Section 24 - Structural Steel

24.1 Design

1. Section 6 of the *AASHTO LRFD Bridge Design Specifications for Highway Bridges* (with current interims), with modifications specified under Section 3 of this Manual, shall govern the design of structural steel members. This will also include the design of curved girder structures.
2. Acceptable Structural Steel superstructure types for New Jersey bridge structures may include rolled beams, welded plate I-girders and steel tubs or boxes. Use of truss and through girder systems are subject to the approval by the Manager, Structural Engineering.
3. The AASHTO/NSBA Steel Bridge Collaboration has published documents that provide guidance on designing for constructability, fabrication detailing as well as other structural steel usage aids. These documents may be referenced and used in preparing contract documents.

   Most notable is the document titled "*Guidelines for Design for Constructability*". Criteria are provided to address designing for member sizes, deflections, stiffener requirements and steel box fabrication.

24.2 Type of Steel

1. Structural steel grades shall conform to the AASHTO M 270 (ASTM A 709), Grades. Table 6.4.1-1 of the *AASHTO LRFD Bridge Design Specifications* provides a listing of designated Grades.

   The use of Grades 36, 50, 50W and HPS designated Grades are permitted. The HPS prefix designates High Performance Steel Grades.

   Grade 50W and HPS Grades are weathering steel grades. Subsection 24.19 of this Manual may be referred to for guidance on the use of weathering steel.

   The use of High Performance Steel (HPS) is strongly encouraged. Economic studies indicate that furnishing of hybrid girders is generally the most economical choice when using HPS for continuous spans. Therefore, girder webs and positive moment top flanges consisting of Grade 50W with negative moment top flanges and all bottom flanges made up of HPS 70W should initially be considered as the ideal girder section.

   The grade or grades of steel to be furnished shall be designated on the plans.
2. All structural steel that is to be used in main load-carrying member components that are subject to tensile stresses shall conform to the applicable Charpy V-notch Impact Test requirements of AASHTO M 270 (ASTM A 709).

   Welded girders made of High Performance Steel (HPS), steels shall be fabricated in accordance with the AASHTO "*Guide for Highway Bridge Fabrication with HPS70W Steel*" which supplements the ANSI/AASHTO/AWS D1.5 Bridge Welding Code.
3. The use of Grades 50W and HPS Grades, given to be weathering steel grades, are subject to the cleaning and painting requirements that are specified in the NJDOT Standard Specifications.
4. All structural steel plans shall have the following note shown thereon:

Structural Steel: AASHTO M 270 Grade ___ (ASTM A 709 Grade___) with Supplementary Requirements for Notch Toughness for all member components marked (T).

5. It shall be the responsibility of the Structural Design Engineer to designate the main load carrying member components that are subject to tensile stress. For this purpose, the designation (T) shall be noted on the contract plans.

   The components to be designated (T) shall include flanges, webs, and splice plates of the welded stringers, girders, or rolled beams (also see Guide Sheet Plate 3.9-21.) The above note and designations shall be verified on the shop drawing plans.

24.3 Span Type Selection

1. Simple and continuous stringers are within the range of span types that can be considered for the majority of structures. The choice should be made on the basis of judgment, economy, appearance and serviceability.

2. Bridges shall be designed to satisfy minimum levels of superstructure redundancy such that the failure of one member would not lead to the collapse of the bridge. Subsection 1.3.4 – Redundancy of the *AASHTO LRFD Bridge Specifications* specifies that multiple load path or continuous structures should be developed.

   Accordingly, redundant type (multiple load path) systems shall always be used. To satisfy this requirement, bridge structures in New Jersey shall be designed and constructed with a minimum of four (4) girder lines.

   Should the need for non-redundant (single load path) systems be unavoidable, their use shall be subject to approval, in writing, by the Manager, Bureau of Structural Engineering. A request for an approval shall include substantial justification why a non-redundant structure is the only structure that will meet the project requirements.

   The approval shall be obtained prior to the Preliminary Plan submission and before beginning final design development. Such approval will be subject to the special design, fabrication, and plant inspection provisions of the AASHTO/AWS Section 12 "*Fracture Control Plan for Non-Redundant Members*" (see Subsection 1.24.5).

3. Structures containing pin and hanger connections for suspended/cantilever spans are not permitted. If somehow deemed necessary, suspended/cantilever span design shall be subject to approval, in writing, by the Manager, Bureau of Structural Engineering prior to the Preliminary plan submission.

   Pin and hanger connections may only be utilized on redundant (multiple load path) systems. Members shall be restrained against lateral movement on the pins and against lateral distortion due to bridge skew or curvature. Pin and hanger connections shall be designed in accordance with the *AASHTO LRFD Bridge Design Specifications*.

24.4 Economics of Stringer Design

1. In the design of welded plate girders, consideration should be given to minimizing the number of transverse intermediate stiffeners.

   This guidance is based on principal for the following reasons:

   a. Welding to the parent metal in itself introduces a discontinuity and should be avoided as much as possible.
   b. Elimination of projections and obstructions and the resulting flat surfaces optimize the chances of improved quality of workmanship in the cleaning and painting of the structural steel both in the fabricating shop, initial field coating and future maintenance painting.
   c. Fabrication cost differentials between welding stiffeners versus use of additional material in the main components of girders; such as, additional web thickness, are not overwhelmingly significant. This should be considered during design.
2. Consideration shall also be given to minimizing the number of butt welded flange plate transitions. Plate size transitions may be located at the field splice so that butt welding requirements are either reduced or eliminated. It is the Designer's responsibility to check the availability of plate sizes in order to determine the location of shop splices for flange plates.
3. Reduction of material mass is not necessarily the ultimate factor in determining span type selection. Material mass of the stringers may represent about 25% of the completed, in-place cost. The bulk of the cost is in fabrication, delivery and erection.
4. Guide Plates contained in Appendix 3 of this Manual, for Structural Steel fabrication, are based on economical fabrication detailing. These details should be studied in project development.
5. Due to the limited availability of rolled beam sections, their use should be carefully studied before their selection as bridge superstructure members.

24.5 Fracture Critical Members

1. Steel bridge members or member components designated as Fracture Critical Members (FCM's) shall conform to the provisions of the most current edition of the AASHTO/AWS D1.5 Bridge Welding Code.
2. Fracture critical members or member components (FCM's) are tension members or tension components of members whose failure would be expected to result in collapse of the bridge.
3. The responsibility for determining which, if any, bridge member or member component is in the FCM category shall rest with the Structural Design Engineer.
4. If it is determined that any member or member component is in the FCM category, the following note shall be shown on the structural steel plans:

   Fracture Critical Members: Members or member components designated as FCM shall be subject to the provisions of the most current Edition of the AASHTO/AWS D1.5 Bridge Welding Code, Section 12.

   Working drawings shall be reviewed by the Structural Design Engineer accordingly.
5. When planning for the rehabilitation or reconstruction of bridges that include the presence of FCM’s, an analysis to verify the capacity of gusset and splice plates shall be made. The analysis shall also consider the potential application of additional dead load or live load, which could result in increased stresses to any member; such as, construction loadings or other temporary conditions. This is

referenced in NJDOT Technical Advisory at http://www.state.nj.us/transportation/eng/structeval/pdf/ADVISORY-GussetPlateEvaluations.pdf and http://www.state.nj.us/transportation/eng/structeval/pdf/ADVISORY-ConstructionLoadonBridges.pdf

24.6 Composite Design

1. Steel stringers with a concrete deck slab shall normally be designed as composite structures, assuming no temporary supports will be provided for the beams or girders during placement of the permanent dead load.
2. Shear connectors shall be 7/8 inch diameter end welded studs. Height of studs depends on concrete haunch dimensions. Shear connectors shall penetrate at least 2 inches into the bottom mat of the deck slab, but the top of the stud head shall be 3 inches minimum below the top of the deck slab. Use of the same height stud on any one bridge is preferred.
3. See Section 3.2 of this Manual (AASHTO Section 6.10.1) for criteria concerning the negative moment area of continuous spans.

24.7 Camber

1. Simple Spans. The various conditions of dead load deflection and camber for each simple span stringer shall be tabulated on the structural steel plans as shown below:

Table 24.1 Dead Load Deflection and Camber for Single Span

Dead Load Deflections (Inches)							Camber (Inches)			
Stringer Number	Location	Structural Steel	Concrete Slab (Including Haunches)	Stay In Place Form And Added Concrete Thickness	Sidewalks Parapets Barriers	Future Paving Allow-ance	Total Dead Load Camber	Vertical Curve Ordinate	Architectural Camber	Total Camber Required
	Mid-Span									
	1/4 Point									

The column headed "Vertical Curve Ordinate" shall be used exclusively for simple span stringers located within the limits of a crest vertical curve, provision for its ordinates must be made within the concrete haunch. Consequently, the tabulation of its ordinates is unnecessary.

Total dead load camber is equal to the sum of the dead load deflections. An architectural camber of L/100 inches, where L is the span length in feet, shall be provided for all simple span stringers unless the vertical curve ordinate meets this, in which case the architectural camber may be omitted. When establishing the depth of the concrete slab and haunch in composite design, the following items shall be considered:

- Total camber required.
- Girder dimensional tolerances per Section 3.5 of the *ANSI/AASHTO/AWS Bridge Welding Code* D1.5.
- A minimum cover of 3 inches over the shear connectors.

When total camber is less than minimum that can be maintained in a beam (W Section) no camber is required but a note stating "Beams shall be placed with any mill camber up" shall be shown on the drawings.

2. Continuous and Cantilevered Spans. The various conditions of dead load deflections and cambers for each stringer shall be tabulated (in the following Camber Table Form) at the tenth point of spans and at the field splice points (at dead load points of contraflexure if field splices are not provided).

   Complete the Camber Table (Table 24.2) and use the example in Figure 24.1 to create the Camber Diagram.

Table 24.2 Camber Table for Continuous and Cantilevered Spans

CAMBER TABLE																																				
		Centerline Brgs. Abut.	SPAN 1										Centerline Brgs. Pier 1	SPAN 2											Centerline Brgs. Pier 2	SPAN 3										Centerline Brgs. Abut.
POINT NUMBER			1	2	3	4	5	6	7	IP_1	8	9	10	11	12	IP_2	13	14	15	16	17	IP_3	18	19	20	21	22	IP_4	23	24	25	26	27	28	29	
CAMBER	Steel	0											0												0											0
	Concrete Slab	0											0												0											0
	SIP Forms and Addl. Concrete Thickness	0											0												0											0
	S.D.L.	0											0												0											0
	V.C.	0																																		
	Architec-tural	0											0												0											0
	TOTAL	0	C1=	C2=	C3=	C4=	C5=	C6=	C7=	C=	C8=	C9=	C10=	C11=	C12=	C=	C13=	C14=	C15=	C16=	C17=	C=	C18=	C19=	C20=	C21=	C22=	C=	C23=	C24=	C25=	C26=	C27=	C28=	C29=	

Camber Table Notes

- The total camber as tabulated is assumed to be measured vertically to the top of the fully cambered web from a straight line drawn from the intersection of top of web and centerline of bearing at one end of the girder to the intersection of top of web and centerline of bearing at the other end of the girder.
- The camber labeled "Steel" in the table is the camber required in the girder to offset the deflection due to the dead load of the steel in the girder plus all necessary diaphragms, cross frames, etc.
- The camber labeled "Conc. Slab" in the table is the camber required in the girder to offset the deflection due to the dead load of the concrete slab.
- The camber labeled "SDL" in the table is the camber required in the girder to offset the deflection due to the superimposed dead load, that is, the curb, sidewalk, railing and future wearing surface.
- The camber labeled "Stay-in-Place forms and added concrete thickness" is the camber required in the girder to offset the deflection due to the weight of the stay-in-place forms and due to the weight of added concrete that is needed to meet the deck grades.
- The camber labeled "VC" in the table is the camber required in the girder to follow the vertical curve. The Vertical Curve value shall be used exclusively for stringers located within the limits of a crest vertical curve. Where such stringers are located within the limits of a sag vertical curve, provision for its value must be made within the concrete haunch. Consequently, the tabulation of its values is unnecessary.
- The camber labeled "Architectural Camber" shall be a value of L/100 inches, where "L" is the span length in feet. If the vertical curve value provides this camber value, the architectural camber may be omitted.
- Cambers listed in the table as positive are upward cambers.
- Cambers listed in the tables as negative are downward cambers.
- The cambers are tabulated in inches.

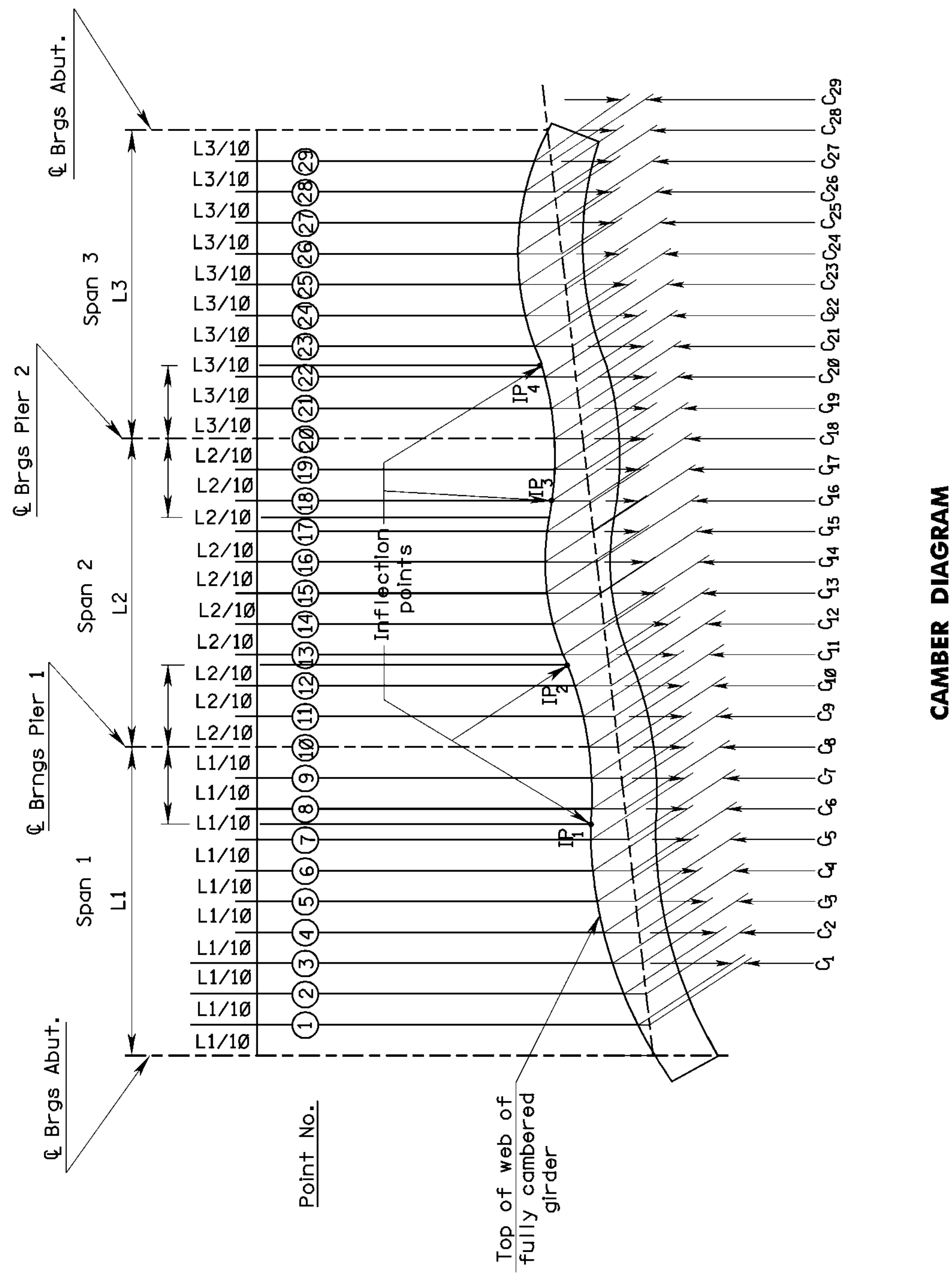

Figure 24.1 Camber Diagram

3. Sag Cambers

   Because of the objectionable appearance of a sag camber in a stringer, sag or negative cambers should be avoided. The following are a few guidelines on possible means of avoiding negative camber in a stringer.

   a. Avoid sag vertical curves on bridges.
   b. Never begin or end a superelevation transition or runoff in the middle of a span. Always begin or end transitions off the structure or, if this is impossible, begin or end the transition at a centerline of bearing or a centerline of pier.
   c. Never place a sag camber in a straight stringer on a curved roadway in order to accommodate the variation in the theoretical bottom of slab elevation. The variation should be taken up in the haunch.
   d. Upward dead load deflection may occur in some areas of continuous girders when the ratio of maximum to minimum span lengths becomes significant. There always is a possibility that computed camber built into the girder is not completely removed with the application of dead load. Camber due to a future wearing surface will remain when construction is completed. Additional camber may remain due to differences between design assumptions and actual girder performance.

24.8 Multiple Span Structures

1. It is desirable that, from an aesthetic viewpoint, a uniform depth of concrete fascia be kept for the full length of the exposed fascia. All fascia beams shall be set so that the bottom of the top flanges will be aligned.
2. Stringers, beams, and girders shall generally be of uniform depth for the full length of the structure, except where changes in depth are absolutely necessary to meet underclearance requirements or where a change in depth is desirable to enhance the appearance of the structure. Changes in depth shall not normally be made in structures with varying spans. Interior stringers shall be made the same depth as the fascia stringer.

24.9 Diaphragms and Crossframes

1. The criteria of Subsection 6.7.4 – Diaphragms and Cross frames of the LRFD Specifications and Section 3 of this Manual shall be followed in analyzing the need for their provision.
2. The structural steel layout should be examined to determine if the location of relatively stiff intermediate diaphragms placed normal to the stringers introduce detrimental stresses in diaphragms and stringers due to twisting. If this condition exists, the spacing of the diaphragms should be staggered.

   Also, the following note should be included on the plans:

   "Intermediate diaphragm connections to stringers shall be limited to finger-tight bolts in oversized holes until the dead loads are in place. The bolts shall be tightened after the deck is in place."
3. Generally, the above note should be provided on final plans for most structural steel erection applications. Especially, final plans that are for those projects where stage construction is involved in the construction process.

24.10 Transverse Intermediate Stiffeners

1. Refer to Subsection 24.5 for Fracture Control Plan criteria.
2. See Guide Plate 3.9-6 for intermediate Stiffener details.

24.11 Bearing Stiffeners

See Guide Plate 3.9-5 for Bearing Stiffener details.

24.12 Connector Plates For Interior Diaphragm X-Frames

See Guide Plate 3.9-14 and 3.9-15 for Connector Plate details.

24.13 Stability During Transportation And Erection

The stability of the stringers and girders during transport and erection is normally the responsibility of the Contractor. However, wherever possible, the design should be such that temporary bracing or diaphragms are not required. In reviewing working drawings, Engineers shall satisfy themselves that the Contractor has properly met his contractual responsibilities in this respect.

24.14 Welded Details

1. Field welding to stringers, plate girders or any major component of the structure shall not be permitted unless approved by the Manager, Structural Engineering, prior to the submission of working drawings.

   Field welding in such cases shall conform to the following Sections of ANSI/AASHTO/AWS Bridge Welding Code D1.5. The following parameters shall be included in the Special Provisions:

   - Pre-qualification of the proposed welding procedures shall be in accordance with Section 5, Part A.
   - Qualifications of the welding operator shall be in accordance with Section 5, Part B.
   - The Quality Control Inspector shall meet the qualifications specified in Section 6 and 12.16.
   - All full penetration welds shall be inspected according to AWS D1.5 and NJDOT Standard Specifications requirements.
   - All fillet welds shall be 100% Magnetic Particle (MT) tested in addition to Visual Inspection.

2. The ANSI/AASHTO/AWS Bridge Welding Code D1.5 promulgates the following concepts of inspection, which, in effect, are separate functions:

   a. Fabrication/Erection Inspection and Testing (Quality Control) is to be performed by the Contractor or Fabricator as a mandatory requirement.

   b. Verification Inspection and Testing (Quality Assurance) is the prerogative of the State.

      Provisions in the ANSI/AASHTO/AWS Bridge Welding Code D1.5 requires that contract documents identify main members and also that contract documents identify groove welds in these members as to category of stress (tension, compression or reversals of stress). Both of these identifications are needed to define the extent of non-destructive testing required by the Contractor as a minimum level under QC inspection specifications.

Identification of the nondestructive inspection required for all welds included in the ANSI/AASHTO/AWS Bridge Welding Code D1.5, shall be accomplished by providing symbols and notes as per paragraph (b) above. This essentially fulfills the requirement of the Bridge Welding Code.

For main member components in structure types such as trusses, bents, towers, box girders etc., it shall be the Structural Design Engineer's responsibility to identify such members and welds as part of the details on the contract drawings with the appropriate welding and NDT symbols.

3. Certain miscellaneous details (supports for screed rails, steel deck forms, miscellaneous connection plates, gussets, etc.) shall normally not be welded by the use of fillet welds (regardless of the direction of weld), plug welds, or tack welds to members or parts subject to tensile stress. At locations where welding cannot be avoided, the maximum stress at the point of attachment shall not exceed nominal fatigue resistance as defined in Subsection 6.6 of the AASHTO LRFD Bridge Design Specifications.

   The attachment of these details shall not be allowed where the stress exceeds the nominal fatigue resistance.

4. The contract plans and shop drawings shall clearly show the flange areas where no welding is permitted and the areas on continuous girders where the stiffeners are to be connected to the top or bottom flanges.

5. In the fabrication of HPS girders, the AASHTO Guide Specifications for Highway Bridge Fabrication with HPS 70W steel recommends that filler metals for Grade 50W base metal be specified for all fillet welding (undermatched fillet welding). When the use of HPS is planned, this criteria should be stated on the Plans.

24.15 Shear Locks

Shear locks shall be provided when a longitudinal expansion joint is located in the roadway area as guided in Subsection 20.6 b. of this Manual. The shear locks shall be located at intermediate diaphragms within the middle half of the span. A minimum of three shear locks shall be provided per span. The shear locks are intended to eliminate differential deflections due to live load and impact.

24.16 Flared Decks

Beams should be laid out parallel as much as practicable. Non-parallel beams shall be kept to a minimum.

24.17 Field Splices

1. To facilitate the fabrication, shipping and the erection of steel girders, one optional field splice will be permitted in spans between 120 and 150 feet in length. This field splice shall be located between the 1/3 and outer 1/4 points of the span length.

   When the span exceeds 150 feet, optional field splices may be located between each of the 1/3 and outer 1/4 points.

   In continuous spans, the bolted field splice shall preferably be made at or near the points of dead load contraflexure.

2. Locations and details of the optional field splice shall be shown on the plans. The Contractor may request modifications subject to approval by the Engineer.

3. Field splices shall be designed and detailed using AASHTO M 164 (ASTM A 325) high strength bolts. The flanges should have sufficient excess area at points where splicing is anticipated to permit a bolted splice to be made.
4. Splice locations are generally selected near transitions in flange thickness or width where there is sufficient flange area to permit hole drilling while still maintaining the required net area.
5. When rolled beams are used for continuous structures, the field splices should be located in areas where no cover plates are required. Consideration should be given to the fact that the fatigue strength of the section adjacent to the bolted connection (Category B*) is less than the fatigue strength of the base metal in areas where there is no splice (Category A*).
6. See Subsection 20.7.7 of this Manual concerning depth of concrete haunch at location of field splices.

24.18 Paint Coating Systems

1. Environmental Zones. Past studies of air pollutants and sea salt and their effects on structural paint has resulted in establishing the State of New Jersey into four environmental zones. These zones are listed herein. The current structural steel paint systems used by the Department are acceptable for use in all four environmental zones.
2. High Pollutant Level Zones. When planning the rehabilitation of structural steel, there is no evidence to show that repainting schedules are adversely affected in areas where, due to the bridge structure's location, pollutant levels may be higher. Thus industrial and rural areas should normally be considered comparable with regard to the use of current structural steel paint systems. Unusual situations such as structures over or near factories may require individual study to access the quality and the extent of required removal of the existing coating system.
3. Salt Splash Zones. The effect of salt splash water on the deterioration of structural paint is dependent upon its salinity. It has been established that waters with salinity high enough to initially require use of paint coating system specifications include all coastal waters (Bays, Harbors, etc.) and coastal parts of tidal rivers. The limits of salt splash zones of tidal rivers in New Jersey are delineated by the following table:

River:	**Salt Splash Zone Limit**
Delaware	Bridgeport, NJ
Mullica	14th mile of River's Length
Hudson	New York Border
All other tidal rivers	15th mile of River

Note: Salt splash zones are defined as areas that are 15 feet or less above the high water level.

4. Salt Intrusion Zones. Certain areas of the State due to their geographic configuration are subjected to high concentrations of sea salt suspended in the air. These areas are designated to be in salt intrusion zones. As illustrated herein salt intrusion is generally limited to a 2 mile coastal region.

5. Steel bridge structures that are located in areas where pollutant levels may be high, or in salt splash zones or in salt intrusion zones will initially require use of a paint coat system. However, as per the guidance provided in Subsection 24.19 the use of weathering steel may be considered. Subsection 24.19 contains guidance on evaluating the use of weathering steel.

ZONAL AREAS OF NEW JERSEY AFFECTED BY SALINITY

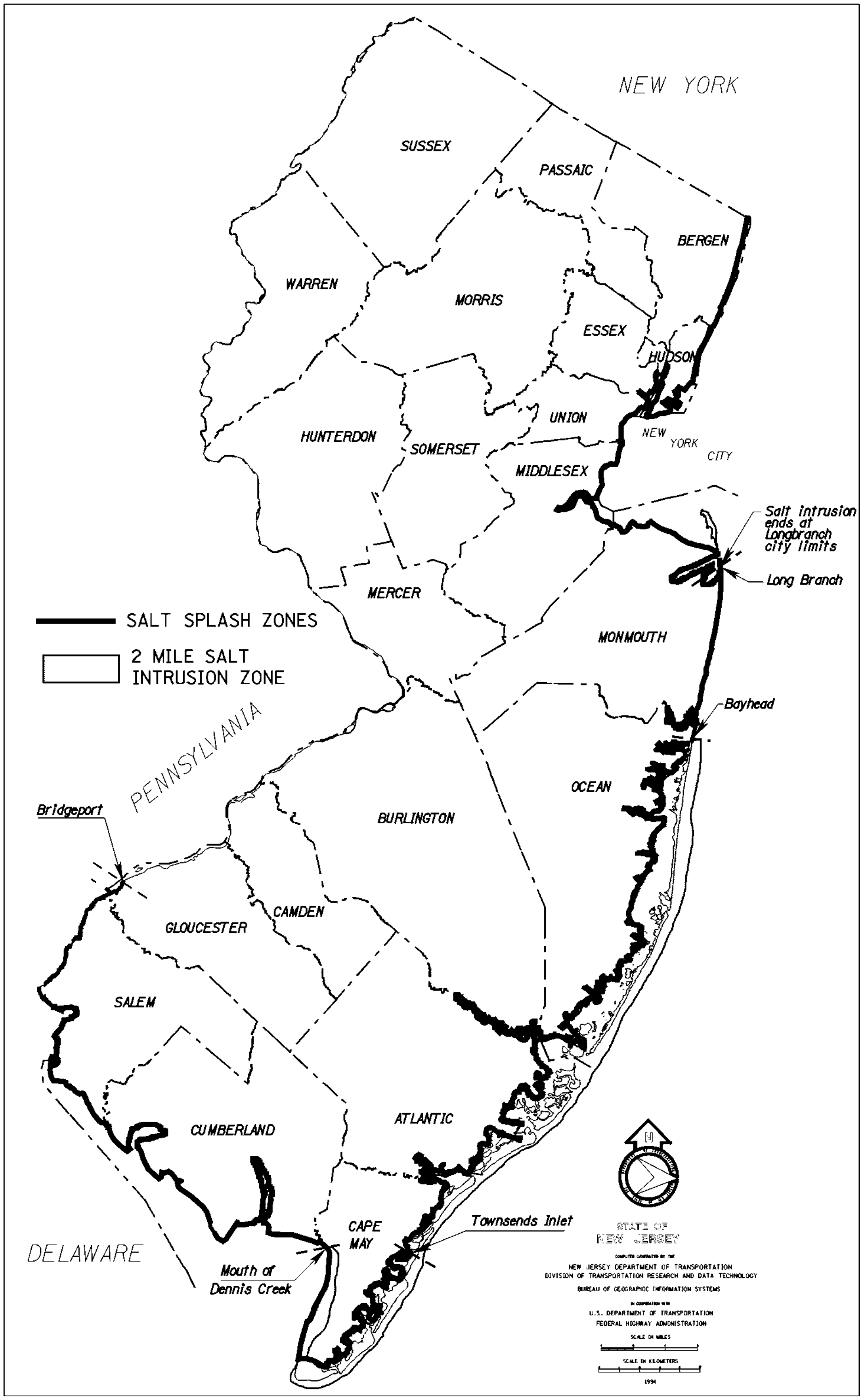

Figure 24.2

The following notes are included to clarify the map on the previous page:

a. A river's point of measurement is to start where the mouth noticeably changes into a bay or ocean.

Examples: Hackensack and Passaic Rivers - Newark Bay
Mullica River - Great Bay
Raritan River - Line parallel from South Amboy to opposite coastline
Shrewsbury River - Sea Bright Bridge

b. Sea salt intrusion areas are surrounded on three sides by salt water (peninsula, protrusion) such that at least one side faces open ocean or, are those land masses completely surrounded by salt water.

c. Except for the Delaware and Hudson Rivers, designated splash zones are only approximations of splash zones on rivers.

e. Dashed lines denote transition points from splash zones to 2 mile intrusion zones.

6. Listed below are the four environmental zones identified in the State of New Jersey:

Zone 1 - Rural or industrial, mild exposure. Where severe corrosion is not a problem.

Zone 2 - Industrial, severe exposure. Area where corrosion is a serious problem. Progressively aggressive industrial locations.

Zone 3A - Marine, mild exposure. Structural steel more than 15 feet above mean high water. Structure located in less severe coastal salt intrusion zone.

Zone 3B - Marine, severe exposure. Structural steel less than 15 feet above mean high water. Structure located in severe coastal salt intrusion zone.

The cleaning and painting systems for new structural steel shall be based on specifications that are contained in the NJDOT Standard Specifications.

For the painting of existing structural steel a written request shall be submitted to the NJDOT Bureau of Maintenance Engineering for direction as to the coating system (Organic Zinc or Epoxy Mastic) that is to be specified for use in the project.

The guidelines listed in the Table 24.3 can be referred to for guidance in planning the work.

Also, as required in the Standard Specifications, a request should be submitted to the Bureau of Project Support to request an EPA ID number. The EPA ID manifest number shall be listed on the manifest for processing of the waste. A minimum of two (2) weeks is required to obtain this information.

The Bureau of Maintenance Engineering and the Bureau of Project Support should be given the structure number, location (highway route number or road name and milepost), municipality, zip code and County.

Table 24.3 Coating Systems

Coating System	Paint System	Surface Preparation	Acceptable Environmental Zones	Selection Criteria
IEU	P: Inorganic Zinc Rich I: Epoxy Polyamide F: Aliphatic Urethane	Near-White Blast Cleaning, SSPC-SP-10	All	Use for the painting of all new structural steel.
OEU	P: Organic Zinc Rich I: Epoxy Polyamide F: Aliphatic Urethane	Near-White Blast Cleaning, SSPC-SP-10	All	Use for all existing structural steel with an ASTM D610 Rust Grade of 6 or less and when no major structural work involving steel replacement is scheduled in the near future.
EU	P: Aluminum Epoxy Mastic I: Aluminum Epoxy Mastic F: Aliphatic Urethane	Hand/Power Tool Cleaning, SSPC-SP-2/3 (with spot commercial blast SSPC-SP-6 if and where directed)	All	Use for the painting of all existing structural steel with an ASTM D610 Rust Grade greater than 6.
Leave Steel Unpainted				Use this option for all existing structural steel with an ASTM D610 Rust Grade of 6 or less and when major structural work, involving steel replacement, is scheduled in the near future. (Use of this option will depend on site conditions)
Key: P=Primer I=Intermediate F=Finish				

7. The Standard Specifications provide color chip numbers for the following finish coat colors:

   Foliage Green; Lake Blue; Brown

   Brown should be specified only at those locations where a significant aesthetic objective is to be achieved. Brown should not be specified for Non-Redundant (Single-Load-Path) type bridges.

   Generally, designation of finished colors should be based on the following considerations: If a bridge is located in an opened area (urban or rural) where it is more exposed to the open sky, then Lake Blue should be designated. If a bridge is located in an area other than this, Foliage Green should be designated.

   Other finish coat colors; such as, gray or off-gray may be used. The color chip number for these colors may be obtained by contacting the Bureau of Maintenance Engineering. The obtained color chip number should be provided in the Project Special Provisions.

8. The following notes are required on Structural Steel plans to compliment the requirements of the Standard Specifications.

   Coating System: ____________________

   Finish Coat Color: ___________________

24.19 Weathering Steel

1. Uncoated weathering grade steels have been available to the bridge engineering industry for many years. The cost-effectiveness of using this material has been demonstrated in both short and long-term savings. The additional cost of this steel is offset by the elimination of the need for initial complete painting. As stated earlier, these steels are currently supplied under AASHTO Specification M270 (ASTM A709) "W" and "HPS W" Grades.

2. The conditions stated below shall render a site not suitable for use of weathering steel.

   a. Grade Separations in "Tunnel-Like" Conditions. Refer to 3.a. below for additional clarification.

   b. Low Level Water Crossings; such as,

      1.) 10 feet or less over stagnant, sheltered water.

      2.) 6 feet or less over moving water.

3. If a proposed structure is to be located at a site with any of the characteristics noted above, the use of uncoated Weathering Grade steel shall not be contemplated. The guidance provided below, shall be considered in analyzing these conditions.

   a. Grade Separations. The so-called "tunnel effect" is produced by the combination of narrow depressed roadway sections between vertical clearances and deep abutments adjacent to the shoulders as are found at urban/suburban grade separations. These roadway/bridge geometrics combine to prevent roadway spray from being dissipated by air currents and can result in excessive salt in the spray being deposited on the bridge steel. In such locations, the use of uncoated weathering steel should be avoided as deicing salts may result in adverse conditions.

b. Low Level Water Crossings. Sufficient clearance over bodies of water must be maintained so that spray or condensation of water vapor does not result in prolonged periods of wetness of the steel. Clearance to bottom flange of at least 10 feet over sheltered, stagnant water and at least 6 feet over running water is recommended.

4. Marine or Industrial Environment. When the project site is located in a marine or industrial environment, a more precise technical evaluation of the suitability of uncoated weathering steel may be obtained from a corrosion consultant, from conducting standardized environmental tests or from both. If serious doubt remains after such an evaluation, then engineering judgment should lean towards coated steel.

5. If weathering steel is approved for use, the following items should be studied in detailing the construction of a bridge:

   a. Elimination of bridge joints where possible.

   b. Expansion joints must be able to control water that is on the deck. Consider the use of a trough under the deck joint to divert water away from vulnerable elements.

   c. When addressing parallel bridge conditions, the distance between the bridges must be assessed to determine if the adjacent fascia girders should be completely painted. If the fascia girders are close together, possible snow accumulation from plowing or debris accumulation could cause a situation where the steel does not completely dry.

   d. Do not use welded drip bars where fatigue stresses may be critical.

   e. Elimination of details that serve as water and debris “traps”.

   f. “Hermetically seal” box members when possible, or provide weep holes to allow proper drainage and circulation of air.

   g. Cover or screen all openings in boxes that are not sealed.

   h. Protection of pier caps and abutment walls to minimize staining.

   i. Seal overlapping surfaces exposed to water (to prevent capillary penetration action).

   j. Provide drip plates (bars) as detailed in Guide Plate 3.9-23.

   k. Paint weathering steel as specified in Section 906.04.03A of the NJDOT Standard Specifications for certain areas.

   l. If non slip-critical bolted joints are used, the faying surfaces should be painted or sealed to prevent the capillary penetration.

24.20 Bearing Devices

The following guidance shall be considered for the design of new structures or for those projects that involve, as applicable, a superstructure replacement. For decision making guidance as to the seismic retrofit of existing bridges, Section 38 of this Manual should be referred to for guidance.

24.20.1 Bearing Selection Evaluation

a. The bearing type selection should be based on achieving the most economical solution that will support all required movements. An initial evaluation will reveal that reinforced elastomeric bearings or elastomeric bearing pads will often be the lowest maintenance and most economic solution as a bearing selection.

b. The use of HLMR bearing systems should be based on satisfying loading conditions that elatomeric bearings cannot.

   Accordingly, economics shall not be the sole category in selecting bearing types. Accommodating longitudinal, transverse and rotational movements as well as consideration of skew conditions should be evaluated in the bearing selection.

c. Subsection 14.6.2 of the AASHTO LRFD Specifications provides a Table that tabulates bearing suitability. This Table can be referred to in determining bearing system selection.

d. Seismic isolation bearings perform all of the service load functions of other bearing types. Also, they will reduce and distribute seismic forces.

24.20.2 Requirements for Bearings

Standard Drawings for "Elastomeric Bearing", "Pot Bearing" and "Seismic Isolation Bearing", as contained in this Manual, may be referred to for a conceptional presentation of these type bearing systems.

A. General

Structural bearings for use on new bridges or for superstructure replacements shall include use of Steel Reinforced Elastomeric Bearings, Elastomeric Pads, either circular or rectangular, High Load Multi-rotational Bearings or Seismic Isolation Bearings.

High Load Multi-Rotational (HLMR) bearing systems shall include those types that consist of a rotational element of the pot type, disc type or spherical type. When expansion is required, HLMR bearing systems may include sliding surfaces.

Components of such bearings shall include masonry, sole and shim plates, bronze or copper alloyed bearing and expansion plates, anchor bolts, guide devices, polytetrafluorethylene (PTFE) sheets or surfacing lubricants and adhesives.

When load conditions indicate that plain Elastomeric Pads or Reinforced Elastomeric bearing systems are sufficient, detailing shall be provided on the final plans for their use. However, when load conditions indicate that HLMR or Isolation Bearing systems are warranted, then the complete design of these bearing assemblies is not required with the final plan submission. As described herein, sufficient information is to be provided on the plans to permit the bearing assembly type to be selected by the Contractor.

Bearings shall be supplied as fixed bearings, guided expansion bearings or non-guided expansion bearings. Bearings shall adequately provide, as applicable, for the thermal expansion and contraction, rotation, camber changes and creep and shrinkage of structural members.

The design, materials, fabrication and installation methods shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* and Sections 14 and Section 18 of the *AASHTO LRFD Bridge Construction Specifications*.

B. Construction Document Requirements

1. Final Plans shall include a “Bearing Table”, as illustrated on the following page that indicates the following information:

    a. A listing of all minimum and maximum vertical and horizontal service loads and transverse and longitudinal rotation requirements for the applicable AASHTO LRFD Load Groups as shown in the following Table.

       As discussed in Subsection 14.4.2 of the *AASHTO LRFD Bridge Design Specifications*, bearings must accommodate movements in addition to supporting loads. Therefore, displacements and, particularly, rotations, shall be listed. This will include all longitudinal forces, transverse forces and seismic forces.

    b. Minimum design rotation requirements of the bearing and construction tolerance.

2. The following information shall also be noted or detailed on the plans.

    a. Magnitude and direction of movements at all bearing support points including seismic, thermal, creep and shrinkage movements.

    b. The location, quantity and type of each bearing (fixed, expansion or guided expansion) and the location of all bearing units. An actual bearing layout is preferred or a bearing framing plan to provide this data may be used.

       To facilitate selection of a bearing system, Table 24.4 should be provided in the design plans. In order to limit the table size, Engineering judgment should be used to eliminate groups which obviously will not control the bearing design.

       Design loads shall be based on the load combinations and load factors that are specified in Subsection 3.4 of the *AASHTO LRFD Bridge Design Specifications*.

Table 24.4 Displacement/ Rotations

		Displacements/ Rotations									
	Load Combination Limit State	**Vertical**		**Horizontal**				**Rotation (RAD)**			
				Transverse		Longitudinal		Transverse		Longitudinal	
		Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Strength I											
Strength II											
Strength III											
Strength IV											
Strength V											
Extreme Event I											
Extreme Event II											

Service I											
Service II											
Service III											
Fatigue											

C. Design Requirement

The Designer is advised that he must provide estimated bridge seat elevations with the submission of final plans. The exact elevations will be determined upon submission of the shop drawings that will, of course, designate the bearing height.

The estimated bridge seat elevation heights should be based on the loads that the bearings must be designed to and the required degree of rotation and displacement.

Additionally, the location of anchor bolts and required grillage reinforcement steel size and spacing shall be indicated on the plans. However, it is to be understood that this detailing may have to be adjusted upon the final bearing system type selection.

Accordingly, notes to this affect shall be provided.

The following note shall be provided when designating bridge seat elevations: "Bridge Seat Elevations are based on a bearing height of _____________".

24.20.3 Permitted Bearing Assembly Types

The following narrative is provided to familiarize the Designer as to the basic features of the permitted bearing assembly types:

A. Elastomeric Bearings. Elastomeric Bearings shall include unreinforced pads (consisting of elastomers only) and reinforced bearings with steel or fabric laminates. Rectangular or circular types are permitted.

1. Elastomeric bearings have been developed to provide a maintenance free device capable of accommodating expansion and rotation by utilizing the unique characteristics of the elastomeric material.
2. Elastomeric bearings are generally placed between sole plates and masonry plates. In some instances, they can be placed directly between the superstructure member and the substructure unit.
3. Elastomeric bearings are available in three basic types as follows:
   - Plain elastomeric pads
   - Steel reinforced elastomeric pads
   - Fabric reinforced pads (usually a fiberglass composition)
4. Laminations can be created in the elastomer by introducing a layer of steel or fabric between the layers of elastomer. The sheets separating the layers of elastomer are completely encased within the elastomeric material. For vertical loads, each layer of the elastomer behaves like an individual pad, while the horizontal strain is additive to each layer. That is, as layers are applied, the allowable horizontal deformations are increased. Therefore,

adding laminations is a convenient way to accommodate larger lateral movements for the same compressive loads.

5. As required by the AASHTO LRFD Bridge Design Specifications, elastomeric materials shall have a hardness of from 50 to 70 durometers.
6. When permitted by design conditions, it is not necessary to bond the elastomeric pads to the superstructure and substructure concrete surfaces. In such cases, restraining lips or keeper plates should be provided around the pads. This will inhibit the potential of the pads walking off the bearing locations. When placed between steel sole plates and masonry plates, the elastomeric material should be bonded vulcanized to the steel plates.

B. Seismic Isolation Bearings

Seismic bearings are permitted if seismic analysis warrants their use. Refer to Section 38 for Seismic analysis criteria.

1. The basic intent of seismic isolation is to increase the fundamental period of vibration such that the structure is subject to significantly lower earthquake forces.
2. The reduction in forces is accompanied by an increase in displacement demand which must be accommodated with a flexible mount.
3. The following elements describe the basic composition of a bridge seismic isolation system:
   a. A flexible mounting so that the period of vibration of the bridge is lengthened sufficiently to reduce the force response.
   b. A damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level.
   c. A means of providing rigidity under low (service) load levels such as wind and braking forces.
4. Rather than resisting the large forces that are generated by earthquakes, seismic isolation systems decouple the bridge deck from the ground motion. When used in combination with a flexible device such as an elastomeric isolation bearing, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces.

   Standard Drawing number 2.2-3 may be referred to for a conceptional presentation of Seismic Isolation Bearings. This drawing is for informational purposes only and is not to be included in a contract set of plans.
5. Seismic design, performance and testing shall be assessed in accordance with the AASHTO Guide Specifications for Seismic Isolation Design.
6. Seismic Isolation bearing assemblies shall include seismic isolation bearings (isolators), sole plates, masonry plates, mounting plates, lead cores, steel shims, bolts, washers and anchor bolts.
7. The following loads will typically govern the design of the various components of the bearing assembly:
   a. Vertical Loads will govern the plan size of the assembly and the internal rubber layer thickness.

b. Short term loads and damping requirements will govern the lead core diameter.

c. Long term displacements and seismic requirements will govern the total rubber height.

d. Imposed rotations will govern the internal rubber layer thickness and the total rubber height.

C. High Load Multi-Rotational (HLMR) Bearing Systems

1. Pot Bearings

a. The rotational elements of a pot bearing shall consist of at least a pot, a piston, an elastomeric disc and sealing rings.

b. Subsection 14.7.4 of the *AASHTO LRFD Bridge Design Specifications* may be referred to for guidance in designing pot bearing systems.

2. Disc Bearings

a. As defined in Subsection 14.7.8 of the LRFD Specifications, a disc bearing functions by deformation of a polyether urethane disc. The disc must be able to resist vertical loads without excessive deformation and be able to accommodate imposed rotations.

b. Disc bearings are typically composed of four fundamental components: the load plates, the upper and lower bearing plates, a horizontal load transfer mechanism and a elastomeric disc.