be added to the overturning moments. Pile loads and soil pressures should be computed with the water table at its lowest normal elevation.

(f) The passive resistance of the soil in front of the structure shall not be considered as resisting either the horizontal forces caused by the earth pressure nor the bending moments in the stem due to those factors.

(g) The resultant horizontal force on each pile shall be computed and shall not exceed the capacity of the pile. The horizontal component of the axial load in the battered piles may be considered as resisting part of the horizontal force. The capacity of the pile to resist unbalanced horizontal force shall be determined from an analysis of the bending forces in the pile taking account of the conditions of fixity of the pile and the modulus of horizontal subgrade reaction of the soil. The combined bending moment and axial load shall be evaluated by the structural design provisions of the AASHTO LRFD and Standard Specifications, as appropriate.

2. Wall Thickness

The minimum thickness of any cast-in-place concrete wall shall be 12 inches for walls up to 10 feet high, 15 inches for walls up to 14 feet high, and 18 inches thick for walls higher than 14 feet. Low walls should be designed with a vertical rear face and higher walls should be battered, with a rear face batter of not less than 1 in 12. Battered faces shall, where possible, be plane, and changes in batter shall be avoided.

3. Backfill

The backfill behind abutments and walls shall consist of a layer of Porous Fill which shall be a minimum of five feet wide, and shall be adequately drained at its base to prevent the build up of water pressure at the back of the structure.

C. Abutments

1. Design Criteria

Abutment foundations shall be sized by the Working Stress Design method in accordance with the AASHTO Standard Specifications. The AASHTO LRFD Specification as modified herein shall govern the design of the abutment's structural concrete.

Abutments shall be designed to conform to the requirements for Earth Retaining Structures (Section V-B of this Manual) and to resist all vertical and horizontal forces from the bridge superstructure and the bridge approach slab. The use of fixed bearings upon abutments should be avoided wherever possible.

2. Approach Slabs

Approach slabs shall be provided for all abutments and shall be constructed for the full width of the roadway including shoulders. Two conditions of support for the approach slab shall be considered in the design of the abutment.

(a) The "as-constructed" condition where the approach slab is supported by the fill as a surcharge load.

(b) The condition where the soil does not provide any support for the approach slab immediately adjacent to the abutment and the slab spans as a beam from the backwall to the soil. The span of the slab shall be assumed to be 25 feet.

3. Construction Condition

Abutments shall be designed for a construction condition where the earth fill is fully placed before the superstructure is erected. The design for this condition shall include a surcharge load for construction equipment. For this condition, the factors of safety given in Section V-B-1 of this Manual may be divided by 1.25.

D. Culverts

Culverts shall be constructed of reinforced concrete Class "B" and shall be analyzed as rigid frames. As a minimum, culverts shall be of sufficient length so that the full roadway section, including shoulders and berms, can be maintained. Footings for culvert wingwalls shall either be placed at the same time as the culvert floor slab or shall be adequately keyed and doweled into it. Toe walls shall be provided along the edge of culvert floor slabs or apron slabs.

VI. SEISMIC DESIGN, ANALYSIS & RETROFIT

A. Design Specifications

The AASHTO LRFD Specifications have *not* been adopted for the seismic design or analysis of Turnpike structures. The seismic design provisions for LRFD are currently under revision as part of and will be issued as a guide specification in the near future.

Except as modified below, the seismic evaluation of all highway bridges shall be governed by Division 1A of the latest edition of the AASHTO Standard Specifications for Highway Bridges, with current interims. Within Division IA, where references are made to Division I Design, the Load Factor Method of shall be used.

In the absence of site-specific response spectra, response spectra shall be developed in accordance with the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1998a and 1998b). The complete map set, Maps 1 - 32, are used with 1997 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (Applied Technology Council, 1997a and 1997b). Response spectra may be obtained by zipcode using the following html: http://geohazards.cr.usgs.gov/eq/html/nehrp.html

It is also noted that site response evaluation (soil amplification) must be applied and shall also be in accordance with the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings.*

B. General Considerations

The most common and significant cause of earthquake damage is ground shaking. In addition to ground shaking, seismic hazards can also include ground failure, liquefaction, lateral spreading, differential settlement and land sliding. Bridges shall be evaluated and proportioned to resist significant damage to such events. As such, two levels of earthquake shaking hazard shall be used to satisfy the basic safety objectives for all Turnpike structures: Functional and Safety Evaluation Event Levels (FEE & SEE).

Under the Functional Evaluation objective, immediate operation is required (following bridge inspection) and minimal damage is permissible. This hazard level has a 10% probability of exceedence in 50 years (return period of approximately 500 years).

For a bridge designated for a Safety Evaluation performance level, significant disruption to service is permissible as is significant damage and has a corresponding 2% probability of exceedence in 50 years (return period of approximately 2,500 years).

These performance objectives represent a rational approach to seismic retrofit as well as new design, and therefore will be used as a basis for evaluating performance of bridges subjected to the functional and safety evaluation level events.

C. Design Requirements

For the design of bridges carrying Turnpike traffic, two events shall be considered, having return periods of approximately 500 years and 2,500 years. Response Modification Factors (R) for the latter event are as follows:

Element	**R**
Wall type pier	2
RC pile bents	
Only vertical piles	3
With battered piles	2
Single Columns	3
Multiple Column bent	5

For the 500 year return period, Response Modification Factors for all substructure elements shall be 1.5.

For the design of local road bridges over the Turnpike, including State and Federal highways, the design shall be based on the lesser of the two events, unless the owner has a published policy which requires otherwise.

For the design of certain special bridges which the Authority deems to be "critical", the design shall be based on the 2,500 year return event and R = 1.5 for all substructure elements.

Bridges that are single span or bridges with less than 25,000 square ft in deck area shall be designed using the NEHRP response spectra, while bridges that exceed 25,000 square ft in deck area require a site-specific evaluation.

Site-specific procedures may be used for any bridge, but shall be used where any of the following apply:

- The bridge is located on Profile Type E or F soils (as defined by NEHRP).
- A time history response analysis will be performed as part of the design / retrofit.

D. Retrofit Requirements

FHWA's Seismic Retrofitting Manual for Highway Bridges shall be used as a guide regarding evaluation procedures and upgrade measures for retrofitting seismically deficient highway bridges. This document is currently provided online at:

http://www.tfhrc.gov//seismic/document.htm

Seismic retrofit of existing bridges differs greatly from the design of new bridges since remaining service life must be considered in establishing a consistent level of safety. The AASHTO LRFD code is based upon an average bridge service life of 75 years, and therefore, seismic retrofit hazard should be a function of this estimate of bridge life. The desired Life-Safety event is taken as 3% probability of exceedence in 75 years, resulting in a return period that is nearly identical to the 2% in 50 year event. Therefore, a consistent level of risk can be proportionally established between bridge retrofit and new design requirements based on a specific accounting of a bridge's remaining life.

Bridges that are single span or bridges with less than 25,000 square ft in deck area shall be retrofitted to resist the 10% in 50 year probability event (return period of approximately 500 years) using the NEHRP response spectra. Isolation strategies, if employed, shall be evaluated for the 2% in 50 year event (return period of approximately 2,500 years) for design and detailing of the isolation bearings.

Bridges that exceed 25,000 square ft in deck area should require a site-specific evaluation based upon both the Functional and Safety Evaluation Event Levels with approximate return periods of 500 years and 2,500 years respectively.

References:

Applied Technology Council, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, Washington, D.C.

Applied Technology Council, 1997b, *NEHRP Commentary for the Seismic Rehabilitation of Buildings*, FEMA 274, Washington, D.C.

Building Seismic Safety Council, 1998a, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 - Provisions*, 1997 Edition, FEMA 302, Washington, D.C.

Building Seismic Safety Council, 1998b, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 - Commentary*, 1997 Edition, FEMA 303, Washington, D.C.