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Chapter 5  Concrete Structures

5.0 General

The provisions in this section apply to the design of cast-in-place (CIP) and precast concrete structures.

5.1 Materials

5.1.1 Concrete

A. **Strength of Concrete** – Pacific NW aggregates have consistently resulted in excellent concrete strengths, which may exceed 10,000 psi in 28 days. Specified concrete strengths should be rounded to the next highest 100 psi.

1. **CIP Concrete Bridges** – Since conditions for placing and curing concrete for CIP components are not as controlled as they are for precast bridge components, Class 4000 concrete is typically used. Where significant economy can be gained or structural requirements dictate, Class 5000 concrete may be used with the approval of the Bridge Design Engineer, Bridge Construction Office, and Materials Lab.

2. **Prestressed Concrete Girders** – Nominal 28-day concrete strength ($f'_c$) for prestressed concrete girders is 7.0 ksi. Where higher strengths would eliminate a line of girders, a maximum of 10.0 ksi can be specified.

    The minimum concrete compressive strength at release ($f'_{ci}$) for each prestressed concrete girder shall be shown in the plans. For high strength concrete, the compressive strength at release shall be limited to 7.5 ksi. Release strengths of up to 8.5 ksi can be achieved with extended curing for special circumstances.

B. **Classes of Concrete**

1. **Class 3000** – Used in large sections with light to nominal reinforcement, mass pours, sidewalks, curbs, gutters, and nonstructural concrete guardrail anchors, luminaire bases.

2. **Class 4000** – Used in CIP post-tensioned or conventionally reinforced concrete box girders, slabs, traffic and pedestrian barriers, approach slabs, footings, box culverts, wing walls, curtain walls, retaining walls, columns, and crossbeams.

3. **Class 4000A** – Used for bridge approach slabs.

4. **Class 4000D** – Used for CIP bridge decks.

5. **Class 4000P** – Used for CIP pile and shaft.

6. **Class 4000W** – Used underwater in seals.

7. **Class 5000 or Higher** – Used in CIP post-tensioned concrete box girder construction or in other special structural applications if significant economy can be gained or structural requirements dictate. Class 5000 concrete is available within a 30-mile radius of Seattle, Spokane, and Vancouver. Outside this 30-mile radius, concrete suppliers may not have the quality control procedures and expertise to supply Class 5000 concrete.
The 28-day compressive design strengths ($f'_c$) are shown in Table 5.1.1-1.

<table>
<thead>
<tr>
<th>Classes of Concrete</th>
<th>$f'_c$ (psi)</th>
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<tbody>
<tr>
<td>COMMERCIAL</td>
<td>2300</td>
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<tr>
<td>3000</td>
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<tr>
<td>4000, 4000A, 4000D</td>
<td>4000</td>
</tr>
<tr>
<td>4000W</td>
<td>2400*</td>
</tr>
<tr>
<td>4000P</td>
<td>3400**</td>
</tr>
<tr>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td>6000</td>
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</table>

*40 percent reduction from Class 4000.
**15 percent reduction from Class 4000 for piles and shafts.

### 28-Day Compressive Design Strength

**Table 5.1.1-1**

#### C. Relative Compressive Concrete Strength

1. During design or construction of a bridge, it is necessary to determine the strength of concrete at various stages of construction. For instance, Section 6-02.3(17)J of the WSDOT Standard Specifications discusses the time at which falsework and forms can be removed to various percentages of the concrete design strength. Occasionally, construction problems will arise which require a knowledge of the relative strengths of concrete at various ages. Table 5.1.1-2 shows the approximate values of the minimum compressive strengths of different classes of concrete at various ages. If the concrete has been cured under continuous moist curing at an average temperature, it can be assumed that these values have been developed.

2. Curing of the concrete (especially in the first 24 hours) has a very important influence on the strength development of concrete at all ages. Temperature affects the rate at which the chemical reaction between cement and water takes place. Loss of moisture can seriously impair the concrete strength.

3. If test strength is above or below that shown in Table 5.1.1-2, the age at which the design strength will be reached can be determined by direct proportion.

For example, if the relative strength at 10 days is 64 percent instead of the minimum 70 percent shown in Table 5.1.1-2, the time it takes to reach the design strength can be determined as follows:

Let $x =$ relative strength to determine the age at which the concrete will reach the design strength

$$\frac{x}{70} = \frac{100}{64}$$

Therefore, $x = 110\%$ \hspace{1cm} (5.1.1-1)

From Table 5.1.1-2, the design strength should be reached in 40 days.
### Relative and Compressive Strength of Concrete

#### Table 5.1.1-2

<table>
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<th>Class 4000</th>
<th>Class 3000</th>
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#### D. Modulus of Elasticity

The modulus of elasticity shall be determined as specified in AASHTO LRFD 5.4.2.4. For calculation of the modulus of elasticity, the unit weight of plain concrete \( w_c \) shall be taken as 0.155 kcf for prestressed concrete girders and 0.150 kcf for normal-weight concrete. The correction factor \( K_1 \) shall normally be taken as 1.0.

#### E. Shrinkage and Creep

Shrinkage and creep shall be calculated per AASHTO LRFD 5.4.2.3. The relative humidity, \( H \), may be taken as 75 percent for standard conditions. The maturity of concrete, \( t \), may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, \( V/S \), is given in Table 5.6.1-1 for standard WSDOT prestressed concrete girders.

In determining the maturity of concrete at initial loading, \( t_i \), one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.

The final deflection is a combination of the elastic deflection and the creep effect associated with given loads shown by the equation below:

\[
\Delta_{total} = \Delta_{elastic} [1 + \psi(t, t_i)] \tag{5.1.1-2}
\]

Figure 5.1.1-1 provides creep coefficients for a range of typical initial concrete strength values, \( f'_c \), as a function of time from initial seven day steam cure \( (t_i = 7 \text{ days}) \). The figure uses a volume-to-surface, \( V/S \), ratio of 3.3 as an average for girders and relative humidity, \( H \), equal to 75 percent.


**Creep Coefficient for Standard Conditions as Function of Initial Concrete Strength**

*Figure 5.1.1-1*

F. **Shrinkage** – Concrete shrinkage strain, $\varepsilon_{sh}$, shall be calculated per AASHTO LRFD.

G. **Grout** – Grout is usually a prepackaged cement based grout or nonshrink grout that is mixed, placed, and cured as recommended by the manufacturer. It is used under steel base plates for both bridge bearings and luminaries or sign bridge bases. Should the grout pad thickness exceed 4", steel reinforcement shall be used. For design purposes, the strength of the grout, if properly cured, can be assumed to be equal to or greater than that of the adjacent concrete but not greater than 4000 psi. Nonshrink grout is used in keyways between precast prestressed tri-beams, double-tees, and deck bulb tees (see Standard Specifications Section 6-02.3(25)O for deck bulb tee exception).

H. **Mass Concrete** – Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking. Temperature-related cracking may be experienced in thick-section concrete structures, including spread footings, pile caps, bridge piers, crossbeams, thick walls, and other structures as applicable.

Concrete placements with least dimension greater than 6 feet should be considered mass concrete, although smaller placements with least dimension greater than 3 feet may also have problems with heat generation effects. Shafts need not be considered mass concrete.
The temperature of mass concrete shall not exceed 160°F. The temperature difference between the geometric center of the concrete and the center of nearby exterior surfaces shall not exceed 35°F.

Designers could mitigate heat generation effects by specifying construction joints and placement intervals. Designers should consider requiring the Contractor to submit a thermal control plan, which may include such things as:

1. Temperature monitors and equipment.
2. Insulation.
3. Concrete cooling before placement.
4. Concrete cooling after placement, such as by means of internal cooling pipes.
5. Use of smaller, less frequent placements.
6. Other methods proposed by the Contractor and approved by the Engineer.

Concrete mix design optimization, such as using low-heat cement, fly ash or slag cement, low-water/cement ratio, low cementitious materials content, larger aggregate, etc. is acceptable as long as the concrete mix meets the requirements of the Standard Specifications for the specified concrete class.

The ACI Manual of Concrete Practice Publication 207 and specifications used for the Tacoma Narrows Bridge Project suspension cable anchorages (2003-2006) can be used as references.

I. **Self-Consolidating Concrete (SCC)** – Self-consolidating concrete (SCC) may be used in structural members such as precast noise wall panels, barriers, three-sided structures, etc. as described in Standard Specifications Section 6-02.3(27).

SCC may be used in prestressed concrete girders.

SCC may be specified for cast-in-place applications where the use of conventional concrete could be challenging and problematic. Examples are where new concrete is being cast up against an existing soffit, or in members with very dense/congested reinforcing steel. Use of SCC for primary structural components such as columns, crossbeams, slabs, etc. requires the approval of the WSDOT Bridge Design Engineer.

J. **Shotcrete** – Shotcrete could be used as specified in WSDOT Standard Plans. Shotcrete may not be suitable for some critical applications unless approved by the Engineer of Record.

Substitution of CIP conventional concrete in the contract document with shotcrete requires the approval of the Engineer of Record.

Some potential shortfalls of shotcrete as compared to conventional CIP concrete include:

- **Durability** – Conventional concrete is placed in forms and vibrated for consolidation. Shotcrete, whether placed by wet or dry material feed, is pneumatically applied to the surface and is not consolidated as conventional concrete. Due to the difference in consolidation, permeability can be affected. If the permeability is not low enough, the service life of the shotcrete will be affected and may not meet the minimum of 75 years specified for conventional concretes.
Observation of some projects indicates the inadequate performance of shotcrete to properly hold back water. This results in leaking and potential freezing, seemingly at a higher rate than conventional concrete. Due to the method of placement of shotcrete, air entrainment is difficult to control. This leads to less resistance of freeze/thaw cycles.

- **Cracking** – There is more cracking observed in shotcrete surfaces compared to conventional concrete. Excessive cracking in shotcrete could be attributed to its higher shrinkage, method of curing, and lesser resistance to freeze/thaw cycles. The shotcrete cracking is more evident when structure is subjected to differential shrinkage.

- **Corrosion Protection** – The higher permeability of shotcrete places the steel reinforcement (whether mesh or bars) at a higher risk of corrosion than conventional concrete applications. Consideration for corrosion protection may be necessary for some critical shotcrete applications.

- **Safety** – Carved shotcrete and shotcrete that needs a high degree of relief to accent architectural features lead to areas of 4”-6” of unreinforced shotcrete. These areas can be prone to an accelerated rate of deterioration. This, in turn, places pedestrians, bicyclists, and traffic next to the wall at risk of falling debris.

- **Visual Quality and Corridor Continuity** – As shotcrete is finished by hand, standard architectural design, as defined in the Design Manual M 22-01, typically cannot be met. This can create conflicts with the architectural guidelines developed for the corridor. Many times the guidelines are developed with public input. If the guidelines are not met, the public develops a distrust of the process. In other cases, the use of faux rock finishes, more commonly used by the private sector, can create the perception of the misuse of public funds.

K. **Lightweight Aggregate Concrete** – Lightweight aggregate concrete may be used for precast and CIP members upon approval of the WSDOT Bridge Design Engineer.

L. **Concrete Cover to Reinforcement** – Concrete cover to reinforcement shall conform to AASHTO LRFD 5.12.3.

   1. **Precast Prestressed Concrete Girders** – Cover to prestressing strands in precast prestressed concrete girders may be measured to the center of the strand.

   Cover to mild steel reinforcement in precast prestressed concrete girders shall conform to AASHTO LRFD 5.12.3. However, cover to ties and stirrups may be reduced to 1.0 inch in “Exterior other than above” applications. See Section 5.6.7.A for additional cover requirements for deck girders.
5.1.2 Reinforcing Steel

A. Grades – Reinforcing bars shall be deformed and shall conform to Section 9-07.2 of the Standard Specifications. ASTM A706 Grade 60 reinforcement is preferred for WSDOT bridges and structures.

1. Grade 80 Reinforcement – Reinforcement conforming to ASTM A706 Grade 80 may be used in Seismic Design Category (SDC) A for all components. For SDCs B, C and D, ASTM A706 Grade 80 reinforcing steel shall not be used for elements and connections that are proportioned and detailed to ensure the development of significant inelastic deformations for which moment curvature analysis is required to determine the plastic moment capacity of ductile concrete members and expected nominal moment capacity of capacity protected members.

ASTM A706 Grade 80 reinforcing steel may be used for capacity-protected members such as footings, bent caps, oversized shafts, joints, and integral superstructure elements that are adjacent to the plastic hinge locations if the expected nominal moment capacity is determined by strength design based on the expected concrete compressive strength with a maximum usable strain of 0.003 and a reinforcing steel yield strength of 80 ksi with a maximum usable strain of 0.090 for #10 bars and smaller, 0.060 for #11 bars and larger. The resistance factors for seismic related calculations shall be taken as 0.90 for shear and 1.0 for bending.

ASTM A706 Grade 80 reinforcing steel shall not be used for oversized shafts where in-ground plastic hinging is considered as a part of the Earthquake-Resisting System (ERS).

ASTM A706 Grade 80 reinforcing steel shall not be used for transverse and confinement reinforcement.

For seismic hooks, $f_y$ shall not be taken greater than 75 ksi.

B. Sizes – Reinforcing bars are referred to in the contract plans and specifications by number and vary in size from #3 to #18. For bars up to and including #8, the number of the bar coincides with the bar diameter in eighths of an inch. The #9, #10, and #11 bars have diameters that provide areas equal to 1″ × 1″ square bars, 1¼″ × 1¾″ square bars and 1½″ × 1¾″ square bars respectively. Similarly, the #14 and #18 bars correspond to 1½″ × 1½″ and 2″ × 2″ square bars, respectively. Appendix 5.1-A3 shows the sizes, number, and various properties of the types of bars used in Washington State.

C. Development

1. Tension Development Length – Development length or anchorage of reinforcement is required on both sides of a point of maximum stress at any section of a reinforced concrete member. Development of reinforcement in tension shall be per AASHTO LRFD 5.11.2.1.

Appendix 5.1-A4 shows the tension development length for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 4.0 to 6.0 ksi.
2. **Compression Development Length** – Development of reinforcement in compression shall be per AASHTO LRFD 5.11.2.2. The basic development lengths for deformed bars in compression are shown in Appendix 5.1-A5. These values may be modified as described in AASHTO. However, the minimum development length shall be 1'-0".

3. **Tension Development Length of Standard Hooks** – Standard hooks are used to develop bars in tension where space limitations restrict the use of straight bars. Development of standard hooks in tension shall be per AASHTO LRFD 5.11.2.4. Tension development lengths of 90° & 180° standard hooks are shown in Appendix 5.1-A6.

D. **Splices** – The Contract Plans shall clearly show the locations and lengths of splices. Splices shall be per AASHTO LRFD 5.11.5.

   Lap splices, for either tension or compression bars, shall not be less than 2'-0".

   1. **Tension Lap Splices** – Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar’s development length, \( d \). There are two classes of tension lap splices: Class A and B. Designers are encouraged to splice bars at points of minimum stress and to stagger lap splices along the length of the bars.

      Appendix 5.1-A7 shows tension lap splices for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 4.0 to 6.0 ksi.

   2. **Compression Lap Splices** – Compression lap splice lengths are shown in Appendix 5.1-A5 for concrete strengths greater than or equal to 3.0 ksi.

   3. **Mechanical Splices** – Mechanical splices are proprietary splicing mechanisms. The requirements for mechanical splices are found in Standard Specifications Section 6-02.3(24)F and in AASHTO LRFD 5.5.3.4 and 5.11.5.2.2.

   4. **Welded Splices** – ASHTO LRFD 5.11.5.2.3 describes the requirements for welded splices. On modifications to existing structures, welding of reinforcing bars may not be possible because of the non-weldability of some steels.

E. **Hooks and Bends** – For hook and bend requirements, see AASHTO LRFD 5.10.2. Standard hooks and bend radii are shown in Appendix 5.1-A1.

F. **Fabrication Lengths** – Reinforcing bars are available in standard mill lengths of 40' for bar sizes #3 and #4 and 60' for bar sizes of #5 and greater. Designers shall limit reinforcing bar lengths to the standard mill lengths. Because of placement considerations, designers should consider limiting the overall lengths of bar size #3 to 30' and bar size #5 to 40'.

   Spirals of bar sizes #4 through #6 are available on 5,000 lb coils. Spirals should be limited to a maximum bar size of #6.
G. **Placement** – Placement of reinforcing bars can be a problem during construction. Sometimes it may be necessary to make a large scale drawing of reinforcement to look for interference and placement problems in confined areas. If interference is expected, additional details are required in the contract plans showing how to handle the interference and placement problems. Appendix 5.1-A2 shows the minimum clearance and spacing of reinforcement for beams and columns.

H. **Joint and Corner Details**

1. **T-Joint** – The forces form a tension crack at 45° in the joint. Reinforcement as shown in Figure 5.1.2-1 is more than twice as effective in developing the strength of the corner than if the reinforcement was turned 180°.

2. “**Normal**” **Right Corners** – Corners subjected to bending as shown in Figure 5.1.2-2 will crack radially in the corner outside of the main reinforcing steel. Smaller size reinforcing steel shall be provided in the corner to distribute the radial cracking.

3. **Right or Obtuse Angle Corners** – Corners subjected to bending as shown in Figure 5.1.2-3 tend to crack at the reentrant corner and fail in tension across the corner. If not properly reinforced, the resisting corner moment may be less than the applied moment.

Reinforced as shown in Figure 5.1.2-3, but without the diagonal reinforcing steel across the corner, the section will develop 85 percent of the ultimate moment capacity of the wall. If the bends were rotated 180°, only 30 percent of the wall capacity would be developed.

Adding diagonal reinforcing steel across the corner, approximately equal to 50 percent of the main reinforcing steel, will develop the corner strength to fully resist the applied moment. Extend the diagonal reinforcement past the corner each direction for anchorage. Since this bar arrangement will fully develop the resisting moment, a fillet in the corner is normally unnecessary.
I. **Welded Wire Reinforcement in Prestressed Concrete Girders, Walls, Barriers and Deck Panels** –

Welded wire reinforcement may be used to replace steel reinforcing bars in prestressed concrete girders, walls, barriers, and deck panels.

Welded wire reinforcement shall meet all AASHTO requirements (see AASHTO LRFD 5.8.2.6, 5.8.2.8, 5.10.3, 5.10.6.3, 5.10.7, 5.10.8, 5.10.10, 5.11.2.5, 5.11.2.6.3, 5.11.6, etc.).

Welded wire reinforcement shall be deformed. The yield strength shall be limited to a maximum of 75 ksi.

Longitudinal wires and welds shall be excluded from regions with high shear demands, including girder webs, and are limited to the flange areas as described in AASHTO LRFD 5.8.2.8. Longitudinal wires for anchorage of welded wire reinforcement shall have an area of 40 percent or more of the area of the wire being anchored as described in ASTM A497 but shall not be less than D4.

Epoxy-coated wire and welded wire reinforcement shall conform to Std. Spec. Section 9-07.3 with the exception that ASTM A884 Class A Type I shall be used instead of ASTM A775.

J. **Headed Steel Reinforcing Bars** – Headed steel reinforcing bars conforming to ASTM A970 Class HA may be used to develop reinforcement in tension. Use and development length shall be in accordance with ACI 318 (see Section 25.4.4 for development length). Minimum concrete cover and clearances to headed steel reinforcing bars shall also be provided to the outermost part of the head of the bar. Designers shall provide main bar (unheaded portion) location requirements in contract documents and verify that cover and clearance requirements to the head of the bar can be satisfied. ASTM A970 Class HA requires that the net bearing area of the head shall not be less than four times the nominal cross-sectional area of the bar. However, the head shape and an upper limit to the head net bearing area are not specified. A gross head area of ten times the bar area (a net bearing area of the head of nine times the bar area) could be used as an estimate of the upper limit of the head area.

5.1.3 **Prestressing Steel**

A. **General** – Three types of high-tensile steel used for prestressing steel are:

1. **Strands** – AASHTO M 203 Grade 270, low relaxation or stress relieved
2. **Bars** – AASHTO M 275 Type II
3. **Parallel Wires** – AASHTO M 204 Type WA

All WSDOT designs are based on low relaxation strands using either 0.5” or 0.6” diameter strands for girders, and ⅜” or ⅞” diameter strands for stay-in-place precast deck panels. Properties of uncoated and epoxy-coated prestressing stands are shown in Appendix 5.1-A8. 0.62” and 0.7” diameter strands may be used for top temporary strands in prestressed concrete girders.

Provide adequate concrete cover and consider use of epoxy coated prestressing reinforcement in coastal areas or where members are directly exposed to salt water.
B. **Allowable Stresses** – Allowable stresses for prestressing steel are as listed in AASHTO LRFD 5.9.3.

C. **Prestressing Strands** – Standard strand patterns for all types of WSDOT prestressed concrete girders are shown throughout Appendix 5.6-A and Appendix 5.9-A.

  1. **Straight Strands** – The position of the straight strands in the bottom flange is standardized for each girder type.

  2. **Harped Strands** – The harped strands are bundled between the harping points (the 0.4 and 0.6 points of the girder length). The girder fabricator shall select a bundle configuration that meets plan centroid requirements.

There are practical limitations to how close the centroid of harped strands can be to the bottom of a girder. The minimum design value for this shall be determined using the following guide: Up to 12 harped strands are placed in a single bundle with the centroid 4” above the bottom of the girder. Additional strands are placed in twelve-strand bundles with centroids at 3” spacing vertically upwards.

At the girder ends, the strands are splayed to a normal pattern. The centroid of strands at both the girder end and the harping point may be varied to suit girder stress requirements.

The slope of any individual harped strands shall not be steeper than 8 horizontal to 1 vertical for 0.6” diameter strands, and 6 horizontal to 1 vertical for 0.5” diameter strands.

The harped strand exit location at the girder ends shall be held as low as possible while maintaining the concrete stresses within allowable limits.

  3. **Temporary Strands** – Temporary strands in the top flanges of prestressed concrete girders may be required for shipping (see Section 5.6.3). These strands may be pretensioned and bonded only for the end 10 feet of the girder, or may be post-tensioned prior to lifting the girder from the form. These strands can be considered in design to reduce the required transfer strength, to provide stability during shipping, and to reduce the “A” dimension. These strands must be cut before the CIP intermediate diaphragms are placed.

D. **Development of Prestressing Strand** –

  1. **General** – Development of prestressing strand shall be as described in AASHTO LRFD 5.11.4.

      The development length of bonded uncoated & coated prestressing strands are shown in Appendix 5.1-A8.

  2. **Partially Debonded Strands** – Where it is necessary to prevent a strand from actively supplying prestress force near the end of a girder, it shall be debonded. This can be accomplished by taping a close fitting PVC tube to the stressed strand from the end of the girder to some point where the strand can be allowed to develop its load. Since this is not a common procedure, it shall be carefully detailed on the plans. It is important when this method is used in construction that the taping of the tube is done in such a manner that concrete cannot leak into the tube and provide an undesirable bond of the strand.
3. **Strand Development Outside of Prestressed Concrete Girders** –

Extended bottom prestress strands are used to connect the ends of girders with diaphragms and resist loads from creep effects, shrinkage effects, and positive moments.

Extended strands must be developed in the short distance within the diaphragm (between two girder ends at intermediate piers). This is normally accomplished by requiring strand chucks and anchors as shown in Figure 5.1.3-1. Strand anchors are normally installed at 1’-9” from the girder ends.

The designer shall calculate the number of extended straight strands needed to develop the required capacity at the end of the girder. The number of extended strands shall not be less than four.

For fixed intermediate piers at the Extreme Event I limit state, the total number of extended strands for each girder end shall not be less than:

\[
N_{ps} = 12[M_{sei} \cdot K - M_{SIDL}] \cdot \frac{1}{0.9\phi A_{ps}f_{py}d} \tag{5.1.3-1}
\]

Where:
- \(M_{sei}\) = Moment due to overstrength plastic moment capacity of the column and associated overstrength plastic shear, either within or outside the effective width, per girder, kip-ft
- \(M_{SIDL}\) = Moment due to superimposed dead loads (traffic barrier, sidewalk, etc.) per girder, kip-ft
- \(K\) = Span moment distribution factor as shown in Figure 5.1.3-2 (use maximum of K1 and K2)
- \(A_{ps}\) = Area of each extended strand, in²
- \(f_{py}\) = Yield strength of prestressing steel specified in AASHTO LRFD Table 5.4.1.1-1, ksi
- \(d\) = Distance from top of deck slab to c.g. of extended strands, in
- \(\phi\) = Flexural resistance factor, 1.0

The plastic hinging moment at the c.g. of the superstructure is calculated using the following:

\[
M_{po}^{CG} = M_{po}^{top} + \frac{\left(M_{po}^{top} + M_{po}^{base}\right)}{L_c}h \tag{5.1.3-2}
\]

Where:
- \(M_{po}^{top}\) = Plastic overstrength moment at top of column, kip-ft
- \(M_{po}^{base}\) = Plastic overstrength moment at base of column, kip-ft
- \(h\) = Distance from top of column to c.g. of superstructure, ft
- \(L_c\) = Column clear height used to determine overstrength shear associated with the overstrength moments, ft
For prestressed concrete girders with cast-in-place deck slabs, where some girders are outside the effective widths and the effective widths for each column do not overlap, two-thirds of the plastic hinging moment at the c.g. of the superstructure shall be resisted by girders within the effective width. The remaining one-third shall be resisted by girders outside the effective width. The plastic hinging moment per girder is calculated using the following:

\[ M_{\text{set}}^{\text{int}} = \frac{2M_{pG}}{3N_{g}^{\text{int}}} \text{ For girders within the effective width} \]  \hfill (5.1.3-3)

\[ M_{\text{set}}^{\text{ext}} = \frac{M_{pG}}{3N_{g}^{\text{ext}}} \text{ For girders outside the effective width} \]  \hfill (5.1.3-4)

\[ \text{If } M_{\text{set}}^{\text{int}} \geq M_{\text{set}}^{\text{ext}} \text{ then } M_{\text{set}} = M_{\text{set}}^{\text{int}} \]  \hfill (5.1.3-5)

\[ \text{If } M_{\text{set}}^{\text{int}} < M_{\text{set}}^{\text{ext}} \text{ then } M_{\text{set}} = \frac{M_{pG}}{N_{g}^{\text{int}} + N_{g}^{\text{ext}}} \]  \hfill (5.1.3-6)

Where:
\[ N_{g}^{\text{int}} = \text{ Number of girders encompassed by the effective width} \]
\[ N_{g}^{\text{ext}} = \text{ Number of girders outside the effective width} \]

For prestressed concrete girders with cast-in-place deck slabs, where all girders are within the effective width or the effective widths for each column overlap, the plastic hinging moment at the c.g. of the superstructure shall be resisted by all girders within the effective width. The plastic hinging moment per girder is calculated using the following:

\[ M_{\text{set}} = \frac{M_{pG}}{N_{g}^{\text{int}}} \]  \hfill (5.1.3-7)

The effective width for the extended strand calculation shall be taken as:

\[ B_{\text{eff}} = D_{c} + D_{s} \]  \hfill (5.1.3-8)

Where:
\[ D_{c} = \text{ Diameter or width of column, see Figure 5.1.3-3} \]
\[ D_{s} = \text{ Depth of superstructure from top of column to top of deck slab, see Figure 5.1.3-3} \]

See Appendix 5-B10 for a design example.
Strand Development

*Figure 5.1.3-1*

\[ K_1 = \frac{L_2}{L_1 + L_2} \quad K_2 = \frac{L_1}{L_1 + L_2} \]

Extended Strand Design

*Figure 5.1.3-2*

Effective Superstructure Width for Extended Strand Design

*Figure 5.1.3-3*
Continuity of extended strands is essential for all prestressed concrete girder bridges with fixed diaphragms at intermediate piers. Strand continuity may be achieved by directly overlapping extended strands as shown in Figure 5.1.3-4, by use of strand ties as shown in Figure 5.1.3-5, by the use of the crossbeam ties as shown in Figure 5.1.3-6 along with strand ties, or by a combination of all three methods. The following methods in order of hierarchy shall be used for all prestressed concrete girders for creating continuity of extended strands:

**Method 1** – Direct extended strands overlapping shall be used at intermediate piers without any angle point due to horizontal curvature and for any crossbeam width. This is the preferred method of achieving extended strand continuity. Congestion of reinforcement and girder setting constructability shall be considered when large numbers of extended strands are required. In these cases, strand ties may be used in conjunction with extended strands.

**Method 2** – Strand ties shall be used at intermediate piers with a girder angle point due to horizontal curvature where extended strands are not parallel and would cross during girder placement. Crossbeam widths shall be greater than or equal to 6 ft measured along the skew. It is preferable that strand ties be used for all extended strands, however if the region becomes too congested for rebar placement and concrete consolidation, additional forces may be carried by crossbeam ties up to a maximum limit as specified in equation 5.1.3-8.

**Method 3** – For crossbeams with widths less than 6’ and a girder angle point due to horizontal curvature, strand ties shall be used if a minimum of 8” of lap can be provided between the extended strand and strand tie. In this case the strand ties shall be considered fully effective. For cases where less than 8” of lap is provided, the effectiveness of the strand tie shall be reduced proportional to the reduction in lap. All additional forces not taken by strand ties must be carried by crossbeam ties up to the maximum limit as specified in equation 5.1.3-8. If this limit is exceeded, the geometry of the width of the crossbeam shall be increased to provide sufficient lap for the strand ties.

The area of transverse ties considered effective for strand ties development in the lower crossbeam ($A_s$) shall not exceed:

$$A_s = \frac{1}{2} \left( \frac{A_{ps} f_{ps} n_s}{f_{ye}} \right)$$

(5.1.3-9)

Where:
- $A_{ps}$ = Area of strand ties, $\text{in}^2$
- $f_{ps}$ = Yield strength of extended strands, ksi
- $n_s$ = Number of extended strands that are spliced with strand and crossbeam ties
- $f_{ye}$ = Expected yield strength of transverse tie reinforcement, ksi

Two-thirds of $A_s$ shall be placed directly below the girder and the remainder of $A_s$ shall be placed outside the bottom flange width as shown in Figure 5.1.3-6.

The size of strand ties shall be the same as the extended strands, and shall be placed at the same level and proximity of the extended strands.
Overlapping Extended Strand

Figure 5.1.3-4

Strand Ties

Figure 5.1.3-5
5.1.4 Prestress Losses

AASHTO LRFD Specifications outline the method of predicting prestress losses for usual prestressed concrete bridges that shall be used in design except as noted below.

A. Instantaneous Losses

1. Elastic Shortening of Concrete – Transfer of prestress forces into the prestressed concrete girder ends results in an instantaneous elastic loss. The prestress loss due to elastic shortening shall be added to the time dependent losses to determine the total losses. The loss due to elastic shortening shall be taken as per AASHTO LRFD 5.9.5.2.3.

For pretensioned member and low-relaxation strands, \( f_{cgp} \) may be calculated based on 0.7\( f_{pu} \). For post-tensioned members with bonded tendons, \( f_{cgp} \) may be calculated based on prestressing force after jacking at the section of maximum moment.

2. Anchorage Set Loss – The anchor set loss shall be based on \( \frac{3}{8} \)" slippage for design purposes. Anchor set loss and the length affected by anchor set loss is shown in Figure 5.1.4-1.

\[
\Delta f_{pa} = \frac{2x (p_{j-left} - p_{j-right})}{A_{pfl}}
\]  
(5.1.4-2)
3. **Friction Losses** – Friction losses occurring during jacking and prior to anchoring depend on the system and materials used. For a rigid spiral galvanized ferrous metal duct system, \( \mu \) shall be 0.20 and \( K = 0.0002 \). For plastic ducts, the designer shall use the values shown in AASHTO LRFD Table 5.9.5.2.2b.

To avoid the substantial friction loss caused by sharp tendon curvature in the end regions where the tendons flare out from a stacked arrangement towards the bearing plates, use 0.10 times the span length or 20 feet as the minimum flare zone length. The recommended minimum radius (horizontal or vertical) of flared tendons is 200 feet. In the special cases where sharp curvature cannot be avoided, extra horizontal and vertical ties shall be added along the concave side of the curve to resist the tendency to break through the web.

\[
\Delta f_{PF} = f_{pj}(1 - e^{-(kx + \mu)})
\]  

(5.1.4-3)

When summing the \( \alpha \) angles for total friction loss along the structure, horizontal curvature of the tendons as well as horizontal and vertical roadway curvature shall be included in the summation. The \( \alpha \) angles for horizontally and vertically curved tendons are shown in Figure 5.1.4-2.
\[ \alpha = \sqrt{(\alpha_H)^2 + (\alpha_V)^2} \]

where: \[ \alpha_V = \frac{2\delta}{L} \]

\[ \alpha_H = \frac{S}{R} \]

The \( \alpha \) Angles for Curved PT Tendons

**Figure 5.1.4-2**

B. **Approximate Estimate of Time-Dependent Losses** – The Approximate Estimate of Time-Dependent Losses of AASHTO LRFD 5.9.5.3 may be used for preliminary estimates of time-dependent losses for prestressed concrete girders with composite decks as long as the conditions set forth in AASHTO are satisfied.

C. **Refined Estimates of Time-Dependent Losses** – Final design calculations of time-dependent prestress losses shall be based on the Refined Estimates of Time-Dependent Losses of AASHTO LRFD 5.9.5.4.

D. **Total Effective Prestress** – For standard precast, pretensioned members with CIP deck subject to normal loading and environmental conditions and pretensioned with low relaxation strands, the total effective prestress may be estimated as:

\[ f_{pe} = f_{pj} - \Delta f_{pLT} - \Delta f_{pES} - \Delta f_{pED} - \Delta f_{pSS} \]  \hfill (5.1.4-4)

The total prestress loss may be estimated as:

\[ \Delta f_{pLT} = \Delta f_{pRO} + \Delta f_{pLT} \]  \hfill (5.1.4-5)

Initial relaxation that occurs between the time of strand stressing and prestress transfer may be estimated as:

\[ \Delta f_{pRO} = \frac{0.40}{40} \left( \frac{f_{pj}}{f_{py}} - 0.55 \right) f_{pj} \]  \hfill (5.1.4-6)

Where:

- \( t \) = Duration of time between strand stressing and prestress transfer, typically 1 day.
- \( f_{pj} \) = Jacking stress
- \( f_{py} \) = Yield strength of the strand

Long term time dependent losses, \( \Delta f_{pLT} \), are computed in accordance with the refined estimates of AASHTO LRFD 5.9.5.4 or a detailed time-step method. Elastic gain due to deck shrinkage shall be considered separately.

Elastic shortening, \( \Delta f_{pES} \), is computed in accordance with AASHTO LRFD 5.9.5.2.3a.
The elastic gain due to deck placement, superimposed dead loads and live loads is taken to be:

\[ \Delta f_{pED} = \frac{E_p}{E_c} \left[ \frac{(M_{slab}+M_{diaphragms})e_{ps}}{I_g} - \frac{(M_{sidl}+\gamma_{LL}M_{LL}+IM)(Y_{bc}-Y_{bg}+e_{ps})}{I_c} \right] \quad (5.1.4-7) \]

Where:

- \( E_p \) = Modulus of elasticity of the prestressing strand
- \( E_c \) = Modulus of elasticity of the concrete at the time of loading
- \( M_{slab} \) = Moment caused by deck slab placement
- \( M_{diaphragms} \) = Moment caused by diaphragms and other external loads applied to the non-composite girder section
- \( M_{sidl} \) = Moment caused by all superimposed dead loads including traffic barriers and overlays
- \( M_{LL}+IM \) = Moment caused by live load and dynamic load allowance
- \( \gamma_{LL} \) = Live load factor (1.0 for Service I and 0.8 for Service III)
- \( e_{ps} \) = Eccentricity of the prestressing strand
- \( I_g \) = Moment of inertia of the non-composite girder
- \( I_c \) = Moment of inertia of the composite girder
- \( Y_{bg} \) = Location of the centroid of the non-composite girder measured from the bottom of the girder
- \( Y_{bc} \) = Location of the centroid of the composite girder measured from the bottom of the girder

The elastic gain due to slab shrinkage, \( \Delta f_{pSS} \), shall be computed in accordance with AASHTO LRFD 5.9.5.4.3d. Deck shrinkage shall be considered as an external force applied to the composite section for the Service I, Service III, and Fatigue I limit states. This force is applied at the center of the deck with an eccentricity from the center of the deck to the center of gravity of the composite section. This force causes compression in the top of the girder, tension in the bottom of the girder, and an increase in the effective prestress force (an elastic gain). The deck shrinkage strain shall be computed as 50% of the strain determined by AASHTO LRFD Equation 5.4.2.3.3-1.

**E. Temporary Losses** – For checking stresses during release, lifting, transportation, and erection of prestressed concrete girders, the elastic and time-dependent losses may be computed based on the following assumptions.

1. **Lifting of Girders From Casting Beds** – For normal construction, forms are stripped and girders are lifted from the casting bed within one day.

2. **Transportation** – Girders are most difficult to transport at a young age. The hauling configuration causes reduced dead load moments in the girder and the potential for overstress between the harping points. Overstress may also occur at the support points depending on the prestressing and the trucking configuration. This is compounded by the magnitude of the prestress force not having been reduced by losses. For an aggressive construction schedule girders are typically transported to the job site around day 10.
When losses are estimated by the Approximate Estimate of AASHTO LRFD 5.9.5.3, the losses at the time of hauling may be estimated by:

\[ \Delta f_{pTH} = \Delta f_{pRO} + \Delta f_{pES} + \Delta f_{pH} \]  

(5.1.4-8)

Where:
- \( \Delta f_{pTH} \) = total loss at hauling
- \( \Delta f_{pH} \) = time dependent loss at time of hauling =

\[ 3 \frac{f_{p1} A_{ps}}{\Delta_s} \gamma_h Y_{st} + 3 \gamma_h Y_{st} + 0.6 \]

3. **Erection** – During construction the non-composite girders must carry the full weight of the deck slab and interior diaphragms. This loading typically occurs around 120 days for a normal construction schedule.

4. **Final Configuration** – The composite slab and girder section must carry all conceivable loads including superimposed dead loads such as traffic barriers, overlays, and live loads. It is assumed that superimposed dead loads are placed at 120 days and final losses occur at 2,000 days.

### 5.1.5 Prestressing Anchorage Systems

There are numerous prestressing systems. Most systems combine a method of prestressing the strands with a method of anchoring it to concrete.

WSDOT requires approval of all multi-strand and/or bar anchorages used in prestressed concrete bridges as described in *Standard Specifications* 6-02.3(26).

### 5.1.6 Post-Tensioning Ducts

Post-tensioning ducts shall meet the requirements of *Standard Specifications* 6-02.3(26)E.

Ducts for longitudinal post-tensioning tendons in spliced prestressed concrete I-girders shall be made of rigid galvanized spiral ferrous metal to maintain standard girder concrete cover requirements.
5.2 Design Considerations

5.2.1 Service and Fatigue Limit States

A. General – Service limit state is used to satisfy allowable stresses, deflection, and control of cracking requirements. Design aids for tensile stress in reinforcement at the service limit state, $f_{ss}$, are provided in Appendices 5.2-A1, 5.2-A2, and 5.2-A3.

B. Control of Cracking – Reinforcement shall be provided and spaced to meet the requirements in AASHTO LRFD 5.7.3.4 “Control of Cracking by Distribution of Reinforcement.” The exposure factor shall be based upon a Class 2 exposure condition.

C. Allowable Stresses in Prestressed Concrete Members – Under service limit state the tensile stresses in the precompressed tensile zone shall be limited to zero. This prevents cracking of the concrete during the service life of the structure and provides additional stress and strength capacity for overloads. Allowable concrete stresses for the service and fatigue limit states are shown in Table 5.2.1-1.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Stress</th>
<th>Location</th>
<th>Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary Stress at Transfer and at Lifting from Casting Bed</td>
<td>Tensile</td>
<td>In areas other than the precompressed tensile zone and without bonded reinforcement</td>
<td>$0.0948 \sqrt{f'_{ct}} \leq 0.2 \text{ (ksi)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>In areas with bonded reinforcement sufficient to resist tensile force in the concrete</td>
<td>$0.19 \sqrt{f'_{ct}} \text{ (ksi)}$</td>
</tr>
<tr>
<td>Compressive</td>
<td>All Locations</td>
<td></td>
<td>$0.65 f'_{ct}$</td>
</tr>
<tr>
<td>Temporary Stress at Shipping and Erection</td>
<td>Tensile</td>
<td>In areas other than the precompressed tensile zone and without bonded reinforcement</td>
<td>$0.0948 \sqrt{f'_{ct}} \text{ (ksi)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>In areas other than the precompressed tensile zone and with bonded reinforcement, plumb girder with impact</td>
<td>$0.19 \sqrt{f'_{ct}} \text{ (ksi)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>In areas other than the precompressed tensile zone and with bonded reinforcement, inclined girder without impact</td>
<td>$0.24 \sqrt{f'_{ct}} \text{ (ksi)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>In areas other than the precompressed tensile zone and with bonded reinforcement, after temporary top strand detensioning</td>
<td>$0.19 \sqrt{f'_{ct}} \text{ (ksi)}$</td>
</tr>
<tr>
<td>Compressive</td>
<td>All locations</td>
<td></td>
<td>$0.65 f'_{ct}$</td>
</tr>
<tr>
<td>Final Stresses at Service Load</td>
<td>Tensile</td>
<td>Precompressed tensile zone</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Compressive</td>
<td>Effective prestress and permanent loads</td>
<td>$0.45 f'_{ct}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Effective prestress, permanent loads and transient loads</td>
<td>$0.60 f'_{ct}$</td>
</tr>
<tr>
<td>Final Stresses at Fatigue Load</td>
<td>Compressive</td>
<td>Fatigue I Load Combination plus one-half effective prestress and permanent loads per AASHTO LRFD 5.5.3.1</td>
<td>$0.40 f'_{ct}$</td>
</tr>
</tbody>
</table>

Allowable Stresses in Prestressed Concrete Members

*Table 5.2.1-1*
5.2.2 Strength-Limit State

A. Flexure – Design for flexural force effects shall be per AASHTO LRFD 5.7.

For prestressed concrete girders, the approximate methods of AASHTO LRFD 5.7.3 underestimate the flexural strength of the composite deck-girder system. Strain compatibility approaches such as the PCI Bridge Design Manual method (PCI BDM Section 8.2.2.5) and the Nonlinear Strain Compatibility Analysis method in the PCI Journal are recommended. In addition to the effective area of the deck, the top flange of the girder and the mild reinforcement in the deck and the top flange of the girder may be included in the analysis.

The typical section for computation of prestressed concrete girder composite section properties is shown in Figure 5.6.2-1.

1. Flexural Design of Nonprestressed Singly-Reinforced Rectangular Beams – For design purposes, the area of reinforcement for a nonprestressed singly-reinforced rectangular beam or slab can be determined by letting:

\[ M_u = \phi M_n = \phi A_s f_y \left( d - \frac{a}{\alpha} \right) \]  \hspace{1cm} (5.2.2-1)

However, if:

\[ a = \frac{A_s f_y}{\alpha f_c' b} \]  \hspace{1cm} (5.2.2-2)

Equation (2) can be substituted into equation (1) and solved for \( A_s \):

\[ A_s = \left( \frac{\alpha f_c' b}{f_y} \right) \left[ d - \sqrt{d^2 - \frac{2M_u}{\alpha f_c' b \Phi}} \right] \]  \hspace{1cm} (5.2.2-3)

Where:

- \( A_s \) = Area of tension reinforcement (in²)
- \( M_u \) = Factored moment (kip·in)
- \( f_c' \) = Specified compressive strength of concrete (ksi)
- \( f_y \) = Specified minimum yield strength of tension reinforcement (ksi)
- \( b \) = Width of the compression face (in)
- \( d \) = Distance from compression face to centroid of tension reinf. (in)
- \( \Phi \) = 0.9
- \( \alpha \) = From AASHTO LRFD 5.7.2.2

The resistance factor should be assumed to be 0.9 for a tension-controlled section for the initial determination of \( A_s \). This assumption must then be verified by checking that the tensile strain in the extreme tension steel is equal to or greater than 0.005. This will also assure that the tension reinforcement has yielded as assumed.

\[ \epsilon_t = 0.003 \left( \frac{d - c}{c} \right) \geq 0.005 \]  \hspace{1cm} (5.2.2-4)

Where:

- \( \epsilon_t \) = Tensile strain in the extreme tension steel
- \( d_t \) = Distance from extreme compression fiber to centroid of extreme tension reinforcement (in)
- \( c \) = \( \frac{A_s f_y}{\alpha f_c' b \beta_1} \)
- \( \beta_1 \) = From AASHTO LRFD 5.7.2.2
B. **Shear** – AASHTO LRFD 5.8 addresses shear design of concrete members.

1. The shear design of prestressed members shall be based on the general procedure of AASHTO LRFD 5.8.3.4.2.

2. The shear design of all non-prestressed members shall be based on either the general procedure, or the simplified procedure of AASHTO LRFD 5.8.3.4.1.

3. The strut-and-tie model shall be employed as required by AASHTO LRFD 5.8.1.1 & 2 for regions adjacent to abrupt changes in cross-section, openings, draped ends, deep beams, corbels, integral bent caps, c-bent caps, outrigger bents, deep footings, pile caps, etc.

4. AASHTO LRFD 5.8.3.4.3 "Simplified Procedure for Prestressed and Nonprestressed Sections" shall not be used.

5. The maximum spacing of transverse reinforcement is limited to 18 inches.

For prestressed concrete girders, shear for the critical section at $d_v$ from the internal face of the support and at the harping point are of particular interest.

C. **Interface Shear** – Interface shear transfer (shear friction) design is to be performed in accordance with AASHTO LRFD 5.8.4.

If a roughened surface is required for shear transfer at construction joints in new construction, they shall be identified in the plans. See *Standard Specifications* Section 6-02.3(12)A.

When designing for shear transfer between new and existing concrete, the designer shall consider the high construction cost associated with roughening existing concrete surfaces. Whenever practical, the design for placing new concrete against existing concrete shall be completed such that roughening of the existing concrete surfaces is not required (i.e. use cohesion and friction factors for a surface that is not intentionally roughened).

When the additional capacity provided by a roughened surface is required, the surface roughening shall meet the requirements specified in AASHTO LRFD 5.8.4.3 (i.e. uniform ¼" minimum amplitude). See *Standard Specifications* Section 6-02.3(12)B and applicable WSDOT special provisions for concrete removal for reference.

The spall pattern roughening detail shown in Figure 5.2.2-1 may be included on plans as an alternative to the default uniform ¼” amplitude roughening.
Interface shear in prestressed concrete girder design is critical at the interface connection between deck slab and girder, and at the end connection of the girder to a diaphragm or crossbeam. Shear in these areas is resisted by roughened or saw-tooth shear keyed concrete as well as reinforcement extending from the girder.

1. **Interface Shear Between Deck Slab and Girder** – The top surfaces of prestressed concrete girders with cast-in-place decks shall be roughened as described in Standard Specifications Section 6-02.3(25)H. The interface shear is resisted by the girder stirrups which extend up into the deck slab as well as the roughened top surface of the girder top flange.

   It is conservative to compute the interface shear force using the full factored loading applied to the composite deck slab and girder. However, the interface shear force need only be computed from factored loads applied to the composite section after the deck slab is placed such as superimposed dead loads and live loads.

   For SIP deck systems, only the roughened top flange surface between SIP panel supports (and the portion of the permanent net compressive force $P_c$ on that section) is considered engaged in interface shear transfer.

2. **Interface Shear Friction at Girder End** – A prestressed concrete girder may be required to carry shears at the end surface of the girder.

   An end condition at an intermediate pier crossbeam is shown in Figure 5.2.2-2. The shear which must be carried along the interface A-A is the actual factored shear acting on the section. The portion of the girder end that is roughened with saw-toothed shear keys shown on the standard girder plans may be considered as a “surface intentionally roughened to an amplitude of 0.25 inches”. Shear
resistance must be developed using interface shear theory assuming the longitudinal bars and the extended strands are actively participating. The main longitudinal deck slab reinforcement is already fully stressed by negative bending moments and thus cannot be considered for shear requirements. All bars, including the extended strands, must be properly anchored in order to be considered effective. This anchorage requirement must be clearly shown on the plans.

Similar requirements exist for connecting the end diaphragm at bridge ends where the diaphragm is cast on the girders (girder End Type A). In this case, however, loads consist only of the factored diaphragm dead load, approach slab dead load, and those wheel loads which can distribute to the interface. Longitudinal reinforcement provided at girder ends shall be identical in both ends of the girder for construction simplicity.

The program PGSuper does not check interface shear friction at girder ends. Standard girder plan details are adequate for girder End Types A and B. Standard girder plan details shall be checked for adequacy for girder End Types C and D.
D. **Shear and Torsion** – The design for shear and torsion is based on ACI 318-02 Building Code Requirements for Structural Concrete and Commentary (318F-02) and is satisfactory for bridge members with dimensions similar to those normally used in buildings. AASHTO LRFD 5.8.3.6 may also be used for design.

According to Hsu, utilizing ACI 318-02 is awkward and overly conservative when applied to large-size hollow members. Collins and Mitchell propose a rational design method for shear and torsion based on the compression field theory or strut-and-tie method for both prestressed and non-prestressed concrete beams. These methods assume that diagonal compressive stresses can be transmitted through cracked concrete. Also, shear stresses are transmitted from one face of the crack to the other by a combination of aggregate interlock and dowel action of the stirrups.

For recommendations and design examples, the designer can refer to the paper by M.P. Collins and D. Mitchell, Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams, PCI Journal, September-October 1980, pp. 32-100.

**5.2.3 Strut-and-Tie Model**

Strut-and-tie models shall be used near regions of discontinuity or where beam theory is not applicable. Design and detailing considerations for strut-and-tie modeling is covered in AASHTO LRFD 5.6.3. See Appendix 5-B for a strut-and-tie design example for a pier cap.

**5.2.4 Deflection and Camber**

A. **General** – Flexural members are designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load plus impact. The minimum superstructure depths are specified in AASHTO LRFD Table 2.5.2.6.3-1 and deflections shall be computed in accordance with AASHTO LRFD 5.7.3.6.2.

Accurate predictions of deflections are difficult to determine, since modulus of elasticity of concrete, $E_c$, varies with stress and age of concrete. Also, the effects of creep on deflections are difficult to estimate. For practical purposes, an accuracy of 10 to 20 percent is often sufficient. Prestressing can be used advantageously to control deflections; however, there are cases where excessive camber due to prestress has caused problems.

B. **Preliminary Estimate for Prestressed Concrete Members** – For preliminary design, the long term deflection and camber of prestressed concrete members may be estimated using the procedure given in the PCI Design Handbook 10 4.8.4.

C. **Deflection Calculation for Prestressed Concrete Girders** – The “$D$” dimension is the computed girder deflection at midspan (positive upward) immediately prior to deck slab placement.

*Standard Specifications* Section 6-02.3(25)K defines two levels of girder camber at the time the deck concrete is placed, denoted $D @ 40$ Days and $D @ 120$ Days. They shall be shown in the plans to provide the contractor with lower and upper bounds of camber that can be anticipated in the field.
PGSuper calculates estimated cambers at 40 days ($D_{40}$) and 120 days ($D_{120}$). Due to variations in observed camber, these estimated cambers are generally considered to be upper bounds at their respective times. This is based on measured girder cambers of prestressed concrete girders compared with the estimated cambers from PGSuper.

$D_{@120}$ Days is the upper bound of expected camber range at a girder age of 120 days after the release of prestress and is primarily intended to mitigate interference between the top of the cambered girder and the placement of concrete deck reinforcement. It is also used to calculate the “A” dimension at the girder ends. The age of 120 days was chosen because data has shown that additional camber growth after this age is negligible. $D_{@120}$ Days may be taken as $D_{120}$, the estimated camber at 120 days reported by PGSuper.

$D_{@40}$ Days is the lower bound of expected camber range at a girder age of 40 days (30 days after the earliest allowable girder shipping age of 10 days). To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths, such as the WF series, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the contractor to assess the risk of increased concrete quantities and mitigates claims for additional material. $D_{@40}$ Days shall be taken as 50 percent of $D_{40}$, the estimated camber at 40 days reported by PGSuper.

Figure 5.2.4-1 shows a typical pattern of girder deflection with time at centerline span. Portions of this characteristic curve are described below. The subparagraph numbers correspond to circled numbers on the curve.

1. **Elastic Deflection Due to Release of Prestress** – The prestress force produces moments in the girder tending to bow the girder upward. Resisting these moments are girder section dead load moments. The result is a net upward deflection.

2. **Creep Deflection Before Cutting Temporary Strands** – The girder continues to deflect upward due to the effect of creep. This effect is computed using the equation stated in Section 5.1.1E.

3. **Deflection Due to Cutting of Temporary Strands** – Cutting of temporary strands results in an elastic upward deflection. The default time interval for creep calculations for release of top temporary strands is 90 days after the release of prestress during girder fabrication for $D_{120}$ (10 days for $D_{40}$).

4. **Diaphragm Load Deflection** – The load of diaphragm is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing diaphragms is 90 days after the release of prestress during girder fabrication for $D_{120}$ (10 days for $D_{40}$).

5. **Creep Deflection After Casting Diaphragms** – The girder continues to deflect upward for any time delay between diaphragms and deck slab casting.
6. **Deck Slab Load Deflection** – The load of the deck slab is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing the deck slab is 120 days after the release of prestress during girder fabrication for $D_{120}$ (40 days for $D_{40}$).

7. **Superimposed Dead Load Deflection** – The load of the traffic barriers, sidewalk, overlay, etc. is applied to the composite girder section resulting in an elastic downward deflection.

8. **Final Camber** – It might be expected that the above deck slab dead load deflection would be accompanied by a continuing downward deflection due to creep. However, many measurements of actual structure deflections have shown that once the deck slab is poured, the girder tends to act as though it is locked in position. To obtain a smooth riding surface on the deck, the deflection indicated on Figure 5.2.4-1 as “Screed Camber” (known as “C”) is added to the profile grade elevation of the deck screeds. The “C” dimension and the “Screed Setting Dimensions” detail shall be given in the plans.

D. **Pre-camber** – Prestressed concrete girders may be precambered to compensate for the natural camber and for the effect of the roadway geometry.

![Prestressed Concrete Girder Camber](Figure 5.2.4-1)
5.2.5 Construction Joints

Construction joints must divide the structure into a logical pattern of separate elements which also permit ease of manufacture and assembly.

The joint surfaces should be oriented perpendicular to the centroidal axis of the precast element.

A. Types of Joints

Joints are either wide or match cast. Depending on their width, they may be filled with CIP concrete or grout. Match cast joints are normally bonded with an epoxy bonding agent. Dry match cast joints are not recommended.

B. Shear Keys

In order to assist shear transmission in wide joints, use a suitable system of keys. The shape of the keys may be chosen to suit a particular application and they can be either single keys or multiple keys. Single keys are generally large and localized whereas multiple keys generally cover as much of the joint surface area as is practical.

Single keys provide an excellent guide for erection of elements. Single keys are preferred for all match cast joints.

For all types of joints, the surfaces must be clean, free from grease and oil, etc. When using epoxy for bonding, the joints shall be lightly sandblasted to remove laitance. For CIP or other types of wide joints, the adjacent concrete surfaces shall be roughened and kept thoroughly wet, prior to construction of the joint. CIP joints are generally preferred.

5.2.6 Inspection Lighting and Access

A. Confined Spaces – See Section 10.8.1 for design requirements for confined spaces.

B. Access Hatch, Air Vent Holes and Inspection Lighting

Box girders with inside clear height of less than or equal to 4 feet do not require access, lighting, receptacles and ventilation. Utilities, longitudinal restrainers and other components requiring inspection or maintenance are not permitted inside the box girder cells.

Box girders with inside clear height greater than 4 feet but less than 6.5 feet shall have access, lighting, receptacles and ventilation provided inside each box girder cell containing utilities, longitudinal restrainers and other components requiring inspection or maintenance.

Box girders with inside clear height greater than or equal to 6.5 feet shall have access, lighting, receptacles and ventilation provided inside.

Access, lighting, receptacles and ventilation shall not be provided inside prestressed concrete tub girder cells. Utilities, longitudinal restrainers and other components requiring inspection or maintenance are not permitted inside the girder cells.
Access doors shall have a minimum 2’-6” diameter or 2’-6” square clear opening. Lock box latches shall be installed on all access doors accessible from ground level. Access hatches shall swing into the box girders and shall be placed at locations that do not impact traffic. Lighting and receptacle requirements shall conform to WSDOT Design Manual Chapter 1040. Air vents shall conform to Figures 5.2.6-1 and 5.2.6-2.

Box girder penetrations greater than one inch in diameter through the exterior shall be covered with galvanized wire mesh screen to prevent vermin and birds from accessing the penetration and the interior of the box girder. The wires shall have a maximum spacing of 1 inch in both directions.
Access Hatch Details

Figure 5.2.6-1
Air Vent Opening Detail

*Figure 5.2.6-2*
Chapter 5 Concrete Structures

5.3 Reinforced Concrete Box Girder Bridges

Post-tensioning shall be required for all new CIP reinforced concrete single-span or multi-span box girder bridges.

The use of CIP reinforced concrete (RC) box girder bridges without post-tensioning shall be restricted to widening existing RC box girder bridges. RC box girder bridges may also be used for bridges with tight curvatures or irregular geometry upon the WSDOT Bridge Design Engineer’s approval. Partial prestressing shall not be considered for design of RC box girders.

The performance and longevity of RC box girder bridges have been a major concern. Cracking in RC box girders is flexural in nature and are an inherent part of reinforced concrete design. RC box girders are designed for ultimate strength and checked for distribution of reinforcement for service conditions and control of cracking. This means that the concrete cracks under applied loads but the cracks are under control. Open cracks in RC box girders result in rebar corrosion and concrete deterioration, affecting the bridge longevity. Post-tensioning RC box girders eliminates cracks, limits corrosion, and improves structural performance.

The above requirements apply equally to RC T-beam and slab bridges. However, these types of superstructures are not encouraged. See also Sections 2.4.1.C & D.

5.3.1 Box Girder Basic Geometries

A. Web Spacing – The most economical web spacing for ordinary box girder bridges varies from about 8 to 12 feet. Greater girder spacing requires some increase in both top and bottom slab thickness, but the cost of the additional concrete can be offset by decreasing the total number of girder stems. Fewer girder stems reduces the amount of form work required and can lower costs.

The number of girder stems can be reduced by cantilevering the top slab beyond the exterior girders. A deck overhang of approximately one-half the girder spacing generally gives satisfactory results. This procedure usually results in a more aesthetic as well as a more economical bridge.

For girder stem spacing in excess of 12 feet or cantilever overhang in excess of 6 feet, transverse post-tensioning shall be used.

B. Basic Dimensions – The basic dimensions for concrete box girders with vertical and sloped exterior webs are shown in Figures 5.3.1-1 & 2, respectively.

1. Top Slab Thickness, T1 – (includes \(\frac{1}{2}\)" wearing surface)

\[ T_1 = \frac{12(S + 10)}{30} \]

but not less than 7" with overlay or 7.5" without overlay.

2. Bottom Slab Thickness, T2 –

a. Near center span

\[ T_2 = \frac{12S_{etc}}{16} \]

but not less than 5.5" (normally 6.0" is used).

b. Near intermediate piers

Thickening of the bottom slab is often used in negative moment regions to control compressive stresses that are significant.

Transition slope = 24:1 (see T2 in Figure 5.3.1-1).
3. **Girder Stem (Web) Thickness, T3**
   a. **Near Center Span**
      Minimum T3 = 9.0” — vertical
      Minimum T3 = 10.0” — sloped
   b. **Near Supports** – Thickening of girder stems is used in areas adjacent to supports to control shear requirements.
      Changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.
      Maximum T3 = T3 + 4.0” maximum
      Transition length = 12 x (difference in web thickness)

4. **Intermediate Diaphragm Thickness, T4 and Diaphragm Spacing**
   a. For tangent and curved bridge with R > 800 feet
      T4 = 0” (diaphragms are not required.)
   b. For curved bridge with R < 800 feet
      T4 = 8.0”
      Diaphragm spacing shall be as follows:
      For 600’ < R < 800’ at ½ pt. of span.
      For 400’ < R < 600’ at ⅓ pt. of span.
      For R < 400’ at ¼ pt. of span.

C. **Construction Considerations** – Review the following construction considerations to minimize constructability problems:
   1. Construction joints at slab/stem interface or fillet/stem interface at top slab are appropriate.
   2. All construction joints to have roughened surfaces.
   3. Bottom slab is parallel to top slab (constant depth).
   4. Girder stems are vertical.
   5. Dead load deflection and camber to nearest ⅛”.
   6. Skew and curvature effects have been considered.
   7. Thermal effects have been considered.
   8. The potential for falsework settlement is acceptable. This always requires added stirrup reinforcement in sloped outer webs.
D. **Load Distribution**

1. **Unit Design** – According to the AASHTO LRFD Specifications, the entire slab width shall be assumed effective for compression. It is both economical and desirable to design the entire superstructure as a unit rather than as individual girders. When a reinforced box girder bridge is designed as an individual girder with a deck overhang, the positive reinforcement is congested in the exterior cells. The unit design method permits distributing all girder reinforcement uniformly throughout the width of the structure.

2. **Dead Loads** – Include additional D.L. for top deck forms:
   - 5 lbs. per sq. ft. of the area.
   - 10 lbs. per sq. ft. if web spacing > 10′-0”.

3. **Live Load** – See Section 3.9.4 for live load distribution to superstructure and substructure.
Basic Dimensions - Vertical Webs

*Figure 5.3.1-1*
Basic Dimensions - Sloped Webs

**Figure 5.3.1-2**

**Basic Dimensions - Sloped Webs**

*Dimensions are shown for demonstration only.*
5.3.2 Reinforcement

This section discusses flexural and shear reinforcement for top slab, bottom slab, webs, and intermediate diaphragms in box girders.

A. Top Slab Reinforcement

1. Near Center of Span – Figure 5.3.2-1 shows the reinforcement required near the center of the span and Figure 5.3.2-2 shows the overhang reinforcement.
   a. Transverse reinforcing in the top and bottom layers to transfer the load to the main girder stems.
   b. Bottom longitudinal “distribution reinforcement” in the middle half of the deck span in $S_{eff}$ is provided to aid distributing the wheel loads.
   c. Top longitudinal “temperature and shrinkage reinforcement.”

2. Near Intermediate Piers – Figure 5.3.2-3 illustrates the reinforcement requirement near intermediate piers.
   a. Transverse reinforcing same as center of span.
   b. Longitudinal reinforcement to resist negative moment (see Figure 5.3.2-3).
   c. “Distribution of flexure reinforcement” to limit cracking shall satisfy the requirement of AASHTO LRFD 5.7.3.4 for class 2 exposure condition.

3. Bar Patterns
   a. Transverse Reinforcement – It is preferable to place the transverse reinforcement normal to bridge center line and the areas near the expansion joint and bridge ends are reinforcement by partial length bars.
   b. Longitudinal Reinforcement

\[
P = \frac{220}{\sqrt{S}} \quad (\text{MAX. = .67})
\]

Partial Section Near Center of Span

*Figure 5.3.2-1*
Overhang Detail
*Figure 5.3.2-2*

Area "A"/2 Bars
= SQ. INCH PER BAR
A = $A_1 \times t_1$

* All rebars shall be epoxy coated, bend stirrups 135 degrees. Do not epoxy coat stirrups.

Top Slab Flexural Reinforcing Near Intermediate Pier
*Figure 5.3.2-3*

Partial Plans at Abutments
*Figure 5.3.2-4*
B. Bottom Slab Reinforcement

1. Near Center of Span – Figure 5.3.2-5 shows the reinforcement required near the center of the span.
   a. Minimum transverse “distributed reinforcement.”
      \[ A_s = 0.005 \times \text{flange area with } \frac{1}{2}A_s \text{ distributed equally to each surface.} \]
   b. Longitudinal “main reinforcement” to resist positive moment.
   c. Check “distribution of flexure reinforcement” to limit cracking per AASHTO LRFD 5.7.3.4 for class 2 exposure condition.
   d. Add steel for construction load (sloped outer webs).

2. Near Intermediate Piers – Figure 5.3.2-6 shows the reinforcement required near intermediate piers.
   a. Minimum transverse reinforcement same as center of span.
   b. Minimum longitudinal “temperature and shrinkage reinforcement.”
      \[ A_s = 0.004 \times \text{flange area with } \frac{1}{2}A_s \text{ distributed equally to each face.} \]
   c. Add steel for construction load (sloped outer webs).

3. Bar Patterns
   a. Transverse Reinforcement – All bottom slab transverse bars shall be bent at the outside face of the exterior web. For a vertical web, the tail splice will be 1′-0″ and for sloping exterior web 2′-0″ minimum splice with the outside web stirrups. See Figure 5.3.2-7.
   b. Longitudinal Reinforcement – For longitudinal reinforcing bar patterns, see Figures 5.3.2-5 & 6.

C. Web Reinforcement

1. Vertical Stirrups – Vertical stirrups for a reinforced concrete box section is shown in Figure 5.3.2-8.

   The web reinforcement shall be designed for the following requirements:
   Vertical shear requirements.
   • Out of plane bending on outside web due to live load on cantilever overhang.
   • Horizontal shear requirements for composite flexural members.
   • Minimum stirrups shall be:
     \[ \frac{A_v}{s} = 50 \frac{b_w}{f_y} \]  \hspace{1cm} (5.3.2-1)
     but not less than #5 bars at 1′-6″,
     Where: \( b_w \) is the number of girder webs x T3
2. **Web Longitudinal Reinforcement** – Web longitudinal reinforcement for reinforced concrete box girders is shown in Figures 5.3.2-8 & 9. The area of skin reinforcement $A_{sk}$ per foot of height on each side face shall be:

$$A_{sk} \geq 0.012(d - 30)$$  \hspace{1cm} (5.3.2-2)

Reinforcing steel spacing < Web thickness (T3) or 12”.

The maximum spacing of skin reinforcement shall not exceed the lesser of $d/6$ and 12”. Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one half of the required flexural tensile reinforcement.

For CIP sloped outer webs, increase inside stirrup reinforcement and bottom slab top transverse reinforcement as required for the web moment locked-in during construction of the top slab. This moment about the bottom corner of the web is due to tributary load from the top slab concrete placement plus 10 psf form dead load. See Figure 5.3.2-10 for typical top slab forming.

D. **Intermediate Diaphragm** – Intermediate diaphragms are not required for bridges on tangent alignment or curved bridges with an inside radius of 800 feet or greater.
Web Reinforcement

Figure 5.3.2-7
Use 2 #10 (2 #8 & 2 #6 for 10" or less webs) at dead load negative moment region near piers. The length of shall be extended 35 diameters beyond the dead load point of inflection. Do not splice the #10 (#8) bars near the pier. The #6 bars may be spliced at center pier. Use 2 #8 only for the positive moment region.

Web Reinforcement Details
Figure 5.3.2-8
1. Stirrup hanger must be placed above longitudinal steel when diaphragm is skewed and slab reinforcement is placed normal to center of roadway. (Caution: Watch for the clearance with longitudinal steel.)

2. The reinforcement should have at least one splice to facilitate proper bar placement.
Notes:
1. The diagonal brace supports web forms during web pour. After cure, the web is stiffer than the brace, and the web attracts load from subsequent concrete placements.
2. The tributary load includes half the overhang because the outer web form remains tied to and transfers load to the web which is considerably stiffer than the formwork.
3. Increase web reinforcement for locked-in construction load due to top slab forming for sloped web box girders.

Typical Top Slab Forming for Sloped Web Box Girder

*Figure 5.3.2-10*
5.3.3 Crossbeam

A. **General** – Crossbeam shall be designed in accordance with the requirements of strength limit state design of AASHTO LRFD Specifications and shall satisfy the serviceability requirements for crack control.

B. **Basic Geometry** – For aesthetic purposes, it is preferable to keep the crossbeam within the superstructure so that the bottom slab of the entire bridge is a continuous plane surface interrupted only by the columns. Although the depth of the crossbeam may be limited, the width can be made as wide as necessary to satisfy design requirements. Normally, it varies from 3 feet to the depth of box but is not less than the column size plus 1’-0” to allow placement of the column reinforcement as shown in see Figures 5.3.3-1 and 2.

Crossbeams on box girder type of construction shall be designed as a T beam utilizing the flange in compression, assuming the deck slab acts as a flange for positive moment and bottom slab a flange for negative moment. The effective overhang of the flange on a cantilever beam shall be limited to six times the flange thickness.

The bottom slab thickness is frequently increased near the crossbeam in order to keep the main box girder compressive stresses to a desirable level for negative girder moments as shown in Figures 5.3.3-1 & 2. This bottom slab flare also helps resist negative crossbeam moments. Consideration should be given to flaring the bottom slab at the crossbeam for designing the cap even if it is not required for resisting main girder moments.

C. **Loads** – For concrete box girders the superstructure dead load shall be considered as uniformly distributed over the crossbeam. For concrete box girders the live load shall be considered as the truck load directly to the crossbeam from the wheel axles. Truck axles shall be moved transversely over the crossbeam to obtain the maximum design forces for the crossbeam and supporting columns.

D. **Reinforcement Design and Details** – The crossbeam section consists of rectangular section with overhanging deck and bottom slab if applicable. The effective width of the crossbeam flange overhang shall be taken as the lesser of:
   - 6 times slab thickness,
   - $\frac{1}{10}$ of column spacing, or
   - $\frac{1}{20}$ of crossbeam cantilever as shown in Figure 5.3.3-3.

The crossbeam shall have a minimum width of column dimension plus 6”.

Crossbeam is usually cast to the fillet below the top slab. To avoid cracking of concrete on top of the crossbeam, construction reinforcement shall be provided at approximately 3” below the construction joint. The design moment for construction reinforcement shall be the factored negative dead load moment due to the weight of crossbeam and adjacent 10′ of superstructure each side. The total amount of construction reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment $M_{cr}$. 


Crossbeam Top Reinforcement for Skew Angle ≤ 25°
*Figure 5.3.3-1*

Crossbeam Top Reinforcement for Skew Angle > 25°
*Figure 5.3.3-2*
Effective Width of Crossbeam

*Figure 5.3.3-3*
Special attention should be given to the details to ensure that the column and crossbeam reinforcement will not interfere with each other. This can be a problem especially when round columns with a great number of vertical bars must be meshed with a considerable amount of positive crossbeam reinforcement passing over the columns.

1. **Top Reinforcement** – The negative moment critical section shall be at the \( \frac{1}{4} \) point of the square or equivalent square columns.

   a. **When Skew Angle \( \leq 25^\circ \)** – If the bridge is tangent or slightly skewed deck transverse reinforcement is normal or radial to centerline bridge, the negative cap reinforcement can be placed either in contact with top deck negative reinforcement (see Figure 5.3.3-1) or directly under the main deck reinforcement.

   b. **When Skew Angle > 25°** – When the structure is on a greater skew and the deck steel is normal or radial to the longitudinal centerline of the bridge, the negative cap reinforcement should be lowered to below the main deck reinforcement (see Figure 5.3.3-2).

   c. **To avoid cracking of concrete** – Interim reinforcement is required below the construction joint in crossbeams.

2. **Skin Reinforcement** – Longitudinal skin reinforcement shall be provided per AASHTO LRFD 5.7.3.4.

### 5.3.4 End Diaphragm

A. **Basic Geometry** – Bearings at the end diaphragms are usually located under the girder stems to transfer loads directly to the pier as shown in Figure 5.3.4-1. In this case, the diaphragm width shall be equal to or greater than bearing grout pads as shown Figure 5.3.4-2.

   Designer shall provide access space for maintenance and inspection of bearings.

   Allowance shall be provided to remove and replace the bearings. Lift point locations, jack capacity, number of jacks, and maximum permitted lift shall be shown in the plan details.

![Bearing Locations at End Diaphragm](Figure 5.3.4-1)
The most commonly used type of end diaphragm is shown in Figure 5.3.4-3. The dimensions shown here are used as a guideline and should be modified if necessary. This end diaphragm is used with a stub abutment and overhangs the stub abutment. It is used on bridges with an overall length less than 400 feet. If the overall length exceeds 400 feet, an L-shape abutment should be used.
B. Reinforcing Steel Details – Typical reinforcement details for an end diaphragm are shown in Figure 5.3.4-4.

Typical End Diaphragm Reinforcement

5.3.5 Dead Load Deflection and Camber

Camber is the adjustment made to the vertical alignment to compensate for the anticipated dead load deflection and the long-term deflection caused by shrinkage and creep. Estimating long-term deflection and camber for reinforced concrete flexural members shall be based on the creep coefficient given in Section 5.1.1E. Alternatively, Table 5.3.5-1 may be used for long-term camber multipliers.

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<tr>
<th>Girder Adjacent to Existing/Stage Construction</th>
<th>Multiplier Coefficient</th>
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<tr>
<td>Deflection (downward) — apply to the elastic deflection due to the weight of member</td>
<td>1.90</td>
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<tr>
<td>Deflection (downward) — apply to the elastic deflection due to superimposed dead load only</td>
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</table>

<table>
<thead>
<tr>
<th>Girder Away From Existing/Stage Construction</th>
<th>Multiplier Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection (downward) — apply to the elastic deflection due to the weight of member</td>
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</tr>
<tr>
<td>Deflection (downward) — apply to the elastic deflection due to superimposed dead load only</td>
<td>3.00</td>
</tr>
</tbody>
</table>

Long-term Camber Multipliers

Table 5.3.5-1
In addition to dead load deflection, forms and falsework tend to settle and compress under the weight of freshly placed concrete. The amount of this take-up is dependent upon the type and design of the falsework, workmanship, type and quality of materials and support conditions. The camber shall be modified to account for anticipated take-up in the falsework.

5.3.6 Thermal Effects

Concrete box girder bridges are subjected to stresses and/or movements resulting from temperature variation. Temperature effects result from time-dependent variations in the effective bridge temperature and from temperature differentials within the bridge superstructure.

A. Effective Bridge Temperature and Movement – Proper temperature expansion provisions are essential in order to ensure that the structure will not be damaged by thermal movements. These movements, in turn, induce stresses in supporting elements such as columns or piers, and result in horizontal movement of the expansion joints and bearings. For more details, see Chapter 8.

B. Differential Temperature – Although time-dependent variations in the effective temperature have caused problems in both reinforced and prestressed concrete bridges, detrimental effects caused by temperature differential within the superstructure have occurred only in prestressed bridges. Therefore, computation of stresses and movements resulting from the vertical temperature gradients is not included in this chapter. For more details, see AASHTO Guide Specifications, Thermal Effects on Concrete Bridge Superstructures dated 1989.

5.3.7 Hinges

Hinges are one of the weakest links of box girder bridges subject to earthquake forces and it is desirable to eliminate hinges or reduce the number of hinges. For more details on the design of hinges, see Section 5.4.

Designer shall provide access space or pockets for maintenance and inspection of bearings.

Allowance shall be provided to remove and replace the bearings. Lift point locations, maximum lift permitted, jack capacity, and number of jacks shall be shown in the hinge plan details.

5.3.8 Drain Holes

Drain holes shall be placed in the bottom slab at the low point of each cell to drain curing water during construction and any rain water that leaks through the deck slab. Additional drains shall be provided as a safeguard against water accumulation in the cell (especially when waterlines are carried by the bridge). In some instances, drainage through the bottom slab is difficult and other means shall be provided (i.e., cells over large piers and where a sloping exterior web intersects a vertical web). In this case, a horizontal drain shall be provided through the vertical web. Figure 5.3.8-1 shows drainage details for the bottom slab of concrete box girder bridges with steel wire screen.
Drain Hole Details

Figure 5.3.8-1
5.4 Hinges and Inverted T-Beam Pier Caps

Hinges and inverted T-beam pier caps require special design and detailing considerations. Continuous hinge shelves (both top and bottom projecting shelves) and continuous ledges of inverted T-beam pier caps, which support girders, are shown in Figure 5.4-1. In each case, vertical tensile forces (hanger tension) act at the intersection of the web and the horizontal hinge shelf or ledge. In the ledges of inverted T-beam pier caps, passage of live loads may also cause reversing torsional stresses which together with conventional longitudinal shear and bending produce complex stress distributions in the ledges\textsuperscript{7,8}.

Figure 5.4-2 provides minimum shelf or ledge support lengths (N) and provides positive longitudinal linkage (e.g., earthquake restrainers) in accordance with the current AASHTO LRFD specifications for seismic design requirements. Design considerations for beam ledges, inverted T-beam and hinges are given in AASHTO LRFD 5.13.2.5.

Inverted T-beam pier caps shall not be used for prestressed concrete girder bridges unless approved by the WSDOT Bridge Design Engineer.
Hinge and Inverted T-Beam Pier Cap

Figure 5.4-1
In-Span Hinge

Figure 5.4-2
5.5 Bridge Widening

This section provides general guidance for the design of bridge widenings. Included are additions to the substructure and the superstructure of reinforced concrete box girder, flat slab, T-beam, and prestressed concrete girder bridges. For additional information, see ACI Committee Report, Guide for Widening Highway Bridges.

5.5.1 Review of Existing Structures

A. General – Obtain the following documents from existing records for preliminary review, design, and plan preparation:

1. The “As-Built” contract plans, usually available from the ”Bridge Engineering Information System” on the Bridge and Structures Office website.

2. The original contract plans and special provisions, which can be obtained from Engineering Records (Plans Vault), Records Control or the ”Bridge Engineering Information System” on the Bridge and Structures Office website.

3. Check with the Bridge Preservation Unit for records of any unusual movements/rotations and other structural information.

4. Original design calculations, which are stored in State Archives.

5. Current field measurements. Current field measurements of existing pier crossbeam locations are recommended so that new prestressed concrete girders are not fabricated too short or too long. This is particularly important if piers have been constructed with different skews.

6. Original and current Foundation Reports from the Materials Lab or from the Plans Vault.

7. Change Order files to the original bridge contract in Records Control Unit.

B. Original Contract Plans and Special Provisions – Location and size of reinforcement, member sizes and geometry, location of construction joints, details, allowable design soil pressure, and test hole data are given on the plans. Original contract plans can be more legible than the microfilm copies.

The special provisions may include pertinent information that is not covered on the plans or in the AASHTO LRFD Specifications.

C. Original Calculations – The original calculations should be reviewed for any “special assumptions” or office criteria used in the original design. The actual stresses in the structural members, which will be affected by the widening, should be reviewed. This may affect the structure type selected for the widening.

D. Final Records – For major widening/renovation projects, the Final Records should be reviewed particularly for information about the existing foundations and piles. Sometimes the piles indicated on the original plans were omitted, revised, or required preboring. Final Records are available from Records Control or Bridge Records (Final Records on some older bridges may be in storage at the Materials Lab).
5.5.2 Analysis and Design Criteria

A. General – Each widening represents a unique situation and construction operations may vary between widening projects. The guidelines in this section are based on years of WSDOT design experience with bridge widenings.

1. Appearance – The widening of a structure should be accomplished in such a manner that the existing structure does not look “added on to.” When this is not possible, consideration should be given to enclosure walls, cover panels, paint, or other aesthetic treatments. Where possible and appropriate, the structure’s appearance should be improved by the widening.

2. Materials – Preferably, materials used in the construction of the widening shall have the same thermal and elastic properties as the materials in the original structure.

3. Load Distribution and Construction Sequence

The members of the widening should be proportioned to provide similar longitudinal and transverse load distribution characteristics as the existing structure. Normally this can be achieved by using the same cross sections and member lengths that were used in the existing structure.

The construction sequence and degree of interaction between the widening and the existing structure, after completion, shall be fully considered in determining the distribution of the dead load for design of the widening and stress checks for the existing structure.

A suggested construction sequence or stage construction shall be clearly shown in the plans to avoid confusion and misinterpretation during construction. A typical construction sequence may involve placing the deck concrete, removing the falsework, placing the concrete for the closure strip, and placing the concrete for the traffic barrier.

4. Specifications – The design of the widening shall conform to the current AASHTO LRFD Specifications and the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction.

5. Geometrical Constraints – The overall appearance and geometrical dimensions of the superstructure and columns of the widening should be the same or as close as possible to those of the existing structure. This is to ensure that the widening will have the same appearance and similar structural stiffness as the original structure.

6. Overlay – It should be established at the preliminary plan stage if an overlay is required as part of the widening.

7. Strength of the Existing Structure – A review of the strength of the main members of the existing structure shall be made for construction conditions utilizing AASHTO LRFD specifications.

A check of the existing main members after attachment of the widening shall be made for the final design loading condition.

If the existing structural elements do not have adequate strength, consult your Design Unit Manager or in the case of consultants, contact the Consultant Liaison Engineer for appropriate guidance.
If significant demolition is required on the existing bridge, consideration should be given to requesting concrete strength testing for the existing bridge and including this information in the contract documents.

8. **Special Considerations**

   a. For structures that were originally designed for HS-20 loading, HL-93 shall be used to design the widening. For structures that were originally designed for less than HS-20, consideration should be given to replacing the structure instead of widening it.

   b. Longitudinal joints are not permitted in order to eliminate potentially hazardous vehicle control problems.

   c. The *Standard Specifications* do not permit falsework to be supported from the existing structure unless the Plans and Specifications state otherwise. This requirement eliminates the transmission of vibration from the existing structure to the widening during construction. The existing structure may still be in service.

   d. For narrow widenings where the Plans and Specifications require that the falsework be supported from the original structure (e.g., there are no additional girders, columns, crossbeams, or closure strips), there shall be no external rigid supports such as posts or falsework from the ground. Supports from the ground do not permit the widening to deflect with the existing structure when traffic is on the existing structure. This causes the uncured concrete of the widening to crack where it joins the existing structure. Differential dead load deflection during construction shall be given consideration.

   e. Precast members may be used to widen existing CIP structures. This method is useful when the horizontal or vertical clearances during construction are insufficient to build CIP members.

   f. The alignment for diaphragms for the widening shall generally coincide with the existing diaphragms.

   g. When using battered piles, estimate the pile tip elevations and ensure that they will have ample clearance from all existing piles, utilities, or other obstructions. Also check that there is sufficient clearance between the existing structure and the pile driving equipment.

B. **Seismic Design Criteria for Bridge Widens** – Seismic design of bridge widenings shall be per Section 4.3.

C. **Substructure**

   1. **Selection of Foundation**

      a. The type of foundation to be used to support the widening shall generally be the same as that of the existing structure unless otherwise recommended by the Geotechnical Engineer. The effects of possible differential settlement between the new and the existing foundations shall be considered.
b. Consider present bridge site conditions when determining new foundation locations. The conditions include: overhead clearance for pile driving equipment, horizontal clearance requirements, working room, pile batters, channel changes, utility locations, existing embankments, and other similar conditions.

2. **Scour and Drift** – Added piles and columns for widenings at water crossings may alter stream flow characteristics at the bridge site. This may result in pier scouring to a greater depth than experienced with the existing configuration. Added substructure elements may also increase the possibility of trapping drift. The Hydraulics Engineer shall be consulted concerning potential problems related to scour and drift on all widenings at water crossings.

D. **Superstructure**

1. **Camber** – Accurate prediction of dead load deflection is more important for widenings than for new bridges, since it is essential that the deck grades match.

   To obtain a smooth transition in transverse direction of the bridge deck, the camber of the girder adjacent to the existing structure shall be adjusted for the difference in camber between new and existing structure. A linear interpolation may be used to adjust the camber of the girders located away from the existing structure. The multipliers for estimating camber of new structure may be taken as shown in Table 5.3.5-1.

2. **Closure Strip** – Except for narrow deck slab widenings a closure strip is required for all CIP widenings. The width shall be the minimum required to accommodate the necessary reinforcement and for form removal. Reinforcement, which extends through the closure strip shall be investigated. Shear shall be transferred across the closure strip by shear friction and/or shear keys.

   All falsework supporting the widening shall be released and formwork supporting the closure strip shall be supported from the existing and newly widened structures prior to placing concrete in the closure strip. Because of deck slab cracking experienced in widened concrete decks, closure strips are required unless the mid-span dead load camber is ½” or less.

3. **Stress Levels and Deflections in Existing Structures** – Caution is necessary in determining the cumulative stress levels, deflections, and the need for shoring in existing structural members during rehabilitation projects.

   For example, a T-beam bridge was originally constructed on falsework and the falsework was released after the deck slab concrete gained strength. As part of a major rehabilitation project, the bridge was closed to traffic and the entire deck slab was removed and replaced without shoring. Without the deck slab, the stems behave as rectangular sections with a reduced depth and width. The existing stem reinforcement was not originally designed to support the weight of the deck slab without shoring. After the new deck slab was placed, wide cracks from the bottom of the stem opened, indicating that the reinforcement was overstressed. This overstress resulted in a lower load rating for the newly rehabilitated bridge. This example shows the need to shore up the remaining T-beam stems prior to placing the new deck slab so that excessive deflections do not occur and over stress in the existing reinforcing steel is prevented.
It is necessary to understand how the original structure was constructed, how the rehabilitated structure is to be constructed, and the cumulative stress levels and deflections in the structure from the time of original construction through rehabilitation.

E. **Stability of Widening** – For relatively narrow box girder and T-beam widenings, symmetry about the vertical axis should be maintained because lateral loads are critical during construction. When symmetry is not possible, use pile cap connections, lateral connections, or special falsework. A minimum of two webs is generally recommended for box girder widenings. For T-beam widenings that require only one additional web, the web should be centered at the axis of symmetry of the deck slab. Often the width of the closure strip can be adjusted to accomplish this.

In prestressed concrete girder bridge widenings with one or two lines of new girders, the end and intermediate diaphragms shall be placed prior to the deck slab casting to ensure the stability of the girders during construction. The closure shall be specified for deck slab but shall not be required for diaphragms. The designer shall investigate the adequacy of the existing girder adjacent to the widening for the additional load due to the weight of wet deck slab transferred through the diaphragms, taking into account the loss of removed overhang and barrier. The diaphragms must be made continuous with existing diaphragms.

### 5.5.3 Removing Portions of the Existing Structure

Portions of the existing structure to be removed shall be clearly indicated on the plans. Where a clean break line is required, a ¾” deep saw cut shall be specified for a deck slab with normal wear and a ½” deep saw cut for worn deck slabs. In no case, however, shall the saw blade cut or nick the main transverse top slab reinforcement. The special provisions shall state that care will be taken not to damage any reinforcement which is to be saved. Hydromilling is preferred where reinforcing bar cover is shallow and can effectively remove delaminated decks because of the good depth control it offers. When greater depths of slab are to be removed, special consideration should be given to securing exposed reinforcing bars to prevent undue vibration and subsequent fatigue cracks from occurring in the reinforcing bars.

The current General Special Provisions should be reviewed for other specific requirements on deck slab removal.

Removal of any portion of the main structural members should be held to a minimum. Careful consideration shall be given to the construction conditions, particularly when the removal affects the existing frame system. In extreme situations, preloading by jacking is acceptable to control stresses and deflections during the various stages of removal and construction. Removal of the main longitudinal deck slab reinforcement should be kept to a minimum. See “Slab Removal Detail” Figure 5.5.4-1 for the limiting case for the maximum allowable removal.

The plans shall include a note that critical dimensions and elevations are to be verified in the field prior to the fabrication of precast units or expansion joint assemblies.
In cases where an existing sidewalk is to be removed but the supporting slab under the sidewalk is to be retained, Region personnel should check the feasibility of removing the sidewalk. Prior to design, Region personnel should make recommendations on acceptable removal methods and required construction equipment. The plans and specifications shall then be prepared to accommodate these recommendations. This will ensure the constructibility of plan details and the adequacy of the specifications.

5.5.4 Attachment of Widening to Existing Structure

A. General

1. Lap and Mechanical Splices – To attach a widening to an existing structure, the first choice is to utilize existing reinforcing bars by splicing new bars to existing. Lap splices or mechanical splices should be used. However, it may not always be possible to splice to existing reinforcing bars and spacing limitations may make it difficult to use mechanical splices.

2. Welding Reinforcement – Existing reinforcing steel may not be readily weldable. Mechanical splices should be used wherever possible. If welding is the only feasible means, the chemistry of the reinforcing steel must be analyzed and acceptable welding procedures developed.

3. Drilling Into Existing Structure – It may be necessary to drill holes and set dowels in epoxy resin in order to attach the widening to the existing structure. When drilling into heavily reinforced areas, chipping should be specified to expose the main reinforcing bars. If it is necessary to drill through reinforcing bars or if the holes are within 4 inches of an existing concrete edge, core drilling shall be specified. Core drilled holes shall be roughened before resin is applied. If this is not done, a dried residue, which acts as a bond breaker and reduces the load capacity of the dowel, will remain. Generally, the drilled holes are ⅛" in diameter larger than the dowel diameter for #5 and smaller dowels and ¼" in diameter larger than the dowel diameter for #6 and larger dowels.

In special applications requiring drilled holes greater than 1½" diameter or deeper than 2’, core drilling shall be specified. These holes shall also be intentionally roughened prior to applying epoxy resin.

Core drilled holes shall have a minimum clearance of 3" from the edge of the concrete and 1” clearance from existing reinforcing bars in the existing structure. These clearances shall be noted in the plans.

4. Dowelling Reinforcing Bars Into the Existing Structure

a. Dowel bars shall be set with an approved epoxy resin. The existing structural element shall be checked for its adequacy to transmit the load transferred to it from the dowel bars.

b. Dowel spacing and edge distance affect the allowable tensile dowel loads. Allowable tensile loads, dowel bar embedment, and drilled hole sizes for reinforcing bars (Grade 60) used as dowels and set with an approved epoxy resin are shown in Table 5.5.4-1. These values are based on an edge clearance greater than 3", a dowel spacing greater than 6", and are shown for both uncoated and epoxy coated dowels. Table 5.5.4-2 lists dowel
embedment lengths when the dowel spacing is less than 6”. Note that in Table 5.5.4-2 the edge clearance is equal to or greater than 3”, because this is the minimum edge clearance for a drilled hole from a concrete edge.

If it is not possible to obtain these embedments, such as for traffic railing dowels into existing deck slabs, the allowable load on the dowel shall be reduced by the ratio of the actual embedment divided by the required embedment.

c. The embedments shown in Table 5.5.4-1 and Table 5.5.4-2 are based on dowels embedded in concrete with $f_c' = 4,000$ psi.

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<th>Bar Size</th>
<th>Allowable Design Tensile Load, $T^*$ (kips)</th>
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<th>Required Embedment, $L_{e}$</th>
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<td>#9</td>
<td>60.0</td>
<td>1½</td>
<td>16</td>
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<tr>
<td>#10</td>
<td>73.6</td>
<td>1¾</td>
<td>20</td>
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<tr>
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<td>1¾</td>
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* Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.

Allowable Tensile Load for Dowels Set With Epoxy Resin $f_c' = 4,000$ psi, Grade 60 Reinforcing Bars, Edge Clearance ≥ 3”, and Spacing ≥ 6”

Table 5.5.4-1

<table>
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<tr>
<th>Bar Size</th>
<th>Allowable Design Tensile Load, $T^*$ (kips)</th>
<th>Drill Hole Size (in)</th>
<th>Required Embedment, $L_{e}$</th>
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<td></td>
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<td>Uncoated (in)</td>
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<td>1</td>
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<td>13½</td>
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<tr>
<td>#8</td>
<td>47.4</td>
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<td>16½</td>
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<tr>
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<td>60.0</td>
<td>1⅛</td>
<td>20</td>
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<tr>
<td>#10</td>
<td>73.6</td>
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<tr>
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<td>1½</td>
<td>30</td>
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*Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.

Allowable Tensile Load for Dowels Set With Epoxy Resin, $f_c'=4,000$ psi, Grade 60 Reinforcing Bars, Edge Clearance ≥ 3”, and Spacing < 6”

Table 5.5.4-2
5. **Shear Transfer Across a Dowelled Joint** – Shear shall be carried across the joint by shear friction. The existing concrete surface shall be intentionally roughened. Both the concrete and dowels shall be considered effective in transmitting the shear force. Chipping shear keys in the existing concrete can also be used to transfer shear across a dowelled joint, but is expensive.

6. **Preparation of Existing Surfaces for Concreting** – See “Removing Portions of Existing Concrete” in the General Special Provisions and *Standard Specifications* Section 6-02.3(12) for requirements. Unsound, damaged, dirty, porous, or otherwise undesirable old concrete shall be removed, and the remaining concrete surface shall be clean, free of laitance, and intentionally roughened to ensure proper bond between the old and new concrete surfaces.

7. **Control of Shrinkage and Deflection on Connecting Reinforcement** – Dowels that are fixed in the existing structure may be subject to shear as a result of longitudinal shrinkage and vertical deflection when the falsework is removed. These shear forces may result in a reduced tensile capacity of the connection. When connecting the transverse reinforcing bars across the closure strip is unavoidable, the interaction between shear and tension in the dowel or reinforcing bar shall be checked. The use of wire rope or sleeved reinforcement may be acceptable, subject to approval by your Design Unit Manager.

Where possible, transverse reinforcing bars shall be spliced to the existing reinforcing bars in a blocked-out area which can be included in the closure strip. Nominal, shear friction, temperature and shrinkage, and distribution reinforcing bars shall be bent into the closure strip.

Rock bolts may be used to transfer connection loads deep into the existing structure, subject to the approval of your Design Unit Manager.

8. **Post-tensioning** – Post-tensioning of existing crossbeams may be utilized to increase the moment capacity and to eliminate the need for additional substructure. Generally, an existing crossbeam can be core drilled for post-tensioning if it is less than 30′ long. The amount of drift in the holes alignment may be approximately 1” in 20′. For crossbeams longer than 30′, external post-tensioning should be considered.

For an example of this application, refer to Contract 3846, Bellevue Transit Access – Stage 1.
B. **Connection Details** – The details on the following sheets are samples of details which have been used for widening bridges. They are informational and are not intended to restrict the designer’s judgment.

1. **Box Girder Bridges** – Figures 5.5.4-1 through 5.5.4-6 show typical details for widening box girder bridges.

   Welding or mechanical butt splice are preferred over dowelling for the main reinforcement in crossbeams and columns when it can be done in the horizontal or flat position. It shall be allowed only when the bars to be welded are free from restraint at one end during the welding process.

   ![Deck Slab Removal Detail](Figure 5.5.4-1)
STAY IN PLACE FORM DETAIL
FOR BOX GIRDER STAGED CONSTRUCTION OR WIDENING

* STAY IN PLACE FORMS SHALL BE SOLID GALVANIZED SHEET METAL. FORMS MUST BE VERTICALLY BRACED AS NECESSARY TO PREVENT BOWING DURING CONCRETE PLACEMENT. TIMBER BRACING MUST BE REMOVED. IF STEEL WALES OR TIES ARE USED, THEY MAY BE LEFT IN PLACE. THE CONTRACTOR SHALL SUBMIT DESIGN CALCULATIONS IN ACCORDANCE WITH STANDARD SPECIFICATIONS 6-02.3(16) AND 6-02.3(17).

Box Girder Section in Span
Figure 5.5.4-2
Box Girder Section Through Crossbeam

Figure 5.5.4-3
Box Girder Section in Span at Diaphragm Alternate I

Figure 5.5.4-4

SEE "BOX GIRDER - SECTION IN SPAN" FOR ADDITIONAL DETAILS.

** SEE TABLE 5.5.4-1 OR 5.5.4-2 FOR MINIMUM DOWEL EMBEDMENT
**Box Girder Section in Span at Diaphragm Alternate II**

*Figure 5.5.4-5*

*IF LAP SPlice EXCEEDS 2'-0", INCREASE WIDTH OF CLOSURE Strip TO ACCOMMODATE INCREASED LAP SPlice.*
Narrow Box Girder Widening Details

Figure 5.5.4-6

NOTE: THIS ALTERNATE APPLIES TO NARROW WIDENINGS WHERE SHEAR IN THE EXTERIOR WEB IS NOT CRITICAL. THIS IS TYPICAL FOR SHORT TO MEDIUM SPANS OR WHERE THE EXISTING SLAB OVERHANG IS CONSIDERABLY LESS THAN HALF THE WEB SPACING.

EMBEDMENT LENGTH (PER TABLE 5.5.4-1, 5.5.4-2, OR MANUFACTURER'S RECOMMENDATION)

DEVELOPMENT LENGTH

L = THREADED COUPLER LENGTH
L/2
L/2

NOTE: INSTALL ANCHOR BOLT WITH EPOXY RESIN SYSTEM PER MANUFACTURER'S RECOMMENDATIONS IN DRY CONDITIONS.
2. **Flat Slab Bridges** – It is not necessary to remove any portion of the existing slab to expose the existing transverse reinforcing bars for splicing purposes, because the transverse slab reinforcement is only distribution reinforcement. The transverse slab reinforcement for the widening may be dowelled directly into the existing structure without meeting the normal splice requirements.

For the moment connection details, see Figure 5.5.4-7.

*Note:* Falsework shall be maintained under pier crossbeams until closure pour is made and cured for 10 days.
3. **T-Beam Bridges** – Use details similar to those for box girder bridges for crossbeam connections. See Figure 5.5.4-8 for slab connection detail.

![T-Beam - Section in Span](Figure 5.5.4-8)
4. **Prestressed Concrete Girder Bridges** – Use details similar to those for box girder bridges for crossbeam moment connections and use details similar to those in Figure 5.5.4-9 for the slab connection detail.

\[
X = \frac{\text{Top Flange Width}}{2} \quad \text{for } 4' \leq 6'
\]

* IF EXISTING TRANSVERSE BOTTOM SLAB BARS ARE TOO SHORT FOR A CONVENTIONAL LAP SPlice THEY SHOULD BE BUTT SPLICED WITH A MECHANICAL COUPLER.
5.5.5 Expansion Joints

The designer should determine if existing expansion joints can be eliminated. It will be necessary to determine what modifications to the structure are required to provide an adequate functional system when existing joints are eliminated.

For expansion joint design, see Section 9.1 Expansion Joints. Very often on widening projects it is necessary to chip out the existing concrete deck and rebuild the joint. Figures 5.5.5-1 & 2 show details for rebuilding joint openings for compression seal expansion joints.

If a widening project includes an overlay, the expansion joint may have to be raised, modified or replaced. See the Joint Specialist for plan details that are currently being used to modify or retrofit existing expansion joints.

![Expansion Joint Detail Shown for Compression Seal](image)

**Expansion Joint Detail Shown for Compression Seal**

*With Existing Reinforcing Steel Saved*

*Figure 5.5.5-1*
5.5.6 Possible Future Widening for Current Designs

For current projects that include sidewalks, provide a smooth rather than a roughened construction joint between the sidewalk and the slab.

5.5.7 Bridge Widening Falsework

For widenings which do not have additional girders, columns, crossbeams, or closure pours, falsework should be supported by the existing bridge. There should be no external support from the ground. The reason is that the ground support will not allow the widening to deflect the existing bridge when traffic is on the bridge. This will cause the “green” concrete to crack where it joins the existing bridge. The designer should contact the Bridge Construction Support Unit regarding falsework associated with widenings.

5.5.8 Existing Bridge Widening

Appendix 5-B3 lists bridge widenings projects that may be used as design aids for the designers. These should not be construed as the only acceptable methods of widening; there is no substitute for the designer’s creativity or ingenuity in solving the challenges posed by bridge widenings.
5.6 Prestressed Concrete Girder Superstructures

The prestressed concrete girder bridge is an economical and rapid type of bridge construction and often preferred for WSDOT bridges.

Precast sections are generally fabricated in plant or somewhere near the construction site and then erected. Precasting permits better material quality control and is often more economical than CIP concrete.

Pre-tensioning is accomplished by stressing strands to a predetermined tension and then placing concrete around the strands, while the stress is maintained. After the concrete has hardened, the strands are released and the concrete, which has become bonded to the tendon, is prestressed as a result of the strands attempting to relax to their original length. The strand stress is maintained during placing and curing of the concrete by anchoring the ends of strands to abutments that may be as much as 500' apart. The abutments and appurtenances used in the prestressing procedure are referred to as a pre-tensioning bed or bench.

5.6.1 WSDOT Standard Prestressed Concrete Girder Types

A girder type consists of a series of girder cross sections sharing a common shape. The numbers within girder series generally refer to the depth of the section in inches. Refer to Standard Specifications Section 6-02.3(25) for a comprehensive list of Standard WSDOT girder types. Standard WSDOT girder types include:

**Prestressed Concrete I Girders** – Washington State Standard I Girders were adopted in the mid-1950s. The original series was graduated in 10 foot increments from 30 feet to 100 feet. In 1990, revisions were made incorporating the results of the research done at Washington State University on girders without end blocks. The revisions included three major changes: a thicker web; end blocks were eliminated; and strand spacing was increased. The current Series of this type include W42G, W50G, W58G, and W74G.

**Prestressed Concrete Wide Flange (WF) I Girders and Spliced Prestressed Concrete Girders** – In 1999, deeper girders, commonly called “Supergirders” were added to the WSDOT standard concrete girders. These new supergirders may be pretensioned or post-tensioned. The pretensioned Series are designated as WF74G, WF83G and WF95G and the post-tensioned (spliced) Series are designated as WF74PTG, WF83PTG and WF95PTG.

In 2004 Series WF42G, WF50G, and WF58G were added to the prestressed concrete girder standards. In 2008, Series WF66G, WF100G, and WF100PTG were added to the prestressed concrete girder standards. In 2009, Series WF36G was added to the prestressed concrete girder standards.

**Prestressed Concrete Wide Flange Deck Girders** – In 2015, the top flanges of Wide Flange I Girders were widened and thickened to support traffic loads without a CIP concrete deck. The top flanges are mechanically connected at the flange edges to adjacent girders. This Series includes the WF39DG through the WF103DG.
Prestressed Concrete Wide Flange Thin Deck Girders – In 2015, the top flanges of wide flange I girders were widened to create a girder which would support a CIP concrete deck placement without formwork. This Series includes the WF36TDG through the WF100TDG.

Deck Bulb Tee Girders – This type of girder has a top flange designed to support traffic loads and are mechanically connected at the flange edges to adjacent girders. They include Series W35DG, W41DG, W53DG and W65DG.

Prestressed Concrete Slab Girders – Prestressed concrete slab girders are available in heights ranging from 12 inches to 36 inches.

Prestressed Concrete Tub Girders – In 2004 prestressed concrete tub girders were added as standard girders.

All WSDOT prestressed concrete girders are high performance high strength concrete girders. They generally rely on high strength concrete to be effective for the spans expected as a single piece. The approximate ranges of maximum span lengths are as shown in Table 5.6.1-1 and Appendix 5.6-A1.

Standard drawings for WSDOT prestressed concrete girders are shown in Appendix 5.6-A and 5.9-A.
### Section Properties of WSDOT Standard Prestressed Concrete Girders

#### Table 5.6.1-1

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<th>Type</th>
<th>Depth (in)</th>
<th>Area (in²)</th>
<th>Iz (in⁴)</th>
<th>Yb (in)</th>
<th>Wt (k/ft)</th>
<th>Volume to Surface Ratio (in)</th>
<th>Max. Span Capability (ft)</th>
<th>Max. Length (252 kips Limit) (ft)</th>
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5.6.2 Design Criteria

WSDOT design criteria for prestressed concrete girder superstructures are given in Table 5.6.2-1.

AASHTO LRFD 5.14.1.4 “Bridges Composed of Simple Span Precast Girders Made Continuous” allows for some degree of continuity for loads applied on the bridge after the continuity diaphragms have been cast and cured. This assumption is based on the age of the girder when continuity is established, and degree of continuity at various limit states. Both degree of continuity and time of continuity diaphragm casting may result in contractual and design issues. Designing these types of bridges for the envelope of simple span and continuous spans for applicable permanent and transient loads is the approach used by WSDOT as it has yielded good results.

<table>
<thead>
<tr>
<th>Design Specifications</th>
<th>AASHTO LRFD Specifications and WSDOT Bridge Design Manual M 23-50</th>
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<tbody>
<tr>
<td>Design Method</td>
<td>Prestressed concrete members shall be designed for service limit state for allowable stresses and checked for strength limit state for ultimate capacity.</td>
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<td>Superstructure Continuity</td>
<td>Prestressed concrete girder superstructures shall be designed for the envelope of simple span and continuous span loadings for all permanent and transient loads. Loads applied before establishing continuity (typically before placement of continuity diaphragms) need only be applied as a simple span loading. Continuity reinforcement shall be provided at supports for loads applied after establishing continuity.</td>
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<td>Loads and Load Factors</td>
<td>Service, Strength, Fatigue, and Extreme Event Limit State loads and load combinations shall be per AASHTO LRFD Specifications</td>
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<td>Allowable Stresses</td>
<td>WSDOT Bridge Design Manual M 23-50 Table 5.2.1-1</td>
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<td>WSDOT Bridge Design Manual M 23-50 Section 5.6.3</td>
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<tr>
<td>Continuous Structure Configuration</td>
<td>Girder types and spacing shall be identical in adjacent spans. Girder types and spacing may be changed at expansion joints.</td>
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<tr>
<td>Girder End Support Skew Angle</td>
<td>Girder end support skew angles shall be limited to 45° for all prestressed concrete girders. Skew angles for prestressed concrete slabs, deck bulb-tees and tubs shall be limited to 30°.</td>
</tr>
</tbody>
</table>
| Intermediate Diaphragms | CIP concrete intermediate diaphragms shall be provided for all prestressed concrete girder bridges (except slabs) as shown below:  
  - ½ points of span for span length > 160'-0".  
  - ¼ points of span for 120'-0" < span length ≤ 160'-0".  
  - Midpoint of span for 40'-0" < span length ≤ 80'-0".  
  - No diaphragm requirement for span length ≤ 40'-0".  
  Intermediate diaphragms shall be either partial or full depth as described in Section 5.6.4.C.4. |

Design Criteria for Prestressed Concrete Girders  
Table 5.6.2-1
A. **Support Conditions** – The prestressed concrete girders are assumed to be supported on rigid permanent simple supports. These supports can be either bearing seats or elastomeric pads. The design span length is the distance center to center of bearings for simple spans. For continuous spans erected on falsework (raised crossbeam), the effective point of support for girder design is assumed to be the face of the crossbeam. For continuous spans on crossbeams (dropped or semi-dropped crossbeam), the design span length is usually the distance center to center of temporary bearings.

B. **Composite Action**

1. **General** – The sequence of construction and loading is extremely important in the design of prestressed concrete girders. The composite section has a much larger capacity than the basic girder section but it cannot take loads until the deck slab has obtained adequate strength. Assumptions used in computing composite section properties are shown in Figure 5.6.2-1.

2. **Load Application** – The following sequence and method of applying loads is typically used in girder analysis:
   a. Girder dead load is applied to the girder section.
   b. Diaphragm dead load is applied to the girder section.
   c. Deck slab dead load is applied to the girder section.
   d. Superimposed dead loads (such as barriers, sidewalks and overlays) and live loads are applied to the composite section.

   The dead load of one traffic barrier or sidewalk may be divided among a maximum of three girder webs.

3. **Composite Section Properties** – A CIP concrete bridge deck forms the top flange of the composite girder in prestressed concrete girder bridge construction.

   a. **Effective and Transformed Flange Width** – The effective flange width of a concrete bridge deck for computing composite section properties shall be per AASHTO LRFD 4.6.2.6. The effective flange width shall be reduced by the ratio $E_{slab}/E_{girder}$ to obtain the transformed flange width. The effective modulus of the composite section with the transformed flange width is then $E_{girder}$.

   b. **Effective Flange Thickness** – The effective flange thickness of a concrete bridge deck for computing composite section properties shall be the deck thickness reduced by $\frac{1}{2}”$ to account for wearing. Where a bridge will have an overlay applied prior to traffic being allowed on the bridge, the full deck thickness may be used as effective flange thickness.
Concrete Structures

Section as Detailed

\[ \text{Effective Flange Width} = W_{EF} \]

\[ W_T = W_{EF} \times \frac{E_{SLAB}}{E_{GIRDER}} \]

Section for Computation of Composite Section Properties

Typical Section for Computation of Composite Section Properties

\( t = T - \frac{1}{2}'' \)

 slug

A-T for Dead Load and for Composite Section for Negative Moment.

0.0 for Composite Section for Positive Moment.

\( A'' \text{ at } \& \text{ BRG.} \)

\( T \text{ (7}'\frac{1}{2}'' \text{ MIN.)} \)

\( \frac{3}{4}'' \text{ FILLET (TYP.)} \)

\( \frac{1}{2}'' \text{ WEARING SURFACE} \)

\( \text{CIP Deck Slab, assumed to be horizontal.} \)
c. **Flange Position** – An increased dimension from top of girder to top of bridge deck at centerline of bearing at centerline of girder shall be shown in the Plans. This is called the “A” dimension. It accounts for the effects of girder camber, vertical curve, deck cross slope, etc. See Appendix 5-B1 for method of computing.

For purposes of calculating composite section properties for negative moments, the pad/haunch height between bottom of deck and top of girder shall be taken as the “A” dimension minus the flange thickness “T” at intermediate pier supports and shall be reduced by girder camber as appropriate at other locations.

For purposes of calculating composite section properties for positive moments, the bottom of the bridge deck shall be assumed to be directly on the top of the girder. This assumption may prove to be true at center of span where excess girder camber occurs.

d. **Section Dead Load** – The bridge deck dead load to be applied to the girder shall be based on the full bridge deck thickness. The full effective pad/haunch weight shall be added to that load over the full length of the girder. The full effective pad or haunch height is typically the “A” dimension minus the flange thickness “T”, but may be higher at midspan for a crest vertical curve.

C. **Design Procedure**

1. **General** – The WSDOT Prestressed concrete girder design computer program PGSuper is the preferred method for final design.

2. **Stress Conditions** – The designer shall ensure that the stress limits as described in Table 5.2.1-1 are not exceeded for prestressed concrete girders. Each condition is the result of the summation of stresses with each load acting on its appropriate section (such as girder only or composite section).

Dead load impact need not be considered during lifting.

During shipping, girder stresses shall be checked using two load cases. The first load case consists of a plumb girder with dead load impact of 20% acting either up or down. The second load case consists of an inclined girder with no dead load impact. The angle of inclination shall be the equilibrium tilt angle computed for lateral stability (see BDM 5.6.3.D.6 and equation (12) in reference12) with a roadway superelevation of 6%.

D. **Standard Strand Locations** – Standard strand locations of typical prestressed concrete girders are shown in Figure 5.6.2-2 and Appendices 5.6-A and 5.9-A.
Typical Prestressed Concrete Girder Configuration

Figure 5.6.2-2
E. **Girder End Types** – There are four typical end types for prestressed concrete girders. Due to the extreme depth of the WF83G, WF95G, and WF100G girders, and possible end of girder tilt at the piers for profile grades, the designer will need to pay particular attention to details to assure the girders will fit and perform as intended.

The end types designs may require modification for bridge security. The space between girders at the abutment may require omission by extending the diaphragm to the face of the abutment stem. Coordinate with the State Bridge and Structures Architect during final design where required.

The four end types are shown as follows:

1. **End Type A** – End Type A as shown in Figure 5.6.2-3 is for cantilever end piers with an end diaphragm cast on the end of the girders. End Type A has a recess at the bottom of the girder near the end for an elastomeric bearing pad. See Appendix 5.6-A4-12 and 5.6-A9-9 for bearing pad details. The recess at the centerline of bearing is 0.5” deep. This recess is to be used for profile grades up to and including 4 percent. The recess is to be replaced by an embedded steel plate flush with the bottom of the girder for grades over 4 percent. A tapered bearing plate, with stops at the edges to contain the elastomeric pad, can be welded or bolted to the embedded plate to provide a level bearing surface.

Reinforcing bars and pretensioned strands project from the end of the girder. The designer shall assure that these bars and strands fit into the end diaphragm. Embedment of the girder end into the end diaphragm shall be a minimum of 3” and a maximum of 6”. For girder ends where the tilt would exceed 6” of embedment, the girder ends shall be tilted to attain a plumb surface when the girder is erected to the profile grade.

The gap between the end diaphragm and the stem wall shall be a minimum of 1½” or ½” greater than required for longitudinal bridge movement.
There are four end types shown on the standard girder sheets. Due to the extreme depth of the WF83G, WF95G, and WF100G girders, and possible end of girder tilt at the piers for profile grades, the designer will need to pay particular attention to details to assure the girders will fit and perform as intended. The four end types are shown as follows:

1. **End Type A**
   - End Type A as shown in Figure 5.6.2-4 is for cantilever end piers with an end diaphragm cast on the end of the girders. End Type A has a recess at the bottom of the girder near the end for an elastomeric bearing pad. See Appendix 5.6-A7-9 and 5.6-A9-12 for bearing pad details.
   - The recess at the centerline of bearing is 0.5″ deep. This recess is to be used for profile grades up to and including 4%. The recess is to be replaced by an embedded steel plate flush with the bottom of the girder for grades over 4%. A tapered bearing plate, with stops at the edges to contain the elastomeric pad, can be welded or bolted to the embedded plate to provide a level bearing surface.
   - Reinforcing bars and pretensioned strands project from the end of the girder. The designer shall assure that these bars and strands fit into the end diaphragm. Embedment of the girder end into the end diaphragm shall be a minimum of 3″ and a maximum of 6″. For girder ends where the tilt would exceed 6″ of embedment, the girder ends shall be tilted to attain a plumb surface when the girder is erected to the profile grade.
   - The gap between the end diaphragm and the stem wall shall be a minimum of 1½″ or ½″ greater than required for longitudinal bridge movement.

**Figure 5.6.2-3**

End Type A (End Diaphragm on Girder)
2. **End Type B** – End Type B as shown in Figure 5.6.2-4 is for “L” type abutments. End Type B also has a recess at the bottom of the girder for an elastomeric bearing pad. Notes regarding the bearing recess on End Type A also apply to End Type B. End Type B typically does not have reinforcing or strand projecting from the girder end.

The centerline of the diaphragm is normal to the roadway surface. The centerline of the bearing is coincident with the centerline of the diaphragm at the top of the elastomeric pad.
3. **End Type C** – End Type C as shown in Figure 5.6.2-5 is for continuous spans and an intermediate hinge diaphragm at an intermediate pier. There is no bearing recess and the girder is temporarily supported on oak blocks. In the past, this detail was used in low seismic areas such as east of the Cascade Mountains. This end type shall not be used on new structures and is restricted to the widening of existing bridges with hinge diaphragms at intermediate piers.

The designer shall check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting cross beam for load from the oak block including dead loads from girder, deck slab, and construction loads.

For **prestressed concrete girders** with intermediate hinge diaphragms, designers shall:

a. Check size and minimum embedment in crossbeam and diaphragm for hinge bars.

b. Check interface shear friction at girder end (see Section 5.2.2.C.2).

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**End Type C (Intermediate Hinge Diaphragm)**

*Figure 5.6.2-5*
4. **End Type D** – End Type D as shown in Figure 5.6.2-6 is for continuous spans fully fixed to columns at intermediate piers. There is no bearing recess and the girder is temporarily supported on oak blocks.

The designer shall check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting cross beam for load from the oak block including dead loads from girder, deck slab, and construction loads. The designer shall check interface shear friction at the girder end (see Section 5.2.2.C.2).

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**End Type D**

*Figure 5.6.2-6*

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**F. Splitting Resistance in End Regions of Prestressed Concrete Girders** – The splitting resistance of pretensioned anchorage zones shall be as described in AASHTO LRFD 5.10.10.1. For pretensioned I-girders or bulb tees, the end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 2½”. The remaining splitting reinforcement not fitting within the h/4 zone may be placed beyond the h/4 zone at a spacing of 2½”.

**G. Confinement Reinforcement in End Regions of Prestressed Concrete Girders** – Confinement reinforcement per AASHTO LRFD 5.10.10.2 shall be provided.
H. **Girder Stirrups** – Girder stirrups shall be field bent over the top mat of reinforcement in the deck slab.

Girder stirrups may be prebent, but the extended hook shall be within the core of the slab (the inside edge of the hook shall terminate above the bottom mat deck slab bars).

I. **Transformed Section Properties** – Transformed section properties shall not be used for design of prestressed concrete girders. Use of gross section properties remains WSDOT’s standard methodology for design of prestressed concrete girders including prestress losses, camber and flexural capacity.

In special cases, transformed section properties may be used for the design of prestressed concrete girders with the approval of the WSDOT Bridge Design Engineer. The live load factor at the Service III load combination shall be as follows:

- \( \gamma_{LL} = 0.8 \) when gross section properties are used
- \( \gamma_{LL} = 1.0 \) when transformed section properties are used

### 5.6.3 Fabrication and Handling

A. **Shop Plans** – Fabricators of prestressed concrete girders are required to submit shop plans which show specific details for each girder. These shop plans are reviewed for conformance with the Contract Plans and specifications.

B. **Special Problems for Fabricators**

1. **Strand Tensioning** – The method selected for strand tensioning may affect the design of the girders. The strand arrangements shown in the office standard plans and included in the PGSuper computer program are satisfactory for tensioning methods used by fabricators in this state. Harped strands are normally tensioned by pulling them as straight strands to a partial tension. The strands are then deflected vertically as necessary to give the required harping angle and strand stress. In order to avoid overtensioning the harped strands by this procedure, the slope of the strands is limited to a maximum of 6:1 for 0.5” \( \phi \) strands and 8:1 for 0.6”\( \phi \) strands. The straight strands are tensioned by straight jacking.

2. **Hold Down Forces** – Forces on the hold-down units are developed as the harped strands are raised. The hold-down device provided by the fabricator must be able to hold the vertical component of the harping forces. Normally a two or more hold-down unit is required. Standard commercial hold-down units have been preapproved for use with particular strand groups.

3. **Numbers of Strands** – Since the prestressing beds used by the girder fabricators can carry several girders in a line, it is desirable that girders have the same number of strands where practical. This allows several girders to be set up and cast at one time.

For pretensioned concrete girders, the number of permanent prestressing strands (straight and harped) shall be limited to 100 total 0.6”\( \phi \) strands.
C. Handling of Prestressed Concrete Girders

1. **In-Plant Handling** – The maximum weight that can be handled by precasting plants in the Pacific Northwest is 252 kips. Pretensioning lines are normally long enough so that the weight of a girder governs capacity, rather than its length. Headroom is also not generally a concern for the deeper sections.

2. **Lateral Stability during Handling** – The designer shall specify the lifting embedment locations (3’ minimum from ends - see *Standard Specifications* Section 6-02.3(25)L) and the corresponding concrete strength at release that provides an adequate factor of safety for lateral stability. The calculations shall conform to methods as described in references 2, 11, 12, 13. Recommended factors of safety of 1.0 against cracking, and 1.5 against failure shall be used.

Lateral stability can be a concern when handling long, slender girders. Lateral bending failures are sudden, catastrophic, costly, pose a serious threat to workers and surroundings, and therefore shall be considered by designers. When the girder forms are stripped from the girder, the prestressing level is higher and the concrete strength is lower than at any other point in the life of the member. Lifting embedment/support misalignment, horizontal girder sweep and other girder imperfections can cause the girder to roll when handling, causing a component of the girder weight to be resisted by the weak axis.

Lateral stability may be improved using the following methods:

a. Move the lifting embedments away from the ends. This may increase the required concrete release strength, because decreasing the distance between lifting devices increases the concrete stresses at the harp point. Stresses at the support may also govern, depending on the exit location of the harped strands.

b. Select a girder section that is relatively wide and stiff about its vertical (weak) axis.

c. Add temporary prestressing in the top flange.

d. Brace the girder.

e. Raise the roll axis of the girder with a rigid yoke.

For stability analysis of prestressed concrete girders during in-plant handling, in absence of more accurate information, the following parameters shall be used:

1. Height of pick point above top of girder = 0.0”

2. Lifting embedment transverse placement tolerance = 0.25”

3. Maximum girder sweep tolerance at midspan = 0.000521 in/in of total girder length
D. **Shipping Prestressed Concrete Girders**

1. **General** – The ability to ship girders can be influenced by a large number of variables, including mode of transportation, weight, length, height, and lateral stability. The ability to ship girders is also strongly site-dependent. For large or heavy girders, routes to the site shall be investigated during the preliminary design phase. To this end, on projects using large or heavy girders, WSDOT can place an advisory in their special provisions including shipping routes, estimated permit fees, escort vehicle requirements, Washington State Patrol requirements, and permit approval time.

2. **Mode of Transportation** – Three modes of transportation are commonly used in the industry: truck, rail, and barge. In Washington State, an overwhelming percentage of girders are transported by truck, so discussion in subsequent sections will be confined to this mode. However, on specific projects, it may be appropriate to consider rail or barge transportation.

   Standard rail cars can usually accommodate larger loads than a standard truck. Rail cars range in capacity from approximately 120 to 200 kips. However, unless the rail system runs directly from the precasting plant to the jobsite, members must be trucked for at least some of the route, and weight may be restricted by the trucking limitations.

   For a project where a large number of girders are required, barge transportation is usually the most economical. Product weights and dimensions are generally not limited by barge delivery, but by the handling equipment on either end. In most cases, if a product can be made and handled in the plant, it can be shipped by barge.

3. **Weight Limitations** – The net weight limitation with trucking equipment currently available in Washington State is approximately 190 kips, if a reasonable delivery rate (number of pieces per day) is to be maintained. Product weights of up to 252 kips can be hauled with currently available equipment at a limited rate.

   Long span prestressed concrete girders may bear increased costs due to difficulties encountered during fabrication, shipping, and erection. Generally, costs will be less if a girder can be shipped to the project site in one piece. However, providing an alternate spliced-girder design to long span one-piece pretensioned girders may reduce the cost through competitive bidding.

   When a spliced prestressed concrete girder alternative is presented in the Plans, the substructure shall be designed and detailed for the maximum force effect case only (no alternative design for substructure).

   Local carriers should be consulted on the feasibility of shipping large or heavy girders on specific projects.
4. **Support Locations** – The designer shall provide shipping support locations in the plans to ensure adequate girder stability. Shipping support locations shall be no closer than the girder depth to the ends of the girders or the girder centerline. The overhangs at the leading and trailing ends of the girders should be minimized and equal if possible. Generally, the leading end overhang should not exceed 15‘ to avoid interference with trucking equipment. Local carriers should be consulted if a larger leading end overhang is required. Shipping support locations shall maintain the concrete stresses within allowable limits.

Length between shipping support locations may be governed by turning radii on the route to the jobsite. Potential problems can be circumvented by moving the support points closer together (away from the ends of the girder), or by selecting alternate routes. Up to 130‘ between supports is typically acceptable for most projects.

5. **Height Limitations** – The height of a deep girder section sitting on a jeep and steerable trailer is of concern when considering overhead obstructions on the route to the jobsite. The height of the support is approximately 6‘ above the roadway surface. When adding the depth of the girder, including camber, the overall height from the roadway surface to the top of concrete can rapidly approach 14‘. Overhead obstructions along the route should be investigated for adequate clearance in the preliminary design phase. Obstructions without adequate clearance must be bypassed by selecting alternate routes.

Expectations are that, in some cases, overhead clearance will not accommodate the vertical stirrup projection on deeper WSDOT standard girder sections. Alternate stirrup configurations can be used to attain adequate clearance, depending on the route from the plant to the jobsite.

6. **Lateral Stability during Shipping** – The designer shall specify support locations in the Plans that provide an adequate factor of safety for lateral stability during shipping. The calculations shall conform to methods as described in references 2, 11, 12, 13. Recommended factors of safety of 1.0 against cracking, and 1.5 against failure (rollover of the truck) shall be used. See the discussion above on lateral stability during handling of prestressed concrete girders for suggestions on improving stability.

For lateral stability analysis of prestressed concrete girders during shipping, in absence of more accurate information, the following parameters shall be used:

a. Roll stiffness of entire truck/trailer system:

\[ K_\theta = \text{the maximum of} \left\{ \frac{28,000 \, \text{kip} \cdot \text{in}}{\text{rad}} \right\} \]

\[ \left( \frac{4,000 \, \text{kip} \cdot \text{in}}{\text{rad} \cdot \text{axle}} \right) \cdot N \]

Where:

- \( N = \) required number of axles = \( W_g/W_a \), rounded up to the nearest integer
- \( W_g = \) total girder weight (kip)
- \( W_a = \) 18 (kip/axle)
b. Height of girder bottom above roadway = 72"

     c. Height of truck roll center above road = 24"

     d. Center to center distance between truck tires = 72"

     e. Maximum expected roadway superelevation = 0.06

     f. Maximum girder sweep tolerance at midspan = 0.001042 in/in of total girder length

     g. Support placement lateral tolerance = ±1"

     h. Increase girder C.G. height over roadway by 2% for camber

E. **Erection** – A variety of methods are used to erect prestressed concrete girders, depending on the weight, length, available crane capacity, and site access. Lifting girders during erection is not as critical as when they are stripped from the forms, particularly when the same lifting devices are used for both. However, if a separate set of erection devices are used, the girder shall be checked for stresses and lateral stability. In addition, once the girder is set in place, the free span between supports is usually increased. Wind can also pose a problem. Consequently, when girders are erected, they shall immediately be braced. The temporary bracing of the girders is the contractor’s responsibility.

F. **Construction Sequence for Multi-Span Prestressed Concrete Girder Bridges** – For multi-span prestressed concrete girder bridges, the sequence and timing of the superstructure construction has a significant impact on the performance and durability of the bridge. In order to maximize the performance and durability, the “construction sequence” details shown in Appendix 5.6-A2 shall be followed for all new WSDOT multi-span prestressed concrete girder bridges. Particular attention shall be paid to the timing of casting the lower portion of the pier diaphragms/crossbeams (30 days minimum after girder fabrication) and the upper portion of the diaphragms/crossbeams (10 days minimum after placement of the deck slab). The requirements apply to multi-span prestressed concrete girder bridges with monolithic and hinge diaphragms/crossbeams.

**5.6.4 Superstructure Optimization**

A. **Girder Selection** – Cost of the girders is a major portion of the cost of prestressed concrete girder bridges. Much care is therefore warranted in the selection of girders and in optimizing their position within the structure. The following general guidelines should be considered.

1. **Girder Series Selection** – All girders in a bridge shall be of the same series unless approved otherwise by the Bridge and Structures Engineer. If vertical clearance is no problem, a larger girder series, utilizing fewer girder lines, may be a desirable solution.

   Fewer girder lines may result in extra reinforcement and concrete but less forming cost. These items must also be considered.
2. **Girder Concrete Strength** – Higher girder concrete strengths should be specified where that strength can be effectively used to reduce the number of girder lines, see Section 5.1.1.A.2. When the bridge consists of a large number of spans, consideration should be given to using a more exact analysis than the usual design program in an attempt to reduce the number of girder lines. This analysis shall take into account actual live load, creep, and shrinkage stresses in the girders.

3. **Girder Spacing** – Consideration must be given to the deck slab cantilever length to determine the most economical girder spacing. This matter is discussed in Section 5.6.4.B. The deck slab cantilever length should be made a maximum if a line of girders can be saved. It is recommended that the overhang length, from edge of slab to center line of exterior girder, be less than 40% of girder spacing; then the exterior girder can use the same design as that of the interior girder. The following guidance is suggested.

   a. **Tapered Spans** – On tapered roadways, the minimum number of girder lines should be determined as if all girder spaces were to be equally flared. As many girders as possible, within the limitations of girder capacity should be placed. Deck slab thickness may have to be increased in some locations in order to accomplish this.

   b. **Curved Spans** – On curved roadways, normally all girders will be parallel to each other. It is critical that the exterior girders are positioned properly in this case, as described in Section 5.6.4.B.

   c. **Geometrically Complex Spans** – Spans which are combinations of taper and curves will require especially careful consideration in order to develop the most effective and economical girder arrangement. Where possible, girder lengths and numbers of straight and harped strands should be made the same for as many girders as possible in each span.

   d. **Number of Girders in a Span** – Usually all spans will have the same number of girders. Where aesthetics of the underside of the bridge is not a factor and where a girder can be saved in a short side span, consideration should be given to using unequal numbers of girders. It should be noted that this will complicate crossbeam design by introducing torsion effects and that additional reinforcement will be required in the crossbeam.

B. **Bridge Deck Cantilevers** – The exterior girder location is established by setting the dimension from centerline of the exterior girder to the adjacent curb line. For straight bridges this dimension will normally be no less than 2′-6” for W42G, W50G, and W58G; 3′-0” for W74G; and 3′-6” for WF74G, WF83G, WF95G and WF100G. Some considerations which affect this are noted below.

   1. **Appearance** – Normally, for best appearance, the largest bridge deck overhang which is practical should be used.

   2. **Economy** – Fortunately, the condition tending toward best appearance is also that which will normally give maximum economy. Larger curb distances may mean that a line of girders can be eliminated, especially when combined with higher girder concrete strengths.
3. **Bridge Deck Strength** – It must be noted that for larger overhangs, the bridge deck section between the exterior and the first interior girder may be critical and may require thickening.

4. **Drainage** – Where drainage for the bridge is required, water from bridge drains is normally piped across the top of the girder and dropped inside of the exterior girder line. A large bridge deck cantilever length may severely affect this arrangement and it must be considered when determining exterior girder location.

5. **Bridge Curvature** – When straight prestressed concrete girders are used to support curved roadways, the curb distance must vary. Normally, the maximum bridge deck overhang at the centerline of the long span will be made approximately equal to the overhang at the piers on the inside of the curve. At the point of minimum curb distance, however, the edge of the girder top flange should be no closer than 1'0" from the bridge deck edge. Where curvature is extreme, other types of bridges should be considered. Straight girder bridges on highly curved alignments have a poor appearance and also tend to become structurally less efficient.

### C. Diaphragm Requirements

1. **General** – Diaphragms used with prestressed concrete girder bridges serve multiple purposes. During the construction stage, the diaphragms help to provide girder stability for the bridge deck placement. During the life of the bridge, the diaphragms act as load distributing elements, and are particularly advantageous for distribution of large overloads. Diaphragms also improve the bridge resistance to over-height impact loads.

Diaphragms for prestressed concrete girder bridges shall be cast-in-place concrete. Standard diaphragms and diaphragm spacings are given in the office standards for prestressed concrete girder bridges. For large girder spacings or other unusual conditions, special diaphragm designs shall be performed.

Inserts may be used to accommodate the construction of intermediate diaphragms for connections between the diaphragm and the web of prestressed concrete girders. The designer shall investigate the adequacy of the insert and the connection to develop the tensile capacity of diaphragm reinforcement. The designer shall also investigate the interface shear capacity of the diaphragm-to-web connections for construction and deck placement loads.

Open holes should be provided for interior webs so reinforcement can be placed through.

2. **Design** – Diaphragms shall be designed as transverse beam elements carrying both dead load and live load. Wheel loads for design shall be placed in positions so as to develop maximum moments and maximum shears.

3. **Geometry** – Diaphragms shall normally be oriented parallel to skew (as opposed to normal to girder centerlines). This procedure has the following advantages:
   a. The build-up of higher stresses at the obtuse corners of a skewed span is minimized. This build-up has often been ignored in design.
b. Skewed diaphragms are connected at points of approximately equal girder deflections and thus tend to distribute load to the girders in a manner that more closely meets design assumptions.

c. The diaphragms have more capacity as tension ties and compression struts are continuous. Relatively weak inserts are only required at the exterior girder.

On curved bridges, diaphragms shall normally be placed on radial lines.

4. **Full or Partial Depth Intermediate Diaphragms** – Full depth intermediate diaphragms as shown in the office standard plans shall be used for all deck bulb tee and wide flange deck girder superstructures.

Based on research done by WSU on damage by over-height loads (see reference 24), the use of intermediate diaphragms for I-shaped prestressed concrete girder bridges with CIP concrete bridge decks (including WF, wide flange thin deck, etc.) shall be as follows:

a. Full depth intermediate diaphragms as shown in the office standard plans shall be used for bridges crossing over roads of ADT > 50000.

b. Either full depth or partial depth intermediate diaphragms as shown in the office standard plans may be used for all bridges not included in item 1.

The use of full or partial depth intermediate diaphragms in bridge widenings shall be considered on a case-by-case basis depending on the width of the widening and number of added girders.

5. **Tub Girder Intermediate Diaphragms** – Intermediate diaphragms shall be provided both inside and between prestressed concrete tub girders.

The diaphragms inside the tub may be cast in the field or at the fabrication plant. The bottom of the diaphragm inside the tub shall be at least 3 inches above the top of the bottom flange.

The diaphragms between the tubs shall be cast in the field. For diaphragms between the tubs, the roughened surface or shear keys on the sloped web faces may not be effective in resisting interface shear. All diaphragm and construction loads on the diaphragm before the deck cures and gains strength will then be resisted by the reinforcement or inserts alone.

D. **Skew Effects** – Skew in prestressed concrete girder bridges affects structural behavior and member analysis and complicates construction.

1. **Analysis** – Normally, the effect of skew on girder analysis is ignored. It is assumed that skew has little structural effect on normal spans and normal skews. For short, wide spans and for extreme skews (values over 30°), the effect of the skew on structural action shall be investigated. All trapezoidal tub, slab, wide flange deck, wide flange thin deck and deck bulb-tee girders have a skew restriction of 30°.

Skews at ends of prestressed concrete girders cause prestressing strand force transfer to be unbalanced about the girder centerline at girder ends. In some cases, this effect has caused bottom flange cracking.
2. **Detailing** – To minimize labor costs and to avoid stress problems in prestressed concrete girder construction, the ends of girders for continuous spans shall normally be made skewed. Skewed ends of prestressed concrete girders shall always match the piers they rest on at either end.

E. **Grade and Cross Slope Effects** – Large cross slopes require an increased amount of the girder pad dimension (‘A’ dimension) necessary to ensure that the structure can be built. This effect is especially pronounced if the bridge is on a horizontal or vertical curve. Care must be taken that deck drainage details reflect the cross slope effect.

Girder lengths shall be modified for added length along grade slope.

F. **Curve Effect and Flare Effect** – Curves and tapered roadways each tend to complicate the design of straight girders. The designer must determine what girder spacing to use for dead load and live load design and whether or not a refined analysis, that considers actual load application, is warranted. Normally, the girder spacing at centerline of span can be used for girder design, especially in view of the conservative assumptions made for the design of continuous girders.

G. **Girder Pad Reinforcement** – Girders with a large “A” dimension may require a deep pad between the top of the girder and the bottom of the deck. When the depth of the pad at the centerline of the girder exceeds 6”, reinforcement shall be provided in the pad as shown in Figure 5.6.4-1.

![Girder Pad Reinforcement](image)

**Figure 5.6.4-1**
5.6.5 **Repair of Damaged Girders at Fabrication**

When girders suffer defects during fabrication or damage before becoming part of a final structure, the girder repairs shall be addressed with pre-approved repair procedures from the current Annual Plant Approval document for the fabricator (see *Standard Specifications* Section 6-02.3(25)A). If the repairs cannot be addressed by this document, the fabricator will initiate the Fax Resolution process from the current Annual Plant Approval document to address contract specific repairs with the Project Office and HQ Bridge Construction. Normally, no designer action is required. When evaluating repairs for unusual situations not covered, the designer must ensure that the required strength and appearance of the girder can be maintained. If stressing will occur after the repair is made, normally no test loading is required; however, such a test should be considered. See reference 14 for guidance.

5.6.6 **Repair of Damaged Girders in Existing Bridges**

A. **General** – This section is intended to cover repair of damaged girders on existing bridges. For repair of newly constructed girders, see Section 5.6.5. Overheight loads are a fairly common source of damage to prestressed concrete girder bridges. The damage may range from spalling and minor cracking of the lower flange of the girder to loss of a major portion of a girder section. Occasionally, one or more strands may be broken. The damage is most often inflicted on the exterior or first interior girder.

B. **Repair Procedure** – The determination of the degree of damage to a prestressed concrete girder is largely a matter of judgment. Where the flange area has been reduced or strands lost, calculations can aid in making this judgment decision. The following are general categories of damage and suggested repair procedures 15, 16.

1. **Minor Damage** – If the damage is slight and concerns only spalling of small areas of the outside surface of the concrete, repair may be accomplished by replacing damaged concrete areas with concrete grout. The area where new concrete is to be applied shall first be thoroughly cleaned of loose material, dried, and then coated with epoxy.

2. **Moderate Damage** – If damage is moderate, consisting of loss of a substantial portion of the flange and possibly loss of one or more strands, a repair procedure must be developed using the following guidelines. It is probable that some prestress will have been lost in the damaged area due to reduction in section and consequent strand shortening or through loss of strands. The following repair procedure is recommended to assure that as much of the original girder strength as possible is retained:
   
   a. **Determine Condition** – Sketch the remaining cross section of the girder and compute its reduced section properties. Determine the stress in the damaged girder due to the remaining prestress and loads in the damaged state. If severe overstresses are found, action must be taken to restrict loads on the structure until the repair has been completed. If the strand loss is so great that AASHTO prestress requirements cannot be met with the remaining strands, consideration should be given to replacing the girder.
b. **Restore Prestress If Needed** – If it is determined that prestress must be restored, determine the stress in the bottom fiber of the girder as originally designed due to $DL + LL + I + \text{Prestress}$. (This will normally be about zero psi). Determine the additional load ($P$) that, when applied to the damaged girder in its existing condition, will result in this same stress. Take into account the reduced girder section, the effective composite section, and any reduced prestress due to strand loss. Should the damage occur outside of the middle one-third of the span length, the shear stress with the load ($P$) applied should also be computed. Where strands are broken, consideration should be given to coupling and jacking them to restore their prestress.

c. **Prepare a Repair Plan** – Draw a sketch to show how the above load is to be applied and specify that the damaged area is to be thoroughly prepared, coated with epoxy, and repaired with grout equal in strength to the original concrete. Specify that this load is to remain in place until the grout has obtained sufficient strength. The effect of this load is to restore lost prestress to the strands which have been exposed.

d. **Test Load** – Consideration should be given to testing the repaired girder with a load equivalent to $1.0DL + 1.5(LL + IM)$. The $LL$ Live Load for test load is HL-93.

3. **Severe Damage** – Where the damage to the girder is considered to be irreparable due to loss of many strands, extreme cracking, etc., the girder may need to be replaced. This has been done several times, but involves some care in determining a proper repair sequence.

   In general, the procedure consists of cutting through the existing deck slab and diaphragms and removing the damaged girder. Adequate exposed reinforcement steel must remain to allow splicing of the new bars. The new girder and new reinforcement is placed and previously cut concrete surfaces are cleaned and coated with epoxy. New deck slab and diaphragm portions are then poured.

   It is important that the camber of the new girder be matched with that in the old girders. Excessive camber in the new girder can result in inadequate deck slab thickness. Girder camber can be controlled by prestress, curing time, or dimensional changes.

   Pouring the new deck slab and diaphragms simultaneously in order to avoid overloading the existing girders in the structure should be considered. Extra bracing of the girder at the time of deck slab pour shall be required.

   Methods of construction shall be specified in the plans that will minimize inconvenience and dangers to the public while achieving a satisfactory structural result. High early strength grouts and concretes should be considered.

   In case of replacement of a damaged girder, the intermediate diaphragms adjacent to the damaged girder shall be replaced with full depth diaphragms as shown in Figure 5.6.6-1.

   In case of replacement of a damaged girder, the replacement girder shall preferably be the same type as the original damaged girder.
In case of repair of a damaged girder with broken or damaged prestressing strands, the original damaged strands shall be replaced with similar diameter strands. Restoration of the prestress force as outlined in BDM 5.6.6 B-2b shall be considered.

Existing bridges with pigmented sealer shall have replacement girders sealed. Those existing bridges without pigmented sealer need not be sealed.

**Figure 5.6.6-1**

Full Depth Intermediate Diaphragm Replacement

*ANCHOR DETAIL*

ASTM A-307
4. **Repair vs. Replacement of Damaged Girder** – Several factors need to be considered when evaluating whether to repair or to replace a damaged girder. Among them are the level of concrete damage, number of broken strands, location and magnitude of web damage, permanent offset of the original girder alignment, and overall structural integrity. Other considerations include fresh damage to previously damaged girders, damage to adjacent girders, and cost of repair versus replacement. Ultimately, the evaluation hinges on whether the girder can be restored to its original capacity and whether the girder can be repaired sufficiently to carry its share of the original load.

The following guidelines describe damaged girder conditions which require replacement:

- **Strand Damage** – More than 25% of prestressing strands are damaged/severed. If over 25% of the strands have been severed, replacement is required. Splicing is routinely done to repair severed strands. However, there are practical limits as to the number of couplers that can be installed in the damaged area.

- **Girder Displacements** – The bottom flange is displaced from the horizontal position more than ½” per 10’ of girder length. If the alignment of the girder has been permanently altered by the impact, replacement is required. Examples of non-repairable girder displacement include cracks at the web/flange interface that remain open. Abrupt lateral offsets may indicate that stirrups have yielded. A girder that is permanently offset may not be restorable to its original geometric tolerance by practical and cost-effective means.

- **Concrete Damage at Harping Point** – Concrete damage at harping point resulting in permanent loss of prestress. Extreme cracking or major loss of concrete near the harping point may indicate a change in strand geometry and loss in prestress force. Such loss of prestress force in the existing damaged girder cannot be restored by practical and cost-effective means, and requires girder replacement.

- **Concrete Damage at Girder Ends** – Severe concrete damage at girder ends resulting in permanent loss of prestress or loss of shear capacity. Extreme cracking or major loss of concrete near the end of a girder may indicate unbonding of strands and loss in prestress force or a loss of shear capacity. Such loss of prestress force or shear capacity in the existing damaged girder cannot be restored by practical and cost-effective means, and requires girder replacement.

There are other situations as listed below which do not automatically trigger replacement, but require further consideration and analysis.

- **Significant Concrete Loss** – For girder damage involving significant loss of concrete from the bottom flange, consideration should be given to verifying the level of stress remaining in the exposed prestressing strands. Residual strand stress values will be required for any subsequent repair procedures.
• **Adjacent Girders** – Capacity of adjacent undamaged girders. Consideration must be given as to whether dead load from the damaged girder has been shed to the adjacent girders and whether the adjacent girders can accommodate the additional load.

• **Previously Damaged Girders** – Damage to a previously damaged girder. An impact to a girder that has been previously repaired may not be able to be restored to sufficient capacity.

• **Cost** – Cost of repair versus replacement. Replacement may be warranted if the cost of repair reaches 70% of the replacement project cost.

C. **Miscellaneous References** – The girder replacement contracts and similar jobs listed in Table 5.6.6-1 should be used for guidance:

<table>
<thead>
<tr>
<th>Contract</th>
<th>Project Name</th>
<th>Bridge Number</th>
<th>Total Bridge Length (ft)</th>
<th>Year work planned</th>
<th>Work Description</th>
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<tr>
<td>C-7425</td>
<td>I-5 Bridge 005/518 Girder Replacement</td>
<td>5/518</td>
<td>322</td>
<td>2008</td>
<td>Replace damaged PCG</td>
</tr>
<tr>
<td>C-7637</td>
<td>SR 520/ W Lake Sammamish Pkwy To SR 202 HOV And SR</td>
<td>11/1</td>
<td>287</td>
<td>2009</td>
<td>Replace damaged PCG in one span</td>
</tr>
<tr>
<td>C-7095</td>
<td>SR 14, Lieser Road Bridge Repair</td>
<td>14/12</td>
<td>208</td>
<td>2006</td>
<td>Replace damaged PCG</td>
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<td>C-7451</td>
<td>I-90 Bridge No. 90/121- Replace Portion Of Damaged</td>
<td>90/121</td>
<td>250</td>
<td>2007</td>
<td>Replace damaged PCG</td>
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<tr>
<td>C-7567</td>
<td>Us395 Col Dr Br &amp; Court St Br - Bridge Repair</td>
<td>395/103</td>
<td>114</td>
<td>2008</td>
<td>Replace damaged PCG</td>
</tr>
<tr>
<td>C-7774</td>
<td>SR 509, Puyallup River Bridge Special Repairs</td>
<td>509/11</td>
<td>3584</td>
<td>2010</td>
<td>Replace fire damage PCG span</td>
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<td>C-9593</td>
<td>Columbia Center IC Br. 12/432(Simple Span)</td>
<td>Repair</td>
<td></td>
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<td>16th Avenue IC Br. 12/344 (Continuous Span)</td>
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<td>Golden Givens Road Bridge 512/10</td>
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<tr>
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<td>Anderson Hill Road Bridge 3/130W</td>
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</tr>
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</table>

**Girder Replacement Contracts**

*Table 5.6.6-1*
5.6.7 Deck Girders

A. General – The term “deck girder” refers to a girder whose top flange or surface is the driving surface, with or without an overlay. They include slab, double-tee, ribbed, deck bulb-tee, wide flange deck and wide flange thin deck girders.

Deck girders without a composite CIP concrete deck shall have a minimum concrete cover of 2” over the top mat. The top mat of reinforcement in the deck (top flange) shall be epoxy-coated.

B. Slab Girders – Slab girder spans between centerline bearings shall be limited to the prestressed concrete girder height multiplied by 30 due to unexpected variations from traditional beam camber calculations.

Standard configurations of slab girders are shown in the girder standard plans. The width of slab girders should not exceed 8’-0”.

A minimum 5” composite CIP bridge deck shall be placed over slab girders. The CIP concrete deck shall be either Class 4000D concrete with one layer of #4 epoxy coated reinforcement in both the transverse and longitudinal directions spaced at 1’-0” maximum. Welded ties are still required.

The AASHTO LRFD 2.5.2.6.2 deflection criteria shall be satisfied for slab girders.

A minimum of two permanent top strands shall be provided for slab girders, one adjacent to each edge. Additional permanent top strands can be used if required to control girder end tensile stresses as well as concrete stresses due to plant handling, shipping and erection.

In some cases it may be necessary to use temporary top strands to control girder end tensile stresses as well as concrete stresses due to plant handling, shipping and erection. These strands shall be bonded for 10’ at both ends of the girder, and unbonded for the remainder of the girder length. Temporary strands shall be cut prior to equalizing girders and placing the CIP bridge deck. Designers may also consider other methods to control girder stresses including debonding permanent strands at girder ends and adding mild steel reinforcement.

The specified design compressive strength (f’c) of slab girders should be kept less than or equal to 8 ksi to allow more fabricators to bid.

C. Double-Tee and Ribbed Deck Girders – Double-tee and ribbed deck girders shall be limited to widening existing similar structures. An HMA overlay with membrane shall be specified. These sections are capable of spanning up to 60’.

D. Deck Bulb-Tee Girders – Deck bulb-tee girders have standard girder depths of 35, 41, 53, and 65 inches. The top flange/deck may vary from 4-feet 1-inch to 6-feet wide. They are capable of spanning up to 145 feet. Deck bulb-tee girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

Deck bulb-tee girders shall have an HMA or concrete overlay. A waterproofing membrane shall be provided with an HMA overlay.

Deck bulb-tee girders shall be installed with girder webs plumb. Bridge deck superelevation shall be accommodated by varying the top flange thickness.
Superelevation should be limited so that lifting embedments can be located at the center of gravity of the girder to prevent complications with lifting, hauling and erection. Use of deck bulb-tee girders should be avoided when superelevation transitions occur within the span.

Girder size and weight shall be evaluated for shipping and hauling to the project site.

E. **Wide Flange Deck Girders** – Wide flange deck girders have standard girder depths ranging from 39 inches to 103 inches. The top flange/deck may vary from 5-feet to 8-feet wide. They are capable of spanning up to 195 feet for a 5-foot top flange width and 165 feet for an 8-foot top flange width. Wide flange deck girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

Wide flange deck girders shall have an HMA or concrete overlay. A waterproofing membrane shall be provided with an HMA overlay.

Wide flange deck girders shall be installed with girder webs plumb. Bridge deck superelevation shall be accommodated by varying the top flange thickness. Superelevation should be limited so that lifting embedments can be located at the center of gravity of the girder to prevent complications with lifting, hauling and erection. Use of wide flange deck girders should be avoided when superelevation transitions occur within the span.

Girder size and weight shall be evaluated for shipping and hauling to the project site.

F. **Wide Flange Thin Deck Girders** – Wide flange thin deck girders have standard girder depths ranging from 36 inches to 100 inches. The top flange may vary from 5-feet to 8-feet wide. They are capable of spanning up to 225 feet for a 5-foot top flange width and 200 feet for an 8-foot top flange width.

Wide flange thin deck girders shall have a minimum 5” thick composite CIP bridge deck. Welded ties and grouted keys at flange edges are not required. The CIP bridge deck thickness shall be capable of accommodating expected girder camber variations and tolerances. Transverse reinforcement in the CIP bridge deck shall be designed to resist live loads and superimposed dead loads. Additional transverse reinforcement may be necessary over the flange edge joints to help control concrete cracking in the CIP bridge deck.

Wide flange thin deck girders shall be installed with girder webs plumb. Bridge deck superelevation shall be accommodated by varying the CIP bridge deck thickness. Use of wide flange thin deck girders should be avoided with large superelevations in order to limit CIP bridge deck thickness.

Girder size and weight shall be evaluated for shipping and hauling to the project site.
5.6.8 Prestressed Concrete Tub Girders

A. General – Prestressed concrete tub girders (U and UF sections) are an option for moderate bridge spans.

The standard tub girders (U sections) have 4'-0" or 5'-0" bottom flange widths and are 4'-6", 5'-6" or 6'-6" deep. A 6" deep top flange can be added to tub girders (UF sections) to improve structural efficiency and to accommodate placement of stay-in-place precast deck panels.

Drain holes shall be provided at the low point of the tub girders at the centerline of the bottom flange.

B. Curved Tub Girders – Curved tub girders may be considered for bridges with moderate horizontal radiuses. I-girders may not be curved.

Curved tub girders can either be designed in one piece or in segments depending on span configurations and shipping limitations. Curved tub girders are post-tensioned at the fabrication plant and shipped to the jobsite. Additional jobsite post-tensioning may be required if segment assembly is necessary, or if continuity over intermediate piers is desired. Closure joints at segment splices shall meet the requirements of Section 5.9.4.C.

The following limitations shall be considered for curved tub girders:

1. The overall width of curved segments for shipment shall not exceed 16 feet.
2. The location of the shipping supports shall be carefully studied so that the segment is stable during shipping. The difference in dead load reactions of the shipping supports within the same axle shall not exceed 5 percent.
3. The maximum shipping weight of segments may be different depending on the size of the segments. The shipping weight shall meet the legal axle load limits set by the RCW, but in no case shall the maximum shipping weight exceed 275 kips.
4. The minimum web thickness shall be 10". Other cross-sectional dimensions of WSDOT standard tub girders are applicable to curved tub girders.
5. Effects of curved tendons shall be considered per Section 5.8.1.F.
6. The clear spacing between ducts shall be 2" min. The duct diameter shall not exceed 4½".
5.6.9  **Prestressed Concrete Girder Checking Requirement**

A. Shear reinforcing size and spacing shall be determined by the designer.

B. Determine lifting location and required concrete strength at release to provide adequate stability during handling. Generally temporary strands provide additional stability for lifting and transportation, and reduce the camber. Less camber allows for less “A” dimension and concrete pad dead weight on the structure. Temporary strands are cut after the girders are erected and braced and before the intermediate diaphragms are cast.

C. Due to the extreme depth of the WF83G, WF95G, and WF100G girders, and possible tilt at the piers for profile grades, the designer will need to pay particular attention to details to assure the girders will fit and perform as intended.

D. Check edge distance of supporting cross beam.

5.6.10  **Review of Shop Plans for Pretensioned Girders**

Pretensioning shop drawings shall be reviewed by the designer. Shop drawings, after review by the designer, shall be stamped with the official seal and returned to the bridge construction support office. The review must include:

A. All prestressing strands shall be of ½” or 0.6” diameter grade 270 low relaxation uncoated strands.

B. Number of strands per girder.

C. Jacking stresses of strands shall not exceed $0.75f_{pu}$.

D. Strand placement patterns and harping points.

E. Temporary strand pattern, bonded length, location and size of blockouts for cutting strands.

F. Procedure for cutting temporary strands and patching the blockouts shall be specified.

G. Number and length of extended strands and rebars at girder ends.

H. Locations of holes and shear keys for intermediate and end diaphragms.

I. Location and size of bearing recesses.

J. Saw tooth at girder ends.

K. Location and size of lifting loops or lifting bars.

L. All horizontal and vertical reinforcement.

M. Girder length and end skew.
5.7 **Bridge Decks**

Concrete bridge decks shall be designed using the Traditional Design of AASHTO LRFD 9.7.3 as modified by this section.

The following information is intended to provide guidance for bridge deck thickness and transverse and longitudinal reinforcement of bridge decks. Information on deck protection systems is given in Section 5.7.4.

5.7.1 **Bridge Deck Requirements**

A. **Minimum Bridge Deck Thickness** – The minimum bridge deck thickness (including 0.5” wearing surface) shall be 7.5” for concrete bridges, 8.0” for steel girder bridges, and 8.5” for concrete girder bridges with SIP deck panels. This minimum bridge deck thickness may be reduced by 0.5” for bridges with Deck Protection Systems 2, 3 and 5.

The minimum CIP bridge deck thickness for prestressed concrete slab girders is 5”.

Minimum bridge deck thicknesses are established in order to ensure that overloads will not result in premature bridge deck cracking.

The minimum clearance between top and bottom reinforcing mats shall be 1”.

B. **Computation of Bridge Deck Strength** – The design thickness for usual bridge decks are shown in Figures 5.7.1-1 & 2.

The thickness of the bridge deck and reinforcement in the area of the cantilever may be governed by traffic barrier loading. Wheel loads plus dead load shall be resisted by the sections shown in Figure 5.7.1-2.

Design of the cantilever is normally based on the expected depth of the bridge deck at centerline of girder span. This is usually less than the dimensions at the girder ends.

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**Diagram:**

**SLAB THICKNESS**

**DIMENSION "A" AT CENTERLINE OF GIRDLE BEARING.**

**Depths for Bridge Deck Design at Interior Girder**

*Figure 5.7.1-1*
C. **Computation of “A” Dimension** – The distance from the top of the bridge deck to the top of the girder at centerline bearing at centerline of girder is represented by the “A” Dimension. It is calculated in accordance with the guidance of Appendix 5-B1. This ensures that adequate allowance will be made for excess camber, transverse deck slopes, vertical and horizontal curvatures. Where temporary prestress strands at top of girder are used to control the girder stresses due to shipping and handling, the “A” dimension must be adjusted accordingly.

The note in the left margin of the layout sheet shall read: “A” Dimension = X” (not for design).

### 5.7.2 Bridge Deck Reinforcement

A. **Transverse Reinforcement** – The size and spacing of transverse reinforcement may be governed by interior bridge deck span design and cantilever design. Where cantilever design governs, short hooked bars may be added at the bridge deck edge to increase the reinforcement available in that area. Top transverse reinforcement is always hooked at the bridge deck edge unless a traffic barrier is not used. Top transverse reinforcement is preferably spliced at some point between girders in order to allow the clearance of the hooks to the bridge deck edge forms to be properly adjusted in the field. Usually, the bridge deck edge hooks will need to be tilted in order to place them. On larger bars, the clearance for the longitudinal bar through the hooks shall be checked. Appendices 5.3-A5 through 5.3-A8 can be used to aid in selection of bar size and spacing.

For skewed spans, the transverse bars are placed normal to bridge centerline and the areas near the expansion joints and bridge ends are reinforced by partial length bars. For raised crossbeam bridges, the bottom transverse bridge deck reinforcement is discontinued at the crossbeam.

The spacing of bars over the crossbeam must be detailed to be large enough to allow concrete to be poured into the crossbeam. For typical requirements, see Section 5.3.3.D.
For bridge decks with a crowned roadway, the bottom surface and rebar shall be flat, as shown in Figure 5.7.2-1.

**Figure 5.7.2-1**

**Bottom of Bridge Deck at Crown Point**

B. **Longitudinal Reinforcement** – This section discusses reinforcement requirements for resistance of longitudinal moments in continuous multi-span prestressed concrete girder bridges and is limited to reinforcement in the bridge deck since capacity for resisting positive moment is provided by the girder reinforcement.

1. **Simple Spans** – For simple span bridges, longitudinal bridge deck reinforcement is not required to resist negative moments and therefore the reinforcement requirements are nominal. Figure 5.7.2-2 defines longitudinal reinforcement requirements for these decks. The bottom longitudinal reinforcement is defined by AASHTO LRFD 9.7.3.2 requirements for distribution reinforcement. The top longitudinal reinforcement is based on current office practice.

**Figure 5.7.2-2**

**Nominal Longitudinal Bridge Deck Reinforcement**
2. **Continuous Spans** – Continuity reinforcement shall be provided at supports for loads applied after establishing continuity. The longitudinal reinforcement in the bridge deck at intermediate piers is dominated by the negative moment requirement. Where these bars are cut off, they are lapped by the nominal top longitudinal reinforcement described in Section 5.7.2.D. The required bridge deck thickness for various bar combinations is shown in Table 5.7.2-1.

C. **Distribution of Flexural Reinforcement** – The provision of AASHTO LRFD 5.7.3.4 for class 2 exposure condition shall be satisfied for both the top and bottom faces of the bridge deck.

<table>
<thead>
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<th>Longitudinal Bar</th>
<th>#5</th>
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<th>#7</th>
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<tr>
<td>#4</td>
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<td>--</td>
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<tr>
<td>#5</td>
<td>7½</td>
<td>7½</td>
<td>7¾</td>
</tr>
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<td>8¾</td>
<td>--</td>
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</tr>
</tbody>
</table>

**Note:**
Deduct ½" from minimum bridge deck thickness shown in table when an overlay is used.

**Minimum Bridge Deck Thickness for Various Bar Sizes**

*Table 5.7.2-1*
D. **Bar Patterns** – Figure 5.7.2-3 shows two typical top longitudinal reinforcing bar patterns. Care must be taken that bar lengths conform to the requirements of Section 5.1.2.

The symmetrical bar pattern shown should normally not be used when required bar lengths exceed 60 feet. If the staggered bar pattern will not result in bar lengths within the limits specified in Section 5.1.2, the method shown in Figure 5.7.2-4 may be used to provide an adequate splice. All bars shall be extended by their development length beyond the point where the bar is required.

Normally, no more than 33% of the total area of main reinforcing bars at a support (negative moment) or at midspan (positive moment) shall be cut off at one point. Where limiting this value to 33% leads to severe restrictions on the reinforcement pattern, an increase in figure may be considered. Two reinforcement bars shall be used as stirrup hangers.
E. **Concrete Bridge Deck Design and Detailing** – These requirements are primarily for beam-slab bridges with main reinforcement perpendicular to traffic:

- Minimum cover over the top layer of reinforcement shall be 2.5" including 0.5" wearing surface (Deck Protection Systems 1 and 4). The minimum cover over the bottom layer reinforcement shall be 1.0".
- The minimum clearance between top and bottom reinforcing mats shall be 1".
- A maximum bar size of #5 is preferred for longitudinal and transverse reinforcement in the bridge deck except that a maximum bar size of #7 is preferred for longitudinal reinforcement at intermediate piers.
- The minimum amount of reinforcement in each direction shall be 0.18 in.²/ft for the top layer and 0.27 in.²/ft for the bottom layer. The amount of longitudinal reinforcement in the bottom of bridge decks shall not be less than \( \frac{220}{380} \) percent of the positive moment as specified in AASHTO LRFD 9.7.3.2.
- Top and bottom reinforcement in longitudinal direction of bridge deck shall be staggered to allow better flow of concrete between the reinforcing bars.
- The maximum bar spacing in transverse and longitudinal directions for the top mat, and transverse direction of the bottom mat shall not exceed 12". The maximum bar spacing for bottom longitudinal within the effective length, as specified in AASHTO LRFD 9.7.2.3, shall not exceed the deck thickness.
- Allow the Contractor the option of either a roughened surface or a shear key at the intermediate pier diaphragm construction joint.
• Both, top and bottom layer reinforcement shall be considered when designing for negative moment at the intermediate piers.
• Reduce lap splices if possible. Use staggered lap splices for both top and bottom in longitudinal and transverse directions.

5.7.3 Stay-in-place Deck Panels

A. General – The use of precast, prestressed stay-in-place (SIP) deck panels for bridge decks may be investigated at the preliminary design stage. The acceptance evaluation will consider such items as extra weight for seismic design and the resulting substructure impacts.

The composite deck system consisting of precast prestressed concrete deck panels with a CIP topping has advantages in minimizing traffic disruption, speeding up construction and solving constructability issues on certain projects. Contractors, in most cases, prefer this composite deck panel system for bridge decks in traffic congested areas and other specific cases.

SIP deck panels may be used on WSDOT bridges with WSDOT Bridge and Structures Office approval. Details for SIP deck panels are shown in Appendix 5.6-A10-1.

Steel deck forms are not permitted in order to allow inspection of deck soffits and to avoid maintenance of a corrosion protection system.

B. Design Criteria – The design of SIP deck panels follows the AASHTO LRFD Specifications and the PCI Bridge Design Manual. The design philosophy of SIP deck panels is identical to simple span prestressed concrete girders. They are designed for Service Limit State and checked for Strength Limit State. The precast panels support the dead load of deck panels and CIP topping, and the composite SIP deck panel and CIP cross-section resists the live load and superimposed dead loads. The tensile stress at the bottom of the panel is limited to zero per WSDOT design practice.

C. Limitations on SIP Deck Panels – The conventional full-depth CIP bridge deck shall be used for most applications. However, the WSDOT Bridge and Structures Office may allow the use of SIP deck panels with the following limitations:

1. SIP deck panels shall not be used in negative moment regions of continuous conventionally reinforced bridges. SIP deck panels may be used in post-tensioned continuous bridges.

2. Bridge widening. SIP deck panels are not allowed in the bay adjacent to the existing structure because it is difficult to set the panels properly on the existing structure, and the requirement for a CIP closure. SIP deck panels can be used on the other girders when the widening involves multiple girders.

3. Phased construction. SIP deck panels are not allowed in the bay adjacent to the previously placed deck because of the requirement for a CIP closure.

4. Prestressed concrete girders with narrow flanges. Placement of SIP deck panels on girders with flanges less than 12” wide is difficult.
5. A minimum bridge deck thickness of 8.5”, including 3.5” precast deck panel and 5” CIP concrete topping shall be specified.

6. SIP deck panels are not allowed for steel girder bridges.

5.7.4 Bridge Deck Protection

All bridge decks, precast or cast-in-place slabs, or deck girder structures shall use a deck protection system as described in this section to reduce the deterioration of the bridge deck and superstructure. The WSDOT Bridge Management Unit shall determine the type of protection system during the preliminary plan or Request For Proposal (RFP) stage for structures not described in this section. Special conditions (i.e. a widening) where it may be desirable to deviate from the standard deck protection systems require approval of the WSDOT Bridge Management Unit.

Preliminary plans shall indicate the protection system in the left margin per BDM Section 2.3.8.

Saw cutting or grinding pavement items are not allowed on the bridge decks. Rumble strips and recessed pavement markers shall not be placed on bridge decks, or approach slab surfaces whether they are concrete or asphalted as stated in Section 8-08 and 8-09 of the Standard Specifications, respectively.

Traffic detection loops shall not be located in an existing bridge surface. They may be installed during the construction of bridge decks prior to placing the deck concrete in accordance with Standard Plan J-50.16.

A. Deck Protection Systems – The following paragraphs describe five WSDOT protective systems used to protect a concrete bridge deck design.

1. Type 1 Protection System – This is the minimum default protection system for cases where a protection system has not been specified on a structure. Type 1 protection system shall be used for cast-in-place bridge decks with two layers of reinforcement, see Figure 5.7.4-1. This also applies to CIP slab bridges, deck replacements and the widening of existing decks. System 1 consists of the following:

a. A minimum 2 1/2” of concrete cover over top bar of deck reinforcing. The cover includes a ½” wearing surface and ¼” tolerance for the placement of the reinforcing steel.

b. Both the top and bottom mat of deck reinforcing shall be epoxy-coated.

c. Girder stirrups and horizontal shear reinforcement do not require epoxy-coating.
2. **Type 2 Protection System** – This protection system consists of concrete overlays, see Figure 5.7.4-2. Concrete overlays are generally described as a 1.5" unreinforced layer of modified concrete used to rehabilitate an existing deck. Overlay concrete is modified to provide a low permeability that slows or prevents the penetration of water into the bridge deck, but also has a high resistance to rutting.

WSDOT Bridge Management Unit shall determine the type of concrete overlay placed on all new or existing decks; and may specify similar overlays such as a polyester or RSLMC in special cases when rapid construction is cost effective. Brief descriptions of common overlays are as follows.

a. **1½” Modified Concrete Overlay** – These overlays were first used by WSDOT in 1979 and have an expected life between 20-40 years. There are more than 600 bridges with concrete overlays as of 2010. This is the preferred overlay system for deck rehabilitation that provides long-term deck protection and a durable wearing surface. In construction, the existing bridge deck is hydromilled ½” prior to placing the 1.5” overlay. This requires the grade to be raised 1”. The modified concrete overlay specifications allow a contractor to choose between a Latex, Microsilica or Fly ash mix design. Construction requires a deck temperature between 45°F - 75°F with a wind speed less than 10 mph. Traffic control can be significant since the time to construct and cure is 42 hours.
b. **¾” Polyester Modified Concrete Overlay** – These overlays were first used by WSDOT in 1989 and have an expected life between 20-40 years with more than 20 overlay as of 2010. This type of overlay uses specialized polyester equipment and materials. Construction requires dry weather with temperatures above 50°F and normally cures in 4 hours. A polyester concrete overlay may be specified in special cases when rapid construction is needed.

c. **1½” Rapid Set Latex Modified Concrete Overlay** – A rapid set latex modified concrete (RSLMC) overlay uses special cement manufactured by the CTS Company based in California. RSLMC is mixed in a mobile mixing truck and applied like a regular concrete overlay. The first RSLMC overlay was applied to bridge 162/20 South Prairie Creek in 2002 under contract 016395. Like polyester, this overlay cures in 4 hours and may be specified in special cases when rapid construction is needed.

d. **½” Thin Polymer Overlay** – Thin polymer overlays are built up layers of a polymer material with aggregate broad cast by hand. The first thin overlay was placed in 1986 and after placing 25 overlays, they were discontinued in the late 1998 due to poor performance.

![Diagram of Concrete Overlay](image)

**Type 2 Protection System**

**Figure 5.7.4-2**

3. **Type 3 Protection System** – This protection system consists of a Hot Mixed Asphalt (HMA) overlay wearing surface and requires the use of a waterproofing membrane, see Figure 5.7.4-3. HMA overlays provide a lower level of deck protection and introduce the risk of damage by planing equipment during resurfacing. Asphalt overlays with a membrane were first used on a WSDOT bridges in 1971 and about ⅓ of WSDOT structures have HMA. The bridge HMA has an expected life equal to the roadway HMA when properly constructed.

Waterproof membranes are required with the HMA overlay. Unlike roadway surfaces, the HMA material collects and traps water carrying salts and oxygen at the concrete surface deck. This is additional stress to an epoxy protection system or a bare deck and requires a membrane to mitigate the penetration of salts and oxygen to the structural reinforcement and cement paste. See *Standard Specifications* for more information on waterproof membranes.
HMA overlays may be used in addition to the Type 1 Protection System for new bridges where it is desired to match roadway pavement materials. New bridge designs using HMA shall have a depth of overlay between 0.15’ (1.8”) and 0.25’ (3”). Designers should consider designing for a maximum depth of 0.25’ to allow future overlays to remove and replace 0.15’ HMA without damaging the concrete cover or the waterproof membrane. Plan sheet references to the depth of HMA shall be in feet, since this is customary for the paving industry. WSDOT roadway resurfacing operations will normally plane and pave 0.15’ of HMA which encourages the following design criteria.

Existing structures may apply an HMA overlay in accordance with the Bridge Paving Policies, Section 5.7.5.

Standard Plan A-40.20.00, Bridge Transverse Joints Seals for HMA provides some standard details for saw cutting small relief joints in HMA paving. Saw cut joints can have a longer life, better ride, and help seal the joint at a location known to crack and may be used for small bridge expansion joints less than 1 inch.

WSDOT prohibits the use of a Type 3 Protection System for prestressed concrete slab girder or deck girder bridges managed by WSDOT except for pedestrian bridges or for widening existing similar structures with an HMA overlay. The HMA with membrane provides some protection to the connections between girders, but can be prone to reflective cracking at the joints. It is not uncommon for voided slabs to fill with water and aggressively corrode the reinforcement. Prestressed concrete members with a Type 3 Protection System shall have a minimum cover of 2” over an epoxy coated top mat.
4. **Type 4 Protection System** – This system is a minimum 5” cast-in-place (CIP) topping with one mat of epoxy coated reinforcement and placed on prestressed concrete slab girder and deck girder members, see Figure 5.7.4-4. This system eliminates girder wheel distribution problems, provides a quality protection system and provides a durable wearing surface.

   a. A minimum concrete cover of 1” applies to the top mat of the top flange of the prestressed member.

   b. Epoxy coating the prestressed member top mat reinforcement is not required.

   ![Type 4 Protection System](image)

5. **Type 5 System** – This system requires a layered, 3” concrete cover for double protection, see Figure 5.7.4-5. All segmentally constructed bridges shall use this system to protect construction joints and provide minor grade adjustments during construction. Bridge decks with transverse or longitudinal post-tensioning in the deck shall use this system since deck rehabilitation due to premature deterioration is very costly. The 3” cover consists of the following:

   a. The deck is constructed with a 1¾” concrete cover.

   b. Both the top and bottom mat of deck reinforcing are epoxy-coated. Girder/web stirrups and horizontal shear reinforcement does not require epoxy-coating.

   c. The deck is then scarified ¼” prior to the placement of a modified concrete overlay. Scarification shall be diamond grinding to preserve the integrity of the segmental deck and joints.

   d. A Type 2a, 1½” Modified Concrete Overlay is placed as a wearing surface.
B. **Existing Bridge Deck Widening** – New deck rebar shall match the existing top layer. This provides steel at a uniform depth which is important when removing concrete during future rehab work. Bridges prior to the mid 1980’s used 1½” concrete cover. New and widened decks using a Type 1 Protection System have 2½” cover.

When an existing bridge is widened, the existing concrete or asphalt deck may require resurfacing. WSDOT is forced to rehab concrete decks based on the condition of the existing deck or concrete overlay. If a deck or overlay warrants rehabilitation, then the existing structure shall be resurfaced and included in the widening project.

By applying the stated design criteria, the following policies shall apply to bridge widening projects which may require special traffic closures for the bridge work.

1. **Rebar** – The deck or cast-in-place slab of the new widened slab shall use the Type 1 Protection System, even though the existing structure has bare rebar. The top mat of new rebar shall match the height of existing rebar. Variations in deck thickness are to be obtained by lowering the bottom of the deck or slab.

2. **Concrete Decks** – If the existing deck is original concrete without a concrete overlay, the new deck shall have a Type 1 Protection System and the existing deck shall have a 1½” concrete overlay or Type 2 Protection System. This matches the rebar height and provides a concrete cover of 2.5” on both the new and old structure.

   If the existing deck has a concrete overlay, the new deck shall have a Type 1 Protection System and the existing overlay shall be replaced if the deck deterioration is greater than 1% of the deck area.

3. **Concrete Overlays** – It is preferred to place a concrete overlay from curb to curb. If this is problematic for traffic control, then Plans shall provide at least a 6” offset lap where the overlay construction joint will not match the deck construction joint.
4. **HMA Overlays** – The depth of existing asphalt must be field measured and shown on the bridge plans. This mitigates damage of the existing structure due to removal operations and reveals other design problems such as: improper joint height, buried construction problems, excessive weight, or roadway grade transitions adjustments due to drainage.

The new deck must meet the rebar and cover criteria stated above for Concrete Decks and deck tinning is not required. Type 3 Protection system shall be used and HMA shall be placed to provide a minimum 0.15’ or the optimum 0.25’.

5. **Small Width Widening** – With approval of the WSDOT Bridge Management Unit, smaller width widening design that has traffic on the new construction can match existing 1½” concrete cover for the widened portion, if the existing deck deterioration is greater than 1% of the deck area.

6. **Expansion Joints** – All joints shall be in good condition and water tight for the existing bridge and the newly constructed widened portion. The following joint criteria applies:
   a. The existing expansion joint shall be replaced if:
      1. More than 10% of the length of a joint has repairs within 1′-0” of the joint.
      2. Part of a joint is missing.
      3. The joint is a non-standard joint system placed by maintenance.
   b. All existing joint seals shall be replaced.
   c. When existing steel joints are not replaced in the project, the new joint shall be the same type and manufacturer as the existing steel joint.
   d. Steel joints shall have no more than one splice and the splice shall be at a lane line. Modular joints shall not have any splices.

### 5.7.5 Bridge Deck HMA Paving Design Policies

This Section of the BDM establishes the criteria used to provide bridge paving design options for paving projects. Bridge paving design options are customized for each bridge based on the existing conditions and previous paving. Paving designers including paving consultants are required to request a Bridge deck Condition Report (BCR) for each bridge which contains the paving design options and other relevant bridge information for each bridge within the project limits.

An asphalt wearing surface is a recognized method to manage concrete rutting, improve the ride on HMA roadways, and is a form of deck protection. Bridges may or may not have the capacity to carry the additional dead load of an asphalt wearing surface. The design options are documented for the paving project in a BCR. The Bridge Office will provide bridge sheets for structural items and the required Special Provisions for the WSDOT projects. The Bridge Office Projects and Design Engineer should be notified early in the paving design to allow time to complete engineering and plan sheets.
All WSDOT structures within the defined project limits must be evaluated for paving or Bituminous Surface Treatment (BST or chip seal). All bridges shall be identified in the Plans as “INCLUDED IN PROJECT” or “NOT INCLUDED” per WSDOT Plan Preparation Manual, Section 4 “Vicinity Map”, paragraph (n). This includes all state bridges and not limited to:

1. Off the main line. Typical locations include bridges on ramps, frontage roads, or bridges out of right-of-way.
2. Bridges where the main line route crosses under the structure.
3. Bridges at the beginning and ending stations of the project. It is not necessary to include the bridge when it was recently resurfaced, but it should be included if incidental joint maintenance repairs are necessary.

Region is responsible for field evaluation of paving condition and the depth of asphalt provided by the last paving contract. Asphalt depths can vary on the concrete deck and from bridge to bridge. In most cases, asphalt depth measurements at the fog line on the four corners of the deck are sufficient to establish a design depth for contracts. The Bridge Asset Manager shall be informed of the measurements. Paving shown in the Plans would use an approximate or averaged value of the measurements. Some situations may require a Plan Detail showing how the depth varies for the Planing contractor.

A standard Microstation detail is available to simplify detailing of bridge paving in the Plans, see “SH_DT_RDSECBridgeDeckOverlay_Detail”. The table format is copied from the BCR and allows the bridge paving design requirements to be listed in the table. All bridges within the limits of the project must be listed in the table to clarify which structures do not have paving and facilitate data logging for the Washington State Pavement Management System and the Bridge Office.

The following bridge paving policies have been developed with the concurrence of WSDOT Pavement Managers to establish bridge HMA Design options available for state managed structures.

1. **Maximum HMA Depth** – Bridge decks shall be 0.25’ or 3”. A greater depth may be allowed if the structure is specifically designed for more than 0.25’, such as structures with ballast or as approved by the WSDOT Load Rating Engineer. Paving designs that increase the HMA more than 3” require a new Load Rating analysis and shall be submitted to the WSDOT Load Rating engineer.
   a. Concrete bridge decks with more than 0.21’ HMA may be exempted from paving restrictions for mill/fill HMA design.
   b. Prestressed concrete deck girders and slabs with less than 0.25’ HMA require paving restrictions to avoid planing the supporting structure.
   c. A paving grade change will be required when more than 0.25’ of asphalt exists on a structure in order to reduce the weight on the structure and meet acceptable rail height standards.
2. **Grade limited/0.15’** – For bridge decks with 0.15’ HMA and the grade is limited by bridge joint height or other considerations, resurfacing must provide full depth removal of HMA or mill/fill the minimum 0.12’.
3. **Grade Transitions** – When raising or lowering the HMA grade profile on/off or under the bridge, the maximum rate of change or slope shall be 1″/40′ (1′/500′) as shown in Standard Plan A60.30.00, even if this means extending the project limits. Incorrect transitions are the cause of many “bumps at the bridge” and create an undesired increase in truck loading. The following items should be considered when transitioning a roadway grade:

   a. Previous HMA overlays that raised the grade can significantly increase the minimum transition length.

   b. Drainage considerations may require longer transitions or should plane to existing catch basins.

   c. Mainline paving that raises the grade under a bridge must verify Vertical Clearance remains in conformance to current Vertical Clearance requirements. Mill/Fill of the roadway at the bridge is generally desired unless lowering the grade is required.


4. **Full Removal** – Full depth removal and replacement of the HMA is always an alternate resurfacing design option. Full depth removal may be required by the Region Pavement Manager or the Bridge Office due to poor condition of the HMA or bridge deck. Bridge Deck Repair and Membrane Waterproofing (Deck Seal) standard pay items are required for this option and the Bridge Office will provide engineering estimates of the quantity (SF) and cost for both.

   a. Bridge Deck Repair will be required when the HMA is removed and the concrete is exposed for deck inspection. Chain Drag Testing is completed and based on the results, the contractor is directed to fix the quantity of deck repairs. The Chain Drag results are sent to the Bridge Asset Manager and used by the Bridge Office to monitor the condition of the concrete deck and determine when the deck needs rehabilitation or replacement.

   b. Membrane Waterproofing (Deck Seal) is Std. Item 4455 and will be required for all HMA bridge decks, except when the following conditions are met.

      i. HMA placed on a deck that has a Modified Concrete Overlay which acts like a membrane.

      ii. The bridge is on the P2 replacement list or deck rehabilitation scheduled within the next 4 years or two bienniums.

5. **Bare Deck HMA** – Paving projects may place HMA on a bare concrete deck, with concurrence of the WSDOT Bridge Asset Manager, if the bridge is on an HMA route and one of the following conditions apply.

   a. Rutting on the concrete deck is ½” or more.

   b. The Region prefers to simplify paving construction or improve the smoothness at the bridge.
When the concrete bridge deck does not have asphalt on the surface, Region Design should contact the Region Materials lab and have a Chain Drag Report completed and forwarded to the Bridge Asset Manager during design to establish the Bridge Deck Repair quantities for the project. Pavement Design should then contact Region Bridge Maintenance to request the repairs be completed prior to contract; or the repairs may be included in the paving contract. Small amounts of Bridge Deck Repair have an expensive unit cost by contract during paving operations.

6. **Bridge Transverse Joint Seals** – Saw cut pavement joints shown in Std. Plan A-40.20.00 perform better and help prevent water problems at the abutment or in the roadway. Typical cracking locations where pavement joint seals are required: End of the bridge; End of the approach slab; or HMA joints on the deck. Std. Plan A-40.20.00, Detail 8 shall be used at all truss panel joint locations. However, if Pavement Designers do not see cracking at the ends of the bridge, then sawcut joints may be omitted for these locations. HQ Program management has determined this work is “incidental” to P1 by definition and should be included in a P1 paving project and use Std. Item 6517. The following summarizes the intended application of the Details in Std. Plan A-40.20.00.

a. **Detail 1** – Applies where HMA on the bridge surface butts to the HMA roadway.

b. **Detail 2, 3, & 4** – Applies where concrete bridge surface butts to the HMA roadway.

c. **Detail 8** – Applies at truss panel joints or generic open concrete joints.

d. **Detail 5, 6 &7** – For larger 1″ sawcut joints instead of ½″ joints provided in details 2, 3, & 4.

7. **BST (chip seal)** – Bituminous Surface Treatments ½″ thick may be applied to bridge decks with HMA under the following conditions.

a. Plans must identify or list all structures bridges included or excepted within project limits and identify bridge expansion joint systems to be protected.

b. BST is not allowed on weight restricted or posted bridges.

c. Planing will be required for structures at the maximum asphalt design depth or the grade is limited.

It is true that BSTs are not generally a problem but only if the structure is not grade limited by for structural reasons. BCRs will specify a ½″ chip seal paving depth of 0.03’ for BST Design to be consistent with Washington State Pavement Management System. Plans should indicate ½″ chip seal to be consistent with *Standard Specifications* and standard pay items.

8. **Culverts and Other Structures** – Culverts or structures with significant fill and do not have rail posts attached to the structure generally will not have paving limitations. Culverts and structures with HMA pavement applied directly to the structure have bridge paving design limits.
5.8  Cast-in-place Post-Tensioned Bridges

5.8.1  Design Parameters

A. **General** – Post-tensioning is generally used for CIP construction and spliced prestressed concrete girders since pretensioning is generally practical only for fabricator-produced structural members. The Post-tensioned Box Girder Bridge Manual\(^{17}\) published by the Post-tensioning Institute in 1978 is recommended as the guide for design. This manual discusses longitudinal post-tensioning of box girder webs and transverse post-tensioning of box girder slabs, but the methods apply equally well to other types of bridges. The following recommendations are intended to augment the PTI Manual and the AASHTO LRFD Specifications and point out where current WSDOT practice departs from practices followed elsewhere.

The AASHTO criteria for reinforced concrete apply equally to bridges with or without post-tensioning steel. However, designers should note certain requirements unique to prestressed concrete such as special \(\phi\)-factors, load factors and shear provisions.

Post-tensioning consists of installing steel tendons into a hollow duct in a structure after the concrete sections are cast. These tendons are usually anchored at each end of the structure and stressed to a design strength using a hydraulic jacking system. After the tendon has been stressed, the duct is filled with grout which bonds the tendon to the concrete section and prevents corrosion of the strand. The anchor heads are then encased in concrete to provide corrosion protection.

B. **Bridge Types** – Post-tensioning has been used in various types of CIP bridges in Washington State with box girders predominating. See Appendix 5-B4 for a comprehensive list of box girder designs. The following are some examples of other bridge types:

- Kitsap County, Contract 9788, Multi-Span Slab
- Peninsula Drive, Contract 5898, Two-Span Box Girder
- Covington Way to 180th Avenue SE, Contract 4919, Two-Span Box Girder Longitudinal Post-tensioning
- Snohomish River Bridge, Contract 4444, Multi-Span Box Girder Longitudinal Post-tensioning

See Section 2.4.1 of this manual for structure type comparison of post-tensioned concrete box girder bridges to other structures. In general, a post-tensioned CIP bridge can have a smaller depth-to-span ratio than the same bridge with conventional reinforcement. This is an important advantage where minimum structure depth is desirable. However, structure depth must be deep enough to accommodate anchorages.
1. **Slab Bridge** – Structure depth can be quite shallow in the positive moment region when post-tensioning is combined with haunching in the negative moment region. However, post-tensioned CIP slabs are usually more expensive than when reinforced conventionally. Designers should proceed with caution when considering post-tensioned slab bridges because severe cracking in the decks of bridges of this type has occurred. The Olalla Bridge (Contract 9202) could be reviewed as an example. This bridge has spans of 41.5’ - 50’ - 41.5’, a midspan structure depth of 15 inches, and some haunching at the piers.

2. **T-Beam Bridge** – This type of bridge, combined with tapered columns, can be structurally efficient and aesthetically pleasing, particularly when the spacing of the beams and the columns are the same. A T-Beam bridge can also be a good choice for a single-span simply-supported structure.

When equally spaced beams and columns are used in the design, the width of beam webs should generally be equal to the width of the supporting columns. See SR 16, Union Avenue O’Xings, for an example. Since longitudinal structural frame action predominates in this type of design, crossbeams at intermediate piers can be relatively small and the post-tensioning tendons can be placed side-by-side in the webs, resulting in an efficient center of gravity of steel line throughout. For other types of T-Beam bridges, the preferred solution may be smaller, more closely spaced beams and fewer, but larger pier elements. If this type of construction is used in a multispans, continuous bridge, the beam cross-section properties in the negative moment regions need to be considerably larger than the properties in the positive moment regions to resist compression.

Larger section properties can be obtained by gradually increasing the web thickness in the vicinity of intermediate piers or, if possible, by adding a fillet or haunch. The deck slab overhang over exterior webs should be roughly half the web spacing.

3. **Box Girder Bridge** – This type of bridge has been a popular choice in this state. The cost of a prestressed box girder bridge is practically the same as a conventionally-reinforced box girder bridge, however, longer spans and shallower depths are possible with prestressing.

The superstructure of multi-cell box girders shall be designed as a unit. The entire superstructure section (traffic barrier excluded) shall be considered when computing the section properties.

For criteria on distribution of live loads, see Section 3.9.4. All slender members subjected to compression must satisfy buckling criteria.

Web spacing should normally be 8 to 11 feet and the top slab overhang over exterior girders should be approximately half the girder spacing unless transverse post-tensioning is used. The apparent visual depth of box girder bridges can be reduced by sloping all or the lower portion of the exterior web. If the latter is done, the overall structure depth may have to be increased. Web thickness should be 12 inches minimum, but not less than required for shear.
horizontal and vertical reinforcing, duct placement, and for concrete placing clearance. Providing 2½” of clear cover expedites concrete placement and consolidation in the heavily congested regions adjacent to the post-tensioning ducts. Webs should be flared at anchorages. Top and bottom slab thickness should normally meet the requirements of Section 5.3.1.B, but not less than required by stress and specifications. Generally, the bottom slab would require thickening at the interior piers of continuous spans. This thickening should be accomplished by raising the top surface of the bottom slab at the maximum rate of ½” per foot.

C. **Strand and Tendon Arrangements** – The total number of strands selected should be the minimum required to meet the strength and service limit state requirements at all points. Duct sizes and the number of strands they contain vary slightly, depending on the supplier. Chapter 2 of the PTI Post-tensioned Box Girder Bridge Manual, and shop drawings of the recent post-tensioned bridges kept on file in the Construction Plans Section offer guidance to strand selection. In general, a supplier will offer several duct sizes and associated end anchors, each of which will accommodate a range of strand numbers up to a maximum in the range. Present WSDOT practice is to indicate only the design force and cable path on the contract plans and allow the post-tensioning supplier to satisfy these requirements with tendons and anchors. The most economical tendon selection will generally be the maximum size within the range. Commonly-stocked anchorages for ½” diameter strands include 9, 12, 19, 27, 31, and 37 strands. Commonly-stocked anchorages for 0.6” diameter strands include 4, 7, 12, 19, 22, and 27 strands. The design should utilize commonly-stocked items. For example, a design requiring 72 strands per web would be most economically satisfied by two standard 37-strand tendons. A less economical choice would be three standard 27-strand tendons containing 24 strands each. Tendons shall not be larger than (37) ½” strand units or (27) 0.6” strand units, unless specifically approved by the WSDOT Bridge Design Engineer. The duct area shall be at least 2.5 times the net area of the prestressing steel. In the regions away from the end anchorages, the duct placement patterns indicated in Figures 5.8.1-1 through 5.8.1-3 shall be used.

Although post-tensioning steel normally takes precedence in a member, sufficient room must be provided for other essential mild steel and placement of concrete, in particular near diaphragms and cross-beams.

More prestress may be needed in certain portions of a continuous superstructure than elsewhere, and the designer may consider using separate short tendons in those portions of the spans only. However, the savings on prestressing steel possible with such an arrangement should be balanced against the difficulty involved in providing suitable anchoring points and sufficient room for jacking equipment at intermediate locations in the structure. For example, torsion in continuous, multigirder bridges on a curve can be counter-balanced by applying more prestress in the girders on the outside of the curve than in those on the inside of the curve.
Some systems offer couplers which make possible stage construction of long bridges. With such systems, forms can be constructed and concrete cast and stressed in a number of spans during stage 1, as determined by the designer. After stage 1 stressing, couplers can be added, steel installed, concrete cast and stressed in additional spans. To avoid local crushing of concrete and/or grout, the stress existing in the steel at the coupled end after stage 1 stressing shall not be exceeded during stage 2 stressing.

**TYPICAL SECTION**

* 2½" MIN. CLR. TO ANY REINF. (TO PERMIT POURING OF CONCRETE)
A SINGLE TIER OF TENDONS CENTERED IN THE WEB WILL GENERALLY PERMIT THE USE OF THINNER WEBs THAN USING DOUBLE TIERS.

**Tendon Placement Pattern for Box Girder Bridges**

*Figure 5.8.1-1*
Tendon Placement Pattern for Box Girder Bridges

Figure 5.8.1-2
D. **Layout of Anchorages and End Blocks** – Consult industry brochures and shop plans for recent bridges before laying out end blocks. To encourage bids from a wider range of suppliers, try to accommodate the large square bearing plate sizes common to several systems.

Sufficient room must be allowed inside the member for mild steel and concrete placement and outside the member for jacking equipment. The size of the anchorage block in the plane of the anchor plates shall be large enough to provide a minimum of 1” clearance from the plates to any free edge.

The end block dimensions shall meet the requirements of the AASHTO LRFD Specifications. Note that in long-span box girder superstructures requiring large bearing pads, the end block should be somewhat wider than the bearing pad beneath to avoid subjecting the relatively thin bottom slab to high bearing stresses. When the piers of box girder or T-beam bridges are severely skewed, the layout of end blocks, bearing pads, and curtain walls at exterior girders become extremely difficult as shown in Figure 5.8.1-4. Note that if the exterior face of the exterior girder is in the same plane throughout its entire length, all the end block widening must be on the inside. To lessen the risk of tendon break-out through the side of a thin web, the end block shall be long enough to accommodate a horizontal tendon curve of 200 feet minimum radius. The radial component of force in a curved tendon is discussed in AASHTO LRFD 5.10.4.3.
Layout of Anchorages and End Blocks

Figure 5.8.1-4

All post-tensioning anchorages in webs of box girder or multi stem superstructures shall be vertically aligned. Special Anchorage Devices may be used to avoid a staggered anchorage layout. If a staggered layout must be used, the plans shall be reviewed and approved by the WSDOT Bridge Design Engineer.

To ensure maximum anchorage efficiency, maximum fatigue life and prevention of strand breakage, a minimum tangent length at the anchorage is required to ensure that the strands enter the anchorage without kinking.

To prevent excessive friction loss and damage to the prestressing sheathings, adherence to the minimum tendon radii is required.
Table 5.8.1-1 and Figure 5.8.1-5 present the required minimum radius of curvature along with the required minimum tangent lengths at stressing anchorages. Deviation from these requirements needs the approval of the WSDOT Bridge Design Engineer.

<table>
<thead>
<tr>
<th>Anchor Types</th>
<th>Radii, ft.</th>
<th>Tangent Length, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>½” Diameter Strand Tendons</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-4</td>
<td>7.5</td>
<td>2.6</td>
</tr>
<tr>
<td>5-7</td>
<td>9.8</td>
<td>2.6</td>
</tr>
<tr>
<td>5-12</td>
<td>13.5</td>
<td>3.3</td>
</tr>
<tr>
<td>5-19</td>
<td>17.7</td>
<td>3.3</td>
</tr>
<tr>
<td>5-27</td>
<td>21.0</td>
<td>3.3</td>
</tr>
<tr>
<td>5-31</td>
<td>22.3</td>
<td>4.9</td>
</tr>
<tr>
<td>5-37</td>
<td>24.0</td>
<td>4.9</td>
</tr>
<tr>
<td>0.6” Diameter Strand Tendons</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6-4</td>
<td>10.6</td>
<td>3.3</td>
</tr>
<tr>
<td>6-7</td>
<td>12.8</td>
<td>3.3</td>
</tr>
<tr>
<td>6-12</td>
<td>16.4</td>
<td>3.3</td>
</tr>
<tr>
<td>6-19</td>
<td>20.7</td>
<td>4.9</td>
</tr>
<tr>
<td>6-22</td>
<td>22.6</td>
<td>4.9</td>
</tr>
<tr>
<td>6-31</td>
<td>26.4</td>
<td>4.9</td>
</tr>
</tbody>
</table>

Minimum Tendon Radii and Tangent Length

Table 5.8.1-1

E. Superstructure Shortening – Whenever members such as columns, crossbeams, and diaphragms are appreciably affected by post-tensioning of the main girders, those effects shall be included in the design. This will generally be true in structures containing rigid frame elements. For further discussion, see Section 2.6 of reference 17.

Past practice in the state of Washington regarding control of superstructure shortening in post-tensioned bridges with rigid piers can be illustrated by a few examples. Single-span bridges have been provided with a hinge at one pier and longitudinal slide bearings at the other pier. Two-span bridges have been detailed with longitudinal slide bearings at the end piers and a monolithic middle pier. On the six-span Evergreen Parkway Undercrossing (Bridge Number 101/510), the center pier (pier 4) was built monolithic with the superstructure, and all the other piers were constructed with slide bearings. After post-tensioning, the bearings at piers 3 and 5 were converted into fixed bearings to help resist large horizontal loads such as earthquakes.

Superstructures which are allowed to move longitudinally at certain piers are typically restrained against motion in the transverse direction at those piers. This can be accomplished with suitable transverse shear corbels or bearings allowing motion parallel to the bridge only. The casting length for box girder bridges shall be slightly longer than the actual bridge layout length to account for the elastic shortening of the concrete due to prestress.
F. Effects of Curved Tendons – AASHTO LRFD 5.10.4.3 shall be used to consider the effects of curved tendons. In addition, confinement reinforcement shall be provided to confine the PT tendons when $R_{in}$ is less than 800 ft or the effect of in-plane plus out-of-plane forces is greater than or equal to 10 k/ft:

$$\frac{P_u}{R_{in}} + \frac{P_u}{\pi R_{out}} \geq 10 \frac{k}{ft}$$ (5.8.1-1)

Where:

- $P_u$ = Factored tendon force = 1.2 $P_{jack}$ (kips)
- $R_{in}$ = Radius of curvature of the tendon at the considered location causing in-plane force effects (typically horizontal) (ft)
- $R_{out}$ = Radius of curvature of the tendon at the considered location causing out-of-plane force effects (typically vertical) (ft)

Curved tendon confinement reinforcement, when required, shall be as shown in Figure 5.8.1-6. Spacing of the confinement reinforcement shall not exceed either 3.0 times the outside diameter of the duct or 18.0 in.

G. Edge Tension Forces – If the centroid of all tendons is located outside of the kern of the section, spalling and longitudinal edge tension forces are induced. Evaluate per AASHTO LRFD 5.10.9.6.4.
5.8.2 Analysis

A. General – The procedures outlined in Section 2.1 through 2.5 of reference 17 for computation of stress in single and multispan box girders can be followed for the analysis of T-beams and slab bridges as well.

The WinBDS program available on the WSDOT system will quickly perform a complete stress analysis of a box girder, T-beam, or slab bridge, provided the structure can be idealized as a plane frame. For further information, see the program user instructions.

STRUDL or CSIBridge is recommended for complex structures which are more accurately idealized as space frames. Examples are bridges with sharp curvature, varying superstructure width, severe skew, or slope-leg intermediate piers. An analysis method in Chapter 10 of reference18 for continuous prestressed beams is particularly well adapted to the loading input format in STRUDL. In the method, the forces exerted by cables of parabolic or other configurations are converted into equivalent vertical linear or concentrated loads applied to members and joints of the superstructure. The vertical loads are considered positive when acting up toward the center of tendon curvature and negative when acting down toward the center of tendon curvature. Forces exerted by anchor plates at the cable ends are coded in as axial and vertical concentrated forces combined with a concentrated moment if the anchor plate group is eccentric. Since the prestress force varies along the spans due to the effects of friction, the difference between the external forces applied at the end anchors at opposite ends of the bridge must be coded in at various points along the spans in order for the summation of horizontal forces to equal zero. With correct input, the effects of elastic shortening and secondary moments are properly reflected in all output, and the prestress moments printed out are the actual resultant (total) moments acting on the structure. For examples of the application of STRUDL to post-tensioning design, see the calculations for I-90 West Sunset Way Ramp and the STRUDL/CSI Bridge manuals.

B. Section Properties – As in other types of bridges, the design normally begins with a preliminary estimate of the superstructure cross-section and the amount of prestress needed at points of maximum stress and at points of cross-section change. For box girders, see Figures 2-0 through 2-5 of Reference17. For T-beam and slab bridges, previous designs are a useful guide in making a good first choice.

For frame analysis, use the properties of the entire superstructure regardless of the type of bridge being designed. For stress analysis of slab bridges, calculate loads and steel requirements for a 1’ wide strip. For stress analysis of T-beam bridges, use the procedures outlined in the AASHTO LRFD Specifications.

Note that when different concrete strengths are used in different portions of the same member, the equivalent section properties shall be calculated in terms of either the stronger or weaker material. In general, the concrete strength shall be limited to the values indicated in Section 5.1.1.
C. **Preliminary Stress Check** – In accordance with AASHTO, flexural stresses in prestressed members are calculated at service load levels. Shear stresses, stirrups, moment capacities vs. applied moments are calculated at ultimate load levels.

During preliminary design, the first objective should be to satisfy the allowable flexural stresses in the concrete at the critical points in the structure with the chosen cross-section and amount of prestressing steel, then the requirements for shear stress, stirrups, and ultimate moment capacity can be readily met with minor or no modifications in the cross-section. For example, girder webs can be thickened locally near piers to reduce excessive shear stress.

In the AASHTO formulas for allowable tensile stress in concrete, bonded reinforcement should be interpreted to mean bonded auxiliary (nonprestressed) reinforcement in conformity with Article 8.6 of the 2002 ACI Code for Analysis and Design of Reinforced Concrete Bridge Structures. The refined estimate for computing time-dependent losses in steel stress given in the code shall be used. To minimize concrete cracking and protect reinforcing steel against corrosion for bridges, the allowable concrete stress under final conditions in the precompressed tensile zone shall be limited to zero in the top and bottom fibers as shown in Figure 5.8.2-1.

In all cases where tension is allowed in the concrete under initial or final conditions, extra mild steel (auxiliary reinforcement) shall be added to carry the total tension present. This steel can be computed as described in Section 9-5 of Reference 18.

**Box Girder Stresses**

*Figure 5.8.2-1*
In case of overstress, try one or more of the following remedies: adjust tendon profiles, add or subtract prestress steel, thicken slabs, revise strength of concrete of top slab, add more short tendons locally, etc.

D. **Camber** – The camber to be shown on the plans shall include the effect of both dead load and final prestress.

E. **Expansion Bearing Offsets** – Figure 5.8.1-4 indicates expansion bearing offsets for the partial effects of elastic shortening, creep, and shrinkage. The initial offset shown is intended to result in minimal bearing eccentricity for the majority of the life of the structure. The bearing shall be designed for the full range of anticipated movements: \( ES + CR + SH + TEMP \) including load factors specified in AASHTO for deflections.

### 5.8.3 Post-tensioning

A. **Tendon Layout** – After a preliminary estimate has been made of the concrete section and the amount of prestressing needed at points of maximum applied load, it may be advantageous in multispans to draw a tendon profile to a convenient scale superimposed on a plot of the center of gravity of concrete (c.g.c.) line. The most efficient tendon profile from the standpoint of steel stress loss will normally be a series of rather long interconnected parabolas, but other configurations are possible. For continuous bridges with unequal span lengths, the tendon profile (eccentricity) shall be based on the span requirement. This results in an efficient post-tensioning design. The tendon profile and c.g.c. line plot is strongly recommended for superstructures of variable cross-section and/or multiple unsymmetrical span arrangements, but is not necessary for superstructures having constant cross-section and symmetrical spans. The main advantages of the tendon profile and c.g.c. plot are:

1. The primary prestress moment curves (prestress force times distance from c.g.c. line to center of gravity of steel (c.g.s.) lines) at all points throughout all spans are quickly obtained from this plot and will be used to develop the secondary moment curves (if present) and, ultimately, to develop the resultant total prestress moment curve.

2. Possible conflicts between prestressing steel and mild steel near end regions, crossbeams, and diaphragms may become apparent.

3. Possible design revisions may be indicated. For example, camber in bridges with unequal spans can be balanced by adjusting tendon profiles.

The tendon profile and c.g.c. line diagram shall also contain a sketch of how the end bearing plates or anchors are to be arranged at the ends of the bridge. Such a sketch can be useful in determining how large the end block in a girder bridge will have to be and how much space will be required for mild steel in the end region. In general, the arrangement of anchor plates should be the same as the arrangement of the ducts to which they belong to avoid problems with duct cross-overs and to keep end blocks of reasonable width.
B. **Prestress Losses** – Prestress losses shall be as indicated in Section 5.1.4.

C. **Jacking End** – Effective prestressing force in design of post-tensioned bridges depends on the accumulation of friction losses due to the horizontal and vertical curvature of the tendons as well as the curvature of the bridge. Although jacking ends of post-tensioned bridges is important to achieve more effective design, consideration shall be given to the practicality of jacking during construction. The following general stressing guidelines shall be considered in specifying jacking end of post-tensioned bridges.

   - All simple or multiple span CIP or precast concrete bridges with total length of less than 350' shall be stressed from one end only.
   - All CIP or precast concrete post tensioned bridges with total length between 350' to 600', may be stressed from one end or both ends if greater friction losses due to vertical or horizontal curvature are justified by the designer.
   - All CIP or precast concrete bridges with total length of greater than 600' shall be stressed from both ends.

When stressing tendons from both ends or when alternating a single pull from both ends (half tendons pulled from one end with the other half pulled from the other end), all tendons shall be stressed on one end before all tendons are stressed on the opposite end.

Stressing at both ends shall preferably be done on alternate tendons, and need not be done simultaneously on the same tendon. In rare cases, tendons can be stressed from both ends to reduce large tendon losses but is undesirable due to worker safety issues and a reduction in stressing redundancy.

D. **Steel Stress Curve** – Steel stresses may be plotted either as the actual values or as a percentage of the jacking stresses. A steel stress diagram for a typical two-span bridge is shown in Figure 5.8.3-1. Spans are symmetrical about pier 2 and the bridge is jacked from both ends.
Accurate plotting of steel stress variation due to local curvature is normally not necessary, and straight lines between intersection points on the diagram as shown in Figure 5.8.3-1 are usually sufficient. When tendons are continuous through the length of the bridge, the stress for design purposes at the jacked end should be limited to $0.79f_{pu}$ or 213 ksi for 270 ksi low relaxation strands. This would permit the post-tensioning contractor to jack to the slightly higher value of $0.81f_{pu}$ for low relaxation strands as allowed by the AASHTO LRFD Specifications in case friction values encountered in the field turn out somewhat greater than the standard values used in design. Stress loss at jacked end shall be calculated from the assumed anchor set of ½", the normal slippage during anchoring in most systems. At the high points on the initial stress curve, the stress shall not exceed $0.74f_{pu}$ for low relaxation strands after seating of the anchorage. If these values are exceeded, the jacking stress can be lowered or alternately the specified amount of anchor set can be increased.
When the total tendon length \( (L) \) is less than the length of cable influenced by anchor set \( (x) \) and the friction loss is small, as in short straight tendons, the \( 0.70f_{pu} \) value at the anchorage immediately after anchor set governs. In these cases, the allowable jacking stress value at the anchorage cannot be used and a slightly lower value shall be specified.

In single-span, simply supported superstructures friction losses are so small that jacking from both ends is normally not warranted. In the longer multispans bridges where the tendons experience greater friction losses, jacking from both ends will usually be necessary. Jacking at both ends need not be done simultaneously, since final results are virtually the same whether or not the jacking is simultaneous. If unsymmetrical two-span structures are to be jacked from one end only, the jacking must be done from the end of the longest span.

In the absence of experimental data, the friction coefficient for post-tensioning tendons in rigid and semi-rigid galvanized metal sheathing shall be taken as shown in Table 5.8.3-1. For tendon lengths greater than 1,000 feet, investigation is warranted on current field data of similar length bridges for appropriate values of \( \mu \). In the absence of experimental data, the friction coefficient for post-tensioning tendons in polyethylene ducts shall be taken as shown in the AASHTO LRFD Bridge Design Specifications.

<table>
<thead>
<tr>
<th>Tendon Length</th>
<th>( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 ft or less</td>
<td>0.15</td>
</tr>
<tr>
<td>Over 500 ft to 750 ft</td>
<td>0.20</td>
</tr>
<tr>
<td>Over 750 ft to 1,000 ft</td>
<td>0.25</td>
</tr>
</tbody>
</table>

**Friction Coefficients for Post-tensioning Tendons in Metal Ducts**  
*Table 5.8.3-1*

For tendon lengths greater than 1,000 feet, investigation is warranted on current field data of similar length bridges for appropriate values of \( \mu \).

**E. Flexural Stress in Concrete** – Stress at service load levels in the top and bottom fibers of prestressed members shall be checked for at least two conditions that will occur in the lifetime of the members. The initial condition occurs just after the transfer of prestress when the concrete is relatively fresh and the member is carrying its own dead load. The final condition occurs after all the prestress losses when the concrete has gained its full ultimate strength and the member is carrying dead load and live load. For certain bridges, other intermediate loading conditions may have to be checked, such as when prestressing and falsework release are done in stages and when special construction loads have to be carried, etc. The concrete stresses shall be within the AASHTO LRFD Specification allowable except as amended in Section 5.2.1.

In single-span simply supported superstructures with parabolic tendon paths, flexural stresses at service load levels need to be investigated at the span midpoint where moments are maximum, at points where the cross-section changes, and near the span ends where shear stress is likely to be maximum (see Section 5.8.4 Shear). For tendon paths other than parabolic, flexural stress shall be investigated at other points in the span as well.
In multspan continuous superstructures, investigate flexural stress at points of maximum moment (in the negative moment region of box girders, check at the quarter point of the crossbeam), at points where the cross section changes, and at points where shear is likely to be maximum. Normally, mild steel should not be used to supplement the ultimate moment capacity. It may be necessary, however, to determine the partial temperature and shrinkage stresses that occur prior to post-tensioning and supply mild steel reinforcing for this condition.

In addition, maximum and minimum steel percentages and cracking moment shall be checked. See Section 2.3.8 of Reference\textsuperscript{17}.

F. Prestress Moment Curves

1. **Single-Span Bridges, Simply Supported** – The primary prestress moment curve is developed by multiplying the initial steel stress curve ordinates by the area of prestressing steel times the eccentricity of steel from the center of gravity of the concrete section at every tenth point in the span. The primary prestress moment curve is not necessary for calculating concrete stresses in single-span simply supported bridges. Since there is no secondary prestress moment developed in the span of a single span, simply supported bridge which is free to shorten, the primary prestress moment curve is equal to the total prestress moment curve in the span. However, if the single span is rigidly framed to supporting piers, the effect of elastic shortening shall be calculated. The same would be true when unexpected high friction is developed in bearings during or after construction.

2. **Multispan Continuous Bridges** – Designers shall take into account the elastic shortening of the superstructure due to prestressing. To obtain the total prestress moment curve used to check concrete stresses, the primary and secondary prestress moment curves must be added algebraically at all points in the spans. As the secondary moment can have a large absolute value in some structures, it is very important to obtain the proper sign for this moment, or a serious error could result.

G. Partial prestressing – Partial prestressing is not allowed in WSDOT bridge designs. However, mild reinforcement could be added to satisfy the ultimate flexural capacity under factored loads if the following requirements are satisfied:

1. Allowable stresses, as specified in this manual for Service-I and Service-III limit states, shall be satisfied with post-tensioning only. The zero-tension policy remains unchanged.

2. Additional mild reinforcement could be used if the ultimate flexural capacity cannot be met with the prestressing provided for service load combinations. The mild reinforcement is filling the gap between the service load and ultimate load requirements. This should be a very small amount of mild reinforcement since adequate post-tensioning is already provided to satisfy the service load requirement for dead load and live loads.

3. If mild reinforcement is added, the resistance factor for flexural design shall be adjusted per AASHTO LRFD 5.5.4.2.1 to account for the effect of partial prestressing. The section will still be considered uncracked and requirements for crack control, and side skin reinforcement do not apply.
5.8.4 Shear and Anchorages

A. Shear Capacity – Concrete box girder and T-beam bridges with horizontal construction joints (which result from webs and slabs being cast at different times) shall be checked for both vertical and horizontal shear capacity. Generally, horizontal shear requirements will control the stirrup design.

Vertical concrete shear capacity for prestressed or post-tensioned structural members is calculated per AASHTO LRFD 5.8.3. Minimum stirrup area and maximum stirrup spacing are subject to the limitations presented in AASHTO LRFD 5.8.2.5 and 5.8.2.7. For further explanation, refer to Section 11.4 of the ACI 318-02 Building Code Requirements for Reinforced Concrete and Commentary. Chapter 27 of Notes on ACI 318-02 Building Code Requirements for Reinforced Concrete with Design Applications presents two excellent example problems for vertical shear design.

B. Horizontal Shear – Horizontal shear stress acts over the contact area between two interconnected surfaces of a composite structural member. AASHTO LRFD 5.8.4 shall be used for shear-friction design.

C. End Block Stresses – The highly concentrated forces at the end anchorages cause bursting and spalling stresses in the concrete which must be resisted by reinforcement. For a better understanding of this subject, see Chapter 7 of Reference 18 and 19, and Section 2.82 of Reference 17.

Note that the procedures for computing horizontal bursting and spalling steel in the slabs of box girders and T-beams are similar to those required for computing vertical steel in girder webs, except that the slab steel is figured in a horizontal instead of a vertical plane. In box girders, this slab steel should be placed half in the top slab and half in the bottom slab. The anchorage zones of slab bridges will require vertical stirrups as well as additional horizontal transverse bars extending across the width of the bridge. The horizontal spalling and bursting steel in slab bridges shall be placed half in a top layer and half in a bottom layer.

D. Anchorage Stresses – The average bearing stress on the concrete behind the anchor plate and the bending stress in the plate material shall satisfy the requirements of the AASHTO LRFD Specification. In all sizes up to the 31-strand tendons, the square anchor plates used by three suppliers (DSI, VSL, AVAR, Stronghold) meet the AASHTO requirements, and detailing end blocks to accommodate these plates is the recommended procedure. In the cases where nonstandard (rectangular) anchor plates must be specified because of space limitations, assume that the trumpet associated with the equivalent size square plate will be used. In order to calculate the net bearing plate area pressing on the concrete behind it, the trumpet size can be scaled from photos in supplier brochures. Assume for simplicity that the concrete bearing stress is uniform. Bending stress in the steel should be checked assuming bending can occur across a corner of the plate or across a line parallel to its narrow edge. See Appendix 5-B2 for preapproved anchorages for post-tensioning.
E. **Anchorage Plate Design** – The design and detailing of the anchorage block in CIP post-tensioned box girders should be based on Normal Anchorage Devices as defined in *Standard Specifications* Section 6-02.3(26)C. Special Anchorage Devices as defined in *Standard Specifications* Section 6-02.3(26)D could be used if stacking of Normal Anchorage Devices within the depth of girder is geometrically not possible. Anchorage plates shall not extend to top and bottom slab of box girders. If Special Anchorage Devices are used, they shall be specified in the contract plans and bridge special provisions.

### 5.8.5 Temperature Effects

Most specifications for massive bridges call for a verification of stresses under uniform temperature changes of the total bridge superstructure. Stresses due to temperature unevenly distributed within the cross-section are not generally verified. In reality, however, considerable temperature gradients are set up within the cross-section of superstructures. Such temperature differences are mostly of a very complex nature, depending on the type of cross-section and direction of solar radiation.

Solar radiation produces uniform heating of the upper surface of a bridge superstructure which is greater than that of the lower surface. An inverse temperature gradient with higher temperatures at the lower surface occurs rarely and involves much smaller temperature differences. In statically indeterminate continuous bridge beams, a temperature rise at the upper surface produces positive flexural moments which cause tensile stresses in the bottom fibers. When the temperature gradient is constant over the entire length of a continuous beam superstructure, positive flexural moments are induced in all spans. These moments are of equal constant magnitude in the interior spans and decrease linearly to zero in the end spans. The most critical zones are those which have the lowest compressive stress reserve in the bottom fibers under prestress plus dead load. Normally, these are the zones near the interior supports where additional tensile stresses develop in the bottom fibers due to

- A concentrated support reaction, and
- Insufficient curvature of prestressed reinforcement.

Studies have shown that temperature is the most important tension-producing factor, especially in two-span continuous beams in the vicinity of intermediate supports, even when the temperature difference is only 10°C between the deck and bottom of the beam. In practice, a box girder can exhibit a ΔT=30°C. The zone at a distance of about 0.3 to 2.0d on either side of the intermediate support proved to be particularly crack-prone.

Computation of stresses induced by vertical temperature gradients within prestressed concrete bridges can become quite complex and are ignored in typical designs done by WSDOT. It is assumed that movements at the expansion devices will generally relieve any induced stresses. However, such stresses can be substantial in massive, deep bridge members in localities with large temperature fluctuations. If the structure being designed falls within this category, a thermal stress investigation shall be considered. See Reference[17] and the following temperature criteria for further guidance.

1. A mean temperature 50°F with rise 45°F and fall 45°F for longitudinal analysis using one-half of the modulus of elasticity. (Maximum Seasonal Variation.)
2. The superstructure box girder shall be designed transversely for a temperature differential between inside and outside surfaces of ±15°F with no reduction in modulus of elasticity (Maximum Daily Variation).

3. The superstructure box girder shall be designed longitudinally for a top slab temperature increase of 20°F with no reduction in modulus of elasticity. (In accordance with Post-tensioning Institute Manual, Precast Segmental Box Girder Bridge Manual, Section 3.3.4.)

5.8.6 Construction

A. General – Construction plans for conventional post-tensioned box girder bridges include two different sets of drawings. The first set (contract plans) is prepared by the design engineer and the second set (shop plans) is prepared by the post-tensioning materials supplier (contractor).

B. Contract Plans – The contract plans shall be prepared to accommodate several post-tensioning systems, so only prestressing forces and eccentricity should be detailed. The concrete sections shall be detailed so that available systems can be installed. Design the thickness of webs and flanges to facilitate concrete placement. Generally, web thickness for post-tensioned bridges shall be as described in Section 5.8.1.B. See section 5.8.7 for design information to be included in the contract plan post-tensioning notes.

C. Shop Plans – The shop plans are used to detail, install, and stress the post-tensioning system selected by the Contractor. These plans must contain sufficient information to allow the engineer to check their compliance with the contract plans. These plans must also contain the location of anchorages, stressing data, and arrangement of tendons.

D. Review of Shop Plans for Post-tensioned Girder – Post-tensioning shop drawings shall be reviewed by the designer (or Bridge Technical Advisor for non-Bridge Office projects) and consulted with the Concrete Specialist if needed. Review of shop drawing shall include:

1. All post-tensioning strands shall be of ½” or 0.6” diameter grade 270 low relaxation uncoated strands.

2. Tendon profile and tendon placement patterns.

3. Duct size shall be based on the duct area at least 2.5 times the total area of prestressing strands.

4. Anchor set shall conform to the contract plans. The post-tensioning design is typically based on an anchor set of ½”.

5. Maximum number of strands per tendon shall not exceed (37) ½” diameter strands or (27) 0.6” diameter strands per Standard Specifications 6-02.3(26)F.


8. Number of strands per web.
9. Anchorage system shall conform to *Standard Specifications* 6-02.3(26)B to D. The anchorage assembly dimensions and reinforcement detailing shall conform to the corresponding post-tensioning catalog.

10. The curvature friction coefficient and wobble friction coefficient. The curvature friction coefficient shall be per Section 5.8.3.D. The wobble friction coefficient of \( k = 0.0002/\text{ft} \) is often used. These coefficients may be revised by the post-tensioning supplier if approved by the design engineer and conform to the *Standard Specifications* 6.02.3(26)G.

11. Post-tensioning stressing sequence.

12. Tendon stresses shall not exceed the following limits for low relaxation strands as specified in Section 5.8.3.D:
   1. \( 0.81f_{pu} \) at anchor ends immediately before seating.
   2. \( 0.70f_{pu} \) at anchor ends immediately after seating.
   3. \( 0.74f_{pu} \) at the end point of length influenced by anchor set.

13. Elongation calculations for each jacking operation shall be verified. If the difference in tendon elongation exceeds 2%, the elongation calculations shall be separated for each tendon per *Standard Specifications* 6-02.3(26) A.

14. Vent points shall be provided at all high points along tendon per *Standard Specifications* 6-02.3(26)E4.

15. Drain holes shall be provided at all low points along tendon per *Standard Specifications* 6-02.3(26)E4.

16. The concrete strength at the time of post-tensioning, \( f'_{ci} \) shall not be less than 4,000 psi or the strength specified in the plans per *Standard Specifications* 6-02.3(26)G. Different concrete strength may be used if specified in the contract plans.

17. Concrete stresses at the anchorage shall be checked per *Standard Specifications* 6-02.3(26)C for normal anchorage devices. For special anchorage devices, if not covered in the Appendix 5-B2 for pre-approved list of post-tensioning system, testing per *Standard Specifications* 6-02.3(26)D is required.

E. **During Construction**

1. If the measured elongation of each strand tendon is within \( \pm 7\% \) of the approved calculated elongation, the stressed tendon is acceptable.

2. If the measured elongation is greater than \( 7\% \), force verification after seating (lift-off force) is required. The lift-off force shall not be less than 99% of the approved calculated force nor more than 70% \( f_{pu} A_s \).

3. If the measured elongation is less than \( 7\% \), the bridge construction office will instruct the force verification.
4. One broken strand per tendon is usually acceptable. (Post-tensioning design shall preferably allow one broken strand). If more than one strand per tendon is broken, the group of tendon per web should be considered. If the group of tendons in a web is under-stressed, then the adequacy of the entire structure shall be investigated by the designer and consulted with the Bridge Construction Office.

5. Failed anchorage is usually taken care of by the Bridge Construction Office.

6. Over or under elongation is usually taken care of by the Bridge Construction Office.

7. In case of low concrete strength the design engineer shall investigate the adequacy of design with lower strength.

8. Other problems such as unbalanced and out of sequence post-tensioning, strands surface condition, strand subjected to corrosion and exposure, delayed post-tensioning due to mechanical problems, Jack calibration, etc. should be evaluated on a case-by-case basis and are usually taken care by Bridge Construction Office.

5.8.7 Post-tensioning Notes — Cast-in-place Girders

A. General – The design plans shall contain the following information for use by the post-tensioned and state inspector:

1. Tendon jacking sequence,
2. Friction coefficients
3. Duct type
4. Elastic and time-dependent losses
5. Anchor set
6. Prestress forces
7. Falsework construction and removal
8. Minimum number of strands, if required for ultimate moment capacity

If jacking is done at both ends of the bridge, the minimum strand elongation due to the specified jacking load for the end jacked first as well as the end jacked last shall be indicated. The calculated strand elongations at the ends of the bridge are compared with the measured field values to ensure that the friction coefficients (and hence the levels of prestressing throughout the structure) agree with the values assumed by the designer.

The tendons shall be jacked in a sequence that avoids causing overstress or tension in the bridge.

The standard post-tensioning notes for the sequence of stressing of longitudinal tendons shall be shown in the Contract Plans.
5.9 Spliced Prestressed Concrete Girders

5.9.1 Definitions

The provisions herein apply to precast girders fabricated in segments that are spliced longitudinally to form the girders in the final structure. The cross-section for this type of bridge is typically composed of bulb tee girders or trapezoidal tub girders with a composite CIP deck. WSDOT standard drawings for spliced I-girders are shown in Appendices 5.9-A1 through 5.9-A3, and for spliced-tub girders are shown in Appendices 5.9-A4 and 5.9-A5. Span capabilities of spliced prestressed concrete girders are shown in Appendices 5.6-A1-8 for I-girders and 5.6-A1-9 for trapezoidal tub girders.

Prestressed concrete deck bulb tee girder bridges may also be fabricated in segments and spliced longitudinally. Splicing in this type of girder may be beneficial because the significant weight of the cross-section may exceed usual limits for handling and transportation. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of AASHTO LRFD 5.14.4.3.

Spliced prestressed concrete girder bridges may be distinguished from what is referred to as “segmental construction” in bridge specifications by several features which typically include:

• The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span.

• Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.

• Wet-cast closure joints are usually used to join girder segments rather than match-cast joints.

• The bridge cross-section is composed of girders with a CIP concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be integrally cast with each girder. Connecting the girders across the longitudinal joints completes a bridge of this type.

• Girder sections are used, such as bulb tee, deck bulb tee or tub girders, rather than closed cell boxes with wide monolithic flanges.

• Provisional ducts are required for segmental construction to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced prestressed concrete girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

• The method of construction and any required temporary support is of paramount importance in the design of spliced prestressed concrete girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.
All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents. Stresses due to changes in the structural system, in particular the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude.

Prestress losses in spliced prestressed concrete girder bridges shall be estimated using the provisions of Section 5.1.4. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered. When required, the effects of creep and shrinkage in spliced prestressed concrete girder bridges shall be estimated using the provisions of Section 5.1.1.

5.9.2 WSDOT Criteria for Use of Spliced Girders

See Section 5.6.3.D.3 for criteria on providing an alternate spliced-girder design for long span one-piece pretensioned girders.

5.9.3 Girder Segment Design

A. **Design Considerations** – Stress limits for temporary concrete stresses in girder segments before losses and stress limits for concrete stresses in girder segments at the service limit state after losses specified in Section 5.2.1.C shall apply at each stage of pretensioning or post-tensioning with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for \( f'_{ci} \) in the stress limits.

The designer shall consider requirements for bracing of the girder segments once they have been erected on the substructure. Any requirements for temporary or permanent bracing during subsequent stages of construction, along with the contractor's responsibilities for designing and placing them, shall be specified in the contract documents.

Effects of curved tendons shall be considered per Section 5.8.1.F.

B. **Post-tensioning** – Post-tensioning may be applied either before and/or after placement of bridge deck concrete. Part of the post-tensioning may be applied prior to placement of the deck concrete, with the remainder placed after deck concrete placement. In the case of multi-stage post-tensioning, ducts for tendons to be tensioned before the deck concrete shall not be located in the deck.

All post-tensioning tendons shall be fully grouted after stressing. Prior to grouting of post-tensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers.

Where some or all post-tensioning is applied after the bridge deck concrete is placed, fewer post-tensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the post-tensioning is applied before the deck concrete is placed, a greater number of post-tensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary.
5.9.4 Joints Between Segments

A. General – Cast-in-place closure joints are typically used in spliced girder construction. The sequence of placing concrete for the closure joints and bridge deck shall be specified in the contract documents. Match-cast joints shall not be specified for spliced girder bridges unless approved by the Bridge Design Engineer. Prestress, dead load, and creep effects may cause rotation of the faces of the match-cast joints prior to splicing. If match cast joint is specified, the procedures for splicing the girder segments that overcome this rotation to close the match-cast joint shall be shown on the contract plans.

B. Location of Closure Joints – The location of intermediate diaphragms shall be offset by at least 2′-0″ from the edge of cast-in-place closure joints.

In horizontally curved spliced girder bridges, intermediate diaphragms could be located at the CIP closure joints if straight segments are spliced with deflection points at closures. In this case, the diaphragm could be extended beyond the face of the exterior girder for improved development of diaphragm reinforcement.

The final configuration of the closures shall be coordinated with the State Bridge and Structures Architect on all highly visible bridges, such as bridges over vehicular or pedestrian traffic.

C. Details of Closure Joints – The length of a closure joint between concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of post-tensioning ducts. The length of a closure joint shall not be less than 2′-0″. A longer closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

Web reinforcement within the joint shall be the larger of that in the adjacent girders. The face of the segments at closure joints shall be specified as intentionally roughened surface.

Concrete cover to web stirrups at the CIP closures of pier diaphragms shall not be less than 2½″. If intermediate diaphragm locations coincide with CIP closures between segments, then the concrete cover at the CIP closures shall not be less than 2½″. This increase in concrete cover is not necessary if intermediate diaphragm locations are away from the CIP closures. See Figures 5.9.4-1 to 5.9.4-3 for details of closure joints.

Adequate reinforcement shall be provided to confine tendons at CIP closures and at intermediate pier diaphragms. The reinforcement shall be proportioned to ensure that the steel stress during the jacking operation does not exceed 0.6fy.

The clear spacing between ducts at CIP closures of pier diaphragms shall be 2.0″ minimum. The duct diameter for WSDOT standard spliced girders shall not exceed 4.0″ for spliced I-girders and 4½″ for spliced tub girders.

On the construction sequence sheet indicate that the side forms at the CIP closures and intermediate pier diaphragms shall be removed to inspect for concrete consolidation prior to post-tensioning and grouting.

Self-consolidating concrete (SCC) may be used for CIP closures.
D. **Joint Design** – Stress limits for temporary concrete stresses in joints before losses specified in Section 5.2.1.C shall apply at each stage of post-tensioning. The concrete strength at the time the stage of post-tensioning is applied shall be substituted for $f'_{ci}$ in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in Section 5.2.1.C shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for $f'_c$ in the stress limits. The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations.
CIP Closure at Pier Diaphragm

*Figure 5.9.4-1*
CIP Closure Away from Intermediate Diaphragm

*Figure 5.9.4-2*
CIP Closure at Intermediate Diaphragm

Figure 5.9.4-3
5.9.5 **Review of Shop Plans for Spliced Prestressed Concrete Girders**

Shop drawings for spliced prestressed concrete girders shall be reviewed by the designer (or Bridge Technical Advisor for non-Bridge Office projects) and consulted with the Concrete Specialist if needed. Review of shop drawing shall include:

1. All prestressing strands shall be of ½" or 0.6” diameter grade 270 low relaxation uncoated strands.
2. Number of strands per segment.
3. Pretensioning strands jacking stresses shall not exceed $0.75f_{pu}$.
4. Strand placement patterns.
5. Temporary strand placement patterns, location and size of blockouts for cutting strands.
6. Procedure for cutting temporary strands and patching the blockouts shall be specified.
7. Number and length of extended strands and rebars at girder ends.
8. Location of holes and shear keys for intermediate and end diaphragms.
9. Location and size of bearing recesses.
10. Saw tooth at girder ends.
11. Location and size of lifting loops or lifting bars.
12. Number and size of horizontal and vertical reinforcement.
13. Segment length and end skew.
14. Tendon profile and tendon placement pattern.
15. Duct size shall be based on the duct area at least 2.5 times the total area of prestressing strands.
16. Anchor set. The post-tensioning design is typically based on an anchor set of ½”.
17. Maximum number of strands per tendon shall not exceed (37) ½” diameter strands or (27) 0.6” diameter strands per *Standard Specifications* 6-02.3(26)F.
18. Jacking force per girder.
20. Number of strands per web.
21. Anchorage system shall conform to pre-approved list of post-tensioning system per Appendix 5-B4. The anchorage assembly dimensions and reinforcement detailing shall conform to the corresponding post-tensioning catalog.
22. The curvature friction coefficient and wobble friction coefficient. The curvature friction coefficient of $\mu = 0.15$ for bridges less than 400 feet, $\mu = 0.2$ for bridges between 400 feet and 800 feet, and $\mu = 0.25$ for bridges longer than 800 feet. The wobble friction coefficient of $k = 0.0002/\text{ft}$ is often used. These coefficients may be revised by the post-tensioning supplier if approved by the design engineer and conform to the Standard Specifications 6.02.3(26)G.

23. Post-tensioning stressing sequence.

24. Tendon stresses shall not exceed the following limits for low relaxation strands as specified in Section 5.8.3.D:
   - $0.81f_{pu}$ at anchor ends immediately before seating.
   - $0.70f_{pu}$ at anchor ends immediately after seating.
   - $0.74f_{pu}$ at the end point of length influenced by anchor set.

25. Elongation calculations for each jacking operation shall be verified. If the difference in tendon elongation exceeds 2%, the elongation calculations shall be separated for each tendon per Standard Specifications Section 6-02.3(26)A.

26. Vent points shall be provided at all high points along tendon.

27. Drain holes shall be provided at all low points along tendon.

28. The concrete strength at the time of post-tensioning, $f'_{ci}$ shall not be less than 4,000 psi per Standard Specifications 6-02.3(26)G. Different concrete strength may be used if specified in the contract plans.

29. Concrete stresses at the anchorage shall be checked per Standard Specifications Section 6-02.3(26)C for bearing type anchorage. For other type of anchorage assemblies, if not covered in the Appendix 5-B2 for pre-approved list of post-tensioning system, testing per Standard Specifications Section 6-02.3(26)D is required.

30. Concrete stresses at CIP closures shall conform to allowable stresses of Table 5.2.1-1.

5.9.6 Post-tensioning Notes — Spliced Prestressed Concrete Girders

1. The CIP concrete in the bridge deck shall be Class 4000D. The minimum compressive strength of the CIP concrete at the wet joint at the time of post-tensioning shall be …. ksi.

2. The minimum prestressing load after seating and the minimum number of prestressing strands for each girder shall be as shown in post-tensioning table.

3. The design is based on ….. inch diameter low relaxation strands with a jacking load for each girder as shown in post-tensioning table, an anchor set of $\frac{3}{8}''$ a curvature friction coefficient, $\mu = 0.20$ and a wobble friction coefficient, $k = 0.0002/\text{ft}$. The actual anchor set used by the contractor shall be specified in the shop plans and included in the transfer force calculations.

4. The design is based on the estimated prestress loss of post-tensioned prestressing strands as shown in post-tensioning table due to steel relaxation, elastic shortening, creep and shrinkage of concrete.
5. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations. The stressing sequence shall meet the following criteria:

A. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the bridge.

B. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent webs. At no time during stressing operation will more than one-sixth of the total prestressing force is applied eccentrically about the centerline of bridge.

6. The maximum outside diameter of the duct shall be …. inches. The area of the duct shall be at least 2.5 times the net area of the prestressing steel in the duct.

7. All tendons shall be stressed from pier …. 
5.99 References


3. PCI Bridge Design Manual, Precast/Prestressed Concrete Institute, Chicago, IL, 1997.

4. ACI 318-02, Building Code Requirements for Reinforced Concrete and Commentary, American Concrete Institute, 1989, pp. 353.


14. Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products, Precast/Prestressed Concrete Institute, Chicago, IL, 2006.

15. Transportation Research Board Report No. 226 titled, Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members.

16. Transportation Research Board Report No. 280 titled, Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members.

17. Post-tensioned Box Girder Bridge Manual, Post-tensioning Institute, 301 West Osborn, Phoenix, Arizona.


24. TRAC Report WA-RD 696.1, "Effect of Intermediate Diaphragms to Prestressed Concrete Bridge Girders in Over-Height Truck Impacts” completed on April 2008 by the Washington State University.

Appendix 5.1-A1 Standard Hooks

RECOMMENDED END HOOKS
All Grades
D = Finished bend diameter

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STIRRUP AND TIE HOOK DIMENSIONS
All Grades

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135° SEISMIC STIRRUP/TIE HOOK DIMENSIONS
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FOR BEAMS AND COLUMNS.
(DISTANCES IN INCHES)

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## Appendix 5.1-A4

### Tension Development Length of Deformed Bars

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Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. Top bars are horizontal bars placed so that more than 12” of fresh concrete is cast below the reinforcement.
4. The minimum tension development length = 12”.
5. $\lambda_{rc}$ is the Reinforcement Confinement Factor.
### Notes:
1. Values based on use of normal weight concrete.
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4. The minimum tension development length = 12".
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Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. Top bars are horizontal bars placed so that more than 12” of fresh concrete is cast below the reinforcement.
4. The minimum tension development length = 12”.
5. $\lambda_{rc}$ is the Reinforcement Confinement Factor.
## Compression Development Length and Appendix 5.1-A5 Minimum Lap Splice of Grade 60 Bars

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**Notes:**

1. Where excess bar area is provided, the development length may be reduced by the ratio of required area to provided area.
2. Where reinforcement is enclosed within a spiral composed of a bar of not less than 0.25 inches in diameter and spaced at not more than a 4.0 inch pitch, the compression development length may be multiplied by 0.75.
3. The minimum compression development length is 12 inches.
4. Where bars of different size are lap spliced in compression, the splice length shall not be less than the development length of the larger bar or the splice length of the smaller bar.
5. Where ties along the splice have an effective area not less than 0.15 percent of the product of the thickness of the compression component times the tie spacing, the compression lap splice may be multiplied by 0.83.
6. Where the splice is confined by spirals, the compression lap splice may be multiplied by 0.75.
7. The minimum compression lap splice length is 24 inches.
# Appendix 5.1-A6

## Tension Development Length of 90° and 180° Standard Hooks

### Standard Hook Tension Development Length $l_{dh}$ (in)

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Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. The basic development length $l_{hb}$ shall be multiplied by 1.2 for epoxy coated reinforcement.
4. The basic development length $l_{hb}$ may be reduced by the ratio of required area to provided area where excess bar area is provided.
5. The basic development length $l_{hb}$ may be multiplied by 0.8 for #11 and smaller bars for hooks with side cover normal to plane of the hook not less than 2.5 inches, and for 90 degree hook with cover on the bar extension beyond hook not less than 2.0 inches.
6. The basic development length $l_{hb}$ may be multiplied by 0.8 for 90 degree hooks of #11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length, $l_{dh}$, of the hook; or enclosed within ties or stirrups parallel to the bar being developed spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend, and in both cases the first tie or stirrup enclosing the bent portion of the hook is within $2d_b$ of the outside of the bend.
7. The basic development length $l_{hb}$ may be multiplied by 0.8 for 180 degree hooks of #11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length, $l_{dh}$, of the hook, and the first tie or stirrup enclosing the bent portion of the hook is within $2d_b$ of the outside of the bend.
8. Minimum tension development length is the larger of $8d_b$ and 6 inches.
## Tension Lap Splice Lengths of Grade 60 Bars – Class B

### Appendix 5.1-A7

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Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. Top bars are horizontal bars placed so that more than 12” of fresh concrete is cast below the reinforcement.
4. The minimum tension lapsplice length = 24”.
5. $\lambda_{rc}$ is the Reinforcement Confinement Factor.
6. Class A tension lap splices may be used where the area of reinforcement provided is at least twice that required by analysis over the entire length of the lap splice and one-half or less of the total reinforcement is spliced within the required lap splice length. The Class A modification factor is 0.77.
### Class B Tension Lap Splice Length of Epoxy Coated Deformed Bars (in)

(cover less than $3_{db}$ or clear spacing between bars less than $6_{db}$)

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Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. Top bars are horizontal bars placed so that more than 12" of fresh concrete is cast below the reinforcement.
4. The minimum tension lap splice length = 24".
5. $\lambda_{rc}$ is the Reinforcement Confinement Factor.
6. Class A tension lap splices may be used where the area of reinforcement provided is at least twice that required by analysis over the entire length of the lap splice and one-half or less of the total reinforcement is spliced within the required lap splice length. The Class A modification factor is 0.77.
### Class B Tension Lap Splice Length of Epoxy Coated Deformed Bars (in)
*(cover not less than 3\(\text{db}\) and clear spacing between bars not less than 6\(\text{db}\))

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<td>168.10 129.31</td>
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**Notes:**
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. Top bars are horizontal bars placed so that more than 12” of fresh concrete is cast below the reinforcement.
4. The minimum tension lap splice length = 24”.
5. \(\lambda_{rc}\) is the Reinforcement Confinement Factor.
6. Class A tension lap splices may be used where the area of reinforcement provided is at least twice that required by analysis over the entire length of the lap splice and one-half or less of the total reinforcement is spliced within the required lap splice length. The Class A modification factor is 0.77.
### AASHTO M203 Grade 270 Uncoated Prestressing Strands

#### Prestressing Strand Properties and Development Length

<table>
<thead>
<tr>
<th>Strand Diameter (in)</th>
<th>Weight (lbs/ft)</th>
<th>Nominal Diameter (in)</th>
<th>Area (in(^2))</th>
<th>Transfer length (in)</th>
<th>Develop. Length (k = 1.0) (ft)</th>
<th>Develop. Length (k = 1.6) (ft)</th>
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<tr>
<td>(\frac{3}{8})</td>
<td>0.290</td>
<td>0.375</td>
<td>0.085</td>
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<td>0.438</td>
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<tr>
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<td>30.0</td>
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<tr>
<td>(\frac{1}{2}) S</td>
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<td>0.520</td>
<td>0.167</td>
<td>31.2</td>
<td>7.01</td>
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<tr>
<td>(\frac{9}{16})</td>
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<td>0.563</td>
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<td>0.700</td>
<td>0.294</td>
<td>42.0</td>
<td>9.43</td>
<td>15.09</td>
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</table>

**Assumptions for determining development length:**

\[
\begin{align*}
    f_{ps} &= f_{pu} = 270 \text{ ksi} \\
    f_{pe} &= (270 \text{ ksi} \times 0.75) - 40 \text{ ksi} = 162.5 \text{ ksi}
\end{align*}
\]
Appendix 5.2-A1

Working Stress Design

Service Load — Concrete Stresses and Constants

<table>
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<th>Class</th>
<th>Class</th>
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<tr>
<td>$n$</td>
<td>$f_c'$</td>
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<tr>
<td>$f_c$</td>
<td>$f' c$</td>
</tr>
<tr>
<td>$f_{c}(Compression)$</td>
<td>$f_{c}(Tension)$ Use only with special permission</td>
</tr>
<tr>
<td>$f_s$ (Grade 40)</td>
<td>$f_s$ (Grade 50)</td>
</tr>
<tr>
<td>$V_c$ (With web reinf.)</td>
<td>$V_c$ (With web reinf.)</td>
</tr>
<tr>
<td>$k$ Balanced rectangular sections</td>
<td>$K = \frac{f_s}{f' c + 0.0002}$</td>
</tr>
<tr>
<td>$j$</td>
<td>$\frac{0.0156}{174,000}$</td>
</tr>
<tr>
<td>$p$</td>
<td>$0.0125$</td>
</tr>
<tr>
<td>$E_c$ (for stress calc.) $(n$ as above)</td>
<td>$E_c$ (for short term defl due to E.Q., etc.) $(n + 8)$</td>
</tr>
<tr>
<td>$E_c$ (for D.L. Camber of Slabs, Ft5 Mmrs., Settlement) $(n + 16)$</td>
<td>$E_c$ (for D.L. Camber, except slabs) $(n + 24)$</td>
</tr>
</tbody>
</table>

Temp. Coeff. = .000006 V/s, $\Delta T$ ~ 45°F Drop to 35°F Rise — All climates.
Shrink age Coeff. = .0002% (Temp. rise & shrinkage cancel).

*For more detailed analysis $V = 0.9(f' c)^{0.5} + 1000 0.4(V_c)$

See 1974 A.A.S.H.D. Interim 1.5.29 (B)(2).

Stirrup spacing: $S = \frac{A_s \times f_s \times j \times d}{V - V_c} \times \frac{V}{V_c} \times \frac{1.750}{A_s \times d}$

(kips/inch units)

$A_s = $ Total area of stirrup legs.
$V_s = $ Total shear taken by stirrups.
$V = $ Total shear on section.
$V_c = $ Total shear by conc. $V_c = bjd$

$d = \sqrt{\frac{My}{bK}}$ (Balanced rectangular section)

$f_c = \frac{2M}{k/4b d^2}$ (Rectangular section)

$f_s = \frac{M}{A_s \times d}$

$v = \frac{V}{b d}$

$n_c = \frac{E_s}{E_c}$
### Appendix 5.2-A3 Working Stress Design

#### COEFFICIENTS (K, k, j, j) FOR RECTANGULAR SECTIONS

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<tr>
<th>$f'_c$ and n</th>
<th>$f_c$</th>
<th>K</th>
<th>k</th>
<th>j</th>
<th>p</th>
<th>$f'_c$</th>
<th>K</th>
<th>k</th>
<th>j</th>
<th>p</th>
</tr>
</thead>
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<tr>
<td>2500</td>
<td>875.</td>
<td>137.</td>
<td>.356</td>
<td>.881</td>
<td>.0097</td>
<td>128.</td>
<td>.329</td>
<td>.890</td>
<td>.0080</td>
<td>1000.</td>
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<td>1125.</td>
<td>201.</td>
<td>.415</td>
<td>.862</td>
<td>.0146</td>
<td>190.</td>
<td>.387</td>
<td>.871</td>
<td>.0121</td>
<td>1250.</td>
<td>235.</td>
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<tr>
<td>3000.</td>
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<td>.875</td>
<td>.0154</td>
<td>162.</td>
<td>.349</td>
<td>.884</td>
<td>.0102</td>
<td>1500.</td>
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<td>.842</td>
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<td>536.</td>
<td>.545</td>
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<td>.0409</td>
<td>513.</td>
<td>.516</td>
<td>.828</td>
<td>.0344</td>
<td>2500.</td>
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</table>

**Table continues...**

*$^a$Balanced steel ratio* applies to problems involving bending only.
Appendix 5.3-A2  
Negative Moment Reinforcement

\[ M = 0 \]

Adjusted Negative Moment Curve

RESIST. M

Inflection Point for Negative Moment

\[ \geq \frac{d}{3} \text{ or } \frac{15d}{50} \text{ or } \frac{5d}{20} \]

Two bars extended for stirrup hangers, anchor beyond \( \frac{d}{3} \) END SUPPORT.

\[ \text{Bar embed.} \geq \frac{d}{10} \]

\[ L_n \text{ (clear span)} \]

\[ \frac{1}{5} \text{ of } A_b \text{ NEG.} \]

\[ \text{Bars } b \]

\[ \text{Inf. Pier} \]

\[ \text{End Support} \]

\[ \text{MIN. PER GIRDER EXTEND } d + \frac{15d}{5} \text{ or } \frac{35d}{2} \text{ or } \frac{5d}{20} \text{ EXCEPT 2 BARS D} \]
Adjusted Negative Moment Case I
(Design for $M$ at Face of Support)

CASE I (DESIGN FOR $M$ AT FACE OF EFFECTIVE SUPPORT) APPLIES TO GIRDERs, BEAMS OR X-BEAMS WHERE THE SUPPORT INCREASES THE DEPTH OF THE BEAM EXCEPT FOR CASES WHERE:

1. THE INCREASE IN DEPTH DUE TO THE SUPPORT IS INSUFFICIENT TO RESIST THE MOMENT AT $\ell$ SUPPORT; THAT IS
   $d \ell < d_{face} \frac{M_{face}}{\ell}$
2. CONTINUOUS BEAMS WHERE ONE-HALF THE LENGTH OF SUPPORT DIVIDED BY THE SPAN IS GREATER THAN 0.1:
   \[ \frac{W/2}{SPAN} > 0.1 \]

WHERE CASE 1. OR 2. APPLIES USE CASE II.

PROVIDE MINIMUM FLEXURAL REINFORCEMENT PER AASHTO 8.17

CALCULATE $A_e$ REQUIRED FOR THIS MOMENT USING $a$ & $d$ VALUES AT FACE, CHECK THAT $A_e \leq 75\%$ OF BALANCED REINF. FOR TAPERED BEAMS A MORE CRITICAL SECTION MAY EXIST AT OTHER POINTS ALONG THE BEAM.
Adjusted Negative Moment Case II
(Design for M at ¼ Point)

CASE II (DESIGN FOR M ¼ POINT OF SUPPORT) APPLIES TO GIRDERS, BEAMS, OR X-BEAMS WHERE ONE OF THE FOLLOWING SUPPORT CONDITIONS EXIST:

1. NO INCREASE IN BEAM DEPTH CAN BE ATTRIBUTED TO THE SUPPORT.
2. THE INCREASE IN DEPTH DUE TO THE SUPPORT IS INSUFFICIENT TO RESIST THE MOMENT AT ¼ SUPPORT; THAT IS
   \[ d\bar{e} \leq \frac{M}{4\text{ }\text{face}} \]
3. CONTINUOUS BEAMS WHERE ONE-HALF THE LENGTH OF SUPPORT DIVIDED BY THE SPAN IS GREATER THAN 0.1:
   \[ \left( \frac{W/2}{\text{SPAN}} > 0.1 \right) \]

TYPICAL SECTION

CALCULATE \( A_s \) REQUIRED FOR THIS MOMENT USING \( a \) & \( d \) VALUES AT FACE. CHECK THAT \( A_s \leq 75\% \) OF BALANCED REINF. FOR TAPERED BEAMS A MORE CRITICAL SECTION MAY EXIST AT OTHER POINTS ALONG THE BEAM.
Cast-In-Place Deck Slab Design for Appendix 5.3-A5 Positive Moment Regions $f'_c = 4.0$ ksi

Required Bar Spacing for Girder Spacings and Slab Thicknesses for the Positive Moment Region

- Maximum Bar Spacing = 12
- #6 Bars
- #5 Bars

Note: Control of cracking by distribution of reinforcement is not shown.
Cast-In-Place Deck Slab Design for Appendix 5.3-A6 Negative Moment Regions

\( f'_{c} = 4.0 \text{ ksi} \)

Required Bar Spacing for Girder Spacings and Slab Thicknesses for the Negative Moment Region

Maximum Bar Spacing = 12"

#6 Bars

#5 Bars

Note: Control of cracking by distribution of Reinforcement is not checked.

Girder Spacing in Feet

Bar Spacing in Inches

4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5 9.0 9.5 10.0 10.5 11.0 11.5 12.0

4 5 6 7 8 9 10 11 12 13 14

7.5" Slab
8.0" Slab
8.5" Slab
9.0" Slab
Slab Overhang Required Reinforcement for Vehicle Impact - Interior Barrier Segment - LRFD A13.4.1 Design Case 1

Notes:
1. Top and bottom mats each carry one-half the tension impact load.
2. Only Design Case 1 of LRFD A13.4.1 is considered. Designer must also check Design Cases 2 and 3.
3. Section considered is a vertical section through the slab overhang at the toe of the barrier.
Slab Overhang Required Reinforcement for Vehicle Impact - End Barrier Segment - LRFD A13.4.1 Design Case 1

Notes:
1. Top and bottom mats each carry one-half the tension impact load.
2. Only Design Case 1 of LRFD A13.4.1 is considered. Designer must also check Design Cases 2 and 3.
3. Section considered is a vertical section through the slab overhang at the toe of the barrier.
## Appendix 5.6-A1-1  Span Capability of W Girders

<table>
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<th>Girder Type</th>
<th>Girder Spacing (ft)</th>
<th>CL Bearing to CL Bearing (ft)</th>
<th>“A” Dim. (in)</th>
<th>Deck Thickness (in)</th>
<th>Shipping Weight (kips)</th>
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<td>98</td>
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</tbody>
</table>

Design Parameters:
- PGSuper version 2.2.3.0
- Girder $f'_{ci} = 7.5$ ksi, $f'_{c} = 9.0$ ksi
- Slab $f'_{c} = 4.0$ ksi
- No horizontal or vertical curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
### Appendix 5.6-A1-2  Span Capability of WF Girders

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**Design Parameters:**
- PGSuper Version 2.2.3.0
- Girder $f_{ci} = 7.5$ ksi, $f_{c} = 9.0$ ksi
- Slab $f_{c} = 4.0$ ksi.
- No vertical or horizontal curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
Span Capability of
Deck Bulb Tee Girders

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Design Parameters:
- PGSuper version 2.2.3.0
- Girder $f'_{ci} = 7.5$ ksi, $f'_c = 9.0$ ksi
- Slab $f'_c = 4.0$ ksi
- No horizontal or vertical curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
## Appendix 5.6-A1-4

### Span Capability of WF Thin Deck Girders

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Design Parameters:
- PGSuper Version 2.8.0
- Girder f′ci = 7.5 ksi, f′c = 9.0 ksi
- 1/2 D40 ≥ C
- Shipping weight < 270 kip
- Slab f′c = 4.0 ksi.
- No vertical or horizontal curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
## Appendix 5.6-A1-5

### Span Capability of WF Deck Girders

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<th>Girder Type</th>
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Design Parameters:
- PGSuper Version 2.8.0
- Girder $f_{ci} = 7.5$ ksi, $f_c = 9.0$ ksi
- 1/2 D40 ≥ C
- Shipping weight < 270 kip
- No vertical or horizontal curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- 1½” concrete overlay
## Span Capability of Trapezoidal Tub Girders without Top Flange

**Appendix 5.6-A1-6**

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# Span capability exceeds maximum shipping weight of 252 kips

**Design Parameters:**
- PGSuper Version 2.2.3.0
- Girder f'ci = 7.5 ksi, f'c = 9.0 ksi
- Slab f'c = 4.0 ksi.
- No vertical or horizontal curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
### Span Capability of Trapezoidal Tub Girders with Top Flange

**Appendix 5.6-A1-7**

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*Span capability exceeds maximum shipping weight of 252 kips*

**Design Parameters:**
- PGSuper version 2.2.3.0
- Girder $f'_{ci} = 7.5$ ksi, $f'_{c} = 9.0$ ksi
- CIP slab $f'_{c} = 4.0$ ksi
- No horizontal or vertical curve
- 2% roadway crown slope
- Standard WSDOT "F" shape barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- Includes 2" future HMA overlay with density of 140 pcf
- Deck includes a 3.5" SIP panel with a 5" CIP slab
Span Capability of
Appendix 5.6-A1-8
Post-tensioned Spliced I-Girders

\[ f'c = 6.0 \text{ ksi}, \ f'c = 9 \text{ ksi} \]
Strand diameter = 0.6" Grade 270 ksi low relaxation

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</table>
Concrete Structures Chapter 5

* Controlled by over-reinforced section (see LRFD Sec. 5.7.3.3)
** Total force calculated at jacking end of post-tensioned girder (rounded to the nearest 10)

Design Parameters:
- PGSplice V. 0.3
- WSDOT BDM LRFD design criteria
- No vertical or horizontal curve
- 2.0% roadway crown slope
- Interior girder with barrier load (6 girder bridge)
- Only flexural service and strength checked; lifting and hauling checks not necessarily satisfied
- Simple girder span lengths are CL bearing to CL bearing
- Slab \( f'_c = 4.0 \text{ ksi} \)
- Standard WSDOT “F” shape barrier
- Under normal exposure condition and 75% relative humidity
- Spans reported in 5'-0" increments
- Designs based on “normally” reinforced sections (c/de < 0.42 LRFD 5.7.3.3)
- Designs based on 22 strands/duct
- For 6'-10' girder spacing -- 7.5" slab
- For 12’ girder spacing -- 8.0" slab
- For 14’ girder spacing -- 8.75" slab
- Girders post-tensioned before slab pour are assumed to be post-tensioned adjacent to structure.
- All spec checks at wet joints have been ignored. It is assumed that the designer can modify the wet joints to reach the required span as shown in the table. These modifications are outside the scope of this table.
<table>
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<tr>
<th>Girder Type</th>
<th>Girder Spacing (ft)</th>
<th>Span Length (ft)</th>
<th>End Segments</th>
<th>Middle Segment</th>
<th>Spliced Post-tensioned Girder</th>
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</table>
Concrete Structures Chapter 5

Total force calculated at jacking end of post-tensioned girder
# Span capability exceeds maximum shipping weight of 200 kips

Design Parameters:
- PGSplice V. 0.3
- WSDOT BDM LRFD design criteria
- No vertical or horizontal curve
- 2.0% roadway crown slope
- Interior girder with barrier load (6 girder bridge)
- Only flexural service and strength checked; lifting and hauling checks not necessarily satisfied
- Simple girder span lengths are CL bearing to CL bearing
- Standard WSDOT “F” shape barrier
- Under normal exposure condition and 75% humidity
- Spans reported in 5'-0" increments
- “A” dimension = deck thickness + 2"
- Closure pour for spliced girders is 2', $f'_{ci} = 6.0$ ksi, $f'_c = 9$ ksi
- Girder $f'_{ci} = 6.0$ ksi, $f'_c = 9.0$ ksi, slab $f'_c = 4.0$ ksi
- Girders are spliced in-place after slab is cast
- Prestressing and post-tensioning steel is 0.6" diameter, Grade 270
- End segments are 25% of total length; center segment is 50% of total length
- Range of applicability requirements in LRFD ignored; span lengths may be longer than allowed by LRFD
- Designs are based on a 22 diameter strand limit per 4" duct for high pressure grout
- All spec checks at wet joints have been ignored. It is assumed that the designer can modify the wet joints to reach the required span as shown in the table. These modifications are outside the scope of this table.
## Prestressed Concrete I and WF Girders

### Wide Flange (WF) Girders

<table>
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<tr>
<th>WF36G</th>
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<th>WF58G</th>
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<th>WF74G</th>
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<td>SPAN LENGTH = 110 FT.</td>
<td>SPAN LENGTH = 135 FT.</td>
<td>SPAN LENGTH = 135 FT.</td>
<td>SPAN LENGTH = 155 FT.</td>
<td>SPAN LENGTH = 170 FT.</td>
<td>SPAN LENGTH = 180 FT.</td>
<td>SPAN LENGTH = 190 FT.</td>
<td>SPAN LENGTH = 210 FT.</td>
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</table>

### Span Lengths Shown

Span lengths shown are based on concrete compressive strengths of 7.5 ksi at transfer and 10.0 ksi at final.
SPAN LENGTHS SHOWN ARE THE MAXIMUM EXPECTED FOR EACH TYPE OF GIRDER USING POSTPLICE PROGRAM.

THE CONCRETE COMPRESSIVE STRENGTHS FOR STANDARD DESIGNS ARE LIMITED TO 7.5 ksi AT TRANSFER AND 9.0 ksi AT FINAL.

THE DESIGN IS BASED ON 0.6” DIAM. LOW RELAXATION PRESTRESSING STRANDS.

STRENGTH OF CONCRETE AT THE CLOSURES SHALL NOT EXCEED 6.0 ksi FOR POST-TENSIONING BEFORE BRIDGE DECK CASTING AND 4.0 ksi FOR POST-TENSIONING AFTER BRIDGE DECK CASTING.

POST-TENSIONED TUB SECTIONS MAY BE CURVED. POST-TENSIONED BEFORE BRIDGE DECK CASTING.

POST-TENSIONED AFTER BRIDGE DECK CASTING.

NOTES:
1. SPAN LENGTHS SHOWN ARE THE MAXIMUM EXPECTED FOR EACH TYPE OF GIRDER USING POSTPLICE PROGRAM.
2. THE CONCRETE COMPRESSIVE STRENGTHS FOR STANDARD DESIGNS ARE LIMITED TO 7.5 ksi AT TRANSFER AND 9.0 ksi AT FINAL.
3. THE DESIGN IS BASED ON 0.6” DIAM. LOW RELAXATION PRESTRESSING STRANDS.
4. STRENGTH OF CONCRETE AT THE CLOSURES SHALL NOT EXCEED 6.0 ksi FOR POST-TENSIONING BEFORE BRIDGE DECK CASTING AND 4.0 ksi FOR POST-TENSIONING AFTER BRIDGE DECK CASTING.
5. POST-TENSIONED TUB SECTIONS MAY BE CURVED.
6. POST-TENSIONED BEFORE BRIDGE DECK CASTING.
   ** POST-TENSIONED AFTER BRIDGE DECK CASTING.
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**NOTE:**
1. SPAN LENGTHS SHOWN ARE THE MAXIMUM FOR EACH TYPE OF GIRDER USING FOSUPER PROGRAM.
2. THE CONCRETE COMPRESSIVE STRENGTHS FOR STANDARD DESIGNS ARE LIMITED TO 7.5 ksi AT TRANSFER AND 9.0 ksi AT FINAL.
3. THE DESIGN IS BASED ON 0.6" DIAM. LOW RELAXATION PRESTRESSING STRANDS.
**INSTALL TEMPORARY BRACING FOR ERECTION IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F4.**

**INSTALL TEMPORARY BRACING FOR DIAPHRAGM AND DECK PLACEMENT IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F5.**

1. ERECT AND BRACE GIRDERs.
2. JUST PRIOR TO CUTTING THE TEMPORARY STRANDS, REMOVE EXPANDED POLYSTYRENE IN RECESSES IN THE TOP FLANGES OF GIRDER.
3. CUT STRANDS IN RECESSES. STRANDS MAY BE CUT BY USING A CUTTING TORCH AND MOVING THE FLAME BACK AND FORTH OVER THE LENGTH OF EXPOSED STRAND TO GET INDIVIDUAL WIRES TO BREAK ONE AT A TIME TO LESS THE SHOCK TO THE GIRDER. STRANDS SHALL BE RELEASED IN A SYMMETRICAL MANNER ABOUT THE GIRDER CENTERLINE STARTING WITH THOSE NEAREST THE CENTERLINE AND WORKING OUTWARDS.
4. DO NOT ALLOW ANY MOISTURE TO ENTER THE RECESSES.
5. REMOVE ALL MOISTURE IN RECESSES PRIOR TO FILLING THEM WITH GROUT.
6. WITHIN 24 HOURS OF CUTTING THE TEMPORARY STRANDS, FILL THE RECESSES WITH A GROUT CONFORMING TO STD. SPEC. 9-20.3(2).

**TEMPORARY STRAND CUTTING SEQUENCE**

1. ERECT AND BRACE GIRDERS.
2. CUT STRANDS IN RECESSES. STRANDS MAY BE CUT BY USING A CUTTING TORCH AND MOVING THE FLAME BACK AND FORTH OVER THE LENGTH OF EXPOSED STRAND TO GET INDIVIDUAL WIRES TO BREAK ONE AT A TIME TO LESS THE SHOCK TO THE GIRDER. STRANDS SHALL BE RELEASED IN A SYMMETRICAL MANNER ABOUT THE GIRDER CENTERLINE STARTING WITH THOSE NEAREST THE CENTERLINE AND WORKING OUTWARDS.
3. DO NOT ALLOW ANY MOISTURE TO ENTER THE RECESSES.
4. REMOVE ALL MOISTURE IN RECESSES PRIOR TO FILLING THEM WITH GROUT.
5. WITHIN 24 HOURS OF CUTTING THE TEMPORARY STRANDS, FILL THE RECESSES WITH A GROUT CONFORMING TO STD. SPEC. 9-20.3(2).

**CONSTRUCTION SEQUENCE - SUPERSTRUCTURE**

**STAGE 1**

**SET GIRDERs IN PLACE**

INSTALL TEMPORARY BRACING FOR ERECTION IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F4.

**STAGE 2**

**CAST DIAPHRAGMS AND PLACE BRIDGE DECK REINFORCEMENT**

INSTALL TEMPORARY BRACING FOR DIAPHRAGM AND DECK PLACEMENT IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F5.

**STAGE 3**

**CAST BRIDGE DECK**

CAST BRIDGE DECK (OR PLACE PRECAST DECK PANELS) WHEN DIAPHRAGM CONCRETE COMPRESSIVE STRENGTH HAS REACHED 3000 PSI (MIN).

**STAGE 4**

**CAST TRAFFIC BARRIERS**

TRAFFIC BARRIER SHALL NOT BE CAST UNTIL THE DECK CONCRETE COMPRESSIVE STRENGTH HAS REACHED 3000 PSI (MIN).
MULTIPLE SPAN PRESTRESSED GIRDER CONSTRUCTION SEQUENCE

1. ERECT AND BRAZE GIRDERS.

2. JUST PRIOR TO CUTTING THE TEMPORARY STRANDS, REMOVE EXPANDED POLYSTYRENE IN RECESSES IN TOP FLANGE OF GIRDERS.

3. CUT STRANDS IN RECESSES. STRANDS MAY BE CUT BY USING A CUTTING TORCH AND MOVING THE FLAME BACK AND FORTH OVER THE LENGTH OF EXPOSED STRAND TO LET INDIVIDUAL WIRES BREAK ONE AT A TIME TO LESSEN THE SHOCK TO THE GIRDER. STRANDS SHALL BE RELEASED IN A SYMMETRICAL MANNER ABOUT THE GIRDER CENTERLINE STARTING WITH THOSE NEAREST THE CENTERLINE AND WORKING OUTWARDS.

4. DO NOT ALLOW ANY MOISTURE TO ENTER THE RECESSES. REMOVE ALL MOISTURE IN RECESSES PRIOR TO FILLING THEM WITH GROUT.

5. WITHIN 24 HOURS OF CUTTING THE TEMPORARY STRANDS, FILL THE RECESSES WITH A GROUT CONFORMING TO STD. SPEC. 9-20.3(2).

NOTE: NO LIVE LOAD SHALL BE ALLOWED ON THE SPANS UNTIL THE COMPRESSIVE STRENGTH OF THE TOP PORTION OF THE PIER DIAPHRAGM HAS REACHED 3000 PSI (MIN.).
INSTALL TEMPORARY BRACING FOR ERECTION IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F4.

INSTALL TEMPORARY BRACING FOR DIAPHRAGM AND DECK PLACEMENT IN ACCORDANCE WITH STD. SPEC. SECTION 6-02.3(17)F5.

ERECT AND BRACE GIRDERS. JUST PRIOR TO CUTTING THE TEMPORARY STRANDS, REMOVE EXPANDED POLYSTYRENE IN RECESSES IN TOP FLANGE OF GIRDERS.

CUT STRANDS IN RECESSES. STRANDS MAY BE CUT BY USING A CUTTING TORCH AND MOVING THE FLAME BACK AND FORTH OVER THE LENGTH OF EXPOSED STRAND TO LET INDIVIDUAL WIRES BREAK ONE AT A TIME TO LESSENTHE SHOCK TO THE GIRDER. STRANDS SHALL BE RELEASED IN A SYMMETRICAL MANNER ABOUT THE GIRDER CENTERLINE STARTING WITH THOSE NEAREST THE CENTERLINE AND WORKING OUTWARDS.

DO NOT ALLOW ANY MOISTURE TO ENTER THE RECESSES. REMOVE ALL MOISTURE IN RECESSES PRIOR TO FILLING THEM WITH GROUT.

WITHIN 24 HOURS OF CUTTING THE TEMPORARY STRANDS, FILL THE RECESSES WITH A GROUT CONFORMING TO STD. SPEC. 9-20.3(2).

NOTE: NO LIVE LOAD SHALL BE ALLOWED ON THE SPANS UNTIL THE COMPRESSIVE STRENGTH OF THE TOP PORTION OF THE GIRDER HAS REACHED 3000 PSI (MIN.).

CAST DIAPHRAGMS AND PLACE BRIDGE DECK REINFORCEMENT.

CAST BRIDGE DECK (OR PLACE PRECAST DECK PANELS) WHEN DIAPHRAGM CONCRETE COMPRESSIVE STRENGTH HAS REACHED 3000 PSI (MIN.) AND ALL FALSEWORK HAS BEEN REMOVED.

TRAFFIC BARRIER SHALL NOT BE CAST UNTIL THE DECK AND INFERIOR PIER DIAPHRAGM CONCRETE COMPRESSIVE STRENGTH HAS REACHED 3000 PSI (MIN.) AND ALL FALSEWORK HAS BEEN REMOVED.

NOTE: NO LIVE LOAD SHALL BE ALLOWED ON THE SPANS UNTIL THE COMPRESSIVE STRENGTH OF THE TOP PORTION OF THE DIAPHRAGM HAS REACHED 3000 PSI (MIN.).
**GIRDER NOTES**

1. Plan length shall be increased as necessary to compensate for shortening due to prestressing and shrinkage.
2. All pretensioned and temporary strands shall be 0.6"ø low relaxation strands (AASHTO M203 Grade ST).
3. For end types A, C, and D cut all strands flush with the girder ends and Paint with an approved epoxy paint as shown. For end type B all strands shall be below concrete surface and group with an approved epoxy cement.
4. The top surface of the girder flange shall be roughened in accordance with Section 6-02.3(25) of the Standard Specifications.
5. Lifting embedments shall be installed in accordance with Section 6-02.3(25)L of the Standard Specifications.
6. Caution shall be exercised in handling and placing girders. All girders shall be checked by the contractor to ensure that they are braced adequately to prevent tipping and to control lateral bending during shipment. Once erected, all girders shall be braced laterally to prevent tipping until the diaphragms are cast and cured.
7. For temporary pad reorders shall be constructed and fastened in such a manner as to not cause damage to the order during the strand release operation.
8. Temporary top strands shall be either pretensioned or post-tensioned. The lifting location "C" and concrete release strength "F" are shown in the order schedule. Assume that the temporary top strands are pretensioned. Alternatively, post-tensioned temporary top strands may be used if the lifting points in the order schedule are maintained and the strands are stressed prior to lifting the order from the form.
9. For diaphragms, cut holes and place inserts on the interior face of exterior girders. Place holes and inserts parallel to skew. Inserts shall be 1"ø Meadow Burke MX-3 Hi-Tensile, 1"ø x 5½" Williams F22 open ferrule or approved equal.

**GIRDER ELEVATION**

- Extend straight strands (1) through (4) unless noted otherwise on strand extension detail. Order details 3.0 through 3.
- End bars ahead on station.
- End bars back on station.
- Order bars.
- Interchangeable bars.
- Splay to clear harped strands.
- B1 = 1½" (G4, G8)
- B2 = 3" (G5)
- G5 bars right of¢ girder.
- G7 harped strands bundled.
- G7, G9 and G10 bars may be used interchangeably as top flange ties.

**TYPICAL END ELEVATION**

- End type C shown, other end types similar.
- Field bending required to obtain 18" concrete cover at pavement seat.
- B1 = 1½" (G4, G8)
- B2 = 3" (G5)
- G5 bars right of¢ girder.
- G7 harped strands bundled.
- G7, G9 and G10 bars may be used interchangeably as top flange ties.

**INTERMEDIATE DIAPHRAGM**

- End type A
- End type B
- End type C
- End type D
- End type E
- End type F
- End type G
- End type H
- End type I
- End type J
- End type K
- End type L
- End type M
- End type N
- End type O
- End type P
- End type Q
- End type R
- End type S
- End type T
- End type U
- End type V
- End type W
- End type X
- End type Y
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- End typeAY
- End type AZ
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- End type BC
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- End type BJ
- End type BK
- End type BL
- End type BM
- End type BN
- End type BO
- End type BP
- End type BQ
- End type BR
- End type BS
- End type BT
- End type BU
- End type BV
- End type BW
- End type BX
- End type BY
- End type BZ

**DIAPHRAGM**

- Apply approved retardant for ¼" etch to side forms or ¼" roughened surface treatment by approved mechanical method.
GIRDER NOTES

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL PRETENSIONED AND TEMPORARY STRANDS SHALL BE 0.6"Ø LOW RELAXATION INSERTS ON THE INTERIOR FACE OF EXTERIOR GIRDER.

3. FIELD BENDING REQUIRED TO OBTAIN 1½" DIA. WORKING TIES, INSERTS SHALL BE 1/2" Ø MEADOWBURKE MX-3 HI-TENSILE, LANCASTER MALLEABLE, DAYTON-SUPERIOR F-62 FLARED FERRULE INSERT OR APPROVED EQUAL.

4. THE TOP SURFACE OF THE GIRDER FLANGE SHALL BE ROUNDED IN ACCORDANCE WITH SECTION 6.02.3(25) OF THE STANDARD SPECIFICATIONS.

5. LIFTING RAMPMENTS SHALL BE INSTALLED IN ACCORDANCE WITH SECTION 6.02.3(25) OF THE STANDARD SPECIFICATIONS.

6. CAUTION SHALL BE EXERCISED IN HANDLING AND PLACING GIRDERS. ALL GIRDERS SHALL BE CHECKED BY THE CONTRACTOR TO ENSURE THAT THEY ARE BRACED TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING. ONCE ERECTED, ALL GIRDERS SHALL BE BRACED LATERALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

7. FORM HANGERS SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE GIRDER DURING THE STRAND RELEASE OPERATION.

8. TEMPORARY TOP STRANDS SHALL BE EITHER PRETENSIONED OR POST-TENSIONED IN ACCORDANCE WITH SECTION 6.02.3(25) OF THE STANDARD SPECIFICATIONS AND THE ORDER DETAILS SHEETS. THE LIFTING LOCATION "C" AND CONCRETE RELEASE DIAPHRAM "CA" SHOWN IN THE ORDER SCHEDULE ARE SCHEDULED TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING. ONCE ERECTED, ALL GIRDERS SHALL BE BRACED LATERALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

9. FOR DIAPHRAGMS, OMIT HOLE PLATE INSERTS HANGERS SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE GIRDER DURING THE STRAND RELEASE OPERATION.

10. TEMPORARY TOP STRANDS MAY BE USED IF THE LIFTING POINTS IN THE ORDER SCHEDULE ARE SCHEDULED TO THE DIAPHRAGMS AND THE STRANDS ARE DEERED PRIOR TO LIFTING THE GIRDER FROM THE FORM.
**PLAN PRETENSIONED TEMPORARY TOP STRANDS**

Post-tensioned temporary top strands similar, except 10'-0" length of bonding occurs at one end only. The opposing end is anchored with plates and strand chuck. See "Girder Schedule" for number of temporary strands required.

1. **Alternate #1**
   - 5/8" (or 0.62) strand chuck, tack weld to anchor.
   - Prior to installing on strand, thread strand through anchor plate. Anchor strand with two piece wedges before order erection.
   - Verify wedges are seated tightly immediately before placing diaphragm concrete.

2. **Alternate #2**
   - Extend straight strands (3) through (6) at end ahead on station. Extend straight strands (7) through (10) at end back on station.

**END VIEW**

Temporary strands pretensioned.

**PLAN TEMPORARY STRAND POST-TENSIONED ALTERNATE**

Adjust bars to clear the steel plate.

**SECTION A**

Strawberry extension detail.

Number of extended strands shall be determined by the designer.
STRAND EXTENSION DETAIL

Number of extended strands shall be determined by the designer.
WF GIRDER DETAILS 1 OF 5

MARCH 2015

GIRDER SCHEDULE

<table>
<thead>
<tr>
<th>GIRDER SERIES</th>
<th>GIRDER TYPE</th>
<th>END 1 TYPE</th>
<th>END 2 TYPE</th>
<th>INT. DIAGRAM TYPE</th>
<th>PLAN LENGTH (ACCORDING TO GRADE)</th>
<th>MIN. CONC. COMP. STRENGTH (SEE GIRDER NOTE 2)</th>
<th>NUMBER OF STRANDS</th>
<th>LOCATION OF CG STRANDS</th>
<th>STRAIGHT STRANDS TO EXTEND</th>
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</table>

GIRDER NOTES

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.
2. ALL PRETENSIONED AND TEMPORARY STRANDS SHALL BE GFP AASHTO ME50 GRADE 270 LOW RELAXATION STRANDS, JACKED TO 202.5 KSI.

SCREED SETTING DIMENSIONS

FOR DIMENSION "C" SEE GIRDER SCHEDULE

NOTE TO DESIGNER:
PLACE DETAIL ON BRIDGE DECK REINFORCEMENT PLAN.
Prestressed Concrete Superstructure

GIRDER PLAN

SECTION A

SECTION C

GIRDER ELEVATION

<table>
<thead>
<tr>
<th>GIRDER SERIES</th>
<th>H</th>
<th>I</th>
<th>D</th>
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<td>WF50G</td>
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<td>1'-2&quot;</td>
<td>1'-2&quot;</td>
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<td>1'-0&quot;</td>
<td>1'-2&quot;</td>
<td>1'-2&quot;</td>
</tr>
</tbody>
</table>


details of ASHRAE STANDARD 6-02.3(25)

SYMMETRICAL ABOUT GIRDER C.G.

...
**WF Girder Details 3 of 5**

**Bearing Recess and Bottom Flange Spall Protection Detail**

**End Type A**
- Extend straight strands identified in order schedule.
- Bearing recess parallel to $\frac{3}{4}$ order.

**End Type B**
- Measured normal to $\frac{3}{4}$ diaphragm.
- **STRAND CHUCK TACK WELDED TO A36 1 1/2 x 4 x 0'-4 with $\frac{3}{4}$ hole prior to installing strands, ASME A36 3/4 x 1/2 steel strand anchor, or approved equal (TYP.). Verify strand grips are seated tightly immediately before placing diaphragm concrete. SECURELY TIE ANCHOR TO THE REBAR CAGE TO PREVENT DISPLACEMENT DURING CONCRETE PLACEMENT.**

**End Type D**
- Extend straight strands identified in order schedule.
- Extension length parallel to $\frac{3}{4}$ order.

**Flange Spall Protection Detail**
- Bearing recess forms shall be constructed and fastened to avoid girder damage during strand release.

**WF GIRDER DETAILS 3 OF 5**

---

**Washington State Department of Transportation**

**Bridges**

**PRESTRESSED CONCRETE GIRDERS**

**Details 3 of 5**

---

**WF Girder**

---

**BRIDGE DESIGN MANUAL**

**March 2015**

---

**Appendix A**

**Precompressed Concrete Superstructure**
GIRDER REINFORCEMENT NOTES

- Deformed welded wire reinforcement may be substituted for mild reinforcement in accordance with Standard Specification 6-02.3(25A).

<table>
<thead>
<tr>
<th>ORDER SERIES</th>
<th># OF BARS</th>
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<td>WF95G</td>
<td>11</td>
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<tr>
<td>WF100G</td>
<td>12</td>
</tr>
</tbody>
</table>

BENDING DIAGRAM

(ALL DIMENSIONS ARE OUT TO OUT)

- Varies for skewed ends.
- #3 or #4 may be substituted. Field bending is optional.
- Pairs of #4 bars, or #3 and #5 bars, may be used interchangeably as bottom flange ties.
- #1 bars may be substituted for 2 #3 bars within Zone 1.
TEMPORARY STRAND NOTES

1. TEMPORARY TOP STRANDS SHOULD BE EITHER PRETENSIONED OR POST-TENSIONED IN ACCORDANCE WITH STANDARD SPECIFICATION SECTION 6-02.3(25).L.

2. 2" x 6" x 2½" DEEP BLOCKOUT FOR STRAND DETENSIONING, FORM WITH EXPANDED POLYSTYRENE (TYP.)

3. 3'-0" MIN. OFFSET BETWEEN SYMMETRICAL BLOCKOUT PAIRS (TYP.)

4. TEMPORARY STRANDS IN PLASTIC SLEEVE (TYP.)

5. TEMPORARY STRAND IN PLASTIC SLEEVE AT LIVE END AND BONDED AT DEAD END (TYP.)

6. PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER (TYP.)

7. TEMPORARY STRAND NOTES

8. NOTES:

---

PRETENSIONED TEMPORARY TOP STRANDS ALTERNATE

SEE ORDER SCHEDULE FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

POST-TENSIONED TEMPORARY TOP STRANDS ALTERNATE

SEE ORDER SCHEDULE FOR NUMBER OF TEMPORARY STRANDS REQUIRED.
ADDITIONAL STRAND EXTENSION DETAIL

\[ n = \text{total number of additional extended strands} \]

* All additional strands in excess of 5 shall be placed in this group.

**ANCHOR PLATE DETAIL**

- Prestressing strands 2\(\frac{3}{4}\) in. SP-A (Typ. – not tensioned)
- 2\(\frac{3}{8}\) x 1\(\frac{1}{16}\) steel strand anchor
- Anchor strand with two-piece wedges after girder erection
- Verify wedges are seated tightly immediately before placing diaphragm concrete (Typ.)

**VIEW B**
GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED. IT MAY BE NECESSARY TO THREAD #7 REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDER. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.

1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. IT MAY BE NECESSARY TO THREAD #7 REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDER.
3. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE.
4. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.
NOTES:
1. ORDER SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. CUT/RELEASE ORDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM AND BRIDGE DECK. SEE "TEMPORARY STRAND CUTTING SEQUENCE".
3. STEEL DIAPHRAGM AND ORDER REINFORCING NOT SHOWN FOR CLARITY.
4. CONSTRUCTION DETAILS ARE NORMAL TO ORDER. FOR CONCRETE PLACEMENT SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.
5. ORDER PLACED PARALLEL TO FACE OF CROSSBEAM, FULL WIDTH OF BOTTOM FLANGE, TO REMAIN IN PLACE. HEIGHT SHALL NOT EXCEED WIDTH AT ORDER."
GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE. LONGITUDINAL DIMENSIONS ARE NORMAL TO SKEW. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.

NOTES:
1. 1'-6" 1½" MIN. 6" MAX. THREAD ANCHOR DETAIL ASTM A-307
2. SEE FRAMING PLAN 1'-0" 111 1'-6"
3. #4 TIE (TYP.) 1"ø BOLT (TYP.) FACE OF WEB 3" FILLET BETWEEN GIRDERS CONSTRUCTION JOINT WITH ROUGHENED SURFACE #4 BETWEEN GIRDERS (TYP.) 2 #4 STIRRUPS
4. SEE "ANCHOR DETAIL" THIS SHEET (TYP.) TOP OF GIRDER NOTE TO DESIGNER: Insert appropriate dimension value for "D" NOTE TO DESIGNER: Full depth intermediate diaphragms are required for:
- I-5 bridges
- Other bridges crossing over roads of ADT>50,000
5. NOTE TO DETAILER: Revise Details to show correct girder height.

NOTE TO DESIGNER:
Insert appropriate dimension increase for vertical curve effect if necessary.

NOTE TO DESIGNER:
Insert appropriate dimension for "D"

NOTE TO DESIGNER:
Full depth intermediate diaphragms are required for:
- I-5 bridges
- Other bridges crossing over roads of ADT>50,000

NOTES:
1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS.
3. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE.
4. CONSTRUCTION JOINT WITH ROUGHENED SURFACE.
5. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.
Full Depth Intermediate Diaphragm Details

Girders shall be held rigidly in place when diaphragms are placed. It may be necessary to thread reinforcing bars through holes in girders prior to placing exterior girders. Cut/release girder temporary strands before casting diaphragm. See temporary strand cutting sequence.

Notes:
1. Girders shall be held rigidly in place when diaphragms are placed.
2. It may be necessary to thread reinforcing bars through holes in girders prior to placing exterior girders.
3. Cut/release girder temporary strands before casting diaphragm. See temporary strand cutting sequence.
4. Longitudinal dimensions are normal to grade.
5. For concrete placement procedure see "Superstructure Construction Sequence" sheet.

Face of Web

Anchor Detail

Anchor A-400

NOTE TO DESIGNER:
Insert appropriate dimension. Increase for vertical curve effect if necessary.

NOTE TO DETAILER:
Avoid details so they do not conflict with girders.

Elevation

Full Depth Intermediate Diaphragm
Dimensions are along diaphragm.
I GIRDER BEARING DETAILS

GROUT PAD DETAIL

GIRDER NOT SHOWN FOR CLARITY

Skew angle shown at 30°.

For WF girders the edge of the bearing pad shall be set at 1" minimum to 9" maximum from the edge of the bottom flange.

For W girders, bulb tee and deck bulb tee girders the edge of the bearing pad shall be set at 1" from the edge of the bottom flange.

NOTES:

1. GIRDER STOPS SHALL BE CONSTRUCTED AFTER GIRDER PLACEMENT.

2. THE ELASTOMERIC GIRDER STOP PADS SHALL BE BONDED TO THE GIRDER STOP WITH AN APPROVED ADHESIVE.

SECTION A

SECTION B

ELASTOMERIC BEARING PAD

LOCATION: ELASTOMERIC BEARING PAD

Shear angles shown at 30°.

For WF girders the edge of the bearing pad shall be set at 1" minimum to 9" maximum from the edge of the bottom flange.

For W girders, bulb tee and deck bulb tee girders the edge of the bearing pad shall be set at 1" from the edge of the bottom flange.

ELASTOMERIC GIRDER STOP PAD

SHEAR MODULUS = 165 PSI

BEARING DESIGN TABLE

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Washington State Department of Transportation

Prestressed Concrete Superstructure

Bridge and Structures Office

H. A. G. 12

5-6-12
WF Thin Deck Girder

Details 1 of 3

**Bridging Design Manual**

MARCH 2015

---

**Girder Schedule**

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<th>Segment</th>
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<th>Ends 1 Type</th>
<th>Ends 2 Type</th>
<th>Int. Diaphragm</th>
<th>Type</th>
<th>Spacing</th>
<th>Width</th>
<th>Location of End 1 Type (C.G. Strands)</th>
<th>Location of End 2 Type (C.G. Strands)</th>
<th>Number of Strands</th>
<th>Straight Strands to Extend</th>
<th>D</th>
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**Notes to Designer:**

1. WF Thin Deck Girder Detail Sheets 2 to 5 are intended to be used as is without need for modification for most projects. Project specific girder details are then limited to the girder schedule. WF Thin Deck Girder Detail Sheet 5 may be omitted if temporary top strands are not used.

2. Zone 1 is intended to be the splitting resistance zone defined by BDM 5.6.2.F.

3. Zone 2 is intended to be the confinement reinforcement zone defined by BDM 5.6.2.G.

4. Dimensions in the girder schedule shall be shown to the nearest ¹/₁₆" except the "A" dimension which shall be shown to the nearest ¹/₄".

5. The number of harped strands should not exceed half the number of straight strands unless the straight strand pattern is full.

6. Minimum width "W" shall be 6'-0" to allow for inspection access. Maximum width "W" shall be 8'-0".

7. Provide a longitudinal #4 in cp deck inside extended hooks in addition to the deck reinforcement.

---

**Girder Notes**

1. Plan length shall be increased as necessary to compensate for shortening due to prestress and shrinkage.

2. All prestressed and temporary strands shall be ¹/₈" AASHTO M203 Grade 270 Low Relaxation strands, jacked to 202.5 KSI.

---

**Screed Setting Dimensions**

For dimension "C" see girder schedule.

---

**Washington State Department of Transportation**

Prestressed Concrete Superstructure

WF Thin Deck Girder

Details 1 of 3
WF Thin Deck Girder

DETAILS 2 OF 5

STRAIGHT STRAND LOCATION SEQUENCE
SHALL BE AS SHOWN

HARPED STRAND LOCATION SEQUENCE
SHALL BE AS SHOWN

4½" 2Ù"
1 3 5 7 9
2 4 6 8 10
11

C.G. TOTAL STRAIGHT STRANDS

C.G. TOTAL HARPED STRANDS

3" ø OPEN HOLE. ADJUST HOLE LOCATION VERTICALLY TO MISS HARPED STRANDS. OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR GIRDERS. PLACE HOLES AND INSERTS PARALLEL TO DIAPHRAGM CENTERLINE. INSERTS SHALL BE 1" ø MEADOWBURKE MX-3HI-TENSILE, 1" ø x 5½" WILLIAMS F22 OPEN FERRULE INSERT, 1" ø x 4½" DAYTON-SUPERIOR F-62 FLARED SLAB FERRULE INSERT OR APPROVED EQUAL.

TEXTURE TOP OF GIRDER IN ACCORDANCE WITH STANDARD SPECIFICATION 6-02.3(25)H

APPLY APPROVED RETARDANT FOR ¼" ETCH TO SIDE FORMS OR ¼" ROUGHENED SURFACE TREATMENT BY APPROVED MECHANICAL METHOD. Omit at exterior face of exterior girder.

INSTALL LIFTING EMBEDMENTS IN ACCORDANCE WITH STANDARD SPECIFICATION 6-02.3(25)L. REMOVE FROM TOP OF GIRDER AFTER ERECTION.

PROVIDE END TYPE DETAILS IDENTIFIED IN THE GIRDER SCHEDULE.

À OF INTERMEDIATE DIAPHRAGM, SEE STRAND PLAN FOR LOCATION.

É HOLE THROUGH TOP FLANGE, OMIT ON EXTERIOR SIDE OF EXTERIOR GIRDER, HOUST RIGHT, AS REQUIRED (TYP.)

GIRDER ELEVATION

GIRDER PLAN

SECTION A
HARPED STRAND LOCATION SEQUENCE SHALL BE AS SHOWN

C.G. TOTAL STRAIGHT STRANDS

C.G. TOTALHARPED STRANDS

C.G. TOTAL TEMPORARY STRANDS

APPLY APPROVED RETARDANT FOR ¼" ETCH TO SIDE FORMS OR ¼" ROUGHENED SURFACE TREATMENT BY APPROVED MECHANICAL METHOD, OMIT AT EXTERIOR FACE OF EXTERIOR GIRDERS.

MULTIPLE UNIT HOLD DOWN TO STRADDLE HARPING POINT

FOR W. ½" OPEN HOLE, ADJUST HOLE LOCATION VERTICALLY TO MISS HARPED STRANDS. OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR GIRDERS. PLACE HOLES AND INSERTS PARALLEL TO DIAPHRAGM CENTERLINE. INSERTS SHALL BE 1" ø MEADOWBURKE MX-3HI-TENSILE, 1" ø x 5½" WILLIAMS F22 OPEN FERRULE INSERT, 1" ø x 4½" DAYTON-SUPERIOR F-62 FLARED SLAB FERRULE INSERT OR APPROVED EQUAL.

TEXTURE TOP OF GIRDER IN ACCORDANCE WITH STANDARD SPECIFICATION 6-02.3(25)H

APPLY APPROVED RETARDANT FOR ¼" ETCH TO SIDE FORMS OR ¼" ROUGHENED SURFACE TREATMENT BY APPROVED MECHANICAL METHOD, OMIT AT EXTERIOR FACE OF EXTERIOR GIRDERS.

INSTALL LIFTING EMBEDMENTS IN ACCORDANCE WITH STANDARD SPECIFICATION 6-02.3(25)L. REMOVE FROM TOP OF GIRDER AFTER ERECTION.

PROVIDE END TYPE DETAILS IDENTIFIED IN THE GIRDER SCHEDULE.

À OF INTERMEDIATE DIAPHRAGM. SEE "FRAMING PLAN" FOR LOCATIONS.

6" ø HOLE THROUGH TOP FLANGE. OMIT ON EXTERIOR SIDE OF EXTERIOR GIRDER. ADJUST REINF. AS REQUIRED (TYP.)

¼" (TYP.)

1'-11" (TYP.)
WF Thin Deck Girder

Details 3 of 5

### Bearing Recess and Bottom Flange Spall Protection Detail

- **Bearing recess forms** shall be constructed and fastened to avoid girder damage during strand release.

#### END TYPE A

- Extend straight strands identified in order schedule parallel to % order.
- Bearing recess extends straight strands identified in order schedule.
- Sawteeth detail.

#### END TYPE B

- multiply normal to % diaphragm.

#### END TYPE D

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### Sawteeth Detail

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View E

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View F

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View G

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View H

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View I

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.

### View J

- Sawteeth shall be full width over area shown.
- % expanded polystyrene for skew angle greater than 10°.
WF THIN DECK GIRDER
Details 4 of 5

GIRDER REINFORCEMENT NOTES

1. Deformed welded wire reinforcement may be substituted for mild reinforcement in accordance with Standard Specification 6-02.3(25)A.

FIELD BENDING OF G1 AND G4 TO OBTAIN 1½" COVER AT PAVEMENT SEAT OR EXPANSION JOINT IF NECESSARY

CONT. #4 (2'-0" MIN. SPLICE) INSIDE BARRIER REINF. HOOK (TYP.)

END OF GIRDER

10'-0" 1'-10" (TYP.)

FIELD BEND ALTERNATIVE SIDES

R=2"

VARIES FOR SKEWED ENDS.

#3 OR #4 MAY BE SUBSTITUTED. FIELD BENDING IS OPTIONAL.

PAIRS OF G7 BARS, OR G9 AND G10 BARS, MAY BE USED INTERCHANGEABLY AS BOTTOM FLANGE TIES.

1 G2 MAY BE SUBSTITUTED FOR 2 G1 WITHIN ZONE 1.

WF66TDG SHOWN, OTHERS SIMILAR

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<thead>
<tr>
<th>GPR SERIES</th>
<th>NO. OF G5 BARS</th>
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<tr>
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<td>WF100TDG</td>
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STRAIGHT AND TEMPORARY STRANDS NOT SHOWN FOR CLARITY. WF66TDG SHOWN, OTHERS SIMILAR.
TEMPORARY STRAND NOTES

1. TEMPORARY STRANDS SHALL BE EITHER PRETENSIONED OR POST-TENSIONED IN ACCORDANCE WITH STANDARD SPECIFICATION SECTION 6-02.3(25).

2. 2" x 6" x 2½" DEEP BLOCKOUT FOR STRAND DETENSIONING. FORM WITH EXPANDED POLYSTYRENE (TYP).

3. 3'-0" MIN. OFFSET BETWEEN SYMMETRICAL BLOCKOUT PAIRS (TYP.)

4. TEMPORARY STRANDS IN PLASTIC SLEEVE (TYP.)

5. PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER (TYP.)

6. 10'-0" BOND TEMPORARY STRANDS (TYP.)

7. TEMPORARY STRANDS AT LIVE END AND BONDED AT DEAD END. (TYP.)

8. PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER. (TYP.)

9. 2" x 6" x 2½" DEEP BLOCKOUT FOR STRAND DETENSIONING. FORM WITH EXPANDED POLYSTYRENE. REMOVE POLYSTYRENE JUST PRIOR TO CUTTING THE TEMPORARY STRANDS AND PREVENT MOISTURE FROM ENTERING THE BLOCKOUT AS DESCRIBED IN THE TEMPORARY STRAND CUTTING SEQUENCE (TYP.).
/bridge design manual

wf thin deck girder
end diaphragm details

bridge and structures office

notes:
1. girders shall be held rigidly in place when diaphragms are placed.
2. cut/prepare girder temporary strands before casting diaphragm. see temporary strand cutting sequence.
3. extended strands and girder reinforcing not shown for clarity.
1. Girders shall be held rigidly in place when diaphragms are placed.
2. It may be necessary to thread #7 reinforcing bars through holes in girders prior to placing exterior girders.
3. Cut/release order temporary strands before casting diaphragm. See temporary strand cutting sequence.
4. For concrete placement procedure see construction sequence sheets.

---

**Deflection Joint Details**

**Anchor Details**

ASTM A-307

---

**Construction Joint with Roughened Surface**

Top of girder to match top edges of top flanges for full width of bridge.

---

**Bend in Field**

90° (Typ.)

---

**Edge Beam Soffit**

9" at § girder

---

**Face of Web**

1" Ø Bolt (Typ.)

---

**Top of Bridge Deck**

$3\overline{3}$ (Typ.)

---

**Minimum Splice**

2'-7" min. splice between girders when required.

---

**Spackle**

$3\overline{3}$ (Typ.)

---

**Edge Beam**

$3\overline{3}$ (Typ.)

---

**Note to Detailer:**

Revise Details to show correct girder shape and girder spacing.

---

**Note to Designer:**

Adjust girder stop height to provide 2" min. gap to bottom of diaphragm.

---

**Section A**

Longitudinal dimensions are normal to diaphragm. Girder stop not shown for clarity.

---

**Elevation**

End Diaphragm

Dimensions are along § diaphragm.

---

**Anchor Detail**

ASTM A-307

---

**Notes:**

1. Girders shall be held rigidly in place when diaphragms are placed.
2. It may be necessary to thread #7 reinforcing bars through holes in girders prior to placing exterior girders.
3. Cut/release order temporary strands before casting diaphragm. See temporary strand cutting sequence.
4. For concrete placement procedure see construction sequence sheets.
WF Thin Deck Girders Diaphragm at Intermediate Pier Details

ELEVATION AT TOP OF OAK BLOCK

NOTES:
1. Orders shall be held rigidly in place when diaphragms are placed.
2. Cutcrease order temporary strands before casting diaphragm and bridge deck.
3. Extended strands and order reinforcing not shown for clarity.
4. Longitudinal dimensions are normal to diaphragm.
5. For concrete placement procedure see "Superstructure Construction Sequence" sheet.

NOTE TO DESIGNER:
Actual dimensions and bar sizes shall be determined by the designer.

NOTE TO DETAILER:
Reinforce details to show actual dimensions.

OAK BLOCK DETAIL

Face of diaphragm and crossbeam

Bridge and Structures Office

Washington State Department of Transportation

Prestressed Concrete Girders

WF Thin Deck Girders Diaphragm at Intermediate Pier Details
Girders shall be held rigidly in place when diaphragms are placed. It may be necessary to thread reinforcing bars through holes in girders prior to placing exterior girders. Cut/release girder temporary strands before casting diaphragm. See temporary strand cutting sequence. Longitudinal dimensions are normal to ¢ diaphragm.

Dimensions are along diaphragm.

ELEVATION
PARTIAL DEPTH INTERMEDIATE DIAPHRAGM

Notes:
1. Girders shall be held rigidly in place when diaphragms are placed.
2. It may be necessary to thread reinforcing bars through holes in girders prior to placing exterior girders.
3. Cut/release girder temporary strands before casting diaphragm. See temporary strand cutting sequence.
4. Longitudinal dimensions are normal to ¢ diaphragm.
5. For concrete placement procedure see "Superstructure Construction Sequence" sheet.
6. See framing plan.

NOTE TO DESIGNER:
- Full depth intermediate diaphragms are required for:
  - I-5 bridges
  - Other bridges crossing over roads of ADT>50,000

NOTE TO DESIGNER:
- Insert appropriatedimension value for "D"
GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE. LONGITUDINAL DIMENSIONS ARE NORMAL TO SKEW. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.

NOTES:
1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS.
3. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE.
4. LONGITUDINAL DIMENSIONS ARE NORMAL TO SKEW.
5. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.

NOTE TO DETAILER:
- Full depth intermediate diaphragms are required for:
  - I-5 bridges
  - Other bridges crossing over roads of ADT>50,000

NOTE TO DESIGNER:
- See "Anchor Detail" this sheet (typ.)
WF Deck Girder Details 1 of 4

GIRDER SCHEDULE

<table>
<thead>
<tr>
<th>SERIAL</th>
<th>PLAN LENGTH (ALONG GIRDER GRADE)</th>
<th>LOCATION OF C.S. STRANDS</th>
<th>STRAIGHT STRANDS TO EXTEND</th>
<th>PLAN LENGTH (ALONG GIRDER GRADE)</th>
<th>LOCATION OF C.S. STRANDS</th>
<th>STRAIGHT STRANDS TO EXTEND</th>
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<th>LOCATION OF C.S. STRANDS</th>
<th>STRAIGHT STRANDS TO EXTEND</th>
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</table>

NOTES TO DESIGNER:

1. WF Deck Girder Detail Sheets 2 to 4 are intended to be used as is without need for modification for most projects. Project-specific girder details are then limited to the girder schedule.

2. Zone 1 is intended to be the splitting resistance zone defined by BDM 5.6.2.F. Zone 2 is intended to be the confinement reinforcement zone defined by BDM 5.6.2.G.

3. Dimensions in the Schedule shall be shown to the nearest "A" except the "A" dimension which shall be shown to the nearest "B".

4. The number of harped strands should not exceed half the number of straight strands unless the straight strand pattern is full.

5. Minimum width "W" shall be 5'-0" to allow for inspection access. Maximum width "W" shall be 8'-0".

6. Ensure harped strands exit girder end below blockout for end type A.

7. Girder ends skew is limited to 30°.

8. It is assumed that the final profile grade is provided by varying the overlay thickness. Instead, the designer could add a "girder flange thickening" detail to account for profile grade and prestressing camber effects.

GIRDER NOTES

1. Plan length shall be increased as necessary to compensate for shortening due to prestress and shrinkage.

2. All pretensioned and temporary strands shall be 0.62 AASHTO M203 Grade 270 low-relaxation strands, jacked to 202.5 KSI.

3. Structural steel shapes and connections shall be ASTM A and they shall be painted with a primer coat in accordance with Std. Spec. 6.07.5.B. Mild tiob shall be painted with a field primer coat of an organic zinc paint after field welding.

Washington State Department of Transportation

Prestressed Concrete Group, Bridge and Structures Office

STANDARD PRESTRESSED CONCRETE GIRDERS

WF Deck Girder Details 1 of 4
GIRDER REINFORCEMENT NOTES

1. Deformed welded wire reinforcement may be substituted for mild reinforcement in accordance with Standard Specification 6-02.3(26)A.

Note to Designer:
- Determine G3 spacing
- 10'-0" (TYP.)
- 1'-10" (TYP.)
- Field bending is optional.
- 1½" cover at pavement seat or expansion joint if necessary.
- Cont. #4 (2'-0" min. splice) inside barrier reinforcement. Hook (TYP.)
- Rotate hooks as req'd to achieve minimum cover (TYP.)
- Harped strands

WF DECK GIRDER DETAILS 4 OF 4
GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED. IT MAY BE NECESSARY TO THREAD #7 REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.

NOTES:
1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. IT MAY BE NECESSARY TO THREAD #7 REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS.
3. CUT/RELEASE ORDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE.
4. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.

NOTE TO DETAILER:
- Adjust order stop height to provide 2" MIN. GAP TO BOTTOM OF DIAPHRAGM.
- Adjust order stop height to provide 2" MIN. GAP TO BOTTOM OF DIAPHRAGM.
WF Deck Girder Diaphragm at Intermediate Pier Details

ELEVATION END DIAPHRAGM

DIMENSIONS ARE ALONG ¥ DIAPHRAGM
NOTE TO DETAILER:
Revise Details to show correct girder shape and girder spacing.

SECTION A

LONGITUDINAL DIMENSIONS ARE NORMAL TO DIAPHRAGM.
GIRDER STOP NOT SHOWN FOR CLARITY.

ELEVATION

NOTE TO DETAILER:
Revise Details to show correct girder shape and girder spacing.

NOTE TO DESIGNER:
Adjust Girder Stop Height to provide 2" Min. Gap to Bottom of Diaphragm.

BF DECK GIRDER DIAPHRAGM AT INTERMEDIATE PIER DETAILS

M:STANDARDS\Girders\WFDG\WFDG INTERMEDIATE PIER DIAPHRAGM.MAN

GIRDER STOP (TYP.)
#7 (TYP.)
#4 BETWEEN GIRDERS (TYP.)

NOTE:
1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. IT MAY BE NECESSARY TO THREAD ¥ REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS.
3. CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM. SEE TEMPORARY STRAND CUTTING SEQUENCE.
4. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.

ANCHOR DETAIL

FACE OF WEB
TYP. BOLT (TYP.)
5/8" MIN.
0" MIN (THREAD)
5/8" MIN.
3" MIN.

5.6-A6-7

BIDG.

STANDARDS
PRESTRESSED CONCRETE GIRDERS

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

BRIDGE AND STRUCTURES OFFICE

1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
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4. FOR CONCRETE PLACEMENT PROCEDURE SEE CONSTRUCTION SEQUENCE SHEETS.

ANCHOR DETAIL

FACE OF WEB
TYP. BOLT (TYP.)
5/8" MIN.
0" MIN (THREAD)
5/8" MIN.
3" MIN.

5.6-A6-7

BIDG.

STANDARDS
PRESTRESSED CONCRETE GIRDERS

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

BRIDGE AND STRUCTURES OFFICE

NOTES:
GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS. LONGITUDINAL DIMENSIONS ARE NORMAL TO SKEW. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.

NOTES:
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2. IT MAY BE NECESSARY TO THREAD REINFORCING BARS THROUGH HOLES IN GIRDERS PRIOR TO PLACING EXTERIOR GIRDERS.
3. LONGITUDINAL DIMENSIONS ARE NORMAL TO SKEW.
4. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.
DG GIRDER DETAIL SHEETS 1 TO 2 ARE INTENDED TO BE USED AS IS WITHOUT NEED FOR MODIFICATION FOR MOST PROJECTS. PROJECT SPECIFIC GIRDER DETAILS ARE THEN LIMITED TO THE GIRDER SCHEDULE.

V1 SPA. @ V2 IS INTENDED TO BE THE SPLITTING RESISTANCE ZONE DEFINED BY BDM 5.6.2.F.

V3 SPA. @ V4 IS INTENDED TO BE THE CONFINEMENT REINFORCEMENT ZONE DEFINED BY BDM 5.6.2.G.

GIRDER END SKEW IS LIMITED TO 30°.

DIMENSIONS IN THE GIRDER SCHEDULE SHALL BE SHOWN TO THE NEAREST \frac{1}{8} INCH.

THE NUMBER OF HARPED STRANDS SHOULD NOT EXCEED HALF THE NUMBER OF STRAIGHT STRANDS UNLESS THE STRAIGHT STRAND PATTERN IS FULL.

IT IS ASSUMED THAT THE FINAL PROFILE GRADE IS PROVIDED BY VARYING THE OVERLAY THICKNESS. INSTEAD, THE DESIGNER COULD ADD A "GIRDER FLANGE THICKENING" DETAIL TO ACCOUNT FOR PROFILE GRADE AND PRESTRESSING CAMBER EFFECTS.

THIS STANDARD IS BASED ON THE USE OF AN HMA OVERLAY. USE OF A 5" CIP CONCRETE DECK REQUIRES MODIFICATIONS.

NOTES TO DESIGNER:

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL PRESTRESSED STRANDS SHALL BE 0.6"Ø AASHTO M203 GRADE 270 LOW RELAXATION STRANDS, JACED TO 202.5 KSI.

3. CUT ALL STRANDS FLUSH WITH THE GIRDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN.

4. THE TOP SURFACE OF THE GIRDER FLANGE SHALL BE FINISHED IN ACCORDANCE WITH SECTION 6-02.3(25)H OF THE STANDARD SPECIFICATIONS.

5. LIFTING EMBEDMENTS SHALL BE INSTALLED IN ACCORDANCE WITH SECTION 6-02.3(25)L OF THE STANDARD SPECIFICATIONS. AFTER ERECTION, CUT OFF LIFTING EMBEDMENTS 1 INCH BELOW THE TOP OF THE FLANGE AND FILL WITH AN APPROVED GROUT.

6. CAUTION SHALL BE EXERCISED IN HANDLING AND PLACING GIRDERS. ALL GIRDERS SHALL BE CHECKED BY THE CONTRACTOR TO ENSURE THAT THEY ARE BRAZED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPMENT. ONCE ERECTED, ALL GIRDERS SHALL BE BRAZED UNEQUALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

7. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE GIRDER DURING THE STRAND RELEASE OPERATION.

8. STRUCTURAL STEEL SHAPES AND ASSEMBLIES SHALL BE ASTM A36. THEY SHALL BE PAINTED WITH A PRIMER COAT IN ACCORDANCE WITH STD. SPEC. 6-07.3(9). WELD TIES SHALL BE PAINTED WITH A FIELD PRIMER COAT OF AN ORGANIC ZINC PAINT AFTER FIELD WELDING.

9. FOR DIAPHRAGMS, MILD STEEL REINFORCEMENT CONFORMING TO SECTION 9-07.7 WITH DEFORMED WIRE CONFORMING TO SECTION 9-07.8 MAY BE SUBSTITUTED FOR MILD STEEL REINFORCEMENT IF AASHTO LRFD BRIDGE DESIGN SPECIFICATION REQUIREMENTS (INCLUDING DEVELOPMENT AND ANCHORAGE) ARE MET. WELDED WIRE REINFORCEMENT SHALL HAVE THE SAME AREA AND SPACING AS THE MILD STEEL REINFORCEMENT THAT IT REPLACES AND THE YIELD STRENGTH SHALL BE GREATER THAN OR EQUAL TO 60 KSI.

SHEAR STIRRUP LONGITUDINAL WIRES AND TACK WELDS SHALL BE EXCLUDED FROM GIRDER WEBS. LONGITUDINAL WIRES FOR ANCHORAGE OF WELDED WIRE REINFORCEMENT SHALL HAVE AN AREA OF 40% OR MORE OF THE AREA OF THE WIRE BEING ANCHORED BUT SHALL NOT BE LESS THAN 4A. 

1. DO ORDER DETAIL SHEETS 1 TO 2 ARE INTENDED TO BE USED AS IS WITHOUT NEED FOR MODIFICATION FOR MOST PROJECTS. PROJECT SPECIFIC GIRDER DETAILS ARE THEN LIMITED TO THE GIRDER SCHEDULE.

2. V1 SPA. @ V2 IS INTENDED TO BE THE SPLITTING RESISTANCE ZONE DEFINED BY BDM 5.6.2.F.

3. V3 SPA. @ V4 IS INTENDED TO BE THE CONFINEMENT REINFORCEMENT ZONE DEFINED BY BDM 5.6.2.G.

4. GIRDER END SKEW IS LIMITED TO 30°.

5. DIMENSIONS IN THE GIRDER SCHEDULE SHALL BE SHOWN TO THE NEAREST \frac{1}{8} INCH.

6. THE NUMBER OF HARPED STRANDS SHOULD NOT EXCEED HALF THE NUMBER OF STRAIGHT STRANDS UNLESS THE STRAIGHT STRAND PATTERN IS FULL.

7. IT IS ASSUMED THAT THE FINAL PROFILE GRADE IS PROVIDED BY VARYING THE OVERLAY THICKNESS. INSTEAD, THE DESIGNER COULD ADD A "GIRDER FLANGE THICKENING" DETAIL TO ACCOUNT FOR PROFILE GRADE AND PRESTRESSING CAMBER EFFECTS.

8. THIS STANDARD IS BASED ON THE USE OF AN HMA OVERLAY. USE OF A 5" CIP CONCRETE DECK REQUIRES MODIFICATIONS.
### GIRDER SCHEDULE

<table>
<thead>
<tr>
<th></th>
<th>GIRDER HEIGHT H</th>
<th>GIRDER WIDTH W</th>
<th>#22 Voids</th>
<th>#22 End Type A</th>
<th>#22 End Type B</th>
<th>#22 Voids End 2 Type A</th>
<th>#22 Voids End 2 Type B</th>
<th>MIN. CONC. COMP. STRENGTH</th>
<th>TRANVERSE REINFORCEMENT</th>
<th>LONGITUDINAL REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
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</tbody>
</table>

### GIRDER NOTES

1. **Plan length** shall be increased as necessary to compensate for shortening due to prestress and shrinkage.
2. All strands shall be 
   - 4/8 in. Atlanta Mega Grade 70 low relaxation strands, jacketed to 
   - G20 No. 6 strands shall be symmetrical about the centerline. Externally stranded strands in each row shall be fully bonded.
3. Space extended strands symmetrically and evenly across order width. Strands located with respect to girder in adjacent spans.
4. Extended strands shall be extended at each girder end for the indicated (length parallel to the centerline). Extended strands shall not be extended past girder ends. Extended strands shall be symmetrically placed about the order centerline. Extended lengths of pairs of strands that are symmetrically positioned about the girder centerline shall be equal.
5. Structural steel shapes and assemblies shall be ASTM A36 unless noted otherwise. They shall be painted with a primer coat in accordance with STD SPEC 607.391. Welds shall be painted with a field primer coat of an organic zinc paint after field welding. Stainless steel shapes and assemblies shall not be painted.

---

**NOTE TO DESIGNER:**

1. Slab girder detail sheets 2 to 4 are intended to be used as is without need for modification for most projects. Project-specific girder details are then limited to the girder schedule.
2. Zone 1 is intended to be the splitting resistance zone defined by BDM 5.6.2.F. Zone 2 is intended to be the confinement reinforcement zone defined by BDM 5.6.2.G.
3. Dimensions in the girder schedule shall be shown to the nearest ¥" except the "A" dimension which shall be shown to the nearest ¥".
4. Provide a longitudinal ¥"A in CIP deck inside extended and voids in addition to the deck reinforcement.
5. Maximum slab angle is 30° (60° ≤ φ ≤ 120°).
6. This standard is intended to be used with a 5" minimum CIP concrete deck. Modifications are required if this standard is used with an HMA overlay.
7. Place a minimum of 2 fully bonded strands in both Row 1 and the top row (all 4 corners).
8. ¥"A and ¥"B should be a minimum of ¥"A at 1'-0" spacing and ¥"B are not necessary at end Type B.

---

**TYPICAL SLAB GIRDER CONFIGURATIONS**

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<tr>
<th>H</th>
<th>W</th>
<th>MAX. SPAN LENGTH</th>
<th>¥&quot;A</th>
<th>¥&quot;B</th>
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<td>30'</td>
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<tr>
<td>16</td>
<td>4'-0&quot;</td>
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<td>60'</td>
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<td>15.7'</td>
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<td>4'-0&quot;</td>
<td>2'</td>
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**SCREED SETTING DIMENSIONS**

For dimension C- See Girder Schedule.
PLAN

Interior girders shown, exterior girders similar.

Provide end type details identified in the drawing schedule.

Install lifting embeddings in accordance with Std. Spec. 6-02.3(25), remove to top of girders after erection.

TEXTURE TOP OF GIRDER IN ACCORDANCE WITH Std. SPEC. 6-02.3(25).

END 1

PLAN LENGTH (ALONG GIRDER)

END 2

PROVIDE END TYPE DETAILS IDENTIFIED IN THE DRAWING SCHEDULE (TYP.).

INSTALL LIFTING EMBEDMENTS IN ACCORDANCE WITH STD. SPEC. 6-02.3(25), REMOVE TO TOP OF GIRDER AFTER ERECTION (TYP.).

TEXTURE TOP OF GIRDER IN ACCORDANCE WITH Std. SPEC. 6-02.3(25).
TEMPORARY STRANDS
SEE GIRDER SCHEDULE FOR REQUIRED
NUMBER OF TEMPORARY STRANDS.
TEMPORARY STRANDS SHALL BE
PLACED IN THE TOP ROW.

TEMPORARY STRANDS IN PLASTIC SLEEVE
¢ GIRDER
10'-0"
BOND TEMPORARY
STRANDS
10'-0"
BOND TEMPORARY
STRANDS
3'-0" MIN. OFFSET BETWEEN
SYMMETRICAL BLOCKOUT PAIRS (TYP.)

EXTEND STRAIGHT
STRANDS IDENTIFIED
IN GIRDER SCHEDULE

CUT ALL STRANDS FLUSH
WITH THE GIRDER END
AND PAINT WITH AN
APPROVED EPOXY RESIN,
EXCEPT FOR EXTENDED
STRANDS AS SHOWN.

CUT ALL STRANDS FLUSH
WITH THE GIRDER END
AND PAINT WITH AN
APPROVED EPOXY RESIN,
EXCEPT FOR EXTENDED
STRANDS AS SHOWN.

CUT ALL STRANDS 1" BELOW
CONCRETE SURFACE AND GROUT
WITH AN APPROVED EPOXY GROUT

REBAR CHUCK TACK WELDED TO
ASTM A36
¢ x 4 x 0'-4

ASTM A108
2¾"ø x 1" STEEL STRAND
ANCHOR, OR APPROVED
EQUAL (TYP.).

VERIFY STRAND GRIPS ARE SEATED TIGHTLY
IMMEDIATELY BEFORE PLACING DIAPHRAGM CONCRETE.
SECURELY TIE ANCHOR TO THE REBAR CAGE TO PREVENT
DISPLACEMENT DURING CONCRETE PLACEMENT.

SAWTEETH ARE FULL WIDTH
SAWTEETH DETAIL

1" 1"

END OF GIRDER

90°

2" x 6" x 2½" DEEP BLOCKOUT FOR
STRAND DETENSIONING. FORM WITH
EXPANDED POLYSTYRENE. REMOVE
POLYSTYRENE JUST PRIOR TO CUTTING
THE TEMPORARY STRANDS AND PREVENT
MOISTURE FROM ENTERING THE BLOCKOUT
AS DESCRIBED IN THE TEMPORARY
STRAND CUTTING SEQUENCE (TYP.).

2" x 6" x 28" DEEP BLOCKOUT FOR
STRAND DETENSIONING. FORM WITH
EXPANDED POLYSTYRENE. REMOVE
POLYSTYRENE JUST PRIOR TO CUTTING
THE TEMPORARY STRANDS AND PREVENT
MOISTURE FROM ENTERING THE BLOCKOUT
AS DESCRIBED IN THE TEMPORARY
STRAND CUTTING SEQUENCE (TYP.).
GIRDER REINFORCEMENT NOTES

1. Deformed welded wire reinforcement may be substituted for mild reinforcement in accordance with Standard Specification 6-02.3(25)A.

TRANSVERSE REINFORCEMENT OPTIONS

2. CLR. (TYP.) may be reduced to 1" CLR. for sloped edges at crown locations.

3. Zone 1, Zone 2, and Zone 3 transverse reinforcement spaced at (H - 3") or 1'-6" whichever is smaller.

4. Continuous #4 at bend in traffic barrier reinforcement (2'-0" min. lap)

FIELD BEND G1 TO OBTAIN 1½" COVER AT PAVEMENT SEAT IF NECESSARY. DO NOT EXTEND AND PROVIDE 1½" CLR. TO GIRDER END FOR END TYPE B.

SEE TRAFFIC BARRIER SHEETS FOR DETAILS AND LOCATIONS.

PARALLEL TO END

G4 #4
SPACED AT 1'-0" MAX.

G5 AND G6 SHOWN, 2 G7 AND 2 G8 SIMILAR.

STRANDS NOT SHOWN.

SEE TRAFFIC BARRIER SHEETS FOR DETAILS AND LOCATIONS.

OTHER END SIMILAR.

10'-0" (TYP.)

2'-0" (TYP.)

SPLAY AS SHOWN

PLAN

TRANSVERSE REINFORCEMENT OPTIONS

TRAFFIC BARRIER BARS NOT SHOWN FOR CLARITY.

SEE TRAFFIC BARRIER SHEETS FOR DETAILS AND LOCATIONS.

OTHER END SIMILAR STRANDS NOT SHOWN.

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

BRIDGE AND STRUCTURES OFFICE

STANDARD
PRESTRESSED CONCRETE GIRDER

SLAB GIRDER
DETAILS 4 OF 4
**NOTE TO DESIGNER:**
Actual dimensions and bar sizes to be determined by the Designer.

**NOTE TO DETAILER:**
Revise Details to show actual Geometry.

**CONSTRUCTION JOINT WITH ROUGHENED SURFACE OR SHEAR KEY**
See GIRDER SCHEDULE.

**REINF. FOR CONC. ROADWAY SLAB**

**LEVEL PERPENDICULAR TO CROSSBEAM**

**CONSTRUCTION JOINT WITH ROUGHENED SURFACE**
See "OAK BLOCK DETAIL" THIS SHEET (TYP.)

**CROSSBEAM**

**SLAB GIRDER (TYP.)**

**SEE "OAK BLOCK DETAIL" THIS SHEET (TYP.)**

**OAK BLOCK PLACED PARALLEL TO FACE OF CROSSBEAM, FULL WIDTH OF GIRDER. REMOVE AFTER PLACING TRAFFIC BARRIER.**

**NOTES:**
1. **CUT/RELEASE GIRDER TEMPORARY STRANDS BEFORE CASTING DIAPHRAGM AND BRIDGE DECK. SEE TEMPORARY STRAND CUTTING SEQUENCE.**
2. **EXTENDED STRANDS AND GIRDER REINFORCING NOT SHOWN FOR CLARITY.**
3. **LONGITUDINAL DIMENSIONS ARE NORMAL TO ¥ PIER. FOR CONCRETE PLACEMENT PROCEDURE SEE "SUPERSTRUCTURE CONSTRUCTION SEQUENCE" SHEET.**

---

**APPENDIX A**

**PRESTRESSED CONCRETE SUPERSTRUCTURE**

**BRIDGE DESIGN MANUAL**

**SLAB GIRDER**

**FIXED DIAPHRAGM**

**WASHINGTON STATE DEPARTMENT OF TRANSPORTATION**

**STANDARD PRESTRESSED CONCRETE ORDERS**

**SLAB GIRDER FIXED DIAPHRAGM**
### GIRDER SCHEDULE

<table>
<thead>
<tr>
<th>GIRDER</th>
<th>GIRDER SERIES</th>
<th>END 1 TYPE</th>
<th>END 2 TYPE</th>
<th>&quot;A&quot; DIMENSION AT PLAN LENGTH</th>
<th>V1 SPA. @</th>
<th>V2 SPA. @</th>
<th>V3 SPA. @</th>
<th>V4 SPA. @</th>
<th>L</th>
<th>Ld</th>
<th>LL</th>
<th>2P1 «1 «2</th>
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### GIRDER NOTES

1. **Plan Length shall be increased as necessary to compensate for shortening due to prestress and shrinkage.**
2. **All pretensioned and temporary strands shall be 0.6"ø AASHTO M203 Grade 270 low relaxation strands, jacked to 202.5 ksf.**
3. **For end types A, C, and D cut all strands flush with the order ends and paint with an approved epoxy resin except for extended strands as shown. For end type B cut all strands 1" below concrete surface and grout with an approved epoxy grout.**
4. **The top surface of the order flange shall be roughened in accordance with Section 6-02.3(25) of the standard specifications.**
5. **Lifting embedments shall be installed in accordance with Section 6-02.3(25) of the standard specifications.**
6. **Caution shall be exercised in handling and placing girders.** All girders shall be checked by the contract to ensure that they are braced adequately to prevent tipping and to control lateral bending during shipping. Once erected, all girders shall be braced laterally to prevent tipping until the diaphragms are cast and cured.
7. **Forms for beams and receivers shall be constructed and fastened in such a manner as to not cause damage to the girder during the strand release operation.**
8. **Temporary top strands shall be either pretensioned or post-tensioned in accordance with Section 6-02.6.1 of the standard specifications and the girder details sheet.**
9. **Temporary top strands are pretensioned. Alternatively, post-tensioned temporary top strands may be used if the lifting points in the order schedule are maintained and the strands are stressed prior to lifting the girder from the form.**
10. **Temporary strands shall be either pretensioned or post-tensioned in accordance with Section 6-02.6.1 of the standard specifications and the girder details sheet.**

### Notes to Designer

1. Tub girder detail sheets 1 to 3 are intended to be used as is without need for modification for most projects. Project specific girder details are then limited to the girder schedule. Tub girder detail sheet 3 may be omitted if temporary top strands are not used.
2. V1 SPA. @ V2 is intended to be the splitting resistance zone defined by BDM 5.6.2.
3. V3 SPA. @ V4 is intended to be the confinement reinforcement zone defined by BDM 5.6.2.
4. Dimensions in the order schedule shall be shown to the nearest 1/8th inch.
5. The number of harped strands should not exceed half the number of straight strands unless the straight strand pattern is full.
6. Temporary top strands require top flanges.
7. Delete unused rows in the order series table.
TUB GIRDER DETAILS 2 OF 3

BEARING RECESS AND BOTTOM FLANGE SPALL PROTECTION DETAIL

SAWTOOTH DETAILS
SAWTOOTH ARE FULL WIDTH - USE SAWTOOTH KEYS FROM BOTTOM OF BOTTOM FLANGE TO BOTTOM OF LOWEST HARPED STRAND AS WELL AS TOP FLANGE ADJACENT TO HARPED STRANDS.

BOTTOM SLAB DRAIN HOLE DETAIL
3"Ø CIRCULAR DRAIN HOLE. SEE FRAMING PLAN FOR LOCATIONS.

STRAND EXTENSION DETAIL
ANCHOR STRAND WITH TWO PIECE WEDGES BEFORE GIRDER ERECTION. VERIFY WEDGES ARE SEATED TIGHTLY IMMEDIATELY BEFORE PLACING DIAPHRAGM CONCRETE.

GIRDER SCHEDULE LEGEND
I. AND II. ARE SHIPPIG SUPPORT LOCATIONS AT LEADING AND TRAILING ENDS, RESPECTIVELY.

TRANVERSE REINFORCING
SKewed ENDs
LENGTH OF AND EARY WITH SKEW. ONLY TRANSVERSE REINFORCEMENT SHOWN.

PLAN LENGTH ALONG GIRDER GRADE

BEARING RECESS (TYP.)

END 1 END 2
GIRDER SCHEDULE LEGEND
I. AND II. ARE SHIPPIG SUPPORT LOCATIONS AT LEADING AND TRAILING ENDS, RESPECTIVELY.

STRAND PATTERN
AT GIRDER END
HARPED STRAND LOCATION SEQUENCE SHALL BE AS SHOWN etc.

C.G. TOTAL HARPED STRANDS

3½" DRAIN HOLE WITH T X T NO. 6 STEEL WIRE SCREEN
NO. 6 METALLIC PIPE
C.G. TOTAL STRAIGHT STRANDS

STRAND PATTERN
AT 6' SPAN
STRAIGHT STRAND LOCATION SEQUENCE SHALL BE AS SHOWN etc.
A FOOT WIDE BOTTOM FLANGE SHOWN OTHERS SIMILAR.

BEARING RECESS AND BOTTOM FLANGE SPALL PROTECTION DETAIL

BEARING RECESS
BEARING WIDTH + 2" - SEE BEARING DETAILS SHEET

3½" Ø CIRCULAR DRAIN HOLE.

GIRDER SLOPE
½" EXPANDED POLYSTYRENE UNDER FLANGE GREATER THAN 15°

1½" CHAMFER ON WEB FOR SKEWS GREATER THAN 15°

3½" EXPANDED POLYSTYRENE UNDER FLANGE GREATER THAN 15°

LEVEL (AFTER CASTING DECK)

BEARING RECESS (TYP.)

BOTTOM OF GIRDER
4" (MIN.)

3½" Ø HOLE PRIOR TO INSTALLING ON STRAND OR 2½" x 1½" STEEL STRAND ANCHOR (TYP.)

SYMMETRICAL ABOUT Ø GIRDER C.G. TOTAL HARPED STRANDS
HARPED STRAND BUNDLES (TYP.)

GIRDER END

BEARING RECESS
BEARING WIDTH + 2" - SEE BEARING DETAILS SHEET

90° (TYP.)

END OF P.S. GIRDER
90°

BEARING RECESS
BEARING WIDTH + 2" - SEE BEARING DETAILS SHEET

1½" CHAMFER ON WEB FOR SKEWS GREATER THAN 15°

3½" EXPANDED POLYSTYRENE UNDER FLANGE GREATER THAN 15°

LEVEL (AFTER CASTING DECK)

ORDER SLOPE

3½" ORDER SLOPE
SECTION A

PRETENSIONED TEMPORARY
TOP STRANDS ALTERNATE

SEE "GIRDER SCHEDULE" FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

PLAN

POST-TENSIONED TEMPORARY
TOP STRANDS ALTERNATE

SEE "GIRDER SCHEDULE" FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

DETAIL 1

TEMPORARY STRAND LOCATION SEQUENCE SHALL BE AS SHOWN 1, 2, ETC.

BOND TEMPORARY STRANDS

3'-0" DEAD END

4" normal to % ORDER

PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER.

G4 #5 (TYP.)

TEMPORARY STRAND IN PLASTIC SLEEVE AT LIVE END AND BONDED AT DEAD END.

PLAN

PRETENSIONED TEMPORARY
TOP STRANDS ALTERNATE

SEE "GIRDER SCHEDULE" FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

SECTION A

TEMPORARY STRAND LOCATION SEQUENCE SHALL BE AS SHOWN 1, 2, ETC.

BOND TEMPORARY STRANDS

3'-0" DEAD END

4" normal to % ORDER

PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER.

G4 #5 (TYP.)

TEMPORARY STRAND IN PLASTIC SLEEVE AT LIVE END AND BONDED AT DEAD END.

PLAN

POST-TENSIONED TEMPORARY
TOP STRANDS ALTERNATE

SEE "GIRDER SCHEDULE" FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

DETAIL 1

TEMPORARY STRAND LOCATION SEQUENCE SHALL BE AS SHOWN 1, 2, ETC.

BOND TEMPORARY STRANDS

3'-0" DEAD END

4" normal to % ORDER

PT ANCHOR PLATE TO BE DETERMINED BY GIRDER MANUFACTURER.

G4 #5 (TYP.)

TEMPORARY STRAND IN PLASTIC SLEEVE AT LIVE END AND BONDED AT DEAD END.
NOTE TO DESIGNER

If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.

1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. REINFORCING BAR SHALL BE THREADED THROUGH HOLES IN GIRDERS PRIOR TO PLACING OF EXTERIOR GIRDERS. SEE PLANS FOR "TRAFFIC BARRIER DIMENSIONS AND LOCATION." SEE "GIRDER DETAILS" SHEET FOR DIMENSION "A."

NOTE:

3. END DIAPHRAGM MAY BE CAST ON GRADE. IF SO, THE UPPER LEG OF THE JOINT FILLER SHALL FORM THE BOTTOM FACE FULL WIDTH.
4. JOINT FILLER TYPE 1 SHALL BE USED TO COVER ALL VERTICAL END DIAPHRAGM JOINTS. EITHER JOINT FILLER TYPE 1 OR JOINT FILLER TYPE 2 SHALL BE USED TO COVER ALL HORIZONTAL END DIAPHRAGM JOINTS.

TYPICAL END TYPE "A" DIAPHRAGM AT END PIERS

ELEVATION

PLAN VIEW

END DIAPHRAGM GEOMETRY

SECTION F

BUTYL RUBBER AT DIAPHRAGM

BUTYL RUBBER AT VERTICAL JOINTS

DIAPHRAGM

GIRDER SEAT & RECESS - LEVEL

BEARING PAD

TOP OF ORDER

2" FILLET

1'-3" BUTYL RUBBER AT DIAPHRAGM

4'-2"

½" RECESS

3" UNDER DIAPHRAGM

9" THICK DIAMONDRRubber Sheet

BOND WITH ADHESIVE THIS SURFACE ONLY

1'-3" STIRRUP @ 1'-3"

#4 (TYP.)

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

2" CHAMFER

SEE JOINT FILLER DETAIL

END DIAPHRAGM GEOMETRY

SECTIONS THROUGH END DIAPHRAGMS AT END PIERS

SEE "GIRDER DETAILS" SHEET FOR DIMENSION "A.

ALL LONGITUDINAL DIMENSIONS ARE NORMAL TO SKIN.
BRIDGE DESIGN MANUAL

MARCH 2015

End Diaphragm on Girder Details

TYPICAL END TYPE "A" DIAPHRAGM AT END PIERS

ELEVATION

PLAN VIEW

BYUTYL RUBBER AT DIAPHRAGM

BYUTYL RUBBER AT VERTICAL JOINTS

NOTE TO DESIGNER

If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.

# NOTE TO DESIGNER

1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.

2. REINFORCING BAR SHALL BE THREADED THROUGH HOLES IN GIRDERS PRIOR TO PLACING OF EXTERIOR GIRDERS. SEE PLANS FOR "TRAFFIC BARRIER" DIMENSIONS AND LOCATION. SEE "GIRDER DETAILS" SHEET FOR DIMENSION "A".

3. END DIAPHRAGM MAY BE CAST ON GRADE. IF SO, THE OUTER LEG OF THE JOINT FILLER SHALL FORM THE BOTTOM FACE FULL WIDTH.

4. JOINT FILLER TYPE 1 SHALL BE USED TO COVER ALL VERTICAL END DIAPHRAGM JOINTS. OTHER JOINT FILLER TYPE 1 OR JOINT FILLER TYPE 2 SHALL BE USED TO COVER ALL HORIZONTAL END DIAPHRAGM JOINTS.

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# NOTE TO DESIGNER

If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.

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If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.

# NOTE TO DESIGNER

If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.
TUB GIRDER BEARING DETAILS

GROUT PAD DETAIL

1/2" RECESS AT ¢ BEARING

¾" RECESS

¾" CHAMFER (TYP.)

¢ BEARING ALONG PIER

GROUT PAD DETAIL

½" GAP BETWEEN ELASTOMERIC STOP PAD AND GIRDER

9/16" THICK ELASTOMERIC STOP PAD

¢ BEARING ALONG PIER

GROUT PAD ELEVATION

¾" CHAMFER (TYP.)

¢ BEARING

SKEW ANGLE

LEVEL

ELASTOMERIC BEARING PAD

CONSTRUCTION JOINT WITH ROUGHENED SURFACE

1¼" MIN. 1½" MAX. (TYP.)

ELASTOMERIC STOP PAD

GROUT PAD ELEVATION

9/16" GAP BETWEEN ELASTOMERIC STOP PAD AND GIRDER

1/2" CHAMFER (TYP.)

Elastomeric bearing pad shall be set at 1" from the edge of the girder.

Notes:
1. Girder stops shall be constructed after girder placement.
2. The elastomeric stop pads shall be cemented to girder stops with approved adhesive.

Elastomeric Stop Pad

Shear modulus = 165 psi

Section A

Section B

BEARING DESIGN TABLE

AASHTO METHOD B DESIGN

SERVICE - UNIT STATE

DEAD LOAD (25% REACTION) kips

LIVE LOAD REACTION (150% IMPACT) kips

UNLOADED HEIGHT in

LOADED HEIGHT (DL) in

SHEAR MODULUS kips/in

BEARING DESIGN TABLE

Washington State Department of Transportation

PRESTRESSED CONCRETE GIRDERS

TUB GIRDER BEARING DETAILS

Appendix A

Bridge Design Manual

March 2015

Pre-Stressed Concrete Superstructure
NOTES:

1. PRETENSIONING STRANDS, LEVELING BOLTS AND GROUT FOR GROUT PAD UNDER SIP DECK PANELS SHALL BE AS SPECIFIED IN THE SPECIAL PROVISIONS.

2. LOOSEN THE LEVELING BOLT BY TWO TURNS AFTER THE GROUT HAS REACHED THE DESIGN STRENGTH SPECIFIED IN SECTION 9-20.3(2). LEVELING BOLT SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH AASHTO M232.

3. FOR SKewed END PANELS, ADJUST THE LEVELING BOLT LOCATIONS LONGITUDINALLY ACROSS THE E OF GIRDER SUCH THAT EACH PANEL WILL HAVE 4 BOLTS AFTER THE PANEL IS SAWCUT. THE PANEL MAY BE CAST SQUARE AND SAWCUT TO FIT THE PLAN DECK.

4. THE CONTRACTOR MAY SUBMIT FOR APPROVAL ALTERNATE LIFT POINT LOCATIONS, LIFTING EMBEDMENTS AND DEVICES IN ACCORDANCE WITH SECTION 6-02.3(28)G. LIFT POINT LOCATIONS AND LIFTING EMBEDMENTS AND DEVICES SHALL BE SHOWN ON THE SHOP PLANS SUBMITTED FOR APPROVAL. DESIGN CALCULATIONS SHALL BE SUBMITTED WITH THE SHOP PLANS.

5. THE CONTRACTOR MAY SUBMIT AN ALTERNATE METHOD FOR FORMING GROUT PAD UNDER SIP DECK PANELS AT EXTERIOR FACE OF GIRDER FLANGE REFER ALSO TO SPECIAL PROVISIONS.
The minimum compressive strength of the cast-in-place concrete at the closure at the time of post-tensioning shall be as shown in the post-tensioning table. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations. The stressing sequence shall meet the following criteria:

A. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the bridge.

B. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent webs. At no time during stressing operation will more than 1/6 of the total prestressing force be applied eccentrically about the centerline of the bridge.

The design is based on ½" or 0.6" low relaxation strands with an anchor set of ½", a curvature friction coefficient, $\mu = 0.20$ and a working friction coefficient, $\mu = 0.05$. The actual anchor set and jacking force used by the contractor shall be specified in the shop plans and included in the transfer force calculations.

4. The design is based on the estimated prestress loss of post-tensioning strands shown in the post-tensioning table due to steel relaxation, elastic shortening creep and shrinkage of concrete.

The maximum outside diameter of the duct shall be ¾" inches. The area of the duct shall be at least 2.5 times the net area of the prestressing steel in the duct.

5. All tendons shall be stressed from one end.

6. Temporary strands shall be post-tensioned in accordance with Section 6-02.3(25)(D) of the Standard Specifications. Temporary strands may be post-tensioned on the same day the pretensioning is released into the girder.
PRE-TENSIONING NOTES

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL PRETENSIONED AND TEMPORARY STRANDS SHALL BE [½"Ø OR 0.6"Ø] LOW RELAXATION STRANDS (AASHTO M203 GRADE 270.)

3. FOR END TYPES A, C, D AND E CUT ALL STRANDS FLUSH WITH THE GIRDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN. FOR END TYPE B CUT ALL STRANDS 1" BELOW CONCRETE SURFACE AND GROUT WITH AN APPROVED EPOXY GROUT.

4. THE TOP SURFACE OF THE ORDER PLANE SHALL BE ROUGHENED IN ACCORDANCE WITH SECTION 6-02.3(25)H OF THE STANDARD SPECIFICATIONS.

5. LIFTING EMBEDMENTS SHALL BE INSTALLED IN ACCORDANCE WITH SECTION 6-02.3(25)L OF THE STANDARD SPECIFICATIONS. CONTRACTOR TO DESIGN OTHER LIFTING MECHANISM IF THE GIRDER SECTION WEIGHT EXCEEDS 200 KIPs.

6. CAUTION SHALL BE EXERCISED IN HANDLING AND PLACING GIRDERS. ALL GIRDERS SHALL BE CHECKED BY THE CONTRACTOR TO ENSURE THAT THEY ARE BRACED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING. ONCE ERECTED, ALL GIRDERS SHALL BE BRACED LATERALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

7. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE ORDER DURING THE STRAND RELEASE OPERATION.
PT ANCHOR PLATE TO BE INSTALLED PERPENDICULAR TO TOP OF GIRDER.

2 ~ #5 - TOP AND BOTTOM OF STUDS

PLASTIC DUCTS FOR TEMPORARY STRANDS (TYP.)

½"ø x 6" STUDS (TYP.)

2" x 2" x 2½" DEEP EXPANDED POLYSTYRENE FILLED BLOCKOUT (TYP.)

2" Ø STRAND CHUCK. TACK WELD TO ANCHOR PRIOR TO INSTALLING ON STRAND. THREAD STRAND THROUGH ANCHOR PRIOR TO GIRDER ERECTION. VERIFY WEDGES ARE SEATED TIGHTLY IMMEDIATELY BEFORE PLACING DIAPHRAGM CONCRETE.

EXTEND STRAIGHT STRANDS (1) THROUGH (8) AT END AHEAD ON STATION. EXTEND STRAIGHT STRANDS (9) THROUGH (16) AT END BACK ON STATION.

2¾"ø x 1" STEEL STRAND ANCHOR. ANCHOR STRAND WITH TWO PIECE WEDGES BEFORE GIRDER ERECTION. VERIFY WEDGES ARE SEATED TIGHTLY IMMEDIATELY BEFORE PLACING DIAPHRAGM CONCRETE.

STEEL ANCHOR ¡ ½ x 4 x 0'-4 WITH ¾"ø HOLE

¾"ø STRAND CHUCK. FIELD BEND ALT. SIDES

LEVEL (AFTER CASTING SLAB)

END VIEW

BOTTOM FLANGE

PLAN VIEW OF TEMPORARY STRANDS

SEE "POST-TENSIONING TABLE" FOR NUMBER OF TEMPORARY STRANDS REQUIRED.

FOR END TYPES "C" AND "E" ONLY

END AHEAD ON STATION

END BACK ON STATION

GIRDER END STIRRUPS

GIRDER TOP FLANGE

GIRDER LONGIT. FULL LENGTH

GIRDER END TIES

GIRDER BOT. FLANGE TIES

GIRDER END LONGIT. STR.

GIRDER TOP LONGIT. STR.

GIRDER STIRRUPS

NOTE: FOR DIMENSION "A", SEE "GIRDER SCHEDULE"
Girder Elevation
Mid-Section

1. Mark holes and place inserts on the interior face of exterior girders. Place holes and inserts parallel to skew. Inserts shall be 1" Ø Burke Hi-Tensile, Lancaster Malleable, Dayton-Superior F-62 Flared Thin Slab (1" x 4") Ferrule Insert or Approved Equal (Typ.).

2. Lift lifting bars, HD threaded bars with anchor plates and nuts at bottom, position to avoid PT ducts.

3. Extends straight strands (9) through (16).

4. Adjust hole location vertically to avoid PT ducts. (Typ.)
**GIRDER PLAN**

**STRAND PATTERN**

- **PRECAST END SEGMENTS**
  - STRAIGHT STRAND LOCATION SEQUENCE SHALL BE AS SHOWN: (1), (2), (3), ETC.
  - - INDICATES DEBONDED STRAND.

- **PRECAST MID-SEGMENTS**
  - STRAIGHT STRAND LOCATION SEQUENCE SHALL BE AS SHOWN: (1), (2), (3), ETC.
  - - INDICATES DEBONDED STRAND.

**SAWTEETH DETAIL**

SAWTEETH ARE FULL WIDTH - USE SAWTOOTH KEYS IN AREA OUTSIDE OF PT. DUCTS AS SHOWN IN VIEWS B AND E - GIRDER DETAILS 2 OF 5

**GIRDER SCHEDULE**

<table>
<thead>
<tr>
<th>SPAN</th>
<th>MINIMUM CONCRETE COMPR. STRENGTH</th>
<th>C.G. TOTAL STRAIGHT STRANDS</th>
<th>C.G. TOTAL STRAIGHT STRANDS</th>
<th>NO. OF STRANDS</th>
<th>JACKING FORCE (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2P1</td>
<td>END SEGMENT 1</td>
<td>END SEGMENT 2</td>
<td>END SEGMENT 1</td>
<td>END SEGMENT 2</td>
<td>END SEGMENT 1</td>
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**"A" DIMENSION TABLE**

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<tr>
<th>PIPE</th>
<th>&quot;A&quot; (40 DAYS) (IN)</th>
<th>&quot;A&quot; (120 DAYS) (IN)</th>
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**NOTE:**

Dimensions shall be shown in Imperial units to the nearest Â\(\frac{1}{8}\) inch.
PRE-TENSIONING NOTES

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL PRETENSIONED AND TEMPORARY STRANDS SHALL BE [½" Ø OR 0.6" Ø] LOW RELAXATION STRANDS (AASHTO M203 GRADE 270.)

3. FOR END TYPES A, C, D AND E CUT ALL STRANDS FLUSH WITH THE ORDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN. FOR END TYPE B CUT ALL STRANDS 1" BELOW ConCRETE SURFACE AND GROUT WITH AN APPROVED EPOXY GROUT.

4. THE TOP SURFACE OF THE ORDER FLANGE SHALL BE ROUNDED IN ACCORDANCE WITH SECTION G-02(2)(B) OF THE STANDARD SPECIFICATIONS. CONTRACTOR TO DESIGN OTHER LIFTING MECHANISM IF THE ORDER WEIGHT EXCEEDS 200 KIPS.

5. LIFTING EMBEDMENTS SHALL BE INSTALLED IN ACCORDANCE WITH SECTION G-02(2)(B) OF THE STANDARD_SPECIFICATIONS. CONTRACTOR TO DESIGN OTHER LIFTING MECHANISM IF THE ORDER SECTION WEIGHT EXCEEDS 200 KIPS.

6. CAUTION SHALL BE EXERCISED IN HANDLING AND PLACING ORDERS. ALL ORDERS SHALL BE CHECKED BY THE CONTRACTOR TO ENSURE THAT THEY ARE BRACED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING. ONCE ERECTED, ALL ORDERS SHALL BE BRACED LATERALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

7. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE GIRDER DURING THE STRAND RELEASE OPERATION.
GIRDER ELEVATION
MID-SEGMENT

- Onset, holes and place inserts on the interior face of exterior girders. Place holes and inserts parallel to skew. Inserts shall be Tutone 310, Lancaster Malleable, Dayton Superior F-62 Flared thin slab (1" x 4") ferrule insert or approved equal (TYP.)

INTERMEDIATE DIAPHRAGM

- Position lifting bars to avoid P.T. ducts.

- Adjust hole locations vertically to avoid P.T. strands (TYP.)

- 5.9-A2-4

- 3" @ 1'-6"

- 10 @ 6" = 5'-0"

- 6 @ 3"

- 10 @ 6" = 5'-0"

- SAWTEETH

- EXTEND STRAIGHT STRANDS (9) THRU (16)

- C.G. TOTAL STRAIGHT STRANDS

- LIFTING BARS - H.S. THREADED BARS WITH ANCHOR PLATES AND NUTS AT BOTTOM. POSITION TO AVOID P.T. DUCTS.

- 90° & 10°
**POST-TENSIONING TABLE**

<table>
<thead>
<tr>
<th>SPAN</th>
<th>ORDER</th>
<th>D.O.C. CONCRETE COMPRESSIVE STRENGTH (KSI)</th>
<th>NUMBER OF STRANDS</th>
<th>PRESTRESSING LOAD (KIPS)</th>
<th>AFTER SEATING</th>
<th>TOTAL PRESTRESS LOSS (KSI)</th>
<th>E₁ (IN.)</th>
<th>E₂ (IN.)</th>
<th>E₃ (IN.)</th>
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</table>

**POST-TENSIONING NOTES**

1. The minimum compressive strength of the cast-in-place concrete at the closure at the time of post-tensioning shall be as shown in Post-Tensioning Table.

2. The maximum outside diameter of the duct shall be 3/4 inch. The area of the duct shall be at least 0.5 times the net area of the post-tensioning steel in the duct.

3. The design is based on [½"Ø or 0.6"Ø] low relaxation strands with an anchor set of [½"Ø], a curvature friction coefficient, μ = 0.25 and a mobilization coefficient, k = 0.0002/FT. The actual anchor set and jacking force used by the contractor shall be specified in the shop plans and included in the transfer force calculations.

4. The design is based on the estimated prestress loss of post-tensioning strands shown in the Post-Tensioning Table due to steel relaxation, elastic shortening creep and shrinkage of concrete.

5. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations. The stressing sequence shall meet the following criteria:

   a. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the bridge.

   b. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent webs. At no time during stressing operations must more than 1/6 of the total prestressing force be applied eccentrically about the centerline of the bridge.

6. All tendons shall be stressed from one end.

7. Temporary strands shall be post-tensioned in accordance with Section 6-02.3(25) of the Standard Specifications. Temporary strands may be post-tensioned on the same day the pretensioning is released into the girder.
PRE-TENSIONING NOTES

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL PRETENSIONED AND TEMPORARY STRANDS SHALL BE [½" OR 0.6"] LOW RELAXATION STRANDS (AASHTO M203 GRADE 270).

3. FOR END TYPES A, C, D AND E CUT ALL STRANDS FLUSH WITH THE GIRDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN. FOR END TYPE B CUT ALL STRANDS 1" BELOW CONCRETE SURFACE AND GROUT WITH AN APPROVED EPOXY GROUT.

4. THE TOP SURFACE OF THE GIRDER FLANGES SHALL BE ROUGHENED IN ACCORDANCE WITH SECTION 6-02.2H OF THE STANDARD SPECIFICATIONS.

5. LIFTING EMBEDMENTS SHALL BE INSTALLED IN ACCORDANCE WITH SECTION 6-02.3(25) OF THE STANDARD SPECIFICATIONS. CONTRACTOR TO DESIGN OTHER LIFTING MECHANISM IF THE GIRDER SECTION WEIGHT EXCEEDS 250 KIPS.

6. CAUTION SHALL BE EXERCISED IN HANDLING AND PLACING GIRDERS. ALL GIRDERS SHALL BE CHECKED BY THE CONTRACTOR TO ENSURE THAT THEY ARE BRACED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING. ONCE ERECTED, ALL GIRDERS SHALL BE BRACED LATERALLY TO PREVENT TIPPING UNTIL THE DIAPHRAGMS ARE CAST AND CURED.

7. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS TO NOT CAUSE DAMAGE TO THE GIRDER DURING THE STRAND RELEASE OPERATION.
LIFTING BARS - HD. THREADED BARS WITH ANCHOR PLATES AND NUTS AT BOTTOM POSITION TO AVOID P.T. DUCTS.

INTER. DIAPHR. PICKUP FORCE

GIRDER & INTERMEDIATE DIAPHRAGM

INTERMEDIATE DIAPHRAGM PICKUP FORCE

8" X 9" X 7" SHEAR KEYS (OMIT AT EXTERIOR FACE OF EXTERIOR GIRDERS)

3"Ø OPEN HOLE. ADJUST HOE LOCATION VERTICALLY TO MISS P.T. STRANDS, (TYP.)

* OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR GIRDERS. PLACE HOLES AND INSERTS PARALLEL TO SKEW. INSERTS SHALL BE 3/8" BURKE HI-TENSILE, LANCASTER MALLEABLE, DAYTON-SUPERIOR F-62 FLARED THRU SHEAR (1" X 4") FERRULE INSERT OR APPROVED EQUAL, (TYP.)

INSERTS ON THE INTERIOR FACE OF EXTERIOR GIRDERS (TYP.)

GIRDER ELEVATION MID-SEGMENT

EXTEND STRAIGHT STRANDS (9) THROUGH (16), (10 SPANS @ 6" = 5'-0"

10 SPANS @ 6" = 5'-0"

10 SPANS @ 6" = 5'-0"

11" SPA. @ 3" = 1'-0"

10 SPA. @ 6" = 5'-0"

6 SPA. @ 3" = 1'-0"

EXTEND STRAIGHT STRANDS (9) THROUGH (16).

Y (TYP.)

Z (TYP.)

3" SPA. @ 1'-0"

2 G5 #5 - ADJUST LOCATION TO CLEAR P.T. DUCTS (TYP.)

2 G2 #5, G3 #5 & 2 G7 #3

6 SPA. @ 3" = 1'-0"

10 SPA. @ 6" = 5'-0"

3"6 SPA. @ 3" = 1'-6"

5" SPA. @ 3" = 1'-6"

10 SPA. @ 6" = 5'-0"

3" SPA. @ 1'-0"

GIRDERS ELEVATION MID-SEGMENT

MID-SEGMENT REINFORCING

ELEVATION MID-SEGMENT REINFORCING

INTERCEPTOR

3"6 SPA. @ 3" = 1'-6"

10 SPA. @ 6" = 5'-0"

6 SPA. @ 3" = 1'-0"

-10 SPANS @ 6" = 5'-0"

6 SPA. @ 3" = 1'-0"

10 SPA. @ 6" = 5'-0"

2 G7 #3 - ADJUST LOCATION TO CLEAR P.T. DUCTS (TYP.)

2 G5 #5 - ADJUST LOCATION TO CLEAR P.T. DUCTS (TYP.)

2 G2 #5, G3 #5 & 2 G7 #3

6 SPA. @ 3" = 1'-0"

10 SPA. @ 6" = 5'-0"

3" SPA. @ 1'-0"
**Notation:**
1. Girder stops shall be constructed after girder placement.
2. The elastomeric stop pads shall be cemented to girder stops with approved adhesive.

---

**Bearing Pad Details**

**Grout Pad Detail**

- ½" recess at Φ bearing
- ¾" chamfer (Typ.)
- ½" thick elastomeric stop pad
- ¾" gap between elastomeric stop pad and girder stop
- Construction joint with roughened surface
- Girder stop
- Elastomeric bearing pad
- Grout pad
- 1½" at Φ bearing
- 1¼" min. 1½" max. (Typ.)

**Girder Stop Pad**

- ¼" outer layer (Typ.)
- ½" inner layer (Typ.)
- ½" for height ≤ 5" ¾" for height > 5"
- 1½" at Φ bearing
- Grout pad elevation
- Elastomeric bearing pad
- Laminated elastomeric bearing pad (shims)
- 90° skew angle
- The edge of the bearing pad shall be set at 1" from the edge of the girder.
Appendix A

BRIDGE DESIGN MANUAL

March 2015

Precast Concrete Superstructure

Tub Spliced Girder

Details 1 of 5

**1.** THE CAST-IN-PLACE CONCRETE IN DECK SLAB SHALL BE CLASS 4000D. THE MINIMUM COMPRESSIVE STRENGTH OF THE CAST-IN-PLACE CONCRETE AT THE WET JOINT AT THE TIME OF POST-TENSIONING SHALL BE ??? ksi.

**2.** THE MINIMUM PRESTRESSING LOAD AFTER SEATING AND THE MINIMUM NUMBER OF PRESTRESSING STRANDS FOR EACH GIRDER SHALL BE AS SHOWN IN POST-TENSIONING TABLE.

**3.** THE DESIGN IS BASED ON ???? INCH DIAMETER LOW RELAXATION STRANDS WITH A JACKING LOAD FOR EACH GIRDER AS SHOWN IN POST-TENSIONING TABLE, AN ANCHOR SET OF ???? INCH OF CURVATURE FRICTION COEFFICIENT, \( \varepsilon = 0.20 \) AND A WOBBLE FRICTION COEFFICIENT, \( k = 0.0002/\text{ft} \). THE ACTUAL ANCHOR SET USED BY THE CONTRACTOR SHALL BE SPECIFIED IN THE SHOP PLANS AND INCLUDED IN THE TRANSFER FORCE CALCULATIONS.

**4.** THE DESIGN IS BASED ON THE ESTIMATED PRESTRESS LOSS OF POST-TENSIONING STRANDS SHOWN IN POST-TENSIONING TABLE DUE TO STEEL RELAXATION, ELASTIC SHORTENING CREEP AND SHRINKAGE OF CONCRETE.

**5.** THE CONTRACTOR SHALL SUBMIT THE STRESSING SEQUENCE AND ELONGATION CALCULATIONS TO THE ENGINEER FOR APPROVAL. ALL LOSSES DUE TO TENDON VERTICAL AND HORIZONTAL CURVATURE MUST BE INCLUDED IN ELONGATION CALCULATIONS.

A. THE PRESTRESSING FORCE SHALL BE DISTRIBUTED WITH AN APPROXIMATELY EQUAL AMOUNT IN EACH WEB AND SHALL BE PLACED SYMMETRICALLY ABOUT THE CENTERLINE OF THE TUB.

B. NO MORE THAN ONE-HALF OF THE PRESTRESSING FORCE IN ANY WEB MAY BE STRESSED BEFORE AN EQUAL FORCE IS STRESSED IN THE ADJACENT WEBS.

AT NO TIME DURING STRESSING OPERATION WILL MORE THAN ONE-SIXTH OF THE TOTAL PRESTRESSING FORCE BE APPLIED ECCENTRICALLY ABOUT THE CENTERLINE OF BRIDGE.

**6.** THE MAXIMUM OUTSIDE DIAMETER OF THE DUCT SHALL BE ??? INCHES. THE AREA OF THE DUCT SHALL BE AT LEAST 2.5 TIMES THE NET AREA OF THE PRESTRESSING STEEL IN THE DUCT.

**7.** ALL TENDONS SHALL BE STRESSED FROM PIER ??.

**8.** SIDE FORMS FROM INSIDE & OUTSIDE OF THE CLOSURES & CROSSBEAM SHALL BE REMOVED PRIOR TO POST-TENSIONING.

---

**Post-Tensioning Notes**

1. The strand pattern at ? span varies.

2. The strand location detail shows the tendon in sag curve.

3. The duct splice detail includes extended strands (1) - (8).

4. The strand pattern at 6 span is shown.

---

**Post-Tensioning Table**

<table>
<thead>
<tr>
<th>Span</th>
<th>Girder</th>
<th>Diameter</th>
<th>In.</th>
<th>Jacking Load</th>
<th>After Seating</th>
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<tbody>
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<td>4</td>
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</table>
PRE-TENSIONING NOTES:

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL STRANDS FOR PRETENSIONING SHALL BE 1/8 INCH DIAMETER LOW RELAXATION STRANDS (AASHTO M203, GRADE 270).

3. FOR END TYPES A, C AND D, CUT ALL STRANDS FLUSH WITH THE GIRDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN. FOR END TYPES B AND E CUT ALL STRANDS 1" BELOW CONCRETE SURFACE AND GROUT WITH AN APPROVED EPOXY GROUT.

4. EXTENDED STRANDS AND BARS ARE PARALLEL TO GIRDER.

5. LIFTING BARS SHALL BE 1/2 INCH DIAMETER HIGH STRENGTH THREADED BARS (AASHTO M275, GRADE 150 MINIMUM). LIFTING HARDWARE THAT CONNECTS TO THREADED BARS SHALL BE VERTICAL ONLY AND WITHIN 10 DEGREES OF PERPENDICULAR TO A LINE BETWEEN PICK POINTS. CONTRACTOR SHALL SUBMIT CALCULATIONS FOR APPROVAL BY THE ENGINEER IF LIFTING FORCES ARE TO BE OTHERWISE.

6. EXTRA CAUTION MUST BE EXERCISED IN HANDLING AND PLACING ALL GIRDERS. ALL GIRDERS SHALL BE CHECKED TO ENSURE THAT THEY ARE BRACED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING.

7. THE TOP SURFACE OF THE GIRDER FLANGE SHALL BE ROUGHENED IN ACCORDANCE WITH SECTION 6-02.3(25)H.

8. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS NOT TO CAUSE DAMAGE TO THE GIRDER DURING STRAND RELEASE OPERATION.
TUB SPLICED GIRDER

DETAILS 3 OF 5

 Fri Mar 06 10:15:41 2015

M:\STANDARDS\Girders\PT Trapezoidal Tubs\PT_TRAPEZOIDAL TUB 3.MAN

2½" EXPANDED POLYSTYRENE BLOCK

TRANSVERSE REINFORCING

AT SKEWED ENDS

WEB TIE LONGITUDINAL

TOP WEB LONGITUDINAL

LONGITUDINAL BOTTOM FLANGE

STIRRUPS, INSIDE WEB

STIRRUPS, OUTSIDE WEB

WEB TIE LONGITUDINAL

¾" CHAMFER ON WEB FOR SKEWS GREATER THAN 15°

EXPANDED POLYSTYRENE FOR SKEWS EQUAL OR LESS THAN 15°

TOP WEB PLT.

BOTTOM WEB PLT.

WEB PLT.

WEB

1½" ø H.S. THREADED LIFTING BAR W/ANCHOR @ BOTTOM POSITION TO AVOID STRANDS (TYP.)

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

SAWTEETH DETAILS

SAWTEETH (SEE SAWTOOTH DETAILS THIS SHEET)

SAWTEETH ARE FULL WIDTH - USE SAWTEETH FULL HEIGHT ON END SECTIONS.

SAWTEETH DETAILS

SAWTEETH ARE FULL WIDTH - USE SAWTEETH FULL HEIGHT ON END SECTIONS.

TOP WEB PLT.

BOTTOM WEB PLT.

WEB PLT.

WEB

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

BOTTOM OF TUB SPALL PROTECTION

END TYPE "C" INTERMEDIATE HINGE DIAPHRAGM AND END TYPE "D" INTERMEDIATE FIXED DIAPHRAGM

TOP WEB PLT.

BOTTOM WEB PLT.

WEB PLT.

WEB

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

SYMM. ABOUT ½ TRAPEZOIDAL TUB SPLICED GIRDER

BOTTOM OF TUB SPALL PROTECTION

END TYPE "C" INTERMEDIATE HINGE DIAPHRAGM AND END TYPE "D" INTERMEDIATE FIXED DIAPHRAGM
TUB SPLICED GIRDER DETAILS 4 OF 5

Fri Mar 06 10:19:29 2015

M:\STANDARDS\Girders\PT Trapezoidal Tubs\PT_TRAPEZOIDAL TUB 4.MAN

SAWTEETH AREA OUTSIDE OF PT. DUCTS.
EXTEND STRAIGHT STRANDS (9) THROUGH (16).

EXTEND  G4 #6

1½" CLR.
6 SPA. @ 3" = 1'-6"
10 SPA. @ 6" = 5'-0"

9" SPA. @ 1'-6"

G7 #4 & G1 #5, G2 #5

1½" CLR.
6 SPA. @ 3" = 1'-6"
10 SPA. @ 6" = 5'-0"

2 7" CLR

G5 #5 - ADJUST LOCATION TO CLEAR P.T. DUCTS (TYP.)

INTERMEDIATE DIAPHRAGM

LIFTING BARS 2 - 1½" & NO. THREAD BARS W/ANCHOR NUT AT BOTTOM

PICKUP FORCE

GIRDER ELEVATION - MID-SEGMENT

* Omit holes and place inserts on the interior face of exterior web, place holes and inserts parallel to skew. Inserts shall be 1" Rocket/Holker, Hi-Tensile, Lancaster Malleable or approved equal.

PICKUP FORCE

GIRDER ELEVATION ~ MID-SEGMENT

¢ INTERMEDIATE DIAPHRAGM
¢ GIRDER & INTERMEDIATE DIAPHRAGM
¢ PICKUP FORCE

4½" 1'-1"

2'-7½" ¾" x 4½" x 7" SHEAR KEYS (OMIT EXTERIOR FACE OF EXTERIOR GIRDER)

* 3" OPEN HOLE

B=90° ± 10°

1'-9"

C.G. TOTAL STRAIGHT STRANDS

LIFTING BARS 2 ~ 1½" ø H.S. THREADBARS W/ANCHOR NUT AT BOTTOM

* OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR WEB. PLACE HOLES AND INSERTS PARALLEL TO SKEW. INSERTS SHALL BE 1" ROCKETS/HOLKER, Hi-TENSILE, LANCASTER MALLEABLE OR APPROVED EQUAL.

TYPICAL MID-SEGMENT ELEVATION
NOTES TO DESIGNER:

1. This strand extension detail is to be used for continuous spans at moment resisting diaphragms only. The detail is not applicable for continuous spans using hinge diaphragms.

2. Designer shall calculate the exact number of extended strands needed to develop the required moment capacity at the end of the girder. This calculation shall be based on the tensile strength of the strands, the stresses imposed on the anchor, and concrete bearing against the projected area of the anchor.

3. The total number of extended strands shall not be less than:

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   where:
   - \( M_c \): The lesser of Elastic or Plastic hinging moment of the column
   - \( V_c \): The shear capacity of the column
   - \( h \): Distance from top of column to c.g. of superstructure
   - \( N_c \): Number of columns
   - \( N_{ps} \): Number of extended strands
   - \( k \): Stress imposed on the anchor
   - \( A_{ps} \): Area of each extended strand
   - \( f_{ps} \): Ultimate strength of strands, ksi
   - \( d \): Distance from top of slab to c.g. of extended strand
   - \( M \): Moment due to SIDL (Traffic barrier, sidewalk, etc.)

   The total number of extended strands shall not be less than:

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \( k \) for \( L_1 = L_2 = 0.5 \)

   \( k \) for \( L_1 = 2L_2 = 0.67 \)

   \[
   M = \frac{ Ng }{ A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
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   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
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   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
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   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
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   \]

   \[
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   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
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   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
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   k = \frac{Ng}{A_{ps} f_{ps} d}
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   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
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   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
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   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
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   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
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   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
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   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   M = \frac{Ng}{A_{ps} f_{ps} d}
   \]

   \[
   N_{ps} = \frac{12(M_c + V_c X h - M)}{N_c X k}
   \]

   \[
   k = \frac{Ng}{A_{ps} f_{ps} d}
   \]
# NOTE TO DESIGNER

1. GIRDERS SHALL BE HELD RIGIDLY IN PLACE WHEN DIAPHRAGMS ARE PLACED.
2. REINFORCING BAR SHALL BE THREADED THROUGH HOLES IN GIRDERS PRIOR TO PLACING OF EXTERIOR GIRDERS. SEE PLANS FOR "TRAFFIC BARRIER" DIMENSIONS AND LOCATION. SEE "GIRDER DETAILS" SHEET FOR DIMENSION "A".
3. END DIAPHRAGM MAY BE CAST ON GRADE. IF SO, THE UPPER LEG OF THE JOINT FILLER SHALL FORM THE BOTTOM FACE FULL WIDTH.
4. JOINT FILLER TYPE 1 SHALL BE USED TO COVER ALL VERTICAL END DIAPHRAGM JOINTS. EITHER JOINT FILLER TYPE 1 OR JOINT FILLER TYPE 2 SHALL BE USED TO COVER ALL HORIZONTAL END DIAPHRAGM JOINTS.

NOTE: SEE "BUTYL RUBBER @ DIAPHRAGM DETAIL"
**BRIDGE DESIGN MANUAL**

**Prestressed Concrete Superstructure**

**MARCH 2015**

**Tub Spliced Girder Raised Crossbeam Details**

**Plan**
- Skewed
- No Skew

**Elevation**
- Temporary Support Detail
- Construction Joint with Roughened Surface

**Construction Sequence**
1. Column & Temp. Support
2. Place Girder on Temporary Support
3. Cast Diaphragm Stage 1
4. Cast Roadway Slab
5. Complete Diaphragm
6. Remove Temporary Support

**Detail C**
- Top of Bridge Deck
- Top of Crossbeam Reinforcing
- Temporary Support Detail This Sheet
- Elastomeric Bearing Pad

**Section A**
- Construction Joint with Roughened Surface
- End of Trapezoidal Tub Girder

**Section B**
- Cast Bridge Deck Reinforcing
- 3" Gap in Spiral Continuity for Placement of Crossbeam Longitudinal Reinforcement

**Section**
- Top of Crossbeam Reinforcing
- End of Precast Trapezoidal Tub Girder

**Eisen**
- Straight Strands
- Humped Strands

**Dimensions**
- 1½" Min.
- 2'-6" Min.
- 6" Min.

**Reinforcement**
- H1 #5 @ 1'-0" Max.
- H2 #5 (Typ.)
- H3 #5
- H4 #5
- H5 #5 (Typ.)

**Concrete**
- Class 4000
- Class 4000D
- 45° Skew

**Materials**
- Prestressed Trapezoidal Tub Girder
- Bridge Deck Reinforcing
- Substructure
- Superstructure

**Notes**
- 3" Gap in Spiral Continuity for Placement of Crossbeam Longitudinal Reinforcement
- Varies
- **Temp. Support (Typ.)**
- **Cast Roadway Slab**
- **Remove Temporary Support**

**Washington State Department of Transportation**

**Bridge and Structures Office**

**Appendix A**

**Sheet 5.9-A-6**
**Post-Tensioning Table**

<table>
<thead>
<tr>
<th>Span</th>
<th>Girder Diameter In.</th>
<th>Strand Diameter In.</th>
<th>Jacketing After Seating</th>
<th>Total Prestressing Loss ksi (DT+ES+FR+AS)</th>
<th>$E_1$ (kn)</th>
<th>$E_2$ (kn)</th>
<th>$E_3$ (kn)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Post-Tensioning Notes**

1. The cast-in-place concrete in deck slab shall be Class 4000D. The minimum compressive strength of the cast-in-place concrete at the wet joint at the time of post-tensioning shall be 37 ksi.

2. The minimum prestressing load after seating and the maximum number of prestressing strands for each girder shall be as shown in the post-tensioning table.

3. The design is based on 1/4 inch diameter low-relaxation strands with a landing load for each tendon as shown in the post-tensioning table. An anchor set of 1/8 inch of curvature friction coefficient, $A = 0.20$ and a working friction coefficient, $A = 0.001$, of the actual anchor set used by the contractor shall be specified in the shop plans and included in the transfer force calculations.

4. The design is based on the estimated prestressing losses of the tendons, including those shown in the post-tensioning table due to steel relaxation, elastic shortening, creep, and shrinkage of concrete.

5. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations.

6. A. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the web.

7. B. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent web. At no time during stressing operation will more than one-sixth of the total prestressing force be applied eccentrically about the centerline of the web.

8. The maximum outside diameter of the duct shall be 1½ inches. The area of the duct shall be at least 20 times the net area of the prestressing steel in the duct.

9. The tendon shall be stressed from pier to pier.

10. Outside forms from inside and outside of the closures and crossbeams shall be removed prior to post-tensioning.
PRE-TENSIONING NOTES:

1. PLAN LENGTH SHALL BE INCREASED AS NECESSARY TO COMPENSATE FOR SHORTENING DUE TO PRESTRESS AND SHRINKAGE.

2. ALL STRANDS FOR PRETENSIONING SHALL BE 3/8" DIAMETER LOW RELAXATION STRANDS (AASHTO M203, GRADE 270).

3. FOR END TYPES A, C AND D, CUT ALL STRANDS FLUSH WITH THE GIRDER ENDS AND PAINT WITH AN APPROVED EPOXY RESIN, EXCEPT FOR EXTENDED STRANDS AS SHOWN. FOR END TYPES B AND E CUT ALL STRANDS 1" BELOW CONCRETE SURFACE AND GROUT WITH AN APPROVED EPOXY GROUT.

4. EXTENDED STRANDS AND BARS ARE PARALLEL TO GIRDER.

5. LIFTING BARS SHALL BE 1 1/2" DIAMETER HIGH STRENGTH THREADING BARS (AASHTO M275, GRADE 150 MINIMUM). LIFTING HARDWARE THAT CONNECTS TO THREADING BARS SHALL BE DESIGNED AND DETAILED BY THE CONTRACTOR. LIFTING FORCES ON THREADING BARS SHALL BE VERTICAL ONLY AND WITHIN 10 DEGREES OF PERPENDICULAR TO A LINE BETWEEN PICK POINTS. CONTRACTOR SHALL SUBMIT CALCULATIONS FOR APPROVAL BY THE ENGINEER IF LIFTING FORCES ARE TO BE OTHERWISE.

6. EXTRA CAUTION MUST BE EXERCISED IN HANDLING AND PLACING ALL GIRDERS. ALL GIRDERS SHALL BE CHECKED TO ENSURE THAT THEY ARE BRACED ADEQUATELY TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING SHIPPING.

7. THE TOP SURFACE OF THE GIRDER FLANGE SHALL BE ROUGHENED IN ACCORDANCE WITH SECTION 6-02.3(25)H.

8. FORMS FOR BEARING PAD RECESSES SHALL BE CONSTRUCTED AND FASTENED IN SUCH A MANNER AS NOT TO CAUSE DAMAGE TO THE GIRDER DURING STRAND RELEASE OPERATION.
GIRDER ELEVATION - MID-SEGMENT

- OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR WEB. PLACE HOLES AND INSERTS PARALLEL TO SKEW. INSERTS SHALL BE T & S KOLBE, BURKE, INTENSILE, LANCASTER MALLEABLE OR APPROVED EQUAL.

TYPICAL MID-SEGMENT ELEVATION

- PICKUP FORCE
- INTER. DIAPHRAGM
- GIRDER & INTERMEDIATE DIAPHRAGM
- LIFTING BARS 2 - 1/8" #4 HS. THREAD BARS WINCH OR NUT AT BOTTOM
- G3 #4

* OMIT HOLES AND PLACE INSERTS ON THE INTERIOR FACE OF EXTERIOR WEB. PLACE HOLES AND INSERTS PARALLEL TO SKEW. INSERTS SHALL BE T & S KOLBE, BURKE, INTENSILE, LANCASTER MALLEABLE OR APPROVED EQUAL.
NOTES TO DESIGNER:

1. This strand extension detail is to be used for continuous spans at moment resisting diaphragms only. This detail is not applicable to continuous spans using hinge diaphragms.

2. Designer shall calculate the exact number of extended straight strands needed to develop the required moment capacity at the end of the girder. This calculation shall be based on the tensile strength of the strands, the stresses imposed on the anchor, and concrete bearing against the projected area of the anchor.

3. The total number of extended strands shall not be less than:

\[
N_{ps} = \frac{12(M_c + V_c/h - M_t)}{F_t N_c} K
\]

where:

- \(M_c, V_c\) = The lesser of Elastic or Plastic hinging moment & shear capacity of column, Ft-kips, kips respectively,
- \(h\) = distance from top of column to c.g. of superstructure, ft
- \(N_c\) = number of columns
- \(N_g\) = number of girders
- \(A_{ps}\) = area of each extended strand, in²
- \(F_t\) = ultimate strength of strands, ksi
- \(d\) = distance from top of slab to c.g. of extended strands, in
- \(M_t\) = moment due to SIDL (Traffic barrier, sidewalk, etc.)
- \(K\) = 0.5 for \(L_1 = L_2\), 0.67 for \(L_1 = 2L_2\)

NOTE: Dimensions shall be shown in imperial units to the nearest 1/8 inch.

3.6”Ø DRAIN HOLE (SEE “FRAMING PLAN” FOR LOCATION)

STRAIGHT STRAND EXTENSION DETAIL
ALTERNATE # 2

END OF PRECAST SEGMENT

PREVENT STRAND BOND

END SEGMENT 1

STRAIGHT

C.G. STRANDS

END SEGMENT 2

STRAIGHT

C.G. STRANDS

STRAND PATTERN

STRAIGHT STRAND LOCATION SEQUENCE SHALL BE AS SHOWN (1), (2), ETC.

NOTES TO DESIGNER:

1. This strand extension detail is to be used for continuous spans at moment resisting diaphragms only. This detail is not applicable to continuous spans using hinge diaphragms.

2. Designer shall calculate the exact number of extended straight strands needed to develop the required moment capacity at the end of the girder. This calculation shall be based on the tensile strength of the strands, the stresses imposed on the anchor, and concrete bearing against the projected area of the anchor.

3. The total number of extended strands shall not be less than:

\[
N_{ps} = \frac{12(M_c + V_c/h - M_t)}{F_t N_c} K
\]

where:

- \(M_c, V_c\) = The lesser of Elastic or Plastic hinging moment & shear capacity of column, Ft-kips, kips respectively,
- \(h\) = distance from top of column to c.g. of superstructure, ft
- \(N_c\) = number of columns
- \(N_g\) = number of girders
- \(A_{ps}\) = area of each extended strand, in²
- \(F_t\) = ultimate strength of strands, ksi
- \(d\) = distance from top of slab to c.g. of extended strands, in
- \(M_t\) = moment due to SIDL (Traffic barrier, sidewalk, etc.)
- \(K\) = 0.5 for \(L_1 = L_2\), 0.67 for \(L_1 = 2L_2\)

NOTE: Dimensions shall be shown in imperial units to the nearest 1/8 inch.

3.6”Ø DRAIN HOLE (SEE “FRAMING PLAN” FOR LOCATION)
# NOTE TO DESIGNER

1. If ground line is less than 2'-0" minimum below the bottom of girder at front face of abutment, a curtain wall shall be provided.

## TYPICAL END TYPE "A" DIAPHRAGM

**AT END PIERS**

![Diagram of typical end type "A" diaphragm at end piers]

### ELEVATION

- 1/2" THICK BUTYL RUBBER SHEETING
- 1'-0" UNDER DIAPHRAGM

### PLAN VIEW

- BUTYL RUBBER @ DIAPHRAGM
- BUTYL RUBBER @ VERTICAL JOINTS

### END DIAPHRAGM GEOMETRY

- DIAPHRAGM - BUTYL RUBBER SHEETING
- WALL
- BEARING
- BEARING

**NOTE:**

- Joint filler shall be used to cover all vertical end diaphragm joints.
- Joint filler type 1 shall be used to cover all horizontal end diaphragm joints.

### SUMMARY OF JOINTS

- **OPEN JOINT**
- **CONSTRUCTION JOINT**
- **Butyl rubber @ diaphragm**
- **Butyl rubber @ vertical joints**

---

**DIAPHRAGM DETAILS SHEET**

**MISCELLANEOUS DIAPHRAGM DETAILS SHEET**

**GIRDER DETAILS SHEET**
Introduction

The slab haunch is the distance between the top of a girder and the bottom of the roadway slab. The haunch varies in depth along the length of the girder accommodating the girder camber and geometric effects of the roadway surface including super elevations, vertical curves and horizontal curves.

The basic concept in determining the required “A” dimension is to provide a haunch over the girder such that the top of the girder is not less than the fillet depth (typically ¾") below the bottom of the slab at the center of the span. This provides that the actual girder camber could exceed the calculated value by 1¾” before the top of the girder would interfere with the bottom mat of slab reinforcement.

It is desirable to have points of horizontal and vertical curvature and super elevation transitions off the bridge structure as this greatly simplifies the geometric requirements on the slab haunch. However, as new bridges are squeezed into the existing infrastructure it is becoming more common to have geometric transitions on the bridge structure.

Each geometric effect is considered independently of the others. The total geometric effect is the algebraic sum of each individual effect.

Fillet Effect

The distance between the top of the girder and the top of the roadway surface, must be at least the thickness of the roadway slab plus the fillet depth.

\[ \Delta_{deck} = t_{slab} + t_{fillet} \]
Excessive Camber Effect

The girder haunch must be thickened to accommodate any camber that remains in the girder after slab casting. This is the difference between the “D” and “C” dimensions from the Girder Schedule Table. Use a value of 2 ½” at the preliminary design stage to determine vertical clearance.

Profile Effect

The profile effect accounts for changes in the roadway profile along the length of the girder. Profile changes include grade changes, vertical curve effects, and offset deviations between the centerline of girder and the alignment caused by flared girders and/or curvature in the alignment.

When all of the girders in a span are parallel and the span is contained entirely within the limits of a vertical and/or horizontal curve, the profile effect is simply the sum of the Vertical Curve Effect and the Horizontal Curve Effect.

\[
\Delta_{\text{profile effect}} = \Delta_{\text{vertical curve effect}} + \Delta_{\text{horizontal curve effect}} \tag{5-B1.1}
\]

The horizontal curve effect is, assuming a constant super elevation rate along the length of the span,

\[
\Delta_{\text{horizontal curve effect}} = \frac{1.5S^2m}{R} \tag{5-B1.2}
\]

Where:
\[S\] = The length of curve in feet
\[R\] = The radius of the curve in feet
\[m\] = The crown slope

The horizontal curve effect is in inches.
The horizontal curve effect is, assuming a constant super elevation rate along the length.

\[ \Delta = \frac{S}{R} \]  
(5-B1.3)

\[ \phi = \frac{\Delta}{R} \]  
(5-B1.4)

\[ \phi = \frac{S}{4R} \]  
(5-B1.5)

\[ \tan \phi \approx \phi \]  
(5-B1.6)

\[ \tan \phi \approx \frac{2H}{S} \]  
(5-B1.7)

\[ H = \frac{S}{2} \tan \phi \approx \frac{S}{2} \phi = \frac{S}{2} \times \frac{S}{4R} = \frac{S^2}{8R} \]  
(5-B1.8)

\[ \Delta_{\text{horizontal curve effect}} = \frac{S^2}{8R} \times 12 \text{ in} = \frac{1.5S^2}{R} \text{m (inches)} \]  
(5-B1.9)
The vertical curve effect is

\[ \Delta_{\text{vertical curve effect}} = \frac{1.5GL^2_g}{100L} \text{ (inches)} \]  \hspace{1cm} (5-B1.10)

Where:
- \( G \) = The algebraic difference in profile tangent grades \( G = g_2 - g_1 \) (%)
- \( L^2_g \) = The girder length in feet
- \( L \) = The vertical curve length in feet

The vertical curve effect is in inches and is positive for sag curves and negative for crown curves.

![Diagram of vertical curve effect](image)

\[ K = \frac{100G}{2L} \hspace{1cm} (5-B1.11) \]

\[ \Delta_{\text{vertical curve effect}} = K \frac{L^2_g}{40,000} \times 12 \text{ in} = \frac{G}{2L} \frac{L^2_g}{400} \times 12 = \frac{1.5GL^2_g}{100L} \hspace{1cm} (5-B1.12) \]

If one or more of the following roadway geometry transitions occur along the span, then a more detailed method of computation is required:
- Change in the super elevation rate
- Grade break
- Point of horizontal curvature
- Point of vertical curvature
- Flared girders
The exact value of the profile effect may be determined by solving a complex optimization problem. However it is much easier and sufficiently accurate to use a numerical approach.

The figure below, while highly exaggerated, illustrates that the profile effect is the distance the girder must be placed below the profile grade so that the girder, ignoring all other geometric effects, just touches the lowest profile point between the bearings.

In the case of a crown curve the haunch depth may reduced. In the case of a sag curve the haunch must be thickened at the ends of the girder.

To compute the profile effect:

1. Create a chord line parallel to the top of the girder (ignoring camber) connecting the centerlines of bearing. The equation of this line is

   \[ y_c(x_i) = y_a(x_s, z_s) + (x_i - x_s) \left( \frac{y_a(x_e, z_e) - y_a(x_s, z_s)}{x_e - x_s} \right) \]  

   Where:
   - \( x_i \) = Station where the elevation of the chord line is being computed
   - \( x_s \) = Station at the start of the girder
   - \( x_e \) = Station at the end of the girder
   - \( z_s \) = Normal offset from alignment to centerline of the girder at the start of the girder at station \( x_s \)
   - \( z_e \) = Normal offset from the alignment to the centerline of the girder at the end of the girder at station \( x_e \)
   - \( y_a(x_s, z_s) \) = Elevation of the roadway profile at station \( x_s \) and offset \( z_s \)
   - \( y_a(x_e, z_e) \) = Elevation of the roadway profile at station \( x_e \) and offset \( z_e \)
   - \( y_c(x_i) \) = Elevation of the chord line at station \( x_i \)
2. At 10th points along the span, compute the elevation of the roadway surface directly above the centerline of the girder, \( y_c(x_i, z_i) \), and the elevation of the line paralleling the top of the girder, \( y_a(x_i) \). The difference in elevation is the profile effect at station \( x_i \),

\[
\Delta_{\text{profile effect}}(i) = y_c(x_i) - y_a(x_i, z_i)
\]  

(5-B1.14)

**Girder Orientation Effect**

The girder orientation effect accounts for the difference in slope between the roadway surface and the top of the girder. Girders such as I-beams are oriented with their Y axis plumb. Other girders such as U-beam, box beam, and slabs are oriented with their Y axis normal to the roadway surface. The orientation of the girder with respect to the roadway surface, and changes in the roadway surface along the length of the girder (super elevation transitions) define the Girder Orientation Effect.

If the super elevation rate is constant over the entire length of the span and the Y-axis of the girder is plumb, the girder orientation effect simplifies to the Top Width Effect,

\[
\Delta_{\text{girder orientation effect}} = \Delta_{\text{top width effect}} = m \left( \frac{W_{\text{top}}}{2} \right)
\]  

(5-B1.15)

If the super elevation rate varies along the span, the girder orientation effect may be computed at 10th points using this equation.

If there is a change in super elevation rate and/or the Y-axis of the girder is not plumb, then once again a more complex computation is required.
To compute the girder orientation effect at each 10th point along the girder, when the girder is not plumb:

1. Determine the cross slope, $m$, of the roadway surface at station $x_i$. If there is a crown point over the girder the cross slope is taken as

$$m(x_i, z_i) = \frac{y_a(x_i, z_i^\text{left}) - y_a(x_i, z_i^\text{right})}{z_i^\text{left} - z_i^\text{right}}$$  \hspace{1cm} (5-B1.16)$$

Where:
- $x_i$ = The station where the cross slope is being computed
- $z_i$ = Normal offset from the alignment to the centerline of the girder at the end of the girder at station $x_i$
- $z_i^\text{left}$ = Offset from the alignment to the top left edge of the girder
- $z_i^\text{right}$ = Offset from the alignment to the top right edge of the girder
- $y_a(x_i, z_i^\text{left})$ = Roadway surface elevation at station $x_i$ and normal offset $z_i^\text{left}$
- $y_a(x_i, z_i^\text{right})$ = Roadway surface elevation at station $x_i$ and normal offset $z_i^\text{right}$
2. Determine the girder orientation effect at station

\[ x_i = \frac{W_{\text{top}}}{Z} \frac{m - m_g}{\sqrt{1 + m_g^2}} \]  \hspace{1cm} (5-B1.17)

“A” Dimension

The “A” dimension is the sum of all these effects.

\[ A = \Delta_{\text{fillet}} + \Delta_{\text{excess camber}} + \Delta_{\text{profile effect}} + \Delta_{\text{girder orientation effect}} \]  \hspace{1cm} (5-B1.18)

If you have a complex alignment, determine the required “A” dimension for each section and use the greatest value.

Round “A” to the nearest ¼".

The minimum value of “A” is

\[ A_{\text{min}} = \Delta_{\text{fillet}} + \Delta_{\text{girder orientation effect}} \]  \hspace{1cm} (5-B1.19)

If a Drain Type 5 crosses the girder, “A” shall not be less than 9".
Limitations

These computations are for a single girder line. The required haunch should be determined for each girder line in the structure. Use the greatest “A” dimension.

These computations are also limited to a single span. A different haunch may be needed for each span or each pier. For example, if there is a long span adjacent to a short span, the long span may have considerably more camber and will require a larger haunch. There is no need to have the shorter spans carry all the extra concrete needed to match the longer span haunch requirements. With the WF series girders, the volume of concrete in the haunches can add up quickly. The shorter span could have a different haunch at each end as illustrated below.

![Diagram of Limitations](image)

*Same "A" dimension at all piers*

*Pier by pier "A" dimensions*

*Span by span "A" dimensions*
Stirrup Length and Precast Deck Leveling Bolt Considerations

For bridges on crown vertical curves, the haunch depth can become excessive to the point where the girder and diaphragm stirrups are too short to bend into the proper position. Similarly the length of leveling bolts in precast deck panels may need adjustment.

Stirrup lengths are described as a function of “A” on the standard girder sheets. For example, the G1 and G2 bars of a WF74G girder are 6’-5”+ “A” in length. For this reason, the stirrups are always long enough at the ends of the girders. Problems occur when the haunch depth increases along the length of the girder to accommodate crown vertical curves and super elevation transitions.

If the haunch depth along the girder exceeds “A” by more than 2”, an adjustment must be made. The haunch depth at any section can be compute as

\[ A - \Delta_{profile
effect} - \Delta_{excess\camber} \]  

(5-B1.20)

“A” Dimension Worksheet - Simple Alignment

Fillet Effect

| Slab Thickness (t_{slab}) | = _____ in |
| Fillet Size (t_{fillet}) | = _____ in |
| \( \Delta_{fillet} = t_{slab} + t_{fillet} \) | = _____ in |

Excess Camber Effect

| “D” Dimension from Girder Schedule (120 days) | = _____ in |
| “C” Dimension from Girder Schedule | = _____ in |
| \( \Delta_{excess\camber} = "D" - "C" \) | = _____ in |

Profile Effect

| Horizontal Curve Effect, \( \Delta_{horizontal\ curve\ effect} = \frac{1.5S^2m}{R} \) | = _____ in |
| Vertical Curve Effect, \( \Delta_{vertical\ curve\ effect} = \frac{1.5GL^2}{100L} \) | = _____ in (+ for sag, – for crown) |
| \( \Delta_{profile} = \Delta_{horizontal\ curve\ effect} + \Delta_{vertical\ curve\ effect} \) | = _____ in |

Girder Orientation Effect

| Girder must be plumb. |
| \( \Delta_{girder\ orientation} = 0 \) for U-beams inclined parallel to the slab |
| \( \Delta_{girder\ orientation} = \Delta_{top\ flange\ effect} = m \left( \frac{W_{top}}{2} \right) \) | = _____ in |

“A” Dimension

| \( \Delta_{fillet} + \Delta_{excess\camber} + \Delta_{profile\ effect} + \Delta_{girder\ orientation\ effect} \) | = _____ in |
| Round to nearest \( \frac{1}{4}'' \) | = _____ in |
| Minimum “A” Dimension, \( \Delta_{fillet} + \Delta_{girder\ orientation\ effect} \) | = _____ in |

| “A” Dimension = _____ in |
Example

Slab: Thickness = 7.5", Fillet = 0.75"
WF74G Girder: W_{top} = 49"
Span Length = 144.4 ft
Crown Slope = 0.04 ft/ft
Camber: D = 7.55", C = 2.57"
Horizontal Curve Radius = 9500 ft through centerline of bridge
Vertical Curve Data: \( g_1 = 2.4\% \), \( g_2 = -3.2\% \), L = 800 ft

Fillet Effect

Slab Thickness (t_{slab}) = 7.5"
Fillet Size (t_{fillet}) = 0.75"
\( \Delta_{fillet} = t_{slab} + t_{fillet} \) = 8.25"

Excess Camber Effect

“D” Dimension from Girder Schedule (120 days) = 7.55"
“C” Dimension from Girder Schedule = 2.57"
\( \Delta_{excess\ camber} = "D" - "C" \) = 4.98"

Profile Effect

Horizontal Curve Effect
Chord Length = 144.4 ft, \( C = 2R \sin \frac{\Delta}{2} \)
144.4 = 2(9500)\( \sin \frac{\Delta}{2} \)
\( \Delta = 0.87" \)

Curve Length
\( R \Delta \left( \frac{\pi}{180} \right) = 9500(0.87) \left( \frac{\pi}{180} \right) = 144.4 ft \)
\( \Delta_{horizontal\ curve\ effect} = \frac{1.5S^2m}{R} = \frac{1.5(144.4)^2}{9500} = 0.13" \)

Vertical Curve Effect \( \Delta_{vertical\ curve\ effect} = \frac{1.5GL^2}{100L} = \frac{1.5(-5.6)(-144.4)^2}{100(800)} = -2.19" \)
(+ for sag, – for crown)
\( \Delta_{profile} = \Delta_{horizontal\ curve\ effect} + \Delta_{vertical\ curve\ effect} = 0.13 - 2.19 = -2.06" \)

Girder Orientation Effect
\( \Delta_{girder\ orientation} = \Delta_{top\ flange\ effect} = m \left( \frac{W_{top}}{2} \right) = 0.04 \frac{49}{2} = 0.98" \)

“A” Dimension
\( \Delta_{fillet} + \Delta_{excess\ camber} + \Delta_{profile\ effect} + \Delta_{girder\ orientation\ effect} = 8.25 + 4.98 - 2.06 + 0.98 = 12.15" \)
Round to nearest \( \frac{1}{4}" = 12.25" \)
Minimum “A” Dimension, \( \Delta_{fillet} + \Delta_{girder\ orientation\ effect} = 8.25 + 0.98 = 9.23" \)
“A” Dimension = 12\( \frac{1}{4} " \)
### Appendix 5-B3

#### Existing Bridge Widening

The following listed bridge widenings are included as aid to the designer. These should not be construed as the only acceptable methods of widening; there is no substitute for the designer’s creativity or ingenuity in solving the challenges posed by bridge widenings.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Contract No.</th>
<th>Type of Bridge</th>
<th>Unusual Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>NE 8th Street U’Xing</td>
<td>405 9267</td>
<td>Ps. Gir.</td>
<td>Pier replacements</td>
</tr>
<tr>
<td>Higgins Slough</td>
<td>536 9353</td>
<td>Flat Slab</td>
<td></td>
</tr>
<tr>
<td>ER17 and AR17 O-Xing</td>
<td>5 9478</td>
<td>Box Girder</td>
<td>Middle and outside widening.</td>
</tr>
<tr>
<td>SR 538 O-Xing</td>
<td>5 9548</td>
<td>T-Beam</td>
<td>Unbalanced widening section support at diaphragms until completion of closure pour.</td>
</tr>
<tr>
<td>B-N O’Xing</td>
<td>5 9566</td>
<td>Box Girder</td>
<td>Widened with P.S. Girders, X-beams, and diaphragms not in line with existing jacking required to manipulate stresses, added enclosure walls.</td>
</tr>
<tr>
<td>Blakeslee Jct. E/W</td>
<td>5 9638</td>
<td>T-Beam and Box Girder</td>
<td>Post-tensioned X-beam, single web.</td>
</tr>
<tr>
<td>B-N O’Xing</td>
<td>18 9688</td>
<td>Box Girder</td>
<td></td>
</tr>
<tr>
<td>SR 536</td>
<td>9696</td>
<td>T-Beam</td>
<td>Similar to Contract 9548.</td>
</tr>
<tr>
<td>LE Line over Yakima River</td>
<td>90 9806</td>
<td>Box Girder</td>
<td>Pier shaft.</td>
</tr>
<tr>
<td>SR 18 O-Xing</td>
<td>90 9823</td>
<td>P.S. Girder</td>
<td>Lightweight concrete.</td>
</tr>
<tr>
<td>Hamilton Road O-Xing</td>
<td>5 9894</td>
<td>T-Beam</td>
<td>Precast girder in one span.</td>
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<tr>
<td>Dillenbauch Creek</td>
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<td>Flat Slab</td>
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<tr>
<td>Longview Wye SR 432 U-Xing</td>
<td>5</td>
<td>P.S. Girder</td>
<td>Bridge lengthening.</td>
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<td>Klickitat River Bridge</td>
<td>142</td>
<td>P.S. Girder</td>
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<td>Skagit River Bridge</td>
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<td>Steel Truss</td>
<td>Rail modification.</td>
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<td>B-N O’Xing at Chehalis</td>
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<td></td>
<td>Replacement of thru steel girder span with stringer span.</td>
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<td>Bellevue Access EBCD Widening and Pier 16 Modification</td>
<td>90 3846</td>
<td>Flat Slab and Box Girder</td>
<td>Deep, soft soil. Straddle best replacing single column.</td>
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<td>Totem Lake/NE 124th I/C</td>
<td>405 3716</td>
<td>T-Beam</td>
<td>Skew = 55 degrees.</td>
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<td>Pacific Avenue I/C</td>
<td>5 3087</td>
<td>Box Girder</td>
<td>Complex parallel skewed structures.</td>
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<td>SR 705/SR 5 SB Added Lane</td>
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<td>Tapered widening of flat slab outrigger pier, combined footings.</td>
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<td>3845</td>
<td>CIP Conc. Box Girder</td>
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<td>Fishtrap Creek Bridge 546/8</td>
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<td>P.C. Units</td>
<td>Widening of existing P.C. Units. Tight constraints on substructure.</td>
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<td>Widening/Deck replacement using standard rolled sections.</td>
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<td>SR</td>
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<td>Tye River Bridges 2/126 and 2/127</td>
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<td>Obdashian Bridge 2/275</td>
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### Post-tensioned Box Girder Bridges

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<th>Width Curb</th>
<th>Span/Depth</th>
<th>Skew Deg.</th>
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<td>182</td>
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</table>

**Middle 3 spans of 7-span bridge are post-tensioned.**
<table>
<thead>
<tr>
<th>Contract No.</th>
<th>Name</th>
<th>County</th>
<th>Award Date</th>
<th>Span</th>
<th>Span/</th>
<th>Slew Deg.</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>1439</td>
<td>SR 516 Oxing</td>
<td>King</td>
<td>3/79</td>
<td>63.5</td>
<td>42</td>
<td>24.2</td>
<td>40</td>
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<tr>
<td>1580</td>
<td>Ahlanum Creek Oxing</td>
<td>Yakima</td>
<td>9/79</td>
<td>167</td>
<td>28</td>
<td>25.1</td>
<td>Curved 1200R</td>
</tr>
<tr>
<td></td>
<td>SB</td>
<td></td>
<td>5/172</td>
<td>167</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>NB</td>
<td></td>
<td>137</td>
<td>166</td>
<td></td>
<td></td>
<td>Curved 1200R</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>6/172</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1950</td>
<td>Yakima River Bridges</td>
<td>Benton</td>
<td>10/60</td>
<td>140+</td>
<td>Varies</td>
<td>48-100'</td>
<td>Curve 6000R</td>
</tr>
<tr>
<td></td>
<td>North Bridge</td>
<td></td>
<td>161</td>
<td>161</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>South Bridge</td>
<td></td>
<td>140+</td>
<td>161</td>
<td></td>
<td></td>
<td>Curve 5000R</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>161</td>
<td>161</td>
<td></td>
<td></td>
<td>Transverse post-tensioning, 10’ bicycle and pedestrian path on one side.</td>
</tr>
<tr>
<td></td>
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<td>215</td>
<td>215</td>
<td></td>
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<td>147</td>
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<tr>
<td>2156</td>
<td>14-D Line</td>
<td>Clark</td>
<td>11/81</td>
<td>163</td>
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<tr>
<td>2156</td>
<td>14-D Line (South)</td>
<td>Clark</td>
<td>11/81</td>
<td>138</td>
<td>25</td>
<td>24.4</td>
<td>Curved 6000R</td>
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<td></td>
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<td></td>
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<td>138</td>
<td>138</td>
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</tr>
<tr>
<td>2207</td>
<td>GE Line Over G Line</td>
<td>Benton</td>
<td>4/82</td>
<td>90</td>
<td>38</td>
<td>23.5</td>
<td>Curved 14000R</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>168</td>
<td></td>
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<td></td>
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<td>90</td>
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<tr>
<td>Contract No.</td>
<td>Name</td>
<td>County</td>
<td>Award Date</td>
<td>Span</td>
<td>Width Curb (ft.)</td>
<td>Span/Depth</td>
<td>Slew Deg.</td>
</tr>
<tr>
<td>-------------</td>
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</tr>
<tr>
<td>2207</td>
<td>RA Line Over ER Line</td>
<td>Benton</td>
<td>4/82</td>
<td>47</td>
<td>104</td>
<td>17.3</td>
<td>20</td>
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<td>47</td>
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<tr>
<td>2245</td>
<td>Pearl Street Overpass</td>
<td>Pierce</td>
<td>4/82</td>
<td>49</td>
<td>54</td>
<td>22.7</td>
<td></td>
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<tr>
<td>2245</td>
<td>6th Avenue Overpass</td>
<td>Pierce</td>
<td>4/82</td>
<td>43</td>
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<td>2327</td>
<td>Spokane River Bridge</td>
<td>Spokane</td>
<td>6/82</td>
<td>175</td>
<td>76</td>
<td>Varies</td>
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<td>175</td>
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<td>***</td>
<td>Green River Bridge</td>
<td>King</td>
<td>118</td>
<td>118</td>
<td>74</td>
<td>Varies</td>
<td>22</td>
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<td>3794</td>
<td>Sen. Sam C. Guess Memorial</td>
<td>King</td>
<td>5/90</td>
<td>126</td>
<td>77</td>
<td>Varies</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>(Division St. 21644)</td>
<td></td>
<td></td>
<td>126</td>
<td></td>
<td>(depth 5.5 to 8.5 at piers)</td>
<td></td>
</tr>
</tbody>
</table>

***Not yet to contract.
Appendix 5-B5  
Prestressed Girder Design

References
1. WSDOT BDM M23-50, Aug 2010  
2. WSDOT Bridge Office Design Memorandums  
3. AASHTO LRFD Bridge Design Specifications with Interim Revisions through 2010  
4. PCI Design Handbook, 5th Ed  
5. PCI Bridge Design Manual (PCI BDM)  
6. PG Super Theoretical Manual  
8. PCI Journal, Jan-Feb 2005, Flexural Strength of Reinforced and Prestressed Concrete T-Beams

Unit Definitions and Mathcad System Constants

\[ k_{cf} := \text{kip} \div \text{ft}^3 \quad \text{ORIGIN} := 1 \]

Design Outline

1. Material Properties  
2. Structure Definition  
3. Computation of Section Properties  
4. Loading and Limit State Parameters  
5. Dead and Live Load Force Effects  
6. Computation of Stresses for Dead and Live Loads  
7. Prestressing Forces and Stresses  
8. Stresses at Service and Fatigue Limit States  
9. Strength Limit State  
10. Shear & Longitudinal Reinf Design  
11. Deflection and Camber  
12. Lifting, Shipping, and General Stability  
13. Check Results
1. Material Properties

1.1 Concrete - Prestressed Girder

Minimum compressive strength at release
\[ f'_{ci} := 7.5 \text{ ksi} \] BDM 5.1.1.A.2

Nominal 28-day compressive strength
\[ f'_{c} := 8.5 \text{ ksi} \] BDM 3.8

Unit weight of girder concrete (for dead load)
\[ w_{c} := 0.165 \text{kcf} \] BDM 5.1.1.D

Unit weight of girder concrete for elastic modulus
\[ w_{cE} := 0.155 \text{kcf} \] BDM 5.1.1.D; LRFD 5.4.2.4

Aggregate correction factor
\[ K_{1} := 1.0 \] BDM 5.1.1.D; LRFD 5.4.2.4

Concrete modulus of elasticity
\[ E_{c} := \begin{cases} 33000 \cdot K_{1} \left( \frac{w_{cE}}{\text{kcf}} \right)^{1.5} \cdot \frac{f'_{c}}{\text{ksi}} \text{ ksi if } f'_{c} \leq 15 \text{ksi} & = 5871 \text{ksi} \\ \text{"error" otherwise} & \text{ } \end{cases} \]

Concrete modulus of elasticity at transfer
\[ E_{ci} := \begin{cases} 33000 \cdot K_{1} \left( \frac{w_{cE}}{\text{kcf}} \right)^{1.5} \cdot \frac{f'_{ci}}{\text{ksi}} & = 5515 \text{ksi} \\ \text{"error" otherwise} & \text{ } \end{cases} \] BDM 5.1.1.D; LRFD 5.4.2.4

Concrete modulus of rupture for flexure
\[ f'_{r} := 0.24 \cdot \frac{f'_{c}}{\text{ksi}} = 0.700 \text{ksi} \] LRFD 5.4.2.6

Concrete modulus of rupture for flexure at lifting
\[ f'_{rL} := 0.24 \cdot \frac{f'_{ci}}{\text{ksi}} = 0.657 \text{ksi} \] LRFD 5.4.2.6

Concrete modulus of rupture to calculate minimum reinforcement
\[ f_{r,Mcr.min} := 0.37 \cdot \sqrt{\frac{f'_{c}}{\text{ksi}}} = 1.079 \text{ksi} \]

1.2 Concrete - CIP Slab

Nominal 28-day compressive strength
\[ f'_{cs} := 4 \text{ ksi} \] BDM 5.1.1.B

Unit weight of CIP concrete (for dead load)
\[ w_{cs} := 0.155 \text{kcf} \] BDM 3.8

Unit weight of CIP concrete for elastic modulus
\[ w_{csE} := 0.150 \text{kcf} \] BDM 5.1.1.D

Concrete modulus of elasticity
\[ E_{cs} := \begin{cases} 33000 \cdot K_{1} \left( \frac{w_{csE}}{\text{kcf}} \right)^{1.5} \cdot \frac{f'_{cs}}{\text{ksi}} \text{ ksi if } f'_{cs} \leq 15 \text{ksi} & = 3834 \text{ksi} \\ \text{"error" otherwise} & \text{ } \end{cases} \]

Stress Block Factor
\[ \beta_{1} := \begin{cases} 0.85 \text{ if } f'_{cs} \leq 4 \text{ksi} & = 0.85 \text{ LRFD 5.7.2.2} \\ 0.65 \text{ if } f'_{cs} \geq 8 \text{ksi} & \text{ } \\ 0.85 - 0.05 \cdot \frac{f'_{cs} - 4 \text{ksi}}{1 \text{ksi}} & \text{otherwise} \end{cases} \]

1.3 Reinforcing steel - deformed bars

This function returns a bar diameter:

This function returns a bar area:


\[ \text{dia(bar)} := \begin{cases} 
0.375\text{-in} & \text{if bar} = 3 \\
0.500\text{-in} & \text{if bar} = 4 \\
0.625\text{-in} & \text{if bar} = 5 \\
0.750\text{-in} & \text{if bar} = 6 \\
0.875\text{in} & \text{if bar} = 7 \\
1.000\text{in} & \text{if bar} = 8 \\
1.128\text{in} & \text{if bar} = 9 \\
1.270\text{in} & \text{if bar} = 10 \\
1.410\text{in} & \text{if bar} = 11 \\
1.693\text{in} & \text{if bar} = 14 \\
2.257\text{in} & \text{if bar} = 18 \\
\text{"ERROR" otherwise} 
\end{cases} \]

\[ \text{area(bar)} := \begin{cases} 
0.110\text{-in}^2 & \text{if bar} = 3 \\
0.197\text{-in}^2 & \text{if bar} = 4 \\
0.309\text{-in}^2 & \text{if bar} = 5 \\
0.445\text{-in}^2 & \text{if bar} = 6 \\
0.600\text{-in}^2 & \text{if bar} = 7 \\
0.786\text{-in}^2 & \text{if bar} = 8 \\
1.003\text{-in}^2 & \text{if bar} = 9 \\
1.270\text{-in}^2 & \text{if bar} = 10 \\
1.561\text{-in}^2 & \text{if bar} = 11 \\
2.251\text{-in}^2 & \text{if bar} = 14 \\
3.998\text{-in}^2 & \text{if bar} = 18 \\
\text{"ERROR" otherwise} 
\end{cases} \]

Yield strength \[ f_y := 60\text{-ksi} \]

Elastic modulus \[ E_k := 29000\text{-ksi} \]

**1.4 Prestressing Steel - AASHTO M-203, Uncoated, 7 Wire, Low-Relaxation Strands**

Tensile strength \[ f_{pu} := 270\text{-ksi} \]

Yield strength \[ f_{py} := 0.90f_{pu} = 243.0\text{-ksi} \]

Strand modulus of elasticity \[ E_p := 28500\text{-ksi} \]

Nominal strand diameter \[ d_b := 0.6\text{in} \]

Area of wire strand \[ A_p := \begin{cases} 
0.153\text{-in}^2 & \text{if } d_b = 0.5\text{in} \\
0.217\text{-in}^2 & \text{if } d_b = 0.6\text{in} 
\end{cases} \]

Transfer Length \[ l_t := 60\cdot d_b = 36.0\text{in} \]
2. Structure Definition

2.1 Bridge Geometry

Select "interior" or "exterior" girder

Bridge width (inside curb to inside curb)

Girder spacing

Number of girder lines

Skew angle (for girders round to 5 deg)

Design span, CL bearing to CL bearing

Distance from end of girder to CL bearing

Girder length (see BDM end diaphragm geometry)

Curb width on deck (see Standard Plans)

Deck overhang (from CL of exterior girder to end of deck)

Overhang thickness at edge of slab

Overhang thickness at exterior edge of top flange

2.2 Concrete Deck Slab

Slab depth for design

Depth of wearing surface

Slab depth for weight

2.3 Intermediate Diaphragms

Intermediate Diaphragm Thickness

Intermediate Diaphragm Height (excluding deck)

2.4 Prestressing

Number of harping strands

Number of straight strands

Number of temporary strands

Harping location from girder end

Distance from girder bottom to lowest straight strand

2.5 Site Data

Average annual relative humidity

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April 2015
3. Computation of Section Properties

3.1 Girder Properties

(collapsible region containing BDM Table 5.6.1-1)

Washington standard girder
Row in BDM Table 5.6.1-1 for this girder
Girder depth
Girder cross-section area
Girder moment of inertia (strong-axis)
Girder c.g. from girder bottom
Girder Volume-to-surface ratio
Girder Weight
Girder web width
Girder top flange width
Girder bottom flange width
Girder moment of inertia (weak axis)
Lifting Point from both ends of girder
Shipping Point from Front (left) end of girder
Shipping Point from Back (right) end of girder

Calculated section properties
Girder c.g. to girder top
Section modulus to top of girder
Section modulus to bottom of girder

Shear Stirrup Reinforcement

Since the reaction force in the direction of the applied shear introduces compression into the end region, the critical section for shear may be taken at \( d_1 \) from interior face of support. \( d_1 \) may be estimated using LRFD 5.8.2.9 where \( d_1 \) need not be taken less than 0.72 h. Place live load
vehicle with heavy axle at $d_v$ from support.

Estimate of $d_v$ to determine critical section for shear calculations

$$d_{est} := 0.72 \cdot (d_g + t_s) = 4.86 \text{ ft}$$

Vertical stirrup bar size

$$\text{bar}_v := 5$$

Define stirrup spacing for entire girder by giving stirrup reinforcing zone lengths and spacing of stirrups within each zone. Zones are defined sequentially from front of girder to the end. The sum of the zone lengths must equal the total girder length. Additional rows may be added if necessary. The first and last zones should be the clearance to the first stirrup from the end of the girder. A pair of stirrups is assumed located at the transition locations between zones.

### Front End of Girder

<table>
<thead>
<tr>
<th>Zone 1 Length (end clr)</th>
<th>$\text{VR}_{1,1} := 1.5 \text{ in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1 Stirrup Spacing</td>
<td>$\text{VR}<em>{1,2} := \text{VR}</em>{1,1}$</td>
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</tbody>
</table>

### Back End of Girder

<table>
<thead>
<tr>
<th>Zone 11 Length (end clr)</th>
<th>$\text{VR}<em>{11,1} := \text{VR}</em>{1,1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 11 Stirrup Spacing</td>
<td>$\text{VR}<em>{11,2} := \text{VR}</em>{1,2}$</td>
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</tbody>
</table>

### Zone 2 Length

<table>
<thead>
<tr>
<th>Zone 2 Length</th>
<th>$\text{VR}_{2,1} := 20 \text{ in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 2 Stirrup Spacing</td>
<td>$\text{VR}_{2,2} := 2.5 \text{ in}$</td>
</tr>
</tbody>
</table>

### Zone 3 Length

<table>
<thead>
<tr>
<th>Zone 3 Length</th>
<th>$\text{VR}_{3,1} := 72 \text{ in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 3 Stirrup Spacing</td>
<td>$\text{VR}_{3,2} := 6 \text{ in}$</td>
</tr>
</tbody>
</table>

### Zone 4 Length

<table>
<thead>
<tr>
<th>Zone 4 Length</th>
<th>$\text{VR}_{4,1} := 120 \text{ in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 4 Stirrup Spacing</td>
<td>$\text{VR}_{4,2} := 12 \text{ in}$</td>
</tr>
</tbody>
</table>

### Zone 5 Length

<table>
<thead>
<tr>
<th>Zone 5 Length</th>
<th>$\text{VR}_{5,1} := 120 \text{ in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 5 Stirrup Spacing</td>
<td>$\text{VR}_{5,2} := 12 \text{ in}$</td>
</tr>
</tbody>
</table>

### Zone 6 Length (at girder midspan)

$$\text{VR}_{6,1} := \text{GL} - \sum_{i=1}^{5} \text{VR}_{i,1} - \sum_{i=7}^{11} \text{VR}_{i,1}$$

### Zone 6 Stirrup Spacing (at girder midspan)

$$\text{VR}_{6,2} := 18 \text{ in}$$

<table>
<thead>
<tr>
<th>Length of Region</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>1.5</td>
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<tr>
<td>2</td>
<td>20.00</td>
<td>2.50</td>
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<td>3</td>
<td>72.00</td>
<td>6.00</td>
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<td>4</td>
<td>120.00</td>
<td>12.00</td>
</tr>
<tr>
<td>5</td>
<td>120.00</td>
<td>12.00</td>
</tr>
</tbody>
</table>

$$\text{VR} = \text{in}$$
Check if lengths of user defined shear reinforcement regions sum to the total girder length.

### 3.2 "A" Dimension

- **Fillet Effect**
  
  \[ A_{fi} := 0.75\text{in} \]

- **Excessive Camber Effect (estimate)**
  
  \[ A_{Ex} := 2.5\text{in} \]

- **Superelevation Rate**
  
  \[ \text{Super} := 0.02 \]

- **Length of Horizontal Curve**
  
  \[ S_H := 0\text{ft} \]

- **Radius of Horizontal Curve**
  
  \[ R_H := 0\text{ft} \]

- **Vertical Curve Length**
  
  \[ L_{VC} := 1000\text{ft} \]

- **Entrance Grade**
  
  \[ g_1 := 0 \text{ } \% \]

- **Exit Grade**
  
  \[ g_2 := -0 \text{ } \% \]

- **Horizontal Curve Effect**
  
  \[ A_{HC} := \frac{1.5 \left( S_H \div \text{ft} \right)^2 \cdot \text{Super}}{R_H \div \text{ft}} \text{in} = 0.000 \text{in} \]

- **Vertical Curve Effect**
  
  \[ A_{VC} := \frac{1.5 \left( g_2 - g_1 \right) \left( \frac{GL}{\text{ft}} \right)^2}{100 \cdot \frac{L_{VC}}{\text{ft}}} \text{in} = 0.000 \text{in} \]

- **Girder Orientation Effect**
  
  \[ A_{Orient} := \frac{b_f}{2} = 0.490 \text{in} \]

- **Calculated "A" dimension**
  
  \[ A := \text{max} \left( A_{P1} \cdot t_{s2} + A_{fi} \right) = 0.000 \text{in} \]

### 3.3 Span-to-Depth Ratio (Optional Criteria)

Minimum depth (for simple span prestressed girder, including deck)

\[ \text{depth}_{min} := 0.045 \cdot L = 70.2 \text{in} \]

LRFD 2.5.2.6.3
Check minimum depth

\( \text{chk}_{2} := \text{if} \left( \text{depth}_{\text{min}} \leq d_{g} + t_{s} \quad \theta_{sk} \leq 75\,\text{deg} \right) \) = "OK"

### 3.4 Composite Section Properties

**Effective flange width**

Check if Refined Analysis is required

\( \text{chk}_{3} := \text{if} \left( \theta_{sk} > 75\,\text{deg} \right) \) = "OK"

Effective flange width for interior girder

\( b_{i} := S = 78.00\,\text{in} \)

Effective flange width for exterior girder

\( b_{e} := 0.5 \cdot S + \text{overhang} = 82.50\,\text{in} \)

Effective flange width

\( b_{e} := \begin{cases} b_{i} \quad \text{if} \quad \text{girder} = \text{"interior"} = 78.00\,\text{in} \\ b_{e} \quad \text{if} \quad \text{girder} = \text{"exterior"} \end{cases} \)

**Transformed Slab Properties**

Modular ratio

\( n := \frac{E_{s}}{E_{c}} = 0.65 \)

BDM 5.6.2.B.3

Slab transformed flange width

\( b_{e,\text{trans}} := b_{e} \cdot n = 50.94\,\text{in} \)

Slab moment of inertia (transformed)

\( I_{\text{slab}} := b_{e,\text{trans}} \cdot t_{s}^{3} / 12 = 1456.0\,\text{in}^{4} \)

Area of slab (transformed)

\( A_{\text{slab}} := b_{e,\text{trans}} \cdot t_{s} = 356.6\,\text{in}^{2} \)

c.g. of slab to bottom of girder

\( Y_{bs} := d_{g} + 0.5 \cdot t_{s} = 77.5\,\text{in} \)

**Composite Section**

c.g. to bottom of girder

\( Y_{b} := \frac{A_{\text{slab}} \cdot Y_{bs} + A_{g} \cdot Y_{bg}}{A_{\text{slab}} + A_{g}} = 47.31\,\text{in} \)

c.g. to top of girder

\( Y_{t} := d_{g} - Y_{b} = 26.69\,\text{in} \)

c.g. to top of slab

\( Y_{ts} := t_{s} + Y_{t} = 33.69\,\text{in} \)

Slab moment of inertia about composite N.A.

\( I_{\text{slabc}} := A_{\text{slab}} \cdot \left( Y_{ts} - 0.5 t_{s} \right)^{2} + I_{\text{slab}} = 326347\,\text{in}^{4} \)

Girder moment of inertia about composite N.A.

\( I_{g} := A_{g} \cdot \left( Y_{b} - Y_{bg} \right)^{2} + I_{g} = 859801\,\text{in}^{4} \)

Composite section moment of inertia

\( I_{c} := I_{\text{slabc}} + I_{g} = 1186148\,\text{in}^{4} \)

Section modulus to bottom of girder

\( S_{b} := I_{c} / Y_{b} = 25069\,\text{in}^{3} \)

Section modulus to top of girder

\( S_{t} := I_{c} / Y_{t} = 44450\,\text{in}^{3} \)

Section modulus to top of slab (modified by modular ratio to get stress for correct slab effective width)

\( S_{ts} := \frac{I_{c}}{Y_{ts} \cdot \left( 1 / n \right)} = 53919\,\text{in}^{3} \)
4. Loading and Limit State Parameters

4.1 Live Load Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formula</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle width</td>
<td>6 ft</td>
<td>LRFD 3.6.1.2</td>
</tr>
<tr>
<td>Lane load</td>
<td>0.64 kip/ft</td>
<td>LRFD 3.6.1.2.4</td>
</tr>
<tr>
<td>Number of Design Lanes</td>
<td>( N_L = \begin{cases} \text{floor} \left( \frac{BW}{12 \text{ ft}} \right) &amp; \text{if } BW &gt; 24 \text{ ft} = 3.0 \ 2 &amp; \text{if } 24 \text{ ft} \geq BW \geq 20 \text{ ft} \ 1 &amp; \text{otherwise} \end{cases} )</td>
<td>LRFD 3.6.1.1.1</td>
</tr>
<tr>
<td>Multiple Presence Factor</td>
<td>( m_p = \begin{cases} 1.20 &amp; \text{if } N_L = 1 \ 1.00 &amp; \text{if } N_L = 2 \ 0.85 &amp; \text{if } N_L = 3 \ 0.65 &amp; \text{otherwise} \end{cases} )</td>
<td>LRFD 3.6.1.1.2</td>
</tr>
</tbody>
</table>

4.2 Service Limit States

Limit states relating to stress, deformation, and crack width under regular service conditions. LRFD 5.5.2

Service I - Load combination relating to the normal operational use of the bridge. Compression in prestressed components is investigated using this load combination.

\[ 1.0 \, (DC + DW) + 1.0 \, (LL+IM) \]

Service III - Load combination relating only to tension in prestressed concrete superstructures with the objective of crack control.

\[ 1.0 \, (DC + DW) + 0.8 \, (LL+IM) \]

Notes:
1. Force effects due to temperature, shrinkage and creep, because of the free movement at end piers, are considered to be zero.
2. Force effects due to temperature gradient, wind, friction at bearings, and settlement are ignored.

Service III Limit State Live Load Factor \( C_{LLserIII} := 0.8 \)

4.3 Strength Limit States

Load Combinations LRFD 3.4.1
- Strength I load combination shall be satisfied in final operational condition.
- The force effects due to temperature shrinkage and creep are ignored.

Resistance factors BDM 5.2.4.B.1
- Tension-controlled precast/prestressed concrete \( \phi_f := 1.0 \)
- Precast/prestressed concrete - transition region \( \phi_{p\text{Trans}}(d_t, c) := 0.583 + 0.25 \left( \frac{d_t}{c} - 1 \right) \)
Concrete Structures Chapter 5

Compression-controlled concrete with spirals or ties

\[ \phi_c := 0.75 \]

Axial/Flexure for precast/prestressed concrete

\[ \phi_p(d_t, c) := \text{if} \left( c = 0, \phi_f, \max(\phi_p^{\text{Trans}}(d_t, c), \phi_f, \phi_c) \right) \]

Shear and torsion of normal weight concrete

\[ \phi_v := 0.90 \]

Load Factors

Dead load - Structure and Attachments

\[ \gamma_{DC} := 1.25 \]

Dead load - Wearing Surfaces and Utilities

\[ \gamma_{DW} := 1.5 \]

Live load

\[ \gamma_{LL} := 1.75 \]

Load Modifier

Ductility Factor

\[ \eta_D := 1.00 \]

Redundancy Factor

\[ \eta_R := 1.00 \]

Operational Importance Factor

\[ \eta_I := 1.00 \]

Load Modifier

\[ \eta := \max(\eta_D \eta_R \eta_I \cdot 0.95) = 1.0 \]

4.4 Fatigue Limit State

The compressive stress due to the Fatigue 1 load combination and one-half the sum of effective prestress and permanent loads shall not exceed 0.40 f_c after losses.

Fatigue 1 Limit State Live Load Factor

\[ \gamma_{LL,\text{fat}} := 1.5 \]
5. Dead and Live Load Force Effects

Define Sections for Computation of Forces and Stresses

Define the section locations along the girder length where moments, shears and stresses are to be computed:

Girder End

| P2 + 0.1L | 14.973 |
| P2 + 0.2L | 27.973 |
| P2 + 0.3L | 40.973 |
| P2 + 0.4L | 53.973 |

Midspan SE := P2 + 0.5L SE = 66.973 ft

| P2 + 0.6L | 79.973 |
| P2 + 0.7L | 92.973 |
| P2 + 0.8L | 105.973 |
| P2 + 0.9L | 118.973 |

Girder End

| GL | 133.946 |

Add Support, Harp, Critical Shear, Transfer, Lifting and Shipping Support Points to the Section Vector

Add these points only if they are not there already.

SE :=

Sections ← SE

ADD := (P2 GL P2 0.4GL 0.6GL P2 + d \text{est} GL - P2 - d \text{est} l_t GL - l_t L_L GL - L_L GL - L_T )

for j ∈ 1..cols(ADD)

Match ← 0

for i ∈ 1..rows(Sections)

Match ← 1 if Sections_i = ADD_{1,j}

Sections ← stack(Sections, ADD_{1,j}) if ¬Match

return Sections

Sort vector SE in ascending order

SE := sort(SE)

Find Row Numbers for Points of Interest

Row number of left support

rs_L := match(P2, SE)_1 = 2.0

Row number of right support

rs_R := match(GL - P2, SE)_1 = 22.0

Row number of left PS Transfer point

rp := match(l_t, SE)_1 = 3.0

Row number of left critical section for shear

rc := match(P2 + d \text{est}, SE)_1 = 5.0

Row number of left harp point

rh := match(0.4GL, SE)_1 = 10.0

SE = 118.973 ft

| 1 | 0.000 |
| 2 | 1.973 |
| 3 | 3.000 |
| 4 | 5.000 |
| 5 | 6.833 |
| 6 | 10.000 |
| 7 | 14.973 |
| 8 | 27.973 |
| 9 | 40.973 |
| 10 | 53.578 |
| 11 | 53.973 |
| 12 | 66.973 |
| 13 | 79.973 |
| 14 | 80.368 |
| 15 | 92.973 |
Row number of midspan \( \text{rm} := \text{match}(P2 + 0.5L, \text{SE})_1 = 12.0 \)

Row number of left lifting point \( \text{rl}_1 := \text{match}(L_1, \text{SE})_1 = 4.0 \)

Row number of right lifting point \( \text{rl}_2 := \text{match}(GL - L_1, \text{SE})_1 = 20.0 \)

Row number of left shipping (bunk) point \( \text{rb}_L := \text{match}(L_L, \text{SE})_1 = 6.0 \)

Row number of right shipping (bunk) point \( \text{rb}_R := \text{match}(GL - L_T, \text{SE})_1 = 18.0 \)

Range variable for rows of SE \( i := 1..\text{rows(SE)} \)

Functions for Shear and Moment

Function for moment on simple span with uniform load \( M_{\text{uniform}}(w, L, x) := \frac{w \cdot x}{2} (L - x) \)

Function for shear on simple span with uniform load \( V_{\text{uniform}}(w, L, x) := \frac{w \cdot (L - x)}{2} \)

Function for moment on simple span with point load \( M_{\text{point}}(P, a, L, x) := \begin{cases} 0 \text{kip-ft} & \text{if } x < 0 \text{ft } v x > L \\ P \cdot (L - x) & \text{if } 0 \text{ft} \leq a \leq x \\ P \cdot (L - a) & \text{if } x < a \leq L \end{cases} \)

Function for shear on simple span with point load. When \( a = x \), the positive value is returned.

\( V_{\text{point}}(P, a, L, x) := \begin{cases} 0 \text{kip} & \text{if } x < 0 \text{ft } v x > L \\ 0 \text{kip-ft} & \text{if } a < 0 \text{ft } v a > L \\ \frac{P \cdot a}{L} & \text{if } 0 \text{ft} \leq a < x \\ \frac{P \cdot (L - a)}{L} & \text{if } x \leq a \leq L \end{cases} \)

Function for moment on simple span with cantilevered ends with uniform load

\( w = \text{Uniform Load} \)
\( a = \text{Front cantilever length by left support} \)
\( b = \text{Back cantilever length by right support} \)
\( L = \text{Simple Span Length (between supports)} \)
\( x = \text{Location to determine moment measured from left (front) end} \)
\[ M_{\text{cant}}(w, a, b, L, x) := \begin{cases} 
\text{return } 0 \text{kip-ft} & \text{if } x < 0 \text{ft} \lor x > a + L + b \\
\text{return } -\frac{w \cdot x^2}{2} & \text{if } x \leq a \\
\text{return } \frac{w}{L} \cdot \frac{(a + L + b) \cdot \left( \frac{a + L + b}{2} - b \right) \cdot (x - a) - \frac{w \cdot x^2}{2}}{2} & \text{if } a < x < a + L \\
\text{return } -\frac{w \cdot (a + L + b - x)^2}{2} & \text{if } a + L \leq x 
\end{cases} \]

### 5.1 Dead Load - Girder

Moments when on span supports

\[ M^{(i)} := \begin{cases} 
\text{Mom}_i \leftarrow M_{\text{uniform}}(w_g \cdot L, SE_i - P2) & \text{for } i \in r_{S_L} \ldots r_{S_R} 
\end{cases} \]

Moments at Casting Yard (Release)

\[ M^{(1)} := \begin{cases} 
\text{Mom}_i \leftarrow M_{\text{uniform}}(w_g \cdot GL, SE_i) & \text{for } i \in 1 \ldots \text{rows(SE)} 
\end{cases} \]

Shears when on span supports

\[ V^{(i)} := \begin{cases} 
v_i \leftarrow V_{\text{uniform}}(w_g \cdot L, SE_i - P2) & \text{for } i \in r_{S_L} \ldots r_{S_R} 
\end{cases} \]

### 5.2 Dead Load - Intermediate Diaphragms

Number of Intermediate Diaphragms

\[ n_{\text{dia}} := \begin{cases} 
\text{return } 4 & \text{if } L > 160\text{ft} \\
\text{return } 3 & \text{if } 160\text{ft} \geq L > 120\text{ft} \\
\text{return } 2 & \text{if } 120\text{ft} \geq L > 80\text{ft} \\
\text{return } 1 & \text{if } 80\text{ft} \geq L > 40\text{ft} \\
0 & \text{otherwise} 
\end{cases} \]

Spacing of Intermediate Diaphragms along girder span

\[ \text{DiaSpacing} := \frac{L}{n_{\text{dia}} + 1} = 32.5 \text{ ft} \]

Intermediate Diaphragm Length

\[ \text{Dia}_L := \frac{S - b_w}{\cos(\theta_{sk})} = 82.99 \text{ in} \]

Approximate Weight of Intermediate Diaphragm

\[ \text{DiaWt} := \begin{cases} 
-w_c \cdot \text{Dia}_L \cdot t_{\text{dia}} \cdot h_{\text{dia}} & \text{if } \text{girder} = \text{"interior"} \\
-w_c \cdot \text{Dia}_L \cdot t_{\text{dia}} \cdot h_{\text{dia}} \cdot 0.5 & \text{if } \text{girder} = \text{"exterior"} 
\end{cases} \]

\[ \text{DiaWt} = 2.859 \text{ kip} \]
Moments

\[ M^{(2)} := \begin{cases} 
  \text{for } i \in r_{L..R} \\
  a \leftarrow 0 \text{ ft} \\
  \text{Mom}_{i} \leftarrow 0 \text{kip-ft} \\
  \text{for } j \in 1..n_{\text{dia}} \\
  a \leftarrow a + \text{DiaSpacing} \\
  \text{Mom}_{i} \leftarrow \text{Mom}_{i} + M_{\text{point}}(\text{DiaWt}, a, L, SE_{i} - P2) \\
\end{cases} \]

Shears

\[ V^{(2)} := \begin{cases} 
  \text{for } i \in r_{L..R} \\
  a \leftarrow 0 \text{ ft} \\
  v_{i} \leftarrow 0 \text{kip} \\
  \text{for } j \in 1..n_{\text{dia}} \\
  a \leftarrow a + \text{DiaSpacing} \\
  v_{i} \leftarrow v_{i} + V_{\text{point}}(\text{DiaWt}, a, L, SE_{i} - P2) \\
\end{cases} \]

### 5.3 Dead Load - Pad

The full effective pad ("A"-t) weight shall be applied over the full length of the girder.  

BDM 5.6.2.B.3.d

- Depth of slab pad is "A" dimension minus full deck thickness
  
  \[ t_{pu} := A - t_{s2} = 3.75 \text{-in} \]

- Weight of pad
  
  \[ w_{pu} := t_{pu} b_{f} \cdot w_{cs} = 0.198 \text{kip/ft} \]

Moments

\[ M^{(3)} := \begin{cases} 
  \text{for } i \in r_{L..R} \\
  \text{Mom}_{i} \leftarrow M_{\text{uniform}}(w_{pu}, L, SE_{i} - P2) \\
\end{cases} \]

Shears

\[ V^{(3)} := \begin{cases} 
  \text{for } i \in r_{L..R} \\
  v_{i} \leftarrow V_{\text{uniform}}(w_{pu}, L, SE_{i} - P2) \\
\end{cases} \]

### 5.4 Dead Load - Slab

- Weight of slab
  
  \[ w_{s} := \begin{cases} 
  t_{s2} S \cdot w_{cs} \text{ if girder } = \text{"interior"} \\
  t_{s2} \left( \frac{S}{2} + \frac{b_{f}}{2} \right) \left[ \frac{\alpha_{c} + \alpha_{f}}{2} \left( \text{overhang} - \frac{b_{f}}{2} \right) \right] \cdot w_{cs} \text{ if girder } = \text{"exterior"} \\
\end{cases} \]

\[ = 0.630 \text{kip/ft} \]
Moments

\[
M^{(\phi)} := \begin{cases} 
\text{for } i \in rs_L..rs_R \\
\text{Mom}_i \leftarrow M_{\text{uniform}}(w_s \cdot L, SE_i - P2) \text{ \text{Mom}}
\end{cases}
\]

Shears

\[
V^{(\phi)} := \begin{cases} 
\text{for } i \in rs_L..rs_R \\
v_i \leftarrow V_{\text{uniform}}(w_s \cdot L, SE_i - P2) \text{ \text{v}}
\end{cases}
\]

5.5 Dead Load - Barrier

Dead load of one traffic barrier is divided among a maximum of three girders. If the bridge has less than 6 girders, then the weight of the two barriers should be divided equally between all girders.

Weight of one 32" F shape traffic barrier is

\[
tb := 0.460 \text{kip} \div \text{ft}
\]

Weight of traffic barrier per girder

\[
w_b := \begin{cases} 
2 \frac{tb}{N_b} \text{ if } N_b < 6 = 0.153 \frac{\text{kip}}{\text{ft}} \\
\frac{tb}{3} \text{ otherwise}
\end{cases}
\]

The Functions below assumes the bridge is a simple span. If the weight of barrier is to be superimposed upon spans made continuous, the function must be modified.

Moments

\[
M^{(s)} := \begin{cases} 
\text{for } i \in rs_L..rs_R \\
\text{Mom}_i \leftarrow M_{\text{uniform}}(w_b \cdot L, SE_i - P2) \text{ \text{Mom}}
\end{cases}
\]

Shears

\[
v^{(s)} := \begin{cases} 
\text{for } i \in rs_L..rs_R \\
v_i \leftarrow V_{\text{uniform}}(w_b \cdot L, SE_i - P2) \text{ \text{v}}
\end{cases}
\]

5.6 Dead Load - Future Overlay

For deck protection system 1, include the weight of a future 2" HMA overlay

\[
w_o := \begin{cases} 
2\text{in} \cdot S \cdot 0.140\text{kcf} \text{ if girder = "interior"} = 0.152 \frac{\text{kip}}{\text{ft}} \\
2\text{in} \left(S + \text{overhang} - \text{cw}\right) \cdot 0.140\text{kcf} \text{ if girder = "exterior"}
\end{cases}
\]

The Functions below assume the bridge is a simple span. If the weight of future overlay is to be superimposed upon spans made continuous, the functions must be modified.

Moments

\[
M^{(o)} := \begin{cases} 
\text{for } i \in rs_L..rs_R \\
\text{Mom}_i \leftarrow M_{\text{uniform}}(w_o \cdot L, SE_i - P2) \text{ \text{Mom}}
\end{cases}
\]
Shears

\[ V^{i(\phi)} := \begin{cases} v_i & \text{for } i \in r_{L}..r_{R} \\ V_{\text{uniform}}(w_o \cdot L, SE_i - P^2) & \end{cases} \]

5.7 Live Load - AASHTO Design truck

Bending Moment

The following function finds the maximum moment due to an AASHTO HL93 Truck Load at a section a distance "x" along a simple span of length "L". A truck moving both directions is checked.

\[
\text{HL93TruckM}(x, L) := \left\{ \begin{array}{l}
\text{Axles} \leftarrow \begin{pmatrix} 8\text{kip} \\ 32\text{kip} \\ 32\text{kip} \end{pmatrix} \\
\text{Locations} \leftarrow \begin{pmatrix} 0\text{ft} \\ -14\text{ft} \\ -28\text{ft} \end{pmatrix} \\
\text{rows} \leftarrow \text{rows(Locations)} \\
\text{Loc} \leftarrow \text{Locations} \\
\text{Moment} \leftarrow 0\text{kip ft} \\
\text{while } \text{Loc}_{\text{rows}} \leq L \\
\text{for } i \leftarrow 1..\text{rows} \\
\quad M_i \leftarrow M_{\text{point}}(\text{Axles}_i, \text{Loc}_i, L, x) \\
\quad \text{Loc}_i \leftarrow \text{Loc}_i + 0.01\text{ft} \\
\quad \text{Moment} \leftarrow \max \left( \sum M, \text{Moment} \right) \\
\text{Loc} \leftarrow \text{Locations} \\
\text{x} \leftarrow L - x \\
\text{while } \text{Loc}_{\text{rows}} \leq L \\
\text{for } i \leftarrow 1..\text{rows} \\
\quad M_i \leftarrow M_{\text{point}}(\text{Axles}_i, \text{Loc}_i, L, x) \\
\quad \text{Loc}_i \leftarrow \text{Loc}_i + 0.01\text{ft} \\
\quad \text{Moment} \leftarrow \max \left( \sum M, \text{Moment} \right) \\
\text{Moment}
\end{array} \right.
\]

Range Variable for Graph

\[ z := 0\text{ft}, 10\text{ft}..L \]
Chapter 5 Concrete Structures

Maximum Bending Moments Along Span - HL93 Truck

Distance Along Span (ft)

Moments

\[ M(\gamma) := \begin{cases} \text{Mom}_i & \text{for } i \in r_{L..} \cdot r_{R} \\ \text{Mom}_i & \leftarrow \text{HL93TruckM}(SE_i - P_2, L) \\ \text{Mom} \end{cases} \]

Shear

The following function finds the maximum positive shear due to an AASHTO HL93 Truck Load at a section a distance "x" along a simple span of length "L".  

\[
\text{HL93TruckVP}(x, L) := \begin{cases} \text{8kip} & \text{Axles} \\ \text{32kip} & \text{32kip} \\ \text{0ft} & \text{Loc} \\ \text{-14ft} & \text{-28ft} \\ \text{rows} \leftarrow \text{rows} \text{(Loc)} \\ \text{Shear} \leftarrow 0\text{kip} \\ \text{while } \text{Loc}_{\text{rows}} \leq L \\ \text{for } i \in 1..\text{rows} \\ \text{V}_i \leftarrow \text{V}_{\text{point}}(\text{Axles}_i, \text{Loc}_i, L, x) \\ \text{Loc}_i \leftarrow \text{Loc}_i + 0.01\text{ft} \\ \text{Shear} \leftarrow \max \left( \sum \text{V}, \text{Shear} \right) \\ \text{Shear} \end{cases} \]

The following function finds the maximum negative shear due to an AASHTO HL93 Truck Load at a section a distance "x" along a simple span of length "L".  

LRFD 3.6.1.2.2
The maximum shear, positive on the first half and negative on the second half, will be returned by the following function. At the centerline of girder, only the positive value is returned.

\[
\chi_i^{(2)} = \begin{cases} 
\text{for } i \in r_{L}..r_{R} \\
\begin{cases} 
\text{if } \left( SE_i \leq \frac{GL}{2} \right) \text{, } \text{HL93TruckVP}(SE_i - P2, L) \text{, } \text{HL93TruckVN}(SE_i - P2, L) 
\end{cases}
\end{cases}
\]

\[
v_i \leftarrow \begin{cases} 
\text{if } \left( SE_i \leq \frac{GL}{2} \right) \text{, } \text{HL93TruckVP}(SE_i - P2, L) \text{, } \text{HL93TruckVN}(SE_i - P2, L) 
\end{cases}
\]

### 5.8 Live Load - AASHTO Tandem

**Bending Moment**

The following function finds the maximum moment due to an AASHTO HL93 Tandem Load at a section \( \varepsilon \).  

LRFD 3.6.1.2.3
distance "x" along a simple span of length "L".

\[ \text{HL93TandemM}(x, L) := \begin{cases} 
\text{Axles} & \begin{pmatrix} 25 \text{kip} \\ 25 \text{kip} \end{pmatrix} \\
\text{Locations} & \begin{pmatrix} 0 \text{ft} \\ -4 \text{ft} \end{pmatrix} \\
\text{rows} & \text{rows}(\text{Locations}) \\
\text{Moment} & 0 \text{kip} \cdot \text{ft} \\
\text{while} & \text{Locations}_{\text{rows}} \leq L \\
\text{for} & i \in 1..\text{rows} \\
M_i & \leftarrow M_{\text{point}}(\text{Axles}_i, \text{Locations}_i, L, x) \\
\text{Locations}_i & \leftarrow \text{Locations}_i + 0.01 \text{ft} \\
\text{Moment} & \leftarrow \max \left( \sum M_i, \text{Moment} \right)
\end{cases} \]

**Maximum Bending Moments Along Span - HL93 Tandem**

**Shear**

The following function finds the maximum positive shear due to an AASHTO HL93 Tandem Load at a section a distance "x" along a simple span of length "L".
The following function finds the maximum negative shear due to an AASHTO HL93 Tandem Load at a section a distance "x" along a simple span of length "L".

**HL93TandemVP(x, L)** :=

\[
\text{Axles} \leftarrow \begin{pmatrix} 25 \text{kip} \\ 25 \text{kip} \end{pmatrix},
\]

\[
\text{Locations} \leftarrow \begin{pmatrix} 0 \text{ft} \\ -4 \text{ft} \end{pmatrix},
\]

\[
\text{rows} \leftarrow \text{rows}(\text{Locations}),
\]

\[
\text{Shear} \leftarrow 0 \text{kip},
\]

\[
\text{while } \text{Locations}_\text{rows} \leq L
\]

\[
\text{for } i \in 1..\text{rows}
\]

\[
V_i \leftarrow V_{\text{point}}(\text{Axles}_i, \text{Locations}_i, L, x)
\]

\[
\text{Locations}_i \leftarrow \text{Locations}_i + 0.01 \text{ft},
\]

\[
\text{Shear} \leftarrow \max\left(\sum V, \text{Shear}\right)
\]

\[
\text{Shear}
\]

**LRFD 3.6.1.2.3**

**HL93TandemVN(x, L)** :=

\[
\text{Axles} \leftarrow \begin{pmatrix} 25 \text{kip} \\ 25 \text{kip} \end{pmatrix},
\]

\[
\text{Locations} \leftarrow \begin{pmatrix} 0 \text{ft} \\ -4 \text{ft} \end{pmatrix},
\]

\[
\text{rows} \leftarrow \text{rows}(\text{Locations}),
\]

\[
\text{Shear} \leftarrow 0 \text{kip},
\]

\[
\text{while } \text{Locations}_\text{rows} \leq L
\]

\[
\text{for } i \in 1..\text{rows}
\]

\[
V_i \leftarrow V_{\text{point}}(\text{Axles}_i, \text{Locations}_i, L, x)
\]

\[
\text{Locations}_i \leftarrow \text{Locations}_i + 0.01 \text{ft},
\]

\[
\text{Shear} \leftarrow \min\left(\sum V, \text{Shear}\right)
\]

\[
\text{Shear}
\]
Chapter 5 Concrete Structures

The maximum shear, positive on the first half and negative on the second half, will be returned by the following function. At the centerline of girder, only the positive value is returned.

\[ v^{(g)} \left[ \frac{\text{kip}}{\text{kip}} \right] := \begin{cases} v_i & \text{for } i \in rs_{L..rs_R} \\ v & \text{for all other points} \end{cases} \]

\[ v_i \leftarrow \begin{cases} \text{if } SE_i \leq \frac{GL}{2} \cdot \text{HL93TandemVP} \left( SE_i - P2, L \right), \text{HL93TandemVN} \left( SE_i - P2, L \right) & \text{max } v^{(g)} \\ \text{otherwise} & \text{min } v^{(g)} \end{cases} \]

5.9 Live Load - AASHTO Lane Load

Moments

\[ M^{(q)} \left[ \frac{\text{kip} \cdot \text{ft}}{\text{kip} \cdot \text{ft}} \right] := \begin{cases} \text{Mom}_i & \text{for } i \in rs_{L..rs_R} \\ \text{Mom} & \text{for all other points} \end{cases} \]

\[ \text{Mom}_i \leftarrow \text{Mom}_{\text{uniform}} \left[ \frac{w_{\text{lane}}}{L}, SE_i - P2 \right] \]

Maximum positive shear at a point on the span occurs when the lane load occupies the part of the span to the right of that point. Maximum negative shear at a point on the span occurs when the lane load occupies the part of the span to the left of that point.

Shears

\[ v^{(q)} \left[ \frac{\text{kip}}{\text{kip}} \right] := \begin{cases} v_i & \text{for } i \in rs_{L..rs_R} \\ v & \text{for all other points} \end{cases} \]

\[ v_i \leftarrow \begin{cases} \text{if } SE_i \leq \frac{GL}{2} \cdot \frac{w_{\text{lane}} \left[ \frac{L - \left( SE_i - P2 \right)}{2 \cdot L} \right]^2}{2 \cdot L} & \text{max } v^{(q)} \\ \text{otherwise} & \text{min } v^{(q)} \end{cases} \]

5.10 Maximum Live Load including Dynamic Load Allowance, per lane

The dynamic load allowance shall not applied to pedestrian loads or to the design lane load.

Dynamic Load All. for all limit states except Fatigue  \[ IM := 33\% \]

LRFD 3.6.2.1
Concrete Structures Chapter 5

Moments

\[
M^{(10)}_{\text{\text{Mom}}} := \begin{cases} 
\text{Mom}_i \leftarrow \max \left( M_{i, 7}, M_{i, 8} \right) \cdot (1 + \text{IM}) + M_{i, 9} \\
\text{Mom} 
\end{cases}
\]

The maximum shear, positive on the first half and negative on the second half, will be returned by the following function. At the centerline of girder, only the positive value is returned.

\[
V^{(10)}_{\text{\text{Mom}}} := \begin{cases} 
V_i \leftarrow \text{SE}_i \cdot \max \left( V_{i, 7}, V_{i, 8} \right) \cdot (1 + \text{IM}) + V_{i, 9}, \min \left( V_{i, 7}, V_{i, 8} \right) \cdot (1 + \text{IM}) + V_{i, 9} \\
\end{cases}
\]

Create final row with zeros for shear matrix

\[
V_{\text{\text{rows(SE)}}, 1} := 0 \text{kip}
\]

5.11 Fatigue Live Load

Bending Moment

The following function finds the maximum moment due to an AASHTO Fatigue Truck Load with 30 foot LRFD 3.6.1.4 spacing between 32kip axles at a section a distance "x" along a simple span of length "L". A truck moving both directions is checked.

\[
\text{HL93TruckMFat}(x, L) := \begin{aligned}
\text{Axles} &\leftarrow \begin{cases} 
8 \text{kip} \\
32 \text{kip} \\
32 \text{kip}
\end{cases} \\
\text{Locations} &\leftarrow \begin{cases} 
0 \text{ft} \\
-14 \text{ft} \\
-44 \text{ft}
\end{cases} \\
\text{rows} &\leftarrow \text{rows(\text{Locations})} \\
\text{Loc} &\leftarrow \text{Locations} \\
\text{Moment} &\leftarrow 0 \text{kip-ft} \\
\text{while} &\left\{ \text{Loc}_{\text{rows}} \leq L \right\} \\
&\left\{ \text{for} \ i \in 1.. \text{rows} \\
&M_i \leftarrow \text{M}_{\text{point}}(\text{Axles}_i, \text{Loc}_i, L, x) \\
&\text{Loc}_i \leftarrow \text{Loc}_i + 0.01 \text{ft} \\
&\text{Moment} \leftarrow \max \left( \sum \text{M}, \text{Moment} \right) \\
\text{Loc} &\leftarrow \text{Locations} \\
x &\leftarrow L - x \\
\text{while} &\left\{ \text{Loc}_{\text{rows}} \leq L \right\} \\
&\left\{ \text{for} \ i \in 1.. \text{rows} \\
&M_i \leftarrow \text{M}_{\text{point}}(\text{Axles}_i, \text{Loc}_i, L, x) \\
\end{cases}
\end{aligned}
\]
Mom \leftarrow \text{HL93TruckMFat}(z, L) \times \text{SE}

\begin{align*}
\text{Mom} &:= 0 \text{kip-ft} \\
M_{\text{FAT}} &:= \left\{ \begin{array}{ll}
0 & \text{for } i \in [r_L, r_R] \\
\text{HL93TruckMFat}(\text{SE}, L) & \text{otherwise}
\end{array} \right.
\end{align*}

\begin{align*}
M_{\text{FAT}} &= 15 \text{kip-ft} \\
\text{IM}_{\text{FAT}} &= 15\%
\end{align*}

Dynamic Load for Fatigue limit state

5.12 Summary of Moments and Shears
Concrete Structures

Chapter 5

| Row number of left support | \( r_s_L = 2.0 \) | Column 1 = Dead Load of Girder between supports after erection |
| Row number of right support | \( r_s_R = 22.0 \) | Column 2 = Dead Load of Intermediate Diaphragms |
| Row number of left PS Transfer point | \( r_p = 3.0 \) | Column 3 = Dead Load of Pad |
| Row number of left critical section for shear | \( r_c = 5.0 \) | Column 4 = Dead Load of Slab |
| Row number of left harp point | \( r_h = 10.0 \) | Column 5 = Dead Load of Barrier |
| Row number of midspan | \( r_m = 12.0 \) | Column 6 = Dead Load of Future Overlay |
| Row number of left lifting point | \( r_l_1 = 4.0 \) | Column 7 = Live Load of AASHTO Design Truck |
| Row number of right lifting point | \( r_l_2 = 20.0 \) | Column 8 = Live Load of AASHTO Design Tandem |
| Row number of left shipping (bunk) point | \( r_b_L = 6.0 \) | Column 9 = Live Load of AASHTO Design Lane |
| Row number of right shipping (bunk) point | \( r_b_R = 18.0 \) | Column 10 = Maximum Live Load Effect including Impact, per lane |
| Column 11 = Dead Load of Girder between ends after release |

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\[ M = \text{kip-ft} \]

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5.13 Moment Distribution of Live Load

Applicability for use of Live Load Distribution Factors

For typical cross section, use case k

- Width of deck is constant
- Beams are parallel
- Beams have approximately the same stiffness
- Curvature in plan is less than the limit specified in LRFD 4.6.1.2.4

Multiple presence factors shall not be applied except for exterior girders with special requirement.

\[ d_{\text{bar}} := \text{overhang} - \text{cw} = 2.75 \text{ ft} \]

Roadway overhang check

\[ \text{chk}, 1 := \text{if} \left( d_{\text{bar}} \leq 3 \text{ ft}, \text{"OK"}, \text{"NG"} \right) = \text{"OK"} \]

Minimum beam count check

\[ \text{chk}, 2 := \text{if} \left( N_h \geq 4, \text{"OK"}, \text{"NG"} \right) = \text{"OK"} \]

Distance between the centers of gravity of the basic beam and deck

\[ e_g := Y_{bs} - Y_{bg} = 41.84 \text{ in} \]

Longitudinal stiffness parameter

\[ K_g := \frac{1}{n} \left( I_g + A_g e_g^2 \right) = 3599953 \text{ in}^4 \]

Distribution Factor (DF) for Moment on interior girder

LRFD 4.6.2.2b
Concrete Structures Chapter 5

Girder spacing check

\[ \text{chk}_{53} := \text{if } (3.5 \text{ ft} \leq S \leq 16.0 \text{ ft}, "OK", "NG") = "OK" \]

Slab thickness check

\[ \text{chk}_{54} := \text{if } (4.5 \text{ in} \leq t_s \leq 12.0 \text{ in}, "OK", "NG") = "OK" \]

Beam span check

\[ \text{chk}_{55} := \text{if } (20 \text{ ft} \leq L \leq 240 \text{ ft}, "OK", "NG") = "OK" \]

Longitudinal stiffness parameter check

\[ \text{chk}_{56} := \text{if } (10^4 \text{ in}^4 \leq K_g \leq 7 \cdot 10^6 \text{ in}^4, "OK", "NG") = "OK" \]

DF for interior girder

\[
DF_1 := \begin{cases} 
0.075 + \left( \frac{S}{9.5 \text{ ft}} \right)^{0.6} \cdot \left( \frac{S}{L} \right)^{0.2} \cdot \left( \frac{K_g}{L \cdot t_s^3} \right)^{0.1} & \text{if } N_L > 1 = 0.604 \\
0.06 + \left( \frac{S}{14 \text{ ft}} \right)^{0.4} \cdot \left( \frac{S}{L} \right)^{0.3} \cdot \left( \frac{K_g}{L \cdot t_s^3} \right)^{0.1} & \text{if } N_L = 1
\end{cases}
\]

Distribution Factor (DF) for Moment on exterior girder

For exterior girder design with slab cantilever length equal or less than one-half of the adjacent interior girder spacing, use the live load distribution factor for interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.

For exterior girder design with slab cantilever length exceeding one-half of the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall not be less than the live load used for the adjacent interior girder.

Minimum distance to curb from LL wheel

\[ \text{curb}_{\text{min.sp}} := 2 \text{ ft} \]

\[
x \leftarrow S + d_{\text{bar}} - \text{curb}_{\text{min.sp}} \\
\text{Numerator} \leftarrow 0 \text{ ft} \\
\text{UseAxleWidth} \leftarrow 1 \\
\text{while } x > 0 \text{ ft} \\
\quad \text{Numerator} \leftarrow \text{Numerator} + x \\
\quad \text{if } \text{UseAxleWidth} \\
\quad \quad x \leftarrow x - \text{axlewidth} \\
\quad \quad \text{UseAxleWidth} \leftarrow 0 \\
\quad \text{otherwise} \\
\quad x \leftarrow x - (12 \cdot \text{ft} - \text{axlewidth}) \\
\quad \text{UseAxleWidth} \leftarrow 1
\]

Lever rule distribution

\[ DF_{\text{lever}} := \frac{\text{Numerator}}{2 \cdot S} = 0.654 \]

DF for exterior girder

\[ DF_e := \begin{cases} 
DF_1 & \text{if overhang } \leq 0.5 \cdot S = 0.654 \\
\max(DF_{\text{lever}}, DF_1) & \text{otherwise}
\end{cases} \]

Reduction in Moment DF for Skewed Bridges (LRFD 4.6.2.2e, case k)
Chapter 5 Concrete Structures

**Note:** Applied when the difference between skew angles of two adjacent lines of support $\leq 10$ deg. LRFD 4.6.2.2.2e

Check on skew angle

$chk_{53,7} := \text{if } \left(30 \text{ deg} \leq \theta_{sk} \leq 60 \text{ deg}, "OK", "NG" \right) = "OK"$

Check on girder spacing

$chk_{53,8} := \text{if } \left(3.5\text{-ft} \leq S \leq 16.0\text{-ft}, "OK", "NG" \right) = "OK"$

Check on girder span

$chk_{53,9} := \text{if } \left(20\text{-ft} \leq L \leq 240\text{-ft}, "OK", "NG" \right) = "OK"$

Check on girder count

$chk_{53,10} := \text{if } \left(N_{b} \geq 4, "OK", "NG" \right) = "OK"$

Parameters for skew equation

$c_{1} := \begin{cases} 0.0 & \text{if } \theta_{sk} < 30 \text{ deg} \\ 0.25 \left(\frac{K_{g}}{S/L} - \frac{t_{s}^{3}}{L} \right) & \text{otherwise} \end{cases} = 0.090$

Reduction Factor for skew

$SK := 1 - c_{1} \left(\tan\left(\min\left(\theta_{sk}, 60\text{deg}\right)\right)\right)^{1.5} = 0.961$

Reduced DF for moment

$DF := \begin{cases} SK \cdot DF_{i} & \text{if } \text{girder = "interior"} \\ SK \cdot DF_{e} & \text{if } \text{girder = "exterior"} \end{cases} = 0.580$

Distribution Factor for Fatigue Load LRFD 3.6.1.4.3b

LRFD 3.6.1.1.2

$DF_{iFAT} := \frac{0.06 + \left(\frac{S}{14\text{-ft}}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_{g}}{L \cdot t_{s}^{3}}\right)^{0.1}}{1.2} = 0.352$

DF for interior girder (one lane loaded)

$DF_{eFAT} := \begin{cases} DF_{iFAT} & \text{if } \text{overhang} \leq 0.5S \\ \max\left(DF_{lever}, DF_{iFAT}\right) & \text{otherwise} \end{cases} = 0.654$

DF for exterior girder

Reduced DF for moment - Fatigue Loading

$DF_{FAT} := \begin{cases} SK \cdot DF_{iFAT} & \text{if } \text{girder = "interior"} \\ SK \cdot DF_{eFAT} & \text{if } \text{girder = "exterior"} \end{cases} = 0.338$

5.14 Shear Distribution of Live Load

Distribution Factor (DF) Method for Shear on interior girder LRFD 4.6.2.2.3a

Range of applicability (LRFD Table 4.6.2.2.3a-1), case k checks are the same as those for moment so checks above are sufficient.

Shear LL distribution factor - interior girder

$DF_{vi} := \max\left[0.36 + \frac{S}{25\text{-ft}}, 0.2 + \frac{S}{12\text{-ft}}, \left(\frac{S}{35\text{-ft}}\right)^{2.0}\right] = 0.707$

Distribution Factor (DF) Method for Shear on exterior girder BDM 3.9.4.A
For exterior girder design with slab cantilever length equal or less than one-half of the adjacent interior girder spacing, use the live load distribution factor for interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.

For exterior girder design with slab cantilever length exceeding one-half of the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall not be less than the live load used for the adjacent interior girder.

\[
DF_{ve} := \begin{cases} 
DF_{vi} & \text{if overhang } \leq 0.5\ S \\
\max(DF_{lever}, DF_{vi}) & \text{otherwise}
\end{cases} = 0.707
\]

**Correction Factor for Shear DF for Skewed Bridges**  

LRFD 4.6.2.2.3c

Check on skew angle - other checks for range of applicability are same as for moment with skew

\[
\text{chk}_{sk} := \begin{cases} 
(0\ \text{deg} \leq \theta_{sk} \leq 60\ \text{deg}, \text{"OK"}) & \text{if } 0\ \text{deg} \leq \theta_{sk} \leq 60\ \text{deg} \\
\text{"NG"} & \text{otherwise}
\end{cases}
\]

Skew Correction Factor - Shear

\[
SK_v := 1.0 + 0.20\left(\frac{L \cdot t_s}{K_g}\right)^{0.3} \tan(\theta_{sk}) = 1.065
\]

Distribution Factor for Shear

\[
DF_v := \begin{cases} 
SK_v \cdot DF_{vi} & \text{if girder } = \text{"interior"} \\
SK_v \cdot DF_{ve} & \text{if girder } = \text{"exterior"}
\end{cases} = 0.753
\]
6. Computation of Stresses for Dead and Live Loads

6.1 Summary of Stresses

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<tr>
<th>Component</th>
<th>Noncomposite</th>
<th>Composite</th>
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<tr>
<td>S - top of slab</td>
<td>S_{ts} = 53919-in³</td>
<td></td>
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<tr>
<td>S - top of girder</td>
<td>S_{tg} = 19154-in³</td>
<td>S_{t} = 44450-in³</td>
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<tr>
<td>S - bottom of girder</td>
<td>S_{bg} = 20593-in³</td>
<td>S_{b} = 25069-in³</td>
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Columns 1-4 and 11 act upon the noncomposite section
Columns 5-6 act upon the composite section
Columns 7-10 act upon the composite section and are multiplied by the distribution factor

Negative stress indicates compression.

Stress at the top of the girder: 
Stress at the bottom of the girder: 
Stress at the top of the CIP Slab:
### Concrete Structures

#### Chapter 5

**ST**: for \( j \in 1..4 \)

- for \( i \in 1..\text{rows(SE)} \)
  - \( \text{Stress}_{i,j} \leftarrow \frac{M_{i,j}}{S_{tg}} \)

- for \( j \in 5..6 \)
  - for \( i \in 1..\text{rows(SE)} \)
    - \( \text{Stress}_{i,j} \leftarrow \frac{M_{i,j}}{S_t} \)

- for \( j \in 7..10 \)
  - for \( i \in 1..\text{rows(SE)} \)
    - \( \text{Stress}_{i,j} \leftarrow \frac{M_{i,j} \cdot \text{DF}}{S_t} \)

**SB**: for \( j \in 1..4 \)

- for \( i \in 1..\text{rows(SE)} \)
  - \( \text{Stress}_{i,j} \leftarrow \frac{M_{i,j}}{S_{bg}} \)

**SS**: for \( j \in 1..4 \)

- for \( i \in 1..\text{rows(SE)} \)
  - \( \text{Stress}_{i,j} \leftarrow \frac{M_{i,j}}{S_{bg}} \)

**Stress**

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**ST** = Stress, **SB** = Stress, **SS** = Stress

\( S_t \) for Stress in kpsi

\( S_{tg} \) for Stress in kpsi

\( S_{ts} \) for Stress in kpsi

\( S_{bg} \) for Stress in kpsi

\( M_{i,j} \) for Stress in kpsi

\( DF \) for Stress in kpsi
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7. Prestressing Forces and Stresses

7.1 Stress Limits for Prestressing Strands

- Service limit state after all losses
  \[ f_{p,e}\text{.lim} := 0.80 \cdot f_{p,y} = 194.4\text{-ksi} \]

- Stress limit immediately prior to transfer (after relaxation losses prior to transfer)
  \[ f_{p,bt}\text{.lim} := 0.75 \cdot f_{p,u} = 202.5\text{-ksi} \]

- Initial stress in PS at jacking. WSDOT practice is to set the jacking force equal to the AASHTO limit immediately prior to transfer.
  \[ f_{pj} := f_{p,bt}\text{.lim} = 202.5\text{-ksi} \]

7.2 Allowable Concrete Stresses at Service Limit State

- Compressive Stress Limits in PS Concrete After PS Losses
  \[ f_{c,TL}\text{.lim} := -0.65 \cdot f'_{c} = -4.875\text{-ksi} \]
  \[ f_{c,SH}\text{.lim} := -0.65 \cdot f'_{c} = -5.525\text{-ksi} \]
  \[ f_{c,PP}\text{.lim} := -0.45 \cdot f'_{c} = -3.825\text{-ksi} \]
  \[ f_{c,PPT}\text{.lim} := -0.60 \cdot f'_{c} = -5.100\text{-ksi} \]
  \[ f_{c,FA}\text{.lim} := -0.40 \cdot f'_{c} = -3.400\text{-ksi} \]

- Tensile Stress Limits in PS Concrete
  \[ f_{t,TL}\text{.lim} := 0.19 \cdot \sqrt{f'_{c} \cdot \text{ksi}} = 0.520\text{-ksi} \]
  \[ f_{t,SP}\text{.lim} := 0.19 \cdot \sqrt{f'_{c} \cdot \text{ksi}} = 0.554\text{-ksi} \]
  \[ f_{t,SL}\text{.lim} := 0.24 \cdot \sqrt{f'_{c} \cdot \text{ksi}} = 0.700\text{-ksi} \]
  \[ f_{t,PCT}\text{.lim} := 0\text{-ksi} \]

7.3 Jacking Forces

- Jacking force for straight strands
  \[ P_{js} := f_{pj} \cdot N_{s} \cdot A_{p} = 1142.5\text{-kip} \]

- Jacking force for harped strands
  \[ P_{jh} := f_{pj} \cdot N_{h} \cdot A_{p} = 527.3\text{-kip} \]
Jacking force for temporary strands \[ P_{jt} := f_{pj} N_t A_p = 263.7 \text{kip} \]
Total jacking force \[ P_{jack} := P_{jh} + P_{js} + P_{jt} = 1933.5 \text{kip} \]

### 7.4 C.G. of Prestress

Final number of permanent prestress strands \[ N_p := N_s + N_h = 38 \]
Total area of permanent prestress strands \[ A_{ps} := A_p N_p = 8.246 \text{in}^2 \]
Area of temporary strands \[ A_{temp} := A_p N_t = 1.302 \text{in}^2 \]
Area of final plus temporary strands \[ A_{pstem} := A_p (N_t + N_p) = 9.548 \text{in}^2 \]

c.g. to straight strands from bottom of girder, \( E \) (check specific girder pattern in BDM girder details - WF girder shown)

\[
E :=
\begin{align*}
&\frac{4 \text{in}}{N_s} & \text{if } N_s \leq 2 \\
&\frac{2 \cdot 4 \text{in} + (N_s - 2) \cdot 2 \text{in}}{N_s} & \text{if } 3 \leq N_s \leq 6 \\
&\frac{4 \cdot 2 \text{in} + (N_s - 4) \cdot 4 \text{in}}{N_s} & \text{if } 7 \leq N_s \leq 8 \\
&\frac{4 \cdot 4 \text{in} + (N_s - 4) \cdot 2 \text{in}}{N_s} & \text{if } 9 \leq N_s \leq 20 \\
&\frac{16 \cdot 2 \text{in} + (N_s - 16) \cdot 4 \text{in}}{N_s} & \text{if } 21 \leq N_s \leq 32 \\
&\frac{16 \cdot 2 \text{in} + 16 \cdot 4 \text{in} + (N_s - 32) \cdot 6 \text{in}}{N_s} & \text{if } 33 \leq N_s \leq 42 \\
&\frac{16 \cdot 2 \text{in} + 16 \cdot 4 \text{in} + 10 \cdot 6 \text{in} + (N_s - 42) \cdot 8 \text{in}}{N_s} & \text{if } 43 \leq N_s \leq 46 \\
&\text{"error" otherwise}
\end{align*}
\]

\[ E = 2.769 \text{in} \]

c.g. of straight strands to c.g. of girder \[ e_s := Y_{bg} - E = 32.891 \text{in} \]
c.g. of temporary strands to c.g. of girder \[ e_{temp} := 2 \text{in} - Y_{tg} = -36.340 \text{in} \]

**Eccentricity for harped strand at Midspan**

c.g. to harped strands from bottom of girder, \( F_{CL} \), at midspan \[ F_{CL} := 4 \text{in} \]
Minimum $F_{CL} \text{ per construction constraints}$

$$F_{CL, \text{lim}} := \begin{cases} 4 \text{in} & \text{if } 1 \leq N_h \leq 12 \\ 12 \cdot 4 \text{in} + \left( N_h - 12 \right) \cdot 6 \text{in} & \text{if } 13 \leq N_h \leq 24 \\ 12 \cdot 4 \text{in} + 12 \cdot 6 \text{in} + \left( N_h - 24 \right) \cdot 8 \text{in} & \text{if } 25 \leq N_h \leq 36 \\ \text{“error” otherwise} \end{cases}$$

$$F_{CL, \text{lim}} = 4.000 \text{ in} \quad \text{BDM 5.1.3.C.2}$$

Check if $F_{CL}$ is too close to bottom of girder

$$\text{chk}_{1, 1} := \text{if} \left(F_{CL} \geq F_{CL, \text{lim}}, \text{"OK"}, \text{"NG"} \right) = \text{"OK"}$$

**Eccentricity for harped strand at end of girder**

Distance from the top of girder to the c.g. of the harped strands at the end of girder

$$F_o := 9 \text{in}$$

Limit to how close $F_o$ may be to top of girder per strand pattern shown in the standard plans

$$F_{o, \text{lim}} := \begin{cases} \text{increment} \leftarrow 1 & = 9.000 \text{ in} \\ e \leftarrow 2 \text{in} & \text{for } i \in 1..N_h \\ \text{if increment} \\ \text{increment} \leftarrow 0 \\ e \leftarrow e + 2 \text{in} & \text{increment} \leftarrow 1 \text{ otherwise} \\ \text{Product} \leftarrow \text{Product} + e \\ \text{return} \frac{\text{Product}}{N_h} \end{cases}$$

Check if $F_o$ is too close to top of girder

$$\text{chk}_{1, 2} := \text{if} \left(F_o \geq F_{o, \text{lim}}, \text{"OK"}, \text{"NG"} \right) = \text{"OK"}$$

**Strand Eccentricity Table**

Harped strand slope for c.g. of strands

$$\text{slope}_h := \frac{d - F_o - F_{CL}}{x_h} = 0.094877$$

Maximum slope on individual strand

$$\text{maxslope}_h := \frac{d - F_o - F_{CL} + \left(F_{o, \text{lim}} - 4\text{in}\right)}{x_h} = 0.1027$$

Limit for maximum slope on individual strand

$$\text{slope}_{\text{lim}} := \begin{cases} \frac{1}{6} & \text{if } d_b = 0.5\text{in} \quad = 0.1250 \quad \text{BDM 5.1.3.C.2} \\ \frac{1}{8} & \text{if } d_b = 0.6\text{in} \\ \text{“error” otherwise} \end{cases}$$

Check slope of harped strands

$$\text{chk}_{1, 3} := \text{if} \left(\text{maxslope}_h \leq \text{slope}_{\text{lim}}, \text{"OK"}, \text{"NG"} \right) = \text{"OK"}$$
Holddown force at jacking (for shop drawing check)

\[ P_{hd} := P_{jh} \sin(\text{atan}(\text{slope}_h)) = 49.8 \text{ kip} \]

Eccentricity of harped, total permanent, and total permanent + temporary strands at each girder section. Measured from girder neutral axis (positive toward bottom of girder)

\begin{align*}
\text{EC} &= \text{for } i \in 1..\text{rows(SE)} \nonumber \\
\text{EC}_{i,1} &\leftarrow Y_{bg} - [F_{CL} + \text{slope}_h(x_h - SE_i)] \text{ if } SE_i < x_h \\
\text{EC}_{i,1} &\leftarrow Y_{bg} - F_{CL} \text{ if } x_h \leq SE_i \leq GL - x_h \\
\text{EC}_{i,1} &\leftarrow Y_{bg} - [F_{CL} + \text{slope}_h(SE_i + x_h - GL)] \text{ if } SE_i > GL - x_h \\
\text{EC}_{i,2} &\leftarrow \frac{e_s N_s + EC_{i,1} \cdot N_h}{N_p} \\
\text{EC}_{i,3} &\leftarrow \frac{EC_{i,2} \cdot N_p + e_{\text{temp}} N_t}{N_p + N_t}
\end{align*}

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7.5 Loss of Prestress

**Stress in strands before prestress transfer**

Time at transfer

\[ t_0 := 1\text{ day} \]

\[ \Delta f_{pR0} := -\log\left(\frac{24.0 - t_0}{\text{day}}\right) \left(\frac{f_{pj}}{f_{py}} - 0.55\right) f_{pj} \]

\[ \Delta f_{pR0} := \text{BDM 5.1.4.D} \]
\[ \Delta f_{pR0} = -1.980 \text{ ksi} \]

Prestress immediately before transfer
\[ f_{pbt} := f_{pj} + \Delta f_{pR0} = 200.520 \text{ ksi} \]

**Initial Loss due to Elastic Shortening at Midspan**

Estimate of elastic shortening in permanent strands immediately after transfer
\[ x_1 := -(f_{pbt} - 0.7 \cdot f_{pu}) = -11.5 \text{ ksi} \]

BDM 5.1.4.A.1

Estimate of elastic shortening in temporary strands immediately after transfer
\[ x_2 := -(f_{pbt} - 0.7 \cdot f_{pu}) = -11.5 \text{ ksi} \]

BDM 5.1.4.A.1

Estimate of total prestressing force P
\[ P := A_{ps} (f_{pbt} + x_1) + A_{temp} (f_{pbt} + x_2) = 1804.6 \text{ kip} \]

Solve Block for prestress after elastic shortening
Given
LRFD 5.9.5.2.3a

Total prestressing force P
\[ P = A_{ps} (f_{pbt} + x_1) + A_{temp} (f_{pbt} + x_2) \]

Elastic shortening in perm. strands immediately after transfer
\[ x_1 = \left[ \frac{E_p}{E_{ci}} \left( \frac{P}{A_g} + \frac{P \cdot EC_{rm,3} \cdot EC_{rm,2}}{I_g} - \frac{M_{rm,11} \cdot EC_{rm,2}}{I_g} \right) \right] \]

Elastic shortening in temp. strands immediately after transfer
\[ x_2 = \left[ \frac{E_p}{E_{ci}} \left( \frac{P}{A_g} + \frac{P \cdot EC_{rm,3} \cdot e_{temp}}{I_g} - \frac{M_{rm,11} \cdot e_{temp}}{I_g} \right) \right] \]

Solve for the 3 unknowns in the 3 equations above
\[ \begin{cases} P_{ps} \\ \Delta f_{pES} := \text{Find}(P, x_1, x_2) \\ \Delta f_{pEST} \end{cases} \]

Stress in prestress strands immediately after transfer
\[ P_{ps} = 1798.2 \text{ kip} \]

Initial loss in perm. strands due to elastic shortening
\[ \Delta f_{pES} = -13.055 \text{ ksi} \]

Initial loss in temp. strands due to elastic shortening
\[ \Delta f_{pEST} = -6.716 \text{ ksi} \]

**Elastic Gain due to Diaphragms, Deck and SIDL at Midspan:**

BDM 5.1.4.D

Elastic gain due to diaphragms and deck
\[ \Delta f_{pED1} := \frac{E_p}{E_c} \left[ (M_{rm,2} + M_{rm,3} + M_{rm,4}) \frac{EC_{rm,2}}{I_g} \right] = 4.986 \text{ ksi} \]

Elastic gain due to SIDL (including barrier weight but not traffic overlay)
\[ \Delta f_{pED2} := \frac{E_p}{E_c} \left[ M_{rm,5} \frac{(Y_b - Y_{bg} + EC_{rm,2})}{I_c} \right] = 0.702 \text{ ksi} \]

**Approximate Lump Sum Estimate of Time Dependent Losses**

The Approximate Estimate of Time-Dependent Losses of AASHTO LRFD 5.9.5.3 may be used for precast
BDM 5.1.4.B
Concrete Structures

Chapter 5

prestressed girders with composite decks as long as the conditions set forth in AASHTO are satisfied: LRFD 5.9.5.3

Normal density concrete
Concrete is either steam or moist cured
Prestressing is by low relaxation strands
Sit in average exposure condition and temperatures

Concrete density check

\[ \text{chk} = \begin{cases} 1 & \text{if } (0.155 \text{ kcf} \geq w_{cE} \geq 0.135 \text{ kcf}, "OK", "NG") = "OK" \end{cases} \]

Correction factor for ambient air RH

\[ \gamma_h := 1.7 - 0.01(H \div \%) = 0.950 \]

Correction factor for concrete strength at transfer

\[ \gamma_{st} := \frac{5}{1 + f_{ci} \div \text{ksi}} = 0.588 \]

Approx lump sum long term PS losses at shipping

\[ \Delta f_{pLT} := -\left(10.0 \cdot \frac{f_{pbt} \cdot A_{ps}}{\text{ksi} \cdot A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + 2.4 \right) \text{ksi} = -19.111 \cdot \text{ksi} \]

LRFD 5.9.5.3

Loss due to Removal of Temporary Strands at Midspan

Force in temporary strands before removal

\[ P_{tr} := A_{temp} \left( f_{pbt} + \Delta f_{pEST} + \Delta f_{pLTH} \right) = 245.5 \cdot \text{kip} \]

Change in stress at c.g. permanent strands after removal of temporary strands

\[ f_{ptr} := \frac{P_{tr} \cdot E_{c} \cdot \text{temp} \cdot E_{cm} \cdot 2}{I_g} = -0.129 \cdot \text{ksi} \]

Loss in permanent strands due to removal of temporary strands

\[ \Delta f_{ptr} := \frac{E_{r}}{E_{c}} \cdot f_{ptr} = -0.626 \cdot \text{ksi} \]

Total Prestress Losses - Permanent Strands at Midspan BDM 5.1.4.D

Total PS loss by lump sum estimate

\[ \Delta f_{pT} := \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{ptr} + \Delta f_{pED1} + \Delta f_{pED2} + \Delta f_{pLT} \]

Effective Prestress at Midspan

Effective prestress at midspan

\[ f_{pe} := f_{pj} + \Delta f_{pT} = 173.416 \cdot \text{ksi} \]

Check effective prestress limit

\[ \text{chk}_{-5} := \begin{cases} \text{if } (f_{pe} \leq f_{pe}\text{.lim} , "OK", "NG") = "OK" \end{cases} \]

Effective prestress force at midspan

\[ P_e := A_{ps} \cdot f_{pe} = 1430.0 \cdot \text{kip} \]

7.6 Effective Prestress Modifier for Sections within Transfer Length

Multiply effective prestress force by modifier below at each section to account for force in prestressing within the transfer length. The prestressing force may be assumed to vary linearly from 0.0 at the point where bonding commences (free end of strand) to a maximum at the transfer length.
\[
\text{TRAN} := \begin{cases}
\text{for } i \in 1..\text{rows(SE)} & \\
\text{if } SE_i < l_t & \text{then } TR_i := \frac{SE_i}{l_t} \\
\text{if } l_t \leq SE_i \leq GL - l_t & \text{then } TR_i := 1 \\
\text{if } GL - l_t < SE_i & \text{then } TR_i := \frac{GL - SE_i}{l_t}
\end{cases}
\]

\[
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16 & \ldots
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\]
8. Stresses at Service and Fatigue Limit States

Negative stress indicates compression.

8.1 Service I for Casting Yard Stage (At Release)

Effective Prestress in Permanent Strands

\[ f_{peP1} := f_{pj} + \Delta f_{pR0} + \Delta f_{pES} = 187.5 \text{ ksi} \]

Effective Prestress in Temporary Strands

\[ f_{peT1} := f_{pj} + \Delta f_{pR0} + \Delta f_{pEST} = 193.8 \text{ ksi} \]

Stress in girder due to prestressing:

\[
PS1 := \begin{cases} 
\text{for } i \in 1.. \text{rows(SE)} \\
  P_p & \leftarrow f_{peP1} \cdot \text{TRAN}_i \cdot A_{ps} \\
  P_t & \leftarrow f_{peT1} \cdot \text{TRAN}_i \cdot A_{\text{temp}} \\
  PS_{i,1} & \leftarrow \frac{P_p}{A_g} - \frac{P_p \cdot \text{EC}_{i,2}}{S_{tg}} + \frac{P_t}{A_g} - \frac{P_t \cdot e_{\text{temp}}}{S_{tg}} \\
  PS_{i,2} & \leftarrow \frac{P_p}{A_g} + \frac{P_p \cdot \text{EC}_{i,2}}{S_{bg}} + \frac{P_t}{A_g} + \frac{P_t \cdot e_{\text{temp}}}{S_{bg}} 
\end{cases}
\]

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<tr>
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<tr>
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</tr>
<tr>
<td>15</td>
<td>-0.169</td>
</tr>
<tr>
<td>16</td>
<td>-0.546</td>
</tr>
</tbody>
</table>

Find total Service I stress which includes:
- Prestress
- Girder Dead Load between ends

\[
\text{STRESS1} := \begin{cases} 
\text{for } i \in 1.. \text{rows(SE)} \\
  \text{STR}_{i,1} & \leftarrow PS_{i,1} + \text{ST}_{i,11} \\
  \text{STR}_{i,2} & \leftarrow PS_{i,2} + \text{SB}_{i,11} 
\end{cases}
\]

<table>
<thead>
<tr>
<th>\text{Top Stress}</th>
<th>\text{Bottom Stress}</th>
</tr>
</thead>
<tbody>
<tr>
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<td>3</td>
<td>-1.401</td>
</tr>
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<td>4</td>
<td>-1.426</td>
</tr>
<tr>
<td>5</td>
<td>-1.447</td>
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<tr>
<td>6</td>
<td>-1.478</td>
</tr>
<tr>
<td>7</td>
<td>-1.513</td>
</tr>
<tr>
<td>8</td>
<td>-1.528</td>
</tr>
<tr>
<td>9</td>
<td>-1.431</td>
</tr>
<tr>
<td>10</td>
<td>-1.230</td>
</tr>
</tbody>
</table>
Maximum compressive stress allowed:

\[ f_{c,TL,\text{lim}} = -4.875 \text{ ksi} \]

Maximum tensile stress allowed:

\[ f_{t,TL,\text{lim}} = 0.520 \text{ ksi} \]

Check compressive stress

\[ \text{chk}_{\text{c,1}} := \text{if} \left( \min(\text{STRESS1}) \geq f_{c,TL,\text{lim}} \right) \text{"OK", "NG"} = \text{"OK"} \]

Check tensile stress (with bonded reinforcement)

\[ \text{chk}_{\text{c,2}} := \text{if} \left( \max(\text{STRESS1}) \leq f_{t,TL,\text{lim}} \right) \text{"OK", "NG"} = \text{"OK"} \]

### 8.2 Service I after Temporary Strand Removal

**Effective Prestress in Permanent Strands**

\[ f_{\text{peP}_2} := f_{pj} + \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pLTH} + \Delta f_{\text{ptr}} = 181.6 \text{ ksi} \]

#### Stress in girder due to prestressing:

\[
\begin{align*}
\text{PS2} &:= \text{for } i \in 1.. \text{rows(SE)} \\
P_p &\leftarrow f_{\text{peP}_2} \cdot \text{TRAN}_i \cdot A_{ps} \\
\text{PS}_{i,1} &\leftarrow \left( \frac{P_p}{A_g} - \frac{P_{p\text{-EC}_i,2}}{S_{tg}} \right) \\
\text{PS}_{i,2} &\leftarrow \left( \frac{P_p}{A_g} + \frac{P_{p\text{-EC}_i,2}}{S_{bg}} \right)
\end{align*}
\]

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>-0.349</td>
<td>-1.733</td>
</tr>
<tr>
<td>3</td>
<td>-0.502</td>
<td>-2.662</td>
</tr>
<tr>
<td>4</td>
<td>-0.446</td>
<td>-2.714</td>
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<tr>
<td>5</td>
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</tr>
<tr>
<td>6</td>
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<td>-2.845</td>
</tr>
<tr>
<td>7</td>
<td>-0.166</td>
<td>-2.975</td>
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<tr>
<td>8</td>
<td>0.200</td>
<td>-3.315</td>
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<tr>
<td>9</td>
<td>0.565</td>
<td>-3.655</td>
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<tr>
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<td>0.919</td>
<td>-3.984</td>
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<tr>
<td>11</td>
<td>0.919</td>
<td>-3.984</td>
</tr>
<tr>
<td>12</td>
<td>0.919</td>
<td>-3.984</td>
</tr>
<tr>
<td>13</td>
<td>0.919</td>
<td>-3.984</td>
</tr>
<tr>
<td>14</td>
<td>0.919</td>
<td>-3.984</td>
</tr>
<tr>
<td>15</td>
<td>0.565</td>
<td>-3.655</td>
</tr>
<tr>
<td>16</td>
<td>0.200</td>
<td>...</td>
</tr>
</tbody>
</table>

Find total Service I stress which includes:

- Prestress
- Girder Dead Load between supports

<table>
<thead>
<tr>
<th></th>
<th>Top Stress</th>
<th>Bottom Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

\[ f_{\text{peP}_2} := f_{pj} + \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pLTH} + \Delta f_{\text{ptr}} = 181.6 \text{ ksi} \]
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STRESS2 := for i ∈ 1..rows(SE)

\[
\begin{align*}
&\text{STR}_{i,1} \leftarrow \text{PS}_{2,1} + \text{ST}_{i,1} \\
&\text{STR}_{i,2} \leftarrow \text{PS}_{2,2} + \text{SB}_{i,1}
\end{align*}
\]

STRESS2 =

<table>
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<tr>
<th></th>
<th>1</th>
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</thead>
<tbody>
<tr>
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<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>-0.349</td>
<td>-1.733</td>
</tr>
<tr>
<td>3</td>
<td>-0.546</td>
<td>-2.621</td>
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<tr>
<td>4</td>
<td>-0.573</td>
<td>-2.596</td>
</tr>
<tr>
<td>5</td>
<td>-0.596</td>
<td>-2.575</td>
</tr>
<tr>
<td>6</td>
<td>-0.630</td>
<td>-2.543</td>
</tr>
<tr>
<td>7</td>
<td>-0.670</td>
<td>-2.506</td>
</tr>
<tr>
<td>8</td>
<td>-0.697</td>
<td>-2.481</td>
</tr>
<tr>
<td>9</td>
<td>-0.611</td>
<td>-2.560</td>
</tr>
<tr>
<td>10</td>
<td>-0.422</td>
<td>-2.737</td>
</tr>
<tr>
<td>11</td>
<td>-0.425</td>
<td>-2.734</td>
</tr>
<tr>
<td>12</td>
<td>-0.481</td>
<td>-2.682</td>
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<td>-0.425</td>
<td>-2.734</td>
</tr>
<tr>
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<tr>
<td>15</td>
<td>-0.611</td>
<td>-2.560</td>
</tr>
<tr>
<td>16</td>
<td>-0.697</td>
<td>...</td>
</tr>
</tbody>
</table>

Maximum compressive stress allowed:

\[ f_{c,SH,\text{lim}} = -5.525 \text{ ksi} \]

Maximum tensile stress allowed:

\[ f_{t,SP,\text{lim}} = 0.554 \text{ ksi} \]

Check compressive stress:

\[ \text{chk}_{3} := \text{if} \left( \text{min}(\text{STRESS2}) \geq f_{c,SH,\text{lim}}, "OK", "NG" \right) = "OK" \]

Check tensile stress (with bonded reinforcement):

\[ \text{chk}_{4} := \text{if} \left( \text{max}(\text{STRESS2}) \leq f_{t,SP,\text{lim}}, "OK", "NG" \right) = "OK" \]

8.3 Service I after Deck and Diaphragm Placement

Effective Prestress in Permanent Strands

\[ f_{p\text{eP3}} := f_{p\text{j}} + \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pLT} + \Delta f_{ptr} + \Delta f_{pED1} = 172.7 \text{ ksi} \]

Stress in girder due to prestressing:

\[
\begin{align*}
\text{PS3} &:= \text{for } i \in 1..\text{rows(SE)} \\
&\text{P}_{p} \leftarrow f_{p\text{eP3}}\text{TRAN}_{i}\text{A}_{ps} \\
&\text{PS}_{i,1} \leftarrow \left( \frac{\text{P}_{p}}{\text{A}_{g}} - \frac{\text{P}_{p}\text{EC}_{i,2}}{\text{S}_{tg}} \right) \\
&\text{PS}_{i,2} \leftarrow \left( \frac{\text{P}_{p}}{\text{A}_{g}} + \frac{\text{P}_{p}\text{EC}_{i,2}}{\text{S}_{bg}} \right)
\end{align*}
\]

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>0.000</td>
</tr>
<tr>
<td>2</td>
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<td>-1.649</td>
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<td>-2.532</td>
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<td>0.538</td>
<td>-3.477</td>
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<tr>
<td>10</td>
<td>0.875</td>
<td>-3.790</td>
</tr>
<tr>
<td>11</td>
<td>0.875</td>
<td>-3.790</td>
</tr>
<tr>
<td>12</td>
<td>0.875</td>
<td>-3.790</td>
</tr>
</tbody>
</table>

\[ \text{PS3} = \]
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8.4 Service I for Superimposed Dead Load (SIDL) - Bridge Site 2

Effective Prestress in Permanent Strands

\[ f_{\text{peP4}} := f_{\text{pj}} + \Delta f_{\text{pR0}} + \Delta f_{\text{pES}} + \Delta f_{\text{pLT}} + \Delta f_{\text{ptr}} + \Delta f_{\text{PED1}} + \Delta f_{\text{PED2}} = 173.4 \text{ ksi} \]

Stress in girder due to prestressing:

<table>
<thead>
<tr>
<th>Top Stress</th>
<th>Bottom Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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</tr>
<tr>
<td>1</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
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<tr>
<td>8</td>
<td>-1.477</td>
</tr>
<tr>
<td>9</td>
<td>-1.652</td>
</tr>
<tr>
<td>10</td>
<td>-1.620</td>
</tr>
<tr>
<td>11</td>
<td>-1.626</td>
</tr>
<tr>
<td>12</td>
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<td>14</td>
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</tr>
<tr>
<td>15</td>
<td>-1.652</td>
</tr>
<tr>
<td>16</td>
<td>-1.477</td>
</tr>
</tbody>
</table>

Maximum compressive stress allowed:

\[ f_{c,\text{PP.lim}} = -3.825 \text{ ksi} \]

Maximum tensile stress allowed:

\[ f_{L,\text{PCT.lim}} = 0.000 \text{ ksi} \]

Check compressive stress

\[ \text{chk}_{5,\text{c}} := \text{if}(\text{min(STRESS3)} \geq f_{c,\text{PP.lim}} \text{"OK","NG"}) = \text{"OK"} \]

Check tensile stress (with bonded reinforcement)

\[ \text{chk}_{6,\text{t}} := \text{if}(\text{max(STRESS3)} \leq f_{L,\text{PCT.lim}} \text{"OK","NG"}) = \text{"OK"} \]
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PS4 := for i \in 1..\text{rows(SE)}
\begin{align*}
P_p &\leftarrow f_{peP4} \cdot \text{TRAN}_{i} \cdot A_{ps} \\
PS_{i, 1} &\leftarrow \left( \frac{P_p}{A_g} - \frac{P_p \cdot EC_{i, 2}}{Stg} \right) \\
PS_{i, 2} &\leftarrow \left( \frac{P_p}{A_g} + \frac{P_p \cdot EC_{i, 2}}{Sbg} \right)
\end{align*}

Find total Service I stress which includes:
- Prestress
- Girder Dead Load between supports
- Diaphragm Dead Load
- Slab and Pad Dead Load
- Barrier SIDL

STRESS4 := for i \in 1..\text{rows(SE)}
\begin{align*}
STR_{i, 1} &\leftarrow PS_{i, 1} + \sum_{j=1}^{5} ST_{i, j} \\
STR_{i, 2} &\leftarrow PS_{i, 2} + \sum_{j=1}^{5} SB_{i, j} \\
STR_{i, 3} &\leftarrow \sum_{j=1}^{5} SS_{i, j}
\end{align*}

Maximum compressive stress allowed - girder:
\[ f_{c, PP, \text{lim}} = -3.825 \text{ksi} \]

Maximum tensile stress allowed:
\[ f_{t, PCT, \text{lim}} = 0.000 \text{ksi} \]

<table>
<thead>
<tr>
<th></th>
<th>Top Stress</th>
<th>Bottom Stress</th>
<th>Slab Top Stress</th>
</tr>
</thead>
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<td>0.000</td>
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<tr>
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<td>-1.655</td>
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<td>-2.543</td>
<td>-2.543</td>
</tr>
<tr>
<td>4</td>
<td>-0.426</td>
<td>-2.593</td>
<td>-2.593</td>
</tr>
<tr>
<td>5</td>
<td>-0.377</td>
<td>-2.638</td>
<td>-2.638</td>
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<tr>
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<td>-2.717</td>
<td>-2.717</td>
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<td>-2.842</td>
<td>-2.842</td>
</tr>
<tr>
<td>8</td>
<td>0.191</td>
<td>-3.166</td>
<td>-3.166</td>
</tr>
<tr>
<td>9</td>
<td>0.540</td>
<td>-3.491</td>
<td>-3.491</td>
</tr>
<tr>
<td>10</td>
<td>0.878</td>
<td>-3.805</td>
<td>-3.805</td>
</tr>
<tr>
<td>11</td>
<td>0.878</td>
<td>-3.805</td>
<td>-3.805</td>
</tr>
<tr>
<td>12</td>
<td>0.878</td>
<td>-3.805</td>
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<td>0.878</td>
<td>-3.805</td>
<td>-3.805</td>
</tr>
<tr>
<td>15</td>
<td>0.540</td>
<td>-3.491</td>
<td>-3.491</td>
</tr>
<tr>
<td>16</td>
<td>0.191</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>
Check compressive stress in girder:

\[ \text{chk}_c, s := \text{if} \left( \min \left( \text{STRESS}_4(1), \text{STRESS}_4(2) \right) \geq f_{c, \text{PP}, \text{lim}} \text{"OK", "NG"} \right) = \text{"OK"} \]

Check tensile stress in girder (with bonded reinforcement):

\[ \text{chk}_t, s := \text{if} \left( \max \left( \text{STRESS}_4(1), \text{STRESS}_4(2) \right) \leq f_{t, \text{PP}, \text{lim}} \text{"OK", "NG"} \right) = \text{"OK"} \]

### 8.5 Service I for Final with Live Load - Bridge Site 3 - Compressive Stresses

**Effective Prestress in Permanent Strands**

\[ f_{\text{pe}5} := f_{\text{pe}} = 173.4 \text{ ksi} \]

**Stress in girder due to prestressing:**

\[
\begin{align*}
\text{PS5} := & \quad \text{for } i \in \text{row}(\text{SE}) \\
\text{P}_p & := f_{\text{pe}5} \cdot \text{TRAN}_i \cdot A_{\text{ps}} \\
\text{PS}_{1,1} & := \left( \frac{\text{P}_p}{A_g} - \frac{\text{P}_p \cdot \text{EC}_{1,2}}{S_{t_g}} \right) \\
\text{PS}_{1,2} & := \left( \frac{\text{P}_p}{A_g} + \frac{\text{P}_p \cdot \text{EC}_{1,2}}{S_{b_g}} \right) \\
\text{PS} & := \text{P}_p
\end{align*}
\]

<table>
<thead>
<tr>
<th>Stress</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>-0.333</td>
<td>-1.655</td>
</tr>
<tr>
<td>3</td>
<td>-0.480</td>
<td>-2.543</td>
</tr>
<tr>
<td>4</td>
<td>-0.426</td>
<td>-2.593</td>
</tr>
<tr>
<td>5</td>
<td>-0.377</td>
<td>-2.638</td>
</tr>
<tr>
<td>6</td>
<td>-0.292</td>
<td>-2.717</td>
</tr>
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<td>7</td>
<td>-0.158</td>
<td>-2.842</td>
</tr>
<tr>
<td>8</td>
<td>0.191</td>
<td>-3.166</td>
</tr>
<tr>
<td>9</td>
<td>0.540</td>
<td>-3.491</td>
</tr>
<tr>
<td>10</td>
<td>0.878</td>
<td>-3.805</td>
</tr>
<tr>
<td>11</td>
<td>0.878</td>
<td>-3.805</td>
</tr>
<tr>
<td>12</td>
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<tr>
<td>15</td>
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<td>-3.491</td>
</tr>
<tr>
<td>16</td>
<td>0.191</td>
<td>...</td>
</tr>
</tbody>
</table>

**Find total Service I stress which includes:**

- Prestress
- Girder Dead Load between supports
- Diaphragm Dead Load
- Slab and Pad Dead Load
- Barrier SIDL
- Traffic Overlay
- Live Load

To maximize bottom compressive stress, the Live Load is left off.
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\[
\text{STRESS5} := \text{for } i \in 1.. \text{rows(SE)}
\]

\[
\text{STR}_{i,1} \leftarrow \text{PS5}_{i,1} + \sum_{j=1}^{6} ST_{i,j} + ST_{i,10}
\]

\[
\text{STR}_{i,2} \leftarrow \text{PS5}_{i,2} + \sum_{j=1}^{6} SB_{i,j}
\]

\[
\text{STR}_{i,3} \leftarrow \sum_{j=1}^{6} SS_{i,j} + SS_{i,10}
\]

\[
\text{ST}_{ij} = + \text{ST}_{i,10}, + \text{STR}_{i,2}, \text{PS5}_{i,2}, 1, \text{SB}_{i,j}, \text{SS}_{i,j}, \text{SS}_{i,10}, + \text{IM FAT}
\]

\[
\text{ST}_{ij} = + \text{STR}_{i,3}, \text{SS}_{i,10}, + \text{PS5}_{i,1}, \text{ST}_{i,10}, + \text{SB}_{i,j}, \text{IM FAT}
\]

\[
\text{f}_{c,PPT,\text{lim}} = -5.100 \text{ ksi}
\]

\[
\text{chk}_{\text{gir}} := \text{if } \left( \text{min}(\text{STRESS5}^{(1)}, \text{STRESS5}^{(2)}) \right) \geq f_{c,PPT,\text{lim}}, \text{"OK", \"NG\"} = \text{"OK"}
\]

**8.6 Fatigue I for Final with Live Load - Bridge Site 3 - Compressive Stresses**

Live Load Stresses from the factored Fatigue Load:

\[
\text{SFATLL} := \text{for } i \in 1.. \text{rows(SE)}
\]

\[
\text{Stress}_{i,1} \leftarrow \frac{\gamma_{LLfat}}{\text{M}_{FAT} \cdot \text{DFAT} \cdot (1 + \text{IM FAT})} \text{St}_{i}
\]

\[
\text{Stress}_{i,2} \leftarrow \frac{\gamma_{LLfat}}{\text{M}_{FAT} \cdot \text{DFAT} \cdot (1 + \text{IM FAT})} \text{S}_{b}
\]

\[
\text{SFATLL} = \text{for } i \in 1.. \text{rows(SE)}
\]

\[
\text{Top} \quad \text{Stress} \quad \text{Bottom} \quad \text{Stress}
\]

\[
\begin{array}{cccc}
1 & 0.000 & 0.000 \\
2 & 0.000 & 0.000 \\
3 & -0.010 & 0.018 \\
4 & -0.029 & 0.051 \\
5 & -0.045 & 0.080 \\
6 & -0.073 & 0.129 \\
7 & -0.112 & 0.199 \\
8 & -0.194 & 0.345 \\
9 & -0.252 & 0.446 \\
10 & -0.282 & 0.500 \\
11 & -0.283 & 0.501 \\
12 & -0.284 & 0.504 \\
13 & -0.283 & 0.501 \\
14 & -0.282 & 0.500 \\
15 & -0.252 & 0.446 \\
16 & -0.194 & ...
\end{array}
\]
Find total Fatigue I stress which includes:
- 1/2 Prestress
- 1/2 Girder Dead Load between supports
- 1/2 Diaphragm Dead Load
- 1/2 Slab and Pad Dead Load
- 1/2 Barrier SIDL
- 1/2 Future Overlay SIDL
- Fatigue Live Load

To maximize bottom compressive stress, the Fatigue Live Load is left off.

\[
\text{PS5}_{i,1} = \sum_{j=1}^{6} \text{ST}_{i,j} + S_{\text{FATLI}i,1}
\]

\[
\text{PS5}_{i,2} = \sum_{j=1}^{6} \text{SB}_{i,j}
\]

\[
\text{STR}_{i,1} = \left( \frac{\text{PS5}_{i,1}}{2} \right) + S_{\text{FATLI}i,1}
\]

\[
\text{STR}_{i,2} = \left( \frac{\text{PS5}_{i,2}}{2} \right)
\]

\[
\text{STRESS6} := \begin{cases} \text{for } i \in 1..\text{rows(SE)} \end{cases}
\]

\[
\text{STRESS6} = \begin{array}{c|c|c}
\text{Top} & \text{Bottom} & \text{ksi} \\
\hline
1 & 0.000 & 0.000 \\
2 & -0.167 & -0.828 \\
3 & -0.293 & -1.229 \\
4 & -0.367 & -1.173 \\
5 & -0.432 & -1.124 \\
6 & -0.539 & -1.044 \\
7 & -0.689 & -0.931 \\
8 & -0.988 & -0.709 \\
9 & -1.150 & -0.598 \\
10 & -1.173 & -0.595 \\
11 & -1.177 & -0.592 \\
12 & -1.238 & -0.534 \\
13 & -1.177 & -0.592 \\
14 & -1.173 & -0.595 \\
15 & -1.150 & -0.598 \\
16 & -0.988 & ... \\
\end{array}
\]

Maximum compressive stress allowed - girder: \( f_{c,\text{FA,lim}} = -3.400 \text{-ksi} \)

Check compressive stress in girder

\[
\text{chk}_{i,10} := \text{if} \left( \text{min(STRESS6)} \geq f_{c,\text{FA,lim}} \right) \text{"OK", "NG"} = \text{"OK"}
\]

### 8.7 Service III for Final with Live Load - Bridge Site 3 - Tensile Stresses

Find total Service III stress which includes:
- Prestress
- Girder Dead Load between supports
- Diaphragm Dead Load
- Slab and Pad Dead Load
- Barrier SIDL
- Traffic Overlay
- Live Load (factored)

\[
\text{STRESS7} := \begin{cases} \text{for } i \in 1..\text{rows(SE)} \end{cases}
\]

\[
\text{STRESS7} = \begin{array}{c|c|c}
\text{Top} & \text{Bottom} & \text{ksi} \\
\hline
1 & 0.000 & 0.000 \\
2 & -0.333 & -1.655 \\
3 & -0.583 & -2.428 \\
4 & -0.725 & -2.260 \\
5 & -0.850 & -2.112 \\
6 & -1.055 & -1.870 \\
7 & -1.344 & -1.525 \\
8 & -1.923 & -0.823 \\
9 & -2.232 & -0.421 \\
10 & -2.277 & -0.313 \\
11 & -2.286 & -0.304 \\
12 & -2.421 & -0.158 \\
13 & -2.286 & -0.304 \\
\end{array}
\]
Maximum tensile stress allowed - girder:

\[ f_{t,PCT}\text{.lim} = 0.000 \text{ ksi} \]

Check tensile stress in girder (with bonded reinforcement):

\[
\text{chk}_{\text{t,PCT},11} := \text{if} \left( \max\left(\text{STRESS}^{(1)}, \text{STRESS}^{(2)}\right) \leq f_{t,PCT}\text{.lim}, \text{"OK"}, \text{"NG"} \right) = \text{"OK}
\]
Chapter 5 Concrete Structures

9. Strength Limit State

9.1 Ultimate Moments

Factored bending moments for Strength 1 Limit State (ultimate):

\[ M_u := \begin{cases} 
\text{for } i \in 1..\text{rows(SE)} \\
UM_i & \left( \gamma_{DC} \sum_{j=1}^{5} M_{i,j} + \gamma_{DW} M_{i,6} + \gamma_{LL} \cdot DF \cdot M_{i,10} \right) 
\end{cases} \]

\[ UM := \begin{array}{c}
1 \\
1 \\
2 \\
3 \\
4 \\
5 \\
6 \\
7 \\
8 \\
9 \\
10 \\
11 \\
12 \\
13 \\
14 \\
15 \\
16 \\
\end{array} \]

\[ M_u = 9 \text{ kip-ft} \]

9.2 Flexural Resistance

The approximate method using the rectangular stress distribution of AASHTO LRFD 5.7.3 is used below. It is known that this method underestimates the flexural resistance due to factors such as not accounting for higher strength concrete for the girder, not accounting for the top flange of the precast girder, excessive "c" dimensions causing the flexural resistance factor to be reduced, etc. If higher capacity or improved accuracy is needed, it is recommended to use the Nonlinear Strain Compatibility Analysis procedure described in the PCI Journal, Jan-Feb 2005, "Flexural Strength of Reinforced and Prestressed Concrete T-Beams". Areas of mild steel tension and compression reinforcement are conservatively assumed to be zero.

Check for validity of \( f_{ps} \) eqn at midspan

\[ \text{chk1} = \text{if } (f_{pe} \geq 0.5 \cdot f_{pu} \text{"OK"}, \text{"NG"}) = \text{"OK"} \]

Factor for determination of \( c \)

\[ k := 2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) = 0.28 \]

Depth of compression flange

\[ h_{F} := t_{S} = 7.0 \text{ in} \]

Find stress in prestressing steel at nominal flexural resistance

Strands at all sections are assumed to be fully developed.
Concrete Structures Chapter 5

Distance from extreme compression fiber to the centroid of the prestressing tendons

\[ d_{p_i} := h_f + Y_{tg} + EC_{i,2} \]

Distance between neutral axis and compression face for flanged (T) section behavior

\[ c_{fl_i} := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f_{cs} \cdot (b_e - b_w) \cdot h_f}{0.85 \cdot f_{cs} \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_{p_i}}} \]  LRFD 5.7.3.1.1

Distance between neutral axis and compression face for rectangular section

\[ c_{rfl_i} := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f_{cs} \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_{p_i}}} \]  LRFD 5.7.3.1.1

Neutral axis distance:
If the compression block for the rectangular section behavior is contained within the top flange, use the \( c \) for rectangular section behavior. Otherwise, use the \( c \) for T section behavior.

\[
c_i := \begin{cases} 
\text{for } i \in 1..\text{rows(SE)} & \text{LRFD 5.7.3.2.2} \\
\beta_1 \cdot c_{rfl_i} & \text{if } \beta_1 \cdot c_{rfl_i} \leq h_f \\
c_i & \text{otherwise} 
\end{cases}
\]

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\[
d_{p_i} = 86.64 \text{ in} \quad c_{fl_i} = 8.192.258 \text{ in} \quad c_{rfl_i} = 8.9.494 \text{ in} \quad c_{l} = 8.19.258 \text{ in} \quad c_{1} = 9.19.683 \text{ in} \]

Average stress in prestressing steel at nominal flexural resistance

\[ f_{psl_i} := f_{pu} \left( 1 - k \cdot \frac{c_i}{d_{p_i}} \right) \]  LRFD 5.7.3.1.1

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Chapter 5 Concrete Structures

Development Length Factor

\[ \kappa := \text{if} \left( d_g > 24\text{in}, 1.6, 1 \right) = 1.6 \quad \text{LRFD 5.11.4.2} \]

Required development length at midspan (conservative to use for entire girder)

\[ l_d := \kappa \left( \frac{f_{ps,\text{rm}}}{\text{ksi}} - \frac{2}{3} \frac{f_{pe}}{\text{ksi}} \right) d_b = 129.51\text{-in} \quad \text{LRFD 5.11.4.2} \]

Reduced stress in prestressing steel at nominal flexural resistance at ends of girder

Within the transfer and development lengths at the ends of the girder, the stress in the prestressing steel at nominal flexural resistance must be reduced as shown in AASHTO LRFD Figure C5.11.4.2-1.

\[
f_{ps} := \begin{cases} 
    f_{pe} \cdot \text{TRAN}_i & \text{if } SE_i \leq l_t \\
    f_{pe} + \frac{SE_i - l_t}{l_d - l_t} \left( f_{ps,\text{rm}} - f_{pe} \right) & \text{if } l_t < SE_i \leq l_d \\
    f_{ps,\text{rm}} & \text{if } l_d < SE_i < GL - l_d \\
    GL - l_t - SE_i & \text{if } GL - l_d \leq SE_i < GL - l_t \\
    f_{pe} + \frac{GL - l_t - SE_i}{l_d - l_t} \left( f_{ps,\text{rm}} - f_{pe} \right) & \text{if } GL - l_t \leq SE_i < GL - l_t \\
    f_{pe} \cdot \text{TRAN}_i & \text{if } SE_i \geq GL - l_t 
\end{cases}
\]

Recalculate stress block depth based on reduced stress in prestressing steel

\[
f_{ps} := \begin{cases} 
    247.789 & \text{if } i = 7 \\
    248.788 & \text{if } i = 8 \\
    249.702 & \text{if } i = 9 \\
    250.516 & \text{if } i = 10, 11, 12, 13, 14, 15, 16 \\
\end{cases}
\]
Concrete Structures Chapter 5

Distance between neutral axis and compression face for flanged (T) section behavior

$$c_f := \frac{A_{ps} \cdot f_{ps_i} - 0.85 \cdot f'_{cs} \cdot (b_e - b_w) \cdot h_f}{0.85 \cdot f'_{cs} \cdot \beta_1 \cdot b_w}$$

Distance between neutral axis and compression face for rectangular section

$$c_r := \frac{A_{ps} \cdot f_{ps_i}}{0.85 \cdot f'_{cs} \cdot \beta_1 \cdot b_e}$$

Neutral axis distance:
If the compression block for the rectangular section behavior is contained within the top flange, use the c for rectangular section behavior. Otherwise, use the c for T section behavior.

Depth of equivalent stress block

$$a_i := \beta_1 \cdot c_i$$

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Nominal flexural resistance

$$M_n := \begin{cases} \text{for } i \in 1 \ldots \text{rows(SE)} \\ MN_i \leftarrow A_{ps} \cdot f_{ps_i} \left(d_p_i - \frac{a_i}{2}\right) + 0.85 \cdot f'_{cs} \left(b_e - b_w\right) h_f \left(\frac{a_i}{2} - \frac{h_f}{2}\right) \text{ if } h_f < a_i \\ MN_i \leftarrow A_{ps} \cdot f_{ps_i} \left(d_p_i - \frac{a_i}{2}\right) \text{ otherwise} \\ MN \end{cases}$$
Distance from extreme compression fiber to the centroid of the extreme tension steel element

\[ d_t := h_f + d_g - s_{\text{bottom}} = 79.000\text{ in} \]

LRFD 5.5.4.2.1

Flexure resistance factor

\[ \phi_i := \text{if} \left(c_i > 0, \phi_p(d_t, c_i), 1.0 \right) \]

Factored flexural resistance

\[ M_{t_i} := \phi_i M_{n_i} \]

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Ultimate Moment Factored Resistance vs. Factored Loading

Distance Along Girder (ft)
Check flexural strength at all sections

\[
\text{chk}_i, 2 := \begin{cases} 
\text{CH} \leftarrow \text{"OK"} & \text{if } i \in 1..\text{rows(SE)} \\
\text{CH} \leftarrow \text{"NG"} & \text{if } M_{r_i} < M_{u_i} \\
\text{CH} & \end{cases}
\]

9.3 Minimum Reinforcement

Modulus of rupture

\[ \sigma_{\text{mcr.min}} = 1.079 \text{ ksi} \]

Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (bottom of girder)

Total unfactored dead load moment acting on the monolithic or noncomposite girder

\[ M_{\text{dnc}} := \sum_{j=1}^{4} M_{1,j} \]

Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads

\[ S_c := S_b = 25069 \cdot \text{in}^3 \]

Section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads

\[ S_{nc} := S_{bg} = 20593 \cdot \text{in}^3 \]

\[ \gamma_1 := 1.56 \]

\[ \gamma_2 := 1.1 \]

\[ \gamma_3 := 1.0 \]

\[ M_{\text{cr.mod}} := \gamma_3 \left( \gamma_1 \cdot \sigma_{\text{mcr.min}} + \gamma_2 \cdot f_{\text{cpe}} \right) S_c - M_{\text{dnc}} \cdot \left( \frac{S_c}{S_{nc}} - 1 \right) \]

1.0 for prestressed concrete structures

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Check if minimum reinforcement is provided. This check need not be satisfied if section is compression controlled.

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\[
\text{chk} \left( \begin{array}{c} 92 \\ CH \text{ "OK"} \end{array} \right) = \text{"OK"}
\]

\[
\text{for } i \in 1: \text{rows}(SE) \\
\quad \text{CH} \leftarrow \text{"NG" if } M_{r,i} < \min(M_{cr.mod,i}, 1.33 \cdot M_{u,i}) \\
\quad \text{CH}
\]

\[
M_{dnc,i} \cdot S_c \cdot S_{nc} - M_{r,i} \cdot S_{c}\left( S_{nc} - 1 \right) = M_{cr.mod,i} \gamma_3 \
\]

\[
\gamma_1 = 1.56 \\
\gamma_2 = 1.1 \\
\gamma_3 = 1.0
\]
10. Shear & Longitudinal Reinforcement Design

10.1 Factored Shear Loads

Factored shears for Strength 1 Limit State (ultimate):

\[ V_u := \begin{cases} \text{for } i \in 1 \ldots \text{rows(SE)} \\ UV_i \leftarrow \sum_{j=1}^{5} \left( \gamma_{DC} V_{i,j} + \gamma_{DW} V_{i,6} + \gamma_{LL} \cdot D F \cdot V_{i,10} \right) \\ UV \end{cases} \]

\[ V_u = \begin{array}{c|c|c} \text{Row} & 1 & \text{kip} \\ \hline 1 & 0.00 & \text{kip} \\ 2 & 357.81 & \text{kip} \\ 3 & 353.11 & \text{kip} \\ 4 & 343.95 & \text{kip} \\ 5 & 335.59 & \text{kip} \\ 6 & 321.18 & \text{kip} \\ 7 & 298.69 & \text{kip} \\ 8 & 240.65 & \text{kip} \\ 9 & 180.14 & \text{kip} \\ 10 & 125.97 & \text{kip} \\ 11 & 124.30 & \text{kip} \\ 12 & 69.55 & \text{kip} \\ 13 & 124.29 & \text{kip} \\ 14 & 125.97 & \text{kip} \\ 15 & 180.13 & \text{kip} \\ 16 & \ldots & \text{kip} \end{array} \]

10.2 Critical Section Location

The shear design of prestressed members shall be based on the general procedure of AASHTO LRFD 5.8.3.4.2. The minimum angle \( \theta \) shall be 25 degrees.

Compute effective shear depth

Effective depth from extreme compression fiber to the centroid of the tensile force (mild steel reinforcement is neglected)

\[ d_{ei} := d_{pi} \quad \text{LRFD 5.8.2.9} \]

Check if sectional shear model is appropriate. If not, use strut and tie.

\[ \text{chk} := \begin{cases} \text{if } \left( \frac{L}{2} \geq 2 \cdot d_{ei}, "OK", "NG" \right) = "OK" \quad \text{LRFD 5.8.1.1} \\ \end{cases} \]

Distance between resultants of tensile and compressive flexure forces

\[ d_{VI} := \frac{M_{ii}}{A_{ps} \cdot t_{ps}} \quad \text{LRFD C5.8.2.9-1} \]

Section total depth

\[ h := d_g + t_s = 81.0 \text{ in} \]

Effective shear depth

\[ d_{vi} := \max(d_{VI}, 0.9 \cdot d_{ei}, 0.72 \cdot h) \quad \text{LRFD 5.8.2.9} \]

\[ \begin{array}{c|c} \text{Row} & \text{Value} \\ \hline 1 & 58.579 \\ 2 & 59.288 \\ \ldots & \end{array} \]

\[ \begin{array}{c|c} \text{Row} & \text{Value} \\ \hline 1 & 0.00 \\ 2 & 57.52 \\ 3 & \ldots \end{array} \]

\[ \begin{array}{c|c} \text{Row} & \text{Value} \\ \hline 1 & 58.32 \\ 2 & 58.32 \\ 3 & \ldots \end{array} \]
Critical Section Location

Distance to critical section from support when reaction force introduces compression into the end region (use centerline of support instead of face to be conservative)

Modify d_{est} above to recompute section forces and stresses at the correct critical section for shear, if necessary.

10.3 Shear Design

Calculate longitudinal strain

Angle of harped strands inclination

\[ \theta_{harp} := \arctan(slope_h) = 5.420 \text{ deg} \]

Effective PS Force in harped strands

\[ P_{h_i} := f_p \cdot \text{TRAN} \cdot A_p \cdot N_h \]

Vert component of Eff PS Force in harp strnds

\[ V_{P_i} := \begin{cases} 0 \text{kip} & \text{if } 0.4 \text{-GL} \leq SE_i \leq 0.6 \text{-GL} \\ P_{h_i} \cdot \sin(\theta_{harp}) & \text{otherwise} \end{cases} \]

For usual levels of prestressing

\[ f_{po} := 0.7 \cdot f_{pu} = 189.0 \text{-ksi} \]

Factored axial force (positive for tension)

\[ N_u := 0.0 \text{-kip} \]
\[ p_h = \begin{array}{c|c}
7 & 451.6 - \text{kip} \\
8 & 451.6 - \text{kip} \\
9 & 451.6 - \text{kip} \\
10 & 451.6 - \text{kip} \\
11 & 451.6 - \text{kip} \\
12 & 451.6 - \text{kip} \\
13 & 451.6 - \text{kip} \\
14 & 451.6 - \text{kip} \\
15 & 451.6 - \text{kip} \\
16 & \ldots
\end{array} \]

\[ V_p = \begin{array}{c|c}
7 & 42.7 - \text{kip} \\
8 & 42.7 - \text{kip} \\
9 & 42.7 - \text{kip} \\
10 & 0.0 - \text{kip} \\
11 & 0.0 - \text{kip} \\
12 & 0.0 - \text{kip} \\
13 & 0.0 - \text{kip} \\
14 & 0.0 - \text{kip} \\
15 & 42.7 - \text{kip} \\
16 & \ldots
\end{array} \]

Area of prestressing steel on the flexural tension side of the member

\[ A_{psv} = \text{if} \left( t_s + Y_{tg} + EC_{i,1} \geq \frac{h}{2} \cdot N_p \cdot A_p \cdot N_s \cdot A_p \right) \]

Reduction factor for \( A_{psv} \) if strand is not fully developed at section under consideration

\[ RF_i = \frac{f_{ps_i}}{f_{psI_i}} \]

Area of non-prestressed reinforcing steel on the flexural tension side

\[ A_s = 0.0 - \text{in}^2 \]

Factored Moment - longitudinal strain calculation

\[ M_{uv,i} = \max \left( \left| M_{u,i} \right|, \left| V_{u,i} - V_{p,i} \right| \cdot d_v \right) \]

Calculated Longitudinal strain

\[ \varepsilon_{s,i} = \min \left( \max \left( \frac{M_{uv,i}}{d_v}, 0.5 \cdot N_u + \left| V_{u,i} - V_{p,i} - A_{psv,i} \cdot RF_i \cdot f_{po} \cdot TRAN_{i} \right| \right), 0.006 \right) \]

For sections closer than \( d_y \) to the face of the support, the strain at \( d_y \) may be used

\[ \varepsilon_{s,i} = \begin{cases} 
\varepsilon_{s,rc} & \text{if } SE_{i} \leq SE_{rc} \\
\varepsilon_{rows(SE) - rc + 1} & \text{if } SE_{i} \geq SE_{rows(SE) - rc + 1} \\
\varepsilon_{s,i} & \text{otherwise}
\end{cases} \]
Theta and beta factors for shear

Angle of inclination of diagonal compressive stresses \[ \theta_i = \left( \frac{29 + 3500 \varepsilon_{SI}}{1 + 750 \varepsilon_{SI}} \right) \text{deg} \]

Factor indicating ability of diagonally cracked concrete to transmit tension for sections containing at least the minimum amount of transverse reinforcement

Factor of safety of the diagonal compressive stresses \[ \beta_i = \left( \frac{4.8}{1 + 750 \varepsilon_{SI}} \right) \]

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Nominal Shear Resistance

Effective girder web width \[ b_v := b_w = 6.125 \text{ in} \]

Area of shear reinforcement within a distance "s" \[ A_v := 2 \cdot \text{area (bar)} = 0.618 \text{ in}^2 \]

Stirrup spacing at each section. If section is in the first or last stirrup zones (the clearance to the first set of stirrups from the ends of the girder) then use the spacing for the adjacent zone.
Concrete Structures

Chapter 5

Concrete Structures Chapter 5

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\[ s := \begin{cases} \text{return VR}_{2,2} & \text{if } SE_1 \leq VR_{1,1} \\ \text{return VR}_{\text{rows(VR)}-1,2} & \text{if } SE_i \geq GL - VR_{\text{rows(VR)}},1 \\ \text{for } k \in 1..\text{rows(VR)} & \text{return VR}_{k,2} & \text{if } SE_i \leq \sum_{j=1}^{k} VR_{j,1} \end{cases} \]

Nominal shear resistance provided by tensile stress in concrete

\[ V_{c,i} := 0.0316 \cdot \beta_i \cdot \sqrt{\frac{f'c}{\text{ksi}}} \cdot b_v \cdot d_{v,i} \]

Nominal shear resistance provided by transverse reinforcement (LRFD 5.8.3.3)

\[ V_{s,i} := \frac{A_v f_y \cdot d_{v,i} \cdot \cot(\theta)}{s_i} \]

Design shear resistance

\[ V_n := \min \left( \frac{V_{c,i} + V_{s,i} + V_{p,i}}{s_i}, \frac{0.25 f'c \cdot b_v \cdot d_{v,i} + V_{p,i}}{s_i} \right) \]

\[
\begin{array}{|c|c|}
\hline
\text{V}_c & \text{V}_s & \text{V}_n \\
\hline
8 & 174.3 & 1560.5 \\
9 & 147.6 & 2531 & 325.1 \\
10 & 128.1 & 3330 & 525.7 \\
11 & 127.4 & 4561 & 537.4 \\
12 & 125.0 & 4635 & 483.7 \\
13 & 127.4 & 4800 & 396.0 \\
14 & 128.1 & 390.0 & 439.0 \\
15 & 147.6 & 336.8 & 336.8 \\
16 & ... & ... & ... \\
\hline
\end{array}
\]
Check adequacy in shear

\[
\text{Minimum Transverse Reinforcement}
\]

Min shear reinforcement (LRFD 5.8.2.5)

\[
A_{v,\text{min}_i} := 0.0316 \cdot \frac{f_c}{\text{ksi}} \cdot \frac{b_v s_i}{f_y}
\]

Check minimum reinforcement limit

\[
\text{Maximum Spacing of Transverse Reinforcement}
\]
Concrete Structures Chapter 5

Shear stress on concrete
\[ v_{u_i} := \frac{V_{u_i} - \phi_v V_{p_i}}{\phi_v b_v d_{v_i}} \]

Max shear reinforcement spacing
\[ s_{\text{max}_i} := \begin{cases} \min(0.8 d_{v_i}, 18 \text{in}) & \text{if } v_{u_i} < 0.125 f_c \\ \min(0.4 d_{v_i}, 12 \text{in}) & \text{otherwise} \end{cases} \]

Check maximum shear reinforcement spacing

\[ v_u = 0.570 \text{ ksi} \]

\[ s_{\text{max}} = 8.0 \text{ in} \]

10.4 Longitudinal Reinforcement

Resistance Factor for axial load (compression)
\[ \phi_{cN} := \phi_c = 0.75 \]

Required area of prestressing
\[ A_{\text{ps,req}_i} := \begin{cases} a \leftarrow 0\text{in}^2 & \text{if } SE_i = 0\text{ft} \lor SE_i = \text{GL} \\ a \leftarrow \frac{M_{u_i}}{d_{v_i} \phi_i} + 0.5 \cdot \frac{N_u}{\phi_{cN}} + \left( \frac{V_{u_i}}{\phi_v} - V_{p_i} \right) - 0.5 \cdot \min \left( V_{s_i}, \frac{V_{u_i}}{\phi_v} \right) \cdot \cot\left( \theta_i \right) \cdot \frac{1}{f_{ps_i}} & \text{if } SE_{rc} \leq SE_i \leq GL - SE_{rc} \\ a \leftarrow \left( \frac{V_{u_i}}{\phi_v} - 0.5 \cdot \min \left( V_{s_i}, \frac{V_{u_i}}{\phi_v} \right) - V_{p_i} \right) \cdot \frac{1}{f_{ps_i}} \cdot \cot\left( \theta_i \right) \cdot \frac{1}{f_{ps_i}} & \text{otherwise} \end{cases} \]

Check if required area is provided

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10.5 Horizontal Interface Shear between Girder and Slab

It is conservative to compute the interface shear force using the full factored loading applied to the composite BDM 5.2.2.C deck slab and girder. Compute actual shear stress using mechanics of materials rather than use AASHTO LRFD 5.8.4.2.

First Moment of Transformed Slab from Neutral Axis

\[ Q_{\text{slab}} := A_{\text{slab}} \left( Y_t + \frac{t_s}{2} \right) = 10763.3 \text{ in}^3 \]

Permanent Net Compressive Force Normal to the Shear Plane

\[ P_c := w_{cs} \cdot t_s \cdot b_e = 0.588 \text{ kip/ft} \]

Area of stirrups crossing interface per foot

\[ a_{vf_i} := \frac{A_v}{s_i} \]

Shear Force at Girder/Slab Interface per foot

\[ V_{ui_i} := \frac{V_{ui} \cdot Q_{\text{slab}}}{I_c} \]

Cohesion Factor

\[ c_{vi} := 0.28 \text{ ksi} \]

Friction Factor

\[ \mu := 1.0 \]

Fraction of Concrete Strength Available

\[ K_{1vi} := 0.3 \]

Limiting Interface Shear Resistance

\[ K_2 := 1.8 \text{ ksi} \]

Nominal Interface Shear Resistance

\[ V_{ni_i} := \min \left[ c_{vi} b_f + \mu \left( a_{vf_i} f_y + P_c \right), K_{1vi} f_{cs} b_f, K_2 b_f \right] \]

Factored Interface Shear Resistance

\[ V_{ri_i} := \phi_V V_{ni_i} \]
Check adequacy in interface shear

\[
\text{chk} \begin{align*}
0, 7 & \Rightarrow \begin{cases} 
\text{CH} & \leftarrow \text{"OK"} = \text{"OK"} \quad \text{for } i \in 1..\text{rows(SE)} \\
\text{CH} & \leftarrow \text{"NG"} \text{ if } V_{\text{ri}_i} < V_{\text{ui}_i} \\
\text{CH} & = \text{CH} 
\end{cases}
\end{align*}
\]

Check stirrup spacing adequacy

\[
\text{chk} \begin{align*}
0, 8 & \Rightarrow \begin{cases} 
\text{CH} & \leftarrow \text{"OK"} = \text{"OK"} \quad \text{for } i \in 1..\text{rows(SE)} \\
\text{CH} & \leftarrow \text{"NG"} \text{ if } 24\text{in} < s_i \\
\text{CH} & = \text{CH} 
\end{cases}
\end{align*}
\]

Minimum Area of Interface Shear Reinforcement

\[
a_{\text{vf, min}_i} = \begin{aligned}
& \text{return } 0 \text{ in}^2/\text{ft} \quad \text{if } V_{\text{ui}_i}/b_f < 0.210\text{ksi} \\
& \min \left[ 0.05 \cdot b_f, \frac{1}{f_y} \cdot \left[ 1 + \frac{1.33 \cdot V_{\text{ui}_i}}{\phi_v} - c_{\text{vi}} b_f - P_c \right] \cdot \frac{\text{in}^2}{\text{ft}} \right] \quad \text{otherwise}
\end{aligned}
\]

Check minimum area of interface shear reinforcement

\[
\text{chk} \begin{align*}
0, 9 & \Rightarrow \begin{cases} 
\text{CH} & \leftarrow \text{"OK"} = \text{"OK"} \quad \text{for } i \in 1..\text{rows(SE)} \\
\text{CH} & \leftarrow \text{"NG"} \text{ if } a_{\text{vf}_i} < a_{\text{vf, min}_i} \\
\text{CH} & = \text{CH} 
\end{cases}
\end{align*}
\]

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10.6 Pretensioned Anchorage Zone

Factored Splitting Resistance

Distance from end contributing to splitting resistance

\[ l_{\text{split}} := \frac{d_g}{4} = 18.50 \text{ in} \]

Total area of vertical reinforcement located within bursting length \( h/4 \)

\[ A_{s,\text{burst}} := \text{for } i \in 1.. \text{rows(VR)} \]
\[ x \leftarrow i \text{ if } l_{\text{split}} > \sum_{j=1}^{i} VR_{j,1} \]
\[ \text{for } i \in 1..x+1 \]
\[ AV \leftarrow AV + A_{v,\text{ceil}} \left( \frac{VR_{i,1}}{VR_{i,2}} \right) \text{ if } i \leq x \]
\[ AV \leftarrow AV + A_{v,\text{floor}} \left( \frac{l_{\text{split}} - \sum_{j=1}^{i-1} VR_{j,1}}{VR_{i,2}} \right) \text{ otherwise} \]

Maximum stress in steel

\[ f_s := 20 \text{ ksi} \]

Splitting Resistance

\[ P_r := f_s \cdot A_{s,\text{burst}} = 86.52 \text{ kip} \]

Minimum required splitting resistance

\[ P_{r,\text{min}} := 0.04 \cdot f_{pbt} \cdot A_{pstemp} = 76.58 \text{ kip} \]

Check if adequate splitting resistance is required. If not, required additional reinforcement can be provided at 2.5\(^\circ\) spacing beyond the bursting length.

Confinement Reinforcement

Confinement reinforcement shall be provided at the ends of beams to confine the prestressing steel in the bottom flange.

Minimum length of PS confinement in bottom flange

\[ l_{\text{confin}} := 1.5d_g = 9.250 \text{ ft} \]
11. **Deflection and Camber**

Positive deflection is defined upward (in direction of camber).

### 11.1 Deflections Due to Prestress

The following function finds camber induced by straight strands, where:

- \( P \) = Prestressing Force
- \( e \) = Eccentricity of Prestressing Force from C.G. (positive upwards)
- \( E \) = Modulus of Elasticity
- \( I \) = Moment of Inertia
- \( x \) = Distance from left support to compute deflection
- \( L \) = Span Length between supports

\[
\text{Straight}\Delta(P, e, E, I, x, L) := \begin{cases} 
0 & \text{if } x < 0 \text{ in } x > L \\
\frac{P e}{2 E I} (x - L) & \text{otherwise}
\end{cases}
\]

The following function finds camber induced by harped strands, where:

- \( P \) = Prestressing Force
- \( e_1 \) = Eccentricity of Straight Midspan Portion of Prestressing Force from C.G. (positive upwards)
- \( e_2 \) = Eccentricity of Prestressing Force at support from C.G. (positive upwards)
- \( E \) = Modulus of Elasticity
- \( I \) = Moment of Inertia
- \( x \) = Distance from left support to compute deflection
- \( L \) = Span Length between supports
- \( b \) = Distance between support and harp point (assumed symmetrical)

\[
\text{Harp}\Delta(P, e_1, e_2, E, I, x, L, b) := \begin{cases} 
\text{return } 0 & \text{if } x < 0 \text{ in } x > L \\
e \leftarrow (e_2 - e_1) \\
\text{return } \frac{P e x}{6 E I b} \left( x^2 + 3 b^2 - 3 b L \right) + \frac{P e_2}{2 E I} x (x - L) & \text{if } x \le b \\
\text{return } \frac{P e x}{6 E I b} \left( 3x^2 + b^2 - 3 L x \right) + \frac{P e_2}{2 E I} x (x - L) & \text{if } b < x < L - b \\
\text{return } \frac{P e (L - x)}{6 E I b} \left[ (L - x)^2 + 3 b^2 - 3 b L \right] + \frac{P e_2}{2 E I} x (x - L) & \text{if } L - b \le x
\end{cases}
\]

Deflections due to straight strands

\[
\Delta S_i := \text{Straight}\Delta \left( f_{peP1} N_S A_p, -e_s, E_{ci}, I_g, S E_i - P2, L \right)
\]

Deflections due to temporary strands

\[
\Delta T_i := \text{Straight}\Delta \left( f_{peT1} N_t A_p, -e_{temp}, E_{ci}, I_g, S E_i - P2, L \right)
\]

Deflections due to release of temporary strands

\[
\Delta TR_i := -\text{Straight}\Delta \left( f_{peT1} + \Delta f_{pL, TH} N_t A_p, -e_{temp}, E_{ci}, I_g, S E_i - P2, L \right)
\]

Deflections due to harp strands

\[
\Delta H_i := \text{Harp}\Delta \left( f_{peP1} N_h A_p, -E_{C_{rm}, 1}, -E_{C_{rs}, L, 1}, E_{ci}, I_g, S E_i - P2, L, x_h - P2 \right)
\]
11.2 Deflections due to Dead Loads

The following function returns the deflection of a simple span due to a concentrated load at any point:

\[ \Delta \text{POINT}(P, a, x, L, E, I) := \begin{cases} 
0 & \text{if } x < 0 \text{in} \lor x > L \\
0 & \text{if } a < 0 \text{in} \lor a > L \\
\frac{P \cdot (L - a) \cdot x}{6 \cdot E \cdot I \cdot L} \cdot \left[ L^2 - (L - a)^2 - x^2 \right] & \text{if } x < a \\
\frac{P \cdot a \cdot (L - a)^2}{3 \cdot E \cdot I \cdot L} & \text{if } x = a \\
\frac{P \cdot a \cdot (L - x)}{6 \cdot E \cdot I \cdot L} \cdot \left[ L^2 - a^2 - (L - x)^2 \right] & \text{otherwise}
\end{cases} \]

The following function returns the deflection of a simple span due to a uniform load:

\[ \Delta \text{TR}(w, x, L, E, I) := \frac{w \cdot x^2}{2 \cdot E \cdot I \cdot L} \cdot \left[ L^2 - x^2 \right] \]

\[ \Delta \text{H}(w, x, L, E, I) := \frac{w \cdot x^2}{2 \cdot E \cdot I \cdot L} \cdot \left[ a^2 - (a - x)^2 \right] \]

\[ \Delta \text{S}(P, a, x, L, E, I) := \frac{P \cdot a \cdot (L - a)^2}{6 \cdot E \cdot I \cdot L} \cdot \left[ L^2 - a^2 - (L - x)^2 \right] \]

\[ \Delta \text{T}(w, x, L, E, I) := \frac{w \cdot x^2}{2 \cdot E \cdot I \cdot L} \cdot \left[ a^2 - (a - x)^2 \right] \]

\[ \Delta \text{H}(w, x, L, E, I) := \frac{w \cdot x^2}{2 \cdot E \cdot I \cdot L} \cdot \left[ a^2 - (a - x)^2 \right] \]
\[ \Delta_{\text{UNIFORM}}(w, x, L, E, I) := \begin{cases} 0 & \text{if } x < 0 \text{ or } x > L \\ \frac{wx}{24E I} \left( L^3 - 2Lx^2 + x^3 \right) & \text{otherwise} \end{cases} \]

Deflection Due to girder dead load
\[ \Delta G := -\Delta_{\text{UNIFORM}}(w_g, SE_i - P2, L, E_c, I_g) \]

Deflection Due to pad and slab dead load
\[ \Delta SL_i := -\Delta_{\text{UNIFORM}}(w_p u + w_s, SE_i - P2, L, E_c, I_g) \]

Deflection Due to barrier dead load
\[ \Delta BAR_i := -\Delta_{\text{UNIFORM}}(w_h, SE_i - P2, L, E_c, I_c) \]

Due to intermediate diaphragms
\[ \Delta DIA := \begin{cases} \text{for } i \in rsL..rsR \\ a \leftarrow 0 \text{ft} \\ \Delta_i \leftarrow 0 \text{in} \\ \text{for } j \in 1..n_{\text{dia}} \\ a \leftarrow a + \text{DiaSpacing} \\ \Delta_i \leftarrow \Delta_i - \Delta_{\text{POINT}}(\text{DiaWt, a, } SE_i - P2, L, E_c, I_g) \\ \Delta_{\text{row}(SE)} \leftarrow 0 \text{in} \end{cases} \]

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\[ \Delta G = \text{in} \quad \Delta SL = \text{in} \quad \Delta BAR = \text{in} \quad \Delta DIA = \text{in} \quad \Delta P = \text{in} \]

11.3 Deflections Due to Creep

The following functions determine the creep coefficient where
\[ t = \text{Maturity of Concrete (days), age of concrete between time of loading and time for analysis of creep effect} \]
\[ t_1 = \text{Age of concrete (days) at time of load application} \]
Chapter 5 Concrete Structures

\( f = \) Specified compressive strength of concrete at time of prestressing

**Volume/Surface Area Factor**

\[ k_s := \max \left( 1.45 - 0.13 \times \frac{V_{S_f}}{in}, 1.0 \right) = 1.035 \]

**Humidity Factor**

\[ k_{hc} := 1.56 - 0.008 \left( \frac{H}{\%}\right) = 0.960 \]

**Concrete Strength Factor**

\[ k_f(f) := \frac{5}{1 + \frac{f}{ksi}} \]

**Time Development Factor**

\[ k_{td}(t, f) := \frac{t}{61 - 4 \left( \frac{f}{ksi}\right) + \frac{t}{day}} \]

**Creep Coefficient**

\[ \psi_{cr}(t, t_i, f) := 1.9 \times k_s \times k_{hc} \times k_f(f) \times k_{td}(t, f) \left( \frac{t_i}{day}\right)^{-0.118} \]

**Time Intervals for Construction and Creep Coefficients**

**Note:** 1 day of accelerated curing is treated as 7 days for concrete creep

<table>
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<th>Time Intervals (days)</th>
<th>Construction Timing</th>
<th>Time Intervals (days)</th>
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<td>Maximum timing ( D_{120} )</td>
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1 - Casting Girder to Releasing Strands
2 - Releasing Strands to Cutting Temporary Strands and Casting Diaphragms
3 - Releasing Strands to Placing Deck

**Creep Coefficients for Minimum timing**

\( \psi_{10.7} := \psi_{cr}(10\text{day}, 7\text{day}, f_{ci}) = 0.215 \)

\( \psi_{40.7} := \psi_{cr}(40\text{day}, 7\text{day}, f_{ci}) = 0.497 \)

\( \psi_{30.10} := \psi_{cr}(30\text{day}, 10\text{day}, f_{ci}) = 0.399 \)

**Creep Coefficients for Maximum timing**

\( \psi_{90.7} := \psi_{cr}(90\text{day}, 7\text{day}, f_{ci}) = 0.657 \)

\( \psi_{120.7} := \psi_{cr}(120\text{day}, 7\text{day}, f_{ci}) = 0.702 \)

\( \psi_{30.90} := \psi_{cr}(30\text{day}, 90\text{day}, f_{ci}) = 0.308 \)

Deflections due to creep between girder fabrication and temp strand removal / diaphragm placement for

Minimum Timing

\[ \Delta CR_{1_{\min}} := \psi_{10.7} \left( \Delta S_i + \Delta H_i + \Delta T_i + \Delta G_j \right) \]

Deflections due to creep between girder fabrication and temp strand removal / diaphragm placement for

Maximum Timing

\[ \Delta CR_{1_{\max}} := \psi_{90.7} \left( \Delta S_i + \Delta H_i + \Delta T_i + \Delta G_j \right) \]

Deflections due to creep between temp strand removal / diaphragm placement and deck placement f0
Minimum Timing

\[ \Delta CR_{2\min_i} := (\psi_{40.7} - \psi_{10.7}) \left( \Delta S_i + \Delta H_i + \Delta T_i + \Delta G_i \right) + \psi_{30.10} \left( \Delta DIA_i + \Delta TR_i \right) \]

Deflections due to creep between temp strand removal / diaphragm placement and deck placement for Maximum Timing

\[ \Delta CR_{2\max_i} := (\psi_{120.7} - \psi_{90.7}) \left( \Delta S_i + \Delta H_i + \Delta T_i + \Delta G_i \right) + \psi_{30.90} \left( \Delta DIA_i + \Delta TR_i \right) \]

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11.4 "D" and "C" Dimensions

"D" dimension at 40 days

\[ D_{40_i} := \Delta S_i + \Delta T_i + \Delta H_i + \Delta G_i + \Delta CR_{1\min_i} + \Delta TR_i + \Delta DIA_i + \Delta CR_{2\min_i} \]

"D" dimension at 120 days

\[ D_{120_i} := \Delta S_i + \Delta T_i + \Delta H_i + \Delta G_i + \Delta CR_{1\max_i} + \Delta TR_i + \Delta DIA_i + \Delta CR_{2\max_i} \]

Screed setting dimension "C" = - elastic deflection due to slab, traffic barrier, and overlay on noncomposite

\[ C := - (\Delta SL_i + \Delta BAR_i) \]

Excess girder camber at 120 days to find "A" dim.

\[ \Delta EXCESS_{120_i} := D_{120_i} - C_i \]

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11.5 Deflections Due to Live Load

Live load deflection criteria is based upon the following:
1. The vehicular load shall include the dynamic load allowance of LRFD 3.6.2.1
2. The live load deflection should be taken as the larger of (LRFD 3.6.1.3.2):
   That resulting from the design truck alone, or
   that resulting from 25% of the design truck taken together with the design lane load
3. The provision of LRFD 3.6.1.1.2 (multiple presence of live load) shall be applied.
4. For straight girder systems, all design lanes should be loaded and all supporting elements should be assumed to deflect equally.
5. For composite design, the stiffness of the design cross-section should include the entire width of the roadway and the structurally continuous portions of the barriers. For simplicity and to be conservative, neglect the barriers.

Live load deflection limit (Vehicular Bridge) \( \Delta_{LL,\text{lim}} = \frac{L}{800} = 1.950\text{-in} \)

Composite Section Properties for Entire Superstructure

Slab transformed flange width \( b_{\text{slab, trans}} := (B + 2\cdot CW)\cdot n = 311.51\text{-in} \)

Slab moment of inertia (transformed) \( I_{\text{slab2}} := \frac{b_{\text{slab, trans}}^3}{3} \div 12 = 8904.1\text{-in}^4 \)

Area of slab (transformed) \( A_{\text{slab2}} := \frac{b_{\text{slab, trans}}^3}{4} = 2180.6\text{-in}^2 \)

c.g. of slab to bottom of girder \( y_{bs} = 77.500\text{-in} \)

Lower Bound D at 40 Days \( 0.5\cdot D_{40_{\text{rm}}} = 1.067\text{-in} \)

Upper Bound D at 120 Days \( D_{120_{\text{rm}}} = 2.282\text{-in} \)

Screed Camber "C" \( C_{\text{rm}} = 1.375\text{-in} \)

Check that Final Excess Camber is less than that assumed to estimate the A dimension. A dimension estimate should be revised if necessary.
c.g. to bottom of girder

\[ Y_{b2} := \frac{A_{slab2} \cdot Y_{bs} + N_b \cdot A_g \cdot Y_{bg}}{A_{slab2} + N_b \cdot A_g} = 47.48 \text{ in} \]

c.g. to top of girder

\[ Y_{t2} := d_g - Y_{b2} = 26.52 \text{ in} \]

c.g. to top of slab

\[ Y_{ts2} := t_s + Y_{t2} = 33.52 \text{ in} \]

Slab moment of inertia about composite N.A.

\[ I_{slab2} := A_{slab2} \left( Y_{ts2} - 0.5t_s \right)^2 + I_{slab2} = 1974622 \cdot \text{in}^4 \]

Girder moment of inertia about composite N.A.

\[ I_{gc2} := N_b \cdot A_g \left( Y_{b2} - Y_{bg} \right)^2 + N_b \cdot I_g = 5179723 \cdot \text{in}^4 \]

Composite section moment of inertia

\[ I_{c2} := I_{slab2} + I_{gc2} = 7154345 \cdot \text{in}^4 \]

**Maximum Live Load Deflection due to Design Truck**

The following function finds the maximum deflection due to an AASHTO HL93 Truck Load at a section a distance "x" along a simple span of length "L". A truck moving both directions is checked.

\[
\text{HL93Truck}(\Delta, x, L) := \begin{cases} 
\text{Axles} & \left[ \begin{array}{c}
8\text{kip} \\
32\text{kip} \\
32\text{kip}
\end{array} \right] \\
\text{Locations} & \left[ \begin{array}{c}
0\text{ft} \\
-14\text{ft} \\
-28\text{ft}
\end{array} \right] \\
\text{rows} & \text{rows(Locations)} \\
\text{Loc} & \text{Locations} \\
\text{Deflection} & 0\text{in} \\
\text{while} & \text{Loc}_{\text{rows}} \leq L \\
\text{for} & i \in 1..\text{rows} \\
\Delta_i & \Delta_{\text{POINT}}(\text{Axles}_i, \text{Loc}_i, x, L, E_c, I_{c2}) \\
\text{Loc}_i & \text{Loc}_i + 0.01\text{ft} \\
\text{Deflection} & \max \left( \sum \Delta, \text{Deflection} \right) \\
\text{Loc} & \text{Locations} \\
\text{x} & L - x \\
\text{while} & \text{Loc}_{\text{rows}} \leq L \\
\text{for} & i \in 1..\text{rows} \\
\Delta_i & \Delta_{\text{POINT}}(\text{Axles}_i, \text{Loc}_i, x, L, E_c, I_{c2}) \\
\text{Loc}_i & \text{Loc}_i + 0.01\text{ft} \\
\text{Deflection} & \max \left( \sum \Delta, \text{Deflection} \right) \\
\text{Deflection} & \end{cases}
\]
Deflections due to one truck loading on entire superstructure

$$\Delta\text{TRUCK}_i := \text{HL93Truck} \Delta \left( SE_i - P_2, L \right)$$

Deflections due to one lane loading on entire superstructure

$$\Delta\text{LANE}_i := \Delta\text{UNIFORM} \left( w_{\text{lane}}, SE_i - P_2, L, E_c, t; 2 \right)$$

Maximum Superstructure Deflections

$$\Delta\text{SUPER}_i := N_L \cdot m_p \cdot \max \left[ \Delta\text{TRUCK}_i \cdot (1 + IM), 0.25 \cdot \Delta\text{TRUCK}_i \cdot (1 + IM) + \Delta\text{LANE}_i \right]$$

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<td>16</td>
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<td>16</td>
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Check LL Deflection Limit

$$\text{chk}_{1, 2} := \text{if} \left( \max (\Delta\text{SUPER}) < \Delta_{LL\lim} \right. $$ "OK", "NG") = “OK”
12. Lifting, Shipping, and General Stability

12.1 Lifting Stresses

Dead load bending moment and stress

Impact is not applied during the lifting stage

BDM 5.6.2 C.2.

Dead load moments during lifting

\[ M_{\text{Lift}i} := M_{\text{cant}}(w_g \cdot L_1 \cdot L_1 \cdot GL - 2L_1 \cdot SE_i) \]

Dead load stresses at top of girder during lifting

\[ ST_{\text{Lift}i} := \frac{M_{\text{Lift}i}}{S_{\text{tg}}} \]

Dead load stresses at bottom of girder during lifting

\[ SB_{\text{Lift}i} := \frac{M_{\text{Lift}i}}{S_{\text{bg}}} \]

<table>
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<tr>
<th>( i )</th>
<th>( M_{\text{Lift}i} ) (kip ft)</th>
<th>( ST_{\text{Lift}i} ) (ksi)</th>
<th>( SB_{\text{Lift}i} ) (ksi)</th>
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<td>-1.265</td>
<td>1.176</td>
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<td>1.124</td>
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Service I for Casting Yard Stage (At Lifting)

Effective Prestress in Permanent Strands

\[ f_{\text{pLift}} = f_{\text{pj}} + \Delta f_{\text{pR0}} + \Delta f_{\text{pES}} = 187.5 \text{ ksi} \]

Effective Prestress in Temporary Strands

\[ f_{\text{pT.Lift}} = f_{\text{pj}} + \Delta f_{\text{pR0}} + \Delta f_{\text{pEST}} = 193.8 \text{ ksi} \]

Stress in girder due to prestressing:

<table>
<thead>
<tr>
<th>Top Stress</th>
<th>Bottom Stress</th>
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<tbody>
<tr>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>
### Chapter 5 Concrete Structures

#### 12.1 Lifting Stresses

Find total Service I stress which includes:
- Prestress
- Girder Dead Load between lift points

**STRESS\text{Lift} :=**
\[
\begin{align*}
\text{STR}_{i,1} & := \text{PSLift}_{i,1} + \text{STLift}_i \\
\text{STR}_{i,2} & := \text{PSLift}_{i,2} + \text{SBLift}_i
\end{align*}
\]

<table>
<thead>
<tr>
<th></th>
<th>Top Stress</th>
<th>Bottom Stress</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
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</tr>
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<td>-0.546</td>
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</tr>
<tr>
<td>9</td>
<td>-0.169</td>
<td>-3.601</td>
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<tr>
<td>10</td>
<td>0.197</td>
<td>-3.942</td>
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<tr>
<td>11</td>
<td>0.197</td>
<td>-3.942</td>
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<tr>
<td>12</td>
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<td>-3.601</td>
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<td>16</td>
<td>-0.546</td>
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</tbody>
</table>

Maximum compressive stress allowed:
\[ f_{c,TL..lim} = -4.875 \text{ksi} \]

Maximum tensile stress allowed:
\[ f_{t,TL..lim} = 0.520 \text{ksi} \]

Check compressive stress

\[ \text{chk}_{2,1} := \text{if } \left( \min(\text{STRESS}_{\text{Lift}}) \geq f_{c,TL..lim} \right) \text{ "OK", "NG" } = \text{ "OK"} \]
Check tensile stress (with bonded reinforcement)

$$\text{chk}_{2.2} := \text{if } \left( \max \{\text{STRESS}_{\text{Lift}}\} \leq f_{\text{LT, lim}} \right) \text{"OK", "NG" } = \text{"OK"}$$

### 12.2 Girder Stability During Lifting

**References**
1. PCI Journal Jan/Feb 1989 and Jan/Feb 1993, Lateral Stability of Long Prestressed Concrete Beams Parts 1 and 2, Robert F. Mast
2. PCI Journal Jul/Aug 1998, New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girder Bridges
3. PCI Journal Fall 2009, Design Optimization for Fabrication of Pretensioned Concrete Bridge Girders
4. BDM 5.6.3.C.2

Length of girder between lift points

$$L_{\text{Lift}} := GL - 2L_1 = 123.95 \text{ ft}$$

Initial eccentricity caused by lift loop placement tolerance

$$e_{\text{lift}} := 0.25 \text{ in}$$

Std. Spec. 6-02.3(25)I
BDM 5.6.3.C.2

Initial eccentricity caused by sweep (horizontal) tolerance at midspan of girder

$$e_{\text{sweep}} := \frac{0.125 \sin 0.688 \text{ in}}{10 \text{ ft}} = 0.837 \text{ in}$$

Std. Spec. 6-02.3(25)J
BDM 5.6.3.C.2

Offset Factor that determines the distance between the roll axis and the c.g. of the arc of a curved girder

$$F_{oL} := \left( \frac{L_{\text{Lift}}}{GL} \right)^2 - \frac{1}{3} = 0.523$$

Initial eccentricity of the c.g. from the roll axis

$$e_i := e_{\text{lift}} + e_{\text{sweep}} F_{oL} = 0.688 \text{ in}$$

Downward deflection due to self weight (midspan). The first term is deflection caused by self weight between lifting supports. The second term is deflection caused by overhangs.

$$\Delta_{\text{self}} := -\Delta_{\text{UNIFORM}} \left( \frac{w_g \cdot SE_{\text{rm}}}{L_1 \cdot L_{\text{Lift}} \cdot E_{\text{ci}} \cdot I_{\text{g}}} \right) + \frac{w_g L_1^2 \cdot L_{\text{Lift}}^2}{16 \cdot E_{\text{ci}} \cdot I_{\text{g}}} = -1.377 \text{ in}$$

Deflection due to prestress (midspan)

$$\Delta_{\text{ps}} := \text{Straight} \Delta \left( f_{\text{peP1}} N_s A_p - e_s E_{\text{ci}} I_{\text{g}} \cdot SE_{\text{rm}} \cdot GL \right) \ldots = 2.769 \text{ in}$$

$$+ \text{Straight} \Delta \left( f_{\text{peT1}} N_t A_p - e_{\text{temp},E_{\text{ci}} I_{\text{g}} \cdot SE_{\text{rm}} \cdot GL} \right) \ldots$$

$$+ \text{Harp} \Delta \left( f_{\text{peP1}} N_h A_p - E_{\text{cm},1} E_{\text{ci}} I_{\text{g}} \cdot SE_{\text{rm}} \cdot GL \cdot \chi_h \right)$$

Vertical distance from the roll center to the c.g.

$$y_r := Y_{ig} - \left( \Delta_{\text{self}} + \Delta_{\text{ps}} \right) F_{oL} = 37.612 \text{ in}$$

Initial roll angle of a rigid beam

$$\theta_i := \frac{e_i}{y_r} = 0.018 \text{ rad}$$

Theoretical deflection at the girder c.g. assuming full weight is applied about the weak axis

$$z_o := \frac{w_g}{12 E_{ci} I_y \cdot GL} \left( \frac{1}{10} L_{\text{Lift}}^5 - L_1^2 L_{\text{Lift}}^3 + 3 \cdot L_1^4 L_{\text{Lift}}^5 + \frac{6}{5} L_1^5 \right)$$

$$z_o = 8.243 \text{ in}$$

Lateral bending moment to cause cracking in corner of top or bottom flange from biaxial bending

$$M_{\text{lat},i} := \min \left( \left( f_r L - \text{STRESS}_{\text{Lift},i,1} \right) \frac{2I_y}{b_f}, \left( f_r L - \text{STRESS}_{\text{Lift},i,2} \right) \frac{2I_y}{b_f \cdot \text{bot}} \right)$$
Tilt angle at cracking

\[ \theta_{\text{max},i} := \begin{cases} \text{return } \min \left( \frac{M_{\text{lat},l}}{M_{\text{Lift},l}}, \frac{\pi}{2} \right) & \text{if } SE_i \leq SE_{rl_1} \\ \text{return } \min \left( \frac{M_{\text{lat},l}}{M_{\text{Lift},l}}, \frac{\pi}{2} \right) & \text{if } SE_{rl_1} < SE_i < SE_{rl_2} \\ \text{return } \min \left( \frac{M_{\text{lat},l}}{M_{\text{Lift},l}}, \frac{\pi}{2} \right) & \text{if } SE_i \geq SE_{rl_2} \end{cases} \]

Factor of Safety against cracking during lifting

\[ FS_{cr,i} := \left( \frac{z_o}{y_r} + \frac{\theta_i}{\theta_{\text{max},i}} \right)^{-1} \]

<table>
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<tr>
<th>i</th>
<th>( M_{\text{lat},i} ) (kip·ft)</th>
<th>( \theta_{\text{max},i} ) (rad)</th>
<th>FS_{cr,i}</th>
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Check if minimum FS against cracking is greater than 1.0

\[ \text{chk}_{2,i} := \text{if } (\min(FS_{cr,i}) \geq 1.0, \text{"OK", "NG"}) = \text{"OK"} \]

Tilt angle at which the maximum FS against failure occurs

\[ \theta'_{\text{max}} := \frac{e_i}{2.5 \cdot z_o} = 0.1827 \text{ rad} \]

Effective theoretical deflection

\[ z'_o := z_o \left( 1 + 2.5 \theta'_{\text{max}} \right) = 12.008 \text{ in} \]

Maximum Factor of Safety against failure

\[ FS_i := \frac{y_r \cdot \theta'_{\text{max}}}{z'_o} = 2.385 \]

If Maximum FS against failure is less than the minimum FS against cracking, then set it equal to

\[ FS_i := \max(\min(FS_{cr,i}), FS_i) = 3.262 \]
the minimum FS against cracking

Check lifting

\[
\text{chk}_{2,4} := \text{if} \left( FS_f \geq 1.5, "OK", "NG" \right) = "OK"
\]

### 12.3 Shipping Weight and Stresses

**Girder weight limit for truck shipping**

**Total weight**

\[ W_g := w_g \cdot GL = 141.7 \text{ kip} \]

Check allowable shipping weight (BDM 5.6.3 D.3)

\[
\text{chk}_{2,5} := \text{if} \left( W_g \leq 240 \text{ kip}, "OK", "NG" \right) = "OK"
\]

**Dead load bending moment and stress**

Length of girder between shipping points

\[ L_S := GL - L_L - L_T = 113.95 \text{ ft} \]

Dead load moments during shipping

\[ M_{Ship_i} := M_{can} \left( w_g, L_L, L_T, L_S, SE_i \right) \]

Dead load stresses at top of girder during shipping

\[ S_{T_{Ship_i}} := \frac{M_{Ship_i}}{S_{tg}} \]

Dead load stresses at bottom of girder during shipping

\[ S_{B_{Ship_i}} := \frac{M_{Ship_i}}{S_{bg}} \]

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<td>16</td>
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**Prestressing Stresses**

Effective Prestress in Permanent Strands

\[ f_{peP.Ship} := f_{pj} + \Delta f_{PR0} + \Delta f_{PES} + \Delta f_{PLTH} = 182.2 \text{ ksi} \]
Effective Prestress in Temporary Strands

\[ f_{pt,\text{Ship}} := f_{pj} + \Delta f_{pR0} + \Delta f_{pEST} + \Delta f_{pLTH} = 188.5 \text{ ksi} \]

Stress in girder due to prestressing:

\[
\begin{align*}
PS_{\text{Ship}} := & \text{ for } i \in 1..\text{rows(SE)} \\
P_p & \leftarrow f_{peP,\text{Ship}} \cdot \text{TRAN}_i \cdot A_{ps} \\
P_t & \leftarrow f_{peT,\text{Ship}} \cdot \text{TRAN}_i \cdot A_{temp} \\
PS_{1,1} & \leftarrow \left( \frac{P_p}{A_g} - \frac{P_p \cdot E_{ci,2}}{S_{tg}} + \frac{P_t}{A_g} - \frac{P_t \cdot e_{temp}}{S_{tg}} \right) \\
PS_{1,2} & \leftarrow \left( \frac{P_p}{A_g} + \frac{P_p \cdot E_{ci,2}}{S_{bg}} + \frac{P_t}{A_g} + \frac{P_t \cdot e_{temp}}{S_{bg}} \right)
\end{align*}
\]

\[
\begin{array}{c|cc}
& \text{Top Stress} & \text{Bottom Stress} \\
\hline
1 & 0.000 & 0.000 \\
2 & -0.831 & -1.629 \\
3 & -1.235 & -2.504 \\
4 & -1.179 & -2.556 \\
5 & -1.127 & -2.604 \\
6 & -1.038 & -2.687 \\
7 & -0.898 & -2.818 \\
8 & -0.531 & -3.159 \\
9 & -0.164 & -3.500 \\
10 & 0.191 & -3.830 \\
11 & 0.191 & -3.830 \\
12 & 0.191 & -3.830 \\
13 & 0.191 & -3.830 \\
14 & 0.191 & -3.830 \\
15 & -0.164 & -3.500 \\
16 & -0.531 & ...
\end{array}
\]

\[ PS_{\text{Ship}} = \text{ksi} \]

Service I for Shipping - Plumb Girder with Impact

Find total Service I stress which includes:
- Prestress
- Girder Dead Load between bunk points including impact up or down

Impact during the shipping stage which shall be applied either up or down

\[ IM_{SH} = 20\% \]

BDM 5.6.2 C.2.
Concrete Structures Chapter 5

STRESS_{Ship1} := for i ∈ 1..rows(SE)

\[
\begin{align*}
\text{STR}_{i,1} & \leftarrow \text{PS}_{Ship_{i,1}} + \text{ST}_{Ship_{i,1}}(1 - \text{IM}_{SH}) \\
\text{STR}_{i,2} & \leftarrow \text{PS}_{Ship_{i,1}} + \text{ST}_{Ship_{i}} \\
\text{STR}_{i,3} & \leftarrow \text{PS}_{Ship_{i,1}} + \text{ST}_{Ship_{i,1}}(1 + \text{IM}_{SH}) \\
\text{STR}_{i,4} & \leftarrow \text{PS}_{Ship_{i,2}} + \text{SB}_{Ship_{i}}(1 - \text{IM}_{SH}) \\
\text{STR}_{i,5} & \leftarrow \text{PS}_{Ship_{i,2}} + \text{SB}_{Ship_{i}} \\
\text{STR}_{i,6} & \leftarrow \text{PS}_{Ship_{i,2}} + \text{SB}_{Ship_{i}}(1 + \text{IM}_{SH})
\end{align*}
\]

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<th>Top Stress + IM</th>
<th>Top Stress - IM</th>
<th>Top Stress + IM</th>
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Maximum compressive stress allowed:

\[ f_{c,SH,\text{lim}} = -5.525 \text{ ksi} \]

Maximum tensile stress allowed:

\[ f_{t,\text{SP,lim}} = 0.554 \text{ ksi} \]

Check compressive stress

\[ \text{chk}_{2,6} := \text{if} (\min\{\text{STRESS}_{\text{Ship1}}\} \geq f_{c,SH,\text{lim}}, "OK", "NG") = "OK" \]

Check tensile stress (with bonded reinforcement)

\[ \text{chk}_{2,7} := \text{if} (\max\{\text{STRESS}_{\text{Ship1}}\} \leq f_{t,\text{SP,lim}}, "OK", "NG") = "OK" \]

Service I for Shipping - Girder on Superelevation without Impact

BDM 5.6.3 D.6
Chapter 5 Concrete Structures

References
1. PCI Journal Jan/Feb 1989 and Jan/Feb 1993, Lateral Stability of Long Prestressed Concrete Beams Parts 1 and 2, Robert F. Mast
2. PCI Journal Jul/Aug 1998, New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girder Bridges
3. PCI Journal Fall 2009, Design Optimization for Fabrication of Pretensioned Concrete Bridge Girders
4. BDM 5.6.3.C.2

Maximum expected roadway superelevation
\[ se := 6\% \]

Superelevation angle
\[ \alpha := \text{atan} (se) = 0.0599 \cdot \text{rad} \]
\[ \alpha = 3.434 \cdot \text{deg} \]

Rotational Stiffness of Support
\[ K_0 := \max \left( \frac{28000 \text{kip-in}}{\text{rad}}, 4000 \frac{\text{kip-in}}{\text{rad}} \right) \cdot \text{ceil} \left( \frac{W_g}{18 \text{kip}} \right) = 32000 \frac{\text{kip-in}}{\text{rad}} \]

Height at which beam weight \( W_g \) could be placed to cause neutral equilibrium
\[ r := \frac{K_0}{W_g} = 225.77 \cdot \text{in} \]

Initial eccentricity caused by shipping support placement tolerance
\[ e_{\text{ship}} := 1 \text{in} \]

Initial eccentricity caused by sweep (horizontal) tolerance at midspan of girder
\[ e_{s,\text{ship}} := 0.125 \text{in} \cdot \text{GL} = 1.674 \cdot \text{in} \]

Offset Factor that determines the distance between the roll axis and the c.g. of the arc of a curved girder
\[ F_{\text{Ol,ship}} := \left( \frac{L_s}{\text{GL}} \right)^2 - \frac{1}{3} = 0.390 \]

Initial eccentricity of the c.g. from the roll axis
\[ e_{i,\text{ship}} := e_{\text{ship}} + e_{s,\text{ship}} \cdot F_{\text{Ol,ship}} = 1.654 \cdot \text{in} \]

Height of roll center over roadway
\[ h_r := 24 \text{in} \]

Horizontal distance from roll center to center of tire support
\[ z_{\text{max}} := \frac{72 \text{in}}{2} = 36.0 \cdot \text{in} \]

Distance from the roll center to the c.g. of girder along roll axis (add 2% for camber)
\[ y := (Y_{bg} + 72 \text{in} - h_r) \cdot 1.02 = 85.333 \cdot \text{in} \]

Theoretical deflection at the girder c.g. assuming full weight is applied about the weak axis. Equation for \( z_{o,\text{ship}} \) derived for unequal overhangs.
\[ z_{o,\text{ship}} := \frac{w_g}{24 \cdot E_c \cdot I_y \cdot \text{GL}} \left( \frac{-6 \cdot L_L^5}{5} - 2L_L^4 \cdot L_S + L_L^2 \cdot L_S^2 - \frac{2}{3} \cdot L_L^2 \cdot L_S \cdot L_T^2 + 
L_S^3 \cdot L_T^2 - \frac{L_S^5}{5} - 2L_S \cdot L_T^4 - \frac{6 \cdot L_T^5}{5} \right) \]
\[ z_{o,\text{ship}} = 4.779 \cdot \text{in} \]

Equilibrium Tilt Angle
\[ \theta_{eq} := \frac{\alpha \cdot r + e_{i,\text{ship}}}{r - y - z_{o,\text{ship}}} = 0.1119 \cdot \text{rad} \]

Lateral bending moment during shipping for inclined girder on superelevation
\[ M_{\text{lat,INCL}} := M_{\text{ship}} \cdot \theta_{eq} \]
Concrete Structures

Find total Service I stress which includes:
- Prestress
- Girder Dead Load between bunk points in biaxial bending due to superelevation

\[
\text{STRESS}_{\text{Ship2}} := \begin{cases} 
\text{STR}_{i,1} & \text{if } i \in 1..\text{rows(SE)} \\
\text{STR}_{i,1} & = \text{PS}_{\text{Ship1},1} + \text{ST}_{\text{Ship1}} - \frac{\text{M}_{\text{lat INCL},i}}{2y} \\
\text{STR}_{i,2} & = \text{PS}_{\text{Ship1},1} + \text{ST}_{\text{Ship1}} + \frac{\text{bf}}{2y} \\
\text{STR}_{i,3} & = \text{PS}_{\text{Ship1},2} + \text{SB}_{\text{Ship1}} - \frac{\text{M}_{\text{lat INCL},i}}{2y} \\
\text{STR}_{i,4} & = \text{PS}_{\text{Ship1},2} + \text{SB}_{\text{Ship1}} + \frac{\text{bf,bot}}{2y} \\
\end{cases}
\]

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<td>16</td>
<td>-1.462</td>
<td>-0.677</td>
<td>-2.966</td>
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</tbody>
</table>

\[ f_{c,\text{SH,lim}} = -5.525 \text{ ksi} \]

\[ f_{t,\text{SI,lim}} = 0.700 \text{ ksi} \]

Check compressive stress

Check tensile stress (with bonded reinforcement)
12.4 Girder Stability During Shipping

Lateral bending moment to cause cracking in corner of top or bottom flange from biaxial bending

\[ M_{latSh_i} := \min \left( \left( f_r - \text{STRESS}_i \right) \frac{21_y}{b_f} (f_r - \text{STRESS}_i) \frac{21_y}{b_f} \right) \]

Tilt angle at cracking

\[ \theta_{\text{maxSh}_i} := \begin{cases} \min \left( \frac{M_{\text{latSh}_i}}{M_{\text{Ship}_i}} \frac{\pi}{2} \right) & \text{if } SE_i \leq SE_{rbL} \\ \min \left( \frac{M_{\text{latSh}_i}}{M_{\text{Ship}_i}} \frac{\pi}{2} \right) & \text{if } SE_{rbL} < SE_i < SE_{rbR} \\ \min \left( \frac{M_{\text{latSh}_i}}{M_{\text{Ship}_i}} \frac{\pi}{2} \right) & \text{if } SE_i \geq SE_{rbR} \end{cases} \]

Factor of Safety against cracking during lifting

\[ FS_{cr,2_i} := \frac{r \cdot \left( \theta_{\text{maxSh}_i} - \alpha \right)}{z_{o,\text{ship}} \cdot \theta_{\text{maxSh}_i} + e_{i,\text{ship}} + y \theta_{\text{maxSh}_i}} \]

<table>
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<th>( M_{latSh} )</th>
<th>( \theta_{\text{maxSh}} )</th>
<th>( FS_{cr,2} )</th>
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Check if minimum FS against cracking is greater than 1.0

Tilt angle at which the maximum FS against rollover occurs

Effective theoretical deflection

\[ \theta_{\text{maxS}} := \frac{z_{\text{max}} - h_r \alpha}{r} + \alpha = 0.2130 \text{-rad} \]

\[ z'_{oS} := z_{o,\text{ship}} \left( 1 + 2.5 \cdot \theta_{\text{maxS}} \right) = 7.325 \text{-in} \]
Maximum Factor of Safety against rollover

\[
FS_{FS} := \frac{\theta_{\text{maxS}} - \alpha}{z_0 \cdot \theta_{\text{maxS}} + c_{\text{ship}} + y \cdot \theta_{\text{maxS}}} = 1.616
\]

Check FS against rollover

\[
\text{chk\_12,11} := \text{if}(FS_{FS} \geq 1.5, "OK", "NG") = "OK"
\]
13. Check Results

Row # indicates Section of each check and Column # indicates the check number within that Section.

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chk = chk

Check for NG entries. If zero, all checks are satisfied.

\[
\text{Number}_{\text{NG}} := \begin{cases} 
0.0 & \text{for } i \in 1..\text{rows}(\text{chk}) \\
\text{for } j \in 1..\text{cols}(\text{chk}) \\
\text{Num } \leftarrow \text{Num } + 1 \text{ if } \text{chk}_{i,j} = \text{"NG"} \\
\text{Num}
\end{cases}
\]
Appendix 5-B6

1 Structure

Design span \( L := 120 \text{ ft} \)
Roadway width \( BW := 43 \text{ ft} \) barrier face to barrier face
Girder spacing \( S := 9 \text{ ft} \)
Skew angle \( \theta := 0 \text{ deg} \)
No. of girder \( N_b := 5 \)
Curb width on deck, \( cw := 10.5 \text{ in} \)

Deck overhang (centerline of exterior girder to end of deck)
\[
\text{overhang} := \frac{BW - (N_b - 1)S}{2} + cw \quad \text{overhang} = 4.375 \text{ ft}
\]

Future overlay (2” HMA), \( w_{\text{h}ma} := 0.140 \text{ kcf} \cdot 2\text{in} \)
\[
w_{\text{h}ma} = 0.023 \text{ kip/ft}^2
\]

2 Criteria and assumptions

2.1 Design Live Load for Decks

(§3.6.1.3.3, not for empirical design method) Where deck is designed using the approximate strip method, specified in §4.6.2.1, the live load shall be taken as the wheel load of the 32.0 kip axle of the design truck, without lane load, where the strips are transverse.

\[
\text{if } (S \leq 15 \text{ ft}, "OK", "NG") = "OK" \quad (§3.6.1.3.3)
\]

The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than (§3.6.1.3.1) for the design of the deck overhang - 1 ft from the face of the curb or railing, and for the design of all other components - 2 ft from the edge of the design lane.

(§3.6.1.3.4) For deck overhang design with a cantilever, not exceeding 6.0 ft from the centerline of the exterior girder to the face of a continuous concrete railing, the wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity, located 1 ft from the face of the railing.

\[
\text{if } (\text{overhang} - cw \leq 6 \text{ ft}, "OK", "NG") = "OK"
\]

Horizontal loads on the overhang resulting from vehicle collision with barriers shall be considered in accordance with the yield line analysis.

2.2 Dynamic Load Allowance (impact)

\( IM := 0.33 \quad (§3.6.2.1) \)

2.3 Minimum Depth and Cover (§9.7.1)

slab design thickness \( t_{s1} := 7 \text{-in} \)
for D.L. calculation \( t_{s2} := 7.5 \text{-in} \)
min. depth \( \text{if } (t_{s1} \geq 7.0 \text{-in}, "OK", "NG") = "OK" \)
top concrete cover = 1.5 in. (up to #11 bar) ($\S$5.12.4 & Table 5.12.3-1) 
use 2.5 in. (Office Practice)

bottom concrete cover = 1 in. (up to #11 bar)
sacrificial thickness = 0.5 in. ($\S$2.5.2.4)

2.4 Skew Deck ($\S$9.7.1.3 and BDM $\S$5.7.2)

The primary reinforcement shall be placed perpendicular to the main supporting components.

3 Material Properties

3.1 Concrete

$ f'_{c} := 4$ ksi  
Use CLASS 4000D for bridge concrete deck (BDM 5.1.1)

$ f_{r2} := 0.37 \cdot \frac{f'_{c}}{\text{ksi}} \quad f_{r2} = 0.74 \text{ksi} \quad ($§5.4.2.6) \quad \text{for use in §5.7.3.3.2}$

$w_{c} := 0.160 \text{ kcf}$

$ E_{c} := 33000 \left( \frac{w_{c}}{\text{kcf}} \right) \cdot 1.5 \cdot \sqrt{\frac{f'_{c}}{\text{ksi}}} \quad E_{c} = 4224.0 \text{ksi} \quad ($§5.4.2.4)$

3.2 Reinforcing Steel ($\S$5.4.3)

$f_{y} := 60$ ksi  \hspace{1cm} E_{s} := 29000$ ksi

4 Methods of Analysis

Concrete deck slabs may be analyzed by using

Approximate elastic methods of analysis, or
Refined methods of analysis, or
Empirical design.

Per office practice, concrete deck slab shall be designed and detailed for both empirical and traditional design methods.

5 Empirical Design ($\S$9.7.2)

5.1 Limit States ($\S$9.5.1)

For other than the deck overhang, where empirical design is used, a concrete deck maybe assumed to satisfy service, fatigue and fracture and strength limit states requirements.

Empirical design shall not be applied to overhangs ($\S$9.7.2.2).

5.2 Design Conditions ($\S$9.7.2.4)

For the purpose of empirical design method, the effective length $S_{\text{eff}}$ shall be taken as ($\S$9.7.2.3).
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**WEB THICKNESS**
\[ b_w := 6.125 \text{ in} \]

**TOP FLANGE WIDTH**
\[ b_f := 49 \text{ in} \]

**EFFECTIVE DEPTH**
\[ S_{\text{eff}} := S - b_f + \frac{b_f - b_w}{2} \]

**S_{\text{eff}} = 6.7 \text{ ft}**

The design depth of the slab shall exclude the loss that is expected to occur as a result of grinding, grooving, or wear.

\[ \text{if} \left( 18.0 \geq \frac{S_{\text{eff}}}{t_{s1}} \geq 6.0, \"OK\", \"NG\" \right) = \"OK\" \quad \text{max} \frac{S_{\text{eff}} + 10 \text{ ft}}{30} = 6.7 \text{ in} \]

**CORE DEPTH**
\[ \text{if} \left( t_{s2} - 2.5 \text{ in} - 1 \text{ in} \geq 4 \text{ in}, \"OK\", \"NG\" \right) = \"OK\" \]

**OVERHANG**
\[ \text{if} \left( \text{overhang} \geq 3 \times t_{s1}, \"OK\", \"NG\" \right) = \"OK\" \quad \text{overhang} = 52.5 \text{ in} \quad 3 \times t_{s1} = 21 \text{ in} \]

A structurally continuous concrete barrier is made composite with the overhang.

**5.3 Optional deflection criteria for span-to-depth ratio (LRFD Table 2.5.2.6.3.1)**

For slabs with main reinforcement parallel to traffic (however, the criteria is used per Office Practice)

\[ \text{Min. Depth (continuous span) where} \]
\[ \frac{S_{\text{eff}} + 10 \text{ ft}}{30} \leq t_{s2}, \"OK\", \"NG\" \]

\[ \text{Try #5 @ 14 in. for bottom longitudinal and transverse,} \quad \frac{0.31 \text{ in}^2}{14 \text{ in}} = 0.27 \text{ in}^2 \text{ per ft} \]

**Composite construction for steel girder (N/A)**

A minimum of two shear connectors at 2 ft centers shall be provided in the M-region of continuous steel superstructures.

**5.4 Reinforcement Requirement (§9.7.2.5)**

Four layers of reinforcement is required in empirically designed slabs.

The amount of deck reinforcement shall be (§C9.7.2.5)

**0.27 in²/ft for each bottom layer (0.3% of the gross area of 7.5 in. slab)**

**0.18 in²/ft for each top layer (0.2% of the gross area)**
#4 @ 12 in. for top longitudinal and transverse. \(0.2 \text{ in}^2 \cdot \frac{1\text{-ft}}{12\text{ in}} = 0.2\text{ in}^2\) per ft

Spacing of steel shall not exceed 18 in.

\[\text{if}(\theta \geq 25\text{ deg}, \"OK\”, \"NG\") = \"NG\"\]

if OK, double the specified reinforcement in the end zones, taken as a longitudinal distance equal to \(S_{\text{eff}}\).

6  Traditional Design

6.1  Design Assumptions for Approx. Method of Analysis (§4.6.2)

Deck shall be subdivided into strips perpendicular to the supporting components (§4.6.2.1.1). Continuous beam with span length as center to center of supporting elements (§4.6.2.1.6). Wheel load may be modeled as concentrated load or load based on tire contact area. Strips should be analyzed by classical beam theory.

6.2  Width of Equivalent Interior Strip (§4.6.2.1.3)

Strip width calculations are not needed since live load moments from Table A4-1 are used.

Spacing in secondary direction (spacing between diaphragms):

\[L_{\text{d}} := \frac{L}{4}, \quad L_{\text{d}} = 30.0\text{ ft}\]

Spacing in primary direction (spacing between girders):

\[S = 9\text{ ft}\]

Since \(\text{if} \left(\frac{L_{\text{d}}}{S} \geq 1.50, \"OK\”, \"NG\”\right) = \"OK\”\), where \(\frac{L_{\text{d}}}{S} = 3.33\) (§4.6.2.1.5)

Therefore, all the wheel load shall be applied to primary strip. Otherwise, the wheels shall be distributed between intersecting strips based on the stiffness ratio of the strip to sum of the strip stiffnesses of intersecting strips.

6.3  Limit States (§5.5.1)

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied. Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck (§13.6.2). Fatigue need not be investigated for concrete deck slabs in multi-girder applications (§5.5.3.1).

6.4  Strength Limit States

Resistance factors (§5.5.4.2.1)

\[\phi_f := 0.90\]  for flexure and tension of reinforced concrete

\[\phi_v := 0.90\]  for shear and torsion

Load Modifier
\[ \eta_D := 1.00 \quad \text{for conventional design (§1.3.3)} \]
\[ \eta_R := 1.00 \quad \text{for conventional level of redundancy (§1.3.4)} \]
\[ \eta_I := 1.00 \quad \text{for typical bridges (§1.3.5)} \]
\[ \eta := \max \left( \frac{\eta_D \eta_R \eta_I}{0.95} \right) \quad \eta = 1 \quad (§1.3.2) \]

Strength I load combination - normal vehicular load without wind (§3.4.1)

Load factors (LRFD Table 3.4.1-1&2):
\[ \gamma_{dc} := 1.25 \quad \text{for component and attachments} \]
\[ \gamma_{dw} := 1.50 \quad \text{for wearing surface and utilities (max.)} \]
\[ \gamma_L := 1.75 \quad \text{for LL} \]

Multiple presence factor (§3.6.1.1.2):
\[ M_1 := 1.20 \quad \text{1 truck} \]
\[ M_2 := 1.00 \quad \text{2 trucks} \]
\[ M_3 := 0.85 \quad \text{3 trucks} \quad \text{(Note; 3 trucks never control for girder spacings up to 15.5 ft, per training notes)} \]

6.4.1 Moment Force Effects Per Strip (§4.6.2.1.6)

The design section for negative moments and shear forces may be taken as follows:

- Prestressed girder - shall be at 1/3 of flange width < 15 in.
- Steel girder - 1/4 of flange width from the centerline of support.
- Concrete box beams - at the face of the web.

web thickness \( b_w = 6.13 \text{ in} \)

Design critical section for negative moment and shear shall be at \( d_c \) (§4.6.2.1.6)

\[ d_c := \min \left( \frac{b_l}{3} \quad 15-\text{in} \right) \]
\[ d_c = 15 \text{ in} \quad \text{from CL of girder (may be too conservative, see training notes)} \]

Maximum factored moments per unit width based on Table A4-1: for \( S = 9 \text{ ft} \)

(applicable if \[ \min((0.625 \times S \times 6-\text{ft})) \geq \text{overhang} - \text{cw,"OK" ,"NG" ]) = "OK" \]

if \( \left[ N_b \geq 3,"\text{OK" ,"NG" ="} \right] = "\text{OK}" \)
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6.29 kip-ft

$M_{LLp} := \frac{6.29 \text{ kip-ft}}{\text{ft}}$

$3.51 \frac{\text{kip-ft}}{\text{ft}}$

$M_{LLn} := \frac{3.51 \text{ kip-ft}}{\text{ft}}$

(max. -M at d_c from CL of girder)

Dead load moments

$M_{DCp} := \frac{l \cdot w_c \cdot S^2}{10}$

$M_{DCp} = 0.81 \frac{\text{kip-ft}}{\text{ft}}$

(max. +M_{DC})

$M_{DWp} := \frac{w_{hmma} \cdot S^2}{10}$

$M_{DWp} = 0.189 \frac{\text{kip-ft}}{\text{ft}}$

(max. +M_{DW})

$M_{DCn} := M_{DCp}$

$M_{DCn} = 0.81 \frac{\text{kip-ft}}{\text{ft}}$

(max. -M_{DC} at d_c at interior girder)

$M_{DWn} := M_{DWp}$

$M_{DWn} = 0.189 \frac{\text{kip-ft}}{\text{ft}}$

(max. -M_{DW} at d_c at interior girder)

Factored positive moment per ft

$M_{up} := \eta \gamma \left( \gamma_{dc} \cdot M_{DCp} + \gamma_{cw} \cdot M_{DWp} + \gamma_{LC} \cdot M_{LLp} \right)$

$M_{up} = 12.3 \frac{\text{kip-ft}}{\text{ft}}$

Factored negative moment

$M_{un} := \eta \gamma \left( \gamma_{dc} \cdot M_{DCn} + \gamma_{cw} \cdot M_{DWn} + \gamma_{LC} \cdot M_{LLn} \right)$

$M_{un} = 7.44 \frac{\text{kip-ft}}{\text{ft}}$

6.4.2 Flexural Resistance

Normal flexural resistance of a rectangular section may be determined by using equations for a flanged section in which case $b_w$ shall be taken as $b$ (§5.7.3.2.3).

$\beta_1 := \begin{cases} 
\left( \frac{f_c}{4 \cdot \text{ksi}} \cdot 0.85 \cdot 0.85 - 0.05 \left( \frac{f_c - 4.0 \cdot \text{ksi}}{1.0 \cdot \text{ksi}} \right) \right) & \text{if } \beta_1 \geq 0.65 \\
0.85 & \text{otherwise}
\end{cases}$

6.4.3 Design for Positive Moment Region

assume bar #

$\text{bar}_p := 5$

$\text{dia(bar)} := \begin{cases} 
0.5 \text{-in} & \text{if } \text{bar} = 4 \\
0.625 \text{-in} & \text{if } \text{bar} = 5 \\
0.75 \text{-in} & \text{if } \text{bar} = 6
\end{cases}$

$d_p := t_{s1} - 1 \cdot \text{in} - \frac{\text{dia(bar)}_p}{2}$

$d_p = 5.7 \text{in}$

$A_s := \frac{0.85 \cdot f_c \cdot \text{ft}}{f_y} \left( d_p^2 - \frac{\frac{2}{3} \cdot M_{up} \cdot \text{ft}}{0.85 \cdot \phi_f \cdot f_c \cdot \text{ft}} \right)$

$A_s = 0.52 \text{ in}^2$ per ft

use (bottom-transverse) #

$\text{bar}_p = 5$

$s_p := 7.5 \text{-in}$ (max. spa. 12 in. per BDM memo)
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\begin{align*}
A_b(\text{bar}) & := \begin{cases} 
0.20\text{-in}^2 & \text{if bar }= 4 \\
0.31\text{-in}^2 & \text{if bar }= 5 \\
0.44\text{-in}^2 & \text{if bar }= 6 
\end{cases} \\
A_{sp} & := A_b(\text{bar}) \frac{1\text{-ft}}{s_p} \\
& = A_{sp} = 0.5\text{ in}^2 \quad \text{per ft}
\end{align*}

Check min. reinforcement (§5.7.3.3.2),

\[ M_{cr} := f_c \cdot \frac{1}{6} \cdot 12\text{-in} \cdot t_{s2}^2 \quad 1.2 \cdot M_{cr} = 8.325 \text{kip-ft} \quad M_{up} \text{ ft} = 12.30 \text{kip-ft} \]

\[ \text{if } (M_{up} \text{ ft} \geq 1.2 \cdot M_{cr}, \text{"OK"}, \text{"NG"}) = \text{"OK"} \]

6.4.4 Design for Negative Moment Region

\begin{align*}
\text{assume bar } & \quad \text{bar}_n := 5 \\
d_n & := t_{s1} - 2.0\text{-in} - \frac{\text{dia(bar}_n)}{2} \\
& = d_n = 4.69\text{ in} \\
A_s & := \frac{0.85 \cdot f'_c \cdot \text{ft}}{f_y} \left( d_n - \sqrt{d_n^2 - \frac{2 \cdot M_{un} \text{ ft}}{0.85 \cdot \phi_r \cdot f'_c \cdot \text{ft}}} \right) \\
& = A_s = 0.37\text{ in}^2 \quad \text{per ft}
\end{align*}

\[ \text{use (top-transverse) bar } \quad \text{bar}_n = 5 \quad s_n := 7.5\text{-in} \quad (\text{max. spa. 12 in. per BDM memo}) \]

\[ A_{sn} := A_b(\text{bar}_n) \frac{1\text{-ft}}{s_n} \quad A_{sn} = 0.5\text{ in}^2 \quad \text{per ft} \]

Check min. reinforcement (§5.7.3.3.2),

\[ M_{cr} := f_c \cdot \frac{1}{6} \cdot 12\text{-in} \cdot t_{s2}^2 \quad 1.2 \cdot M_{cr} = 8.325 \text{kip-ft} \quad M_{un} \text{ ft} = 7.438 \text{kip-ft} \]

\[ \text{if } (M_{un} \text{ ft} \geq 1.2 \cdot M_{cr}, \text{"OK"}, \text{"NG"}) = \text{"NG"} \]

Design for 1.2 Mer,

\[ \frac{0.85 \cdot f'_c \cdot \text{ft}}{f_y} \left( d_n - \sqrt{d_n^2 - \frac{2 \cdot 1.2 \cdot M_{cr}}{0.85 \cdot \phi_r \cdot f'_c \cdot \text{ft}}} \right) = 0.42\text{ in}^2 \quad \text{Say OK} \]

6.5 Control of Cracking by Distribution of Reinforcement (§5.7.3.4)

Service I load combination is to be considered for crack width control (§3.4.1).
Combined limit state load modifier (§1.3.2)

\[ \eta_s := 1 \]

Load factors (LRFD Table 3.4.1-1):

- \( \gamma_{dc} := 1.00 \) for component and attachments
- \( \gamma_{dw} := 1.00 \) for wearing surface and utilities (max.)
- \( \gamma_L := 1.00 \) for LL

\[ M_{sp} := \eta_s \left( \gamma_{dc} \cdot M_{Dcp} + \gamma_{dw} \cdot M_{Dwp} + \gamma_L \cdot M_{Llp} \right) \]

\[ M_{sn} := \eta_s \left( \gamma_{dc} \cdot M_{Dcn} + \gamma_{dw} \cdot M_{Dwn} + \gamma_L \cdot M_{Lln} \right) \]

\( \gamma_{ep} := 0.75 \) for Class 2 exposure condition for deck (assumed)

\( \gamma_{en} := 0.75 \) for Class 2 exposure condition for deck (assumed)

\( h := t_{s1} = 7 \text{ in} \)

\[ \rho_p := \frac{A_{sp}}{d_p \cdot 12 \text{ in}} \quad \rho_n := \frac{A_{sn}}{d_n \cdot 12 \text{ in}} \]

\[ n := \frac{E_n}{E_c} \quad n = 6.866 \quad n := \max[\text{ceil}(n - 0.495) \cdot 6] \]

set \( n = 7 \) (round to nearest integer, §5.7.1, not less than 6)

\[ k(\rho) := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n - \rho \cdot n} \quad k(\rho_p) = 0.272 \]

\[ j(\rho) := 1 - \frac{k(\rho)}{3} \quad j(\rho_p) = 0.909 \]

\[ f_{sa} := \frac{M_{sp} \cdot \text{ft}}{A_{sp} \cdot j(\rho_p) \cdot d_p} \quad f_{sa} = 34.1 \text{ ksi} \]

for \( \text{bar}_p = 5 \quad s_p = 7.5 \text{ in} \)

\[ d_c := (1 \text{ in}) + \frac{\text{dia(bar}_p)}{2} \quad d_c = 1.3 \text{ in} \] (the actual concrete cover is to be used to compute \( d_c \))

\[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} \quad \beta_s = 1.33 \]

if \[ \left(\frac{s_p}{\beta_s \cdot f_{sa} / \text{ksi}} \leq \frac{700 \cdot \gamma_{ep} \cdot \text{in}}{2 \cdot d_c, "OK", "NG"} \right) = "OK" \]

where \( s_p = 7.5 \text{ in} \)

\[ \frac{700 \cdot \gamma_{ep} \cdot \text{in}}{\beta_s \cdot f_{sa} / \text{ksi}} - 2 \cdot d_c = 9.0 \text{ in} \]
\[ k(\rho_n) = 0.295 \quad j(\rho_n) = 0.902 \]

\[ f_{sa} := \frac{M_{sa} \text{ ft}}{A_{sa} j(\rho_n) d_n} \quad f_{sa} = 25.8 \text{ ksi} \]

for \( b_{an} = 5 \quad s_n = 7.5 \text{ in} \)

\[ d_c := 2 \text{ in} + \frac{\text{dia}[b_{an}]}{2} \quad d_c = 2.31 \text{ in} \] (the actual concrete cover is to be used to compute \( d_c \))

\[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} \quad \beta_s = 1.705 \]

\[ \text{if} \left( \frac{s_n}{\beta_s} \cdot \frac{f_{sa}}{f_y} - 2d_c \right) = "OK", "NG" \]

where \( s_n = 7.5 \text{ in} \)

\[ \beta_s \cdot \frac{f_{sa}}{f_y} - 2d_c = 7.3 \text{ in} \]

say OK

6.6 Shrinkage and Temperature Reinforcement (§5.10.8.2)

For components less than 48 in. thick,

where \( A_g := t_{s2} \cdot 1\text{-ft} \)

\[ A_{tem} := 0.11 \frac{A_g \text{ ksi}}{f_y} \quad A_{tem} = 0.17 \text{ in}^2 \text{ per ft} \]

The spacing of this reinforcement shall not exceed \( 3t_{s1} = 21 \text{ in} \) or 18 in (per BDM memo 12 in.)

\[ \text{top longitudinal} - \quad b_{ar} := 4 \quad s := 12 \text{ in} \quad A_k := A_0(bar) \cdot \frac{1\text{-ft}}{s} \quad A_k = 0.2 \text{ in}^2 \text{ per ft} \quad \text{OK} \]

6.7 Distribution of Reinforcement (§9.7.3.2)

The effective span length \( S_{eff} \) shall be taken as (§9.7.2.3):

\[ S_{eff} = 6.70 \text{ ft} \]

For primary reinforcement perpendicular to traffic:

\[ \text{percent} := \min \left( \frac{220}{S_{eff} \text{ ft}} \right) \quad \text{percent} = 67 \]

Bottom longitudinal reinforcement (per BDM memo < slab thickness): \( t_{s2} = 7.5 \text{ in} \)
6.8 Maximum bar spacing (§5.10.3.2)

Unless otherwise specified, the spacing of the primary reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18 in. The maximum spacing of temperature shrinkage reinforcement shall be as specified in §5.10.8.

\[ 1.5 \times t_{s1} = 10.5 \text{ in} \quad \text{OK} \]

6.9 Protective Coating (§5.12.4)

Epoxy coated reinforcement shall be specified for both top and bottom layer slab reinforcements except only top layer when the slab is with longitudinal post-tensioning.

7 Slab Overhang Design

(§3.6.1.3.4) Horizontal loads resulting from vehicular collision with barrier shall be considered in accordance with the provisions of LRFD Section 13.

(§13.7.3.1.2) Unless a lesser thickness can be proven satisfactory during the crash testing procedure, the min. edge thickness for concrete deck overhangs shall be taken as 8 in. for concrete deck overhangs supporting concrete parapets or barriers.

7.1 Applicable Limit States (§5.5.1)

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied.

Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck (§13.6.2).

7.2 Strength Limit state

Load Modifier

\[ \eta_D := 1.00 \quad \text{for ductile components and connections (§1.3.3 & simplified)} \]

\[ \eta_R := 1.00 \quad \text{for redundant members (§1.3.4)} \]

\[ \eta_I := 1.00 \quad \text{for operationally important bridge (§1.3.5)} \]

\[ \eta := \max \left( \frac{\eta_D \cdot \eta_R \cdot \eta_I}{0.95} \right) \quad \eta = 1 \quad (§1.3.2) \]

Load factors (LRFD Table 3.4.1-1):

\[ \gamma_{dc} := 1.25 \quad \text{for component and attachments} \]

\[ \gamma_{dw} := 1.50 \quad \text{for wearing surface and utilities (max.)} \]

\[ \gamma_{L} := 1.75 \quad \text{for LL} \]
7.3 Extreme Event Limit State II

Extreme event limit state shall apply for the force effect transmitted from the vehicular collision force.

Load Modifier

\[
\eta_D := 1.00 \quad (\S\text{1.3.3}) \\
\eta_R := 1.00 \quad (\S\text{1.3.4}) \\
\eta_I := 1.00 \quad (\S\text{1.3.5}) \\
\eta_e := \max\left(\frac{\eta_D \cdot \eta_R \cdot \eta_I}{0.95}\right) \quad \eta_e = 1 \quad (\S\text{1.3.2})
\]

Load factors (LRFD Table 3.4.1-1):

\[
\gamma_{dc} := 1.25 \quad \text{for component and attachments} \\
\gamma_{dw} := 1.50 \quad \text{for wearing surface and utilities (max.)} \\
\gamma_{CT} := 1.00 \quad \text{for collision force}
\]

7.4 Vehicular Collision Force (\S\text{13.7.2})

Railing test level TL-4 applies for high-speed highways, freeways, and interstate highways with a mixture of trucks and heavy vehicles.

The transverse and longitudinal loads need not be applied in conjunction with vertical loads (\S\text{A13.2}). Design forces for railing test level **TL-4** (LRFD Table A13.2-1),

\[
\begin{align*}
\text{transverse} & : F_t := 54\,\text{kip} \\
\text{longitudinal} & : F_L := 18\,\text{kip} \\
\text{vertical (down)} & : F_v := 18\,\text{kip}
\end{align*}
\]

Effective Distances:

\[
\begin{align*}
\text{transverse} & : L_t := 3.50\,\text{ft} \\
\text{longitudinal} & : L_L := 3.50\,\text{ft} \\
\text{vertical} & : L_v := 18\,\text{ft}
\end{align*}
\]

Min. design height, \(H\), 32 in. (LRFD Table A13.2-1) use \(H := 32\,\text{in}\)

7.5 Design Procedure (\S\text{A13.3})

Yield line analysis and strength design for reinforced concrete may be used.
7.6 Nominal Railing Resistance (§A13.3)

For F-shape barriers, the approximate flexural resistance may be taken as:

Flexural capacity about vertical axis,

\[ M_w := 35.62 \text{kip} \cdot \text{ft} \]

Additional flexural resistance of beam in addition to \( M_w \), if any, at top of wall,

\[ M_b := 10.27 \text{kip} \cdot \text{ft} \]

Flexural capacity about horizontal axis,

\[ M_c := 19.21 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

Critical wall length, over which the yield mechanism occurs, \( L_c \), shall be taken as:

\[
L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H \left(M_b + M_w\right)}{M_c}}
\]

\[ L_c = 9.1 \text{ft} \]

For impact within a barrier segment, the total transverse resistance of the railing may be taken as:

\[
R_w := \left(\frac{2}{2 \cdot L_c - L_t}\right) \left[8 \cdot M_b + 8 \cdot M_w + \frac{M_c L_c^2}{H}\right]
\]

\[ R_w = 131.11 \text{kip} \]

7.7 Design Load Cases (§A13.4.1)

Case 1

Transverse and longitudinal forces at extreme event limit state.

Resistance factor (§A13.4.3.2) \( \phi := 1.0 \)

(§C13.7.3.1.2) Presently, in adequately designed bridge deck overhangs, the major crash-related damage occurs in short sections of slab areas where the barriers is hit.

a. at inside face of parapet

\[
M_s := \min\left(\frac{(R_w \cdot 1.2 \cdot F_t) \cdot H}{L_c + 2 \cdot H}\right)
\]

\[ M_s = 11.97 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

\[ M_{DCa} := 0.45 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

DL M- at edge of curb (see deck.gts STRUDL output),

\[ cw = 0.875 \text{ ft} \]

Design moment

\[
M_u := \eta_c \left(\gamma_{dc} \cdot M_{DCa} + \gamma_{CT} \cdot M_s\right)
\]

\[ M_u = 12.5 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]
(§A13.4.2) Deck overhang may be designed to provide a flexural resistance, \( M_s \), which is acting in coincident with tensile force, \( T \) (see memo),

\[
T := \frac{\min (R_w \left(1.2 \cdot F_t\right)) \cdot \text{ft}}{L_c + 2H} \quad \text{per ft}
\]

\( T = 4.49 \text{ kip per ft} \)

\( \min. \ "\text{haunch+slab}" \text{ dimension,} \)

\[ A := t_{c2} + 0.75 \cdot \text{in} \]

\( d_s \), flexural moment depth at edge of curb,

\[
d_s := \left(7.\text{in} + \frac{A - 7\text{in}}{\text{overhang} - 0.5 \cdot b_f} \cdot \text{cw} - 2.5\cdot\text{in} - \frac{\text{dia}(\text{bar}_o)}{2}\right) \quad \text{d}_s = 4.7 \text{in}
\]

\( A_s \) required for \( M_u \) and \( T \),

\[
A_s := \frac{0.85 \cdot f'_c \cdot \text{ft}}{f_y} \left(d_s - \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot \text{ft}}{0.85 \cdot \phi \cdot f'_c \cdot \text{ft}}} + \frac{T}{f_y}\right) \quad A_s = 0.67 \text{ in}^2 \quad \text{per ft} \quad (1)
\]

Check max. reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where \( d_c \)

\[
d_c := d_s \quad d_c = 4.7 \text{ in}
\]

\[
c := \frac{A_s \cdot f_y - T}{0.85 \cdot \beta_1 \cdot f'_c \cdot 1 \cdot \text{ft}} \quad c = 1 \text{ in}
\]

\[ \text{if } \left(\frac{c}{d_c} \leq 0.42, "\text{OK}" \right) = "\text{OK}" \quad \frac{c}{d_c} = 0.221
\]

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

b. at design section in the overhang

Design critical section for negative moment and shear shall be at \( d_c \), (§4.6.2.1.6)

\[
d_c := \min \left(\frac{b_f}{3} \cdot 15\cdot\text{in}\right) \quad d_c = 15 \text{ in} \quad \text{from CL of girder (may be too conservative, see training notes)}
\]

At the inside face of the parapet, the collision forces are distributed over a distance \( L_c \) for the moment and \( L_c + 2H \) for the axial force. Similarly, assume the distribution length is increased in a 30 degree angle from the base of the parapet,

Collision moment at design section,
\[ M_{se} := \frac{M_s L_c}{L_c + 2 \cdot 0.577 \cdot (\text{overhang} - d_c - \text{cw})} \quad \text{M}_{se} = 9.31 \text{ kip-ft} \]

dead load moment @ \( d_c \) from CL of exterior girder (see deck.gts STRUDL output)

\[ \text{overhang} - d_c = 3.125 \text{ ft} \quad \text{from edge of deck} \]

\[ M_{DCb} := 1.96 \text{ kip-ft} \quad M_{DWb} := 0.06 \text{ kip-ft} \]


design moment

\[ M_u := \eta_e (\gamma_{dc} M_{DCb} + \gamma_{dw} M_{DWb} + \gamma_{CT} M_{se}) \quad M_u = 11.85 \frac{\text{kip-ft}}{\text{ft}} \]

(§A13.4.2) design tensile force, T,

\[ T := \frac{\text{min}(\{R_w \cdot 1.2 \cdot F_t\})}{L_c + 2 \cdot H + 2 \cdot 0.577 \cdot (\text{overhang} - \text{cw} - d_c)} \quad T = 3.81 \text{ kip per ft} \]

\[ d_s, \text{ flexural moment depth at design section in the overhang.} \]

\[ d_s := A - 2.5 \cdot \text{in} - \frac{\text{dia}[\text{bar}_o]}{2} \quad d_s = 5.4 \text{ in} \]

A\textsubscript{s} required for \( M_u \) and T,

\[ \frac{0.85 \cdot f'c \cdot \text{ft}}{f_y} \left( d_s - \frac{d_s^2}{2} - \frac{2 \cdot M_u \cdot \text{ft}}{0.85 \cdot f'c \cdot \text{ft}} \right) + \frac{T}{f_y} = 0.53 \text{ in}^2 \quad \text{per ft} \quad \text{(doesn't control)} \quad (2) \]

c. at design section in first interior span

The collision moment per unit width at the section under consideration can then be determined using the 30° distribution.

\[ M_s = 11.97 \frac{\text{kip-ft}}{\text{ft}} \]

Collision moment at at \( d_c \) from the exterior girder, (see deck.gts output, barrierM factor for 1 kip-ft of Ms),

\[ M_{si} := M_s \cdot 0.824 \quad M_{si} = 9.87 \frac{\text{kip-ft}}{\text{ft}} \]

Using the 30° angle distribution, design moment

\[ M_{si} := \frac{M_{si} L_c}{L_c + 2 \cdot 0.577 \cdot (\text{overhang} - \text{cw} + d_c)} \quad M_{si} = 6.16 \frac{\text{kip-ft}}{\text{ft}} \]

dead load moment @ this section (see deck.gts output) \( d_c = 1.25 \text{ ft} \)
M\text{DCi} := 2.11 \text{kip-ft/ft} \quad \text{M\text{DWi} := 0.03 kip-ft/ft} \\

\text{design moment} \\
\quad M_u := \eta_c \left( \gamma_{dc} \cdot M_{\text{DCi}} + \gamma_{dw} \cdot M_{\text{DWi}} + \gamma_{CT} \cdot M_{\text{si}} \right) \quad M_u = 8.84 \text{ kip-ft/ft} \\
\text{d}_s, \text{flexural moment depth at the design section,} \\
\quad d_s := t_{s1} - 2.0\text{-in} - \frac{\text{dia(bar}_o\text{)}}{2} \quad d_s = 4.69\text{ in} \\

A_s \text{ required for } M_u, \\
\quad \frac{0.85 \cdot f'_c \cdot \text{ft}}{f_y} \left( d_s - \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot \text{ft}}{0.85 \cdot \phi \cdot f'_c \cdot \text{ft}}} \right) = 0.4 \text{ in}^2 \text{ per ft (doesn't control)} \quad (3) \\

\textbf{Case 2 \quad Vertical collision force} \\
For concrete parapets, the case of vertical collision never controls. \\

\textbf{Case 3 \quad Check DL + LL} \\
\quad \text{Resistance factor (§1.3.2.1)} \quad \phi_f := 0.9 \\
\quad \text{For deck overhangs, where applicable, the §3.6.1.3.4 may be used in lieu of the equivalent strip method (§4.6.2.1.3).} \\
\quad \text{a. at design section in the overhang} \\
\quad \text{moment arm for 1.0 kip/ft live load (§3.6.1.3.4)} \quad x := \text{overhang - cw - 1\cdot ft - d}_c \quad x = 15\text{ in} \\
\quad \text{live load moment without impact,} \\
\quad \quad w_L := 1.0 \text{ kip/ft} \quad M_{\text{LL}} := M_1 \cdot w_L \cdot x \quad M_{\text{LL}} = 1.5 \text{ kip-ft/ft} \\
\quad \text{factored moment} \\
\quad \quad M_u := \eta \left[ \gamma_{dc} \cdot M_{\text{DCb}} + \gamma_{dw} \cdot M_{\text{DWb}} + \gamma_{L} \cdot M_{\text{LL}} \cdot (1.0 + IM) \right] \quad M_u = 6.03 \text{ kip-ft/ft} \\
\quad \text{d}_s, \text{flexural moment depth at edge of curb,} \\
\quad \quad d_s := A - 2.5\text{-in} - \frac{\text{dia(bar}_o\text{)}}{2} \quad d_s = 5.44\text{ in}
A_s required for $M_u$,
\[
\frac{0.85 \cdot f' c \cdot \text{ft}}{f_y} \left( d_s = \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot \text{ft}}{0.85 \cdot \phi f \cdot f' c \cdot \text{ft}}} \right) = 0.26 \text{ in}^2 \text{ per ft} \text{ (doesn't control)} \quad (4)
\]

b. at design section in first span

Assume slab thickness at this section, $t_{s1} = 7 \text{ in}$

use the same D.L. + L.L moment as in previous for design (approximately)

factored moment $M_u = 6.03 \text{ kip-ft per ft}$

d_s, flexural moment depth at edge of curb,
\[
d_s := t_{s1} - 2.0 \text{ in} - \frac{\text{dia(bar}_o)}{2} \quad d_s = 4.7 \text{ in}
\]

A_s required for $M_u$,
\[
\frac{0.85 \cdot f' c \cdot \text{ft}}{f_y} \left( d_s = \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot \text{ft}}{0.85 \cdot \phi f \cdot f' c \cdot \text{ft}}} \right) = 0.3 \text{ in}^2 \text{ (doesn't control)} \quad (5)
\]

The largest of (1) to (5), As required, $A_s = 0.67 \text{ in}^2 \text{ per ft}$

use bar #
\[
\begin{align*}
\text{bar}_o &= 5 \quad \text{ @ } s := 22.5 \text{ in} \\
\text{bar}_n &= 5 \quad s_n = 7.5 \text{ in}
\end{align*}
\]

(top transverse) at edge of curb, put 1 #5 between every other top bar in the deck overhang region

A_s := \frac{A_0(\text{bar}_o)}{s} + \frac{A_0(\text{bar}_n)}{s_n} \quad A_s = 0.66 \text{ in}^2 \text{ say OK}

Determine the point in the first bay of the deck where the additional bars are no longer needed,
\[
A_s := \frac{A_0(\text{bar}_o)}{s} \quad A_s = 0.5 \text{ in}^2
\]

c := \frac{A_s \cdot f_y}{0.85 \cdot \beta_1 \cdot f' c \cdot \text{1-ft}} \quad c = 0.9 \text{ in}
\]

d_c := t_{s1} - 2.0 \text{ in} - \frac{\text{dia(bar}_o)}{2} \quad d_c = 4.7 \text{ in}
\]
\[
a := \beta_1 \cdot c \quad a = 0.7 \text{ in}
\]

For the strength limit state,
\[
M_{\text{cap}} := \phi f A_s f_y \left( d_c - \frac{a}{2} \right) \quad M_{\text{cap}} = 9.65 \text{ kip-ft per ft}
\]
For the extreme event limit state,

\[
M_{\text{cap}} := \phi \cdot A_s \cdot f_y \left( d_c - \frac{a}{2} \right) \quad M_{\text{cap}} = 10.72 \text{ kip-ft per ft}
\]

By inspection of (1) to (5), no additional bar is required beyond design section of the first bay.

Cut off length requirement (§5.11.1.2)

\[15 \cdot \text{dia(bar)}_o = 0.781 \text{ ft} \quad (\text{controls by inspection})\]

8 Reinforcing Details

8.1 Development of Reinforcement (§5.11.2.1.1)

basic development length for #11 bar and smaller,

\[L_{db}(d_b, A_b) := \max \left( \frac{1.0}{1.25 \cdot A_b \cdot f_y \cdot \sqrt{f'c}} \right) \]

For #5 bars, \[L_{db}(0.625 \text{ in}, 0.31 \text{ in}^2) = 15 \text{ in}\]

For #6 bars, \[L_{db}(0.75 \text{ in}, 0.44 \text{ in}^2) = 18 \text{ in}\]

For epoxy coated bars (§5.11.2.1.2),

\[\text{with cover less than } 3d_b \text{ or with clear spacing less than } 6d_b \quad \text{times 1.5}\]
\[\text{not covered above} \quad \text{times 1.2}\]

For widely spaced bars..... times 0.8 (§5.11.2.1.3)

bars spaced laterally not less than 6 in. center-to-center, with not less than 3. in clear cover measured in the direction of spacing.

For bundled bars..... times 1.2 for a three-bar bundle (§5.11.2.3)

Lap Splices in Tension (§5.11.5.3.1)

The length of lap for tension lap splices shall not be less than either 12 in. or the following for Class A, B, or C splices:

Class A splice ....... times 1.0
Class B splice ....... times 1.3
Class C splice ....... times 1.7
Flexural Reinforcement (§5.11.1.2)
Except at supports of simple-spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

- the effective depth of the member,
- 15 times the nominal diameter of bar, or
- 1/20 of the clear span.

No more than 50% of the reinforcement shall be terminated at any section, and adjacent bars shall not be terminated in the same section.

Positive moment reinforcement (§5.11.1.2.2)
At least 1/3 the positive moment reinforcement in simple-span members, and 1/4 the positive moment reinforcement in continuous members, shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6 in.

Negative moment reinforcement (§5.11.1.2.3)
At least 1/3 of the total tension reinforcement provided for negative moment at a support shall have an embedment length length beyond the point of inflection (DL + LL) not less than:

- the effective depth of the member, \( d \)
- \( 12.0 \cdot d_b \), and
- \( 0.0625 \) times the clear span.

Moment resisting joints (§5.11.1.2.4)
In Seismic Zones 3 and 4, joint shall be detailed to resist moments and shears resulting from horizontal loads through the joint.

Q.E.D.
Precast Concrete

Appendix 5-B7 Stay-in-place (SIP) Deck Panel

Design Criteria
Loading: HL-93

Concrete:
SIP Panel,
\[ f'_{ci} := 4.0 \text{ ksi} \]
\[ f'_{c} := 5.0 \text{ ksi} \]
(f\text{ci} + 1 \text{ ksi})
CIP slab,
\[ f'_{cs} := 4 \text{ ksi} \]

Reinforcing Steel: (§5.4.3)
AASHTO M-31, Grade 60,
\[ f_y := 60 \text{ ksi} \]
\[ E_s := 29000 \text{ ksi} \]

Prestressing Steel:
AASHTO M-203, uncoated 7 wire, low-relaxation strands (§5.4.4.1)
Nominal strand diameter,
\[ d_b := 0.375 \text{ in} \]
\[ A_p := 0.085 \text{ in}^2 \]
(Trends now are toward the use of 3/8 in. diameter strand, per PCI J., 33(2), pp.67-109)
\[ f_{pu} := 270 \text{ ksi} \]
\[ f_{py} := 0.90 \cdot f_{pu} \]
\[ f_{py} := 243 \text{ ksi} \]
\[ f_{pe} := 0.80 \cdot f_{py} \]
\[ f_{pe} := 194.4 \text{ ksi} \]
\[ E_p := 28500 \text{ ksi} \]

Design Method: LRFD

Mechanical shear ties on the top of panels are not required per PCI, special report, PCI J., 32(2), pp. 26-45.

Structure
Design span
\[ L := 89.07 \text{ ft} \]
Roadway width
\[ BW := 53.0 \text{ ft} \]
barrier face to barrier face
Girder spacing
\[ S := 6.75 \text{ ft} \]
Skew angle
\[ \theta := 14.65 \text{ deg} \]
no. of girder
\[ N_b := 8 \]
curb width on deck,
\[ cw := 10.5 \text{ in} \]
Deck overhang (CL of exterior girder to end of deck)
\[ \text{overhang} := \frac{BW - (N_b - 1) \cdot S}{2} + cw \]
overhang = 3.75 ft
slab design thickness
\[ t_{s1} := 8.0 \text{ in} \]
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for D.L. calculation
\[ t_{s2} := 8.5\text{-in} \]

Panel dimensions:
\[ W_{sip} := 8.0\text{-ft} \quad L_{sip} := 6.34\text{-ft} \quad t_{sip} := 3.5\text{-in} \]

CIP composite slab:

- \[ t_{cs1} := t_{s1} - t_{sip} \quad t_{cs1} = 4.5\text{ in} \] (used for structural design)
- \[ t_{cs2} := t_{s2} - t_{sip} \quad t_{cs2} = 5\text{ in} \] (actual thickness)

\[ w_{c} := 0.160\text{-kcf} \]

Future overlay (2" HMA),
\[ w_{dw} := 0.140\text{kcf} \cdot 2\text{in} \quad w_{dw} = 0.023\frac{\text{kip}}{\text{ft}^2} \]

**Minimum Depth and Cover (§9.7.1)**

- Min. Depth
  \[ \text{if} \left( t_{s2} \geq 7.0\text{-in}, "OK", "NG" \right) = "OK" \]
- Min. SIP thickness
  \[ \text{if} \left( 0.55 \cdot t_{s2} > t_{sip} \geq 3.5\text{-in}, "OK", "NG" \right) = "OK" \]

- Top cover for epoxy-coated main reinforcing steel = 1.5 in. (up to #11 bar)
  \[ = 2.0\text{ in.} \text{ (#14 & #18 bars)} \text{ (§5.12.4 & Table 5.12.3-1)} \]

- Bottom concrete cover (unprotected main reinforcing) = 1 in. (up to #11 bar)
  \[ = 2\text{ in.} \text{ (#14 & #18 bars)} \]

- Sacrificial thickness = 0.5 in. (§2.5.2.4)

Optional deflection criteria for span-to-depth ratio (LRFD Table 2.5.2.6.3-1)

Min. Depth (continuous span) where \( S = 6.75\text{ft} \) (slab span length):

\[
\text{if} \left[ \max \left( \left( \frac{S + 10\text{-ft}}{30} \right) \right) \leq t_{s1}, "OK", "NG" \right] = "OK" \quad \max \left( \left( \frac{S + 10\text{-ft}}{0.54\text{-ft}} \right) \right) = 6.7\text{in} 
\]

**Skew Deck (§9.7.1.3)**

\[ \theta \leq 25\text{-deg} = 1 \] it true, the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components.

**Loads**
The precast SIP panels support their own weight, any construction loads, and the weight of the CIP slabs. For superimposed dead and live loads, the precast panels are analyzed assuming that they act compositely with the CIP concrete.

**Dead load** per foot

- SIP panel
  \[ w_{sip} := t_{sip} \cdot w_{c} \quad w_{sip} = 0.047\frac{\text{kip}}{\text{ft}^2} \]

- CIP slab
  \[ w_{cs} := t_{cs2} \cdot w_{c} \quad w_{cs} = 0.067\frac{\text{kip}}{\text{ft}^2} \]

- Weight of one traffic barrier is
  \[ t_{b} := 0.52\frac{\text{kip}}{\text{ft}} \]
Weight of one sidewalk is \( t_{\text{side}} := 0.52 \, \text{kip/ft} \)

**Wearing surface & construction loads**

Future wearing surface \( w_{dw} = 0.023 \, \text{kip/ft}^2 \)

Construction load \( w_{\text{con}} := 0.050 \, \text{kip/ft} \) \(^{(\text{§9.7.4.1})}\)

Note that load factor for construction load is 1.5 \(^{\text{(§3.4.2)}}\).

**Live loads**

\(^{(\text{§3.6.1.3.3, not for empirical design method})}\) Where deck is designed using the approximate strip method, specified in \(^{\text{§4.6.2.1}}\), the live load shall be taken as the wheel load of the 32.0 kip axle of the design truck, without lane load, where the strips are transverse:

\[
\text{if } (S \leq 15 \, \text{ft}, "OK", "NG") = "OK" \quad (\text{§3.6.1.3.3})
\]

Multiple presence factor:

\[ M_1 := 1.2 \quad M_2 := 1.0 \quad (\text{§3.6.1.1.1.2}) \]

Dynamic Load Allowance (impact) \[ IM := 0.33 \quad (\text{§3.6.2.1}) \]

Maximum factored moments per unit width based on Table A4-1: for \( S = 6.75 \, \text{ft} \)

\[
\text{applicability} \quad \text{if } \left[ \min((0.625 \cdot S - 6 \, \text{ft})) \geq \text{overhang} - \text{cw}, "OK", "NG" \right] = "OK"
\]

\[
\text{if } \left[ N_b \geq 3, "OK", "NG" \right] = "OK"
\]

\[ M_{\text{LLp}} := 5.10 \, \text{kip-ft/ft} \]

\(^{(\text{§3.6.1.3.4})}\) For deck overhang design with a cantilever, not exceeding 6.0 ft from the centerline of the exterior girder to the face of a continuous concrete railing, the wheel loads may be replaced with a uniformly distributed line load of 1.0 KLF intensity, located 1 ft from the face of the railing:

\[
\text{if } (\text{overhang} - \text{cw} \leq 6 \, \text{ft}, "OK", "NG") = "OK"
\]

**Load combination**

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied.

Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck \(^{\text{(§13.6.2)}}\).

Fatigue need not be investigated for concrete deck slabs in multi-girder applications \(^{\text{(§5.5.3.1)}}\).
Strength Limit States

Load Modifier

\[ \eta_D := 1.00 \quad \text{for conventional design (§1.3.3)} \]
\[ \eta_R := 1.00 \quad \text{for conventional level of redundancy (§1.3.4)} \]
\[ \eta_I := 1.00 \quad \text{for typical bridges (§1.3.5)} \]

\[ \eta := \max \left( \eta_D \eta_R \eta_I \right) \cdot 0.95 \quad \eta = 1 \quad (§1.3.2) \]

Strength I load combination - normal vehicular load without wind (§3.4.1)

Load factors (LRFD Table 3.4.1-1&2):

\[ \gamma_{dc} := 1.25 \quad \text{for component and attachments} \]
\[ \gamma_{dw} := 1.50 \quad \text{for DW} \]
\[ \gamma_L := 1.75 \quad \text{for LL} \]

Section Properties

*Non-composite section* per foot

\[ A_{sip} := t_{sip} \cdot 12 \text{ in} \quad A_{sip} = 42 \text{ in}^2 \]
\[ I_{sip} := \frac{12 \cdot t_{sip}^3}{12} \quad I_{sip} = 42.875 \text{ in}^4 \]
\[ Y_{bp} := \frac{1}{2} t_{sip} \quad Y_{bp} = 1.75 \text{ in} \]
\[ Y_{tp} := t_{sip} - Y_{bp} \quad S_{tp} := \frac{I_{sip}}{Y_{tp}} \quad S_{bp} := \frac{I_{sip}}{Y_{bp}} \]
\[ Y_{tp} = 1.75 \text{ in} \quad S_{tp} = 24.5 \text{ in}^3 \quad S_{bp} = 24.5 \text{ in}^3 \]

\[ E_c := 33000 \left( \frac{w_c}{\text{kcf}} \right)^{1.5} \sqrt{\frac{f_c}{\text{ksi}}} \quad E_c = 4722.6 \text{ ksi} \quad (§5.4.2.4) \]

\[ E_{ci} := 33000 \left( \frac{w_c}{\text{kcf}} \right)^{1.5} \sqrt{\frac{f_{ci}}{\text{ksi}}} \quad E_{ci} = 4224.0 \text{ ksi} \]

*Composite Section Properties* (§4.6.2.6)

\[ E_{cs} := 33000 \left( \frac{w_c}{\text{kcf}} \right)^{1.5} \sqrt{\frac{f_{cs}}{\text{ksi}}} \quad E_{cs} = 4224.0 \text{ ksi} \quad (§5.4.2.4) \]
modular ratio, \( n := \frac{f'_c}{\sqrt{f'_{cs}}} \) \( \quad n = 1.118 \)

\( b := 12 \text{ in} \)

\( A_{\text{slab}} := \frac{b}{n} t_{cs1} \)
\( Y_{bs} := t_{sip} + \frac{t_{cs1}}{2} \)
\( AY_{bs} := A_{\text{slab}} Y_{bs} \)

**Area**

CIP slab

\( A_{\text{slab}} = 48.3 \text{ in}^2 \)
\( Y_{bs} = 5.75 \text{ in} \)
\( A_{\text{slab}} Y_{bs} = 277.7 \text{ in}^3 \)

SIP panel

\( A_{\text{sip}} = 42 \text{ in}^2 \)
\( Y_{bp} = 1.75 \text{ in} \)
\( A_{\text{sip}} Y_{bp} = 73.5 \text{ in}^3 \)

\( Y_{b} := \frac{A_{\text{slab}} Y_{bs} + A_{\text{sip}} Y_{bp}}{A_{\text{slab}} + A_{\text{sip}}} \)
\( Y_{b} = 3.89 \text{ in} \) \( @ \) bottom of panel

\( Y_{t} := t_{sip} - Y_{b} \)
\( Y_{t} = -0.39 \text{ in} \) \( @ \) top of panel

\( Y_{ts} := t_{sip} + t_{cs1} - Y_{b} \)
\( Y_{ts} = 4.11 \text{ in} \) \( @ \) top of slab

\( I_{\text{slabc}} := A_{\text{slab}} \left( Y_{ts} - \frac{t_{cs1}}{2} \right)^2 + \frac{(b - \frac{b}{n}) t_{cs1}^3}{12} \)
\( I_{\text{slabc}} = 248.7 \text{ in}^4 \)

\( I_{pc} := A_{\text{sip}} \left( Y_{b} - Y_{bp} \right)^2 + I_{\text{sip}} \)
\( I_{pc} = 235.1 \text{ in}^4 \)

\( I_{c} := I_{\text{slabc}} + I_{pc} \)
\( I_{c} = 483.8 \text{ in}^4 \)

**Section modulous of the composite section**

\( S_{b} := \frac{I_{c}}{Y_{b}} \)
\( S_{b} = 124.4 \text{ in}^3 \) \( @ \) bottom of panel

\( S_{t} := \frac{I_{c}}{|Y_{t}|} \)
\( S_{t} = 1242.1 \text{ in}^3 \) \( @ \) top of panel

\( S_{ts} := n \frac{I_{c}}{Y_{ts}} \)
\( S_{ts} = 131.6 \text{ in}^3 \) \( @ \) top of slab

**Required Prestress**

Assume the span length conservatively as the panel length, \( L_{\text{sip}} = 6.34 \text{ ft} \)

\( M_{\text{sip}} := \frac{w_{\text{sip}} L_{\text{sip}}^2}{8} \)
\( M_{\text{sip}} = 0.234 \text{ ft-kip} \)

\( M_{\text{cip}} := \frac{w_{\text{cs}} L_{\text{sip}}^2}{8} \)
\( M_{\text{cip}} = 0.335 \text{ ft-kip} \)
For the superimposed dead and live loads, the force effects should be calculated based on analyzing the strip as a continuous beam supported by infinitely rigid supports (§4.6.2.1.6)

\[
M_{DW} := 0.10 \text{ ft kip/ft} \\
M_b := 0.19 \text{ kip-ft/ft} \\
(\text{see Strudls-dl output})
\]

\[
f_b := \frac{ (M_{SIP} + M_{cip}) \text{ft}}{S_{bp}} + \frac{ (M_{DW} + M_b + M_{LLp}) \text{ft}}{S_b} \quad f_b = 0.799 \text{ksi}
\]

**Tensile Stress Limits**

\[
0.190 \sqrt{\frac{f_c}{\text{ksi}}} = 0.42 \text{ksi} \quad (\text{§5.9.4.2.2})
\]

0 ksi WSDOT design practice

Required precompression stress at bottom fiber,

\[
f_{creq} := f_b - 0 \text{ ksi} \quad f_{creq} = 0.799 \text{ksi}
\]

If \( P_{se} \) is the total effective prestress force after all losses, and the center of gravity of stands is concentric with the center of gravity of the SIP panel:

\[
P_{se} := f_{creq} W_{sip} t_{sip} \quad P_{se} = 268.43 \text{ kip} \quad \text{per panel}
\]

Assume stress at transfer,

\[
f_{pi} := 0.75 f_{pu} \quad f_{pi} = 202.5 \text{ksi} \quad (\text{LRFD Table 5.9.3-1})
\]

Assume 15% final losses, the final effective prestress,

\[
p_{se} := f_{pi} (1 - 0.15) \quad p_{se} = 172.12 \text{ksi}
\]

The required number of strands,

\[
N_{req} := \frac{P_{se}}{p_{se} A_p} \quad N_{req} = 18.35 \quad N_p := \text{ceil}(N_{req})
\]

Try \( N_p := 19 \)

**Prestress Losses**

Loss of Prestress (§5.9.5)

\[
\Delta f_{PT} = \Delta f_{pES} + \Delta f_{pLT}
\]

where, \( \Delta f_{pLT} = \) long-term prestress loss due to creep of concrete, shrinkage of concrete, and relaxation of steel.
Steel relaxation at transfer (Office Practice)

Curing time for concrete to attain $f'_{ci}$ is approximately 12 hours: set $t := 0.75$ day

\[ f_{pj} := 0.75 \cdot f_{pu} \quad f_{pj} = 202.5 \text{ ksi} \]

immediately prior to transfer+steel relax.

\[(LRFD \text{ Table 5.9.3-1})\]

\[ \Delta f_{pR0} := \frac{\log(24.0 - t)}{40.0} \left( \frac{f_{pj}}{f_{py}} - 0.55 \right) f_{pj} \]

\[ \Delta f_{pR0} = 1.80 \text{ ksi} \]

Given:

\[ A_p = 0.085 \text{ in}^2 \]

straight strands \[ N_p = 19 \]

jacking force, \[ f_{pj} N_p A_p = 327.04 \text{ kip} \]

(note: these forces include initial prestress relaxation loss, see §C5.9.5.4.4b)

\[ A_{ps} := A_p N_p \quad A_{ps} = 1.615 \text{ in}^2 \text{ per panel} \]

\[ A_{psip} := A_{ps} \frac{ft}{W_{sip}} \quad A_{psip} = 0.202 \text{ in}^2 \text{ per ft} \]

c.g. of all strands to c.g. of girder, \[ e_p := 0 \text{ in} \]

Elastic Shortening, $\Delta f_{pES}$ (§5.9.5.2.3a)

\[ f_{cgp} : \text{ concrete stress at c.g. of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the sections of maximum moment.} \]

Guess values:

\[ p_{si} := 194.4 \text{ ksi} \]

prestress tendon stress at transfer (LRFD Table 5.9.3-1)

Given

\[ \left( f_{pj} - \Delta f_{pR0} - p_{si} \right) \frac{E_{ci}}{E_p} = \frac{-p_{si} \cdot A_{psip}}{A_{sip}} \]

(note: used only when \( e_p = 0 \text{ in} \))

\[ p_{si} := \text{Find}(p_{si}) \quad p_{si} = 194.4 \text{ ksi} \]

\[ f_{cgp} := \frac{-p_{si} \cdot (A_{psip})}{A_{sip}} \quad f_{cgp} = -0.93 \text{ ksi} \]

\[ \Delta f_{pES} := f_{pj} - \Delta f_{pR0} - p_{si} \quad \Delta f_{pES} = 6.3 \text{ ksi} \]

Approximate Estimate of Time Dependent Losses (§5.9.5.3)

Criteria:

Normal-weight concrete

Concrete is either steam or moist cured

Prestressing is by low relaxation strands

Are sited in average exposure condition and temperatures
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\[ H := 75 \]
the average annual ambient relative humidity (%)

\[ \gamma_h := 1.7 - 0.01H \]
\[ \gamma_h = 0.95 \]

\[ \gamma_{st} := \frac{5}{1 + \frac{f_{ci}}{ksi}} \]
\[ \gamma_{st} = 1 \]

\[ \Delta f_pR := 2.5 ksi \]
an estimate of relaxation loss for low relaxation strand

Then,

\[ \Delta f_{pLT} := 10.0 \frac{f_{pi} \cdot A_{sip}}{A_{sip}} \gamma_h \gamma_{st} + (12.0 ksi) \gamma_h \gamma_{st} + \Delta f_pR \]
\[ \Delta f_{pLT} = 23.1 ksi \]

Total loss \( \Delta f_{pT} \),

\[ \Delta f_{pT} := \Delta f_{pR0} + \Delta f_{pLT} + \Delta f_{pES} \]
\[ \Delta f_{pT} = 31.25 ksi \]

\[ f_{pe} := f_{pj} - \Delta f_{pT} \]
\[ f_{pe} = 171.25 ksi \]

if \( f_{pe} \leq 0.80 f_{py} \) "OK" , "NG" ) = "OK" (LRFD Table 5.9.3-1)

\[ p_e := \frac{N_p \cdot A_p \cdot f_{pe}}{W_{sip}} \]
\[ p_e = 34.57 \text{ kip per foot} \]

**Stresses in the SIP Panel at Transfer**

**Stress Limits for Concrete**

Compression:  \(-0.60 f_{ci} = -2.4 ksi\)

Tension:  Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

\[ 0.24 \cdot \sqrt[3]{\frac{f_{ci}}{ksi}} \text{ ksi} = 0.48 ksi \]

or w/o bonded reinforcement,

\[ \min \left\{ \frac{0.0948 \cdot \sqrt[3]{f_{ci}} \text{ ksi}}{0.200 \text{ ksi}} \right\} = 0.19 ksi \] (Controls)

Because the strand group is concentric with the precast concrete panel, the midspan section is the critical section that should be checked.
Stress at Midspan

Effective stress after transfer,

\[ P_{si} := \frac{N_{p} A_{p} p_{si}}{W_{sip}} \quad P_{si} = 39.244 \text{ kip} \]

Moment due to weight of the panel,

\[ M_{sip} = 0.234 \text{ kip-ft} \]

At top of the SIP panel,

\[ \frac{M_{sip \cdot ft}}{S_{tp}} \left( - \frac{P_{si \cdot ft}}{A_{sip}} \right) = -1.05 \text{ ksi} \quad < \text{allowable} \quad -0.60 f'_{ci} = -2.4 \text{ ksi} \quad \text{OK} \]

At bottom of the SIP panel,

\[ \frac{M_{sip \cdot ft}}{S_{bp}} \left( - \frac{P_{si \cdot ft}}{A_{sip}} \right) = -0.82 \text{ ksi} \quad < \text{allowable} \quad -0.60 f'_{ci} = -2.4 \text{ ksi} \quad \text{OK} \]

Stresses in SIP Panel at Time of Casting Topping Slab

The total prestress after all losses,

\[ P_{e} = 34.57 \text{ kip} \]

Stress Limits for Concrete

Flexural stresses due to unfactored construction loads shall not exceed 65% of the 28-day compressive strength for concrete in compression or the modulus of rupture in tension for prestressed concrete for m panels (§9.7.4.1).

The construction load shall be taken to be less than the weight of the form and the concrete slab plus 0.050 KSF.

For load combination Service I:

Compression: \[ -0.65 f'_{c} = -3.25 \text{ ksi} \]

Tension: Modulus of rupture,

\[ f_{t} := 0.24 \sqrt{\frac{f'_{c}}{\text{ksi}}} \quad f_{t} = 0.54 \text{ ksi} \]

Stresses at Midspan after all Non-Composite Loads

\[ M_{sip} = 0.23 \text{ ft kip} \]
\[ M_{cip} = 0.33 \frac{\text{ft-kip}}{\text{ft}} \]
\[ M_{const} := 0.050 \frac{\text{kip}}{\text{ft}^2} \times \frac{L_{sip}}{8} \]
\[ M_{const} = 0.25 \frac{\text{ft-kip}}{\text{ft}} \]

At top of the SIP panel,
\[
\left[ \frac{(M_{sip} + M_{cip} + M_{const}) \cdot \text{ft}}{S_{tp}} \right] - \frac{P_c \cdot \text{ft}}{A_{sip}} = -1.23 \text{ksi} < \text{allowable} \quad -0.65 \cdot f' c = -3.25 \text{ksi} \quad \text{OK}
\]

At bottom of the SIP panel,
\[
\left[ \frac{(M_{sip} + M_{cip} + M_{const}) \cdot \text{ft}}{S_{bp}} \right] - \frac{P_c \cdot \text{ft}}{A_{sip}} = -0.42 \text{ksi} < \text{allowable} \quad -0.65 \cdot f' c = -3.25 \text{ksi} \quad \text{OK}
\]

**Elastic Deformation (§9.7.4.1)**

Deformation due to
\[
\Delta := \frac{5}{48} \cdot \frac{(M_{sip} + M_{cip}) \cdot \text{ft} \cdot L_{sip}^2}{E_c I_{sip}} \quad \Delta = 0.02 \text{ in}
\]

\[
\text{if} \quad \Delta \leq \min \left( \frac{L_{sip}}{180}, 0.25 \text{ in} \right) \quad \text{if} \quad L_{sip} \leq 10 \text{ ft} \quad \text{"OK"}, \text{"NG"} = \text{"OK"}
\]
\[
\min \left( \frac{L_{sip}}{240}, 0.75 \text{ in} \right) \quad \text{otherwise}
\]

**Stresses in SIP Panel at Service Loads**

Compression:
- Stresses due to permanent loads
  \[-0.45 \cdot f' c = -2.25 \text{ ksi} \quad \text{for SIP panel} \]
  \[-0.45 \cdot f' cs = -1.8 \text{ ksi} \quad \text{for CIP panel} \]
- Stresses due to permanent and transient loads
  \[-0.60 \cdot f' c = -3 \text{ ksi} \quad \text{for SIP panel} \]
  \[-0.60 \cdot f' cs = -2.4 \text{ ksi} \quad \text{for CIP panel} \]
- Stresses due to live load + one-half of the permanent loads
  \[-0.40 \cdot f' c = -2 \text{ ksi} \quad \text{for SIP panel} \]
  \[-0.40 \cdot f' cs = -1.6 \text{ ksi} \quad \text{for CIP panel} \]
Tension:

\[
0.0948 \sqrt{\frac{f_c}{\text{ksi}}} = 0.21 \text{ ksi}
\]  

\(\text{ksi}\)  

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**Service Load Stresses at Midspan**

- **Compressive stresses at top of CIP slab**

Stresses due to permanent load + prestressing

\[
\frac{(M_{DW} + M_b) \text{ft}}{S_{ts}} = -0.026 \text{ksi} < \text{allowable} \quad -0.45 \cdot f_c = -1.8 \text{ksi} \quad \text{OK}
\]

Stresses due to permanent and transient loads,

\[
\frac{(M_{DW} + M_b + M_{LLp}) \text{ft}}{S_{ts}} = -0.49 \text{ksi} < \text{allowable} \quad -0.60 \cdot f_c = -2.4 \text{ksi} \quad \text{OK}
\]

- **Compressive stresses at top of the SIP panel**

Stresses due to permanent load + prestressing

\[
\frac{P_e \text{ft}}{A_{sip}} - \frac{(M_{sip} + M_{cip}) \text{ft}}{S_{ip}} - \frac{(M_{DW} + M_b) \text{ft}}{S_{l}} = -1.1 \text{ksi} < \text{allowable} \quad -0.45 \cdot f_c = -2.25 \text{ksi} \quad \text{OK}
\]

Stresses due to permanent and transient loads,

\[
\frac{P_e \text{ft}}{A_{sip}} - \frac{(M_{sip} + M_{cip}) \text{ft}}{S_{ip}} - \frac{(M_{DW} + M_b + M_{LLp}) \text{ft}}{S_{l}} = -1.15 \text{ksi} < \text{allowable} \quad -0.60 \cdot f_c = -3 \text{ksi} \quad \text{OK}
\]

Stresses due to live load + one-half the sum of effective prestress and permanent loads,

\[
-0.5 \left(\frac{P_e \text{ft}}{A_{sip}}\right) - \frac{0.5 (M_{sip} + M_{cip}) \text{ft}}{S_{ip}} - \frac{(0.5 M_{DW} + 0.5 M_b + M_{LLp}) \text{ft}}{S_{l}} = -0.6 \text{ksi} < \text{allowable} \quad -0.40 \cdot f_c = -2 \text{ksi} \quad \text{OK}
\]

- **Tensile stresses at bottom of the SIP panel**

Stresses due to permanent and transient loads,

\[
\frac{P_e \text{ft}}{A_{sip}} + \frac{(M_{sip} + M_{cip}) \text{ft}}{S_{bp}} + \frac{(M_{DW} + M_b + M_{LLp}) \text{ft}}{S_{b}} = 0.02 \text{ksi} < \text{allowable} \quad 0.0948 \sqrt{\frac{f_c}{\text{ksi}}} = 0.21 \text{ ksi} \quad \text{OK}
\]
Flexural Strength of Positive Moment Section

Resistance factors (§5.5.4.2.1)

\[ \phi_f := 0.90 \quad \text{for flexure and tension of reinforced concrete} \]

\[ \phi_p := 1.00 \quad \text{for flexure and tension of prestressed concrete} \]

\[ \phi_v := 0.90 \quad \text{for shear and torsion} \]

Ultimate Moment Required for Strength I

Dead load moment,

\[ M_{DC} := M_{sip} + M_{cip} + M_b \quad M_{DC} = 0.76 \text{kip} \cdot \text{ft} \]

Wearing surface load moment,

\[ M_{DW} = 0.1 \text{kip} \cdot \text{ft} \]

Live load moment,

\[ M_{LLp} = 5.1 \text{kip} \cdot \text{ft} \]

\[ M_u := \eta \left( \gamma_{dc} M_{DC} + \gamma_{dw} M_{DW} + \gamma_{L} M_{LLp} \right) \quad M_u = 10.02 \text{kip} \cdot \text{ft} \]

Flexural Resistance (§5.7.3)

Find stress in prestressing steel at nominal flexural resistance, \( f_{ps} \) (§5.7.3.1.1)

\[ f_{pe} = 171.249 \text{ ksi} \quad 0.5 \cdot f_{pu} = 135 \text{ ksi} \]

if \( f_{pe} \geq 0.5 \cdot f_{pu} \), "OK" , "NG" = "OK"

\[ k := 2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) \quad k = 0.28 \quad \text{(LRFD Eq. 5.7.3.1.1-2)} \]

\[ A_s := 0 \text{ in}^2 \]

\[ A'_s := 0 \text{ in}^2 \quad \text{(conservatively)} \]

distance from extreme compression fiber to the centroid of the prestressing tendons,

\[ d_p := t_{s1} - 0.5 \cdot t_{sip} \quad d_p = 6.25 \text{ in} \]

\[ W_{sip} = 96 \text{ in} \quad \text{effective width of compression flange} \]

\[ \beta_1 := \begin{cases} 
0.85 & \text{if} \ f_{cs} \leq 4 \cdot \text{ksi}, 0.85, 0.85 - 0.05 \cdot \left( \frac{f_{cs} - 4.0 \text{ ksi}}{1.0 \text{ ksi}} \right) \\
0.65 & \text{otherwise} 
\end{cases} \]

\[ \beta_1 = 0.85 \quad (§5.7.2.2) \]
Assume rectangular section,

\[ c := \frac{A_{ps} f_{pu}}{0.85 f_{c}' \beta_1 W_{sip} + k A_{ps} f_{pu} \frac{f_{pu}}{d_p}} \]

\[ c = 1.47 \text{ in} \]

Stress in prestressing steel at nominal flexural resistance, \( f_{ps} \) (§5.7.3.1.1),

\[ f_{ps} := f_{pu} \left(1 - k \cdot \frac{c}{d_p}\right) \]

\[ f_{ps} = 252.24 \text{ ksi} \]

Check stress in prestressing steel according to available development length, \( l_d \)

Available development length at midspan of the SIP panel,

\[ l_d := 0.5 L_{sip} \]

\[ l_d = 3.17 \text{ ft} \]

rearranging LRFD eq. 5.11.4.1-1

\[ f_{psld} := \frac{l_d}{1.6 d_b} \cdot \text{ksi} + \frac{2}{3} f_{pe} \]

\[ f_{psld} = 177.57 \text{ ksi} \]

(may be too conservative)

\[ f_{ps} := \min\left(f_{ps} \ f_{psld}\right) \]

\[ f_{ps} = 177.57 \text{ ksi} \]

Flexural Resistance (§5.7.3.2.2 & 5.7.3.2.2),

\[ a := \beta_1 c \]

\[ a = 1.25 \text{ in} \]

\[ A_{ps} = 1.615 \text{ in}^2 \] per panel

\[ M_n := A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) \]

\[ M_n = 134.4 \text{ kip-ft} \]

\[ M_r := \phi_p M_n \]

\[ M_r = 134.4 \text{ kip-ft} \] per panel

\[ M_r := \frac{M_r}{W_{sip}} \]

\[ M_r = 16.81 \text{ kip-ft/ft} \] per ft

\[ M_u \leq M_r = 1 \]

OK where \( M_u = 10.02 \text{ kip-ft/ft} \)

Limits of Reinforcement

**Minimum Reinforcement** (§5.7.3.3.2)

Compressive stress in concrete due to effective prestress force (after all losses) at midspan

\[ f_{peA} := \frac{P_c \cdot \text{ft}}{A_{sip}} \]

\[ f_{peA} = 0.82 \text{ ksi} \] (compression)
Non-composite dead load moment at section, $M_{dnc}$:

$$M_{dnc} := M_{cip} + M_{sip} \quad M_{dnc} = 0.57 \text{ kip-ft}$$

$$f_r = 0.54 \text{ ksi} \quad \text{use SIP panel}$$

$$M_{cr} := \left( f_r + f_{peA} \right) \frac{S_b}{ft} - M_{dnc} \left( \frac{S_b}{S_{bp}} - 1 \right) \quad 1.2 \cdot M_{cr} = 14.13 \text{ kip-ft}$$

$$M_r \geq 1.2 \cdot M_{cr} = 1 \quad \text{OK} \quad \text{where} \quad M_r = 16.81 \text{ kip-ft}$$

**Negative Moment Section Over Interior Beams**

Deck shall be subdivided into strips perpendicular to the supporting components (§4.6.2.1.1). Continuous beam with span length as center to center of supporting elements (§4.6.2.1.6). Wheel load may be modeled as concentrated load or load based on tire contact area. Strips should be analyzed by classical beam theory.

Spacing in secondary direction (spacing between diaphragms):

$$L_d := \frac{L}{1.0} \quad L_d = 89.07 \text{ ft}$$

Spacing in primary direction (spacing between girders):

$$S = 6.75 \text{ ft}$$

Since $$\frac{L_d}{S} \geq 1.50 = 1$$, where $$\frac{L_d}{S} = 13.2$$ (§4.6.2.1.5)

therefore, all the wheel load shall be applied to primary strip. Otherwise, the wheels shall be distributed between intersecting strips based on the stiffness ratio of the strip to sum of the strip stiffnesses of intersecting strips.

**Critical Section**

The design section for negative moments and shear forces may be taken as follows:

Prestressed girder - shall be at 1/3 of flange width < 15 in.

Steel girder - 1/4 of flange width from the centerline of support.

Concrete box beams - at the face of the web.

top flange width $$b_f := 15.06 \text{ in}$$

Design critical section for negative moment and shear shall be at $d_c$ (§4.6.2.1.6)

$$d_c := \min \left( \frac{1}{3} \cdot b_f, 15 \text{ in} \right) \quad d_c = 5 \text{ in}$$

from CL of girder (may be too conservative, see training notes)
Maximum factored moments per unit width based on Table A4-1: for \( S = 6.75 \text{ ft} \)

(Include multiple presence factors and the dynamic load allowance)

applicability
\[
\text{if } [\min((0.625 \cdot S \cdot 6 \cdot \text{ft})) \geq \text{overhang} - \text{cw}, "OK", "NG"] = "OK"
\]

\[
\text{if } (N_b \geq 3, "OK", "NG") = "OK"
\]

\[
M_{\text{LLn}} := 4.00 \frac{\text{kip-ft}}{\text{ft}}
\]

(max. -M at \( d_c \) from CL of girder)

Dead load moment (STUDL s-dl output)

\[
M_{\text{DCn}} := 0.18 \frac{\text{kip-ft}}{\text{ft}}
\]

(Dead load from deck overhang and sidl only, max. -M at \( d_c \) at interior girder, conservative)

\[
\frac{d_c}{S} = 0.062
\]

Service negative moment

\[
M_{\text{sn}} := M_{\text{DCn}} + M_{\text{DWn}} + M_{\text{LLn}}
\]

M_{\text{sn}} = 4.28 \frac{\text{kip-ft}}{\text{ft}}

Factored negative moment

\[
M_{\text{un}} := \eta \left( \gamma_{\text{dc}} \cdot M_{\text{DCn}} + \gamma_{\text{dw}} \cdot M_{\text{DWn}} + \gamma_{\text{L}} \cdot M_{\text{LLn}} \right)
\]

M_{\text{un}} = 7.38 \frac{\text{kip-ft}}{\text{ft}}

**Design of Section**

Normal flexural resistance of a rectangular section may be determined by using equations for a flanged section in which case \( b_w \) shall be taken as \( b \) (**§5.7.3.2.3**).

\[
\beta_1 := \frac{f'_{\text{cs}} - 4 \cdot \text{ksi}}{0.85, 0.85 - 0.05 \left( \frac{f'_{\text{cs}} - 4 \cdot \text{ksi}}{1.0 \cdot \text{ksi}} \right)}
\]

\[
\beta_1 := \begin{cases} 
\beta_1 & \text{if } \beta_1 \geq 0.65 \\
0.65 & \text{otherwise}
\end{cases}
\]

\[
\beta_1 = 0.85 \text{ (§5.7.2.2)}
\]

Conservatively use CIP slab concrete strength

Assume bar #

\[
\text{bar}_n := 5
\]

\[
\text{dia}(\text{bar}) := \begin{cases} 
0.5\text{-in} & \text{if } \text{bar} = 4 \\
0.625\text{-in} & \text{if } \text{bar} = 5 \\
0.75\text{-in} & \text{if } \text{bar} = 6 \\
0.875\text{-in} & \text{if } \text{bar} = 7
\end{cases}
\]

\[
d_n := t_{s2} - 2.5\text{-in} - \frac{\text{dia}(\text{bar}_n)}{2}
\]

\[
d_n = 5.69\text{ in}
\]
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\[
A_s := \frac{0.85 \cdot f'_{cs} \cdot \text{ft}}{f_y} \left( d_n - \sqrt{\frac{d_n^2}{2 \cdot \text{Mun} \cdot \text{ft}}} \right) \quad A_s = 0.3 \text{ in}^2 \quad \text{per ft}
\]

use (top-transverse) bar #

\[
\text{bar}_n = 5 \quad \text{s}_n := 9 \cdot \text{in}
\]

\[
A_b(\text{bar}) := \begin{cases} 
0.20 \cdot \text{in}^2 & \text{if bar } = 4 \\
0.31 \cdot \text{in}^2 & \text{if bar } = 5 \\
0.44 \cdot \text{in}^2 & \text{if bar } = 6 \\
0.60 \cdot \text{in}^2 & \text{if bar } = 7 
\end{cases}
\]

\[
A_{sn} := A_b(\text{bar}_n) \cdot \frac{1 \cdot \text{ft}}{s_n} \quad A_{sn} = 0.41 \text{ in}^2 \quad \text{per ft}
\]

Maximum Reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where

\[
d_e := d_n
\]

\[
c := \frac{A_{sn} \cdot f_y}{0.85 \cdot \beta_1 \cdot f'_{cs} \cdot 1 \cdot \text{ft}} \quad c = 0.72 \text{ in}
\]

\[
\text{if} \left( \frac{c}{d_e} \leq 0.42, "OK", "NG" \right) = "OK" \quad \frac{c}{d_e} = 0.126
\]

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

Minimum Reinforcement (§5.7.3.3.2)

\[
f_{cs} := 0.24 \sqrt{\frac{f_{cs} \cdot \text{ksi}}{\text{ksi}}} \quad f_{ts} = 0.48 \text{ ksi} \quad \text{use SIP panel concrete strength}
\]

\[
n := \frac{E_s}{E_{cs}} \quad n = 6.866 \quad n := \max[(\text{ceil}(n - 0.495)) \cdot 6]
\]

\[
n = 7 \quad \text{set } n = 7 \quad \text{(round to nearest integer, §5.7.1, not less than 6)}
\]

\[
(n - 1)A_{sn} = 2.48 \text{ in}^2
\]

\[
A_{gc} := t_{s2} \cdot \text{ft} \quad A_{gc} = 102 \text{ in}^2
\]

\[
d_s := 2.5 \text{in} + 0.625 \text{in} + 0.5 \cdot 0.75 \text{in} \quad \text{c.g. of reinforcement to top of slab} \quad d_s = 3.5 \text{ in}
\]

\[
Y_{ts} := \frac{A_{gc} \cdot 0.5 \cdot t_{s2} + (n - 1) \cdot A_{sn} \cdot d_s}{A_{gc} + (n - 1) \cdot A_{sn}} \quad Y_{ts} = 4.23 \text{ in}
\]
\[ I_{cg} := \frac{ft \cdot t_{s2}^3}{12} + A_{gc} \left( 0.5 \cdot t_{s2} - Y_{ts} \right)^2 + (n - 1)A_{sn} \left( Y_{ts} - d_{c} \right)^2 \quad I_{cg} = 615.49 \text{ in}^4 \]

\[ M_{cr} := \frac{f_{ts} \cdot I_{cg}}{Y_{ts}} \quad M_{cr} = 5.817 \text{ kip-ft} \quad 1.2 \cdot M_{cr} = 6.98 \text{ kip-ft} \]

\[ \text{if} \left( M_{un} \text{ ft} \geq 1.2 \cdot M_{cr}, "OK", "NG" \right) = "OK" \]

**Crack Control (§5.7.3.4)**

\[ \gamma_e := 0.75 \quad \text{for Class 2 exposure condition for deck (assumed)} \]

\[ d_c := 2.0 \text{ in} + 0.5 \cdot \text{dia} \left( \text{bar}_n \right) \quad d_c = 2.31 \text{ in} \]

\[ h := t_{s1} \quad h = 8 \text{ in} \]

\[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} \quad \beta_s = 1.581 \]

\[ M_{sn} = 4.28 \text{ kip-ft} \frac{\text{ft}}{\text{ft}} \]

\[ n := \frac{E_s}{E_{cs}} \quad n = 6.866 \quad n := \text{ceil} \left( n - 0.495 \right) \quad \text{use slab concrete strength} \]

\[ \text{set } n = 7 \quad \text{(round to nearest integer, §5.7.1)} \]

\[ \rho := \frac{A_{sn}}{ft \cdot d_n} \quad \rho = 6.056 \times 10^{-3} \]

\[ k(\rho) := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n - \rho \cdot n} \quad k(\rho) = 0.252 \]

\[ j(\rho) := 1 - \frac{k(\rho)}{3} \quad j(\rho) = 0.916 \]

\[ f_{sa} := \frac{M_{sn} \cdot ft}{A_{sn} \cdot j(\rho) \cdot d_n} \quad f_{sa} = 23.85 \text{ ksi} \]

\[ \text{if} \left( s_n \leq \frac{700 \cdot \gamma_e \text{ in}}{\beta_s \frac{f_{sa}}{\text{ksi}}} - 2 \cdot d_c, "OK", "NG" \right) = "OK" \quad \text{where} \quad s_n = 9 \text{ in} \quad \frac{700 \cdot \gamma_e \text{ in}}{\beta_s \frac{f_{sa}}{\text{ksi}}} - 2 \cdot d_c = 9.3 \text{ in} \]

**Shrinkage and Temperature Reinforcement (§5.10.8.2)**
Concrete Structures

For components less than 48 in. thick,

\[
A_g := t_s \cdot 1 \cdot \text{ft}
\]

\[
A_{\text{tem}} := 0.11 \cdot \frac{A_g \cdot \text{ksi}}{f_y} \quad A_{\text{tem}} = 0.19 \text{ in}^2 \text{ per ft}
\]

The spacing of this reinforcement shall not exceed \(3 \cdot t_s = 24 \text{ in}\) or 18 in.

**top longitudinal** -

\[
\text{bar} := 4 \quad s := 12 \text{ in} \quad A_s := A_y(\text{bar}) \cdot \frac{1 \cdot \text{ft}}{s} \quad A_s = 0.2 \text{ in}^2 \text{ per ft} \quad \text{OK}
\]

### Distribution of Reinforcement (§9.7.3.2)

The effective span length \(S_{\text{eff}}\) shall be taken as (§9.7.2.3):

web thickness \(b_w := 7 \text{ in}\)

top flange width \(b_f = 15.06 \text{ in}\)

\[
S_{\text{eff}} := S - b_f + \frac{b_f - b_w}{2} \quad S_{\text{eff}} = 5.83 \text{ ft}
\]

For primary reinforcement perpendicular to traffic:

\[
\text{percent} := \min \left( \left( \frac{220}{S_{\text{eff}}} \right), \frac{67}{\sqrt{\text{ft}}} \right) \quad \text{percent} = 67
\]

**Bottom longitudinal** reinforcement (convert to equivalent mild reinforcement area):

\[
A_s := \frac{\text{percent}}{100} \cdot \frac{A_{\text{ps}} \cdot f_{py}}{W_{\text{kip}}} \quad A_s = 0.55 \text{ in}^2 \text{ per ft}
\]

\[
\text{use bar} \# \quad \text{bar} := 5 \quad s := 6.0 \text{ in} \quad A_s := A_y(\text{bar}) \cdot \frac{1 \cdot \text{ft}}{s} \quad A_s = 0.62 \text{ in}^2 \text{ per ft} \quad \text{OK}
\]

### Maximum bar spacing (§5.10.3.2)

Unless otherwise specified, the spacing of the primary reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18 in. The maximum spacing of temperature shrinkage reinforcement shall be as specified in §5.10.8.

\[
1.5 \cdot t_s = 12 \text{ in} \quad \text{OK}
\]

### Protective Coating (§5.12.4)

Epoxy coated reinforcement shall be used for slab top layer reinforcements except when the slab is overlayed with HMA.
Appendix 5-B8  W35DG Deck Bulb Tee 48" Wide

W35DG Deck Bulb Tee, 48" Wide

Flexural Design Example, LRFD 2005

1.0 Material Properties

Precast Concrete

\[ f_c := 8.0 \text{ ksi} \]
\[ f_{ci} := 6.0 \text{ ksi} \]

\[ E_c := 33000 \text{ksi} \left( \frac{\rho}{\text{kcf}} \right)^{3/2} \sqrt{\frac{f_c}{\text{ksi}}} \]
\[ E_c = 5974 \text{ ksi} \]

\[ E_{ci} := 33000 \text{ksi} \left( \frac{\rho}{\text{kcf}} \right)^{3/2} \sqrt{\frac{f_{ci}}{\text{ksi}}} \]
\[ E_{ci} = 5173 \text{ ksi} \]

Rupture Modulus

\[ f_r := 0.24 \text{ksi} \sqrt{\frac{f_c}{\text{ksi}}} \]
\[ f_r = 0.679 \text{ksi} \]

Prestressing Steel (low-relaxation)

\[ f_{pu} := 270 \text{ksi} \]
\[ f_{py} := 243 \text{ksi} \]
\[ E_p := 28500 \text{ksi} \]

\[ A_{\text{strand}} := 0.217 \text{in}^2 \]
\[ d_{\text{strand}} := 0.6 \text{in} \]

\[ n := \frac{E_p}{E_c} \]
2.0 Geometric Properties

Span Length (bearing to bearing)

\[ L := 85\text{ft} \]

Top flange width (i.e. girder spacing)

\[ b := 48\text{in} \quad t_s := 6\text{in} \]

Section depth

\[ h := 35\text{in} \]

Gross area (used for dead weight calculations)

\[ A_g := 669\text{in}^2 \]

Section Properties

\[ y_b := 20.9\text{in} \]

\[ I_g := 100096\text{in}^4 \]

\[ I_p := 169341\text{in}^4 \]

\[ J := 29572\text{in}^4 \]

\[ S_b := \frac{I_g}{y_b} \quad S_b = 4789\text{in}^3 \]

\[ S_t := \frac{I_g}{(h - y_b)} \quad S_t = 7099\text{in}^3 \]

3.0 Permanent Loads

DC: Girder self-weight
The moment distribution factor is:

\[ w_{dl} := A_g \cdot \rho \]

\[ w_{dl} = 0.743 \text{kip/ft} \]

**DC: Diaphragms (at 1/3 points)**

\[ P_{dia} := 8\text{in} \cdot (b - 6\text{in}) \cdot (h - 12\text{in}) \cdot \rho \]

\[ P_{dia} = 0.716 \text{kip} \]

**DC: Traffic Barriers (1/3 of F-shape)**

\[ w_{sdl} := \frac{0.450 \text{kip}}{3 \text{ft}} \]

\[ w_{sdl} = 0.150 \text{kip/ft} \]

**DW: Overlay (3" ACP)**

\[ w_{dw} := 3\text{in} \cdot b \cdot 0.140\text{kcf} \]

\[ w_{dw} = 0.140 \text{kip/ft} \]

### 4.0 Live Loads

**HL-93 loading is travelling in 2 traffic lanes; for the maximum force effect taken at midspan:**

\[ M_{HL} := 
\left( 0.08 \cdot L^2 + 24 \cdot L \cdot ft - \frac{1120}{3} \cdot ft^2 \right) \text{kip/ft} \]

\[ M_{HL} = 2245 \text{kip-ft} \]

*This includes a 33% dynamic load allowance and a multiple presence factor of 1.0*

**Live Load Distribution Factor (design for interior beam):**

**Number of lanes**

\[ N_L := 2 \]

**From AASHTO Table 4.6.2.2b-1**

\[ e_g := \frac{h \cdot \gamma \cdot b - t_s}{2} \]

\[ e_g = 11.100 \text{in} \]

\[ K_g := \left( I_g + A_g \cdot e_g^2 \right) \]

\[ K_g = 182523 \text{in}^4 \]

**The moment distribution factor is:**

\[ g_{LL} := 0.075 + \left( \frac{b}{9.5\text{ft}} \right)^{0.6} \cdot \left( \frac{b}{L} \right)^{0.2} \cdot \left( \frac{K_g}{12L \cdot t_s^3} \right)^{0.1} \]

\[ g_{LL} = 32.2\% \]
**5.0 Flexural Load Combinations**

*DC; Component load effects*

\[
M_{dl} := \frac{w_{dl}}{8} \cdot L^2 \quad M_{dl} = 671 \text{ kip}\cdot\text{ft}
\]

\[
M_{sdl} := \frac{w_{sdl}}{8} \cdot L^2 + \frac{P_{dia}}{3} \cdot L \quad M_{sdl} = 156 \text{ kip}\cdot\text{ft}
\]

\[
M_{DC} := M_{dl} + M_{sdl} \quad M_{DC} = 827 \text{ kip}\cdot\text{ft}
\]

*Dead load moment at harping point (for stress at release)*

\[
M_{harp} := \frac{3 \cdot w_{dl} L^2}{25} \quad (at \ 0.4L \ point) \quad M_{harp} = 644 \text{ kip}\cdot\text{ft}
\]

*DW; Overlay load effects*

\[
M_{DW} := \frac{w_{dw}}{8} \cdot L^2 \quad M_{DW} = 126 \text{ kip}\cdot\text{ft}
\]

*LL+I; Live load effects*

\[
M_{LL} := g_{LL} \cdot M_{HL} \quad M_{LL} = 723 \text{ kip}\cdot\text{ft}
\]

*Conservatively, the design moment will be the maximum dead and live load moments at midspan*

**Service I**

\[
M_{\text{serviceI}} := M_{DC} + M_{DW} + M_{LL} \quad M_{\text{serviceI}} = 1677 \text{ kip}\cdot\text{ft}
\]

**Service III**

\[
M_{\text{serviceIII}} := M_{DC} + M_{DW} + 0.8 \cdot M_{LL} \quad M_{\text{serviceIII}} = 1532 \text{ kip}\cdot\text{ft}
\]

**Strength I**

\[
M_u := 1.25 \cdot M_{DC} + 1.5 \cdot M_{DW} + 1.75 \cdot M_{LL} \quad M_u = 2489 \text{ kip}\cdot\text{ft}
\]
6.0 Prestress Layout

Prestressed strand layout:

\[ N_{st} := 16 \]
\[ N_{harp} := 6 \]
\[ F_o := 9\text{in} \]

\[ F_{cl} := \begin{cases} (4.0\text{in}) \text{ if } N_{harp} \leq 12 \\ \left[ 12 - 4.0\text{in} + \left( \frac{N_{harp} - 12}{N_{harp}} \right) \cdot 8.0\text{in} \right] \text{ if } N_{harp} > 12 \end{cases} \]

\[ F_{cl} = 4.00\text{in} \]

\[ E := \begin{cases} 2\text{in} \text{ if } N_{st} \leq 10 \\ \left[ 4\text{in} - \left( \frac{N_{st} - 5}{N_{st}} \right) \right] \text{ if } 10 < N_{st} \leq 18 \\ \left[ \frac{6\text{in} \cdot N_{st} - 56\text{in}}{N_{st}} \right] \text{ if } 18 < N_{st} \leq 22 \\ \left[ 4\text{in} - \left( \frac{N_{st} - 3}{N_{st}} \right) \right] \text{ if } 22 < N_{st} \leq 24 \\ \left[ 6\text{in} - \left( \frac{N_{st} - 10}{N_{st}} \right) \right] \text{ if } 24 < N_{st} \leq 26 \end{cases} \]

\[ E = 2.75\text{in} \]

Distance to the prestressing steel C.G. measured from the bottom of the girder at midspan:

\[ A_{harp} := A_{\text{strands}} \cdot N_{harp} \]
\[ A_{st} := A_{\text{strands}} \cdot N_{st} \]
\[ A_{ps} := A_{st} + A_{harp} \]
\[ N_{strands} := N_{harp} + N_{st} \]
\[ A_{ps} = 4.774 \text{ in}^2 \]
\[ y_{bps} := \frac{N_{harp} \cdot F_{cl} + N_{st} \cdot E}{N_{strands}} \]
\[ y_{bps} = 3.091 \text{in} \]
Which gives a midspan strand eccentricity:

\[ e := y_b - y_{bps} \quad e = 17.8 \text{ in} \]

The prestressing geometry at end of girder is:

Transfer Length

\[ l_t := 60 \cdot d_{\text{strand}} \quad l_t = 36.0 \text{ in} \]

Self-weight moment at transfer point

\[ M_{lt} := w_{dl} \cdot \frac{l_t \cdot (L - l_t)}{2} \quad M_{lt} = 91 \text{ kip} \cdot \text{ft} \]

Prestress offset of harped strands at bottom of girder end

\[ y_{bh} := h - F_0 \quad y_{bh} = 26.0 \text{ in} \]

Prestress offset at transfer point

Offset of harped strands from girder bottom

\[ y_{bhlt} := y_{bh} - \frac{l_t}{0.4L} \cdot (y_{bh} - F_{cl}) \quad y_{bhlt} = 24.1 \text{ in} \]

Offset of the C.G. of all strands from girder bottom

\[ y_{bslt} := \frac{y_{bhlt} \cdot N_{\text{harp}} + E \cdot N_{st}}{N_{\text{strand}}} \quad y_{bslt} = 8.6 \text{ in} \]

Prestress eccentricity at transfer point

\[ e_{lt} := y_b - y_{bslt} \quad e_{lt} = 12.3 \text{ in} \]
7.0 Prestress Force and Losses

Jacking PS force:

\[
  f_{pi} := 0.75 f_{pu} \\
  P_{jack} := A_{ps} f_{pi}
\]

\[P_{jack} = 967 \text{ kip}\]

Estimate of initial PS force after release, \(P_{si}\):

\[
  P_{si} := 0.69 f_{pu} A_{ps} \quad \quad P_{si} = 889 \text{ kip}
\]

Elastic Shortening Losses

\[
  f_{cgp} := \left( \frac{P_{si}}{A_{g}} \right) + \left( \frac{P_{si} e^2}{I_g} \right) - \left( \frac{M_{dl} e}{I_g} \right) \quad f_{cgp} = 2.714 \text{ ksi}
\]

Steel Relaxation Losses (for low-relaxation strands)

\[
  t := 1 \quad \text{(days before transfer)}
\]

\[
  \Delta f_{RE} := \frac{\log(24 \cdot t)}{40} \left( \frac{f_{pi}}{f_{pu}} - 0.55 \right) f_{pi} \quad \Delta f_{RE} = 1.40 \text{ ksi}
\]
\[ \Delta f_{\text{p, instant}} := \Delta f_{\text{p, ES}} + \Delta f_{R} \]

**Release PS force**

\[ P_{sr} := (f_{pi} - \Delta f_{\text{p, instant}}) A_{ps} \]

\[ \Delta f_{\text{td}} := 33 \text{ksi} \left[ 1 - 0.15 \left( \frac{f_{c} - 6 \text{ksi}}{6 \text{ksi}} \right) \right] + 6 \text{ksi} - 8 \text{ksi} \]

**Time Dependent Losses (for low-relaxation strands)**

\[ \Delta f_{\text{total}} := \Delta f_{\text{p, instant}} + \Delta f_{\text{td}} \]

**Effective PS force**

\[ P_{se} := (f_{pi} - \Delta f_{\text{total}}) A_{ps} \]

\[ \Delta f_{\text{p, instant}} = 16.4 \text{ ksi} \]

\[ P_{sr} = 889 \text{ kip} \]

\[ \Delta f_{\text{td}} = 29.35 \text{ ksi} \]

\[ \Delta f_{\text{total}} = 45.70 \text{ ksi} \]

\[ P_{se} = 749 \text{ kip} \]

---

**8.0 Concrete Stresses at Release**

**Allowable stresses:**

**Compression:**

\[ 0.6 \cdot f_{cl} = 3.600 \text{ ksi} \]

**Tension:**

\[ \max (-0.2 \text{ksi}, -\sqrt{f_{cl} \text{ksi}}) = -0.200 \text{ ksi} \]

**Stress at transfer point:**

\[ f_{\text{ttt}} := P_{sr} \left( \frac{1}{A_g} - \frac{e_{lt}}{S_t} \right) + \frac{M_{lt}}{S_t} \]

\[ f_{\text{ttt}} = -0.06 \text{ ksi} \quad \text{OK} \]
Concrete and Steel Stresses at Service

Allowable concrete stress at midspan

Compression; Cases I, II, and III:

0.45 $f_c = 3.600$ ksi (Under total dead load)

0.4 $f_c = 3.200$ ksi (Under half of permanent loads and full live load)

0.6 $f_c = 4.800$ ksi (Under full Service I load)

Tension (per BDM):

0 ksi (Tension check under Service III load)

Concrete stress at midspan:

\[
 f_{tI} := P_e \left( \frac{1}{A_g} - \frac{e}{S_t} \right) + \frac{M_{DC} + M_{DW}}{S_t} \]

\[
 f_{tII} := \frac{1}{2} P_e \left( \frac{1}{A_g} - \frac{e}{S_t} \right) + \left( \frac{M_{DC} + M_{DW}}{S_t} \right) + \frac{M_{LL}}{S_t} \]

\[
 f_{tIII} := P_e \left( \frac{1}{A_g} - \frac{e}{S_t} \right) + \frac{M_{ServiceI}}{S_t} \]

\[
 f_{blt} := P_{sr} \left( \frac{1}{A_g} + \frac{e_t}{S_b} \right) - \frac{M_{lt}}{S_b} \]

\[
 f_{tharp} := P_{sr} \left( \frac{1}{A_g} - \frac{e}{S_t} \right) + \frac{M_{harp}}{S_t} \]

\[
 f_{bharp} := P_{sr} \left( \frac{1}{A_g} + \frac{e}{S_b} \right) - \frac{M_{harp}}{S_b} \]

\[
 f_{blt} = 3.39 \text{ ksi } \quad \text{OK} \]

\[
 f_{tharp} = 0.19 \text{ ksi } \quad \text{OK} \]

\[
 f_{bharp} = 3.02 \text{ ksi } \quad \text{OK} \]

\[
 \text{Stress at harp point:} \]

\[
 f_{tharp} := P_{sr} \left( \frac{1}{A_g} - \frac{e}{S_t} \right) + \frac{M_{harp}}{S_t} \]

\[
 f_{bharp} := P_{sr} \left( \frac{1}{A_g} + \frac{e}{S_b} \right) - \frac{M_{harp}}{S_b} \]
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Steel stress at service

Allowable steel stress; AASHTO LRFD 5.9.3:

\[ 0.8 f_{py} = 194 \text{ ksi} \]

\[ \Delta f_{ps} := n \left( \frac{e}{y_b} \right) \left( \frac{M_{sdl} + M_{DW} + M_{LL}}{S_b} \right) \]

\[ \Delta f_{ps} = 10.2 \text{ ksi} \]

\[ f_{psservice} = f_{pl} - \Delta f_{total} + \Delta f_{ps} \]

\[ f_{psservice} = 167 \text{ ksi} \]

10.0 Flexural Strength Check

As calculated above, the factored load is:

\[ M_u = 2489 \text{ kip-ft} \]

Bonded Steel Stress

\[ \beta_1 := 0.65 \quad k := 0.28 \]

\[ c_{rec} := \frac{(A_{ps} f_{pu})}{0.85 \beta_1 f_c \cdot b + k \cdot A_{ps} f_{pu} (h - y_{bps})} \]

\[ c_{rec} = 5.768 \text{ in} \]

\[ c_{flange} := \frac{A_{ps} f_{pu} - 0.85 \beta_1 f_c (b - 6\text{ in}) \cdot t_S}{0.85 \beta_1 f_c \cdot 6\text{ in} + k \cdot A_{ps} f_{pu} (h - y_{bps})} \]

\[ c_{flange} = 4.630 \text{ in} \]

\[ c := \begin{cases} c_{rec} & \text{if } c_{rec} \leq t_S \\ c_{flange} & \text{otherwise} \end{cases} \]

\[ a := \beta_1 c \]
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Moment capacity at midspan

\[ f_{ps} := f_{pu} \left( 1 - k \cdot \frac{c}{h - y_{bps}} \right) \]

\[ f_{ps} = 256.3 \text{ ksi} \]

\[ \phi := 1.0 \]

\[ \phi M_n := \begin{cases} \phi \cdot A_{ps} f_{ps} \left( h - y_{bps} - \frac{a}{2} \right) & \text{if } c_{rec} \leq 6 \text{ in} \\ \phi \cdot A_{ps} f_{ps} \left( h - y_{bps} - \frac{a}{2} \right) + 0.85 \beta_1 f_c \left( b - 6 \text{ in} \right) \cdot 6 \text{ in} \left( \frac{a}{2} - \frac{6 \text{ in}}{2} \right) & \text{otherwise} \end{cases} \]

\[ = 3063 \text{ kip-ft} \]

11.0 Reinforcement Limits

Maximum RF

\[ \frac{c}{h - y_{bps}} = 0.181 \]

0.42 maximum \( (LRFD 5.7.3.3.1-1) \) OK

Minimum RF

\[ f_{cpe} := P_{se} \left( \frac{1}{A_g} + \frac{e}{S_b} \right) \]

\[ f_{cpe} = 3.902 \text{ ksi} \]

\[ M_{cr} := S_b \left( f_{cpe} + f_r \right) \]

\[ M_{cr} = 1828 \text{ kip-ft} \]

\[ \phi M_n \text{ must be greater than the lesser of } 1.2 M_{cr} \text{ and } 1.33 M_u \text{ } (LRFD 5.7.3.3.2) \]

\[ 1.2M_{cr} = 2194 \text{ kip-ft} \]

\[ 1.33M_u = 3311 \text{ kip-ft} \]

\[ \phi M_n = 3063 \text{ kip-ft} \] OK

12.0 Camber and Deflection

Self-Weight Effect:
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\[ \Delta_{dc} := \frac{5 \cdot w_{dl} \cdot L^4}{384 \cdot E_c \cdot I_g} \]

\[ \Delta_{dc} = -1.686 \text{ in} \]

\[ a_c := 0.4 \cdot L \]

\[ e_h := e - \left[ \frac{y_b - \left( \frac{N_{harp} + E \cdot N_{st}}{N_{strand}} \right)}{N_{strand}} \right] \]

\[ \Delta_{ps} := \frac{P_{sr}}{E_c \cdot I_g} \cdot \left[ \frac{e \cdot L^2}{8} - \frac{e_h (a_c)^2}{6} \right] \]

\[ \Delta_{ps} = 3.689 \text{ in} \]

Superimposed Loads

\[ \Delta_{sdl} := \frac{-5 \left( w_{sdl} + w_{dw} \right) \cdot L^4}{384 \cdot E_c \cdot I_g} - \frac{23P_{dia} \cdot L^3}{648 \cdot E_c \cdot I_g} \]

\[ \Delta_{sdl} = -0.615 \text{ in} \]

Long-term deflections from BDM multiplier method (Table 5-20):

Camber at Transfer

\[ C_i := \Delta_{ps} + \Delta_{dc} \]

\[ C_i = 2.00 \text{ in} \]

Camber at 2000 days

\[ C_{final} := 2.50 \cdot \Delta_{dc} + 2.25 \cdot \Delta_{ps} \]

\[ C_{final} = 4.09 \text{ in} \]

Deflection from barrier and overlay

\[ C_{sdl} := 2.75 \cdot \Delta_{sdl} \]

\[ C_{sdl} = -1.69 \text{ in} \]

Final Camber

\[ C_{fsdl} := C_{final} + C_{sdl} \]

\[ C_{fsdl} = 2.39 \text{ in} \]
# Prestressed Voided Slab with Cast-in-Place Topping

## General Input

<table>
<thead>
<tr>
<th>Specification</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder type:</td>
<td>18</td>
</tr>
<tr>
<td>Span Length:</td>
<td>L = 58.00 ft</td>
</tr>
<tr>
<td>Girder Length:</td>
<td>L&lt;sub&gt;g&lt;/sub&gt; = 58.83 ft</td>
</tr>
<tr>
<td>Bridge Width:</td>
<td>W = 42.75 ft</td>
</tr>
<tr>
<td>Number of Lanes</td>
<td>2</td>
</tr>
<tr>
<td>Skew Angle:</td>
<td>θ&lt;sub&gt;skew&lt;/sub&gt; = 0.00 degrees</td>
</tr>
</tbody>
</table>

## Girder Section Properties

| Girder Width: | b = 4.00 ft = 48.00 in | BDM 5-A-XX |
| Girder Depth: | d = 18.00 in = 1.50 ft | |
| Height:       | h = 23.00 in | |
| Top Flange Thickness: | t<sub>tf</sub> = 4.50 in (from top of void to bottom of slab) | |
| Bottom Flange Thickness: | t<sub>bf</sub> = 4.50 in (from bottom of void to bottom of girder) | |
|                   | h<sub>f</sub> = t<sub>tf</sub> + t<sub>bf</sub> = 9.50 in | |
| Width of each Void: | b<sub>eachV</sub> = 9.00 in | |
| Net Width of Girder: | b<sub>w</sub> = 21.00 in | |
| Number of Voids: | N<sub>v</sub> = 3 | |
| Area of Each Void: | A<sub>each v</sub> = 63.62 in<sup>2</sup> | |
| Void Area: | A<sub>v</sub> = 190.85 in<sup>2</sup> | |
| Area of Girder: | A<sub>g</sub> = 673.15 in<sup>2</sup> | |
| Area of Deck + Leg: | A<sub>d</sub> = 240.00 in<sup>2</sup> | |
| Area of Comp. Sect.: | A<sub>comp</sub> = 913.15 in<sup>2</sup> | |
| Number of girders: | N<sub>g</sub> = 10 | |
| Wt of barrier: | w<sub>TB</sub> = 0.50 k/ft | BDM 5.1.1-D |
| Thickness of deck: | t<sub>d</sub> = 5.00 in | |
| Wt of Concrete: | w<sub>c</sub> = 0.155 kcp for calculating E<sub>c</sub> | |
| Wt of Concrete: | w<sub>cd</sub> = 0.160 kcp for dead load calculations | BDM 3.1.1 |

## Strength of Concrete

| Deck | f<sub>c′</sub> = 4.0 ksi | BDM 5.1.1-A.1 |
| Final | f<sub>c</sub> + = 8.5 ksi | BDM 5.1.1-A.2 |

### Modulus of Elasticity

- Girder: 
  \[ E_c = 33000w \left( \frac{1.5}{f_c} \right) \sqrt{f_{c'}}, = 5871.1 \text{ ksi} \]
- Deck: 
  \[ E_c = 33000w \left( \frac{1.5}{f_c} \right) \sqrt{f_{c'}}, = 4027.6 \text{ ksi} \]
- Transfer: 
  \[ E_{c'} = 33000w \left( \frac{1.5}{f_{c'}}, \right) \sqrt{f_{c'}}, = 328.0 \text{ ksi} \]

\[ nc = 1.46 \text{ ratio for transformed section} \]

### Modulus of Rupture

- Final: 
  \[ f_r = 0.24 \sqrt{f_{c'}}, = 0.700 \text{ ksi} \]
- Minimum: 
  \[ f_{r\text{Min}} = 0.37 \sqrt{f_{c'}}, = 0.08 \text{ ksi} \]

### Poisson's Ratio

- m = 0.2

---

Prestressed Voided Slab with Cast-in-Place Topping | AASHTO LRFD Specifications | Reference |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder type: 18</td>
<td>Prestressed Precast Flat Slab</td>
<td>Prelim.Plan, Sh 1</td>
</tr>
<tr>
<td>Span Length: L = 58.00 ft</td>
<td>C.L. to C.L. Bearing</td>
<td>BDM 5.6.2-A</td>
</tr>
<tr>
<td>Girder Length: L&lt;sub&gt;g&lt;/sub&gt; = 58.83 ft</td>
<td>End to End</td>
<td>BDM fig. 5-A-XX</td>
</tr>
<tr>
<td>Bridge Width: W = 42.75 ft</td>
<td>Deck Width</td>
<td>Prelim.Plan, Sh 1</td>
</tr>
<tr>
<td>Number of Lanes: 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Skew Angle: θ&lt;sub&gt;skew&lt;/sub&gt; = 0.00 degrees</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

WSDOT Bridge Design Manual  M 23-50.14
April 2015
### Reinforcing Steel - deformed bars

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength</td>
<td>f'_y = 60.00 ksi</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>E_s = 29000.00 ksi</td>
</tr>
</tbody>
</table>

### Prestressing Input

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand diam.</td>
<td>d_b = 0.60 in</td>
</tr>
<tr>
<td>Area</td>
<td>0.217 in²</td>
</tr>
<tr>
<td>Ultimate Strength</td>
<td>f_{pu} = 270.00 ksi</td>
</tr>
<tr>
<td>Yield Strength</td>
<td>f_{py} = 0.9 f_{pu} = 243.00 ksi</td>
</tr>
<tr>
<td>Prior to Transfer</td>
<td>f_{pbt} = 0.75 f_{pu} = 202.50 ksi</td>
</tr>
<tr>
<td>Effective Stress Limit</td>
<td>f_{pe} = 0.8 f_{py} = 194.40 ksi</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E_p = 28500 ksi</td>
</tr>
</tbody>
</table>

### Number of Strands

<table>
<thead>
<tr>
<th>Location</th>
<th>Bonded Strands</th>
<th>Debonded Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>~2 in from Bottom</td>
<td>14</td>
<td>OK</td>
</tr>
<tr>
<td>~4 in from Bottom</td>
<td>6</td>
<td>OK</td>
</tr>
<tr>
<td>~6 in from Bottom</td>
<td>0</td>
<td>OK</td>
</tr>
</tbody>
</table>

### Eccentricities of Prestress Strands

<table>
<thead>
<tr>
<th>Eccentricity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C. G. of bottom strands to bottom of girder</td>
<td>2.50 in</td>
</tr>
<tr>
<td>C. G. of top strands to bottom of girder</td>
<td>15.00 in</td>
</tr>
<tr>
<td>C. G. of bonded bottom strands to C.G. of girder</td>
<td>6.40 in</td>
</tr>
<tr>
<td>C. G. of debonded strands to C.G. of girder</td>
<td>7.00 in</td>
</tr>
<tr>
<td>C. G. of all bottom strands to C.G. of girder</td>
<td>6.50 in</td>
</tr>
<tr>
<td>C. G. of top strands to C.G. of girder</td>
<td>6.00 in</td>
</tr>
</tbody>
</table>

### E = C. G. of all strands to C.G. of girder = 4.71 in.

### Output

#### HS20-44 Force Effect:
- **Jacking Force, P_j** = 1230.4 kips

#### Live Load Force Effect:
- Moment = 1294.23 ft-kips per lane
- Reaction = 95.69 kips per lane

### Service Limit State

<table>
<thead>
<tr>
<th>Concrete Stresses at Transfer</th>
<th>Load Case</th>
<th>Top of Girder</th>
<th>Bottom of Girder</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>At d_v</td>
<td>DL+P/S</td>
<td>0.099</td>
<td>0.503</td>
<td>OK</td>
</tr>
<tr>
<td>At mid-span</td>
<td>DL+P/S</td>
<td>-1.031</td>
<td>0.503</td>
<td>OK</td>
</tr>
</tbody>
</table>

### Concrete Stresses at const.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Calculated-Allowable</th>
<th>Calculated-Allowable</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>At mid-span</td>
<td>DL+P/S</td>
<td>-1.639</td>
<td>0.503</td>
</tr>
</tbody>
</table>

### Concrete Stresses at Service

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Limit State I</th>
<th>Limit State III</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>At mid-span</td>
<td>DL+LL+P/S</td>
<td>-2.430</td>
<td>-2.584</td>
</tr>
<tr>
<td></td>
<td>DL+P/S</td>
<td>-1.639</td>
<td>-3.825</td>
</tr>
<tr>
<td></td>
<td>LL+1/2DL+1/2P/S</td>
<td>-1.556</td>
<td>-3.400</td>
</tr>
</tbody>
</table>

### Strength Limit State

| Moment at Mid-span, ft-kips | Mu = 1386 | f Mn = 1856.9 | OK |

---

**Reference**
- LRFD 5.4.3
- BDM 5.1.2
- LRFD 5.4.3.2
- LRFD Table 5.4.1-1
- LRFD Table 5.9.3-1
- LRFD 5.4.4.2
- BDM fig. 5-A-XX
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- Compressive stress limits after all losses

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   Flexural forces
   Flexural resistance
   Nominal flexural resistance
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   Development of prestressing strand

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   Effective Web Width, bv, and Effective Shear Depth, dv
   Component of Prestressing Force in Direction of Shear Force, Vp
   Shear Stress Ratio
   Factored shear force
   fpo
   Factored moment
   Longitudinal Strain (Flexural Tension)
   Determination of b and q
   Shear strength
   Required shear strength
   Maximum spacing of shear reinforcement
   Minimum shear reinforcement
   Longitudinal reinforcement

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   Deflection due to prestressing forces at Transfer
   Deflection due to weight of Girder
   Deflection due to weight of Traffic Barrier TB
   Deflection due to weight of Deck and Legs
   Deflection (Camber) at transfer, Ci
   Creep Coefficients (Table 13-1)
   Final Deflection Due to All Loads and Creep
   Time Verses Deflection Curve (fig. 13-1)

References
Prestressed Voided Slab Design

AASHTO LRFD Specifications

1 Structure:

Project XL2526, Name Br #539/858E

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Length</td>
<td>58.00 ft</td>
<td>Prelim Plan, Sh 1</td>
</tr>
<tr>
<td>Girder Length</td>
<td>58.83 ft</td>
<td>BDM fig. 5-A-XX</td>
</tr>
<tr>
<td>Bridge Width</td>
<td>42.75 ft</td>
<td>Prelim Plan, Sh 1</td>
</tr>
<tr>
<td>Girder Width</td>
<td>4.00 ft</td>
<td></td>
</tr>
<tr>
<td>Number of girders</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

2 Live load

HL-93

Vehicular live load designated as " HL-93 " shall consist of a combination of:

- Design truck or design tandem, plus
- Design lane load

Design truck is equivalent to AASHTO HS20-44 truck.

The design lane shall consist of a 0.64 klf, uniformly distributed in the longitudinal direction. Design lane load shall be assumed to be uniformly distributed over 10 ft width in the transverse direction.

Design tandem shall consist of a pair of 25.0 kip axles spaced at 4'-0" apart

Number of design lanes:

- Integer part of: Width / (12 ft lane) = 3 Lanes
- and 2 lanes if width is 20-24 ft.

3 Material Properties

Concrete

LRFD Specifications allows a concrete compressive strength with a range of 2.4 to 10.0 ksi at 28 days. Compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

4 Allowable Concrete Stresses at Service Limit State

Tensile stress limit

For service loads which involve traffic loading, tensile stress in members with bonded or unbonded prestressing strands shall be investigated using Service - III load combination.

Tension in other than precompressed tensile zone assuming uncracked section:

$$ f_t = 0.19 \sqrt{f_{ci}} $$

Transfer & Lifting

$$ f_t = 0.19 \sqrt{f_c} $$

Shipping

BDM 5.2.3-B
Tension in precompressed tensile zone:
\[ f_t = 0.00 \text{ ksi} \]

**Compressive stress limits after all losses**
Compression shall be investigated using Service - I load combination: LRFD 5.9.4.2

- \( f_c = 0.45 f'_c \) Due to permanent loads
- \( f_c = 0.60 f'_c \) To all load combinations
- \( f_c = 0.40 f'_c \) Due to transient loads and one-half of permanent loads BDM 5.2.3-B

5 Computation of Section Properties
Girder and Composite Section Properties

<table>
<thead>
<tr>
<th>Table 5-1: Moment of Inertia, I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
</tr>
<tr>
<td>(in^3)</td>
</tr>
<tr>
<td>Girder</td>
</tr>
<tr>
<td>Legs</td>
</tr>
<tr>
<td>Deck</td>
</tr>
<tr>
<td>Composite</td>
</tr>
</tbody>
</table>

\[ Y_{bg} = 9.00 \text{ in} \]
\[ Y_{tg} = 9.0 \text{ in} \]
\[ Y_{bc} = 12.02 \text{ in} \]
\[ Y_{tgc} = 6.0 \text{ in} \]
\[ Y_{tsc} = 11.0 \text{ in} \]

Torsional Moment of Inertia
\[ J = 55820 \text{ in}^4 \]

Section Modulus:
Girder
- Bottom: \[ S_b = \frac{I_x}{y_{be}} = 2446.8 \text{ in}^3 \]
- Top: \[ S_t = \frac{I_x}{y_{tg}} = 2446.8 \text{ in}^3 \]

Composite
- Bottom: \[ S_b = \frac{I_{comp}}{y_{bc}} = 3819.5 \text{ in}^3 \]
- Top girder: \[ S_t = \frac{I_{comp}}{y_{tgc}} = 7682.1 \text{ in}^3 \]
- Top slab: \[ S_t = \frac{I_{comp}}{y_{tsc}} = 4183.06 \text{ in}^3 \]

Transformed section properties
Deck
\[ b_c = \frac{b}{nc} = 32.93 \text{ in} \]

Legs
\[ A_{lege} = 0.00 \text{ in}^2 \]
\[ Y_{blege} = 0.00 \text{ in} \]
\[ I_{klege} = 0.0 \text{ in}^4 \]
### Table 5-2: Moment of Inertia Transformed section, I

<table>
<thead>
<tr>
<th>Area</th>
<th>Y_b</th>
<th>Ix</th>
<th>d</th>
<th>Ad^2</th>
<th>Ix</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>673.15</td>
<td>9.00</td>
<td>22021.6</td>
<td>2.3</td>
<td>3438.0</td>
</tr>
<tr>
<td>Legs</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Deck</td>
<td>164.64</td>
<td>20.50</td>
<td>500.0</td>
<td>-9.2</td>
<td>14056.6</td>
</tr>
<tr>
<td>Composite</td>
<td>837.8</td>
<td>11.26</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

- Ybgt = 9.00 in
- Ytgt = 9.0 in
- Ybct = 11.26 in
- Ytgct = 6.7 in
- Ytsct = 11.7 in

\[
\begin{align*}
\text{Composite (Bottom)} & \quad S_{bt} = \frac{I_{\text{compt}}}{y_{bcr}} = 3553.9 \text{ in}^3 \\
\text{(Top girder)} & \quad S_{tg} = \frac{I_{\text{compt}}}{y_{tgct}} = 5937.1 \text{ in}^3 \\
\text{(Top slab)} & \quad S_{ts} = \frac{I_{\text{compt}}}{y_{tsct}} = 3408.52 \text{ in}^3
\end{align*}
\]

### 6 Limit State

Each component and connection shall satisfy the following equation for each limit state:

\[
\sum \eta_i \gamma_i Q_i \leq \phi R_s = R_i
\]

Where:

- Load Modifier for Ductility, Red for loads which a max. value of \( g_i \) is appropriate
  \[ h_i = \frac{\eta_i}{\eta D} \geq 0.95 \]

- Load Modifier for Redundancy, Rf for loads which a min. value of \( g_i \) is appropriate
  \[ h_i = \frac{1}{\eta_D \eta R \eta I} \leq 1.00 \]

- Ductility factor
- Redundancy factor
- Operational Importance factor

\[ h_i = 1.00 \quad \text{for any ordinary structure} \]

Therefore the Limit State Equation simplifies to:

\[
\sum \gamma_i Q_i \leq \phi R_s = R_i
\]

Where:

- \( g_i \) = Load Factor, statistically based multiplier applied to force effects
- \( Q_i \) = Force Effect (Moment or Shear)
- \( f \) = Resistance Factor
- \( R_n \) = Nominal Resistance
- \( R_e \) = Factored Resistance
Concrete Structures

Chapter 5

Service limit state

Service limit state shall be taken as restriction on stress, deformation and crack width under regular service conditions.

Load combinations and load factors

The Total Factored Force Effect shall be taken as:

\[ Q = \sum \gamma_i Q_i \]

Where:

- \( g_i \) = Load Factors specified in Tables 1 & 2
- \( Q_i \) = Force Effects from loads specified in LRFD

Strength-I load combination relating to the normal vehicle use of the bridge without wind.

Service-I load combination relating to the normal operational use of the bridge. 

Service-III load combination relating only to tension in prestressed concrete structures with the objective of crack control.

\[ Q_{\text{Strength-I}} = \gamma_{DC} D + \gamma_{DW} D + 1.75(\text{LL} + \text{IM}) \]

\[ Q_{\text{Service-I}} = 1.0 (\text{DC + DW}) + 1.0 (\text{LL + IM}) \]

\[ Q_{\text{Service-III}} = 1.0 (\text{DC + DW}) + 0.8 (\text{LL + IM}) \]

Effects due to shrinkage and creep are not considered.

7 Vehicular Live Load

Design vehicular live load

Design live load designated as HL-93 shall be taken as:

\[ \text{LL} = [\text{Truck or tandem}] (1 + \text{IM}) + \text{Lane} \]

- Single Span Length = 58.00 ft
- HS-20 Truck Axles 32.00 32.00 8.00 kips
- HS-20 Truck Axle Spacing 14.00 14.00 ft
- Tandem Truck Axles 25.00 25.00 kips
- Tandem Truck Axle Spacing 4.00 ft
- Lane load density, \( w_L \) = 0.64 k/ft

Maximum live load force effect

Max Shear, \( V_{\text{max}} \), occurs at the horizontal distance of \( d_v \) from the face of support where \( d_v \) is the effective depth between the tensile and compressive resultant forces in the member and is \( \geq \text{Max} [0.72 h \text{ or } 0.9de] \).

Max Moment, \( M_{\text{max}} \), occurs near midspan (CL) underneath the nearest concentrated load (P1) when that load is the same distance to midspan as the center of gravity (+ CG) is to midspan. Use the Truck or Tandem (Near Midspan) and the Lane (At Midspan) maximum moments together to be conservative.
So the HL-93 Live Load, \( LL = HS-20 \text{ Truck}(1+IM)+\text{Lane Load} \) Governs

Near Center line

\[
\begin{align*}
M_{\text{max}} &= 770.759 \text{ ft-kips} & \text{Corresponding } V_{@M_{\text{max}}} &= 25.10 \text{ kips} \\
M_{@dv} &= 25.4199 \text{ ft-kips} & \text{Corresponding } V_{@dv} &= 17.66 \text{ kips} \\
M_{@CL} &= 269.12 \text{ ft-kips} & \text{Corresponding } V_{@CL} &= 0.00 \text{ kips}
\end{align*}
\]

Lane Loading

\[
\begin{align*}
V_{\text{max}} &= 58.67 \text{ kips} & M_{@V_{\text{max}}} &= 82.35 \text{ ft-kips} \\
M_{@dv} &= 25.4199 \text{ ft-kips} & \text{Corresponding } V_{@dv} &= 17.66 \text{ kips}
\end{align*}
\]

Dynamic load allowance (Impact, IM)

\[
\begin{align*}
\text{IM} &= 33\% & \text{For bridge components (girder)} \\
\text{LRFD 3.6.2.1-1}
\end{align*}
\]

\[
\begin{align*}
\text{At } dv \\
M_{(L+IM)} &= 134.9 \text{ k-ft} \\
V_{(L+IM)} &= 95.7 \text{ kips}
\end{align*}
\]

Distribution of Live Load, \( D_f \) (Beam Slab Bridges)

For Multibeam deck bridges with conditions as follows, the approximate method of live load distribution applies with the following conditions:

\[
\begin{align*}
\text{Width of deck is constant} & \quad \text{LRFD 4.6.2.2.1-1} \\
\text{Number of Beams, } N_b & \geq 4 \\
\text{Beams are parallel} & \quad \text{"} \\
\text{Beams have approximately the same stiffness} & \quad \text{"} \\
\text{Roadway overhang, } d_e & \leq 3.0 \text{ ft} \\
\text{Curvature in plane is less than } 12 \text{ degree} & \quad \text{"} \\
\text{X-section is one consistent with one listed in LRFD Table 4.6.2.2.1-1} & \quad \text{"}
\end{align*}
\]

The multiple presence factor shall not be applied in conjunction with approximate load distribution except for exterior beams.

The typical x-section applies to voided and solid slabs w P.T.

The composite deck makes the section significantly connected to act as a unit.

Distribution Factor for Moment Interior Girder, \( D_{f_{\text{Min}}} \)

For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies:

\[
\begin{align*}
\text{Range of applicability: Width of beam (b), } 35 &\leq 48 \leq 60 \text{ in} \\
\text{Span length (L), } 20 &\leq 58.00 \leq 120 \text{ ft} \\
\text{Number of Beams (N_{by}), } 5 &\leq 10.00 \leq 20 \\
\text{LRFD 4.6.2.2.2b-1}
\end{align*}
\]

\[
\begin{align*}
k &= 2.5 N_b^{-0.2} \geq 1.5 = 1.58 \\
D_{f_{i}} &= k(b/305)^{0.6} (b/12L)^{0.2} (I/J)^{0.06} = 0.301
\end{align*}
\]
Skew Reduction Factor for Moments

Range of applicability: \( \theta_{\text{skew}}, 0 \leq 60^\circ \) LRFD 4.6.2.2e-1
if \( \theta_{\text{skew}} \geq 60^\circ \) then use \( \theta_{\text{skew}} = 60^\circ \)
Reduction Factor = 1.05 - 0.25 \( \tan(q) \) \leq 1.0
Reduction Factor = 1.000

Moment Distribution Factor for Skewed Interior girder, 
\[ DF_{\text{MInt}} = 0.301 \]

Moment Distribution Factor for Exterior girder, \( DF_{\text{MExt}} \) LRFD 4.6.2.2d
For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies:

\[ DF_{\text{MExt}} = e \times DF_{\text{MInt}} \]

Where Skew Reduction Factor is included in \( DF_{\text{MInt}} \) and Correction Factor LRFD 4.6.2.2d-1
\[ e = 1.04 + \frac{d_e}{25} \geq 1.0 \]
\[ \text{barrier footprint} = 18.50 \text{ in} \]
\[ d_e = 2.00 \text{ ft} \]
\[ e = 1.12 \]

Moment Distribution Factor for Skewed Exterior Girder, 
\[ DF_{\text{MExt}} = 0.337 \]

Shear Distribution Factors
Shear Distribution Factor for Interior Girder, \( DF_{\text{VInt}} \) LRFD 4.6.2.2.3a
For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies:

Range of applicability:
- Width of beam (b), \( 35 \leq 48 \leq 60 \text{ in} \) LRFD Table 4.6.2.2.3a-1
- Span length (L), \( 20 \leq 58.00 \leq 120 \text{ ft} \)
- Number of Beams (\( N_b \)), \( 5 \leq 10 \leq 20 \)
- St Venant Torsional Inertia (\( J \)), \( 25000 \leq 55820 \leq 610000 \text{ in}^4 \)
- Net Moment of Inertia (\( I_c \)), \( 40000 \leq 45919 \leq 610000 \text{ in}^4 \)

By substituting the above pre-determined values, the approximate live load distribution factor for shear may be taken as the greater of:

One design Lane Loaded:
\[ DF_{\text{VInt}} = \left( \frac{b}{130L} \right)^{0.15} \left( \frac{I_c}{J} \right)^{0.05} \]
\[ = 0.447 \]

Two or more Lanes Loaded:
\[ DF_{\text{VInt}} = \left( \frac{b}{156} \right)^{0.4} \left( \frac{b}{12.0L} \right)^{0.1} \left( \frac{I_c}{J} \right)^{0.05} \left( \frac{b}{48} \right) \]
\[ = 0.456 \]
Skew Reduction Factor for Shear

Range of applicability:

- Skew \( \left( q_{skew} \right) \), \( 0 \leq 0.00 \leq 60^\circ \)
- Span length (L), \( 20 \leq 58.00 \leq 120 \text{ ft} \)
- Depth of beam or stringer (d), \( 17 \leq 18 \leq 60 \text{ in} \)
- Width of beam (b), \( 35 \leq 48 \leq 60 \text{ in} \)
- Number of Beams \( (N_b) \), \( 5 \leq 10 \leq 20 \)

\[
RF_\theta = 1.0 + \frac{12.0L}{90d} \sqrt{\frac{\tan \theta}{100}} = 1.000
\]

Shear Distribution Factor for Skewed Interior Girder,
\( DF_{VInt} = 0.456 \)

Shear Distribution Factor for Skewed Exterior Girder
For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies:

Range of applicability:

- Overhang, \( x_e = 2.00 \leq 2.0 \text{ ft} \)
- Width of Beam (b), \( 35 \leq 48.00 \leq 60 \)

One design lane loaded:

\[
e = 1.25 + \frac{d_x}{20} \geq 1.0 = 1.06
\]

\( DF_{VExt} = e \times DF_{VInt} \)

Two or more lanes loaded:

\[
e = 1 + \left( \frac{d_x + b / 12 - 2}{40} \right)^{0.5} \geq 1.0 = 1.32
\]

\( DF_{VExt} = e \times DF_{VInt} \times 48/b \)

Shear Distribution Factor for Skewed Exterior Girder,
\( DF_{VExt} = 0.600 \)

Table 7-2: Summary of Live Load Distribution Factors:

<table>
<thead>
<tr>
<th>Girder</th>
<th>Moment</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior Girder</td>
<td>( DF_{MInt} = 0.301 )</td>
<td>( DF_{VInt} = 0.456 )</td>
</tr>
<tr>
<td>Exterior Girder</td>
<td>( DF_{MExt} = 0.337 )</td>
<td>( DF_{VExt} = 0.600 )</td>
</tr>
</tbody>
</table>
Table 7-3: Distributed Live Load

<table>
<thead>
<tr>
<th>Simple span</th>
<th>Moment, ft-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dv = 3'</td>
</tr>
<tr>
<td>Interior Girder</td>
<td>40.63</td>
</tr>
<tr>
<td>Exterior Girder</td>
<td>45.51</td>
</tr>
</tbody>
</table>

Simple span | Shear, kips

|               | dv = 3' | 6'  | 9'  | 0.5 L |
| Interior Girder | 43.63    | 41.96  | 38.83 | 35.70  | 14.80  |
| Exterior Girder  | 57.43    | 55.23  | 51.11 | 46.98  | 19.49  |

8 Computation of Stresses

Sign convention: + Tensile stress
- Compressive stress

Stresses due to Weight of Girder

Unit weight girder, \( w_g = 0.72 \) k/ft
Transfer Length = \( d_b \) (60) = 36.00 in

**LRFD 3.3.2**

**LRFD 5.11.4.1**

### Table 8-1

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>( V_G ) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>20.01</td>
</tr>
<tr>
<td>3.00</td>
<td>18.84</td>
</tr>
<tr>
<td>6.00</td>
<td>16.67</td>
</tr>
<tr>
<td>9.00</td>
<td>14.49</td>
</tr>
<tr>
<td>mid-span = 29.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**AISC LRFD p 5-162**

### Table 8-2

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>( M_G ) (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>28.31</td>
</tr>
<tr>
<td>3.00</td>
<td>59.78</td>
</tr>
<tr>
<td>6.00</td>
<td>113.03</td>
</tr>
<tr>
<td>9.00</td>
<td>159.77</td>
</tr>
<tr>
<td>mid-span = 29.00</td>
<td>304.68</td>
</tr>
</tbody>
</table>

At Transfer Using Full Length of the Girder

**LRFD 3.3.2**

### Table 8-3

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>( V_G ) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.80</td>
<td>20.01</td>
</tr>
<tr>
<td>3.00</td>
<td>19.14</td>
</tr>
<tr>
<td>6.00</td>
<td>16.97</td>
</tr>
<tr>
<td>9.00</td>
<td>14.79</td>
</tr>
<tr>
<td>mid-span = 29.42</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**AISC LRFD p 5-162**

### Table 8-4

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>( M_G ) (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.80</td>
<td>37.13</td>
</tr>
<tr>
<td>3.00</td>
<td>60.68</td>
</tr>
<tr>
<td>6.00</td>
<td>113.03</td>
</tr>
<tr>
<td>9.00</td>
<td>162.48</td>
</tr>
<tr>
<td>mid-span = 29.4167</td>
<td>313.50</td>
</tr>
</tbody>
</table>
Table 8-5: Stresses due to Girder Dead Load, $s_G$

<table>
<thead>
<tr>
<th>dv</th>
<th>3 ft</th>
<th>6 ft</th>
<th>9 ft</th>
<th>0.5 L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of girder ksi</td>
<td>-0.139</td>
<td>-0.293</td>
<td>-0.554</td>
<td>-0.784</td>
</tr>
<tr>
<td>Bottom of girder ksi</td>
<td>0.139</td>
<td>0.293</td>
<td>0.554</td>
<td>0.784</td>
</tr>
<tr>
<td>Top of girder at transfer ksi</td>
<td>-0.182</td>
<td>-0.298</td>
<td>-0.563</td>
<td>-0.797</td>
</tr>
<tr>
<td>Bottom of girder at transfer ksi</td>
<td>0.182</td>
<td>0.298</td>
<td>0.563</td>
<td>0.797</td>
</tr>
</tbody>
</table>

Concrete stresses due to Traffic Barrier (DC)

Weight of Traffic Barrier over three girders, $w_{TB3G} = \frac{W_{TB}}{3} = 0.17 \text{ k/ft}$

Table 8-6

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>$V_{TB}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>4.60</td>
</tr>
<tr>
<td>3.00</td>
<td>4.33</td>
</tr>
<tr>
<td>6.00</td>
<td>3.83</td>
</tr>
<tr>
<td>9.00</td>
<td>3.33</td>
</tr>
<tr>
<td>mid-span</td>
<td>29.00</td>
</tr>
</tbody>
</table>

Table 8-7

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>$M_{TB}$ (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>6.51</td>
</tr>
<tr>
<td>3.00</td>
<td>13.75</td>
</tr>
<tr>
<td>6.00</td>
<td>26.00</td>
</tr>
<tr>
<td>9.00</td>
<td>36.75</td>
</tr>
<tr>
<td>mid-span</td>
<td>29.00</td>
</tr>
</tbody>
</table>

Table 8-8: Stresses due to Traffic Barrier, $s_{TB}$ Comp.

<table>
<thead>
<tr>
<th>dv</th>
<th>3 ft</th>
<th>6 ft</th>
<th>9 ft</th>
<th>0.5 L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of slab ksi</td>
<td>-0.019</td>
<td>-0.039</td>
<td>-0.075</td>
<td>-0.105</td>
</tr>
<tr>
<td>Top of girder ksi</td>
<td>-0.010</td>
<td>-0.021</td>
<td>-0.041</td>
<td>-0.057</td>
</tr>
<tr>
<td>Bottom of girder ksi</td>
<td>0.020</td>
<td>0.043</td>
<td>0.082</td>
<td>0.115</td>
</tr>
</tbody>
</table>

Concrete stresses due to Concrete Deck and Legs (D+L)

Area of deck + Legs = 240.00 in$^2$
Extra Concrete from A dimension = 24.00 in$^2$
Total Deck = 264.00 in$^2$
Weight of Concrete Deck, $w_{SIDL} = 0.29 \text{ k/ft}$

Table 8-9

<table>
<thead>
<tr>
<th>x (ft)</th>
<th>$V_{D+L}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>8.10</td>
</tr>
<tr>
<td>3.00</td>
<td>7.63</td>
</tr>
<tr>
<td>6.00</td>
<td>6.75</td>
</tr>
<tr>
<td>9.00</td>
<td>5.87</td>
</tr>
<tr>
<td>mid-span</td>
<td>29.00</td>
</tr>
</tbody>
</table>
Table 8-10

<table>
<thead>
<tr>
<th>dv (ft)</th>
<th>( M_{SDL} ) (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.38</td>
<td>11.46</td>
</tr>
<tr>
<td>3.00</td>
<td>24.20</td>
</tr>
<tr>
<td>6.00</td>
<td>45.76</td>
</tr>
<tr>
<td>9.00</td>
<td>64.68</td>
</tr>
<tr>
<td>mid-span</td>
<td>123.35</td>
</tr>
</tbody>
</table>

\[
M_{DL+L} = \frac{w_{DL+L} x (L - x)}{2}
\]

Table 8-11: Stresses due to Deck and Legs, \( s_{DW} \)

<table>
<thead>
<tr>
<th>dv</th>
<th>3 ft</th>
<th>6 ft</th>
<th>9 ft</th>
<th>0.5 L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of girder ksi</td>
<td>-0.056</td>
<td>-0.119</td>
<td>-0.224</td>
<td>-0.317</td>
</tr>
<tr>
<td>Bottom of girder ksi</td>
<td>0.056</td>
<td>0.119</td>
<td>0.224</td>
<td>0.317</td>
</tr>
</tbody>
</table>

Stresses in Girder due to LL+IM (composite section):

Table 8-12:

<table>
<thead>
<tr>
<th>dv</th>
<th>3 ft</th>
<th>6 ft</th>
<th>9 ft</th>
<th>0.5 L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Slab ksi</td>
<td>-0.131</td>
<td>-0.270</td>
<td>-0.506</td>
<td>-0.707</td>
</tr>
<tr>
<td>Top of Girder ksi</td>
<td>-0.071</td>
<td>-0.147</td>
<td>-0.275</td>
<td>-0.385</td>
</tr>
<tr>
<td>Bottom of Girder ksi</td>
<td>0.143</td>
<td>0.296</td>
<td>0.554</td>
<td>0.774</td>
</tr>
</tbody>
</table>

Summary of stresses at dv

Table 8-13:

<table>
<thead>
<tr>
<th>Stresses, ksi</th>
<th>Top of girder</th>
<th>Bottom of girder</th>
<th>Top of slab</th>
<th>Bottom of girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Girder</td>
<td>-0.139</td>
<td>0.139</td>
<td>-0.019</td>
<td>-0.010</td>
</tr>
<tr>
<td>Weight Traffic Barrier</td>
<td>--</td>
<td>--</td>
<td>-0.131</td>
<td>-0.071</td>
</tr>
<tr>
<td>Weight of Deck</td>
<td>-0.056</td>
<td>0.056</td>
<td>-0.104</td>
<td>0.143</td>
</tr>
<tr>
<td>Live Load plus Impact Service - I</td>
<td>--</td>
<td>--</td>
<td>-0.131</td>
<td>0.143</td>
</tr>
<tr>
<td>Live Load plus Impact Service - III</td>
<td>--</td>
<td>--</td>
<td>-0.104</td>
<td>0.114</td>
</tr>
</tbody>
</table>

Summary of stresses at Mid-Span

Table 8-14:

<table>
<thead>
<tr>
<th>Stresses, ksi</th>
<th>Top of girder</th>
<th>Bottom of girder</th>
<th>Top of slab</th>
<th>Bottom of girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Girder</td>
<td>-1.494</td>
<td>1.494</td>
<td>-0.201</td>
<td>-0.109</td>
</tr>
<tr>
<td>Weight Traffic Barrier</td>
<td>--</td>
<td>--</td>
<td>-0.605</td>
<td>0.605</td>
</tr>
<tr>
<td>Weight of Deck</td>
<td>-0.605</td>
<td>0.605</td>
<td>-1.252</td>
<td>-0.682</td>
</tr>
<tr>
<td>Live Load plus Impact Service - I</td>
<td>--</td>
<td>--</td>
<td>-1.002</td>
<td>-0.545</td>
</tr>
<tr>
<td>Live Load plus Impact Service - III</td>
<td>--</td>
<td>--</td>
<td>1.097</td>
<td></td>
</tr>
</tbody>
</table>

9 Approximate Evaluation of Pre-Stress Losses

For Prestress losses in members constructed and prestressed in a single stage, relative to the stress immediately before transfer, in pretensioned members, with low relaxation strands, the Total Lump Sum Losses may be taken as:

\[
\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pL}
\]
Time Dependent Losses

For normal weight concrete pretensioned by low-relaxation strands, approximate lump-
sum time dependent losses resulting from creep and shrinkage of concrete and relaxation
of prestressing steel may be used as follows:

\[ \Delta f_{pLT} = 10 \frac{f_{pR}}{A_g} Y_h \gamma_{st} + 12 \gamma_{st} \Delta f_{pR} = 19.53 \text{ ksi} \]  

\[ Y_h = 1.7 - 0.01H = 0.9 \]  

\[ Y_{st} = 5/(1+f_{ci}) = 0.625 \]

Losses due to elastic shortening should be added to time-dependent losses to
determine the total losses.

Loss due to strand relaxation

\[ \Delta f_{pR} = 2.50 \]  

Loss due to elastic shortening

\[ f_{cgp} = \text{Stress due to prestressing and girder weight at Centroid of prestressing} \]

strands, at section of maximum moment

Concrete stress at Centroid of prestressing

\[ P_i = N A_{ps} .7f_{pu} = 1148.4 \text{ kips} \]  

\[ f_{ps} = -\frac{P_i}{A_g} - \frac{P_i e^2}{I_g} = 2.86 \text{ ksi} \]  

\[ f_g = \frac{M e}{I_g} = 0.78 \text{ ksi} \]  

\[ f_{cgp} = f_g + f_{ps} = -2.08 \text{ ksi} \]  

Elastic shortening loss,

\[ \Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} = 11.14 \text{ ksi} \]  

\[ \Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 30.67 \text{ ksi} \]

Above relaxation losses not added to Time Dependent Losses, but will be used for
Service Limit State Total Transfer PS Losses, Section 10.
10 Stresses at Service Limit State

Stresses after elastic shortening and relaxation:

\[ \text{Force per strand} \quad \frac{P}{N} = A_{ps}(f_{pt}-Df_{pEs}-\Delta f_{pR}) = 40.98 \text{ kips} \]

Table 10-1:

<table>
<thead>
<tr>
<th>Strands</th>
<th>Force per Strand, kips</th>
<th>Total force</th>
<th>Eccent. in.</th>
<th>Moment in-kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Strands</td>
<td>20</td>
<td>40.98</td>
<td>819.66</td>
<td>6.40</td>
</tr>
<tr>
<td>Debonded Strands @ 3'</td>
<td>0</td>
<td>40.98</td>
<td>0.00</td>
<td>7.00</td>
</tr>
<tr>
<td>Debonded Strands @ 6'</td>
<td>2</td>
<td>40.98</td>
<td>81.97</td>
<td>7.00</td>
</tr>
<tr>
<td>Debonded Strands @ 9'</td>
<td>2</td>
<td>40.98</td>
<td>81.97</td>
<td>7.00</td>
</tr>
<tr>
<td>Top Strands</td>
<td>4</td>
<td>40.98</td>
<td>163.93</td>
<td>-6.00</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
\frac{dv}{P (\text{kips})} &= 984 \quad \text{Mp(in-k)} = 4262 \\
3 & \quad \frac{P (\text{kips})}{\text{kips}} = 984 \quad \text{Mp(in-k)} = 4262 \\
6 & \quad \frac{P (\text{kips})}{\text{kips}} = 984 \quad \text{Mp(in-k)} = 4262 \\
9 & \quad \frac{P (\text{kips})}{\text{kips}} = 1066 \quad \text{Mp(in-k)} = 4836 \\
\text{mid} & \quad \frac{P (\text{kips})}{\text{kips}} = 1148 \quad \text{Mp(in-k)} = 5410 \\
\end{align*}
\]

Prestressing stress (using the full length of the girder)

\[ f_{p(top)} = -\frac{P}{A_c} + \frac{M_{ps}}{S_c} = \text{ksi} \]

<table>
<thead>
<tr>
<th></th>
<th>1.40</th>
<th>3.00</th>
<th>6.00</th>
<th>9.00</th>
<th>29.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>dv</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.39</td>
<td>0.51</td>
</tr>
<tr>
<td>mid</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.39</td>
<td>0.51</td>
</tr>
<tr>
<td>3</td>
<td>-3.20</td>
<td>-3.20</td>
<td>-3.20</td>
<td>-3.56</td>
<td>-3.916</td>
</tr>
</tbody>
</table>

Stresses after Losses (noncomposite section)

\[ \text{Force per strand} \quad \frac{P}{N} = A_{ps}(f_{pEs}Df_{pE}^2-\Delta f_{pR}) = 37.29 \text{ kips} \]

BDM 5.1.4-A.1

Table 10-2:

<table>
<thead>
<tr>
<th>Strands</th>
<th>Force per Strand, kips</th>
<th>Total force</th>
<th>Eccent. in.</th>
<th>Moment in-kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Strands</td>
<td>20</td>
<td>37.29</td>
<td>745.74</td>
<td>6.40</td>
</tr>
<tr>
<td>Debonded Strands @ 3'</td>
<td>0</td>
<td>37.29</td>
<td>0.00</td>
<td>7.00</td>
</tr>
<tr>
<td>Debonded Strands @ 6'</td>
<td>2</td>
<td>37.29</td>
<td>74.57</td>
<td>7.00</td>
</tr>
<tr>
<td>Debonded Strands @ 9'</td>
<td>2</td>
<td>37.29</td>
<td>74.57</td>
<td>7.00</td>
</tr>
<tr>
<td>Top Strands</td>
<td>4</td>
<td>37.29</td>
<td>149.15</td>
<td>-6.00</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
\frac{dv}{P (\text{kips})} &= 895 \quad \text{Mp(in-k)} = 3878 \\
3 & \quad \frac{P (\text{kips})}{\text{kips}} = 894.89 \quad \text{Mp(in-k)} = 3878 \\
6 & \quad \frac{P (\text{kips})}{\text{kips}} = 895 \quad \text{Mp(in-k)} = 3878 \\
9 & \quad \frac{P (\text{kips})}{\text{kips}} = 969 \quad \text{Mp(in-k)} = 4400 \\
\text{mid} & \quad \frac{P (\text{kips})}{\text{kips}} = 1044 \quad \text{Mp(in-k)} = 4922 \\
\end{align*}
\]

Prestressing stress after all losses (noncomposite)

\[ f_{p(top)} = -\frac{P}{A_c} + \frac{M_{ps}}{S_c} = \text{ksi} \]

<table>
<thead>
<tr>
<th></th>
<th>0.26</th>
<th>0.26</th>
<th>0.26</th>
<th>0.36</th>
<th>0.46</th>
</tr>
</thead>
<tbody>
<tr>
<td>dv</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
<td>0.36</td>
<td>0.46</td>
</tr>
<tr>
<td>3</td>
<td>-2.91</td>
<td>-2.91</td>
<td>-2.91</td>
<td>-3.24</td>
<td>-3.563</td>
</tr>
<tr>
<td>mid</td>
<td>-2.91</td>
<td>-2.91</td>
<td>-2.91</td>
<td>-3.24</td>
<td>-3.563</td>
</tr>
</tbody>
</table>

WSDOT Bridge Design Manual M 23-50.14

April 2015
### Summary of Stresses at Service Limit States

#### Table 10-3:

<table>
<thead>
<tr>
<th>dv</th>
<th>3'</th>
<th>6'</th>
<th>9'</th>
<th>mid-span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing stress + self wt of girder (using full length of girder)</td>
<td>girder + ps</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( f_{g,topDL+PS} ) (ksi)</td>
<td>0.10</td>
<td>-0.02</td>
<td>-0.28</td>
<td>-0.40</td>
</tr>
<tr>
<td>( f_{g,botDL+PS} ) (ksi)</td>
<td>-3.02</td>
<td>-2.91</td>
<td>-2.64</td>
<td>-2.76</td>
</tr>
</tbody>
</table>

| Construction stress at top & bottom of girder (noncomposite) | gir + ps + deck |
| \( f_{g,topDL+PS} \) | 0.06 | -0.16 | -0.52 | -0.74 | -1.64 |
| \( f_{g,botDL+PS} \) | -2.72 | -2.50 | -2.14 | -2.14 | -1.46 |

| Stresses due to all loads plus prestressing: | gir+ps+deck+barr+(LL+im) |
| \( f_{g,topDL+LL+PS} \) | -0.02 | -0.32 | -0.84 | -1.19 | -2.43 |
| \( f_{g,botDL+LL+PS} \) | -2.56 | -2.16 | -1.50 | -1.25 | 0.13 |

| Stresses due to Transient loads and one-half of permanent loads plus prestressing: | gir+ps+deck+barr+(1/2DL+PS) |
| \( f_{g,topLL+(1/2DL+PS)} \) | -0.05 | -0.24 | -0.56 | -0.78 | -1.56 |
| \( f_{g,botLL+(1/2(DL+PS))} \) | -1.21 | -0.93 | -0.47 | -0.24 | 0.75 |

| Stresses due at service III load combination: | gir+ps+deck+barr+0.8(LL+im) |
| \( f_{g,bot.8*LL+(DL+PS)} \) | -2.58 | -2.22 | -1.61 | -1.40 | -0.15 |

**Tensile stress limit in areas without bonded reinforcement (at dv):**

\[ f_t = 0.0948 \sqrt{f'_{ci}} \leq 0.200 \text{ ksi} > 0.099 \text{ ksi} \]

**Compressive stress limit in pretensioned components:**

\[ f_{ci} = 0.60f'_{ci} = 4.20 \text{ ksi} > -3.02 \text{ ksi} \]

**Compressive stress limit at service - I load combinations**

- Due to permanent loads (DL + PS):
  \[ f_{comp} = 0.45f_c = -3.83 \text{ ksi} \]
- Due to permanent loads and transient loads (DL + PS + LL):
  \[ f_{comp} = 0.60f_c = -5.10 \text{ ksi} \]
- Due to transient loads and one-half of permanent loads (LL + 1/2DL + 1/2PS):
  \[ f_{comp} = 0.40f_c = -3.40 \text{ ksi} \]

**Tensile stress limit at service - III load combination**

\[ f_{tens} = 0.00 \text{ ksi} > -0.146 \text{ ksi} \]

**Stresses at transfer**

The prestressing force may be assumed to vary linearly from zero at free end to a maximum at transfer length, \( l_t \).

\[ l_t = 60 \times d_{strand}/12 = 3.00 \text{ ft} = 36.00 \text{ in.} \]

**Notes:**

- BDM 5.2.3-B
- LRFD 5.5.4.1
11 Strength Limit State

Strength limit state shall be considered to satisfy the requirements for strength and stability.

\[ \eta \sum (\gamma Q_i) < \phi R_u = R_r \]  

Resistance factors

- \( \phi = 0.90 \)  \( \) Flexural in reinforced concrete \( \) LRFD 5.5.4.2.1
- \( \phi = 1.00 \)  \( \) Flexural in prestressed concrete
- \( \phi = 0.90 \)  \( \) Shear
- \( \phi = 0.75 \)  \( \) Axial Compression

Flexural forces

Strength - 1 load combination is to be considered for normal vehicular load without wind.

Load factors:

- \( \gamma_{DC} = 1.25 \)  \( \) Components and attachments (Girder + TB + Deck) \( \) LRFD TABLE 3.4.1-1
- \( \gamma_{DW} = 1.50 \)  \( \) Wearing surface (SIDL or ACP)
- \( \gamma_{LL} = 1.75 \)  \( \) Vehicular load (LL + Impact)

Flexural moment = \( 1.0 \left[ 1.25 \text{ DC} + 1.5 \text{ DW} + 1.75 (\text{LL}+\text{IM}) \right] \)

\[ M_u = 1386.5 \text{ ft.-kips} \]

Checked using QConBridge program,

\[ M_u = 200.0 \text{ ft.-kips} \]  \( \) NG Mu No Check

Flexural resistance

For practical design an equivalent rectangular compressive stress distribution of \( 0.85 f'_c \) overall depth of \( a = b_1 c \) may be considered.

\[ \beta_1 = 0.65 \text{ for } f'_c = 8.5 \text{ ksi} \]

The average stress in prestressing strands, \( f_{ps} \), may be taken as:

\[ f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right) \]  \( \) LRFD 5.7.3.1.1-1

\[ k = 2 \left( 1.04 - \frac{f_{ps}}{f_{pu}} \right) = 0.28 \]  \( \) LRFD 5.7.3.1.1-2

Location of neutral axis of composite transformed section:

For rectangular section without mild reinforcement:

\[ c = \frac{A_{ps} f_{ps} + A_x f_x + A_y f_y}{0.85 f'_c \beta_1 b + kA_{ps} \frac{f_{ps}}{d_p}} \]  \( \) LRFD 5.7.3.1.1-4

- \( A_x = A'_x = 0.00 \text{ in.}^2 \)  \( \) with no partial prestressing considered
- \( A_{ps} = 6.08 \text{ in.}^2 \)  \( \) Area prestressing strands
- \( d_p = 18.71 \text{ in.} \)  \( \) Distance from extreme compression fiber to Centroid of prestressing strands.

\[ c_1 = 9.156 \text{ in} \]
Deck Thickness + Top Flange = 9.50 in  

For T-section without mild reinforcement:

\[
c = \frac{A_{ps} f_{pu} + A_{f} f_{y} - 0.85f_{c} (b - b_{w})h_{f}}{0.85f_{c} \beta_{w} P_{w} + kA_{ps} f_{pu}}
\]

\[
b_{w} = 21.00 \text{ in} \\
h_{f} = 9.50 \text{ in} \\
c = 6.67 \text{ in}
\]

\[
c = 9.156 \text{ in} \\
a = \beta_{1} c = 5.95 \text{ in} < t_{f} = 9.50 \text{ in.}
\]

Average stress in prestressing steel:

\[
f_{ps} = f_{pu} \left(1 - k \frac{c}{d_{p}}\right) = 233.0 \text{ ksi.}
\]

Tensile stress limit at strength limit state, \(f_{pu} = 270.0 \text{ ksi}\)  

**Nominal flexural resistance**

Rectangular:

\[
M_{n} = A_{ps} f_{ps} \left(d_{p} - \frac{a}{2}\right) = 1857 \text{ ft.-kips}
\]

T-shaped:

\[
M_{n} = A_{ps} f_{ps} \left(d_{p} - \frac{a}{2}\right) + 0.85f_{c} (b - b_{w})h_{f} (a / 2 - h_{f} / 2) = 1583 \text{ ft.-kips}
\]

\[
M_{n} = 1857 \text{ ft.-kips}
\]

Flexural resistance, \(M_{r} = \Phi M_{n} = 1857 \times 1386 \text{ ft.-kips}\)

**Minimum reinforcement**

The amount of prestressing and non-prestressing steel shall be adequate to develop flexural resistance greater than or equal to the least 1.2 times the cracking moment or 1.33 times the factored moment required by Strength Limit State 1.

Flexural resistance,

\[
M_{cr} = \Phi M_{n} \geq \text{The Lesser of:} \quad 1.2M_{cr} = 613.4 \text{ ft-kips, governs} \\
1.33M_{u} = 1844.0 \text{ ft-kips}
\]

\[
M_{cr} = S_{c} (f_{c} + f_{pe}) - M_{d} \left(\frac{S_{c}}{S_{b}} - 1\right) S_{c} f_{c} = 222.71 \text{ ft-kips}
\]

\[
f_{pe} = -3.56 \quad \text{Stress at extreme fiber due to prestressing} \quad \text{LRFD 5.4.2.6}
\]

\[
M_{cr} = 511 \text{ ft.-kips}
\]

\[
M_{r} = 1857 > 1.2 M_{cr} = 613.4 \text{ ft.-kips} \quad \text{LRFD 5.7.3.3.2}
\]
Development of prestressing strand

Pretension strand shall be bonded beyond the critical section for a development length taken as:

\[ l_d \geq K \left( f_{ps} - \frac{2}{3} f_{pe} \right) d_b \]

\[ f_{ps} = 233.01 \text{ ksi} \]
\[ f_{pe} = 171.83 \text{ ksi} \]
\[ d_b = 0.60 \text{ in.} \]

\[ l_d \geq 5.92 \text{ ft} \]

\[ l_d = 5.92 \text{ ft} < \frac{1}{2} \text{ Span} \quad \frac{L}{2} = 29.00 \quad \text{OK developed} \]

12 Shear Design

Design procedure

The shear design of prestressed members shall be based on the general procedure of AASHTO - LRFD Bridge Design Specifications article 5.8.3.4.2 using the Modified Compression Field Theory.

Shear design for prestressed girder will follow the (replacement) flow chart for LRFD Figure C.5.8.3.4.2-5. This procedure eliminates the need for q angle and b factor iterations.

Effective Web Width, \( b_v \), and Effective Shear Depth, \( d_v \)

Effective web width shall be taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the compressive and tensile forces due to flexure

\[ b_v = \text{Net Web} = \text{Width} - \text{Voids} \]
\[ b_v = 21.0 \text{ in.} \]

Effective shear depth shall be taken as the distance between resultant of tensile and compressive forces due to flexure but it need not to be taken less than the greater of: 0.9\(d_e\) OR 0.72\(h\).

\[ d_v = d_e - a/2 = 15.7 \text{ in.} \]
\[ d_v = 0.9d_e = 16.8 \text{ in.} \quad \text{governs} \]
\[ d_v = 0.72h = 16.6 \text{ in.} \]

use \[ d_v = 16.8 \text{ in.} \quad \text{or} \quad 1.40 \text{ ft} \]

Component of Prestressing Force in Direction of Shear Force, \( V_p \)

The prestressing in PCPS Slabs are horizontal only, there is no vertical component

\[ V_p = 0.00 \text{ kips} \]
Shear Stress Ratio
Where the Shear Stress (ksi) on the concrete is,

\[ V_u = \frac{V_u - \phi V_p}{\phi b_i d_v} = 0.444 \text{ ksi} \]

LRFD 5.8.2.9-1

Where the

Factored shear force
\[ V_u = S (h_i g_i V_i) \]

LRFD 3.4.1-1

\[ h_i = 1.00 \text{ Limit state factor for any ordinary structure} \]

LRFD TABLES 3.4.1-1 & 3.4.1-2

\[ g_{DC} = 1.25 \text{ Components and attachments (Girder + TB + Deck)} \]

\[ g_{DW} = 1.50 \text{ Wearing surface (SIDL or ACP)} \]

\[ g_{LL+IM} = 1.75 \text{ Vehicular load (LL + Impact)} \]

\[ V_g = 20.0 \text{ kips} \]

\[ V_{tb} = 4.6 \text{ kips} \]

\[ V_{D+L} = 8.1 \text{ kips} \]

\[ V_{DC} = 32.7 \text{ kips} \]

\[ V_{LL+IM} \times DF_{Vext} = 57.4 \text{ kips} \]

Shear force effect,
\[ V_u = 1.00(1.25 V_{DC} + 1.5 V_{DW} + 1.75 V_{LL+IM}) \]

\[ V_u = 141.36 \text{ kips} \]

If the (critical) section (for shear) is within the transfer length of any (prestress) strands, calculate the effective value of \( f_{po} \), the parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked in difference in strain between the prestressing tendons and the surrounding concrete.
\[ f_{po} = 0.70 f_{pu} \]

LRFD 5.8.3.4.2

\[ f_{po} = \left[ \frac{x + d_v}{l_t} \right] 0.70 f_{pu} \text{ governs, } dv \text{ is within the transfer length of the prestressed strands} \]

Where the distance between the edge of girder (or beginning of prestress) and the CL of Bearing (BRG)
\[ x = 5.00 \text{ in.} \]

accounting for bridge skew gives a long. distance from the face of girder as,
\[ x = 5.00 \text{ in.} \]

\[ f_{po} = 114.68 \text{ ksi} \]
Factored Moment

Where: Factored moment is not to be taken less than $V_u d_v$

$$M_u = \sum (\eta_i \gamma_i M_i)$$  \hspace{1cm} LRFD 5.8.3.4.2

Ultimate moment at $d_v$ from support, $M_u$

- Girder, $M_g = 28.8$ ft-kips
- Traffic Barrier, $M_{tb} = 6.6$ ft-kips
- Deck + Legs, $M_{D+L} = 11.7$ ft-kips

$$M_{DC} = 47.0 \text{ ft-kips}$$
$$M_{LL+IM \times DF_{Ext}} = 45.5 \text{ ft-kips}$$

Moment Force Effect,

$$M_u = 1.00(1.25M_{DC} + 1.50M_{DW} + 1.75M_{LL+IM})$$  \hspace{1cm} LRFD TABLE 3.4.1-1

$$M_u = 138.5 \text{ ft-kips} \approx 1661.5 \text{ in.-kips}$$

Check which value governs:

$$V_u d_v = 2381.0 \text{ in.-kips} \hspace{0.5cm} \text{govers}$$
$$M_u = 1661.5 \text{ in.-kips}$$

Longitudinal Strain (Flexural Tension)

The section contains at least the minimum transverse reinforcement as specified in Article 5.8.2.5. Longitudinal strain in the "web reinforcement" on the flexural tension side of the member,

$$\varepsilon_x = \left[ \frac{M_u}{d_v} - 0.5N_u + \left| V_u - V_p \right| \right] \leq 0.002$$

$$2(E_s A_s + E_p A_{ps})$$

Applied Factored Axial forces,

$$N_u = 0.00 \text{ kips}$$

Factored Shear,

$$V_u = 141.36 \text{ kips}$$

Vertical Component of Prestress Forces,

$$V_p = 0.00$$

Area of prestressing steel on the flexural tension side of the member,

$$A_{ps(T)} = N_{tb} \times A_{ps} = 4.34 \text{ in.}^2$$

Prestress/Concrete Modulus of Elasticity Parameter

$$f_{po} = 114.68 \text{ ksi}$$

Modulus of Elasticity of Mild Reinforcement,

$$E_s = 29000 \text{ ksi}$$

Area of Mild Reinforcement in flexural tension side of the member,

$$A_{s(bottom)} = n_s(bottom) A_s$$

Where there are 4 No. 4 bars  \hspace{1cm} BDM fig. 5-A-XX

$$A_{s(bottom)} = 0.80 \text{ in.}^2$$

Modulus of Elasticity of Prestress Strands,

$$E_p = 28500 \text{ ksi}$$

Substitution gives,

$$\varepsilon_x = -0.0007317 \hspace{1cm} < 0, \text{so use the following Equation 3:}$$

LRFD 5.8.3.4.2-1
If the value of $e_x$ from LRFD Equations 5.8.3.4.2-1 or 2 is negative, the strain shall be taken as:

$$e_x = \frac{M_u + 0.5N_u + (V_u - V_p)}{2(E_cA_c + E_sA_s + E_pA_p)}$$

Where: Modulus of Elasticity of Concrete,

$$E_c = 5871.1 \text{ ksi}$$

Area of concrete on the flexural tension side of the member,

$$A_c = 216.00 \text{ in.}^2$$

Substitution gives,

$$e_x = -0.0000760 \quad \text{Equation 3 Governs}$$

$$e_x = -0.0000760$$

**Determination of $\beta$ and $\theta$**

Shear Stress Ratio of: 0.052 Is a value just $\leq$ 0.075

1000 x the Long. Strain: -0.076 Is a value just $\leq$ -0.05

From Table 1:

| $\theta$ | 21.00 deg. |
| $\beta$  | 4.10       |

**Shear strength**

$$V_r = fV_n$$

Nominal shear strength shall be taken as:

$$V_n = V_c + V_s + V_p$$

Shear resistance provided by concrete:

$$V_c = 0.0316\beta\sqrt{f_c' b_v d_v}$$

Shear taken by shear reinforcements:

$$V_s = V_n - V_c - V_p$$

$$f = 0.90 \quad \text{for shear}$$

$$V_n = \text{Nominal shear strength}$$

**Required shear strength**

Nominal shear strength shall be taken as the lesser of:

$$V_n = V_c + V_s + V_p = 221.4 \text{ kips} \quad \text{governs}$$

$$V_n = 0.25f_c' b_v d_v + V_p = 751.6 \text{ kips}$$
Initial Shear from stirrups, based on 12" spacing of #4 bars

\[ V_s = \frac{A_v f_y d_v \cot \theta}{s_{gov}} = 87.75 \text{ kips} \]

LRFD C5.8.3.3-1

Shear resistance provided by concrete:

\[ V_s = 0.0316 \beta b_d d_v = 133.6 \text{ kips} \]

LRFD 5.8.3.3-3

Shear taken by shear reinforcement:

\[ V_{seq} = V_d - V_c - V_p = 23.5 \text{ kips} \]

LRFD 5.8.3.3-1

Spacing of shear reinforcements:

Try 2 legs of \# 4

Av = 0.40 in.²

Required Spacing, \( s_{req'd} = \frac{A_v f_y d_v \cot \theta}{V_s} = 44.87 \text{ in.} \]

LRFD C5.8.3.3-1

**Maximum spacing of shear reinforcement**

- if \( v_u < 0.125 f'c \) then \( s_{max} = 0.8 \text{ dv} < 24 \text{ in.} \)
- LRD5 5.8.2.7-1

Maximum spacing of shear reinforcement, WSDOT Practice = 18.00 in

if \( v_u \geq 0.125 f'c \) then \( s < 0.4 \text{ dv} < 12 \text{ in.} \)

\[ v_u = 0.444 \text{ ksi} \]

\[ 0.125 f'c = 1.063 \text{ ksi} > v_u = 0.444 \text{ ksi} \]

Maximum spacing, \( s_{max} = 13.5 \text{ in.} \)

Governs spacing

Governing spacing, \( s_{gov} = 13.0 \text{ in.} \)

\[ A_v(\text{provided}) = 0.40 \text{ in.²} \]

Assuming two #4 legs

\[ V_s = \frac{A_v f_y d_v \cot \theta}{s_{gov}} = 81.00 \text{ kips} \]

LRFD C5.8.3.3-1

Shear reinforcement is required if:

\[ 0.5 f(V_c + V_p) < V_u \]

LRFD 5.8.2.4-1

\[ 0.5(V_c + V_p) = 60.1 < V_u = 141.4 \text{ kips} \]

Yes, Shear/Transverse Reinf. Is Required

**Minimum shear reinforcement**

When shear reinforcement is required by design, the area of steel provided,

\[ A_v(\text{provided}) \geq 0.0316 \sqrt{\frac{f'_c b_d s_{gov}}{f_y}} \]

Use Spacing: 12.00 in \( \leq 13.0 \text{ in.} \)

LRFD 5.8.2.5-1

where:

- \( s = 12.0 \text{ in.} \)

Required Area of Steel,

\[ 0.0316 \sqrt{\frac{f'_c b_d s_{gov}}{f_y}} = 0.39 \text{ in.²} \]

0.40 > 0.39 in.²

OK for Min. Transverse Reinf.
Longitudinal reinforcement

Longitudinal reinforcement shall be provided so that at each section the following equations are satisfied:

\[ A_s f_y + A_{ps} f_{ps} \left( \frac{d_v}{l_t} \right) \geq T = \frac{M_u}{d_v \phi} + \frac{0.5N_u}{\phi} + \left( \frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot \theta \]

- \( A_s = 0.80 \text{ in}^2 \)
- \( f_y = 60.00 \text{ ksi} \)
- \( A_{ps} = 4.34 \text{ in}^2 \)
- \( f_{ps} = 233.01 \text{ ksi} \)
- \( d_v = 16.84 \text{ in.} \)
- \( l_t = 3.00 \text{ ft} = 36.00 \text{ in} \)
- \( M_u = 198.4 \text{ ft-kips} \)
- \( f = 1.00 \text{ Flexural in prestressed concrete} \)
- \( f = 0.90 \text{ Shear} \)
- \( f = 0.75 \text{ Axial Compression} \)
- \( N_u = 0.00 \)
- \( V_u = 141.36 \text{ kips} \)
- \( V_s = 81.00 \text{ kips} \)
- \( V_p = 0.00 \text{ kips} \)
- \( q = 21.00 \text{ degree} \)

by substitution:

\[ 521.135 \geq 445.0 \text{ kips} \]

**OK for Longitudinal Reinforcement**

13 Deflection and Camber

Let downward Deflection be Positive +
Let upward Deflection, Camber, be Negative -

Deflection due to prestressing forces at Transfer

Deflection due to bottom strands is computed from a combination of fully bonded strands and the partially bonded or "debonded" strands which are sleeved at the ends of the girder. Each type has their own eccentricity.

\[ \Delta p_{ps} = \Delta p_{bb} + \Delta p_{db} \]

\[ \Delta p_{bb} + \Delta p_{db} = \left( P_{bb} e_{bb} + k_{db} P_{db} e_{db} \right) \frac{L^2}{8E_{ci} I_c} \]

Force and eccentricity due to the bonded bottom prestress strands are:

\( P_{bb} = 819.66 \text{ kips} \)
\( e_{bb} = 6.40 \text{ in} \)
Reduction factor for the partially bonded or debonded strands

\[ k_{db} = \frac{L - 2l_{db}}{L} = 0.847 \]

The average sleeved length of the debonded strands,

\[ l_{db} = 4.5 \text{ ft} = 54.0 \text{ in} \]

Force and eccentricity due to the debonded bottom prestress strands are:

\[ P_{db} = 0.00 \text{ kips} \]
\[ e_{db} = 7.00 \text{ in} \]
\[ D_{p_{shot}} = -2.786 \text{ in. upward} \]

Deflection due to top strands is computed from:

\[ \Delta_{ps_{top}} = \frac{P_te_tL_t^2}{8E_{ci}I_c} \]

Prestressing force and eccentricity of top strands.

\[ P_t = 163.9 \text{ kips} \]
\[ e_t = -6.00 \text{ in.} \]
\[ \Delta_{ps_{top}} = 0.522 \text{ in. downward} \]

Total deflection due to prestressing:

\[ \Sigma \Delta_{ps} = -2.79 + 0.52 = -2.263 \text{ in. upward} \]

**Deflection due to weight of Girder**

\[ \Delta_g = \frac{5w_g L^4}{384 E_{ci} I_c} = 1.66 \text{ in. downward} \]

(AISC LRFD page 4-190)

**Deflection due to weight of Traffic Barrier TB**

\[ \Delta_{tb} = \frac{5w_{tb(3G)}}{384 E_{ci} I_c} = 0.18 \text{ in. downward} \]

**Deflection due to weight of Deck and Legs**

\[ \Delta_{SIDL} = \frac{5w_{SIDL} L^4}{384 E_{ci} I_c} = 0.58 \text{ in. downward} \]

**Deflection (Camber) at transfer, C_i**

Deflection accounted at transfer are due to prestressing and weight of girder:

At transfer: \[ \Delta_i = -2.26 + 1.66 = -0.60 \text{ in} \]

**Creep Coefficients**

\[ C_F = -[(\Delta_{ps} + \Delta_g)(\psi_{(l,ii)} + 1)] \]

Creep Coefficient:
\[ \Psi_{(t,0)} = 1.9 \times k_v k_h k_t k_{td} t^{0.118} \]

**LRFD 5.4.2.3.2-1**

\[ k_v = 1.45 - 0.13(v/s) \geq 1.0 \]

**LRFD 5.4.2.3.2-2**

\[ k_h = 1.56 - 0.008H \quad H = 80.00 \] average humidity (AASHTO fig. 5.4.2.3.3-1)

**LRFD 5.4.2.3.2-3**

\[ k_t = \frac{5}{(1+f'_c)} \]

**LRFD 5.4.2.3.2-4**

\[ k_{td} = \frac{t}{(61-4f'_c+t)} \]

**LRFD 5.4.2.3.2-5**

\[ \frac{v}{s} = 4.03 \text{ in} \quad \text{Void end from end of girder} = 15 \text{ in.} \]

**LRFD 5.4.2.3.2**

<table>
<thead>
<tr>
<th>Table 13-1:</th>
<th>( t )</th>
<th>( t_i )</th>
<th>( k_v )</th>
<th>( k_h )</th>
<th>( k_t )</th>
<th>( k_{td} )</th>
<th>( \Psi_{(t,0)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Psi_{(7,30)} )</td>
<td>7.00</td>
<td>30.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.48</td>
<td>0.41</td>
</tr>
<tr>
<td>( \Psi_{(30,40)} )</td>
<td>30.00</td>
<td>40.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.55</td>
<td>0.40</td>
</tr>
<tr>
<td>( \Psi_{(7,40)} )</td>
<td>7.00</td>
<td>40.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.55</td>
<td>0.48</td>
</tr>
<tr>
<td>( \Psi_{(7,90)} )</td>
<td>7.00</td>
<td>90.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.73</td>
<td>0.64</td>
</tr>
<tr>
<td>( \Psi_{(90,120)} )</td>
<td>90.00</td>
<td>120.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.78</td>
<td>0.50</td>
</tr>
<tr>
<td>( \Psi_{(7,120)} )</td>
<td>7.00</td>
<td>120.00</td>
<td>1.00</td>
<td>0.92</td>
<td>0.63</td>
<td>0.78</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Assume 1 Day of accelerated cure by radiant heat or steam. 1 Day accelerated cure = 7 normal Days of cure. Age of Concrete when load is initially applied, 

**Final Deflections Due to All Loads and Creep**

"D" Parameters for Minimum Timing

\[ \Delta c_{\text{cr.min}} = \Psi(7,40)(\Delta p_{\text{bot}}+\Delta p_{\text{stop}}+\Delta g) = -0.28 \text{ in} \]

\[ \Delta c_{\text{cr.min}} = \Delta p_{\text{bot}}+\Delta p_{\text{stop}}+\Delta g = -0.88 \text{ in} \]

"D" Parameters for Maximum Timing

\[ \Delta c_{\text{cr.max}} = \Psi(7,120)(\Delta p_{\text{bot}}+\Delta p_{\text{stop}}+\Delta g) = -0.41 \text{ in} \]

\[ \Delta c_{\text{cr.max}} = \Delta p_{\text{bot}}+\Delta p_{\text{stop}}+\Delta g = -1.01 \text{ in} \]

Elastic deflection due to slab and Traffic barrier

\[ C = \Delta s_{\text{slab}}+\Delta t = 0.76 \text{ in} \]
Excess girder camber

\[ \Delta_{\text{excess}} = D40 + C = -0.13 \text{ in} \]

\[ \Delta_{\text{excess}} = D120 + C = -0.25 \text{ in} \]

Time period to display (40, 120) = 40.00

Deck thickness at Piers = 5.13 in.

Fig. 13-1 Time Vs. Deflection Curve
Design Specifications:


Strand for Positive EQ Moment:

For girders made continuous for live load, extended bottom prestress strands are used to carry positive EQ load, creep, and other restrained moments from one span to another.

Strands used for this purpose must be developed in the short distance between the two girder ends. The strand end anchorage device used, per WSDOT Standard Plan, is a 2'-0" strand extension with strand chuck and steel anchor plate.

The number of strands to be extended cannot exceed the number of straight strands available in the girder and shall not be less than four.

The design moment at the center of gravity of superstructure is calculated using the following:

\[
M_{po}^{CG} = M_{po}^{top} + \left( M_{po}^{top} + M_{po}^{Base} \right) \frac{h}{L_c}
\]

Where:

- \( M_{po}^{top} \) = plastic overstrength moment at top of column, kip-ft.
- \( M_{po}^{Base} \) = plastic overstrength moment at base of column, kip-ft.
- \( h \) = distance from top of column to c.g. of superstructure, ft.
- \( L_c \) = column clear height used to determine overstrength shear associated with the overstrength moments, ft.

This moment is resisted by the bent cap through torsion forces. The torsion in the bent cap is distributed into the superstructure based on the relative flexibility of the superstructure and the bent cap.

Hence, the superstructure does not resist column overstrength moments uniformly across the width. To account for this, an effective width approximation is used, where the maximum resistance per unit of superstructure width of the actual structure is distributed over an equivalent effective width to provide an equivalent resistance.
It has been suggested that for concrete bridges, with the exception of box girders and solid superstructure, this effective width should be calculated as follows:

\[ B_{\text{eff}} = D_c + D_s \]

Where:

- \( D_c \) = diameter of column
- \( D_s \) = depth of superstructure including cap beam

Based on the structural testing conducted at the University of California at San Diego La Jolla, California in the late 1990's (Holombo 2000), roughly two-thirds of the column plastic moment to be resisted by the two girders adjacent to the column (encompassed by the effective width) and the other one-third to be resisted by the non-adjacent girders.

Based on the effective width, the moment per girder line is calculated as follows:

**adjacent girders (encompassed by the effective width):**

- \( M_{\text{int}}^\text{sei} = \frac{2M_{\text{po}}^\text{CG}}{3N_{\text{int}}^g} \)
- \( M_{\text{ext}}^\text{sei} = \frac{M_{\text{po}}^\text{CG}}{3N_{\text{ext}}^g} \)

**Seismic Moment:**

If \( M_{\text{int}}^\text{sei} \geq M_{\text{ext}}^\text{sei} \), then \( M_{\text{sei}} = M_{\text{int}}^\text{sei} \)

If \( M_{\text{int}}^\text{sei} < M_{\text{ext}}^\text{sei} \), then \( M_{\text{sei}} = \frac{M_{\text{po}}^\text{CG}}{N_{\text{int}}^g + N_{\text{ext}}^g} \)

where:

- \( N_{\text{int}}^g \) = Number of girder encompassed by the effective width.
- \( N_{\text{ext}}^g \) = Number of girder outside the effective width.

Number of extended straight strands needed to develop the required moment capacity at the end of girder is based on the yield strength of the strands.

\[ N_{ps} = 12[M_{\text{sei}}^\text{CG} \cdot K - M_{\text{SIDL}}] \cdot \frac{1}{0.9\phi f_{py}p_d} \]

Where:

- \( A_{ps} \) = area of each extended strand, in^2
- \( f_{py} \) = yield strength of prestressing steel specified in LRFD Table 5.4.1-1
- \( d \) = distance from top of slab to c.g. of extended strands, in.
- \( M_{\text{SIDL}} \) = moment due to SIDL (traffic barrier, sidewalk, etc.) per girder
- \( K \) = span moment distribution factor use maximum of \( K_1 \) and \( K_2 \)
- \( \phi \) = flexural resistance factor
Assume EI is constant and Girders have fixed-fixed supports for both spans.

\[
K_1 = \frac{L_1}{L_1 + L_2} \quad K_2 = \frac{L_2}{L_1 + L_2}
\]

References:

Given:
- \(D_c\) = 5.00 ft. diameter of column
- \(D_s\) = 12.93 ft. depth of superstructure including cap beam
- \(B_{\text{eff}}\) = 5 + 12.93 = 17.93 ft.
- \(f_{c}'\) = 4.00 ksi, specified compressive strength of deck concrete, Class 4000D.
- \(d_b\) = 0.6" nominal strand diameter =0.217 in^2
- \(f_{pu}\) = 270 ksi specified tensile strength of prestressing strands. LRFD Table 5.4.4.1-1
- \(f_{pys}\) = 243 ksi ksi for low relaxation strand
- \(\phi\) = 1.00 resistance factor (LRFD C 1.3.2.1, for extreme event limit state)
- \(N_{s''}^\text{ext}\) = 3 number of girders encompassed by the effective width
- \(N_{s'}^\text{ext}\) = 2 number of prestressed girders in the pier
- \(G\) = W83G H = 82.625" girder depth
- \(A\) = 9.50 " "A" Dimension including 1/2" Integral W.S.
- \(t_s\) = 7.50 " effective slab thickness (not including 1/2" Integral W.S.)
- \(Y_{\text{slab}}\) = 36.86 " c.g. of superstructure to top of slab (see PGSuper output)
- \(b\) = 81.00 " effective flange width (PGSuper Output & LRFD 4.6.2.6.1)
- \(h\) = 116.64 " distance from top of column to c.g. of superstructure
- \(L_1\) = 176.63 ft. Span length of span 1. Factor = 1.33
- \(L_2\) = 180.00 ft. Span length of span 2. Factor = 1.00

<table>
<thead>
<tr>
<th>Far End Condition</th>
<th>Pin</th>
<th>Fixed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor</td>
<td>1.33</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Design Steps:

Step 1:

Calculate the design moment at the center of gravity of superstructure

\[ M_{CG}^{po} = 16000 + (16000 + 16500) / 25 \times 116.64 / 12 = 28636 \text{ kip-ft} \]

Step 2:

Calculate the design moment per girder.

\[ M_{int}^{po} = 2/3 \times 28636 / 3 = 6363.56 \text{ kip-ft} \]
\[ M_{ext}^{po} = 1/3 \times 28636 / 2 = 4772.67 \text{ kip-ft} \]
\[ M_{avg}^{po} = 28636 / (3 + 2) = 5727.2 \text{ kip-ft} = 6363.56 \text{ kip-ft}. \]
\[ L_1 = 234.92 \text{ ft. (Modified)} \quad K_1 = 180 / (234.92 + 180) = 0.434 \]
\[ L_2 = 180.00 \text{ ft. (Modified)} \quad K_2 = 234.92 / (234.92 + 180) = 0.566 \]
\[ K = 0.566 \]

Design Moment per girder

\[ M_{des} = 0.566 \times 6363.56 - 0.9 \times 517 = 3137.61 \text{ ft-kips per girder} \]

Step 3:

Calculate the number of extended strand required

\[ cs = 3.00" \text{ c.g. of extended strands to bottom of girder} \]
\[ d = 9.5 - 0.5 + 82.625 - 3 = 88.625" \]

assume \( f_{py} = 243 \text{ ksi} \)

Number of extended strand required =

\[ 12 \times 3137.61 / (0.9 \times 1 \times 0.217 \times 243 \times 88.625) = 9 \text{ strands} \]

Use = \( N_{ps} \) 10 extend strands - Use even number of strands
Step 4:

Check moment capacity of extended strands

\( c_s = 3.00'' \) c.g. of extended strands to bottom of girder

Per LRFD 5.7.3.2 The factored flexural resistance

\[ M_r = \phi M_n \]
\[ M_n = A_{ps} f_{py} \left( d_p - \frac{a}{2} \right) \]

where:

\( A_{ps} = \) area of prestressing steel, in\(^2\) = \( 10 \times 0.217 = 2.17 \text{ in}^2 \)
\( d_p = \) distance from extreme compression fiber to the centroid of prestressing tendons (in.)
\( d_p = 9.5 - 0.5 + 82.625 - 3 = 88.625'' \)

Assume rectangular behavior:

\[ c = \frac{A_{ps} f_{py}}{0.85 f_{ce} \beta_1 b} \]
\[ \beta_1 = 0.85 \text{ for } f_{ce} \leq 4000 \text{ psi} \]
\[ \beta_1 = 0.85 - 0.05 \frac{f_{ce}}{4000} \geq 0.65 \text{ for } f_{ce} > 4000 \text{ psi} \]

\( f_c = 4.00 \text{ ksi} = 1.3 \times 4 = 5.2 \text{ ksi} = 0.79 \)
\( c = 2.17 \times 243 / 0.85 \times 5.2 \times 0.79 \times 81 = 1.864'' \)
\( a = 0.79 \times 1.864 = 1.473'' \) depth of the equivalent stress block (in.)
\( M_n = 2.17 \times 243 \times (88.625 - 1.473/2) / 12 = 3862.04 \text{ kip-ft.} \)
\( M_r = 1 \times 3862.04 = 3862.04 \text{ kip-ft.} > = 3137.61 \text{ ft-kips OK} \)
Problem Description: A wingwall with traffic barrier is to be checked for moment capacity at a vertical section at the abutment for a vehicular impact.

AASHTO LRFD Specifications Extreme Event-II Limit State (Test Level TL-4)

- **W** := 15ft
  - Wingwall Length
- **h** := 2.5ft
  - Height of wingwall at end away from pier.
- **S** := 2ft
  - Traffic surcharge (given in height of soil above road). See LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.
- **GroundSlope** := 2
  - to 1
- **W** := 45 - lbf ft² ft⁻²
  - Lateral Earth Pressure (equivalent fluid pressure per foot)
- **F** := 54kip
  - Transverse Collision Load § Table A13.2-1 LRFD AASHTO
- **Lt** := 3.5ft
  - Collision Dist. Width § Table A13.2-1 LRFD AASHTO
- **γCT** := 1
  - Collision Load Factor § Table 3.4.1-1 LRFD AASHTO
- **γEH** := 1.35
  - Horizontal Earth Load Factor § Table 3.4.1-2 LRFD AASHTO
- **γLS** := 0.5
  - Live Load Surcharge Load Factor § Table 3.4.1-2 LRFD AASHTO
  - for Extreme Event II

**Transverse Collision Force Moment Arm**

\[ \text{MomentArm} := L - \frac{L_t}{2} \]

\[ \text{MomentArm} = 13.25 \text{ ft} \]

**Wall Height at Abutment**

\[ H := h + \left( \frac{L}{\text{GroundSlope}} \right) \]

\[ H = 10.00 \text{ ft} \]

**Flexural Moment due to Collision Load and Earth Pressure**

\[
\text{FlexuralMoment} := \gamma_{CT} \cdot F_t \cdot \text{MomentArm} + \gamma_{EH} \cdot \frac{W \cdot L^2}{24} \left[ 3 \cdot h^2 + \left( H + 4 \cdot S \cdot \frac{\gamma_{LS}}{\gamma_{EH}} \right) \left( H + 2 \cdot h \right) \right]
\]

\[ \text{FlexuralMoment} = 836.92 \text{ kip-ft} \]

\[
M_u := \frac{\text{FlexuralMoment}}{H} = 83.69 \frac{\text{kip-ft}}{\text{ft}}
\]
Problem Description: A wingwall with traffic barrier is to be checked for moment capacity at a vertical section at the abutment for a vehicular impact.

AASHTO LRFD Specifications Extreme Event-II Limit State (Test Level TL-4)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>L (_w)</td>
<td>15ft</td>
<td>Wingwall Length</td>
</tr>
<tr>
<td>h</td>
<td>2.5ft</td>
<td>Height of wingwall at end away from pier.</td>
</tr>
<tr>
<td>S</td>
<td>2ft</td>
<td>Traffic surcharge (given in height of soil above road). See LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.</td>
</tr>
<tr>
<td>GroundSlope</td>
<td>:= 2 (\frac{1}{1})</td>
<td></td>
</tr>
<tr>
<td>W</td>
<td>(\frac{45 \text{lbf}}{ft^2\cdot\text{ft}})</td>
<td>Lateral Earth Pressure (equivalent fluid pressure per foot)</td>
</tr>
<tr>
<td>(F_t)</td>
<td>:= 54kip</td>
<td>Transverse Collision Load</td>
</tr>
<tr>
<td>L (_t)</td>
<td>:= 3.5ft</td>
<td>Collision Dist. Width</td>
</tr>
<tr>
<td>(\gamma_{CT})</td>
<td>:= 1</td>
<td>Collision Load Factor</td>
</tr>
<tr>
<td>(\gamma_{EH})</td>
<td>:= 1.35</td>
<td>Horizontal Earth Load Factor</td>
</tr>
<tr>
<td>(\gamma_{LS})</td>
<td>:= 0.5</td>
<td>Live Load Surcharge Load Factor for Extreme Event II</td>
</tr>
</tbody>
</table>

Transverse Collision Force Moment Arm

\[
\text{MomentArm} := L - \frac{L_t}{2}
\]

MomentArm = 13.25 ft

Wall Height at Abutment

\[
H := h + \left(\frac{L}{\text{GroundSlope}}\right)
\]

H = 10.00 ft

Flexural Moment due to Collision Load and Earth Pressure

\[
\text{FlexuralMoment} := \gamma_{CT} \cdot F_t \cdot \text{MomentArm} + \gamma_{EH} \cdot \frac{W \cdot L^2}{24} \left[3 \cdot h^2 + \left(H + 4 \cdot S \cdot \frac{\gamma_{LS}}{\gamma_{EH}}\right)(H + 2 \cdot h)\right]
\]

FlexuralMoment = 836.92 kip-ft

\[
M_u := \frac{\text{FlexuralMoment}}{H}
\]

\(M_u = 83.69 \text{kip-ft/ft}\)
Define Units

\[ \text{ksi} \equiv 1000\cdot\text{psi} \quad \text{kip} \equiv 1000\cdot\text{lbf} \quad \text{kcf} \equiv \text{kip}\cdot\text{ft}^{-3} \quad \text{klf} \equiv 1000\cdot\text{lbf}\cdot\text{ft}^{-1} \]

\[ \text{MPa} \equiv \text{Pa}\cdot10^{6} \quad \text{N} \equiv 1\cdot\text{newton} \quad \text{kN} \equiv 1000\cdot\text{N} \]
Find the flexural strength of a W83G girder made composite with a 7.50 in. thick cast-in-place deck, of which the top 0.50 in. is considered to be a sacrificial wearing surface. The girder spacing is 6.0 ft. To simplify the calculations, ignore the contribution of any non-prestressed reinforcing steel and the girder top flange. The girder configuration is shown in Figure 1 with 70-0.6 in. diameter strands, and concrete strengths of 6000 psi in the deck and 15000 psi in the girder.

![Figure 1](image)

**Bare W83G Bridge Girder Data**

- Depth of girder: $h = 82.68$ in.
- Width of girder web: $b_w = 6.10$ in.
- Area of prestressing steel: $A_{ps} = 15.19$ in.$^2$
- Specified tensile strength of prestressing steel: $f_{pu} = 270.00$ ksi
- Initial jacking stress: $f_{pj} = 202.50$ ksi
- Effective prestress after all losses: $f_{pe} = 148.00$ ksi
- Modulus of Elasticity of prestressing steel: $E_p = 28,600$ ksi
Design concrete strength  
\[ f'_c = 15000 \text{ psi} \]

### Composite W83G Bridge Girder Data

Overall composite section depth  
\[ H = 89.68 \text{ in.} \]

Deck slab width  
\[ b = 72.00 \text{ in.} \]

Deck slab thickness  
\[ t = 7.50 \text{ in.} \]

Structural deck slab thickness  
\[ h_f = 7.00 \text{ in.} \]

Depth to centroid of prestressing steel  
\[ d_p = 85.45 \text{ in.} \]

Design concrete strength  
\[ f'_c = 6000 \text{ psi} \]

\[
\varepsilon_{ps} = 0.003 \left( \frac{d_p}{c} - 1 \right) + \left( \frac{f_{pe}}{E_p} \right) = 0.003 \left( \frac{85.45}{30.75} - 1 \right) + \left( \frac{148.00}{28,600} \right)
\]

\[
f_{si} = \varepsilon_{ps} \left[ 887 + \frac{27,613}{\left( 1 + (112.4 \varepsilon_{ps})^{7.36} \right)^{7.36}} \right] \leq \sum A_{si} F_{si} = A_{ps} f_{si} = (15.19)(246.56) \quad a = \beta_{1(ave)} c = (0.719)(30.75)
\]

\[
\beta_{1(ave)} = \sum \left( f'_c A_c \beta_1 \right)_j / \sum \left( f'_c A_c \right)_j = \frac{\left(6(7)(72)(0.75) + (15)(22.1 - 7)(6.10)(0.65)\right)}{\left(6(7)(72) + (15)(22.1 - 7)(6.10)\right)}
\]

\[
\sum F_{cj} = 0.85 f'_c (a - h_f) b_w = 0.85(6)(7)(72) + 0.85(15)(22.10 - 7)(6.10)
\]

\[
M_n = 0.85 f'_c (a - h_f) b_w \left( d_p - \frac{h_f}{2} \right) + 0.85 f'_c (a - h_f) b_w \left( d_p - h_f - \frac{a - h_f}{2} \right)
\]

\[
= 0.85(6)(7)(72) \left( 85.45 - \frac{7}{2} \right) + 0.85(15)(22.1 - 7)(6.10) \left( 85.45 - 7 - \frac{22.1 - 7}{2} \right)
\]

\[
d_t = H - 2 = 89.68 - 2 \phi = 0.5 + 0.3 \left( \frac{d_t}{c} - 1 \right) = 0.5 + 0.3 \left( \frac{87.68}{30.75} - 1 \right)
\]
\[
\phi M_n = 1.00(293.931) \varepsilon_{ps} = 0.003 \left( \frac{d_p}{c} - 1 \right) + \left( \frac{f_{pe}}{E_p} \right) = 0.003 \left( \frac{85.45}{32.87} - 1 \right) + \left( \frac{148.00}{28.600} \right)
\]

\[
f_{si} = \varepsilon_{ps} \left[ 887 + \frac{27,613}{\left(1 + (112.4\varepsilon_{ps})^{0.36} \right)^{0.36}} \right] \leq
\]

\[
= \left( 0.009974 \right) \left[ 887 + \frac{27,613}{\left(1 + (112.4(0.009974))^{0.36} \right)^{0.36}} \right]
\]

\[
\sum A_{si} F_{si} = A_{ps} f_{si} = (15.19)(242.83) a = \beta c = (0.65)(32.87)
\]

\[
\sum F_{cj} = 0.85 f'_{c(deck)} h_f b + 0.85 f'_{c(girder)} (a - h_f) b_w
\]

\[
= 0.85(6)(7)(72) + 0.85(15)(21.37 - 7)(6.10)
\]

\[
M_n = 0.85 f'_{c(deck)} h_f b \left( d_p - \frac{h_f}{2} \right) + 0.85 f'_{c(girder)} (a - h_f) b_w \left( d_p - h_f - \frac{(a - h_f)}{2} \right)
\]

\[
= 0.85(6)(7)(72) \left( 85.45 - \frac{7}{2} \right) + 0.85(15)(21.37 - 7)(6.10) \left( 85.45 - 7 - \frac{(21.37 - 7)}{2} \right)
\]

\[
d_t = H - 2 = 89.68 - 2 \phi = 0.5 + 0.3 \left( \frac{d_t}{c} - 1 \right) = 0.5 + 0.3 \left( \frac{87.68}{32.87} - 1 \right)
\]

\[
\phi M_n = 1.00(290.323)
\]

**Flexural Strength – Strain Compatibility with Non-Linear Concrete Stress Block**

The concrete stress-strain curves for both the deck and girder concrete are taken from Collins and Mitchell (see BDM 5.1.1). The “power formula” of the PCI BDM (see BDM 5.1.3) is used to determine the stress in the prestressing steel for each iteration.

The concrete compression block is divided into (100) slices, (21) equal slices in the flange and (79) equal slices in the web for this case. The strain at the center of each slice was used to determine the average stress within that slice, which was multiplied by the area of the slice to determine the force in each slice.

The product of these forces and the distance to the center of each force from the top of the deck was used to calculate the resultant forces and eccentricities in the flange and
web. Example calculations for the stresses in the slice at the top of the deck, at the interface between the deck and girder, and the prestressing steel are as follows:

For the deck concrete,

\[ E_c = \frac{\left(40,000 \sqrt{f_{c}' + 1,000,000}\right)}{1000} = \frac{\left(40,000 \sqrt{6000 + 1,000,000}\right)}{1000} \]

\[ = 4098 \text{ ksi} \]

\[ n = 0.8 + \frac{f_{c}'}{2500} = 0.8 + \frac{6000}{2500} = 3.20 \]

\[ k = 0.67 + \frac{f_{c}'}{9000} = 0.67 + \frac{6000}{9000} = 1.337 \]

\[ \varepsilon_{c}' \times 1000 = \frac{f_{c}'}{E_c} \frac{n}{n-1} = \frac{6000}{4098} \frac{3.2}{3.2-1} = 2.129 \]

For the top slice of deck,

\[ y = \frac{7}{21(2)} = 0.167 \text{ in.} \]

\[ \varepsilon_{cf} = \frac{0.003}{c} (c - y) = \frac{0.003}{34.42} (34.42 - 0.167) = 0.002985 \]

\[ f_{c} = \left(\frac{f_{c}'}{n-1}\right) = \frac{n(\varepsilon_{cf} / \varepsilon_{c}')}{(\varepsilon_{cf} / \varepsilon_{c}')^{3.2}} \]

\[ = 4.18 \text{ ksi (28.8 MPa)} \]

The contribution of this slice to the overall resultant compressive force is

\[ C_1 = (4.18kksi)(72in)\left(\frac{7}{21}\text{ in}\right) = 100.32\text{kip} \]

For bottom slice of deck,

\[ y = \frac{7}{21(20)} + \frac{7}{21(2)} = 6.833 \text{ in.} \]

\[ \varepsilon_{cf} = \frac{0.003}{c} (c - y) = \frac{0.003}{34.42} (34.42 - 6.833) = 0.002404 \]

\[ f_{c} = \left(\frac{f_{c}'}{n-1}\right) = \frac{n(\varepsilon_{cf} / \varepsilon_{c}')}{(\varepsilon_{cf} / \varepsilon_{c}')^{3.2}} \]

\[ = 5.59 \text{ ksi} \]
The contribution of this slice to the overall resultant compressive force is

\[ C_{21} = (5.59 \text{ksi})(72\text{in}) \left( \frac{7}{21} \text{ in} \right) = 134.16 \text{kip} \]

For girder concrete,

\[ E_c = \left( \frac{40,000 \sqrt{f'_c} + 1,000,000}{1000} \right) = \left( \frac{40,000 \sqrt{15000} + 1,000,000}{1000} \right) \]

\[ = 5899 \text{ ksi (40674 MPa)} \]

\[ n = 0.8 + \frac{f'_c}{2500} = 0.8 + \frac{15000}{2500} = 6.80 \]

\[ k = 0.67 + \frac{f'_c}{9000} = 0.67 + \frac{15000}{9000} = 2.337 \]

\[ \varepsilon'_c \times 1000 = \frac{f'_c}{E_c} \cdot \frac{n}{n-1} = \frac{15000}{5899} \cdot \frac{6.8}{6.8-1} = 2.981 \]

For the top slice of girder,

\[ y = 7 + \frac{27.42}{79(2)} = 7.174 \text{ in.} \]

\[ \varepsilon_{cf} = \frac{0.003}{c} (c - y) = \frac{0.003}{34.42} (34.42 - 7.174) = 0.002375 \]

Since \( \varepsilon_{cf} / \varepsilon'_c = 0.002375 / 0.002981 = 0.797 < 1.0, \) \( k = 1.0 \)

\[ f_c = \left( f'_c \right) \frac{n \left( \varepsilon_{cf} / \varepsilon'_c \right)}{n-1 + \left( \varepsilon_{cf} / \varepsilon'_c \right)^n k} = (15) \frac{6.8(0.002375 / 0.002981)}{6.8 - 1 + (0.002375 / 0.002981)^{6.8(1.0)}} \]

\[ = 13.51 \text{ ksi} \]

The contribution of this slice to the overall resultant compressive force is

\[ C_{22} = (13.51 \text{ksi})(6.10\text{in}) \left( \frac{34.42 - 7.0}{79} \text{ in} \right) = 28.60 \text{kip} \]

For the prestressing steel:

\[ \varepsilon_{ps} = 0.003 \left( \frac{d_p}{c} - 1 \right) + \left( \frac{f_{pe}}{E_p} \right) = 0.003 \left( \frac{85.45}{34.42} - 1 \right) + \left( \frac{148}{28500} \right) = 0.00964 \]
The resultant force in the prestressing steel is
\[ T = (239.93 ksi)(15.19 in^2) = 3644.6 kip \]

The overall depth to the neutral axis, c, was varied until the sum of the compressive force in all the concrete slices equaled the tension force in the prestressing steel.

Equilibrium was achieved at a compressive force in the slab of 2473 kip, 3.68” below the top of slab and a compressive force in the girder of 1169 kip, 16.20” below the top of slab.

Summing moments about the centroid of the prestressing steel,
\[ M_n = 2473(85.45 – 3.68) + 1169(85.45 – 7 – 9.20) = 283,170 \text{ kip-in}. \]

To calculate \( \phi \),

Assume the lowest row of prestressing strands is located 2” from the bottom of the girder. The depth to the extreme strands is
\[ d_t = H - 2 = 89.68 - 2 = 87.68 \text{ in}. \]
\[ \phi = 0.5 + 0.3 \left( \frac{d_t}{c} - 1 \right) = 0.5 + 0.3 \left( \frac{87.68}{34.42} - 1 \right) = 0.96 \]
\[ \phi M_n = 0.96(283,170) = 273,034 \text{ kip-in}. \]

**Effects of Refinement – Strain Compatibility Analysis**

A significant amount of additional capacity may be realized for this member by including the top flange of the W83G girder. The top flange is 49” wide and approximately 6” deep. The large area and high strength of the top flange provide a considerable compression contribution to the capacity analysis. The resulting depth to neutral axis, c, is 13.6” and the nominal capacity, \( M_n \), is 321,362 kip-in. The capacity reduction factor is 1.0. Accounting for the top flange results in 14% additional capacity.
Appendix 5-B13

Example for Hammerhead Pier

The hammerhead pier shown in Figure 5.1 consists of a rectangular pier and a variable depth cap beam that supports 5 lines of precast, pretensioned girders. The girders sit on neoprene pads, which in turn are supported by concrete bearing blocks having dimensions of 18 x 36 in. The Strength I factored loads acting on the 5 bearing blocks include allowances for the factored self-weight of the cap beam.

The specified concrete compressive strength, $f'_c$, is 4 ksi and the specified yield strength of the reinforcing steel is 60 ksi.

Design the hammerhead pier using the AASHTO LRFD Specifications.

![Diagram of a hammerhead pier](image)

**Figure 5.1. Details of hammerhead pier.**

The three central loads are located at a distance which is less than twice the member depth from the supporting reaction. Hence the central 20 ft of the hammerhead pier is a D-Region and will be designed using the strut-and-tie method. The outer portions of the hammerhead pier are flexural regions (B-Regions) which can be designed for shear using either the sectional model or the strut-and-tie model. For this example, the strut-and-tie model will be used.

\[\text{§5.6.3} \quad \text{§5.8.1.1}\]
Step 1 - Draw Idealized Truss Model and Solve for Member Forces

The idealized truss model shown in Figure 5.2 represents the flow of forces in the hammerhead pier. The dashed lines coincide with the centerlines of the compressive struts that represent compressive stresses in different areas of the concrete. The solid lines coincide with the centroids of tension ties, which represent tension forces in different groups of reinforcing bars.

Under the action of the girder loads the ends of the cap beam will bend down causing tension near the top face of the hammerhead pier and compression near the sloping bottom faces. To allow appropriate room for placement of the longitudinal reinforcement, it has been assumed that the centroid of the tension tie near the top face is located 6 in. below the top face. To provide an appropriate space for the concrete compression zone, it has been assumed that the centerline of the bottom compression strut is located 9 in. above the sloping bottom face and is parallel to this face. The compression force in the pier is represented by 3 vertical struts. The central strut carries the 585 kip load, while the outer two struts carry 1075 kips each. Assuming that the pier is subjected to uniform compressive stresses, the width of each outer strut must be:

\[
\frac{1075}{585 + 2 \times 1075} \times 8 = 3.14 \text{ ft}
\]

Hence the centerline of the outer struts will be 0.50 x 3.14 = 1.57 ft from the outer faces of the pier.

The distributed stirrups in the cap beam are represented by the vertical tension Ties AB, CD, EF, and GH. To solve the statics of the truss model it is convenient to know the lengths of these 4 truss members. As can be seen from Figure 5.1 and Figure 5.2, the vertical distance between the top tie, ACEG, and the bottom strut, BDFH, increases by 0.2432 ft for every additional foot traveled away from the free end of the cantilever. As shown in Figure 5.2, the resulting lengths of the 4 vertical ties are 3.858 ft, 5.074 ft, 6.29 ft, and 8.132 ft.

The member forces shown in Figure 5.2 were determined by the method of joints. Thus at Joint A, the vertical component from Member AD must push the joint upwards with 530 kips. The member must also push the joint to the left with a force of 530 x 5.00 / 5.074 = 522 kips. The square root of the sum of the squares of these two components is the force in Member AD, namely a compression of 744 kips. Member AC must have a tension force of 522 kips to balance the horizontal component of Member AD. Considering horizontal and vertical equilibrium for Joints D, C, F, E, H, and G enables all of the member forces to be computed.
Figure 5.2. Truss idealization.

It is of interest to note that the vertical component of the compression force in the sloping bottom strut, BDFH, carries a significant portion of the vertical shear force. Thus if Member BDFH were horizontal, the forces in Members CD and EF, which represent the tensions in the stirrups, would both be 530 kips, rather than 403 kips and 325 kips, respectively.

Step 2 – Check Size of Bearings

The concrete in the vicinity of Joint E, that is nodal region E, must anchor vertical Tie EF and horizontal Ties EC and EG. The bearing stress on such a region (CTT node) is limited to $0.65f_c'$. Hence the minimum bearing area required to support the 545 kip load is:

$$\text{bearing area required} = \frac{P_u}{0.65f_c'} = \frac{545}{0.65 \times 0.70 \times 4} = 299 \text{ in}^2$$

Therefore, the bearing area chosen, 18 x 36 in., is satisfactory (648 in.$^2$).
Step 3 – Design Reinforcement for Main Tension Tie ACEGI

(a) At the highest tension locations, EGI

The required area of tension tie reinforcement, $A_{st}$, is:

$$A_{st} = \frac{P_u}{\phi f_y} = \frac{1653}{0.9 \times 60} = 30.61 \text{ in.}^2$$

§5.6.3.4.1

Use 20 No. 11 bars, $A_{st} = 20 \times 1.56 = 31.2 \text{ in.}^2$

As shown in Figure 5.3, the required 20 No. 11 bars can be provided in 2 layers of 10 bars. If No. 5 stirrups are used the centroid of the 20 No. 11 bars will be about 4.7 in. from the top face. Hence the assumption that the centroid of the tension tie would be 6 in. below the top face was conservative.

(b) At lowest tension location, AC

The required area of tension tie reinforcement is:

$$A_{st} = \frac{P_u}{\phi f_y} = \frac{522}{0.9 \times 60} = 9.67 \text{ in.}^2$$

§5.6.3.4.1

Therefore, use 8 No. 11 bars, $A_{st} = 8 \times 1.56 = 12.48 \text{ in.}^2$.

Figure 5.3. Layout of 20 – No. 11 top bars near pier.
(c) Development of bars

The development length for a straight top horizontal No. 11 bar with $f_y = 60$ ksi and $f_c' = 4$ ksi is 82 in. If $90^\circ$ hooks with at least 2.5 in. of side cover are used the development length is reduced to 19 in. Hence terminate the 10 bars in the lower layer at a location 19 in. beyond point E. Terminate the remaining 10 bars with $90^\circ$ hooks at a location 27 in. beyond point A.

§5.11.2.4

Step 4 – Design Tension Ties Representing Stirrups

Try using No. 5 stirrups with 4 legs (see Figure 5.3).

(a) Stirrup spacing required for Tie CD

Vertical Tie CD has the highest tension. Hence the number of stirrups required in stirrup band 2 (see Figure 5.2), is:

$$n = \frac{P_u}{\phi A_{st} f_y} = \frac{403}{0.9 \times 4 \times 0.31 \times 60} = 6.02$$

§5.6.3.4.1

Hence, the required spacing, $s$, within the 5-ft band is:

$$s \leq \frac{60}{6.02} = 9.97 \text{ in.}$$

Try a spacing of 9 in.

In the flexural region between A and E the minimum transverse reinforcement, assuming a stirrup spacing of 9 in., is:

$$A_v = 0.0316 \sqrt{\frac{f_c'}{f_y}} b_v s = 0.0316 \times \sqrt{4} \times \frac{42 \times 9}{60} = 0.39 \text{ in.}^2$$

§5.8.2.5

Since $A_v = 4 \times 0.31 = 1.24 \text{ in.}^2$, an amount greater than minimum has been provided in stirrup band 2 (see Figure 5.2). While No. 5 stirrups with 2 legs could be used in stirrup band 1, which will be governed by the minimum area requirement, it would be more practical to continue the 4-legged No. 5 stirrups at a spacing of 9 in. throughout this region.
(b) Stirrup spacing required for Tie EF

Vertical Tie EF must resist a tension of 325 kips. Hence the number of stirrups required in stirrup band 3 (see Figure 5.2), is:

\[ n = \frac{P_u}{\phi A_{st} f_y} = \frac{325}{0.9 \times 4 \times 0.31 \times 60} = 4.85 \]  
\( \text{§5.6.3.4.1} \)

Hence, the required spacing, s, within the 5-ft band is:

\[ s \leq \frac{60}{4.85} = 12.37 \text{ in.} \]

Try a spacing of 12 in.

For crack control in this disturbed region, the ratio of reinforcement area to cross-sectional area shall not be less than 0.003 in both the vertical and horizontal directions. Hence:

\[ \frac{A_{st}}{b_s} \geq 0.003 \]

Therefore:

\[ s \leq \frac{A_{st}}{0.003 b} = \frac{4 \times 0.31}{0.003 \times 42} = 9.84 \text{ in.} \]

Thus use No. 5 stirrups with 4 legs spaced at 9 in. throughout the length of the beam.

Step 5 – Check Capacity of Bottom Strut BDFH

The highest compressive force in the bottom Strut BDFH is 867 kips in Member FH (see Figure 5.2).

As this strut will be crossed by vertical stirrups, the compressive capacity of this strut may need to be reduced. The area of Tie EF is \((60/9) \times 4 \times 0.31 = 8.27 \text{ in.}^2\). Hence the strain in this stirrup under the 325 kip tension is:

\[ \varepsilon_s = \frac{P_u}{A_{st} E_s} = \frac{325}{8.27 \times 29,000} = 1.36 \times 10^{-3} \]

As the smallest angle between the strut and the tension tie is 90 - 13.7 = 76.3\(^\circ\), the principal strain, \(\varepsilon_1\), can be determined as:  
\( \text{§5.6.3.3.3} \)

\[ \varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s = 1.36 \times 10^{-3} + (1.36 \times 10^{-3} + 0.002)\cot^2 76.3^\circ = 1.56 \times 10^{-3} \]
And, the limiting compressive stress, $f_{cu}$, in the strut is:

$$f_{cu} = \frac{f'_{c}}{0.8 + 170 \varepsilon_{1}} \leq 0.85 f'_{c} = \frac{4}{0.8 + 170 \times 1.56 \times 10^{-3}} = 3.76 \text{ ksi} \leq 0.85 \times 4 = 3.40 \text{ ksi}$$

The centroid of the strut was assumed to be at 9 in. vertically from the bottom face (see Figure 5.2); hence the thickness of the strut perpendicular to the sloping bottom face is $2 \times 9 \times \cos 13.7^\circ = 17.5$ in. The nominal resistance of the strut is:

$$P_{n} = f_{cu}A_{cs} = 3.40 \times 42 \times 17.5 = 2499 \text{ kips}$$

The factored resistance of the strut is:

$$P_{f} = \phi P_{n} = 0.70 \times 2499 = 1749 \text{ kips}$$

As the factored resistance exceeds the 867 kip compression due to factored loads, the strut capacity is adequate.

While the truss geometry could be adjusted by reducing the thickness of the bottom strut and the member forces recalculated, the changes in forces will be rather small, resulting in perhaps the saving of only one bar in the main tension tie. Thus the original conservative assumptions are acceptable.

**Step 6 -- Check Capacity of Diagonal Struts of AD, CF, and EH**

Of the three diagonal struts crossing the web, AD, CF, and EH, Member EH has the highest compression. The details of the member at end E, where it crosses the tension ties, are shown in Figure 5.4.

The strains in Ties CE and EG due to factored loads are shown in Figure 5.3. For determining the strut capacity, the average value of these two strains has been assumed, giving $\varepsilon_{s} = 1.85 \times 10^{-3}$.

The principal strain, $\varepsilon_{1}$, can be determined as:

$$\varepsilon_{1} = \varepsilon_{s} + (\varepsilon_{s} + 0.002) \cot^{2} \alpha_{s} = 1.85 \times 10^{-3} + (1.85 \times 10^{-3} + 0.002) \cot^{2} 47.0^\circ = 5.20 \times 10^{-3}$$

and the limiting compressive stress, $f_{cu}$, in the strut is:

$$f_{cu} = \frac{f'_{c}}{0.8 + 170 \varepsilon_{1}} \leq 0.85 f'_{c} = \frac{4}{0.8 + 170 \times 5.20 \times 10^{-3}} = 2.38 \text{ ksi} \leq 0.85 \times 4 = 3.40 \text{ ksi}$$
The cross-sectional dimension of strut EH in the plane of the pier is 19.6 in. (see Figure 5.4), while the effective thickness of the strut at end E could be conservatively taken as 36 in. which is the width of the bearing block. However, the good anchorage conditions provided by the No. 11 bars in the corner of the stirrups enable the effective thickness of the strut to be increased.

\[ \varepsilon_s = \frac{843}{10 \times 1.56 \times 25000} = 1.86 \times 10^{-3} \]

\[ \alpha_s = 47.0^\circ \]

\[ 18 \sin 47.0^\circ + 9.4 \cos 47.0^\circ = 19.6^\circ \]

**Figure 5.4. Details of Strut EH near Node E.**

As can be seen from Figure 5.3, the center-to-center distance of the vertical stirrups across the 42-in. width of the hammerhead pier is 12.5 in. As this distance is less than 2 × 6d_{ba} = 2 × 6 × 1.410 = 16.9 in., the full 42-in. width of the pier cap is effective. Hence the nominal resistance of the strut is:

\[ P_n = f_{cu} A_{cs} = 2.38 \times 42 \times 19.6 = 1959 \text{ kips} \]

The factored resistance of the strut is:

\[ P_r = \phi P_n = 0.70 \times 1959 = 1357 \text{ kips} \geq 1189 \text{ kips required} \]

Therefore, the strut capacity is adequate.
Step 7 – Provide Crack Control Reinforcement

In Step 4, the stirrup spacing was adjusted to satisfy the crack control requirements for reinforcement in the vertical direction, but crack control reinforcement also must be provided in the horizontal direction. The vertical spacing between these horizontal bars must not exceed 12 in. If this maximum spacing is used, the area of horizontal bars in each layer needs to be:

\[ A_{st} = 0.003bs = 0.003 \times 12 \times 42 = 1.51 \text{ in.}^2 \]

Therefore, use 4 No. 6 horizontal bars at 12 in. spacing \((4 \times 0.44 = 1.76 \text{ in.}^2\) provided), arranged as shown in Figure 5.5.

Step 8 – Sketch the Required Reinforcement

The resulting reinforcement for the hammerhead pier is shown in Figure 5.5. For clarity the pier reinforcement is not shown.

![Reinforcement details for hammerhead pier.](image)

*Figure 5.5. Reinforcement details for hammerhead pier.*
Shear and Torsion Capacity of a Reinforced Concrete Beam

Define Units:
\[ \begin{align*}
\text{ksi} & = 1000\text{-psi} \\
\text{kip} & = 1000\text{-lbf} \\
\text{kip-ft}^3 & = \text{kip} \cdot \text{ft}^{-3} \\
\text{kip-ft} & = \text{kip} \cdot \text{ft}^{-1}
\end{align*} \]

Problem Description:
Find the torsion and shear capacity of a reinforced concrete beam of width 37in and height 90in. Clear cover for all sides equals 1.625in. Shear and torsion reinforcement consists of #6 stirrups spaced at 5in. Longitudinal moment steel consists of 4 #18 bars in one row in the top and in the bottom. Factored loads are \( V_u = 450 \) kips and \( T_u = 500 \) kip-ft.

Concrete Properties:
\[ f_c' := 4\cdot\text{ksi} \]

Reinforcement Properties:

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<th>Bar Diameter (in)</th>
<th>Bar Area (in^2)</th>
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<table>
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<tr>
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<td>4.00</td>
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</table>

\[ f_y := 40\cdot\text{ksi} \]

\[ E_s := 29000\text{ksi} \]

\[ E_p := 28500\text{ksi} \]

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<tr>
<td>18</td>
<td>Longitudinal - Bottom</td>
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<tr>
<td>6</td>
<td>Transverse</td>
</tr>
<tr>
<td>5</td>
<td>Spacing of Transverse Reinforcement</td>
</tr>
</tbody>
</table>

\[ \text{Spacing of Transverse Reinforcement} := \frac{\text{bar}}{\text{bar}} \text{ in} \]

LRFD 5.4.3.2

LRFD 5.4.4.2 for strands

Longitudinal - Top

Longitudinal - Bottom

Transverse

Spacing of Transverse Reinforcement
Concrete Structures Chapter 5

Factored Loads:

\[ V_u := 450 \text{ kip} \]
\[ T_u := 500 \text{ kip-ft} \]
\[ M_u := 0 \text{ kip-ft} \]
\[ N_u := 0 \text{ kip} \]

Torsional Resistance Investigation Requirement:

Torsion shall be investigated where: \( LRFD \ 5.8.2.1 \)

\[ T_u > 0.25 \phi \cdot T_{cr} \]

For Torsion and Shear - Normal weight concrete \( LRFD \ 5.5.4.2 \)

\[ \phi := 0.90 \]

\[ A_{cp} := b \cdot h \]
\[ A_{cp} = 3330 \text{ in}^2 \]

\[ p_c := (b + h) \cdot 2 \]
\[ p_c = 254 \text{ in} \]

\[ f_{pc} := 0 \text{ ksi} \]

\[ T_{cr} := 0.125 \sqrt{\frac{f_c}{\text{ksi}}} \left( \frac{A_{cp}}{\text{in}^2} \right)^2 \sqrt{1 + \frac{f_{pc}}{\text{ksi}}} \cdot \text{kip-in} \]
\[ T_{cr} = 10914 \text{ kip-in} \]
\[ T_{cr} = 909.5 \text{ kip-ft} \]

Beam Section Properties:

\[ d_{LT} := \text{dia}(\text{bar}_{LT}) \]
\[ d_{LT} = 2.257 \text{ in} \]
\[ A_{LT} := A_b(\text{bar}_{LT}) \]
\[ A_{LT} = 4 \text{ in}^2 \]

\[ d_{LB} := \text{dia}(\text{bar}_{LB}) \]
\[ d_{LB} = 2.257 \text{ in} \]
\[ A_{LB} := A_b(\text{bar}_{LB}) \]
\[ A_{LB} = 4 \text{ in}^2 \]

\[ d_T := \text{dia}(\text{bar}_T) \]
\[ d_T = 0.75 \text{ in} \]
\[ A_T := A_b(\text{bar}_T) \]
\[ A_T = 0.44 \text{ in}^2 \]

\[ b := 37 \text{ in} \]
\[ h := 90 \text{ in} \]
\[ \text{bottomcover} := 1.625 \cdot \text{ in} \]
\[ \text{sidecover} := 1.625 \cdot \text{ in} \]
\[ \text{topcover} := 1.625 \cdot \text{ in} \]
\[ 0.25 \cdot \phi \cdot T_{cr} = 204.6 \text{kip}\cdot\text{ft} \]

\[ T_u > 0.25 \cdot \phi \cdot T_{cr} = 1 \]

Torsion shall be investigated.

Since torsion shall be investigated, transverse reinforcement is required as per LRFD 5.8.2.4. The minimum transverse reinforcement requirement of LRFD 5.8.2.5 shall be met.

**Minimum Transverse Reinforcement:**

\[ b_v := b \]

\[ b_v = 37 \text{in} \]

\[ A_v := 2A_T \]

\[ A_v = 0.88 \text{in}^2 \]

\[ A_{vmin} := 0.0316 \sqrt{\frac{f_c}{\text{ksi}} \cdot \frac{b_v \cdot s}{f_y}} \]

\[ A_{vmin} = 0.29 \text{in}^2 \]

\[ A_v \geq A_{vmin} = 1 \quad \text{OK} \]

**Equivalent Factored Shear Force:**

\[ p_h := 2 \left[ b - 2 \left( \text{sidecover} + \frac{d_T}{2} \right) \right] + \left( h - \text{topcover} - \text{bottomcover} - d_T \right) \]

\[ p_h = 238 \text{in} \]

\[ A_{oh} := b - 2 \left( \text{sidecover} + \frac{d_T}{2} \right) \cdot \left( h - \text{topcover} - \text{bottomcover} - d_T \right) \]

\[ A_{oh} = 2838 \text{in}^2 \]

\[ A_o := 0.85 \cdot A_{oh} \]

\[ A_o = 2412.3 \text{in}^2 \]

\[ V_{ust} := \sqrt{V_u^2 + \left( \frac{0.9 \cdot p_h \cdot T_u}{2 \cdot A_o} \right)^2} \]

\[ V_{ust} = 522.9 \text{kip} \]

\[ V_{ust} \] shall be used to determine \( \beta \) and \( \theta \).
**Concrete Structures**

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**Determination of $\beta$ and $\theta$:**

\[ \beta \approx \frac{V_{u}}{V_{p}} \]

\[ \theta = 30.5 \cdot \text{deg} \]

\[ V_{p} := 0 \cdot \text{kip} \]

\[ A_{ps} := 0 \cdot \text{in}^2 \]

\[ A_{s} := 4 \cdot A_{LB} \]

\[ f_{po} := 0 \cdot \text{ksi} \]

\[ d_{e} := h - \text{bottomcover} - d_{T} - \frac{d_{LB}}{2} \]

\[ d_{v} := \max(0.9 \cdot d_{e}, 0.72 \cdot h) \]

\[ d_{e} = 86.5 \text{in} \]

\[ d_{v} = 77.85 \text{in} \]

\[ V_{u} := 0.202 \text{ksi} \]

\[ \frac{V_{u}}{f'_{c}} = 0.05 \]

---

From Table 5.8.3.4.2-1, Find $\beta$ and $\theta$

\[ \theta := 30.5 \cdot \text{deg} \quad \text{Value is close to original guess. OK.} \]

\[ \beta := 2.59 \]

---

LRFD 5.8.3.4

LRFD 5.8.2.9

LRFD 5.8.3.4.2-1

LRFD 5.8.2.9-1

---
Torsional Resistance:

The factored Torsional Resistance shall be: LRFD 5.8.2.1

\[ T_r = \phi \cdot T_n \]

\[ A_t := A_T \quad A_t = 0.44 \text{ in}^2 \quad \text{LRFD 5.8.3.6.2} \]

\[ T_n := \frac{2 \cdot A_T \cdot f_y \cdot \cot(\theta)}{s} \]

\[ T_n = 28831 \text{ kip}\cdot\text{in} \quad \text{LRFD 5.8.3.6.2-1} \]

\[ T_n = 2403 \text{ kip}\cdot\text{ft} \]

\[ T_r := \phi \cdot T_n \]

\[ T_r = 25948 \text{ kip}\cdot\text{in} \quad \text{LRFD 5.8.2.1} \]

\[ T_r = 2162 \text{ kip}\cdot\text{ft} \]

\[ T_r \geq T_u = 1 \quad \text{OK} \]

Shear Resistance:

The factored Shear Resistance shall be: LRFD 5.8.2.1

\[ V_r = \phi \cdot V_n \]

\[ V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f_c}{\text{ksi}}} \cdot b_v \cdot d_v \cdot \text{ksi} \]

\[ V_c = 471.5 \text{ kip} \quad \text{LRFD 5.8.3.3} \]

\[ \alpha := 90 \cdot \text{deg} \]

\[ V_s := \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s} \]

\[ V_s = 930.4 \text{ kip} \quad \text{LRFD 5.8.3.3} \]

\[ V_n := \min(V_c + V_s + V_p \cdot 0.25 \cdot f_c \cdot b_v \cdot d_v + V_p) \]

\[ V_n = 1402 \text{ kip} \]

\[ V_r := \phi \cdot V_n \]

\[ V_r = 1262 \text{ kip} \]

\[ V_r \geq V_u = 1 \quad \text{OK} \]
Check for Longitudinal Reinforcement:  

\[ f_{ps} := 0 \cdot \text{ksi} \]

\[ X_1 := A_s \cdot f_y + A_{ps} \cdot f_{ps} \quad X_1 = 640 \text{ kip} \]

For a Solid Section:

\[ X_2 := \frac{|M_u|}{\phi \cdot d_v} + \frac{0.5 \cdot N_u}{\phi} + \cot(\theta) \cdot \sqrt{\left(\frac{V_u}{\phi} - V_p - 0.5 \cdot V_s\right)^2 + \left(\frac{0.45 \cdot p_h \cdot T_u}{2 \cdot A_o \cdot \phi}\right)^2} \]

\[ X_2 = 258 \text{ kip} \]

\[ X_1 \geq X_2 = 1 \quad \text{OK} \]

Maximum Spacing of Transverse Reinforcement:  

\[ v_u = 0.202 \text{ksi} \]

\[ 0.125 \cdot f'c = 0.5 \text{ksi} \]

\[ s_{\text{max}} := \text{if} \left(v_u < 0.125 \cdot f'c \cdot \min(0.8 \cdot d_v, 24\text{in}), \min(0.4 \cdot d_v, 12\text{in})\right) \]

\[ s_{\text{max}} = 24 \text{in} \]

\[ \text{if} \left(s \leq s_{\text{max}}, "OK", "NG" \right) = "OK" \]
Sound Wall Design
Appendix 5-B15 – Type D-2k

This design is based upon:
- AASHTO Standard Specifications for Highway Bridges 17th Ed. - 2002
- USS Steel Sheet Piling Design Manual - July 1984
- WSDOT Bridge Design Manual
- Caltrans Trenching and Shoring Manual - June 1995

This design doesn’t account for the loads of a combined retaining wall / noisewall. A maximum of 2 ft of retained fill above the final ground line is suggested.


Define Units:
- ksi ≡ 1000·psi
- kip ≡ 1000·lbf
- kcf ≡ kip·ft⁻³
- klf ≡ kip·ft⁻¹
- plf ≡ lbf·ft⁻¹
- psf ≡ lbf·ft⁻²
-pcf ≡ lbf·ft⁻³

Concrete Properties:

\[ w_c := 160\text{ pcf} \quad \text{BDM 4.1.1} \]

\[ f'c := 4000\text{ psi} \]

\[ E_c := \left( \frac{w_c}{pcf} \right)^{1.5} \cdot 33 \cdot \sqrt[3]{f'c\text{ psi}} \quad E_c = 4.224 \times 10^6 \text{ psi} \quad \text{Std Spec. 8.7.1} \]

\[ \beta_1 := \begin{cases} f'c \leq 4000 \text{ psi} & , 0.85, \max \left( 0.85 - \frac{f'c - 4000 \text{ psi}}{1000 \text{ psi}} \cdot 0.05, 0.65 \right) \end{cases} \quad \beta_1 = 0.85 \quad \text{Std Spec. 8.16.2.7} \]

\[ f_r := 7.5 \cdot \sqrt[3]{f'c\text{ psi}} \quad f_r = 474.3\text{ psi} \quad \text{Std Spec. 8.15.2.1.1} \]
Reinforcement Properties:

**Diameters:**
- $0.375\text{ in}$ if $\text{bar} = 3$
- $0.500\text{ in}$ if $\text{bar} = 4$
- $0.625\text{ in}$ if $\text{bar} = 5$
- $0.750\text{ in}$ if $\text{bar} = 6$
- $0.875\text{ in}$ if $\text{bar} = 7$
- $1.000\text{ in}$ if $\text{bar} = 8$
- $1.128\text{ in}$ if $\text{bar} = 9$
- $1.270\text{ in}$ if $\text{bar} = 10$
- $1.410\text{ in}$ if $\text{bar} = 11$
- $1.693\text{ in}$ if $\text{bar} = 14$
- $2.257\text{ in}$ if $\text{bar} = 18$

**Areas:**
- $0.11\text{ in}^2$ if $\text{bar} = 3$
- $0.20\text{ in}^2$ if $\text{bar} = 4$
- $0.31\text{ in}^2$ if $\text{bar} = 5$
- $0.44\text{ in}^2$ if $\text{bar} = 6$
- $0.60\text{ in}^2$ if $\text{bar} = 7$
- $0.79\text{ in}^2$ if $\text{bar} = 8$
- $1.00\text{ in}^2$ if $\text{bar} = 9$
- $1.27\text{ in}^2$ if $\text{bar} = 10$
- $1.56\text{ in}^2$ if $\text{bar} = 11$
- $2.25\text{ in}^2$ if $\text{bar} = 14$
- $4.00\text{ in}^2$ if $\text{bar} = 18$

$f_y := 60000 \text{ psi}$

$E_s := 29000000 \text{ psi}$

Std. Spec. 8.7.2

---

**FIGURE A: SHAFT LATERAL SOIL PRESSURES**
### Wall Geometry:

- **Wall Height:** \( H := 24 \text{ ft} \)  
  \( H \) should be \( \leq 28 \text{ ft} \)
- **Half of Wall Height:** \( h := H \cdot 0.5 \)  
  \( h = 12 \text{ ft} \)
- **Shaft Diameter:** \( b := 2.50 \text{ ft} \)
- **Shaft Spacing:** \( L := 12 \text{ ft} \)

### Wind Load (Guide Spec. Table 1-2.2.2.C):

- **Wind Exposure B2**: Wind Exposure B1 or B2 - Provided by the Region
- **Wind Velocity 90 mph**: Wind Velocity 80 or 90 mph - Provided by the Region

\[
\text{WindPressure}(\text{WindExp}, \text{WindVel}) :=
\begin{align*}
12 \text{ psf} & \quad \text{if } (\text{WindExp} = \text{"B1"} \land \text{WindVel} = 80 \text{ mph}) \\
16 \text{ psf} & \quad \text{if } (\text{WindExp} = \text{"B1"} \land \text{WindVel} = 90 \text{ mph}) \\
20 \text{ psf} & \quad \text{if } (\text{WindExp} = \text{"B2"} \land \text{WindVel} = 80 \text{ mph}) \\
25 \text{ psf} & \quad \text{if } (\text{WindExp} = \text{"B2"} \land \text{WindVel} = 90 \text{ mph}) \\
\text{"error"} & \quad \text{otherwise}
\end{align*}
\]

Wind Pressure: \( P_w := \text{WindPressure}(\text{WindExp}, \text{WindVel}) \)  
\( P_w = 25 \text{ psf} \)

### Seismic Load (Guide Spec. 1-2.1.3):

- **Acceleration Coefficient**: \( A := 0.35 \)  
  BDM 4.4-A2
- **DL Coefficient, Wall**: \( f := 0.75 \)  
  Not on bridge condition
- **Panel Plan Area**: \( A_{pp} := 4\text{in} \cdot L + 13\text{in} \cdot 16\text{in} \)  
  \( A_{pp} = 5.44 \text{ ft}^2 \)
- **Seismic Force EQD (perp. to wall surface)**: \( \text{EQD} := \max(A \cdot f, 0.1) \cdot \left(\frac{A_{pp} \cdot w_c}{L}\right) \)  
  \( \text{EQD} = 19.1 \text{ psf} \)

### Factored Loads (Guide Spec. 1-2.2.2):

- **Wind**: \( 1.3 \cdot P_w \cdot 2 \cdot h \cdot L \)  
  \( \text{Wind} = 9360 \text{ lbf} \)
- **EQ**: \( 1.3 \cdot \text{EQD} \cdot 2 \cdot h \cdot L \)  
  \( \text{EQ} = 7134 \text{ lbf} \)
- **P**: \( \max(\text{Wind}, \text{EQ}) \)  
  \( P = 9360 \text{ lbf} \)  
  Factored Design load acting at mid height of wall "h".
Soil Parameters:

- **Soil Friction Angle:** $\phi \:= \, 38 \cdot \text{deg}$
  - Provided by the Region

- **Soil Unit Weight:** $\gamma \:= \, 125 \cdot \text{pcf}$
  - Provided by the Region

- **Top Soil Depth:** $y \:= \, 2.0 \cdot \text{ft}$
  - From top of shaft to ground line

- **Ineffective Shaft Depth:** $d_0 \:= \, 0.5 \cdot \text{ft}$
  - Depth of neglected soil at shaft

- **Isolation Factor for Shafts:** $\text{Iso} := \min \left( 3.0, 0.08 \cdot \frac{\phi}{\text{deg}} \cdot \frac{L}{b} \right)$
  - Factor used to amplify the passive resistance based on soil wedge behavior resulting from shaft spacing - Caltrans pg 10-2.
  - $\text{Iso} = 3.00$

- **Factor of Safety:** $\text{FS} := \, 1.00$

- **Angle of Wall Friction:** $\delta := \frac{2}{3} \cdot \phi$
  - $\delta = 25.333 \text{deg}$

- **Correction Factor for Horizontal Component of Earth Pressure:** $HC := \cos (\delta)$
  - $HC = 0.904$

- **Foundation Strength Reduction Factors:**
  - $\phi_{fa} := \, 1.00$ (Active)
    - Guide Spec. 1-2.2.3
  - $\phi_{fp} := \, 0.90$ (Passive)
    - Guide Spec. 1-2.2.3
Fig. 5(a) – Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel)
Concrete Structures

Chapter 5

Side 1:

Backfill Slope Angle: \( \beta_s := -\arctan \left( \frac{1}{2} \right) \)
\( \beta_s = -26.5651 \) deg
\( \frac{\beta_s}{\phi} = -0.70 \)

Using the USS Steel Sheet Piling Design Manual, Figure 5(a):

For \( \phi = 38 \) deg and \( \beta_s = 0 \) deg:
\( K_a = 0.234, K_p = 14.20, R_p = 0.773 \)

For \( \phi = 32 \) deg and \( \beta_s = 0 \) deg:
\( K_a = 0.290, K_p = 7.85, R_p = 0.8366 \)

For \( \phi = 38 \) deg and \( \beta_s = -26.5651 \) deg:
\( K_a = 0.190, K_p = 3.060, R_p = 0.773 \)

For \( \phi = 32 \) deg and \( \beta_s = -26.5651 \) deg:
\( K_a = 0.230, K_p = 1.82, R_p = 0.8366 \)

Active Earth Pressure Coeff:
\( K_{a1} := 0.190 \) USS Fig. 5(a)

Passive Earth Pressure Coeff:
\( K_{p1} := 3.060 \) USS Fig. 5(a)

Reduction for Kp:
\( R_{p1} := 0.773 \) USS Fig. 5(a)

Active Pressure:
\[ \phi P_{a1} := K_{a1} \cdot \gamma \cdot HC \cdot \phi_{fa} \]
\[ \phi P_{a1} = 21 \text{ psf/ft} \]

Passive Pressure:
\[ \phi P_{p1} := \frac{K_{p1} \cdot R_{p1} \cdot \gamma \cdot HC \cdot Iso \cdot \phi_{fp}}{FS} \]
\[ \phi P_{p1} = 722 \text{ psf/ft} \]

Side 2:

Backfill Slope Angle: \( \beta_s := -\arctan \left( \frac{1}{2} \right) \)
\( \beta_s = -26.5651 \) deg
\( \frac{\beta_s}{\phi} = -0.70 \)

Active Earth Pressure Coeff:
\( K_{a2} := 0.190 \) USS Fig. 5(a)

Passive Earth Pressure Coeff:
\( K_{p2} := 3.060 \) USS Fig. 5(a)

Reduction for Kp:
\( R_{p2} := 0.773 \) USS Fig. 5(a)

Active Pressure:
\[ \phi P_{a2} := K_{a2} \cdot \gamma \cdot HC \cdot \phi_{fa} \]
\[ \phi P_{a2} = 21 \text{ psf/ft} \]

Passive Pressure:
\[ \phi P_{p2} := \frac{K_{p2} \cdot R_{p2} \cdot \gamma \cdot HC \cdot Iso \cdot \phi_{fp}}{FS} \]
\[ \phi P_{p2} = 722 \text{ psf/ft} \]

Allowable Net Lateral Soil Pressure:

\[ R_1 := \phi P_{p1} - \phi P_{a2} \]
\[ R_1 = 700 \text{ psf/ft} \] Side 1

\[ R_2 := \phi P_{p2} - \phi P_{a1} \]
\[ R_2 = 700 \text{ psf/ft} \] Side 2
Depth of Shaft Required:

The function "ShaftD" finds the required shaft depth "d" by increasing the shaft depth until the sum of the moments about the base of the shaft "Msum" is nearly zero. See Figure A for a definition of terms.

\[
\text{ShaftD}(d_0, P, R_1, R_2, b, h, y) :=
\begin{align*}
&d \leftarrow 0 \text{ ft} \\
&M_{\text{Sum}} \leftarrow 100 \text{ lbf-ft} \\
\text{while } &M_{\text{Sum}} \geq 0.001 \text{ lbf-ft} \\
&d \leftarrow d + 0.00001 \text{ ft} \\
&z \leftarrow \frac{2}{d(R_1 + R_2)} \left( \frac{R_2 d^2}{2} - \frac{R_2 d_0^2}{2} - \frac{P}{b} \right) \\
&x \leftarrow \frac{R_2 z (d - z)}{R_1 d + R_2 (d - z)} \\
&P_1 \leftarrow (R_2 d_0) (d - d_0 - z) \\
&P_2 \leftarrow R_2 (d - d_0 - z)^2 \cdot \frac{1}{2} \\
&P_3 \leftarrow R_2 (d - z) \cdot x \cdot \frac{1}{2} \\
&P_4 \leftarrow R_1 \cdot d \cdot (z - x) \cdot \frac{1}{2} \\
&X_1 \leftarrow \frac{z + d - d_0}{2} \\
&X_2 \leftarrow \frac{2 z + d - d_0}{3} \\
&X_3 \leftarrow z - \frac{x}{3} \\
&X_4 \leftarrow \frac{1}{3} (z - x) \\
&M_{\text{Sum}} \leftarrow P \cdot (h + y + d) + b \cdot ( -P_1 \cdot X_1 - P_2 \cdot X_2 - P_3 \cdot X_3 + P_4 \cdot X_4) \\
\end{align*}
\]
Check for 2 load cases. Case 1 has load \( P \) acting as shown on Figure A. Case 2 has load \( P \) acting in the opposite direction.

**Case 1:**

\[
\begin{align*}
d_{c1} := & \text{ShaftD}(d_0, P, R_1, R_2, b, h, y) \\
z_{c1} := & \frac{2}{d_{c1} \cdot (R_1 + R_2)} \left( \frac{R_2 \cdot d_{c1}^2}{2} - \frac{R_2 \cdot d_0^2}{2} - \frac{p}{b} \right) \\
x_{c1} := & \frac{R_2 \cdot z_{c1} \cdot (d_{c1} - z_{c1})}{R_1 \cdot d_{c1} + R_2 \cdot (d_{c1} - z_{c1})} \\
P_{4c1} := & R_1 \cdot d_{c1} \cdot \left( z_{c1} - x_{c1} \right) \cdot \frac{1}{2}
\end{align*}
\]

\( d_{c1} = 11.18 \text{ ft} \)

\( z_{c1} = 5.102 \text{ ft} \)

\( x_{c1} = 1.797 \text{ ft} \)

\( P_{4c1} = 12935 \text{ ft}^2 \text{ psf} \)

**Case 2:**

\[
\begin{align*}
d_{c2} := & \text{ShaftD}(d_0, P, R_2, R_1, b, h, y) \\
z_{c2} := & \frac{2}{d_{c2} \cdot (R_1 + R_2)} \left( \frac{R_1 \cdot d_{c2}^2}{2} - \frac{R_1 \cdot d_0^2}{2} - \frac{p}{b} \right) \\
x_{c2} := & \frac{R_1 \cdot z_{c2} \cdot (d_{c2} - z_{c2})}{R_2 \cdot d_{c2} + R_1 \cdot (d_{c2} - z_{c2})} \\
P_{4c2} := & R_2 \cdot d_{c2} \cdot \left( z_{c2} - x_{c2} \right) \cdot \frac{1}{2}
\end{align*}
\]

\( d_{c2} = 11.18 \text{ ft} \)

\( z_{c2} = 5.102 \text{ ft} \)

\( x_{c2} = 1.797 \text{ ft} \)

\( P_{4c2} = 12935 \text{ ft}^2 \text{ psf} \)

**Determine Shaft Lateral Pressures and Moment Arms for Controlling Case:**

\[
\begin{align*}
d := & \max(d_{c1}, d_{c2}) \\
R_a := & \text{if}(d_{c2} \geq d_{c1}, R_1, R_2) \\
R_a := & 700 \text{ psf} \\
R_b := & \text{if}(d_{c2} \geq d_{c1}, R_2, R_1) \\
R_b := & 700 \text{ psf}
\end{align*}
\]

\[
\begin{align*}
z := & \frac{2}{d \cdot (R_a + R_b)} \left( \frac{R_a \cdot d_0^2}{2} - \frac{R_a \cdot d_0^2}{2} - \frac{p}{b} \right) \\
z := & 5.102 \text{ ft} \\
x := & \frac{R_a \cdot z \cdot (d - z)}{R_b \cdot d + R_a \cdot (d - z)} \\
x := 1.797 \text{ ft}
\end{align*}
\]

\[
\begin{align*}
P_1 := & (R_a \cdot d_0) \cdot (d - d_0 - z) \\
P_1 := & 1953 \text{ lbf ft} \\
X_1 := & \frac{z + d - d_0}{2} \\
X_1 := 7.892 \text{ ft}
\end{align*}
\]

\[
\begin{align*}
P_2 := & R_a \cdot (d - d_0 - z)^2 \cdot \frac{1}{2} \\
P_2 := & 10901 \text{ lbf ft} \\
X_2 := & \frac{2 \cdot z + d - d_0}{3} \\
X_2 := 6.962 \text{ ft}
\end{align*}
\]

\[
\begin{align*}
P_3 := & R_a \cdot (d - z) \cdot x \cdot \frac{1}{2} \\
P_3 := 3825 \text{ lbf ft} \\
X_3 := & z - \frac{x}{3} \\
X_3 := 4.503 \text{ ft}
\end{align*}
\]

\[
\begin{align*}
P_4 := & R_b \cdot d \cdot (z - x) \cdot \frac{1}{2} \\
P_4 := 12935 \text{ lbf ft} \\
X_4 := & \frac{1}{3} \cdot (z - x) \\
X_4 := 1.102 \text{ ft}
\end{align*}
\]

\[
M_{sum} := P \cdot (h + y + d) + b \cdot (-P_1 \cdot X_1 - P_2 \cdot X_2 - P_3 \cdot X_3 + P_4 \cdot X_4)
\]

\( M_{sum} = -0.13163 \text{ lbf ft} \)
Shaft Design Values:

The Maximum Shear will occur at the bolts or at the top of area 4 on Figure A:

\[ V_{shaft} := \max(P, P_4c_1 \cdot b, P_4c_2 \cdot b) \quad V_{shaft} = 32339 \text{ lbf} \]

The Maximum Moment in the shaft will occur where the shear = 0.

Assume that the point where shear = 0 occurs in areas 1 and 2 on Figure A.

Check for Case 1:

\[ s_{c1} := -d_o + \sqrt{d_o^2 + \frac{2P}{R_2 \cdot b}} \quad s_{c1} = 2.808 \text{ ft} \]

\[ M_{shaftc1} := P \cdot (h + y + d_o + s_{c1}) - R_2 \cdot d_o \cdot b \cdot s_{c1}^2 \cdot \frac{1}{2} - R_2 \cdot b - s_{c1}^3 \cdot \frac{1}{6} \quad M_{shaftc1} = 152094 \text{ lbf-ft} \]

Check that the point where shear = 0 occurs in areas 1 and 2 on Figure A:

Check1 := if \( s_{c1} \leq (d_{c1} - d_o - z_{c1}) \) \( "OK" \), "NG"

Check1 = "OK"

Check for Case 2:

\[ s_{c2} := -d_o + \sqrt{d_o^2 + \frac{2P}{R_1 \cdot b}} \quad s_{c2} = 2.808 \text{ ft} \]

\[ M_{shaftc2} := P \cdot (h + y + d_o + s_{c2}) - R_1 \cdot d_o \cdot b \cdot s_{c2}^2 \cdot \frac{1}{2} - R_1 \cdot b \cdot s_{c2}^3 \cdot \frac{1}{6} \quad M_{shaftc2} = 152094 \text{ lbf-ft} \]

Check that the point where shear = 0 occurs in areas 1 and 2 on Figure A:

Check2 := if \( s_{c2} \leq (d_{c2} - d_o - z_{c2}) \) \( "OK" \), "NG"

Check2 = "OK"

\[ M_{shaft} := \max(M_{shaftc1}, M_{shaftc2}) \quad M_{shaft} = 152094 \text{ lbf-ft} \]

Anchor Bolt and Panel Post Design Values:

\[ V_{bolt} := P \quad V_{bolt} = 9360 \text{ lbf} \]

\[ M_{bolt} := P \cdot (h + y) \quad M_{bolt} = 131040 \text{ lbf-ft} \]

Panel Design Value (about a vertical axis):

Find Design Moment for a 1 ft wide strip of wall (between panel posts) for the panel flexure design

\[ w_{panel} := \max[P_w \cdot \max(A \cdot f, 0.1) \cdot (4\text{in-}w_o)] \quad w_{panel} = 25.0 \text{ psf} \]
Panel Post Resistance:

\[ \begin{align*}
M_{\text{panel}} &= 1.3 \frac{w_{\text{panel}} \cdot L^2}{8} \\
\text{Check3} &\text{ if } \phi_f M_n \geq M_{\text{bolt}}, "OK", "NG" \\
\end{align*} \]

Check Flexural Resistance (Std. Spec. 8.16.3):

\[ \begin{align*}
\phi_f &= 0.90 \\
d_{\text{pa}} &= h_{\text{pa}} - C_{\text{pa}} - \frac{\text{dia}(\text{bar}_A)}{2} \\
A_s &= 2 \cdot A_b(\text{bar}_A) \\
a &= \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b_{\text{pa}}} \\
M_n &= A_s \cdot f_y \left( \frac{d_{\text{pa}} - a}{2} \right) \\
\phi_f M_n &= 145719 \text{lbf-ft}
\end{align*} \]

Check4 := if (\( \rho \leq 0.75 \cdot \rho_b \), "OK", "NG") [Check4 = "OK"]

Check Shear (Std. Spec. 8.16.6) - Note: Shear Capacity of stirrups neglected:

\[ \phi_v = 0.85 \]
\[ V_{ca} := 2 \cdot \sqrt{\frac{f_c}{\psi \cdot b_{pa} \cdot d_{pa}}} \]

\[ V_{ca} = 18961 \text{ lbf} \]

Check6 := if \( \phi_v \cdot V_{ca} \geq V_{bolt} \), "OK", "NG"

Check6 = "OK"

Panel Post Base Resistance:

\[ b_{pb} := 9 \text{ in} \quad \text{Width of Panel Post Base} \]
\[ h_{pb} := 17.5 \text{ in} \quad \text{Depth of Panel Post Base} \]

Check Flexural Resistance (Std. Spec. 8.16.3):

\( \phi_f = 0.9 \)

\[ d_{pb} := h_{pb} - 0.75 \text{ in} \]
\[ A_s := 2 \cdot A_b (\text{bar}_B) \]

\[ A_s = 2 \text{ in}^2 \]

\[ a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b_{pb}} \]
\[ a = 3.922 \text{ in} \]

\[ M_n := A_s \cdot f_y \left( d_{pb} - \frac{a}{2} \right) \]
\[ M_n = 147892 \text{ lbf} \cdot \text{ft} \]

Check7 := if \( \phi_f \cdot M_n \geq M_{bolt} \), "OK", "NG"

Check7 = "OK"

Check Maximum Reinforcement (Std. Spec. 8.16.3.1):

\[ \rho_b := \frac{0.85 \cdot \beta_1 \cdot f_c}{f_y} \cdot \left( \frac{87000 \cdot \psi}{87000 \cdot \psi + f_y} \right) \]

\[ \rho_b = 0.029 \]

\[ \rho := \frac{A_s}{b_{pb} \cdot d_{pb}} \]
\[ \rho = 0.01327 \]

Check8 := if \( \rho \leq 0.75 \cdot \rho_b \), "OK", "NG"

Check8 = "OK"

Check Minimum Reinforcement (Std. Spec. 8.17.1.1):

\[ S_b := \frac{b_{pb} \cdot h_{pb}^2}{6} \]
\[ S_b = 459.4 \text{ in}^3 \]

\[ M_{crb} := f_r \cdot S_b \]
\[ M_{crb} = 18158 \text{ lbf} \cdot \text{ft} \]

Check9 := if \( \phi_f \cdot M_n \geq \min \{ 1.2 \cdot M_{crb} \cdot 1.33 \cdot M_{bolt} \} \), "OK", "NG"

Check9 = "OK"

Check Shear (Std. Spec. 8.16.6) - Note: Shear Capacity of stirrups neglected:

\[ \phi_v = 0.85 \]

Std. Spec. 8.16.1.2.2
\[ V_{cb} := 2 \cdot \frac{f_c}{\text{psi}} \cdot b_{pb} \cdot d_{pb} \quad V_{cb} = 19069 \text{ lbf} \]

Check10 := if \( \left( \phi_v \cdot V_{cb} \geq V_{bolt} \right) \), "OK", "NG"

Required Splice Length (Std. Spec. 8.25 and 8.32):

Basic Development Length (Std. Spec. 8.25.1):

\[ I_{\text{basic}}(\text{bar}) := \max \left( \frac{0.04 \cdot A_{B(\text{bar})} \cdot f_y}{\sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi-in}}, 0.0004 \cdot \text{dia}(\text{bar}) \cdot \frac{f_y}{\text{psi}} \right) \text{ if bar} \leq 11 \]

\[ \frac{0.085 \cdot f_y}{\sqrt{\frac{f_c}{\text{psi}}} \cdot \text{in}} \text{ if bar} = 14 \]

\[ \frac{0.11 \cdot f_y}{\sqrt{\frac{f_c}{\text{psi}}} \cdot \text{in}} \text{ if bar} = 18 \]

"error" otherwise

\[ I_{\text{basic}}(\text{bar}_A) := I_{\text{basic}}(\text{bar}_B) \quad I_{\text{basic}}(\text{bar}_A) = 4.016 \text{ ft} \]

\[ I_{\text{basic}}(\text{bar}_B) := I_{\text{basic}}(\text{bar}_B) \quad I_{\text{basic}}(\text{bar}_B) = 3.162 \text{ ft} \]

Development Length (Std. Spec. 8.25):

For top reinforcement placed with more than 12 inches of concrete cast below (Std. Spec. 8.25.2.1):

\[ |dA| := I_{\text{basic}}(\text{bar}_A) \cdot 1.4 \quad |dA| = 5.623 \text{ ft} \]

\[ |dB| := I_{\text{basic}}(\text{bar}_B) \cdot 1.4 \quad |dB| = 4.427 \text{ ft} \]

Required Lapssplice (Y):

The required lapssplice Y is the maximum of the required lap splice length of bar A (using a Class C splice), the development length of bar B, or 2'-0" per BDM 5.1.2.D.

\[ \text{LapSplice} := \max \left( 1.7 \cdot |dA|, |dB|, 2 \cdot \text{ft} \right) \quad \text{LapSplice} = 9.558 \text{ ft} \]

Note: Lap Splices are not allowed for bar sizes greater than 11 per AASHTO Std. Spec. 8.32.1.1.

Check11 := if \( (\text{bar}_A \leq 11 \land \text{bar}_B \leq 11) \), "OK", "NG"

Check11 = "OK"
Anchor Bolt Resistance (Std. Spec. 10.56):

\[ V_{\text{bolt}} = 9360 \text{ lbf} \]
\[ M_{\text{bolt}} = 131040 \text{ lbf} \cdot \text{ft} \]
\[ d_{\text{bolt}} := 1.0 \text{ in} \]
\[ A_{\text{bolt}} := \frac{\pi \cdot d_{\text{bolt}}^2}{4} \]
\[ F_t := 30 \text{ ksi} \]
\[ F_v := 18 \text{ ksi} \]
\[ \text{PanelAxialLoad} := \left(4\text{in} \cdot \frac{L}{2} + 13\text{in} \cdot 10\text{in}\right) \cdot (2 \cdot h + y - 3\text{in}) \cdot w_c \]
\[ f_a := \frac{\text{PanelAxialLoad}}{4 \cdot A_{\text{bolt}}} \]
\[ f_v := \frac{V_{\text{bolt}}}{4 \cdot A_{\text{bolt}}} \]

Check12 := if \( f_v \leq F_v, "OK", "NG" \)

\[ f_t := \frac{M_{\text{bolt}}}{13.5\text{in} \cdot 2 \cdot A_{\text{bolt}}} - f_a \]
\[ F_{t1} := \left[ \frac{f_v}{F_v} \leq 0.33, F_t, F_t \cdot \sqrt{1 - \left(\frac{f_v}{F_v}\right)^2} \right] \]

Check13 := if \( f_t \leq F_{t1}, "OK", "NG" \)
Design Summary:

Wall Height: \( H = 24 \text{ ft} \)

Required Shaft Depth: \( d = 11.18 \text{ ft} \)

Maximum Shaft Shear: \( V_{\text{shaft}} = 32339 \text{ lbf} \)

Maximum Shaft Moment: \( M_{\text{shaft}} = 152094 \text{ lbf}\cdot\text{ft} \)

Maximum Shaft Moment Accuracy Check (Case 1): Check1 = "OK"

Maximum Shaft Moment Accuracy Check (Case 2): Check2 = "OK"

Bar A:

Bar A = 10

Post Flexural Resistance (Bar A): Check3 = "OK"

Maximum Reinforcement Check (Bar A): Check4 = "OK"

Minimum Reinforcement Check (Bar A): Check5 = "OK"

Post Shear Check (Bar A): Check6 = "OK"

Bar B:

Bar B = 9

Post Flexural Resistance (Bar B): Check7 = "OK"

Maximum Reinforcement Check (Bar B): Check8 = "OK"

Minimum Reinforcement Check (Bar B): Check9 = "OK"

Post Shear Check (Bar B): Check10 = "OK"

Lap Splice Length: LapSplice = 9.558 ft

Lap Splice Allowed Check: Check11 = "OK"

Bolt Diameter: \( d_{\text{bolt}} = 1 \text{ in} \)

Anchor Bolt Shear Stress Check: Check12 = "OK"

Anchor Bolt Tensile Stress Check: Check13 = "NG"