Section 5  
Bridge Decks

5.1 Concrete Deck Slabs

5.1.1 Composite Design

Concrete deck slabs on steel girders are almost always designed to act compositely with the girders. Composite design provides an advantage in reducing the necessary section of primary members and also serves to significantly stiffen the bridge. The composite action is attained by the use of properly designed stud shear connectors.

Prestressed concrete beams are also designed with a composite deck slab, regardless of whether the beams are spread or adjacent. The composite action is attained by extending reinforcing stirrups from the top of the beams into the slab.

The design thicknesses for monolithic structural slabs neglect the top integral wearing surface portion in structural calculations. This is to account for expected wear and deterioration of the wearing surface. The design thickness for various concrete deck systems are in Table 5-1.

<table>
<thead>
<tr>
<th>Deck system</th>
<th>Deck Thickness</th>
<th>Design Thickness</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monolithic Slab</td>
<td>240 mm</td>
<td>200 mm</td>
<td>Epoxy-Coated or Galvanized Reinforcement</td>
</tr>
<tr>
<td>Monolithic Slab</td>
<td>215 mm</td>
<td>200 mm</td>
<td>Solid Stainless Steel or Stainless Steel Clad Reinforcement</td>
</tr>
<tr>
<td>Two Course Deck</td>
<td>255 or 230 mm</td>
<td>190 mm</td>
<td>See Section 5.1.4</td>
</tr>
<tr>
<td>Deck Slab</td>
<td>150 mm</td>
<td>125 mm</td>
<td>Adjacent Concrete Slab Units or Box Beams</td>
</tr>
</tbody>
</table>

Table 5-1  
Deck Thickness Requirements

Dead load calculations shall always include the full thickness of the deck system. All structures with a monolithic wearing surface shall be designed for a possible future wearing surface weighing 960 N/m².
5.1.2 Monolithic Decks for Spread Girders

5.1.2.1 History

Monolithic bridge decks have been the preferred deck system for many years, although they have gone through a number of detail changes. They have remained the preferred choice because of their general performance and cost when compared to alternate deck systems. The original monolithic deck used in New York was a 7½ inch (190 mm) deck with uncoated reinforcement and a 1½ inch (38 mm) cover on the top bars. This deck system was first used around 1967. Because of concerns about chloride penetration into the deck, the top cover on the reinforcing steel was increased to 3¼ inches (83 mm) and the total deck thickness was increased to 9¼ inches (235 mm) around 1974. Uncoated reinforcing steel was still used.

With the introduction of epoxy-coated reinforcement for the top mat of steel in 1976, the top cover was reduced to 2½ inches (64 mm) and the total deck thickness to 8½ inches (216 mm). In 1992, because of renewed concerns about chloride penetration and deck durability, the top cover on the reinforcement was increased to 3½ inches (90 mm) and the total deck thickness to 9½ inches (240 mm). The top mat of reinforcement remained epoxy-coated.

In 1996, Class HP concrete was introduced for deck slabs. This mix uses a lower water/cement ratio and substituted 20% fly ash and 6% microsilica for cement. The objective was to decrease permeability and cracking of deck slabs and to increase workability and strength. In 1997, the covers on the top and bottom reinforcing steel were adjusted to provide greater protection for the bottom reinforcement and to reduce cracking of the top of the concrete. The top cover was decreased to 75 mm and the bottom cover increased to 35 mm. At the same time isotropic reinforcement was made the preferred deck reinforcement for most situations. See Section 5.1.5.1 for more information on isotropic reinforcement.

The introduction of solid stainless steel and stainless steel clad reinforcement provides designers with the opportunity to reduce the top cover in bridge decks from 75 mm to 50 mm because of their exceptional corrosion resistance. The reduction of cover and slab thickness has the potential to decrease bridge deck load enough to reduce the size or number of girders. Solid stainless steel and stainless steel clad reinforcement is significantly more expensive than plain or epoxy-coated reinforcement and requires D.C.E.S. approval. See Section 15 for more information on stainless steel reinforcement.

5.1.2.2 Current Practice

The standard deck system to be used for new construction with steel girders and spread concrete beams is a monolithic deck with an integral wearing surface and isotropic reinforcement with epoxy-coated or galvanized bars in both mats. The D.C.E.S. will make exceptions to this policy only in unusual circumstances. Bridge deck replacements should use the same deck system, if possible. However, a thinner monolithic deck, a lightweight concrete deck or alternative deck system may be used if it is necessary to reduce dead load or if a thinner deck is required.
The concrete strength and dimensions of the standard monolithic deck for bridges with steel girders or spread concrete beams are as follows:

- 28-day strength: 21 MPa (Class HP concrete)
- Thickness: 240 mm
- Cover on Top Steel: 75 mm
- Cover on Bottom Steel: 35 mm

### 5.1.3 Monolithic Decks for Adjacent Concrete Beams

The standard deck system to be used on adjacent prestressed concrete beams is a 150 mm monolithic deck with epoxy-coated or galvanized reinforcement. The monolithic deck is considered to be an integral wearing surface.

The thickness of bridge decks for prestressed concrete box and slab beams may exceed the 150 mm minimum shown on the plans. This is especially true for structures with superelevated and curved alignments. For these situations, additional thickness information needs to be provided. Maximum as well as minimum thicknesses and their locations need to be shown if the average theoretical slab thickness exceeds 175 mm for a nominal 150 mm minimum deck. When the maximum slab thickness exceeds 225 mm an additional mat of steel reinforcing bars should be provided in the thickened section for crack control, see Section 5.1.5.3.

### 5.1.4 Two-Course Decks

There are two types of two-course decks. One utilizes a 65-mm asphalt concrete wearing surface over a 190 mm structural concrete deck. The other type uses a portland cement concrete wearing surface over a 190-mm structural concrete deck.

The structural concrete deck in a two course deck system uses epoxy-coated reinforcement in the top mat with 40 mm of cover. The bottom reinforcement is uncoated and has 25 mm of cover.

A waterproofing membrane shall be used below all asphalt wearing surfaces.

The concrete overlay is specified to be one of the following, normally at the Contractor's option. In some cases, only certain options will be specified on the plans:

- 40-mm Class DP Concrete
- 40-mm Micro-Silica Concrete

Two-course decks are used by NYSDOT only in unusual circumstances after prior approval by the D.C.E.S. Some localities and authorities use them as their standard deck system. A two-course deck may provide a small increase in deck life in areas of aggressive environments or very heavy traffic, but its increased cost is usually not justified and there have been problems with pavement “shove” on decks with asphalt overlays.
5.1.5 Deck Reinforcement Design

5.1.5.1 Isotropic Decks

The design of isotropic reinforced decks is based on empirical results that show reinforced concrete bridge decks develop an arching action between girders and fail in punching shear rather than flexure when subjected to loads that are significantly higher than factored design loads. Isotropic reinforced decks have lighter reinforcement than traditionally reinforced decks and use equal reinforcement transversely and longitudinally in both top and bottom mats. Reinforcement in deck overhangs is designed for flexure the same as for conventional decks.

Isotropic reinforcement is the preferred method for deck reinforcement. It shall be used when the following conditions are satisfied:

- There must be four or more girders in the final cross section of the bridge. (A stage construction condition with three girders is permissible, however, the temporary overhangs must be reinforced traditionally.)
- The maximum center-to-center spacing of the girders is 3.3 m and the minimum spacing is 1.5 m.
- Design slab thickness shall be a minimum of 200 mm and the total standard deck thickness shall be a minimum of 240 mm. A 215-mm thick deck may be used with solid stainless steel and stainless steel clad reinforcement.
- The deck is fully cast-in-place and water cured. Only permanent corrugated metal and removable wooden forms shall be permitted (prestressed concrete form units are not allowed).
- The supporting components are made of spread steel or concrete I-girders.
- The deck shall be fully composite in both positive and negative moment regions. In negative moment regions, composite section property computations shall only include the area of the longitudinal steel.
- Isotropic reinforcement may be used with spread concrete box beams provided the reinforcement is adequate to resist flexure for the clear span between beam units.
- The minimum overhang, measured from the centerline of the fascia girder to the fascia, is 750 mm. If a concrete barrier composite with the deck is used, the minimum overhang is 600 mm.
- Skew angles up to 45°. Note: For skews above 30° isotropic reinforcement becomes very congested at the end of the slab. Traditional deck slab reinforcement is recommended for skews greater than 30°.
When isotropic reinforcement is used the following details are followed:

- The reinforcement shall be two mats (one top and one bottom) comprised of #13 bars on 200 mm in transverse and longitudinal directions. A less desirable alternate of #16 bars on 300 mm may be used at regional request. The above spacings need to be adjusted when there is a skew as noted below.
- The top and bottom transverse and longitudinal reinforcement shall be staggered so that the top bars are centered between the bottom bar spacing, except in the end zones of decks with a skew angle over 30°.
- The top and bottom mats of reinforcement are epoxy coated or galvanized.
- Top reinforcement cover is 75 mm for epoxy and galvanized bars and 50 mm for solid stainless steel and stainless steel clad; bottom reinforcement cover is 35 mm for all types of bars.
- The longitudinal bars of both mats shall be placed on top of the transverse bars.
- For skew angles greater than 30° additional reinforcement shall be placed in the slab end zones at abutments and conventional deck joints. The additional reinforcement shall double the amount of the reinforcement in both mats and in both directions. This shall be done by cutting the spacing of the reinforcement in half. This additional reinforcement zone shall extend a distance from the end of the slab equal to the girder spacing.
- Fascia overhang reinforcement must be designed traditionally. An effort should be made to use #13 or #16 bars. The isotropic reinforcement extends to the fascia. Its area is included in the overhang design. Additional longitudinal reinforcement shall be placed in the overhang as shown on the BD sheets.
- Longitudinal bars are placed parallel to the girders. Transverse bars are placed parallel to the skew angle for angles up to and including 30°. On structures with curved girders the transverse bars shall be placed radially, maintaining the maximum spacing at the outside fascia girder. When reinforcement is placed on the skew, the perpendicular bar spacing shall be equal to the 200-mm nominal bar spacing times the cosine squared of the skew angle.
- For skew angles greater than 30° the transverse bars shall be placed normal to the girders.
- Additional longitudinal reinforcement in negative moment areas shall be provided as required in Article 6.10.1.7 of the *NYSDOT LRFD Bridge Design Specifications*.
- Welded splices are not permitted. Mechanical connectors are permitted only where stage construction requires their use due to a lack of adequate clearance for a lap splice.
5.1.5.2 Traditional Deck Slab Reinforcement

When the conditions of Section 5.1.5.1 for isotropic reinforcement cannot be satisfied, traditional deck slab reinforcement shall be used. When concrete deck slabs are designed with traditional reinforcement (nonisotropic) the design shall be in accordance with strength limit state design methods of the NYSDOT LRFD Bridge Design Specifications. Service limit states must also be checked in accordance with Article 5.7.3.4. When slabs are continuous over three or more supports, advantage shall be taken of the 0.80 continuity factor to reduce dead load and live load, simple-span bending moments. It is recommended that designers include stud shear connectors in the negative moment regions of continuous girder bridges as permitted by AASHTO. This may serve to lessen deck cracks by providing a more bonded section. Including longitudinal reinforcement in this region in section properties is permitted at the designer’s option.

Transverse reinforcement for a 240-mm monolithic deck is given in the Traditional Deck Slab Reinforcement Table, Table 5-2. This transverse reinforcement is to be used in both the top and bottom mats. Design span is defined as the perpendicular distance between girders less one half the width of the one flange.

Ordinarily, girder spacing should not exceed 3.5 m. Larger spacings are possible but should be used only in special cases with the approval of the D.C.E.S.

Longitudinal reinforcement in the top of the slab shall be #16 bars at 450 mm. Spacing of longitudinal reinforcement in the bottom of the slab shall be in accordance with Article 9.7.3 of the NYSDOT LRFD Bridge Design Specifications. The longitudinal bars shall be placed on top in both mats. No bars need be placed in the bottom of the slab directly over supporting members. Additional longitudinal reinforcement in negative moment areas shall be provided as required by Article 6.10.1.7 of the NYSDOT LRFD Bridge Design Specifications.

Both the top and bottom mat of reinforcement are epoxy coated or galvanized.

For skews up to and including 30° the reinforcement shall be placed parallel to the skew. For skews over 30° the reinforcement shall be placed normal to the girders. Skewed reinforcement shall be detailed with the spacing perpendicular to the bars; not parallel to the girders. This intent needs to be detailed clearly with the use of arrowheads perpendicular to the bars.
Table 5-2
Traditional Deck Slab Reinforcement

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</table>
5.1.5.3 Reinforcement of Decks for Adjacent Concrete Beams

Deck slabs for bridges with adjacent prestressed concrete box beams or units shall be a minimum of 150 mm thick. A single mat of #13 epoxy-coated or galvanized bars spaced at 200 mm in each direction shall be used in the top of the deck. The cover on the top bar shall be 75 mm. The use of welded wire fabric has been discontinued because of the superior performance of bar reinforcement in controlling longitudinal cracking over the shear keys.

When cross slope transitions increase the deck slab thickness above 225 mm, a second, bottom mat of #13 epoxy-coated or galvanized bars spaced at approximately 200 mm in each direction should be used. The second mat of reinforcement should be used only in the areas of increased deck thickness. The bottom mat should have a cover of 35 mm above the top of the beams. The designer will need to adjust the spacing of the bottom mat to avoid the composite bars extending from the beams.

In the negative moment regions of slabs which are continuous over piers, additional reinforcement shall be added in the longitudinal direction to resist the negative moment in the deck slab. This additional reinforcement shall be designed in accordance with the NYSDOT LRFD Bridge Design Specifications Article 5.14.1.4.8. The bar reinforcement shall be long enough to span the region of moment that cannot be handled by the positive moment reinforcement, plus a development length on each end. Appropriate reinforcement shall also be provided in the transverse direction for the length of the corresponding longitudinal bars. All bars shall be placed to provide a minimum clear cover of 75 mm.

5.1.5.4 Deck Overhangs

The recommended maximum overhang of a concrete deck slab beyond the centerline of the steel fascia I-girder is 1.2 m. In addition, the maximum overhang for steel fascia I-girders less than 1.5 m in depth should be limited to 1.0 m. The use of an overhang greater than 1.0 m with steel fascia I-girders less than 1.5 m in depth requires a detailed analysis. See the current BD sheets for recommended overhangs for prestressed concrete Bulb-Tee and AASHTO I-beams.

Forming and bracing systems used to place the concrete for bridge decks with large overhangs induce large horizontal forces in the fascia girder. These forces can cause lateral buckling and deflection problems in the fascia girder resulting in a poor deck profile. See Figure 5.1.

The design of formwork and temporary bracing is the Contractor's responsibility. A properly designed fascia girder within the geometric guidelines mentioned above will handle normal practice construction loads. When the overhang geometry is outside the guidelines, the designer shall evaluate the ability of the fascia beam to safely support the construction loads. Construction loads shall include but not be limited to the forms, bracing, wet concrete, walkway overhangs, workforce, and concrete screeding machines and appurtenances. Assistance in determining typical construction loads and the level of analysis required is available from the Construction Support/Bridge Foundation Unit.

If the investigation of the assumed construction loads determines that bracing beyond that normally necessary is required place Note 16 from Section 17.3 on the plans.
If the structure is designed by a Consultant, a task for checking the fascia girder for the actual construction loads calculated by the Contractor’s engineer should be included in the Construction Support Services Agreement.

When girder depths exceed 1.25 m, another potential overhang related problem can develop. If the brace supporting the overhang form is brought back to bear against the web of the fascia girder above the bottom flange, the horizontal force from the brace can buckle the web. Place Note 51 from Section 17.3 on the plans in this situation.

In cases with large overhangs, shallow beams and long spans the designer may choose to accommodate the temporary construction loads by placing additional permanent bracing (lateral system and more diaphragms) in the fascia bay.

Reinforcement in the top of the structural deck slab in overhang regions needs to be designed to resist wheel loads on the overhang as well as impact loads on the railing or barrier. Requirements for overhang reinforcement are found in the NYSDOT LRFD Bridge Design Specifications.

Slab edge reinforcement should include #16 longitudinal bars in the top mat. See the latest BD sheets.

Top transverse deck slab bars require hooks at each fascia of the slab to provide proper development. When the transverse width is less than 9 m use one bar with hooks at each end. When the transverse width is greater than 9 m and less than 35 m, use two unequal length bars, each with a hook on one end (see Section 15.4.1). When the transverse width is greater than 35 m, provide a long straight bar in the center lapped to shorter bars with hooks on one end. Bottom transverse deck slab bars do not require hooks, and can be straight bars up to 18.29 m.

![Figure 5.1](image)

**Figure 5.1**
Overhang Form Bracing
5.1.6 Haunches

Haunches are to be provided on all bridges with steel girders or prestressed concrete I-beams, bulb-tees or spread box beams. The purpose of the haunch is to provide a means for final adjustment of the deck slab elevation to match the designed roadway profile and cross slope. The haunch allows this adjustment to correct construction and fabrication variations without having the top flange of the girder project into the structural deck.

The calculated depth of haunch shall have a 50-mm minimum concrete thickness as measured at the centerline of beam from the top of beam to the bottom of slab. A deeper minimum is required when the top flange equals or exceeds 400 mm in width to allow for roadway cross slope. The total haunch depth shown on the plans shall include the thickness of the top flange for fabricated steel girders.

At all splice locations for steel girders, the top flange splice plates will reduce the haunch depth. The designer shall verify that a negative haunch will not occur at the splice location. If a negative haunch does occur, the haunch shall be increased to eliminate the negative haunch (such that the distance between the theoretical bottom of slab and the top of the top flange top splice plate will be greater than zero). It is not necessary to provide the full 50-mm minimum haunch at the splice location. Dimension “E” in the haunch table will still be dimensioned from the theoretical bottom of slab elevation to the top of the top flange.

Details of haunches for steel girders are shown in the current BD sheets. For simple span bridges, the calculated depth of the haunch at the centerline of bearings shall be the minimum depth, plus the difference in thickness between the maximum and minimum top flange plates plus increases to account for cross slope and horizontal curvature when straight girders are used.

The haunch shall be reinforced when the depth of the concrete portion of the haunch exceeds 100 mm. Only the section along the girder where the concrete portion of the haunch exceeds 100 mm requires reinforcement in the haunch. See haunch reinforcement details in the current BD sheets.

Steel beams shall have minimum 150-mm stud shear connectors for haunches up to 100 mm in depth. Haunches on steel beams greater than 100 mm shall comply with NYSDOT LRFD Bridge Design Specifications, Article 6.10.10.1.4 or NYSDOT Standard Specifications for Highway Bridges, Article 10.38.2.3. Haunches on fascia beams of multispan bridges shall be set so that the top of the webs of fascia beams in adjacent spans line up.

Do not label the haunch as 50 mm minimum. Label it only as “haunch”. The Contractor shall provide the completed Haunch Table to the EIC.

A haunch table shall be shown on the plans to assist in construction. For spans 20 m and under, the haunch table should be done for span quarter points. For spans over 20 m, the haunch table shall give elevations at span tenth points, but not to exceed a spacing of 6 m. Bridges with curved girders should have a haunch table with the elevations given at the diaphragm lines. The predicted concrete slab and superimposed dead load deflections are shown at these points.
Haunch tables shall always be computed considering stage construction and the assumed pouring sequence for continuous span structures. In addition, the computations shall consider differing dead loads applied to fascia and interior girders, especially with construction loads such as barriers. Field measurements are then taken at the same points shown in the haunch table. The actual haunches are then determined from this information. An example of a partial haunch table to be shown on the contract plans is given in Figure 5.2. A full haunch table is shown in the current BD sheets.

Bridges with complex geometry, haunched girders, and significant superelevation transitions should have a Design Haunch Depth Table providing the “design” haunch depth at the supports.

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<th>DESIGN HAUNCH DEPTH</th>
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</thead>
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<tr>
<td>G2</td>
</tr>
<tr>
<td>G3</td>
</tr>
<tr>
<td>G4</td>
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<tr>
<td>G5</td>
</tr>
</tbody>
</table>

Table 5-3
Design Haunch Table

Except for the fascia side of the fascia girder, haunches shall not extend beyond the flange of the beam. In the past, some bridges were constructed with a haunch detail as shown in Figure 5.3. This detail was commonly employed when removable wooden forms were used for a concrete deck, since it enabled precut material to be used in the forming operation. The cause of cracking and eventual loosening of portions of this unreinforced concrete has been attributed to forces generated by corrosion on the vertical edges of the flanges. Cracking occurs at the top corner of a girder flange and progresses upward and outward through the concrete to the vertical haunch face. This loosened concrete is then prone to separate and fall from the structure.

Figure 5.2
Haunch Table
All structural plans for bridges with concrete decks supported on steel girders, Bulb-Tee and AASHTO I-beams or floor systems shall include Note 52 from Section 17.3 on the plans, in association with the standard haunch detail.

![Haunch Detail (Cracking Problem)](image)

**Figure 5.3**
Haunch Detail (Cracking Problem)

5.1.7 Forming

Current specifications permit the forming of structural slabs using removable forms, permanent corrugated metal forms and prestressed concrete form units. If one or more options is not permitted on a particular job, the remaining permitted option(s) shall be listed on the Plans.

Individual railroads and the Thruway Authority may not permit the use of permanent corrugated metal forms above their facilities. Use of these forms must be approved by the railroad or agency involved.

When permanent corrugated metal forms are specified, a small detail shall be included in the plans indicating the presence of corrugations on the bottom of the structural slab and that the bottom row of reinforcement shall be placed with 35 mm cover above the crest of the corrugations. Such a detail is shown on the current BD sheet titled, “Structural Slab Concrete (Optional Forming Systems).”

The additional weight of permanent corrugated metal forms with the corrugations filled with Styrofoam shall be taken as 192 N/m². The stringers or girders shall be designed for this additional weight.

No additional weight shall be considered when using precast prestressed concrete form units. Their details are shown on the current BD Sheet titled “Structural Slab Concrete (Optional Forming Systems).”
5.1.8 Continuous Structure Deck Slab Placements

Deck slabs on continuous structures are subject to transverse cracking during construction. The cracking can be found in negative moment areas where the concrete has already set and the placement has continued into positive moment areas. The cracking is caused by additional deflection of the beams when the concrete in the remaining positive moment area is placed.

The frequency of the cracking can be reduced if proper construction methods are used and strict control over the timing and sequencing of the deck placement operation is exercised.

Deflection cracks usually occur for one of the following three reasons:

1. Slow Rate of Placement

When the concreting progresses slowly, some of the already-placed concrete may take its initial set prior to full deflection of the steel. As additional concrete is placed during the same placement operation, cracks will occur in the concrete that has already set. To prevent this from happening, either the duration of the placement should be decreased or the time to initial set of the concrete should be lengthened.

The time required to complete a placement depends on its size and complexity, concrete delivery logistics, available rate of supply, and Contractor efficiency. Responsibility for attaining the highest practical rate of placement, and the shortest possible placement time at any particular project location rests with the Contractor.

The setting time for concrete can vary widely. It depends on many factors, such as mix design, use of admixtures, and atmospheric conditions. Retarding admixtures are intended to lengthen the time to initial set of the concrete.

To avoid cracking caused by the occurrence of initial set prior to completion of the placement, the duration of each placement shall be kept to a minimum, and no concrete shall be placed without sufficient retarding admixture to assure that initial set will not occur prior to completion of the placement.

2. Incorrect Loading Sequence

Many continuous structures require a total volume of concrete which is too large to be placed prior to the occurrence of initial set at some point in the deck. In cases where the total volume of concrete exceeds 275 m³, the total concrete volume must be divided into a sequence of placements. Although this method lessens the probability of cracking related to initial set, cracking may still occur if the sequence of applying concrete loads is incorrect.

When a sequence of placements is used, the location of the first placements is vital. Concrete cannot be placed in negative moment areas first because subsequent placements will impose tensile stresses on this concrete, resulting in transverse cracking.
Further, if any placement results in the upward deflection of concrete previously placed in a positive moment area, the concrete in that area may crack. Consequently, it is necessary to place concrete in each positive moment area during the initial placement. This may be difficult if the volume of concrete required to fully place all positive moment areas is very large. Therefore, either the concrete volume or the placement rate must be modified. In some cases, the placement rate can be increased by the use of an additional finishing machine. The volume can be decreased by adding some of the positive area to the negative area, to improve the balance between placement volumes. As a last resort, the positive moment area placement can be divided and placed in separate placements, but, in such a way as to minimize the potential for cracking.

On skewed structures, the placement of the concrete and the operation of the finishing machine should parallel the skew angle. Loading the structure in this manner equalizes the steel deflections. It may be necessary to operate the finishing machine at a reduced skew angle on certain very wide or highly skewed structures.

3. Early Application of Loads

Immediately after initial set, deck concrete has little or no compressive (or tensile) strength. At this time, minor loads or deflections can cause serious cracking in the new deck. However, compressive strength increases rapidly to a point where moderate stresses (due to loads or deflections) can be resisted. For this reason, new deck concrete that will have any measurable effect on recently placed concrete shall not be placed until adequate early strength may be assumed. A waiting period of 72 acceptable curing hours, measured from completion of previous placement to start of next placement, is considered sufficient.

Instructions to Designer

The Contract Plans for every continuous steel structure where the total volume of deck concrete exceeds 275 m³ shall include a deck placement sequence. Two placements shall be shown, except for structures comprised of unusually long or numerous spans which will require special treatment. Continuous spans with the total volume of deck concrete less than 275 m³ may be placed in a single placement. A placement is defined as the total volume of concrete placed during a continuous work period. It may result from one placement operation in one area, one placement operation in several areas sequentially; or two or more placement operations in several areas simultaneously.

“Placement 1” shall include the positive moment areas (except as noted below) in all spans. “Placement 2” shall include all the negative moment areas. Construction joint locations shall be shown in the deck placement sequence details. These joints shall be located at or near points of dead load contraflexure (see Figure 5.4). In addition to delineating the placements, this information may be helpful to the Engineer should it be necessary to terminate the Contractor's deck placement operation due to unforeseen circumstances.

When the total volume of deck concrete to be placed during Placement 1 exceeds 275 m³, two simultaneous placement operations shall be used. The designer should specify this by including a note in the deck placement sequence details.
At a few project locations, the available supply of concrete will not support the use of two placement operations. The designer must determine that sufficient concrete is available before specifying the use of two placement operations on the plans. The determination may be obtained by asking the Regional Materials Engineer. When the use of two placement operations is impractical, or when special treatment is in order due to unusual length or number of spans, Placement 1 may be divided into Placement 1A and Placement 1B. The plans should show Placement 1A to be comprised of end span positive moment areas only. A note shall be added stating that the segments labeled Placement 1B shall not be placed until a minimum of 72 acceptable curing hours after the completion of Placement 1A. This procedure confines the risk of deflection cracking to end span areas near the points of contraflexure only.

In certain instances, where the concrete volume is very large, the designer may elect to modify the Placement 1 segment lengths such that Placement 2 includes some positive moment area. This may be accomplished in either of two ways:

1. Move the location of construction joints up to 5% of the span length into the positive moment area (see Figure 5.5).
2. Introduce an additional construction joint within 20% of the span length from the abutment, in end span positive moment areas only (See Figure 5.6).

Either, or both, of these methods will reduce the duration of Placement 1. The total placement volumes of Placement 1 and Placement 2 will also become more equal, thus facilitating the Contractor's operations.

Construction joints shall be shown parallel to the skew angle, regardless of the orientation of the reinforcement.

Longitudinal construction joints shall not be used to reduce placement size.

The direction of placement shall be shown on the plans. The direction of placement shall preferably be uphill and always uphill when the true (not theoretical) grade exceeds 3%. Also applies to simple spans.

Camber-deflection data shown on the plans shall be based on the placement sequence shown on the plans. The loads imposed by Placement 1 will be supported by the noncomposite beam section, and partial deflections shall be computed accordingly. The loads imposed by Placement 2 will be supported by the composite beam section, n=27 (assuming a modular ratio=3n), in positive moment areas covered by Placement 1, and by the noncomposite section in negative moment areas. Partial deflections from the various placements included in Placement 2 shall be computed, assuming simultaneous placement.

The Designer shall check for uplift at bearings. Where uplift is anticipated, a load vector shall be shown at the free end bearing line (usually an abutment) towards which Placement 1 is progressed, It shall be accompanied by a note reading:

Provide uplift restraint equal to ________kN/Bearing. The cost of this restraint shall be included in the price for the appropriate concrete deck item. (See Figure 5.4)

See section 17 for additional slab placement notes to be shown on the contract plans.
Figure 5.4
Slab Placement Sequence - A
(For Decks Over 275 m³)

Figure 5.5
Slab Placement Sequence - B
(For Decks Over 275 m³)

Figure 5.6
Slab Placement Sequence - C
(For Decks over 275 m³)
5.1.9 Stage Construction Deck Slabs

5.1.9.1 General Considerations

Stage construction should be used only if absolutely necessary. It increases construction time, maintenance and protection of traffic costs, and overall construction cost. The resulting deck slab has the potential of having lower quality than if placed in one continuous placement. However, because site conditions often necessitate stage construction it is a common strategy employed when replacing existing bridge decks, superstructures and complete bridges. It allows structures to remain in service during all or most of the replacement process, thereby avoiding detours or expensive temporary bridges.

During the construction operation, a portion of the proposed bridge width is built as an independent bridge for a specific stage of that construction process. Thus, a "bridge" exists in service for some period of time that may have different performance characteristics than the finished full width structure. It is extremely important that the bridge resulting from each stage of construction be evaluated to ensure the serviceability required during that stage. It is also important that the bridge be analyzed for the various construction loads to which it will be subjected, including, but not limited to, erection operations and slab placement operations.

Attention to the design and service behavior of these partially complete structures will avoid construction problems, unanticipated costs and delays, and potential failures. It will also provide a better engineered structure during the various stages and eventually through the bridge's entire service life.

A third placement (Closure Placement) between the stages should be used if possible. This will help to isolate the second stage deck slab during the curing process from undesirable vibrations caused by traffic on the first stage deck slab. In addition, the closure placement permits a smooth transition between the top surfaces of the deck placements should they be misaligned due to variation from the theoretical deflection of one or both groups of girders. The closure placement should be wide enough to accommodate the transverse bar splice. If it cannot be made wide enough, mechanical connectors shall be utilized on the transverse reinforcement. Consideration should also be given to increasing its width to keep the first and/or second stage overhang from becoming too large.

Notes from Section 17 will be placed on the plans where applicable. They also contain instructions concerning the installation of the diaphragms between the stages.

Eccentric construction stage loads (particularly on stage widths supported by 2 or 3 girders) can cause the superstructure to noticeably move laterally during the deck placement. When lateral movements are anticipated, additional permanent or temporary bracing to resist such movements should be considered. It may also be possible to brace against the adjacent existing structure (or previously completed adjacent stage). When bracing against an adjacent structure, the bracing must allow for freedom of vertical movement so the construction stage deck pour deflection will occur as predicted. Top struts shall be included in all cross frames located in temporary fascia bays of each stage of construction.
For longer spans (over 40 m) combined with narrow construction stage widths (2 or 3 girders), special treatment of Superimposed Dead Loads (SDLs) may have to be considered to maximize the match between work completed in different construction stages being connected with closure diaphragms and a subsequent deck closure placement. Specifically, the sequence of SDLs being applied must be evaluated. For example, some SDL is often applied to the first stage in the form of concrete traffic barriers while the second stage may have a lesser, or no, SDL applied to it at the time of closure. A procedure to calculate these sequenced SDL deflections to be used on the Camber and Haunch Tables is outlined in Section 5.1.9.3.

If special conditions of loading are anticipated during stage construction operations which may require the Contractor to perform an engineering analysis during construction, ensure that this is clearly presented by note on the plans.

Curved girder bridges and bridges with high skews (>30°) require special attention since the stage deck placement displacements can be very different from those where stage construction is not used. A grid analysis or three-dimensional analysis is recommended for computing stage construction behavior. For curved girder structures, each stage must be analyzed independently in addition to analyzing the final structure configuration. Individual stage conditions often produce the controlling design loads and displacements on some or all of the girders.

The designer is also reminded to check the load capacity of the existing structure if it will be used to carry traffic during a construction stage. Partial removal of the structure and/or modifications to the lane configurations and superimposed loads for stage traffic may require load restrictions or strengthening measures.

### 5.1.9.2 Steel Superstructures

The preferable minimum number of girders per construction stage is three. However, it is recognized that it may be necessary to utilize a construction stage with only two girders. If a construction stage is to be supported on two girders, the girder spacing should be increased to a reasonable maximum considering deck design requirements. The use of bottom lateral bracing with a two girder stage system is also recommended for spans greater than 35 m. Isotropic deck reinforcing shall not be used on decks supported by two girders.

Deck overhangs should be equalized where possible to avoid having an eccentric deck concrete load applied to the stage girder system. Eccentric deck placement loads can cause lateral twisting and/or unequal girder deflections during the deck placement. The weight of the slab haunches and the added thickness of the slab fascia overhang must be considered.

It is preferable to position the construction stage line at approximately the one-third point of the girder spacing between stages.

The deck dead load deflections based on stage construction considerations should be computed. The actual deck load is applied to each stage to compute individual girder deflections. In many cases, the stage deck placement load per girder will be less than the full design deck load of a typical interior girder due to the reduced stage placement overhang. Any load eccentricity applied to the girder transverse section for the construction stage must be accounted for. As an alternative to computing individual girder loads and deflections, a grid analysis computer program to model the individual construction stages may be used.
5.1.9.3 Stage Construction Deflection Calculations for Steel Structures

The following procedure may be used to calculate staged Superimposed Dead Load deflections to be used in the Girder Camber and Haunch Tables:

1. Compute the total Design SDL uniform load for the entire completed bridge \([W_{SDL}]\). This load would include final sidewalks, railings, design future wearing surface as well as weight of the deck closure placement(s).

2. Compute the Stage SDL uniform load actually applied to each individual construction stage at the time of stage closure. \([WS_{STAGE1}, WS_{STAGE2}...]\). The deck closure placements(s) are not included in the Stage SDL(s). Each Stage SDL will often be different from each other. Compute the individual girder Stage SDL deflections \([ds_{STAGE1}, ds_{STAGE2}...]\). If a heavy Stage SDL is applied highly eccentric to the stage girder framing layout, (e.g., a concrete traffic barrier on one side only) the load eccentricity should be accounted for in computing the individual girder \(ds_{STAGEX}\) values. Otherwise, \(WS_{STAGEX}\) can be distributed equally to all girders supporting the individual stage.

3. Compute the Final SDL uniform load applied to the completed bridge after stage closure \([wf_{SDL}]\), where \(wf_{SDL} = W_{SDL} - (WS_{STAGE1} + WS_{STAGE2}...).\) This consists of only that portion of the Design SDL that is applied after the stages are structurally connected to each other. Compute the individual girder deflections attributed to the Final SDL \([df_{SDL}]\). The girder \(df_{SDL}\) values are computed by distributing \(wf_{SDL}\) equally to all girders in the final bridge section.

4. The individual girder SDL deflections for each stage’s girders \([dSDL]\) are computed as follows:
   
   For (an individual) Stage X girder: \(d_{SDL} = ds_{STAGEX} + df_{SDL}\)
   
   \(d_{SDL}\) shall be the value used on the girder Camber Table for the SDL deflection incorporated into the girder Haunch Table.

5.1.9.4 Prestressed Concrete Superstructures

A complete discussion may be found in Section 9 of this Manual.
5.1.10 Deck Sealers

Sealers are an effective means of protecting concrete from the ingress of water and chlorides, while having minimal effect on the concrete's ability to breathe (transfer water vapor). Applying sealers to new concrete protects the concrete while it matures and becomes less permeable. Sealers protect existing structural concrete from corrosion-related distress when reinforcing steel is subjected to chlorides.

There are two types of sealers: surface and penetrating types. Only penetrating sealers are used to seal decks.

When penetrating sealers are applied to concrete they penetrate the surface, chemically bond to the concrete, and prevent water and chlorides from entering. Because the sealers bond below, not on, the surface, they cannot be abraded away easily. Good surface preparation prior to applying the sealer is essential to achieve the desired maximum penetration. Contaminants must be totally removed and the surface allowed to dry. When the surface is properly prepared, a five-year service life of the sealer can be achieved.

Penetrating sealer should be applied to all new and concrete overlaid bridge decks, to protect the surface from scaling due to early exposure to deicing chemicals. This is recommended because the majority of bridge deck and overlay placements occur late in the construction season thereby making them prone to early exposure to deicing chemicals, and because the concrete, regardless of age, will receive some benefit from the application of a sealer.

Parapets and barriers allow the use of curing compounds. Because curing compounds prevent penetration of sealers into concrete, sealers should not be used unless the membrane cured surfaces are allowed to cure and then are sandblasted.

Usage Guidelines

New Bridge Decks: To protect new, “green,” concrete from scaling, a penetrating type sealer (which does not contain an aqueous solvent/carrier) shall be applied to the top surface of all newly constructed bridge decks, bridge deck rehabilitations, and concrete approach slabs, in accordance with Item 559.1896_18.

Existing Bridge Decks: Application of sealers to the top surface of existing bridge decks shall be in accordance with Item 559.1796_18.

Existing decks with good quality concrete and epoxy-coated reinforcing steel should generally not be considered for sealer application. Decks with such protection are usually only sealed as a remediation for construction, material, or other problems, such as hairline cracks or an open surface. The use of sealers in these situations should be decided on a case by case basis, in consultation with the Regional Materials Engineer. Sealers are not a viable alternative for protecting improperly air entrained concrete.

Sealers may be used on existing decks with uncoated steel reinforcing bars or less than 75 mm of cover to slow down any existing corrosion and postpone more costly repairs. Sealers do not stop corrosion, but the corrosion process is slowed by reducing intrusion of water and chlorides.
5.1.11 Aggregate Requirements for Concrete Decks and Approach Slabs

To provide adequate wet-weather friction, a concrete wearing surface must have sufficient macrotexture and microtexture. Macrotexture is provided by manipulating the concrete surface during or after construction (e.g., Astroturf drag and saw-cut grooving). Microtexture is the texture on the surface of the exposed aggregate particles.

As concrete decks and approach slabs are subjected to traffic loads the cement paste abrades away, reducing macrotexture. If wear becomes excessive before the slab reaches the end of its structural life, macrotexture can be improved through relatively inexpensive treatments such as saw-cut grooving.

Traffic also reduces the microtexture of the concrete surface by “polishing” the exposed aggregate surfaces. The hardness of the aggregate determines its resistance to polishing under traffic. Once compromised, microtexture cannot be restored through inexpensive treatments, and in most cases the only remedy is to overlay the surface. Therefore, it is essential that appropriate aggregate be used during initial construction of the slab. Since harder aggregates are more expensive and of limited supply, it is not appropriate to simply use the hardest aggregates in every situation.

The required aggregate hardness depends on the traffic volume and site geometry. High traffic volume (AADT), braking traffic, or turning traffic will polish aggregate more quickly than straight rolling traffic. The NYSDOT Standard Specifications for Construction and Materials contains requirements for four types of friction aggregate; Types 1, 2, 3, and 9. Each type is intended for use under specific traffic and geometric conditions. The aggregate requirements are in addition to all surface texture requirements such as turf drag or saw-cut grooving. Increasing the macrotexture from these treatments does not compensate for using inappropriate aggregate.

If any portion of the bridge deck or approach slabs meets any one of the criteria listed below, use the Aggregate Type Selection table (Table 5-4) to determine the appropriate aggregate. If the bridge deck or approach slabs do not meet any of the criteria, use Type 9 aggregate. The designer shall specify only one type of aggregate for each bridge and its approach slabs by selecting the appropriate pay item.

- The deck or approach slabs are ≤150 m before a stop sign, traffic signal, or yield sign, as measured from the stop bar or yield sign.
- The deck or approach slabs are in a location where vehicles regularly queue regardless of distance from a traffic control device.
- The deck or approach slabs are ≤150 m from the point of curvature of a curve requiring reduced speed limit, chevrons, advisory speed, advisory curve or other warning signs or signals as defined in the Manual of Uniform Traffic Control Devices (MUTCD).
- The deck or approach slab is ≤150 m before an exit ramp, as measured from the initiation of the taper for the deceleration lane.
- The deck or approach slab is ≤150 m after an entrance ramp, as measured from the terminus of the taper for the acceleration lane.
The deck or approach slab is located on an entrance or exit ramp.

Any location where the ratio of wet weather accidents to total accidents is greater than the state average for the same facility type.

<table>
<thead>
<tr>
<th>Traffic</th>
<th>Location</th>
<th>Aggregate Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Volume ¹</td>
<td>Downstate ²</td>
<td>Type 1</td>
</tr>
<tr>
<td></td>
<td>Upstate ²</td>
<td>Type 2</td>
</tr>
<tr>
<td>Low Volume ¹</td>
<td>All</td>
<td>Type 3</td>
</tr>
</tbody>
</table>

¹“High Volume” refers to single lane bridges with design year AADT over 4000, 2 or 3 lane bridges with two-way design year AADT over 8,000, or bridges with 4 or more lanes with two-way design year AADT over 13,000. “Low Volume” refers to bridges not meeting the aforementioned criteria.

² The City of New York and the surrounding counties of Dutchess, Nassau, Orange, Putnam, Rockland, Suffolk, and Westchester are referred to as “Downstate”. All other areas are referred to as “Upstate”.

Table 5-4
Aggregate Type Selection

5.2 Jointless Decks at Abutments

A jointless bridge deck at an abutment is one where the bridge superstructure is normally supported on conventional bearings and the deck slab is continuous with the approach slab over the abutment backwall. All expansion and contraction of the superstructure is, therefore, transmitted into horizontal movement through the expansion bearings and the sliding of the approach slab over the abutment backwall. A bond breaker is used over the backwall and over the approach fills at the expansion end. The deck slab should not haunch down to an end diaphragm at a jointless abutment. Haunching the deck slab to an end diaphragm designed to carry live loads serves no useful purpose. Because the deck slab is supported directly by the very stiff backwall, the end diaphragm would actually carry very little load.

Bridges with jointless decks do not rigidly connect superstructure and substructure as integral abutment bridges do. Bridges with jointless decks are supported on conventional abutments. If it is possible to construct an integral or semi-integral abutment, it is preferable to do so rather than construct a bridge with a jointless deck using conventional abutments. Integral abutments are more cost effective because of their simpler details. Situations where integral abutments cannot be used include locations where the footing is on rock, sufficient pile penetration is not possible, or a high wall abutment is necessary. In these situations, the possibility of a semi-integral abutment should be investigated before a jointless deck is used.
The advantage to using jointless decks is the considerable benefit gained by eliminating the deck expansion joint system. Leaking deck joint systems are one of the most significant causes of bridge deterioration. Although deck joint design has improved considerably in recent years, it is unlikely that any deck joint system will ever be completely reliable. Therefore, there is a strong motivation to eliminate all deck joints whenever possible.

Jointless bridge decks at abutments can be used under the following criteria:

- Approach slabs must be used. See Section 13 of this manual for appropriate details.
- Maximum skew of 30° at the expansion end. It is difficult for the slab to slide over the backwall when the skew exceeds 30°.
- Jointless deck details may be used at abutments with U-wingwalls if the skew is less than 15°. See Section 13.1.3 for approach slab width criteria.
- Maximum skew of 60° at the fixed end.
- Jointless deck details may be used at the fixed end of the span even if a conventional expansion joint is used at the expansion end.
- On a curved girder bridge, jointless deck details may be used at the fixed end.
- Maximum expansion length at the abutment of 60 m. (Expansion length is defined as the distance from the $t$ of the expansion bearing to the $t$ of the nearest fixed bearing.)
- When the expansion length at an abutment exceeds 20 m, provision for expansion must be provided at the end of the approach slab by using the appropriate sleeper slab detail shown on the current BD sheet.

5.3 Other Deck Types

Concrete decks are almost always used on bridges but other deck types can be used in special circumstances. Some of these deck types are discussed below:

**Timber** - Timber decks should only be used on low-volume rural roads. Timber decks can be of plank construction where timbers are fastened to stringers with their wide dimension horizontal. Timber decks can also be glue laminated or nail laminated with their narrow edge horizontal. There are many variations in details for timber decks. Timber decks will usually need some kind of wearing surface, in most cases asphalt, to make them more skid resistant.

**Open Steel Flooring** - This deck uses open steel grating supported on steel stringers. It should not be used for new construction because its open construction leaves the underlying structure vulnerable to corrosive attack. These decks also have low skid resistance. Open steel flooring is, however, a lightweight deck and is sometimes used in rehabilitation projects where reduction of dead load is important. Open steel flooring should be galvanized to increase its service life.

**Filled Steel Flooring** - Similar to open steel flooring except the grating is filled with a Class D (small aggregate) concrete, which improves protection of the structure and skid resistance of the deck.

**Composite Unfilled Grid Decks** - Composite unfilled grid decks, commonly referred to as Exodermic™ decks, are a lightweight, modular deck system comprised of a reinforced concrete
slab with an unfilled steel grid. These decks can be cast-in-place or precast. Deck thicknesses may vary from 190 mm to 250 mm.

Advantages are lighter weight without sacrificing stiffness or strength and speed of construction. Precast panels can often be erected during a short, overnight work window.

The specification for this product does not provide for design delegation. Therefore, it is the designer's responsibility to design all aspects of the superstructure and provide all appropriate details in the contract plans. Use of composite unfilled grid decks requires approval of the D.C.E.S. Justification for using this system should include comparisons to other lightweight deck systems.

**Precast Concrete Decks** - There are a number of variations of this type. Their principal advantage is to shorten construction time. They can be advantageous for deck replacement projects in high traffic volume areas where detours and lane restrictions are limited. These decks can be full-depth concrete panels or a concrete deck supported by an unfilled steel grid (see Composite Unfilled Grid Decks above).

**Precast Precompressed Concrete/Steel Composite Superstructure** - This system, commonly referred to as Inverset™, is a combined superstructure and deck system made up of steel beams and a concrete slab. The deck is cast in the shop either in an inverted position or with the beams shored in an upright position. The casting process results in the steel beams being prestressed and the concrete deck being precompressed. The advantages of this system include quicker construction, reduced superstructure depth and increased deck durability.

The specification for this product does not provide for design delegation. Therefore, it is the designer's responsibility to design all aspects of the superstructure and provide all appropriate details in the contract plans.

**Fiber Reinforced Polymer Decks** - These decks consist of E-glass fibers embedded in a resin matrix. Although their use is new, they show great promise of increased durability. However, they are significantly more expensive than conventional concrete decks. They can be a great advantage on rehabilitation projects because of their extreme light weight (about 20% - 25% the weight of concrete). See Structures Design Advisory SDA 02-003.

### 5.4 Deck Drainage

It is important to provide good deck drainage on all structures primarily for traffic safety reasons, but also to prevent structure deterioration from ponding water and improperly directed drainage.

To facilitate runoff and provide better skid resistance, the surface of all concrete bridge decks and approach slabs is to be finished with longitudinal saw cut grooving. Grooves are 2.5 mm wide and 4 mm deep, spaced 19 mm on center, and are cut after the concrete has cured.

The most effective way to provide bridge deck drainage is to use curbless details. The required drainage must be balanced with railing/barrier requirements for the type of facility. Water quality issues must be considered before proposing to use curbless railing systems over waterways.
Good drainage design includes provisions to remove as much water as possible that would flow onto the bridge at the high end of the structure. This can be accomplished by locating drainage inlets approximately 3 m before either the further of the wingwall end or approach slab end when a curbed highway section exists. If there are no curbs, drainage should be handled with sod, asphalt or stone lined gutters.

If a bridge has curbs or traffic barriers it may be necessary to check the deck drainage design. Bridge deck drainage needs to be designed in accordance with FHWA Circular HEC No. 21 - Design of Bridge Deck Drainage, May 1993. The design is to be based on rainfall intensity of the most severe storm of five-minute duration likely to occur in a ten-year period.

Design criteria for bridge deck drainage are based on maintaining the following conditions:

- Maximum width for the spread of water is 3.6 m.
- Maximum spread depth is 10 mm less than the curb height.
- For highways with design speeds less than 75 km/hr, puddles may encroach into a travel lane only to a point where 2.5 m of the lane remains unencroached by the puddle width.
- For highways with design speeds greater than or equal to 75 km/hr, puddles should not encroach into any portion of a travel lane.

If any of the above conditions cannot be met, scuppers (drains) must be provided. Scuppers typically become necessary with a combination of a long (over 100 m), wide (over 15 m) bridge and a flat grade (less than 2%). The average bridge typically does not require scuppers. They should not be used unless needed because of their tendency to contribute to deck and superstructure deterioration.

Consider scupper locations prior to finalizing girder spacing to avoid interference between the outlet and the girder flanges.

When used, scuppers should be located so they do not discharge onto travel lanes, sidewalks or railroad rights-of-way. Scuppers should be midway between cross frames or diaphragms and away from abutments and piers, if possible. Scuppers should have Fiberglass or PVC downspouts extending at least 300 mm below the superstructure. Diffusers should be used over land unless erosion protection is provided or the free fall exceeds 8 m. Scuppers can discharge into downspouts carried down to ground level or to a closed drainage system. However, this method is discouraged because of the susceptibility of the downspouts to freezing or becoming plugged with debris. Bends in downspouts should be kept to a minimum. A clean out fitting should be located at each bend. Scupper details are shown on the current BD sheet.

Scupper grates should be of a bicycle-safe design. These are usually reticuline grates or parallel bar grates with welded transverse bars. See the FHWA publication Bicycle-Safe Grate Inlet Study for additional guidance.
In urban areas, if downspouts extend to the ground, and the potential exists for malicious damage, steel pipe may be used. Fiberglass downspout systems have more impact resistance than PVC systems.

Occasionally, downspouts have been encased in the substructure concrete. This practice should be avoided whenever possible, because it usually creates clean out problems and can also result in chloride damage to the concrete. If used, the installation shall include a 25 mm compressible protective covering between the pipe and the concrete to accommodate expansion of the pipe and shrinkage of the concrete.

Downspouts shall be placed at the least objectionable location by attempting to hide them from view behind columns. The surface below the outfall shall be protected by the use of a stone, concrete slab, or grouted block paving.

5.5 Deck Expansion Joints

5.5.1 Transverse Expansion Joints

Many deck joints and details have been used over the years, with varying results. The one constant result is that nearly all joint systems leaked after a short duration in service. Therefore, their use should be avoided whenever possible through the use of continuous spans, jointless abutments, and semi-integral or integral abutments.

Joint systems currently in use include armorless joints, armored joints and modular joints. See the current BD sheets for selection criteria for each joint system.

5.5.1.1 Armorless Joint Systems

Armorless joint systems are preferred for superstructure movement of 64 mm or less. This range of movement has historically been handled by armored joint systems, which are no longer the preferred system (see Section 5.5.1.2). Armorless bridge joint systems are expected to alleviate many problems associated with armored joints and compression seals.

Armorless joint systems have been used by NYSDOT Bridge Maintenance for many years with excellent results. There are no skew limitations for armorless joint systems but skews over 45° require close attention to sizing criteria on the current BD sheets.

The elastomeric concrete used in armorless joint systems offers a durable header material that cures much faster than traditional concrete. This minimizes lane closure times, reduces Maintenance and Protection of Traffic costs and shortens delays to the traveling public. Unlike traditional concrete, fresh elastomeric concrete bonds extremely well to previously placed fully cured material. It can be installed in segments, making it adaptable to stage construction as well as staged repairs or replacements. Elastomeric concrete headers shall not overhang the concrete slab.
The poured liquid sealant or closed-cell, cross-linked foam seals of armorless joint systems are easily placed in their entirety or in segments. They require very little time to place and/or cure allowing restoration of traffic in a matter of hours.

When replacing an existing armored joint and header only for a rehabilitation and the opening between the deck and the backwall or deck slabs exceeds the maximum opening given in the BD sheets it may still be possible to use an armorless joint without doing additional deck work. If the maximum opening (set opening + design movement) does not exceed 125 mm an armorless joint can still be used.

5.5.1.2 Armored Joint Systems

Persistent maintenance problems with armored joints have been routinely encountered. During initial construction, proper consolidation of concrete under the horizontal leg of the armoring angle is difficult. The resulting voids lead to water collecting under the angle. When this water freezes it lifts up the armoring angle and increases the likelihood of snow plow impact.

An additional problem is corrosion of the steel angle. On the vertical face, corrosion creates a gap at the seal to angle interface which allows water to leak onto the superstructure and substructure elements below. On the horizontal face, corroding steel causes the concrete in contact with the angle to spall away, creating a larger gap for water to get under the angle. This causes leakage behind the angle in even when the seal remains watertight.

Repair of damaged armored joint systems is time consuming and difficult. Damaged compression seals cannot be repaired and must be replaced in their entirety. Typically the whole system needs replacing which requires removal and replacement of the concrete header and armoring angles. This requires jack-hammering, cutting out the steel angles, and placing new steel angles and concrete. The repaired section cannot be opened to traffic until the concrete has cured, requiring long term lane closures.

There are skew limitations for armored joint systems. See the current BD sheets for allowable skews and selection criteria.

5.5.1.3 Modular Joint Systems

Modular joint systems are used for larger movements. Single-cell modular joint systems may be used for up to 50 mm of superstructure movement. Multicell modular joint systems are used for superstructure movement over 50 mm. There are no skew limitations for modular joint systems but skews over 45° require close attention to sizing criteria on the current BD sheets.
5.5.2 Longitudinal Joints

When the bridge width exceeds 27.5 m, a longitudinal deck joint should be considered. This is especially true for bridges whose width approaches or exceeds the bridge span. The type and placement of this joint should take the following bridge characteristics into consideration:

- Bridge deck drainage pattern (i.e., crossslope).
- Likelihood that traffic will have to traverse the joint.
- The existence of a raised or flush median.
- The existence and location of any median barrier.

A 25 mm joint is recommended if traffic is likely to traverse the joint. If a raised median with or without concrete traffic barrier is present, a 50 mm joint is recommended. If the joint is at or near the roadway surface, it should be sealed. If half-section adjacent concrete traffic barriers are used, the closure of the joint is optional. A compression-type seal is the recommended closure material in either case.

5.6 Sidewalk and Brush Curb Overlays

All sidewalks and brush curb overlays should be paid for under Item 557.30, Sidewalks and Safety Walks. The advantage of this item is that it includes the steel reinforcement and provides for a wet cure of the concrete.