SECTION 2
STRUCTURES DESIGN

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SECTION 2
STRUCTURES DESIGN

2.0 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.

AUTHORITY: The New Jersey Turnpike Authority (NJTA), with its principal office in Woodbridge, New Jersey, or its duly authorized representative in cases of Projects entrusted to the Authority's General Consultant or Program Manager, acting on behalf of the Authority.

BRIDGE MAINTENANCE WORK: Work intended to preserve a structure, which typically includes local area repairs to the concrete deck and / or riding surface (or overlay), resetting of bearings, joint repair, drainage cleaning, bridge seat cleaning, crack sealing, and miscellaneous repairs to the superstructure and substructure with spot painting, as required. Also included is select/localized concrete deck panel replacement.

BRIDGE REHABILITATION: Rehabilitation of any existing bridge which generally includes replacement of a large portion or all of the concrete deck and joints, zone painting of the bridge superstructure, and/or miscellaneous superstructure and substructure repairs. Bridge Rehabilitation Projects are generally intended to extend the service life of a bridge for 25 to 35 years. In special instances, a Bridge Rehabilitation may include repair of designated critical items to extend the useful service life of bridges which are considered in poor condition and are scheduled for replacement within 10 to 15 years.

COMPREHENSIVE BRIDGE REHABILITATION: Rehabilitation of any existing bridge (Major or Routine) which generally includes, but is not limited to, complete replacement of the concrete deck and joints, extensive superstructure repairs (including strengthening), full/zone repainting, bearing replacement, seismic retrofit, bridge widening, and/or miscellaneous substructure repairs. Complete/ or partial superstructure replacement may also be considered. Comprehensive Bridge Rehabilitation projects generally are intended to extend the service life of a bridge for an additional 60 to 75 years.

CONTRACT DOCUMENTS: Advertisement for Proposal, Proposal Guarantee, Contract Agreement, Contract Bond, Power of Execution, Standard Specifications, Supplementary Specifications, Plans, Addenda, and other transmitted documents to the prospective bidders prior to the receipt of bids, Change Orders, Field Orders, and Supplementary Agreements, all of which are to be treated as one instrument whether or not set forth at length in the written Contract Agreement.

CONSULTANT: An engineering consulting firm retained by the Authority to perform engineering and plan preparation work.

CONTRACT PLANS: The standard drawings, the official approved drawings specially prepared for the Project, profiles, cross-sections, and any supplemental drawings, or exact reproductions thereof, and that are current on the date the bids are received, and were furnished by the Authority, that indicate the location, character, dimensions, and details of the Work to be done.

CROSS-BEAM: A Closed-box or open-shape steel pier cap, either free spanning or cantilevering out, for the purposes of supporting Girders or Stringers. Cross-Beams are always considered Fracture Critical Members.

CROSS-FRAME: A Fabricated open truss member that acts as bracing that spans between deeper Stringers of a bridge and assists in the distribution of loads. Cross-Frames shall be considered as primary members in curved structures.

CULVERT: A structure including supports erected over a depression, drainage path, or waterway having an opening measured perpendicular to the flow of water (or equivalent measure) of 5' to 20' between faces of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. It may also include multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening.

DESIGN ENGINEER: The Design Engineer is an authorized representative of the Engineer of Record, with at least 10 years of direct bridge structural engineering design experience and at least 5 years working as a structural engineer in New Jersey, who is licensed to practice in New Jersey. Design Engineers shall have practical experience with all the design and construction issues required for the Project including, but not limited to, steel fabrication including welding and field erection, basic metallurgical understanding of common construction materials, basic concrete mix design and chemistry, and an understanding of current trends in construction methods and materials related to heavy bridge work. The Design Engineer may or may not be the Engineer of Record. However, the Design Engineer's initials shall always be the Supervisor in the "supervised box" lower left-hand corner of the Plans related to the structural design aspects of the Project.

DIAPHRAGM: A solid body member such as a C-channel, W-shape, fabricated welded plate, or concrete beam element that acts as bracing and assists in load distribution spans between the Stringers of a bridge. Diaphragms shall be considered as primary members in curved structures and heavily skewed structures.

ENGINEER: The Chief Engineer of the Authority or his/her duly authorized representative acting within the scope of the authority vested in him/her.

ENGINEER OF RECORD (EOR): Professional Engineer licensed to practice in New Jersey, responsible for the preparation of the Contract Documents. All communications with the Authority shall be through the EOR.

FAILURE CRITICAL MEMBER: A bridge member whose failure would be expected to result in a partial or full collapse of the bridge. Failure Critical Members may be constructed of any material designed to withstand loads including non-tensile loads. An example of a Failure Critical Member is a truss chord compression member or non-redundant pier column. This definition is meant to identify bridge members which may not be subjected to significant tension or flexure (such as pier columns), but their loss would still adversely affect the functionality and safe use of the structure. Non-redundant pier columns meeting the impact provisions of Subsection 2.2.2 of this Manual are exempt from this definition.

FLOORBEAM: Horizontal members that span transversely to, and are supported on, Girders or trusses and are used to support the Stringers or deck.

FRACTURE CRITICAL MEMBER: A bridge member subjected to tension or flexure that lacks redundancy. i.e., in the event of failure of just that element, the collapse of an entire span or bridge is likely to occur. Fracture critical members are noted as (FCM) for the purposes of identifying Fracture Critical Members in the Contract Plans. This definition is expanded beyond current FHWA guidance to include members other than steel components, such as non-redundant free spanning pier caps under direct flexure load from the superstructure. Two column concrete pier bents are exempted from this provision where they meet the impact resistance design provisions of Subsection 2.2.2

GIRDER: A horizontal structural member carrying vertical loads by resisting bending. A Girder is comprised of multiple plates and/or angles which are riveted or welded together and is usually interconnected by a system of Floorbeams, which in turn support a floor system that is typically comprised of Stringers, that directly supports a concrete bridge deck. In certain rare instances within the Authority, some Girder bridges may have decks supported solely on the Floorbeams or the deck may be supported directly on the Girder. These details are not to be reproduced in New Bridges. Girder ends are supported directly on abutments and/or piers. Girders may simply be referred to as “beams” in this section.

MAJOR BRIDGE: A bridge carrying Turnpike or Parkway roadways which merits additional consideration for design, maintenance, or rehabilitation effort. The minimum criteria for qualifying as a Major Bridge is that the bridge must support mainline traffic, must feature at least one span longer than 180 feet, or is designated by the Authority to be considered a Major Bridge.

NEW BRIDGE: A complete replacement of an existing bridge or an additional bridge added to the inventory. For the purposes of this Section, the limits of this work include all aspects of the construction from the level of the light standard mounts on the parapet to the bottom of the substructure components (piles / drilled shafts), and the full length of the bridge from the outer limits of the bridge approach slabs (or abutment back walls), including the wingwalls. Also included is the design of all permanent materials left in place and consideration of all temporary works and staging required to construct the New Bridge.

PROJECT: The entire Work to be performed under the Contract, including the furnishing and execution of all things necessary or proper therefore or incidental thereto for completion of the Work.

REFERENCE DRAWING: Any Plan sheet from a previous contract, a concurrent contract, or a future contract, included with the Contract Documents which will aid the Contractor in performing the work of the Project. Absolutely no changes are to be made to any Reference Drawings.

ROUTINE BRIDGE: Any bridge which is not a Major Bridge that is 20' or longer. A Culvert with a clear span of 20 feet or longer is also considered to be a Routine Bridge.

SPECIFICATIONS: The Standard Specifications, the Supplementary Specifications and Addenda, if issued, pertaining to the method or manner of performing the Project and to the qualities of the materials to be furnished for the Project.

STRINGER: A horizontal structural member carrying the vertical loads which supports the bridge deck by resisting bending. Stringer ends may be supported directly on abutments and piers, or may be supported only by a Floorbeam system of a Girder or truss bridge. Stringers may simply be referred to as "beams" in this section.

SUBSTANTIAL MODIFICATION: A Substantial Modification shall be defined as the alteration of any primary load carrying member within a bridge superstructure, including, but not limited to, Girders, Floorbeams, or Stringers, where the incurred load upon, or load capacity of, these members is changed by more than 10% of the existing condition.

2.1 PURPOSE AND INTENT

Section 2 of the Authority's Design Manual provides guidance, policies, standard practice and procedures for the development of bridge and/or structure Projects. The primary goal of this Section is to provide direction to Design Engineers to ensure that bridges constructed for the Authority are, in descending order of priority: (1) highly durable, (2) constructible, and (3) economical.

The instructions found within Section 2 constitute the minimum required level of effort on the part of the EOR. The EOR is encouraged to exceed the minimum required level of effort when best practices dictate. The Authority desires the "best value" structural solution, not the "lowest cost" structural solution, in cases when these two conditions conflict.

Section 2 of the Design Manual shall be used with other Design Manual sections (e.g. Geotechnical-Section 5) and shall be supplemented by current versions of the Authority's Procedures Manual, Load Rating Manual, Construction Manual, Manual for Traffic Control in Work Zones, Standard Specifications, and Standard Supplementary Specifications.

Although this Section provides guidance on design and analysis procedures, it does not preclude the need for additional engineering analysis and design procedures to produce a safe, economical and maintainable structure. Often special conditions will require engineering judgment to be applied and shall be assessed by the EOR and approved by the Authority on a Project specific basis.

2.2 BRIDGES

2.2.1 Design Specifications

2.2.1.1 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 2.2.2 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.

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 5 of this Manual for additional guidance.

2.2.1.2 Load Rating

Load ratings shall be performed by the Design Engineer in parallel with all designs for New Bridges, or where an existing bridge is subjected to a Substantial Modification.

Load ratings of new or rehabilitated structures shall be performed and published in accordance with the Authority's Load Rating Manual Subsection 4.4.

2.2.1.3 Geometry

Shoulder Widths

Shoulder widths shall be established as shown in the following Exhibits at the end of this Section:

- Turnpike Bridges: Exhibits 2-100 to 2-107
- Parkway Bridges: Exhibits 2-108 to 2-111

It is desirable for the shoulder widths on a structure to match the shoulder widths of the approaching roadway for Turnpike and Parkway bridges. However, under certain conditions, it may be acceptable for shoulder widths on a structure to differ from the approach roadways. For bridges carrying local roadways over Turnpike or Parkway roadways, shoulder widths shall be as required by that roadway owner.

Shoulder widths for New Bridges which are less than the shoulder widths of the approach roadways may be permissible in situations where a physical or environmental obstruction beyond the fascias of the bridge prevents widening of the bridge. Approval of substandard shoulder widths shall be only via accepted Design Element Modification Request. Consult with the Authority Engineer where it is anticipated that reduced shoulder widths may be warranted.

The use of a reduced width shoulder will not exempt the Design Engineer from considering future bridge re-decking efforts per Subsection 2.2.4.1 of this Design Manual. Consideration of future re-decking of the structure may still require the use of over-wide shoulders in order to maintain all lanes of traffic during deck replacement or rehabilitation operations.

In the event that the shoulder widths on the structure are less than the shoulder widths on the approach roadway, the approach roadway shall be tapered such that it is tangent a minimum of 100' from the begin / end bridge stations. This will allow the structure cross-section to be considered as matching the approach roadway section, effectively preventing the structure from being categorized as "functionally obsolete" in the NBIS bridge rating system. Refer to

Sections 1A and 1B (Turnpike and Parkway Geometric Design) of this Manual for guidelines on tapering the approach roadway.

For bridge replacement Projects, Consultants shall be required to ensure bridge cross sections meet or exceed the NBIS requirements, effectively preventing the structure from being categorized as “functionally obsolete”.

For Major Bridge Rehabilitation Projects, the existing bridge geometry shall be evaluated per the NBIS requirements. The Consultant shall make recommendations to the Authority for improving the bridge geometrics to prevent “functionally obsolete” NBIS categorization as a part of the Phase A submission. No geometric improvements shall proceed without approval from the Authority.

For Bridge Replacement Projects, Consultants shall be required to confirm that new substructure locations meet the applicable sight distance and lateral clearance requirements for roadways passing under the New Bridge.

For bridge replacement Projects and for Major Bridge Rehabilitation Projects, the bridge cross section and the substructure locations shall be assessed for future roadway capacity requirements of the roadways below the structure and above, as dictated by the Authority’s current accepted Capital Plan. Consult with the Authority Engineer for guidance.

2.2.1.4 Railroad Bridges

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 agency concerned. Where railroad bridges are constructed for separate governing entities (New Jersey Transit, Norfolk-Southern, Conrail, etc.) those bridges shall be designed in accordance with their respective design standards. The New Jersey Turnpike Authority does not assume ownership of or maintenance for rail structures except as determined through an executed agreement. Refer to Subsection 2.2.2 of this Manual regarding modifications to AASHTO Subsection 3.6.5 for railroad impact loading. Consultants are advised that new rail bridges will become the property of the carried rail owner after completion of construction. Consult with the Authority Engineer prior to engaging with rail entities for bridge rehabilitation work.

2.2.1.5 Local and State Highway Bridges

For bridges carrying local roads and State highways over Authority roadways that fall under the Authority’s maintenance jurisdiction, this Design Manual shall be followed for the bridge’s structural design. However, the above roadway geometric requirements as well as the details for sidewalks, parapets, fencing, etc. shall conform to the requirements of the applicable agencies (i.e. NJDOT, local counties).

2.2.1.6 Accelerated Bridge Construction (ABC)

ABC is an emerging form of bridge construction which uses alternative methods of Project planning, design and construction. These alternative methods are geared towards reducing the onsite construction, reducing traffic delays for the duration of a bridge replacement or rehabilitation Project and reducing lane closures and impacts to traffic and local communities. This can be accomplished through use of prefabricated bridge elements to drastically reduce or eliminate time-consuming field operations such as forming and curing of structural concrete. Innovative construction methods such as utilizing slide-out/slide-in techniques which allow a bridge to be built off-line and then quickly moved into place are also effective ABC methods to minimize traffic impacts.

All New Bridge, bridge replacement, superstructure replacement, and bridge deck replacement Projects shall consider the use of ABC.

It is generally understood that ABC Projects will be of a higher initial construction cost than traditionally built Projects. This additional cost may be acceptable when compared to the advantages provided by ABC, including shortened disruptions to traffic, shortened on-site construction duration, reduction or elimination of winter cessation of concrete placement via use of precast elements, and increased quality control for complex aspects of bridge construction. ABC can also be beneficial where extremely tight construction work zones may preclude or complicate the use of traditional construction operations.

In addition, intangible benefits such as increased worker and public safety, decreased environmental impact, and reduced disruption to densely populated areas should be weighed, where appropriate.

The use of ABC is strongly encouraged for bridge work on the Garden State Parkway between Milepost 120 and Milepost 163 where limited shoulder widths, high traffic congestion, and limited work staging areas complicate construction operations.

The Authority has evaluated ABC techniques and details utilized by various agencies and has prepared a reference document titled ‘Policy and Guidelines for Accelerated Bridge Construction, Final Report’, summarizing preferred methodology and tools to evaluate ABC feasibility for a project. The ABC Screening Spreadsheet tool in this reference document shall be utilized in the Phase A investigation to determine if ABC is to be evaluated further for the bridges in the project.

The following ABC candidate evaluation categories were identified as a part of the report. The categories shown below are considered the most critical for the purposes of using the ABC Screening Spreadsheet tool. These categories are discussed in greater depth in the above referenced report.

- Initial Construction Cost
- Road User Cost
- Toll Revenue Impact
- Project Type
- Bridge Type
- Complexity of Geometry
- Maximum Span Dimensions
- Anticipated Detour Route
- Repetition / Economy of Scale
- Public / Road Worker Safety
- Durability / Life Cycle Costs
- Constructability
- Ease of Inspection
- Adherence to the Authority's Standards
- Environmental Impacts
- Utility and Railroad Impacts
- Right-of-Way Impacts

Where Project specific category considerations in addition to the above are identified during the screening process, they shall be brought to the attention of the Authority via the Phase A report.

ABC candidate bridges shall be identified and investigated as a part of the Phase A evaluation. At a minimum, the Phase A report will include:

- A narrative describing how the above considerations affect the recommendation to use ABC for a bridge Project.
- General Plan, Elevation, Bridge Section, and staging sketches for both the traditional and ABC options.
- Cost estimate for both the traditional and ABC options including toll revenue impacts, where relevant.
- Preliminary simplified Gantt style construction schedule in Excel for both the traditional and ABC options.

Certain high cost / high construction duration elements of all bridges, regardless of location, shall consider precast construction methods. These elements include items such as cap beams for bent concrete piers and open bent pier columns for very tall piers.

When considering ABC superstructure replacements, the Design Engineer shall investigate substructure elements for remaining life span per guidance given in Section 5 of the Design Manual prior to recommending them for extended service.

Precast deck panels shall not be used on roadways subject to heavy truck traffic including the Turnpike roadways north of Interchange 15E and the entirety of the Newark Bay Hudson County Extension without the express permission of the Chief Engineer.

The Authority will determine whether a candidate bridge is to be advanced with traditional or ABC type construction as a part of the Phase A review process.

2.2.1.7 Utilities on Structures

Utilities on structures are strongly discouraged. It is the policy of the Authority to prevent the placement of utilities on structures. The Design Engineer shall make every effort to provide alternate roadway crossing options for utilities that would otherwise be supported on a bridge. The alternate options may include directional boring under the roadway or aerial wires spanning over the road. New Bridges are prohibited from having utilities placed upon them without express written permission from the Authority. Comprehensive Bridge Rehabilitation Projects shall investigate the possibility of removing utilities from subject bridges where practical. Refer to Section 7 of the Procedures Manual for additional utility guidance.

2.2.2 Modifications to Current Codes

The following modifications to the current AASHTO LRFD Bridge Design Specifications shall apply:

1.3.5 Operational Importance (AASHTO)

The Operational Importance strength limit state shall classify all Turnpike and Parkway mainline and ramp bridges as “important”, therefore: $\eta I = 1.05$.

2.3.3.2 Clearances (AASHTO)

The minimum vertical clearances for new or replacement bridges shall be limited to 16’-0” to the lowest projection of the underside of the bridge where it crosses over a roadway or roadway shoulder.

The minimum vertical clearance for widened or rehabilitated bridges shall be equal to or greater than the existing bridge clearance. Where existing bridge clearance is less than 14’-6”, consideration shall be given to the following to achieve the minimum bridge replacement clearance noted above.

- Replacing the superstructure
- Raising the superstructure
- Lowering the roadway below.

In instances where this is not practical, the Authority may approve a reduced minimum under clearance of 15’-0”. Consult with the Authority Engineer should this occur.

2.5.2.4 Rideability (AASHTO)

For design purposes, the top 1/2” 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.

Also note that corrective grinding or micro-milling of the concrete deck has become standard practice for the Authority’s bridge Projects to improve rideability and overall deck durability. An additional 1/4” of concrete shall be provided in the Contract Plans for construction, but shall not be considered for dead load as it is assumed that it will be removed during the micro-milling work. Direction should be given in the Contract Plans for the Contractor to set drainage inlets lower to allow for the final micro-milling. Joints should be set in specific end-of-deck and headblock closure pours after the completion of micro-milling to allow for best possible flush fit. However, in instances where this is not practicable, the joint armor may be set to account for the final micro-milling. The Design Engineer is referred to the NJTA Standard and/or Supplementary Specifications for guidance.

2.5.2.6.2 Criteria for Deflection (AASHTO)

Traditionally, bridges have been designed for stress limits using allowable stress design (ASD) or load factor design (LFD) provisions published in the AASHO or AASHTO codes from the first to seventeenth editions. These publications included language limiting deflections due to live load via the limit relationship of span length ‘L’ divided by a factor of 800 for pure vehicular traffic, or ‘L’ divided by 1000 for vehicular plus pedestrian traffic. The anecdotal history of these provisions was ostensibly to mitigate driver and pedestrian discomfort with additional concerns noted for durability of bridges and decks subjected to “excessive” flexure. Many bridge owners still maintain this criterion in their own design guidelines, often without a complete understanding of the need of this provision or its effects on their bridges.

In evaluating the validity of the provision, both the above noted concerns were researched and no conclusive documentation was found to support either supposition. If anything, it is worth pointing out that the inverse may in actuality be true for driver and pedestrian discomfort; consider that no physical sense in the human range is capable of easily detecting so slight a movement as 1/800th of even a relatively modest span length of 100’ (1½”) simply by standing on that span. That being said, the human inner ear does have the ability to detect changes in the ambient rate of acceleration, which is actually how one senses the vertical motion of a bridge while standing on it. Higher rate of acceleration therefore may cause more discomfort than lower rates of acceleration. High rates of acceleration stemming from higher bridge first resonant frequencies with small deflections are more likely to cause more discomfort than lower rates of acceleration stemming from lower bridge first resonant frequencies with large deflections. Regardless, it is often difficult to calculate the first resonant frequency of even a Routine Bridge.

In actual design, deflection can still be a concern. Beams with large cambers can be difficult to erect and correctly align where field splices are concerned, as the erecting contractor must ‘follow the curve’ of the Girder and cannot rely on a straight line view of the side profile of the girder. Shallow Girders may have large cambers that approach or even exceed the depth of the Girder web, creating concerns regarding stability, particularly during plastic concrete deck placement.

After factoring these constructability restrictions, it was determined that minimum practical limits on at least dead load deflection camber must be observed. To that end, the following principles shall apply to deflections:

- When investigating the maximum absolute deflection in tangent (straight beams) bridges, all design lanes should be loaded and all supporting components should be assumed to deflect equally.

- When investigating the maximum absolute deflection in bridges with horizontally curved stringers, all design lanes should be loaded and supporting components should be assumed to deflect unequally. The deflection limit is applied to each individual beam.

- When investigating maximum absolute deflection in transverse members such as closed steel box Cross-Beam pier caps, design lane positions should be loaded as appropriate to create the maximum deflection.

- The live load shall be taken from Article 3.6.1.3.2 and shall not be increased to reflect TP-16 live loading as described in NJTA Design Manual Subsection 2.2.2 (modifications to AASHTO Section 3.6).

- The live load multiple presence provisions of Article 3.6.1.1.2 shall apply.

Deflections of all Turnpike and Parkway bridges shall use the following values as general performance goals. Achieving the actual deflection goals shown below is not required, but encouraged.

- Vehicular load (longitudinal and transverse members) ....Span / 800

- Vehicular load on cantilever arms...Span / 400

The following additional criteria shall also be followed:

- The gross composite section properties in both the negative and positive moment regions of the span shall be used when calculating live load deflections (i.e. uncracked deck slab in negative moment region).

- The span length shall be defined as the distance between the centerlines of supports.

3.5.1 Dead Loads: DC, DW and EV (AASHTO)

Future Wearing Surface
An additional 25 psf of dead load allowance for a future wearing surface shall be applied to New Bridges or replacement bridge deck Projects with one course construction. The 25 psf provision accounts for the difference in weight due to the additional thickness of overlay.

Stay in Place Forms
The dead load for bridges with new or replacement reinforced concrete deck slabs supported by Stringers shall include a provisional 5 psf applied over the deck slab between Stringers to provide for the weight of corrugated metal stay in place (SIP) deck forms. All SIPs shall have their corrugations filled with foam and be oriented such that the minimum clearance to the reinforcing bar is maintained.

3.6 Live Loads (AASHTO)

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

Turnpike Bridges

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.

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 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.
    - Wherever a wheel load is specified, a 25 kip load shall be used.

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
The design vehicular live load to be used for all new Parkway mainline and ramp bridges shall be TP-16 as described above, unless otherwise directed by the Authority. However, the design live load to be used for all new members in Substantial Modifications to existing Parkway bridges shall be HL-93. Wherever a wheel load is specified, a 20 kip load shall be used.

3.6.1.4 Fatigue Load

3.6.1.4.1 Magnitude and Configuration

The first and second paragraphs are changed to:

- The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2. The weights of the axle loads shown in Figure 3.6.1.2.2-1 shall be increased by multiplying by a Dynamic Load Allowance factor of 1.33. A constant spacing of 30.0 ft. shall be used between the 32 kip axles.

3.6.1.4.2 Frequency

The following is added to the first paragraph of Article 3.6.1.4.2:

- For computation of fatigue resistance in accordance with Article 6.6.1.2.5, one way ADTT traffic counts may be taken from Section 3.1.3 of the NJTA Load Rating Manual, current edition.

3.6.5 Vehicular Collision Force: CT (AASHTO)

3.6.5.1 Protection of Structures

The following is added:

Superstructure Impact
Review of historical collision damage at spans of Authority owned structures has shown that the majority of impacts have occurred at the fascia Stringer of the bridge and have been almost exclusively due to over-height vehicles with variable reach features, such as lifted dump bodies, or improperly secured bucket arms of excavators loaded on trailers.

These incidents are difficult to prevent due to the transient height nature of the vehicles. The resulting impact energy of the collision is likewise difficult to evaluate as it varies with vehicle weight, speed, and concentration of the impact area.

Therefore, in lieu of a more refined analysis, spans of Authority owned structures that have less than a 20’ clearance over any roadway shall include fascia Stringers that have a minimum bottom flange thickness of 2” with a web plate of minimum 3/4” thickness. Diaphragms or Cross Frames for fascia beams (if present) shall be of a depth that is as close to the full-height of the web plate as is practicable and shall include stiffeners / connection plates with bolted connections to the bottom flange.

This provision may be waived for bridge superstructures built using ABC methods due to prefabricated element weight lifting concerns, as approved by the Authority.

Substructure Impact

Vehicular collision at traditional solid concrete abutments is not anticipated to be a controlling design load, given the consideration of the following:

- Embankments at stub abutments
- Passive pressure resistance of soil behind the full-height abutment walls

Abutments and piers located within a distance of 25’ to the centerline of a railroad track shall be designed for an equivalent static force of 600 kips or shall be protected by a structurally independent crash wall or embankment that extends not less than 7’ above the top of rail. This provides an allowance of 1’ for future ballasting of the railroad tracks and for potential encroachment during construction or maintenance operations.

Regardless of ADT, all bridges shall consider collision load.

Individual pier columns and wall stems with more than 42 sf of cross-sectional area at the impact height are exempt from impact consideration.

3.10 Earthquake Effects: EQ (AASHTO)

This section shall not be considered in the design of new structures or the rehabilitation of existing structures. Refer to Subsection 2.2.6 of this Design Manual for seismic design and retrofit criteria.

3.11 Earth Pressure: EH, ES, LS and DD (AASHTO)

Geotechnical force effects for walls and abutments including lateral earth pressures shall be computed in accordance with Subsection 5.6.2 of this Design Manual.

3.12 Force Effects Due to Superimposed Deformations: TU, TG, SH, CR, SE, PS (AASHTO)

Design thermal force effects, deformations, and displacements shall be determined per the AASHTO LRFD Bridge Design Specifications, current ed., Article 3.12.2 using Procedure A for Moderate Climate conditions. The load factor for all thermal force effects, deformations, and displacements shall be 1.20 for all applicable limit states. When considering thermal force effects between substructures and superstructures, only the gross moment of inertia of concrete columns or piers shall be considered unless a more detailed analysis is performed, as described below, to verify that the partially cracked moment of inertia can be mobilized.

Forces from thermal effects, such as superstructure expansion between adjacent fixed piers, can cause large moments on pier elements. These moments are carried by the un-cracked gross moment of inertia of the concrete element until internal stress in the bare concrete exceeds the modulus of rupture (fr). After the stresses exceed this limit, the cracked moment of inertia may be used for stiffness and thermal force effect computation. The cracked moment of inertia may be conservatively assumed at 50% of the gross moment of inertia unless a more detailed analysis is performed. Before a partially cracked moment of inertia may be utilized for design, the Design Engineer shall verify that thermal force effects are adequate to exceed the modulus of rupture in the gross moment of inertia of the pier or column and will not cause failures in the bearing mechanisms.

3.15 Blast Loading: BL (AASHTO)

Blast loading shall only be considered for design as directed by the Authority. All design loading required shall be provided to the Design Engineer by the Authority.

4.6.2.2 Beam-Slab Bridges (AASHTO)

Replace the 11th 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 shall be that portion of the floor slab from the fascia to the centerline between the outside Stringer and the first interior Stringer. Parapets, railings, and safety walks, if placed after the slab has cured, shall be assumed to be carried entirely by the fascia Stringer. Sidewalks shall be assumed to be carried proportionally by the Stringers under and directly adjacent to the sidewalk curb using simple span distribution between the Stringers. This distribution ratio is not applicable to noise barriers mounted to superstructures and reference is made to Subsection 2.7 for further guidance for noise barrier loading. Where there is an open joint in a split median barrier, 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 surfaces shall be considered to be carried by the Stringer carrying the slab upon which it is placed.

These provisions are not applicable to bridges where the fascia Stringers are to be reused. Refer to the Authority's Load Rating Manual for dead load distribution for existing bridges.

5.4.2.3.2 Creep (AASHTO)

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

5.9.4.2.2 Tension Stresses (AASHTO)
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 pre-stressing tendons should be investigated using Load Combination Service III and the tension in the pre-compressed tensile zone shall be zero. Stress Limit limitations stated in AASHTO LRFD Specification Table 5.9.4.2.2-1 shall be accounted for in the permit vehicle check.

5.12.3 Concrete Cover (AASHTO)
Article 5.12.3 shall be replaced by the following:

The minimum clear cover for all reinforcement shall be 2" except as given below:

1. Concrete permanently in contact with earth: 3"
2. Concrete exposed to fresh or brackish water or in roadway splash zones:
   Piers and abutments: 4"
   Walls and culverts: 3"
3. Concrete deck slabs:
   Top reinforcement: See Subsection 2.2.4.2
   Bottom reinforcement: 1"

Splash zones noted above shall be any pier or abutment surface within 15" of an active roadway lane. This provision does not apply to MSE precast wall facing panels.

In pile foundations, reinforcement or supports for reinforcement shall be positioned a minimum of 3" clear from the faces of the piles.

6.4.1 Structural Steels (AASHTO)
See Subsection 2.2.3.1 of this Design Manual for additional information.

6.7.2 Dead Load Camber (AASHTO)
In computing cambers, the weight of the concrete deck slab shall include the permanent metal deck forms and the concrete contained in the forms, where present. Refer to Subsection 2.2.4.2 for new decks with foam filled deck forms.

In determining cambers in bridges with 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 concrete slab. New Bridges need not consider a dead load provision for future asphaltic overlays for the purposes of establishing camber. Should need arise to install an asphaltic overlay in the future, the top 1/2" of the concrete deck will be milled away and replaced with the new asphaltic riding surface, thus negligible additional weight will be added to the bridge.

Steelwork shall be cambered to compensate for the weight of utilities where those utility weights incur deflections of 0.1 inches or more. The utility dead load shall be considered as supported by the steel section only.

Simple span bridges shall have a residual architectural camber equal to the length of the span / 1000 or 1", whichever is greater.

These instructions shall apply unless it is known that the construction method will be such as to make them impractical. Consult with the Project Engineer where these deviations are anticipated or warranted.

2.2.3 Materials

2.2.3.1 Structural Steel

Turnpike Bridges
Structural steel for Turnpike bridges shall comply with AASHTO M270, Grade 50W (ASTM A709, Grade 50W) unless otherwise approved by the Authority's Engineering Department. Members classified as FCM's shall be fabricated from ASTM A709 Grade HPS50W.

All structural steel within a distance of 1.5 times the beam depth from a bridge joint shall be metallized in accordance with the NJTA Standard and Supplementary Specifications. The top flanges of Stringers shall be specified as furnished coated in zinc compatible weld-through primer. The exterior metalized surfaces of the fascia stringers shall be painted brown in accordance with the NJTA Standard and Supplementary Specifications. The exterior uncoated surfaces of fascia stringers, including the underside of the bottom flange, shall be blast cleaned in accordance with SSPC-SP-10 to ensure a uniform patina formation.

Parkway Bridges
Structural steel for Parkway bridges shall comply with AASHTO M270, Grade 50 (ASTM A709, Grade 50) unless otherwise approved by the Authority's Engineering Department. Members classified as FCM's shall be fabricated from ASTM A709 Grade HPS50W.

All structural steel for bridges shall be painted contrasting shades of green in accordance with the NJTA Supplementary Specifications and per Exhibit 2-304.

High Strength Steel (HPS70W or higher)
Use of High Strength Steel (Grade 70W or higher) should only be considered on a case-by-case basis and where permitted by the below provisions. Hybrid girder design, where appropriate, will preferably incorporate High Strength Steel in the bottom flanges of girder positive moment regions, and in both the bottom and top flanges of girder negative moment regions. Conventional Grade 50W Steel should be retained in girder positive moment top flanges and all web elements.

Typical design conditions where the Authority may allow High Strength Steel may include but are not limited to:

- Significant anticipated cost savings (10%, min) related to the total Structural Steel bid item.
- Shallow depth girders which may otherwise require overly thick and difficult to fabricate Gr. 50W flanges.
- Negative moment region flanges where drastic changes in flange plate thicknesses (greater than 50%) would otherwise create fabrication or erection complications.

The Design Engineer should be aware that due to the volatile availability of high strength steel, economy of materials should not be used as the sole determining factor for its use. The possibility of mill delivery delays should be investigated by the Design Engineer via direct contact with local steel fabricators prior to specifying its use.

Notch Toughness and Tensile Requirements
All flanges and webs in tension shall be noted as (T) on the plans for supplementary notch toughness testing required in accordance with ASTM A709 Section 10.

All components which are defined as Fracture Critical shall be noted as (FCM) on the plans for supplementary notch toughness testing required in accordance with ASTM A709 Section 10.

For the purposes of testing, Charpy V-Notch (CVN) toughness testing shall be performed for Zone 2 requirements per the ASTM A709 specification.

2.2.3.2 Concrete

The Authority does not allow the use of prestressed concrete for structural beams or piles except at the express written permission of the Supervising Engineer, Structures.

Wherever precast elements are specified for use, they shall be Concrete Class P with a minimum compressive strength at 28 days of 5,500 psi. The Design Engineer shall use a value of 5,000 psi for design.

Concrete for use in deck slabs, headblocks, cast-in-place parapets, and unsurfaced approach slabs for New Bridges, widenings and major deck reconstruction shall be High Performance Concrete (HPC) with a minimum compressive strength at 28 days of 4,400 psi. The Design Engineer shall use a value of 4,000 psi for design.

In situations where small concrete quantities are required for barrier parapet, Class A concrete with concrete penetrating sealer may be used in lieu of High Performance Concrete with the approval of the Engineer.

Concrete Class A, with a minimum compressive strength at 28 days of 4,500 psi, shall be used where Class P and Class HPC are not specified. The Design Engineer shall use a value of 4,000 psi for design.

Concrete Class SCC, with a minimum compressive strength equal to Concrete Class A, shall be considered for drilled shaft foundations when the following conditions exist:

- Very long and / or large diameter shafts.
- Reinforcing requirements dictate congested reinforcement configurations where flow of 3/4" diameter aggregate will be challenging.
- Concrete hold times are excessive due to difficult access or long duration pours.
- Other conditions that, in the opinion of the Engineer, may necessitate the use of Concrete Class SCC.

Concrete Class SCC may also be specified where congested reinforcement is present in above ground concrete members. The Design Engineer is advised that additional notations shall be provided on plans where SCC is used above ground, alerting the contractor to the possible necessity of more robust formwork to resist the higher fluid pressures exerted by the free-flowing nature of the Concrete Class SCC.

2.2.3.3 Reinforcement Steel

Reinforcement steel shall conform to the requirements of ASTM Designation A615, Grade 60 deformed carbon steel. Low-alloy, low-carbon steel conforming to the requirements of ASTM Designation A706, Grade 60, may be substituted in situations where welding is employed to expedite the assembly of reinforcement cages.

Reinforcement steel conforming to the requirements of ASTM Designation A615 shall not be welded. Additionally, welding of intersecting bars shall not be permitted in deck slabs.

All reinforcement steel, regardless of where it is placed within a structure, except for bridge decks, shall be specified as galvanized. For reinforcement within bridge decks, refer to Subsection 2.2.4.2. The designer is advised that for complete protection and to avoid unwanted galvanic response between dissimilar metals, all reinforcement steel accessories and support bars shall be galvanized. All galvanized coatings damaged during installation shall be repaired in accordance with ASTM A780.

2.2.4 Superstructure Design

2.2.4.1 Stringers and Girders

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.

Refer to the provisions of Subsection 2.2.2 of this Manual for additional sizing guidelines relating to fascia beams which may be subjected to impact loading.

2. 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.

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.

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.

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.

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.

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 Cross-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” to 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 2.2.2 Modifications to Current Codes Subsection 3.6.5 in this Manual, 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.

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.

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.

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.

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.

2.2.4.2 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 2.2.2 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 2.2.2 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 2.2.1:

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.

Where the above empirical deck design is found to be impractical for full deck replacement work for existing bridges, the Design Engineer shall reduce the sectional thickness as necessary to carry the above design wheel loading and preserve the load carrying capacity of the superstructure.

2. Reinforcement
   For new deck slab designs for Routine Bridges, galvanized reinforcing steel shall be used throughout. The Design Engineer is advised that for complete protection and to avoid unwanted galvanic response between dissimilar metals, all reinforcement steel accessories as well as all top surfaces of top flanges shall be coated with an inorganic zinc rich primer. Support bars shall be galvanized. All galvanized coatings damaged during installation shall be specified on the plans to be repaired after installation of shear connectors in accordance with ASTM A780.

   For New Bridges and total replacement of bridge decks, stainless steel reinforcement is recommended for use when any of the following apply.

   - Its cost impact on the total Project cost is 5% or less.
   - The redecking Project for Major Bridges that are expected to serve for 75 years or more beyond the completion of the redecking Project.
   - Anticipated complex future staging or non-practicable access to a bridge deck for future repairs warrants its use to avoid correspondingly difficult lane closures.

   Stainless steel reinforcing bar shall not be used for redecking projects without the express written consent of the Authority.

   All reinforcement steel accessories and support bars shall be either stainless steel or non-ferrous in construction where stainless steel reinforcing is specified for decks.

   Solid stainless steel reinforcement where specified shall conform to ASTM A955/A955M – UNS Designations S24000, S24100, S30400, S31603, S31653, S31803, S32101.

   Reinforcement type and protective coating for parapets, medians, and overhangs shall match the deck slab.

   For the purposes of a new deck designed on existing superstructures to remain, additional longitudinal deck reinforcement may be used to increase the negative moment bending capacity of the Stringers.

3. Concrete Haunches
   Haunches shall be provided for all cast in place concrete decks. 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 Stringer due to construction

tolerances, variation in Stringer depth, variation in camber, deflection of the Stringer or other causes. The dimension from the top surface of the slab to the top of the Stringer shall not be less than the nominal slab plus 1” (minimum 1” haunch). The top of the Stringer shall normally be set so as to provide the minimum haunch depth over the thickest flange plate and at the most restrictive side of the Stringer when a cross slope is present. Where field splices in the Stringers are shown on the plans, or permitted by the Specifications, the haunch shall be a minimum depth of 1” over the splice plate. Bolt heads may project into the haunch, but 1” minimum of clear cover shall be maintained between the main steel reinforcement and the bolts.

Haunches that are over 4” high shall be reinforced per details shown on Standard Drawing RP-5. Shear studs in reinforced haunches shall penetrate at least 2” above the top of the haunch reinforcement stirrup. Haunch reinforcement shall be designed as shear reinforcement and shall, in combination with the shear capacity of the unreinforced concrete, meet or exceed the fatigue resistance of the shear connectors as defined in Section 6.10.10.2 of the AASHTO LRFD BDS.

4. Stay in Place Permanent Steel Bridge Deck Forms (SIP forms)
   For New Bridges, complete deck replacement work, and Rehabilitation work, the main reinforcement in the lower mat shall be placed a minimum of 1” above the top surface of the Styrofoam filled SIP forms. The permanent steel bridge deck form corrugations shall be arranged parallel to the direction of transverse reinforcement, except where it is impractical such as in acute corners of the deck at fascia Stringers or in curved bridges where the reinforcement steel may be fanned in a radial arrangement regardless of the SIP corrugation orientation. The concrete cover around the main bars shall not be less than 1” clear in any direction to the surface of the form, see Exhibit 2-301.

5. Slab Corners
   The reinforcing of the acute corners of skewed slabs at a fascia Stringer shall be given special consideration. In these areas, it may be necessary to place additional reinforcement in a fanned arrangement extending into the corner as shown in Exhibit 2-302.

6. Expansion Joints
   Strip seal expansion joints are preferred and shall be used wherever practical. The maximum movement rating permissible for strip seal deck joints shall be 4”. Regardless of calculated thermal movements of less than 4”, all strip seal joints shall be configured to support 4” movement ratings. Strip seal glands shall also support a minimum 4” movement rating and be of the self-cleaning type.

   Where strip seal expansion joints are not practical for use and analysis indicates joint movements greater than 4“, modular type deck joints shall be used. Modular joints are generally 5 to 8 times as expensive as a strip seal expansion dam and they are generally more expensive to maintain. It is the preference of the Authority for Design Engineers to limit continuous span superstructure unit lengths such that modular joints will be either minimized or not needed.

Where widening adjacent to an existing bridge joint that has been retrofitted with an asphaltic type “1P” style joint, it may be acceptable to use a different joint system with a longer service life. Consult with the Authority Engineer.

All deck joints, regardless of type shall be shown in the Contract Documents as being installed after the placement of the majority of the bridge deck. Beam end rotations and translations during construction may complicate the installation and support of the deck joint assemblies. Block-outs shall be utilized to install the deck joint assemblies in their intended alignment for all bridges. The Design Engineer shall provide appropriate details, including all pertinent dimensioning of the closure pours, on the Contract Plans. Closure pours shall be clearly shown on all deck pour sequence diagrams. The Design Engineer shall calculate end rotations and translations for a bridge to determine if the requirements for this construction joint can be relieved.

2.2.4.3 Bearings

1. Standard Drawing Types:
   Standard bearing designs are available for Elastomeric Bearings and High Load Multi Rotational (HLMR) Bearings and are detailed on Standard Drawing Nos. BR-10 to BR-12.

   Steel reinforced Elastomeric Bearings shall be designed in accordance with Method B as outlined in the AASHTO LRFD BDS. Designs shall be coordinated with the notes and Specifications provided both in the NJTA Standard Drawings noted above, and in the Standard Specifications, when analyzing material properties and accounting for fabrication tolerances.

2. New Designs, Widenings and Retrofits:
   Bearings shall be designed in accordance with the appropriate provisions of Section 14 of the AASHTO LRFD BDS.

   Elastomeric Bearings which are fully vulcanized to a masonry plate and a sole plate are preferred where their use is practical and cost effective. Elastomeric Bearings with a sliding surface will be permitted on a case by case basis.

   HLMR Bearings shall be specified for all bridges that qualify as curved in accordance with Article 4.6.1.2.4 of the AASHTO LRFD BDS. HLMR Bearings should be considered and are encouraged for use in situations where skewed, shallow, highly cambered, or unconventional structure framing may induce torsion or transverse rotations at the bearing points.

Seismic Isolation Bearings shall be specified for use as dictated by seismic analysis. Adequacy of Seismic Isolation Bearings shall be evaluated in accordance with the appropriate provisions of the current edition of the AASHTO Guide Specifications for Seismic Isolation Design. Performance metrics for the bearings shall be presented on the Contract Plans.

Regardless of bearing type chosen, the Design Engineer shall evaluate the anticipated construction sequence and alert the Contractor via contract plan notations if temporary bracing, preload jacking for thermal displacement, or other special procedures are required to install the bearings without overstress.

3. Provisions for Substructure Movement:
   Appreciable displacement of bearings can result from settlement of fill under and behind flexible type abutments causing horizontal movement at the top of abutments, as well as small rotations of tall piers. 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 or Cross-Frame shall be positioned and designed to provide for jacking the end of the span. Sufficient additional expansion capacity shall be provided in the bearings to accommodate any anticipated substructure movement, and to 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 load plates so that removal does not entail the demolition of reinforced concrete substructures. Substructures shall be designed to allow adequate space for jacks or other devices for temporarily supporting the superstructure. Superstructures shall be designed to accommodate the loads imposed by these devices.

5. Provisions for Anchor Bolts and Pedestals:
   Anchor bolts shall be fully designed for Elastomeric Bearings. Force effects on anchor bolts to be designed by the bearing fabricator shall be considered where specifying HLMR or Seismic Isolation Bearings for use. Where large horizontal or longitudinal forces are anticipated due to multiple lines of fixed bearings at a single superstructure unit and/or where impact/thrust against guided expansion bearing guides or keeper plates is likely, these forces shall be accounted for in the design of the anchor bolts for Elastomeric Bearings, or included in the Bearing Design Tables provided for HLMR/Seismic Isolation Bearings.

Pedestals, rather than stepped bearing seats, shall be used to support bearings. A nominal height of 6” is preferred. The Design Engineer shall verify by design that a minimum 1’-6” depth of embedment for the anchor bolt is sufficient to secure the bolt into the reinforced concrete mass of the abutment stem or pier cap. Additional pedestal reinforcement, in addition to, or in lieu of extending the anchor bolt embedment, may also be considered where appropriate or required by design. See Exhibit 2-213 for Pedestal Reinforcement Details.

Anchor bolts may be installed by the Contractor via direct casting into the substructure unit concrete, drilling and grouting the bolts in place, or by casting-in oversize holes or preformed holes in the substructure concrete. Drilling and grouting of bars must be explicitly eliminated from use in the Contract Documents if the possibility of damaging the structural integrity of the substructure elements is anticipated. If net uplift or tension is anticipated at the anchor bolts, the casting of oversize holes or use of preformed holes in the substructure concrete shall be explicitly eliminated from use in the Contract Plans.

2.2.5 Substructure Design

2.2.5.1 Piers

1. See Exhibit 2-214 to 2-216 for general guidelines on Turnpike pier details.
   For all New Bridges, hammerhead style wall piers will be specified. Current market trends as of 2016 for labor versus concrete indicate that typical low level open bent pier construction is not cost-efficient. The cost to furnish and install the elaborate steel reinforcing bar arrangements present in the columns of multi-column bents can be considerable. This cost, combined with similar costs to reinforce, form, and place the concrete cap for an open bent pier, have rendered this type of pier uneconomical for low and midlevel structures.

   Open bent piers may be considered on a case-by-case basis where high level structures make this form of construction economical, or where obstructions within the footprint of the pier preclude the use of a solid-wall pier and footing.

   Open bent pier caps, where used, shall be designed such that they are structurally capable of functioning as simple spans between adjacent pier columns to allow for future staged bridge reconstruction. Cover to main reinforcement for pier caps of open bent piers under deck joints shall be 3” minimum.

   For existing bridges requiring rehabilitation, reconstruction or widening, Design Engineers shall utilize the pier type that matches that of the existing bridge.

All new and reconstructed or widened piers shall conform to the impact requirements of Subsection 2.2.2 of this Design Manual.

All reinforcement in all piers shall be galvanized.

2. Footings
Piers shall generally be designed with a continuous footing supporting all walls and /or columns, except where footings must be discontinuous to avoid features such as utilities or roadways. All piers, regardless of type, shall be founded on piles, drilled shafts, or competent rock. Soil bearing piers are not permitted.

All reinforcement in footing shall be galvanized.

2.2.5.2 Abutments

1. Design Criteria
Design guidance for Abutments has been separated between Section 2 and Section 5 of this Design Manual. Refer to Subsection 5.6.2 for additional guidance on retaining wall type selection and design. Unless noted otherwise, the AASHTO LRFD BDS as modified herein shall govern the design of the abutment's structural concrete and foundation.

For clarity, it is repeated in this Section of the Design Manual that the preferred abutment and wingwall type for both the Turnpike and Parkway bridges shall be conventional cast-in-place concrete cantilever semi-gravity retaining walls. Alternate abutment wall types addressed in this Section and Subsection 2.3 shall be considered and approved by the Authority on a case by case basis.

The preferred foundation type for conventional abutment and wingwalls is driven piles, selected and designed in accordance with Section 5 of the Design Manual. Alternate foundation types (spread footings, drilled shafts, etc.) shall be considered and approved by the Authority on a case by case basis.

Foundation and global stability of abutments shall be analyzed in accordance with Sections 10 and 11 of the AASHTO LRFD BDS and Section 5 of this Manual. The Design Engineer is advised that abutment design relies heavily on the input and coordination of the Geotechnical Engineer. Refer to Section 5 of this Manual for definition of, and responsibilities for the Geotechnical Engineer pertaining to the design of abutments.

Abutments shall be designed 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 where practicable in multi span bridges. Abutments shall be specified to be backfilled to the underside of the new approach slab prior to the placement of the superstructure or bearings.

Abutments shall be designed for this sequencing as a temporary 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 resistance factors may be increased by 25%.

For the purposes of standardizing limits, wingwalls for abutments shall be included as the portion of the retaining wall originating from the abutment stem and cast integrally with the abutment up to the joint with the adjoining retaining wall (if present). Where no such clear definition exists, the wingwall shall be defined as the portion of the wall extending 25’ from the corner at the abutment. This definition has been provided to establish uniformity with the Authority’s inspection inventory practices.

2. Approach Slabs
   Approach slabs shall be provided for all abutments and shall be constructed for the full width of the roadway including shoulders. Approach slabs shall be designed assuming they act as simple spans between the abutment backwall and the end of the approach slab for a distance of not less than 25’.

3. Alternate Abutment Walls
   On a case-by-case basis, the Authority may approve the use of alternate abutment configurations which deviate from the preferred configuration described previously in this Section. Where approved by the Authority, Alternate type abutments, such as Mechanically Stabilized Earth (MSE) or Prefabricated Modular (PM) Wall proprietary type systems, may be considered. Where approved by the Authority, analysis and recommendations justifying the deviation shall be included in the Phase A Report. Initial cost shall not be considered as a metric in this analysis.
   a. Proprietary type abutment systems shall be designed based on a 100 year service life.
   b. Refer to Subsection 2.3.3 for MSE and PM wall system requirements. Criteria stated therein shall be applied in such Abutment designs.
   c. The design of proprietary type wall systems supporting abutment caps shall take into account the anticipated movements and loads transmitted from the abutment caps. Plans and / or Specifications for the wall system shall clearly state the additional loads and / or movements that will be imposed on the wall system. On a Project specific basis, Design Engineers may need to consider certain design options, such as adding soil reinforcement straps to the abutment backwall cap, isolating the deep foundation from the wall face, requiring the proprietary system to be designed for higher pressures, etc.
   d. Design Engineers shall contact the approved wall system manufacturers during the design process to discuss Project specific design requirements and details to ensure there will be no conflicts during construction phasing.

e. Under no circumstances may abutment caps on proprietary walls be designed as soil bearing. All abutment caps shall bear directly on piles or drilled shafts assuming no vertical contribution from the proprietary wall system.

4. Integral and Semi-Integral Abutments
   a. Integral abutment design will only be permitted for non-skewed single span bridges.
   b. The joints between abutment stems and independent wingwalls shall always be oriented longitudinally, parallel to the bridge center line.
   c. Loose or compressible fill shall be used behind and beneath any wingwall not independent from the abutment stem.
   d. Provisions shall be made for expansion at the end of relief slabs by installation of sleeper slabs and joints, regardless of the bridge length.
   e. Single span bridges shall have a span length not exceeding 80’.
   f. The minimum reveal between the bottom of the superstructure and the top of the fore slope embankment (if present) shall be 1’-6”.
   g. If Mechanically Stabilized Earth (MSE) wall systems are utilized at integral abutment locations, the following provisions shall apply:
      i. The minimum clear distance between the back of the wall facing and the front edge of the abutment stem foundation piles or pile casings shall be 1’-6”.
      ii. Soil reinforcing straps shall be designed considering the additional loads transmitted from the piles to the reinforced soil backfill.
   h. Manholes, utility valve covers and drainage inlets shall be located beyond the limits of relief and sleeper slabs.

2.2.5.3 Scour Design

1. Bridge substructures and foundations shall be evaluated for scour conditions in accordance with the AASHTO LRFD BDS. Hydraulic and scour analysis shall be performed in accordance with Section 4 of this Manual.

2. In accordance with the AASHTO LRFD BDS, new and replacement bridges shall be designed for the scour condition for a recurrence interval that is expected to produce the most severe adverse condition, up to a 100 year maximum. Other existing bridges scheduled to be significantly rehabilitated or widened will be identified by the Authority on a Project by Project basis. Scour evaluations of existing bridges shall also take into account past history of floods in the Project area. The use of flood criteria greater than 100 years may be necessary and shall be evaluated on an individual bridge basis.

3. In accordance with the AASHTO LRFD BDS, new, replacement and widened bridges shall be checked for the scour condition for a 500-year flood.

4. In accordance with Section 4 of this Manual, a Scour Report shall be submitted which shall include scour countermeasure and resistance recommendations.

5. Scour considerations and / or countermeasures shall be selected and designed as directed in Section 5 of this Manual.

2.2.6 Design for Seismic Events

2.2.6.1 Design Specifications

Except as modified below, the seismic evaluation of all bridges shall be governed by the current editions of the following design codes:

- AASHTO LRFD Bridge Design Specifications, with current interims. (AASHTO LRFD BDS)
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, with current interims. (AASHTO LRFD SBD)
- FHWA Seismic Retrofitting Manual for Highway Structures, Publication No. FHWA-HRT-06-032, January 2006. (FHWA Manual)
- AASHTO Guide Specifications for Seismic Isolation Design (AASHTO GSSID)

The AASHTO Guide Specifications for LRFD Seismic Bridge Design offers a displacement-based design alternative to the force-based design methodology presented in the AASHTO LRFD Bridge Design Specifications. Displacement-based seismic design has the potential to offer a more economical bridge design, especially in regions of high seismic activity. However, the Turnpike and Parkway facilities are contained within a region of relatively low seismic activity where displacement-based designs have generally proven to offer minimal savings as compared to force-based designs. While the Design Engineer is not explicitly discouraged from using a displacement-based seismic design, it should be noted that the potential benefits of such a design may be negligible. However, where site specific spectra in problematic soils may arrive at high peak accelerations, a displacement-based approach to New Bridge design may be warranted.

2.2.6.2 General Considerations

The most common and significant hazard causing earthquake damage is ground shaking. Additional seismic hazards can also include ground failure, liquefaction, lateral spreading, differential settlement and land sliding. All New Bridges shall be designed to resist such hazards in accordance with the AASHTO LRFD BDS. All existing bridges which meet the criteria of Subsection 2.2.6.8 or are otherwise designated by the Authority shall be subjected to a vulnerability analysis and subsequent retrofit design (as required) which considers the above hazards.

2.2.6.3 Seismic Ground Shaking Hazard

For the purposes of both existing bridge vulnerability analysis and new bridge design, the following criteria shall be used when defining the seismic ground shaking hazard. The seismic ground shaking hazard is defined by the design response spectrum.

For the 1,000-year mean return period earthquake, bedrock ground motion parameters shall be taken from the AASHTO LRFD BDS seismic hazard maps and procedures. For the 2,500-year mean return period earthquake, bedrock ground motion parameters for the site shall be taken from the most recent USGS National Seismic Hazard Maps.

For both the 1,000-year and 2,500-year mean return period earthquakes, the design response spectrum shall be computed following the provisions of AASHTO LRFD BDS Article 3.10.4.

The site specific Procedure may be used for any bridge, but shall be mandatory for the following situations:

- Bridges 1,000’ or greater in length.
- Bridges with a deck area exceeding 50,000 sf.
- Bridges designated by the Authority as “Critical”.
- Anywhere a time history response analysis will be performed as part of the overall design / retrofit scheme.

2.2.6.4 Bridge Importance Classification

For the seismic design of New Bridges as well as the seismic vulnerability assessment and retrofit design of existing bridges, all bridges shall be classified as “Essential” bridges unless designated otherwise by the Authority. If the Authority elects to assign “Critical” importance classification to a bridge, the designation will be clearly stated in the Project scope of work.

2.2.6.5 Seismic Performance Criteria

The following seismic performance criteria shall apply for the design of New Bridges as well as for seismic vulnerability assessment and retrofit design of existing bridges. These criteria expand upon and supersede definitions in the AASHTO LRFD BDS and the FHWA Manual.

Bridge Classification	Considered Seismic Event	Mean Return Period	Probability of Exceedance	Acceptable Damage Level	Access Level
Essential	Single Level	1000 years	7% in 75 years	Minimal	Immediate
Critical	Upper Level	2500 years	3% in 75 years	Repairable	Limited
	Lower Level	1000 years	7% in 75 years	Minimal	Immediate

Post-seismic event acceptable damage levels are defined as follows:

- "Minimal" damage means that the bridge should have "essentially elastic" response, meaning minor inelastic response could take place. In reinforced concrete elements, post-earthquake damage should be limited to light flexural cracking. Permanent deformations are not allowed for primary structural members. Minor damage and permanent deformations are permitted in secondary members. No damage to expansion joints is permitted, except for the sealing gland, which may be considered sacrificial for the purposes of seismic performance evaluation.
- "Repairable" damage means that the bridge can be restored to its pre-earthquake condition without replacement of primary structural members. Inelastic response is permitted and may result in concrete cracking, concrete cover spalling, and yielding of reinforcement in concrete members. Where spalling or loss of concrete cover is anticipated, consideration shall be given to where loosened concrete may fall. Falling concrete over active roadways or populated areas will not be considered acceptable. Loosened concrete which may fall over median areas or in roadway shoulders will be considered acceptable. Limited damage will be considered acceptable in secondary members and non-structural components, including expansion joints, provided that such damage will not significantly damage attached primary members or allow the secondary members to fall free of the bridge. Permanent post-event deformations shall be small and no collapse will be permitted. Repairs, where required, shall be possible without completely closing the bridge to traffic, i.e., repairs can be performed with limited lane and shoulder closures. As a part of the Phase A report, the Consultant shall present their detailed seismic design criteria including an inventory of bridge members which are anticipated to receive damage, the anticipated extent of the damage with a conceptual repair scheme, a preliminary estimate of repair costs, and an anticipated construction schedule or time frame in which the repairs can be completed to the point that all active traffic lanes on the bridge can be restored to full service.

Post-earthquake access levels are defined as follows:

- Immediate access means that full service for all vehicles will be available within 72 hours following a design seismic event allowing for inspection and clearance of debris.

- Limited access means that service for emergency vehicles will be available within 72 hours following a design seismic event allowing for inspection and clearance of debris, i.e. steel plates may be required to span over failed joint areas or damaged deck areas. Full service to general traffic for all lanes shall be able to be restored within a matter of three months unless longer timeframes are permitted by the Authority.

2.2.6.6 Analysis for Earthquake Loads

Analysis requirements for earthquake loads presented herein apply to New Bridge design as well as existing bridge seismic vulnerability assessment/retrofit design.

Single Mode or Uniform Load analyses are permitted for all Essential bridges which will not be classified as “Irregular bridges”. Multi-mode analyses shall be used for all Irregular bridges. In addition to the provisions noted in the AASHTO LRFD BDS Article 4.7.4.3, the following bridge types shall be considered to be “Irregular bridges”:

- Bridges with any span curved in plan, where the definition of curvature is as described in Article 4.6.1.2.4b of the AASHTO LRFD BDS.
- Bridges designed with transverse Cross-Beam pier cap elements.
- All bridges designated as “Critical” by the Authority.

Note: seismic analysis does not apply to single span bridges, which are exempt from seismic analysis, with the exception of checking bridge seat length, in accordance with Section 4.7.4.4 of the AASHTO LRFD BDS. This exception does not apply to viaduct bridges composed of a series of single span superstructures.

Extreme Event I Load Combination in Table 3.4.1-1 of the AASHTO LRFD BDS shall consider a Live Load Factor (gEQ) of 0.50. Similarly, 50% of live load lane forces shall be considered simultaneously with dead load and seismic effects when the design and/or analysis is performed in accordance with AASHTO LRFD SBD or the FHWA Manual. The notional truck or tandem portion of the HL-93 or TP-16 live load model shall not be considered. Note that the inertial effects of the live load shall not be included in the dynamic analysis.

2.2.6.7 Design of New Bridges

All New Bridges shall be designed to incorporate minimum support bridge seat lengths, connection designs, and column design / ductility details required for Zone 2 criteria, per the provisions of the AASHTO LRFD BDS. New Bridges designed using AASHTO LRFD SBD shall follow, at a minimum, the design and detailing requirements of Seismic Design Category B. Single-span bridges shall be designed in accordance with Article 3.10.9.1 of the AASHTO LRFD BDS. Forces used in the design of connections between the superstructure and the substructure need not be considered in the design of the substructure for bridges located in Zone 1.

New Bridges designated as “Critical” shall be designed to resist both the lower level and upper level events while maintaining the post-earthquake acceptable damage levels and access levels as defined in Subsection 2.2.6.5.

These general considerations provide for a rational approach to bridge designs that allows the use of simplified analysis methods for the majority of bridges in the Turnpike and Parkway inventories, but requires the inclusion of code mandatory detailing, which offers significant increases in seismic performance, ductility, and redundancy at a relatively incidental increase to the bridge construction cost.

2.2.6.8 Vulnerability Assessment and Retrofit Design

The FHWA Manual shall be used as a guide regarding evaluation procedures and upgrade measures for retrofitting existing seismically deficient highway bridges.

Unless directed otherwise by the Authority, a seismic retrofit shall be considered for all existing Bridge Rehabilitation Projects which meet the following criteria:

- Anticipated Project work includes increasing the bridge deck area by more than 25% and / or replacing the entire bridge deck.
- Anticipated Project work includes replacing or repairing more than 25% of the superstructure bearings.

When the estimated cost of the proposed seismic retrofit strategy exceeds 25% of the estimated replacement cost of the bridge, replacement of the bridge shall be considered.

In addition to the above criteria, the Design Engineer is responsible for rational consideration of all existing bridges within a Project for seismic retrofit eligibility. Development of a retrofit scheme should be considered where the anticipated scale and type of work to an existing bridge offers the opportunity to include cost effective seismic retrofitting measures in the Project.

When an existing bridge has been determined to be a candidate for seismic retrofitting, a conceptual retrofit strategy shall be included as part of the Phase A submission. When the estimated cost of the proposed retrofit strategy indicates that a full bridge replacement may be warranted, the Authority shall be contacted prior to the Phase A submission.

When existing bridges designated by the Authority as “Critical” are investigated for retrofit design, they shall be analyzed to resist both the lower level event and upper level events for maintaining the post-earthquake acceptable damage levels and access levels as defined in Subsection 2.2.6.5. Retrofit strategies shall be prepared for both the lower level event and the upper level event, including a cost estimate for each strategy. Both strategies and their corresponding cost estimates shall be presented as part of the Phase A report and shall include a recommendation to retrofit the structure to either the lower level or the higher-level event.

The Authority reserves the right to reduce the design seismic event to 500 years where the candidate bridge is anticipated to have a remaining service life of 35 years or less.

The majority of existing bridges should not be expected to meet the force and ductile detailing requirements set forth in the AASHTO LRFD codes noted above, as many of these existing bridges were designed with little or no provision for resistance to seismic hazards. The inventory of the Authority's existing bridges generally has limited ductility and are incapable of sustaining stable inelastic cyclic response, which is the basis of current seismic design provisions for New Bridges.

For existing bridge seismic retrofit evaluation, Method C: Component Capacity/Demand Method, as described in Subsection 5.4 for Seismic Retrofit Category C and D, and Appendix D of the Retrofitting Manual shall be used, at a minimum.

Nonlinear static and / or dynamic analyses are recommended, but not required, where bridges with ductile details are to be evaluated, or where member strengthening and/or ductility enhancement are considered as part of the retrofitting concept.

Seismic retrofitting of existing bridges constitutes a substantial structural alteration. The Design Engineer shall perform a complete LRFR load rating analysis of the as-retrofitted bridge in accordance with the NJTA Load Rating Manual unless directed otherwise by the Engineer.

Isolation strategies, if employed, shall be designed in accordance with the AASHTO GSSID. This document is explicitly intended to function in concert with the AASHTO LRFD BDS and the AASHTO LRFD SBD. The use of Load Factor Design or Allowable Stress Design methodologies in concert with these Specifications is not permitted.

2.2.7 Computer Software

(For Design Purposes Only – See Section 2.4 of the NJTA Load Rating Manual for Load Rating Computer Software Requirements)

2.2.7.1 Guidelines on Use

The use of computer software has become a valuable tool to the Design Engineer. However, even the best computer software cannot replace good engineering judgment and design practice. In addition to the guidelines noted below, it is recommended that the checked results be reviewed by senior structural engineers or technical managers as part of the Engineer's QA/QC process.

Design software, with version number and date of release, shall be clearly indicated in the general notes on the Contract Plans. The Authority's Engineering Department approval will not relieve the Design Engineer of the responsibility for the use of the program. The Design Engineer assumes full responsibility for the logic and results of the program.

The following guidelines shall be followed when using any design software:

1. Program input and output shall always be checked by a second Design Engineer. All input and results shall be furnished as .pdf documents and placed in the design calculations. Design Engineers shall be responsible for verifying that the results of the computer software are correct. Verification shall be in the form of a second computer program or sufficient hand calculations to verify results.

2. All computer analysis and design shall be performed under the direct supervision of a structural engineer familiar with the computer software program who has direct experience designing at least two (2) bridges of similar scale and complexity with the software.

3. When utilizing spreadsheets, Mathcad, or software written by or obtained from external sources, the Design Engineer and checking Engineer shall thoroughly check the language and/or formulas to assure the integrity of the structural analysis and/or design.

4. Design calculations shall include as much program documentation as required to ensure that the program input and output can be interpreted by the Authority and future Engineers.

5. Software generated design calculations shall always be validated by hand calculations. At a minimum, the Design Engineer is required to prepare and submit the following hand calculations to validate the software as a part of the overall design calculations submission:

    - Proof calculations for section properties for all Stringer, Girder, Floorbeam, Cross-Frames for curved bridges, and Cross-Beam elements. Calculations shall be provided for non-composite, short-term composite, and long-term composite properties, as applicable.
    - Proof calculations for live load distribution for shear and moment, as applicable.
    - Proof calculations for unfactored live load moments and shears at controlling locations.
    - Proof calculations for field splice and bolted connections where design is performed via software.

Where Finite Element Modeling (FEA) analysis makes the above hand-calculation verification not practicable, the Authority may allow the use of an independent FEA analysis using separate software as sufficient validation.

2.2.8 Permits

For permit requirements to be considered during design, reference is made to the NJTA Procedures Manual.

2.3 RETAINING WALLS

2.3.1 General

Design guidance for retaining walls has been separated between Section 2 and Section 5 of this Design Manual. Refer to Subsection 5.6.2 for additional guidance on retaining wall type selection and design.

For clarity, it is repeated in this Section of the Design Manual that the preferred retaining wall type shall be conventional cast in place concrete semi-gravity retaining walls for both the Turnpike and Parkway roadways. Alternate retaining wall types may be considered and approved by the Authority on a case by case basis.

The preferred foundation type for the conventional retaining walls is driven piles. Alternate foundation types (spread footings, drilled shafts, etc.) may be considered and approved by the Authority on a case by case basis.

2.3.2 Conventional Retaining Structures

1. Foundation and Stability Design
   Foundation and global stability of the retaining structures shall be analyzed in accordance with Sections 10 and 11 of the AASHTO LRFD BDS and Section 5 of this Manual. The Design Engineer is advised that retaining wall design relies heavily on the input and coordination of the Geotechnical Engineer. Refer to Section 5 of this Manual for definition of, and responsibilities for the Geotechnical Engineer pertaining to the design of conventional retaining structures.

2. Conventional Cast in Place Wall Minimum Dimensions
   The minimum thickness at the base of any cast in place concrete wall shall be 12” for walls up to 10’ high, 15” for walls up to 14’ high, and 18” thick for walls higher than 14’. Low walls should be designed with a vertical rear face and higher walls should be battered or stepped, 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. Tops of walls shall be no less than 12” in thickness.

   Footings for conventional cast in place concrete walls shall be no less than 24” thick for soil bearing foundations and 36” for pile bearing foundations with a minimum embedment of 12” for all pile types.

3. Refer to Section 5 of this Manual for the design of driven cantilever walls, such as sheet pile and soldier pile and lagging type walls. These types of

walls should only be used in ‘cut’ situations where the intent of the work is to install the wall and then remove the material in front of the wall to expose down to the finished grade.

2.3.3 Alternate / Proprietary Retaining Walls

1. As approved by the Authority, Design Engineers may consider the use of proprietary retaining wall systems. Proprietary retaining wall systems, Mechanically Stabilized Earth (MSE) Walls and Prefabricated Modular (PM) Walls, generally are more cost effective in specific situations and often provide a shorter construction time than conventional cast-in-place reinforced concrete cantilever retaining wall systems.

   MSE walls are generally not desirable where large excavations are required to construct the wall (cut wall), as the additional cost of temporary walls to support the cut outweighs the cost of other wall types.

2. The current NJTA Standard Specifications provide a list of Wall systems and design and construction criteria that shall be used as guidance when developing the Project Specific Specifications.

3. The Design Engineer shall compare retaining wall types and listed systems to determine which wall configurations best meet Project objectives, i.e., structure cost, functionality, construction time, aesthetics, durability, and other Project specific parameters. Analysis summary and recommendations should be included in the Phase A Report.

4. Alternate retaining walls shall be generically presented in the Contract Documents using the Common Structure Volume (CSV) concept. The CSV is the volume into which all potential wall systems can be placed. All work items required to construct the wall and all appurtenances, ancillary items and all work to complete the Project located within the CSV are not measured but are included in the pay item for the retaining wall. The CSV concept allows the quantification of pay items outside the CSV, such as excavations, embankments, etc., such that those quantities will not vary due to the proprietary wall system selection.

2.3.4 Proprietary Wall Design Guidelines

Except as modified by the current New Jersey Turnpike Authority Standard Specifications and the current New Jersey Turnpike Authority Design Manual, Section 2, designs of MSE and Prefabricated Modular Wall retaining wall systems shall conform to the standards noted in Section 5 of this Manual, with the exception of moment slabs supporting barriers, which shall be designed in accordance with the below document:

2002 AASHTO Standard Specifications for Highway Bridges, 17th Edition and as Modified by the 2007 New Jersey Turnpike Authority Design Manual, Section 2, through Current Updates, for Modifications Appropriate to Load Factor Design (LFD).

a. Design Method
   Load Factor Design (LFD):

Internal Strength and Stability for Barrier Parapet and Moment Slab System

Allowable Stress Design (ASD): External Stability for Moment Slab

IMPACT LOAD

Load Factor Design (LFD) and Allowable Stress Design (ASD): Vehicular Impact Load applied to the Barrier Parapet and the Moment Slab System shall be per AASHTO Standard Specification, Article 2.7.1.3.

Load and Resistance Factor Design (LRFD): Vehicular Impact Load applied from the barrier parapet and moment slab system to the proprietary wall shall be per AASHTO LRFD Bridge Design Specifications, Article 11.10.10.2.

The impact requirements of AASHTO LRFD Article 3.6.5 are waived for MSE or PM abutment walls which envelop pile supported abutment seat beams and for full height cast in place conventional abutments.

b. Design Engineers shall be responsible for developing preliminary design and Contract Documents for MSE Walls including all geometry and loading conditions. Generally speaking, it is the responsibility of the Design Engineer to define the alignment of the retaining wall, indicate anticipated leveling pad steps below the grade, establish Common Structure Volume criteria, and establish elevations of the grade and top of wall at regular stations along its length. Constructability and coordinating with external conflicts such as utilities and adjacent structures is also the responsibility of the Design Engineer. Establishing internal wall design parameters, backfilling requirements, and internal material Specifications is the responsibility of the Geotechnical Engineer. Evaluating external and internal stability (where appropriate) and any associated ground improvements is also the responsibility of the Geotechnical Engineer.

c. Contractors, material suppliers and/or wall vendors will be responsible for developing the final design for MSE Walls including but not limited to, the following:

- Evaluate Internal Stability for Strength limit state and extreme event, and Confirm External Stability
- Select type of soil reinforcement
- Define critical failure surface (for selected soil reinforcement type)
- Define unfactored loads
- Establish vertical layout of soil reinforcements
- Calculate factored horizontal stress and maximum tension at each reinforcement level
- Calculate nominal and factored long-term tensile resistance of soil reinforcements
- Select grade (strength) of soil reinforcement and/or number of soil reinforcement elements at each level
- Calculate nominal and factored pullout resistance of soil reinforcements, and check established layout

- Check connection resistance requirements at facing
- Estimate lateral wall movements (at service limit state)
- Check vertical movement and compression pads
- Design of Facing Elements
- Confirm Overall Global Stability
- Confirm Compound Stability
- Confirm Wall Drainage Systems – working drawings
- Subsurface drainage
- Surface drainage
- Where design parameters are modified by the Contractor, material supplier and/or vendor, they shall also evaluate the external stability for revised wall configurations

Review of shop drawings and designs as provided by the Contractor is the shared responsibility of the Design Engineer and the Geotechnical Engineer.

d. For additional guidance and information, the Design Engineer is referred to the following resources:

- Section 5 of this Design Manual.
- Standard Specifications
- NJTA Sample Plans and Exhibits 2-500 Series of this Manual

e. For MSE wall systems that are located under roadways, a high density polyethylene geo-membrane shall be placed below the pavement and just above the first row of reinforcements to intercept any flows that may contain deicing chemicals. The membrane shall be sloped to drain away from the wall facing. Reference is made to NJTA Standard Specifications for type of material to be used. Refer to the Exhibits at the end of this Section for further details.

Drainage considerations for MSE Walls are discussed in Subsection 5.6.2.3 of this Design Manual.

Where MSE Walls will be constructed in or adjacent to salt or brackish water, refer to Subsection 5.6.2.9 of this Manual for guidance regarding corrosion life estimation or use of stainless / nonmetallic reinforcement.

2.4 CULVERTS

The design of cast-in-place concrete Culverts, precast concrete box Culverts, precast concrete arch structures and precast concrete three-sided rigid frame structures shall conform to Subsection 5.14.5 of this Manual and Section 12 of the AASHTO LRFD BDS.

Culverts shall be constructed as box Culverts or three-sided rigid frames with Class “A” reinforced concrete. Corrugated metal Culverts are not permitted. At a minimum, Culverts shall be of sufficient length so that the full roadway section, including shoulders and berms, can be maintained, including an additional 12’ of available space for Authority Maintenance forces to traverse the waterway when landscaping.

Precast culverts are permitted. Design Engineers shall contact precast manufacturers during the design to discuss Project specific requirements and details to ensure there will be no conflicts during the construction phase. Precast Culverts shall not be used when the top slab is to be used as a riding surface.

2.5 SIGN SUPPORTS

The various types of signs described in this section are either ground mounted or on overhead sign structures. Each of these general categories is sub-divided into various support methods:

1. Ground Mounted:
    - Small Highway Signs (<50 sf)
    - Large Highway Signs (≥50 sf)

2. Overhead Type Structures:
    - Span Type Structures
    - Cantilever Type Structures
    - Butterfly Type Structures
    - Bridge Mounted Structures

2.5.1 General Design Criteria

All Sign Support Structure designs shall be completed in accordance with the Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 6th Edition, with 2015 Interim Revisions (AASHTO LTS), except as noted below. For sign placement layout guidelines and sign panel requirements, see Section 6 of this Manual.

1. Ground Mounted
    a. Small Highway Signs (<50 sf)

    U-Channel Post
    U-channel (flanged) posts are only to be used for mile markers and delineators on the Turnpike and Parkway. Refer to the DE Standard Drawings for design details.

    Turnpike – Timber Post
    See Standard Drawings SL-1 to SL-7 and SI-43 to SI-46.

    Parkway - Timber Post
    See Standard Drawings SL-1 to SL-7 and SI-43 to SI-46.

    All timber post sign structures included in Standard Drawings SI-43 and SI-44 are designed for breakaway in a vehicular collision and do not require roadside protection.

    Turnpike - Single Aluminum Post
    Use of timber posts is preferred. Many signs are of sizes which may be too large for a single timber post support, but are impractically sized for multi column support. These signs shall be installed on a

single extruded aluminum tube having an outside diameter of 4” and a wall thickness of 0.250” following the details as shown in Standard Drawing SI-13. Post sizing shall be validated by design computations.

b. Large Highway Signs (≥50 sf)

Turnpike - Multi-Timber Post
See Standard Drawings SL-1 to SL-7 and SI-43 to SI-47.

All timber post sign structures included in Standard Drawings SI-43 and SI-44 are designed for breakaway in a vehicular collision and do not require roadside protection.

Turnpike - Multi-Aluminum Post
Timber posts are preferred. Aluminum post supports shall only be used as directed or approved by the Authority.

Post spacing and limiting panel sizes are shown on Standard Drawing SI-26. Foundation details are shown on Standard Drawing SI-22.

Where large diameter posts and/or heavy wall thicknesses are required for an atypical installation, it is encouraged to add an additional post, whereby a smaller tube of more commonly available diameter and thickness of wall may be utilized.

Parkway - Multi-Timber Post / Pole
See Standard Drawings SL-1 to SL-7 and SI-43 to SI-47.

Timber double post sign structures included in Standard Drawing SI-44 are breakaway designs and are required when sign structures are to be placed within the roadway clear zone and lack roadside protection measures.

Timber double and triple post sign structures included in Standard Drawing SI-45 are non-breakaway designs and shall be provided with roadside protection measures or shall be located outside of the clear zone.

It is not necessary to perform design computations for sign panel and post configurations shown on the Standard Drawings. Where custom designs are required for configurations not shown on the Standard Drawings, the following wind loading criteria shall be followed when computing loads per AASHTO LTS Section 3:

- Wind Speed of 110 mph
- $K_z = 0.87$
- 10 year anticipated design life
- Note ‘a’ below Table 3-2 is waived

The decision to use a breakaway or non-breakaway sign structure shall be made during design. The Design Engineer shall consider using a breakaway sign structure when reasonable with regards to sign location and message. Non-breakaway signs requiring additional roadside protection may only be installed within the roadway clear zone if directed by the Authority.

2. Overhead Type Structures

a. Span Type Sign Structures

Sign structure type and design varies between the Authority's Turnpike and Parkway roadways. Strictly for reference, Turnpike sign structure design is driven by economy of materials and cost, hence the use of uncoated weathering steel structures composed of commonly available pipe sections. Parkway sign structure design is driven by the need to conform to the historical appearance requirements for the roadway, which does explicitly cite the square weathering steel HSS Vierendeel structures as essential to the character of the roadway.

The Authority exclusively uses weathering steel for all sign structures, regardless of the roadway. This is done as much for economy as it is for aesthetic appearances. Specific permitted grades and Specifications for various weathering steels are published on the relevant Standard Drawings.

Uncoated weathering steel is generally more cost effective than paint or galvanized coatings. Furthermore, where other owners may specify hot dip galvanizing, their component sizes are limited by commonly available galvanizer dipping kettle sizes. Weathering steel structures are not limited by kettle size and can be made as large as structural efficiency dictates, hence why Authority structure trusses are 6' wide and larger, and adjacent agency structures are limited to 5' wide or less. This information is provided both as historical background and as an explanation why weathering steel structures cannot be substituted with galvanized structures in the event that weathering steel shapes or plates are difficult to procure for fabricators. Authority structures are simply too large to fit in commonly available galvanizer dipping kettles.

Sign structure placement shall generally locate the centerline of one end frame and foundation in the median between any adjacent roadways whenever possible and practical. The use of single span sign structures that span over multiple mainline roadways is prohibited, unless directed by the Authority Engineer. Additionally, sign structures located in any adjacent roadways shall be spaced apart, or staggered, a minimum of 35' as measured centerline to centerline of end frames in a direction parallel to traffic.

A minimum underclearance to the lowest obstruction (sign panel or structure member) of 17'-0" over the high point of the roadway shall be provided for all static sign structures and 18'-0" for Variable Message Sign (VMS) or Changeable Message Sign (CMS) structures.

Sign structures with static messages generally shall not be lit unless shown to require it as outlined below. Where lighting is not required, lighting brackets, handrail, and walkway shall be omitted. Turnpike sign structures shall be fabricated with wiring provisions for lighting, should it ever be required, or should the sign structure be re-used at a different location in the future.

Static sign panels on sign structures without lighting shall be placed such that the physical sign panels (not the text) are bottom edge justified with the bottom edge of all sign panels meeting the 17'-0" minimum clearance to the highpoint of the roadway limit. Sign structure end frame heights shall be chosen such that the truss is located as nearly as practical to the mid-height of the largest sign on the structure.

Static sign panels on sign structures with lighting shall be placed such that the physical sign panels (not the text) are vertically centered on the truss. Centerline elevation of the truss shall be placed assuming an 18" additional roadway clearance (total of 18'-6" from sign edge) to allow for the sign lighting support brackets. Where signs are to be provided with lighting, the tallest available standard end frame shall be used. Where it is not possible to use the standard end frames, the Design Engineer shall design a special end frame of the required height.

<u>Turnpike Sign Structures General Criteria</u>
Turnpike sign structure layout shall conform to Exhibits 2-401 to 2-404 shown in this Section of the Design Manual and Standard Drawings SI-13 through SI-25. The Standard Drawings for the span-type sign structures provide three (3) standard heights of end frames and six (6) standard lengths for box truss sections. Span lengths from 45' to 135', in increments of 5' can be obtained by using the standard box truss sections in appropriate combinations, as indicated on the Standard Drawings. Sign structure spans shall be set in increments of 5' wherever possible. Where this is not possible, the standard truss sections shall be combined with a minimum number of special sections to obtain the necessary length.

<u>Parkway Sign Structures General Criteria</u>
Parkway sign structure layout shall be in accordance with Exhibits 2-405 to 2-408 and Standard Drawings SI-28 through SI-34, SI-39, SI-40 and SI-42. Standard Drawings are organized into two ranges of standard spans with 60' to 90' span structures being of single plane Vierendeel truss design, and 91' to 156' span designs being of two plane (box) Vierendeel truss design. Parkway sign structures are typically custom fabricated for each location and are not similar to the modular construction typical of Turnpike signs.

The Design Engineer is made aware that large signs spanning Parkway roadways can potentially create oversize / overweight sign structure component shipping issues. Field splices for the trusses shall be located to accommodate shipping to site on the Parkway with the understanding that portions of the Parkway roadway, particularly above Milepost 105, have tight radius access ramps and low vertical clearances.

b. <u>Cantilever Type Sign Structures General Criteria (Turnpike and Parkway)</u>

This type of sign structure is used to support guide signs adjacent to deceleration lanes in the vicinity of ramp gore areas and may also be used to support overhead signing on narrow roadways and ramps. The sign panel should be located over the lane to which the message applies. The length of cantilever arms shall be detailed to suit the width of the sign proposed to be mounted on the arm. The sign panel shall extend to 6" beyond the outer edge of the arm. The provisions noted above for overhead span sign structures shall also be maintained for cantilever sign structures, as applicable, and in addition to the provisions noted below.

The layout of Turnpike Sign Structures shall be in accordance with Exhibit 2-404. Standard Drawings for Turnpike cantilever sign structures are available as Standard Drawing SI-18A and SI-18B.

The layout of Parkway Sign Structures shall be in accordance with Exhibit 2-408. Standard Drawings for Parkway cantilever sign structures are available as Standard Drawings SI-35 to 38, and SI-41.

c. Butterfly Type Sign Structures General Criteria (Turnpike and Parkway)
This type of sign structure is used to support signs on the Turnpike and Parkway between a pair of adjacent roadways carrying traffic in the same direction. With the approval of the Authority's Engineering Department, a butterfly type structure may be used at the nose of a ramp split and other locations required by design where the use of a span-type structure is not feasible. Structural details of this type of sign structure are similar to cantilever-type sign structures except that cantilever arms are provided on each side of the support post. The layout of the sign structure shall be in accordance with Exhibit 2-404 and 2-408. Standard Drawing for Turnpike butterfly sign structure is available as Standard Drawings SI-18C.

2.5.2 Sign Structure Design

In the event that the standard sign structure configurations do not meet the specific needs of the Project, a custom design will be required. The Design Engineer will follow the provisions of the AASHTO LTS with the following provisions:

Wind Load
Basic Wind Speed as defined in Section 3.8.2 of the AASHTO LTS shall be defined as a minimum of 100 mph for all Turnpike and Parkway sign structures.

Wind Drag Coefficients, Cd, shall be determined in accordance with Table 3-6 of the AASHTO LTS. Where the exact dimensions of a sign panel or sign structure element cannot be determined, the Design Engineer shall select the most conservative Drag Coefficient available for the most appropriate element type denoted in Table 3-6 of the AASHTO LTS. When determining the Wind Drag Coefficient for square shaped tubular truss members, the radius (r) denoted in Table 3-6 of the AASHTO LTS may be assumed as twice the thickness of the square shaped tubular member. VMS/CMS panels may use a drag coefficient of 1.2 for design.

The Height and Exposure Factor, KZ, shall be no less than 0.94 for all parts of the sign structure under normal exposure. Higher values of KZ shall be considered, in accordance with Table 3-5 of the AASHTO LTS, when structures are situated in abnormally exposed conditions on high embankments or on bridge piers or superstructures. The standard designs have been prepared using a value of KZ of 1.0. The Design Engineer shall check the design of the standard sign structures for the particular use intended wherever it is determined that a higher value of the KZ is appropriate.

The Wind Importance Factor and Velocity Conversion Factors defined in Tables 3-2 and 3-4 of the AASHTO LTS shall consider a 50 year design life, unless otherwise directed by the Authority.

<u>Ice Load</u>
Ice load shall be considered as 3.0 psf and shall be applied as directed in Section 3.7 of the AASHTO LTS.

<u>Fatigue Loading</u>
Fatigue loading shall be considered for all span type, cantilever and butterfly sign structures. Dead loads and Ice loads shall not be considered to act in addition to the specific event fatigue loading. The Importance Factor for all fatigue wind loads shall be taken as 1.0.

<u>Foundation Design</u>
Foundations for sign structures shall be fully designed by the Design Engineer. Spread footing and pile foundations shall generally conform to the details shown on Standard Drawing SI-22. Differences in Parkway sign structure pedestal dimensions shall be accommodated in the design of spread footings and pile foundations. Drilled Shaft foundations shall conform to the details shown on Standard Drawings SI-22A, SI-22B and SI-39 through SI-41.

In accordance with the Authority's Procedures Manual and the Design Manual, a soils investigation shall be conducted for all foundations in existing ground and a Geotechnical Engineering Report shall be submitted. The foundation shall be supported on a spread footing, driven piles, or drilled shafts of 30" minimum diameter, with the required analysis and recommendations included in the Geotechnical Engineering Report.

All foundations for sign structures shall have the bottom of the base plate, shown as "Elevation A, AL or AR" on Exhibit 2-401 to 2-408 and "Elevation A" on Standard Drawings SI-14 through SI-41, set at 4' higher than the highest point of the roadway cross section at the transverse centerline of the structure. This elevation shall be the same for both pedestals. Sign structure pedestals may be constructed on three types of foundations; spread footing, driven pile supported footing, or drilled shafts. Drilled shaft foundations have gained popularity with Contractors in recent years and are the preferred foundation type.

Where foundations are to be constructed adjacent to existing Turnpike or Parkway pavement, including shoulders, the width of the foundation shall preferably be that which will not require the removal of paved shoulders for its construction. When excavation for a spread footing or pile bearing footing is required adjacent to Turnpike or Parkway pavement, these excavations shall be protected by steel sheet piling which shall be left in place. When determining the most cost-effective foundation type, the cost of the sheet piling to remain, if required, shall be considered by the Design Engineer.

Where spread footings or driven pile supported footings are to be used, the elevation of sign structure foundations shall be set so that the soil cover over the top of the footing at the centerline of the stem shall be at least 4'. Where these footings are located in embankment slopes, the minimum cover at the outside of the footings shall be 2'. Where it is deemed appropriate to found sign structure pedestals on drilled shafts, the bottom of the pedestal shall be a minimum of 2' below grade.

Where footings carry eccentric gravity loads, as will be the case for cantilever or butterfly signs, the net vertical force and overturning moment at the centerline of the footing shall be considered. The effects of torsion shall also be considered in the foundation design of all butterfly and cantilever structures.

1. Loadings for Design
   The foundations of span-type sign structures shall be designed for a band of signs having a length extending over the entire width of the roadway plus shoulders and having a height equal to the maximum height of sign that can be accommodated by the end frame of the sign structure. The maximum area of sign that need be used for design of structures shall be as specified on the Standard Drawings. Where the actual sign panel(s) width x length is unknown, the most conservative Wind Drag Coefficient (Cd) for sign panels shall be selected from Table 3-6 of the AASHTO LTS.

   The foundations for standard cantilever-type sign structures shall be designed for a band of signs having a length of 18'-9" and a height equal to the maximum height that can be accommodated on the sign structure.

   All Contract Plans prepared for Sign Structures shall have the design sign area clearly published on the General Plan and Elevation Plan sheet.

   The following dead loads may be used for the components of standard Turnpike specific static sign structures:

Truss spans up to 100'	125 plf
Truss span greater than 100'	145 plf
Cantilever/Butterfly Arms	190 plf
End frames and posts	5,000 lbs
Flat sign panels w/stringers	3 psf
Hangers, luminaires, and supports	40 plf

Due to the simplified and more variable nature of Parkway sign structures, individual weight computations will be required for each Parkway structure.

Foundation design loads for VMS and CMS style structures are shown on the Standard Drawings pertaining to those structures.

2.6 LIGHTING

Sign Lighting

Sign structures with static messages generally shall not be lit unless required via lighting warrant analysis performed in accordance with Subsection 7.2.1 of this Design Manual. Guidance associated with the illumination of sign panels is discussed in Section 7 of this Manual.

Roadway Lighting on Bridges

Standard Drawing E-02 includes details for fatigue resistant steel lighting standards, which are to be used exclusively on bridges.

Lighting standards placed on structures shall be located at or near piers and abutments. When locating the lighting features at piers or abutments is not practical, the maximum offset from a pier or abutment bearing line shall be 25% of the length of the span in which the lighting features is to be located. Refer to Section 7 of this Manual to determine bridge lighting requirements.

2.7 NOISE BARRIERS

Noise barriers are only to be provided on new construction (new roads) or on widening projects unless otherwise directed by the Authority. Refer to the Authority's *Policy for Construction of Noise Barriers in Residential Areas* for further guidance.

2.7.1 Preliminary Considerations

1. As a part of the Phase A work for a Project where a Noise barrier is being considered, the following items shall be addressed:
   - Identification of limits of noise barriers.
   - Identification of buried utilities, drainage facilities, and overhead utilities impacted.
   - Protection of the noise barrier when located within the roadway clear zone.

In general, a noise study shall either be performed by the Consultant or will be provided to the Consultant by the Authority in order to determine the height of the noise barrier and recommend noise abatement coatings and/or construction materials. This study will be performed prior to commencement of structural design work.

Where utilities may conflict with the proposed noise barrier alignment, efforts shall be made to realign the barrier or consider revised foundation installation methods (for buried features) prior to investigating relocation of the utilities. The Design Engineer is made aware that drainage facilities or utilities that run parallel to the noise barrier may require access by maintenance forces or utility owners at a future point. Design of the noise barrier shall take this into account by the creation of access points as necessary.

2.7.2 Design Criteria

1. Ground Mounted Noise Barriers - AASHTO LRFD BDS Section 15 shall be used for structural design. Foundation design shall be in accordance with AASHTO LRFD BDS Subsection 15.9 and Section 5 of this Design Manual.

2.7.3 Functional Requirements:

1. Guide rail or concrete barrier curb shall be installed when the noise barrier is located within the clear zone (see Section 1 of this Manual for more information).

2. Stopping sight distance criteria shall apply in determining the location of a noise barrier. Horizontal clearances which reduce the stopping sight distance shall be avoided. In extreme cases where reduced stopping sight distances may be warranted, justification shall be provided and a Design Element Modification Request approval will be required from the Authority's Engineering Department.

3. Noise barrier heights shall be established based on the approved noise study.

4. When the tops of Noise barriers must be stepped, the maximum height of step should not exceed 2'.

5. Noise barriers obstruct light as well as noise. Consideration shall be given to possible roadway icing and other induced environmental conditions caused by shade from the Noise Barrier.

6. Provisions may be necessary to allow access to fire hydrants on the opposite side of the noise barrier. The Consultant shall confer with local fire and emergency officials regarding their specific needs.

7. Noise barriers shall be designed to retain all anticipated differential fills plus an additional 2' of soil as a minimum.

8. Noise Barrier coatings and surfaces are designed primarily to abate sound. They are not designed to be resistant to de-icing salts or be subjected to accumulated plow spoils. Where it may be operationally impractical to place Noise Barriers where they will not be subjected to these conditions, consideration should be given to detailing the bottom 4' of the noise barrier as a solid concrete parapet or wall.

2.7.4 Maintenance Considerations:

1. In some urban areas, noise barriers may be subjected to graffiti being placed on their surfaces. In these locations, the surface texture selected should be such that it is difficult to place the graffiti or such that the graffiti is easily removed. Noise barriers with rough textures and dark colors tend

to discourage graffiti, but final texture and colors should be coordinated with the community or stakeholders protected by the barrier. In the absence of external preference, noise wall panels shall be textured using a 'fractured fin' style form liner similar to Greenstreek form liner number 367. Noise Barrier panels shall be tinted to match color 30045 of the SAE AMS-STD-595C. Noise Barrier surfaces shall be detailed with a silicone based anti-graffiti coating such as recommended by the Qualified Products List (QPL).

2. Access to the back side of the noise barrier should be provided for inspection, litter control, soil erosion monitoring, grass mowing, drainage repairs, maintenance, etc. In subdivision or residential areas, access may be via local streets, where available. If access is not available via local streets, openings in the noise barriers shall be provided as follows:
   - Openings shall be provided at a spacing of 1,000' measured along the roadway; maintenance openings are not required for noise barriers with lengths shorter than 1,000' that can provide access to the residential side from one of the ends.
   - Location of the access openings should be coordinated with the appropriate agency or landowner.
   - Offset barriers concealing the access opening must be overlapped a minimum of 2 1/2 times the offset distance in order to maintain the integrity of the noise attenuation of the main barrier.
   - The offset of the barriers shall allow for a 12'-0" wide vehicular gate.
   - This gate or the barrier end shall be accessible from the right shoulder and not be obstructed by beam guide rail or barrier parapet.
   - The ground leading to the gate or noise barrier end, as well as the ground behind the noise barrier, shall be graded to allow for maintenance vehicle access.

2.7.5 Noise Barriers on Bridges:

1. Provisions for expansion shall be placed in the noise barrier at locations of bridge deck expansion joints and at parapet deflection joints.

2. When a noise barrier is designed to be supported by a New Bridge superstructure, the attachment to the superstructure shall be made to the concrete parapet or directly to the superstructure framing. When designing for wall attachments to the concrete parapet, the behaviors of the bridge superstructure framing and the deck slab overhang shall be analyzed, including the effects of the torsional moments and twist caused by the weight of the noise barrier and wind load. The distribution of the superimposed noise barrier dead load shall be placed entirely on the parapet and distributed to the interior Stringers assuming that the parapet is a cantilever load on a continuous deck beam acting from the fulcrum point of the fascia Stringer bearing. Design Engineers shall be aware that net uplift is possible at the 1st interior Stringer and greater than 100% weight distribution is possible at the fascia Stringer due to the eccentricity created by the Noise Barrier load placed on the outer edge of the deck overhang. A three dimensional analysis of the bridge superstructure shall be performed in order to determine the effective superimposed dead load and wind load distribution to the superstructure elements. Regardless of

analysis results, full-depth Diaphragms shall be placed at all noise barrier post locations between the fascia, 1st interior and 2nd interior Stringers. The Diaphragm bottom flanges shall be framed into the Stringer bottom flanges via gusset plate connection. The use of light weight noise barrier panel systems is strongly encouraged.

3. Noise barrier retrofit is not permitted on existing bridges, except in the case of bridge widening. Where bridge widening is to be performed, the above provisions shall apply to the fascia, 1st interior and 2nd interior Stringers, all of which shall be of new construction and explicitly designed to carry the noise barrier loads.

2.7.6 Types of Noise Barriers

1. Precast reinforced concrete post and panel systems are preferred to be used, except on bridge structures.

2. A number of proprietary noise barrier systems are available for use on bridges or where unusual site conditions prohibit the use of a precast reinforced concrete post and panel system. The materials, load carrying mechanisms and capabilities vary with each system; however, these systems shall conform to the criteria outlined in Subsection 2.7.1 and applicable Project Supplementary Specifications. Proprietary wall systems shall be approved prior to the design of the barrier.

2.7.7 Materials

For ground mounted noise barriers, concrete for cast-in-place foundations and precast/prestressed posts and panels shall conform to the Standard Specifications. Concrete Class P shall be used for precast elements. Concrete class B shall be used for pedestals. Concrete class C shall be used for foundations other than drilled shafts. Concrete class A or SCC shall be used for drilled shaft foundations. All other materials shall conform to the requirements of the Standard Specifications.

2.8 BRIDGE REPAIR AND REHABILITATION

The majority of Authority bridges were built between 1954 and 1971. Their age combined with constant traffic loading have made preservation of the aging infrastructure through regular maintenance and periodic rehabilitation a dominant priority. To that end, the majority of Authority bridge work contracts are rehabilitation or repair driven in nature. The intent of this section is to standardize an approach to the work for Design Engineers and clearly define the performance goals for the bridges, as based on their anticipated remaining life span.

This approach fundamentally differs from prior versions of this section as it implicitly recognizes that few, if any, structures can be practically built for an ‘infinite’ life span and repairing a bridge for an infinite life span is functionally and economically impractical. Furthermore, certain components of any bridge, regardless of design life, should be viewed as ‘wear items’ in need of periodic replacement, including, but not limited to, concrete decks, deck joints, paint, bearings, and the cover layer of concrete for substructures.

In addition, other work is often required on Authority structures due to exceptional circumstances such as repairs from vehicular impact or fire, repairs due to load or distortion derived fatigue damage to structural steel members, and strengthening of superstructure or substructure components to enable them to carry the heavier loading of modern truck traffic. This work is often bundled within larger rehabilitation contracts.

Repair or rehabilitation designs shall be prepared with an understanding of the bridge's Anticipated Service Life (ASL) after the work is performed and to achieve that life span performance goal before the bridge is to be replaced. There are five classifications of ASL which will be assigned to the structure for the design of repair or rehabilitation work:

1. Routine Maintenance Work – This work assumes that the bridge will remain in service for the foreseeable future and generally consists of simple repairs including patching of decks, repairs to safety walks and barriers/parapets, asphaltic and latex modified concrete overlay replacement, minor repairs to steel work, resetting of bearings, spall repairs on substructures, resealing or replacement of deck joints, and localized painting. General 'best practices' cleaning work shall also be incorporated including clearing of scuppers and drainage systems, pressure wash cleaning of joints and bearing seat areas, and application of a sealer coat on bare concrete decks.

2. 15 Year ASL Rehabilitation – This work assumes that the bridge will be replaced within the next 10 to 15 years. The intent of this work is to stabilize structures in poor to fair condition so that they will require minimal additional maintenance until their replacement is planned and constructed. Only advanced deterioration and those areas which are at-risk for a 15-year performance goal are to be repaired. Where bridge decks are in poor to fair condition and heavily patched, isolated or grouped bridge deck panel replacements along with a placement of an asphaltic overlay with membrane waterproofing may be required to extend the service life of the deck until its demolition.

3. Bridge Painting – Bridge coating systems have an ASL of 20 years. Bridge painting is sometimes bundled with larger rehabilitation work or performed as individual contract work. For all bridges which are scheduled for repainting, the Design Engineer is responsible for reviewing the latest Bridge Inspection Report and preparing appropriate repair details for areas of damage or deterioration. The Design Engineer is also to be aware that media blasting of superstructure steel often reveals more extensive deterioration than the hands-on inspection effort prior to blasting operations indicates. Additional allowances should be given during design for additional quantities of repair items. The Design Engineer is directed to physically inspect the steelwork after media blasting and spraying of the prime coat. All defects shall be noted and addressed through original contract "If-And-Where" directed items, or through additional repairs added via Change Order. The Design Engineer, not the Contractor or the Construction Manager, will be responsible for identification of additional areas of deterioration after prime coat application.

4. 35 Year ASL Rehabilitation – This work includes decks/seismic/moderate strengthening, and if practicable, bearing replacement, replacement of heavily deteriorated members, possible superstructure replacement.

5. 75 Year ASL Rehabilitation – Only for NJTA "Major Complex" (Major) bridges. Major Bridges are designed for a 150-year overall service life. A half-life major retrofit of these structures is commonly required to achieve this goal. This type of work typically includes seismic rehabilitation, paint, deck replacement, superstructure replacement, pier modification/replacement, security hardening, fender work and inspection accessibility improvements.

Expected Life Span of Bridge Components

1. Major Bridges – 150 years
2. Routine Bridges and Culverts over 20' in Span– 75 years
3. Concrete Culverts – 50 years
4. Routine Bridge Decks – 35 years to 40 years
5. Major Bridge Decks – 50 to 75 years
6. Superstructure Steel - 75 years, except Stringers in GFS Bridges – 35 to 60 years (replace with the deck unless in good condition)
7. Latex Modified Concrete Overlay – 20 years
8. Asphaltic Overlay – 10 years
9. Joint Sealers – 10 years
10. Deck/Concrete Seal Coats – 5 years
11. Substructure Elements – Routine Bridges at 75 years, Major Bridges at 150 years
12. Bearings – 35 years to 60 years
13. Light standards – 15 years to 20 years
14. Sign Structures – 50 years

General Considerations for Repair or Rehabilitation Contracts

1. Priority of Repair
2. Significance of Asset/Route
3. Utilities
4. Staging and Operational Limitations and Restrictions
5. External Stakeholders
6. Best Value of Work

The Design Engineer shall compile and assess deficiencies and required repairs, and their priority, and provide estimated construction costs within the Phase A report. Prioritization shall be used to select work areas suitable for the Authority's specific budget, and to establish milestones in the contract so high priority repairs are constructed in a timely manner.

Consideration should be given to oversizing repair quantities and details for structural elements. The deterioration at the time of the defect identification during inspection often progresses over the time required to produce the final Contract Drawings. Anticipation of larger repair areas should be incorporated in the contract details and the contract quantities.

2.8.1 Access

The Design Engineer shall consider Contractor Access when preparing Contract Documents for Bridge Rehabilitation work. The contract documents shall include potential access to work areas and staging areas, as determined and coordinated by the Design Engineer prior to completing final design.

2.8.2 Permits

The Design Engineer shall consider NJDEP and US Coast Guard, etc. permit requirements when preparing Contract Documents for Bridge Rehabilitation work. Permits may be required for the construction work activities to be performed or for the Contractors' access requirements. Final design documents shall include permit provisions or restrictions required for the Project.

2.8.3 Other Agency and Railroad Coordination

It is required that, when arranging traffic protection on or over facilities owned by other agencies, including railroads, the Authority's Engineering Department be provided with written statements attesting that all responsible agencies have been informed of the work and have reviewed and concur with the planned schemes for maintaining and protecting traffic on or over their facilities. The Authority's Engineering Department shall also be provided with written statements attesting that all affected utilities have been informed of the work and have reviewed and concur with the planned schemes for protecting their facilities. In scheduling plan preparation, time must be allotted for review by these agencies. The concurrence of these agencies must be obtained in writing.

2.8.4 Structural Inspection Requirements for Rehabilitation and Repair Contracts

The primary sources of deficiencies requiring repairs or rehabilitation required for a bridge Project are the Biennial Inspection Report and the Authority's Bridge Management system (InspectTech, by Bentley Systems, Incorporated). While these resources provide important information, they do not typically provide the details required to prepare detailed repair/rehabilitation plans.

The following guidelines shall be followed regarding when a separate hands-on bridge inspection shall be performed solely for the preparation of bridge repair/rehabilitation drawings:

<u>15 Year ASL Rehabilitation:</u> A separate hands-on bridge inspection shall be performed. The site visit may utilize binoculars, to locate and confirm the limits for repair work, supplemented by local hands-on inspection as required. If catwalks are present on the bridge, access may be available at the permission of the Authority. Challenging terrain may require the use of over-the-side bridge access equipment. Hands-on inspection to determine limits for repair of deficient concrete at splash zones, safety walks, barriers/parapets, substructures, walls, and Stringers, is strongly suggested as deficiencies likely extend beyond the limits observed through visual inspection.

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.

75 Year ASL Rehabilitation: A separate hands-on bridge inspection shall be performed on the entire bridge. If catwalks are present on the bridge, access may be available at the request of the Authority. The intent of this level of rehabilitation is to extend the life of a Major Bridge for the same anticipated remaining life of an all-new routine structure. The Engineer shall inspect/sound all areas of apparent deterioration, and all piers for Major Bridges shall be fully sounded. All defects of any significance shall be fully inspected so that they may be restored to the as-built or better condition.

2.8.5 Traffic Protection

Bridge repair contracts are unique in that they often include short duration, high intensity maintenance level work to limit disruptions to the flow of traffic. In the preparation of repair contracts, all work and repairs must be designed to respect maintenance and protection of traffic (MPT) at, over, under and adjacent to work sites. Closures of the Authority's lanes and shoulders for the purpose of construction shall be planned in accordance with the lane and shoulder closure tables in the Manual for Traffic Control in Work Zones. Special considerations for lane closures occurring during peak travel periods for weekend days are reviewed by the Authority's Operations Department to permit High Intensity Construction Cycles. All MPT provisions in the Contract Documents for traffic and construction staging, including permissible work times, lane closing, and type of protection devices, shall be coordinated with the Authority's Operations Department. Reference is made to the NJTA Standard Specifications and Standard Supplementary Specifications, Standard Drawings, and Manual for Traffic Control in Work Zones for the Authority's general guidelines and construction requirements. Design Engineers shall revise the Specifications on a Project by Project basis as required.

In general, intensive work such as complete or panelized deck replacement shall be sequenced such that on existing bridges carrying mainline roadways of the Turnpike or Parkway, lanes may be closed for construction but typically a minimum of two traffic lanes must be available in every stage in each direction, pending approval from the Authority's Operations Department. For heavier traffic regions of the roadway, it may not be practical to remove any lanes from service for deck replacement work. This is generally accomplished by shifting traffic and using a shoulder as a temporary traffic lane during construction, where a shoulder is available and its condition permits its use as a travel lane. As previously discussed, New Bridges shall be designed such that future deck replacements may be made while traffic is maintained for the full number of active lanes on the bridge.

The Engineer of Record is responsible for reviewing and improving the shoulders, the underlying bridge deck, and protective features where shoulders across the bridge(s) are used as temporary travel lanes. Furthermore, the on-grade approach roadways must be considered when placing traffic in the shoulder.

Authority roadway shoulder pavement is generally designed only to accommodate break-down room for vehicles and emergency use as a travel lane. Shoulder pavement will generally require repaving or reconstruction based on its thickness and condition. While on-grade pavement reconstruction typically is the responsibility of the project highway engineer, this information is published here as it often affects the scope of bridge work.

The following shall be used as a guideline for evaluating and determining replacement for shoulder pavement for traffic staging; however, the Project Highway Engineer shall determine the level of pavement analysis required as it relates to AASHTO guidelines especially if long-term pavement performance is required.

- All shoulder pavement to be used for traffic staging shall be evaluated visually. Ground Penetrating Radar (GPR) shall be used to verify pavement thickness. Falling weight deflectometer testing may be warranted if long-term pavement performance is required.
- Existing shoulder pavement that is 5” thick and in fair or better condition may support traffic for up to two days without improvement.
- Existing shoulder pavement that is less than 5” thick or in poor condition shall be fully reconstructed with full depth pavement.
- Existing shoulder pavement that is 5” thick and in fair or better condition may support traffic for 14 days or less; however, removal and replacement of the surface course (the top 1.5”) may be necessary to maintain vehicle drivability.
- Shoulders which are to support traffic for more than 14 days shall be fully reconstructed with full depth pavement, regardless of condition, unless all of the following apply: the pavement is 5 years old or newer, is at least 5” thick, and has a suitable cross slope for use as a travel lane.
- Portions of shoulders adjacent to drainage inlets, and drainage inlets and chambers shall be carefully evaluated. They may warrant full depth reconstruction, regardless of condition visible at the roadway surface.

The Engineer of Record is also responsible for reviewing and improving shoulder cross slopes and drainage areas to maintain vehicle drivability and proper drainage where shoulder areas are intended for use as temporary lanes during the staged construction (including the approaches to and from), and for reviewing with assistance of the Project Highway Engineer, the clear zone distances and safe recovering areas along adjacent berms and slopes.

It is noted that the Parkway has areas where there are no shoulders and other provisions may be necessary for staged construction based on coordination with the Authority's Operations Department. Traffic protection within these areas requiring slab replacements may require reduction of the number of lanes over short durations/weekends and (High Intensity Construction Cycles) shall be reviewed on a project by project basis.

Lane and shoulder closings for deck slab replacement work along Authority owned roadways and ramps will be done in accordance with standard procedures as outlined in the "TP" series of Standard Drawings and the Manual for Traffic Control in Work Zones. Lane and shoulder closings in interchange areas, ramps and mixing bowl areas may require special details and arrangements of traffic protection devices which must be provided on the Plans.

For the purposes of staging deck replacement and reconstruction work, lanes through staged areas should be 12' wide where practicable. When lane width reductions are necessary, an 11' minimum travel lane is desirable with 10'-6" being the absolute minimum width allowed. Use of a 10'-6" wide travel lane should be limited to an isolated location. Reduced widths of lanes require the approval of the Authority's Operations Department. The minimum travel-way width along curved ramps may be greater than the above values based on the Project Highway Engineer review.

In addition to the above lane width requirements, a minimum 1' buffer shall be provided between the lane line and the face of any barrier where practicable, with 6" being the absolute minimum buffer allowed. Additional guidance is provided in the Authority's Manual for Traffic Control in Work Zones and the TP Standard Drawings.

2.8.6 Repair and Replacement of Decks

The use of temporary shielding for repairs and catches for replacement may be required to prevent debris from falling below. Any deck repair work, including milling of pavement over Authority roadways and partial depth repairs, within a span over or adjacent to live traffic or other public use shall be assumed to require shielding/ catches below the deck, especially where existing stay in place formwork is not present. The Design Engineer shall distinguish (and call out in the plans) the locations of the bridge, or spall repairs, that require temporary shielding or catches.

2.8.6.1 Installation of Replacement Wearing Surfaces

Bridge deck surfaces may be directed for the installation or replacement of an asphaltic or cementitious wearing surface by the Authority. All bridges to receive asphaltic wearing surfaces shall receive a waterproofing membrane prior to installation of the riding surface. Where cementitious wearing surfaces are to be used, the existing deck shall be scarified by 1/2" to promote bond between the deck and wearing surface material.

2.8.6.2 Partial Depth Deck Replacement (Spall Repair)

Bridge deck spall repair, which is the partial depth replacement of the deck, is performed on bridge decks to improve the riding surface and arrest decay. Spall repair generally does not require removal and replacement of reinforcing bars in the deck, except to replace reinforcement that exhibits 25% or more loss in cross-sectional area. Refer to the Standard Drawings for additional guidance on deck spall repair detailing and procedures.

2.8.6.3 Full Depth Deck Repair and Replacement

Complete deck slab replacements shall be designed to meet existing capacity. Isolated deck slab replacements should span between Stringer centerlines. Where this is not practical or possible, a supplemental Stringer shall be installed between the primary Stringers. A supplemental Stringer is typically supported on strengthened existing or new Diaphragms. It need not be made composite with the deck, but shall be left in place after completion of the deck replacement work.

2.8.6.4 Bridges with Existing Surfacing

The new portion of the deck shall be surfaced to match the existing construction. 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. No membrane waterproofing shall be used in the case of an asphalt concrete overlay with ‘Rosphalt’ additive.

Concrete cover for the top reinforcing bars shall be 1 1/2" minimum. Epoxy-coated reinforcement steel shall be used for both the top and bottom reinforcement bar mats. No allowance shall be made for future wearing surface in the design of these slabs. Original deck slabs for Authority bridges vary in thickness from 7” to 10 1/2”. Replacement bridge deck portions shall be preferably designed to match the existing reinforcing bar spacing of the adjacent deck panels that remain wherever possible to facilitate splicing of bars. Instances may exist where matching of the existing deck thickness and maintaining adequate structural design of the deck would be practical. Local thickening of the deck and / or increasing the main reinforcing bar size shall be permitted on a case-by-case basis. Reduction of the design wheel live load to 16 kips per wheel may be permitted on a case-by-case basis.

The design of the permanent metal bridge deck forms shall be the Contractor’s responsibility. Requirements governing the selection, design and fastening of the forms shall be set forth in the Contract Specifications.

The design of catches and shielding shall be the Contractor’s responsibility. Requirements governing the design, installation, maintenance and removal of catches and shielding systems shall be set forth in the Contract Specifications.

2.8.7 Repair of Spalls

Concrete spall repairs are typically performed for substructure elements such as piers and abutments. Spall repairs are performed to eliminate loose or falling concrete, restore environmental protection to exposed steel reinforcement, and to correct aesthetic blemishes. Spall repairs should not be considered structurally significant to the structure as it does not restore strength or structural capacity to the concrete member. Refer to the Standard Drawings for details and procedures regarding general spall repairs for concrete.

2.8.8 Repair of Headblocks, Headers, and Deck Joints

Headblock repairs consist of the reconstruction with concrete of existing abutment headblocks that are spalled or deteriorated or have been surfaced over with asphalt or elastomeric concrete. Generally, at locations where the existing bridge surfacing has been removed adjacent to deck joints, the headers are reconstructed using concrete. Care must be taken during the removal of existing concrete to not displace or damage the joint armoring, reinforcement and other embedded steelwork, where present.

Use of elastomeric concrete at headblocks and headers is no longer an acceptable practice on Authority owned bridges. Remaining elastomeric headblocks and headers that fall within the limits of work zones should be reconstructed with concrete.

All existing steelwork and reinforcement to remain that becomes exposed by the repair work shall be cleaned (by sandblasting) and coated with field anti-corrosion coating when possible.

Steel armored strip seal joints are preferred for retrofit work where full depth headblock repair is justified or where otherwise practical and / or where decks adjacent to a deck joint spanning several Stringer bays are scheduled for replacement. These joints are the only type approved for use on the Authority's Major Bridges.

Compression style epoxy bonded sealers are preferred where the deck edge and abutment headlock (and / or existing joint angle or bulb armor) are still in serviceable condition and do not require reconstruction. Compression style epoxy bonded sealers should be avoided along existing joints retrofit with riser bars, which are typically 1 1/2" to 2" high and present an irregular joint surface.

Asphaltic plug style joints are only permitted on Parkway and local roadway bridges that experience low average daily truck traffic and that currently have or are to receive an asphalt overlay as part of the contract. Any deficient headblock and header concrete underlying the proposed asphaltic plug joint shall be repaired prior to constructing the joint so as to achieve the ASL for the joint. Should the adjacent approach or bridge roadway pavement be scheduled for repair or replacement, the section of the asphaltic plug joint adjacent to that work shall also be replaced regardless of condition.

2.8.9 Drainage on Bridges

The original design philosophy for drainage on bridges was to place inlets in the gutters uphill of a (typically) open joint to eliminate run off through the joint and onto the bridge steel work and substructure elements below. This arrangement is still preferable and should be used in concert with sealing of any open bridge deck joints.

Since the original construction of Turnpike and Parkway roadways, much of the area underlying the bridge has been developed. Thus, existing inlets that are of the air-drop style may now require piping of the runoff to the ground or nearby roadway inlet below.

Open joints may require the installation of drainage troughs to collect the runoff that would otherwise splash on traffic or pedestrians below and funnel it to a remote discharge point. An elastomeric membrane or sheet should be used in the construction of troughs under open joints. This material offers a smooth surface to prevent snagging and is flexible enough, that under the vibration of the bridge due to traffic, it provides a constant flushing action to remove debris.

Drainage features should be designed to be ‘self-cleaning’ with smooth surfaces and edges to prevent snagging of debris. Sharp bends in the piping often causes larger debris such as beverage containers to catch and clog the pipe. All changes in direction of piping should be furnished with a clean-out plug and flat in-span piping should be avoided.

2.8.10 Superstructure Repairs

Superstructure repairs shall be evaluated for:

- Localized deterioration (holes, cracks, severe section loss).
- Members with LFR Inventory rating factor less than 1.0 for the AASHTO Type 3, 3S2 and 3-3 truck legal and Emergency Vehicle (EV) loads.
- Steel members with fatigue cracks.

Superstructure repairs shall consider the bridge's remaining ASL.

Steel superstructure repairs for section losses may be repaired via welded cover plate only at simply supported ends of Stringers, and only within a distance of ‘D’ from the centerline of bearing. The value ‘D’ is defined as the depth of the stringer’s web. Repairs made outside of the ‘D’ limit shall only be made by bolted connection cover plating.

Vehicular impact damage repair shall be performed in accordance with the above provisions except where the damage is limited to deformation in the steel with no cracking or gouges over 1/4” in depth. Where damage is limited to deformation alone, heat straightening of the steel should be considered as an option. Contact the Authority for guidance on design of heat straightening work where considered.

Prestressed concrete Stringers cannot be repaired when damaged or deteriorated to restore their structural capacity. Spall repair for these members is intended only for restoration of protection from the elements and to arrest corrosion of exposed mild steel reinforcing and prestressing strands. While these repairs are still expected to be performed, the Design Engineer is cautioned that structural capacity and condition rating of these ‘repairs’ should not be noted as improved or restored.

Bearing devices should be replaced when more than 50% of an existing bridge deck is also replaced, or when the bridge superstructure is replaced. The Authority prefers the use of laminated Elastomeric Bearings for straight Routine Bridges with minimal skew where practical. Refer to Subsection 2.2.4.3 for additional design and selection criteria for new bearings. Bearing replacement shall be performed in accordance with the Standard Drawings pertaining to field measurement of existing bearings and fabrication of new bearings.

2.8.11 Superstructure Strengthening

At a minimum, all Bridge Rehabilitation Projects which include replacing the bridge deck shall include consideration for strengthening of the superstructure to carry modern live loads commensurate with the ASL performance goals below (based on load and resistance factor rating methodology):

- HL-93 Inventory Rating for ASL of greater than 35 years
- HL-93 Operating Rating for ASL of greater than 15 years to 35 years
- Legal Truck and EV Rating for ASL of 15 years or less

Strengthening to carry TP-16 loading is not considered practical for superstructure rehabilitation Projects except for Turnpike Roadway Projects where superstructure replacement is to be performed. The option of full superstructure replacement is typically considered in situations where a rehabilitation Project would otherwise include deck replacement, repairs to the superstructure, in combination with repainting the existing steel work. Where these or similar conditions are encountered, the Consultant shall provide a cost/benefit analysis of both superstructure replacement and rehabilitation options, with recommendations, as a part of the Phase A submission.

Substructure elements generally need not be considered for the increased live load, unless where directed by the Authority, where the Bridge Inspection Report indicates poor condition, or, where flexural elements in the pier, such as free spanning or cantilever pier caps, are subjected to a more than 10% increase in overall service loading due to the increased live load. It is the responsibility of the Design Engineer to compare the substructure conditions noted in the Bridge Inspection Report to the actual field conditions, which shall include a visit to the site and physically viewing the substructure elements.

2.9 STRUCTURES PLAN PREPARATION

The following are guidelines for the preparation of Contract Plans for new, reconstructed or repaired and other structures (not including sign bridges). These instructions are provided so that a uniform level of engineering (design) information is presented and arranged in a consistent format. Plans will be reviewed by the Authority's Engineering Department at various interim stages of completion as discussed below.

Structure Numbers for new or replacement bridges, sign structures, and Culverts shall be assigned by the Authority's Engineering Department.

2.9.1 Title Sheet

The Authority's standard templates for title sheets are provided at the Authority's website to achieve uniformity of title sheets.

The annual maintenance repair contracts have a consistent presentation between the contracts. The index of drawings on the Title Sheet shall be sufficiently detailed so as to identify and locate each rehabilitation plan, MPT plans, which are organized by Zones and follow the rehabilitation plans and structure/zone specific details of which the zone is associated, and general details.

2.9.2 General Plan and Elevation

As a minimum, a general plan of every structure affected shall be included in the Contract. The scale shall be no less than 1" = 50', preferably 1" = 30' or 1" = 20'. Where applicable, the following data is to be furnished:

- Abutment bearing lines and pier centerlines.
- Turnpike/Parkway continuous stationing at each substructure element.
- North Arrow.
- Lane lines, edge of pavement, roadway protective feature, direction of travel, milepost markers, and roadway designation(s).
- Proposed work areas.
- Drainage facilities.
- Utilities.
- Roadways above and below and right of way lines.
- Railroads tracks and right of way lines; each track shall be identified by the railroad line and branch number, and structure number where applicable, and be identified as active or inactive as determined through coordination with the owner.
- Pertinent existing topographic and planimetric features which may have an effect on the proposed work.
- Vehicular detector loops/pavement sensors for weather and traffic systems and be identified as active or inactive as determined through coordination with the owner.

Maintenance and protection of traffic plans shall also show Mileposts, and a lane closing summary table shall be provided for each plan for each stage of construction.

For the maintenance repair contracts, the Rehabilitation Plan shall act as the General Plan. In addition to the above noted data, structural framing of the superstructure elements shall be depicted on the Rehabilitation Plan for spans and structures that have proposed structural steel repairs, bearing replacements, and/or deck panel replacements. Rehabilitation Plans shall also include designations for each abutment's and pier's existing joint type(s) and joint hardware, the bearing or bearing line designations (as expansion or fixed), and a pilot hole layout and stage designation for each isolated and/or group of deck panel replacements. Depictions of deck panel replacements shall show presence of catches where required.

Also for maintenance repair contracts, in addition to the Estimate of Quantities plan sheets which are organized at the front of the Plans, each Rehabilitation Plan shall be provided with a table summarizing the estimated quantities for the repair pay items separated by stages of construction. Rows for "If and Where Directed by the Engineer" quantities and columns for the insertion of "As-Built" quantities are omitted from these tables.

As a minimum for all Projects, an elevation of every structure affected should depict the general features below the bridge, including those that may affect access such as waterways, ditches, chain link fences, roadway protective features, and other permanent features. Tidal waterways should have mean high and low tide water elevations, and freeboard depicted. Clearances below the structure, including at infield areas, should be shown where they may have an effect on the proposed work.

2.9.3 Plan Content and Format

Minimum expected Plans are defined in this Section. The Contract Plans are the central document for any bridge focused work. Contract Plans are in essence, little more than 'assembly instructions' and should be viewed as such when preparing them. Plans shall be prepared such that they contain adequate information for the Contractor to accurately bid and build all features shown in the plans. How a bridge is constructed is the domain and sole responsibility of the Contractor. It is the responsibility of the Engineer of Record and Design Engineer to investigate and provide adequate information to prove that the Work shown on the Plans is possible to construct. This is typically done through provided staging plans. A Contractor is not required to adhere strictly to the staging shown in the plans, however a plausible way of completing the work must be provided to substantiate that the design of the Project is definitively constructible.

Specifications work in concert with the Plans and are intended to supplement the plans with written descriptions of material requirements, specific methods of work, adherence to testing standards, methods of measuring the work for level of completion and basis of payment.

Specifications for fabricated or furnished materials often require the Contractor to prepare and submit complete shop drawings and details for review before proceeding with fabrication. This will require the Contractor to graphically describe the interpretation of the Plans through more finely detailed drawings which are then utilized during fabrication, erection and construction. The shop drawing and details are reviewed by the Engineer of Record and Construction Supervisor for conformance to the Contract Plans.

Other working drawings will also be required from the Contractor for temporary construction and facilities which are not the responsibility of the Engineer of Record or the Design Engineer during the design and preparation of the Contract Documents (jacking operations, cofferdams, bracing for excavations, forms, catches and shielding etc.). Any permanent features which are the responsibility of the Contractor (MSE Walls, HLMR Bearings, Modular Joint Assemblies, etc.) are to be fully designed, detailed and submitted for review by the Contractor as well. It is the responsibility of the Design Engineer to show these facilities on the plans and provide adequate information to allow the Contractor to design, quantify, bid, and detail them. For the purposes of the Plans, additional information above this level should only be shown where absolutely required for the Contractor to fully understand and execute the work.

2.9.4 New Bridges and Comprehensive Bridge Rehabilitation Projects

New Bridges and Comprehensive Bridge Rehabilitation contracts generally address either new construction or repair of most significant components of a bridge. Thus, the plan set organization for these two types of Projects are usually of similar scale and content. They have therefore been grouped together in this subsection for brevity. Where Comprehensive Rehabilitation Projects differ from New Bridge construction, they are as noted below:

2.9.4.1 Key Plan to Structures

A Key Plan is required upon which each structure site is located and identified within the Contract limits. Ordinarily drawn to a scale of 1” = 200’, the Key Plan is similar to the Plan Reference Drawing typically given in the roadway design plans. Information shown shall include all existing and new construction, stationing at intervals of 500’, and NJ Coordinate Plane grid references. If space permits, the index of drawings for all included structures should be given on this sheet, as well.

2.9.4.2 Estimate of Quantities

For consistent presentation of the maintenance repair contracts, a unique legend of special symbols and general notes shall be used consistently between Projects. It will be the Design Engineer’s responsibility to coordinate with the Authority’s Engineering Department to receive the most current version of the legend and general notes for the repair contracts. For some contracts, this may require a specific separate sheet.

For all Projects, the Estimate of Quantities is an item-by-item tabulation of pay items for each structure site and includes rows for “If and Where Directed by the Engineer” quantities and a column for the insertion of “As-Built” quantities. Separate tabulation of items concerning Maintenance and Protection of Traffic and other general items, for example mobilization, construction layout, and progress schedule, for the overall contract will also be included. The Estimate of Quantities shall include an item number specific to the contract for each pay item, in addition to the Authority’s assigned Unit Code, Description, and Unit.

2.9.4.3 General Plan and Elevation and Bridge Section

This drawing should be prepared using the approved Preliminary Design Plan as a base (see Sample Plans). The scale shall preferably not be less than 1” = 16’, but in no case less than 1” = 30’. Additional General Plan and Elevation sheets may be required for large bridges. The typical section of the bridge superstructure shall be shown on the Bridge Section Sheet directly after the General Plan and Elevation sheet(s).

For Comprehensive Bridge Rehabilitation Projects, the General Plan and Elevation sheets in combination with the typical bridge section should also be used as work location plan sheets for the repair work. For Major Bridges where Girders and Floor-Beams may be very difficult to reach, the Design Engineer should show the repair work locations both on the plan and section sheets so that the Contractor can have a better understanding of the required effort to access the repair areas. Comprehensive Bridge Rehabilitation Projects should also be furnished with a specific Repair Type Description sheet narratively defining the various repair types and providing them with specific symbol call-outs for easy reference within the Plans.

General Notes are also typically shown in the General Plan and Elevation sheet. Checklists for features to be shown is attached as Appendix A.

2.9.4.4 Demolition and Staging Details

The demolition and staging details shall clearly describe the depth and breadth of the work required so that Contractors may understand and accurately bid the work. These plans shall include at a minimum the following:

- Demolition plan(s) and temporary sheeting / wall details
- Staging of demolition work
- Utilities on bridge staging or relocation

2.9.4.5 Substructure Details

Typically, and at a minimum, substructure plans are arranged as follows:

- Foundation Layout Plan(s) with pile or drilled shaft locations and notes
- Foundation Details with pile or drilled shaft details and notes
- Plan and Elevation Views of Abutments
- Detail Views and Sections of Abutments
- Plan and Elevation Views of Piers
- Special Details, Views and Sections of Piers
- Substructure Repair Details*
- Bearing Seat Reconstruction*

*Comprehensive Bridge Rehabilitation Projects only.

Marking and listing of reinforcement bars is not required for Authority Projects. It is generally sufficient to show reinforcement patterns, bends and shapes in elevation views and sectional views. However, special supplementary sections are often necessary to clearly indicate reinforcement shape and placement in corners and other obscure areas. Dimensions for positioning reinforcement beneath bearing areas shall be coordinated in the Plans to avoid conflicts with anchor bolt placement. Bearing pedestal reinforcement details must be shown.

Plan and Elevation views should generally be drawn to a 1/4" scale; preferably, the scale should not be less than 3/16". The scale of sectional views shall be large enough to show a clear representation of reinforcement placement and shape. These views are commonly drawn to 1/2" or 3/4" scale. General substructure detailing criteria of note is described below.

2.9.4.6 Joints

Typical details for construction, contraction and expansion joints are illustrated in Exhibit 205 to 207 of this Manual.

2.9.4.7 Bearing Surfaces

Tops of piers and abutments shall be sloped along their length to minimize the height of concrete bearing pedestals. Pedestals shall be reinforced using the details presented in Exhibit 2-213 to 216 of this Design Manual. Bridge seats shall be sloped longitudinally towards their front face, and pier tops crowned to provide for runoff. Substructure waterproofing membrane shall be applied to the tops of all piers, abutments, and bridge seats, including all sides of pedestals.

2.9.4.8 Damp Proofing

The rear face of all earth retaining structures shall be damp proofed from the top of footing to ground level.

2.9.4.9 Under Bridge Slope Protection

For bridges crossing over the Turnpike or Parkway roadways, under bridge slope protection consists of concrete slope protection or stone slope protection. Construction details are shown in Standard Drawings BR-5 and BR-8. Where the Turnpike or Parkway roadway section approaching the crossing is in cut, the slope normal to the abutment face shall be 2:1, otherwise a slope of 1.5:1 shall be used to determine the structure length and abutment heights.

Where the Turnpike or Parkway crosses local roads or State highways, the type of under bridge slope protection will be specified by the agency having jurisdiction. Usually, a concrete pavement slope protection will be required for these locations, and details will be those of, or approved by, that agency.

2.9.4.10 Drainage Behind Walls and Abutments

Earth retained by walls and abutments shall be drained by the use of porous fill, perforated corrugated metal pipe underdrains, and in some cases, weep holes. Generally, underdrains should connect or discharge into the roadway drainage system. Refer to the details presented in the Exhibits of this Section of the Design Manual.

2.9.4.11 Utility Supports

Utilities shall be supported by steelwork which frames into main members. Supports, which rely on deck inserts or drilled in expansion anchors, shall not be used. Support materials shall be fire resistant and offer secondary measure of restraint against collapse in the event of severe fire damage. Refer to Subsection 2.2.1.7 for further discussion regarding utilities on bridges.

2.9.4.12 Superstructure Details

Typically, and at a minimum, superstructure plans are arranged as follows:

- Framing Plans
- Beam Elevations
- Diaphragm and/or Cross Frame Details
- Field Splice Details
- Steel Details
- Camber Table(s)
- Superstructure Repair Details*
- Bearing Details
- Structural Jacking Details*
- Deck Plans
- Deck Sections
- Parapets
- Deck Joints
- Miscellaneous Details (as required)
- Bridge Painting Details*

*Comprehensive Bridge Rehabilitation Projects only.

No hard and fast rule can be set for determining the amount of information which can be presented on any one sheet, since this depends on the complexity of the structure's framing. However, checklists for information typically required have been provided in Appendix A.

The scale of framing plans should generally be not less than 1" = 20'; Cross section scales should be either 3/8" or 1/2". The scale of detail views will depend on the actual size of the detail and the degree to which that the view would be cluttered by dimensions, material notes, welding symbols, etc. General superstructure detailing criteria of note is described below.

2.9.4.13 Welded Joint Design and Detailing

Welded joint design detailing shall comply with the latest edition of the AASHTO/AWS D1.5 Bridge Welding Code. Information provided on Contract Plans shall conform to Subsection 2.1 of the same text. Weld call-outs and symbols shall conform to AWS A2.4. Finish grinding, where required, shall be shown on the welding symbol in the Contract Plans. Contract Plans shall show PJP or CJP requirements for all groove welds. It is not necessary for the Contract Plans to detail the specific joint designation for the welding procedure, however, the specific joint designation proposed by the Contractor is required for all welding symbols placed on working or shop drawings.

Specific reference is made within this section to the definition of Fracture Critical Members and guidelines related to member identification. A note shall be added to the structural steel plans that Fracture Critical Members and/or member components shall be subject to the provisions of the current Edition of the AASHTO/AWS D1.5 Bridge Welding Code, Section 12.

As a last note, Fatigue of welded connections is a concern for Authority structures. Satisfactory performance of these details has been limited due to the heavy and frequent truck traffic present on the roadways. The Design Engineer is strongly encouraged to consider this when detailing weldments. Weldments of categories below Category D are strongly discouraged, except for cross frame gusset plate weldments and for sign structures where such details may be unavoidable.

2.9.4.14 Splices

Permissible locations of field and shop splices should be given on the Framing Plan or Stringer Elevations. Splices should be fully detailed.

2.9.4.15 Cambers

For simple spans, cambers shall be provided at Stringer quarter points and field splice locations for short spans where the camber is 4" or less, otherwise, camber shall be shown at tenth points and field splice locations. For continuous spans, Cambers shall be provided at

10th point ordinates along each span and at field splice locations. The individual cambers comprising the total value shall be given (i.e., camber for deflection due to steel dead load, camber for deflection due to concrete dead load, camber required for vertical curve), for the correct setting of forms and reinforcement. Ordinarily, beams shall not be cambered for sag vertical curves; the slab haunch shall be varied and the Stringer flange maintained on a vertical tangent, under full dead loads. Rolled section Stringers shall be placed with mill camber up, where practical.

2.9.4.16 Temperature

All details affected by thermal movements shall be designed for a reference temperature of 68°F.

2.9.4.17 Clearances

Unless a greater distance is required by consideration of expansion and live load movements, the ends of fascia Stringers should be set at 6” apart at piers (for simple spans) and within 6” from back walls at abutments. It is customary to detail beams so that ends will be vertical under full dead load.

2.9.4.18 Deck Joints

Bridge deck rehabilitation Projects shall typically consist of replacing failed preformed seals, like compression seals. Strip seal joint systems are preferred for New Bridges. Construction details are shown on Standard Drawings BR-13 and BR-14.

For combinations of span and skew outside the range of applicability of the compression seal or a strip seal, a modular system of multiple elastic sealers may be appropriate. Modular joints may be permitted for use on long continuous superstructure units, but prior approval should be obtained from the Authority before proceeding. The Design Engineer is alerted to the fact that special details must be included in the Plans for these joint systems. Consult with joint system manufacturers.

The type and size of each joint proposed for use shall be indicated on the Preliminary Design Plan.

2.9.4.19 Bridge Drainage

Storm sewer inlet and scupper details, shown on Standard Drawings BR-2A and BR-2B, shall be typically utilized. For bridge Projects where these details cannot be utilized due to superstructure type or other conflicts, the required modifications and/or additional details shall be shown on the Contract Plans.

2.9.5 Bridge Rehabilitation Contracts

Bridge Rehabilitation Contracts generally address limited work on bridges meant to extend their ASL. Their plan set is usually focused on repair of deck, superstructure, and substructure repair, rather than replacement, and often feature limited detailing of existing features intended only to inform the Contractor of the location of work and the expected scale of work at the repair location. Bridge Rehabilitation Project Plans typically have new deck work detailing similar to New Bridge Projects with additional provisions added for work to the Stringer top flanges for removal of existing shear lugs and weldments and staged installations of new shear studs, with top flange lead primer remediation, as appropriate.

2.9.6 Bridge Maintenance Work

Bridge Maintenance Work is organized into four yearly maintenance design and construction contracts.

The Project limits are:

- Garden State Parkway South (Mileposts 0-126)
- Garden State Parkway North (Mileposts 126-172)
- New Jersey Turnpike South (Including the Pearl Harbor Memorial Turnpike Extension)
- New Jersey Turnpike North (Including the Newark Bay-Hudson County Extension)

Bridge maintenance level of work is generally prepared with a minimal level of detailing as the work in these contracts is intended to be simple and repetitive “routine” tasks. Plan preparation is focused on locating the work over a large geographic area with basic standardized detailing provided throughout.

2.9.7 Retaining Walls

Recent retaining wall construction for Authority roadways has predominantly been of Mechanically Stabilized Earth (MSE) construction. However, the Authority has a stated preference for conventional cast-in-place semi-gravity Walls, which generally require more detailing effort on the part of the Design Engineer than MSE walls. This additional detailing effort should always be assumed on the part of the Engineer of Record when planning the design and plan preparation effort for retaining walls. Plan preparation effort for retaining wall types is as noted below.

2.9.8 Cast-in-Place Walls

Cast-in-place concrete walls generally will require the same level of detailing as bridge substructure units:

- General Plan and Elevation with general notes
- Construction or staging details
- Foundation layout with pile layout (if required)
- Typical sections of the retaining walls
- Reinforcement plans, elevations, and sections
- Fencing, handrail, drainage, architectural details

For contracts with long cast-in-place walls or many walls, design information may be conveniently presented in a panel-by-panel tabulation. Panels should be identified numerically on the General Plan and Elevation and referred to in the tabulation. Similarly, various types of wall sections, reinforcement patterns, pile plans, etc. should be detailed once and identified for use in the tabulation. The tabulation should also indicate footing dimensions for each panel, panel end point elevations and footing elevations.

Details such as the placement and arrangement of non-stress reinforcement on wall stems, key construction, porous fill placement, and joint construction are common to all panels and should be presented once in a contract set of plans.

2.9.9 Alternate Walls

Alternate walls are defined as walls, often proprietary, such as Mechanically Stabilized Earth (MSE) Walls and Prefabricated Modular (PM) Walls or any other proprietarily designed wall system other than Cast-in-Place Concrete walls as discussed above.

Design Engineers shall consider the use of alternate retaining wall systems at select Project locations. Proprietary retaining wall systems, MSE Walls and PM Walls, are generally considered to provide a reduced time for construction over standard cast-in-place reinforced concrete cantilever retaining wall system.

Alternate walls shall be presented in the Common Structure Volume (CSV) format. Common Structure Volume is defined in Subsection 2.3.3. Design Engineers shall develop the CSV to encompass all proprietary alternate wall systems applicable to each site and list those systems in the contract specifications. Only wall systems participating in design consultation may be included in the contract Specifications.

The Design Engineer shall develop the General Plan and Elevation to include the CSV, right-of-way, utilities, noise walls, lighting, drainage, staged construction, and other pertinent information. Elevations should show existing and proposed ground lines, minimum foundation elevations and mean high and low water, where appropriate. Cross sections should show limits of CSV, wall batters, pay limits and all pertinent information.

Magnitudes, locations and directions of external loads due to bridges, overhead signs and lighting, noise walls, traffic and other surcharges should be shown on the plans.

Architectural requirements should be identified.

2.9.10 Culverts

Because of their limited plan area, Culvert General Plans should be drawn to as large a scale as may be practical, preferably 1/8” =1’; scale shall not be less than 1” =20’.

2.9.11 Standard Drawings

The Standard Drawings required for each contract will be furnished by the Authority. Absolutely no changes or additions of any kind are to be made to the Standard Drawings. Should changes be required, the Design Engineer shall present the changed drawing on a Project specific border to be signed and sealed by the Engineer of Record for the Project.

2.9.12 Reference Drawings

Design Engineers shall be required to visit the Authority's office as required to obtain and review existing available as-built plans and shop drawings. These existing drawings are defined as Reference Drawings. Inasmuch as repair plans require details of existing construction of many bridges, a considerable number of Reference Drawings may be involved. It has been the practice not to include all Reference Drawings of affected structures as part of the plans, but to include only those specific Reference Drawings necessary to determine the extent of work for the bidding process.

Following the list of Standard Drawings, a separate listing is to be shown on the title sheet as Reference Drawings. An exception to this requirement is made for the annual maintenance repair contracts due to the number of Reference Drawings required for the various repairs in the contract. Direction for annual maintenance repair contracts is provided below.

For contracts listing Reference Drawings on the title sheet, these Reference Drawings will also receive a sheet number and become part of the contract. They will receive sheet numbers after the last Standard Drawing number has been assigned. All boring logs are to be included as Reference Drawings and are to appear in the contract at the end of the "Reference Drawings" division. All Reference Drawings are to have the words "Reference Drawing" placed immediately adjacent to the title box. In the margin below the title box, the contract number, individual sheet number and total number of sheets in the contract are to be added.

If Reference Drawings are required from another contract prepared by the same Consultant, it will be their responsibility to furnish reproduced original copies of such drawings for all contracts to which they apply.

Should the Reference Drawings be from a contract prepared by another Consultant, the Authority's Engineering Department will furnish reproduced original copies of such drawings, provided the Consultant has advised the contract number(s) and sheet description(s) that are required. It will be the Consultant's responsibility to give the Authority's Engineering Department ample notification of which Reference Drawings will be required for each contract so that the Authority's Engineering Department will have time to prepare copies of such sheets.

For the annual maintenance repair contracts, those Projects listed in Subsection 2.9.4, the Design Engineer shall tabularize the Reference Drawings required for each structure in the NJTA Supplementary Specifications Subsection 102.04. These Reference Drawings will not be provided sheet numbers or marked with the words "Reference Drawing".

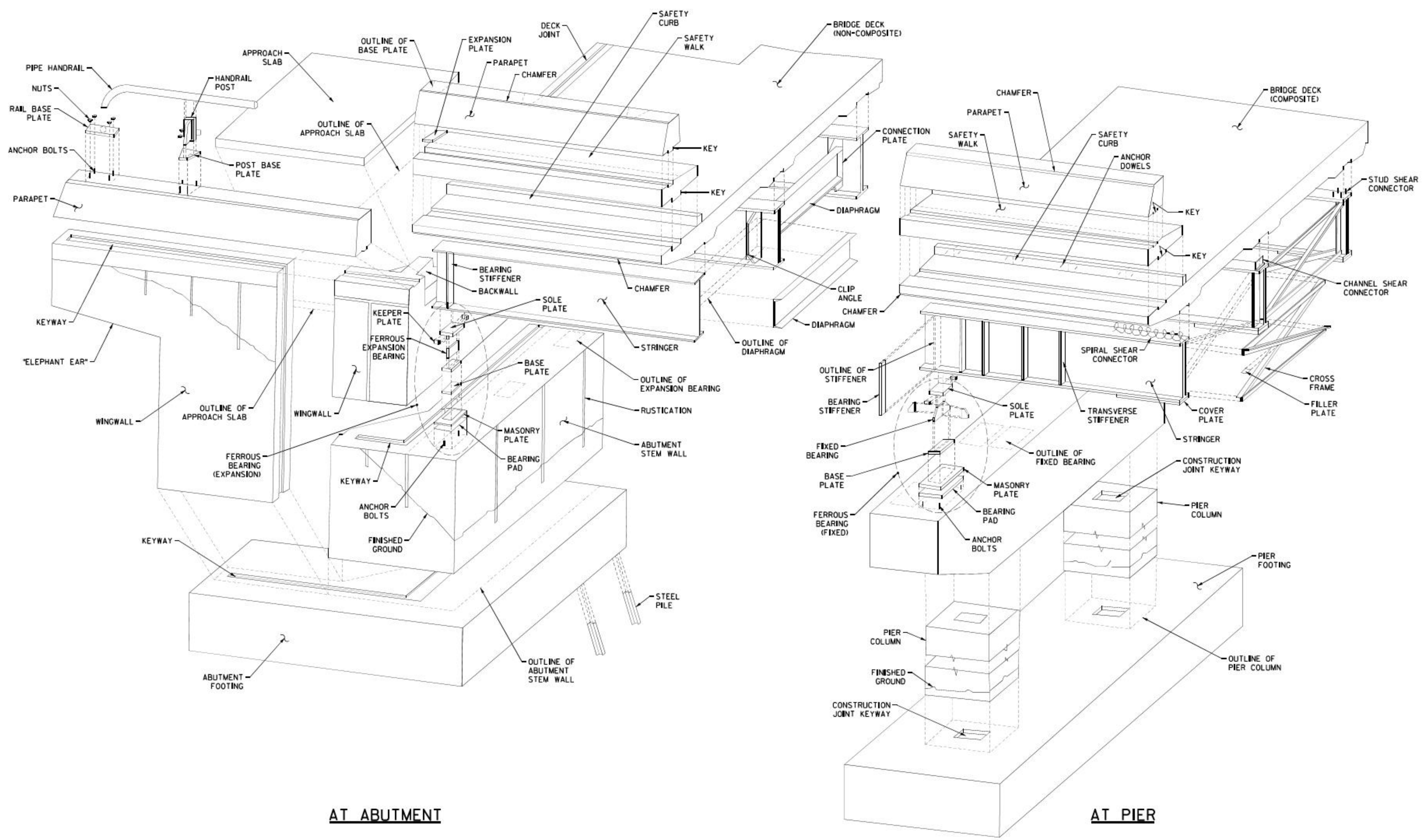
It is the responsibility of the Design Engineer to acquire each Reference Drawing and to compile an electronic PDF file by individual contract for the

Reference Drawings associated with an individual structure. The files by contract shall be organized in subfolders by structure. The Design Engineer shall submit to the Authority the Reference Drawings listed in the NJTA Supplementary Specifications, along with the Standard Drawings listed on the title sheet, for interim submissions of completion and for advertisement.

EXHIBITS

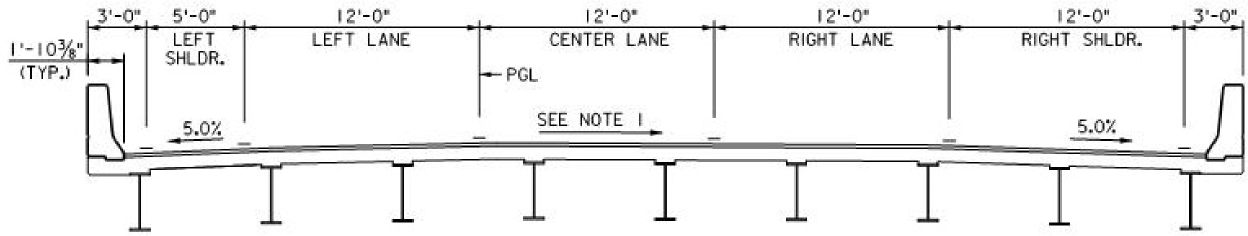
NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-00 TYPICAL BRIDGE ANATOMY AND NOMENCLATURE

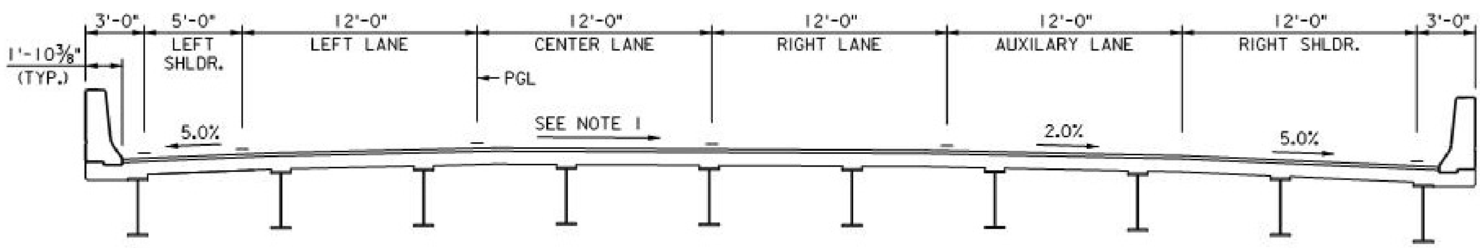


NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-100 TURNPIKE BRIDGE DECK GEOMETRY MAINLINE - 1



NORMAL SECTION



NORMAL SECTION WITH AUXILARY LANE

THREE LANE TURNPIKE ROADWAY

N.T.S.

NOTES:

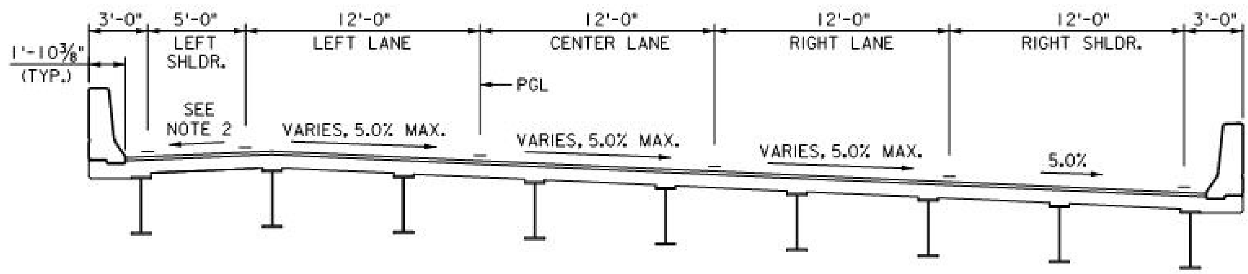
1. FOR LONGITUDINAL GRADES LESS THAN 0.5%, LANE CROSS SLOPES = 2.0% MIN.

   FOR LONGITUDINAL GRADES GREATER THAN OR EQUAL TO 0.5%, LANE CROSS SLOPES = 1.5% MIN.

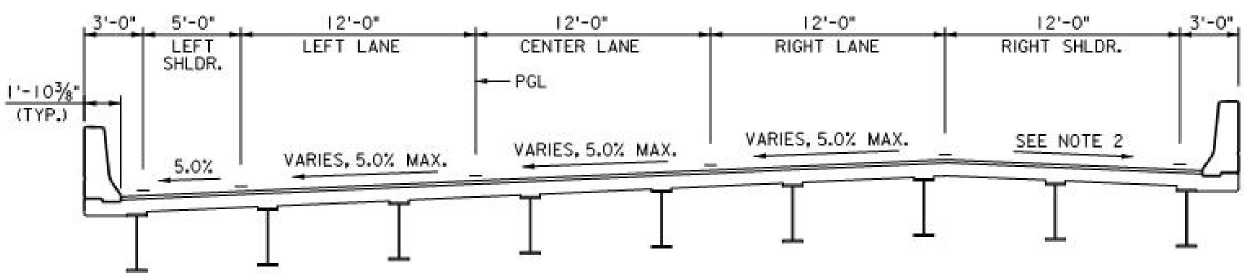
2. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-101 TURNPIKE BRIDGE DECK GEOMETRY MAINLINE - 2



SUPERELEVATED - CURVE RIGHT



SUPERELEVATED - CURVE LEFT

THREE LANE TURNPIKE ROADWAY

N.T.S.

NOTES:

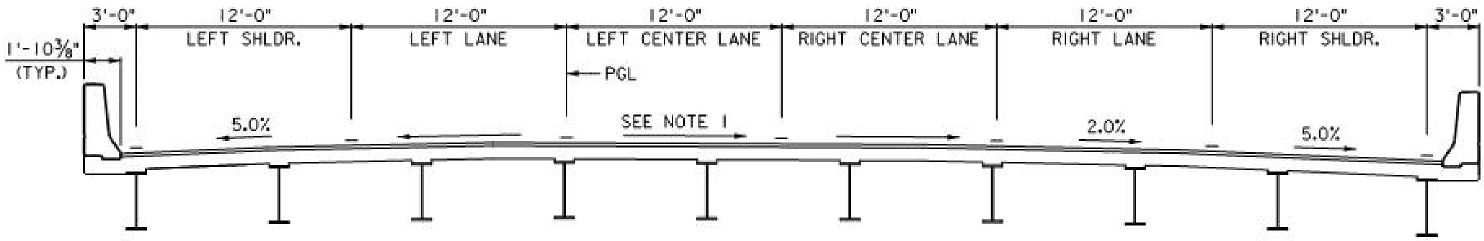
1. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

2. FOR SUPERELEVATED LANE CROSS SLOPE LESS THAN OR EQUAL TO 3.0% SHOULDER CROSS SLOPE SHALL BE 5.0% MAX.

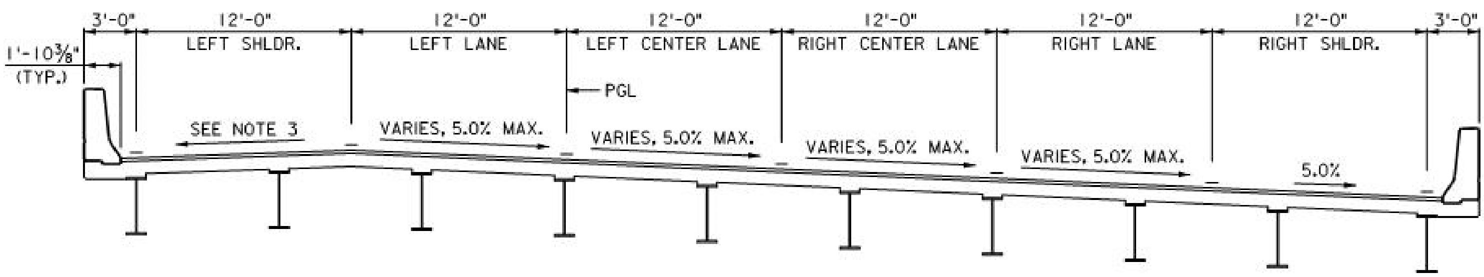
   FOR SUPERELEVATED LANE CROSS SLOPE GREATER THAN 3.0% TO 5.0% MAX. SHOULDER CROSS SLOPE SHALL VARY FROM 5.0% MAX TO 3.0% MIN. ROLLOVER SHALL NOT EXCEED 8.0% MAX.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-102 TURNPIKE BRIDGE DECK GEOMETRY MAINLINE - 3



NORMAL SECTION



SUPERELEVATED - CURVED RIGHT

FOUR LANE TURNPIKE ROADWAY

N.T.S.

NOTES:

1. FOR LONGITUDINAL GRADES LESS THAN 0.5%, LANE CROSS SLOPES = 2.0% MIN.

   FOR LONGITUDINAL GRADES GREATER THAN OR EQUAL TO 0.5%, LANE CROSS SLOPES = 1.5% MIN.

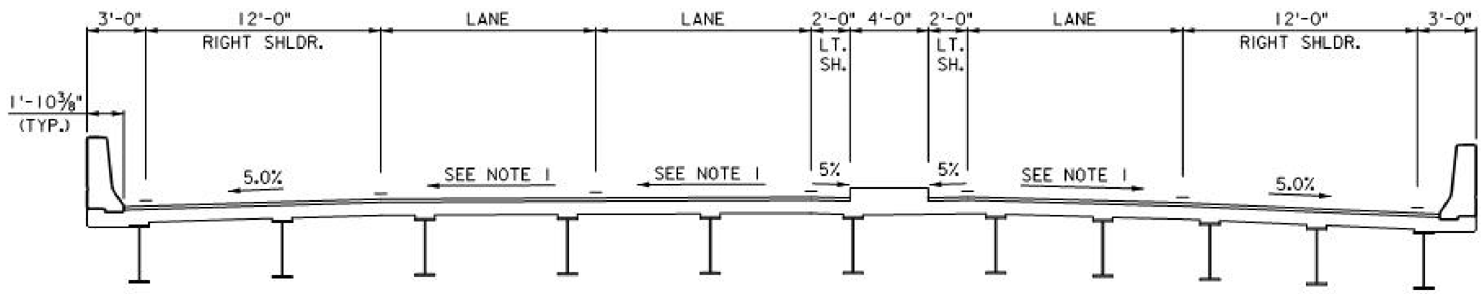
2. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

3. FOR SUPERELEVATED LANE CROSS SLOPE LESS THAN OR EQUAL TO 3.0%, SHOULDER CROSS SLOPE SHALL BE 5.0% MAX.

   FOR SUPERELEVATED LANE CROSS SLOPE GREATER THAN 3.0% TO 5.0% MAX., SHOULDER CROSS SLOPE SHALL VARY FROM 5.0% MAX TO 3.0% MIN. ROLLOVER SHALL NOT EXCEED 8.0% MAX.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-103 TURNPIKE BRIDGE DECK GEOMETRY RAMPS - 1



NORMAL SECTION

TWO WAY TURNPIKE RAMPS

N.T.S.

NOTES:

1. FOR LONGITUDINAL GRADES LESS THAN 0.5%, LANE CROSS SLOPES = 2.0% MIN.

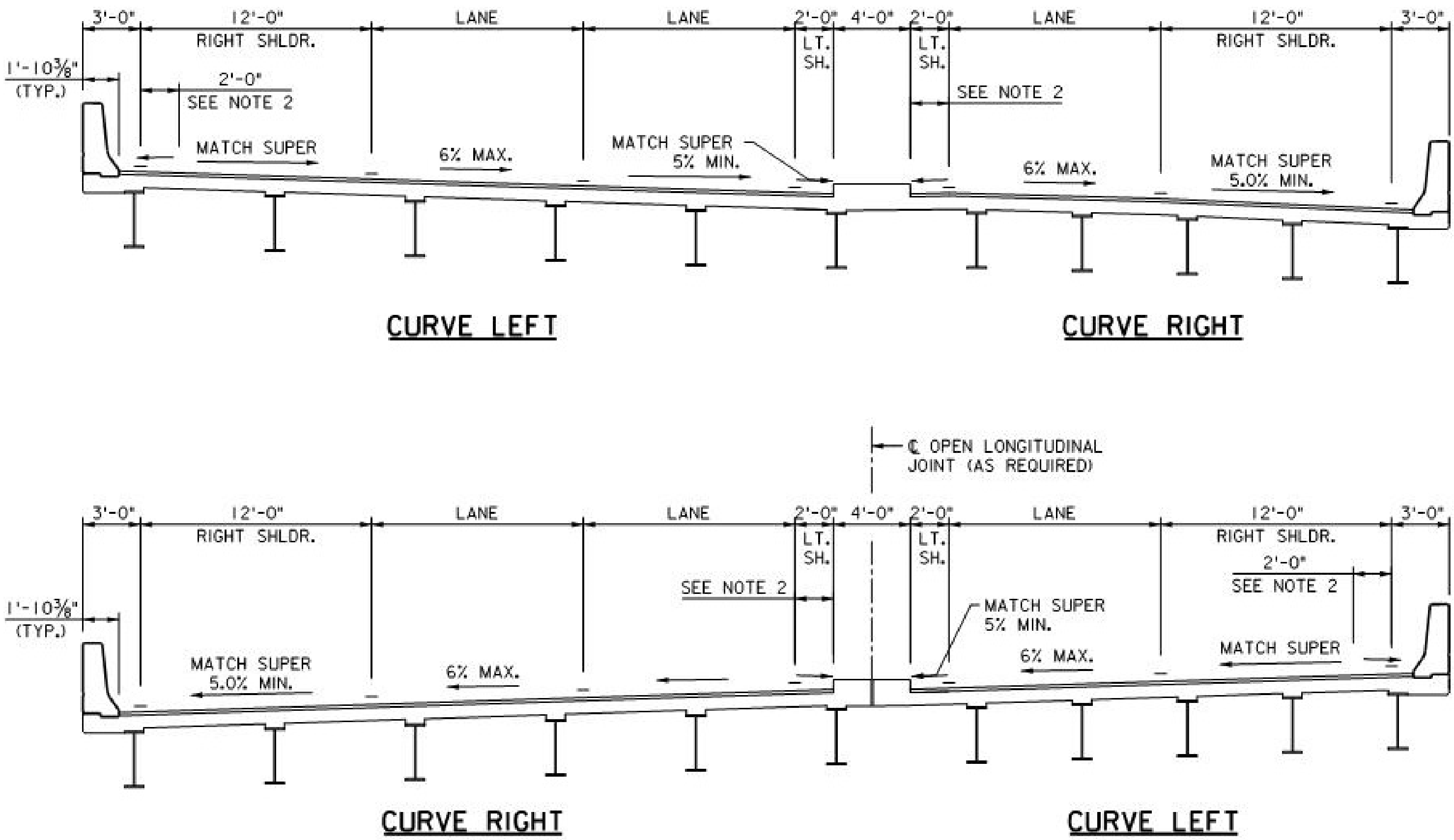
   FOR LONGITUDINAL GRADES GREATER THAN OR EQUAL TO 0.5%, LANE CROSS SLOPES = 1.5% MIN.

2. FOR ROADWAY WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.

3. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-104 TURNPIKE BRIDGE DECK GEOMETRY RAMPS - 2



SUPER GREATER THAN 3.0%
TWO WAY TURNPIKE RAMPS

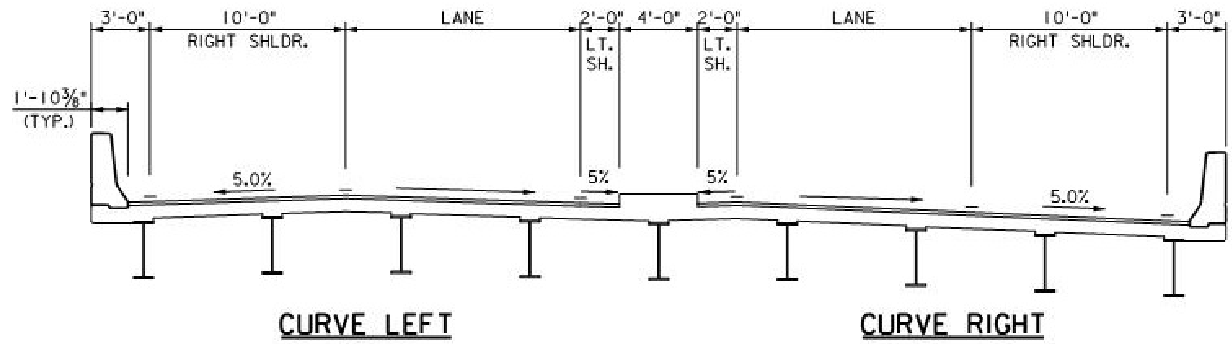
N.T.S.

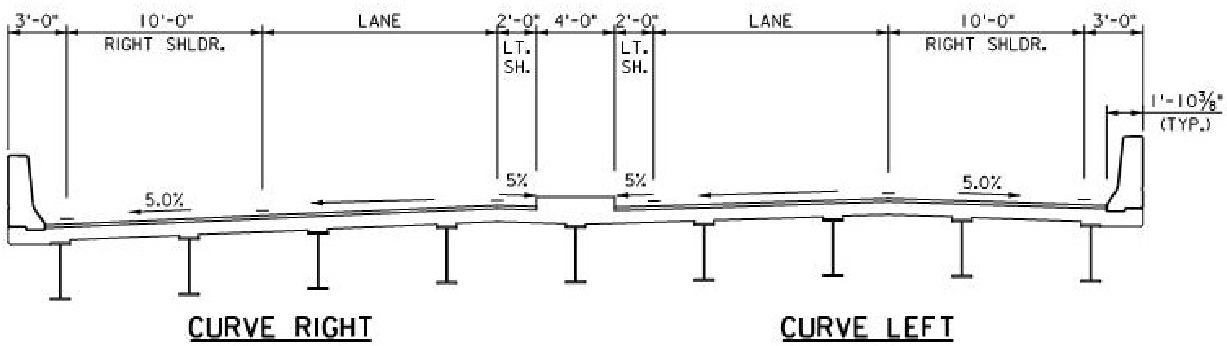
NOTES:

1. FOR ROADWAY WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.
2. FOR SUPERELEVATED ROADWAY CROSS SLOPE GREATER THAN 3.0% TO 6.0% MAX., THE SHOULDER CROSS SLOPE SHALL VARY FROM 5.0% MAX. TO 2.0% MIN. ROLLOVER SHALL NOT EXCEED 8.0% MAX.
3. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-105 TURNPIKE BRIDGE DECK GEOMETRY RAMPS - 3





SUPER LESS THAN OR EQUAL TO 3.0%
TWO WAY TURNPIKE RAMPS

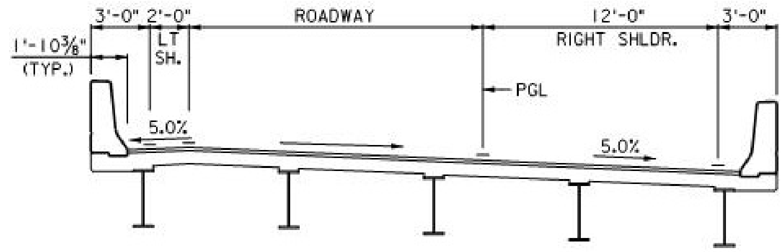
N.T.S.

NOTES:

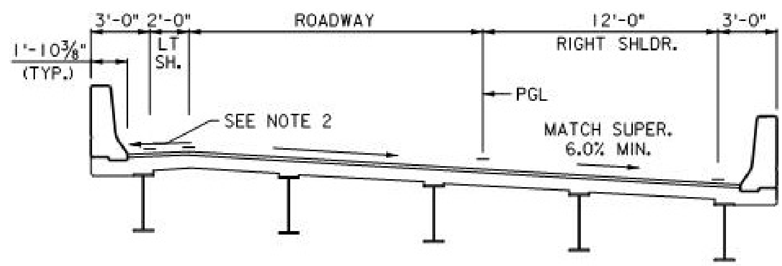
1. FOR ROADWAY WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.
2. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-106 TURNPIKE BRIDGE DECK GEOMETRY RAMPS - 4



NORMAL SECTION



CURVE RIGHT

ONE WAY TURNPIKE RAMPS

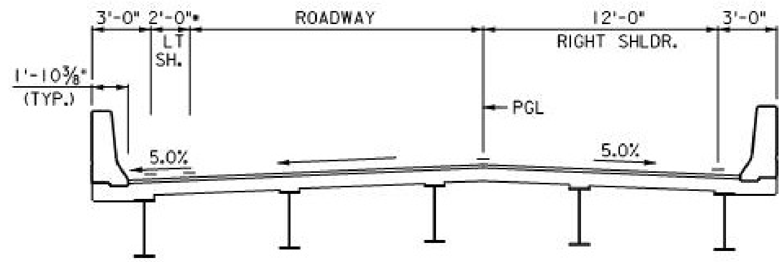
N.T.S.

NOTES:

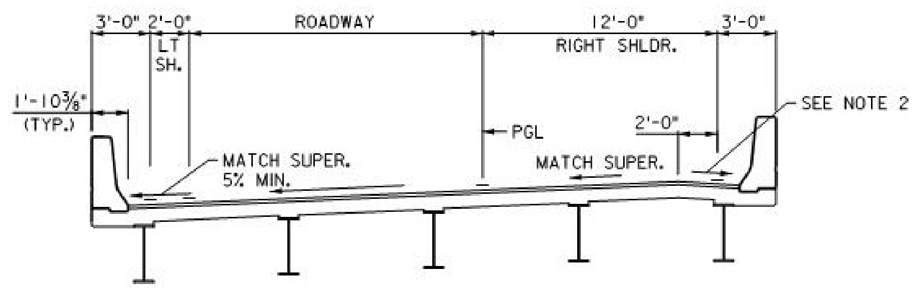
1. FOR ROADWAY WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.
2. FOR SUPERELEVATED ROADWAY CROSS SLOPE LESS THAN OR EQUAL TO 3.0%, SHOULDER CROSS SLOPE = 5.0%.
3. FOR SUPERELEVATED ROADWAY CROSS SLOPE GREATER THAN 3.0% TO 6.0% MAX., SHOULDER CROSS SLOPE SHALL VARY FROM 5% MAX. TO 2.0% MIN.. ROLLOVER SHALL NOT EXCEED 8.0%.
4. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-107 TURNPIKE BRIDGE DECK GEOMETRY RAMPS - 5



SUPER LESS THAN OR EQUAL TO 3.0% CURVE LEFT



SUPER GREATER THAN 3.0% CURVE LEFT

ONE WAY TURNPIKE RAMPS

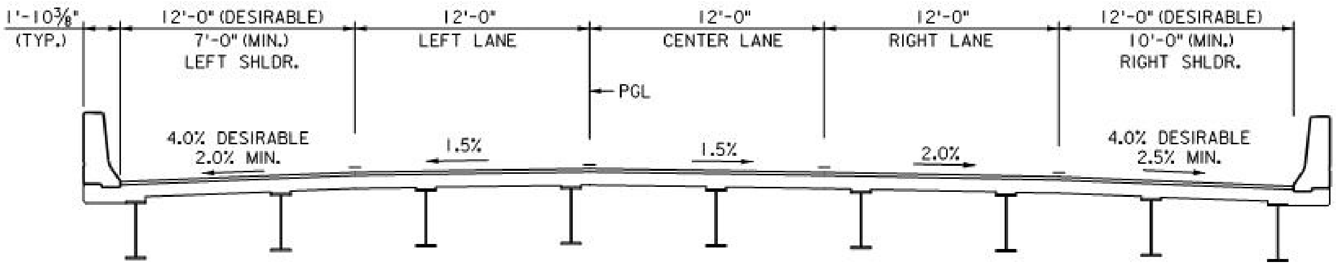
* WIDEN IF REQUIRED FOR HORIZONTAL SIGHT DISTANCE
N.T.S.

NOTES:

1. FOR ROADWAY WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.
2. FOR SUPERELEVATED ROADWAY CROSS SLOPE LESS THAN OR EQUAL TO 3.0%, SHOULDER CROSS SLOPE = 5.0%.
3. FOR SUPERELEVATED ROADWAY CROSS SLOPE GREATER THAN 3.0% TO 6.0% MAX., SHOULDER CROSS SLOPE SHALL VARY FROM 5.0% MAX. TO 2.0% MIN. ROLLOVER SHALL NOT EXCEED 8.0%.
4. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-108 PARKWAY BRIDGE DECK GEOMETRY MAINLINE - 1



NORMAL SECTION

THREE LANE PARKWAY ROADWAY

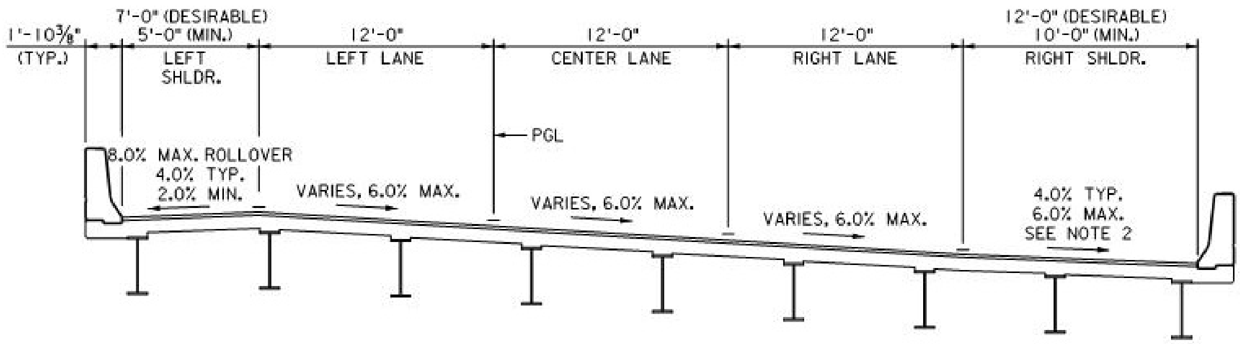
N.T.S.

NOTES:

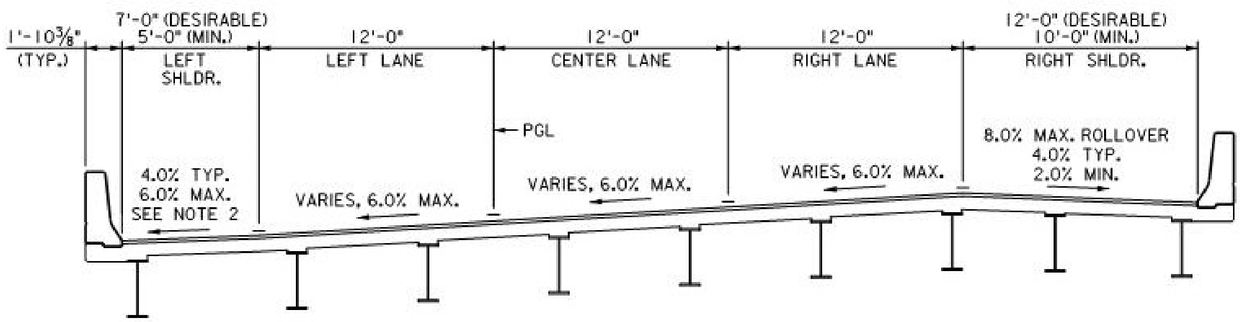
1. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-109 PARKWAY BRIDGE DECK GEOMETRY MAINLINE - 2



SUPERELEVATED - CURVE RIGHT



SUPERELEVATED - CURVE LEFT

THREE LANE PARKWAY ROADWAY

N.T.S.

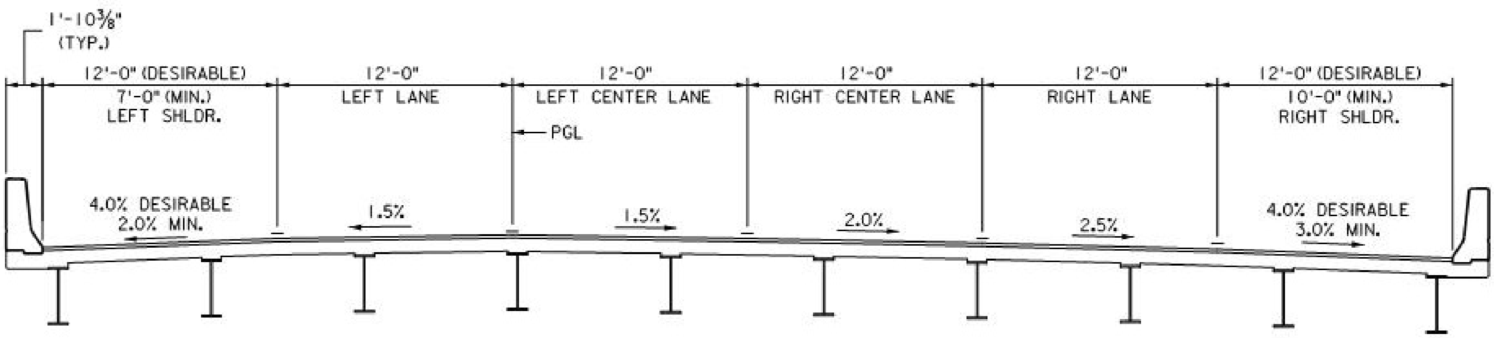
NOTES:

1. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

2. SHOULDER CROSS SLOPE SHALL MATCH ROADWAY CROSS SLOPE WHEN ROADWAY CROSS SLOPE EXCEEDS 4.0%.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-110 PARKWAY BRIDGE DECK GEOMETRY MAINLINE - 3



NORMAL SECTION

FOUR LANE PARKWAY ROADWAY

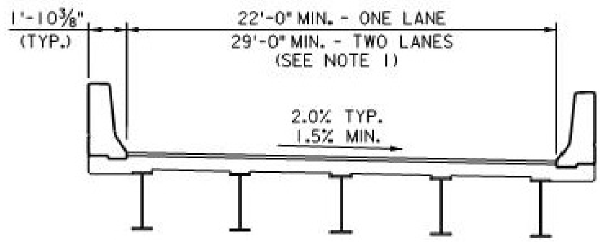
N.T.S.

NOTES:

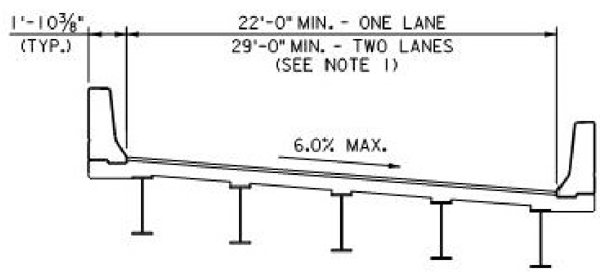
1. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-111 PARKWAY BRIDGE DECK GEOMETRY RAMPS - 1



NORMAL SECTION



CURVE RIGHT
CURVE LEFT SIMILAR

ONE WAY PARKWAY RAMPS

N.T.S.

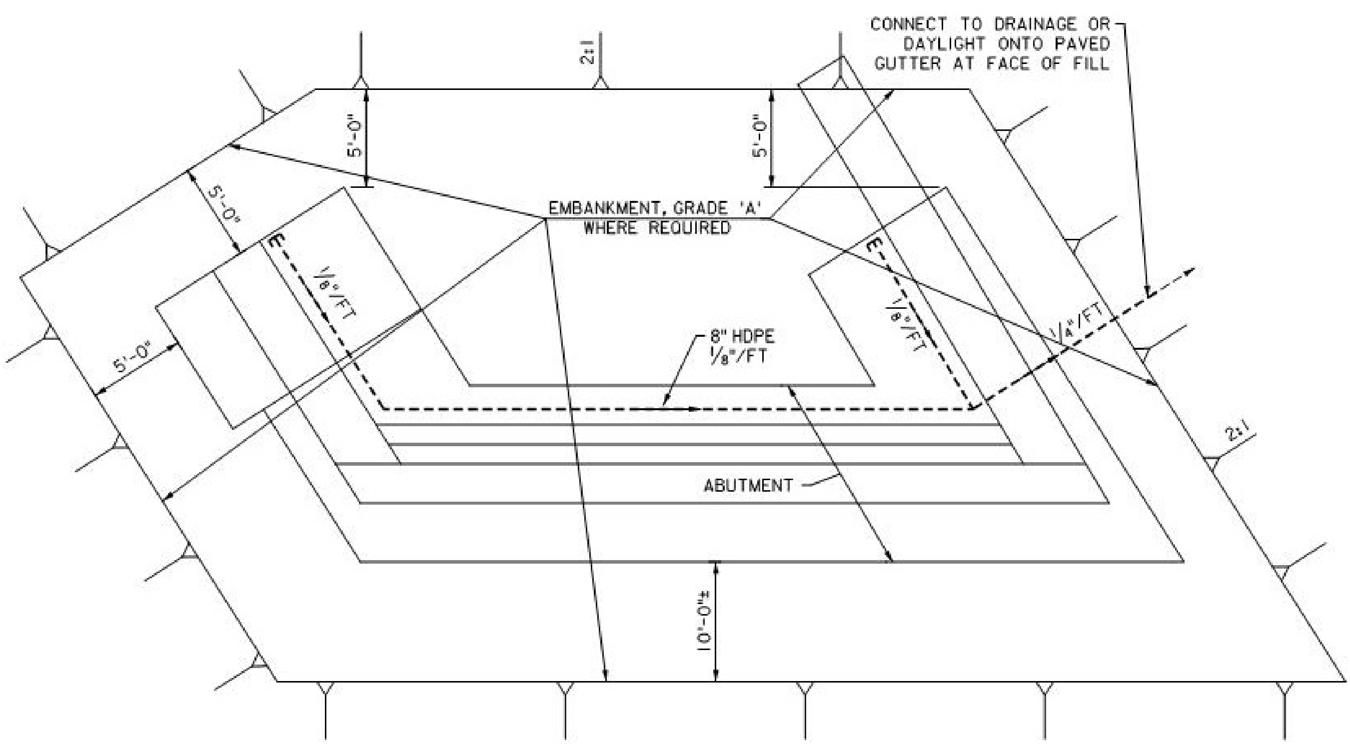
NOTES:

1. FOR ROADWAY WIDTHS, SHOULDER WIDTHS AND CROSS SLOPES FOLLOW ROADWAY STANDARDS.

3. SECTIONS APPLY TO NEW CONSTRUCTION. MODIFICATIONS TO EXISTING STRUCTURES WILL BE REVIEWED ON AN INDIVIDUAL BASIS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-200 SUBSTRUCTURE DETAILS - 1

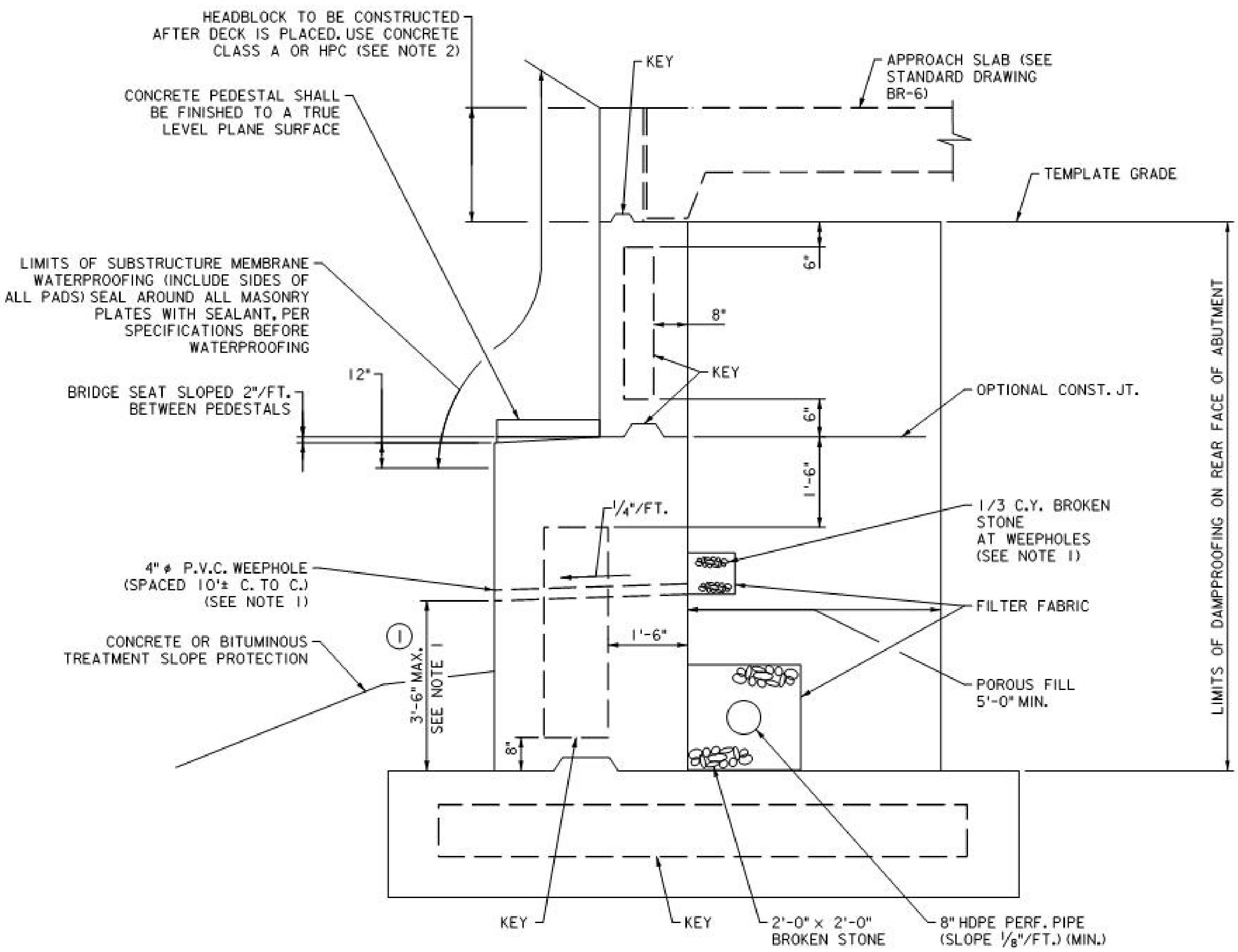


ABUTMENT PLAN

N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-201 SUBSTRUCTURE DETAILS - 2



TYPICAL ABUTMENT SECTION

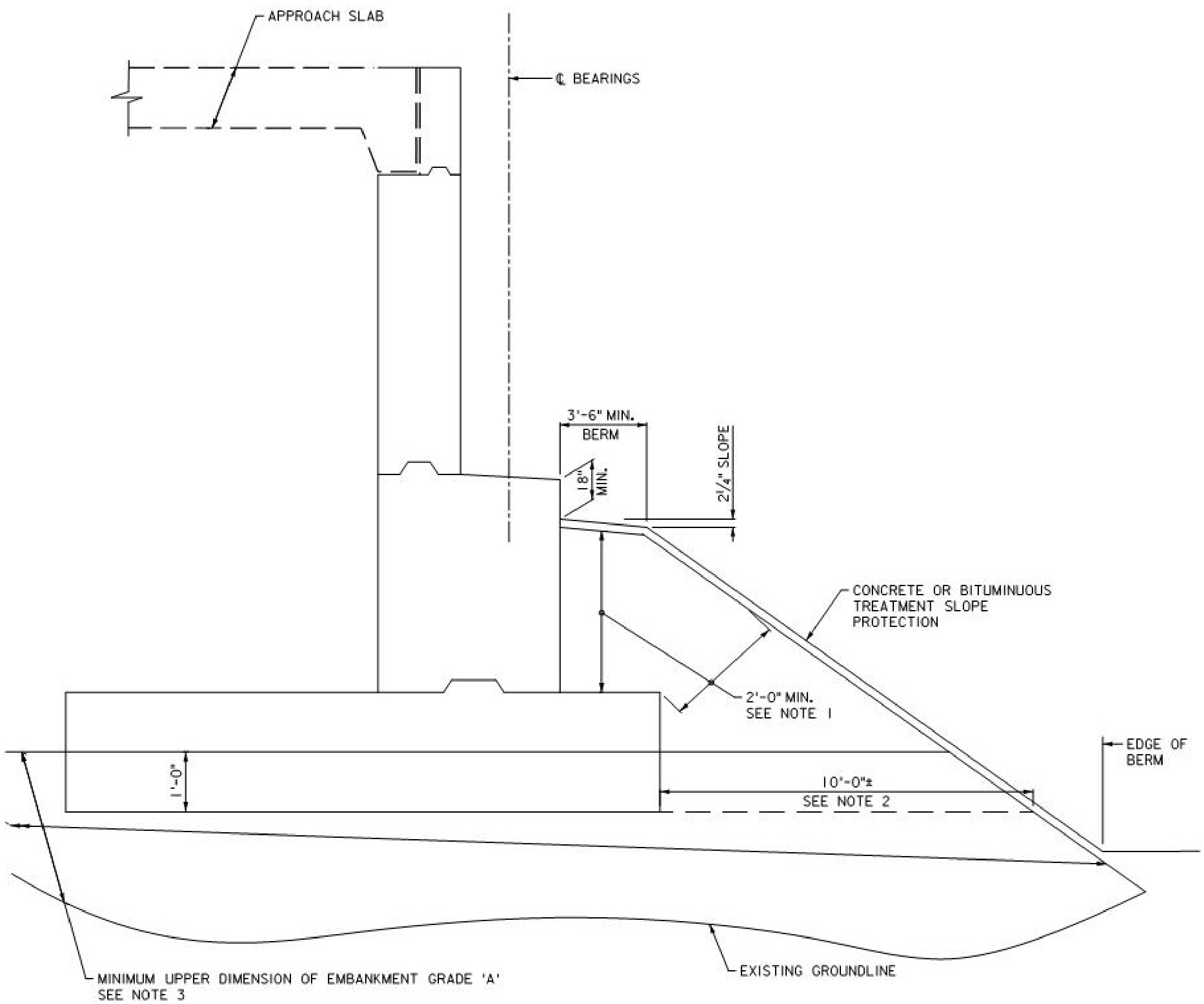
N.T.S.

NOTES:

1. ABUTMENTS AND WALLS WILL NORMALLY BE DRAINED BY WEEPHOLES AND 8" HDPE PIPE, AS SHOWN. THE WEEPHOLES SHOULD NOT BE USED WHERE THEY DRAIN ONTO A SIDEWALK OR ROADWAY, OR WHERE THEY MUST BE PLACED MORE THAN 3'-6" ABOVE THE TOP OF FOOTING.
2. HEADBLOCK SHALL BE HPC WHEN DECK IS CONSTRUCTED OF HPC.
3. PILES NOT SHOWN FOR CLARITY.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-202 SUBSTRUCTURE DETAILS - 3



ABUTMENT IN FILL ON SPECIAL SUBGRADE MATERIAL

ABUTMENT DETAILS

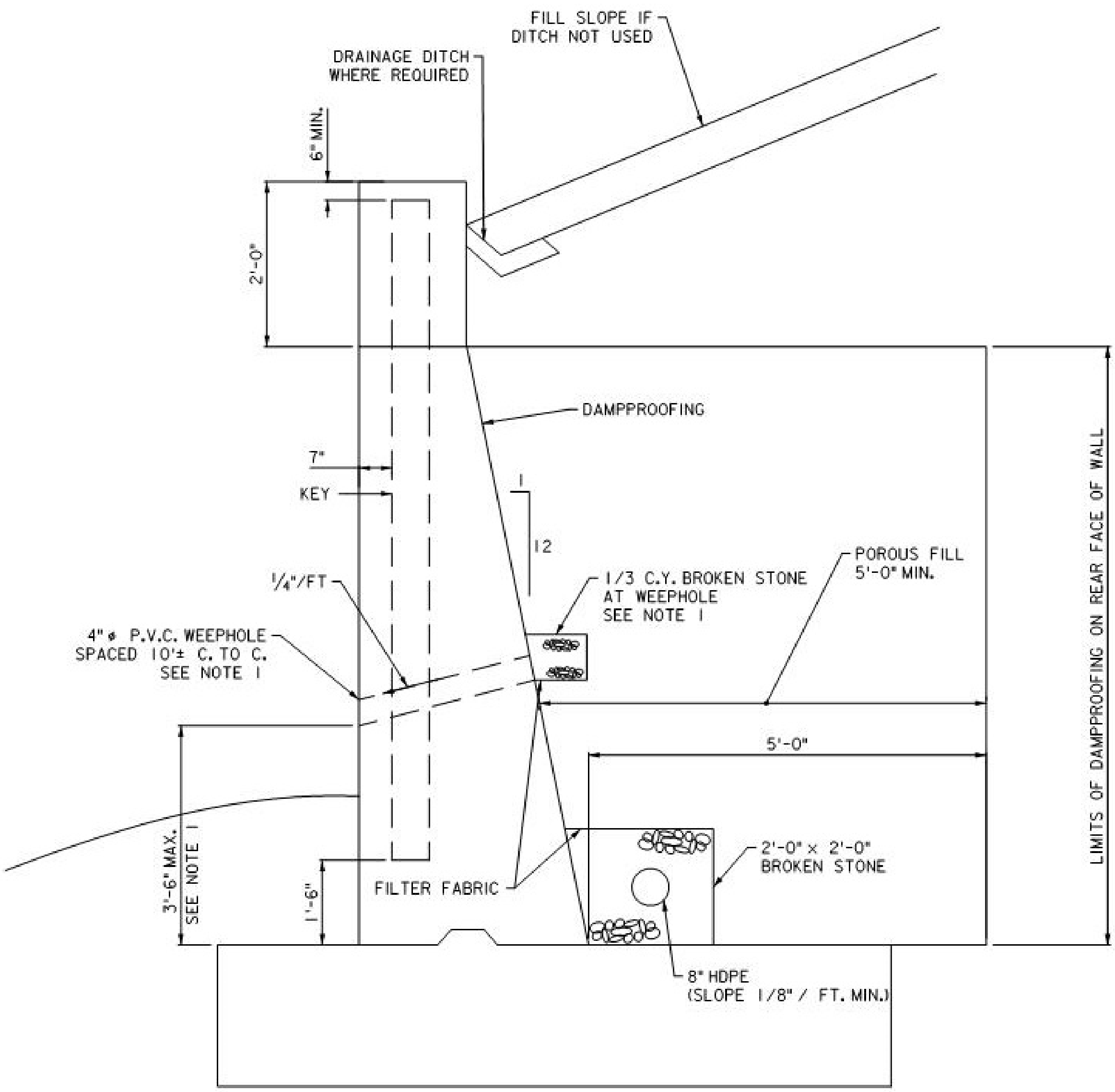
N.T.S.

NOTES:

1. MAINTAIN 2'-0" MIN. COVER TO ALL FOOTINGS. INCREASE BERM WIDTH, OR LOWER FOOTING IF NECESSARY.
2. FOR SOIL BEARING ABUTMENTS MAINTAIN 10FT ±. INCREASE BERM WIDTH, OR LOWER FOOTINGS IF NECESSARY.
3. PLACE COMMON EMBANKMENT IN FILL AREAS UNDER PILE SUPPORTED ABUTMENTS.
4. PILES NOT SHOWN FOR CLARITY.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-203 SUBSTRUCTURE DETAILS - 4



WINGWALL OR RETAINING WALL SECTION WITHOUT PARAPET

WALL DETAILS

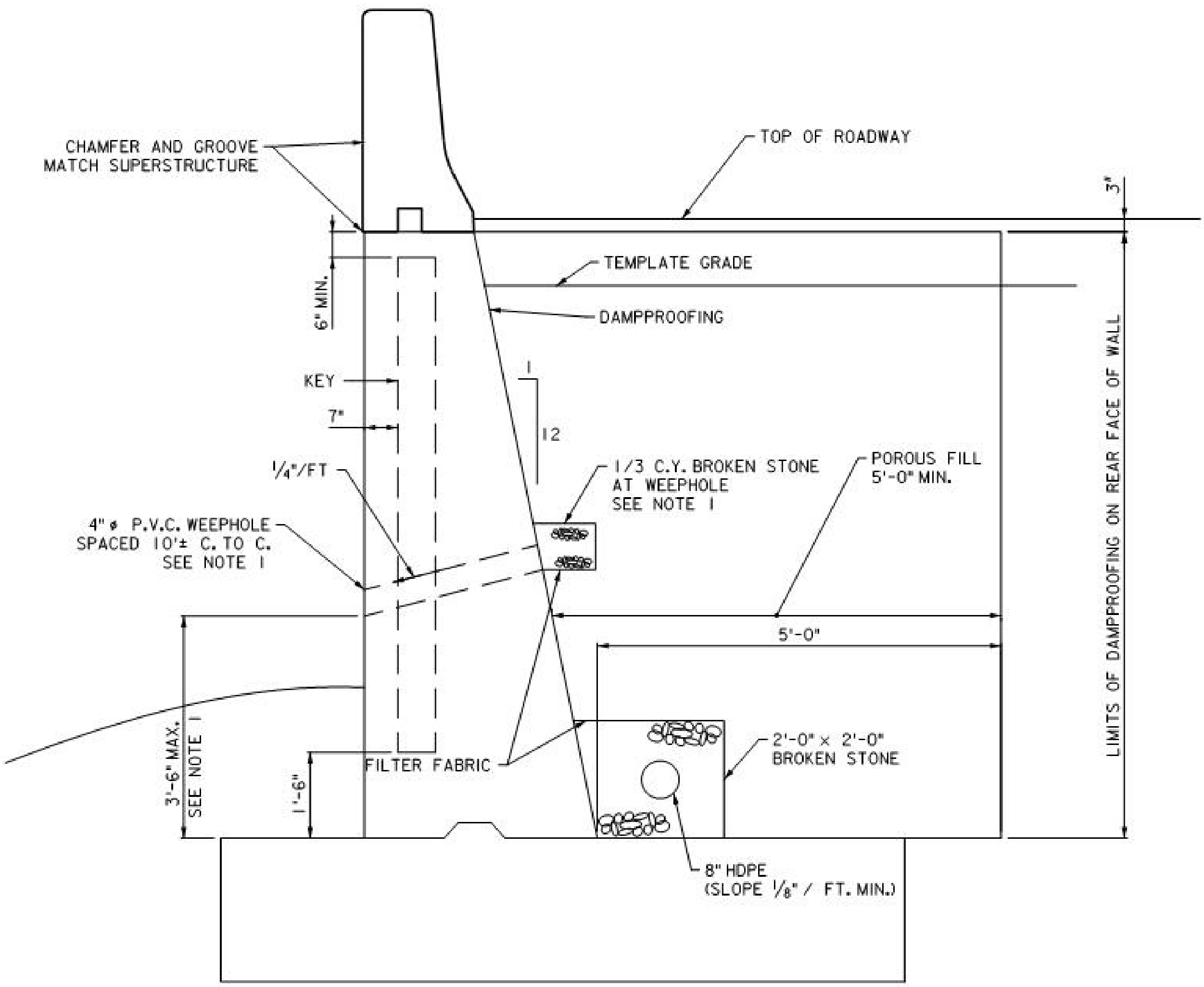
N.T.S.

NOTES:

1. ABUTMENTS AND WALLS WILL NORMALLY BE DRAINED BY WEEPHOLES AND 8" HDPE AS SHOWN. THE WEEPHOLES SHOULD NOT BE USED WHERE THEY DRAIN ONTO A SIDEWALK OR ROADWAY, OR WHERE THEY MUST BE PLACED MORE THAN 3'-6" ABOVE THE TOP OF FOOTING.
2. SEE PLATE EXHIBIT 2-202 FOR ADDITIONAL DETAILS.
3. PILES NOT SHOWN FOR CLARITY.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-204 SUBSTRUCTURE DETAILS - 5



WINGWALL OR RETAINING WALL SECTION WITH A PARAPET

WALL DETAILS

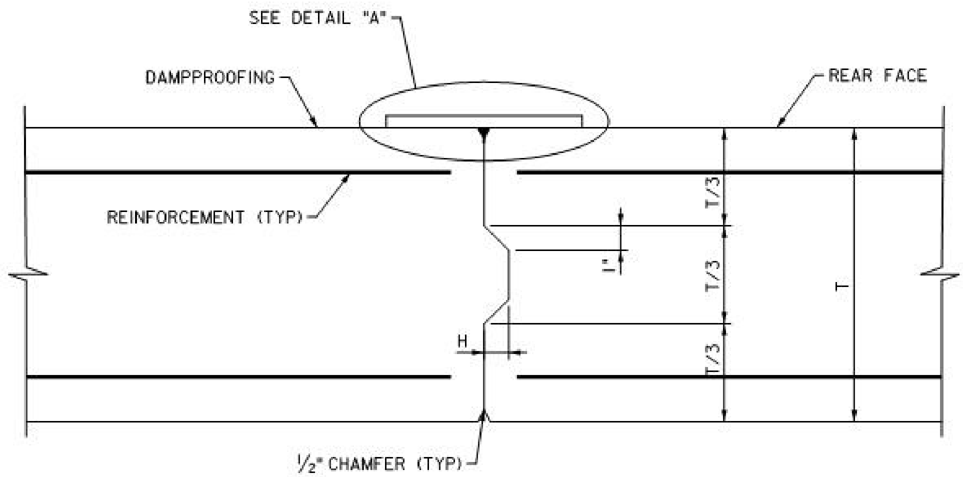
N.T.S.

NOTES:

1. ABUTMENTS AND WALLS WILL NORMALLY BE DRAINED BY WEEPHOLES AND 8" HDPE AS SHOWN. THE WEEPHOLES SHOULD NOT BE USED WHERE THEY DRAIN ONTO A SIDEWALK OR ROADWAY, OR WHERE THEY MUST BE PLACED MORE THAN 3'-6" ABOVE THE TOP OF FOOTING.
2. PILES NOT SHOWN FOR CLARITY.
3. SEE EXHIBIT 2-202 FOR ADDITIONAL DETAILS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-205 SUBSTRUCTURE DETAILS - 6



CONSTRUCTION JOINT

JOINT AND KEY DETAILS

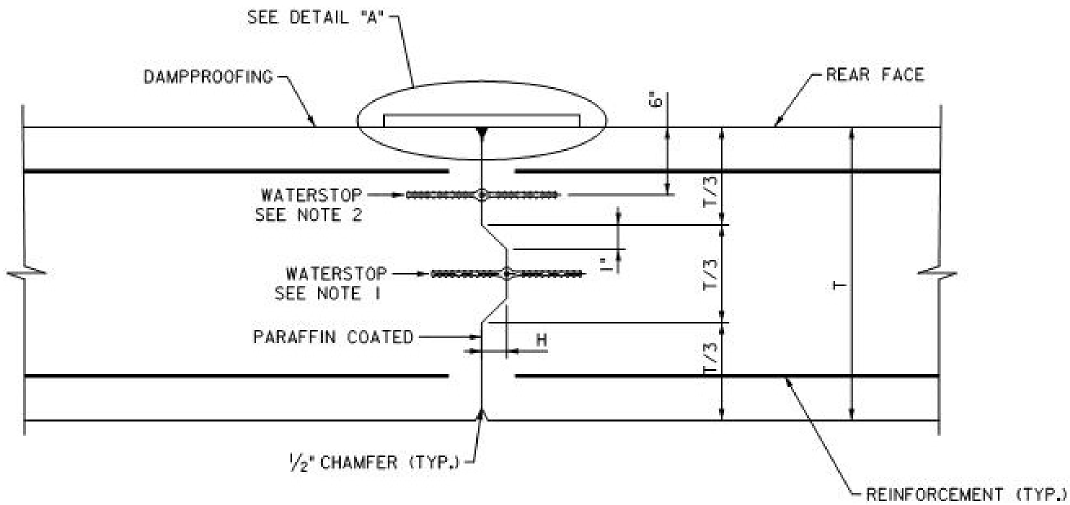
N.T.S.

NOTES:

1. H=2" FOR JOINTS BELOW HEADBLOCK AND 3" FOR ALL OTHER JOINTS.
2. FOR DETAIL A, SEE EXHIBIT 2-209.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-206 SUBSTRUCTURE DETAILS - 7



CONTRACTION JOINT

JOINT AND KEY DETAILS

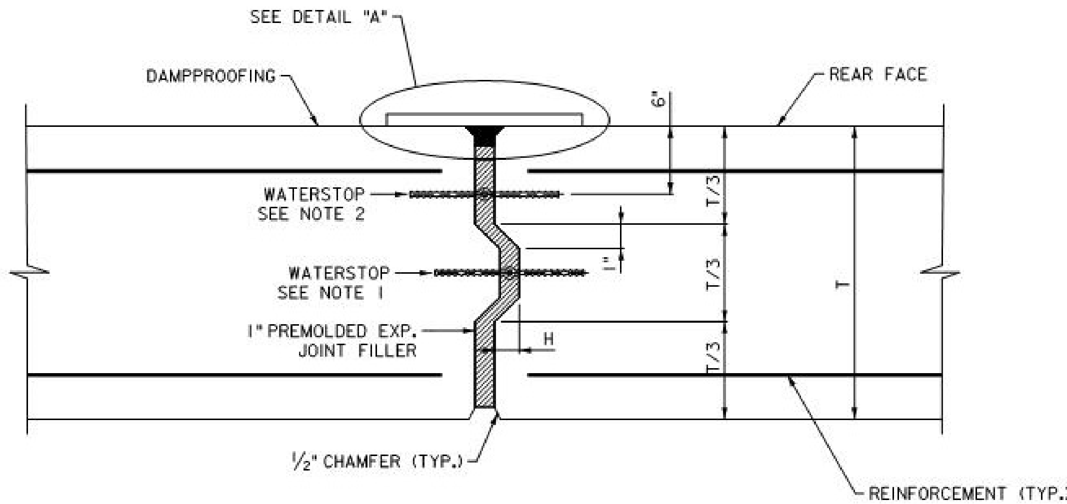
N.T.S.

NOTES:

1. PLACEMENT OF WATERSTOP IN BOX CULVERTS, TOLL PLAZA TUNNELS AND WALL LESS THAN 2'-0" WIDE.
2. PLACEMENT OF WATERSTOP IN ABUTMENTS AND BATTERED RETAINING WALLS.
3. REINFORCEMENT IS DISCONTINUOUS AT CONTRACTION AND EXPANSION JOINTS.
4. CONTRACTION JOINTS SHALL BE TIGHT AND SHALL BE PARAFFIN COATED.
5. H=2" FOR JOINTS BELOW HEADBLOCK AND 3" FOR ALL OTHER JOINTS.
6. FOR DETAIL A, SEE EXHIBIT 2-209.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-207 SUBSTRUCTURE DETAILS - 8



EXPANSION JOINT

JOINT AND KEY DETAILS

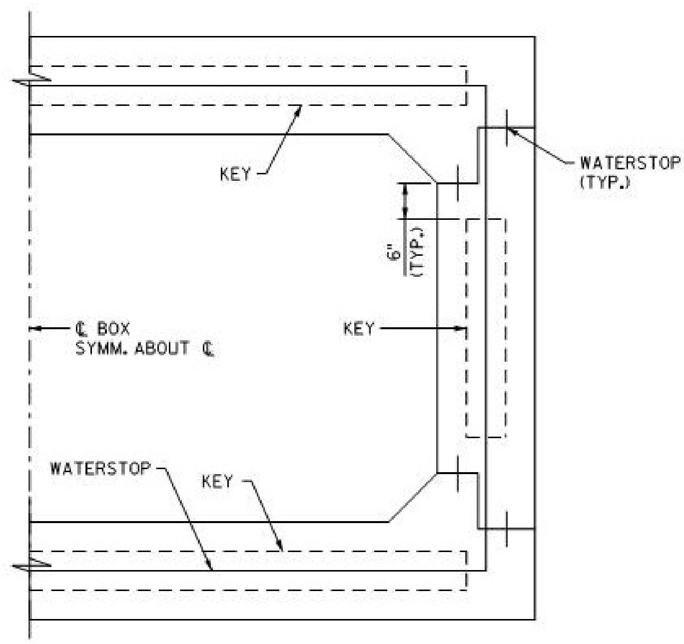
N.T.S.

NOTES:

1. PLACEMENT OF WATERSTOP IN BOX CULVERTS, TOLL PLAZA TUNNELS AND WALL LESS THAN 2'-0" WIDE.
2. PLACEMENT OF WATERSTOP IN ABUTMENTS AND BATTERED RETAINING WALLS.
3. REINFORCEMENT IS DISCONTINUOUS AT CONTRACTION AND EXPANSION JOINTS.
4. H=2" FOR JOINTS BELOW HEADBLOCK AND 3" FOR ALL OTHER JOINTS.
5. FOR DETAIL A, SEE EXHIBIT 2-209.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-208 SUBSTRUCTURE DETAILS - 9



BOX CULVERT OR TUNNEL

JOINT AND KEY DETAILS

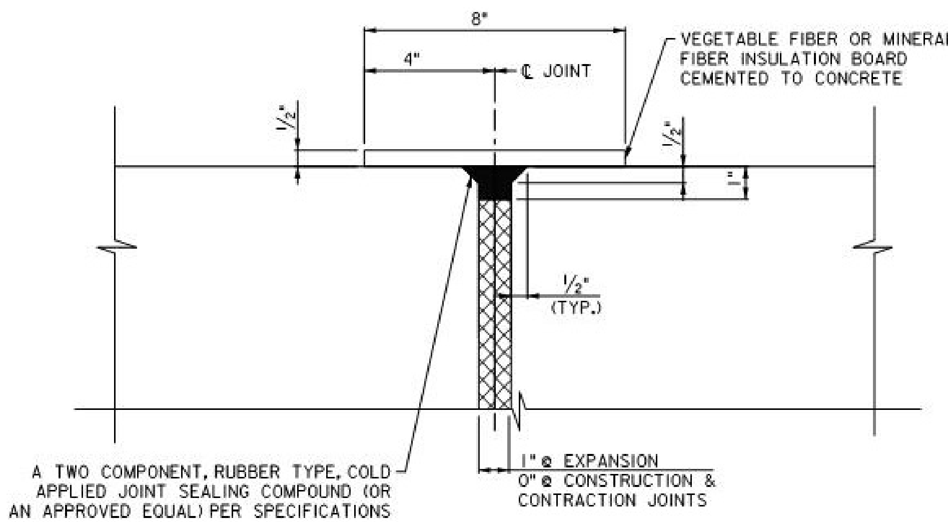
N.T.S.

NOTES:

1. WATERSTOPS SHALL BE CONTINUOUS PLASTIC WATERSTOPS, WITHOUT SPLICES, ALONG THE BOTTOM HALF OF CULVERTS OR TUNNELS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-209 SUBSTRUCTURE DETAILS - 10



DETAIL A

JOINT AND KEY DETAILS

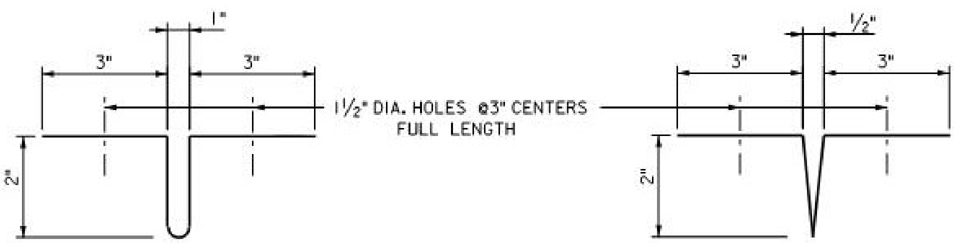
N.T.S.

NOTES:

1. FOR LOCATION OF DETAIL A, SEE EXHIBITS 2-205, 2-206 AND 2-207.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

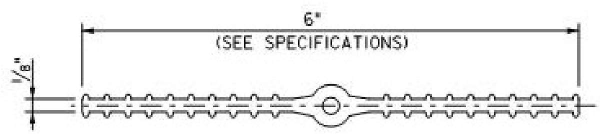
EXHIBIT 2-210 SUBSTRUCTURE DETAILS - 11



EXPANSION

CONTRACTION

16 OZ. COPPER WATERSTOP - 10" WIDE



JOINT AND KEY DETAILS

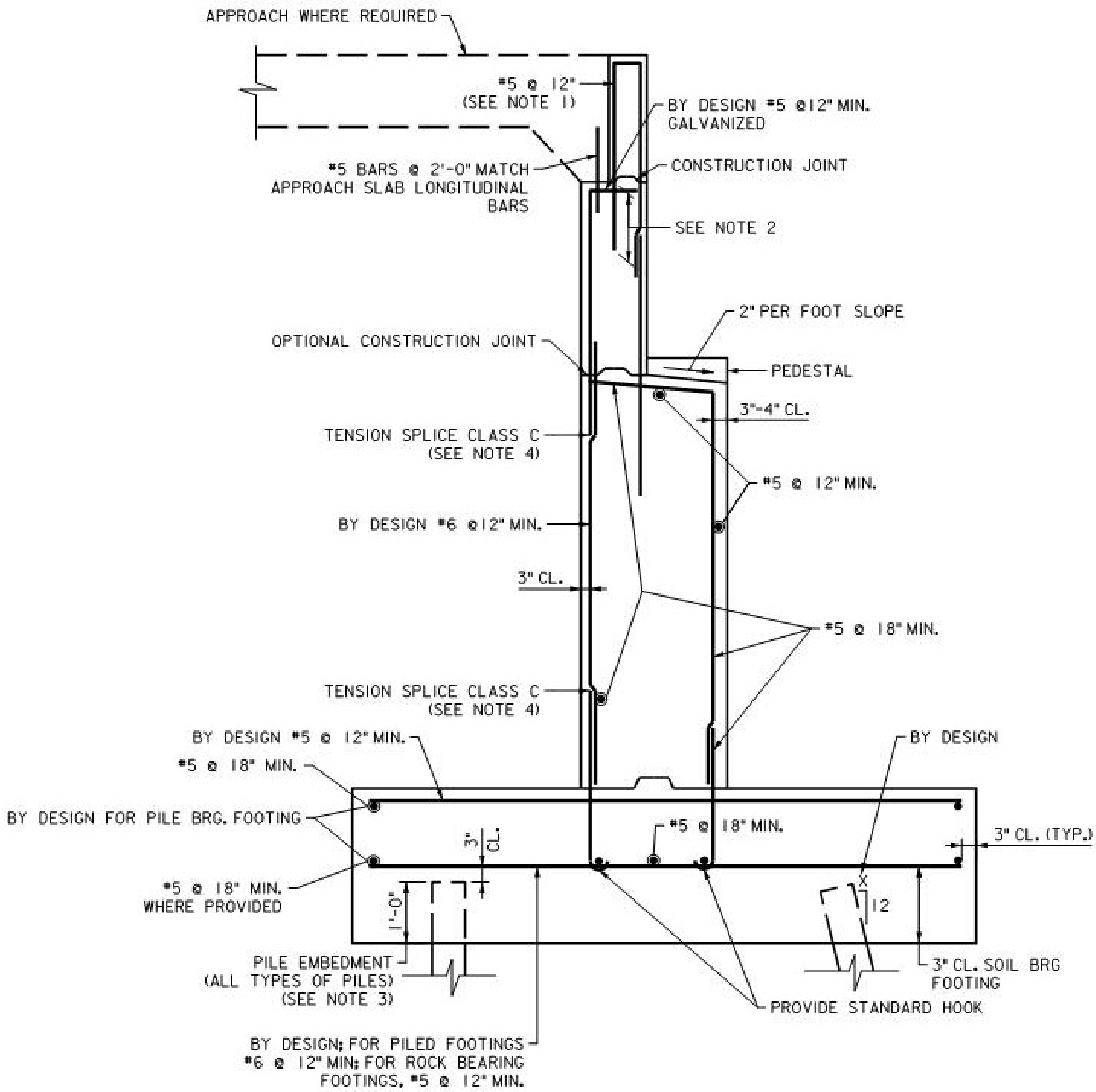
N.T.S.

NOTES:

1. WATERSTOP SHALL BE EITHER 6" PLASTIC, OR 16 OZ. COPPER (10" LONG), AS DIRECTED BY THE ENGINEER.
2. WHEREVER PRACTICABLE, WATERSTOP SHALL BE ONE CONTINUOUS LENGTH.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-211 SUBSTRUCTURE DETAILS - 12



ABUTMENT REINFORCEMENT DETAIL (WALL REINFORCEMENT SIMILAR)

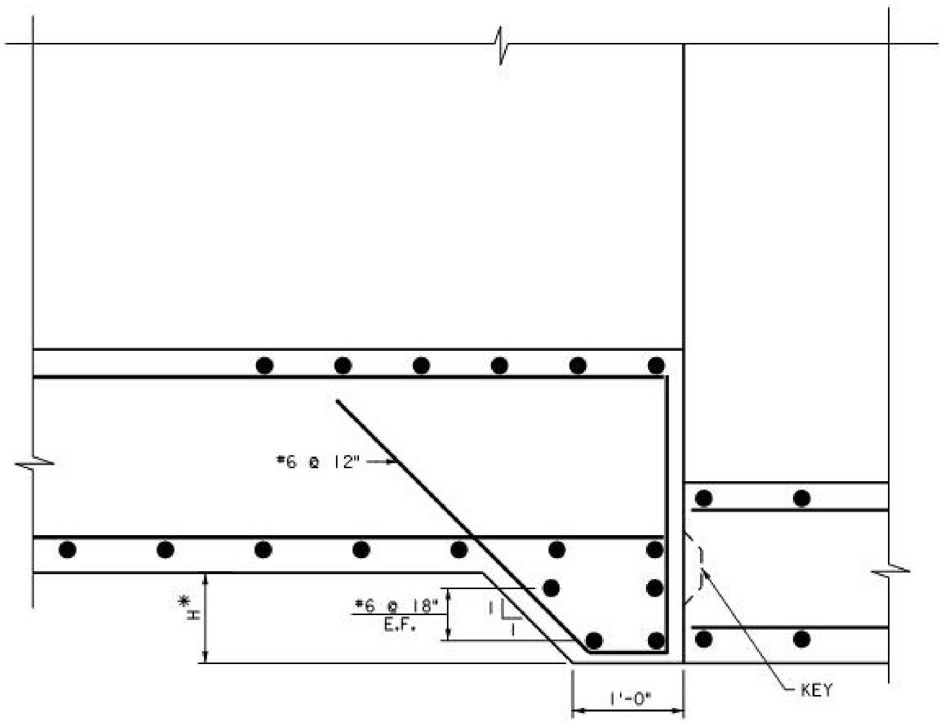
N.T.S.

NOTES:

1. ALL REINFORCEMENT IN HEADBLOCK SHALL BE GALVANIZED.
2. ALL BARS EXTENDING FROM HEADBLOCK INTO ABUTMENT STEM SHALL BE CONSIDERED TENSION SPLICES. REFER TO CURRENT AASHTO FOR REQUIREMENTS.
3. WHEN REQUIRED BY DESIGN THE PILES SHALL BE DESIGNED AND DETAILED TO RESIST UPLIFT.
4. FOR SPLICE CRITERIA REFER TO CURRENT AASHTO REQUIREMENTS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-212 SUBSTRUCTURE DETAILS - 13



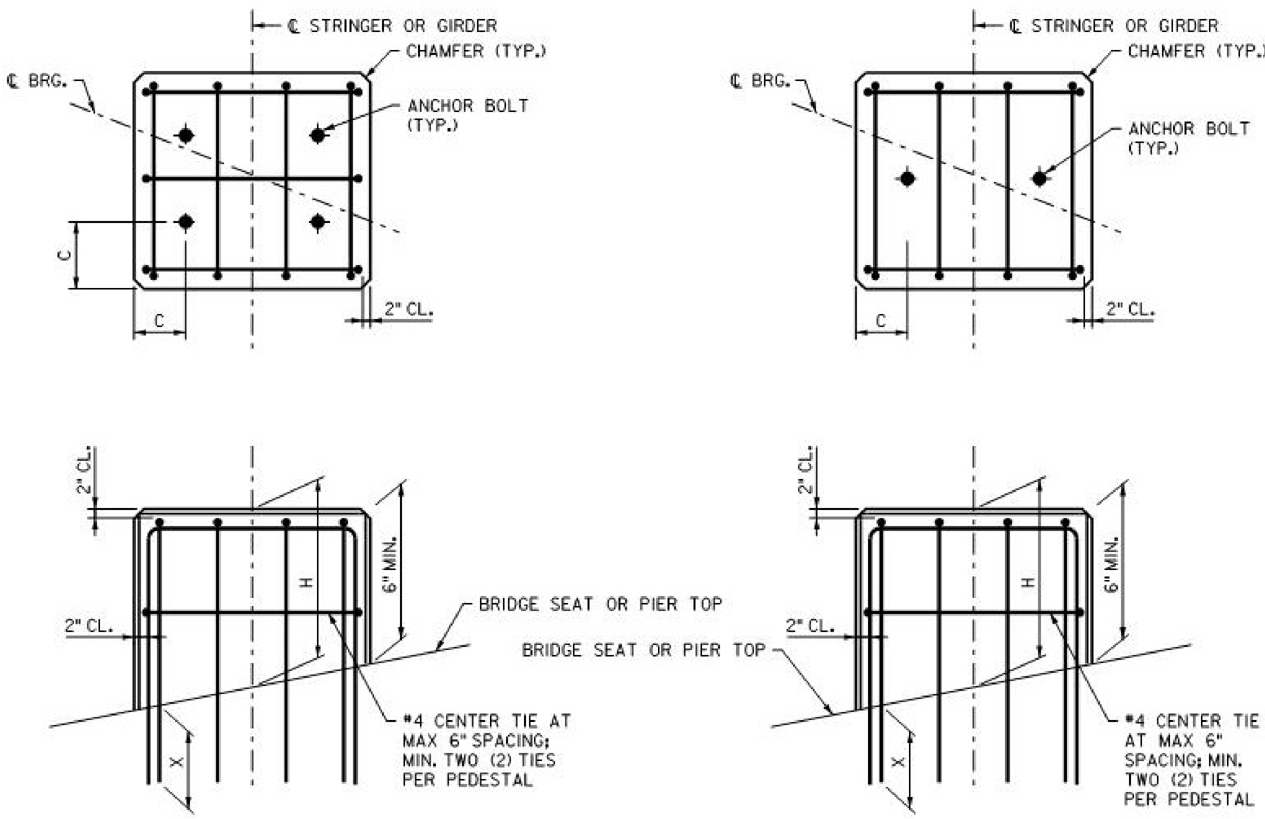
FOOTING HAUNCH DETAIL

N.T.S.

* REINFORCEMENT TO BE PROVIDED WHEN "H" GREATER THAN OR EQUAL TO 2'-0". "H" SHALL NOT BE GREATER THAN DEPTH OF THE FOOTING; EXCEPT THAT WHEN THE FOOTING IS ON PILES, "H" MAY BE TWICE THE DEPTH OF THE FOOTING.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-213 SUBSTRUCTURE DETAILS - 14



FOUR ANCHOR BOLTS

TWO ANCHOR BOLTS

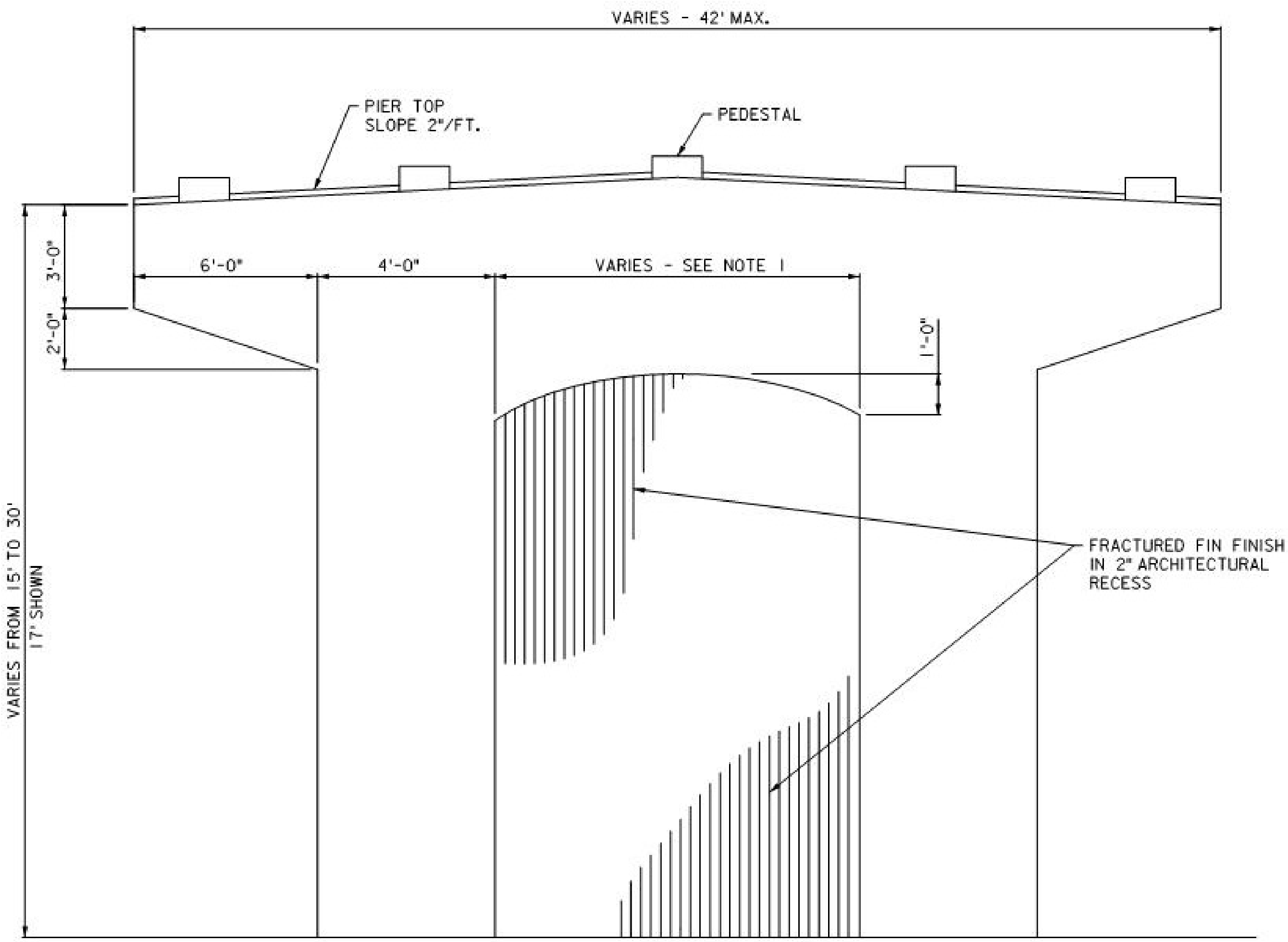
FOR H GREATER THAN 16", PEDESTALS SHALL BE DESIGNED AS COLUMNS.

H	C	BAR SIZE	X
TO 8"	5"	#4	12"
TO 12"	5"	#5	15"
TO 16"	6"	#5	15"

PEDESTAL REINFORCEMENT DETAILS
PIERS AND ABUTMENTS
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-214 SUBSTRUCTURE DETAILS - 15



TURNPIKE TYPICAL 'NARROW' PIER DETAIL GUIDELINES

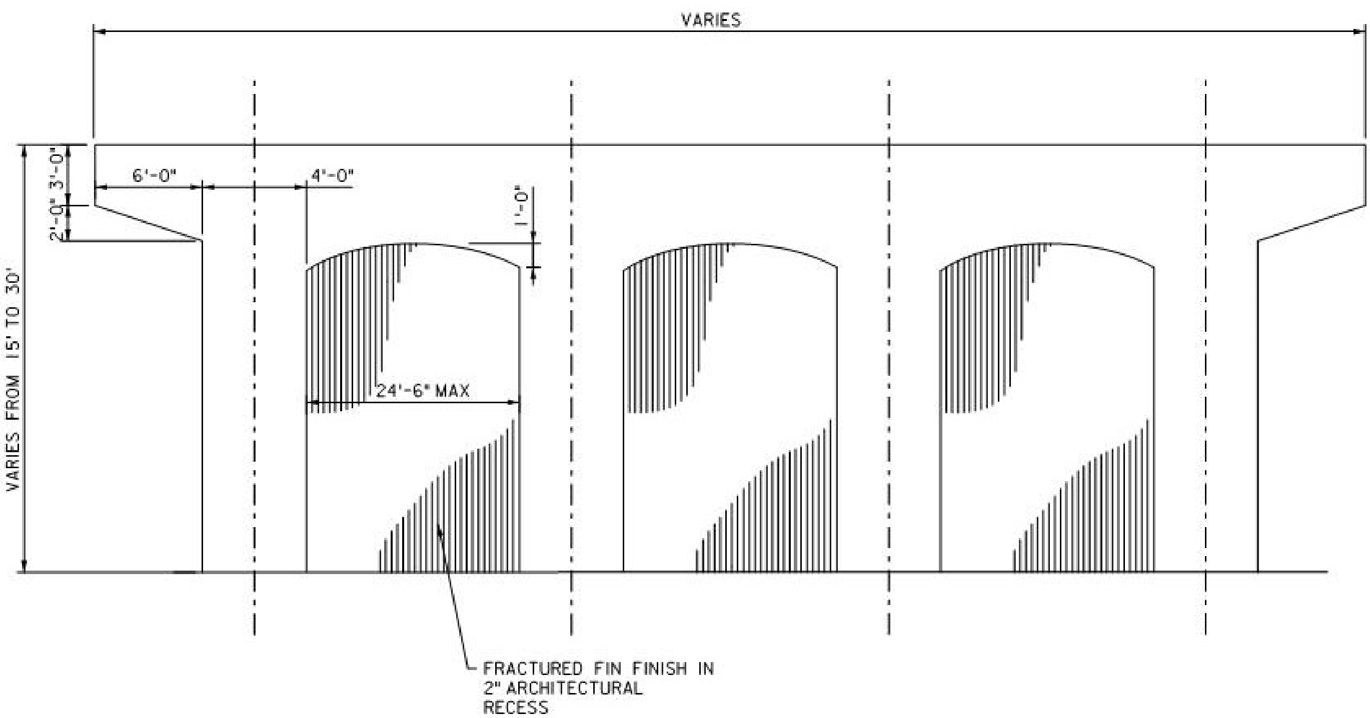
N.T.S.

NOTES:

1. WIDTH OF RECESS MAY VARY FROM 8'-22'. IF LESS THAN 8', OMIT RECESS.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-215 SUBSTRUCTURE DETAILS - 16

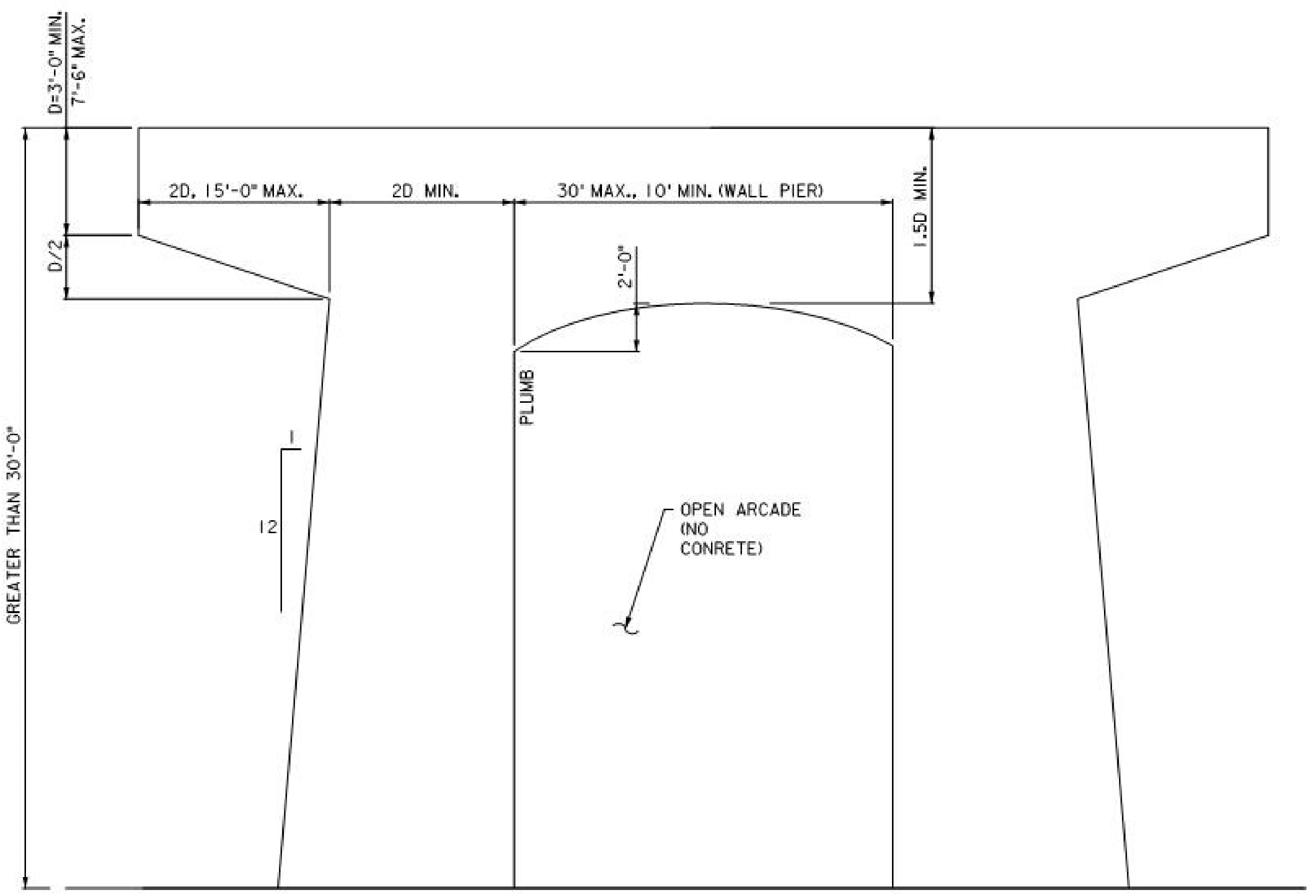


TURNPIKE TYPICAL 'WIDE' PIER DETAIL GUIDELINES

N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-216 SUBSTRUCTURE DETAILS - 17



TYPICAL MAJOR BRIDGE PIER

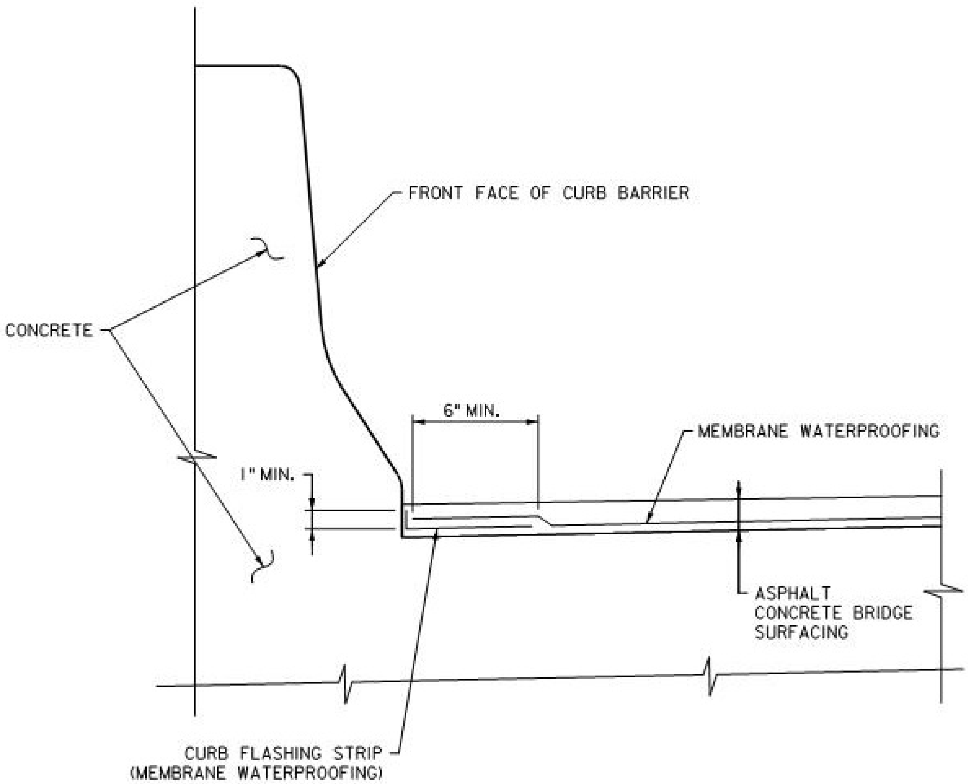
N.T.S.

NOTES:

1. THIS OPTION REQUIRES PRIOR APPROVAL FROM NJTA.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-300 DECK DETAILS - 1

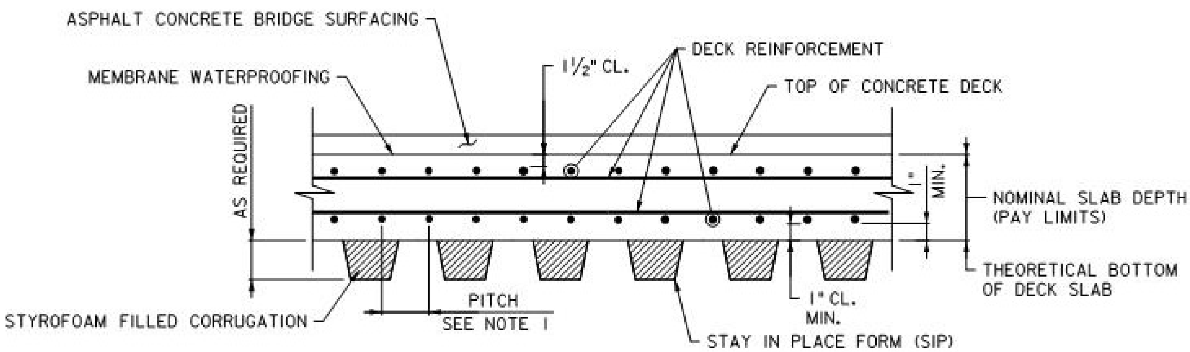


MEMBRANE WATERPROOFING DETAIL AT CURB

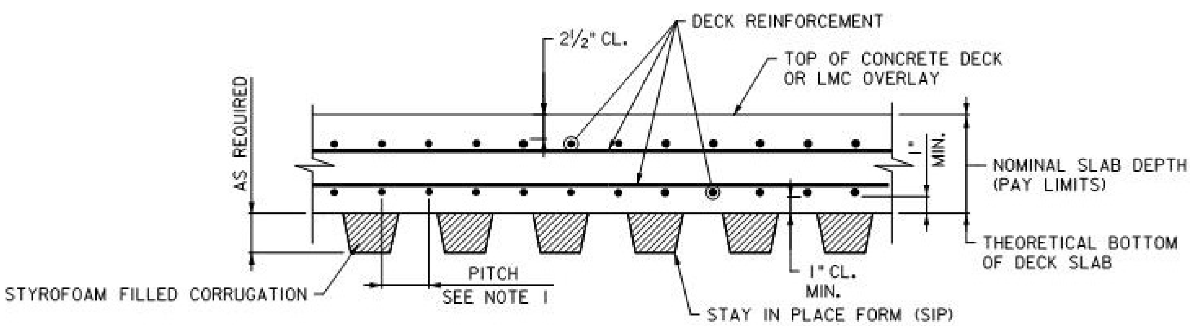
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-301 DECK DETAILS - 2



LONGITUDINAL SECTION
(ASPHALT SURFACED DECKS)



LONGITUDINAL SECTION
(BARE DECKS AND LMC OVERLAYED DECKS)

FORM AND REINFORCEMENT DETAILS

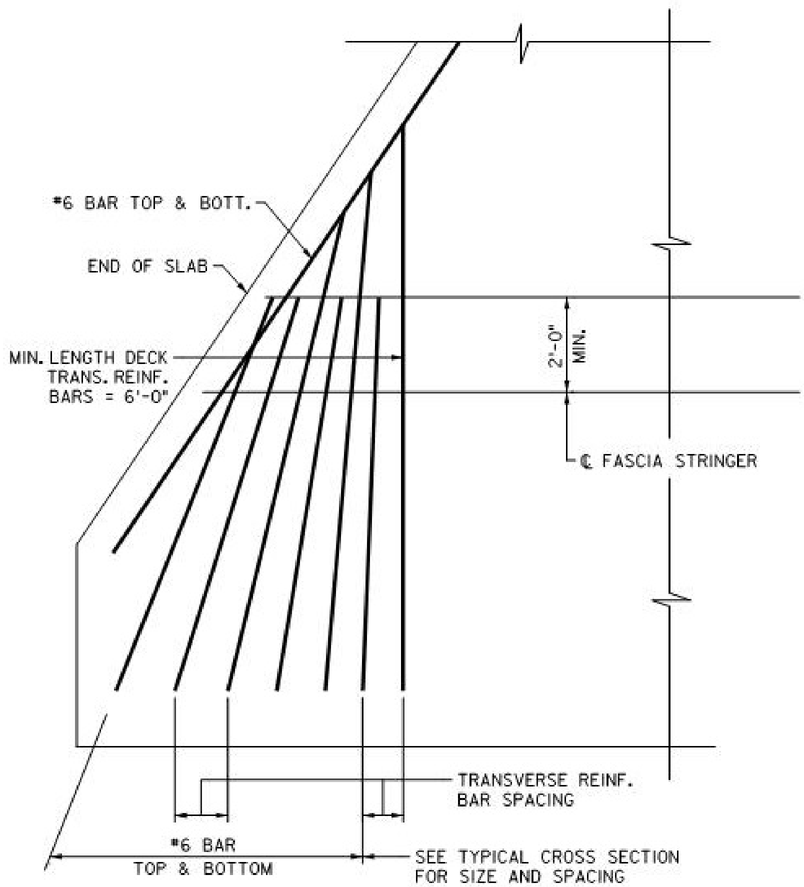
N.T.S.

NOTES:

1. SIP CORRUGATION PITCH NEED NOT MATCH REINFORCEMENT PITCH.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-302 DECK DETAILS - 3

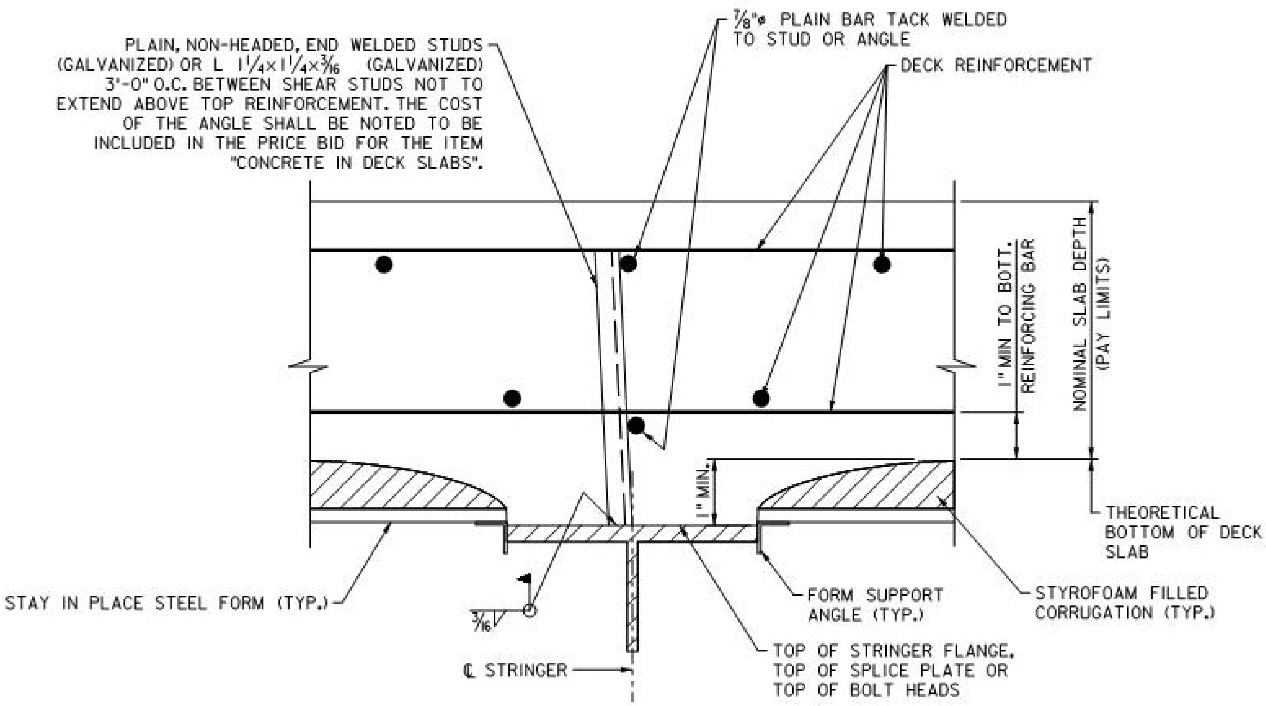


REINFORCEMENT IN CORNERS OF SKEWED SLABS

N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-303 DECK DETAILS - 4



TRANSVERSE SECTION

COMPRESSION FLANGES SHOWN
TENSION FLANGES SIMILAR (SEE BELOW)

FORM AND REINFORCEMENT DETAILS

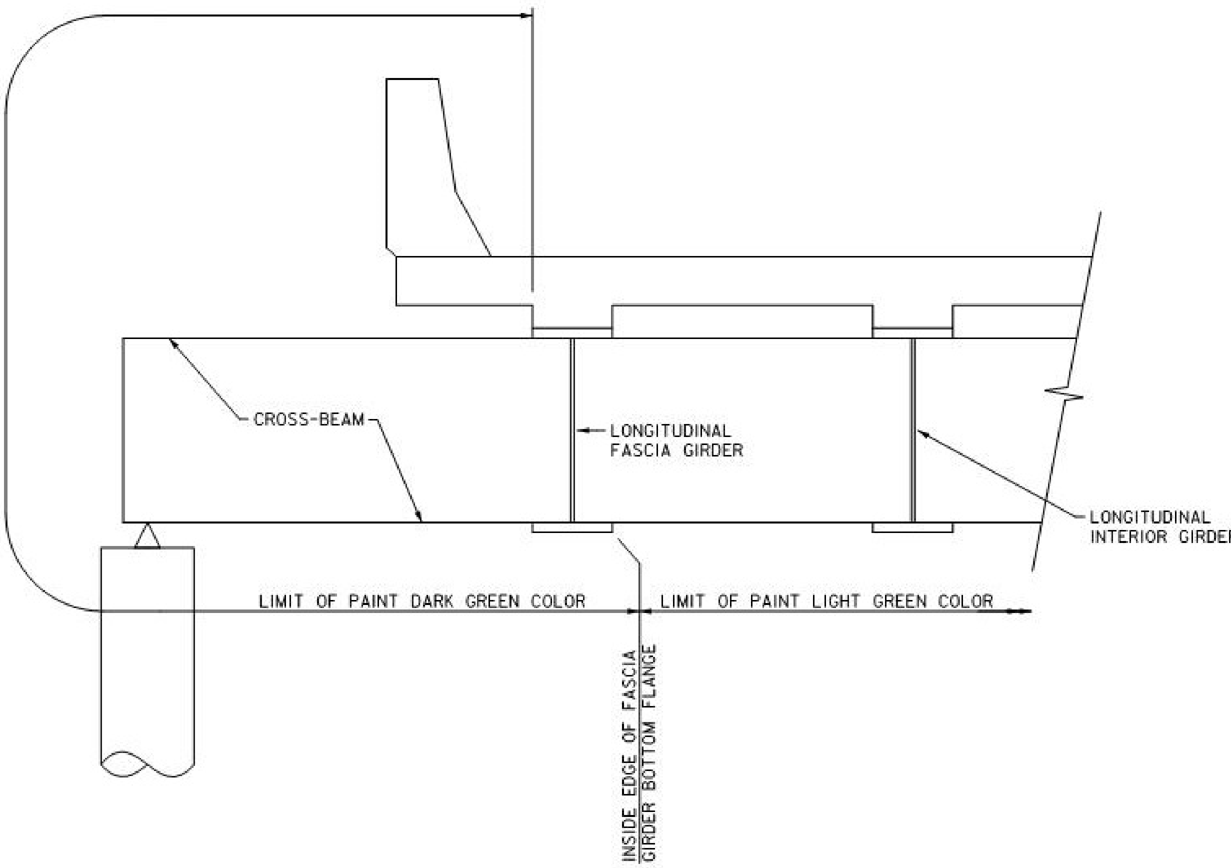
N.T.S.

NOTES:

1. FOR BEAMS WITH TENSION IN THE TOP FLANGE, REINFORCEMENT SUPPORT SHALL NOT BE WELDED DIRECTLY TO THE TOP FLANGE. CONTRACT PLANS SHALL INCLUDE NON WELDED CONNECTION DETAILS.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-304 SUPERSTRUCTURE DETAILS- 1



GARDEN STATE PARKWAY BRIDGES
PAINT COLOR SCHEMATIC
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-400 OVERHEAD SIGN STRUCTURE NOTES

OVERHEAD SIGN STRUCTURE NOTES (FOR PLACEMENT):

A. ON SPAN STRUCTURES WHERE "LANE ENDS" SIGN IS USED, "LANE END" SIGN TO BE CENTERED OVER THE LANE TO BE DROPPED, EXIT DIRECTION SIGN TO BE OVER THE DECEL. LANE OR SHIFTED RIGHT TO OBTAIN 6" MIN. BETWEEN SIGNS IF NECESSARY, THE THREE LANE DESIGNATION SIGN IS TO BE CENTERED OVER THE REMAINING LEFT LANES OR SHIFTED LEFT TO OBTAIN THE 6" MIN. BETWEEN SIGNS, IF NECESSARY.

B. ON OTHER SPAN STRUCTURES, EXIT DIRECTION SIGN TO BE CENTERED OVER RIGHT LANE AND DECEL LANE; THE THRU LANE DESIGNATION SIGN IS TO BE CENTERED OVER THE REMAINING LEFT LANES OR SHIFTED LEFT LANES TO OBTAIN THE 6" MIN. BETWEEN SIGNS, IF NECESSARY.

C. SIGN PANELS FOR STRUCTURES WITH LIGHTING ARE TO BE CENTERED VERTICALLY BETWEEN CHORDS UNLESS OTHERWISE NOTED.

D. LIGHTING FIXTURE SPACING DIMENSIONS "A" AND "B" SHALL BE AS FOLLOWS:

DIM. "A" - 6 FEET NOMINAL, 5 FEET MIN., 6'-6" MAX..
DIM. "B" - $\frac{59}{64}$ (W-XA) WHERE "X" IS ONE LESS THAN THE NUMBER OF FIXTURES AND "W" IS THE WIDTH OF SIGN PANEL.

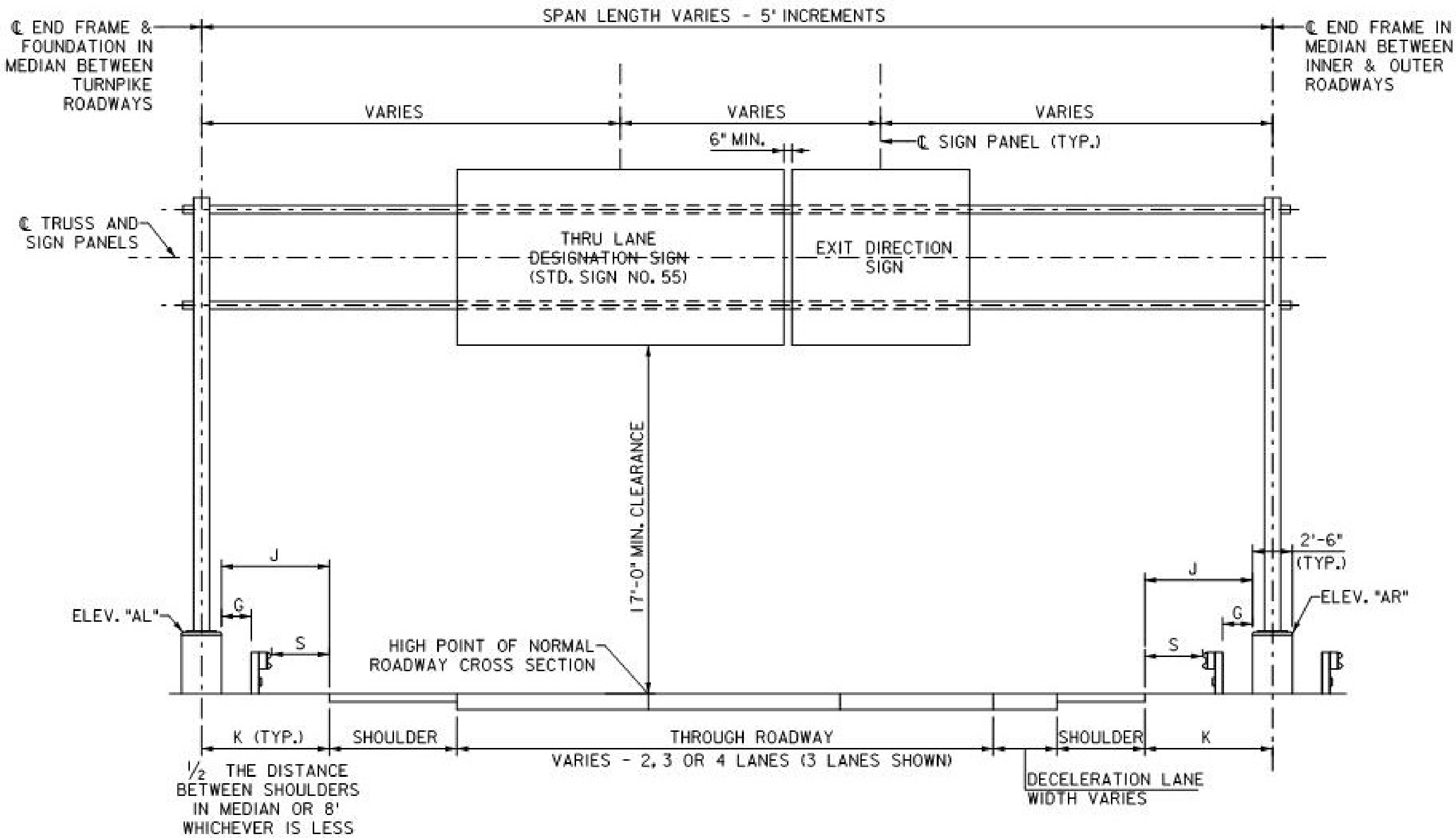
E. ELEV. "A" IS TO BE 4'-0" MIN. ABOVE THE HIGH POINT OF THE TURNPIKE/PARKWAY ROADWAYS, ELEV. "AR" AND "AL" ARE TO BE THE SAME.

F. SIGN ELEVATIONS WILL ALWAYS BE TAKEN IN THE DIRECTION OF THE DRIVER. RIGHT AND LEFT END FRAMES WILL BE REFERRED TO IN THIS MANNER.

G. NORMAL DIMENSIONS FROM FACE OF GUIDE RAIL TO BACK OF POST IS 1'-3".

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-401 TURNPIKE OVERHEAD SIGN STRUCTURES - 1



	WHEN K=4'-6"	WHEN K=8'-0"
G	1'-10"	3'-4"
S	0"	2'-0"
J	3'-3"	6'-9"

WHEN K IS GREATER THAN 8'-0", GUIDE RAIL CRITERIA TO GOVERN.
WHEN K IS LESS THAN 4'-6" CONCRETE BARRIER PROTECTION SHALL BE USED.

SPAN SIGN STRUCTURE

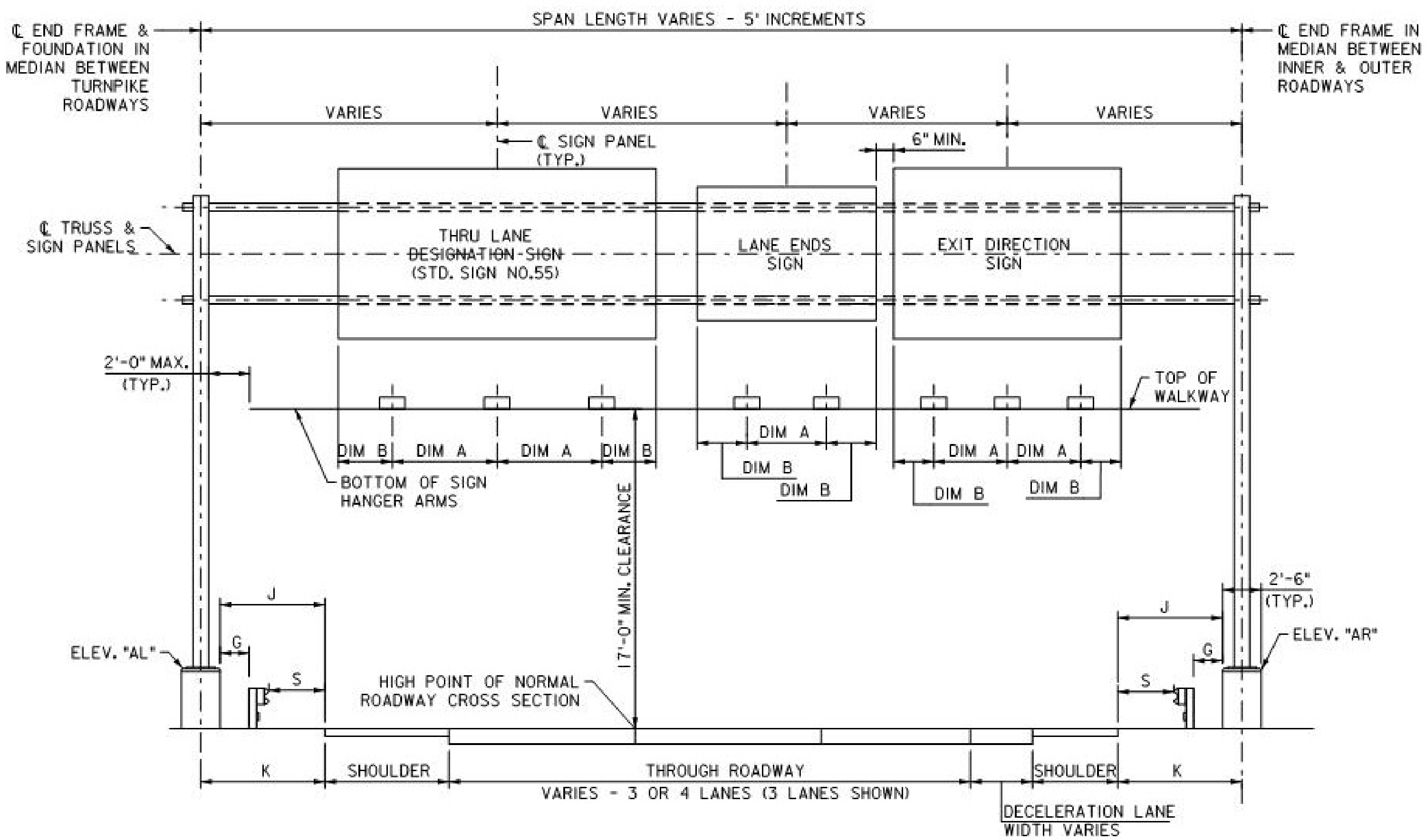
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-402 TURNPIKE OVERHEAD SIGN STRUCTURES - 2



SPAN SIGN STRUCTURE

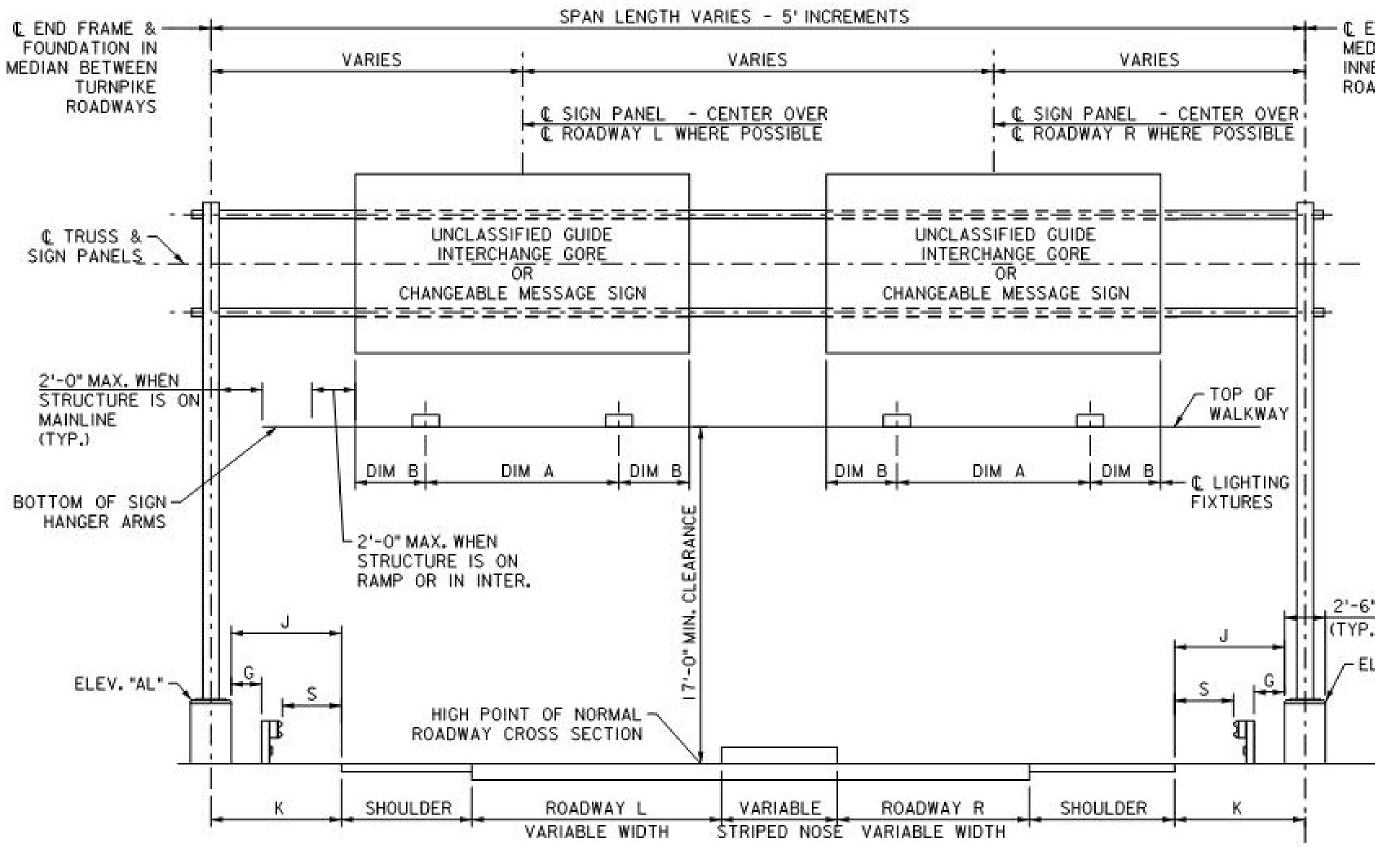
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.
2. SEE EXHIBIT 2-401 FOR ADDITIONAL TABLE.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-403 TURNPIKE OVERHEAD SIGN STRUCTURES - 3



SPAN STRUCTURE - VARIABLE WIDTH ROADWAY

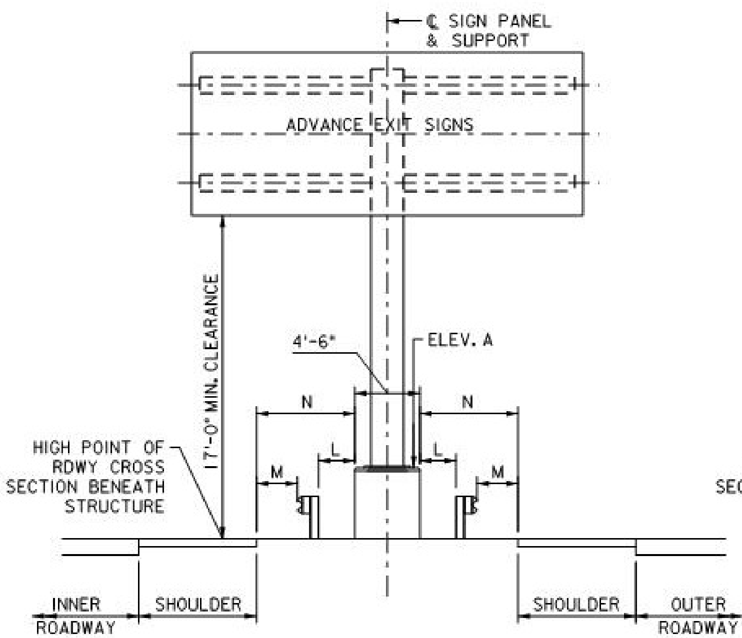
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.
2. SEE EXHIBIT 2-401 FOR ADDITIONAL TABLE.

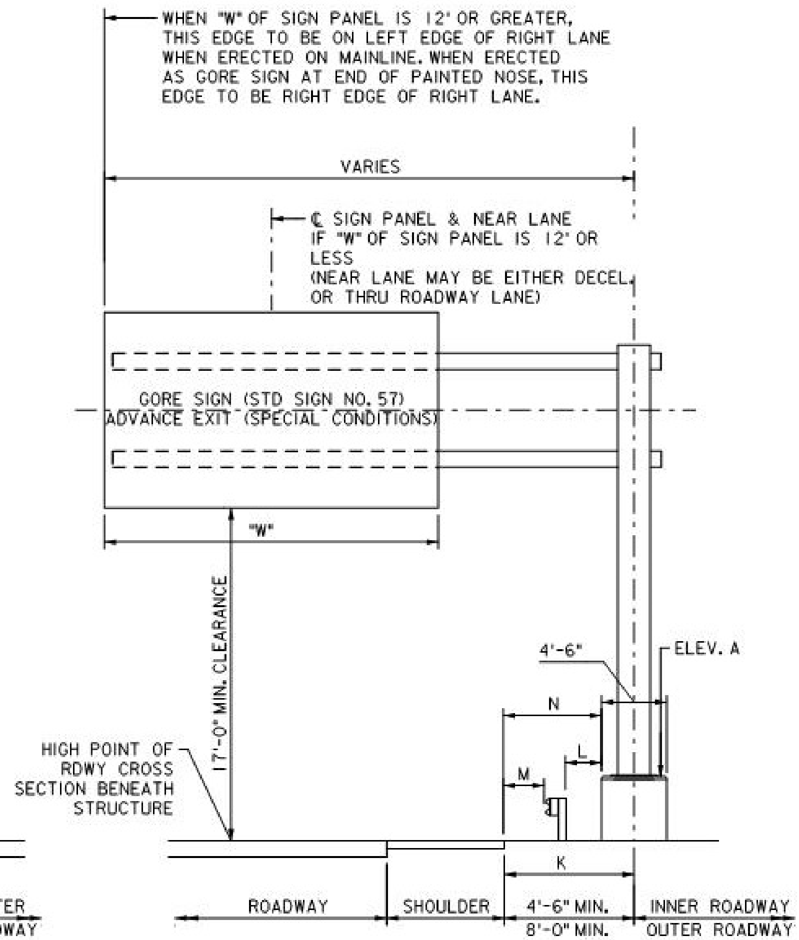
NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-404 TURNPIKE OVERHEAD SIGN STRUCTURES - 4



BUTTERFLY STRUCTURE

N.T.S.



CANTILEVER STRUCTURE

N.T.S.

	WHEN K=4'-6"	WHEN K=8'-0"
L	10"	2'-4"
M	0"	2'-0"
N	2'-3"	5'-9"

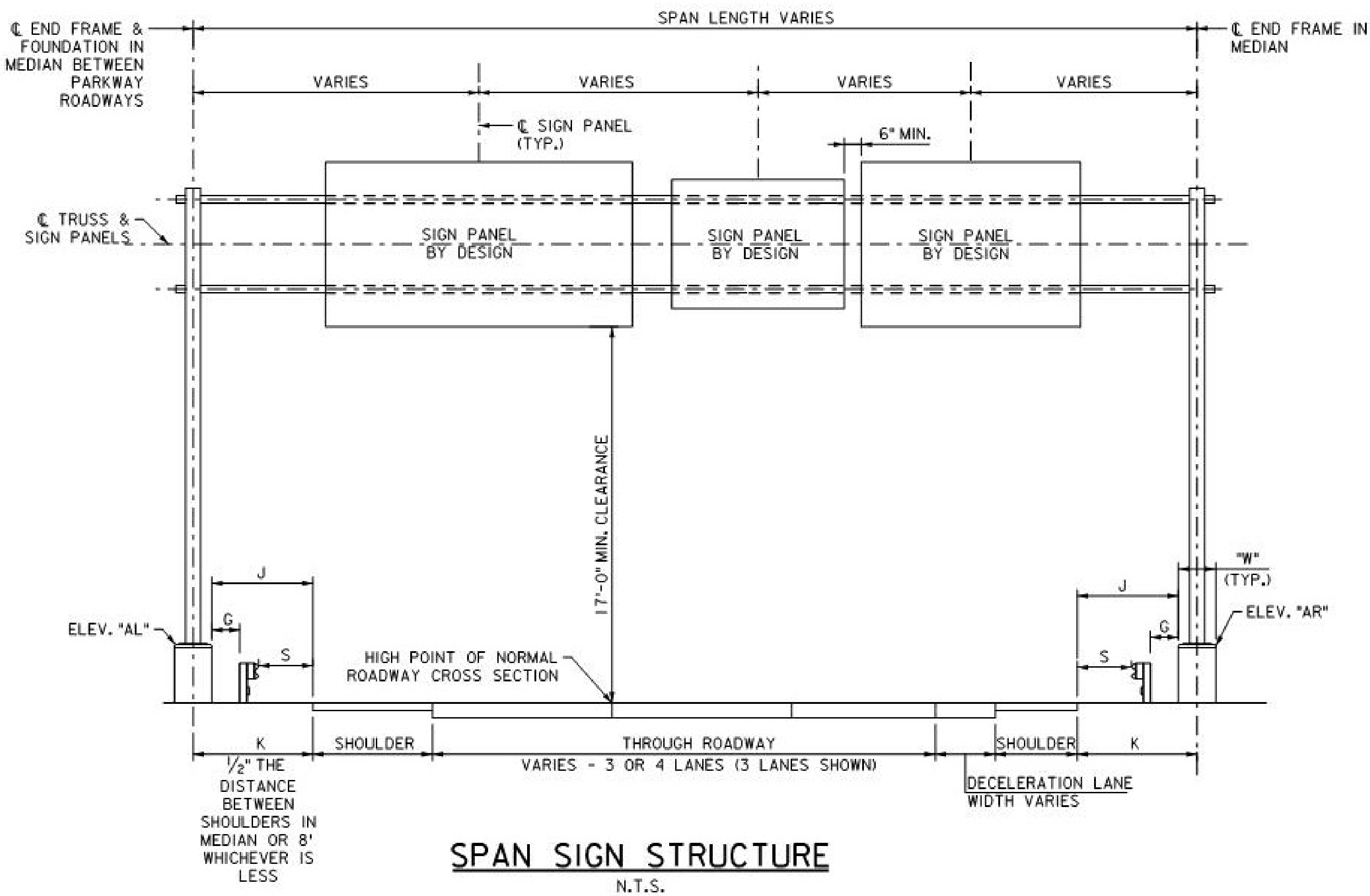
WHEN K IS GREATER THAN 8'-0", GUIDE RAIL CRITERIA TO GOVERN.
WHEN K IS LESS THAN 4'-6" CONCRETE BARRIER PROTECTION SHALL BE USED.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.

NEW JERSEY TURNPIKE AUTHORITY
DESIGN MANUAL

EXHIBIT 2-405 PARKWAY OVERHEAD SIGN STRUCTURES - 1



SPAN SIGN STRUCTURE

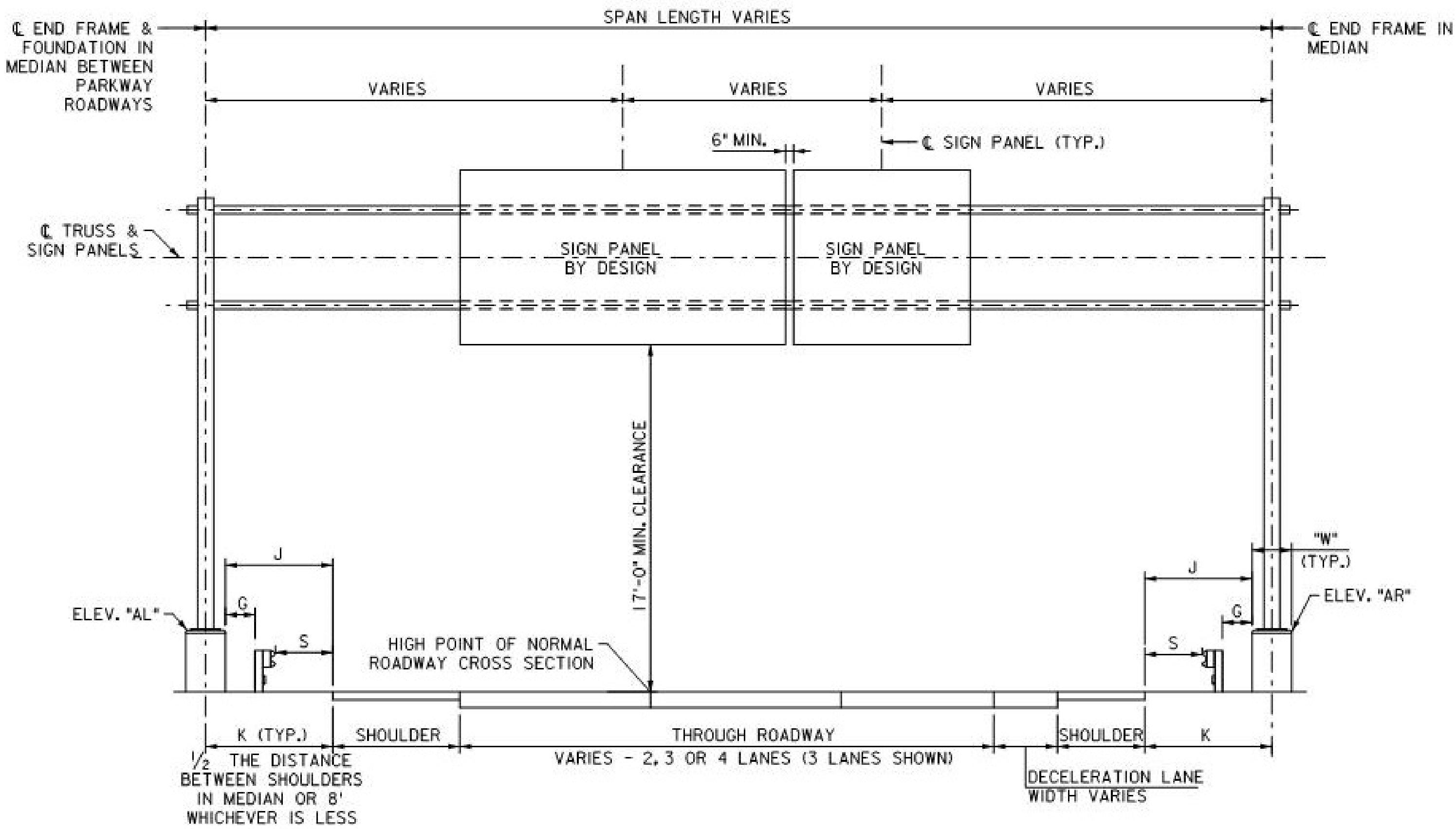
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.
2. SEE EXHIBIT 2-406 FOR ADDITIONAL TABLE.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-406 PARKWAY OVERHEAD SIGN STRUCTURES - 2



	W=3'-6"	
	WHEN K=4'-6"	WHEN K=8'-0"
G	2"	3'-0"
S	1'-10"	2'-0"
J	3'-3"	6'-9"

WHEN K IS GREATER THAN 8'-0", GUIDE RAIL CRITERIA TO GOVERN.
WHEN K IS LESS THAN 4'-6" CONCRETE BARRIER PROTECTION SHALL BE USED.

SPAN SIGN STRUCTURE

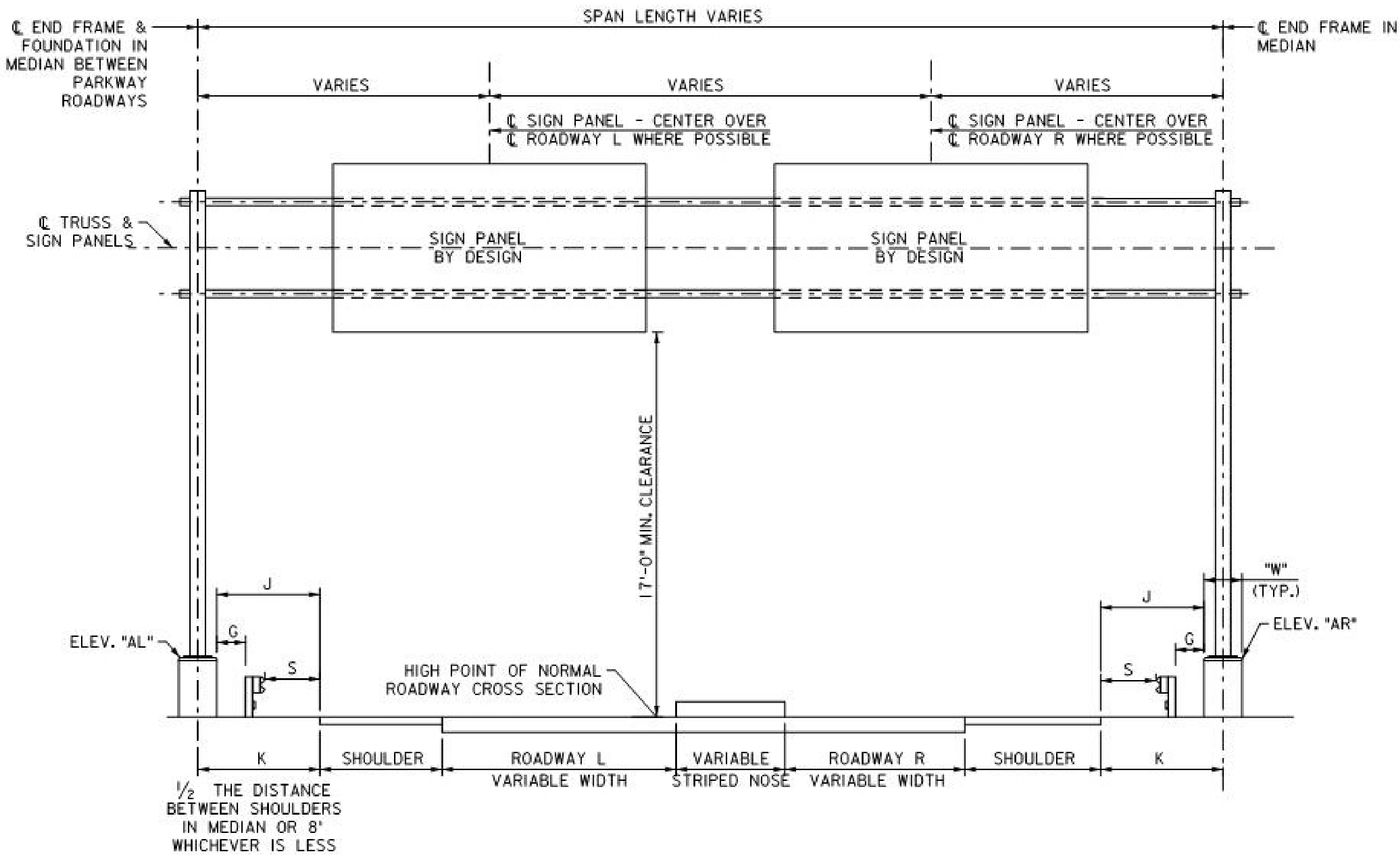
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-407 PARKWAY OVERHEAD SIGN STRUCTURES - 3



SPAN SIGN STRUCTURE - VARIABLE WIDTH ROADWAY

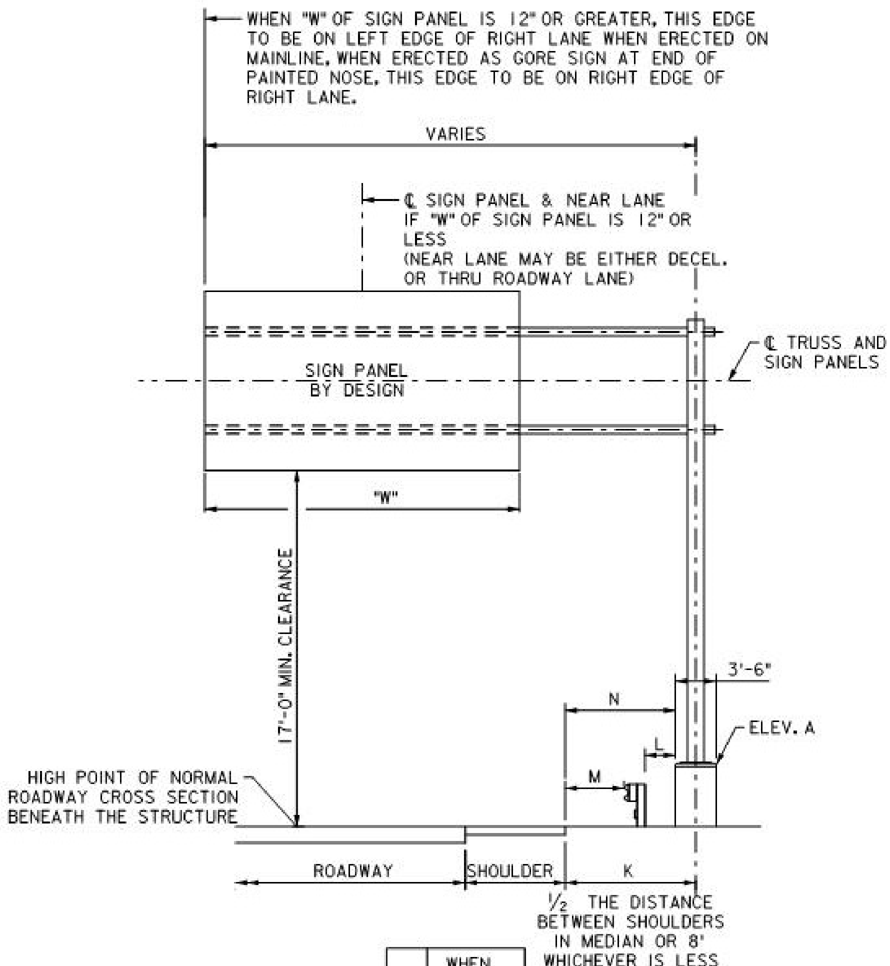
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.
2. SEE EXHIBIT 2-406 FOR ADDITIONAL TABLE.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-408 PARKWAY OVERHEAD SIGN STRUCTURE - 4



	WHEN K=8'-0"
L	3'-0"
M	2'-0"
N	6'-3"

WHEN K IS GREATER THAN 8'-0", GUIDE RAIL CRITERIA TO GOVERN.
WHEN K IS LESS THAN 4'-6" CONCRETE BARRIER PROTECTION SHALL BE USED.

CANTILEVER STRUCTURE

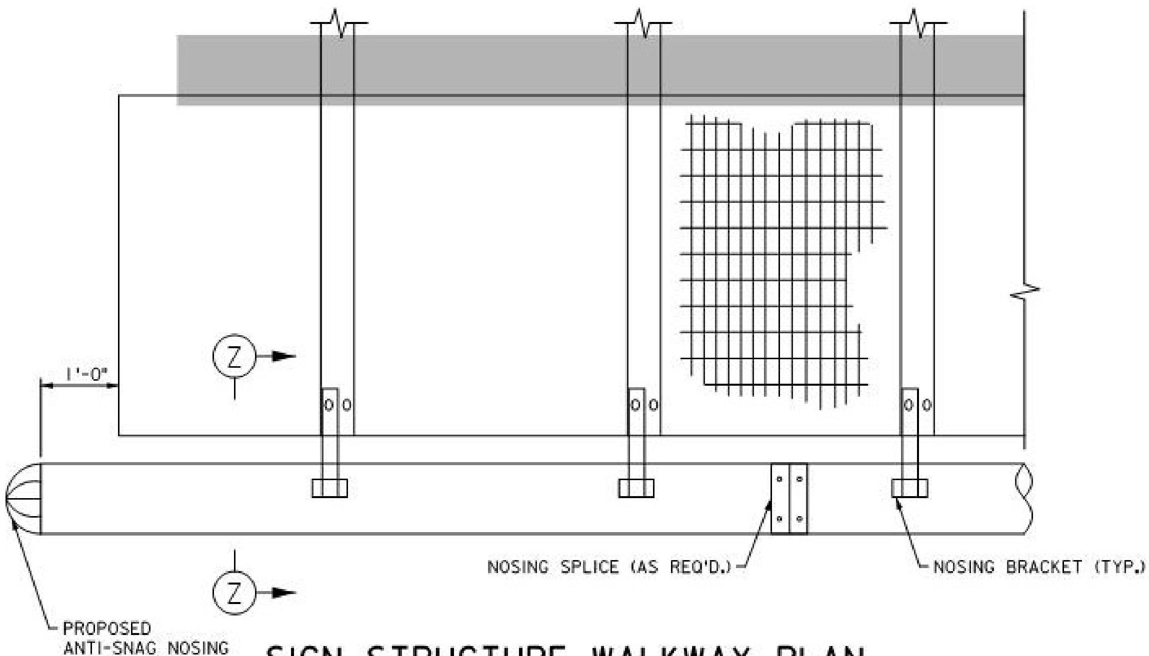
N.T.S.

NOTES:

1. SEE EXHIBIT 2-400 FOR OVERHEAD SIGN STRUCTURE NOTES.

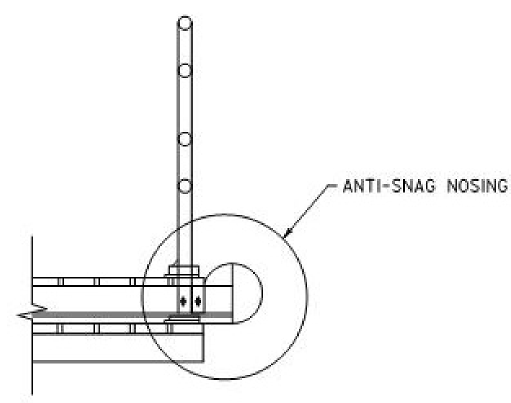
NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-409 ANTI-SNAG NOSING DETAILS - 1



SIGN STRUCTURE WALKWAY PLAN

N.T.S.



SECTION Z-Z

N.T.S.

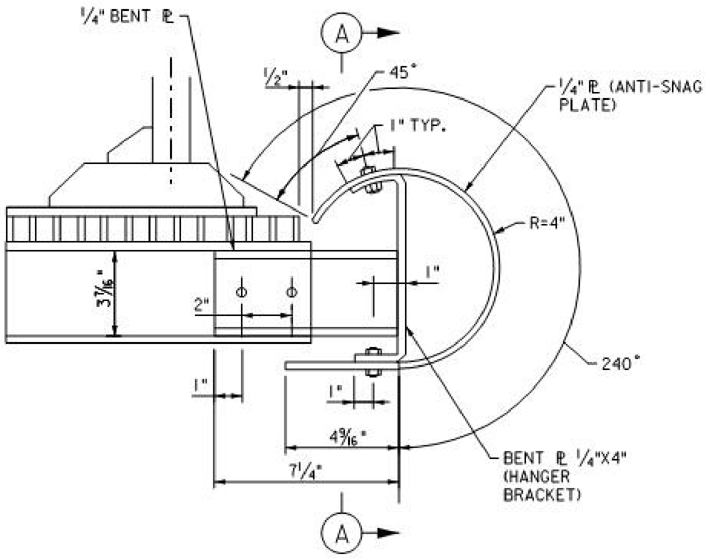
NOTES:

1. ALL SHAPES, PLATES, BRACKETS, HARDWARE AND FABRICATIONS SHOWN FOR THE ANTI-SNAG NOSING SHALL BE MADE FROM 6061-T6 ALUMINUM UNLESS OTHERWISE NOTED.

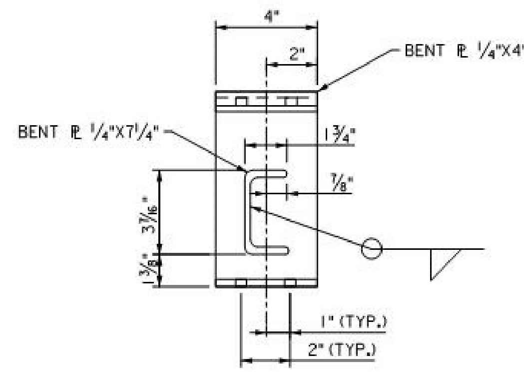
2. ANTI-SNAG ATTACHMENT HARDWARE SHALL BE 18-8 STAINLESS STEEL, BOLTS SHALL BE BUTTON-HEAD CAP SCREWS, McMASTER-CARR PART NO. 92949A587 OR APPROVED EQUAL. LOCKING NUTS, SHALL BE LOW-PROFILE TYPE WITH NYLON INSERTS, McMASTER-CARR PART NO. 90101A237 OR APPROVED EQUAL.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

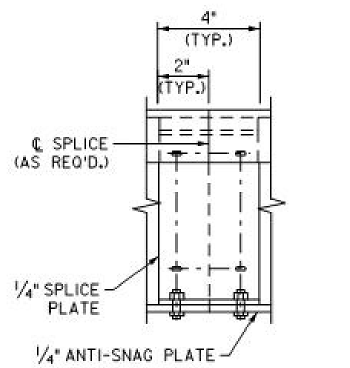
EXHIBIT 2-410 ANTI-SNAG NOSING DETAILS - 2



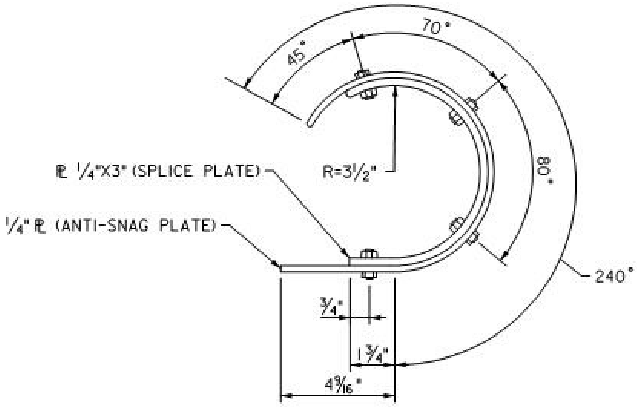
ANTI-SNAG DETAIL SECTION AT HANGER BRACKET
N.T.S.



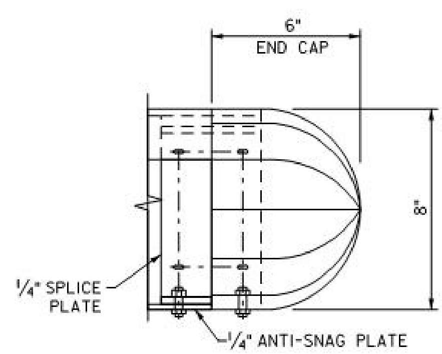
SECTION A-A
(HANGER BRACKET)
ANTI-SNAG PLATE NOT SHOWN FOR CLARITY
N.T.S.



REAR ELEVATION AT SPLICE PLATE
N.T.S.



SECTION AT SPLICE PLATE
(SPLICE PLATE)
N.T.S.

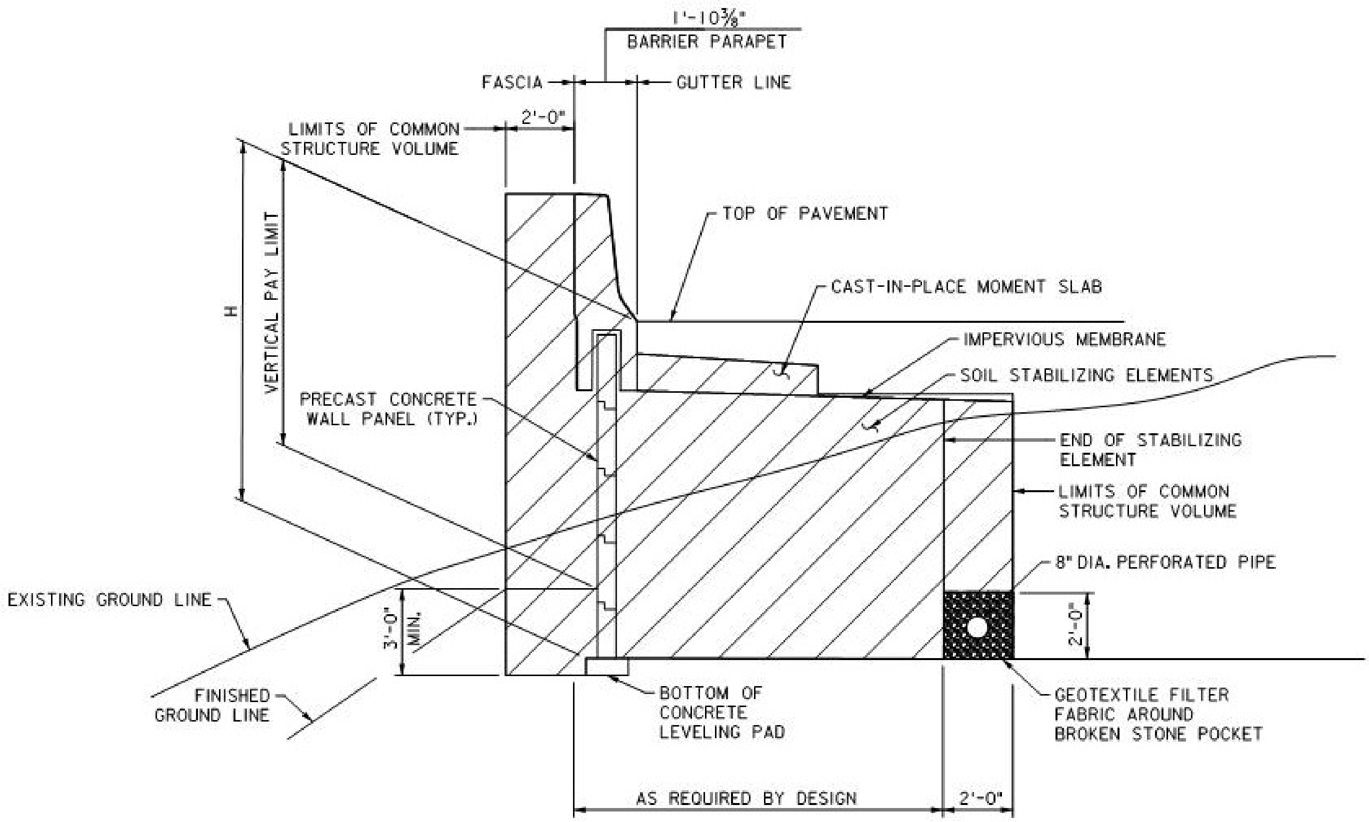


REAR ELEVATION AT END CAP
(TYPICAL)
N.T.S.

NOTES:
1. FOR NOTES, SEE EXHIBIT 2-409.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-500 PROPRIETARY RETAINING WALL DETAILS - 1

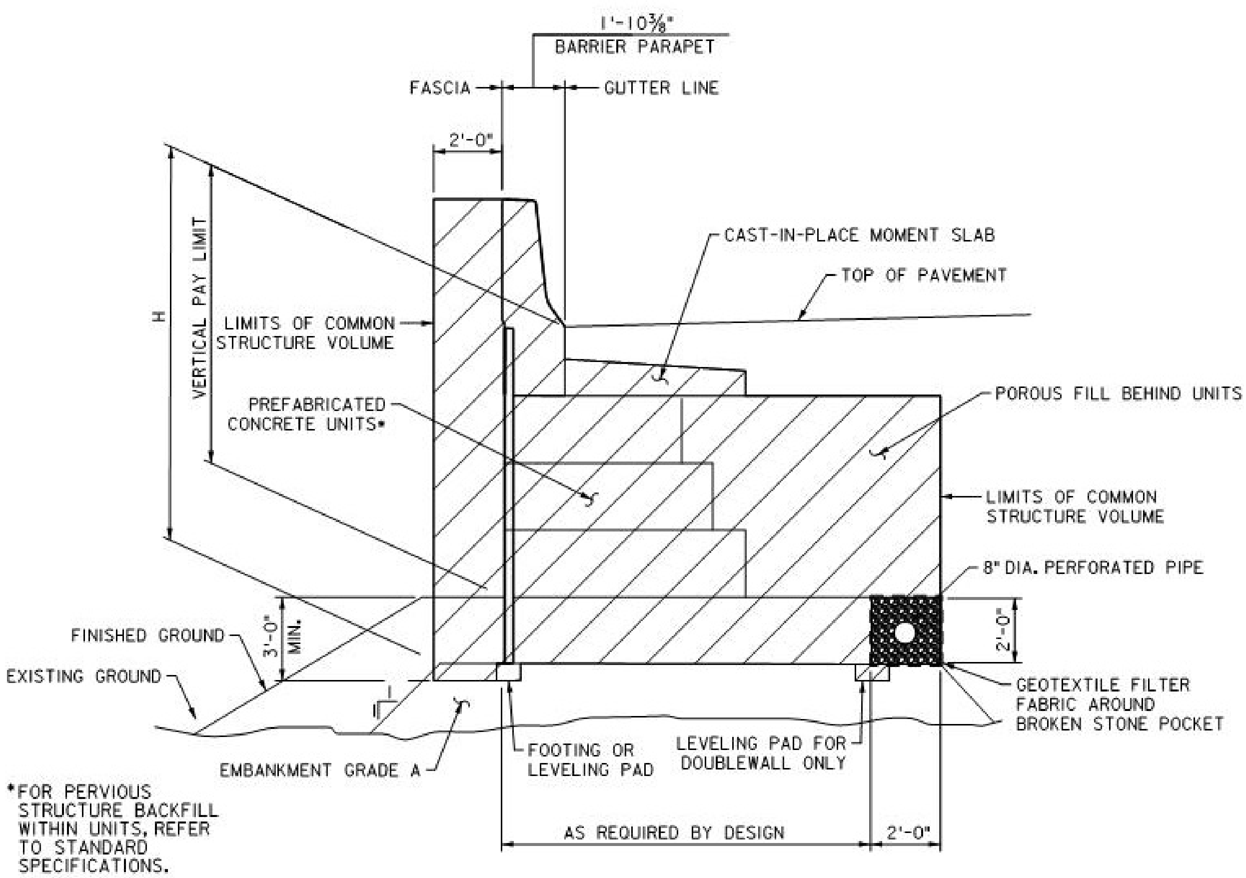


PROPRIETARY WALL SECTION

(MSE SHOWN)
(CUT SHOWN)
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-501 PROPRIETARY RETAINING WALL DETAILS - 2

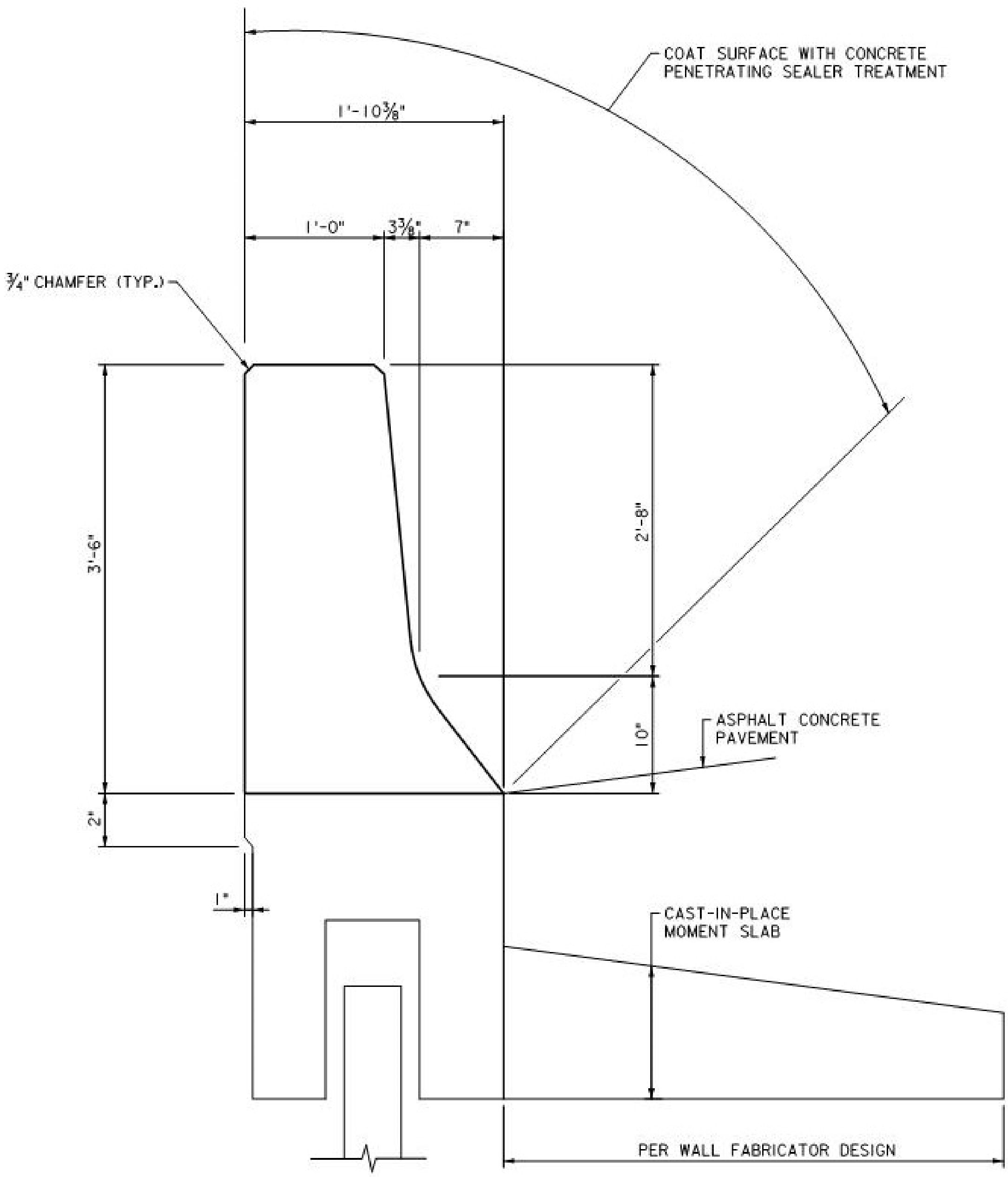


PROPRIETARY WALL SECTION

(PREFABRICATED MODULAR WALL SHOWN)
(FILL SHOWN)
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-502 PROPRIETARY RETAINING WALL DETAILS - 3

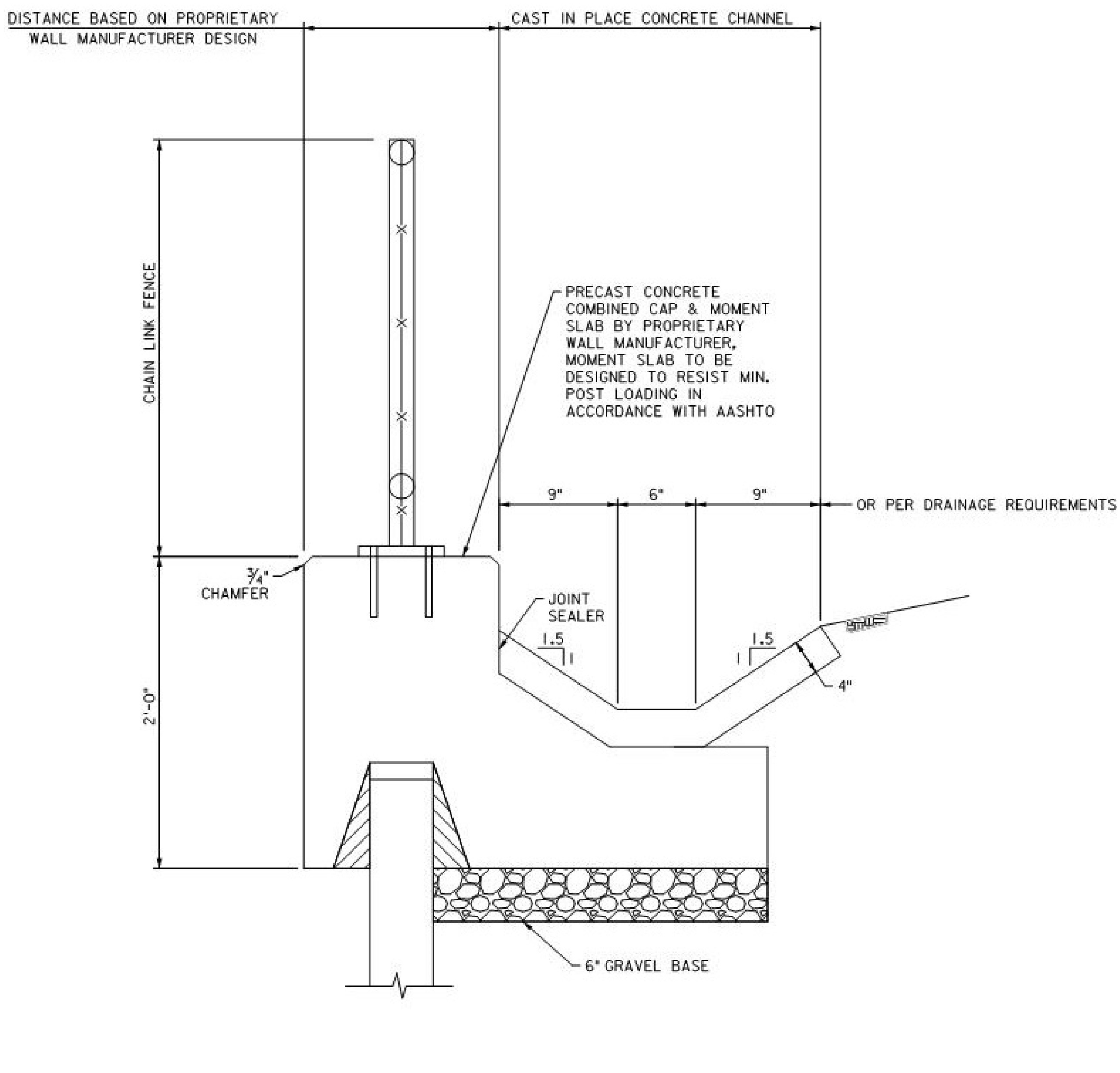


PRECAST CONCRETE BARRIER SECTION

(FOR PROPRIETARY WALLS)
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-503 PROPRIETARY RETAINING WALL DETAILS - 4

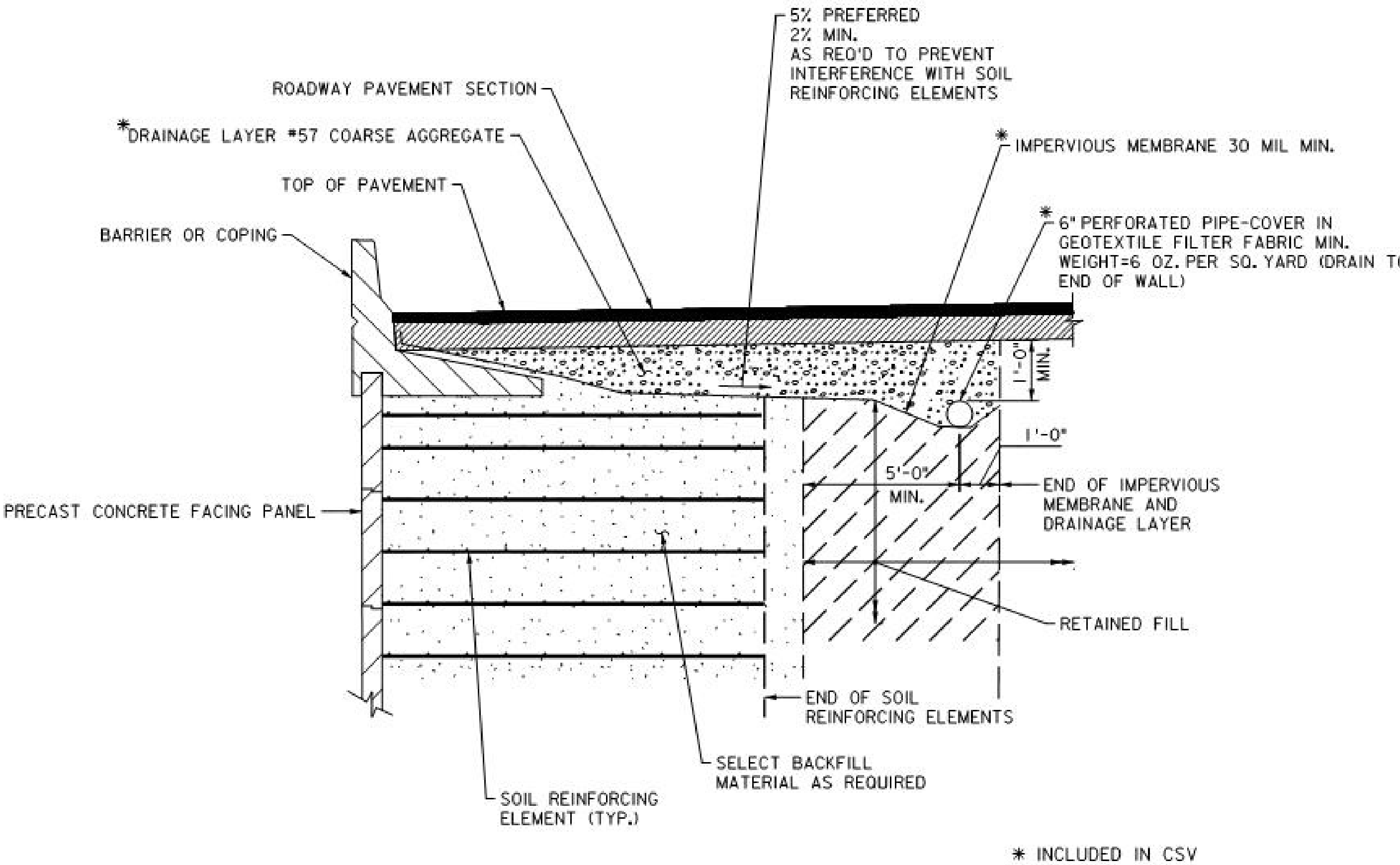


FENCE CAP AND MOMENT SLAB DETAIL FOR PROPRIETARY WALLS

N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

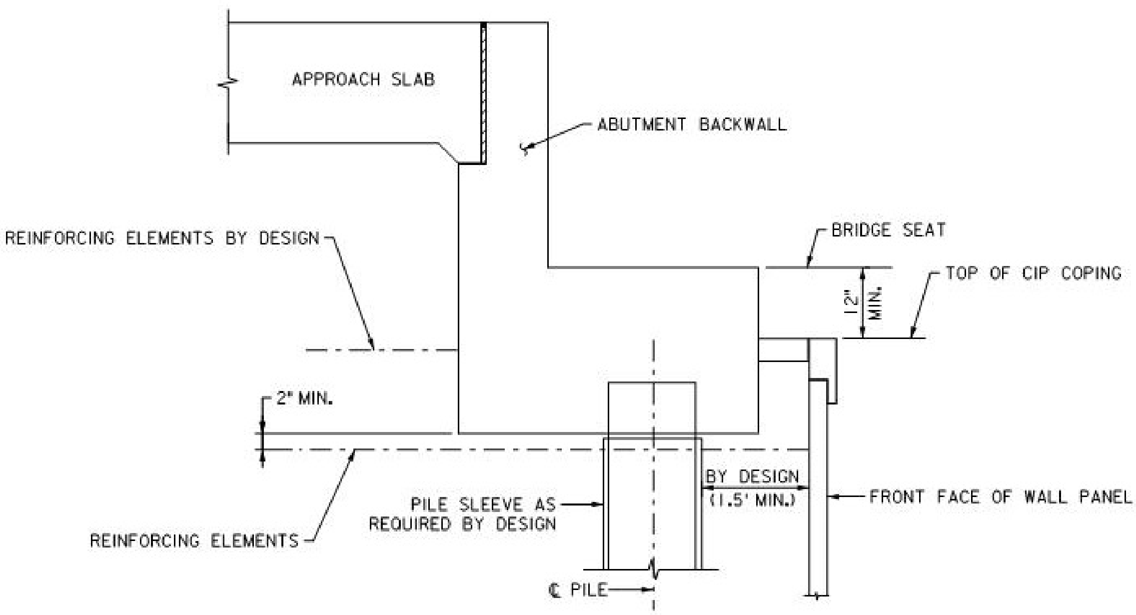
EXHIBIT 2-504 PROPRIETARY RETAINING WALL DETAILS - 5



IMPERVIOUS MEMBRANE DETAIL
(MSE WALLS)
N.T.S.

NEW JERSEY TURNPIKE AUTHORITY DESIGN MANUAL

EXHIBIT 2-505 PROPRIETARY RETAINING WALL DETAILS - 6



MSE ABUTMENT
N.T.S.

Appendix A -Bridge Design Checklist