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, both reinforced and prestressed.

Design of concrete structures shall be based on the requirements and guidance cited herein and in the current AASHTO *LRFD Bridge Design Specifications* (LRFD), AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (SEISMIC), AASHTO *Guide Specification for Accelerated Bridge Construction* (ABC), Special Provisions and the *Standard Specifications for Road, Bridge, and Municipal Construction* (*Standard Specifications*).

5.1 Materials

5.1.1 Concrete

5.1.1.A Strength of Concrete

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

5.1.1.A.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 or higher may be used when it's commercially available near the job site.

5.1.1.A.2 Prestressed Concrete Girders

The recommended specified 28-day concrete compressive strength ($f'c$) is 7.0 ksi. Where higher strengths would eliminate a line of girders, a maximum of 10.0 ksi can be specified. Slab girders should be limited to 8.0 ksi.

The minimum specified concrete compressive strength at release ($f'ci$) is 5.0 ksi for prestressed concrete slab girders and 6.0 ksi for all other prestressed girders. Where higher strengths would eliminate a line of girders, the compressive strength at release may be increased up to 7.5 ksi. Release strengths as high as 8.5 ksi can be achieved with extended curing for special circumstances.

Specified concrete strengths should generally be rounded to the nearest 0.1 ksi above what is required for design and constructability.

5.1.1.B Classes of Concrete

5.1.1.B.1 Class 3000

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

5.1.1.B.2 Class 4000

Used in CIP post-tensioned or conventionally reinforced concrete box girders, slabs, traffic and pedestrian barriers, approach slabs, footings, Buried Structures, wing walls, curtain walls, retaining walls, columns, and crossbeams.

5.1.1.B.3 Class 4000A

Used for bridge approach slabs.

5.1.1.B.4 Class 4000D

Used for CIP bridge decks.

5.1.1.B.5 Class 4000P and 5000P

Used for CIP piles, shafts and deep foundations where vibration is not feasible or practical.

5.1.1.B.6 *Class 4000W*

Used underwater in seals.

5.1.1.B.7 *Class 5000 or Higher*

Used in CIP post-tensioned concrete box girder construction, deep bridge foundations, or in other special structural applications if significant economy can be gained or structural requirements dictate. Class 5000 or higher concrete is generally available near large urban centers. Designers shall confirm availability at the project site before specifying Class 5000 or higher concrete (such as with WACA).

The specified 28-day compressive strengths ($f'c$) are equal to the numerical class of concrete. The compressive strengths for design are shown in Table 5.1.1-1.

Table 5.1.1-1 28-Day Compressive Design Strength

Classes of Concrete	Design Compressive Strength (psi)
COMMERCIAL	2300
3000	3000
4000, 4000A, 4000D, 4000P	4000
4000W	2400*
5000, 5000P	5000
6000	6000

*40 percent reduction from Class 4000.

5.1.1.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, *Standard Specifications* Section 6-02.3(17)J 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 using equation 5.1.1-1 below.

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

$$\frac{x}{70} = \frac{100}{64} \qquad Therefore, x = 110\% \tag{5.1.1-1}$$

From Table 5.1.1-2, the design strength should be reached in 40 days.

Table 5.1.1-2 Relative and Compressive Strength of Concrete

Age	Relative Strength	Class 5000	Class 4000	Class 3000	Age	Relative Strength	Class 5000	Class 4000	Class 3000
Days	%	ksi	ksi	ksi	Days	%	ksi	ksi	ksi
3	35	1.75	1.40	1.05	20	91	4.55	3.64	2.73
4	43	2.15	1.72	1.29	21	93	4.65	3.72	2.79
5	50	2.50	2.00	1.50	22	94	4.70	3.76	2.82
6	55	2.75	2.20	1.65	23	95	4.75	3.80	2.85
7	59	2.95	2.36	1.77	24	96	4.80	3.84	2.88
8	63	3.15	2.52	1.89	25	97	4.85	3.88	2.91
9	67	3.35	2.68	2.01	26	98	4.90	3.92	2.94
10	70	3.5	2.80	2.10	27	99	4.95	3.96	2.97
11	73	3.65	2.92	2.19	28	100	5.00	4.00	3.00
12	75	3.75	3.00	2.25	30	102	5.10	4.08	3.06
13	77	3.85	3.08	2.31	40	110	5.50	4.40	3.30
14	79	3.95	3.16	2.37	50	115	5.75	4.60	3.45
15	81	4.05	3.24	2.43	60	120	6.00	4.80	3.60
16	83	4.15	3.32	2.49	70	125	6.25	5.00	3.75
17	85	4.25	3.34	2.55	80	129	6.45	5.16	3.87
18	87	4.35	3.48	2.61	90	131	6.55	5.24	3.93
19	89	4.45	3.56	2.67	100	133	6.70	5.40	4.00

5.1.1.D Modulus of Elasticity

The modulus of elasticity shall be determined as specified in AASHTO LRFD Section 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 (*K1*) shall normally be taken as 1.0.

5.1.1.E Shrinkage and Creep

Shrinkage and creep shall be calculated in accordance with AASHTO LRFD Section 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.

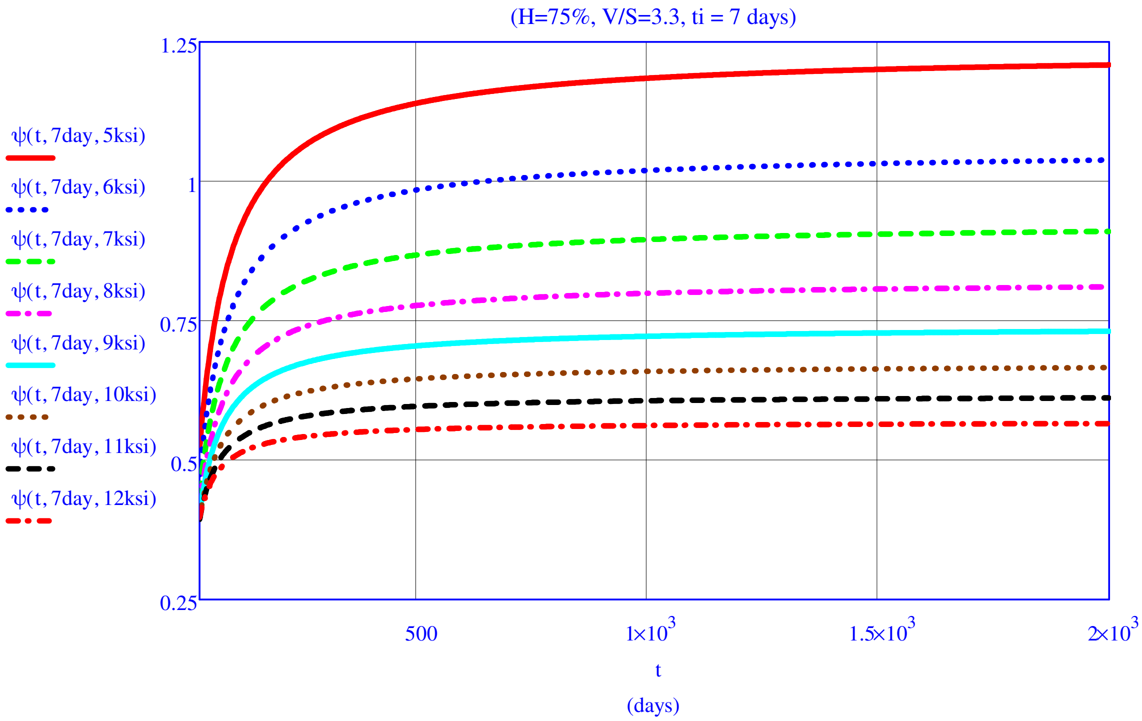
In determining the maturity of concrete at initial loading, *ti*, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.

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′ci*, as a function of time from initial seven day steam cure (*ti* = 7 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.

Figure 5.1.1-1 Creep Coefficient for Standard Conditions as Function of Initial Concrete Strength



5.1.1.F Shrinkage

Concrete shrinkage strain, εsh, shall be calculated in accordance with AASHTO LRFD.

5.1.1.G Grout

Grout is usually a prepackaged cement based grout or non-shrink 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. Non-shrink 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).

5.1.1.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 of Record.

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.

5.1.1.I Self-Consolidating Concrete (SCC)

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

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. Designers should verify the availability of SCC mixes near the job-site.

5.1.1.J Shotcrete

Shotcrete is a pneumatically applied cementitious material that is distinct from conventional cast-in-place and precast concrete. Shotcrete shall only be specified for elements that are explicitly allowed. Shotcrete shall only be considered where it can conform to the aesthetic requirements of the project or corridor. The State Bridge Architect shall be consulted for approval if applicable aesthetic requirements are not established. Shotcrete may be used for:

- Noise barrier walls (i.e. free-standing walls excluding retaining walls)
- Fully supported fascia for retaining walls such as geosynthetic walls, secant pile walls or geofoam walls.
- Fascia for concrete shaft walls where soil arching exists and the clear spacing between shafts does not exceed one-quarter of the shaft diameter or 2 ft, whichever is less.
- Gravity walls (i.e. walls that are not reliant on internal reinforcing for strength)
- Non-structural liners for tunnels and steel plate arch buried structures.
- Rehabilitation of existing structures with restricted overhead clearance (e.g. column jacketing, re-facing of pier walls, etc.)
- Shotcrete when specified in WSDOT *Standard Plans* as long as the facing conforms to the aesthetic requirements of the project or corridor.
- Temporary structures and temporary elements of permanent structures (e.g. cut facing for soil nail wall construction.
- Slope protection and slope face stabilization.

Shotcrete shall not be used for:

- Discretely supported fascia for retaining walls (e.g. soldier pile walls)
- Permanent fascia for anchored walls (e.g. soldier pile tieback walls, soil nails walls)
- Elements (e.g. fascia) that directly resist earth pressure from bridge abutments and piers.
- Traffic barriers and traffic barrier shapes (e.g. differential grade barriers)
- Structures that cannot be constructed in accordance with aesthetic requirements of the project or corridor. This includes most standard concrete finishes such as fractured fin, random board, etc.
- Walls with carved fascia where concrete spalling poses a safety risk to pedestrian or other facilities below the wall. This condition may exist where concrete cover exceeds 4".

Designers are discouraged from specifying shotcrete:

- In marine environments
- For elements subjected to significant saturation and freeze-thaw cycles.
- Deep/heavily reinforced elements.

For shotcrete structures any steel reinforcing or components within 6" of the shotcrete surface shall be galvanized. Epoxy- coated reinforcing could be acceptable if test panels can demonstrate that the shotcreting process does not damage the epoxy coating.

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:

- **Permeability** – 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*, 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.

5.1.1.K Lightweight Aggregate Concrete

Lightweight aggregate concrete shall not be used on bridge decks or other components exposed to traffic wheel loads in service. Lightweight concrete may be used for bridge decks protected by a normal-weight concrete or modified concrete overlay.

5.1.1.L Concrete Cover to Reinforcement

Concrete cover to reinforcement shall conform to AASHTO LRFD Section 5.10.1.

5.1.1.L.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 Section 5.10.1. 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.

5.1.1.L.2 *Concrete Exposed to Salt/Seawater*

Salt/sea water can be an aggressive corrosive environment that significantly shortens the service life of reinforced concrete structures. ACI 201.2R 7.2.1 provides some guidance on severity of exposure: “The severity of marine exposures can vary greatly within a given concrete structure. In general, continuous submersion is the least aggressive exposure. Areas where capillary suction and evaporation are prevalent are the most aggressive because these processes tend to increase the concentration of salts. Examples of such exposures include reclaimed coastal areas with foundations below saline groundwater level, intertidal zones, and splash zones. ”Corrosive water or soil contains greater or equal to 500 part per million (ppm) of chlorides. Sites that are considered corrosive due soley to sulfate content greater than or equal to 2,000 ppm and/or a pH of less than or equal to 5.5 should be considered non-corrosive in determining minimum cover.

Designers shall provide the minimum cover specified in AASHTO LRFD Table 5.10.1-1 to concrete structures with direct exposure to salt/sea water such as the Pacific Ocean and the Puget Sound. However, use of other corrosion mitigation strategies described in ACI 201.2R 7.2.3 and ACI 357.3R could be used to reduce this cover or provide additional protection such as minimizing concrete permeability, using corrosion resistant reinforcement, cathodic protection, treatments that penetrate or are applied on the surface of the concrete to slow the entry of chloride ions, etc. Site shall be classified as Coastal if they conform to the requirements of a marine site in accordance with Section 6.7.1.

5.1.1.M Ultra-High Performance Concrete (UHPC)

Ultra-high performance concrete is allowed for field cast-connections between precast elements. It may be used for repairs, overlays or other uses with State Bridge Design Engineer approval. WSDOT has funded two research projects with the Washington State University and the University of Washington studying the connection of wide flange deck girders using UHPC. The material studied is a high strength, high bond, fiber reinforced, flowable concrete capable of developing non-contact lap splices in a short distance. The material studied does not provide the same properties as common prepackaged commercial UHPC products, but it is capable of developing compact field connections between precast elements using locally available materials.

5.1.2 Reinforcing Steel

5.1.2.A Types and Grades

Steel reinforcement conforming to ASTM A 706 provides controlled ductility and enhanced weldability. Steel reinforcement for cast-in-place components and precast substructure components of bridges shall conform to ASTM A 706 unless noted otherwise. Steel reinforcement for precast bridge superstructure components, precast Buried Structures, retaining walls, barriers and other structures not designed for ductile seismic behavior shall conform to either ASTM A 706 or AASHTO M31 (ASTM A615). Steel reinforcement that is welded shall conform to ASTM A 706.

Grade 60 is the preferred grade for most components and structures. Grade 80 high-strength reinforcing steel may be used selectively to reduce congestion, reduce weight, speed up installation and/or reduce cost where its use is permitted and economical. Bridge decks, crossbeams, spread footings and foundation caps are components where Grade 80 longitudinal reinforcement could be economical. See Section 4.2.20 for additional seismic design requirements.

Designers should consider the need for additional development length when using high strength reinforcing steel. For improved economy, designers should minimize the number of different bar sizes on a job which use high-strength reinforcement. Where high-strength steel reinforcement is used in combination with Grade 60 reinforcement, designers should avoid specifying higher grades for bar sizes that Grade 60 is specified. This practice prevents confusion and improper installation on site. Mechanical couplers are available for high-strength reinforcement, but splices should be staggered and located in regions of low stress.

Transverse steel reinforcement for shear and torsion with a yield strength, f_y, in excess of 75 ksi shall use 75 ksi for resistance calculations. The limit of 75 ksi is intended to maintain the concrete's effectiveness in resisting shear by limiting the size of diagonal cracks that develop.

5.1.2.A.1 *Corrosion Resistant Reinforcement*

Corrosion resistant reinforcing such as stainless steel, chromium steel, galvanized steel or epoxy-coated steel may be used where added corrosion protection is needed. Glass fiber reinforced polymer (GFRP) bars may be used with State Bridge Design Engineer approval.

Epoxy-coated reinforcing is a preferred and economical method of enhanced corrosion protection compared to uncoated steel reinforcing. See Section 5.7.4 for use in Bridge Deck Protection Systems. The epoxy-coating provides an inert barrier to water and chlorides as long as the coating remains bonded and intact. It is important that the epoxy coating be protected from prolonged UV exposure and damage from handling during construction. Epoxy-coated reinforcement typically requires touch-up coating in the field after installation and prior to concrete placement. Field bending should be avoided if possible because it can damage the coating. Epoxy-coated reinforcement also requires longer development and splice lengths that must be considered in design. This may present a challenge when used in compact joints and/or with lightweight concrete. Plans shall designate bar marks with "E".

Galvanized steel reinforcing shall conform to either ASTM A767 Class 1 or ASTM A1094. Galvanized reinforcement provides a level of corrosion resistance similar to if not greater than epoxy-coated reinforcement. Additionally galvanized reinforcement provides a durable coating that will resist damage during handling better than epoxy-coated reinforcement. Galvanized reinforcement can be manufactured using a hot-dip process (ASTM A767) or a continuously galvanized process (ASTM A1094), which are presumed to provide comparable corrosion resistance. Hot-dip galvanized coatings are typically thicker, but continuously galvanized coatings have a higher pure zinc content. Designers should generally specify both types as alternates to provide flexibility in the supply. Galvanized reinforcement requires the same development and splice lengths as uncoated steel, but the of size of hooks is larger for certain bar sizes to accommodate bar that is hot-dipped after fabrication (bending). Chromate treatment of galvanized steel shall be left as optional for the contractor. Plans shall designate bar marks with “G”.

Stainless steel reinforcing shall conform to ASTM A955, UNS S24100, UNS S31653 or UNS31803. Plans shall designate bar marks with "SS".

Corrosion resistant chromium alloyed reinforcing such as ChromX® (alternatively MMFX) shall conform to ASTM A1035 Type CM or CS. Type CS (with 9% chromium and a higher level of corrosion resistance) should be used where corrosion resistance beyond that of epoxy-coated reinforcing is desired. This reinforcement may be used in the design of bridge decks, substructure and foundation members where rebar congestion is a concern or where corrosion protection is needed. This type of reinforcement is high strength and comes with higher initial cost, but it may be cost-effective over the life of a structure if the high strength can be used to reduce the total volume of reinforcement. Contract documents should avoid referring to reinforcement by trade names, such as ChromX® or MMFX. Plans shall designate bar marks with “CR”.

GFRP reinforcing shall conform to the requirements of ASTM D7957. This type of reinforcing bar may be used in bridge decks in any Seismic Design Category, and in crossbeams and foundations in Seismic Design Category A. Design shall be in accordance with *AASHTO LRFD Bridge Design Guide Specifications for GRFP-Reinforced Concrete*, 2nd Edition. Because GFRP reinforcement is vulnerable to damage by mechanical and hydro-demolition equipment, GFRP reinforcement is not ideal for bridge decks with high traffic volume and bridge decks with the potential for future bonded overlays. Designers should consider that GFRP-reinforced structures lack ductility, and that bends in GFRP bars create points of weakness. So GFRP reinforcement is best suited for flat work where ductility is not essential. Plans shall designate bar marks with "GF".

Engineers shall minimize the potential for dissimilar metal corrosion when combining different types of reinforcing steel in a structure. Where corrosion resistant steel is used, specifications shall be provided to require industry standard best-practices for fabrication, handling, placing and protection.

5.1.2.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, 1⅛″ × 1⅛″ square bars and 1¼″ × 1¼″ square bars respectively. Similarly, the #14 and #18 bars correspond to 1½″ × 1½″ and 2″ × 2″ square bars, respectively.

5.1.2.C Development

5.1.2.C.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 in accordance with AASHTO LRFD Section 5.10.8.2.1.

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.

5.1.2.C.2 *Compression Development Length*

Development of reinforcement in compression shall be in accordance with AASHTO LRFD Section 5.10.8.2.2. 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″.

5.1.2.C.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 in accordance with AASHTO LRFD Section 5.10.8.2.4. Tension development lengths of 90° & 180° standard hooks are shown in Appendix 5.1-A6.

5.1.2.D Splices

The Contract Plans shall clearly show the locations and lengths of splices. Splices shall be in accordance with AASHTO LRFD Section 5.10.8.4.

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

5.1.2.D.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, l_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.

5.1.2.D.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.

5.1.2.D.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 Sections 5.5.3.4 and 5.10.8.4.2b.

5.1.2.D.4 *Welded Splices*

AASHTO LRFD Section 5.10.8.4.2c 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.

5.1.2.E Hooks and Bends

For hook and bend requirements, see AASHTO LRFD Section 5.10.2. Standard hooks and bend radii are shown in Appendix 5.1-A1 for steel reinforcing bars with yield strengths up to 100 ksi. Additional tie reinforcement may be required to anchor hooked bars when the yield strength exceeds 75 ksi.

When specifying and detailing galvanized reinforcing, designers should consider that larger bend diameters will be provided if the contractor elects to galvanize bars after fabrication. These diameters (see ASTM A 767) differ from standard CRSI requirements for 180 degree hooks on #7 and #8 bars and all stirrup/tie hooks.

When using GFRP bars, detailed bends should be avoided where possible. If bent bars are necessary, they should be coordinated with suppliers during design. Headed bar may be an acceptable alternative.

5.1.2.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.

Straight galvanized reinforcing bars should be limited to a 40’ maximum length.

Straight stainless steel reinforcing bars should be limited to a 40’ maximum length for #3-#18 bar.

Corrosion resistant reinforcing (ASTM A 1035) is available in 60’ lengths for #4-#18 bar.

GFRP reinforcing bars should be limited to a 40’ maximum length.

For some materials, longer bars are possible. But the designer should coordinate with suppliers during design prior to specifying them. Longer bars can increase lead time and/or limit suppliers. Optional lap splices should be provided at the recommended maximum bar length where possible.

5.1.2.G Placement

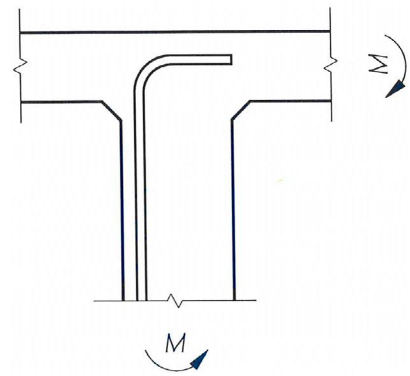
Placement of reinforcing bars can be a challenge during construction. If reinforcement is congested, as is common in column joints, additional details are recommended in the contract plans showing how each bar is placed. Appendix 5.1-A2 shows the minimum clearance and spacing of reinforcement for beams and columns. High-strength reinforcement is one possible method to reduce congestion.

5.1.2.H Joint and Corner Details

5.1.2.H.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°.

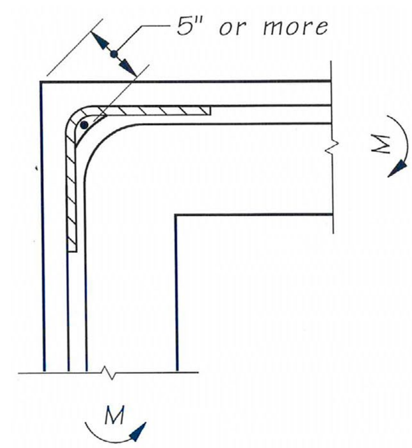
Figure 5.1.2-1 T-Joint Reinforcing Details



5.1.2.H.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.

Figure 5.1.2-2 "Normal" Right Corner Reinforcing Details



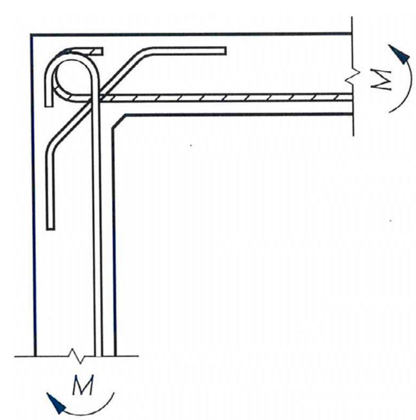
5.1.2.H.3 *Right or Obtuse Angle Corners*

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.

Figure 5.1.2-3 Right or Obtuse Angle Corner Reinforcing Details



5.1.2.I Welded Wire Reinforcement

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

Welded wire shall be deformed and shall conform to the requirements of AASHTO M336/ASTM A 1064. Epoxy-coated wire and welded wire reinforcement shall conform to *Standard Specifications* Section 9-07.3 with the exception that ASTM A884 Class A Type I shall be used instead of ASTM A775. Galvanized welded wire reinforcement shall conform to the requirements of ASTM A1060. Stainless steel welded wire reinforcement shall conform to the requirements of ASTM A1022.

Welded wire reinforcement shall be deformed. The specified minimum 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 Section 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.

5.1.2.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

5.1.3.A General

Three types of high-tensile steel used for prestressing steel are:

1. **Strands**
   AASHTO M 203 Grade 270, low relaxation
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 7⁄16″ 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 when 10 0.6” diameter temporary top strands are not sufficient for temporary handling cases.

Provide adequate concrete cover and consider use of epoxy coated prestressing reinforcement in coastal areas or where members are directly exposed to salt water.

5.1.3.B Stress Limits

Stress limits for prestressing steel are as listed in AASHTO LRFD Section 5.9.2.2.

5.1.3.C Prestressing Strands

Standard strand patterns for all types of WSDOT prestressed concrete girders are shown on the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm).

5.1.3.C.1 *Straight Strands*

The position of the straight strands in the bottom flange is standardized for each girder type.

5.1.3.C.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.

5.1.3.C.3 *Temporary Strands*

Temporary strands in the top flanges of prestressed concrete girders may be required for stability (see Section 5.6.3). These strands shall be considered to be pretensioned for design purposes, but may be post-tensioned at the manufacturer's discretion depending on pretensioning bed capacity. If the strands are post-tensioned, calculations shall be provided by the manufacturer considering the timing of post-tensioning (before or after lifting from the form). It is more efficient to post-tension the strands after removal of the girder from the form if they are not needed for stability during lifting. The calculations shall consider the effects on camber from any revisions to the concrete strength at transfer and timing of post-tensioning. In no case shall the temporary top strands be post-tensioned more than 48 hours after transfer of permanent pretensioning. These strands can be considered in design to reduce the required strength at release, to provide stability during handling and shipping, and to reduce the “A” dimension. These strands must be cut before the CIP intermediate diaphragms are placed.

5.1.3.D Development of Prestressing Strand

5.1.3.D.1 *General*

Development of prestressing strand shall be as described in AASHTO LRFD Section 5.9.4.3.

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

5.1.3.D.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.

Partially debonded strands shall meet the requirements of AASHTO LRFD Section 5.9.4.3.3.

5.1.3.D.3 *Strand Development Outside of Prestressed Concrete Girders*

Extended bottom prestressing strands are used to connect the ends of girders with diaphragms and resist loads from creep effects, shrinkage effects, positive moments due to seismic demand at fixed piers, and seismic connection forces at the abutments on single span bridges.

Extended strands must be developed in the short distance within the diaphragm. Strands shall be extended as far across the diaphragm as practical. And for fixed pier diaphragms or continuity diaphragms they shall be anchored at least 1'-9" from the girder end. The pattern of extended strands and embedded length of extended strands shall be sufficient to resist concrete breakout from the face of the crossbeam, while at the same time minimizing congestion. An explicit concrete breakout check may be unnecessary when all strands are effectively spliced across a crossbeam.

Strands shall be anchored with a strand chuck as shown in Figure 5.1.3-1. Strand chucks shall be a minimum 1$\frac{11}{16}$"ø barrel anchor or similar. The designer shall calculate the number of extended straight strands needed to develop the required moment capacity at the end of each girder. The number of extended strands shall not be less than four.

For fixed intermediate piers in Seismic Design Categories B-D at the Extreme Event I limit state, the girder anchorage with extended strands shall be sufficient to carry a calculated fraction of the plastic overstrength moment demand originating from the nearest column. The required number of extended strands, N_{ps}, for each girder shall be calculated using the following:

$$N_{ps} \geq \frac{M_{u,i}}{0.9\phi A_{ps} f_{py} d} \geq 4 \qquad (5.1.3\text{-}1)$$

Where:

$M_{u,i}$	=	Design moment at the end of each girder (kip-in)
A_{ps}	=	Area of each extended strand (in^2)
f_{py}	=	Yield strength of prestressing steel (ksi)
d	=	Distance from top of deck slab to c.g. of extended strands (in)
ϕ	=	Flexural resistance factor, 1.0

The design moment at the end of each girder shall be calculated using the following:

$$M_{u,i} = M_{g,i} - 0.9M_{SIDL} \tag{5.1.3-2}$$

Where:

Mg,i = The moment demand due to column plastic overstrength in girder i caused by the longitudinal seismic demands (kip-in)

MSIDL = Moment demand due to super imposed dead loads (traffic barrier, sidewalk, etc.) per girder (k-in.)

For spliced prestressed concrete girders, where post-tensioning tendons are installed over intermediate piers, $M_{u,i}$ shall be modified to account for induced moments.

The moment demand due to column plastic overstrength in each girder shall either be determined from the table in Appendix 5.1-A9 or Equation 5.1.3-3. This methodology assumes half the column plastic overstrength moment is resisted by the girders on each side of the column.

$$M_{g,i} = KM_{CG}\frac{sinh\left(\frac{\lambda L_{cb}}{2N_L}\right)}{sinh(\lambda L_{cb})}cosh\left[\lambda L_{cb}\left(1 - \frac{L_{cb,i}}{L_{cb}}\right)\right] \tag{5.1.3-3}$$

Where:

K = Span moment distribution factor. If the span lengths differ, the moment contribution to each span should be modified in accordance with the span lengths, using K_1 and K_2 as shown in Figure 5.1.3-2; otherwise K = 0.5.

MCG = Moment generated by a single column due to the column plastic overstrength and acting at the center of gravity of the superstructure. See Equation 5.1.3-4 (kip-in.)

Lcb,i = Distance from the centerline of nearest column to centerline of the girder (ft.)

λ*Lcb* = Ratio of total stiffness of all girders (within a half column spacing or overhang) to torsional stiffness of half the total length of the crossbeam or half the column spacing. See Equation 5.1.3-5.

Lcb = Half of the crossbeam length for single column bents, or half the column spacing or overhang length for multi-column bents (ft.)

NL = The number of contributing girder lines taken as L_{cb}/S.

S = Girder spacing (ft.)

The moment demand at the center of gravity of the superstructure for each column shall be calculated using the following:

$$M_{CG} = M_{po}^{top} + \frac{M_{po}^{top} + M_{po}^{base}}{L_C}h \tag{5.1.3-4}$$

Where:

M_{po}^{top} = Plastic overstrength moment at top of column, kip-in

M_{po}^{base} = Plastic overstrength moment at base of column (kip-in.)

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 moment (ft.)

The total girder stiffness to crossbeam stiffness ratio shall be calculated using the following:

$$\lambda L_{cb} = \sqrt{\left(\frac{\alpha EI}{L_g}\right)\frac{2N_L}{(GJ/L_{cb})}} \quad (5.1.3\text{-}5)$$

Where:

	=	3 for girders in which far end is free to rotate (expansion piers); and 4 for girders in which far end is fixed against rotation (continuous piers).
EI	=	Flexural stiffness of one girder, including composite deck (kip-in.^{2})
GJ	=	Torsional stiffness of the crossbeam cross-section (kip-in^2)
L_g	=	Girder span length if girders frame into the crossbeam from only one side;
	=	$\frac{2}{\left(1/L_1 + 1/L_2\right)}$, if girders frame into the crossbeam from both sides, where L_1 and L_2 are individual girder span lengths (ft.)

For dropped (two-stage) prismatic crossbeams, the moment distribution is likely to be nearly uniform. For raised (flush) crossbeams, it is likely that λL_{cb} will be > 1.0 and the moment distribution will not be uniform. For tapered crossbeams, Equation 5.1.3-2 may be used if the torsional stiffness is initially defined by the deepest section of the crossbeam, and λL_{cb} is then increased by 20%. This will lead to a less uniform distribution of girder moments than that found with a prismatic crossbeam.

A slight downwards adjustment in the number of extended strands for an individual girder is acceptable if the sum of the adjusted total moment resistance is greater than the ideal total moment resistance. Girders closer to the pier columns shall not have fewer strands than the ideal number required. When girder designs in a span are otherwise identical, the pattern and number of extended strands should also be identical, using the largest number of strands required for any girder.

For cases with uneven girder spacings or girders centered on columns, the designer shall verify that the total combined moment resistance of all girders within the tributary region of the column is greater than the total moment demand at the superstructure CG minus the total factored superimposed dead load moments.

Figure 5.1.3-1 Strand Development

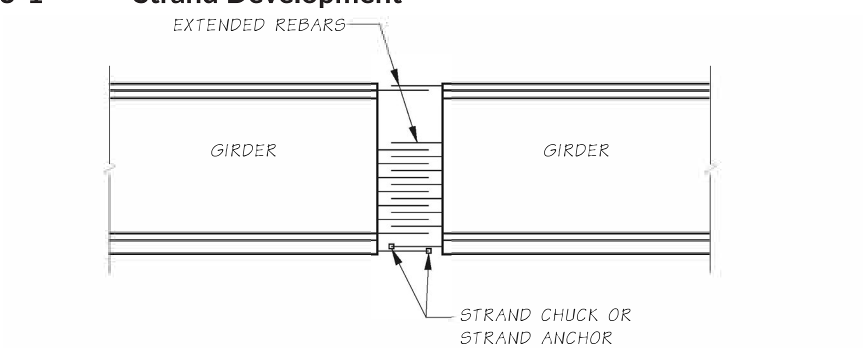
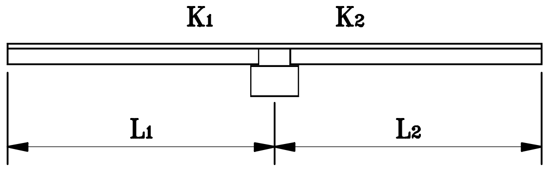
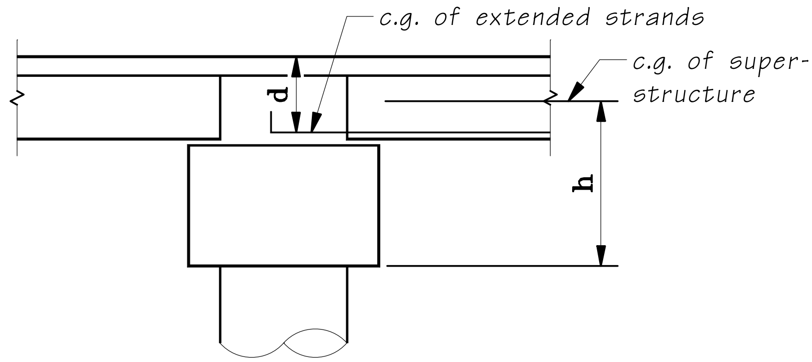


Figure 5.1.3-2 Extended Strand Design



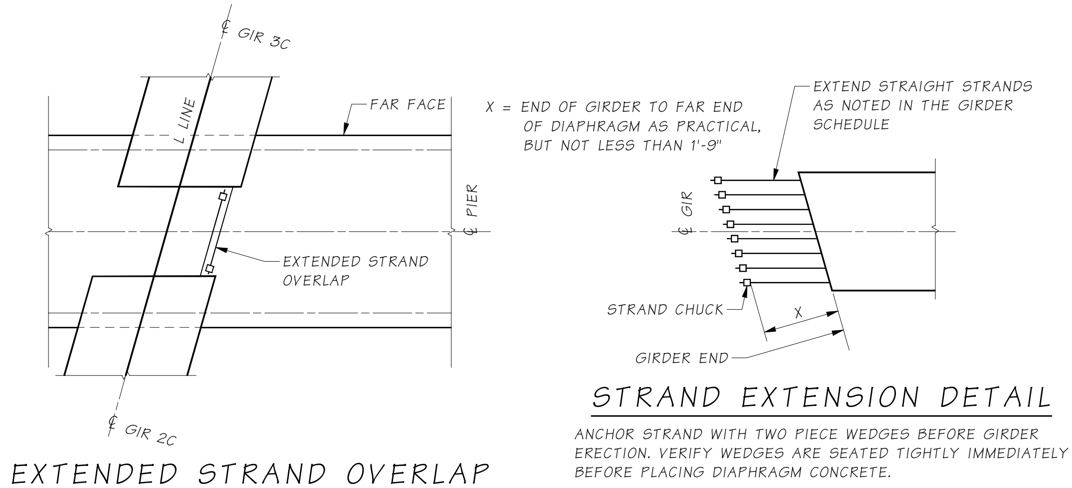
$$K_1 = \frac{L_2}{L_1 + L_2} \qquad K_2 = \frac{L_1}{L_1 + L_2}$$



Anchorage of extended strands is essential for all prestressed concrete girder bridges with fixed diaphragms at intermediate piers. Extended strand anchorage may be achieved by directly overlapping extended strands, by use of strand, by the use of the crossbeam ties 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:

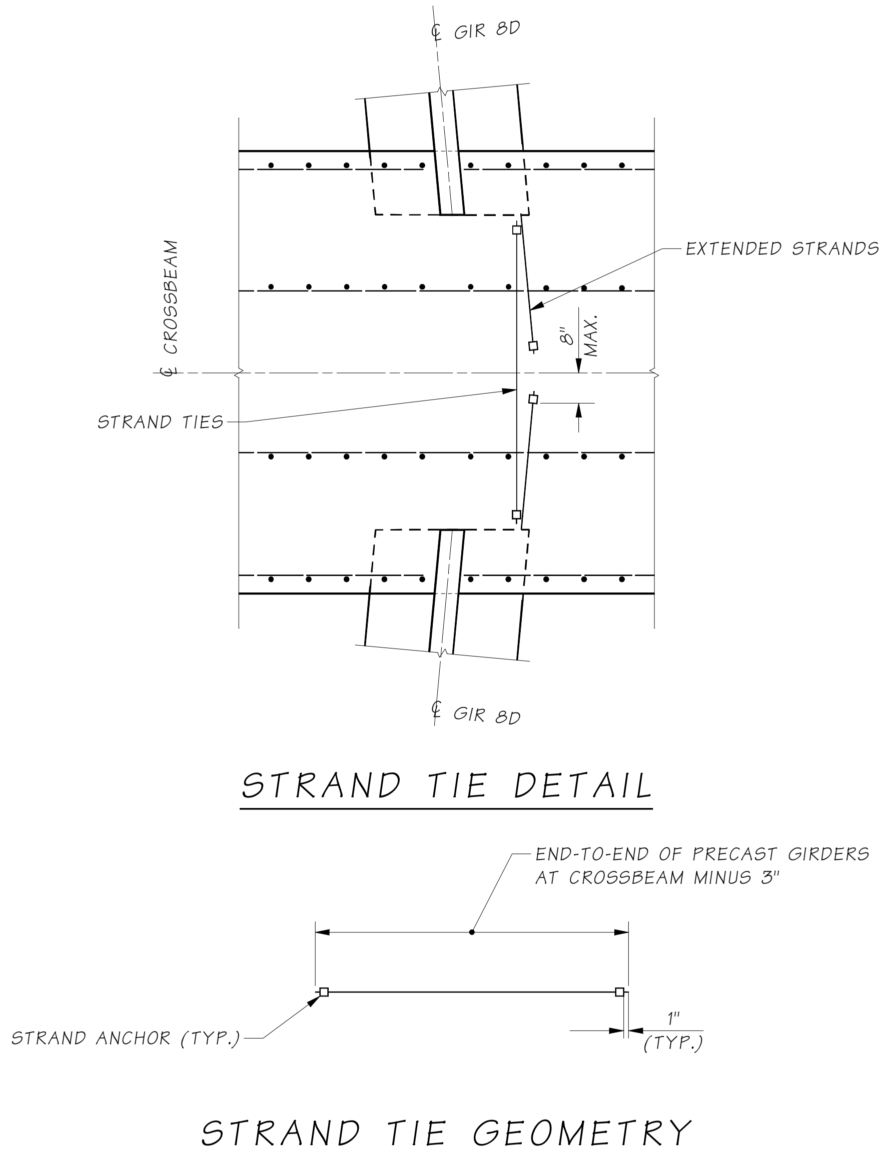
Method 1 – Direct extended strands overlapping shall be used at intermediate piers without any angle point due to horizontal curvature and for any crossbeam width. This is the preferred method of achieving extended strand continuity. Congestion of reinforcement and girder setting constructability shall be considered when large numbers of extended strands are required. In these cases, strand ties may be used in conjunction with extended strands. See Figure 5.1.3-3

Figure 5.1.3-3 Overlapping 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 feet 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 5.1.3-6. See Figure 5.1.3-4.

Figure 5.1.3-4 Stand Ties



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 5.1.3-6. If this limit is exceeded, the geometry of the width of the crossbeam shall be increased to provide sufficient lap for the strand ties. See Figure 5.1.3-5.

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

$$A_s = \frac{1}{2}\left(\frac{A_{ps}f_{py}N_{ps}}{f_{ye}}\right) \tag{5.1.3-6}$$

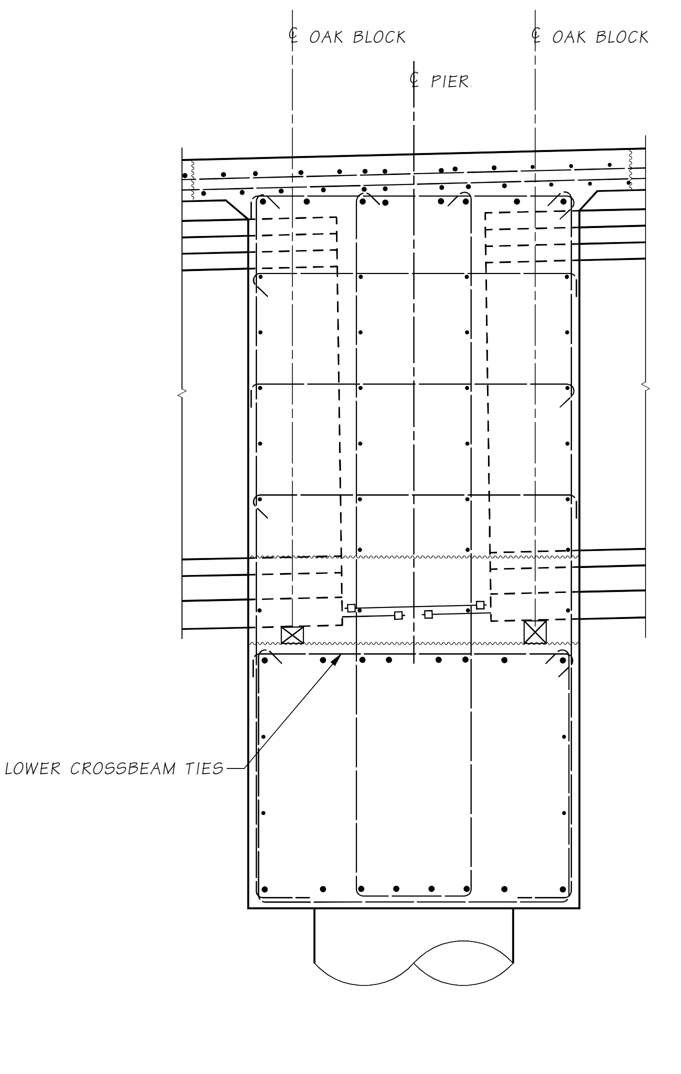
Where:

Aps = Area of strand ties (in^2)
fpy = Yield strength of extended strands (ksi)
Nps = Number of extended strands that are spliced with strand and crossbeam ties
fye = Expected yield strength of transverse tie reinforcement (ksi)

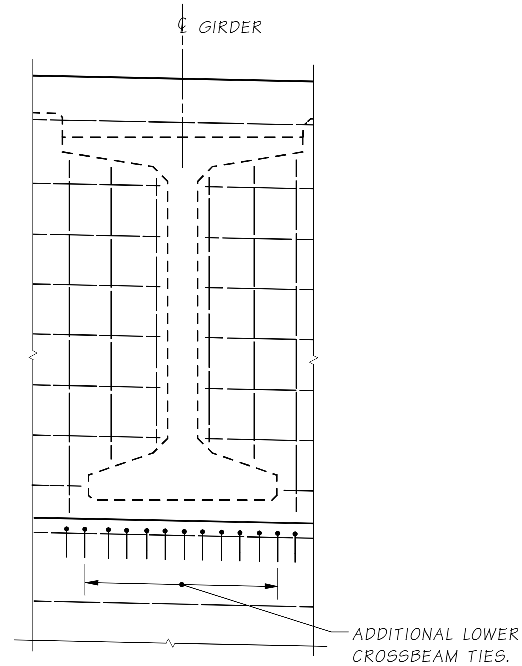
Two-thirds of A_s shall be placed directly below the girder and the remainder of A_s shall be placed outside the bottom flange width as shown in Figure 5.1.3-5.

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.

Figure 5.1.3-5 Lower Crossbeam Ties



INTEGRAL DIAPHRAGM DETAIL



CROSSBEAM TIE DETAIL

LOWER CROSSBEAM STIRRUPS NOT SHOWN FOR CLARITY

5.1.4 Prestress Losses

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

5.1.4.A Instantaneous Losses

5.1.4.A.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 in accordance with AASHTO LRFD Section 5.9.3.2.3.

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

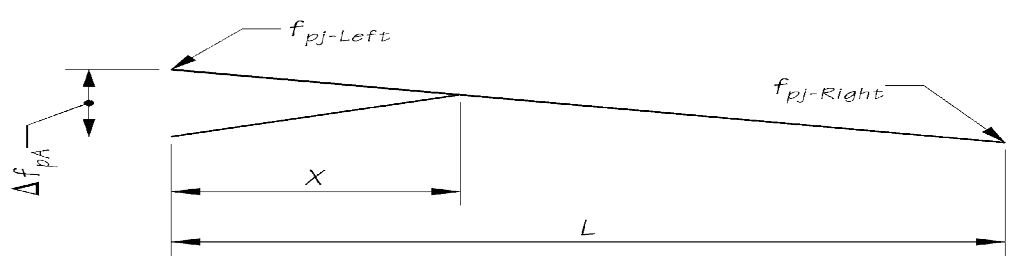
5.1.4.A.2 Anchorage Set Loss

The anchor set loss for multi-strand tendons should be based on ⅜″ slippage for design purposes. For long tendons where the stress along the tendon at jacking may be approximated as linear, anchor set loss and the length affected by anchor set loss may be calculated as shown in Figure 5.1.4-1.

$$x = \sqrt{\frac{\Delta_{set} A_{PT} E_p L}{P_{j\text{-}left} - P_{j\text{-}right}}} \tag{5.1.4-1}$$

$$\Delta f_{pA} = \frac{2x\left(P_{j\text{-}left} - P_{j\text{-}right}\right)}{A_{PT} L} \tag{5.1.4-2}$$

Figure 5.1.4-1 Anchorage Set Loss



5.1.4.A.3 *Friction Losses*

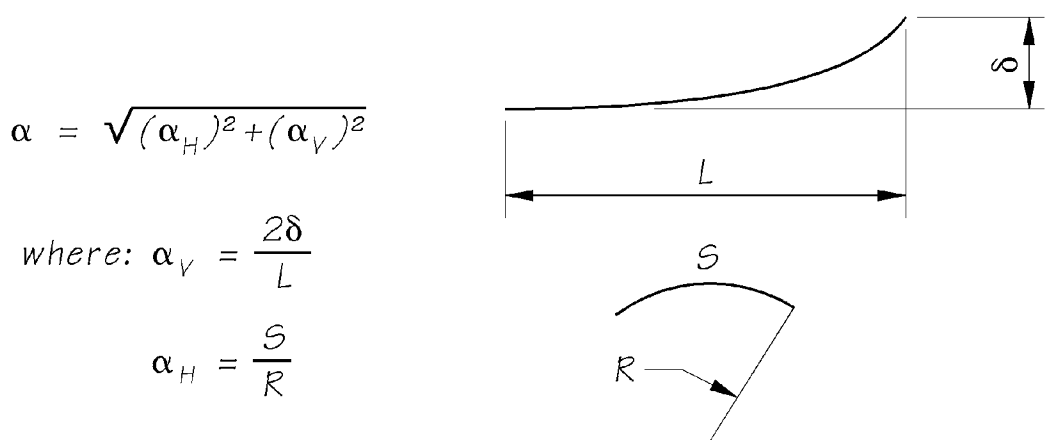
Friction losses occurring during jacking and prior to anchoring depend on the system and materials used. For a rigid spiral galvanized ferrous metal duct system, μ shall be 0.20 and K = 0.0002. For plastic ducts, the designer shall use the values shown in AASHTO LRFD Table 5.9.3.2.2b.

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.

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(kx+\mu\alpha)}\right) \tag{5.1.4-3}$$

When summing the α 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 α angles for horizontally and vertically curved tendons are shown in Figure 5.1.4-2.

Figure 5.1.4-2 The α Angles for Curved PT Tendons



5.1.4.B Approximate Estimate of Time-Dependent Losses

The Approximate Estimate of Time-Dependent Losses of AASHTO LRFD Section 5.9.3.3 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.

5.1.4.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 Section 5.9.3.4.

5.1.4.D Total Effective Prestress

For standard precast, pre-tensioned members with CIP deck subject to normal loading and environmental conditions and pre-tensioned with low relaxation strands, the total effective prestress may be estimated as:

$$f_{pe} = f_{pj} - \Delta f_{pT} - \Delta f_{pES} - \Delta f_{pED} - \Delta f_{pSS} \tag{5.1.4-4}$$

The total prestress loss may be estimated as:

$$\Delta f_{pT} = \Delta f_{pRO} + \Delta f_{pLT} \tag{5.1.4-5}$$

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

$$\Delta f_{pRO} = \frac{log(24t)}{40}\left(\frac{f_{pj}}{f_{py}} - 0.55\right) f_{pj} \tag{5.1.4-6}$$

Where:

t = Duration of time between strand stressing and prestress transfer, typically 1 day
fpj = Jacking stress
fpy = Yield strength of the strand

Long term time dependent losses, Δ*fpLT*, are computed in accordance with the refined estimates of AASHTO LRFD Section 5.9.3.4 or a detailed time-step method. Elastic gain due to deck shrinkage shall be considered separately.

Elastic shortening, Δ*fpES*, is computed in accordance with AASHTO LRFD Section 5.9.3.2.3a.

The elastic gain due to deck placement, superimposed dead loads and live loads is taken to be:

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

Where:

Ep = Modulus of elasticity of the prestressing strand
Ec = Modulus of elasticity of the concrete at the time of loading
Mslab = Moment caused by deck slab placement
Mdiaphragms = Moment caused by diaphragms and other external loads applied to the non-composite girder section
Msidl = Moment caused by all superimposed dead loads including traffic barriers and overlays
M*LL+IM* = Moment caused by live load and dynamic load allowance
γ*LL* = Live load factor (1.0 for Service I and 0.8 for Service III)
eps = Eccentricity of the prestressing strand
Ig = Moment of inertia of the non-composite girder
Ic = Moment of inertia of the composite girder
Ybg = Location of the centroid of the non-composite girder measured from the bottom of the girder
Ybc = Location of the centroid of the composite girder measured from the bottom of the girder

The elastic gain due to slab shrinkage, $\Delta fpSS$, shall be computed in accordance with AASHTO LRFD Section 5.9.3.4.3d. 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 percent of the strain determined by AASHTO LRFD Equation 5.4.2.3.3-1.

5.1.4.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.

5.1.4.E.1 *Lifting of Girders From Casting Beds*

For normal construction, forms are stripped and girders are lifted from the casting bed within one day.

5.1.4.E.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 Section 5.9.3.3, the losses at the time of hauling may be estimated by:

$$\Delta f_{pTH} = \Delta f_{pRO} + \Delta f_{pES} + \Delta f_{pH} \tag{5.1.4-8}$$

Where:

$\Delta fpTH$ = total loss at hauling

ΔfpH = time dependent loss at time of hauling =

$$3\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 3\gamma_h\gamma_{st} + 0.6$$

5.1.4.E.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.

5.1.4.E.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 Post-tensioning Systems

Multistrand grouted tendons with steel strand are the preferred system for post-tensioned concrete bridge superstructures, spliced girders, and bridge components. For post-tensioned concrete bridge decks, unbonded single strand post tensioning systems may be used. The use of other post-tensioning systems and materials shall require the approval of the State Bridge Design Engineer.

Multistrand and grouted post-tensioning systems for permanent construction shall be designed and constructed in accordance with Protection Level 2 (PL-2) practices, as defined by the requirements of PTI/ASBI M50.3-19 *Specification for Multistrand and Grouted Post-Tensioning* and PTI M55.1-19 *Specification for Grouting of Post-Tensioned Structures*. Unbonded single strand post-tensioning systems shall be designed and constructed in accordance with PTI M10.2-17 *Specification for Unbonded Single Strand Tendons*.

Designers should consult post-tensioning system supplier product guides to ensure that multiple suppliers can satisfy the design. Corrugated plastic duct shall be used and shall conform to standard sizes where possible. Common post-tensioning system component sizes and combinations are shown in Table 5.1.5-1. The nominal diameter for plastic duct may be used when interpreting code provisions that reference duct size or diameter without further clarification (i.e. inside diameter, outside diameter, etc.). The interior diameter of plastic duct shall be used when interpreting code provisions that reference duct area without further clarification.

Table 5.1.5-1 Post-tensioning System Sizes

Duct Size (Round)	ID	OD	Corrug. Dia.	Max # Strands (Pull, 0.6”ø)	Max # Strands (Push, 0.6”ø)	Common Anchorage Size (0.6”ø)
3"	2.99”	3.19”	3.63”	12	16	12
3-⅜"	3.35”	3.55”	3.94”	16	20	15, 19
4"	3.93”	4.29”	4.63”	22	27	19, 22
4-½"	4.49”	4.80”	5.28”	29	36	27

Dead end anchorages, defined as anchorages that are not accessible during stressing, shall be avoided where possible. Anchorage pourback materials shall be specified in the plans and should be concrete where possible. Epoxy grout may be used in exposed areas or where improved bond is needed. Non-shrink grout may be specified for small protected pourbacks where proper grout confinement is provided.

5.2 Design Considerations

5.2.1 Service and Fatigue Limit States

5.2.1.A General

Service limit state is used to satisfy stress limit, 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.

5.2.1.B Control of Cracking

Reinforcement shall be provided and spaced to meet the requirements in AASHTO LRFD Section 5.6.7 “Control of Cracking by Distribution of Reinforcement.” The exposure factor shall be based upon a Class 2 exposure condition.

5.2.1.C Stress Limits in Prestressed Concrete Members

Allowable concrete stresses for the service and fatigue limit states are shown in Table 5.2.1-1. For prestressed concrete girders, the concrete stress limits shall be satisfied at all pre-service stages of girder construction and in service in accordance with Section 5.6.2.C. The tensile stress in the precompressed tensile zone for the final service load condition (Service III) is 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.

The controlling locations for temporary compressive stress with and without lateral bending are shown in Figure 5.2.1-1.

For tensile stress limits that require bonded reinforcement sufficient to resist the tensile force in the concrete, the tensile force shall be computed using the procedure illustrated in Figure 5.2.1-2 assuming an uncracked section. The bonded reinforcement is proportioned using a stress of 0.5fy, not to exceed 30 ksi. Individual reinforcing bars are only considered if they are fully developed and are located within the tensile stress region of the member.

The variable λ is the concrete density modification factor calculated in accordance with AASHTO LRFD Section 5.4.2.8.

For precast prestressed segments that are continuous over supports, such as in spliced girders, the tensile stress limits at service limit states shall apply for the precast segment, but need not be applied to a CIP bridge deck.

Table 5.2.1-1 Stress Limits in Prestressed Concrete Members

Condition	Stress	Location	Stress Limit (ksi)
Temporary Stress at Transfer and Lifting from Casting Bed	Tensile	In areas without bonded reinforcement sufficient to resist the tensile force in the concrete	$0.0948\lambda\sqrt{f'_{ci}} \leq 0.2$
		In areas with bonded reinforcement sufficient to resist the tensile force in the concrete	$0.24\lambda\sqrt{f'_{ci}}$
	Compressive	All areas (except as noted below)	$0.65f'_{ci}$
		At section extremities (i.e. flange tips) during handling when lateral bending is explicitly considered	$0.70\ f'_{ci}$
Temporary Stress at Shipping and Erection	Tensile	In areas without bonded reinforcement sufficient to resist the tensile force in the concrete	$0.0948\ \lambda\ \sqrt{f'_c}$ (*ksi*)
		In areas with bonded reinforcement sufficient to resist the tensile force in the concrete	$0.24\lambda\sqrt{f'_c}$
	Compressive	All areas (except as noted below)	$0.65f'_c$
		At section extremities (i.e. flange tips) during handling when lateral bending is explicitly considered	$0.70\ f'_c$
Final Stresses at Service Limit State	Tensile	All Locations	0.0
	Compressive	All areas due to effective prestress and permanent loads	$0.45f'_c$
		All areas, due to effective prestress, permanent loads and transient (live) loads	$0.60f'_c$
Final Stresses at Fatigue Limit State	Compressive	All areas due to the Fatigue I Load Combination plus one-half the sum of effective prestress and permanent loads in accordance with AASHTO LRFD Section 5.5.3.1	$0.40f'_c$

Figure 5.2.1-1 Temporary Compressive Stress Limits With and Without Lateral Bending

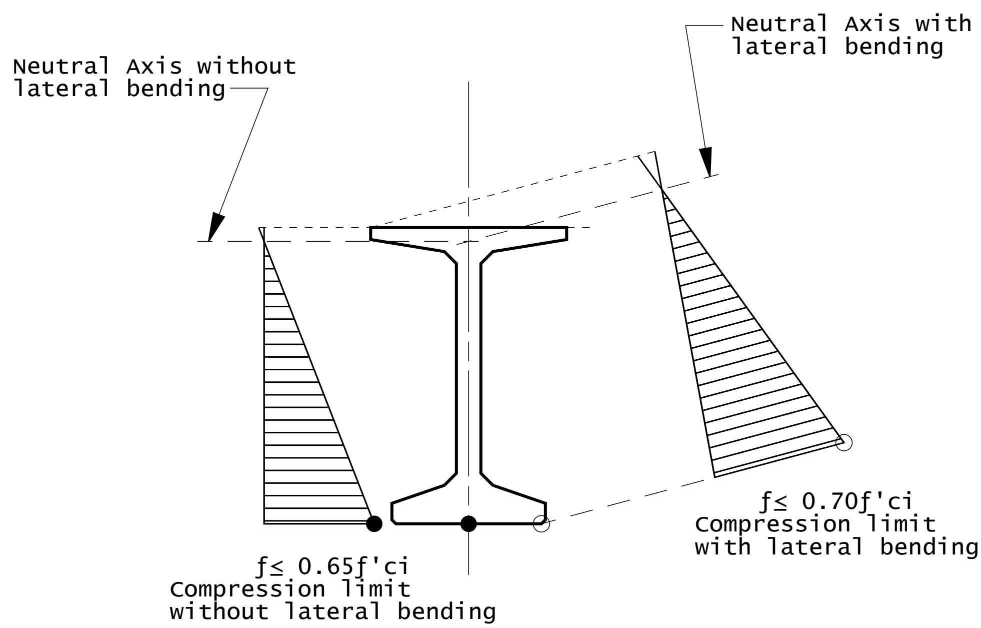
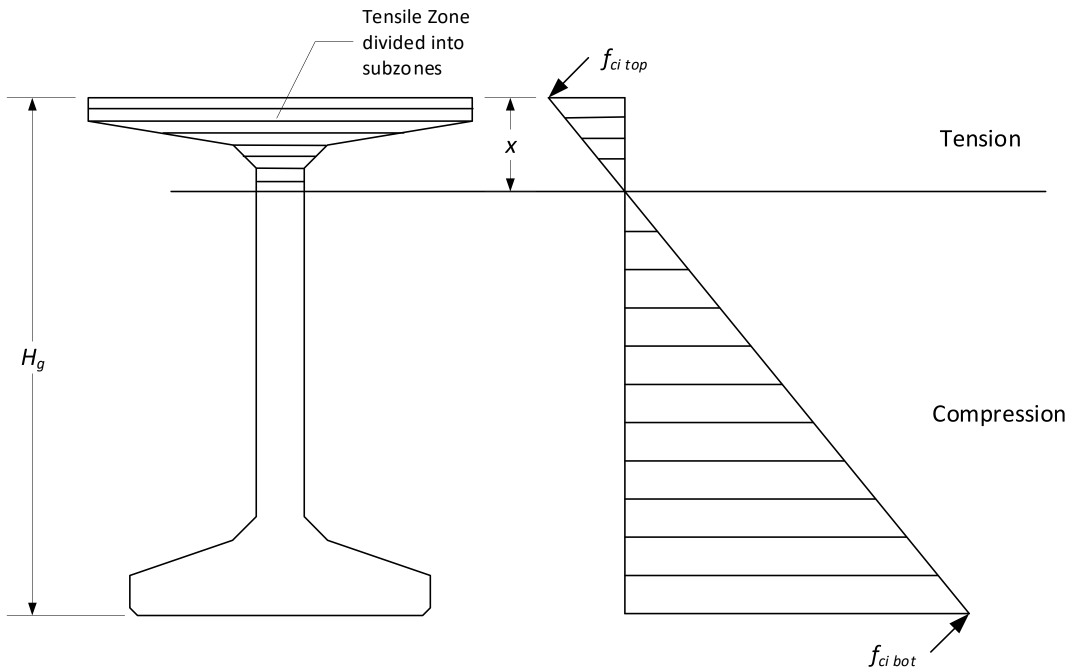


Figure 5.2.1-2 Bonded Reinforcement for Increased Temporary Tensile Stress Limit



$$T = \int f_{ci}(x)dA \approx \sum T_i = \sum f_{ci}(x_i)A_i$$

$$A_s = \frac{T}{f_s}, where\ f_s = 0.5f_y \leq 30.0\ ksi$$

5.2.2 Strength-Limit State

5.2.2.A Flexure

Design for flexural force effects shall be in accordance with AASHTO LRFD Section 5.6.

For prestressed concrete girders, the approximate methods of AASHTO LRFD Section 5.6.3 underestimate the flexural strength of the composite deck-girder system[2, 23]. Strain compatibility approaches such as the PCI *Bridge Design Manual*[3] method (PCI BDM Section 8.2.2.5) and the Nonlinear Strain Compatibility Analysis method in the PCI Journal[23] 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.

5.2.2.A.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:

$$M_u = \phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \tag{5.2.2-1}$$

However, if:

$$a = \frac{A_s f_y}{\propto_1 f_c' b} \tag{5.2.2-2}$$

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

$$A_s = \left(\frac{\propto_1 f_c' b}{f_y}\right)\left[d - \sqrt{d^2 - \frac{2M_u}{\propto_1 f_c' b \phi}}\right] \tag{5.2.2-3}$$

Where:

A_s = Area of tension reinforcement (in^2)
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)
Φ = 0.9
= From AASHTO LRFD Section 5.6.2.2

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.

$$\epsilon_t = 0.003 \left(\frac{d_t - c}{c}\right) \geq 0.005 \tag{5.2.2-4}$$

Where:

ε_t = Tensile strain in the extreme tension steel
d_t = Distance from extreme compression fiber to centroid of extreme tension reinforcement (in)
c = $\dfrac{A_s f_y}{\propto_1 f_c' b \beta_1}$
β_1 = From AASHTO LRFD Section 5.6.2.2

5.2.2.B Shear

AASHTO LRFD Section 5.7 addresses shear design of concrete members.

1. The shear design of prestressed members shall be based on the general procedure of AASHTO LRFD Section 5.7.3.4.2.
2. The shear design of all non-prestressed members shall be based on either the general procedure, or the simplified procedure of AASHTO LRFD Section 5.7.3.4.1.

3. The strut-and-tie model shall be employed as required by AASHTO LRFD Sections 5.7.1.1 and 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. 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.

5.2.2.C Interface Shear

Interface shear transfer (shear friction) design is to be performed in accordance with AASHTO LRFD Section 5.7.4.

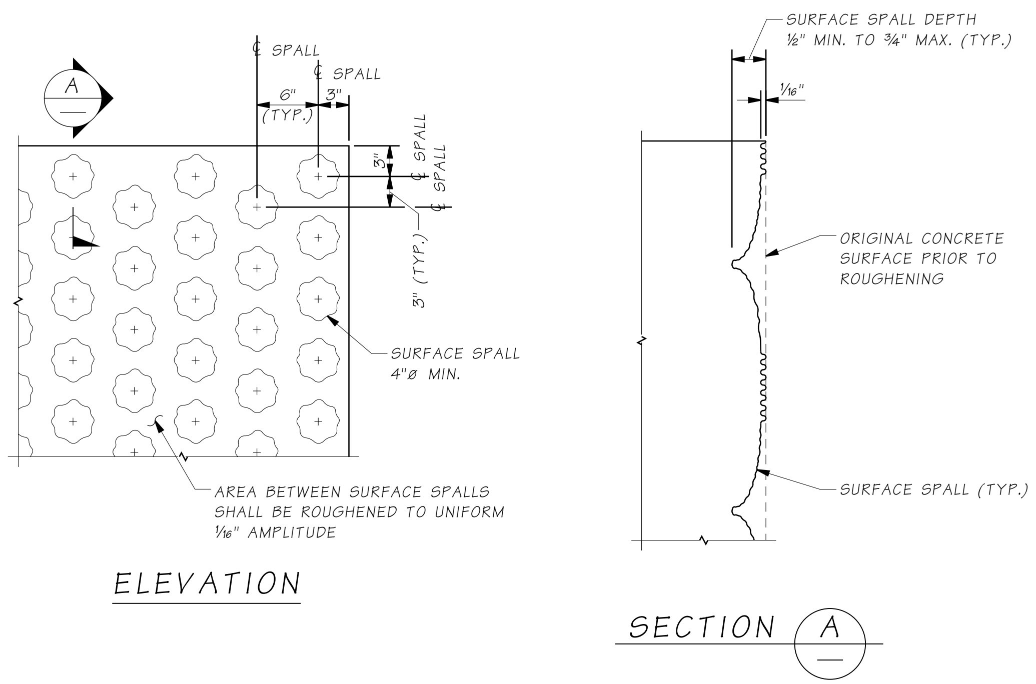
If a roughened surface is required for shear transfer at construction joints in new construction, they shall be identified in the plans. See *Standard Specifications* Section 6-02.3(12)A.

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 Section 5.7.4.4 (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.

Figure 5.2.2-1 Spall Pattern Roughening Detail



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.

5.2.2.C.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 Stay-in-Place (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.

5.2.2.C.2 *Interface Shear Friction at Girder End*

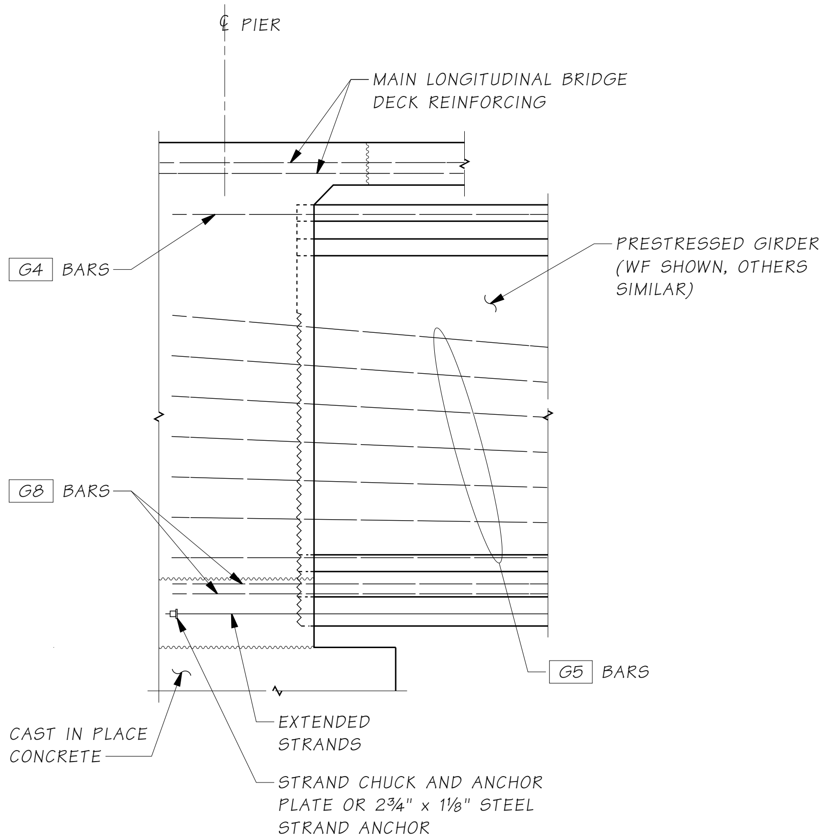
A prestressed concrete girder may be required to carry shears at the end surface of the girder.

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.

Figure 5.2.2-2 End Connection for Continuous Span Girder



5.2.2.D Shear and Torsion

The design for shear and torsion is based on ACI 318-02 *Building Code*[4] *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 Section 5.7.3.6 may also be used for design.

According to Hsu[5], utilizing ACI 318-02 is awkward and overly conservative when applied to large-size hollow members. Collins and Mitchell[6] 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[6].

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 Section 5.8.2.

5.2.4 Deflection and Camber

5.2.4.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 Section 5.6.3.5.2.

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.

5.2.4.B 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 D120, 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 D40, 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.

5.2.4.B.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.

5.2.4.B.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.

5.2.4.B.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}).

5.2.4.B.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}).

5.2.4.B.5 *Creep Deflection After Casting Diaphragms*

The girder continues to deflect upward for any time delay between diaphragms and deck slab casting.

5.2.4.B.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}).

5.2.4.B.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.

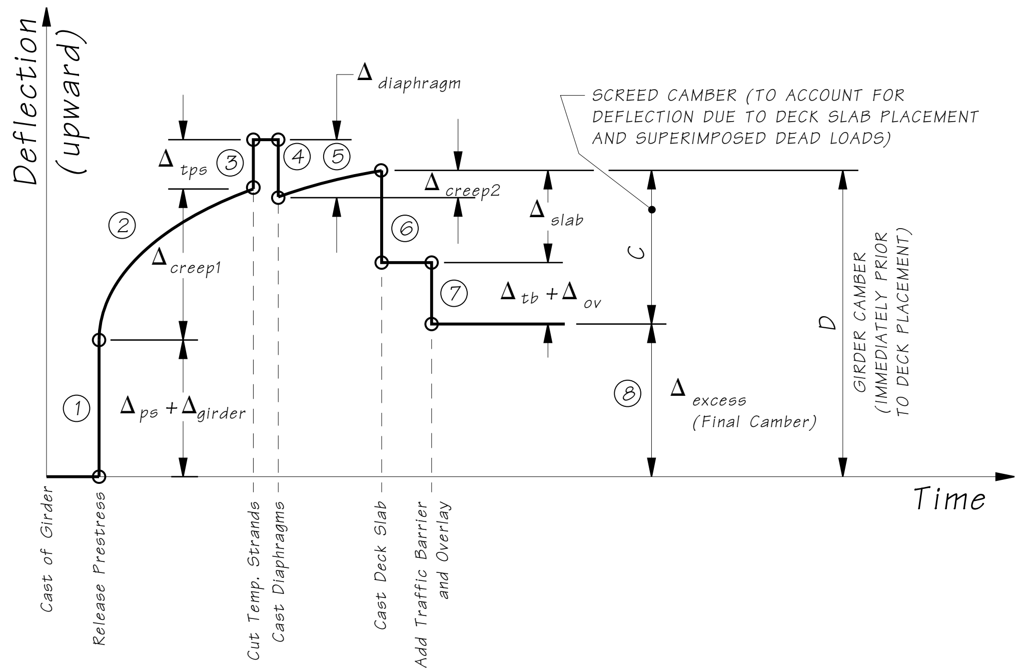
5.2.4.B.8 *Final Camber*

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.

5.2.4.C Pre-camber

Prestressed concrete girders may be precambered to compensate for the natural camber and for the effect of the roadway geometry.

Figure 5.2.4-1 Prestressed Concrete Girder Camber



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 outer face of the element.

When construction joints are shown in the Plans for the convenience of the Contractor and are not structurally required, they shall be indicated as optional.

5.2.5.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.

5.2.5.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 Access and Lighting

5.2.6.A Inspection Access

For girder bridges with bottom flanges, the minimum girder spacing shall be 5’ to permit inspection access between the bottom flanges.

See Section 10.8.1 for design requirements for confined spaces.

5.2.6.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 *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.

Figure 5.2.6-1 Access Hatch Details

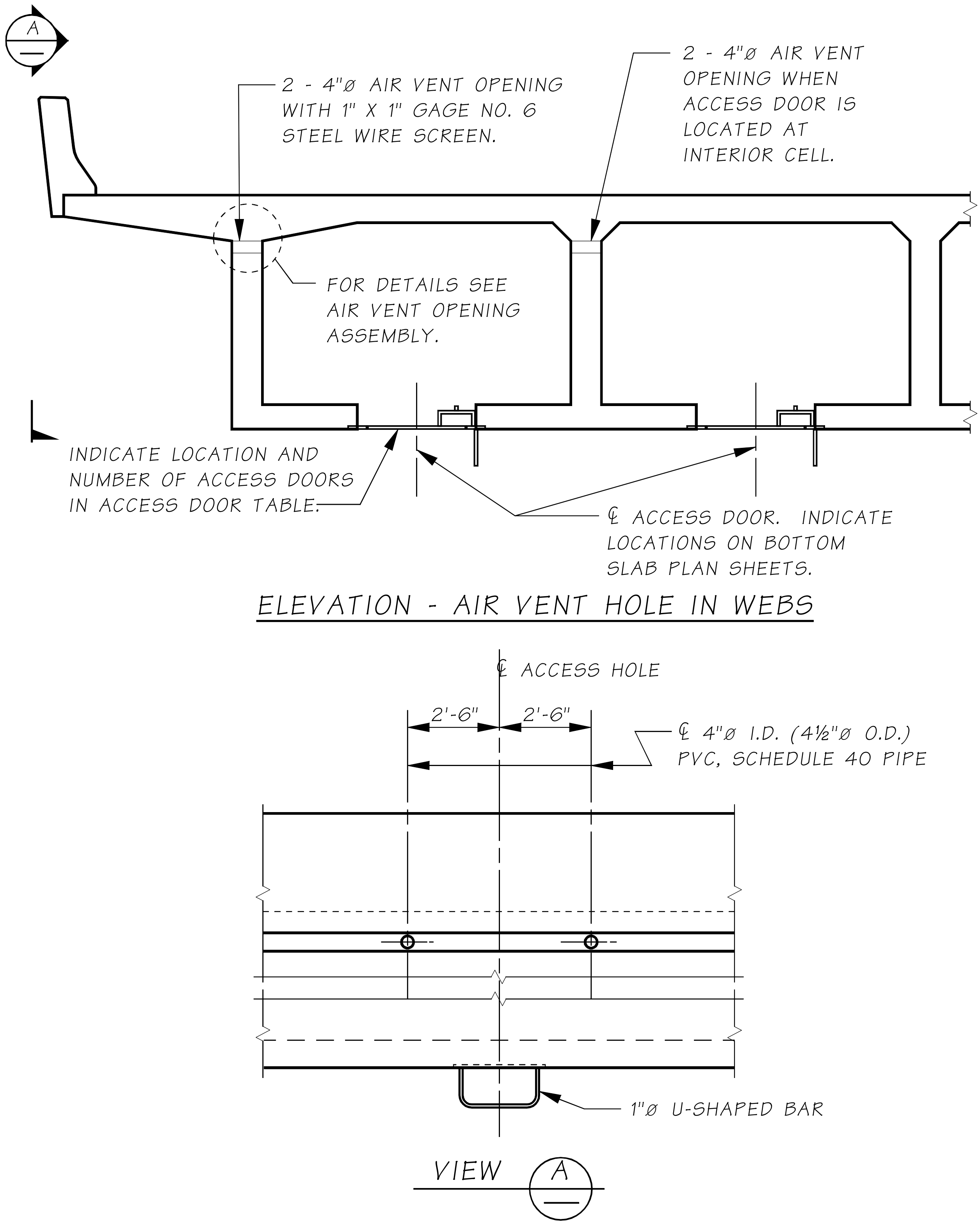
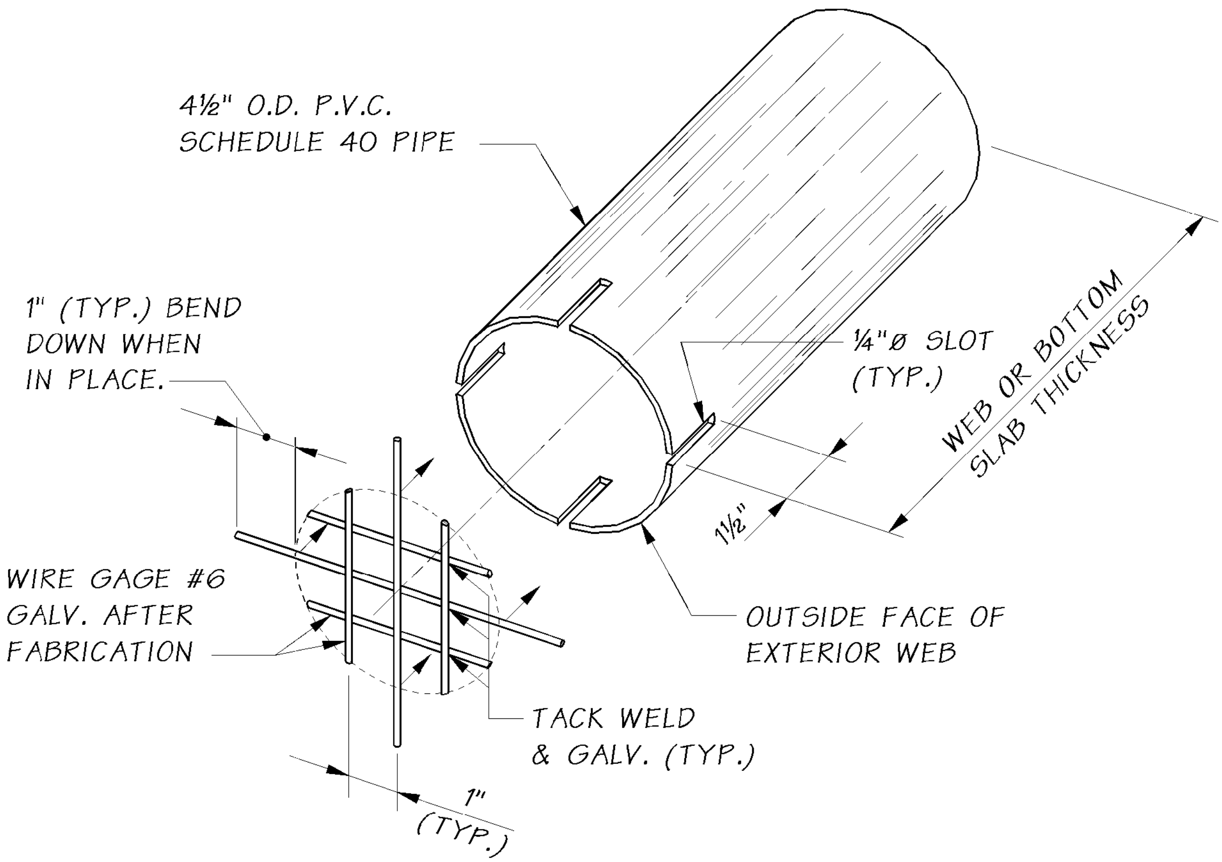


Figure 5.2.6-2 Air Vent Opening Detail



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 State 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 are 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 and 2.4.1.D.

5.3.1 Box Girder Basic Geometries

5.3.1.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.

5.3.1.B Basic Dimensions

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

5.3.1.B.1 *Top Slab Thickness, T1 (includes ½″ wearing surface)*

$T1 = \frac{12(S+10)}{30}$ but not less than 7″ with overlay or 7.5″ without overlay.

5.3.1.B.2 *Bottom Slab Thickness, T2*

1. Near center span

   $T2 = \frac{12\,S_{clr}}{16}$ but not less than 5.5″ (normally 6.0″ is used).

2. 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).

5.3.1.B.3 *Girder Stem (Web) Thickness, T3*

1. **Near Center Span**

   Minimum T3 = 9.0″ — vertical

   Minimum T3 = 10.0″ — sloped

2. **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 × (difference in web thickness)

5.3.1.B.4 *Intermediate Diaphragm Thickness, T4 and Diaphragm Spacing*

1. For tangent and curved bridge with R > 800 feet

   T4 = 0″ (diaphragms are not required.)

2. 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.

5.3.1.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.

5.3.1.D Load Distribution

5.3.1.D.1 *Unit Design*

According to the AASHTO LRFD, 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.

5.3.1.D.2 *Dead Loads*

Include additional D.L. for top deck forms:

5 pounds per square foot of the area.

10 pounds per square foot if web spacing > 10′-0″.

5.3.1.D.3 *Live Load*

See Section 3.9.4 for live load distribution to superstructure and substructure.

Figure 5.3.1-1 Basic Dimensions–Vertical Webs

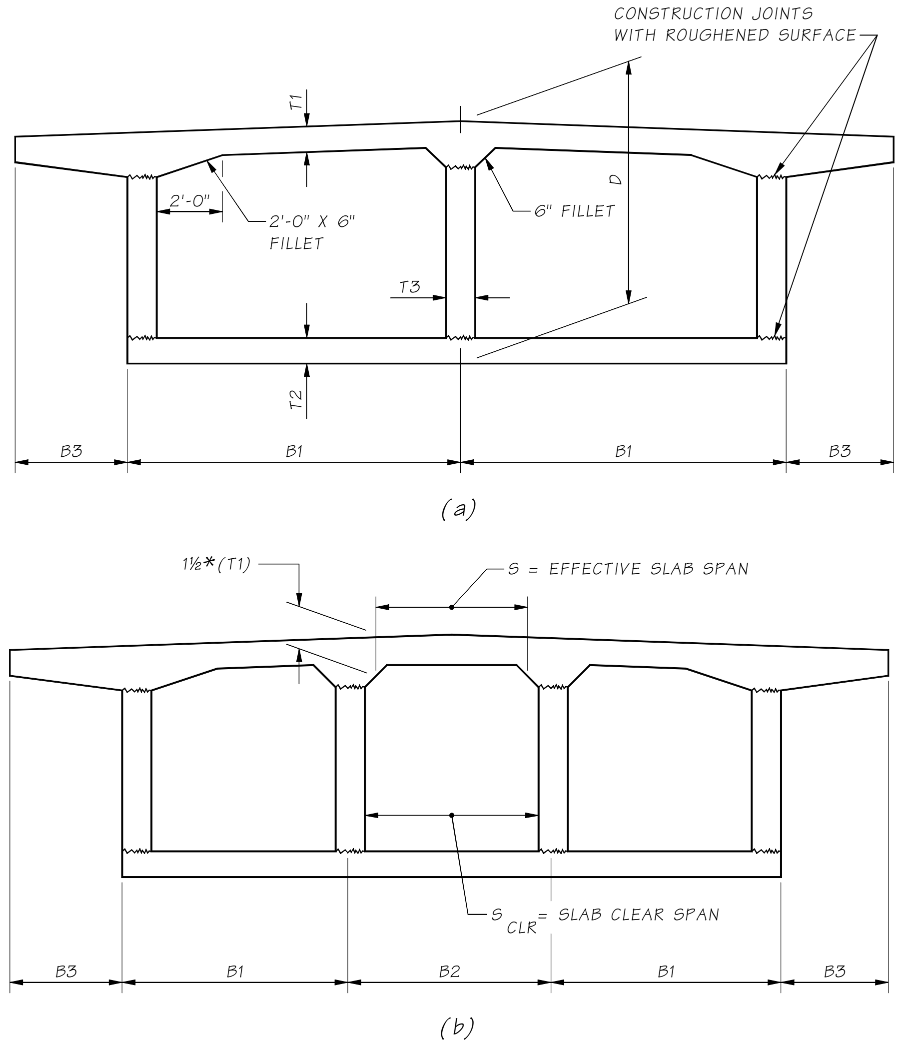
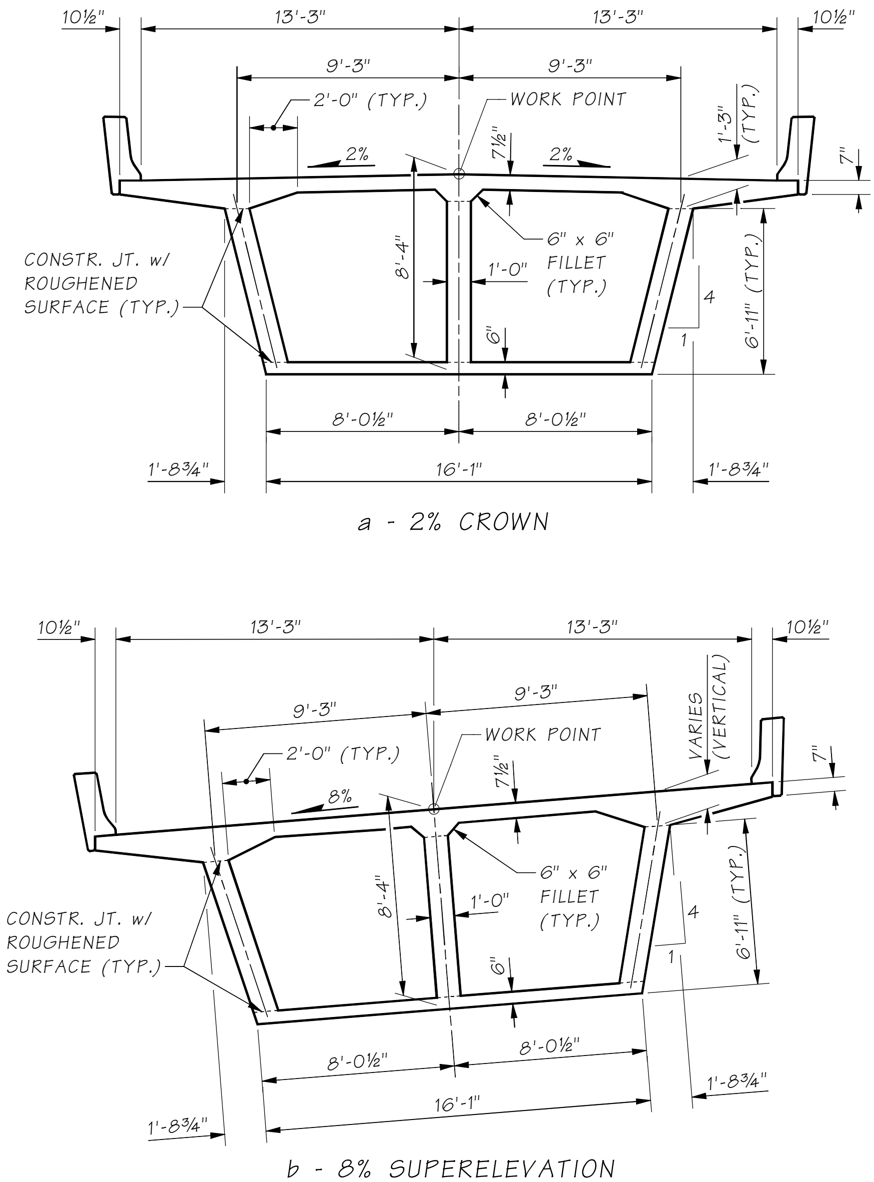


Figure 5.3.1-2 Basic Dimensions–Sloped Webs



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.

5.3.2.A Top Slab Reinforcement

5.3.2.A.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.

1. Transverse reinforcing in the top and bottom layers to transfer the load to the main girder stems.
2. Bottom longitudinal "distribution reinforcement" in the middle half of the deck span in S_{eff} is provided to aid distributing the wheel loads.
3. Top longitudinal "temperature and shrinkage reinforcement."

5.3.2.A.2 *Near Intermediate Piers*

Figure 5.3.2-3 illustrates the reinforcement requirement near intermediate piers.

1. Transverse reinforcing same as center of span.
2. Longitudinal reinforcement to resist negative moment (see Figure 5.3.2-3).
3. "Distribution of flexure reinforcement" to limit cracking shall satisfy the requirement of AASHTO LRFD Section 5.6.7 for class 2 exposure condition.

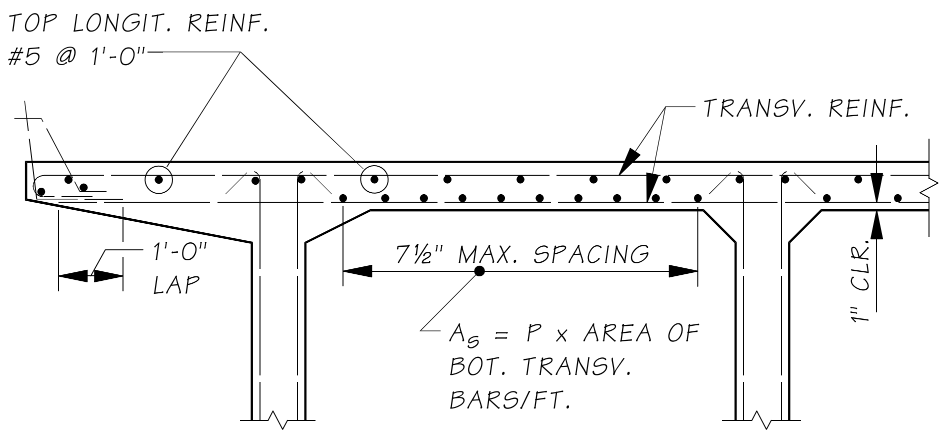
5.3.2.A.3 *Bar Patterns*

1. **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.

2. **Longitudinal Reinforcement**

Figure 5.3.2-1 Partial Section Near Center of Span



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

Figure 5.3.2-2 Overhang Detail

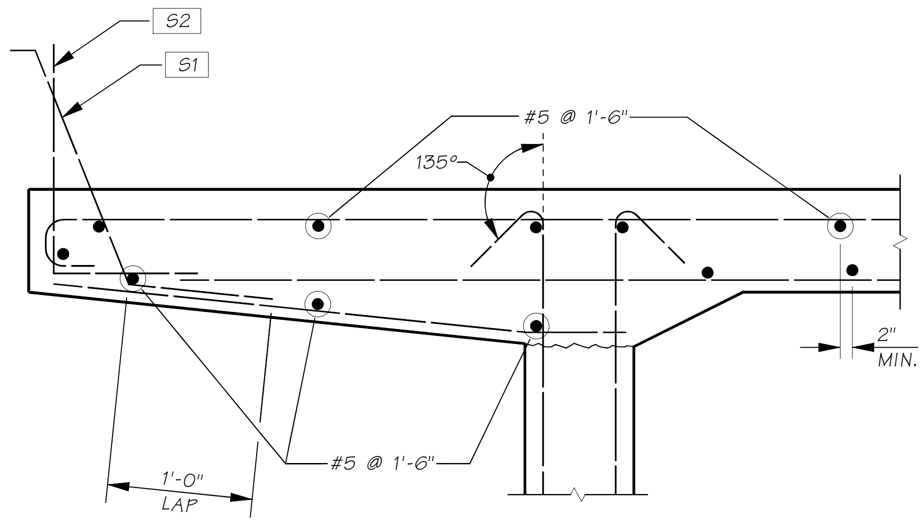
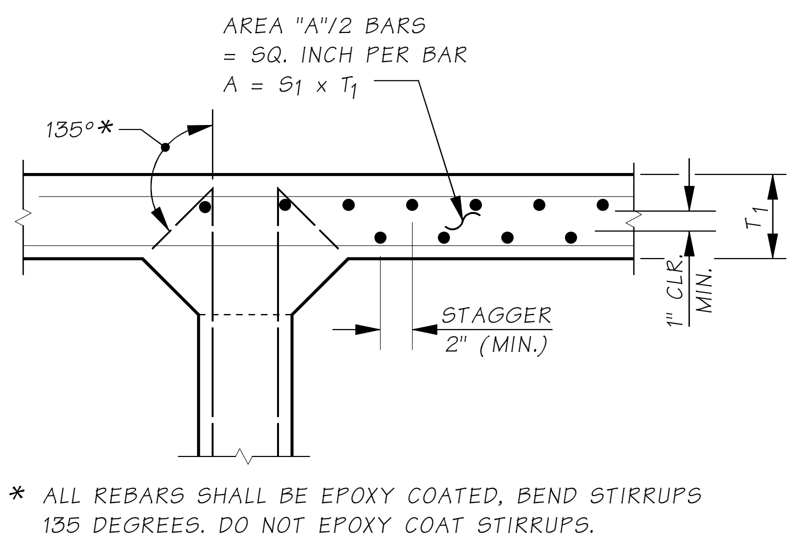


Figure 5.3.2-3 Top Slab Flexural Reinforcing Near Intermediate Pier



5.3.2.B Bottom Slab Reinforcement

5.3.2.B.1 *Near Center of Span*

Figure 5.3.2-5 shows the reinforcement required near the center of the span.

1. Minimum transverse "distributed reinforcement."

   A_s = 0.005 × flange area with ½A_s distributed equally to each surface.
2. Longitudinal "main reinforcement" to resist positive moment.
3. Check "distribution of flexure reinforcement" to limit cracking in accordance with AASHTO LRFD Section 5.6.7 for class 2 exposure condition.
4. Add steel for construction load (sloped outer webs).

5.3.2.B.2 *Near Intermediate Piers*

Figure 5.3.2-6 shows the reinforcement required near intermediate piers.

1. Minimum transverse reinforcement same as center of span.
2. Minimum longitudinal "temperature and shrinkage reinforcement."
   A_s = 0.004 × flange area with ½A_s distributed equally to each face.
3. Add steel for construction load (sloped outer webs).

5.3.2.B.3 *Bar Patterns*

1. **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.

2. **Longitudinal Reinforcement**

   For longitudinal reinforcing bar patterns, see Figures 5.3.2-5 and 5.3.2-6.

5.3.2.C Web Reinforcement

5.3.2.C.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:

$$\frac{A_v}{s} = 50\,\frac{b_w}{f_y} \tag{5.3.2-1}$$

but not less than #5 bars at 1′-6″,

Where: b_W is the number of girder webs x T3

5.3.2.C.2 *Web Longitudinal Reinforcement*

Web longitudinal reinforcement for reinforced concrete box girders is shown in Figures 5.3.2-8 and 5.3.2-9. The area of skin reinforcement A_{sk} per foot of height on each side face shall be:

$$A_{sk} \geq 0.012(d - 30) \quad (5.3.2\text{-}2)$$

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.

5.3.2.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.

Figure 5.3.2-5 Bottom Slab Reinforcement Near Center of Span

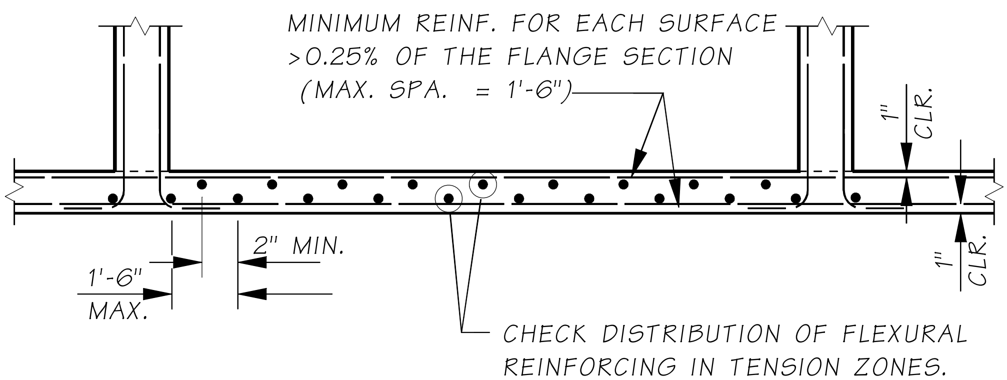


Figure 5.3.2-6 Bottom Slab Reinforcement Near Intermediate Pier

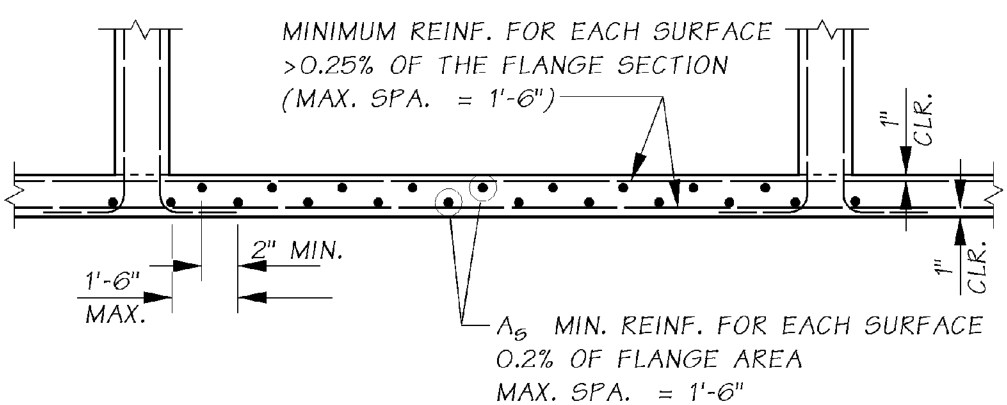


Figure 5.3.2-7 Web Reinforcement

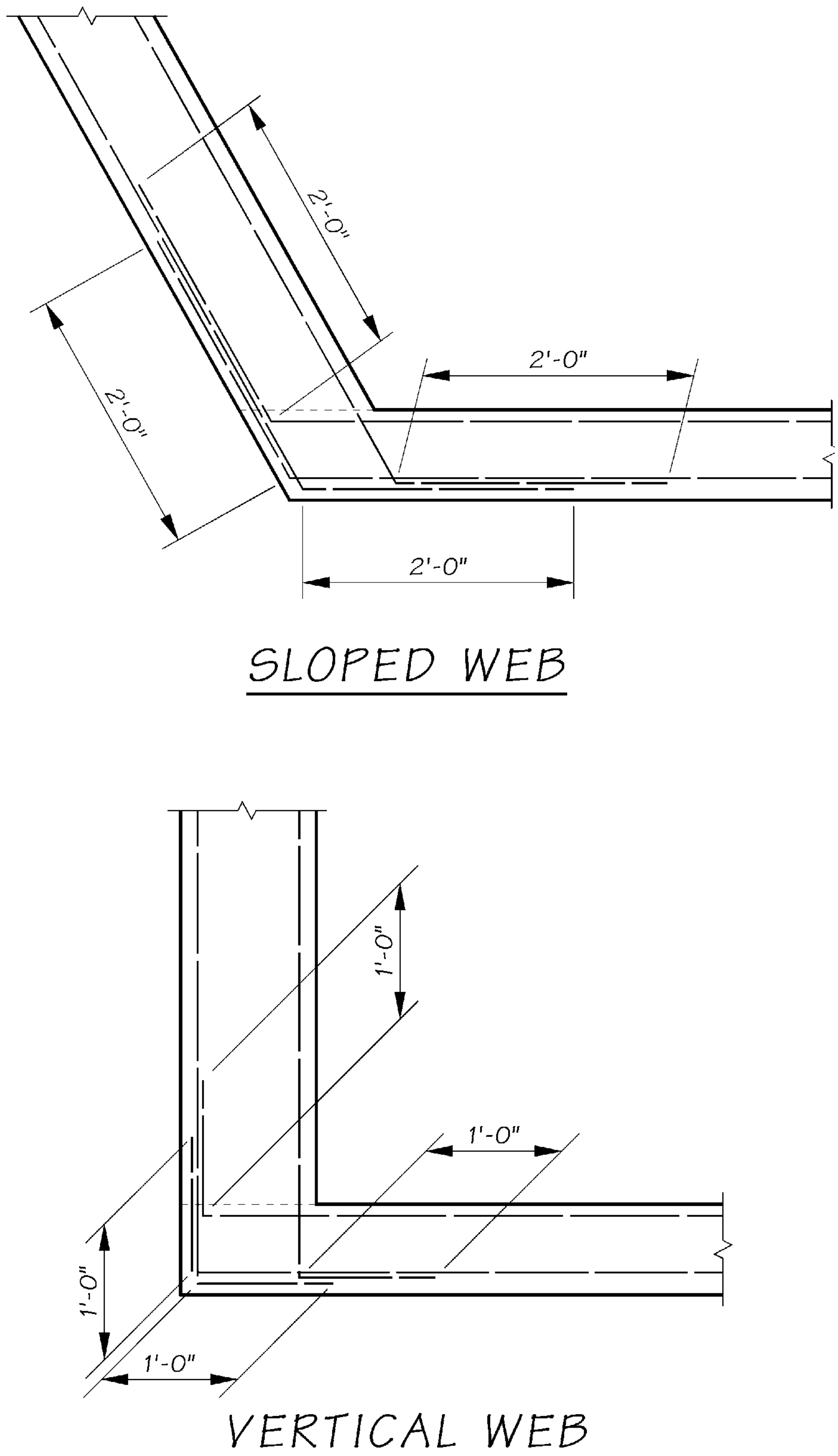
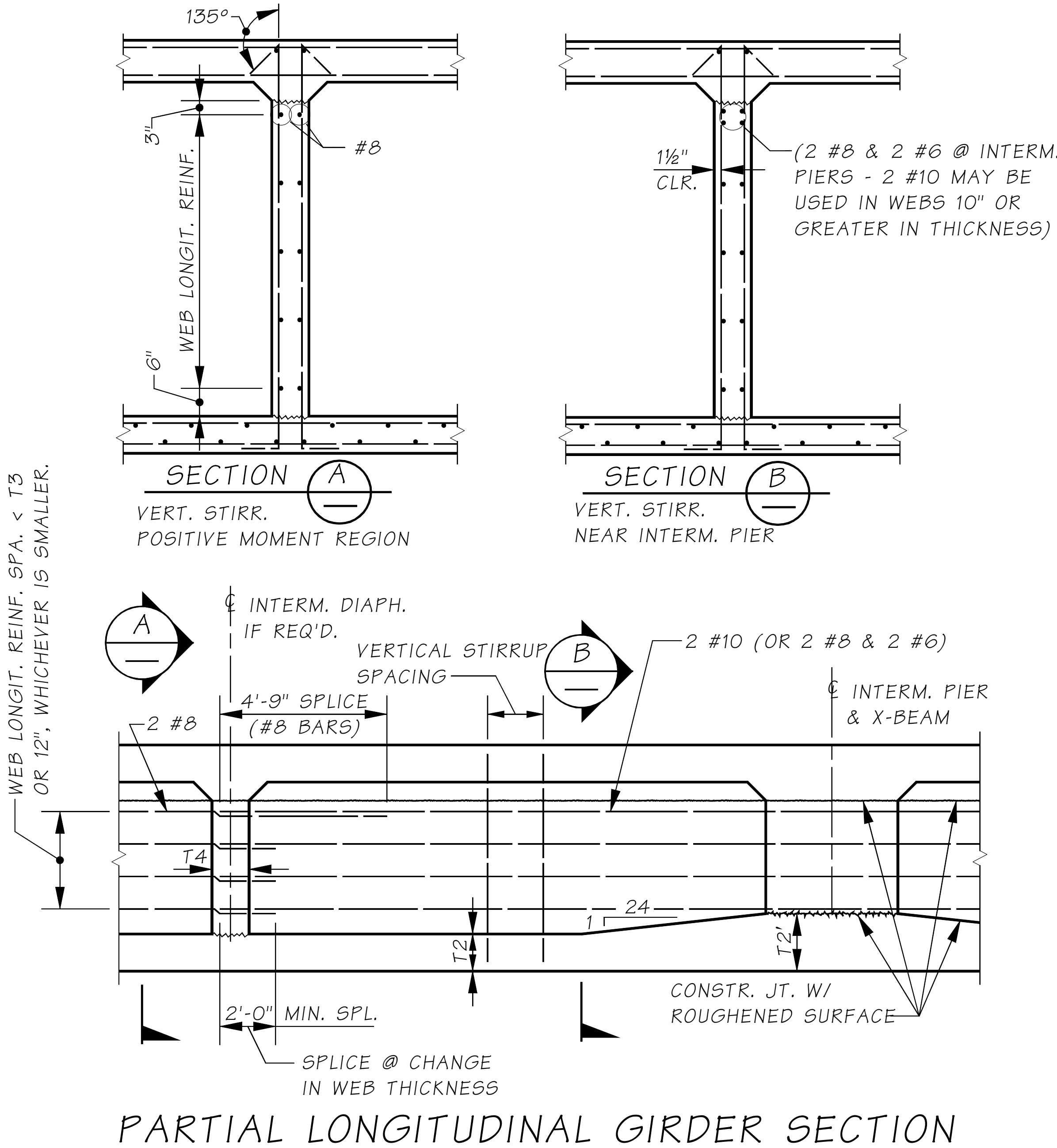
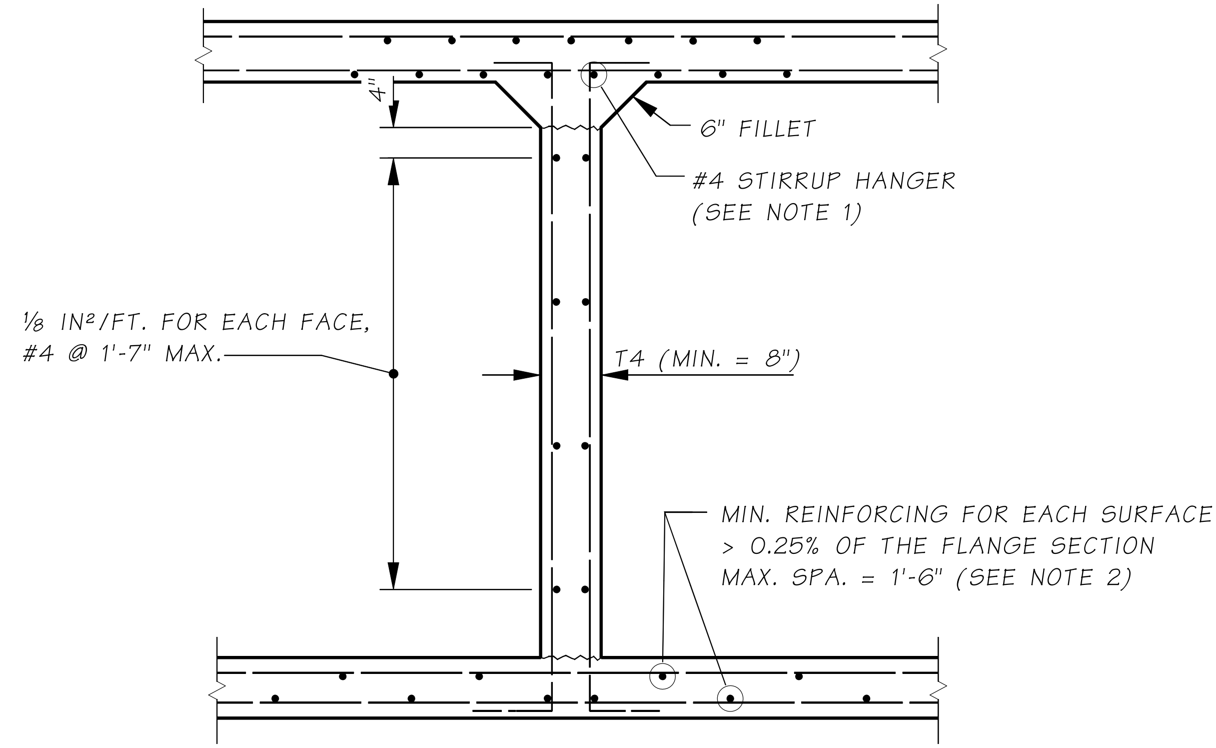


Figure 5.3.2-8 Web Reinforcement Details



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 inflecton. 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.

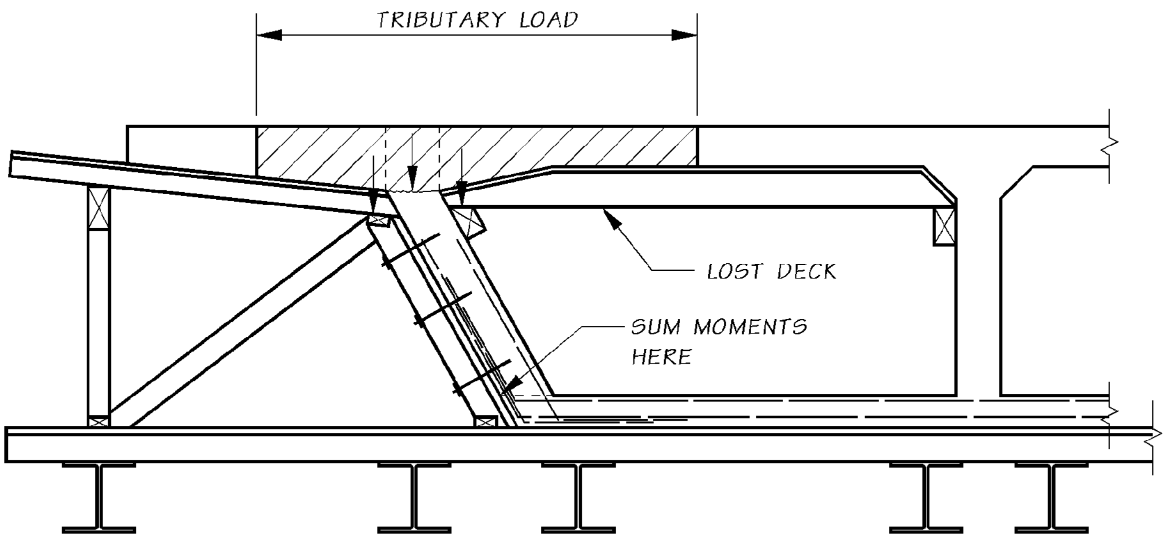
Figure 5.3.2-9 Web Reinforcement Details



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.

Figure 5.3.2-10 Typical Top Slab Forming for Sloped Web Box Girder



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.

5.3.3 Crossbeam

5.3.3.A General

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

5.3.3.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 5.3.3-2.

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 and 5.3.3-2. 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.

5.3.3.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.

5.3.3.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}.

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

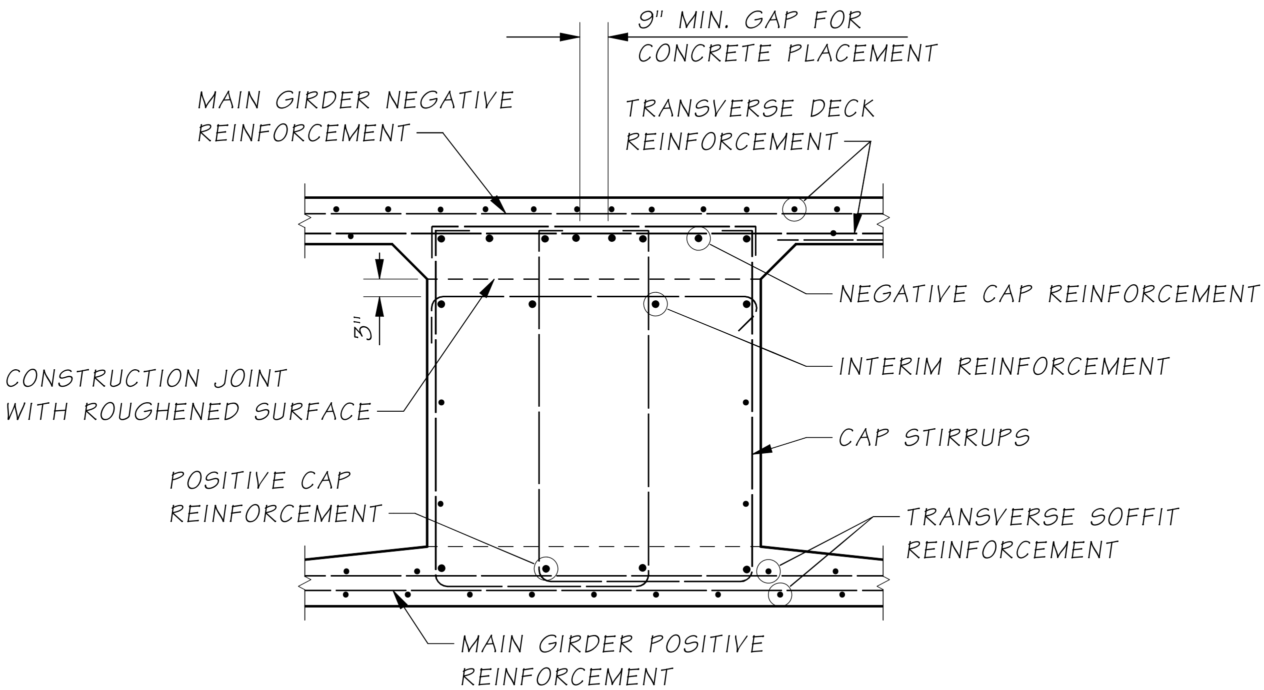


Figure 5.3.3-2 Crossbeam Top Reinforcement for Skew Angle > 25°

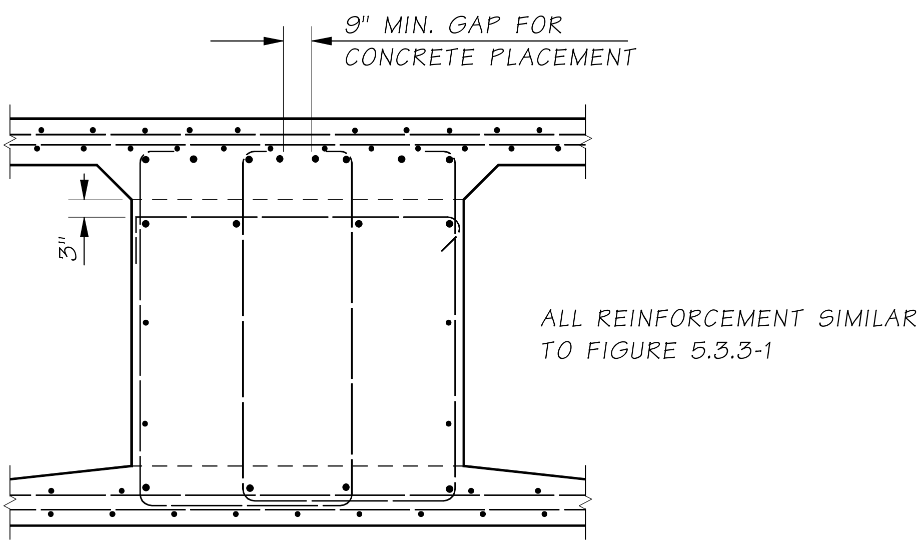
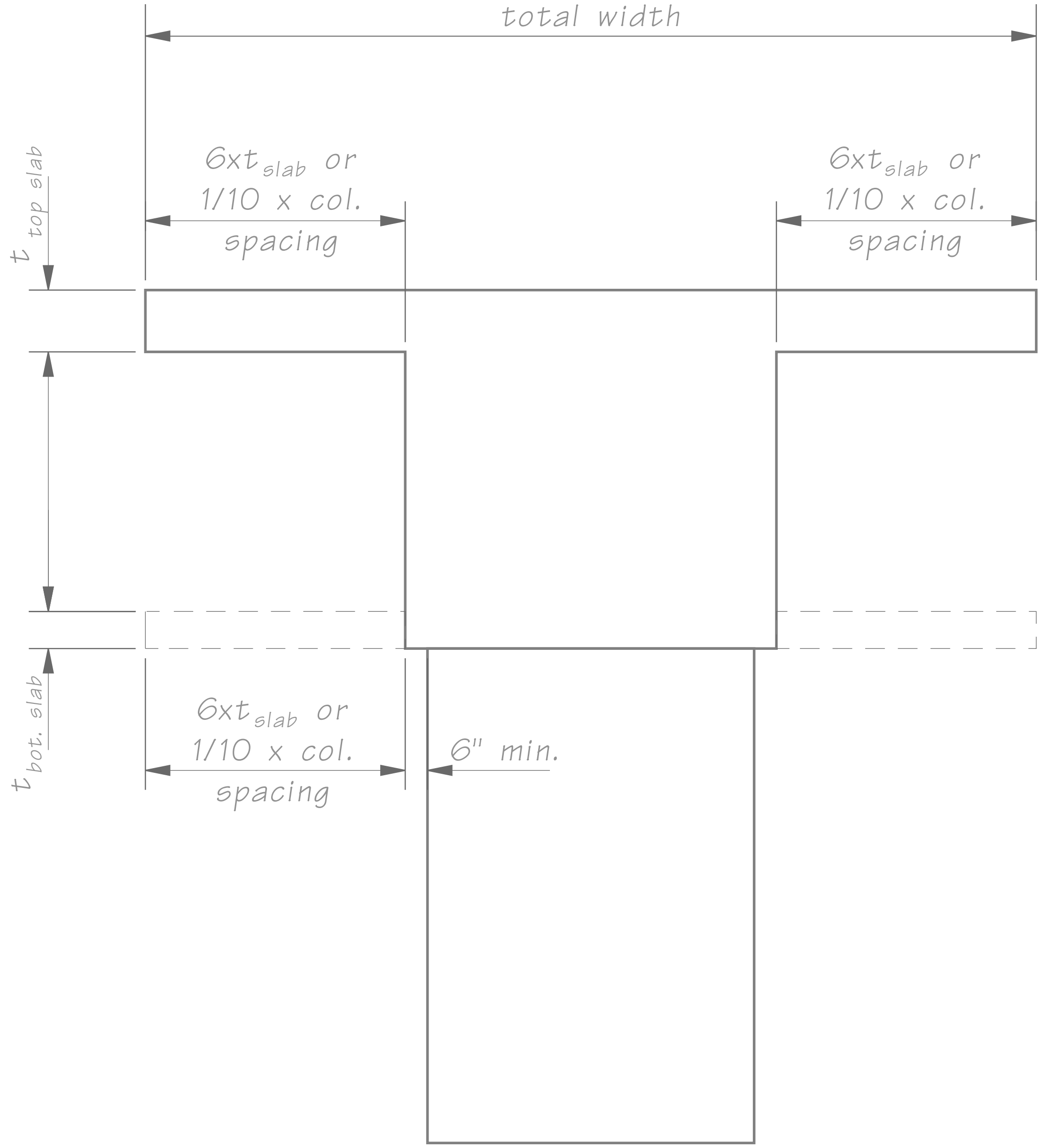


Figure 5.3.3-3 Effective Width of Crossbeam



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.

5.3.3.D.1 *Top Reinforcement*

The negative moment critical section shall be at the ¼ point of the square or equivalent square columns.

1. **When Skew Angle ≤ 25°**

   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.

2. **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).

3. **To avoid cracking of concrete**

   Interim reinforcement is required below the construction joint in crossbeams.

5.3.3.D.2 *Skin Reinforcement*

Longitudinal skin reinforcement shall be provided in accordance with AASHTO LRFD Section 5.6.7.

5.3.4 End Diaphragm

5.3.4.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.

Figure 5.3.4-1 Bearing Locations at End Diaphragm

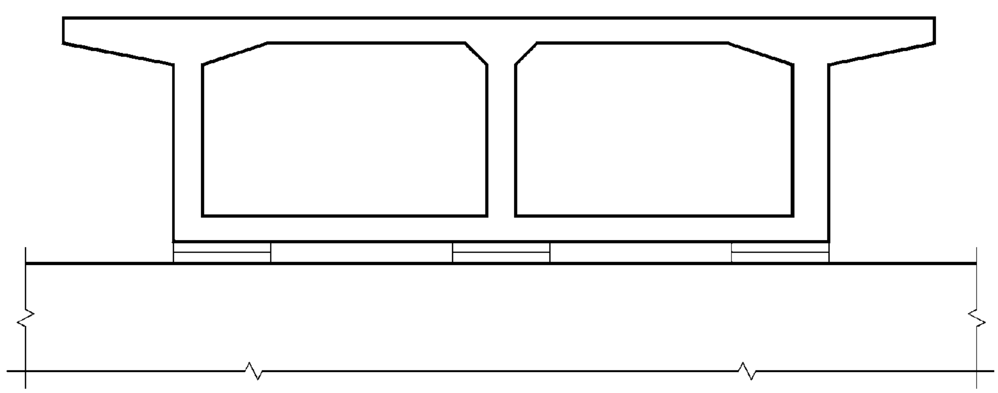
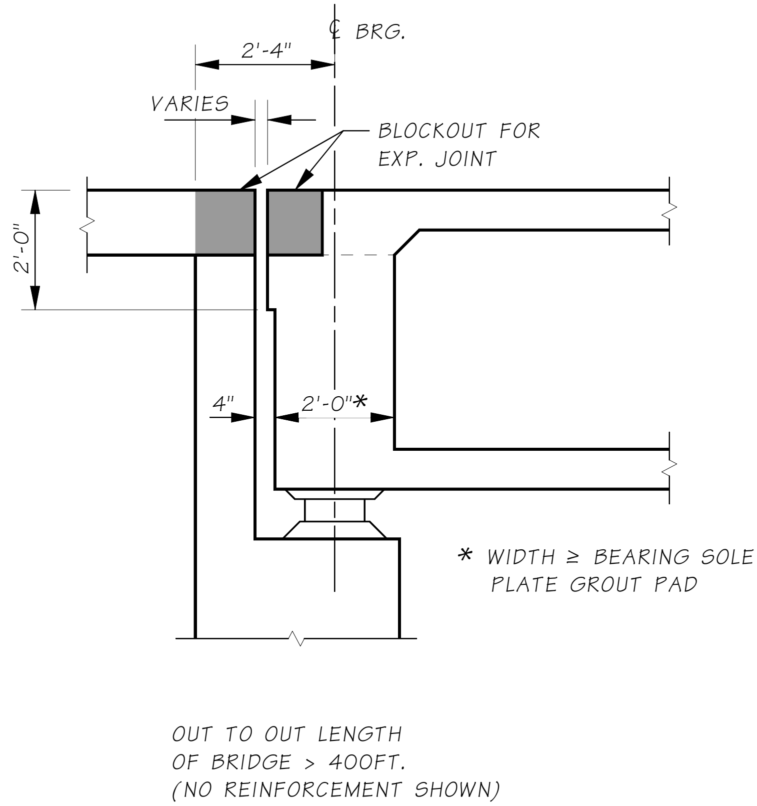
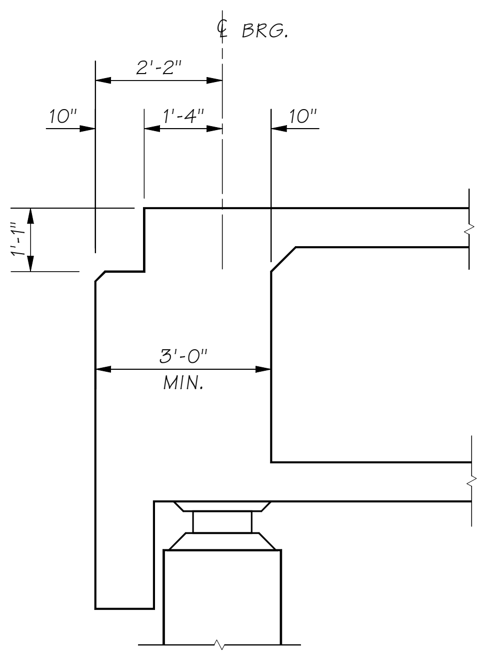


Figure 5.3.4-2 L-shape Abutment End Diaphragm



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.

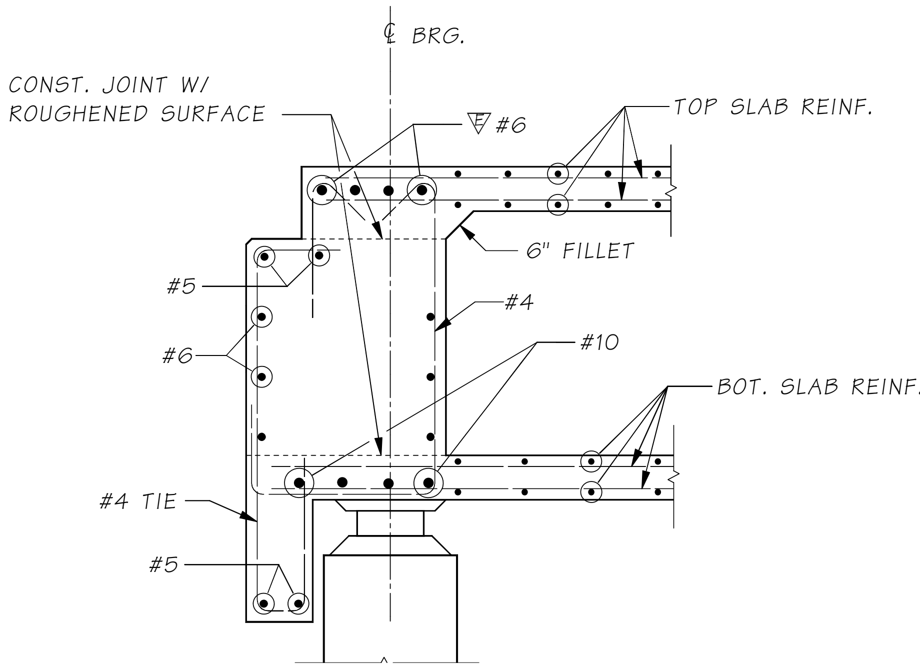
Figure 5.3.4-3 End Diaphragm with Stub Abutment



5.3.4.B Reinforcing Steel Details

Typical reinforcement details for an end diaphragm are shown in Figure 5.3.4-4.

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.

Table 5.3.5-1 Long-term Camber Multipliers

	Multiplier Coefficient
Girder Adjacent to Existing/Stage Construction	
Deflection (downward) — apply to the elastic deflection due to the weight of member	1.90
Deflection (downward) — apply to the elastic deflection due to superimposed dead load only	2.20
Girder Away From Existing/Stage Construction	
Deflection (downward) — apply to the elastic deflection due to the weight of member	2.70
Deflection (downward) — apply to the elastic deflection due to superimposed dead load only	3.00

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.

5.3.6.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.

5.3.6.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.

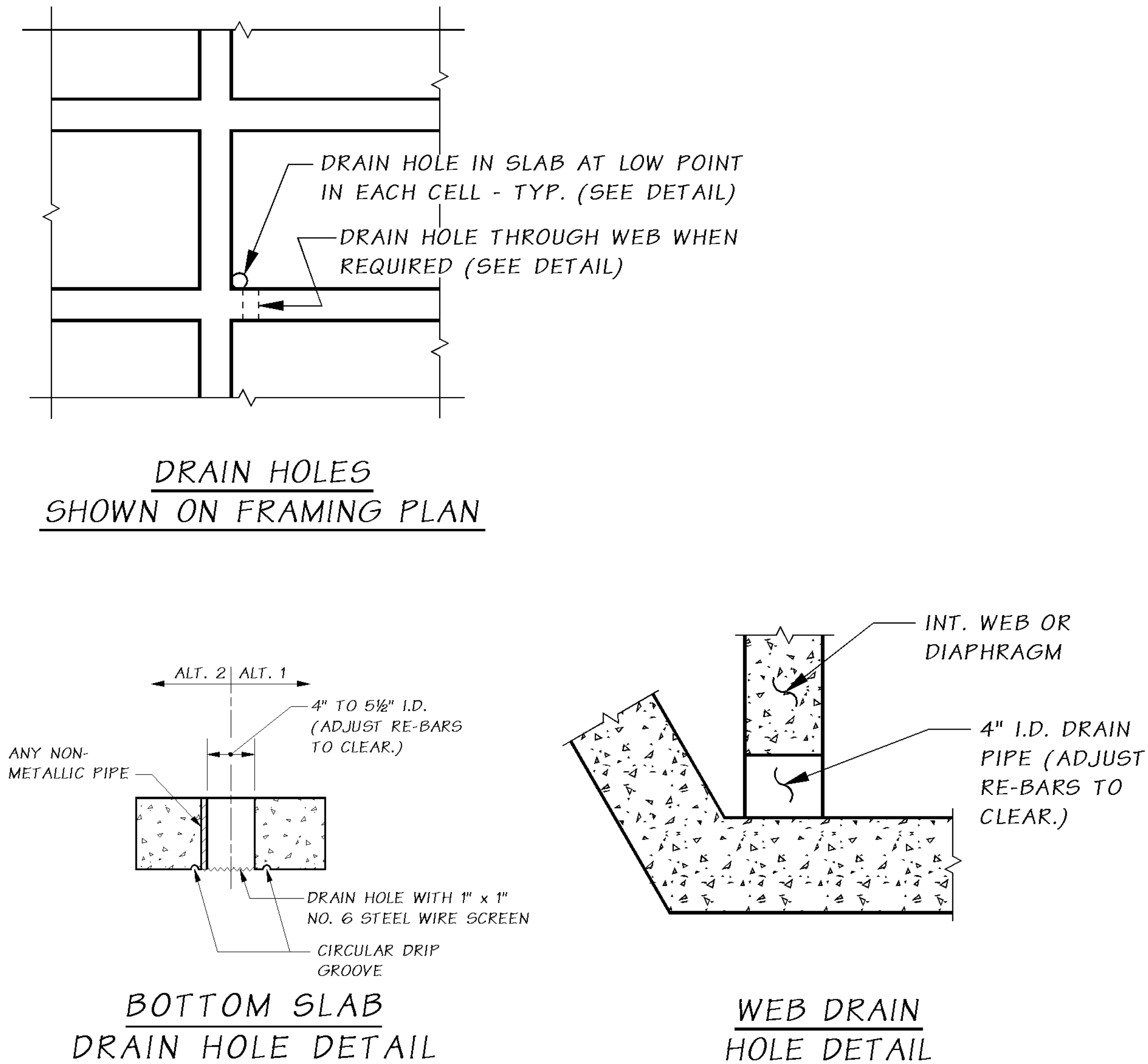
Designer shall provide access space or pockets for maintenance and inspection of bearings.

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.

Figure 5.3.8-1 Drain Hole Details



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[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 *Guide Specifications for Seismic Design* requirements. Design considerations for beam ledges, inverted T-beam and hinges are given in AASHTO LRFD Section 5.8.4.3.

Inverted T-beam pier caps shall not be used for prestressed concrete girder bridges unless approved by the State Bridge Design Engineer.

Figure 5.4-1 Hinge and Inverted T-Beam Pier Cap

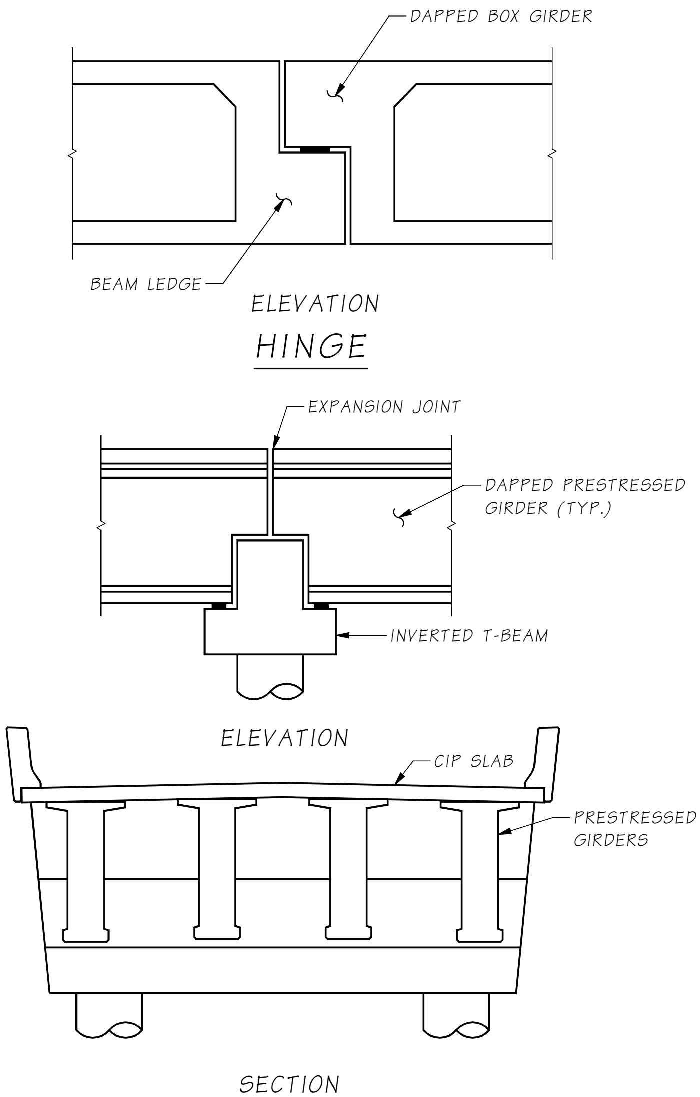
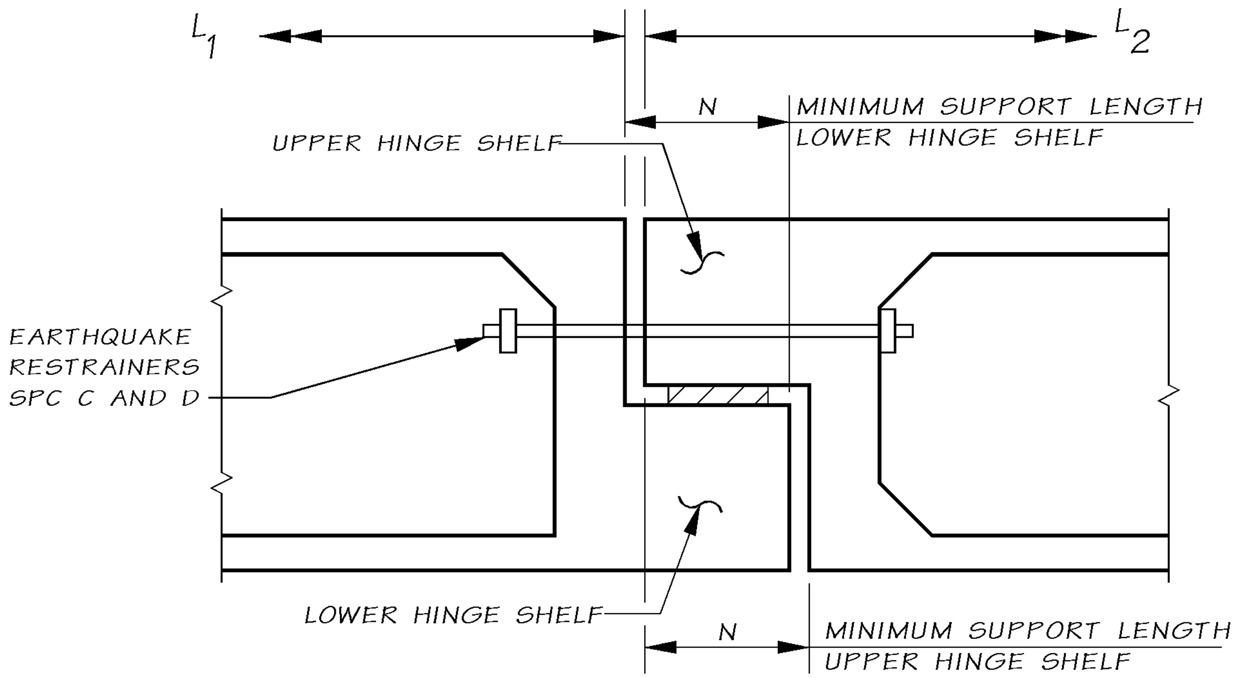


Figure 5.4-2 In-Span Hinge



5.5 Bridge Widenings

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[9].

5.5.1 Review of Existing Structures

5.5.1.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 WSDOT Bridge Preservation Office 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 Bridge Archive Engineer.

5.5.1.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*.

5.5.1.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.

5.5.1.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

5.5.2.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.

5.5.2.A.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.

5.5.2.A.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.

5.5.2.A.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.

5.5.2.A.4 *Specifications*

The design of the widening shall conform to the current AASHTO *LRFD Bridge Design Specifications* and the *Standard Specifications*.

5.5.2.A.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.

5.5.2.A.6 *Overlay*

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

5.5.2.A.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.

5.5.2.A.8 *Special Considerations*

1. 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.
2. Longitudinal joints are not permitted in order to eliminate potentially hazardous vehicle control problems.
3. 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.
4. 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.
5. 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.
6. The alignment for diaphragms for the widening shall generally coincide with the existing diaphragms.
7. 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.

5.5.2.B Seismic Design Criteria for Bridge Widenings

Seismic design of bridge widenings shall be in accordance with Section 4.3.

5.5.2.C Substructure

5.5.2.C.1 *Selection of Foundation*

1. 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.
2. 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.

5.5.2.C.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.

5.5.2.D Superstructure

5.5.2.D.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.

5.5.2.D.2 *Closure Strip*

Except for narrow deck slab widenings a closure strip is required for all 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.

In prestressed concrete girder bridge widenings, the closure shall extend through the intermediate and end diaphragms. The diaphragms shall be made continuous with existing diaphragms.

5.5.2.D.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.

The designer shall investigate the adequacy of the existing structure adjacent to the widening for any additional loads, taking into account the loss of removed components.

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 overstress 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.

5.5.2.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.

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

5.5.4.A General

5.5.4.A.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.

5.5.4.A.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.

5.5.4.A.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.

5.5.4.A.4 *Dowelling Reinforcing Bars Into the Existing Structure*

1. 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.

2. 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.

3. 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.

Table 5.5.4-1 Allowable Tensile Load for Dowels Set With Epoxy Resin f'_c = 4,000 psi, Grade 60 Reinforcing Bars, Edge Clearance ≥ 3″, and Spacing ≥ 6″

Bar Size	Allowable Design Tensile Load, T* (kips)	Drill Hole Size (in)	Required Embedment, L_e	
			Uncoated (in)	Epoxy Coated (in)
#4	12.0	⅝	7	8
#5	18.6	¾	8	9
#6	26.4	1	9	10
#7	36.0	1⅛	11	12
#8	47.4	1¼	13	14.5
#9	60.0	1⅜	16	17½
#10	73.6	1½	20	22
#11	89.0	1⅝	25	28

* Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.

Table 5.5.4-2 Allowable Tensile Load for Dowels Set With Epoxy Resin, f'_c=4,000 psi, Grade 60 Reinforcing Bars, Edge Clearance ≥ 3″, and Spacing < 6″

Bar Size	Allowable Design Tensile Load, T* (kips)	Drill Hole Size (in)	Required Embedment, L_e	
			Uncoated (in)	Epoxy Coated (in)
#4	12.0	⅝	9½	10½
#5	18.6	¾	10½	11½
#6	26.4	1	11½	12½
#7	36.0	1⅛	13½	15
#8	47.4	1¼	16½	18
#9	60.0	1⅜	20	22
#10	73.6	1½	24	27
#11	89.0	1⅝	30	33

*Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.

5.5.4.A.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.

5.5.4.A.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.

5.5.4.A.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 Bridge 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 Bridge Design Unit Manager.

5.5.4.A.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.

5.5.4.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.

5.5.4.B.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.

Figure 5.5.4-1 Deck Slab Removal Detail

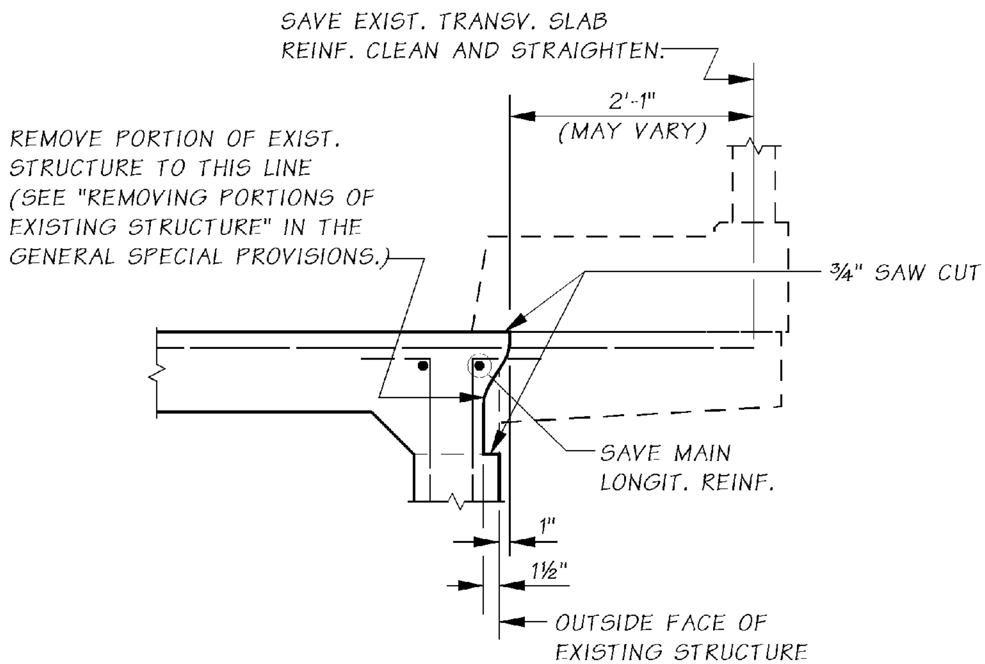
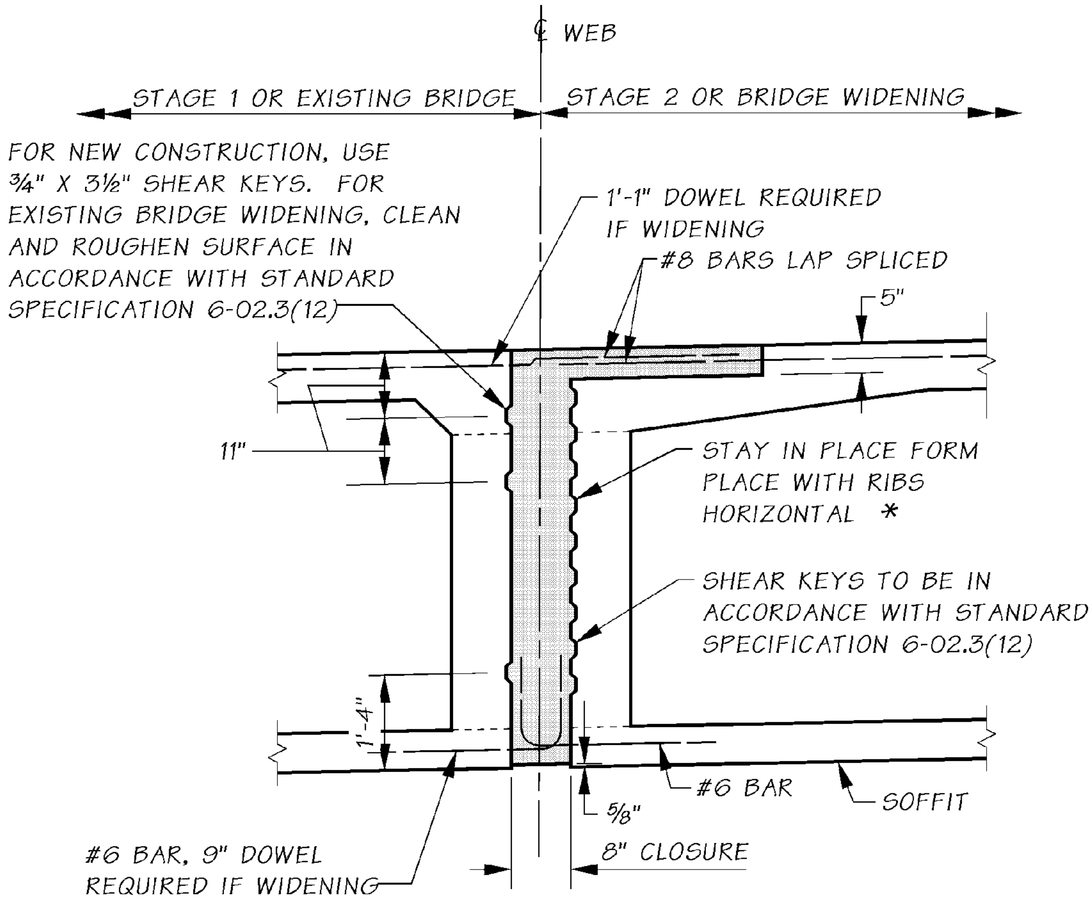


Figure 5.5.4-2 Box Girder Section in Span



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 NECCESSARY 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).

Figure 5.5.4-3 Box Girder Section Through Crossbeam

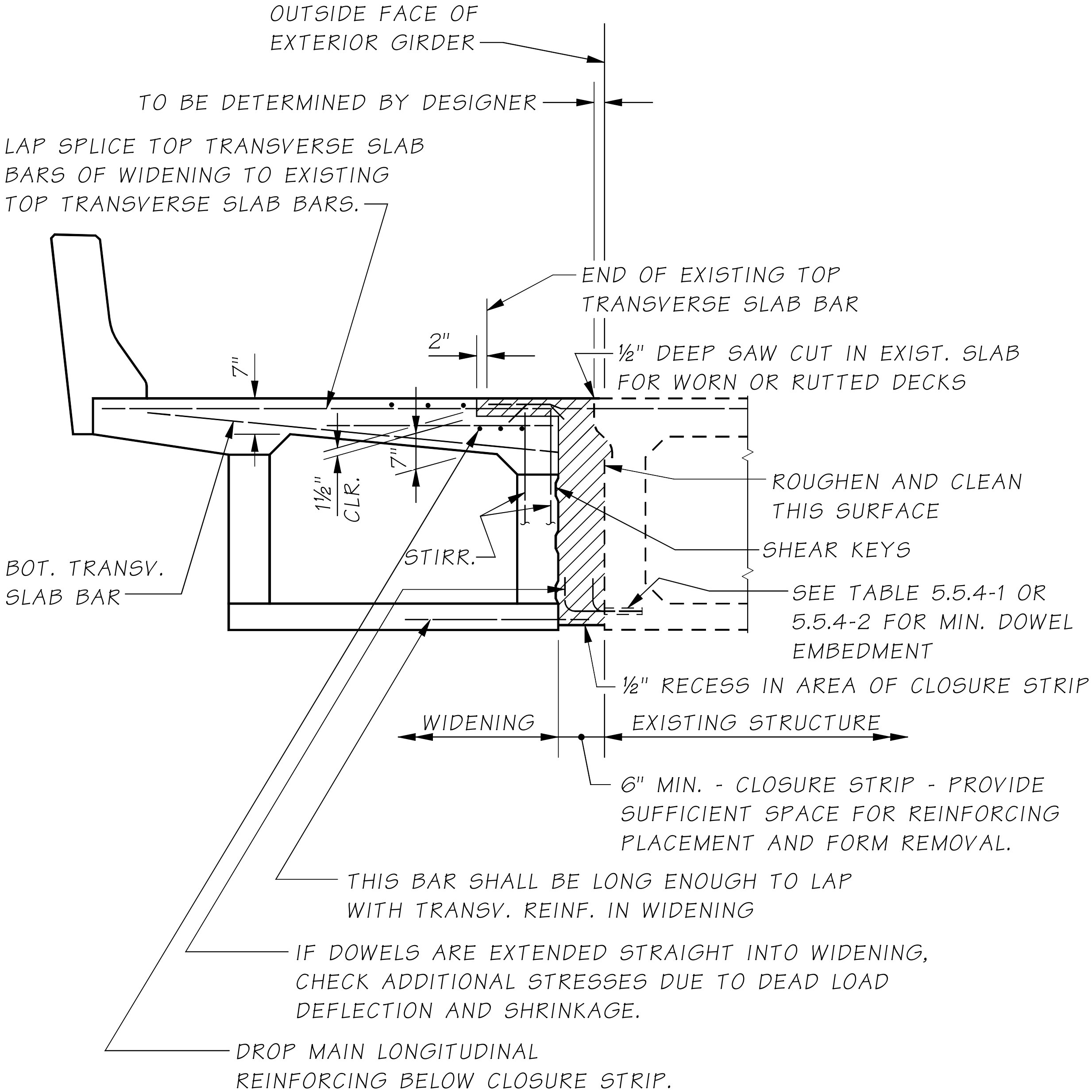
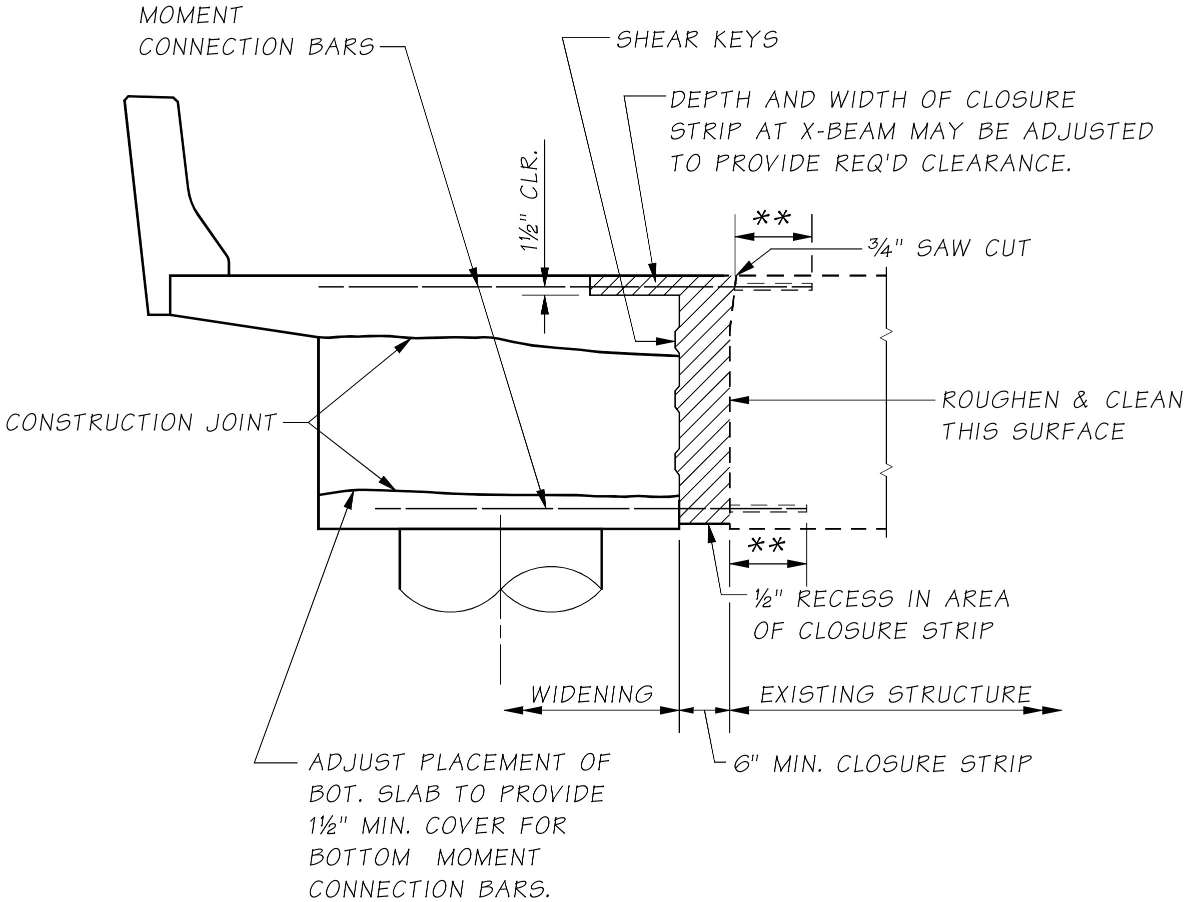


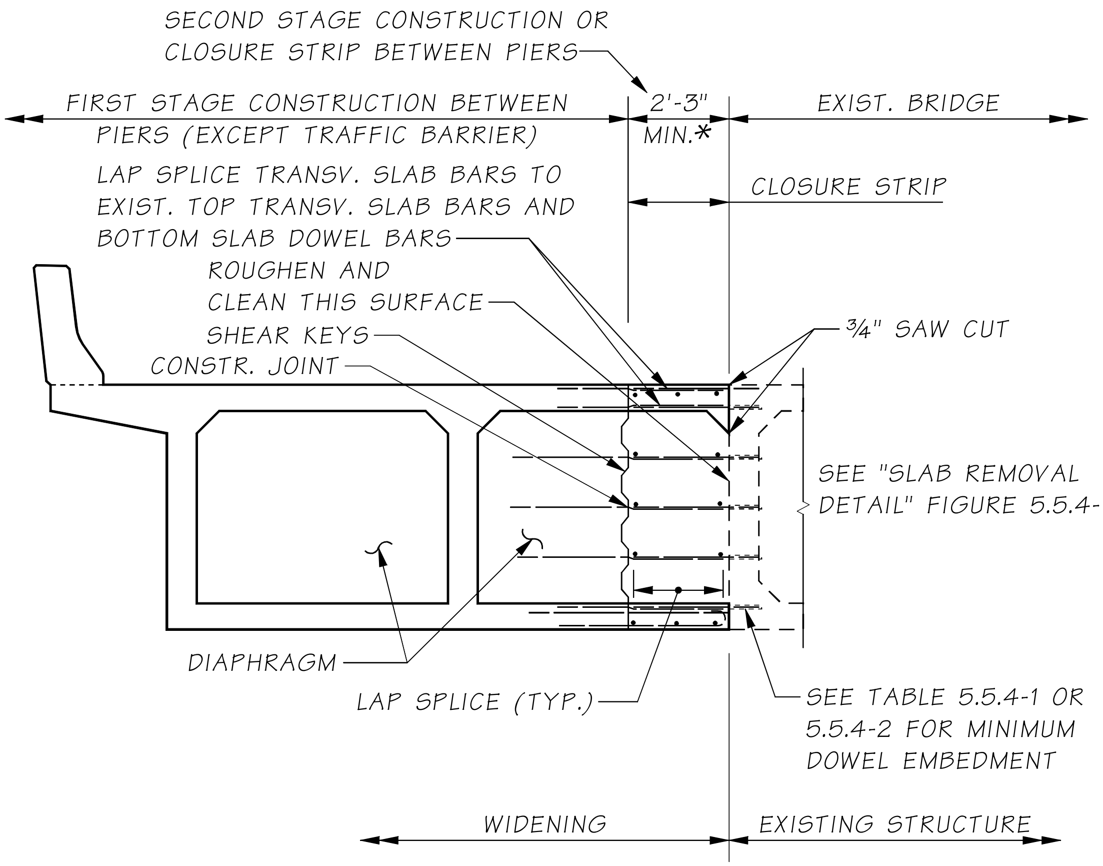
Figure 5.5.4-4 Box Girder Section in Span at Diaphragm Alternate I



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

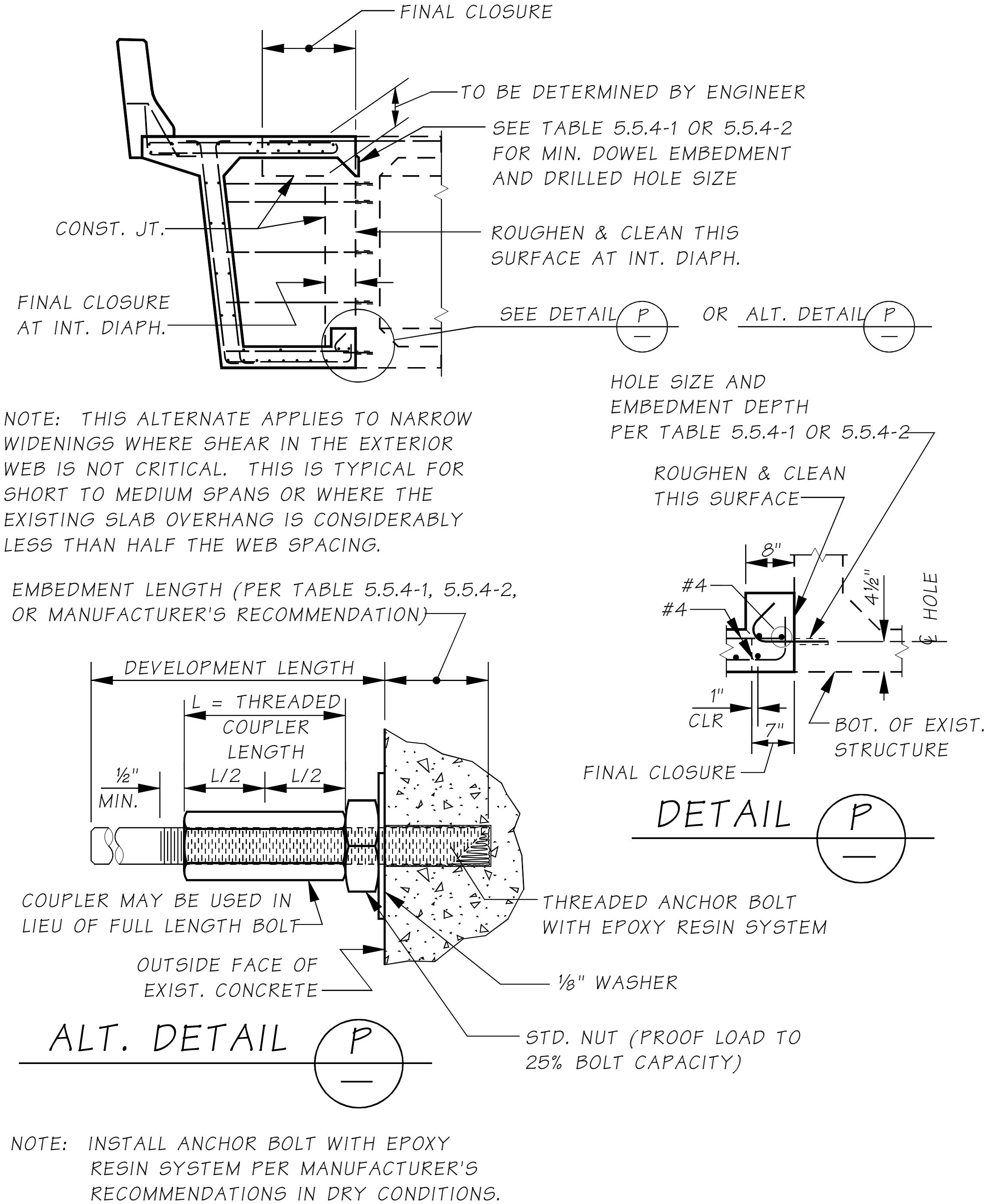
** SEE TABLE 5.5.4-1 OR 5.5.4-2 FOR MINIMUM DOWEL EMBEDMENT

Figure 5.5.4-5 Box Girder Section in Span at Diaphragm Alternate II



* IF LAP SPLICE EXCEEDS 2'-0", INCREASE WIDTH OF CLOSURE STRIP TO ACCOMMODATE INCREASED LAP SPLICE.

Figure 5.5.4-6 Narrow Box Girder Widening Details



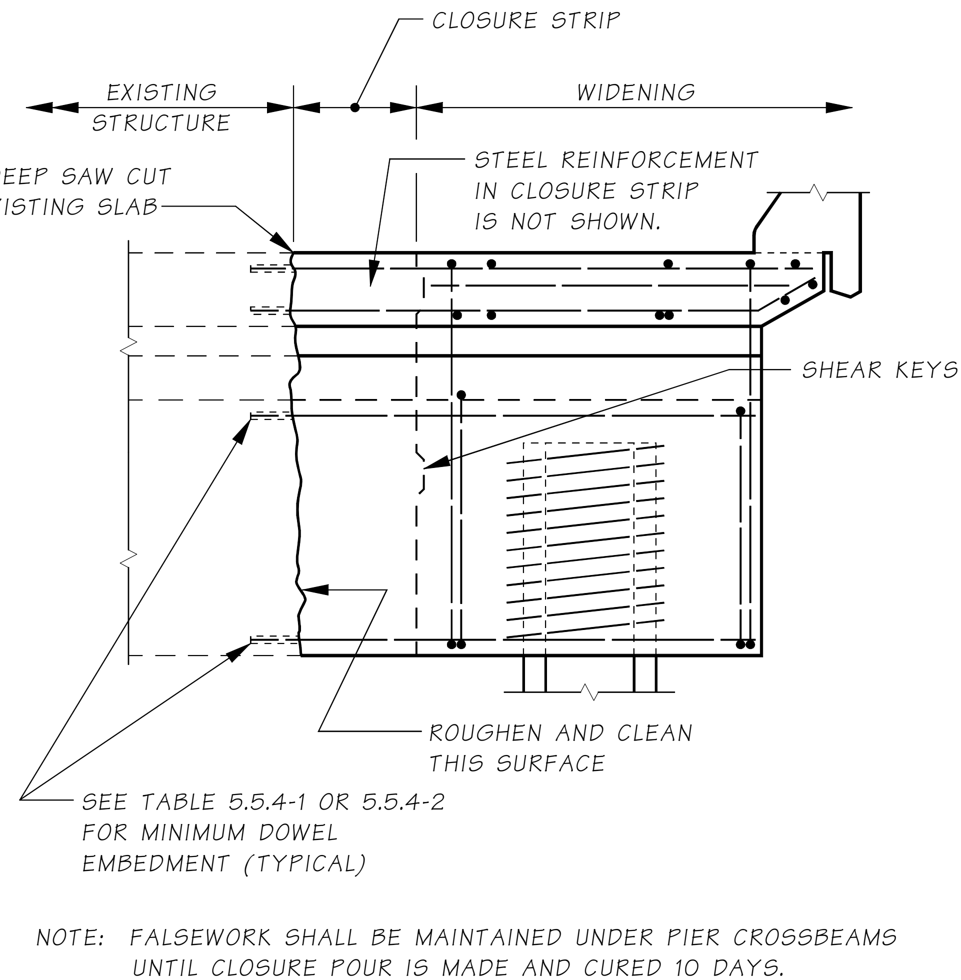
5.5.4.B.2 *Flat Slab Bridges*

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.

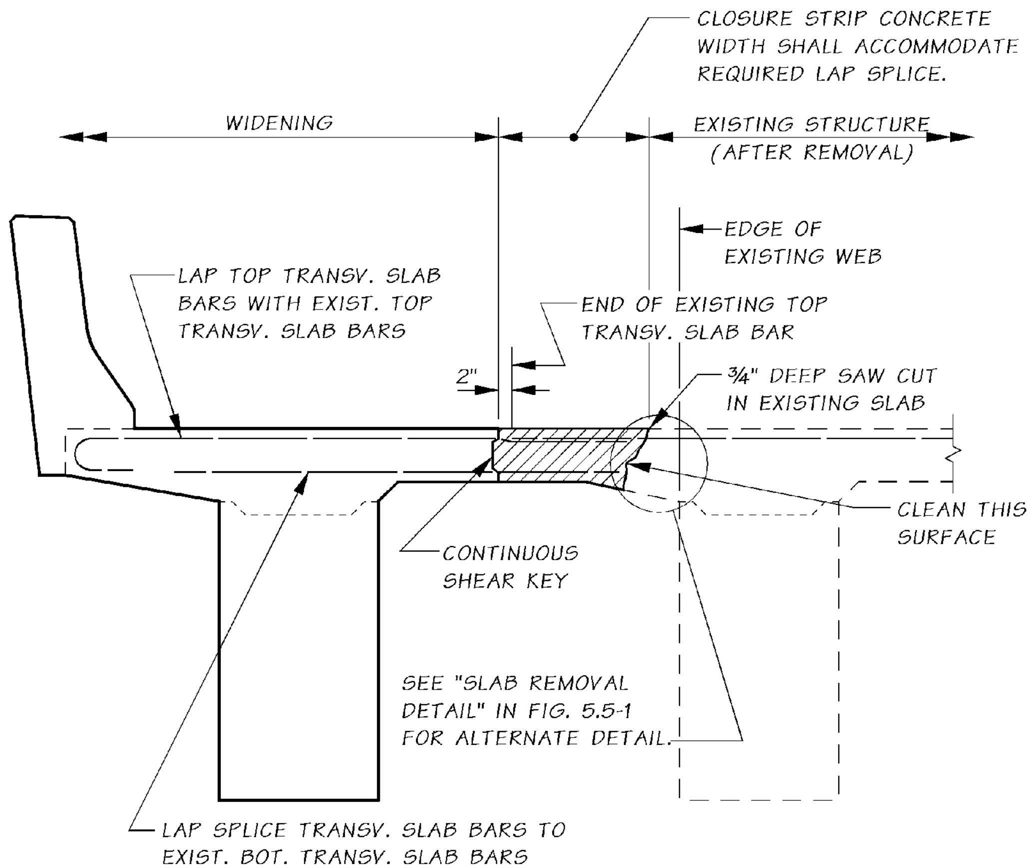
Figure 5.5.4-7 Flat Slab–Section Through Crossbeam



5.5.4.B.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.

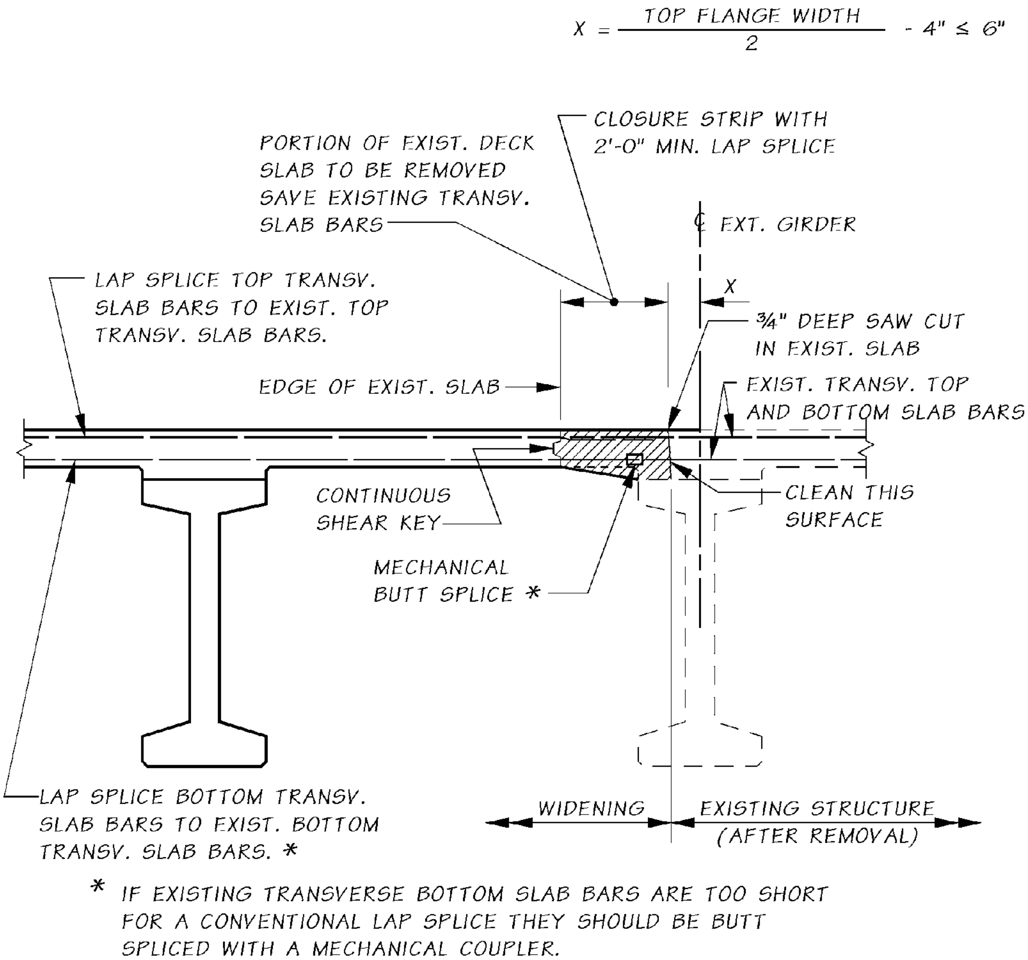
Figure 5.5.4-8 T-Beam–Section in Span



5.5.4.B.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.

Figure 5.5.4-9 Prestressed Concrete Girder–Section in Span



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 and 5.5.5-2 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.

Figure 5.5.5-1 Expansion Joint Detail Shown for Compression Seal With Existing Reinforcing Steel Saved

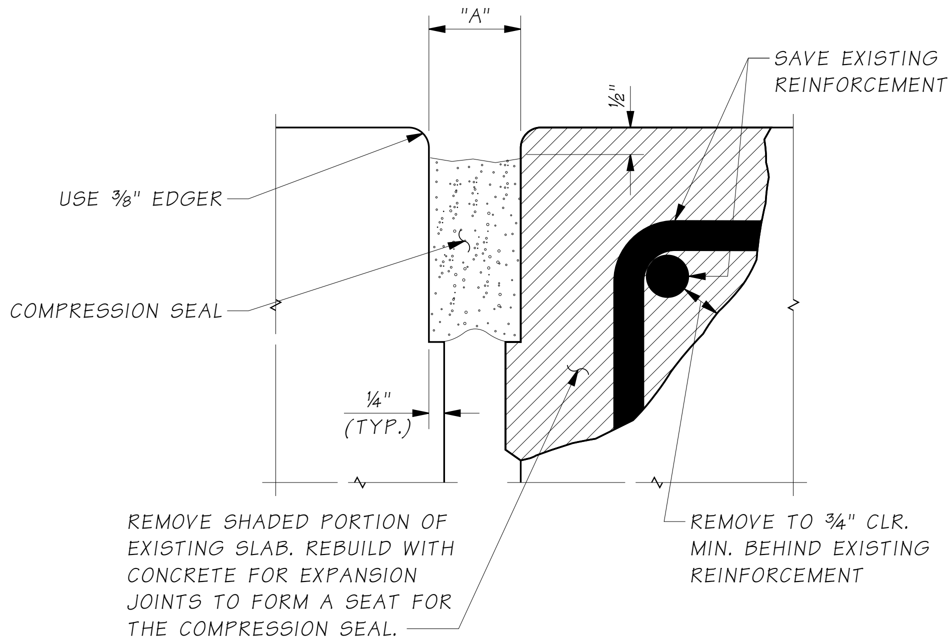
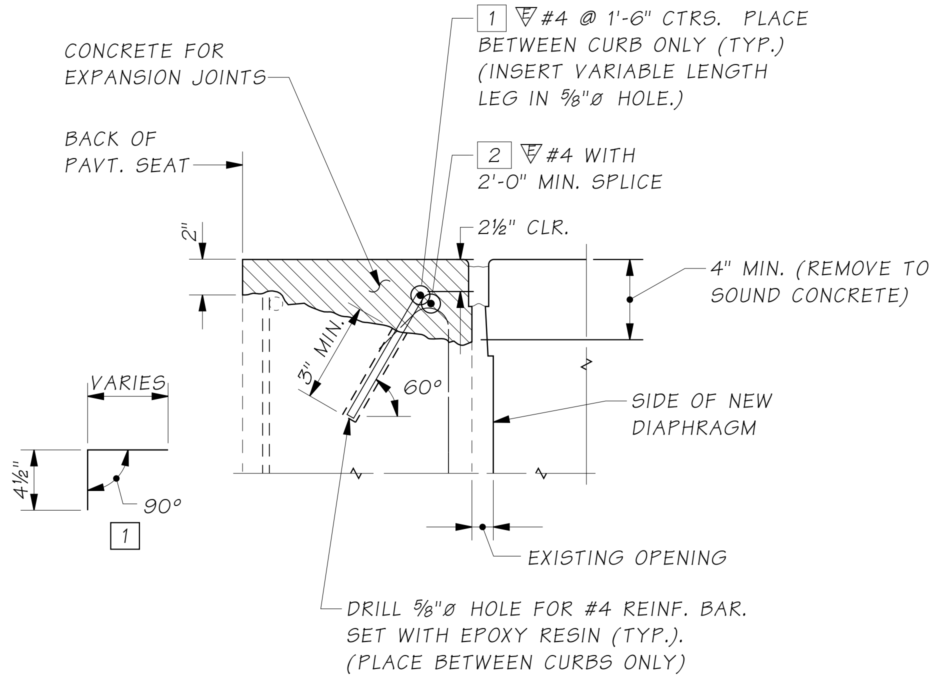


Figure 5.5.5-2 Expansion Joint Detail Shown for Compression Seal With New Reinforcing Steel Added



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.

5.6 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.

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 pre-tensioned or post-tensioned. The pre-tensioned 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 either spliced using ultra high performance concrete or 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 30 inches. 36 inch deep sections may be available but may not be as economical as similar single-web members. Designers should check availability with fabricators.

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 Appendices 5.6-A1-1 to 5.6-A1-9.

Standard drawings for WSDOT prestressed concrete girders are shown in the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm).

Table 5.6.1-1 Section Properties of WSDOT Standard Prestressed Concrete Girders

Type	Depth (in)	Area (in^2)	Y_b (in)	I_x (in4)	I_y (in^4)	J (in^4)	Wt. (k/ft)	Volume-to-Surface Ratio (in)	Max. Span Capability (ft)
W42G	42	373.25	18.94	76092	5408	4670	0.428	2.77	85
W50G	50	525.5	22.81	164958	13363	8509	0.602	3.12	110
W58G	58	603.5	28.00	264609	17065	9499	0.692	3.11	120
W74G	73.5	746.7	38.08	546110	34759	10489	0.856	2.90	150
WF36G	36	690.8	17.54	124772	71291	13997	0.792	3.24	110
WF42G	42	727.5	20.36	183642	71406	14428	0.834	3.23	125
WF50G	50	776.5	24.15	282559	71559	15004	0.890	3.22	135
WF58G	58	825.5	27.97	406266	71712	15580	0.946	3.21	150
WF66G	66	874.5	31.80	556339	71865	16155	1.002	3.20	165
WF74G	74	923.5	35.66	734356	72018	16731	1.058	3.19	175
WF83G	82.625	976.4	39.83	959393	72184	17352	1.119	3.19	180
WF95G	94.5	1049.1	45.60	1328995	72411	18207	1.202	3.18	195
WF100G	100	1082.8	48.27	1524912	72516	18602	1.241 {1.015}	3.17	205 (203*) {210}
WF100G with 5'-1" Top Flange	100	1118.8	49.89	1612834	99849	18714	{1.049}	3.06	{230}
12″ x 4' Solid Slab	12	564	6.0	6768	-	-	0.646	4.78	30
18″ x 4' Voided Slab	18	655	9.0	21876	-	-	0.751	3.05	45
24" x 4' Voided Slab	24	741	12.0	48179	-	-	0.849	3.08	60
26″ x 4' Voided Slab	26	835	13.0	62874	-	-	0.957	3.41	65
30″ x 4'-4" Voided Slab	30	1021	15.0	104444	-	-	1.170	3.71	75
U54G4	54	1038.8	20.97	292423	493926	19520	1.190	3.51	125
U54G5	54	1110.8	19.81	314382	788289	20390	1.273	3.47	130
U66G4	66	1208.5	26.45	516677	637751	22285	1.385	3.51	145
U66G5	66	1280.5	25.13	554262	997354	23153	1.467	3.47	145
U78G4	78	1378.2	32.06	827453	798969	24971	1.579	3.51	155
U78G5	78	1450.2	30.62	885451	1227303	25808	1.662	3.48	165 (152*)
UF60G4	60	1207.7	26.03	483298	639795	22376	1.384	3.48	145
UF60G5	60	1279.7	24.74	519561	999184	23246	1.466	3.45	145
UF72G4	72	1377.4	31.69	787605	800958	25127	1.578	3.48	155
UF72G5	72	1449.4	30.26	844135	1229061	26008	1.661	3.45	160 (152*)

Weight and span capability are based on normal weight concrete with a unit weight of 165 pcf, except as noted. Values in {} are based on lightweight concrete with a unit weight of 135 pcf.

* The span capability figure represents the length at which the section weighs 262 kips.

5.6.2 Design Criteria

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

AASHTO LRFD Section 5.12.3.3 “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 5.6.2-1 Design Criteria for Prestressed Concrete Girders

Design Specifications	AASHTO *LRFD Specifications* and WSDOT *Bridge Design Manual*
Design Method	Prestressed concrete members shall be designed for service limit state for stress limits and checked for strength limit state for ultimate capacity.
Superstructure Continuity	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.
Loads and Load Factors	Service, Strength, Fatigue, and Extreme Event Limit State loads and load combinations shall be in accordance with AASHTO *LRFD Specifications*
Stress Limits	Table 5.2.1-1
Prestress Losses	Section 5.1.4
Shear Design	AASHTO LRFD Section 5.7 and Section 5.2.2.B
Shipping and Handling	Section 5.6.3
Continuous Structure Configuration	Girder type, depth and number of lines shall be identical in adjacent spans. Girder type, depth and number of lines may be changed at expansion piers.
Girder End Skew Angle	Girder end skew angles for prestressed concrete slabs, deck bulb-tees, WFDG girders, WFTDG girders and tubs shall be limited to 30°. Girder end skew angles for all other prestressed concrete girders shall be limited to 45°.
Intermediate Diaphragms	Except for Prestressed Concrete Wide Flange Deck Girder and Prestressed Concrete Slab Girder bridges, CIP concrete intermediate diaphragms shall be provided for prestressed concrete girder bridge spans in the following situations: • Spans crossing a roadway with a minimum vertical clearance of 20'-0" or less. • Spans crossing a railway with a minimum vertical clearance of 23'-4" or less from the top of rail. • Spans crossing a water body or waterway with a minimum vertical clearance of 6'-0" or less from the 100-year MRI water surface level. • Spans that will possibly or likely have vehicular traffic under the span in the future with a minimum vertical clearance of 20'-0" or less. Intermediate diaphragms shall be equally spaced between bearing centerlines at a spacing not to exceed 50'. Intermediate diaphragms shall be either partial or full depth as described in Section 5.6.4.C.4.

5.6.2.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.

5.6.2.B Composite Action

5.6.2.B.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.

5.6.2.B.2 *Load Application*

The following sequence and method of applying loads is typically used in girder analysis:

1. Girder dead load is applied to the girder section.
2. Diaphragm dead load is applied to the girder section.
3. Deck slab dead load is applied to the girder section.
4. Superimposed dead loads (such as barriers, sidewalks and overlays) and live loads are applied to the composite section.

5.6.2.B.3 *Composite Section Properties*

A CIP concrete bridge deck forms the top flange of the composite girder in prestressed concrete girder bridge construction.

1. **Effective and Transformed Flange Width**

   The effective flange width of a concrete bridge deck for computing composite section properties shall be in accordance with AASHTO LRFD Section 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}.

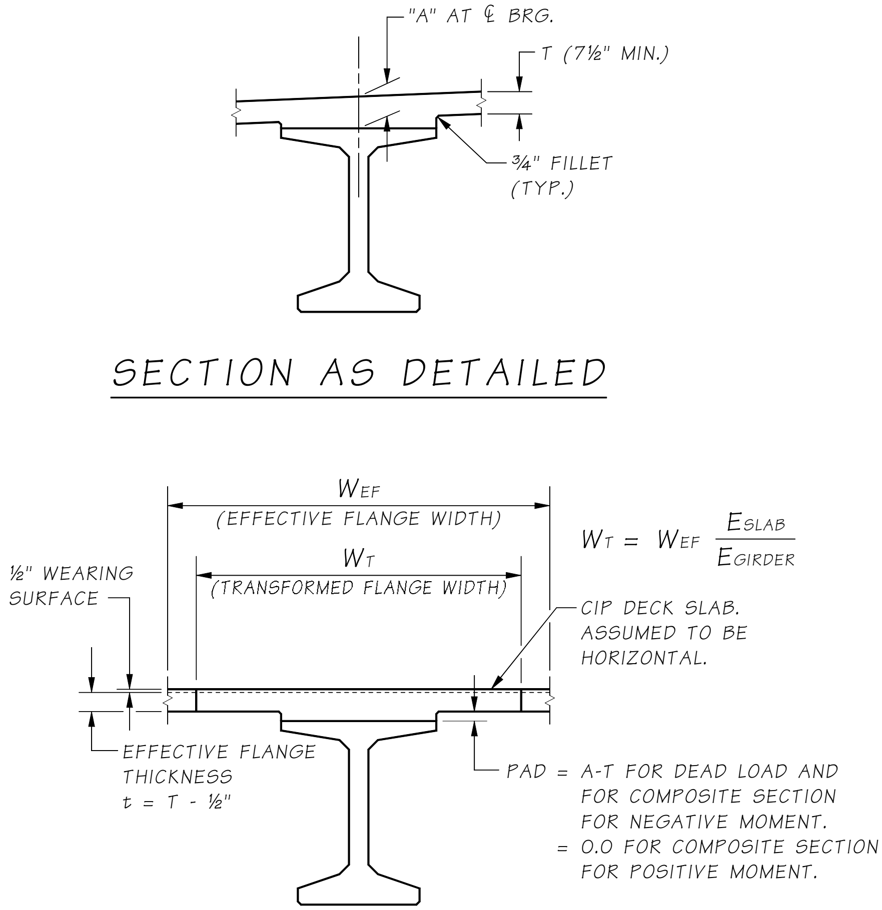
2. **Effective Flange Thickness**

   The effective flange thickness of a concrete bridge deck for computing composite section properties shall be the deck thickness reduced by ½″ 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.

5.6.2.B.4 *Slenderness Ratio*

The lateral slenderness ratio of WF girders *l/r*, where *l* is the girder length and *r* is the radius of gyration about the weak axis, shall not exceed 310. WF100G girders reach this limit at a length of 210 ft. The top flange width of WF100G girders can be increased to 5’-1” for spans greater than 210 ft.

Figure 5.6.2-1 Typical Section for Computation of Composite Section Properties



1. **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.

   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.

2. **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.

5.6.2.C Design Procedure

5.6.2.C.1 *General*

The WSDOT Prestressed concrete girder design computer program PGSuper is the preferred method for design.

5.6.2.C.2 *Stress Conditions*

The stress limits as described in Table 5.2.1-1 shall not be exceeded for prestressed concrete girders at all stages of construction and in service. The stages of construction for which stress limits shall be checked shall include, but not be limited to the following:

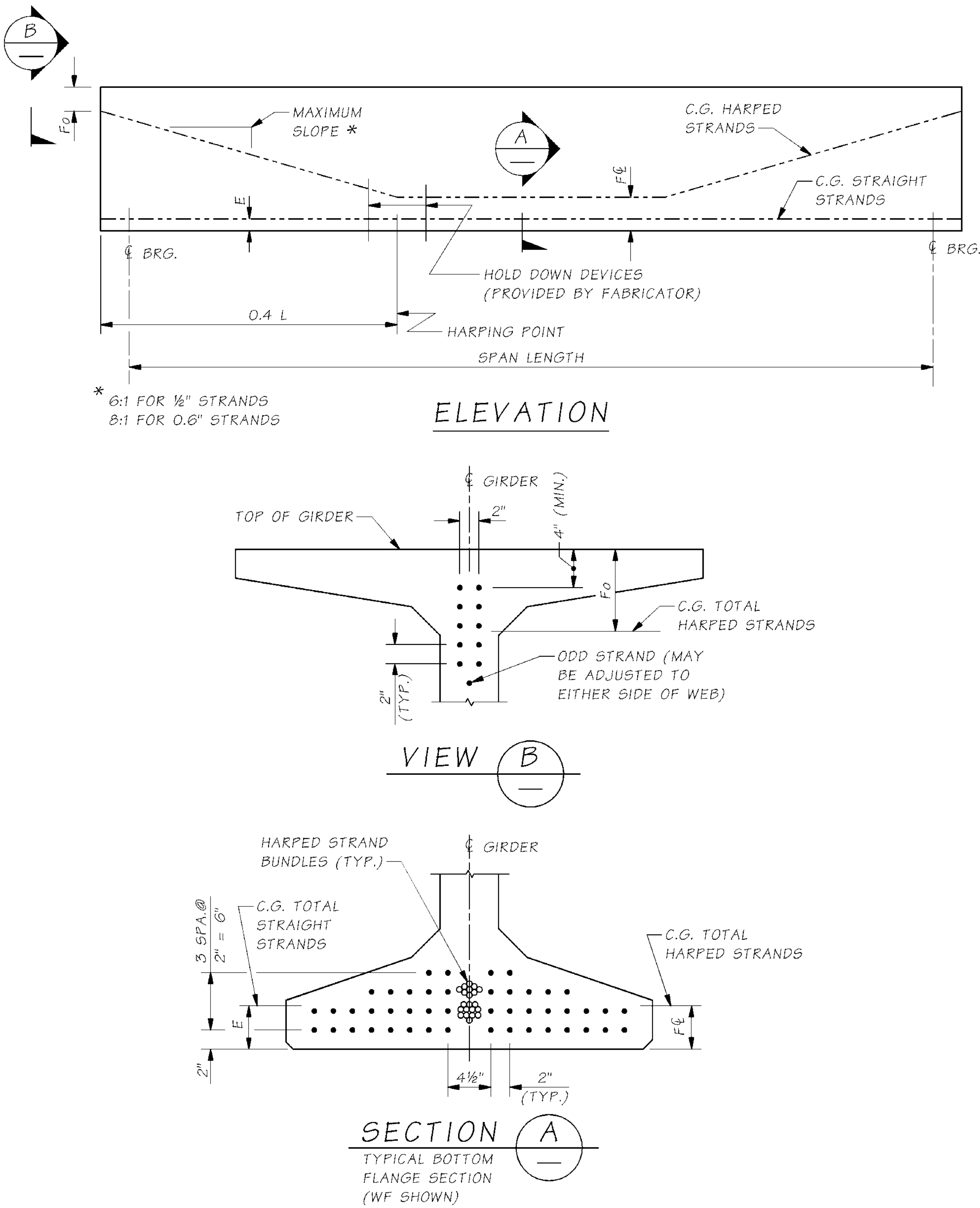
1. Prestressing release at casting yard using Service I Limit State
2. Lifting at casting yard using Service I Limit State. Dead load impact need not be considered during lifting. This check shall be done in accordance with Section 5.6.3.C.2.
3. Shipping for a girder with impact using Service I Limit State. A dead load impact of 20 percent shall be included acting both up and down. The angle of inclination shall be the equilibrium tilt angle computed for lateral stability with a roadway superelevation of 2 percent. This check shall be done in accordance with Section 5.6.3.D.6. This condition represents the girder traveling along a straight road at a typical 2% superelevation with dynamic load effects.
4. Shipping for a girder without impact using Service I Limit State. Dead load impact, wind and centrifugal forces need not be included. The angle of inclination shall be the equilibrium tilt angle computed for lateral stability with a roadway superelevation of 6 percent. This check shall be done in accordance with Section 5.6.3.D.6. This condition represents the girder going slowly through a corner with a 6 percent superelevation.
5. Deck and diaphragm placement using Service I Limit State
6. Final condition without live load using Service I Limit State
7. Final condition with live load using Service I Limit State for compressive stresses and Service III Limit State for tensile stresses
8. Final condition with live load using Fatigue I Limit State

When dead load impact is included in construction checks, the deflection and sweep induced by the dynamic component need not be considered when performing stress and stability checks.

5.6.2.D Standard Strand Locations

Standard strand locations of typical prestressed concrete girders are shown in Figure 5.6.2-2 the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm).

Figure 5.6.2-2 Typical Prestressed Concrete Girder Configuration



5.6.2.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 WSDOT State Bridge and Structures Architect during final design where required.

The four end types are shown as follows:

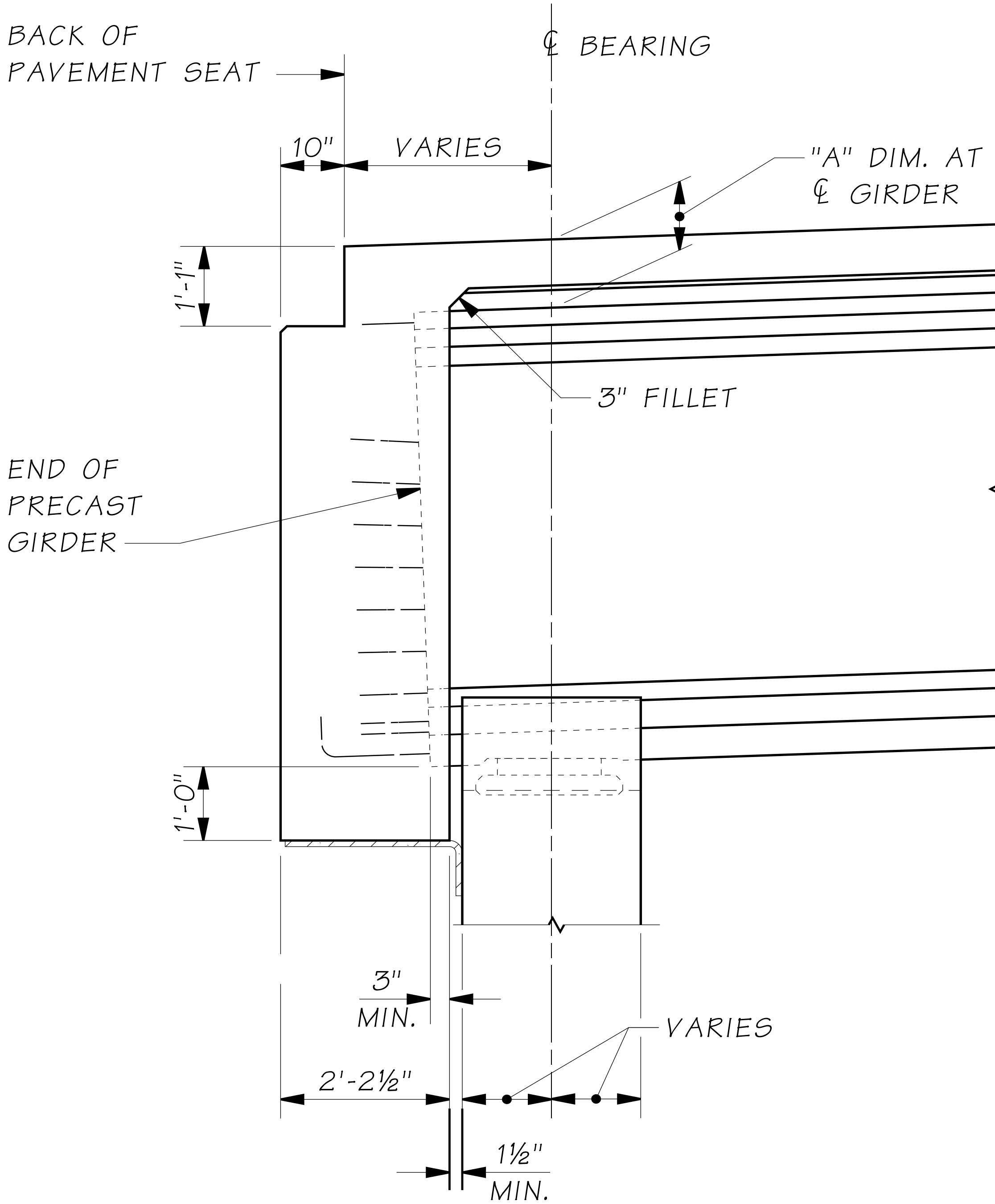
5.6.2.E.1 *End Type A*

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 Bridge Standard Drawings 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 pre-tensioned strands project from the end of the girder. The designer shall assure that these bars and strands fit into the end diaphragm. Extended prestressing strands should be capable of developing seismic connection forces, but at least 4 strands should be extended. 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)

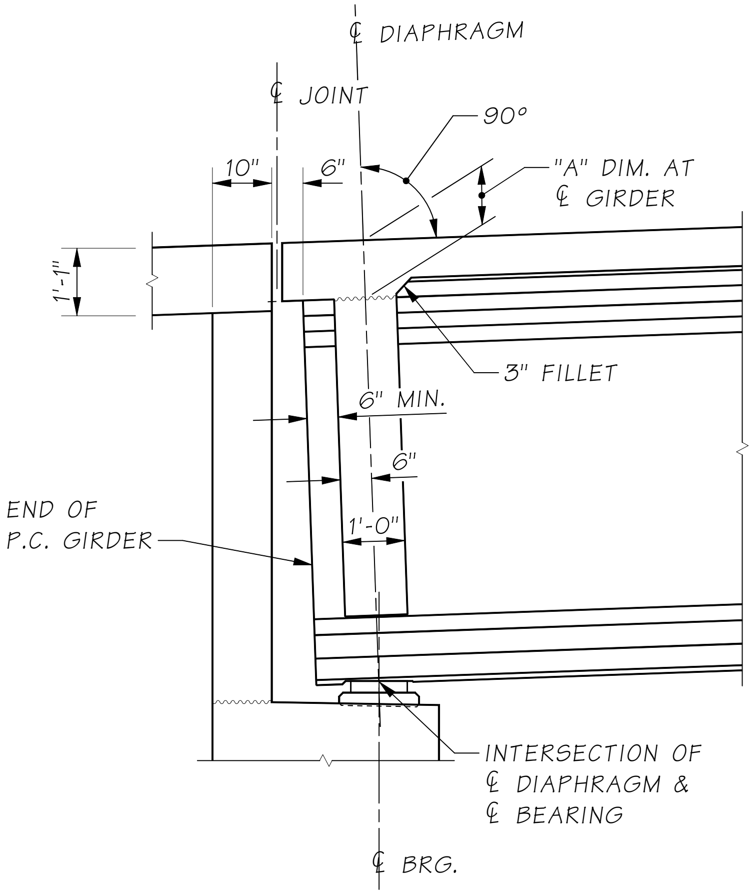


5.6.2.E.2 *End Type B*

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.

Figure 5.6.2-4 End Type B (L-Shape End Pier)



5.6.2.E.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. This detail may be used to reduce the seismic demand at an intermediate pier by allowing rotation about the axis parallel to the crossbeam. The reduced pier stiffness will lower the plastic overstrength shear demand (V_{po}), allow for shorter columns and eliminate the plastic hinge at the top of each column. While the diaphragm hinge is intended to act as a pin, there may be some residual stiffness at the connection that shall be determined by the designer. This stiffness will move the point of inflection down the pier, inducing some plastic overstrength shear demand.

The hinge connection should be assumed pinned to determine the pier displacement and ductility demand for seismic analysis.

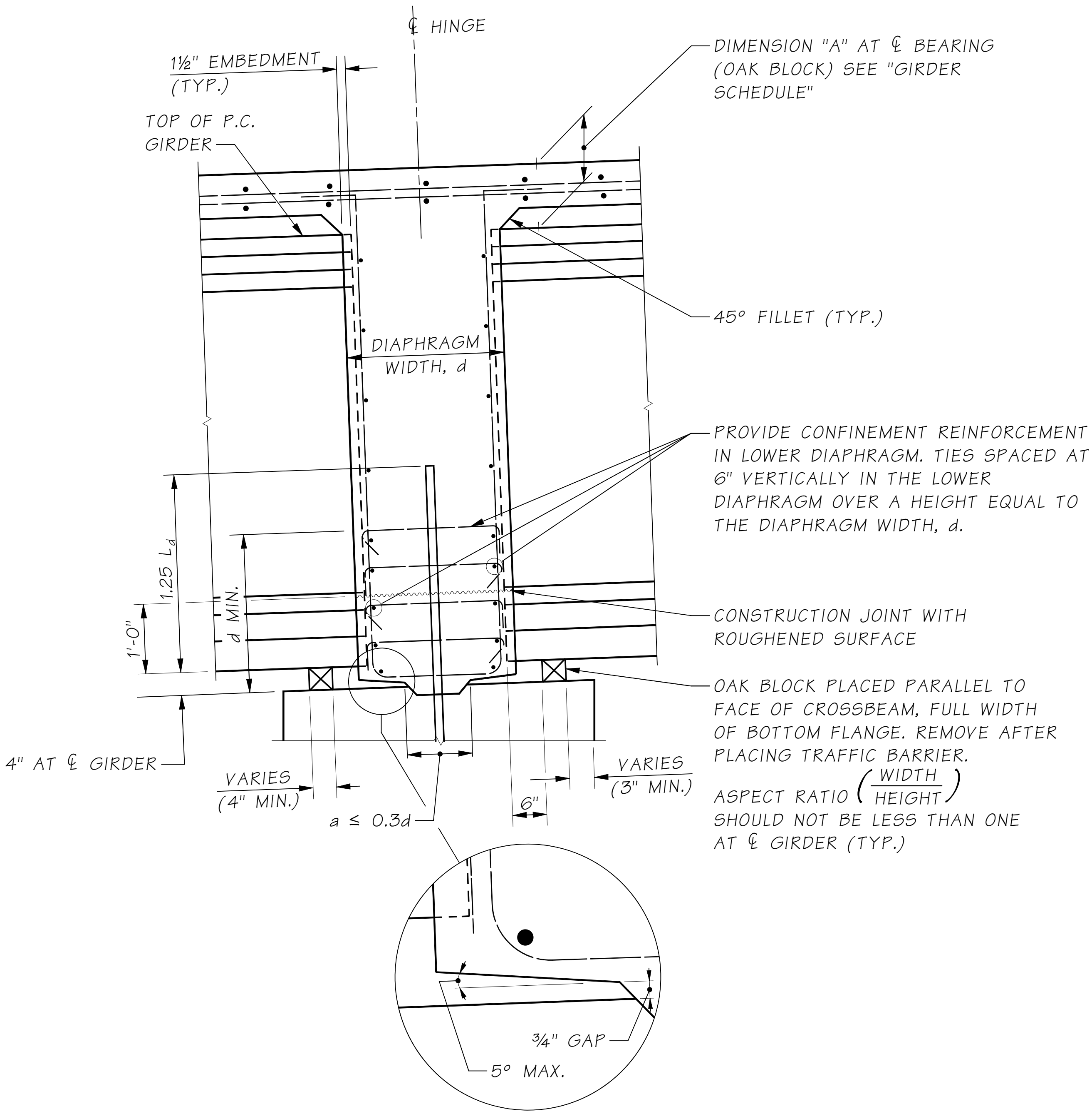
The designer shall check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting cross beam for load from the oak block including dead loads from girder, deck slab, and construction loads.

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

1. Check size and minimum embedment in crossbeam and diaphragm for hinge bars. Bars shall be sized based on interface shear due to calculated plastic overstrength shear force (V_{po}) from the column while ignoring the concrete cohesion and axial load contributions.
2. Design the width of the shear key to take the factored vertical bearing force per AASHTO LRFD Section 5.6.5 at the Strength limit state. The maximum shear key width shall be limited to 0.3d, where d is the width of the diaphragm.
3. Confinement reinforcement shall be added to the diaphragm between the girders over a vertical distance equal to or greater than the diaphragm width. Confinement shall be no less than #4 ties bars spaced at 12 inches longitudinally and staggered 6 inches vertically.
4. The throat of the hinge gap shall be no larger than 0.75 inches The bottom of diaphragm may taper up to 5 degrees maximum to allow for 1.5 times the elastic service, strength or extreme rotation. The material used to form the gap shall be strong enough to support the wet concrete condition and shall be removed after concrete placement.
5. Check interface shear friction at girder end (see Section 5.2.2.C.2).

Design of the pier in the transverse direction (parallel to the crossbeam axis) shall be performed per the AASHTO Seismic Guide Specifications.

Figure 5.6.2-5 End Type C (Intermediate Hinge Diaphragm)

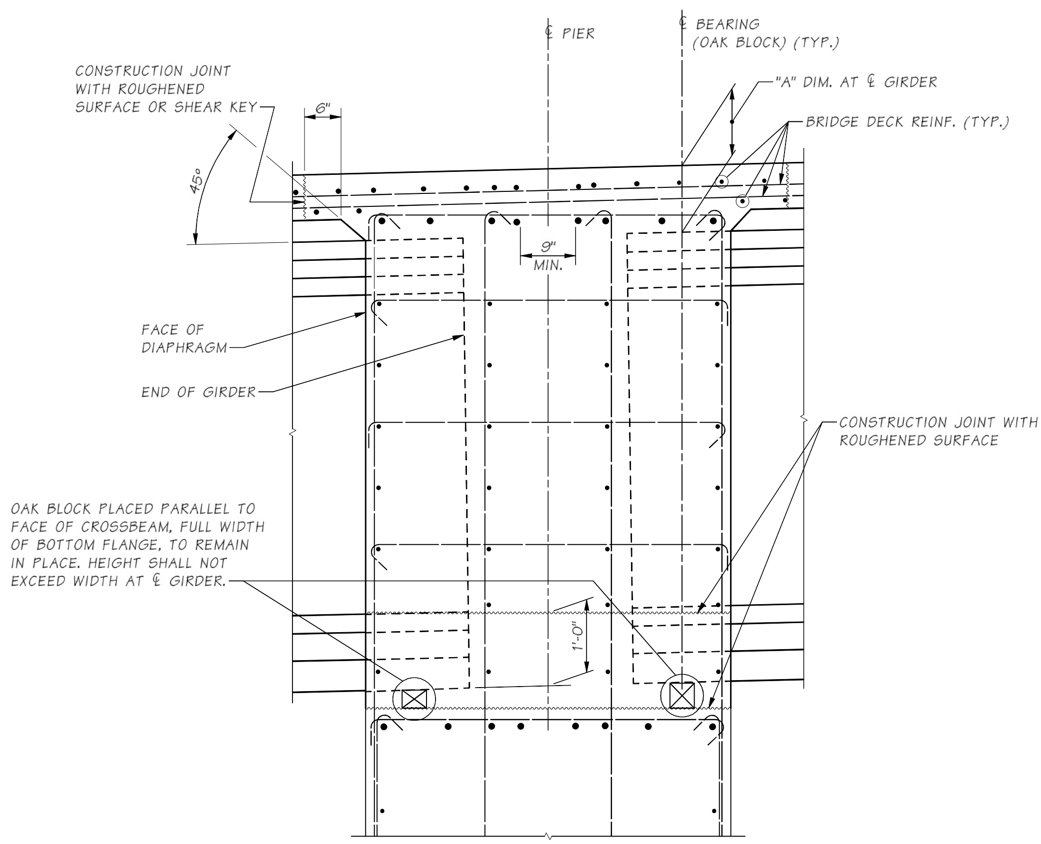


5.6.2.E.4 *End Type D*

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 the girder and construction loads. The designer shall check interface shear friction at the girder end (see Section 5.2.2.C.2).

Figure 5.6.2-6 End Type D



5.6.2.F Splitting Resistance in End Regions of Prestressed Concrete Girders

The splitting resistance of pre-tensioned anchorage zones shall be as described in AASHTO LRFD Section 5.9.4.4.1. For pre-tensioned I-girders or bulb tees, the end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 2½″. For pre-tensioned tub girders, the end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 3″. Any remaining splitting reinforcement not fitting within the h/4 zone may be placed beyond the h/4 zone at the minimum spacing allowed.

5.6.2.G Confinement Reinforcement in End Regions of Prestressed Concrete Girders

Confinement reinforcement in accordance with AASHTO LRFD Section 5.9.4.4.2 shall be provided.

5.6.2.H Girder Stirrups

Except as otherwise permitted, for girders with CIP deck slabs, girder stirrups shall be field bent over the top mat of reinforcement in the bridge deck.

Stirrups for slab and wide flange thin deck girders which shall be bent at the heigh shown in the standard girder plans.

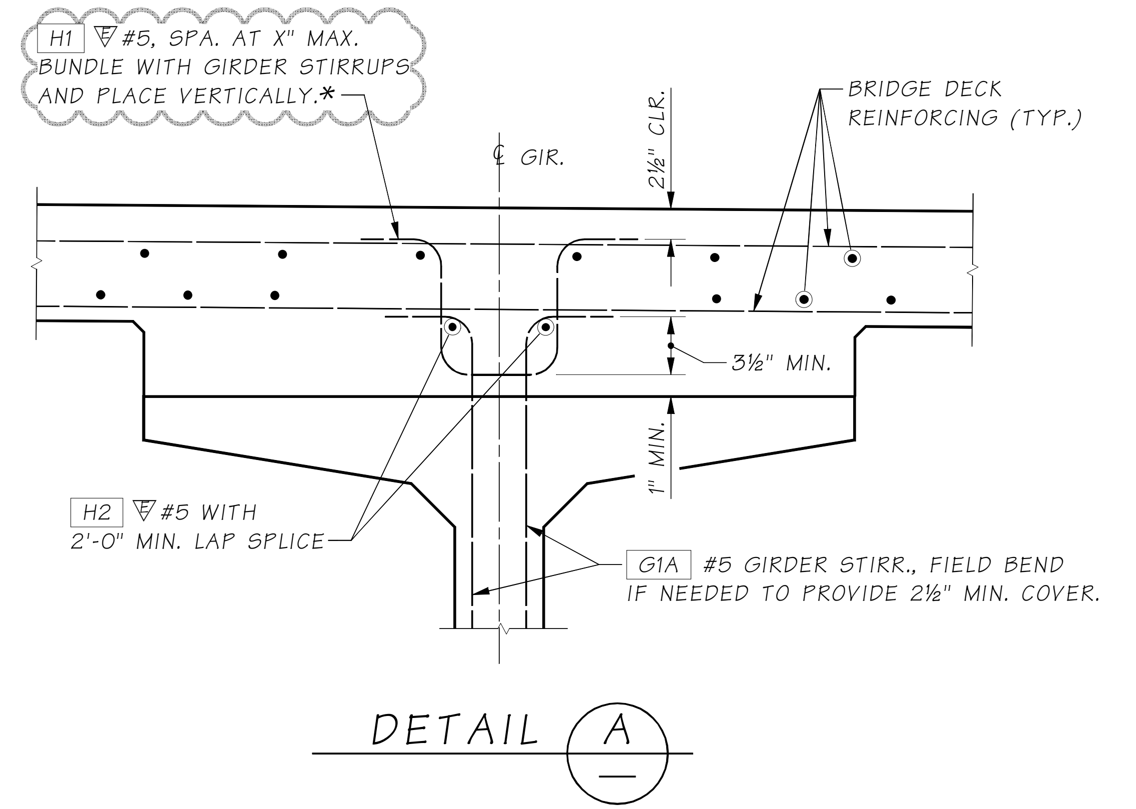
I-girder stirrups may be prebent, but the extended hook shall be within the core of the slab (the inside edge of the hook shall terminate above the bottom mat deck slab bars). For I-girders with a 7½” minimum thickness cast-in-place bridge deck, girder stirrups no larger than #5 bars, and with or without permanent precast prestressed concrete stay-in-place deck panels, prebent stirrups may be used with “hat bar” stirrup extensions. Details shall conform to Figure 5.6.2-7 and the following requirements (see reference 27):

- Girder stirrups shall all extend at least 5” from the top of the girder, but typically no more than the deck thickness minus 2.5”.
- Hat bars shall be epoxy coated and shall be the same bar size as the girder stirrups

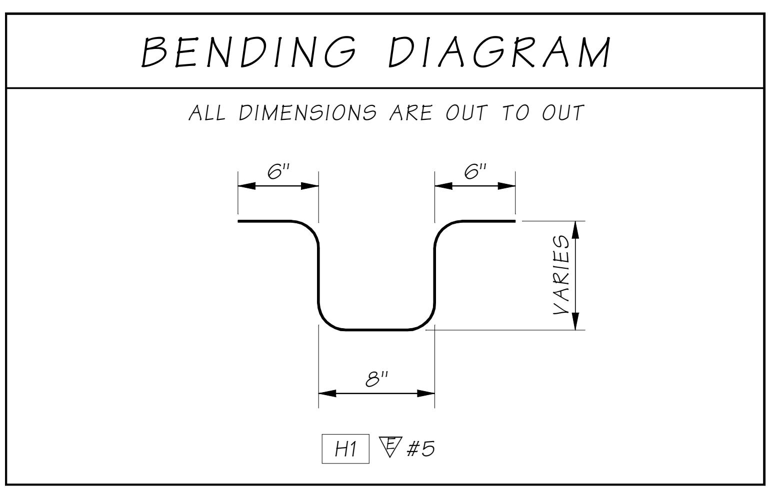
5.6.2.I Section Properties

Gross section properties (including the gross deck area transformed by the girder/deck modular ratio if applicable) should be used for design of precast concrete girders including prestress losses, camber, and flexural capacity.

Figure 5.6.2-7 Hat Bar Plan Details



* H1 #5 MAY BE OMITTED AT LOCTIONS WHERE GIRDER STIRRUPS PROJECT AT LEAST 3" ABOVE THE BOTTOM OF THE TRANSVERSE BAR IN THE BOTTOM MAT OF THE BRIDGE DECK.



5.6.3 Fabrication and Handling

5.6.3.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.

5.6.3.B Special Problems for Fabricators

5.6.3.B.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″ ϕ strands and 8:1 for 0.6″ϕ strands. The straight strands are tensioned by straight jacking.

5.6.3.B.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.

5.6.3.B.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 pre-tensioned concrete girders, the number of permanent prestressing strands (straight and harped) shall be limited to 100 total 0.6″ϕ strands.

5.6.3.C Handling of Prestressed Concrete Girders

5.6.3.C.1 *In-Plant Handling*

The maximum weight that can be handled by precasting plants in the Pacific Northwest is 262 kips. It is possible that products heavier than 262 kips can be manufactured if they will be shipped directly to the jobsite by barge, but special plant lifting arrangements must be investigated. Headroom is also not generally a concern for the deeper sections.

5.6.3.C.2 *Lateral Stability During Handling*

In order to ensure constructability, the designer shall specify the lifting embedment locations (centroid 3′ minimum from ends - see *Standard Specifications* Section 6-02.3(25) L), maximum midspan vertical deflection and the corresponding concrete strength at release that satisfies the stress limits from Section 5.2.1.C and provides an adequate factor of safety for lateral stability. The calculations shall conform to methods as described in *Standard Specifications* Section 6-02.3(25) and reference 26. Factors of safety of 1.0 against cracking and 1.5 against failure shall be used.

Biaxial stresses due to lateral bending at the girder tilt equilibrium condition shall consider the assumed lifting embedment transverse placement tolerance and the girder sweep tolerance. Stresses shall be evaluated for the girder tilt equilibrium condition for a hanging girder as described in *Standard Specifications* Section 6-02.3(25) and reference 26.

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:

1. 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.
2. Select a girder section that is relatively wide and stiff about its vertical (weak) axis.
3. Add temporary prestressing in the top flange.
4. Brace the girder.
5. Raise the roll axis of the girder with a rigid yoke.

5.6.3.D Shipping Prestressed Concrete Girders

5.6.3.D.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.

5.6.3.D.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 may be 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.

5.6.3.D.3 *Weight Limitations*

The net weight limitation with trucking equipment currently available in Washington State is approximately 180 kips, if a reasonable delivery rate (number of pieces per day) is to be maintained. Product weights of up to 262 kips can be hauled with currently available equipment at a limited rate. The hauling of heavier girders may be possible with coordination with hauling subcontractors. Hauling subcontractors should be consulted on the feasibility of shipping large or heavy girders on specific projects.

5.6.3.D.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 girder at 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.

5.6.3.D.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.

5.6.3.D.6 *Lateral Stability During Shipping*

In order to ensure constructability, the designer shall specify concrete strengths, shipping support locations, minimum shipping support rotational spring constants, shipping support center-to-center wheel spacing, maximum midspan vertical deflection at shipping and temporary top strand configurations in the Plans that satisfy the stress limits from Section 5.2.1.C and provide adequate factors of safety for lateral stability during shipping. The calculations shall conform to methods described in *Standard Specifications* Section 6-02.3(25) and reference 26. Factors of safety of 1.0 against cracking and 1.5 against failure and rollover shall be used. The maximum midspan vertical deflection at shipping used to evaluate stability shall be shown in the plans. In order to minimize the need for re-analysis under contract, this value may be conservatively determined using losses at 10 days, camber at 90 days, and a span length equal to the girder length.

The rotational stiffness and center-to-center wheel spacing used in design shall be taken from Table 5.6.3-1. Design the girder for transportation with the least stiff support system as possible while achieving recommended factors of safety.

Table 5.6.3-1 Shipping Support Parameters

Shipping Support Rotational Spring Constant, K_θ (Kip-in/radian)	Shipping Support Center-to-Center Wheel Spacing, W_{cc}
40,000	72
50,000	72
60,000	72 or 96
70,000	96
80,000	96

Design for shipping should not preclude the contractor from making modifications under contract that consider actual conditions, such as fabrication tolerances and the haul route, but the Engineer should confirm that any proposed changes are structurally acceptable in the final in-service structure.

5.6.3.E Erection and Bridge Deck Construction

A variety of methods are used to erect prestressed concrete girders, depending on the weight, length, available crane capacity, and site access. Generally, 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 appurtenances such as deck overhang formwork brackets are attached to the girder, the lifting slings are at an angle other than 90° ± 10° from the top surface of the girder, wind loads are present, the girder is erected in such a manner that one end is supported at the top by a lifting device while the other end is seated on a girder transport vehicle or bearing, or a separate set of erection devices are used, the girder shall be checked for stresses and lateral stability as part of the contractor's erection plan. 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. For tub girders, designers should consider web out-of-plane bending forces that will develop during construction. Examples of these loads include forces developed by lift rigging and loading due to the deck finishing machine. These cases may require temporary struts and govern the design of web stirrups.

For long-span girders and other girders where the lifting loops are placed far from the girder ends to accommodate lifting at release, designers should consider specifying additional lifting loops closer to the girder ends that can be used during erection. When setting girders on bearings during erection, there can be end rotations of the girder that cause horizontal shear in bearings. The magnitude of this shear force or deformation should be considered. Placing lifting loops closer to girder ends minimizes the magnitude of girder end rotation.

5.6.3.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 on the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm) 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

5.6.4.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.

5.6.4.A.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.

5.6.4.A.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.

5.6.4.A.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 percent of girder spacing; then the exterior girder can use the same design as that of the interior girder. The following guidance is suggested.

1. **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.

2. **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.

3. **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.

4. **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.

5.6.4.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.

5.6.4.B.1 *Appearance*

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

5.6.4.B.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.

5.6.4.B.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.

5.6.4.B.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.

5.6.4.B.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.

5.6.4.C Diaphragm Requirements

5.6.4.C.1 *General*

Intermediate diaphragms provide girder stability for the bridge deck placement and improve the bridge resistance to over-height impact loads.

Diaphragms for prestressed concrete girder bridges shall be cast-in-place concrete. 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.

Vertical reinforcement for intermediate diaphragms may be terminated at the top of top flange if SIP deck panels are used for the bridge deck.

5.6.4.C.2 *Design*

Diaphragms shall be designed as transverse beam elements carrying both dead load and live load.

5.6.4.C.3 *Geometry*

Diaphragms shall normally be oriented parallel to skew (as opposed to normal to girder centerlines). This procedure has the following advantages:

1. 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.
2. 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.
3. 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.

5.6.4.C.4 *Full or Partial Depth Intermediate Diaphragms*

Based on research done by Washington State University (WSU) on damage by over-height loads[24], the type of intermediate diaphragms for prestressed concrete girder bridges (including widenings) shall be as follows:

1. Full depth intermediate diaphragms as shown in the office standard plans shall be used for bridges crossing over roads of ADT > 50,000.
2. 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.

5.6.4.C.5 *Tub Girder Intermediate Diaphragms*

Where required, 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.

5.6.4.D Skew Effects

Skew in prestressed concrete girder bridges affects structural behavior and member analysis and complicates construction.

5.6.4.D.1 *Analysis*

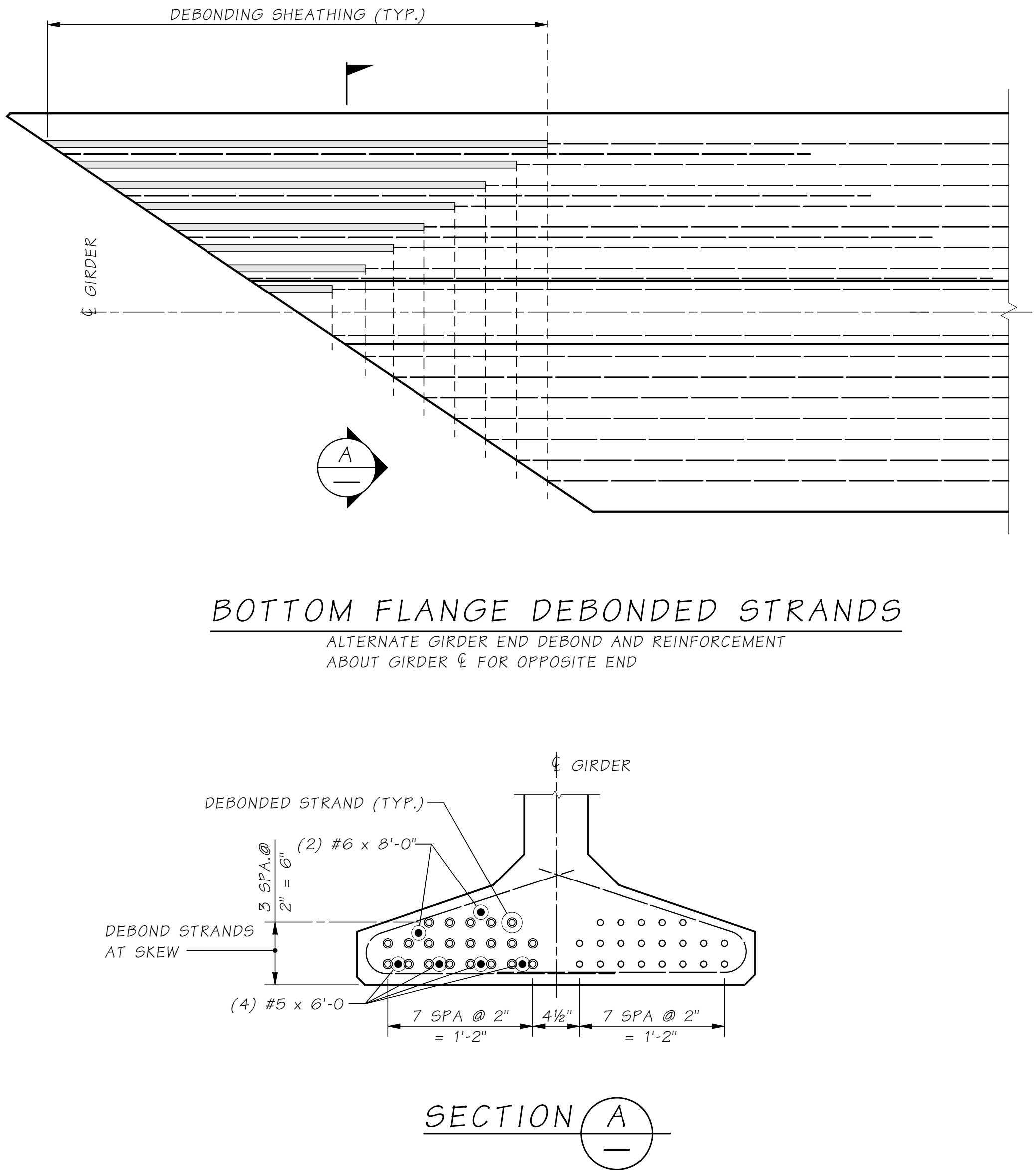
Normally, the effect of skew on girder analysis is ignored. It is assumed that skew has little structural effect on normal spans and normal skews. For short, wide spans and for extreme skews (values over 30°), the effect of the skew on structural action shall be investigated. All trapezoidal tub, slab, 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 has caused bottom flange cracking. Recent projects where this cracking occurred are Contract 8128 (Bridge Number 522/142N has W74G girders with 55 degree skew and 8 bottom flange straight strands) and Contract 8670 (Bridge Number 5/456E has WF100G girders with 56 degree skew and 40 bottom flange straight strands). Details shown in Figure 5.6.4-1 could be used to minimize bottom flange cracking for girders with large skews.

5.6.4.D.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.

Figure 5.6.4-1 Skewed Girder End Details to Prevent Cracking



5.6.4.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.

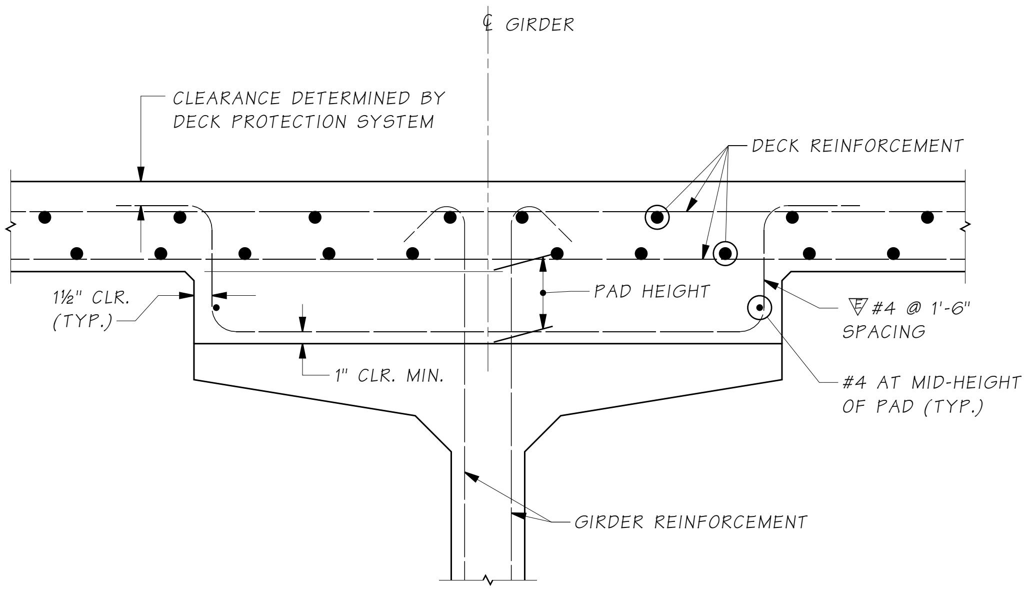
5.6.4.F Curve Effect and Flare Effect

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.

5.6.4.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 5.6.4-2.

Figure 5.6.4-2 Girder Pad Reinforcement



5.6.5 *Repair of Damaged Prestressed Concrete 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 shall 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 Prestressed Concrete Girders in Existing Bridges*

5.6.6.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. Over-height 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.

5.6.6.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 (see references [15, 16]).

5.6.6.B.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.

5.6.6.B.2 *Moderate Damage*

If damage is moderate, (damage does not exceed replacement criteria in Item 4 below), a repair procedure shall 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 steps shall be part of any proposed repair procedure:

1. **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.

2. **Restore Prestress If Needed**

   Prestress in damaged/severed strands can sometimes be restored with mechanical strand couplers. Damaged girders with broken 0.6” diameter strands may need to be repaired with 0.5” diameter strands and additional post-tensioning as needed. Current commercially-available couplers are capable of restoring full prestressing force in strands of up to ½” diameter. Verify that the restoration of full prestress force will not cause overstress in the damaged girder section.

3. **Prepare a Repair Plan**

   Draw a sketch to show the area of concrete removal required for replacement of damaged concrete, and for installation of any mechanical strand couplers required. The damaged area is to be thoroughly prepared, coated with epoxy, and repaired with grout equal in strength to the original concrete.

5.6.6.B.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 shall be replaced. This has been done several times, but involves some care in determining a proper replacement 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.

Casting the new bridge deck 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 casting the bridge deck will 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.

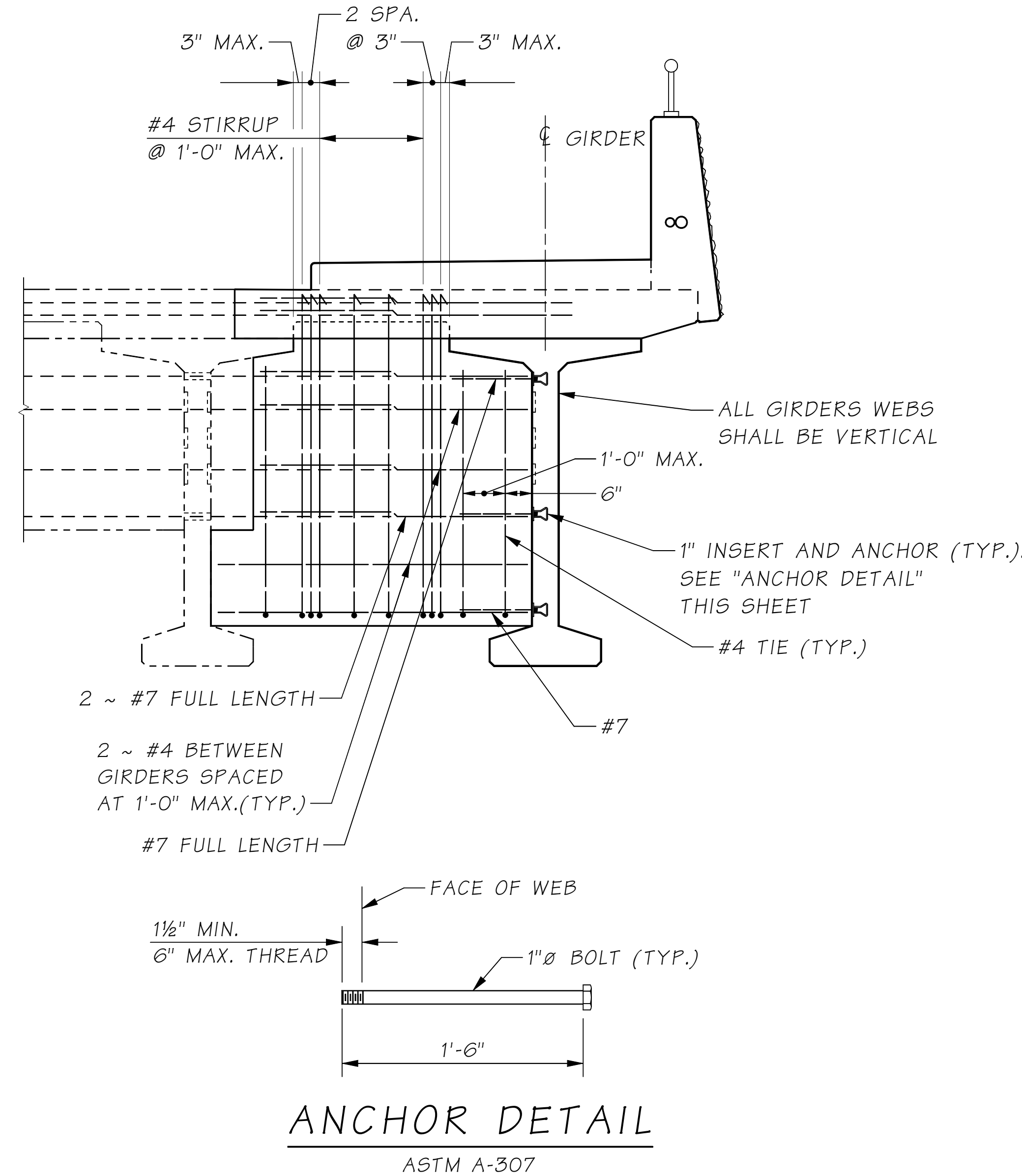
In case of replacement of a damaged girder, the intermediate diaphragms adjacent to the damaged girder shall be replaced with full depth diaphragms as shown in Figure 5.6.6-1.

In case of replacement of a damaged girder, the replacement girder should be of the same type or the same depth 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 Section 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



5.6.6.B.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 percent of prestressing strands are damaged/severed. If over 25 percent 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.
- **Significant Concrete Loss from the Web** – Significant damage of concrete in the web that results in loss of shear capacity shall require girder replacement. The web damage shall be considered significant when more than 25 percent of web section is damaged or when shear reinforcement has yielded.

Damaged girders shall be replaced in accordance with current WSDOT design criteria and with current girder series.

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

- **Significant Concrete Loss from the Bottom Flange** – 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 percent of the replacement project cost.
- **Continuous Girders** – Continuous girders with or without raised crossbeam that requires supporting falsework in the adjacent spans.
- **Superstructure Replacement** – Superstructure replacement shall be considered if more that 50 percent of all girders in the span are damaged or if there is a high risk of future impacts from over-height loads.

5.6.6.C Miscellaneous References

The girder replacement contracts and similar jobs listed in Table 5.6.6-1 should be used for guidance:

Table 5.6.6-1 Girder Replacement Contracts

Contract	Project Name	Bridge Number	Total Bridge Length (ft)	Year work planned	Work Description
7646	I-5 SR 11 Interchange Chuckanut Overcrossing Bridge	11/1	287	2009	Replace damaged PCG
8133	US 395 Court Street Bridge – Replace / Repair Girders	395/103	114	2011	Replace damaged PCG
8251	I-5 113th Ave SW Bridge Special Repair	5/309	204	2012	Replace damaged PCG
8220	SR 16 Olympic Drive NW Bridge Special Repair	16/120	207	2012	Replace damaged PCG
8218	SR 167 24th St. E Bridge Special Repair	167/38	382	2012	Replace damaged PCG
8489	I-5 Chamber Way Bridge Special Repair	5/227	185	2014	Replace damaged PCG
8598	SR 16 Olympic Drive NW Bridge Special Repair	16/120	207	2014	Replace damaged PCG
8801	I-5 NBCD Over 41st Division Dr. Special Repair	5/411NCD	172	2015	Replace damaged PCG
8810	I-5 Birch Bay Lynden Rd Bridge Bridge Repair	5/834	272	2015	Replace damaged PCG
8813	I-90, Front St Bridge 90/66S Girder Replacement	90/66S	231	2015	Replace damaged PCG
8828	I-90 Stampede Pass Interchange – Bridge Repair	90/113	151	2016	Replace damaged PCG span
9465	I-90 Danekas Rd Undercrossing – Bridge Repair	90/332	225	2019	Replace damaged PCG span

5.6.7 Deck Girders

5.6.7.A General

The term “deck girder” refers to a girder whose top flange or surface comprises a continuous driving surface, with or without an overlay or CIP topping. They include slab, double-tee, ribbed, deck bulb-tee, wide flange deck and wide flange thin deck girders

Unless noted otherwise deck girders that are not connected to adjacent girders shall use a Type 1 deck protection system; girders that only have shear connections with adjacent girders shall use a Type 3 or Type 4 deck protection system; and girders that have moment connections with adjacent girders shall use Type 2 or Type 3 deck protection systems. The requirements for bridge deck protection systems are covered in Section 5.7.4.

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

5.6.7.B Slab Girders

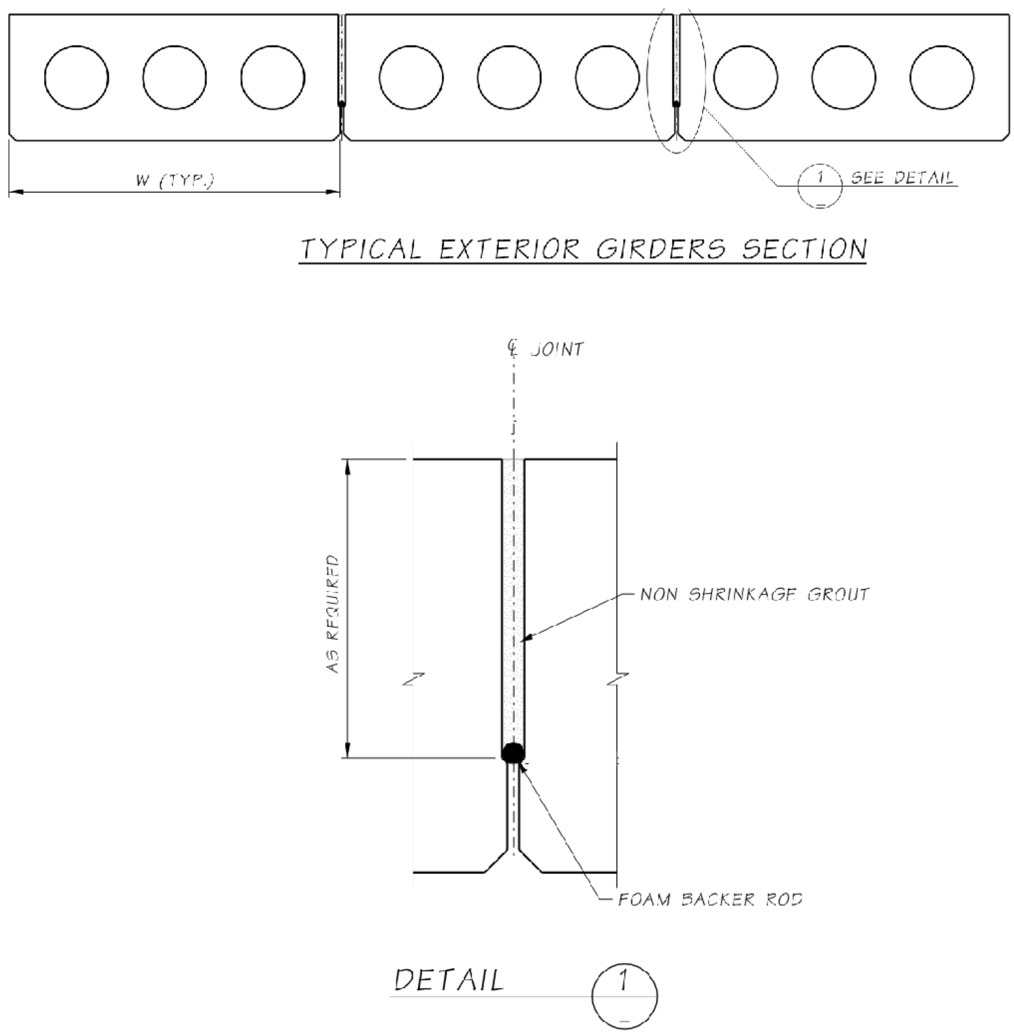
Slab girder spans between centerlines of bearing shall be limited to the prestressed concrete girder height multiplied by 30 due to unexpected variations from traditional beam camber calculations.

Standard configurations of slab girders are shown in the girder standard plans. The width of slab girders should not exceed 8′-0″. Designers should minimize the number of different widths of slabs on projects in order simplify fabrication. For slab girders bearing on reinforced soil, standard details shall be modified per Section 7.5.2.A.

Slab girder spans shall use a Type 4 deck protection system. The CIP topping slab shall be at least 5” thick. For bridges with ADT greater than 30,000, the CIP topping slab shall be at least 7” thick and contain two mats of transverse reinforcement. In the CIP topping slab, the longitudinal reinforcement shall be #5 bars spaced at 12 inches maximum and the transverse reinforcement shall be #5 bars spaced at 6 inches maximum.

Additional reinforcement may be required to resist vehicular collision loads on the traffic barrier. Designers should check the ability of the CIP topping slab to resist collision forces at joints between the girders. The joints between girders near the barrier may need to be filled with grout or concrete to provide a section depth capable of resisting the flexural load effects from collision forces on the barrier. See Figure 5.6.7-1.

Figure 5.6.7-1 Outer Slab Girder Joint



The AASHTO LRFD criteria for deflection 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.

Girder equalization, shear keys and weld ties are not required when a minimum 5″ composite CIP bridge deck is placed over slab girders. Differential camber is expected to be small but the designer should ensure it can be accommodated by the CIP deck.

Designers should ensure that the cross slope of girder supports are the same at both ends of each girder in order to prevent girder torsion, point loads, and gaps between the girder and the bearings.

Lateral restraint of slab girder superstructures with end type A at abutments shall be provided by external girder stops, one on each side of the bridge.

Utilities and appurtenances shall not be run through the voids in slab girders.

5.6.7.C Double-Tee and Ribbed Deck Girders

Double-tee and ribbed deck girders shall be limited to widening existing similar structures. A hot mix asphalt (HMA) overlay with membrane shall be specified. These sections are capable of spanning up to 60′.

5.6.7.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 155 feet. Deck bulb-tee girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

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.

5.6.7.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.

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 on roadways with superelevation transitions or sharp horizontal curvature. They shall be limited to spans where the pier skew angles are within 10° of each other. Designers should balance weight, prestress and camber between adjacent girders to improve fit-up.

Biaxial bending stress and the effect of an eccentric shear center shall be considered when roadway cross-slopes exceed 0.04 ft/ft.

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

1. Wide Flange Deck Girders with Mechanical Connections

   These girders rely on weld ties and a grouted keyway to connect adjacent girders.

   These girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

2. Wide Flange Deck Girders with UHPC Connections

   These girders rely on a short non-contact lap splice between extended transverse reinforcement in cast-in-place closures of ultra high performance concrete. A non-structural overlay shall be used on these bridges to protect the deck girders as well as accommodate differential girder camber. A polyester concrete overlay can be used. An HMA overlay can be used where the approach roadways are HMA. Modified concrete overlays may be considered, but special consideration is required to prevent debonding of the overlay from UHPC.

   These girders shall be limited to simple span bridges with roadway with cross- slopes of 0.04 ft/ft or less. WF39DG, WF45DG, and WF53DG girders may be erected with the web plumb or perpendicular to the roadway surface. Erect all other girders with the web plumb.

   Due to the risk of over height impacts and the difficulty of repairing UHPC connections, these bridges shall be limited to spans with at least 16’-6” of vertical clearance above roadways below.

   Precise fit-up between the top flanges of adjacent girders is necessary for a quality UHPC connection joint. When the ends of girders are skewed, top flange edges are vertically offset relative to one another due to camber. This is commonly known as the “saw tooth” effect. The “saw tooth” effect can be accommodated by negating the effects of camber with longitudinal top flange thickening or precamber or adjusting the bearing elevations so that adjacent top flanges align. Adjustments typically consist of raising one end of the girder and lowering the other to match the profile of the adjacent girder. This approach is only viable if the roadway profile is made to match the camber.

5.6.7.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.

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 using a Type 1 Deck Protection System. The deck shall be assumed to be 7” minimum in preliminary design, but may be reduced to as thin as 6” in final design.

Two mats of transverse reinforcement in the CIP bridge deck shall be designed to resist live loads and superimposed dead loads. The cover to the bottom of the bottom mat shall be 1” minimum. Bottom mat longitudinal bars are not required.

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.

5.6.8 Prestressed Concrete Tub Girders

5.6.8.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 formwork and 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.

5.6.8.B Curved Tub Girders

Curved tub girders may be considered for bridges with moderate horizontal radiuses.

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 in accordance with Section 5.8.1.F.
6. The clear spacing between the outside diameter of ducts shall be 2″ min. The duct diameter shall not exceed 4½″.

5.6.9 Prestressed Concrete Girder Checking Requirement

1. Shear reinforcing size and spacing shall be determined by the designer.
2. 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.
3. 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.
4. Check edge distance of supporting cross beam.

5.6.10 Review of Shop Plans for Pre-tensioned 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:

1. All prestressing strands shall be of ½″ or 0.6″ diameter grade 270 low relaxation uncoated strands.
2. Number of strands per girder.
3. Jacking stresses of strands shall not exceed $0.75f_{pu}$.
4. Strand placement patterns and harping points.
5. Temporary strand pattern, bonded length, 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. Locations 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. All horizontal and vertical reinforcement.
13. Girder length and end skew.

5.7 Bridge Decks

Concrete bridge decks shall be designed using the Traditional Design of AASHTO LRFD Section 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

5.7.1.A Minimum Bridge Deck Thickness

The minimum bridge deck thickness (including 0.5″ wearing surface) shall be 7.5″ for concrete girder 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″.

For bridge deck overhangs that support traffic barriers, the minimum thickness shall be 8”. This minimum is intended to satisfy crashworthiness requirements as well as provide clearance for hooked transverse bars in the deck.

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″.

5.7.1.B Computation of Bridge Deck Strength

The design thickness for usual bridge decks are shown in Figures 5.7.1-1 and 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.

Figure 5.7.1-1 Depths for Bridge Deck Design at Interior Girder

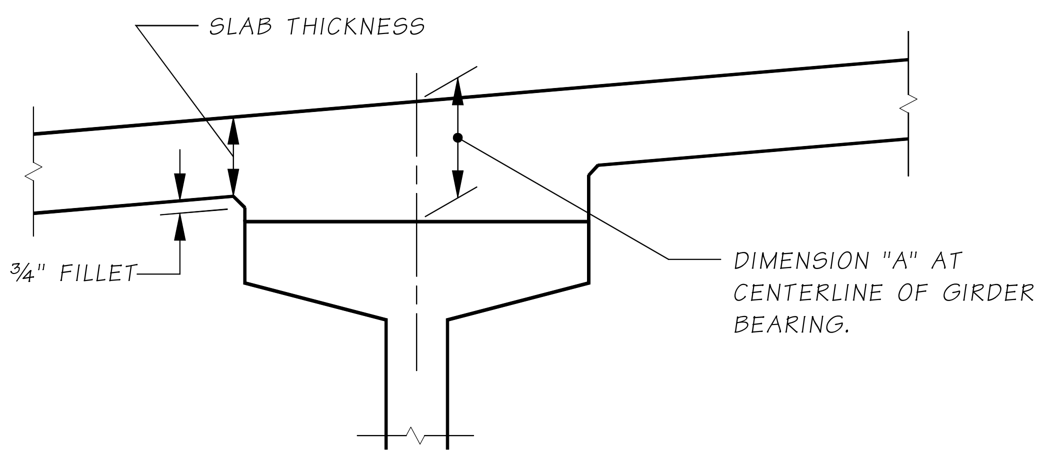
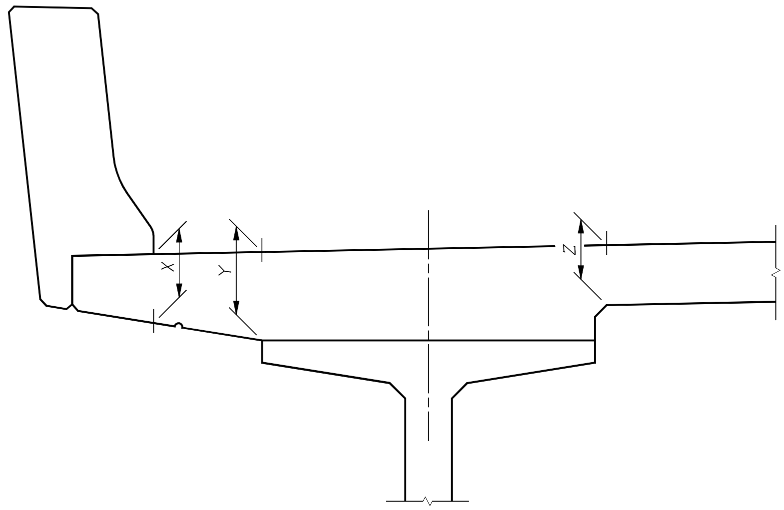


Figure 5.7.1-2 Depths for Bridge Deck Design at Overhang



5.7.1.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. 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).

5.7.1.D Staged Construction

Bridge decks constructed in stages shall have a longitudinal closure pour between adjacent stages. Closure pours allow for dead load deflection, balanced load distribution and isolation of decks from vibration caused by adjacent traffic. After closure pour concrete is placed, traffic should be restricted from early-age loading of the concrete.

5.7.2 Bridge Deck Reinforcement

5.7.2.A Transverse Reinforcement

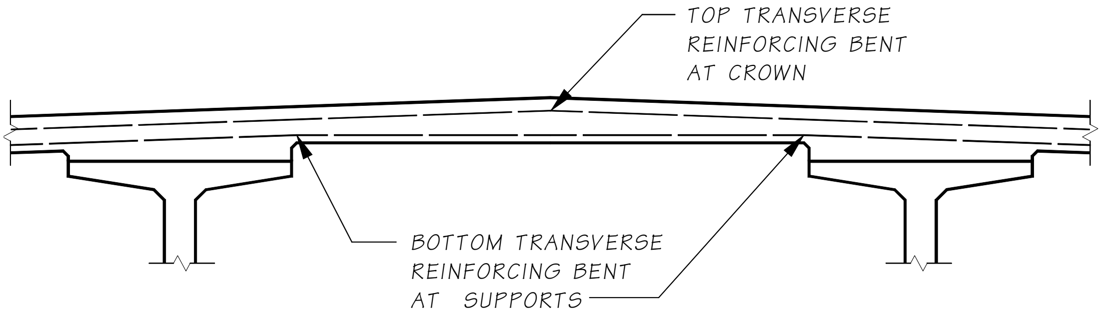
The size and spacing of transverse reinforcement may be governed by interior bridge deck span design and cantilever design. Where cantilever design governs, short hooked bars may be added at the bridge deck edge to increase the reinforcement available in that area. Top transverse reinforcement is always 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



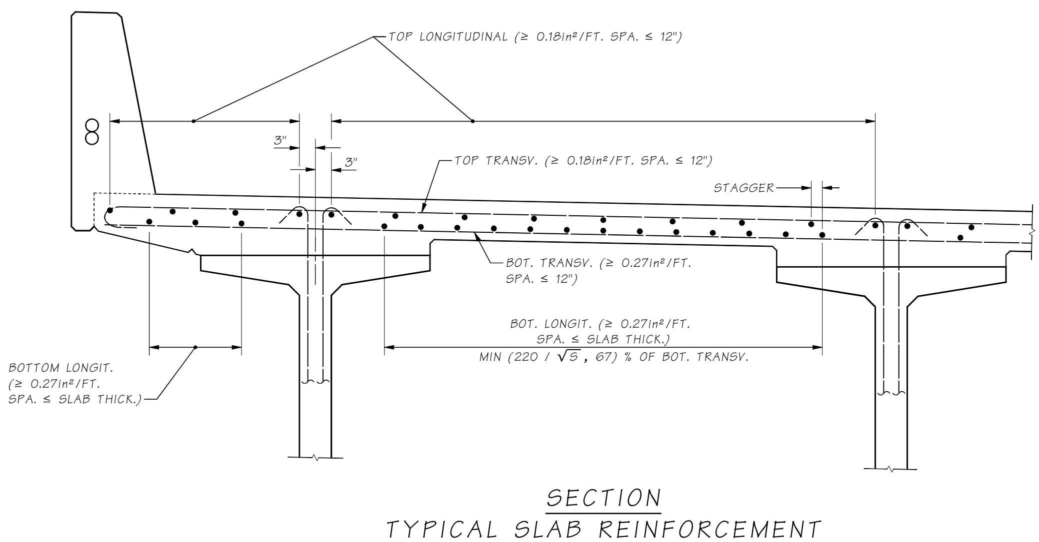
5.7.2.B Longitudinal Reinforcement

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. For longitudinal reinforcement requirements for decks on steel girder bridges, see Chapter 6.

5.7.2.B.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 Section 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 Deck Slab Reinforcement



5.7.2.B.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.

Continuity reinforcement for structures designed as simple spans may be terminated where no longer required by load demands. Reinforcing bars shall be developed beyond the point where the bar is required, but they do not need to be anchored farther as shown in Figure 5.7.2-4.

The required bridge deck thickness for various bar combinations is shown in Table 5.7.2-1.

5.7.2.C Distribution of Flexural Reinforcement

The provision of AASHTO LRFD Section 5.6.7 for class 2 exposure condition shall be satisfied for both the top and bottom faces of the bridge deck.

Table 5.7.2-1 Minimum Bridge Deck Thickness for Various Bar Sizes

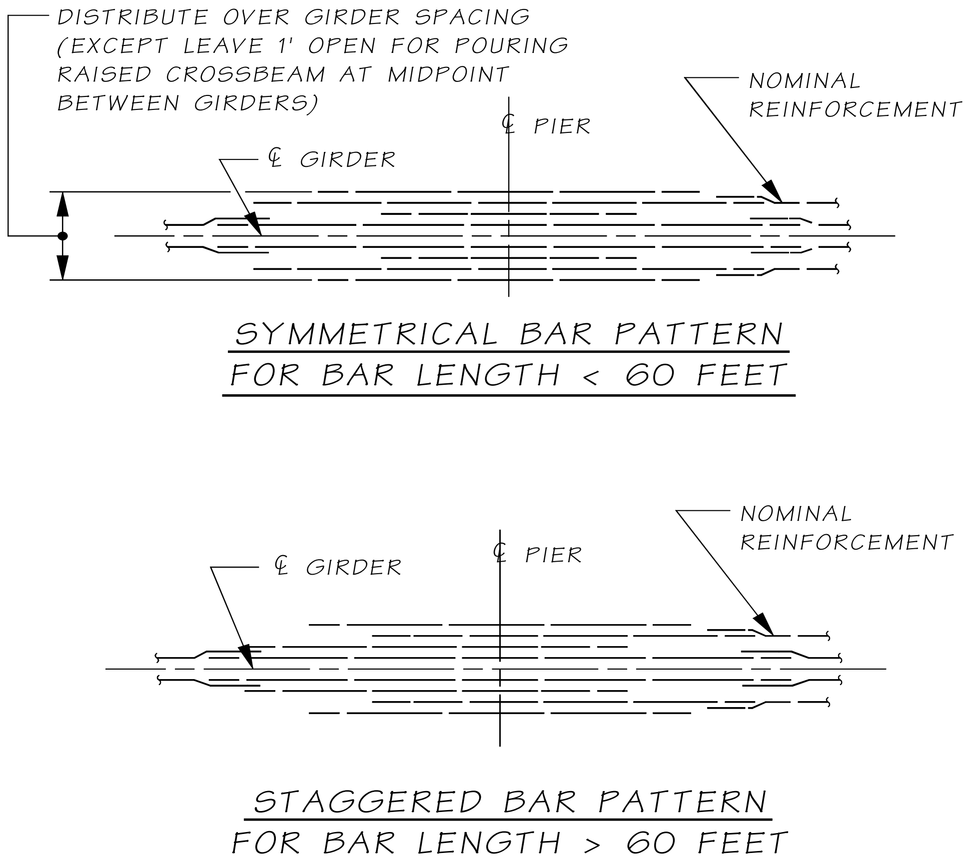
	Minimum Bridge Deck Thickness (Inches) Transverse Bar		
Longitudinal Bar	**#5**	**#6**	**#7**
#4	7½	--	--
#5	7½	7½	7¾
#6	7½	7¾	8
#7	7¾	8	8¼
#8	8	8½	8¾
#9	8½	8¾	9
#10	8¾	--	--

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

5.7.2.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.

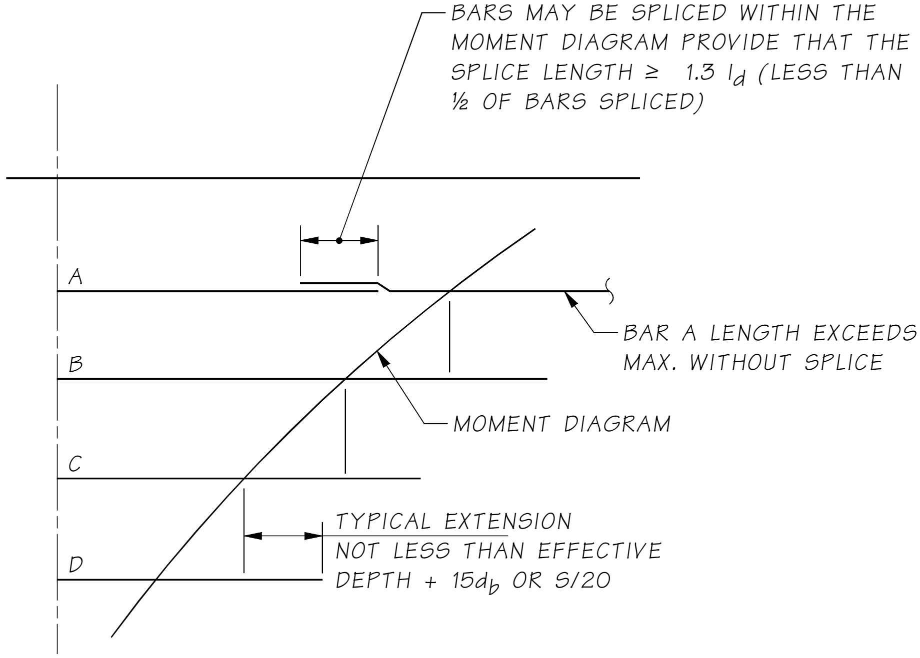
Figure 5.7.2-3 Longitudinal Reinforcing Bar Patterns



All bars shall be extended by their development length beyond the point where the bar is required.

Normally, no more than 33 percent 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 percent leads to severe restrictions on the reinforcement pattern, an increase in figure may be considered. Two reinforcement bars shall be used as stirrup hangers.

Figure 5.7.2-4 Bar Splice Within Moment Envelope



5.7.2.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. #6 bars may be used for transverse reinforcement, but designers should be aware that they may not be fully developed at overhang curblines for all traffic barriers.
- 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 $\frac{220}{\sqrt{S}} \leq 67$ percent of the positive moment as specified in AASHTO LRFD Section 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 Section 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

5.7.3.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.

Precast prestressed stay-in-place deck panels typically do not require air-entrained concrete because they are protected from saturation under service.

SIP deck panels may be used on WSDOT bridges with WSDOT State Bridge and Structures Engineer approval. Details for SIP deck panels are shown on the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm).

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

5.7.3.B Design Criteria

The design of SIP deck panels follows the AASHTO *LRFD Bridge Design 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.

5.7.3.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. Negative moment regions shall be determined as the envelope of moments from all Service and Strength Limit states.
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

The roadway surface for all bridge structures shall conform to one of the listed deck protection systems. 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 Asset Management Unit.

Preliminary plans shall indicate the protection system in the left margin in accordance with Section 2.3.8.

For bridge deck rehabilitation projects, the WSDOT Bridge Asset Management Unit will recommend the Deck Protection System and type of overlay if applicable.

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 *Standard Specifications* Section 8-08 and 8-09, 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.

5.7.4.A Deck Protection Systems

The following paragraphs describe five WSDOT protective systems used to protect a concrete bridge deck design.

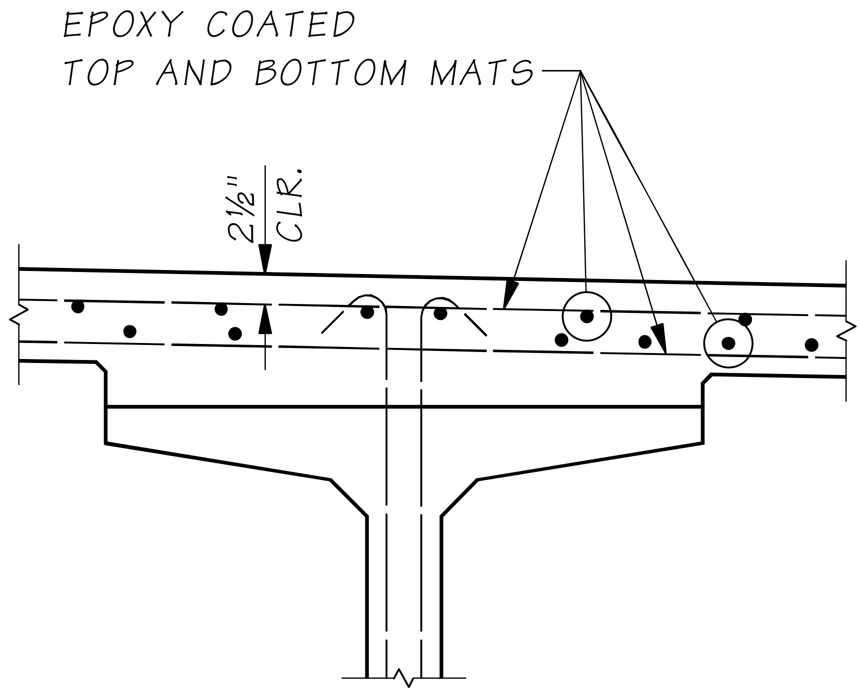
5.7.4.A.1 *Type 1 Protection System*

This is the default deck protection system for cases where a deck protection system has not been specified. 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:

1. A minimum 2½″ of concrete cover over top bars of deck reinforcing for cast-in-place decks. The cover includes a ½″ wearing surface and ¼″ tolerance for the placement of the reinforcing steel. Bottom cover shall be 1” minimum.
2. Both the top and bottom mat of deck reinforcing shall be epoxy-coated or equivalent corrosion protection system as specified in Section 5.1.2.
3. Girder stirrups and horizontal shear reinforcement do not require epoxy-coating or equivalent corrosion protection system as specified in Section 5.1.2.

Bridge decks using partial depth precast prestressed SIP deck panels shall be considered Type 1 protections systems, except that reinforcement and prestressing strand need not be epoxy coated.

Figure 5.7.4-1 Type 1 Protection System



5.7.4.A.2 *Type 2 Protection System*

This protection system consists of cementitious and polymer-based overlays on new and existing bridge decks, see Figure 5.7.4-2 for an example of a modified concrete overlay on a deck rehabilitation project.

For new bridges, a 1½″ modified concrete overlay shall be used.

The common overlays are as follows.

1. **1½″ Modified Concrete Overlay**

   Concrete overlays are generally described as a 1.5″ minimum unreinforced layer of modified concrete. Overlay concrete is modified to provide a low permeability that slows or prevents the penetration of chlorides into the bridge deck, but also has a high resistance to rutting. Ideally, the concrete cover to the top layer of reinforcement should be 2.5″. For new structures, the deck reinforcement shall be epoxy coated or equivalent corrosion protection system as specified in Section 5.1.2.

   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″. For decks with existing concrete overlays, the existing overlay should be removed entirely.

   The modified concrete overlay specifications allow a contractor to choose between a latex, microsilica or fly ash modified 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 cure the concrete overlay alone is 42 hours.

2. **¾″ Polyester Polymer Concrete Overlay**

   These overlays were first used by WSDOT in 1989 and have an expected life between 20-40 years with more than 30 overlay as of 2021. This type of overlay is a polymer-based concrete that uses specialized equipment to place and finish. Construction requires dry weather with temperatures above 50°F and normally cures within 4 hours. A polyester concrete overlay may be specified in special cases when rapid construction is needed. Due to its lower modulus of elasticity and compressive strength than Portland cement based concrete, polyester concrete may not be a good candidate to replace large volumes of structural concrete, such as in decks and edge beams.

3. **3″ Concrete Class 4000D Overlay**

   These are nominally 3” thick concrete overlays placed after the existing bridge deck is scarified down to the top mat of bridge deck reinforcement. The depth of placement may be reduced for structural reasons such as in non-redundant deck overhangs, but the minimum thickness shall be 2” to accommodate the larger aggregate in Concrete Class 4000D.

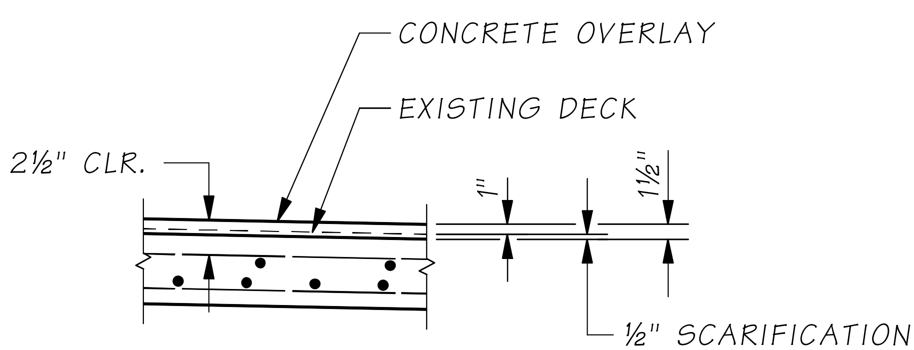
   These overlays were first used in the mid 2010’s on bridges that had previously received a modified concrete overlay. Second generation modified concrete overlays were seen to suffer from debonding, which may have been caused by microcracks in the substrate concrete caused by rotary milling machines and other percussive equipment used to scarify bridge decks in the past. The increased depth of removal using hydromilling equipment ensures the removal of bruised/microcracked concrete in the existing bridge deck.

4. **Historical Overlay Systems**

   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 No. 016395. Like polyester, this overlay cures in 4 hours and may be specified in special cases when rapid construction is needed.

   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.

Figure 5.7.4-2 Type 2 Protection System



5.7.4.A.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 should be applied over decks with 2” of concrete cover and epoxy-coated or other corrosion resistant reinforcing. 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.

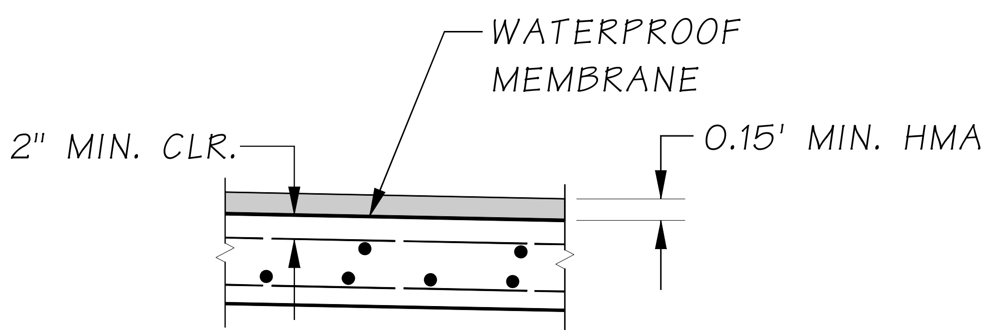
Waterproof membranes are required with the HMA overlay. Unlike roadway surfaces, the HMA material collects and traps water carrying salts and oxygen at the concrete surface deck. This is additional stress to an epoxy protection system or a bare deck and requires a membrane to mitigate the penetration 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 or for simple span WFDG girder bridges connected with UHPC. New bridge designs using HMA shall have a minimum depth of overlay of 0.25' (3″) to allow future resurfacing contracts 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.

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.

Figure 5.7.4-3 Type 3 Protection System

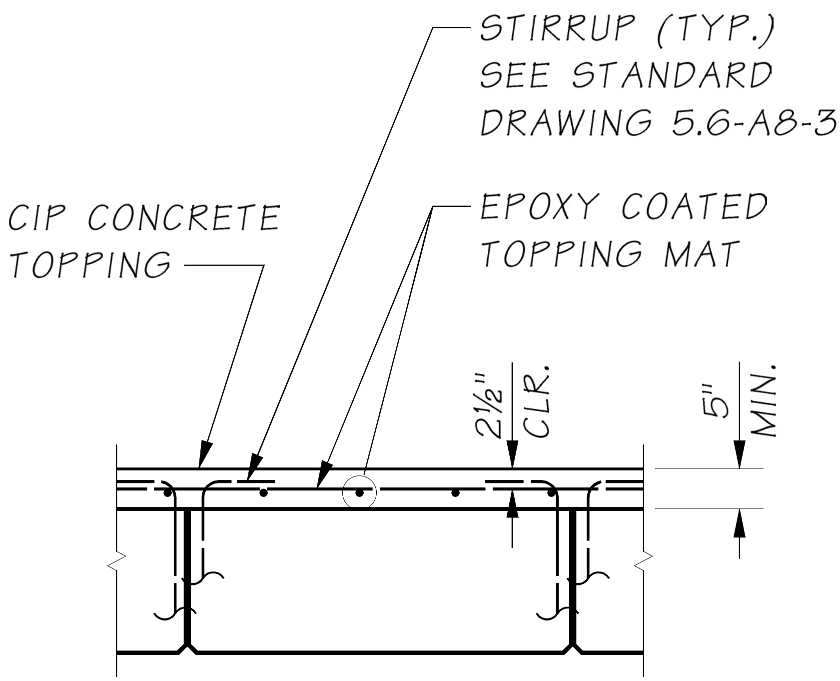


5.7.4.A.4 *Type 4 Protection System*

This system is a minimum 5″ cast-in-place (CIP) topping with at least one mat of epoxy coated reinforcement, see Figure 5.7.4-4. This system eliminates girder wheel distribution problems, provides a quality protection system and provides a durable wearing surface. It is commonly used on slab girder bridges that transfer shear forces between girders with minimal flexure.

1. A minimum concrete cover of 1″ applies to the top mat of the top of the prestressed member.
2. Epoxy coating the prestressed member top mat reinforcement is not required.

Figure 5.7.4-4 Deck Protection System 4

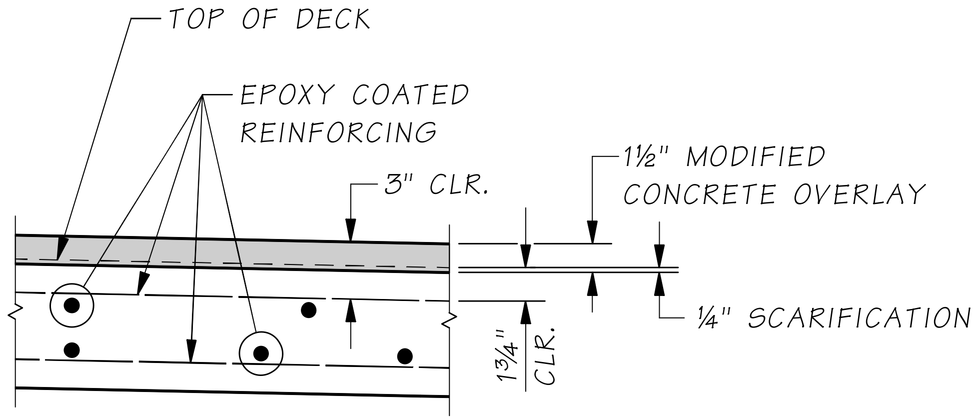


5.7.4.A.5 *Type 5 Protection System*

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. Segmental bridges and bridge decks with transverse 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:

1. Both the top and bottom mat of deck reinforcing are epoxy-coated or equivalent corrosion protection system as specified in Section 5.1.2. Girder/web stirrups and horizontal shear reinforcement does not require epoxy-coating.
2. The deck is constructed with a 1¾″ concrete cover.
3. 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.
4. A 1½″ modified concrete overlay is placed as a wearing surface.

Figure 5.7.4-5 Type 5 Protection System



5.7.4.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 shall 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.

5.7.4.B.1 *Rebar*

The deck or cast-in-place slab of the new widened portion 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.

5.7.4.B.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 percent of the deck area.

5.7.4.B.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.

5.7.4.B.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′.

5.7.4.B.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 less than 1 percent of the deck area.

5.7.4.B.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:

1. The existing expansion joint shall be replaced if:
   - More than 10 percent of the length of a joint has repairs within 1′-0″ of the joint.
   - Part of a joint is missing.
   - The joint is a non-standard joint system placed by maintenance.
2. All existing joint seals shall be replaced.
3. 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.
4. 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.

5.7.4.C Bridge Deck Overlays

Bridge deck overlays are an important means to preserve decks and the underlying structure. Because bridge decks are directly exposed to live loading and the environment, most bridges are expected to need resurfacing of some sort during their service life. The need for an overlay or other resurfacing can be triggered by (among other reasons) excessive wear, rutting, delamination, spalling, patching, corrosion and/or chloride exposure.

For rehabilitation jobs, the type of overlay or resurfacing specified in the Bridge Condition Report will depend on the level of deterioration in the bridge deck, the nature of previous deck rehabilitation efforts, deck expansion joint type, approach roadway material, and the impact of lane closures during construction.

The portion of overlays placed above the top of the original deck are typically not considered part of a structural deck due to the lack of reinforcing and risk of delamination. Portland cement-based overlay materials such as modified concrete and Concrete Class 4000D may be considered as structural concrete when they replace original deck concrete, such as in patches, edge beams, diaphragms and joint headers.

5.7.4.C.1 *Surface Preparation*

Overlays are typically bonded to the underlying deck through a mechanical and/or chemical means. Adequate surface preparation is essential for the long-term durability of overlays.

For Portland cement-based concrete overlays, surface preparation requires the use of hydro-demolition or small hand tool roughening of the concrete substrate. These methods ensure a high degree of mechanical roughness as well as the removal of weakened concrete at the interface zone. These methods also minimize the risk of damage to reinforcing bars when the exposure of reinforcing bars is possible or expected.

Polyester concrete and other lower modulus materials typically only require shot blasting and the use of a chemical primer. Hydro-demolition is typically not used for polyester concrete overlays because polyester concrete is sensitive to water.

5.7.4.C.2 *Existing Structure Evaluation*

For deck rehabilitation projects, longitudinal and transverse deck reinforcement shall be evaluated for the effects of deck concrete removal which impact the load carrying capacity of the superstructure, including areas where the deck is used to resist tension, compression and flexural forces.

Hydro-demolition machines have the potential to remove more depth than intended due to poor calibration or concrete/surface condition. Concrete removal may also expose areas of poorly consolidated or delaminated concrete. Because of this potential, there is risk of section loss to the deck and superstructure during construction. Also, if reinforcing bars become debonded over a significant length, they could lose effectiveness in resisting dead and live load tension or flexure. Designers should consider the risk of inelastic redistribution of dead load.

Limitations shall be placed in the contract to describe removal sequencing and limits as determined by analysis. Limitations typically include requiring the work to be performed in longitudinal strips, which limits the width of potential section loss in the structure. Limits can also be placed on the length of section loss in the superstructure allowed during hydro demolition before a concrete patch (Type 2 Deck Repair) needs to be placed. Further, the weight and placement of construction load equipment may need limits. When establishing the sequence and limits of removal work, designers should consider the following:

- Condition of the deck
- Superstructure type and level of ductility
- Locked-in forces carried by the deck
- Girder/web spacing and redundancy
- Scarification depth relative to deck and concrete cover thickness
- Size, location and development length of deck reinforcing bars
- The presence of prestressing tendons
- Likely methods of concrete delivery and placement
- Proximity of adjacent traffic to work on the bridge
- Construction schedule and other traffic control constraints

While structure needs are critical, designers should consider that overly restrictive limits on the work can significantly extend the duration of the work. In some cases, traffic may need to be completely restricted or detoured.

5.7.4.C.3 *Deck Repair*

Deck repair work is often necessary after scarification to restore the structural integrity of the underlying deck. Shallow repairs below the scarification depth (Type 1 Deck Repair) can often be filled with the overlay material as the overlay is being placed. In the case of deeper repairs, concrete patches should be placed and cured prior to placing of the overlay (Type 2 Deck Repair). Placing deeper repair patches prior to the overlay material avoids the risk of plastic settlement and cracking that can arise when there is a sudden change in the depth of placed concrete. Some Type 2 Deck Repair may also need to be performed prior to the completion of scarification in order to avoid the loss of bridge section or strength.

In order to maintain structural integrity, the material for Type 2 Deck Repair should typically be a structural concrete with a strength and modulus similar to the original deck. This could include Concrete Class M, Modified Concrete, Concrete Class 4000D, or Concrete Class 4000. However, polyester concrete is often used for polyester concrete overlays in order to accelerate the schedule and avoid introducing moisture at the overlay bond zone.

5.7.5 HMA Paving on Bridge Decks

5.7.5.A Design Responsibilities

Bridge paving design options are bridge specific based on the existing conditions and previous paving. All designers, whether WSDOT Bridge and Structures Office, Region PEO, or outside consultants, shall have the following documents in-hand before beginning any bridge deck paving design:

1. *Bridge Condition Report* (BCR) as developed by the WSDOT Bridge and Structures Office for each bridge within the project limits. The BCR specifies the known bridge deck paving conditions present at the bridge, and specifies the paving depths and bridge deck repair requirements as determined by the WSDOT State Bridge Asset Management unit.
2. *Project Resurfacing Report* as developed by the Region Materials Laboratory. The Region PEO is responsible for field evaluation of the current surfacing condition and the current depth of surfacing as confirmed by cores taken by the Region Materials Laboratory. Surfacing depths vary from bridge to bridge and vary within the same bridge deck, so multiple cores at a bridge are necessary to establish a valid current baseline.

Discrepancies in paving depths specified at each bridge between the Project Resurfacing Report and the BCR shall be discussed by the Region PEO and the Bridge Asset Management unit to reach a consensus prior to continuing with bridge deck paving design.

Bridge deck paving PS&E for bridges in HMA paving projects may be prepared in the Region by the Design PEO provided all of the following conditions are satisfied:

1. A minimum of 0.25 feet of competent HMA is present on the bridge deck. Milling operations will leave a minimum of 0.10 feet of HMA on the bridge deck. Filling operations will not add more than 0.15 feet of HMA. Bridge deck repair and a waterproof membrane are not planned.

2. No bridge expansion joint or header repair or replacement work is required.
3. The bridges have an operating load rating equal or greater than 45 tons. Operating ratings are shown on the Bridge Engineering Information System (BEIST) summary sheet: http://beist/InventoryAndRepair/Inventory/BRIDGE
4. The BCR indicates paving weight restrictions are not required for the structure.

Bridge deck paving PS&E for bridges not conforming to all of the criteria above will be prepared by the WSDOT Bridge and Structures Office.

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 to assist the planing operations.

5.7.5.B Design Considerations

An HMA 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 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.

5.7.5.B.1 *HMA Depth*

HMA thickness shall be 0.25′ or 3″. A greater depth may be allowed if structurally acceptable, such as structures with ballast or as approved by the WSDOT Load Rating Engineer. The thickness of HMA shall not reduce the exposed barrier height below minimum requirements. Paving designs that increase the HMA more than 3″ require a new Load Rating analysis and shall be submitted to the WSDOT Bridge Preservation office Load Rating Engineer.

1. Concrete bridge decks with more than 0.21′ HMA may be exempted from paving restrictions for mill/fill HMA design.
2. Prestressed concrete deck girders and slabs with less than 0.25′ HMA require paving restrictions to avoid planing the supporting structure.
3. 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.

5.7.5.B.2 *Grade Controlled Structures*

For bridge decks with an HMA thickness less than 0.25′ 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′.

5.7.5.B.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 A-60.30-00, 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:

1. Previous HMA overlays that raised the grade can significantly increase the minimum transition length.
2. Drainage considerations may require longer transitions or should plane to existing catch basins.
3. 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. See *Design Manual* Section 720.04 Bridge Site Design Elements, (5) Vertical Clearances, (c) Minimum Clearance for Existing Structures, 1. Bridge Over a Roadway.

5.7.5.B.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.

1. 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 WSDOT State Bridge Asset Manager and used by the WSDOT Bridge Office to monitor the condition of the concrete deck and determine when the deck needs rehabilitation or replacement.
2. Membrane Waterproofing (Deck Seal) is Standard Item 4455 and will be required for all HMA bridge decks, except when the following conditions are met.
   A. HMA placed on a deck that has a Modified Concrete Overlay which acts like a membrane.
   B. The bridge is on the P2 replacement list or deck rehabilitation scheduled within the next 4 years or two bienniums.

5.7.5.B.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.

1. Rutting on the concrete deck is ½″ or more.
2. 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.

5.7.5.B.6 *Bridge Transverse Joint Seals*

Saw cut pavement joints shown in Standard Plan A-40.20-04 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 joints on the deck. 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 Standard Item 6517. The following summarizes the intended application of the Details in Standard Plan A-40.20-04.

1. **Detail 1 & 2**
   Applies where HMA on the bridge surface abuts an HMA roadway.
2. **Detail 3 & 4**
   Applies where concrete bridge surface abuts an HMA roadway.
3. **Detail 5, 6 & 7**
   Applies at open concrete joints.
4. **Detail 11**
   Applies to longitudinal staging joints.
5. **Detail 12**
   Applies to pavement repair at pavement seats.

5.7.5.B.7 *Bituminous Surface Treatments (BST)*

Bituminous Surface Treatments (or chip seals) ½″ thick may be applied to bridge decks with HMA under the following conditions.

1. Plans must identify or list all structures bridges included or expected within project limits and identify bridge expansion joint systems to be protected.
2. BST is not allowed on weight restricted or posted bridges.
3. Planing will be required for structures at the maximum asphalt design depth or the grade is limited.

BSTs are generally not a problem if the structure is not grade limited 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.

Equipment used for BST application is generally composed of legal loads and does not require evaluation for load limits. Any equipment used would be governed by the requirements in *Standard Specifications* Section 1-07.7(2).

5.7.5.B.8 *Buried Structures and Other Structures*

Buried Structures or structures with significant fill and do not have rail posts attached to the structure generally will not have paving equipment load limits. Buried Structures and structures with HMA pavement applied directly to the structure shall be evaluated for paving equipment loads.

5.7.5.B.9 *Paving Equipment Load Restrictions*

All structures shall be evaluated for their ability to carry the weight of HMA removal and HMA paving equipment. Modern HMA roadway paving equipment can be quite heavy, and typically does not conform to legal vehicle axle patterns. This is particularly true for material transfer vehicles (MTV's).

Each plan set shall include one plan sheet for HMA removal equipment load restrictions and one plan sheet for HMA paving equipment load restrictions. These limits should be selected to give the paving contractor the most flexibility to select equipment and achieve HMA compaction. In special cases for short span bridges where only one piece of equipment can occupy a span, piece weight limits may be specified by plan note.

Specified paving loads and configurations shall have an operating load rating factor greater than 1.0. An impact factor of 0.1 or greater shall be used. Vibratory methods of compaction shall not be allowed on bridges or other structures.

5.7.5.B.10 *Plans Preparation*

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" in accordance with *Plan Preparation Manual* Section 4 "Vicinity Map", paragraph (n). This applies to all state bridges including but 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.

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.

5.8 Cast-in-place Post-Tensioned Bridges

5.8.1 Design Parameters

5.8.1.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 FHWA *Post-tensioned Box Girder Bridge Manual*[17] 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 FHWA Manual and the AASHTO *LRFD Bridge Design Specifications* and point out where current WSDOT practice departs from practices followed elsewhere.

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 typically 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.

5.8.1.B Bridge Types

Post-tensioning has been used in various types of CIP bridges in Washington State with box girders predominating. 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 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.

5.8.1.B.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 [21, 22, 23].

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.

5.8.1.B.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 multispan, 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.

5.8.1.B.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.

5.8.1.C Strand and Tendon Arrangements

The total number of strands selected should be at least the minimum required to meet the strength and service limit state requirements at all points. Check PT supplier literature for duct sizes and strand capacity. The most economical tendon selection will generally be the maximum duct size within the range than can be fit within the web. 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 66 strands per web would be most economically satisfied by three 22-strand tendons. A less economical choice would be four 17-strand tendons with 19-strand anchorages. The interior cross-sectional area of duct area shall be at least 2.5 times the net cross-sectional 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.

Figure 5.8.1-1 Tendon Placement Pattern for Box Girder Bridges

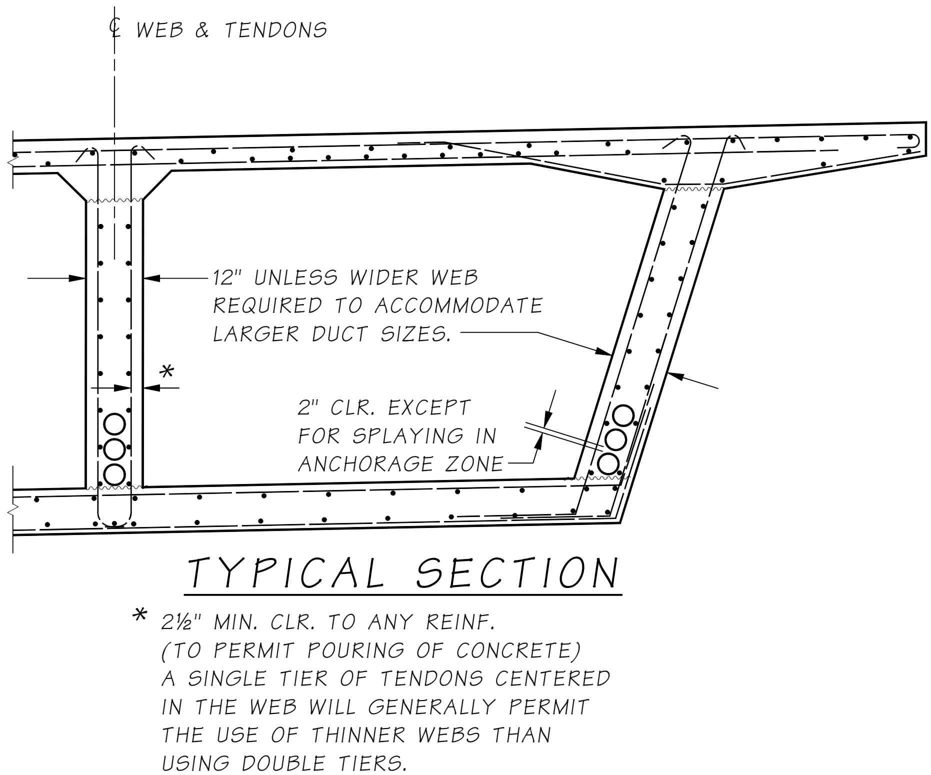


Figure 5.8.1-2 Tendon Placement Pattern for Box Girder Bridges

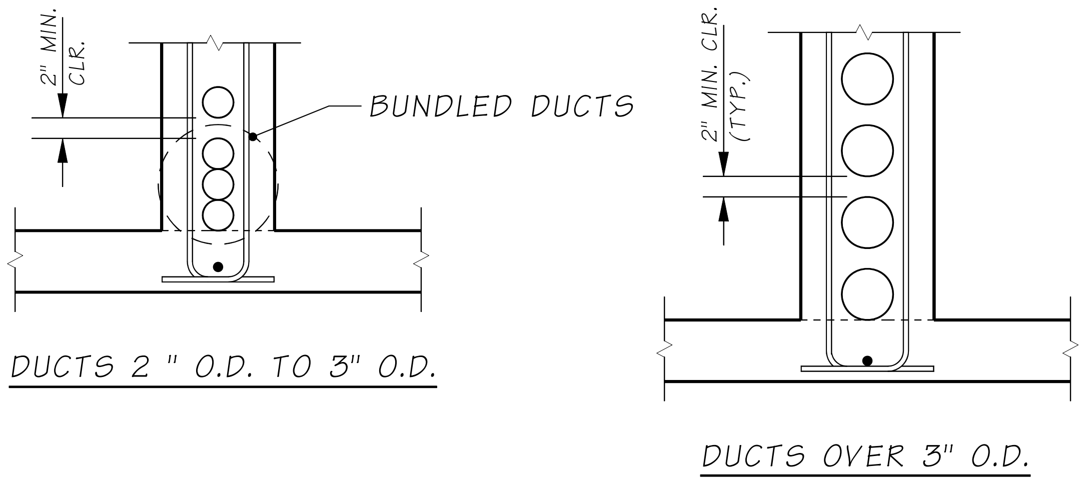
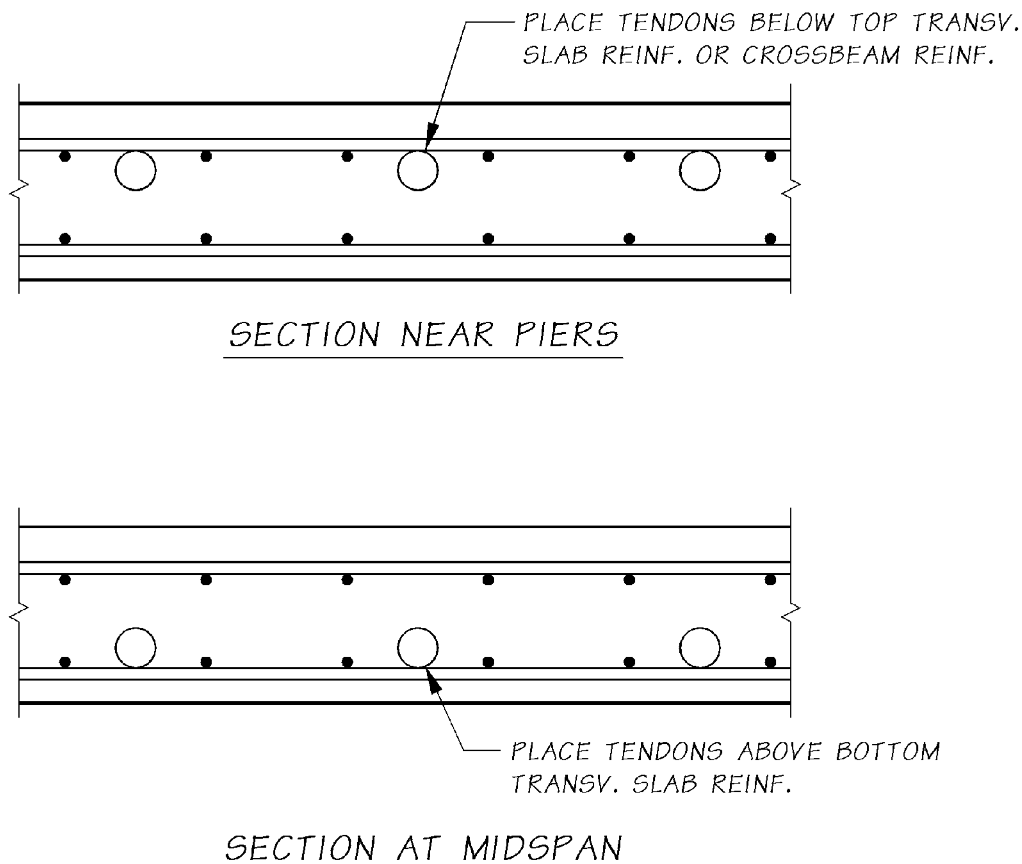


Figure 5.8.1-3 Tendon Placement Pattern for Flat Slab Bridges



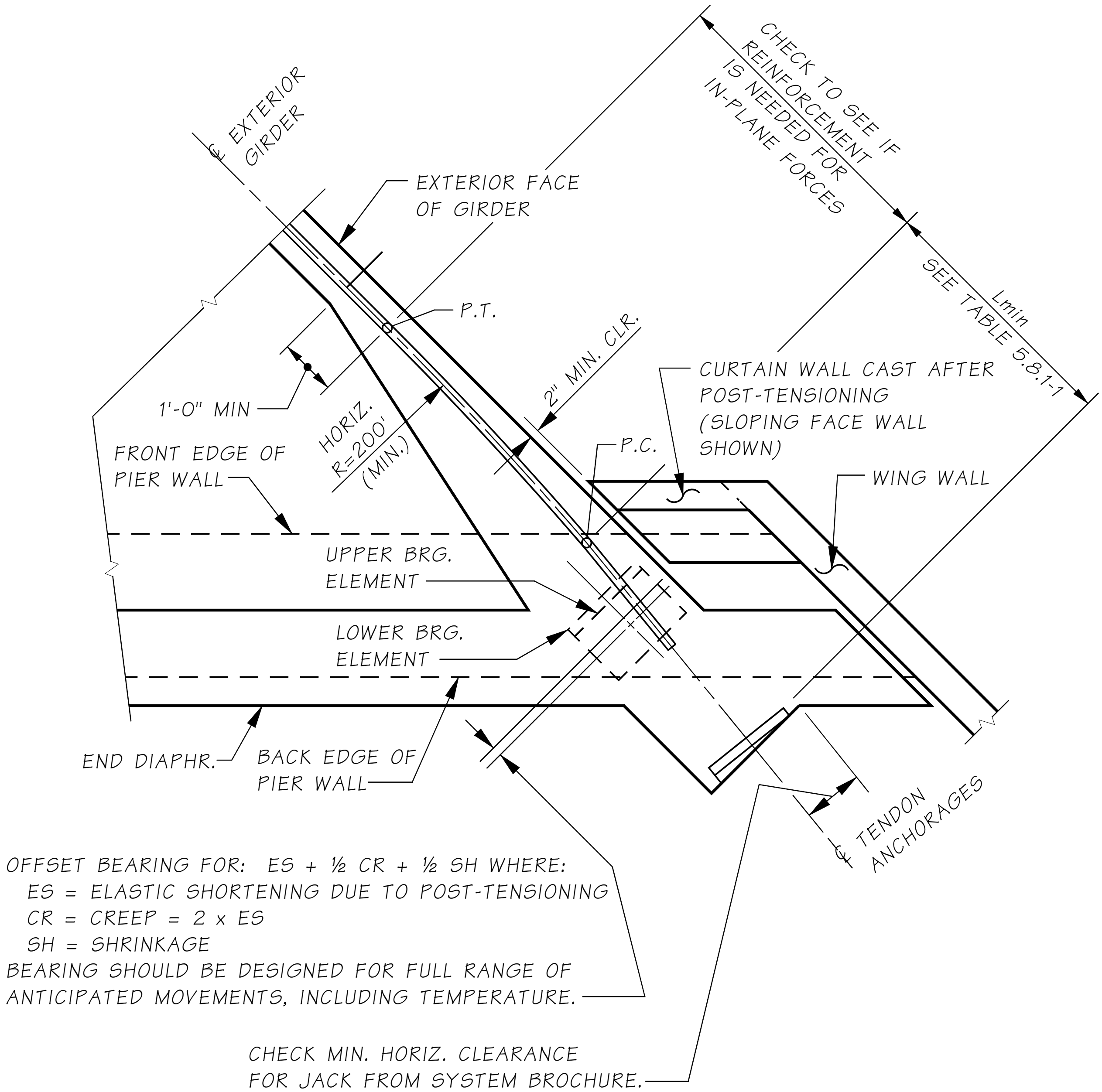
5.8.1.D Layout of Anchorages and End Blocks

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 Section 5.9.5.4.3.

Figure 5.8.1-4 Layout of Anchorages and End Blocks



All post-tensioning anchorages in webs of box girder or multi stem superstructures shall be vertically aligned. Special Anchorage Devices should be used to avoid a horizontally staggered anchorage layout. If anchorages are staggered horizontally, the design of the general zone shall consider the out-of-plane forces.

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.

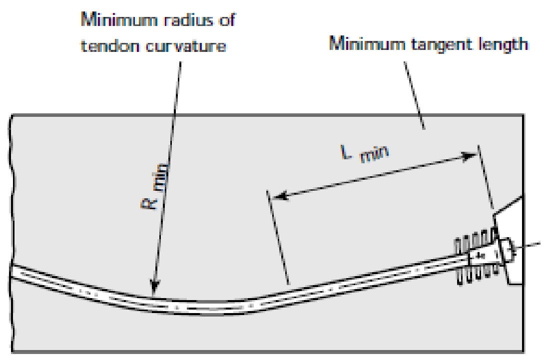
To prevent excessive friction loss and damage to the prestressing sheathings, adherence to the minimum tendon radii is required.

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 State Design Engineer.

Table 5.8.1-1 Minimum Tendon Radii and Tangent Length

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

Figure 5.8.1-5 Tangent Length and Tendon Radii



5.8.1.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.

5.8.1.F Effects of Curved Tendons

AASHTO LRFD Section 5.9.5.4.3 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 feet or the effect of in-plane plus out-of-plane forces is greater than or equal to 10 k/ft:

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

Where:

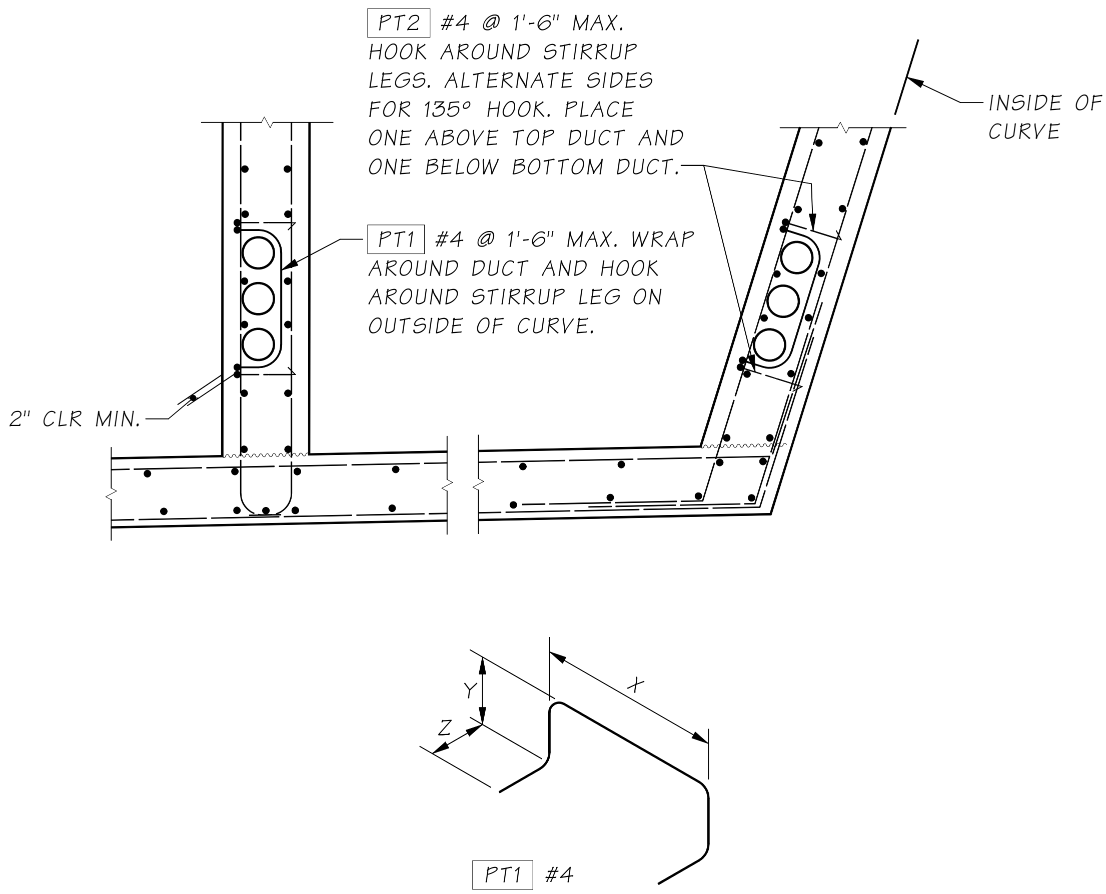
P_u = Factored tendon force = 1.2 P_{jack} (kips)

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 inches.

Figure 5.8.1-6 Curved Tendon Confinement Reinforcement



5.8.1.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 in accordance with AASHTO LRFD Section 5.8.4.5.4.

5.8.2 Analysis

5.8.2.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.

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 reference[18] 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.

5.8.2.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 Reference[17]. 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.

5.8.2.C Preliminary Stress Check

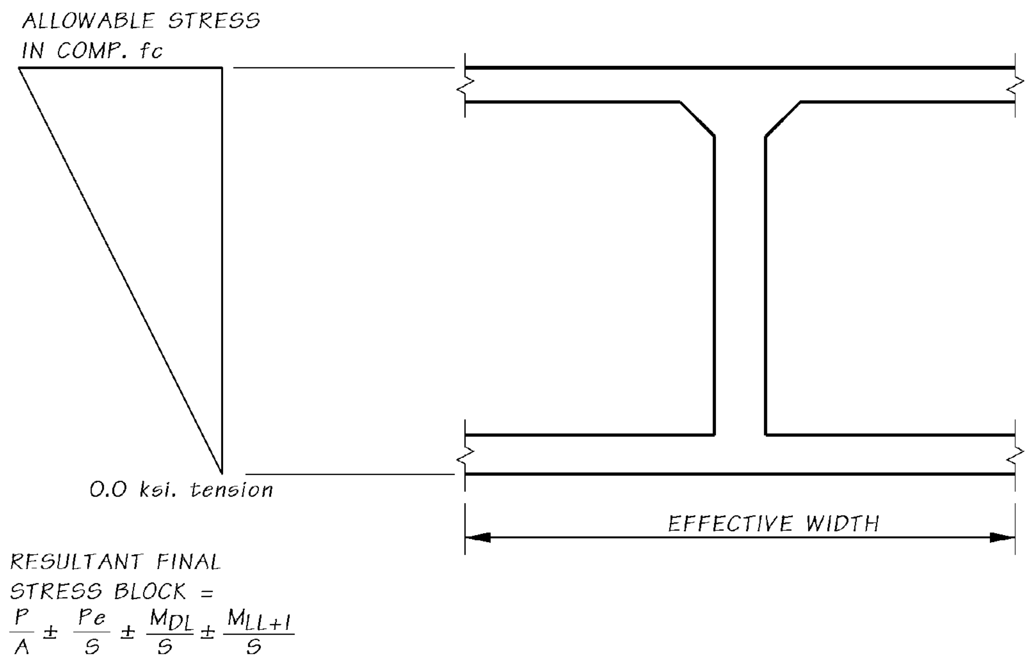
In accordance with AASHTO, flexural stresses in prestressed members are calculated at service load levels. Shear stresses, stirrups, moment capacities vs. applied moments are calculated at ultimate load levels.

During preliminary design, the first objective should be to satisfy the stress limits 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 tensile stress limits 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 concrete stress limits 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].

Figure 5.8.2-1 Box Girder Stresses



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.

5.8.2.D Camber

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

5.8.2.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

5.8.3.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 multispan bridges 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.

5.8.3.B Prestress Losses

Prestress losses shall be as indicated in Section 5.1.4.

5.8.3.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.

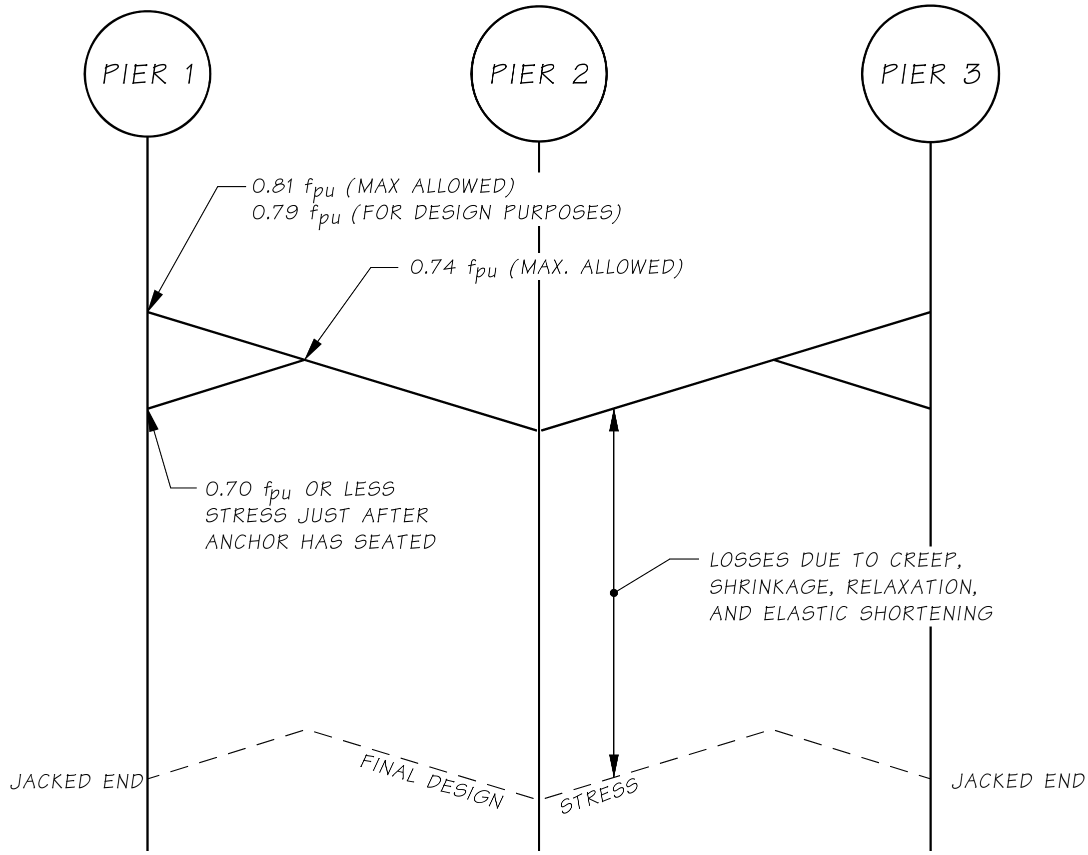
When stressing tendons from both ends or when alternating a single pull from both ends (half tendons pulled from one end with the other half pulled from the other end), all tendons shall be stressed on one end before all tendons are stressed on the opposite end.

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.

5.8.3.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.

Figure 5.8.3-1 Stress Diagram for a 2-Span PT Bridge



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 ⅜″, 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 multispan 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 μ. 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 5.8.3-1 Friction Coefficients for Post-tensioning Tendons in Metal Ducts

Tendon Length	μ
500 ft or less	0.15
Over 500 ft to 750 ft	0.20
Over 750 ft to 1,000 ft	0.25

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

5.8.3.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 multispan 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 [17].

5.8.3.F Prestress Moment Curves

5.8.3.F.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.

5.8.3.F.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.

5.8.3.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. Stress limits, 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 in accordance with AASHTO LRFD Section 5.5.4.2 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

5.8.4.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 in accordance with AASHTO LRFD Section 5.7.3. Minimum stirrup area and maximum stirrup spacing are subject to the limitations presented in AASHTO LRFD Sections 5.7.2.5 and 5.7.2.6. 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.

5.8.4.B Horizontal Shear

Horizontal shear stress acts over the contact area between two interconnected surfaces of a composite structural member. AASHTO LRFD Section 5.7.4 shall be used for shear-friction design.

5.8.4.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.

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 [20].

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 2.0*d* on either side of the intermediate support proved to be particularly crack-prone.

Uniform temperature loads (TU) as well thermal gradients loads (TG) shall be considered in design.

5.8.6 Construction

5.8.6.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).

5.8.6.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.

5.8.6.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.

5.8.6.D Review of Post-tensioning Installation Drawings

Post-tensioning installation drawings shall be reviewed by the designer (or Bridge Technical Advisor) and consulted with the Concrete Specialist if needed. Review of the drawings shall verify that the plans, specifications, applicable PTI requirements, and design assumptions are satisfied. The PTI requirements include a detailed list of most of the required items. Reviewers should pay special attention to the following:

1. Ensure that the special anchorage device test reports for post-tensioning anchorages are included, and that the tendon drawing details for post-tensioning anchorage local zones are validated by the tested anchorage configuration (i.e. minimum spacing, edge distance, and concrete strength) shown in the test reports.

2. Ensure that the layout of the post-tensioning anchorages is consistent with the design of the general zone reinforcing. If not, either the anchorage layout or general zone may need to be revised.
3. Ensure than tendon vents and drains are properly located and oriented.

5.8.6.E During Construction

1. In case of low concrete strength, the design engineer should investigate the adequacy of the design with lower strength if they suspect repairs may be avoidable.
2. If the measured elongation of a strand tendon is within ± 7 percent of the approved calculated elongation, the stressed tendon is acceptable. For tendons shorter than 40 ft, if the measured elongation is within ± 7 percent + ¼ inch, the stressed tendon is acceptable.
3. If the measured elongation is greater than the allowable range, anchorage force verification after seating (lift-off force) should be performed. The lift-off force should not be less than 99 percent of the approved calculated force nor more than 70% $f$pu As.
4. If the measured elongation is less than the allowable range, anchorage force verification of the fixed-end anchorage should be performed. The designer could consider acceptance where the total prestressing force in a web, girder, or even bridge cross-section is greater than 98% of the design prestressing force.
5. One broken strand per tendon may be structurally acceptable. (Post-tensioning design shall preferably allow one broken strand). If more than one strand per tendon is broken, the designed should consider the ability of the other tendons in the structure to provide prestressing and strength.
6. Other problems such as unbalanced and out of sequence post-tensioning, strand surface condition, strand subjected to corrosion and exposure, delayed post-tensioning, jack calibration, etc. should be evaluated on a case-by-case basis in coordination with the contractor, post-tensioning system supplier, HQ Bridge Construction Office, and the Engineer of Record.

5.8.7 Post-tensioning Notes Contract Plans

5.8.7.A Plan Details

The Plans for post-tensioned concrete shall include a longitudinal section showing the vertical profile of the center-of-gravity of the prestressing force, camber diagram for dead load plus prestress force, and the post-tensioning notes as suggested in Section 5.8.7.B. The following information shall typically be included in a post-tensioning table on a per-web basis: minimum required concrete strengths at the time of stressing, minimum number of strands required, jacking load, anchorage load after seating, and long term prestress losses.

5.8.7.B Post-tensioning Notes

1. THE POST-TENSIONING SYSTEM SHALL CONFORM TO THE REQUIREMENTS FOR PROTECTION LEVEL 2 (PL-2).
2. THE DESIGN IS BASED ON 0.6"ø LOW RELAXATION STRANDS WITH THE ESTIMATED AVERAGE LONG TERM PRESTRESS LOSS DUE TO STEEL RELAXATION, ELASTIC SHORTENING, CREEP AND SHRINKAGE OF CONCRETE SHOWN IN THE POST-TENSIONING TABLE.
3. THE ACTUAL ANCHOR SET, INSTANTANEOUS PRESTRESS LOSS AND JACKING FORCE CALCULATED BY THE CONTRACTOR SHALL BE SPECIFIED IN THE POST-TENSIONING INSTALLATION DRAWINGS. THE DESIGN IS BASED ON INSTANTANEOUS PRESTRESS LOSS ASSUMING THE FOLLOWING:
   A. ANCHOR SET OF ⅜”
   B. FRICTION, $\mu = 0.14$
   C. WOBBLE COEFFICIENT, $k = 0.0002$/FT
4. THE DUCT SHALL BE ROUND AND THE MAXIMUM SIZE SHALL BE [???]”. THE INSIDE CROSS-SECTIONAL AREA OF THE DUCT SHALL BE AT LEAST 2.5 TIMES THE NET AREA OF THE PRESTRESSING STEEL IN THE DUCT.
5. THE COMPRESSIVE STRENGTH OF CONCRETE AT THE TIME OF STRESSING SHALL BE AS SHOWN IN POST-TENSIONING TABLE OR THE POST-TENSIONING INSTALLATION DRAWINGS, WHICHEVER IS HIGHER.
6. ALL TENDONS SHALL BE STRESSED FROM [ONE END, BOTH ENDS, ALTERNATING ENDS, . . .].
7. THE TENDON STRESSING SEQUENCE SHALL MEET THE FOLLOWING CRITERIA:
   A. THE PRESTRESSING FORCE SHALL BE APPLIED SYMMETRICALLY ABOUT THE CENTERLINE OF THE BRIDGE.
   B. THE PRESTRESSING FORCE IN ADJACENT WEBS/GIRDERS SHALL NOT DIFFER BY MORE THAN THE TOTAL PRESTRESSING FORCE OF ONE TENDON.
   C. AT NO TIME DURING THE STRESSING OPERATION SHALL MORE THAN ⅙ OF THE TOTAL PRESTRESSING FORCE BE APPLIED ECCENTRICALLY ABOUT THE CENTERLINE OF THE BRIDGE.

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 wide flange I girders or trapezoidal tub girders with a composite CIP deck. WSDOT Bridge and Structure office's standard drawings for spliced I-girders are as shown on the Bridge Standard Drawings website (www.wsdot.wa.gov/Bridge/Structures/StandardDrawings.htm). 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 wide flange deck girder or 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 Section 5.12.2.3.

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 large 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.
- 2'-0" minimum CIP concrete closures are required for connecting spliced girder segments, where segmental bridge segments are often joined with epoxy in 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.

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 pre-tensioned girders.

5.9.3 Girder Segment Design

5.9.3.A Design Considerations

Stress limits for temporary concrete stresses in girder segments specified in Section 5.2.1C shall apply at each stage of pretensioning or posttensioning. The concrete strength at release and initial lifting shall be f'_{ci} and at the time the post-tensioning is applied shall be f'_c in the stress limits.

Stress limits for final concrete stresses at the service load in girder segments as specified in Section 5.2.1C shall apply for intermediate load stages with the concrete strength at the time of loading shall be f'_c in the stress limits.

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 computations shall account for 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. 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.

The designer shall consider requirements for bracing of the girder segments once they have been erected. Any requirements for bracing during subsequent stages of construction that the contractor needs to design shall be specified in the contract documents.

5.9.3.B Post-tensioning

Longitudinal post-tensioning may be applied with the following considerations:

1. Post-tensioning precast segments in their final position before deck casting. This option is recommended by WSDOT for all spliced girder bridges. This option may require higher concrete compressive stress at CIP closures. But this option is more suitable for future deck repairs and deck replacement since the deck is not prestressed.
2. Post-tensioning girder line segments before erecting girders. Handling and shipping of spliced girders with segments post-tensioned prior to erection requires larger cranes and more staging area. This option may be used in some cases where the use of temporary support at the bridge site is not feasible.
3. Post-tensioning after deck casting. This option require lower concrete compressive stress at CIP closure. This option complicates future deck repairs and deck replacements since the deck is prestressed.
4. Two stage post-tensioning where girders are post-tensioned separately for dead load in the first stage, followed by post-tensioning the entire superstructure in a second stage after deck placement.

Designers shall investigate the required concrete compressive strength at the CIP closures. Achieving high strength concrete for CIP closures may be challenging in some locations.

Ducts for longitudinal post-tensioning shall be kept below the bridge deck, and ideally below the top of web when they could be exposed to damage during construction.

Effects of curved tendons shall be considered in accordance with Section 5.8.1.F.

All post-tensioning tendons shall be fully grouted after stressing. For construction cases prior to grouting posttensioning ducts, 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 posttensioning 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

5.9.4.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 and/or butt joints shall not be specified for spliced girder bridges. 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.

5.9.4.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 should 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.

5.9.4.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 or use a sawtooth pattern.

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 $0.6f_y$.

The clear spacing between the outside diameter of ducts at CIP closures of pier diaphragms shall be 2.0″ minimum.

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.

5.9.4.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.

Figure 5.9.4-1 CIP Closure at Pier Diaphragm

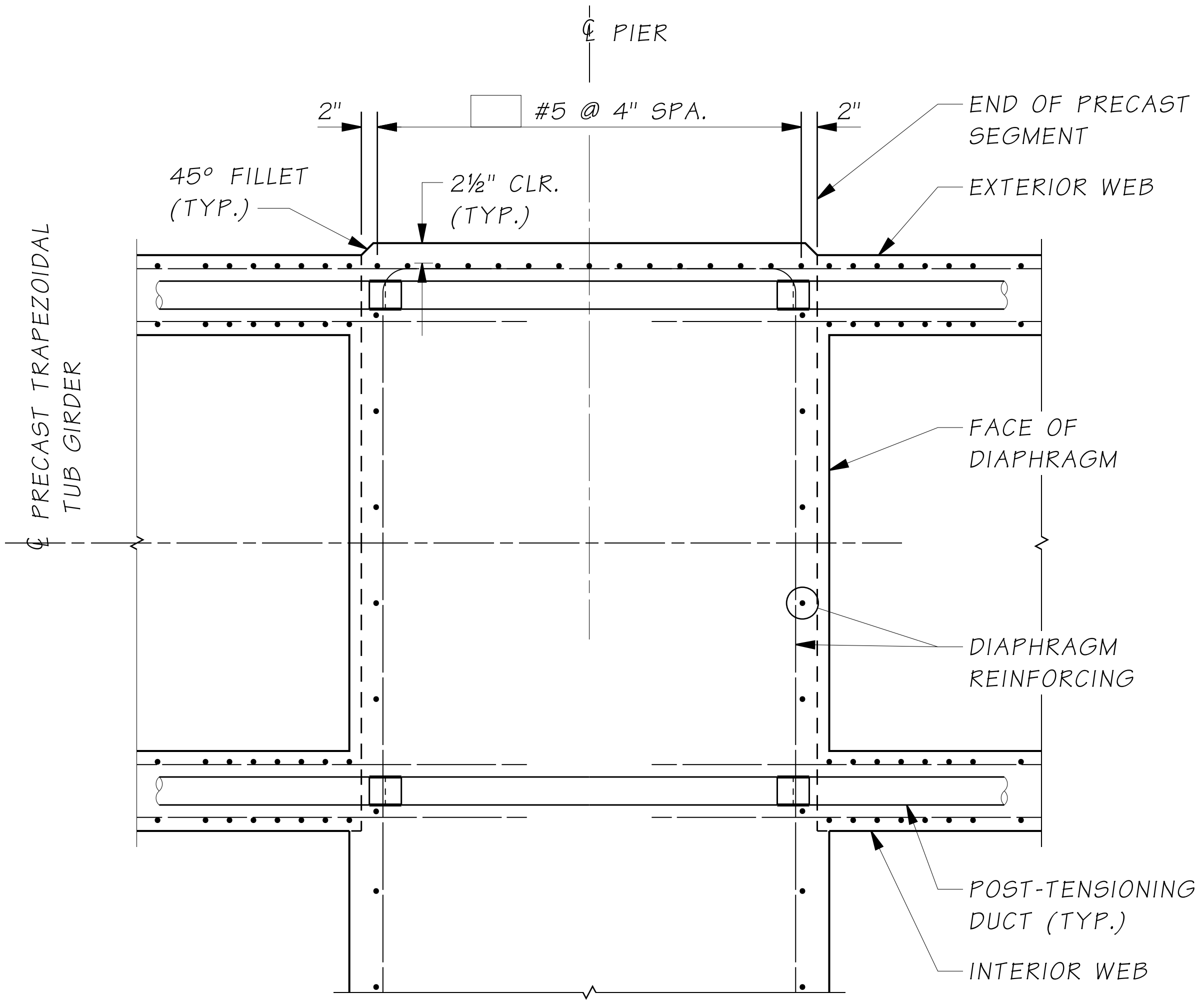


Figure 5.9.4-2 CIP Closure Away from Intermediate Diaphragm

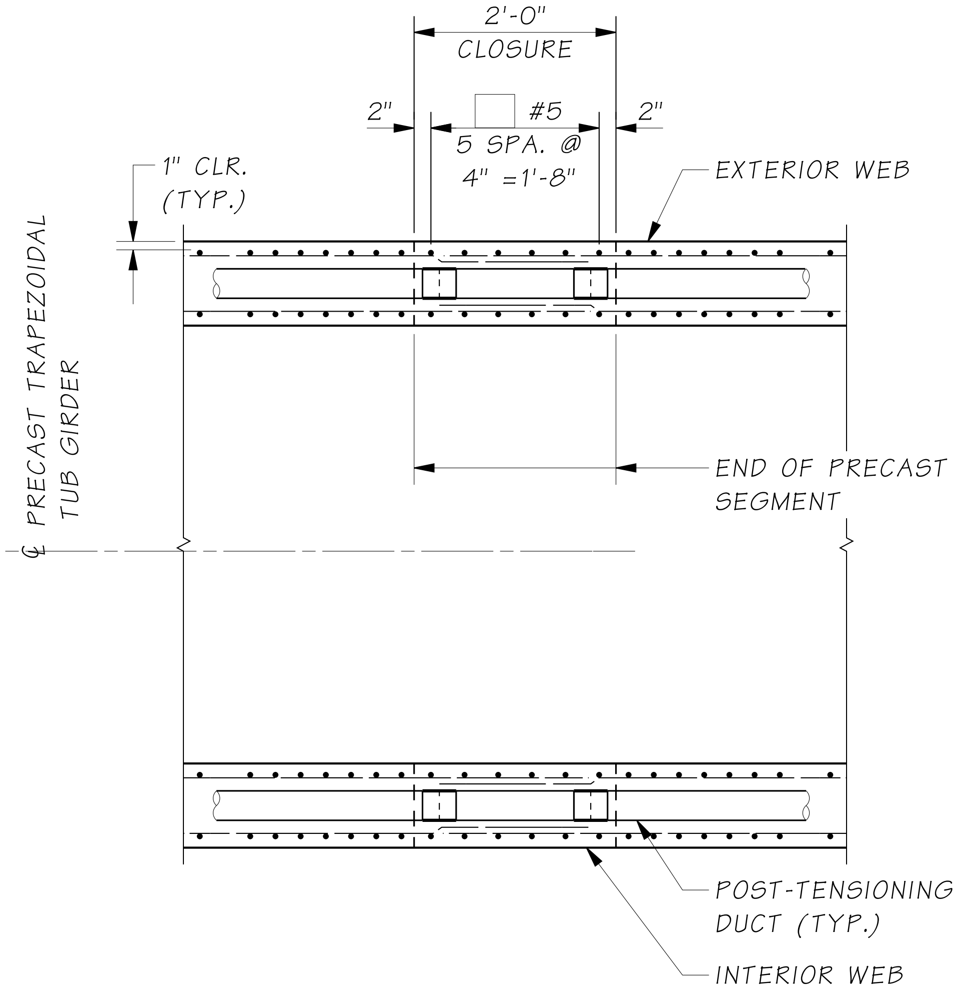
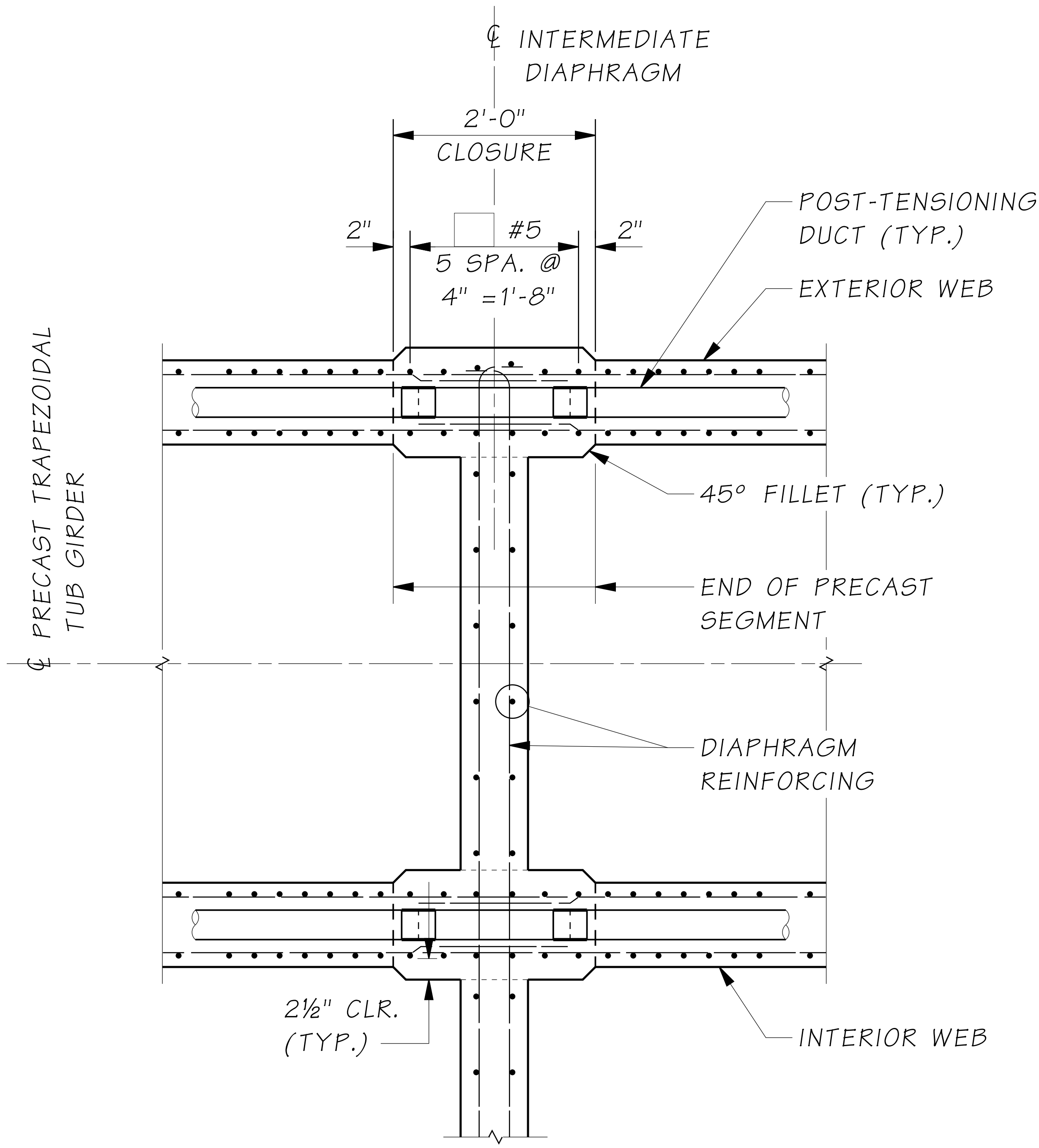


Figure 5.9.4-3 CIP Closure at Intermediate Diaphragm



5.9.5 Review of Shop Plans for Spliced Prestressed Concrete Girders

Shop drawings and post-tensioning installation drawings for spliced prestressed concrete girders shall be reviewed by the designer or Engineer-of-Record and consulted with the Concrete Specialist if needed. See Section 5.6.10 for the review of girder shop drawings, and Section 5.8.6.D for the review of post-tensioning installation drawings.

5.9.6 Post-tensioning Notes — Spliced Prestressed Concrete Girders

Contract plan preparation for spliced prestressed concrete girders should include the details and notes in Section 5.8.7.

5.10 Segmental Bridges

5.10.1 Cross Section Dimensions

For segmental box girder bridges of span length L, girder depth (d_o), top flange width (b) the cross section should satisfy the following:

For constant depth girders: $1/15 > d_o/L > 1/30$

For single cell box girders: $d_o/b \geq 1/6$

Top flange overhangs should not exceed 0.45 times the interior span of the top flange

5.11 Bridge Standard Drawings

Girder Sections

5.6-A1-10 Prestressed Concrete I and WF Girders (PDF 75KB) (DWG 62KB)
5.6-A1-11 Prestressed Concrete Deck Girders (PDF 118KB) (DWG 79KB)
5.6-A1-12 Spliced Prestressed Concrete Girder (PDF 127KB) (DWG 73KB)
5.6-A1-13 Prestressed Concrete Tub Girders (PDF 65KB) (DWG 41KB)

Superstructure Construction Sequences

5.6-A2-1 Single Span Prestressed Girder Construction Sequence (PDF 84KB) (DWG 64KB)
5.6-A2-2 Multiple Span Prestressed Girder Construction Sequence (PDF 80KB) (DWG 51KB)
5.6-A2-3 Raised Crossbeam Prestressed Girder Construction Sequence (PDF 80KB) (DWG 51KB)

W Girders

5.6-A3-1 W42G Girder Details 1 of 2 (PDF 109KB) (DWG 140KB)
5.6-A3-2 W42G Girder Details 2 of 2 (PDF 86KB) (DWG 136KB)
5.6-A3-3 W50G Girder Details 1 of 2 (PDF 110KB) (DWG 136KB)
5.6-A3-4 W50G Girder Details 2 of 2 (PDF 87KB) (DWG 125KB)
5.6-A3-5 W58G Girder Details 1 of 3 (PDF 111KB) (DWG 142KB)
5.6-A3-6 W58G Girder Details 2 of 3 (PDF 85KB) (DWG 184KB)
5.6-A3-7 W58G Girder Details 3 of 3 (PDF 85KB) (DWG 77KB)
5.6-A3-8 W74G Girder Details 1 of 3 (PDF 114KB) (DWG 147KB)
5.6-A3-9 W74G Girder Details 2 of 3 (PDF 85KB) (DWG 117KB)
5.6-A3-10 W74G Girder Details 3 of 3 (PDF 69KB) (DWG 83KB)

WF Girders

5.6-A4-1 WF Girder Details 1 of 5 (PDF 120KB) (DWG 82KB)
5.6-A4-2 WF Girder Details 2 of 5 (PDF 103KB) (DWG 107KB)
5.6-A4-3 WF Girder Details 3 of 5 (PDF 99KB) (DWG 90KB)
5.6-A4-4 WF Girder Details 4 of 5 (PDF 105KB) (DWG 105KB)
5.6-A4-5 WF Girder Details 5 of 5 (PDF 101KB) (DWG 99KB)
5.6-A4-6 Additional Extended Strands (PDF 88KB) (DWG 73KB)
5.6-A4-7 End Diaphragm Details (PDF 143KB) (DWG 115KB)
5.6-A4-8 L Abutment End Diaphragm Details (PDF 126KB) (DWG 106KB)
5.6-A4-9 Diaphragm at Intermediate Pier Details (PDF 135KB) (DWG 153KB)
5.6-A4-10 Partial Depth Intermediate Diaphragm Details (PDF 106KB) (DWG 95KB)
5.6-A4-11 Full Depth Intermediate Diaphragm Details (PDF 99KB) (DWG 99KB)
5.6-A4-12 I Girder Bearing Details (PDF 109KB) (DWG 86KB)

Wide Flange Thin Deck Girders

Wide Flange Deck Girders

Wide Flange Deck Girders (UHPC)

Deck Bulb Tee Girders

5.6-A7-1	Deck Bulb Tee Girder Schedule (PDF 112KB) (DWG 74KB)
5.6-A7-2	Deck Bulb Tee Girder Details 1 of 2 (PDF 126KB) (DWG 153KB)
5.6-A7-3	Deck Bulb Tee Girder Details 2 of 2 (PDF 126KB) (DWG 114KB)

Slabs

5.6-A8-1	Slab Girder Details 1 of 3 (PDF 129KB) (DWG 115KB)
5.6-A8-2	Slab Girder Details 2 of 3 (PDF 82KB) (DWG 134KB)
5.6-A8-3	Slab Girder Details 3 of 3 (PDF 88KB) (DWG 77KB)
5.6-A8-4	Slab Girder Fixed Diaphragm (PDF 90KB) (DWG 80KB)
5.6-A8-5	Slab Girder End Diaphragm (PDF 89KB) (DWG 81KB)
5.6-A8-6	Slab Girder Framing Plan and Typical Section (PDF 124KB) (DWG 49KB)

Tub Girders

5.6-A9-1	Tub Girder Schedule and Notes (PDF 671KB) (DWG 102KB)
5.6-A9-2	Tub Girder Details 1 of 3 (PDF 145KB) (DWG 128KB)
5.6-A9-3	Tub Girder Details 2 of 3 (PDF 69KB) (DWG 49KB)
5.6-A9-4	Tub Girder Details 3 of 3 (PDF 55KB) (DWG 37KB)
5.6-A9-5	Tub Girder End Diaphragm on Girder Details (PDF 63KB) (DWG 59KB)
5.6-A9-6	Tub Girder Raised Crossbeam Details (PDF 74KB) (DWG 55KB)
5.6-A9-7	Tub SIP Deck Panel Girder End Diaphragm on Girder Details (PDF 77KB) (DWG 56KB)
5.6-A9-8	Tub SIP Deck Panel Girder Raised Crossbeam Details (PDF 85KB) (DWG 55KB)
5.6-A9-9	Tub Girder Bearing Details (PDF 62KB) (DWG 32KB)

Stay-In-Place Deck Panel

5.6-A10-1	Stay-In-Place (SIP) Deck Panel Details (PDF 80KB) (DWG 51KB)

Post Tensioned Spliced Girders

5.9-A1-1	WF74PTG Spliced Girder Details 1 of 5 (PDF 76KB) (DWG 52KB)
5.9-A1-2	WF74PTG Spliced Girder Details 2 of 5 (PDF 69KB) (DWG 88KB)
5.9-A1-3	Spliced Girder Details 3 of 5 (PDF 68KB) (DWG 98KB)
5.9-A1-4	WF74PTG Girder Details 4 of 5 (PDF 99KB) (DWG 47KB)
5.9-A1-5	Spliced Girder Details 5 of 5 (PDF 74KB) (DWG 99KB)
5.9-A2-1	WF83PTG Spliced Girder Details 1 of 5 (PDF 70KB) (DWG 65KB)
5.9-A2-2	WF83PTG Spliced Girder Details 2 of 5 (PDF 116KB) (DWG 94KB)
5.9-A2-4	WF83PTG Spliced Girder Details 4 of 5 (PDF 72KB) (DWG 47KB)
5.9-A3-1	WF95PTG Spliced Girder Details 1 of 5 (PDF 69KB) (DWG 103KB)
5.9-A3-2	WF95PTG Spliced Girder Details 2 of 5 (PDF 117KB) (DWG 96KB)
5.9-A3-4	WF95PTG Spliced Girder Details 4 of 5 (PDF 83KB) (DWG 117KB)
5.9-A4-1	Trapezoidal Tub Girder Bearing Details (PDF 76KB) (DWG 57KB)
5.9-A4-2	Tub Spliced Girder Details 1 of 5 (PDF 70KB) (DWG 55KB)
5.9-A4-3	Tub Spliced Girder Details 2 of 5 (PDF 90KB) (DWG 62KB)
5.9-A4-4	Tub Spliced Girder Details 3 of 5 (PDF 69KB) (DWG 65KB)

5.9-A4-5	Tub Spliced Girder Details 4 of 5 (PDF 73KB) (DWG 54KB)
5.9-A4-6	Tub Spliced Girder Details 5 of 5 PDF 72KB) (DWG 50KB)
5.9-A4-7	Tub Girder End Diaphragm on Girder Details (PDF 81KB) (DWG 65KB)
5.9-A4-8	Tub Spliced Girder Raised Crossbeam Details (PDF 90KB) (DWG 59KB)
5.9-A5-1	P.T. Trapezoidal Tub SIP Deck Panel Spliced Girder Details 1 of 5 (PDF 80KB) (DWG 58KB)
5.9-A5-2	P.T. Trapezoidal Tub SIP Deck Panel Spliced Girder Details 2 of 5 (PDF 97KB) (DWG 63KB)
5.9-A5-3	P.T. Trapezoidal Tub SIP Deck Panel Spliced Girder Details 3 of 5 (PDF 74KB) (DWG 59KB)
5.9-A5-4	P.T. Trapezoidal Tub SIP Deck Panel Spliced Girder Details 4 of 5 (PDF 73KB) (DWG 54KB)
5.9-A5-5	P.T. Trapezoidal Tub SIP Deck Panel Spliced Girder Details 5 of 5 (PDF 72KB) (DWG 51KB)
5.9-A5-6	Trapezoidal Tub SIP Deck Panel Girder End Diaphragm on Girder Details (PDF 86KB) (DWG 67KB)
5.9-A5-7	Trapezoidal Tub SIP Deck Panel Spliced Girder Raised Crossbeam Details (PDF 94KB) (DWG 60KB)

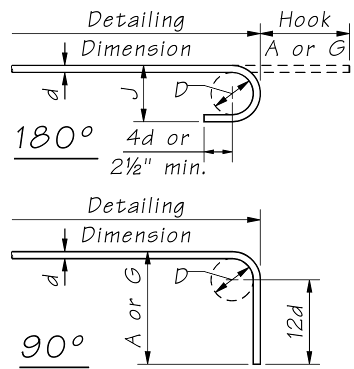
5.12 Appendices

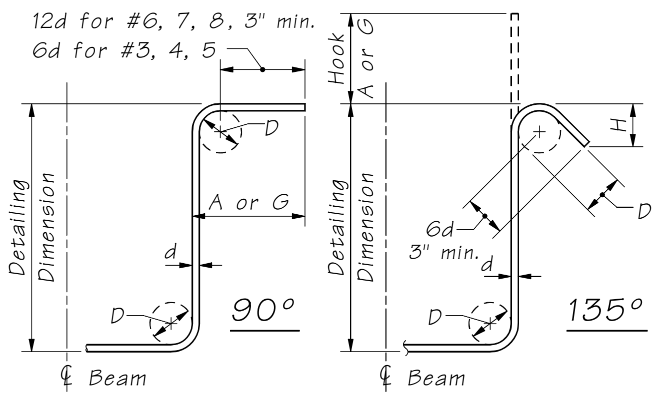
Appendix 5.1-A1	Standard Hooks
Appendix 5.1-A2	Minimum Reinforcement Clearance and Spacing for Beams and Columns
Appendix 5.1-A3	Reinforcing Bar Properties
Appendix 5.1-A4	Tension Development Length of Deformed Bars
Appendix 5.1-A5	Compression Development Length and Minimum Lap Splice of Grade 60 Bars
Appendix 5.1-A6	Tension Development Length of 90° and 180° Standard Hooks
Appendix 5.1-A7	Tension Lap Splice Lengths of Grade 60 Bars – Class B
Appendix 5.1-A8	Prestressing Strand Properties and Development Length
Appendix 5.2-A1	Working Stress Design
Appendix 5.2-A2	Working Stress Design
Appendix 5.2-A3	Working Stress Design
Appendix 5.3-A1	Positive Moment Reinforcement
Appendix 5.3-A2	Negative Moment Reinforcement
Appendix 5.3-A3	Adjusted Negative Moment Case I (Design for M at Face of Support)
Appendix 5.3-A4	Adjusted Negative Moment Case II (Design for M at 1/4 Point)
Appendix 5.3-A5	Cast-In-Place Deck Slab Design for Positive Moment Regions f'_c = 4.0 ksi
Appendix 5.3-A6	Cast-In-Place Deck Slab Design for Negative Moment Regions f'_c = 4.0 ksi
Appendix 5.6-A1-1	Span Capability of W Girders
Appendix 5.6-A1-2	Span Capability of WF Girders
Appendix 5.6-A1-3	Span Capability of Deck Bulb Tee Girders
Appendix 5.6-A1-4	Span Capability of WF Thin Deck Girders
Appendix 5.6-A1-5	Span Capability of WF Deck Girders
Appendix 5.6-A1-6	Span Capability of Trapezoidal Tub Girders without Top Flange
Appendix 5.6-A1-7	Span Capability of Trapezoidal Tub Girders with Top Flange
Appendix 5.6-A1-8	Span Capability of Post-tensioned Spliced I-Girders
Appendix 5.6-A1-9	Span Capability of Post-tensioned Spliced Tub Girders
Appendix 5.6-A1-10	Span Capability of WF Girders with Lighweight Concrete

Appendix 5.1-A1 Standard Hooks

RECOMMENDED END HOOKS
All Grades
D = Finished bend diameter

Bar Size	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#3	2¼"	5"	3"	6"
#4	3"	6"	4"	8"
#5	3¾"	7"	5"	10"
#6	4½"	8"	6"	1'-0"
#7	5¼"	10"	7"	1'-2"
#8	6"	11"	8"	1'-4"
#9	9½"	1'-3"	11¾"	1'-7"
#10	10¾"	1'-5"	1'-1¼"	1'-10"
#11	1'-0"	1'-7"	1'-2¾"	2'-0"
#14	1'-6¼"	2'-3"	1'-9¾"	2'-7"
#18	2'-0"	3'-0"	2'-4½"	3'-5"





STIRRUP AND TIE HOOK DIMENSIONS
All Grades (except galvanized)

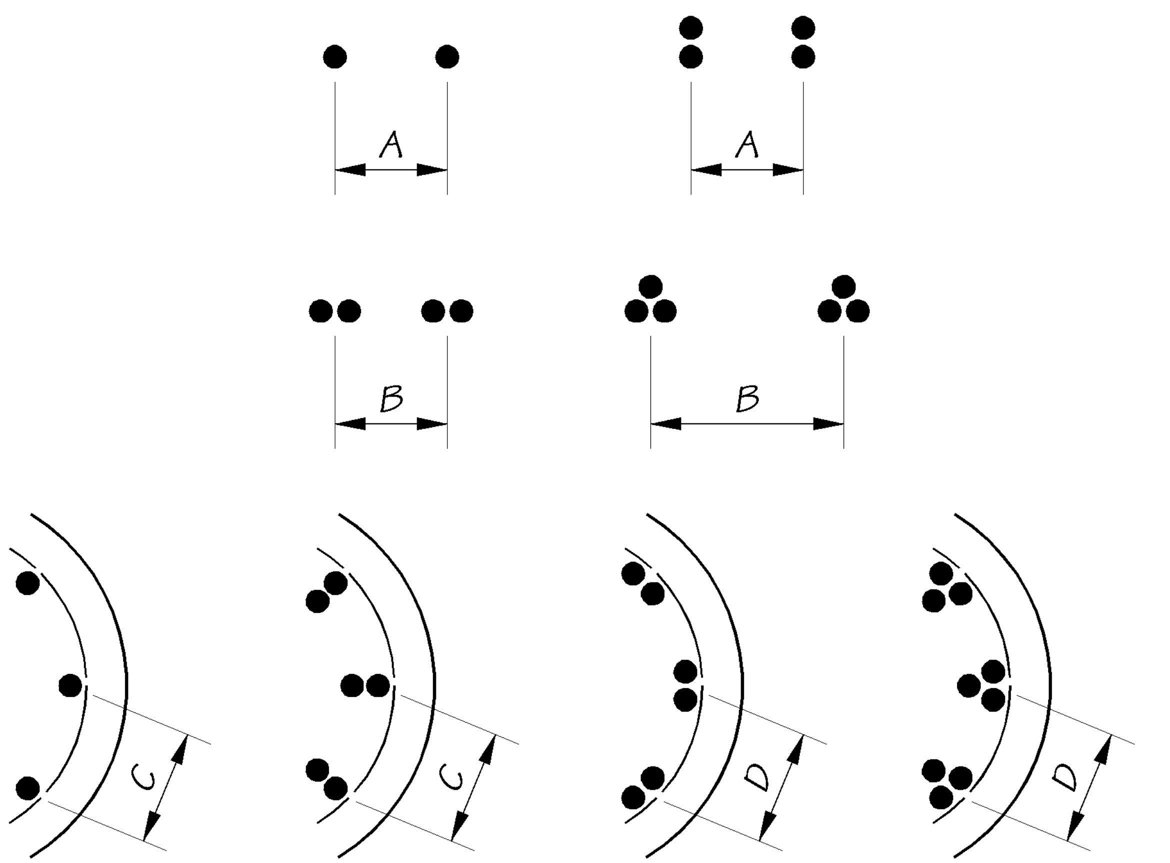
Bar Size	D	90° HOOKS	135° HOOKS	
		Hook A or G	Hook A or G	H Approx.
#3	2"	4½"	4½"	2¾"
#4	2½"	4¾"	5"	3"
#5	3¼"	6"	6"	3¾"
#6	4½"	1'-0"	8"	4½"
#7	5¼"	1'-2"	9"	5¼"
#8	6"	1'-4"	10½"	6"

(Updated 3/2022)

Appendix 5.1-A2 Minimum Reinforcement Clearance and Spacing for Beams and Columns

PREFERED MINIMUM CLEARANCE AND SPACING
FOR BEAMS AND COLUMNS.
(DISTANCES IN INCHES)

Bar Size	A	B	C	D
#4	3¼	4	-	-
5	3½	4¼	-	-
6	3¾	4½	-	-
7	4	4¾	-	-
8	4¼	5	4½	5½
9	4½	5¼	4¾	6
10	4¾	5½	5	6¼
11	5	6	5⅛	6½
14	5½	7	5½	7¼
18	6	8½	6¾	9



Appendix 5.1-A3 Reinforcing Bar Properties

Bar Size (#)	Weight (lbs/ft)	Nominal Diameter (in)	Outside Diameter (in)	Area (in^2)	Standard Mill Length (ft)
3	0.376	0.375	0.42	0.11	40
4	0.668	0.500	0.56	0.20	40
5	1.043	0.625	0.70	0.31	60
6	1.502	0.750	0.83	0.44	60
7	2.044	0.875	0.96	0.60	60
8	2.670	1.000	1.10	0.79	60
9	3.400	1.128	1.24	1.00	60
10	4.303	1.270	1.40	1.27	60
11	5.313	1.410	1.55	1.56	60
14	7.65	1.693	1.86	2.25	60
18	13.60	2.257	2.48	4.00	60

Appendix 5.1-A4 Tension Development Length of Deformed Bars

			Tension Development Length l_d of Uncoated Deformed Bars (in)							
Bar	$f'c$	Basic Dev.	λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
(#)	(ksi)	Len. l_{db} (in)	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	4.0	27.00	14.04	12.00	21.06	16.20	28.08	21.60	35.10	27.00
	5.0	24.15	12.56	12.00	18.84	14.49	25.12	19.32	31.39	24.15
	6.0	22.05	12.00	12.00	17.20	13.23	22.93	17.64	28.66	22.05
4	4.0	36.00	18.72	14.40	28.08	21.60	37.44	28.80	46.80	36.00
	5.0	32.20	16.74	12.88	25.12	19.32	33.49	25.76	41.86	32.20
	6.0	29.39	15.28	12.00	22.93	17.64	30.57	23.52	38.21	29.39
5	4.0	45.00	23.40	18.00	35.10	27.00	46.80	36.00	58.50	45.00
	5.0	40.25	20.93	16.10	31.39	24.15	41.86	32.20	52.32	40.25
	6.0	36.74	19.11	14.70	28.66	22.05	38.21	29.39	47.77	36.74
6	4.0	54.00	28.08	21.60	42.12	32.40	56.16	43.20	70.20	54.00
	5.0	48.30	25.12	19.32	37.67	28.98	50.23	38.64	62.79	48.30
	6.0	44.09	22.93	17.64	34.39	26.45	45.85	35.27	57.32	44.09
7	4.0	63.00	32.76	25.20	49.14	37.80	65.52	50.40	81.90	63.00
	5.0	56.35	29.30	22.54	43.95	33.81	58.60	45.08	73.25	56.35
	6.0	51.44	26.75	20.58	40.12	30.86	53.50	41.15	66.87	51.44
8	4.0	72.00	37.44	28.80	56.16	43.20	74.88	57.60	93.60	72.00
	5.0	64.40	33.49	25.76	50.23	38.64	66.97	51.52	83.72	64.40
	6.0	58.79	30.57	23.52	45.85	35.27	61.14	47.03	76.42	58.79
9	4.0	81.22	42.23	32.49	63.35	48.73	84.46	64.97	105.58	81.22
	5.0	72.64	37.77	29.06	56.66	43.59	75.55	58.11	94.43	72.64
	6.0	66.31	34.48	26.53	51.72	39.79	68.97	53.05	86.21	66.31
10	4.0	91.44	47.55	36.58	71.32	54.86	95.10	73.15	118.87	91.44
	5.0	81.79	42.53	32.71	63.79	49.07	85.06	65.43	106.32	81.79
	6.0	74.66	38.82	29.86	58.24	44.80	77.65	59.73	97.06	74.66
11	4.0	101.52	52.79	40.61	79.19	60.91	105.58	81.22	131.98	101.52
	5.0	90.80	47.22	36.32	70.83	54.48	94.43	72.64	118.04	90.80
	6.0	82.89	43.10	33.16	64.65	49.73	86.21	66.31	107.76	82.89
14	4.0	121.90	63.39	48.76	95.08	73.14	126.77	97.52	158.46	121.90
	5.0	109.03	56.69	43.61	85.04	65.42	113.39	87.22	141.74	109.03
	6.0	99.53	51.75	39.81	77.63	59.72	103.51	79.62	129.39	99.53
18	4.0	162.50	84.50	65.00	126.75	97.50	169.00	130.00	211.26	162.50
	5.0	145.35	75.58	58.14	113.37	87.21	151.16	116.28	188.95	145.35
	6.0	132.68	69.00	53.07	103.49	79.61	137.99	106.15	172.49	132.68

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. λ_{rc} is the Reinforcement Confinement Factor.

			Tension Development Length ld of Epoxy Coated Deformed Bars (in) (cover less than 3_{db} or clear spacing between bars less than 6_{db}) λ_{cf} = 1.5							
Bar (#)	$f'c$ (ksi)	Basic Dev. Len. l_{db} (in)	λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
			Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	4.0	27.00	18.36	16.20	27.54	24.30	36.72	32.40	45.90	40.50
	5.0	24.15	16.42	14.49	24.63	21.73	32.84	28.98	41.05	36.22
	6.0	22.05	14.99	13.23	22.49	19.84	29.98	26.45	37.48	33.07
4	4.0	36.00	24.48	21.60	36.72	32.40	48.96	43.20	61.20	54.00
	5.0	32.20	21.90	19.32	32.84	28.98	43.79	38.64	54.74	48.30
	6.0	29.39	19.99	17.64	29.98	26.45	39.98	35.27	49.97	44.09
5	4.0	45.00	30.60	27.00	45.90	40.50	61.20	54.00	76.50	67.50
	5.0	40.25	27.37	24.15	41.05	36.22	54.74	48.30	68.42	60.37
	6.0	36.74	24.98	22.05	37.48	33.07	49.97	44.09	62.46	55.11
6	4.0	54.00	36.72	32.40	55.08	48.60	73.44	64.80	91.80	81.00
	5.0	48.30	32.84	28.98	49.27	43.47	65.69	57.96	82.11	72.45
	6.0	44.09	29.98	26.45	44.97	39.68	59.96	52.91	74.95	66.14
7	4.0	63.00	42.84	37.80	64.26	56.70	85.68	75.60	107.10	94.50
	5.0	56.35	38.32	33.81	57.48	50.71	76.63	67.62	95.79	84.52
	6.0	51.44	34.98	30.86	52.47	46.30	69.96	61.73	87.45	77.16
8	4.0	72.00	48.96	43.20	73.44	64.80	97.92	86.40	122.40	108.00
	5.0	64.40	43.79	38.64	65.69	57.96	87.58	77.28	109.48	96.60
	6.0	58.79	39.98	35.27	59.96	52.91	79.95	70.55	99.94	88.18
9	4.0	81.22	55.23	48.73	82.84	73.09	110.45	97.46	138.07	121.82
	5.0	72.64	49.40	43.59	74.09	65.38	98.79	87.17	123.49	108.96
	6.0	66.31	45.09	39.79	67.64	59.68	90.19	79.58	112.73	99.47
10	4.0	91.44	62.18	54.86	93.27	82.30	124.36	109.73	155.45	137.16
	5.0	81.79	55.61	49.07	83.42	73.61	111.23	98.14	139.04	122.68
	6.0	74.66	50.77	44.80	76.15	67.19	101.54	89.59	126.92	111.99
11	4.0	101.52	69.03	60.91	103.55	91.37	138.07	121.82	172.58	152.28
	5.0	90.80	61.75	54.48	92.62	81.72	123.49	108.96	154.36	136.20
	6.0	82.89	56.37	49.73	84.55	74.60	112.73	99.47	140.91	124.34
14	4.0	121.90	82.89	73.14	124.33	109.71	165.78	146.28	207.22	182.84
	5.0	109.03	74.14	65.42	111.21	98.12	148.28	130.83	185.35	163.54
	6.0	99.53	67.68	59.72	101.52	89.57	135.36	119.43	169.20	149.29
18	4.0	162.50	110.50	97.50	165.75	146.25	221.01	195.00	276.26	243.76
	5.0	145.35	98.84	87.21	148.25	130.81	197.67	174.42	247.09	218.02
	6.0	132.68	90.23	79.61	135.34	119.42	180.45	159.22	225.56	199.03

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. λ_{rc} is the Reinforcement Confinement Factor.

Bar (#)	$f'c$ (ksi)	Basic Dev. Len. l_{db} (in)	Tension Development Length l_d of Epoxy Coated Deformed Bars (in) (cover not less than 3_{db} and clear spacing between bars not less than 6_{db}) λ_{cf} = 1.2							
			λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
			Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	4.0	27.00	16.85	12.96	25.27	19.44	33.70	25.92	42.12	32.40
	5.0	24.15	15.07	12.00	22.60	17.39	30.14	23.18	37.67	28.98
	6.0	22.05	13.76	12.00	20.63	15.87	27.51	21.16	34.39	26.45
4	4.0	36.00	22.46	17.28	33.70	25.92	44.93	34.56	56.16	43.20
	5.0	32.20	20.09	15.46	30.14	23.18	40.18	30.91	50.23	38.64
	6.0	29.39	18.34	14.11	27.51	21.16	36.68	28.22	45.85	35.27
5	4.0	45.00	28.08	21.60	42.12	32.40	56.16	43.20	70.20	54.00
	5.0	40.25	25.12	19.32	37.67	28.98	50.23	38.64	62.79	48.30
	6.0	36.74	22.93	17.64	34.39	26.45	45.85	35.27	57.32	44.09
6	4.0	54.00	33.70	25.92	50.54	38.88	67.39	51.84	84.24	64.80
	5.0	48.30	30.14	23.18	45.21	34.78	60.28	46.37	75.35	57.96
	6.0	44.09	27.51	21.16	41.27	31.75	55.03	42.33	68.78	52.91
7	4.0	63.00	39.31	30.24	58.97	45.36	78.62	60.48	98.28	75.60
	5.0	56.35	35.16	27.05	52.74	40.57	70.32	54.09	87.90	67.62
	6.0	51.44	32.10	24.69	48.15	37.04	64.20	49.38	80.25	61.73
8	4.0	72.00	44.93	34.56	67.39	51.84	89.86	69.12	112.32	86.40
	5.0	64.40	40.18	30.91	60.28	46.37	80.37	61.82	100.46	77.28
	6.0	58.79	36.68	28.22	55.03	42.33	73.37	56.44	91.71	70.55
9	4.0	81.22	50.68	38.98	76.02	58.48	101.36	77.97	126.70	97.46
	5.0	72.64	45.33	34.87	67.99	52.30	90.66	69.74	113.32	87.17
	6.0	66.31	41.38	31.83	62.07	47.75	82.76	63.66	103.45	79.58
10	4.0	91.44	57.06	43.89	85.59	65.84	114.12	87.78	142.65	109.73
	5.0	81.79	51.03	39.26	76.55	58.89	102.07	78.51	127.59	98.14
	6.0	74.66	46.59	35.84	69.88	53.76	93.18	71.67	116.47	89.59
11	4.0	101.52	63.35	48.73	95.02	73.09	126.70	97.46	158.37	121.82
	5.0	90.80	56.66	43.59	84.99	65.38	113.32	87.17	141.65	108.96
	6.0	82.89	51.72	39.79	77.59	59.68	103.45	79.58	129.31	99.47
14	4.0	121.90	76.06	58.51	114.09	87.77	152.13	117.02	190.16	146.28
	5.0	109.03	68.03	52.33	102.05	78.50	136.07	104.67	170.08	130.83
	6.0	99.53	62.11	47.77	93.16	71.66	124.21	95.55	155.26	119.43
18	4.0	162.50	101.40	78.00	152.10	117.00	202.80	156.00	253.51	195.00
	5.0	145.35	90.70	69.77	136.05	104.65	181.39	139.53	226.74	174.42
	6.0	132.68	82.79	63.69	124.19	95.53	165.59	127.38	206.99	159.22

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. λ_{rc} is the Reinforcement Confinement Factor.

Appendix 5.1-A5 Compression Development Length and Minimum Lap Splice of Grade 60 Bars

Bar (#)	Compression Development Length, l_d (in)			Min. Compression Lap Splice (in)
	f'_c = 3 ksi	f'_c = 4 ksi	$f'_c \geq$ 5 ksi	$f'c \geq$ 3 ksi
3	12.00	12.00	12.00	24.00
4	12.00	12.00	12.00	24.00
5	13.64	12.00	12.00	24.00
6	16.37	14.18	13.50	24.00
7	19.10	16.54	15.75	26.25
8	21.82	18.90	18.00	30.00
9	24.62	21.32	20.30	33.84
10	27.72	24.00	22.86	38.10
11	30.77	26.65	25.38	42.30
14	36.95	32.00	30.47	50.79
18	49.26	42.66	40.63	67.71

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 90º and 180º Standard Hooks

Bar (#)	$f'c$ (ksi)	Basic Dev. Len. l_{hb} (in)	Standard Hook Tension Development Length l_{dh} (in) — Cover Factor λ_{rc} = 1.0 (see Note 5)		Cover Factor λ_{rc} = 0.8 (see Note 5)	
			Reinforcement Confinement Factor λ_{rc} = 1.0 (see Notes 6 and 7)	Reinforcement Confinement Factor λ_{rc} = 0.8 (see Notes 6 and 7)	Reinforcement Confinement Factor λ_{rc} = 1.0 (see Notes 6 and 7)	Reinforcement Confinement Factor λ_{rc} = 0.8 (see Notes 6 and 7)
3	4	7.13	7.13	6.00	6.00	6.00
	5	6.37	6.37	6.00	6.00	6.00
	6	5.82	6.00	6.00	6.00	6.00
4	4	9.50	9.50	7.60	7.60	6.08
	5	8.50	8.50	6.80	6.80	6.00
	6	7.76	7.76	6.21	6.21	6.00
5	4	11.88	11.88	9.50	9.50	7.60
	5	10.62	10.62	8.50	8.50	6.80
	6	9.70	9.70	7.76	7.76	6.21
6	4	14.25	14.25	11.40	11.40	9.12
	5	12.75	12.75	10.20	10.20	8.16
	6	11.64	11.64	9.31	9.31	7.45
7	4	16.63	16.63	13.30	13.30	10.64
	5	14.87	14.87	11.90	11.90	9.52
	6	13.57	13.57	10.86	10.86	8.69
8	4	19.00	19.00	15.20	15.20	12.16
	5	16.99	16.99	13.60	13.60	10.88
	6	15.51	15.51	12.41	12.41	9.93
9	4	21.43	21.43	17.15	17.15	13.72
	5	19.17	19.17	15.34	15.34	12.27
	6	17.50	17.50	14.00	14.00	11.20
10	4	24.13	24.13	19.30	19.30	15.44
	5	21.58	21.58	17.27	17.27	13.81
	6	19.70	19.70	15.76	15.76	12.61
11	4	26.79	26.79	21.43	21.43	17.15
	5	23.96	23.96	19.17	19.17	15.34
	6	21.87	21.87	17.50	17.50	14.00
14	4	32.17	32.17	32.17	32.17	32.17
	5	28.77	28.77	28.77	28.77	28.77
	6	26.26	26.26	26.26	26.26	26.26
18	4	42.88	42.88	42.88	42.88	42.88
	5	38.36	38.36	38.36	38.36	38.36
	6	35.01	35.01	35.01	35.01	35.01

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.

Appendix 5.1-A7 Tension Lap Splice Lengths of Grade 60 Bars – Class B

Bar (#)	$f'c$ (ksi)	Class B Tension Lap Splice Length of Uncoated Deformed Bars (in)							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
		Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	4.0	24.00	24.00	27.38	24.00	36.50	28.08	45.63	35.10
	5.0	24.00	24.00	24.49	24.00	32.65	25.12	40.81	31.39
	6.0	24.00	24.00	24.00	24.00	29.81	24.00	37.26	28.66
4	4.0	24.34	24.00	36.50	28.08	48.67	37.44	60.84	46.80
	5.0	24.00	24.00	32.65	25.12	43.53	33.49	54.42	41.86
	6.0	24.00	24.00	29.81	24.00	39.74	30.57	49.68	38.21
5	4.0	30.42	24.00	45.63	35.10	60.84	46.80	76.05	58.50
	5.0	27.21	24.00	40.81	31.39	54.42	41.86	68.02	52.32
	6.0	24.84	24.00	37.26	28.66	49.68	38.21	62.09	47.77
6	4.0	36.50	28.08	54.76	42.12	73.01	56.16	91.26	70.20
	5.0	32.65	25.12	48.98	37.67	65.30	50.23	81.63	62.79
	6.0	29.81	24.00	44.71	34.39	59.61	45.85	74.51	57.32
7	4.0	42.59	32.76	63.88	49.14	85.18	65.52	106.47	81.90
	5.0	38.09	29.30	57.14	43.95	76.18	58.60	95.23	73.25
	6.0	34.77	26.75	52.16	40.12	69.55	53.50	86.93	66.87
8	4.0	48.67	37.44	73.01	56.16	97.34	74.88	121.68	93.60
	5.0	43.53	33.49	65.30	50.23	87.07	66.97	108.83	83.72
	6.0	39.74	30.57	59.61	45.85	79.48	61.14	99.35	76.42
9	4.0	54.90	42.23	82.35	63.35	109.80	84.46	137.26	105.58
	5.0	49.11	37.77	73.66	56.66	98.21	75.55	122.76	94.43
	6.0	44.83	34.48	67.24	51.72	89.65	68.97	112.07	86.21
10	4.0	61.81	47.55	92.72	71.32	123.63	95.10	154.53	118.87
	5.0	55.29	42.53	82.93	63.79	110.58	85.06	138.22	106.32
	6.0	50.47	38.82	75.71	58.24	100.94	77.65	126.18	97.06
11	4.0	68.63	52.79	102.94	79.19	137.26	105.58	171.57	131.98
	5.0	61.38	47.22	92.07	70.83	122.76	94.43	153.46	118.04
	6.0	56.03	43.10	84.05	64.65	112.07	86.21	140.09	107.76
14		Not Allowed		Not Allowed		Not Allowed		Not Allowed	
18		Not Allowed		Not Allowed		Not Allowed		Not Allowed	

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. λ_{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.

Bar (#)	$f'c$ (ksi)	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}) λ_{cf} = 1.5							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
		Top Bars	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**
3	4.0	24.00	24.00	35.80	31.59	47.74	42.12	59.67	52.65
	5.0	24.00	24.00	32.02	28.25	42.70	37.67	53.37	47.09
	6.0	24.00	24.00	29.23	25.79	38.98	34.39	48.72	42.99
4	4.0	31.82	28.08	47.74	42.12	63.65	56.16	79.56	70.20
	5.0	28.46	25.12	42.70	37.67	56.93	50.23	71.16	62.79
	6.0	25.98	24.00	38.98	34.39	51.97	45.85	64.96	57.32
5	4.0	39.78	35.10	59.67	52.65	79.56	70.20	99.45	87.75
	5.0	35.58	31.39	53.37	47.09	71.16	62.79	88.95	78.49
	6.0	32.48	28.66	48.72	42.99	64.96	57.32	81.20	71.65
6	4.0	47.74	42.12	71.60	63.18	95.47	84.24	119.34	105.30
	5.0	42.70	37.67	64.04	56.51	85.39	75.35	106.74	94.18
	6.0	38.98	34.39	58.46	51.59	77.95	68.78	97.44	85.98
7	4.0	55.69	49.14	83.54	73.71	111.38	98.28	139.23	122.85
	5.0	49.81	43.95	74.72	65.93	99.62	87.90	124.53	109.88
	6.0	45.47	40.12	68.21	60.18	90.94	80.25	113.68	100.31
8	4.0	63.65	56.16	95.47	84.24	127.30	112.32	159.12	140.40
	5.0	56.93	50.23	85.39	75.35	113.86	100.46	142.32	125.58
	6.0	51.97	45.85	77.95	68.78	103.94	91.71	129.92	114.64
9	4.0	71.79	63.35	107.69	95.02	143.59	126.70	179.49	158.37
	5.0	64.22	56.66	96.32	84.99	128.43	113.32	160.54	141.65
	6.0	58.62	51.72	87.93	77.59	117.24	103.45	146.55	129.31
10	4.0	80.83	71.32	121.25	106.98	161.67	142.65	202.08	178.31
	5.0	72.30	63.79	108.45	95.69	144.60	127.59	180.75	159.48
	6.0	66.00	58.24	99.00	87.35	132.00	116.47	165.00	145.59
11	4.0	89.74	79.19	134.62	118.78	179.49	158.37	224.36	197.96
	5.0	80.27	70.83	120.40	106.24	160.54	141.65	200.67	177.06
	6.0	73.28	64.65	109.91	96.98	146.55	129.31	183.19	161.64
14		Not Allowed		Not Allowed		Not Allowed		Not Allowed	
18		Not Allowed		Not Allowed		Not Allowed		Not Allowed	

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. λ_{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.

Bar (#)	$f'c$ (ksi)	Class B Tension Lap Splice Length of Epoxy Coated Deformed Bars (in) (cover not less than 3_{db} and clear spacing between bars not less than 6_{db}) λ_{cf} = 1.2							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
		Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	4.0	24.00	24.00	32.85	25.27	43.80	33.70	54.76	42.12
	5.0	24.00	24.00	29.39	24.00	39.18	30.14	48.98	37.67
	6.0	24.00	24.00	26.82	24.00	35.77	27.51	44.71	34.39
4	4.0	29.20	24.00	43.80	33.70	58.41	44.93	73.01	56.16
	5.0	26.12	24.00	39.18	30.14	52.24	40.18	65.30	50.23
	6.0	24.00	24.00	35.77	27.51	47.69	36.68	59.61	45.85
5	4.0	36.50	28.08	54.76	42.12	73.01	56.16	91.26	70.20
	5.0	32.65	25.12	48.98	37.67	65.30	50.23	81.63	62.79
	6.0	29.81	24.00	44.71	34.39	59.61	45.85	74.51	57.32
6	4.0	43.80	33.70	65.71	50.54	87.61	67.39	109.51	84.24
	5.0	39.18	30.14	58.77	45.21	78.36	60.28	97.95	75.35
	6.0	35.77	27.51	53.65	41.27	71.53	55.03	89.42	68.78
7	4.0	51.11	39.31	76.66	58.97	102.21	78.62	127.76	98.28
	5.0	45.71	35.16	68.57	52.74	91.42	70.32	114.28	87.90
	6.0	41.73	32.10	62.59	48.15	83.46	64.20	104.32	80.25
8	4.0	58.41	44.93	87.61	67.39	116.81	89.86	146.02	112.32
	5.0	52.24	40.18	78.36	60.28	104.48	80.37	130.60	100.46
	6.0	47.69	36.68	71.53	55.03	95.38	73.37	119.22	91.71
9	4.0	65.88	50.68	98.82	76.02	131.76	101.36	164.71	126.70
	5.0	58.93	45.33	88.39	67.99	117.85	90.66	147.32	113.32
	6.0	53.79	41.38	80.69	62.07	107.59	82.76	134.48	103.45
10	4.0	74.18	57.06	111.26	85.59	148.35	114.12	185.44	142.65
	5.0	66.35	51.03	99.52	76.55	132.69	102.07	165.86	127.59
	6.0	60.56	46.59	90.85	69.88	121.13	93.18	151.41	116.47
11	4.0	82.35	63.35	123.53	95.02	164.71	126.70	205.88	158.37
	5.0	73.66	56.66	110.49	84.99	147.32	113.32	184.15	141.65
	6.0	67.24	51.72	100.86	77.59	134.48	103.45	168.10	129.31
14		Not Allowed		Not Allowed		Not Allowed		Not Allowed	
18		Not Allowed		Not Allowed		Not Allowed		Not Allowed	

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. λ_{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.

Appendix 5.1-A8 Prestressing Strand Properties and Development Length

AASHTO M203 Grade 270 Uncoated Prestressing Strands
Properties and Development Length

Strand Diameter (in)	Weight (lbs/ft)	Nominal Diameter (in)	Area (in^2)	Transfer length (in)	Develop. Length *k* = 1.0 (ft)	Develop. Length *k* = 1.6 (ft)
⅜	0.290	0.375	0.085	22.5	5.05	8.08
7⁄16	0.390	0.438	0.115	26.3	5.90	9.44
½	0.520	0.500	0.153	30.0	6.74	10.78
½ S	0.568	0.520	0.167	31.2	7.01	11.21
9⁄16	0.651	0.563	0.192	33.8	7.58	12.14
0.60	0.740	0.600	0.217	36.0	8.08	12.93
0.62	0.788	0.620	0.231	37.2	8.35	13.36
0.70	1.000	0.700	0.294	42.0	9.43	15.09

Assumptions for determining development length:

f_{ps} = f_{pu} = 270 ksi
f_{pe} = (270 ksi x 0.75)–40 ksi = 162.5 ksi

Appendix 5.2-A1 Working Stress Design

Service Load — Concrete Stresses and Constants

	CLASS	CLASS
n (See E_c below)	8	10
f'_c	4000 psi.	3000 psi.
f_c (Compression)	1600	1200
f_t (Tension) Use only with special permission	100	86
f_s (Grade 40)	20,000	20,000
f_s (Grade 60)	24,000	24,000
v_c (With web reinf.)	313	271
v_c	60 *	52 *
Slabs & Footings (Peripheral Shear)		
v_c	114	99
v_c (With web reinf.)	190	164
K ($f_s = 24000$)	246	
K — Balanced rectangular sections	272	197
k — Balanced rectangular sections	.390	.375
j — Balanced rectangular sections	.870	.875
p — Balanced rectangular sections	.0156	.01125
E_c (for stress calc.) (n as above)	522,000 K/□'	418,000 K/□'
E_c (for short term defl. due to E.Q., etc.) ($n = 8$)	522,000	522,000
E_c (for D.L. Camber of Slabs, T&S Moms., Settlement) ($n = 16$)	261,000	261,000
E_c (for D.L. Camber, except slabs) ($n = 24$)	174,000	174,000

Temp. Coeff. = .000006 °/° ~ 45° Drop to 35° Rise ~ All climates.
Shrinkage Coeff. = .0002 °/° (Temp. rise & shrinkage cancel).

* For more detailed analysis $v_c = 0.9(f'_c)^{\frac{1}{2}} + 1100\,p_w\left(\frac{Vd}{M}\right)$
See 1974 A.A.S.H.T.O. Interim 1.5.29 (B)(2).

Stirrup spacing; (Kip & inch units)

$$S = \frac{A_s \times f_s \times jd}{V - V_c} = \frac{A_s \times 20 \times 7/8\, d}{V_s} = \frac{17.50\, A_s \times d}{V_s}$$

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. = $v_c \times bjd$

o

$$d = \sqrt{M/bK} \quad \text{(Balanced rectangular section)}$$

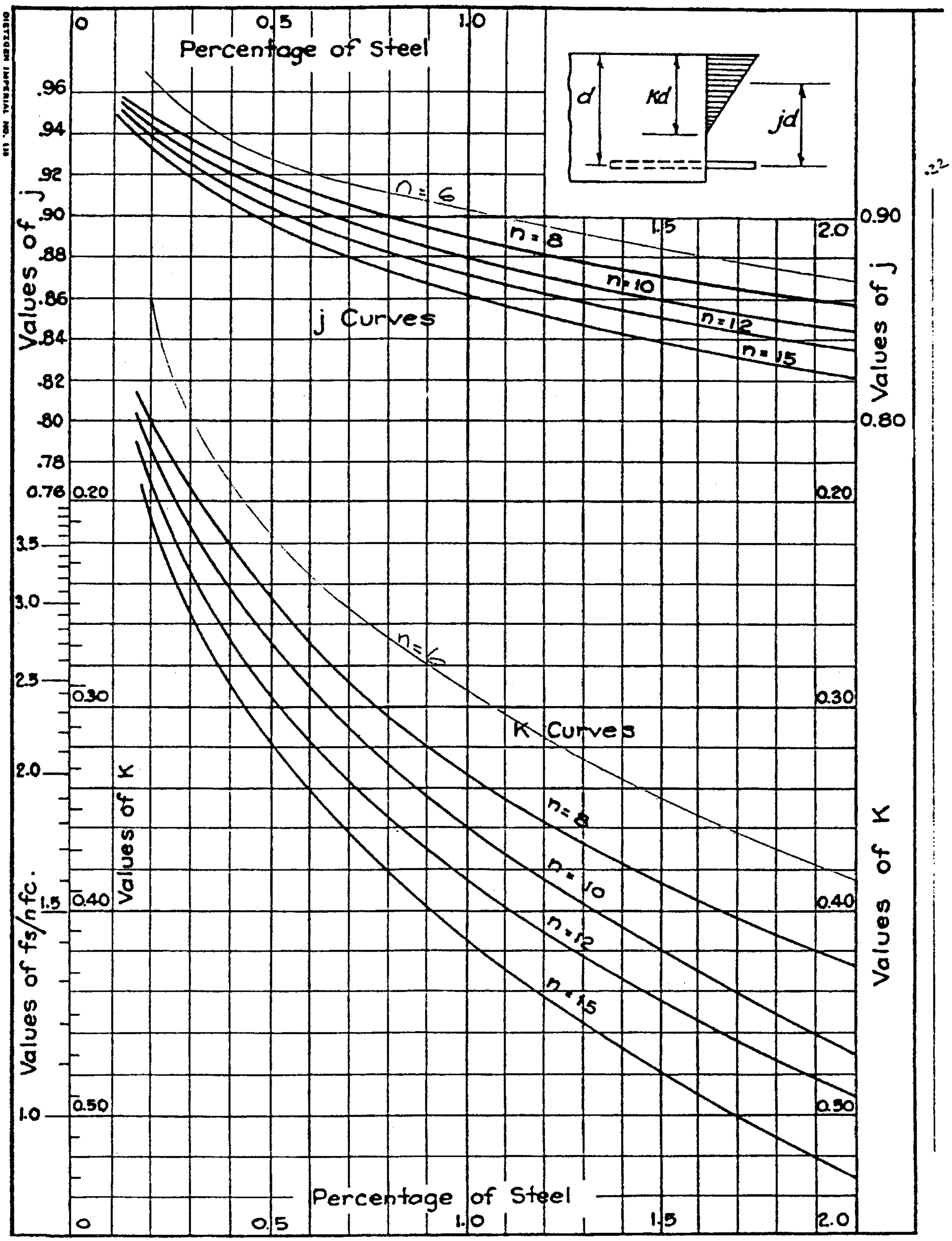
$$f_c = \frac{2M}{kjbd^2} \quad \text{(Rectangular section)}$$

$$f_s = \frac{M}{A_s jd}$$

$$v = \frac{V}{bjd}$$

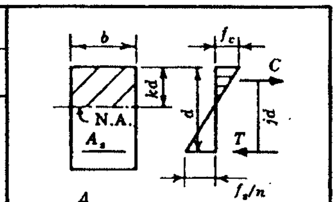
$$n = \frac{E_s}{E_c}$$

Appendix 5.2-A2 Working Stress Design



Appendix 5.2-A3 Working Stress Design

COEFFICIENTS (K, k, j, p) FOR RECTANGULAR SECTIONS



$$p = \frac{A_s}{bd}$$

$$k = \frac{1}{1 + f_s/nf_c} \qquad j = 1 - \tfrac{1}{3}k$$

$$p^{*} = \frac{f_c}{2f_s} \times k \qquad K = \frac{f_c}{2}kj$$

$$a = \frac{f_s}{12{,}000} \times \text{(av. } j\text{-value)}$$

for use in

$$A_s = \frac{M}{ad} \quad \text{or} \quad A_s = \frac{NE}{adi}$$

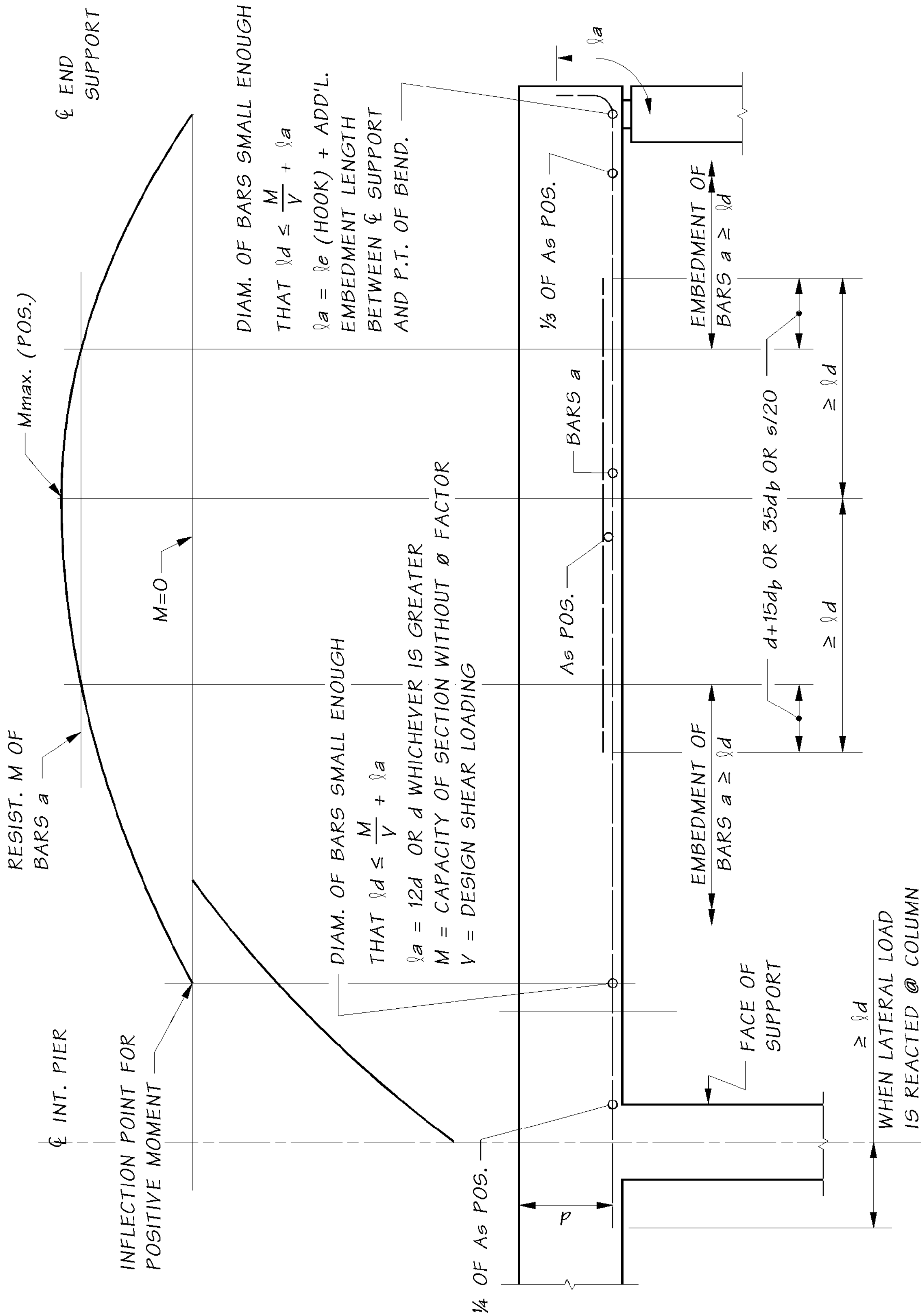
f'_c and n	f_c	K	k	j	p	K	k	j	p
		$f_s = 16{,}000$		$a = 1.13$		$f_s = 18{,}000$		$a = 1.29$	
	875.	137.	.356	.881	.0097	128.	.329	.890	.0080
2500	1000.	169.	.387	.871	.0121	158.	.359	.880	.0100
	1125.	201.	.415	.862	.0146	190.	.387	.871	.0121
10.1	1250.	235.	.441	.853	.0172	222.	.412	.863	.0143
	1500.	306.	.486	.838	.0228	291.	.457	.848	.0190
	1050.	173.	.376	.875	.0124	162.	.349	.884	.0102
3000	1200.	212.	.408	.864	.0153	199.	.380	.873	.0127
	1350.	252.	.437	.854	.0184	238.	.408	.864	.0153
9.2	1500.	294.	.463	.846	.0217	278.	.434	.855	.0181
	1800.	380.	.509	.830	.0286	362.	.479	.840	.0240
	1400.	249.	.412	.863	.0180	234.	.384	.872	.0149
4000	1600.	303.	.444	.852	.0222	286.	.416	.861	.0185
	1800.	359.	.474	.842	.0266	341.	.444	.852	.0222
8.0	2000.	417.	.500	.833	.0313	397.	.471	.843	.0261
	2400.	536.	.545	.818	.0409	513.	.516	.828	.0344
	1750.	327.	.437	.854	.0239	309.	.408	.864	.0199
5000	2000.	397.	.470	.843	.0294	376.	.441	.853	.0245
	2250.	468.	.500	.833	.0351	446.	.470	.843	.0294
7.1	2500.	542.	.526	.825	.0411	518.	.497	.835	.0345
	3000.	694.	.571	.810	.0535	666.	.542	.819	.0452

f'_c and n	f_c	K	k	j	p	K	k	j	p	K	k	j	p
		$f_s = 20{,}000$		$a = 1.44$		$f_s = 22{,}000$		$a = 1.60$		$f_s = 24{,}000$		$a = 1.76$	
	875.	120.	.306	.898	.0067	113.	.287	.904	.0057	107.	.269	.910	.0049
2500	1000.	149.	.336	.888	.0084	141.	.315	.895	.0072	133.	.296	.901	.0062
	1125.	179.	.362	.879	.0102	170.	.341	.886	.0087	161.	.321	.893	.0075
10.1	1250.	211.	.387	.871	.0121	200.	.365	.878	.0104	191.	.345	.885	.0090
	1500.	277.	.431	.856	.0162	264.	.408	.864	.0139	253.	.387	.871	.0121
	1050.	152.	.326	.891	.0085	144.	.305	.898	.0073	136.	.287	.904	.0063
3000	1200.	188.	.356	.881	.0107	178.	.334	.889	.0091	169.	.315	.895	.0079
	1350.	226.	.383	.872	.0129	214.	.361	.880	.0111	204.	.341	.886	.0096
9.2	1500.	265.	.408	.864	.0153	252.	.385	.872	.0131	240.	.365	.878	.0114
	1800.	346.	.453	.849	.0204	331.	.429	.857	.0176	317.	.408	.864	.0153
	1400.	221.	.359	.880	.0126	210.	.337	.888	.0107	199.	.318	.894	.0093
4000	1600.	272.	.390	.870	.0156	258.	.368	.877	.0134	246.	.348	.884	.0116
	1800.	324.	.419	.860	.0188	309.	.396	.868	.0162	295.	.375	.875	.0141
8.0	2000.	379.	.444	.852	.0222	362.	.421	.860	.0191	347.	.400	.867	.0167
	2400.	492.	.490	.837	.0294	472.	.466	.845	.0254	454.	.444	.852	.0222
	1750.	292.	.383	.872	.0168	278.	.361	.880	.0144	265.	.341	.886	.0124
5000	2000.	358.	.415	.862	.0208	341.	.392	.869	.0178	326.	.372	.876	.0155
	2250.	426.	.444	.852	.0250	407.	.421	.860	.0215	390.	.400	.867	.0187
7.1	2500.	496.	.470	.843	.0294	475.	.447	.851	.0254	456.	.425	.858	.0221
	3000.	641.	.516	.828	.0387	617.	.492	.836	.0335	595.	.470	.843	.0294

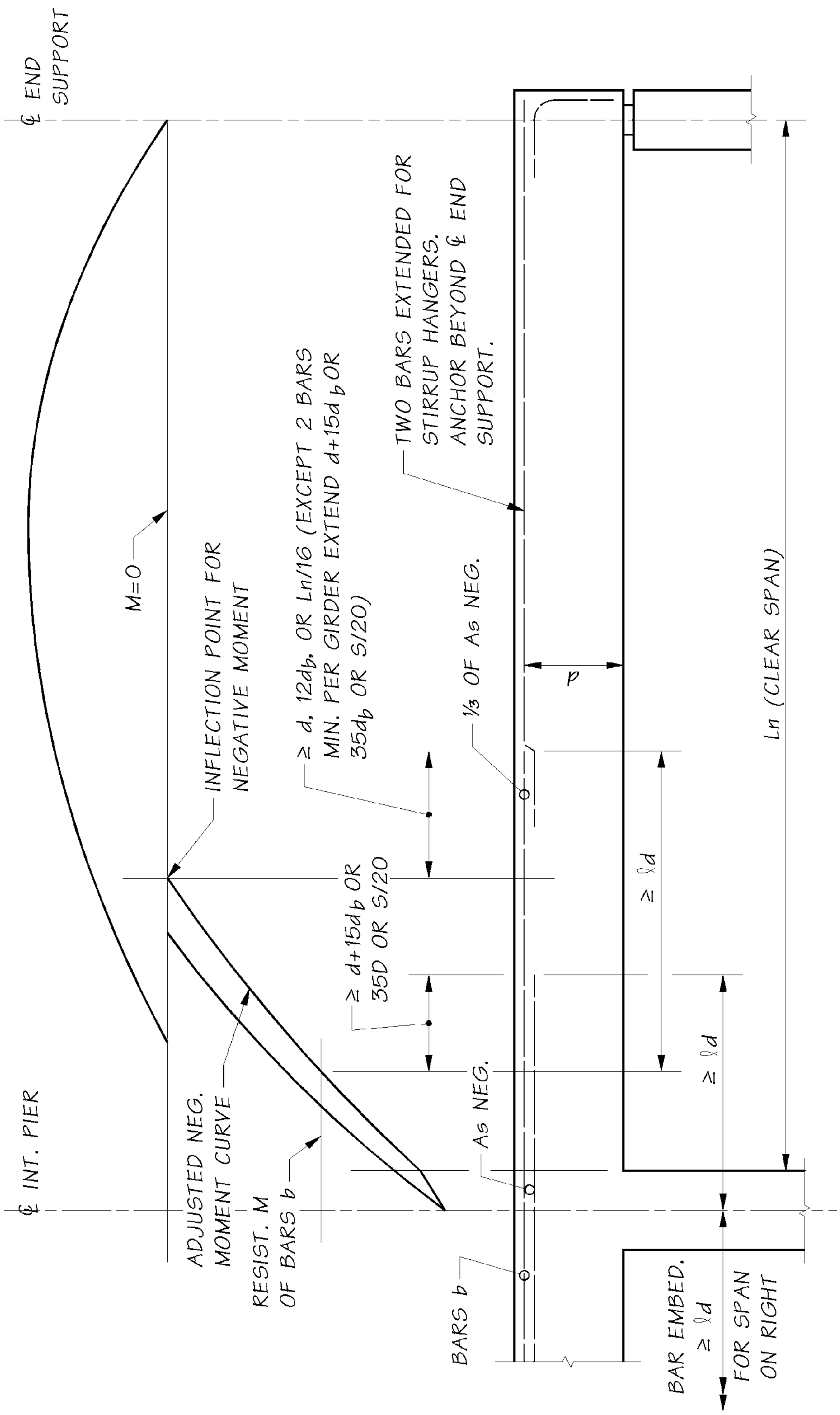
f'_c and n	f_c	K	k	j	p	K	k	j	p	K	k	j	p
		$f_s = 27{,}000$		$a = 2.00$		$f_s = 30{,}000$		$a = 2.24$		$f_s = 33{,}000$		$a = 2.48$	
	875.	99.	.247	.918	.0040	92.	.228	.924	.0033	86.	.211	.930	.0028
2500	1000.	124.	.272	.909	.0050	115.	.252	.916	.0042	108.	.234	.922	.0036
	1125.	150.	.296	.901	.0062	140.	.275	.908	.0052	132.	.256	.915	.0044
10.1	1250.	178.	.319	.894	.0074	167.	.296	.901	.0062	157.	.277	.908	.0052
	1500.	237.	.359	.880	.0100	224.	.336	.888	.0084	211.	.315	.895	.0072
	1050.	126.	.264	.912	.0051	117.	.244	.919	.0043	110.	.226	.925	.0036
3000	1200.	157.	.290	.903	.0064	147.	.269	.910	.0054	138.	.251	.916	.0046
	1350.	190.	.315	.895	.0079	178.	.293	.902	.0066	168.	.273	.909	.0056
9.2	1500.	225.	.338	.887	.0094	211.	.315	.895	.0079	199.	.295	.902	.0067
	1800.	299.	.380	.873	.0127	282.	.356	.881	.0107	267.	.334	.889	.0091
	1400.	185.	.293	.902	.0076	173.	.272	.909	.0063	162.	.253	.916	.0054
4000	1600.	230.	.322	.893	.0095	215.	.299	.900	.0080	203.	.279	.907	.0068
	1800.	277.	.348	.884	.0116	260.	.324	.892	.0097	246.	.304	.899	.0083
8.0	2000.	326.	.372	.876	.0138	308.	.348	.884	.0116	291.	.327	.891	.0099
	2400.	430.	.416	.861	.0185	407.	.390	.870	.0156	387.	.368	.877	.0134
	1750.	247.	.315	.895	.0102	231.	.293	.902	.0085	218.	.274	.909	.0073
5000	2000.	305.	.345	.885	.0128	287.	.321	.893	.0107	271.	.301	.900	.0091
	2250.	366.	.372	.876	.0155	346.	.347	.884	.0130	327.	.326	.891	.0111
7.1	2500.	430.	.397	.868	.0184	407.	.372	.876	.0155	386.	.350	.883	.0132
	3000.	564.	.441	.853	.0245	537.	.415	.862	.0208	511.	.392	.869	.0178

*"Balanced steel ratio" applies to problems involving bending only.

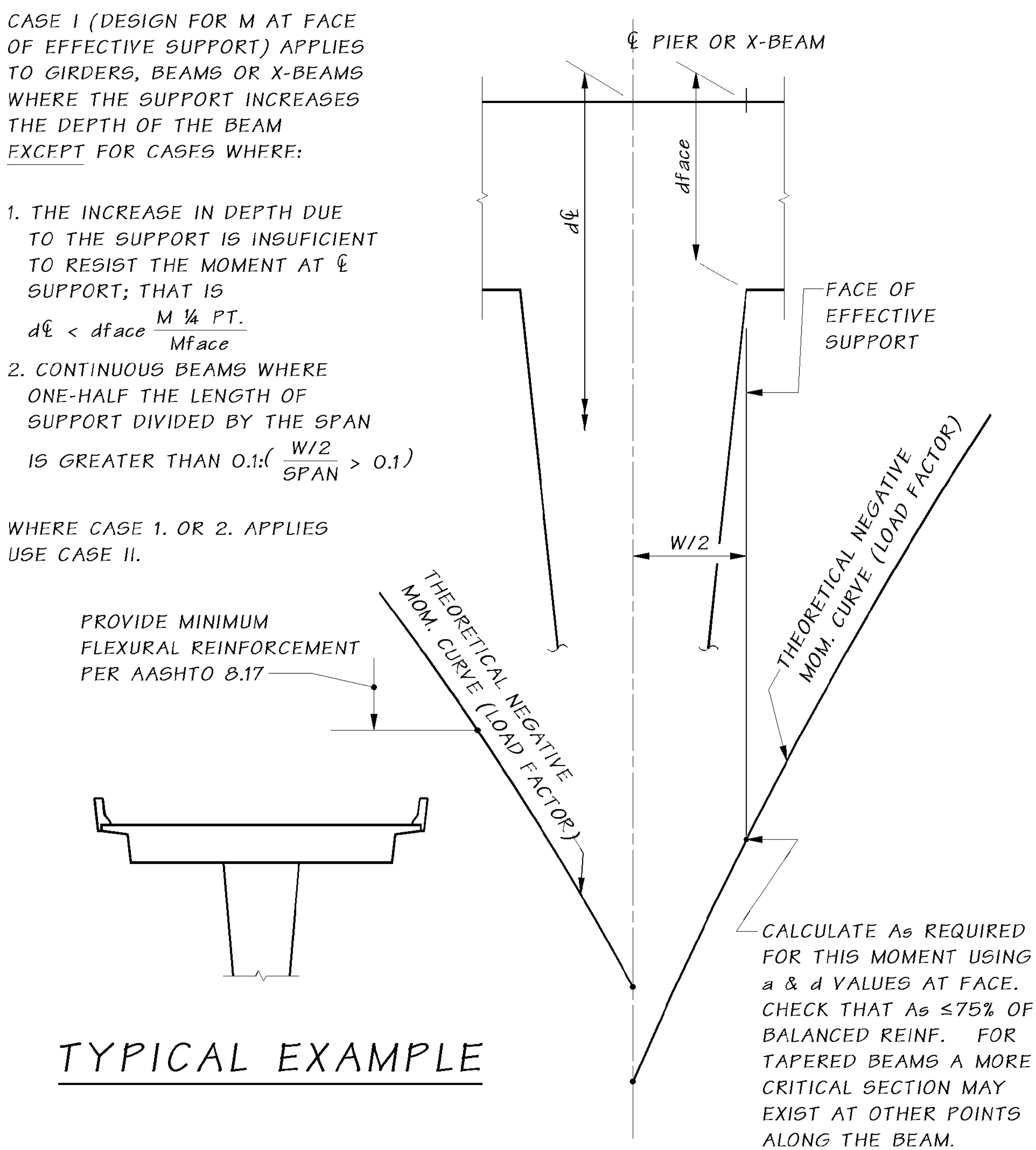
Appendix 5.3-A1 Positive Moment Reinforcement



Appendix 5.3-A2 Negative Moment Reinforcement



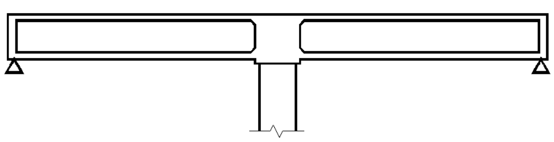
Appendix 5.3-A3 Adjusted Negative Moment Case I (Design for M at Face of Support)



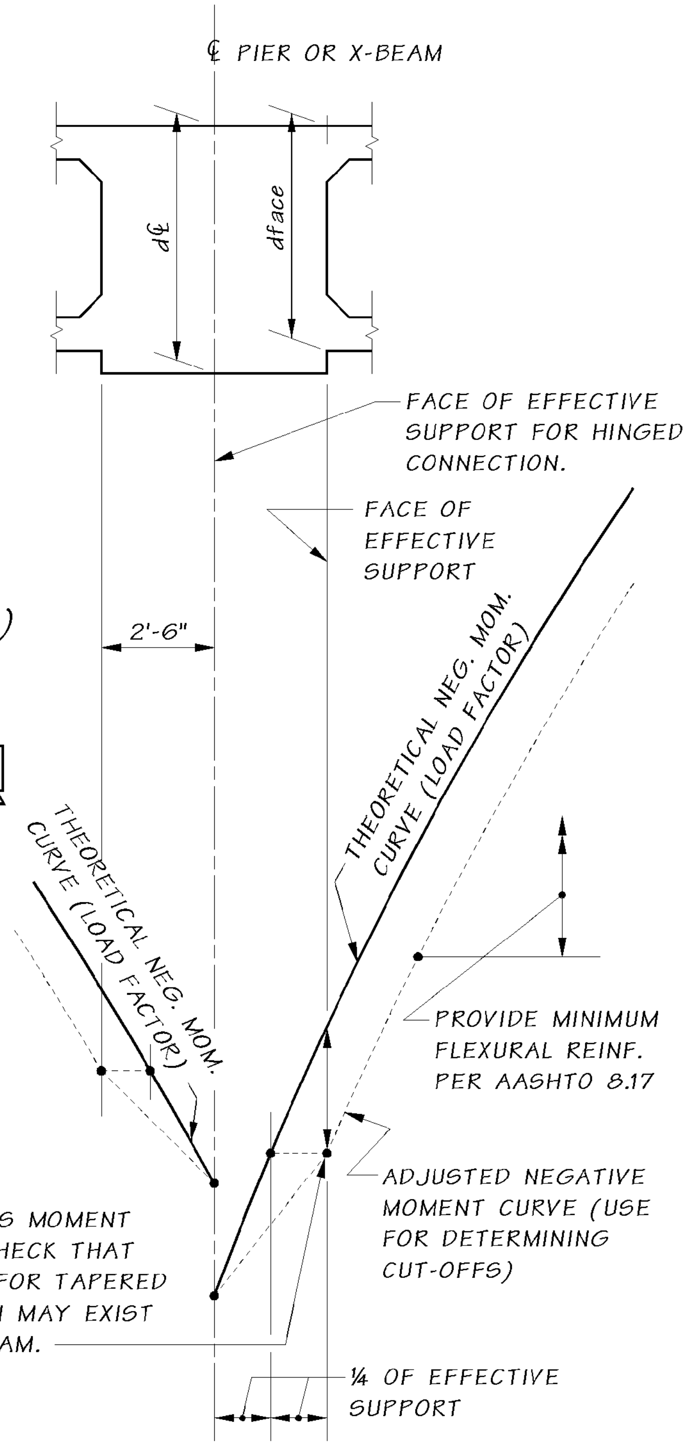
Appendix 5.3-A4 Adjusted Negative Moment Case II (Design for M at 1/4 Point)

CASE II (DESIGN FOR M ½ 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 INSUFICIENT TO RESIST THE MOMENT AT ℄ SUPPORT; THAT IS
   $d℄ \leq dface \dfrac{M\ ¼\ PT.}{Mface}$
3. CONTINUOUS BEAMS WHERE ONE-HALF THE LENGTH OF SUPPORT DIVIDED BY THE SPAN IS GREATER THAN 0.1: $\left(\dfrac{W/2}{SPAN} > 0.1 \right)$

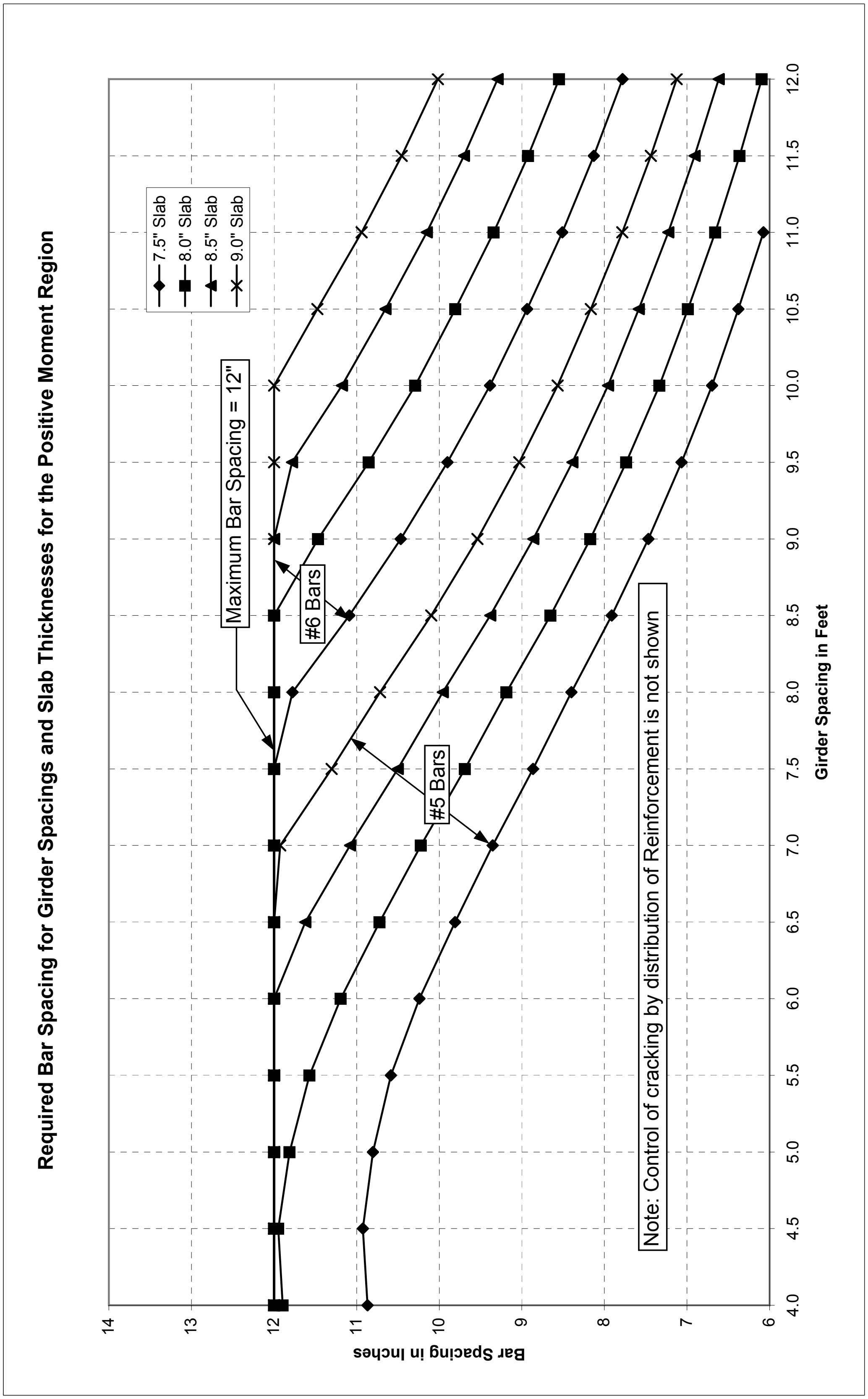


TYPICAL SECTION

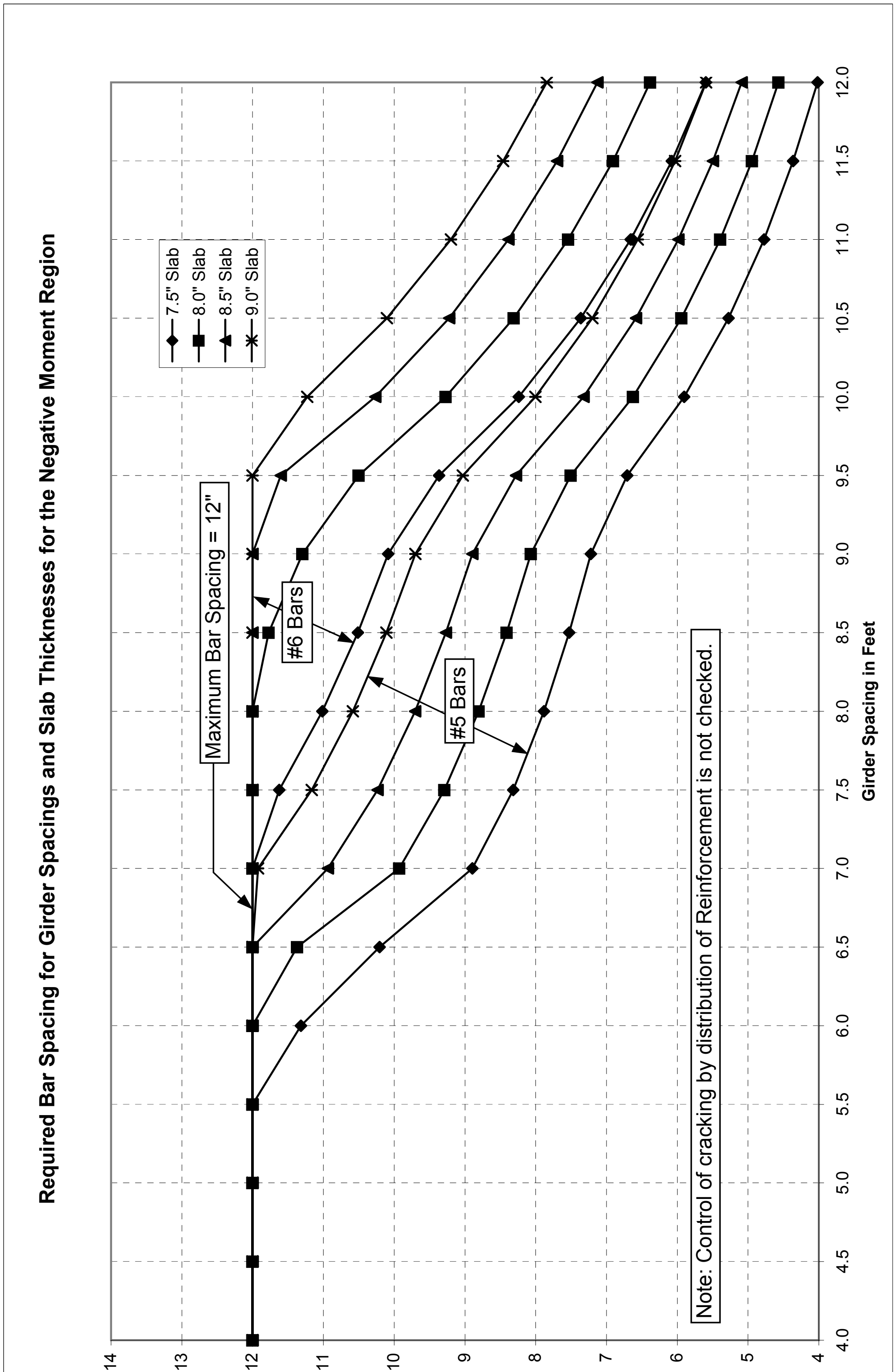


CALCULATE As REQUIRED FOR THIS MOMENT USING a & d VALUES AT FACE. CHECK THAT As ≤ 75% OF BALANCED REINF. FOR TAPERED BEAMS A MORE CRITICAL SECTION MAY EXIST AT OTHER POINTS ALONG THE BEAM.

Appendix 5.3-A5 Cast-In-Place Deck Slab Design for Positive Moment Regions f'_c = 4.0 ksi



Appendix 5.3-A6 Cast-In-Place Deck Slab Design for Negative Moment Regions $f'_c = 4.0$ ksi



Appendix 5.6-A1-1 Span Capability of W Girders

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
W42G	5	90	10.75	7.5	40
	6	85	10.25	7.5	38
	7	80	10.75	7.5	36
	8	75	10.50	7.5	33
	9	70	10.50	7.5	31
	10	65	10.00	7.5	29
	11	60	10.00	8.0	25
	12	50	10.25	8.5	23
W50G	5	115	11.00	7.5	71
	6	110	11.25	7.5	68
	7	105	11.25	7.5	65
	8	100	11.25	7.5	62
	9	95	11.25	7.5	59
	10	90	10.75	7.5	56
	11	85	11.25	8.0	53
	12	80	11.25	8.5	50
W58G	5	130	11.25	7.5	92
	6	125	11.25	7.5	89
	7	120	11.25	7.5	85
	8	115	11.50	7.5	82
	9	110	11.50	7.5	78
	10	105	11.50	7.5	75
	11	100	11.75	8.0	71
	12	95	11.75	8.5	68
W74G	5	155	11.25	7.5	131
	6	145	11.50	7.5	127
	7	140	11.50	7.5	122
	8	135	11.75	7.5	118
	9	130	11.50	7.5	114
	10	125	11.50	7.5	110
	11	120	11.75	8.0	105
	12	115	12.25	8.5	101

Design Parameters:

- PG Super Version 6.1
- Girder: f’ci = 7.5 ksi, f’c = 10 ksi
- Slab: f’c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Average Camber > Final Camber
- 42” Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½” Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder

Appendix 5.6-A1-2 Span Capability of WF Girders

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
WF36G	5	110	11.50	7.5	87
	6	100	12.00	7.5	79
	7	95	12.00	7.5	75
	8	90	11.75	7.5	72
	9	85	11.75	7.5	68
	10	85	11.50	7.5	68
	11	80	11.50	7.5	64
	12	75	11.75	8.0	60
WF42G	5	125	11.75	7.5	107
	6	120	12.25	7.5	103
	7	115	12.25	7.5	99
	8	110	11.75	7.5	94
	9	105	12.25	7.5	90
	10	95	11.50	7.5	82
	11	95	11.50	7.5	82
	12	90	12.00	8.0	78
WF50G	5	135	12.00	7.5	123
	6	135	12.00	7.5	123
	7	130	12.00	7.5	119
	8	125	12.25	7.5	114
	9	120	12.25	7.5	110
	10	115	12.00	7.5	105
	11	110	12.25	7.5	101
	12	100	11.75	8.0	92
WF58G	5	150	11.50	7.5	145
	6	145	10.75	7.5	140
	7	140	11.50	7.5	135
	8	135	12.25	7.5	130
	9	130	12.00	7.5	126
	10	125	11.75	7.5	121
	11	115	12.25	7.5	112
	12	110	12.25	8.0	107
WF66G	5	165	11.25	7.5	168
	6	160	11.50	7.5	163
	7	155	11.50	7.5	158
	8	150	12.00	7.5	153
	9	145	11.50	7.5	148
	10	135	11.75	7.5	138
	11	130	11.75	7.5	133
	12	120	12.50	8.0	123

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
WF74G	5	175	11.00	7.5	188
	6	170	11.25	7.5	183
	7	165	11.50	7.5	178
	8	160	11.75	7.5	172
	9	155	11.75	7.5	167
	10	145	11.75	7.5	157
	11	135	11.25	7.5	146
	12	130	11.75	8.0	141
WF83G	5	185	11.00	7.5	210
	6	180	10.75	7.5	204
	7	175	11.25	7.5	199
	8	170	11.50	7.5	193
	9	165	11.50	7.5	189
	10	160	11.50	7.5	182
	11	155	11.75	7.5	177
	12	150	12.25	8.0	171
WF95G	5	195	10.50	7.5	238
	6	190	10.75	7.5	232
	7	185	10.75	7.5	226
	8	185	11.00	7.5	226
	9	180	11.25	7.5	220
	10	175	11.50	7.5	214
	11	170	11.50	7.5	208
	12	165	12.00	8.0	202
WF100G	5	205	10.50	7.5	257
	6	200	10.50	7.5	252
	7	195	10.50	7.5	245
	8	190	11.00	7.5	239
	9	185	11.00	7.5	233
	10	180	11.25	7.5	227
	11	175	11.25	7.5	221
	12	170	11.75	8.0	214

Design Parameters:

- PG Super Version 6.1
- Girder: f’ci = 7.5 ksi, f’c = 10 ksi
- Slab: f’c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Average Camber > Final Camber
- 42” Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½” Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder

Appendix 5.6-A1-3 Span Capability of Deck Bulb Tee Girders

Girder Type	Girder Width (ft)	CL Bearing to CL Bearing (ft)	Excess Camber at Diaphragm (in)	Shipping Weight (kips)	Governed by Live Load Deflection
W35DG	4	85	2.249	69	Yes
	5	95	5.007	84	
	6	90	4.756	87	
W41DG	4	110	3.411	93	Yes
	5	110	5.041	101	No
	6	105	5.077	106	Yes
W53DG	4	135	3.879	125	Yes
	5	135	4.067	135	No
	6	130	4.864	141	No
W65DG	4	145	3.17	146	No
	5	150	3.889	163	
	6	145	4.067	169	

Design Parameters:

- PG Super Version 6.1
- Girder: f'ci = 7.5 ksi, f'c = 10 ksi
- Slab: f'c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Average Camber > Final Camber
- 42" Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½" Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder

Appendix 5.6-A1-4 Span Capability of WF Thin Deck Girders

Girder Type	Top Flange Width (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Shipping Weight (kips)
WF36TDG	5	110	10.75	94
	6	100	11.00	89
	7	90	11.25	85
	8	85	11.25	84
WF42TDG	5	120	11.25	107
	6	115	11.25	107
	7	105	11.75	103
	8	100	11.50	102
WF50TDG	5	140	10.75	132
	6	130	11.75	129
	7	120	12.00	124
	8	110	11.75	119
WF58TDG	5	155	10.50	155
	6	150	11.00	157
	7	135	11.75	147
	8	125	12.00	141
WF66TDG	5	170	10.00	180
	6	165	10.75	181
	7	150	11.75	171
	8	140	12.00	166
WF74TDG	5	180	9.75	200
	6	175	10.75	202
	7	165	11.50	198
	8	155	12.00	192
WF83TDG	5	195	9.50	229
	6	190	10.25	231
	7	180	11.25	226
	8	170	12.00	221

•• **WF95TDG, & WF100TDG** are available but span lengths are shorter then WF83TDG due to Hauling

Design Parameters:

- PG Super Version 6.1
- Girder: f'ci = 7.5 ksi, f'c = 10 ksi
- Slab: f'c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Slab 7" CIP
- Average Camber > Final Camber
- 42" Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½" Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder
- ½ D40 ≥ C

Appendix 5.6-A1-5 Span Capability of WF Deck Girders

Girder Type	Top Flange Width (ft)	CL Bearing to CL Bearing (ft)	Shipping Weight (kips)
WF39DG	5	115	127
	6	110	133
	7	100	132
	8	95	136
WF45DG	5	130	149
	6	125	156
	7	115	156
	8	110	161
WF53DG	5	145	174
	6	140	182
	7	130	183
	8	125	190
WF61DG	5	160	200
	6	155	210
	7	145	212
	8	140	220
WF69DG	5	175	229
	6	170	240
	7	160	243
	8	155	252
WF77DG	5	185	252
	6	180	#264
	7	175	#275
	8	165	#278

•• **WF86DG, WF98DG, & WF103DG** are available but max length exceeds shipping limits

Shipping Weight over 262 Kips

Design Parameters:

- PG Super Version 6.1
- Girder: f’ci = 7.5 ksi, f’c = 10 ksi
- Slab: f’c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- 9” UHPC Joint
- Average Camber > Final Camber
- 42” Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½” Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder
- ½ D40 ≥ C

Appendix 5.6-A1-6 Span Capability of Trapezoidal Tub Girders without Top Flange

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
U54G4	8	130	10.75	7.5	159
	10	125	11.00	7.5	152
	12	115	10.00	7.5	140
	14	105	9.75	7.5	128
U54G5	9	130	10.00	7.5	169
	11	125	10.75	7.5	163
	13	115	10.25	7.5	150
	15	110	10.50	7.5	143
U66G4*	8	145	10.00	7.5	204
	10	140	10.25	7.5	198
	12	135	10.25	7.5	191
U66G5*	9	150	10.25	7.5	224
	11	140	10.25	7.5	209
	13	135	10.25	7.5	202
	15	130	10.25	7.5	195
U78G4*	8	155	10.00	7.5	249
	10	150	10.00	7.5	241
	12	145	9.75	7.5	233
U78G5*	9	165	10.25	7.5	279#
	11	160	10.25	7.5	270#
	13	150	10.50	7.5	254
	15	145	11.50	7.5	245

Span Capability Exceeds Maximum Ground Shipping Weight of 262 Kips

* Girder exceeds Range of Applicability for Simplified Analysis. Refer to AASHTO Table 4.6.2.2.2b-1 Live Load Distribution Factor for Moment in Interior Beams

Design Parameters:

- PG Super Version 6.1
- Girder: f'ci = 7.5 ksi, f'c = 10 ksi
- Slab: f'c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Average Camber > Final Camber
- 42" Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½" Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder

Appendix 5.6-A1-7 Span Capability of Trapezoidal Tub Girders with Top Flange

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
UF60G4	9*	145	9.25	8.50	204
	11	135	9.50	8.50	190
	13	130	10.00	8.50	184
	15	125	10.25	8.50	177
UF60G5	10*	145	9.25	8.50	216
	12	135	9.75	8.50	202
	14	130	10.00	8.50	194
	16	125	10.25	8.50	187
UF72G4*	9*	155	9.25	8.50	249
	11*	145	9.75	8.50	233
	13*	140	10.00	8.50	225
	15*	135	10.25	8.50	217
UF72G5*	10*	160#	9.50#	8.50	270#
	12*	155#	10.00#	8.50	262#
	14*	150#	10.00#	8.50	253#
	16*	145	10.00	8.50	245

•• ***UF84G4 & UF84G5*** are available but max spans exceed maximum shipping weight

\# Span Capability Exceeds Maximum Ground Shipping Weight of 252 Kips

* Girder exceeds Range of Applicability for Simplified Analysis. Refer to AASHTO Table 4.6.2.2.2b-1 Live Load Distribution Factor for Moment in Interior Beams

Design Parameters:

- PGSuper Version 3.1.3.1
- Girder f'ci = 7.5 ksi, f'c = 10 ksi
- Slab f'c = 4 ksi
- No verticle or horizontal curve
- 2% roadway crown slope
- 8.5" Deck with the option of using a 3.5" SIP panel with a 5" CIP slab
- 42" Single Slope Barrier
- 6% roadway superelevation for shipping check
- Standard WSDOT Abutment End Type A
- 1 ½" concrete overlay or 35 psf HMA overlay
- Typical interior girder

Appendix 5.6-A1-8 Span Capability of Post-tensioned Spliced I-Girders

f'ci = 6.0 ksi, f'c = 9 ksi Strand diameter = 0.6" Grade 270 ksi low relaxation												
Girder Type	Girder Spacing (ft)	Span Length (ft)	Cast-in-place Closures	PT Ducts - Strands/Duct (Duct#4 @ Bottom)				Jacking Force** (kips)	Tendon Force after Seating** (kips)	Tendson Loss* (kips)	E1 (in)	E3 (in)
			Length (ft)	1	2	3	4					
WF74PTG Post-tensioned Before Slab Casting	6	170	2	-	22	22	22	2970	2680	730	36.4	12.7
	8	155	2	-	22	22	22	2970	2670	740	36.4	12.7
	10	140	2	-	22	22	22	2970	2650	760	36.4	12.7
	12	120	2	-	22	22	22	2970	2630	780	36.4	12.7
	14	100	2	-	22	22	22	2970	2590	815	36.4	12.7
WF74PTG Post-tensioned After Slab Casting	6	195	2	-	22	22	22	2960	2690	680	36.4	12.7
	8	185	2	-	22	22	22	2960	2710	680	36.4	12.7
	10	175	2	-	22	22	22	2960	2690	690	36.4	12.7
	12	165	2	-	22	22	22	2990	2720	700	36.4	12.7
	14	155	2	-	22	22	22	3020	2750	710	36.4	12.7
WF83PTG Post-tensioned Before Slab Casting	6	185	2	11	22	22	22	3500	3160	850	33.8	14.5
	8	165	2	-	22	22	22	2985	2710	720	36.4	12.7
	10	155	2	-	22	22	22	2985	2700	730	36.4	12.7
	12	135	2	-	22	22	22	2985	2680	740	36.4	12.7
	14	115	2	-	22	22	22	2985	2620	810	33.8	12.7
WF83PTG Post-tensioned After Slab Casting	6	*205	2	11	22	22	22	3500	3200	810	33.8	14.5
	8	200	2	11	22	22	22	3500	3210	800	37.6	14.5
	10	195	2	22	22	22	22	4000	3640	940	37.6	15.7
	12	185	2	22	22	22	22	4000	3640	940	37.6	15.7
	14	175	2	22	22	22	22	4000	3640	940	37.6	15.7
WF95PTG Post-tensioned Before Slab Casting	6	200	2	11	22	22	22	3500	3150	860	46.1	14.5
	8	185	2	11	22	22	22	3500	3110	980	46.1	14.5
	10	175	2	11	22	22	22	3500	3130	880	46.1	14.5
	12	155	2	8	22	22	22	3360	2990	860	44.9	14.1
	14	135	2	-	22	22	22	3000	2630	810	59.0	12.7
WF95PTG Post-tensioned After Slab Casting	6	235	2	11	22	22	22	3500	3210	800	46.1	14.5
	8	230	2	22	22	22	22	4000	3650	930	37.6	15.7
	10	215	2	22	22	22	22	4000	3640	940	37.6	15.7
	12	205	2	22	22	22	22	4000	3640	950	37.6	15.7
	14	190	2	22	22	22	22	4000	3630	960	37.6	15.7

* 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 percent roadway crown slope
- Interior girder with barrier load (6 girder bridge)
- Only flexural service and strength checked; lifting and hauling checks not necessarily satisfed
- Simple girder span lengths are CL bearing to CL bearing
- Slab f'_c = 4.0 ksi
- Standard WSDOT “F” shape barrier
- Under normal exposure condition and 75 percent 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 isassumed that the designer can modify the wet joints to reach the required span as shown in the table. These modifcations are outside the scope of this table.

Appendix 5.6-A1-9 Span Capability of Post-tensioned Spliced Tub Girders

Girder Type	Girder Spacing (ft)	Span Length (ft)	End Segments	Middle Segment	Spliced Post-tensioned Girder								
			No. of Straight Strands	No. of Straight Strands	PT Ducts ~ Strands/Duct (Duct #4 @ Bottom)				Jacking Force* (kips)	Tendon Force after Seating* (kips)	Tendon Loss* (kips)	E1 (in)	E3 (in)
					1	2	3	4					
U54PTG4	8	135	4	6	-	-	11	22	2904	2636	570	20.0	8.9
	14	150	4	14	-	2	22	22	4048	3708	760	31.5	10.1
U54PTG5	9	135	4	8	-	-	12	22	2992	2764	578	19.7	9.0
	15	150	4	14	-	6	22	22	4400	4032	826	29.8	10.9
U54PTG6	10	135	4	6	-	-	18	22	3520	3200	684	18.3	9.5
	16	145	4	14	-	8	22	22	4576	4196	852	29.0	11.2
U66PTG4	8	155	4	8	-	-	15	22	3256	2944	648	18.9	9.3
	14	170	4	16	-	7	22	22	4488	4088	864	29.4	11.0
U66PTG5	9	155	4	8	-	-	17	22	3432	3110	678	18.5	9.4
	15	170	4	16	-	10	22	22	4752	4334	910	29.3	11.3
U66PTG6	10	155	4	8	-	-	21	22	3784	3434	742	17.7	9.7
	16	165	4	14	-	12	22	22	4928	4500	940	27.7	11.7
U78PTG4	8	175	4	10	-	-	19	22	3608	3262	722	18.1	9.6
	16	190	4	20	-	15	22	22	5192	4718	1014	26.8	12.1
U78PTG5	9	180	4	10	-	2	22	22	4048	3692	776	31.5	10.1
	17	195	4	22	-	21	22	22	5720	5202	1110	25.2	12.6
U78PTG6	10	180	4	10	-	6	22	22	4400	4018	840	29.8	10.9
	18	190#	4	20	2	22	22	22	5896	5400	1104	395.0	12.9

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 percent 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 percent 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 percent of total length; center segment is 50 percent 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.

Appendix 5.6-A1-10 Span Capability of WF Girders with Lighweight Concrete

Table 1. No girder sag present in the final condition

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
WF74G	5	180	12.00	7.5	162
	6	175	12.00	7.5	158
	7	170	12.25	7.5	153
	8	165	12.00	7.5	149
	9	160	12.25	7.5	144
	10	155	12.25	7.5	140
WF83G	5	190	11.75	7.5	181
	6	185	12.25	7.5	176
	7	180	12.25	7.5	171
	8	175	12.00	7.5	167
	9	170	12.50	7.5	162
	10	165	12.25	7.5	157
WF95G	5	205	12.00	7.5	216
	6	200	12.00	7.5	204
	7	195	12.00	7.5	199
	8	190	12.25	7.5	194
	9	185	12.00	7.5	189
	10	180	12.25	7.5	184
WF100G	5[1]	215	11.75	7.5	230
	6[1]	210	10.50	7.5	224
	7	205	11.75	7.5	216
	8	200	11.75	7.5	211
	9	195	12.00	7.5	206
	10	190	12.00	7.5	200

1. 4’-7” wide top flange to satisfy lateral slenderness requirements

Design Parameters:

- PG Super Version 6.1
- Girder: f’ci = 7.5 ksi, f’c = 10 ksi
- Slab: f’c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Average Camber > Final Camber
- Lightweight Concrete Girders
- Normal Concrete Deck
- 42” Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½” Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder
- ½ D40 ≥ C, i.e no sag
- Concrete Unit Weight: 0.128 kip/ft^3
- Concrete w/ Reinf. Unit Weight: 0.138 kip/ft^3

Table 2. Potential girder sag allowed in the final condition

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim (in)	Deck Thickness (in)	Shipping Weight (kips)
WF74G	5	190	10.75	7.5	171
	6	180	11.25	7.5	162
	7	175	11.75	7.5	158
	8	170	11.75	7.5	153
	9	165	12.00	7.5	149
	10	155	12.25	7.5	140
WF83G	5	200	10.75	7.5	190
	6	195	11.00	7.5	185
	7	190	11.25	7.5	181
	8	180	11.75	7.5	171
	9	175	12.00	7.5	167
	10	170	12.25	7.5	162
WF95G	5[1]	220	10.00	7.5	224
	6[1]	210	10.25	7.5	214
	7	205	10.75	7.5	209
	8	195	11.25	7.5	199
	9	190	11.75	7.5	194
	10	180	12.00	7.5	189
WF100G	6[2]	220	9.25	7.5	231
	7[1]	215	9.50	7.5	226
	8	205	9.75	7.5	216
	9	200	10.00	7.5	211
	10	195	10.25	7.5	206

1. 4’-7” wide top flange to satisfy lateral slenderness requirements
2. 5’-1” wide top flange to satisfy lateral slenderness requirements

Design Parameters:

- PG Super Version 6.1
- Girder: f’ci = 7.5 ksi, f’c = 10 ksi
- Slab: f’c = 4 ksi
- No Vertical or Horizontal Curve
- 2% Roadway Crown Slope
- Lightweight Concrete Girders
- Normal Concrete Deck
- Girders at this span length may end up with a sag in the final condition. Precamber may be required.
- 42” Single Slope Barrier
- 6% Roadway Superelevation for Shipping Check
- Standard WSDOT Abutment End Type A
- 1 ½” Concrete Overlay or 35 psf HMA Overlay
- Typical Interior Girder
- Concrete Unit Weight: 0.128 kip/ft^3
- Concrete w/ Reinf. Unit Weight: 0.138 kip/ft^3

5.99 References

1. See Section 1.99.
2. Seguirant, S.J., "New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders," PCI JOURNAL, V. 43, No. 4, July-August 1998, pp. 92-119.
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.
5. Hsu, T. T. C., Torsion of Reinforced Concrete, Van Nostrand Reinhold Co., New York, 1st Ed., 1984, 516 pp.
6. Collins, M. P. and Mitchell, D., Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams, PCI Journal, September-October, 1980, pp. 32-100.
7. Mirza, S.A., and Furlong, R.W., Design of Reinforced and Prestressed Concrete Inverted T Beams for Bridge Structures, PCI Journal, Vol. 30, No. 4, July-August 1985, pp. 112-136.
8. Rabbat, B.G., Reader Comments Design of Reinforced and Prestressed Concrete inverted T Beams for Bridge Structures, PCI Journal, Vol. 31, No. 3, May-June 1986, pp. 157-163.
9. ACI Committee 345, Guide for Widening Highway Bridges, ACI Structural Journal, July/August, 1992, pp. 451-466.
10. PCI Design Handbook, Precast and Prestressed Concrete, Sixth Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 2004.
11. Mast, R.F., "Lateral Stability of Long Prestressed Concrete Beams, Part 1," PCI JOURNAL, V. 34, No. 1, January-February 1989, pp. 34-53.
12. Mast, R.F., "Lateral Stability of Long Prestressed Concrete Beams, Part 2," PCI JOURNAL, V. 38, No. 1, January-February 1993, pp. 70-88.
13. Imper, R.R., and Laszlo, G., "Handling and Shipping of Long Span Bridge Beams," PCI JOURNAL, V. 32, No. 6, November-December 1987, pp. 86-101.
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*, FHWA-HIF-15-016, 2016.
18. Prestressed Concrete Structures T. Y. Lin, Wiley.
19. Prestressed Concrete Vol. I and II, Guyon, Wiley
20. Design of Concrete Bridges for Temperature Gradients, ACI Journal, May 1978.

21. Mast, R. F., “Unified Design Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members”, ACI Structural Journal, V. 89, No. 2, March-April 1992, pp. 185-199. See also discussions by R.K. Devalapura and M.K. Tadros, C.W. Dolan and J.V. Loscheider and closure to discussions in V. 89, No. 5, September-October 1992, pp. 591-593.
22. Weigel, J.A., Seguirant, S.J., Brice, R., and Khaleghi, B., “High Performance Precast, Prestressed Concrete Girder Bridges in Washington State,” PCI JOURNAL, V. 48, No. 2, March-April 2003, pp. 28-52.
23. Seguirant, S. J., Brice, R., and Khaleghi, B., “Flexural Strength of Reinforced and Prestressed Concrete T-Beams,” PCI JOURNAL, V. 50, No. 1, January-February 2005, pp 44-73.
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.
25. NCHRP Report 628, “Self-Consolidating Concrete for Precast, Prestressed Concrete Bridge Elements”, NCHRP Project 18-12, Transportation Research Board, 2009.
26. PCI Publication CB-02-16-E, “Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders”, First Edition, Precast/Prestressed Concrete Institute, February 2016.
27. Chapman, D., “WSDOT Girder Stirrup Hat Bar Test,” Concrete Technology Corporation, 2012.

This page intentionally left blank.