Oregon Department of Transportation

SECTION 1.5
Concrete

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

1.5 Concrete

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1.5.1 Concrete, Design Considerations

Designate the concrete class by the minimum compressive strength at 28 days followed by the maximum aggregate size (e.g., Class 4000 – ¾). Unless otherwise specified, Class 3300 1½, 1 or ¾ is called for by the *Standard Specifications*.

Use high performance IC concrete (HPC-IC) in all cast-in-place concrete decks and deck approach system elements. Use high performance concrete (HPC) in pedestrian bridge decks only when they are located in a snow and ice zone (per *BDM 1.2.4*).

C 1.5.1

The maximum ultimate strength on which allowable stresses are based is 5000 psi, except for prestressed concrete.

Use of IC increases the service life and reduces total life cycle costs of a bridge deck.

Use of IC with lightweight fine aggregate (LWFA) (according to *ASTM C1761)* for concrete can mitigate cracking due to shrinkage in bridge decks.

Internal curing is a practical way of supplying additional curing water throughout the concrete mixture. This water can improve the hydration of cement, reduce autogenous shrinkage, and improve durability.

Curing time before subsequent loading may be shortened.

BDM 1.3.2 for concrete dead loads.

By replacing normal weight fine aggregate with lightweight fine aggregate for IC, the unit weight of the concrete is lighter.

Table 1.5.1-1 Classes of Concrete (For Design and To Be Shown on Plans)

Class of Concrete	Concrete Location
HPC-IC 4500 – 1-1/2	All poured decks (except box girder decks that require greater strength and Pedestrian Bridge decks not in snow and ice zones) Note: This concrete strength works well with both grade 80 and grade 60 rebar and therefore would facilitate use of grade 80 rebar, but still allow contractors to consider grade 60 rebar without the need to change to a different concrete mix. The use of larger maximum aggregate size is to achieve more durable decks.
HPC-IC 4500 – 1½	Approach Slabs
4000 – 3/8	Drilled Shafts
XXXX – ¾	Prestressed members (Does not include poured deck on prestressed members, see above)
XXXX – ½ or 3/8	Post-tensioned box girder bottom slab and stem walls
4000 – ¾	Columns, footings, footing caps, and crossbeams
3300 – 1½, 1 or ¾	All other concrete

1.5.1.1 Lightweight Concrete

When using lightweight concrete, adjust reinforcement development length for lightweight concrete per *LRFD 5.10.8.2.*

C 1.5.1.1

Concrete containing lightweight aggregate has a unit weight of 0.110 - 0.135 kcf. Contact the Technical Specialist and the Structural Materials Engineer when using concrete with a unit weight less than 0.135 kcf (per *Table 1.3.2-1*).

1.5.1.2 Corrosion Protection System

Provide a corrosion protection system for reinforced concrete girders, pile caps, diaphragms, and crossbeams as shown in *Table 1.5.1.2-1*. See environment definition in *BDM 1.2.4*.

Table 1.5.1.2-1 Corrosion Protection System for Precast and Cast-In-Place Concrete Members

Environment	Marine Areas	Coastal Areas
Precast and cast-in-place concrete	HPC	HPC
Non-prestress reinforcement	Stainless steel	ChromX4100
Prestressing strand	Uncoated, black	Uncoated, black
Minimum concrete clear cover for cast-in-place members	2 inches	2 inches

1.5.2 Concrete Finish

C 1.5.2

1.5.2.1 Surface Finish

C1.5.2.1

Concrete finishes are defined in *SP 00540.53* of the *Oregon Standard Specifications for Construction*.

The usual finishes are General Surface Finish and Class 1 Surface Finish. Occasionally, Class 2 Surface Finish is used for a more durable coating and aesthetic enhancement. Class 2 Surface finish will lead to a higher concrete unit cost.

Review selected surface finish with the design team.

For bridges whose superstructure and substructure can be viewed by the public, specify a Class 1 Surface Finish for exposed surfaces.

This includes but is not limited to grade separations and river crossings in or near populated areas. In special situations of high visibility to traffic or people, use of a Class 2 Surface Finish may be considered. Normally, it is limited to the concrete rail sides facing the roadway/bikeway and the tops.

For bridges not viewed by large segments of the public, specify a General Surface Finish for exposed surfaces, except portions of the concrete bridge rail. Specify a Class 1 Surface Finish for the concrete rail sides facing the roadway/bikeway and tops.

This includes but is not limited to stream crossings in sparsely populated areas.

Include details similar to *Figures 1.5.2.1-1, 1.5.2.1-2,* or *1.5.2.1-3* for all contract plans.

Figure 1.5.2.1-1 Concrete Surface Finish Detail for Box Girder Bridges

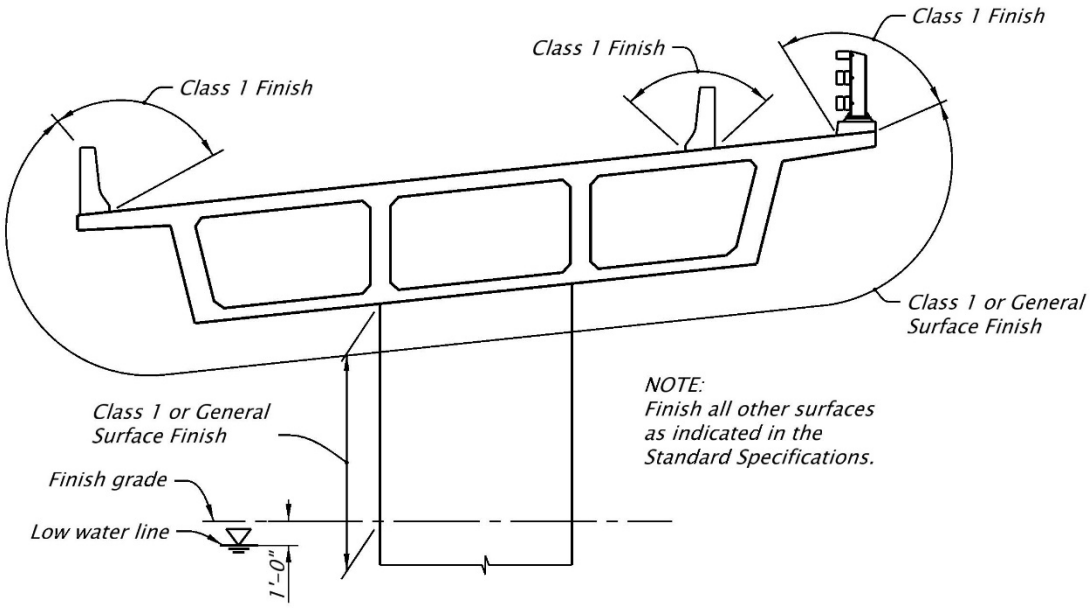


Figure 1.5.2.1-2 Concrete Surface Finish Detail for Precast/Prestressed Concrete Girder Bridges

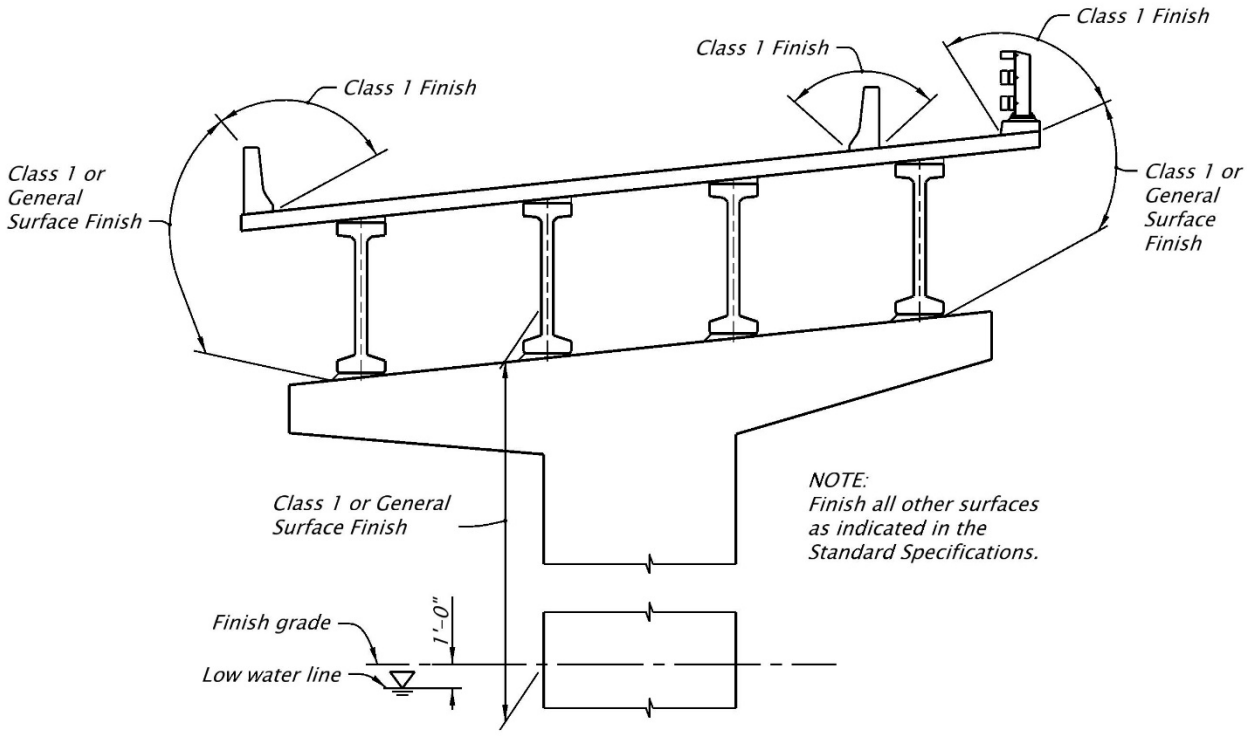
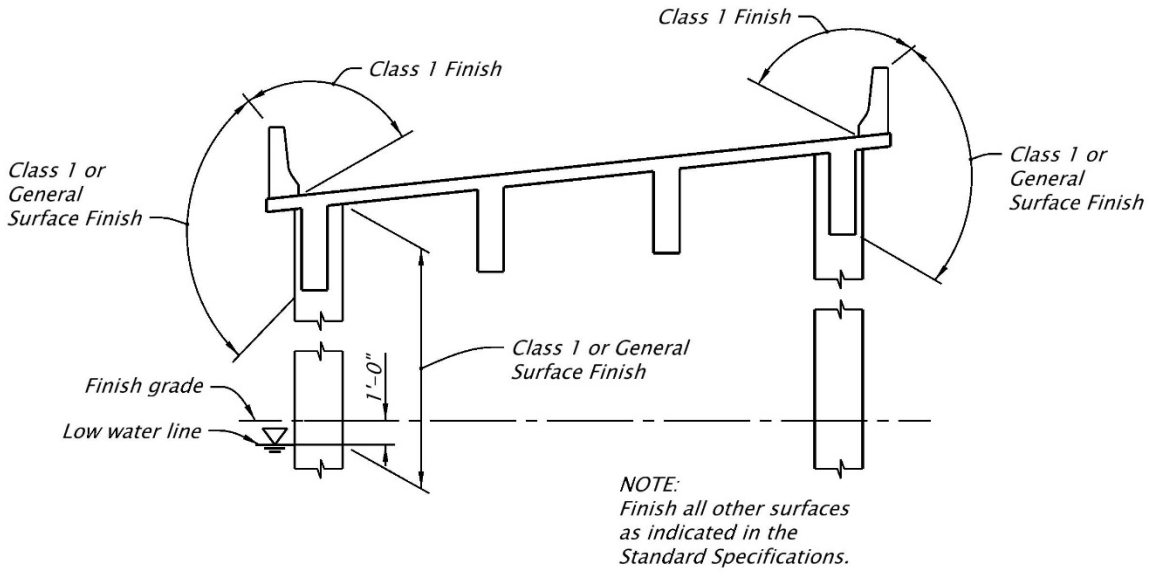


Figure 1.5.2.1-3 Concrete Finish Detail for Reinforced Concrete Deck Girder Bridges



Specify uncoated Class 2 Surface Finish for projects that require the finished surface to have a similar appearance compared to the surrounding concrete.

Projects on existing historic bridges are good candidates for this surface finish.

Most historic bridges have original surface coating that consist of a mix of a paste of fine mortar sand, cement, and water, also known as Moose Milk Concrete Finish. The finish provides smooth surface with a concrete color.

When uncoated Class 2 Surface Finish is specified, utilize the examples in *Figures 1.5.2.1-1* through *1.5.2.1-3* and replace the term "Class 1" with "Uncoated Class 2 Finish" in all locations. Show limits and provide similar notes on other structure types as needed.

Uncoated Class 2 Concrete Surface Finish provide a similar finish to the original coating that hides imperfections and provides similar appearance to surrounding concrete.

Pedestrian concrete bridge decks and sidewalks do not require Deck Roadway Texturing with saw cutting according to *SP 00540.50(c)*. A Deck Sidewalk Finish is specified in *SP 00540.50(d)*. Ensure that all pedestrian surfaces comply with ADA slip resistance and surface requirements.

Do not use color additives in concrete mixes.

Color additives are not used as they can change the characteristics of the mix design which requires review and the effects may not be known without additional laboratory testing. Colored aggregate is generally very expensive to source. In both cases, imperfections due to construction methods are typically visible because the colored concrete or aggregate can't be precisely matched with a topical product such as a coating or slurry/scrub coat.

1.5.2.2 Paint

Use only latex emulsions for finishing. Colors are limited to the concrete gray tones listed in the *00540.10(a)* boilerplate specification.

C 1.5.2.2

When used for Class 1 or Class 2 finishing on cast-in-place concrete, the concrete paint standard specification language allows for the use of either latex emulsion paint or

For latex emulsion paint cure, *00587*, an additional top coat is required when a different color is required.

pigmented sealers at the discretion of the contractor.

Latex emulsions provide schedule benefit to new construction projects as it can be applied to younger concrete. Pigmented sealers require the concrete to be fully cured (generally 28 days, but varies based on mix design).

Coatings are applied to Class 1 finished surfaces to hide imperfections such as patches from form ties, grinding from form bulges and blowouts, air vents (overhead/repairs), and repairs that may be necessary after setting of pre-cast members.

Coatings provide uniform color to both Class 1 and Class 2 Surface Finishes.

Coatings also provide protection against freeze-thaw damage, chemical ingress such as from chlorides or sulfates, and resistance to fungus or algae formation on the surface.

1.5.2.3 Pigmented Sealers

Use pigmented sealers when a color other than the gray tones listed in the boilerplate specification *00540.10(a)* is required on a Class 1 or Class 2 Surface Finish.

When applying architectural treatment using *00543*, include the *00540* boilerplate and use pigmented sealers on adjacent non-textured surfaces which are intended to have a matching color.

When pigmented sealers are used, utilize the examples in *Figures 1.5.2.1-1* through *1.5.2.1-3* and add the language "with pigmented sealer" after "Class 1" or "General Surface Finish" (or Class 2 when warranted). Show limits and provide similar notes on other structure types as needed.

When a single color is used, include in the general notes: "Apply pigmented sealer with color according to *SAE AMS-STD-595* #XXXXX to concrete surfaces as shown."

When multiple colors are used, include in the general notes: "Apply pigmented sealers to concrete surfaces as shown." Indicate limits of individual colors on appropriate plan sheets.

C 1.5.2.3

While ODOT specifications utilize the term pigmented sealer, these products may be submitted under several trade names (e.g. stains, sealers, coatings, or paint).

Alternative materials may be warranted in the following situations (e.g. urethanes, mastics, etc.):

- For areas with exposure to dense pedestrian traffic or high friction surfaces. While acrylics exhibit good impact and abrasion resistance for general use, they may not perform well in high traffic areas and can become slippery when wet if applied to walking, rolling, or riding surfaces.
- High visibility projects with bright colors where color retention is important. Acrylics generally exhibit good color retention and weathering characteristics. For bright and vibrant colors (safety yellow, bright reds, or light blues) certain product formulations (e.g. fluoropolymers, urethanes, siloxanes, etc.) may retain color better than acrylics.

Contact the State Coatings Engineer for additional information on alternative materials.

1.5.3 Concrete Bonding Agents

Bonding agents are used to help new concrete adhere to existing concrete. To obtain better bond with agents the existing surface must be clean, dry and at proper temperature. The surfaces must also be well exposed to facilitate brush application of the bonding agent. Two principal bonding agents are in use today:

- Epoxy - These agents provide the best bond when properly applied. However, they are highly volatile and when the agent is allowed to dry before placement of the new concrete, a bond breaker may be formed. For this reason restrict the use of epoxy agents to critical situations where control can be guaranteed.
- Concrete - These agents have longer pot life and improved bond. They may be applied with greater lead time, but have the same application requirements as epoxy agents.

At normal construction joints, a bonding agent is not generally needed. Mating surfaces prepared to the specifications are considered sufficient to provide acceptable bond and shear transfer through the roughened surface and rebar holding a tight joint.

1.5.4 Curing Concrete

SP 00540.51 in the *Standard Specifications* require cast-in-place concrete to be cured with water. Design all structures assuming concrete is cured using the ODOT standard. Acting as EOR, assure that alternate curing methods are not allowed without prior approval of the Structural Materials Engineer.

Bridge decks must also be cured with water. Although ODOT does use curing compounds for some pavement and sidewalk applications, curing compounds are not be allowed on bridge decks. ODOT experimented with curing compounds in the early 1990s. The results were not consistent from batch to batch. Also, more recent experiments with curing compounds revealed that cylinders cured with a curing compound achieved only 80 percent compressive strength compared to water cured cylinders.

The ODOT water cure requirement also applies to bridge columns, abutments and retaining walls. Since it is difficult to keep vertical surfaces saturated during the cure period, vertical forms must often be left in place for the entire cure period. Contractors will often request to use a curing compound so that forms can be stripped sooner and production increased. However, due to the negative impacts of curing compounds, their use is rarely permitted.

For applications that receive a coating, use of curing compounds can inhibit adherence of the coating. Generally, curing compounds must be removed by sandblasting before subsequent coatings can be applied. Removal of a curing compound would be even more problematic on textured surfaces.

In summary, do not use curing compounds. Exceptions require approval from the Structural Materials Engineer, but do not require a design deviation.

1.5.5 Reinforcement

Ensure there is enough room for bars to fit and to place concrete. Ensure steel can be placed and supported. Show bolster bars on reinforcement details when needed.

Table 1.5.5-1 Standard Bar Chart

Bar #	Nominal Dia. (inch)	Area (in^2)	Weight (lb/ft)
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.650
18	2.257	4.00	13.60

Provide a minimum 2 inch clear cover for situations that are not listed in *Table 1.5.5-2.* When using stainless steel reinforcing, non-ferrous reinforcing or ChromX9100 use *AASHTO Table 5.10.1-1 Category C* cover requirements.

Table 1.5.5-2 Minimum Clear Cover for Coated or Uncoated Reinforcement

Location:	**Cover (inch)**
Top of deck slab (main reinforcing)*	2.5
Bottom of deck slab*	1.5
All superstructure surfaces exposed to the effects of a marine area, *BDM 1.9.2.1.3*	2
Stirrups and ties in T-beams, bottom rebar of slab spans, and curbs and rails *	1.5
Stirrups in box girder stems with non-bundled ducts **	2.5
Stirrup ties in box girder stems with non-bundled ducts **	2
Bottom slab steel in box girders	1
All faces in precast members (slabs, box beams and girders)	1
All cast-in-place members	2
Pier and column spirals, hoops or tie bars+ (increase to 4 inch when exposed to marine area or concrete is deposited in water)	2.5
Footing mats for dry land foundations (use 6 inch when ground water may be a construction problem)	3
Footing mats for stream crossing foundations	6

Tables Notes:

* Except in coastal areas use 2 inch cover.

**For box girder stems with bundled ducts, provide 3 inches clearance to ducts and place stirrups directly against ducts.

+Cover over supplementary crossties may be reduced by the diameter of the tie.

1.5.5.1 Reinforcement for Shrinkage and Temperature

Provide reinforcement for shrinkage and temperature stresses near exposed surfaces and in structural mass concrete according to *LRFD 5.10.8*. Use an area of reinforcement per surface of at least 0.0008 times the gross concrete area with a minimum of #4 at 18 inch centers. Space the reinforcement no farther apart than three times the concrete thickness or a maximum of 18 inch centers.

Table 1.5.5.1-1 Temperature and Shrinkage Reinforcement Spacing

Thickness (inches)	A_s (in^2/foot)	#4 Bar Spacing for One Surface (inches)	#5 Bar Spacing for One Surface (inches)	#6 Bar Spacing for One Surface (inches)	#7 Bar Spacing for One Surface (inches)
6	0.062	#4 @ 18	--	--	--
9	0.091	#4 @ 18	--	--	--
12	0.118	#4 @ 18	--	--	--
15	0.144	#4 @ 15	--	--	--
18	0.170	#4 @ 12	--	--	--
21	0.194	#4 @ 12	#5 @ 18	--	--
24	0.217	#4 @ 10	#5 @ 15	--	--
27	0.239	#4 @ 10	#5 @ 15	--	--
30	0.260	--	#5 @ 12	#6 @ 18	--
36	0.300	--	#5 @ 12	#6 @ 15	--
48	0.371	--	#5 @ 10	#6 @ 12	#7 @ 18
60	0.433	--	#5 @ 10	#6 @ 12	#7 @ 15

Since the amount of reinforcement is somewhat empirical, convenient spacing can be assumed as shown in *Table 1.5.5.1-1*. *Table 1.5.5.1-1* is intended for preliminary purposes only. It is based on a least width dimension of 10 feet.

1.5.5.2 Spacing of Shear Reinforcement

Where shear reinforcement is required and placed perpendicular to the axis of the member, spacing is not to exceed 18 inches.

1.5.5.3 Negative Moment Reinforcement

For cantilever crossbeams with wide bents, extend at least half of the negative reinforcement the full length of the crossbeam.

1.5.5.4 Minimum Bar Spacing

Table 1.5.5.4-1 Minimum Bar Spacing

Bar #	Nominal Dia.(d_b) (inch)	2.5 x d_b or 1.5+d_b (inch)	(1.5x1.5) + d_b for 1.5 agg. (inch)	(1.5x0.75)+ d_b for 0.75 agg. (inch)
3	0.375	1-7/8	2-5/8	1-1/2
4	0.500	2	2-3/4	1-5/8
5	0.625	2-1/8	2-7/8	1-3/4
6	0.750	2-1/4	3	1-7/8
7	0.875	2-3/8	3-1/8	2
8	1.000	2-1/2	3-1/4	2-1/8
9	1.128	2-7/8	3-3/8	2-1/4
10	1.270	3-1/4	3-1/2	2-3/8
11	1.410	3-5/8	3-5/8	2-1/2
14	1.696	4-1/4	4	2-7/8
18	2.257	5-5/8	4-1/2	3-3/8

1.5.5.5 Modified Tension Development Length - GRADE 60 – Uncoated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.11.2.1*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.5-1 Modified Tension Development Length – Grade 60 – Uncoated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement.

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	15.46	14.04	13.24	12.56	12.00
4	20.61	18.72	17.65	16.74	15.28
5	25.76	23.40	22.06	20.93	19.11
6	30.92	28.08	26.47	25.12	22.93
7	36.07	32.76	30.89	29.30	26.75
8	41.22	37.44	35.30	33.49	30.57
9	46.50	42.23	39.82	37.77	34.48
10	52.35	47.55	44.83	42.53	38.82
11	58.12	52.79	49.77	47.22	43.10
14	69.79	63.39	59.76	56.69	51.75
18	93.03	84.50	79.67	75.58	69.00

Table 1.5.5.5-2 Modified Tension Development Length – Grade 60 – Uncoated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	12.00	12.00	12.00	12.00	12.00
4	15.85	14.40	13.58	12.88	12.00
5	19.82	18.00	16.97	16.10	14.70
6	23.78	21.60	20.36	19.32	17.64
7	27.74	25.20	23.76	22.54	20.58
8	31.71	28.80	27.15	25.76	23.52
9	35.77	32.49	30.63	29.06	26.53
10	40.27	36.58	34.48	32.71	29.86
11	44.71	40.61	38.29	36.32	33.16
14	53.68	48.76	45.97	43.61	39.81
18	71.56	65.00	61.28	58.14	53.07

1.5.5.6 Modified Tension Development Length - GRADE 60 – Epoxy Coated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.11.2.1*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.6-1 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement.

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	20.21	18.36	17.31	16.42	14.99
4	26.95	24.48	23.08	21.90	19.99
5	33.69	30.60	28.85	27.37	24.98
6	40.43	36.72	34.62	32.84	29.98
7	47.17	42.84	40.39	38.32	34.98
8	53.90	48.96	46.16	43.79	39.98
9	60.80	55.23	52.07	49.40	45.09
10	68.46	62.18	58.62	55.61	50.77
11	76.00	69.03	65.09	61.75	56.37
14	91.26	82.89	78.15	74.14	67.68
18	121.66	110.50	104.18	98.84	90.23

Tables Notes:

* Cover less than 3db or clear spacing between bars less than 6db.

Table 1.5.5.6-2 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	17.84	16.20	15.27	14.49	13.23
4	23.78	21.60	20.36	19.32	17.64
5	29.73	27.00	25.46	24.15	22.05
6	35.67	32.40	30.55	28.98	26.45
7	41.62	37.80	35.64	33.81	30.86
8	47.56	43.20	40.73	38.64	35.27
9	53.65	48.73	45.94	43.59	39.79
10	60.40	54.86	51.73	49.07	44.80
11	67.06	60.91	57.43	54.48	49.73
14	80.52	73.14	68.95	65.42	59.72
18	107.35	97.50	91.93	87.21	79.61

Table Notes:
* Cover less than 3db or clear spacing between bars less than 6db.

Table 1.5.5.6-3 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	18.55	16.85	15.88	15.07	13.76
4	24.73	22.46	21.18	20.09	18.34
5	30.92	28.08	26.47	25.12	22.93
6	37.10	33.70	31.77	30.14	27.51
7	43.28	39.31	37.06	35.16	32.10
8	49.46	44.93	42.36	40.18	36.68
9	55.80	50.68	47.78	45.33	41.38
10	62.82	57.06	53.80	51.03	46.59
11	69.74	63.35	59.73	56.66	51.72
14	83.74	76.06	71.71	68.03	62.11

18	111.64	101.40	95.60	90.70	82.79

Tables Notes:

* Cover not less than 3db and clear spacing between bars not less than 6db.

Table 1.5.5.6-4 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars – Other Horizontal Bars

Bar #	**L_d (in) f'c =3.3 ksi**	**L_d (in) f'c =4.0 ksi**	**L_d (in) f'c =4.5 ksi**	**L_d (in) f'c =5.0 ksi**	**L_d (in) f'c =6.0 ksi**
3	14.27	12.96	12.22	12.00	12.00
4	19.02	17.28	16.29	15.46	14.11
5	23.78	21.60	20.36	19.32	17.64
6	28.54	25.92	24.44	23.18	21.16
7	33.29	30.24	28.51	27.05	24.69
8	38.05	34.56	32.58	30.91	28.22
9	42.92	38.98	36.75	34.87	31.83
10	48.32	43.89	41.38	39.26	35.84
11	53.65	48.73	45.94	43.59	39.79
14	64.42	58.51	55.16	52.33	47.77
18	85.88	78.00	73.54	69.77	63.69

Tables Notes:

* Cover not less than 3db and clear spacing between bars not less than 6db.

1.5.5.7 Class B Tension Lap Splice (in) - GRADE 60 – Uncoated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.10.8.2.1c*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.7-1 Class B Tension Lap Splice – Grade 60 – Uncoated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement

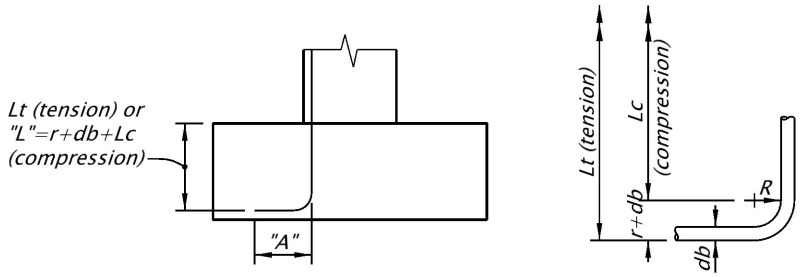
Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	20.09	18.25	17.21	16.33	15.60
4	26.79	24.34	22.94	21.77	19.87
5	33.49	30.42	28.68	27.21	24.84
6	40.19	36.50	34.42	32.65	29.81
7	46.89	42.59	40.15	38.09	34.77
8	53.59	48.67	45.89	43.53	39.74
9	60.45	54.90	51.76	49.11	44.83
10	68.05	61.81	58.28	55.29	50.47
11	75.56	68.63	64.70	61.38	56.03

Table 1.5.5.7-2 Class B Tension Lap Splice – Grade 60 – Uncoated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	15.46	15.60	15.60	15.60	15.60
4	20.61	18.72	17.65	16.74	15.60
5	25.76	23.40	22.06	20.93	19.11
6	30.92	28.08	26.47	25.12	22.93
7	36.07	32.76	30.89	29.30	26.75
8	41.22	37.44	35.30	33.49	30.57
9	46.50	42.23	39.82	37.77	34.48
10	52.35	47.55	44.83	42.53	38.82
11	58.12	52.79	49.77	47.22	43.10

1.5.5.8 Min. Column Bar Lengths in Footings – GRADE 60 & F'C =3.3 KSI

Figure 1.5.5.8-1 Minimum Column Bar Lengths in Footings



NOTE:
Increase Lt or Lc 20 percent or epoxy coated bars "A" and r+db are standard 90° hook dimensions Lc is the compression development length Lt is the tension development length for standard hooks.

Table 1.5.5.8-1 Compression Development Length – Hooked Bars

Basic Compression Development (Length for hooked bars) ldh (inches)	**Bar Size**	**"A" (inches)**	**Compression "L" * (single bar) (inches)**	**Compression "L" * (two bar bundle) (inches)**	**Compression "L" * (three bar bundle) (inches)**
16	6	12	3	16	17
19	7	14	3.5	18	20
21	8	16	4	21	22
24	9	19	6	25	27
27	10	22	7	31	33
30	11	24	7.5	37	40
36	14	31	11	53	58
47	18	41	15	90	98

Table Notes:

* Note: Lc + (r + Db) and including 0.75 modification factor for reinforcement enclosed within a spiral per *LRFD 5.11.2.2.2*.

Table 1.5.5.8-2 Tension Development Length – Hooked Bars

Bar Size	"A" (inches)	Basic Tension Development (length for hooked bars) ldh (inches)	Modified Tension "L" ** 0.7 ldh (inches)
6	12	10	7
7	14	13	9
8	16	17	12
9	19	21	15
10	22	27	19
11	24	33	23
14	31	47	33
18	41	84	59

Table Notes:
** Note: #11 bars and smaller, adequate side and hook extension cover per *LRFD 5.11.2.4.2*.

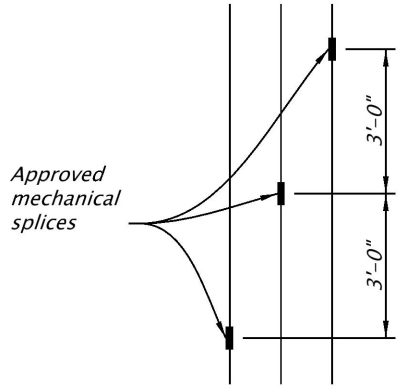
1.5.5.9 Welded Splices and Mechanical Connections

When field welding of reinforcing steel is anticipated, use *ASTM A706* reinforcing steel. Welding of *ASTM A615*, grade 60 reinforcing steel is not permitted without prior approval from the ODOT welding specialist.

Welding of *ASTM A706* for splices for column spiral reinforcing is permitted.

Use approved mechanical splices for #14 and #18 vertical column bars. Stagger splices as shown in *Figure 1.5.5.9-1*, to avoid adjacent bars being spliced in the same plane.

Figure 1.5.5.9-1 Mechanical Splice Staggering



Show lap splices on structure plans with the option of approved mechanical splices available to the contractor.

Special cases such as steel in back walls of abutments of post-tensioned concrete bridges and splicing reinforcement in existing structures may require the use of mechanical splices.

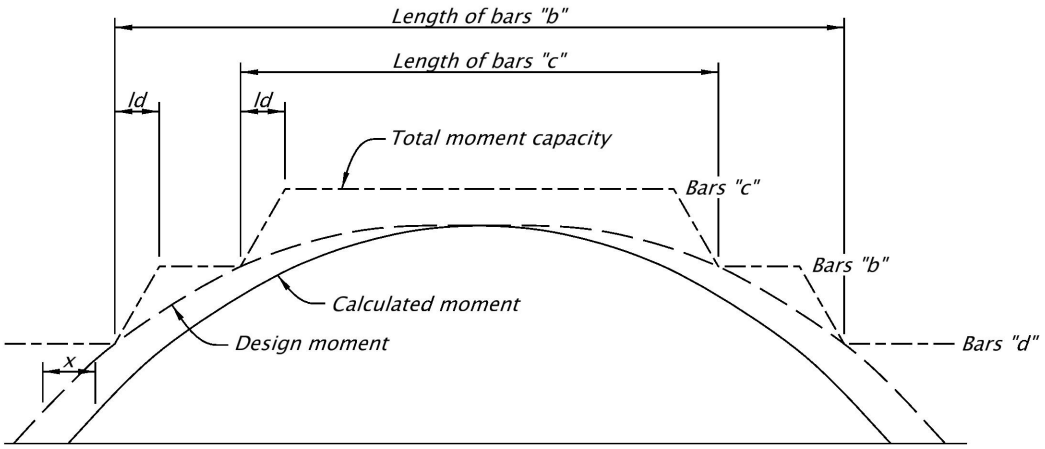
1.5.5.10 *Lap Splices – GRADE 60

(Reserved)

1.5.5.11 Development of Flexural Reinforcement

The added length, **X**, is to provide for unanticipated loading conditions or shifting of the moment diagram due to shear cracking.

Figure 1.5.5.11-1 Development of Flexural Reinforcement



x = effective girder depth,15 bar diameters or span/20
ld = bar development length

1.5.5.12 Distribution of Flexural Reinforcement

In mild areas, as defined in *BDM 1.2.4*, use $\gamma e = 1.0$. In marine, coastal and snow and ice areas, as defined in *BDM 1.2.4*, use $\gamma e = 0.75$. In decks, use $\gamma e = 1.0$.

1.5.5.13 Bundled Bars

Tie bundled bars with #9, or heavier, wire at 4 foot maximum centers. Use of bundled #14 or #18 bars requires the approval of the supervisor.

When bundled bars are used in columns, the minimum clear distance between bundles is 2.5 times the diameter of the largest bar in a bundle.

It is preferred bundled bars not be used in bridge decks. When they are used, increase the thickness of the deck by the diameter of the bar throughout the length where bundling is used.

1.5.5.14 Headed Reinforcement

Headed reinforcement can be used to reduce congestion or reduce development length over a standard hook. Headed reinforcement will always require less development length compared to a standard hook.

Headed rebar is only available for *ASTM A706* and *ASTM A615* applications. It is not available for stainless steel applications. The cost of headed rebar will generally exceed that of a standard hook. Therefore, only use them where the benefit of reduced congestion or shorter development length is significant.

Do not use headed reinforcement where their use will reduce concrete cover below the minimum required. For this reason, it may be necessary to use standard hooked bars in the corners of a rebar cage that otherwise contains headed bars.

Designate bars which require headed reinforcement on the plans. The *SP 00530* boiler plate special provision requires headed reinforcement to meet *ASTM A970*. It also requires headed reinforcement products be selected from the *ODOT QPL*. Therefore, there is no reason to say anything other than **headed bar** on the plans.

Heads may be square, rectangular, round or oval. Minimum head size for square and round heads are provided in *Table 1.5.5.14-1*. Rectangular and oval head area must exceed 10 times the bar area.

Table 1.5.5.14-1 Minimum Head Size for Headed Reinforcement

Bar Size	Min. Width for Square Heads (inch)	Min. Diameter for Round Heads (inch)
4	1-1/2	1.6
5	1-3/4	2.0
6	2-1/8	2.3
7	2-1/2	2.7
8	2-3/4	3.1
9	3-1/8	3.5
10	3-1/2	4.0
11	4	4.4
14	4-3/4	5.3

Headed reinforcement will not require project testing. Testing is required as part the QPL approval process. QC testing by the manufacturer is also required by *ASTM A970*.

When proposed by a contractor, headed reinforcement meeting the minimum head size requirement will generally be acceptable as a direct replacement for standard hooks, except where the head will not allow the required minimum concrete cover.

Use the minimum development lengths for headed reinforcement shown in *Table 1.5.5.14-2*.

Table 1.5.5.14-2 Development Length for Headed Reinforcement, Fy = 60 ksi

Bar #	f'c =3.3 ksi (inch)	f'c =4.0 ksi (inch)	f'c =5.0 ksi (inch)
4	6	5	5
5	7	6	6
6	8	8	7
7	10	9	8
8	12	10	9
9	18	14	10
10	22	18	11
11	25	20	12
14	36	28	15

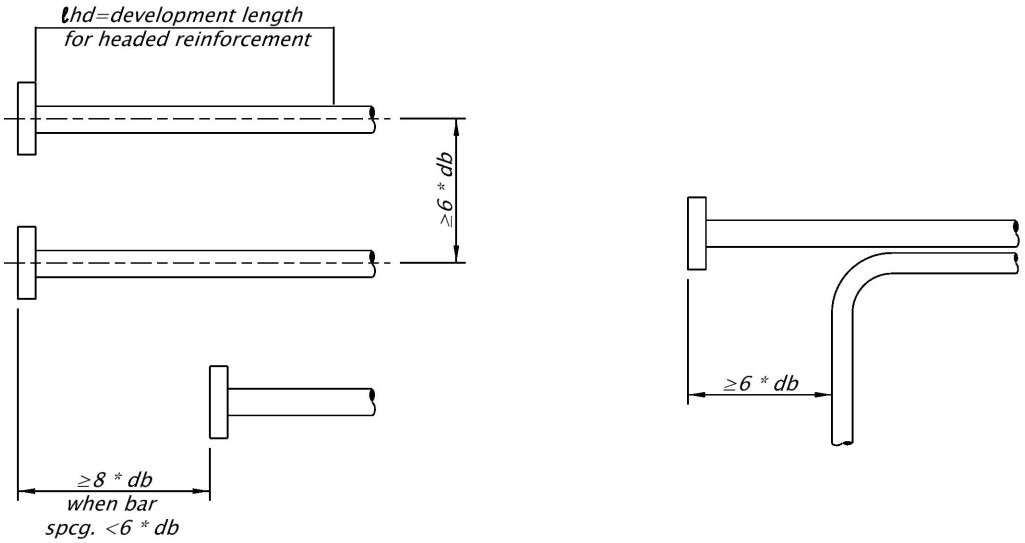
Table Note: Increase lengths for epoxy coated bars per *LRFD 5.11.2.1.2*.

Apply the modification factors and tie requirements in *LRFD 5.11.2.4.2* and *LRFD 5.11.2.4.3* to headed reinforcement also.

Place adjacent headed bars at a minimum spacing of $6 * db$. Spacing less than $6 * db$ can be used when heads from adjacent bars are spaced longitudinally (along the length of the bar) a minimum of $8 * db$ as shown in *Figure 1.5.5.14-1*.

When bundled bars are used, one bar in the bundle may be terminated using headed rebar. Terminate other bars in the bundle using standard hooks as shown in *Figure 1.5.5.14-1*.

Figure 1.5.5.14-1 Terminating Bundled Headed Reinforcement



Use of headed reinforcement can result in high concrete compressive stresses under the bar head. Consider the load path for head compression loads and provide distribution steel perpendicular to a headed bar to ensure satisfactory distribution of compressive stresses. The following articles may be useful to understand the load distribution of headed bars:

- *Strut-and-Tie Models for Headed Bar Development in C-C-T Nodes*, Chun and Hong, ACI Structural Journal, Vol. 106, No. 2, March-April 2009, pg. 123-130.
- *Investigation of Dispersion of Compression in Bottle-Shaped Struts*, Sahoo, Singh and Bhargava, ACI Structural Journal, Vol. 106, No. 2, March-April 2009, pg. 178-186.

LRFD 5.11.3 allows for mechanical devices as anchorage. Headed rebar meeting or exceeding the size required by *ASTM A970* has been extensively tested. A summary of testing can be found in *Texas Research Report 1855-1*, Anchorage Behavior of Headed Reinforcement Literature Review, May 2002.

The minimum development lengths for headed reinforcement are based on the greater of:

- 50 percent of the equivalent hooked bar development length or
- Calculations using a combination of head bearing capacity and bar development.

Development length calculations were based on concrete bearing capacity under the head plus additional straight bar development length as required to fully develop the yield strength of the bar. The concrete bearing capacity was taken from *LRFD equation 5.7.5-2* and was adjusted using a resistance factor of 0.7 for bearing on concrete per *LRFD 5.5.4.2.1*. Some of the proposed development lengths were increased to provide reasonable transitions between different bar sizes.

ACI 318 allows headed reinforcement, but requires a development length equal to 75 percent of the equivalent hooked bar development length. ODOT view this as overly conservative for bridge applications.

Table 1.5.5.14-3 illustrates the difference between ODOT and ACI development length requirements.

Table 1.5.5.14-3 Comparison of Headed Reinforcement Development Length

f'c	3.3 ksi			4.0 ksi			5.0 ksi		
	Calc (inch)	ODOT (inch)	ACI (inch)	Calc (inch)	ODOT (inch)	ACI (inch)	Calc (inch)	ODOT (inch)	ACI (inch)
#4		6	7.8		5	7.1		5	6.4
#5	4	7	9.8	2	6	8.9	0	6	8.0
#6	8	8	11.8	6	8	10.7	3	7	9.6
#7	10	10	13.7	8	9	12.5	4	8	11.2
#8	12	12	15.7	9	10	14.3	3	9	12.7
#9	18	18	17.7	14	14	16.1	7	10	14.4
#10	22	22	19.9	18	18	18.1	9	11	16.2
#11	25	25	22.1	19	20	20.1	7	12	18.0
#14	36	36	26.6	28	28	24.1	12	15	21.6

Development length controlled by 50 percent of equivalent hooked bar development length

Development length based on ODOT calculations, but less than ACI development length

Development length based on ODOT calculations and exceeds ACI development length

ACI development length = 75 percent equivalent hooked bar development length

Calc = Calculated development length from combination of head capacity and bar development

For concrete strengths above 5.0 ksi, the required minimum development length for headed reinforcement can be calculated using 50 percent of the equivalent hooked bar development length.

1.5.5.15 High Strength Reinforcement

1.5.5.15.1 ASTM A706 Grade 80

ASTM A706 grade 80 reinforcement is available on the market. The cost premium for A706 grade 80 reinforcement is approximately 6 to 10 percent above grade 60. Local steel mills (Cascade Steel Rolling Mills) are producing #5, #8, and #11 bars on a regular cycle and stock these bars. Avoid specifying other bar sizes, otherwise a minimum (combined size and length) of 50 tons is required. However, there can be some flexibility for smaller quantities. Contact

Cascade Steel Rolling Mill for requirements when high strength rebar in non-standard rebar sizes is considered for a project with less than 50 tons.

When using A706 grade 80 reinforcement, the design yield strength is 80 ksi. ASTM A706 reinforcement is weldable. Welding would be needed when A706 grade 80 reinforcement is used for confinement hoops. The contractor needs to submit a PQR and WPS for approval as is typical for any rebar welding.

1.5.5.15.2 ASTM A1035 Grade 100

ASTM A1035 grade 100 reinforcement has a design yield strength of 100 ksi. *ASTM A1035* is described as Deformed Plain Low-Carbon, Chromium Steel Bars for Concrete Reinforcement. Proprietary products that meet the requirements of *ASTM A1035* specifications are sold under the brand names of ChromX 9100 (*ASTM A1035 CS*), ChromX 4100 (*ASTM A1035 CM*), and ChromX 2100 (*ASTM A1035 CL*). The main difference between the three products is the chromium content; the higher the number, the greater chromium content.

The reduced chromium content results in lower cost, when high corrosion resistance is not required. The products are not weldable. Currently, Cascade Steel produces ChromX 9100, ChromX 4100, and ChromX 2100 with a cost premium of approximately 192, 70 and 45 percent respectively. Cascade Steel carries some inventory of #5, #8 and #11 bars. For non-stock items, a minimum (combined size and length) of 50 tons is required. However, there can be some flexibility for smaller quantities. Contact Cascade Steel for requirements when high strength rebar in non-stock rebar sizes is considered for a project with less than 50 tons.

When *ASTM A1035* grade 100 bars are specified for a project, ensure that designation letters for the chromium content, i.e. CS, CM, or CL, are included with the ASTM material specifications shown in plan sheets and the project special provisions.

1.5.5.15.3 ASTM A615 Grade 100

Grade 100 reinforcement according to *ASTM A615* requirements is available. The cost premium for *A615* grade 100 reinforcement is approximately 35 percent above grade 60. Similar to other high strength reinforcement products, even though there is a required minimum order of 50 tons for combined size and length, there can be some flexibility for smaller quantities. Contact Cascade Steel for requirements when high strength rebar is considered for a project with less than 50 tons.

The cost premiums shown in this article are preliminary and for rebar production only.

1.5.5.15.4 Application of High Strength Reinforcement

Do not use high strength reinforcement in bridge members where plastic hinging is expected under seismic loads, such as bridge columns. Although A706 grade 80 reinforcement has similar ductile properties compared to A706 grade 60, testing of full-scale seismic models sufficient to satisfy *AASHTO* concerns has not yet been completed.

The stress-strain property of A1035 grade 100 is very different from A706. There is not a well defined yield plateau. More experimental testing is necessary before its full implementation in members designed to form plastic hinges. The overstrength magnifier as defined for A706 in the *Guide Specifications for LRFD Seismic Bridge Design* may not be appropriate. At this time, an overstrength magnifier of 1.4 is recommended when high strength reinforcement is used in capacity-protected members.

Use of high strength reinforcement is recommended in the following areas:

- Bridge decks –Refer to *DET3170* and *DET3171* for deck reinforcement design. See *BDM 1.9.2.1.2* for determining the appropriate use of high strength reinforcement in bridge decks.
- Drilled shafts – Use of high strength reinforcement reduces cost and congestion in drilled shafts thereby making them more constructible. Drilled shafts are designed for elastic seismic performance therefore plastic hinging is not expected.
- Crossbeams & End beams – Use of high strength reinforcement can reduce cost and congestion in negative and positive moment areas of crossbeams and end beams. Normally these members are capacity-protected; therefore they are designed to remain elastic during a seismic event. High strength reinforcement can be used for temperature steel and stirrups as well.

Grade 80 bars are a better option for a replacement of grade 60 bars due to lower cost premium and shorter development length compared to grade 100 bars. In addition, grade 80 has stress-strain behavior similar to grade 60 with greater yield stress and ultimate strength.

Within the same member, do not mix different rebar grades of the same bar size. This policy is to avoid any confusion that may occur during construction. It is acceptable to specify different rebar grades in the same member, when the different grades of bar are also significantly different in bar size (at least two bar sizes apart). For instance, longitudinal #8 bars in a crossbeam can be grade 80 bars, whereas #5 stirrups and temperature bars can be grade 60.

Table 1.5.5.15.4-1 illustrates rebar quantities in the previously mentioned members that are allowed to be reinforced with high strength reinforcement. The quantities shown in *Table 1.5.5.15.4-1* will be different when high strength reinforcement is used. A reduction of 10 to 30 percent in quantities can be anticipated when grade 60 rebar is replaced by grade 80 reinforcement.

All bridges in *Table 1.5.5.15.4-1* consist of precast prestressed concrete girders with a CIP deck. Most spans are simple for dead load and made continuous for live load. The bridges include drilled shafts with different lengths depending on the soil condition at the sites. Several bar sizes are grouped together since these bars can be alternately used in the design to reduce a number of different bar sizes. Specify only a few and commonly available bar sizes in each member.

Table 1.5.5.15.4-1 Comparison of High Strength Rebar Quantities

Bridge No.	Length (feet)	Deck Area (ft2)	No. Drilled Shafts	Span Description	Deck (tons)	Deck (tons)	Cross beams* (tons)	Cross beams* (tons)	Drilled Shafts (tons)	Drilled Shafts (tons)
-	-	-	-	-	#4 - #6	#7 - #9	#4 - #7	#8 - #11	#5 - #6	#9 - #11
22008	968	44015	8 – 8 foot	6 – 150 foot 1 – 50 foot deck girder spans	106	45	54	51	43**	218**
21576	249	31665	15 – 6 foot	2 – 122 foot deck girder spans	87	20	36	55	48	87
21343	524	25152	4 – 8 foot	3 – 180 foot deck girder spans	75	57	29	18	6	37
22248	84	11344	16 – 3 foot	1 - 80 foot deck girder span	33	1	12	8	4	10

Table notes:
* Includes intermediate diaphragms, end beams and cap beams.
** A706 grade 80 rebar.

As shown in *Table 1.5.5.15.4-1*, when non-stock rebar is specified, the amount of deck reinforcement in one bar group can meet the required minimum quantity for the first three bridges. The rebar quantities in crossbeams and end beams are not sufficient when a 20 percent reduction due to the use of high strength rebar is applied. For drilled shafts, the amount of reinforcement in one bar group can meet the required minimum quantity only for the first two bridges. However, when high strength rebar of the same size is also used in other members of the bridges, it is possible that the quantities of each bar size will reach the minimum order requirement.

Showing two options of rebar grades on bridge plans is encouraged to accommodate a contractor that may not be able to obtain high strength bars during a construction project. When this approach is taken, all dimensions need to be prepared to work with both options, especially details related to splice lengths and development lengths. Splice lengths and development lengths for high strength rebar are longer compared to grade 60.

Couplers are available on the market for high strength reinforcement. These couplers are capable of meeting 125 percent of yield strength. The ODOT Materials Lab has the capability to test rebar couplers up to #14 bars in grade 100.

1.5.5.16 Glass Fiber Reinforced Polymer (GFRP) Reinforcement

Glass fibers have an advantage over other fibers for composite materials because of an economical balance of cost and specific strength properties. Glass fibers are commercially available and exhibit good electrical insulation properties. When glass fibers are encapsulated in suitable resin as a system, the composite material is less sensitive to alkaline environment, freezing and thawing condition, and extremely elevated temperature. These properties make GFRP suitable for use as reinforcement for concrete structures. Through a number of research projects, it has been found that GFRP bars exhibit minimal loss in strength when subjected to sustained tension and have good resistance to fatigue. When GFRP bars are embedded inside concrete, there is no UV exposure concern.

ODOT has used GFRP bars on a number of projects particularly in bridge decks and sound walls. Using GFRP bars in the bridge decks was due to good corrosion-resistant and non-conductive properties. The non-conductive property of GFRP bars does not have detrimental effect to cathodic protection system often used for preserving marine and coastal area bridges as long as electrical continuity of adjacent steel reinforcement is maintained. For sound walls, wall weight needed to be minimized. Use of GFRP bars resulted in thickness reduction, since smaller concrete cover could be specified.

Since the release of the 1st Edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* in 2009, there have been advancements in material specifications along with new knowledge and field experiences. *AASHTO* published the 2nd edition of the guide specifications in 2018, which incorporated the new comprehension and extended design provisions for other structural members in addition to bridge decks and traffic railings.

For material specifications, *ASTM D7957* published in 2017 includes requirements for material standards of GFRP reinforcing bars. GFRP bars are commercially available from multiple manufacturers and can be produced for construction with a reasonable lead time. A specification for GFRP reinforcement has been prepared and is available when requested from the BDM technical resource.

GFRP rebar is a good corrosion-resistant reinforcement alternative for reinforced concrete bridges in corrosive environment compared to ferrous reinforcement due to cost and material strength. It is generally less expensive than other alternative reinforcement (stainless steel). Use of GFRP bars does not require electrical isolation to other ferrous reinforcement since it is not conductive. Tensile strength of GFRP bars is slightly higher than mild steel reinforcing bars, however GFRP bars have linear elastic behavior, much lower modulus of elasticity, and smaller ultimate tensile strain. Deformation or surface texture, or both, are required to ensure adequate bond capacity. Nominal material properties used for design are shown in *Table 1.5.5.16-1.*

Table 1.5.5.16-1 Material Properties for Design of GFRP Reinforced Concrete

Material Property	Material Value
Ultimate tensile strength (ksi)	90 ksi (#4 - #6) 75 ksi (#7 - #10)
Tensile modulus of elasticity (ksi)	6,500
Ultimate tensile strain (inch/inch)	0.011

GFRP reinforcement is recommended for structural members that are located in corrosive environment and do not require high ductility. Service limit states often control design over strength limit states. Use GFRP reinforcement only in the following structural members:

- Bridge deck.
- Sound walls.
- Seawalls.
- Bridge Approach Systems.

GFRP bar weight is lighter than steel reinforcement, therefore construction workers are able to handle the reinforcing bars with ease, but more rebar ties are required to maintain GFRP bar position in the rebar cage during concrete pour. GFRP bars are susceptible to abrasion and impact from studded tires and removal tools, therefore use in bridge concrete deck is limited in specific area as shown in *Table 1.26.3-1*.

1.5.5.17 FRP Reinforcement

(Reserved for Future Use)

1.5.5.18 Bar Lengths

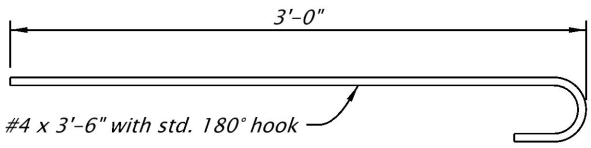
Use stock bar lengths whenever possible without sacrificing economy. Unless absolutely necessary, do not call for bars longer than 60 feet because they are difficult to handle and transport. Bar lengths specified include hook lengths unless detailed otherwise.

Table 1.5.5.18-1 Bar Length Stock Lengths

Bar Size	Stock Length *
#3	20 feet and 40 feet
#4 and #5	20, 30 and 40 feet
#6 thru #18	60 feet

Table notes:
*Only small quantities of #14 and #18 bars are stockpiled by the supplier because of size and weight and may require special mill orders.

Figure 1.5.5.18-1 Specified Length of Bar with Hook



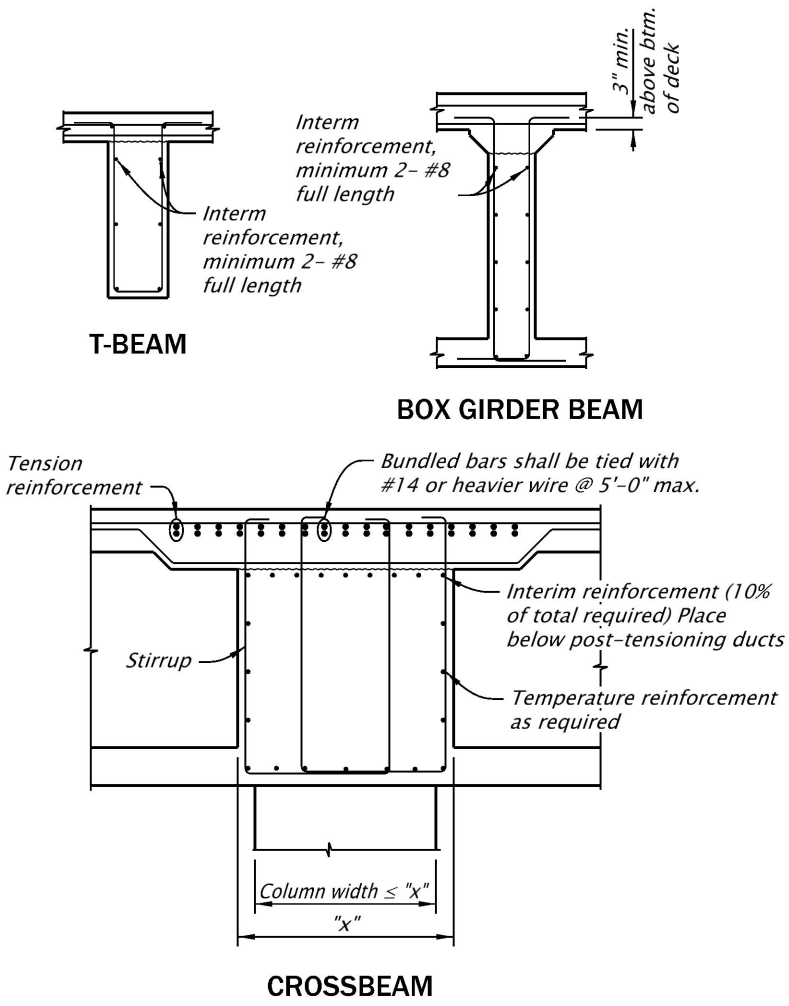
1.5.5.19 Interim Reinforcement for T-Beams and Box Girders

When the deck slab of a continuous T-beam or box girder is placed after the concrete in the stem has taken its set, place at least 10 percent of the negative moment reinforcing steel full length of the longitudinal beam to prevent cracks from falsework settlement or deflection. In lieu of the above requirement, two #8 bars full length of longitudinal girders may be used.

Locate a portion of the negative reinforcement in the stem of the crossbeam below the level of the deck slab construction joint in concrete crossbeams whose principal negative reinforcement lies in the deck slab. Provide sufficient ultimate reinforcement capacity to support 150 percent of the dead load of the crossbeam and superstructure 5 feet along the centerline of the structure either side of the center of bent. Use no less than 10 percent of the total negative reinforcement.

In cases where the bent crossbeams are skewed to the deck steel, place the top crossbeam steel in the top of the stem below the deck (dropped panel). See *Figure 1.5.5.19-1* for typical details.

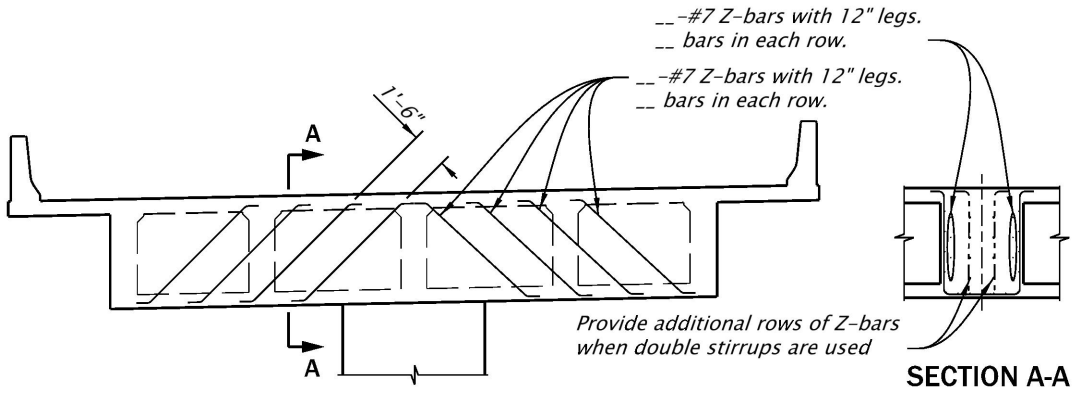
Figure 1.5.5.19-1 Interim Reinforcement



1.5.5.20 Additional Shear Reinforcement

As shown below, provide additional reinforcement to the calculated shear reinforcement in cantilevered portions of crossbeams. Pay careful attention to clearances and possible conflicts with post-tensioning ducts and other reinforcement. Detail the size and number of bars to provide at least 20 percent of the factored Strength I Limit State shear demand at the face of the column. Apply this provision to cantilevered sections of crossbeams when the crossbeam cantilever from the face of the column exceeds the crossbeam depth at the face of the column. This additional reinforcement may be omitted when the shear reinforcement provided from the critical shear section to the face of column provides 20 percent additional capacity above the controlling strength limit state.

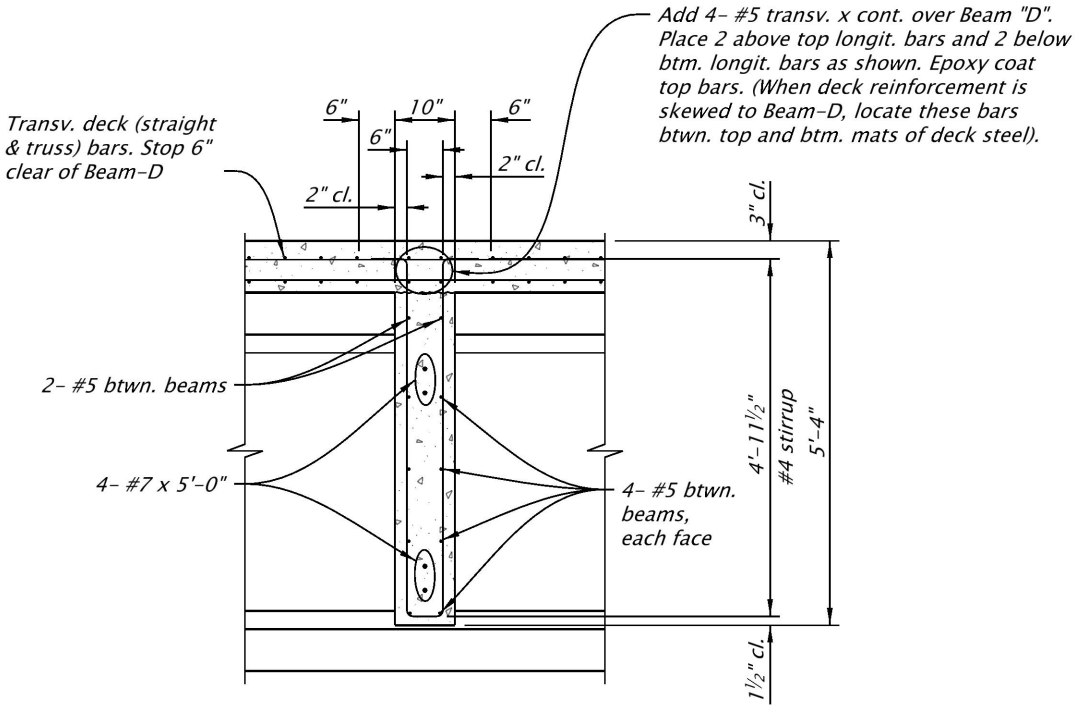
Figure 1.5.5.20-1 Additional Shear Reinforcement in Cantilevered Sections



1.5.5.21 Diaphragm Beam Reinforcement

Figure 1.5.5.21-1 below assumes the deck reinforcement is stopped 6 inches clear of the transverse beams. The added bars provide reinforcement for Beam-D and the deck overhang. When straight bars are used, the spacing of the deck steel will be continuous over the transverse beams and no additional bars will be required.

Figure 1.5.5.21-1 Diaphragm Beam Reinforcement



SECTION AT BEAM-D

1.5.6 Precast Prestressed Concrete Elements

1.5.6.1 Design of Precast Prestressed Elements

C 1.5.6.1

1.5.6.1.1 Girder systems and Deck Requirements

C 1.5.6.1.1

Side-by-side slabs and box beams:
5 inch minimum HPC thickness with a single mat of reinforcement (8 inch maximum centers each way) and transverse tie rods. 7 inch minimum thickness for any portions overhanging the exterior slab or box beam.

Side-by-side Bulb-T and deck Bulb-T girders: 7-1/4 inch minimum HPC thickness with two mats of reinforcement (8 inch maximum centers in each mat and each direction).

Spread slabs and box beams:
8 inch minimum HPC thickness with two mats of reinforcement (8 inch maximum centers in each mat and each direction).

Bulb-T (not side-by-side) and Bulb-I girders: 8 inch minimum HPC thickness.

Deck Bulb-T girders with UHPC connection: Air entrained concrete is required for girder top flange portion (8 inch minimum thickness). The deck girder system and connection details are shown in *Figure 1.5.6.1-1* and *1.5.6.1-2* respectively. ¾ inch thick minimum PPC overlay is applied on top of the deck girder system. Precast concrete Deck Bulb-T girders are connected using UHPC at flange ends to form bridge deck.

Side-by-side Deck Bulb-T girders with weldment connection and ACP: The deck-girder system is not allowed on state-owned bridges. Local Agencies may submit a design deviation explaining the justifications when the following conditions of the bridge are met:

- Not on NHS routes.
- ADTT less than 1,000.
- Located in Mild Area as defined in *BDM 1.2.4.4*.

Weld connection is susceptible to failure, especially when repeated design stresses exceed the allowed stress threshold, quality of the weld is not adequate, the bridge is located in the corrosive environment, or any combination thereof. Even though there were only a few failed weldment connections reported, the weldment was mostly not visible for inspection. Bridge inspectors would identify the weldment issue by using resulting indications, such as girder separation or rotation, uneven deflection, pavement cracking, etc.

See also *BDM 1.9.3.1.6* for requirements regarding asphalt concrete pavement on concrete bridge decks.

Figure 1.5.6.1-1 Section View of Deck Bulb-T Girder System with UHPC Connections

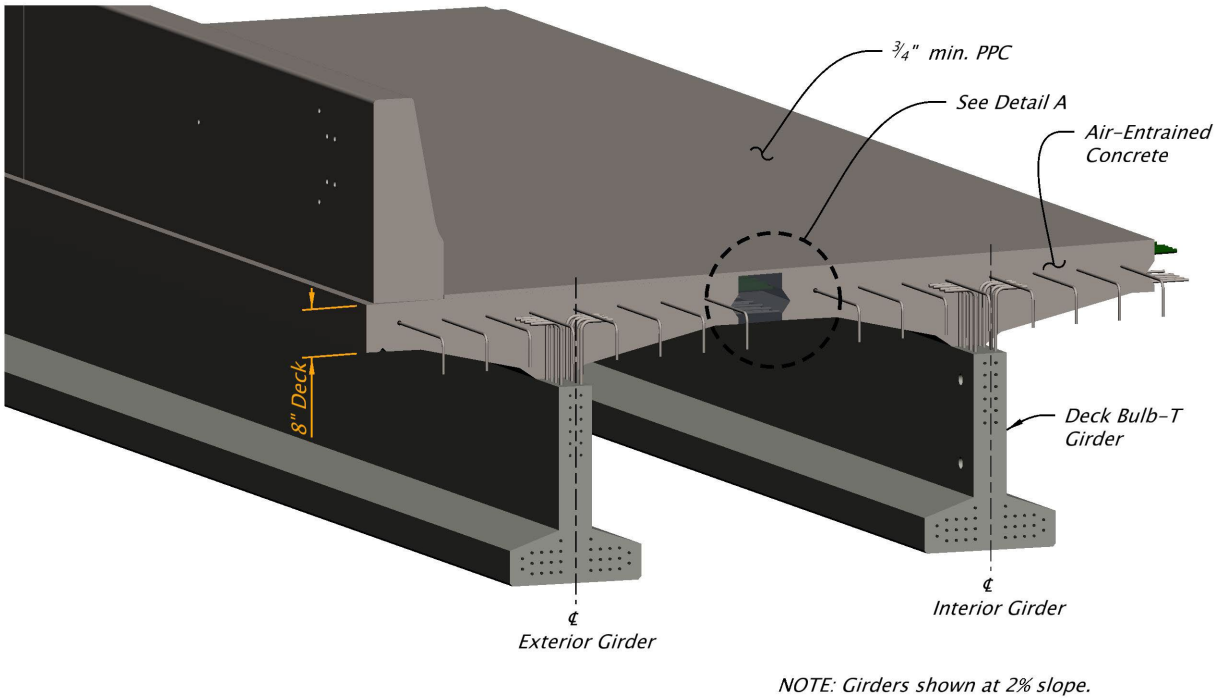
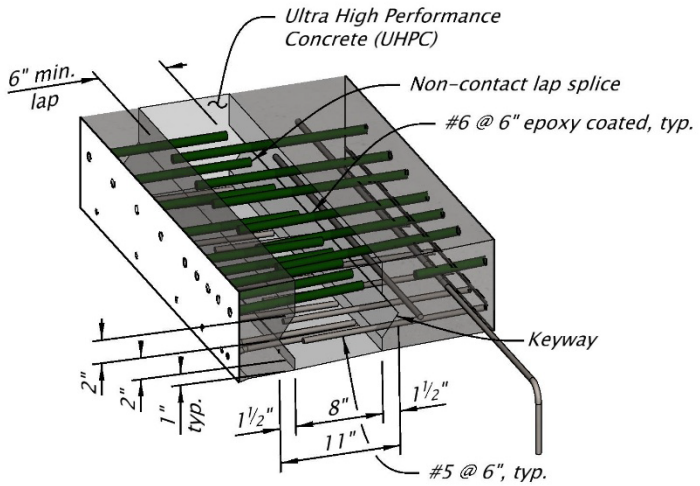


Figure 1.5.6.1-2 Connection Details Between Girder Flanges



1.5.6.1.2 Concrete Strength

Ensure design compressive strengths of concrete are not significantly higher than design requirements and are within the following allowable ranges of design compressive strengths. List the required concrete strengths in the Girder Schedule.

The allowable range of design compressive strengths of concrete at 28 days ($f'c$) to be used are shown in *Table 1.5.6.1-1*.

C 1.5.6.1.2

When precast, prestressed members are used without a cast-in-place deck, the 28 day compressive strength is limited to 8000 psi. This limitation is required to ensure adequate air entrainment and to ensure adequate workability. Higher strength concretes generally are less workable and therefore are more difficult to achieve an acceptable finish suitable for a riding surface.

Table 1.5.6.1-1 Allowable Design Compressive Strength Ranges

Precast Element	Minimum	Maximum
for precast, prestressed girders without CIP deck	4000 psi	8000 psi
for precast, prestressed girders with CIP deck	5000 psi	10,000 psi

The allowable range of design compressive strengths of concrete at release ($f'c$) for all prestressed concrete girders are 4000 psi minimum and 7000 psi maximum.

When a separate concrete mix (8000 psi or less) is used for the top flange, then higher strengths (up to 10,000 psi) may be used for the remainder of the member.

1.5.6.1.3 Concrete Tensile Stress Limits

Modify *LRFD Table 5.9.2.3.1b-1* as follows:

- Modify the 9th bullet to $0.0948 * sqrt(f'c)$, where $f'c$ is in ksi.
- **No Tension** criteria in 6th and 8th bullets still apply.

Modify *LRFD Table 5.9.2.3.2b-1* as follows:

- Modify the 1st and 8th bullets to $0.0948 * sqrt(f'c)$, where $f'c$ is in ksi.
  No tension criteria in 3rd, 5th and 7th bullets still apply.

When simple span precast girders are made continuous for live load design as follows:

- Design the positive moment area as if the girder was simply-supported.
- A maximum concrete tensile stress up to $0.19 * sqrt(f'c)$ in the positive moment area will be allowed for this condition. Also ensure that the maximum concrete tensile stress in the positive moment area does not exceed $0.0948 * sqrt(f'c)$ when the girder is considered continuous for live load.
- Use a load factor of 0.80 for live loads in Service III load combination to check tensile stresses in prestressed concrete members with prestressing strands and reinforcing bars.

C 1.5.6.1.3

PRESTRESS LOSSES

Calculate prestress losses in precast prestressed members according to *LRFD 5.9.3.4* – Refined Estimates of Time-Dependent Losses using gross section properties.

Include prestress gain from loads permanently applied to girders.

Do not include the prestress gain due to application of live load in the total long-term loss calculation.

C PRESTRESS LOSSES

This method of calculating losses is the **Detailed** method presented in NCHRP Report 496 Prestress Losses in Pretensioned High-Strength Concrete Girders. Prestress loss estimates have generally been in the 35 to 45 ksi range. The *LRFD 5.9.3.4* loss calculations appear to be consistent with earlier loss predictions. And these loss levels have resulted in relatively accurate predictions of camber at the time of deck placement. There has also been no record of service cracking in bridges designed using these prestress loss levels.

Permanent loads include the weight of all components and utilities attached to the bridge, such as selfweight, overlay, planned widening and earth fill.

The amount of prestress gain due to application of live load can be more than 20 percent of the total prestress loss. Not including this gain results in a conservative estimate of final girder stresses. Prestress loss affects girder stress but does not change the ultimate strength or capacity to carry permit loads.

1.5.6.1.5 Girder Shape Selection

OREGON BULB-T AND BULB-I

Use Oregon Bulb-T girder shape for bridge projects, where applicable. This shape has a 4 foot wide top flange. This top flange provides safety for workers who must form bridge decks and ensures stability of the girder during shipping.

Use the Bulb-I shape only when it has benefits over a Bulb-T. Since Bulb-I girders have a narrow top flange, it requires less concrete build-up over the girder compared to a Bulb-T. Therefore, bridges with high superelevation (generally, greater than 7 percent) may be candidates for the Bulb-I shape.

Modified Bulb-T girders include those with a non-standard top flange width and those with a wider web. Fabricators are generally able to adjust the top flange width anywhere from 24 inches to 48 inches. At least 3 inches can also be added to the top flange. Discuss any modifications to the top flange with fabricators before placing modified details on plan sheets. Design deviations are not required for top flange modifications.

Adjust web thickness only when necessary to accommodate post-tensioning (such as for spliced girders).

BT90 & BT96 girder sections are the largest in the Oregon inventory. These sections have a 5 foot wide top flange which is necessary to ensure shipping stability of very long girders. Do not consider changes to the top flange width without concurrence from fabricators. The longest girder available

C 1.5.6.1.5

C OREGON BULB-T AND BULB-I

Bulb-I girders are a standard variation of the Bulb-T. To make a Bulb-I, the fabricator will start with the Bulb-T form and add blockouts to portions of the top flange to make the Bulb-I shape.

Since the Bulb-I section is 3 inches taller than the equivalent Bulb-T, it may be preferred for span lengths slightly longer than the equivalent Bulb-T capability. A Bulb-I section may provide benefits over a deeper Bulb-T section. However, due to shipping stability and worker safety concerns, a deeper Bulb-T might still be preferred when the deeper section can be accommodated within the available vertical clearance.

For such cases, increase the web thickness from 6 to 7.5 inches. Increase the top and bottom flange widths by the same amount.

BT96 girders have not yet been used in Oregon. Verify availability of this section before specifying it on a project.

from fabricators is around 185 feet total length. Verify availability for any girder length exceeding 180 feet.

OREGON DECK BULB-T

Roadway surface is directly provided by Deck-Bulb Tee girders with UHPC connection. Because girders are erected plumb, the top flanges are required to be cast on a slope equal to a specified superelevation.

To ensure constructability of flange concrete, a superelevation is limited to 5 percent maximum. Specify thicker PPC overlay (3.75 inches maximum) and thickened flange near beam ends as required to match roadway vertical profile. See *DET3385* and *DET3386* for details.

Due to its function as the roadway surface, construction requires minimal differential camber between adjacent girders in the span. The construction specification requires a tight camber difference, however there will be some residual camber variation due to construction tolerances.

Overfilling of the connection is a common placement method for casting UHPC. As a result, grinding is required to smooth out the uneven surface. Subtract ½ inch flange thickness in the design to account for top surface grinding.

Provide 15 percent minimum over-designed capacities as reserve capacities for locked-in force effects from the girder camber adjustment during construction.

C OREGON DECK BULB-T

AASHTO SHAPES

Use AASHTO shapes only for bridge widenings where the existing bridge has AASHTO shapes.

C AASHTO SHAPES

AASHTO Type II, Type III, Type IV, and Type V shapes do not have the same efficiency as Bulb-T shapes.

There may also be rare cases when an AASHTO shape may provide slightly less vertical clearance compared to the available Bulb-T shape.

WSDOT SHAPES

Specify WSDOT girder section, when there is no Oregon section that can meet an application or provide efficient design for certain situations.

Design and specify WSDOT shape girders according to ODOT design and Specification requirements.

Consult with fabricators for availability of WSDOT girder section forms. Two WSDOT shapes are allowed:

- WF girders: WF36G to WF95G girders. The maximum form length for the WF shapes varies depending on the girder section ranging 80ft – 195ft.
- WF Deck girders (UHPC): WF39DG to WF86DG girders. WF Deck girders are designed to be erected side-by-side and the top flanges are connected using UHPC.

C WSDOT SHAPES

WSDOT girder sections may be an acceptable alternate based on cost and availability. The *Standard Specifications* allow contractors to propose an alternate shape provided it is similar to the specified shape and meets all project requirements (see *SP 00550.03*).

WF Deck girder forms will need modification because of the required thicker top flange due to the ODOT requirements, as a result there will be a cost premium to modify the forms.

SPLICED GIRDERS WITH POST-TENSIONING

Spliced girders with post-tensioning can be used to extend span capabilities of precast concrete girders. Consult with fabricators regarding the appropriate section and segment lengths.

C SPLICED GIRDERS WITH POST-TENSIONING

HAUNCHED AND TRAPEZOIDAL BOX GIRDERS

Consult with fabricators before considering using haunched girders. Although haunches may provide an aesthetic benefit, any structural benefit from haunching a prestressed girder is minimal.

C HAUNCHED AND TRAPEZOIDAL BOX GIRDERS

Trapezoidal box girders are available for applications that require special aesthetic considerations. Trapezoidal box girders can either have a uniform depth or parabolic haunches. Horizontally curved trapezoidal boxes have been used in Colorado.

1.5.6.1.6 Strand Type

Bulb-T and AASHTO girders were developed for use with 0.5 inch diameter prestressing strand. Do not consider use of 0.6 inch diameter strand for these sections without first consulting with fabricators. Modification of the girder section may be needed to accommodate 0.6 inch diameter strand.

BT90/96 and WSDOT shape sections were developed for use with 0.6 inch diameter prestressing strand.

C 1.5.6.1.6

1.5.6.1.7 Shipping routes

Review potential shipping routes and consult with fabricators to make sure the proposed girder type can be shipped to the bridge site.

C 1.5.6.1.7

Narrow roads and sharp curves may restrict the length of girder that can be used.

1.5.6.1.8 Oregon Fabricators

The following northwest precast concrete fabricators can provide precast concrete members to Oregon bridge projects:

- Concrete Technology Corporation in Tacoma, WA.
- Knife River Prestress, Inc. in Harrisburg, OR and Spokane, WA.
- Western States Prestress, Inc. in La Grande, OR.

C 1.5.6.1.8

1.5.6.1.9 Detailing for Precast Concrete Members

C 1.5.9.1.9

CAMBER

See *BDM 1.5.9* for special requirements pertaining to ACP, sidewalk, and rail requirements.

DECK DRAINAGE

See *BDM 1.9.5* for details specific to slab and box beam elements.

AGE OF GIRDER

SP 00550.49 prohibits transportation before seven days and only after the 28 day compressive strength has been achieved.

There may be special construction circumstances when a member needs to be transported and placed before the seven days, but it is not recommended before the 28 day compressive strength has been achieved.

Specify a required wait time on the plan sheet when a design requires a delay for placing the girder on bearing devices to decrease the bearing thickness or encasing the beam ends for fixed connection to reduce restraint moments due to long-term shrinkage and creep effect.

SKEW

Limit skew to 45 degrees for precast slabs and 30 degrees for precast boxes.

Excessively skewed slabs and boxes tend to warp more, making fit and obtaining uniform bearing on the bearing pads more difficult. Stair stepping the bearing pads may be necessary to obtain uniform bearing.

STAGE CONSTRUCTION OF SLABS AND BOXES WITH CAST-IN-PLACE HPC DECKS

Specify spread slabs or boxes with an 8 inch minimum HPC deck (two mats of deck steel) for bridges constructed in stages.

TRANSVERSE CONNECTION FOR SIDE-BY-SIDE SLABS AND BOXES

Connect side-by-side slab and box elements with transverse tie rods as detailed on *BR445*. Alternate connection details, such as intermittent weldments, are not allowed.

Side-by-side slabs or boxes with a 5-inch HPC deck do not allow tie rods to be placed as detailed in *BR445* for staged construction.

TRANSVERSE CONNECTION FOR SIDE-BY-SIDE SLABS AND BOXES

SURFACE FINISH FOR PRECAST MEMBERS

The *Standard Specifications* requires a rake finish on the tops of members that are not used as a roadway surface. A rake finish combined with extending stirrup legs up into the deck is considered sufficient to provide adequate capacity to ensure composite action between the girder and deck. It is not necessary to require additional roughening.

C SURFACE FINISH FOR PRECAST MEMBERS

INTERFACE SHEAR

For all members with a cast-in-place deck, provide interface shear reinforcement full length of the member regardless of whether or not it is required by design. This requirement is satisfied by extending stirrups from the precast member up into the deck slab and will result in minimum reinforcement across the interface shear plane equal to two #4 bars at 18 inch centers.

INTERFACE SHEAR

JOINT AND KEYWAY DETAILS

See standard drawings for recommended details.

C JOINT AND KEYWAY DETAILS

1.5.6.2 Design and Detailing of Precast Prestressed Girders

1.5.6.2.1 Stay-in-Place Forms

Where the spacing between edges of precast concrete girder flanges is no greater than 2 feet, steel stay-in-place deck forms may be used. However, do not use stay-in-place forms in exterior bays.

Steel stay-in-place deck forms may also be used behind end beams where the deck is continuous over interior bents. Hot-dip galvanize all steel stay-in-place forms.

When stay-in-place deck forms are used, provide a minimum section modulus of 0.15 in^3/ft and a maximum form height of 1 ½ inches. Install stay-in-place forms such that the top of the form is at the design bottom of deck thickness. The weight of a form meeting these requirements is likely to be less than 2 psf. This weight is not significant and need not be included in the design. However, add additional non-composite dead load per *Table 1.3.2-1* in the girder design to account for extra concrete weight.

Do not use stay-in-place forms at deck overhang areas or where the edges of girder flanges are greater than 2 feet apart. In such cases, access for inspection and future maintenance of the deck precludes the use of stay-in-place deck forms.

Do not use stay-in-place forms in coastal areas, as defined in *BDM 1.2.4.2*.

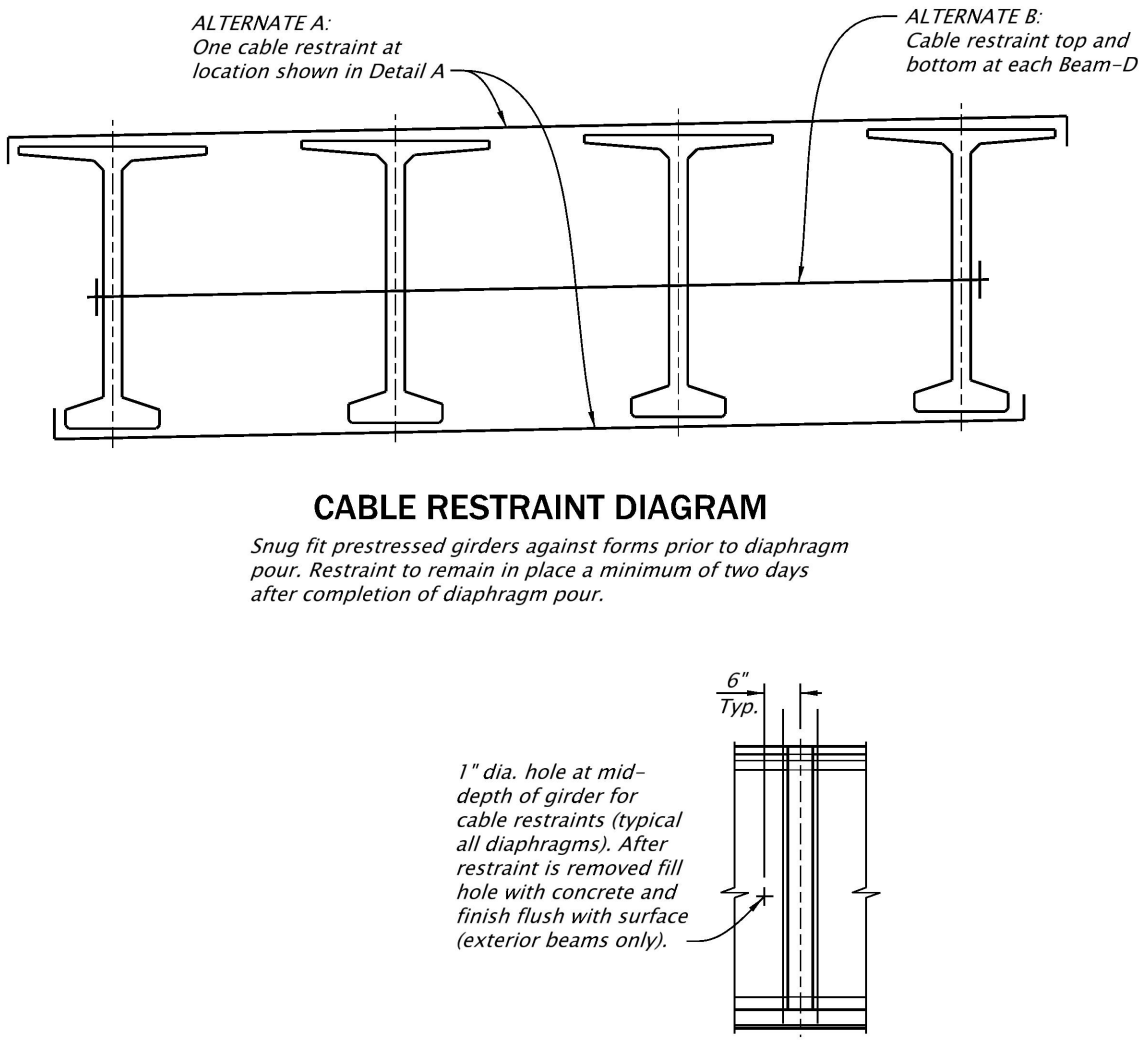
These provisions apply to precast girders, slabs and boxes.

Where stay-in-place forms are considered, add the following statement with the loading section of the general notes:

> Stay-in-place deck forms may be used except for exterior overhangs and between the exterior girder and the first interior girder on each side of the structure. XX psf additional non-composite dead load has been included in the girder design to account for extra concrete and form weight associated with stay-in-place forms.

1.5.6.2.2 Diaphragm Beam Restraint

Figure 1.5.6.2.2-1 Diaphragm Beam Restraint



1.5.6.2.3 Beam Seat or Top of Crossbeam Elevation

Provide a note on the plans indicating when the beam seat (or top of crossbeam) elevations shown are for deck buildups based on three months camber. Adjust the beam seat (or top of crossbeam) elevations during construction to correct for the revised deck buildups.

1.5.6.2.4 Continuous Deck Reinforcement

Simply supported precast prestressed girders with continuous deck over interior piers can be used to create deck continuity for the entire structure. Design the girders as simply supported for superimposed dead loads and live loads, only the deck and its reinforcement are continuous over the pier.

Use the bar size, length and spacing as shown in *Figure 1.5.6.2.4-1*. Specify 20 feet long reinforcing bars for bridge spans with 60-inch tall girders or smaller. Specify 30 feet long reinforcing bars for bridge spans with girders taller than 60 inches.

Do not use the continuity detail for bridges:

- With span lengths greater than 160 ft.
- Carrying interstate routes.
- With ADT greater than 10,000.
- With ADTT greater than 2,500.
- In SDC C and D.

C 1.5.6.2.4

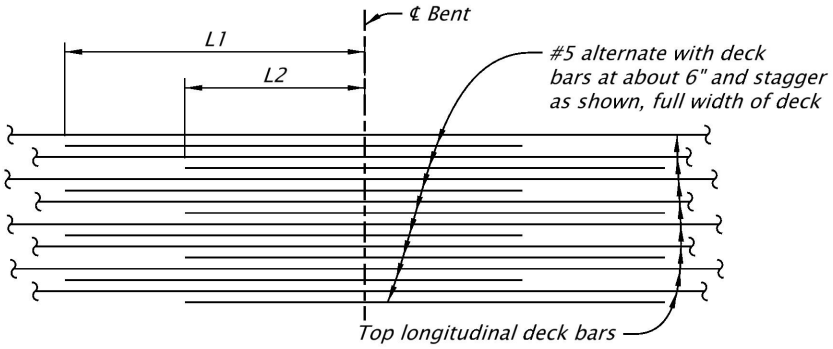
The continuity detail allows simplification of design and detailing and elimination of the expansion joint over the pier. Flexural cracks in the deck are expected at the pier due to live loads. The cracks should close when the live loads disappear.

This detail does not apply to bridges designed simple for dead loads and made continuous for live loads. The girder system requires deck reinforcement to resist negative bending moments generated from the live loads. This results in more deck steel than the detail shown in *Figure 1.5.6.2.4-1*.

NCHRP Report 519 provides design examples for girders made continuous.

This detail is not allowed on bridges with SDC C and D due to unknown performance during a large earthquake.

Figure 1.5.6.2.4-1 Interior Bent with Continuous Deck



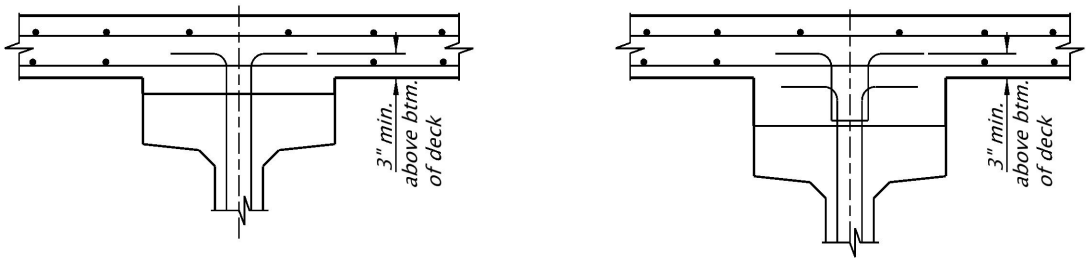
#5 Bar	L1	L2
20' long	12'-6"	7'-6"
30' long	17'-6"	12'-6"

1.5.6.2.5 Beam Stirrups

Bulb-T and Bulb-I standard drawings show stirrups with 90 degree shop bent hooks at the top of the girder. These hooks must protrude at least 3 inches above the bottom of the deck. When they do not, because of excessive build-up, the standard drawing requires the use of **U** bars to fill the gap.

There is no need for the stirrup hooks or **U** bars to extend to the top mat of deck reinforcement, as has been shown in the past. Detail plans to reflect these requirements.

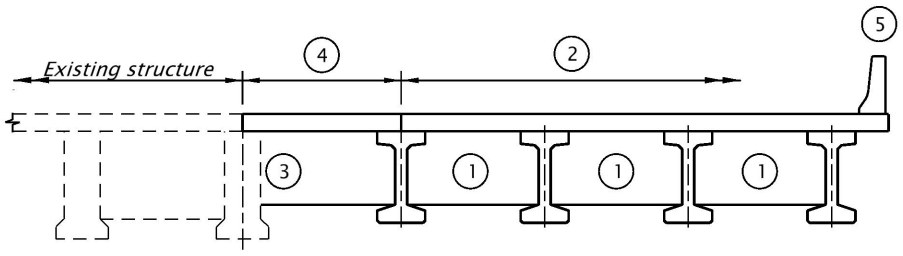
Figure 1.5.6.2.5-1 Beam Stirrup Details



1.5.6.2.6 Structure Widenings, Precast Beam Bridges

Detail connections between superstructures to prevent widening dead loads from being transferred to the existing beams. This may be accomplished by delaying the connection pour (diaphragm and deck) until most of the dead load is applied to the widening. The designer chooses the appropriate placement method. See *BDM 1.9.4.2* for additional closure pour information.

Figure 1.5.6.2.6-1 Closure Pour Method

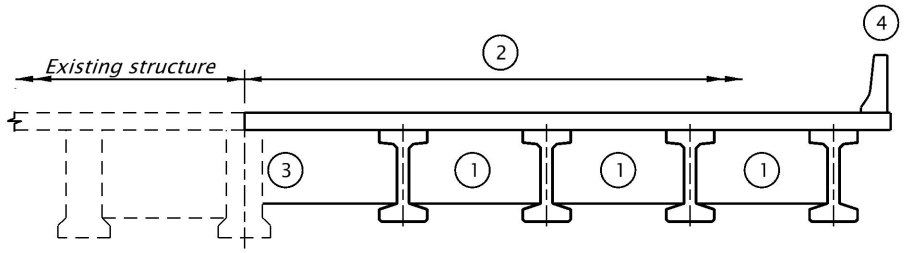


POUR SCHEDULE
(INCLUDING CLOSURE POUR)

1. *Make pour in diaphragms.*
2. *Make pour in deck slab. Delay pour (2) a min. of 3 days after pour (1).*
3. *Make pour in diaphragm of closure pour section.*
4. *Make pour in deck slab of closure pour. Delay a minimum of 3 days after pour (3).*
5. *Make pour in bridge rail.*

In the closure pour method shown in *Figure 1.5.6.2.6-1*, the deck screed machine would normally be placed or supported on the widening beams. As the concrete is placed, the beams tend to deflect equally. This equal deflection normally gives better control of deck thickness and deck steel cover.

Figure 1.5.6.2.6-2 Delayed Diaphragm Pour Method



POUR SCHEDULE
(INCLUDING CLOSURE POUR)

1. *Make pour in diaphragms.*
2. *Make pour in deck slab. Delay pour 2 a min. of 3 days after pour 1. Blockout deck as required to make pour 3.*
3. *Make pour in diaphragm closure pour a minimum of 3 days after pour 2.*
4. *Make pour in bridge rail.*

In *Figure 1.5.6.2.6-2* showing the delayed diaphragm pour method, the deck screed machine rails would normally be placed or supported with one rail on the existing structure and one rail on the widening beams. As the concrete is placed, the new beams would tend to deflect more than the existing composite beams. This unequal deflection makes it more difficult to control deck thickness and deck steel cover, especially at the new beam adjacent to the existing structure.

1.5.6.2.7 Deck Pour Sequence

See *BDM 1.9.4.3*.

1.5.6.2.8 Earthquake Restraint Details

See *Cost Data Books* for sample plans and details.

1.5.6.2.9 Fixed Girder Connections

Where girder ends are designed with a fixed connection to an end beam or bent cap, embed the girder into the end beam (or bent cap) a minimum of 8 inches. Provide transverse bars/rods through the girder ends as shown on the standard drawings (*BR300* & *BR310*). In addition to the above requirements, provide strand extensions or dowels at the end of the girder as needed to ensure adequate transfer of loads to the substructure.

To minimize restraint moments due to girder creep and shrinkage, establish continuity when the age of the girders is at least 30 days after casting. When a certain concrete age is required by

design to minimize the time-dependent effect, include the following note in the plan sheet with continuity details:

> Place continuity diaphragm at least XX days after the girders are manufactured.

1.5.6.2.10 Girder Spacing

Limit girder spacing to 9 feet for girder sections up to BT72 and 1.5 times girder depth for larger girders.

PRECAST MEMBERS TOPPED WITH ACP

Side-by-side elements have been historically topped with ACP over a waterproofing membrane. See *BDM 1.9.3.1.6.* This type of construction works well in a stage construction scenario as long as the elements are placed consecutively from one side to the other.

When using this type of construction, the previous stage precast element at the stage construction joint must carry some of the wearing surface dead load from the subsequent stage since adjacent slabs must have their tie rods connected before the wearing surface is placed for the subsequent stage. This additional load is generally ignored (i.e., the members are designed as if they were all placed in one stage). Long term creep is thought to mitigate this condition. To date, annual inspections have found no distress in precast elements due to this practice.

For cases where elements cannot be placed consecutively from one side to the other, it becomes impossible to place standard transverse tie rods. For this reason, select a different structure type (ex., spread slabs or girders with CIP deck) when elements cannot be placed consecutively. Any side-by-side precast slab or box element must be connected to adjacent elements with transverse tie rods as detailed in *BR445*. Alternate details, such as intermittent weldments, are not allowed.

The use of asphalt concrete pavement is no longer allowed due to long term maintenance concerns. See *BDM 1.9.3.1.6*.

PRECAST MEMBERS TOPPED WITH CIP CONCRETE

Side-by-side elements may be topped with an HPC deck. See *BDM 1.5.6.1* for minimum deck thickness and reinforcing requirements.

For this type of construction, the deck dead load is substantially larger than the PPC or ACP case. For this reason, this type of construction must be detailed to prevent the deck dead load from later stages from being transferred to previous stages.

One solution to this problem is to provide a space (12 to 18 inches) between the stages that is filled with a CIP closure girder which is placed after full deck dead load is applied to both adjacent sections. For this case, design the precast members adjacent to the construction joint as exterior girders. Design the CIP closure girder to carry a contributory portion of live load under the strength limit states.

Use of spread slabs or boxes is another possible solution for stage construction. When so, use 7 ¼ inch minimum deck thickness with two mats of steel as required by *BDM 1.5.6.1*.

Any side-by-side precast slab or box element must be connected to adjacent elements with transverse tie rods as detailed in *BR445*. Alternate details, such as intermittent weldments, are not allowed.

1.5.7 Cast-In-Place Superstructure

1.5.7.1 General Design

1.5.7.1.1 Structure Depths

See *BDM 1.2.7.11* for minimum depth and live load deflection requirements.

1.5.7.1.2 Computations of Deflections

Base computed deflections on the effective moment of inertia of the section.

Estimate long term deflections as instantaneous deflection times a factor of three for reinforced concrete elements.

1.5.7.2 Interim Reinforcement for T-Beams

See *BDM 1.5.5.19*.

1.5.7.3 Diaphragm Beam Steel

See *BDM 1.5.5.21*.

1.5.7.4 Box Girder Stem Flare

Taper changes in girder stem thickness for a minimum distance of 12 times the difference in stem thickness. See Standard Detail *DET3125* for details.

1.5.7.5 Shear Keys and Construction Joints

At construction joints between the stem and slab of concrete girder bridges, use the following note:

> Roughened surface finish. See *SP 00540.43(a)*.

See *BDM 1.14.2.2.1*.

1.5.7.6 Standard Access and Ventilation in Concrete Box Girders

Provide permanent access to all cells of concrete box girders. Access may require using manholes or access holes through bottom slabs, diaphragm beams, crossbeams and longitudinal beams. Standard Drawings *BR135* and *BR136* show standard access and ventilation details. See *BDM 1.2.10* and *BDM 1.15.5* for additional accessibility guidance.

In addition to standard drawings for access holes, draw a section normal to the girder through the access hole showing the relationship of the longitudinal stems, utility lines, and crawl holes to the access hole and ladder. Do not show dimensions when the drawing is to scale.

Use the following guidelines tempered with engineering judgment.

- Deck Access Holes – Do not place access holes through the deck of a structure. There is a potential for the access hole cover to leak. Avoid disruption of traffic and the need for traffic protection and direction.
- Bottom Slab Access Holes - Single span bridges will normally require one access hole per cell. Multiple span bridges will normally require one access hole per cell at each end of the bridge. Locate access holes in accordance with the guidelines shown on the standard drawings. An 8 feet minimum height to the access hole is recommended to discourage unauthorized access into the structure. Keep the inspector in mind when choosing the access locations. Do not place access holes over railroad tracks.
- Girder Stem Access Holes - Provide girder stem access holes through the interior stems at the midpoint of all spans. These lateral access points will allow the inspector to complete their inspection of span or spans without having to exit and reenter the structure.
- Crossbeam Access Holes - These are not detailed on the standard drawing since their design will vary widely because of structural requirements. Only one access hole will be required per crossbeam when the girder stem access holes are provided.
- Bottom Slab Ventilation Holes - These ventilation holes, similar to the bottom slab access holes in design except top opening, are intended to be used in all cells of each span not having access holes. Locate the ventilation holes near the opposite end of the span from an adjacent span having access holes. The holes provide additional ports for removing forms, serves as an exhaust hole when forced ventilation is required and provides additional natural ventilation.
- Stem Ventilation Holes - These holes provide for the escape of lighter-than-air gases and are located near the high point of each span as detailed on the standard drawings.
- Ladder Support – Provides a safe support for the ladder while the inspector unlocks the access hole cover. After the cover is unlocked, reposition the ladder through the access hole so the inspector can grab onto the ladder while entering/leaving the box girder cell.
- Access Cover Prop - The access cover prop is designed to facilitate the opening or closing of the cover when the ladder is supported by the ladder support. Once the ladder is through the access hole, release the prop so the cover will lie flat. The prop would be reengaged upon exiting the box.

1.5.7.7 Form Removal

All forms are to be removed from cells where access is provided.

Deck forms to be removed may be supported off the bottom slab when the bottom slab is fully supported, designed to support the added load and has no detrimental effect on the structure.

Deck forms for non-accessible cells may be left in place. Deck forms left in place are not to be supported off the bottom slab. Web supported deck forms are acceptable. Include an allowance for deck form dead load in the design loads, see *BDM 1.3.2*.

1.5.7.8 Bottom Slab Details

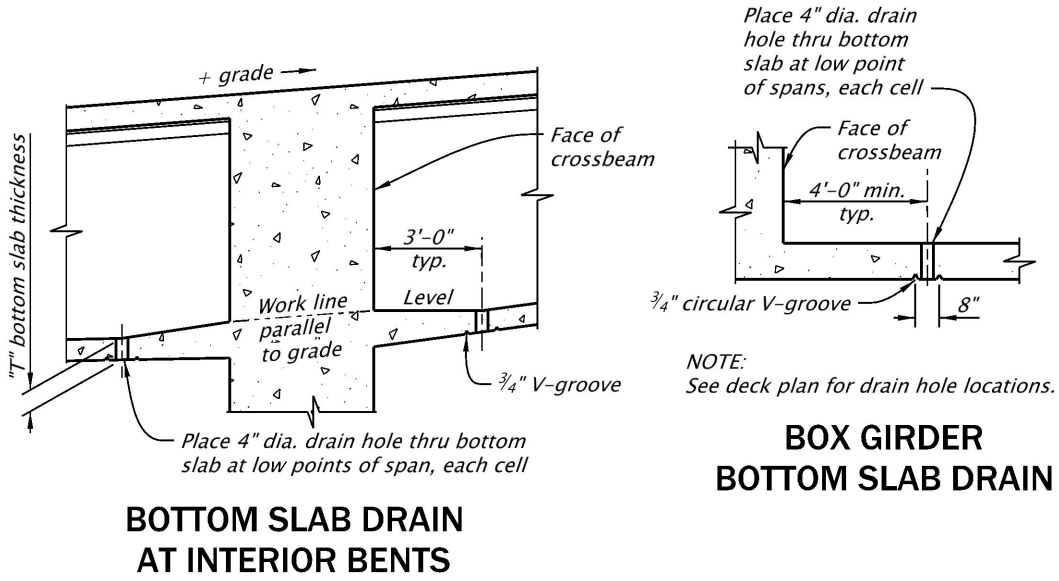
Generally, show the bottom slab of box girders to be parallel to the top slab in transverse section so that all girder stems will be the same depth.

Provide a bottom slab thickness of no less than 6 inches.

For skewed box girders, orient bottom slab transverse bars the same as the deck transverse bars. See *BDM 1.9.1* and *LRFD 9.7.1.3* for requirements.

Place a 4 by 4 inch drain hole through each diaphragm beam at the low point of each cell. Place a 4 inch diameter drain hole through the bottom slab at the low point of each series of cells in a span. For cells that carry water lines, increase 4 inch diameter to 6 inch diameter.

Figure 1.5.7.8-1 Bottom Slab Drain Details



1.5.7.9 CrossBeams

See *BDM 1.5.5.19* and *BDM 1.5.5.20*.

1.5.7.10 Fillets

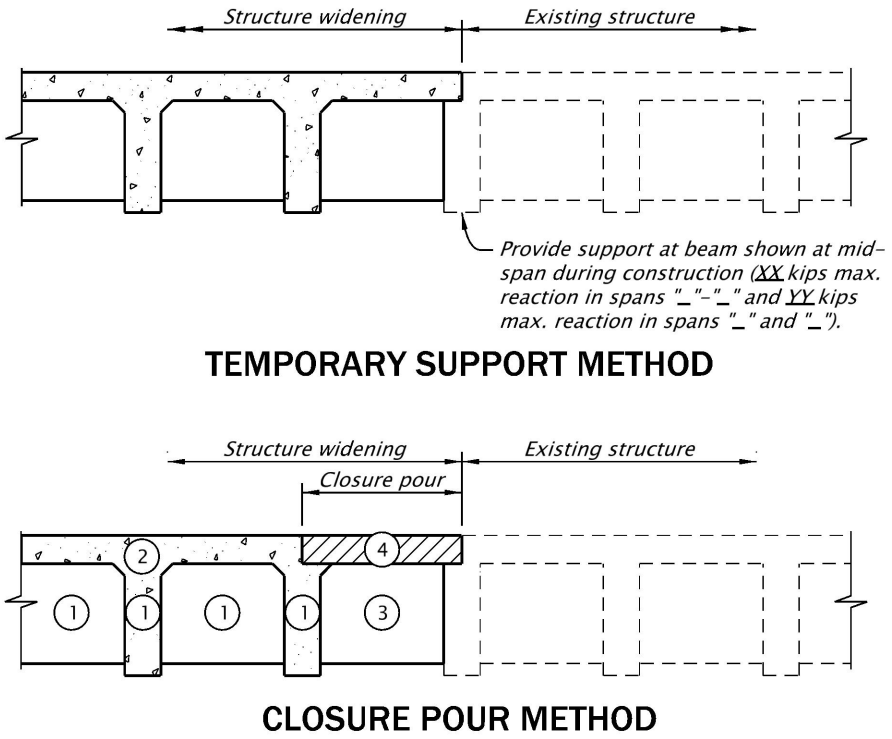
Provide adequate fillets at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required.

Provide a 4 by 4 inch fillet at the intersection of the crossbeam and the deck; and the end diaphragm beam and the deck. For additional detailing requirements see *BDM 1.9.2.*

1.5.7.11 Structure Widenings, Cast-in-Place Superstructures

Detail connections between superstructures to prevent widening dead loads from being transferred to the existing beams. One method is to temporarily support the beam adjacent to the widening during construction. Designate locations where supports are required and expected maximum reactions. An alternate method requires closure pours for the diaphragm and deck slab. See *BDM 1.9.4.2* for additional closure pour information.

Figure 1.5.7.11-1 Closure Pour Method for CIP Superstructures



1.5.7.12 Stay-in-Place Forms for Deck

For deck construction, stay-in-place forms will not be allowed. Loss of access for inspection and future maintenance of the deck preclude the use of stay-in-place deck forms.

1.5.8 Post-Tensioned Structures

1.5.8.1 Design Considerations

1.5.8.1.1 Structure Depths

See *BDM 1.2.7.11* for minimum depth and live load deflection requirements.

1.5.8.1.2 Shrinkage and Creep Stresses

The stresses in the superstructure and substructure of post tensioned concrete bridges which result from elastic shortening may be assumed to remain in the structure indefinitely. The stresses which might be assumed to develop as the result of shrinkage and creep may be assumed to be relieved by creep.

1.5.8.1.3 Shortening of Post Tensioned Bridges

Values in *Table 1.5.8.1.3-1* for shortening of post tensioned, cast in place concrete bridges are based on field measurements by the ODOT Bridge Section. Compare the design values with the field measured values and use the more conservative value.

Table 1.5.8.1.3-1 Shrinkage Prior To Tensioning (Theoretical)

Shrinkage Time Period	**Shortening Length (inch/foot)**
0.4 x .0002 ft/ft x 12 in/ft x 100 ft =	0.10/100
Elastic shortening	0.44/100
Shrinkage and creep after tensioning to 1 year	0.29/100
Shrinkage and creep 1 year to 20 years (anticipated)	0.10/100

These structures were stressed to an average concrete stress of 1200 psi (1000 to 1300 psi). For other values, the elastic shortening and creep is roughly proportional. ODOT data indicates that variation of these values by 50 percent would not be unusual.

1.5.8.1.4 Deflections

Estimate long term deflections as the net instantaneous deflection ($DL + Prestress$) times a factor of two for cast-in-place post-tensioned elements.

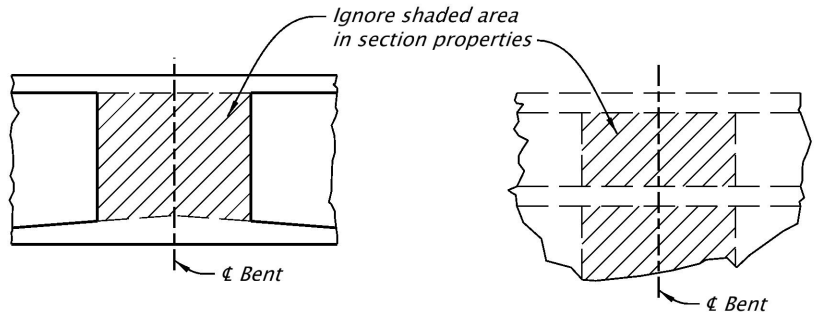
1.5.8.1.5 Curved Post-Tensioned Ducts

Design for the radial prestress forces resulting from curved tendons in post-tensioned structures. Additional shear/flexural reinforcement may be required to resist the lateral web forces and ties to resist the web bursting forces.

1.5.8.1.6 Design Moments at Interior Bents of Post-Tensioned Bridges

For crossbeams with widths less than the distance between the top and bottom slab, do not include the crossbeam in the superstructure section properties. Project the stem and slab dimensions to the centerline of the bent and use those dimensions to calculate section properties. Use the negative moment at the bent centerline for design.

Figure 1.5.8.1.6-1 Crossbeam Section Properties



For greater crossbeam widths, use the above section properties and consider adding supplementary reinforcing steel across the top of the crossbeam to control any theoretical cracking that may occur from live loading.

1.5.8.1.7 Skewed Box Girders

Box girder bridges with skews of over 20 degrees cannot be safely designed without taking into account the effects of skew. These effects generally increase as any of the following increase: skew angle, span length, torsional rigidity of the superstructure. The principal effect of skew is to increase the reactions at the obtuse corner of the structure and to reduce those at the acute corners (sometimes even causing uplift). This increases shear in the beams adjacent to the obtuse corners and produces transverse shear in the deck and bottom slab. These effects can be reduced by reducing the skew, which generally means lengthening the structure or by placing crossbeams at interior bents normal to the centerline of the structure.

When torsion due to skew is a problem, consider reducing the torsional stiffness of the structure. RCDG bridges, either cast in place or with precast girders, are torsionally limber. For additional detailing of skewed decks see *BDM 1.9.2.1.1*.

Do not design box girder bridges with bents skewed more than 45 degrees from the normal to the structure centerline.

Careful design of post tensioning with regard to the deflection and slope of the girder at a skewed end can nullify or reverse the tendency of the obtuse corner of the skewed structure to take a disproportionate part of the dead load. Theoretically, this could be done so that under

full $DL + LL + I$, the reactions would be equal at all bearings. Even an approximation of this condition will benefit the design.

1.5.8.1.8 Concrete Tensile Stress Limits

The concrete tensile stress limits given in *BDM 1.5.6.1* also apply to post-tensioned members.

1.5.8.2 General Details

BDM 1.5.7 generally apply to post-tensioned box girders as well as conventional box girders.

1.5.8.2.1 Conventional Box Girders

See Standard Details *DET 3125* and *DET 3130* for general details.

1.5.8.2.2 Precast Trapezoidal Box Girders

See *BR133* and Standard Details *DET 3131*, *DET 3132* and *DET 3134* for general details.

1.5.8.2.3 Access and Ventilation

See Standard Drawings *BR135* and *BR136* for general details.

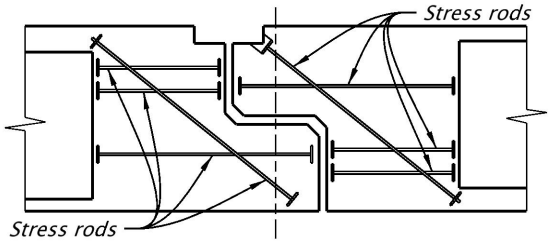
1.5.8.2.4 Overhang Design and reinforcing

See *BDM 1.9.2.2.*

1.5.8.3 Stress Rod Reinforcement of Bearing Seats

An example of a stress-rod reinforced bearing seat is shown below. In order to retain a significant amount of prestressing force, provide a rod with stressed length of not less than 10 feet.

Figure 1.5.8.3-1 Stress Rod Reinforcement of Bearing Seats



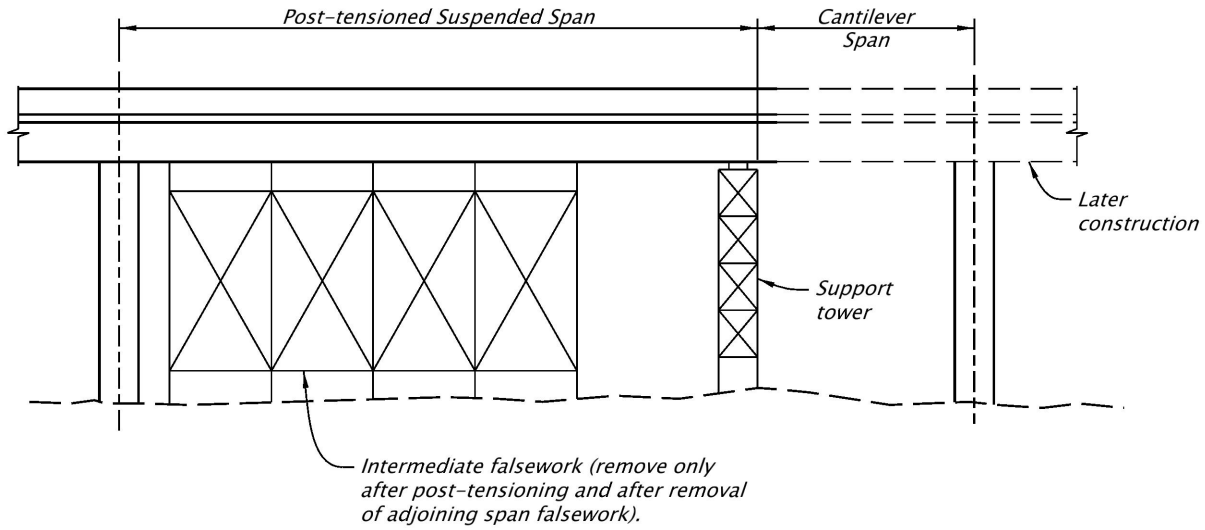
1.5.8.4 Segmental Construction

Where precast and cast-in-place elements are joined in a continuous, segmental structure, chamfer exterior corners of the cast-in-place portion to match precast elements. Standard practice is to chamfer precast elements, even when the chamfer is not shown on our drawings or the shop drawings.

1.5.8.5 Support Tower Details and Notes

Design the support tower at the end of the suspended span to support the reaction from the suspended span including the additional reaction due to post tensioning. Show on the plans the approximate total reaction in kips. Design the tower to accommodate the elastic shortening of the superstructure due to post tensioning. Make provisions so that the superstructure may be returned to the plan elevation (raised or lowered) in the event that actual settlement at the top of the tower differs from the anticipated settlement. Keep the support tower in place until the suspended span is fully supported by the cantilever and adjoining span.

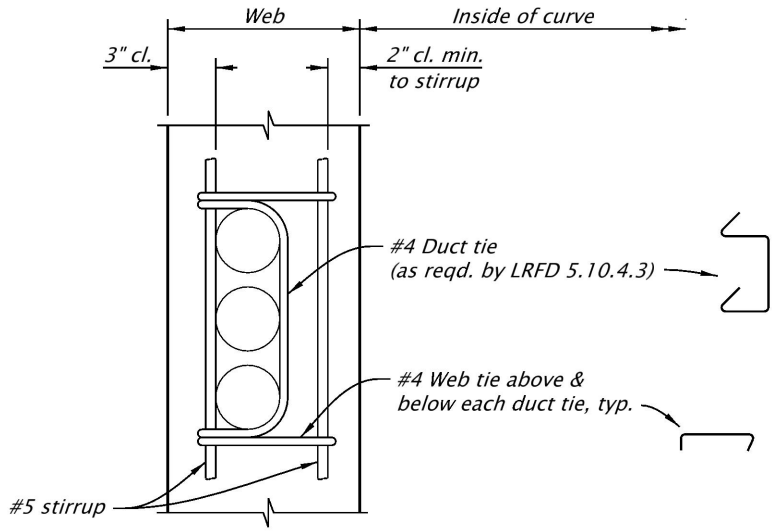
Figure 1.5.8.5-1 Support Tower Details



1.5.8.6 Post-Tension Strand Duct Placement

Place ducts for post-tensioned bridges using the detail provided on *DET3130*. The most common type of duct arrangement has been the bundled duct detail. This detail can be used when the duct size does not exceed 4 ½ inches and when the horizontal curvature of the bridge does not require the use of supplemental ties (see *LRFD 5.10.4.3*). When the horizontal curvature does result in the need for supplemental ties, do not use bundled ducts. When supplemental ties are required due to horizontal curvature, use the following detail:

Figure 1.5.8.6-1 Typical Web and Duct Tie Detail



Detail post tensioned box girders to allow pouring the bottom slab and stems as separate pours. Design the prestressed tendon path to ensure that the ducts do not fall in the area of the bottom slab. See Standard Details *DET 3125* and *DET 3130* for general details. To ensure the ducts are fully encased in concrete, do not place ducts in the bottom slab and keep ducts at least 1 inch below the fillet construction joints near the top slabs. Show *Figures 1.5.8.6-2 and 1.5.8.6-3* on the project plans when needed.

Figure 1.5.8.6-2 Low Point Detail

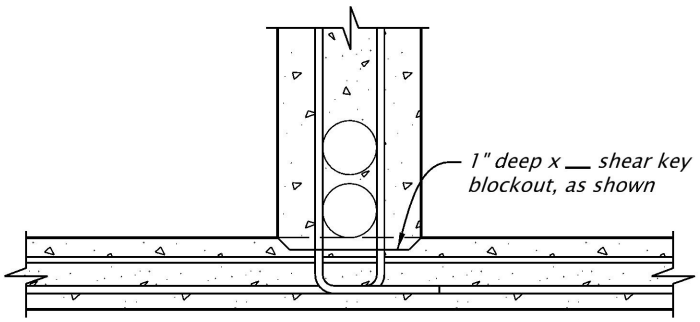
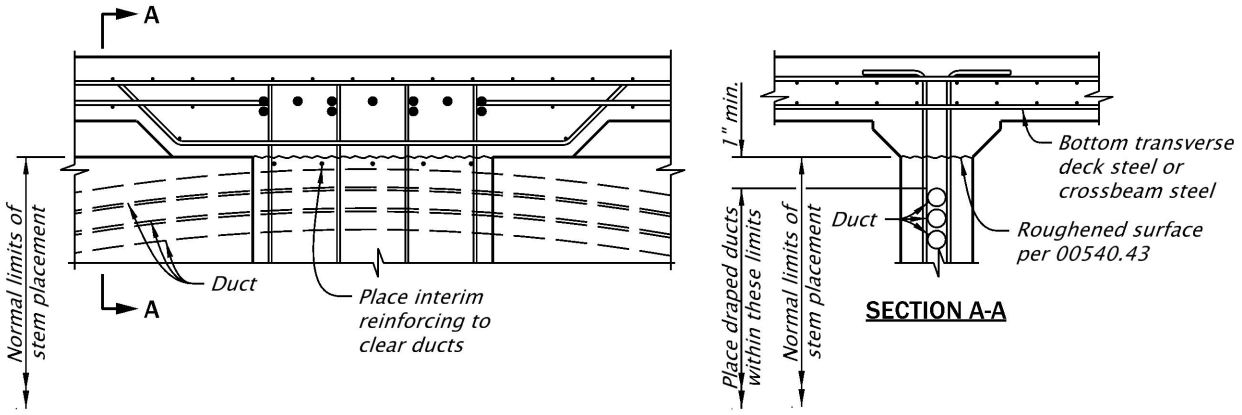


Figure 1.5.8.6-3 High Point Detail



In some cases it may be necessary to place ducts outside the limits shown above. When so, special concrete placement details will normally be needed to ensure the ducts are fully encased in properly consolidated concrete for the entire length of the bridge. For these cases, submit a design deviation request which shows the proposed duct placement detail. Include with the request the details and specification language intended to ensure the concrete will be fully consolidated in areas where the ducts penetrate either into the bottom slab or above the stem fillet construction joint.

1.5.9 Camber Diagrams

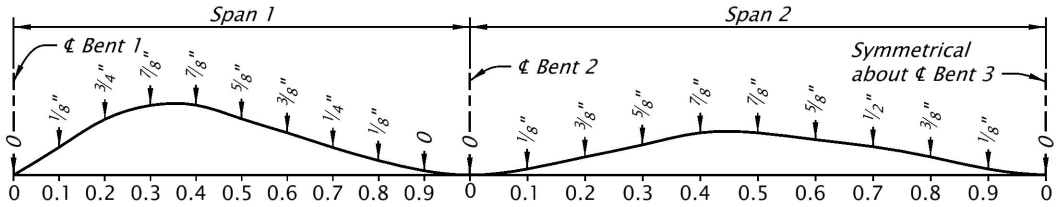
1.5.9.1 Camber Diagrams, General

Show camber diagrams on the plans for all types of cast-in-place concrete structures. The camber diagram shall be titled, **Camber Diagram** and be accompanied by the applicable portions of the following note:

Camber is designed to compensate for deflection due to prestressing, the dead load of all concrete, stay-in-place forms and wearing surface and the long-term effects of shrinkage and creep.

An example of a camber diagram for a cast-in-place structure is shown below in *Figure 1.5.9.1-1*.

Figure 1.5.9.1-1 Camber Diagram Example

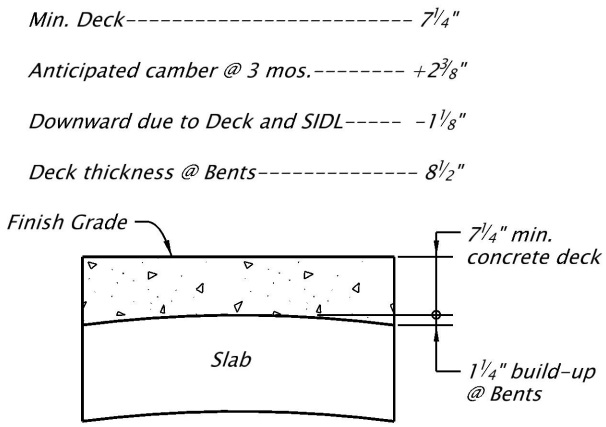


NOTE:
Camber is designed to compensate for deflection due to prestressing, the dead load of all concrete, stay-in-place forms and wearing surface and the long term effects of shinkage and creep.

1.5.9.2 Precast Prestressed Slabs and Box Beams

Camber of precast prestressed elements occurs due to prestressing strands generally located below the center of gravity of slabs or box beams. Reflect allowances for camber and grade correction in top of slab elevations. Increase rail posts lengths and curb heights accordingly near the ends to obtain the proper finish rail height and curb exposure. Note on the typical deck section that post lengths may vary due to camber or superelevation. Include information on the contract plans as shown in *Figure 1.5.9.2-1*. The figure shows slabs with 7 ¼ inch thick concrete deck as an example.

Figure 1.5.9.2-1 Concrete Deck Build-Up Detail



Note to Designer:
When a wearing surface is specified instead of deck concrete, replace "Deck" and "concrete deck" with the wearing surface name, e.g. PPC, MPCO, etc.

1.5.10 Pour Sequences

1.5.10.1 Pour Sequences, General

In order to avoid misunderstanding and claims by the contractor, ensure that construction sequences and pour sequences are clearly described. Particular care is needed when symmetrical structures are covered by sketches showing half of the structure.

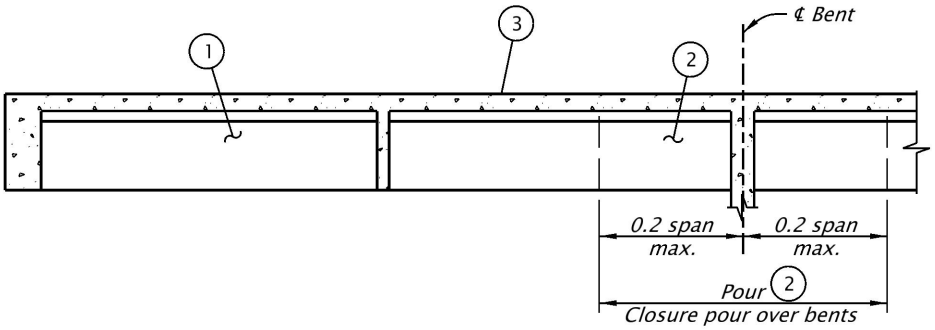
In general, longitudinal pours in continuous spans are stopped near the bents to allow concrete shrinkage to occur in the majority of the span. Closure pours over the bent are generally shorter to minimize shrinkage cracking that could occur between fixed supports or placements.

It is recommended to place bottom slab or beam construction joints at a falsework bent rather than a permanent bent. Cracking may develop at a permanent bent, when the adjacent falsework settles or deflects during the concrete placement. For deck closure pours and deck pour sequence see *BDM 1.9.4.2* and *BDM 1.9.4.3*.

1.5.10.2 T-Beams Supported on Falsework

A typical sketch and pour sequence is shown below in *Figure 1.5.10.2-1*.

Figure 1.5.10.2-1 Pour Sequence Example Detail



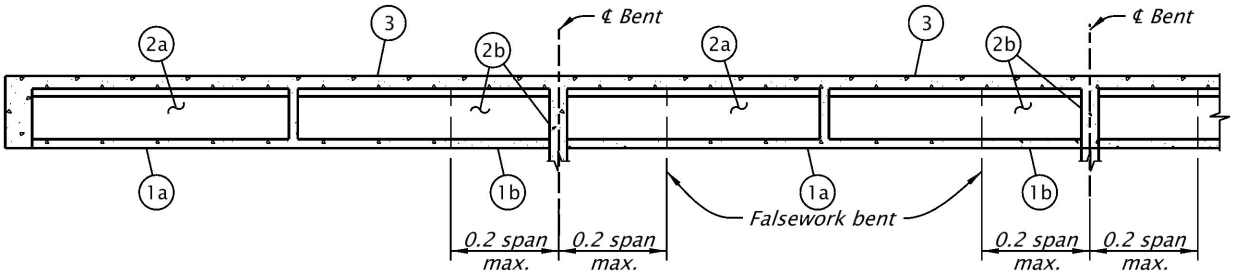
POUR SEQUENCE

1. Pours (1) and (2) are the longitudinal and transverse beams to the bottom of deck (or fillets). Make all pours (1) prior to pours (2). Beam construction joints shall not be near a permanent bent but shall be made at a falsework bent. Delay adjacent beam pours by a minimum of three days.

2. Pour (3) is the (fillets and) deck. Pour (3) to be delayed a minimum of three days after completion of all pours (2). A deck construction joint may be made over any transverse beam. Delay pouring adjacent sections of deck a minimum of five days. Do not remove bulkheads for deck pours until at least three days after completion of pour. Deck pours may extend over any part of a span or spans so long as they meet these requirements.

1.5.10.3 Box Girders on Falsework

Figure 1.5.10.3-1 Pour Sequence Example Detail



POUR SEQUENCE:

1. Pours (1a) and (1b) are the bottom slab. Stop pours (1) at a falsework bent and not at a permanent bent. Delay a minimum of three days between adjacent pours (1). Complete all pours (1a) prior to starting pours (1b). Complete all pours (1) prior to starting pours (2).
2. Pours (2a) and (2b) are the longitudinal and transverse beams to the bottom of the fillets. Stop pours (2) over a falsework bent. Delay the start of pours (2) a minimum of five days after bottom slab pours (1) are complete. Delay a minimum of three days between adjacent pours (2).
3. Pour (3) includes the fillets and deck slab. Pour (3) to be delayed a minimum of three days after completion of all pours (2). Pours (3) may be stopped over any transverse beam, with the use of a deck construction joint. Delay a minimum of five days between adjacent pours (3). Do not remove bulkheads for deck pours until at least three days after completion of the pour. Deck pours may extend over any part of a span or spans as long as they meet these requirements.

Generally, it is preferred that the bottom slab be completely poured first and separately from the longitudinal beams. This ensures a more uniform bottom slab thickness, the slab provides a good base for stem forms, and the continuous bottom slab helps stabilize the falsework system. It also allows the falsework to take its initial settlement without affecting other superstructure components.

1.5.10.4 Drop-In Precast Prestressed Elements

Complicated types of construction require detailed construction sequence notes, such as the following:

Figure 1.5.10.4-1 Pour Sequence Example Detail



POUR SEQUENCE:

1. Make pour (1).

2. Make pour (2), includes Bent 2 column.

3. Make pour (3a), includes bottom slab and webs to bottom of top fillet, Beam **C** to bottom of deck.

4. Make pour (3b), includes deck and top fillets for cast in place section. Delay pour (3b) a minimum of three days after completion of pour (3a).

5. Apply Stage I post tensioning to cast in place section. Stressing to begin a minimum of 14 days after completion of pour (3), but not until concrete in pour (3) has reached its design strength.

6. Place prestressed beams. Beams to be placed so that the number of beams in one span does not exceed by more than four the number in the opposite span.

7. Make pour (4), includes diaphragm beams **D** and end beams **E**.

8. Make pour (5), (no less than 60 days after transfer of stress in precast, prestressed beams), includes deck on prestressed beams to diaphragm beam nearest Bent 2.

9. After pour (5) has been made in Spans 1 and 2, make pour (6a), includes remainder of Beam **C**. Let concrete take initial set, and make Pour (6)b, includes remainder of deck.

10. Apply Stage II post tensioning to assembled Spans 1 and 2. Stressing to begin a minimum of 14 days after completion of Pour (6), but not until concrete in pour (6) has reached its design strength.

11. Pour curbs.

NOTES:

- 1. Bents 1 and 3 footings and walls may be poured any time up to seven days prior to placing of prestressed beams, but concrete must have reached its design strength prior to beam placement. No part shall interfere with post tensioning operations.
- 2. Paving slab and sidewalls may be poured at any time except that no part shall interfere with post tensioning operations.
- 3. Screed deck concrete parallel to bents.
- 4. Composite decks and closure pours shall not be made until at least 60 days have elapsed from the time of transfer of prestressing force in the precast elements.

1.5.10.5 Continuous Cast-in-place Slabs on Falsework

For pours over 600 cy, allow a transverse deck construction joint at $0.2 * (span)$ from the next interior bent.

1.5.10.6 End Bents

When the fit of superstructure elements is critical, be sure to consider the end bent construction sequencing. Normally the end wall construction is delayed until the superstructure elements are in place. Delaying the end wall construction also allows the contractor to compensate for errors in superstructure element lengths and end bent locations. Show a construction sequence diagram, with notes, as needed.

1.5.10.7 Steel Girders

See *BDM 1.6.2.14* for examples.

1.5.11 Concrete Anchors

1.5.11.1 Anchor Bolts / Rods

1.5.11.1.1 Materials

Anchor bolts / rods, including those for bridges, signs, traffic signals, and illumination structures, should normally be specified according to one of the following specifications:

- *ASTM F1554* is the preferred specification.
- *ASTM F1554*, grade 36 for low-strength.
- *ASTM F1554*, grade 55 for medium-strength.
- *ASTM F1554*, grade 105 for high-strength.

Equivalent ASTM designations for anchor bolts / rods are:

- *ASTM A307*, Low-strength carbon steel bolts for general use (non-headed rods conform to *ASTM A36*).
- *ASTM A449*, Medium carbon steel bolts and rods to 3 inch diameter. Proof load requirements are similar to *ASTM F3125* GR A325.

Galvanize anchor bolts or rods full length, when galvanizing is desired.

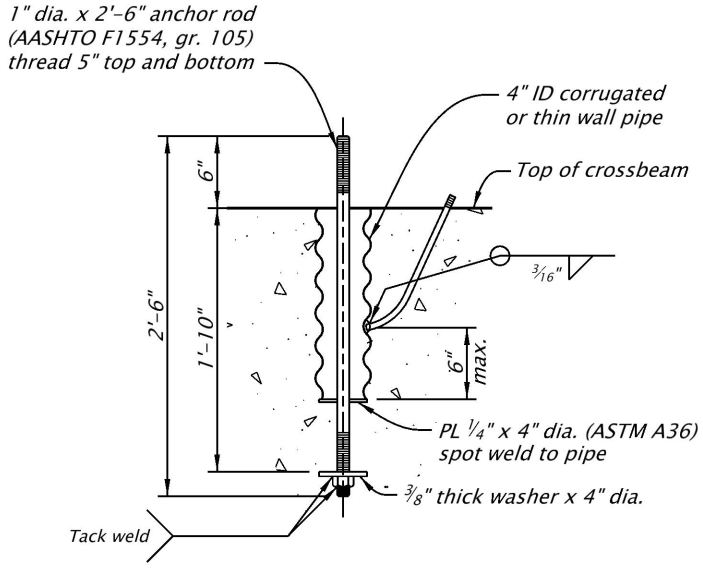
Anchorage of anchor bolts and rods may be accomplished by hooks for *ASTM A307* and grade 36 materials. For higher strength materials, a bearing plate tack welded to a nut or a plate between two nuts should be used.

When tensioning of anchor rods or bolts is desired, load indicator washers may be used up to 1¼ inch diameter (the largest available). Specify load indicator washers on the plans or in the Special Provisions, when required. Recognize that concrete creep and shrinkage may significantly reduce anchor rod stress over time.

1.5.11.1.2 Anchor Bolt Sleeves

To allow for some flexibility in placement and small corrections in bearing locations, an anchor bolt sleeve is often used. The anchor bolt can be field bent slightly to fit the required bearing location. The bearing plate can be temporarily shimmed and then the pad constructed or the pad can be constructed with a blockout around the bolt. The sleeve is grouted at a later time. There are commercially produced anchor bolt sleeves or a fabrication detail can be added to the drawings.

Figure 1.5.11.1.2-1 Anchor Rod Detail



1.5.11.2 Post-Installed Anchors

1.5.11.2.1 Materials

RESIN BONDED ANCHORS

Normally specify *ASTM F1554*, as the anchor rod material. ASTM specifications may be substituted as shown in *Table 1.5.11.2.1-1.*

Table 1.5.11.2.1-1 ASTM Material Substitutions

Anchor Rod Specifications	ASTM Specifications
ASTM F1554 GR 36	A307
ASTM F1554 GR 105	A193 (grade B7) or A449
M31 Rebar, grade 60	A706 or A615

Galvanizing is only required when portions of the anchor are exposed.

Anchor rods do not necessarily need to be fully threaded. Specify the thread length to best fit the particular application.

BONDING MATERIAL

Use a resin bonding system from the Division's QPL for anchor bolts 1 inch diameter or less. For larger anchors, use other types of anchorage such as epoxy grout or cementitious grouts with traditional development lengths.

MECHANICAL ANCHORS

A mechanical anchor system consists of multiple materials and differs from one manufacturer to another. Corrosion-resistant materials are required for mechanical anchors installed in bridge elements. As a minimum, specify hot-dip galvanized coating. Type 316 stainless steel is preferred. All mechanical anchor products on the QPL have an option for providing corrosion-resistant materials. Use the same type of materials for attachments connected to mechanical anchors. When this is unavoidable, provide electrical isolation for all dissimilar metals to avoid galvanic corrosion.

1.5.11.2.2 Design

Ensure that post-installed anchors are embedded in good concrete without active cracks. Avoid using anchors in sections of the bridge with high tensile stresses perpendicular to anchor holes. Drilled holes in concrete attract or even induce cracks at the hole location. Cracks in the concrete will then tend to break down the bond between concrete and epoxy resin for resin bonded anchors or compromise the mechanical anchor system. Do not use post-installed anchors to resist earthquake forces within plastic hinge regions of bridge elements (i.e. columns). Use of post-installed anchors for shear lugs and beam seat extensions is acceptable.

RESIN BONDED ANCHORS

Do not specify anchors larger than 1 inch in diameter using a resin bonded anchor system.

Table 1.5.11.2.2-1 Anchor Diameters and Stress Areas

Diameter (inch)	Stress Area (in^2)	Bar Size	Stress Area (in2)
0.5	0.142	4	0.20
0.625	0.226	5	0.31
0.75	0.334	6	0.44
0.875	0.462	7	0.60
1.00	0.606	8	0.79

FHWA Technical Advisory T5140.34 regarding use of adhesive anchor under sustained tension loads was issued in January 2018. According to the Technical Advisory, FHWA recommends that post-installed adhesive anchors can be used for resisting sustained tension loads only when

specific requirements are met. The recommendations were based on *NCHRP Reports 639* and *757, ACI 318, and ACI 355.4*. Anchoring to concrete design guidance was added to *Section 5 of the AASHTO LRFD design specifications*, which refers to the *ACI design code*. The new requirements apply to all new Federal-aid projects.

Design resin bonded anchors with loads, load factors, and load combinations specified in *LRFD Section 3* and with resistance factors according to *ACI 318 Chapter 17* with the following modifications:

- Uncracked concrete is assumed for design of resin bonded anchor system.
- Use a resin bonded anchor system in concrete with a compressive strength of 2,500 – 8,000 psi.
- Use resistance factors as shown in *Table 1.5.11.2.2-2* for applicable failure modes.

Table 1.5.11.2.2-2 Strength Reduction Factor ϕ for Design of Resin Bonded Anchors

Failure Mode	Strength Limit State: Others	Strength Limit State: Sustained Tension	Extreme Event II Limit State
Reinforcement in tension	0.75	0.75	1.00
Concrete breakout in tension	0.65	0.65	0.90
Adhesive bond in tension	0.65	0.33	0.90
Reinforcement in shear	0.65	0.65	1.00
Concrete breakout in shear	0.70	0.70	0.90
Concrete pryout in shear	0.70	0.70	0.90

- For concrete breakout strength in tension, $kc = 24$. Use $\psi c, N = 1.0$.
- Use characteristic bond stresses from *Table 1.5.11.2.2-3* for bond strength calculation. The bond stresses shown in the table are obtained from the Evaluation Service Report (ESR) of epoxy resin products on the QPL. The reports are approved by the *International Code Council Evaluation Service, Inc. (ICC-ES)*. The recommended bond stresses are based on threaded rod or reinforcing bars installed in holes drilled with a hammer drill and carbide bit.

Table 1.5.11.2.2-3 Characteristic Bond Stress for Bond Strength in Tension Check

Short-Term Peak Temperature (24 hrs.)	Load Application	$\tau uncr$ (psi)
Equal or Less than 130 F	Others	1200
	Sustained Tension	480
Greater than 130 F but less than 176 F	Others	800
	Sustained Tension	320

Before 2019, ODOT used a set of equations developed in-house using historical test data for calculating anchor capacities. The equations are located in *Appendix A1.5.11.2.2*.

MECHANICAL ANCHORS

Each manufacturer establishes its own material strength and it differs for different anchor sizes. Do not specify anchor sizes larger than ¾ inch (nominal) in diameter. Use the nominal material strengths for anchor design found in *Table 1.5.11.2.2-4 and Table 1.5.11.2.2-5*.

Table 1.5.11.2.2-4 Material Properties for Design of Undercut Mechanical Anchors

Material (psi)	Stainless Steel	Hot-Dip Galvanized
Yield strength	87,000	Not on the QPL
Tensile strength	105,000	Not on the QPL

Table 1.5.11.2.2-5 Material Properties for Design of Expansion and Screw Mechanical Anchors

Material (psi)	Stainless Steel	Hot-Dip Galvanized
Yield strength	75,000	55,000
Tensile strength	90,000	75,000

Design mechanical anchors with loads, load factors, and load combinations specified in *LRFD Section 3* and with resistance factors according to *ACI 318 Chapter 17* with the following modifications:

- Assume uncracked concrete for design of mechanical anchor system.
- Use a mechanical anchor system in concrete with a compressive strength of 2,500 – 8,000 psi.
- Use resistance factors as shown in *Table 1.5.11.2.2-6* for applicable failure modes.

Table 1.5.11.2.2-6 Strength Reduction Resistance Factor ϕ for Design of Mechanical Anchors

Failure Mode	Strength Limit State	Extreme Event II Limit State
Reinforcement in tension	0.75	1.00
Concrete breakout in tension	0.65	0.90
Pullout strength in tension	0.65	0.90
Reinforcement in shear	0.65	1.00
Concrete breakout in shear	0.70	0.90
Concrete pryout in shear	0.70	0.90

For concrete breakout strength in tension, $kc = 24$. Use $\psi c, N = 1.0$.

Mechanical anchors from each manufacturer have different details and specifications. The recommended design parameters listed in *Table 1.5.11.2.2-7 thru Table 1.5.11.2.2-9* ensure that an anchor product on the QPL can meet the design requirements.

Table 1.5.11.2.2-7 Design Parameters for Undercut Mechanical Anchor Design

Nominal Diameter (inch)	0.375	0.500	0.625	0.750
Anchor O.D., d_a (inch)	0.625	0.750	1.000	1.125
Effective embedment depth, h_{ef} (inch)	3.75	4.75	7.25	9.75
Effective cross-sectional area, A_{se} (in^2)	0.078	0.131	0.226	0.334
Pullout strength, $N_{p,\ uncr}$ (lbs) Use $\psi c, P = 1.0$.	12600	16000	21000	31000
Hole depth (inch)	4.75	5.75	8.25	10.75
Minimum member thickness (inch)	7.25	8.0	10.75	14.0

Table 1.5.11.2.2-8 Design Parameters for Expansion Mechanical Anchor Design

Nominal Diameter (inch)	0.250	0.375	0.500	0.625	0.750
Anchor O.D., d_a (inch)	0.250	0.375	0.500	0.625	0.750
Effective embedment depth, h_{ef} (inch)	1.5	2.0	3.25	4.0	5.0
Effective cross-sectional area, A_{se} (in^2)	0.020	0.051	0.101	0.162	0.237
Pullout strength, $N_{p,\ uncr}$ (lbs) Use $\psi c, P = 1.0$	1600	3200	5400	7100	11600
Hole depth (inch)	2.75	3.25	4.50	5.25	6.25
Minimum member thickness (inch)	4.0	4.25	5.5	6.25	7.5

Table 1.5.11.2.2-9 Design Parameters for Screw Mechanical Anchor Design

Nominal Diameter (inch)	0.250	0.375	0.500	0.625	0.750
Anchor O.D., d_a (inch)	0.250	0.375	0.500	0.625	0.750
Effective embedment depth, h_{ef} (inch)	1.5	2.0	2.5	3.5	4.0
Effective cross-sectional area, A_{se} (in^2)	0.024	0.099	0.183	0.276	0.414
Pullout strength, $N_{p,\ uncr}$ (lbs) Use $\psi c, P = 1.0$	900	3000	3600	4800	9500
Hole depth (inch)	2.75	3.25	4.0	5.25	6.0
Minimum member thickness (inch)	4.0	4.25	5.0	6.25	7.0

Undercut anchors are good alternative to resin bonded anchors for overhead situation with sustained tension loading.

Overall design calculations for mechanical anchors are similar to resin bonded anchors without bond strength check. Adequate member thickness is important for mechanical anchors to avoid splitting failures. When the member thickness is limited and there is not much space for a longer anchor than the design, add a note on the plan.

For mechanical anchors, the difference between effective embedment depth of anchors (h_{ef}) and total drilled hole depth can vary from ½ to 2 inches depending on each manufacturer. Use the

hole depths shown in *Table 1.5.11.2.2-7* thru *Table 1.5.11.2.2-9* as a guide to ensure that anchors will fit inside the member and around existing rebar during the design.

When any of the above design requirements cannot be met, contact the BDM technical resource for guidance.

1.5.11.2.3 Drilling Holes in Concrete

When existing reinforcing steel is required by design, require bars to be located prior to drilling.

Drill holes according to manufacturer's recommendations. Spalling of adjacent concrete is the main concern when determining the hole location and type of drill to be used. When recommendations to prevent spalling do not exist, use the following drilling methods:

RESIN BONDED ANCHORS

When center of hole is 6 inches or less from the edge of concrete use either a diamond bit core drill or a carbide bit rotary hammer with four cutting edges on the diameter.

When center of hole is more than 6 inches from the edge of concrete use either an air hammer, maximum 9 pound class, or a carbide bit rotary hammer with two cutting edges on the diameter.

MECHANICAL ANCHORS

Use either a diamond bit core drill or a carbide bit rotary hammer with four cutting edges on the diameter.

GROUTED ANCHORS

Any type of drill will normally be acceptable. Grouted anchors should always be placed more than 6 inches from the nearest concrete edge.

1.5.11.2.4 Plan Details

Post-installed anchors are considered critical when failure of the anchors can compromise public safety. Anchors installed in the following members are considered critical.

- Items attached from tunnel ceiling or under bridge deck.
- Bridge rail anchor.
- Shear lug.
- Beam seat extension.
- Sign structure support.
- Structural connection between existing and new concrete or different materials.
- Fencing support.

List all anchors specified for a construction project in *SP 00535.45(c)*. Indicate which tests are required for the specified anchors, especially for critical anchors. For anchors that are not critical and will not see significant loads, tests during construction may be omitted.

RESIN BONDED ANCHORS

For horizontal applications, show drilled holes angled down a minimum of 15 degrees on plan sheets. For thin members, such as bridge decks, a smaller angle of drilled holes may be specified to avoid protrusion of the anchors. Specify a minimum drilled angle of 5 degrees. When any down angle of drilled holes will not work with the design, specify horizontal drilled holes. *SP 00535* will require a certified anchor installer to ensure good quality of installed resin bonded anchors.

When critical resin bonded anchors are used, include the following note on the plans:

> Furnish and install (___" diameter F1554 grade (36) (55) (105)) or (#__ AASHTO M 31, grade 60 rebar) resin bonded anchors with epoxy resin from the QPL. The characteristic bond stress used in the design is ____ psi. The minimum pullout strength is ___ lbs with a minimum embedment (h_{ef}) of _____ in. Install anchors according to the manufacturer's recommendations.

The minimum pullout strength is the smaller unfactored strength of steel reinforcement and adhesive bond in tension obtained from calculation for a single anchor including applicable eccentricity, edge effect, and splitting factors. For anchors subjected to sustained tension, the characteristic bond stress is reduced by a factor of 0.40 in design as shown in *Table 1.5.11.2.2-3*, however use the characteristic bond stress of 1,200 psi in calculation for the minimum pullout strength and show it on the plans for field testing purposes.

MECHANICAL ANCHORS

When critical mechanical anchors are specified, include the following note on the plans:

> Furnish and install ___" nominal diameter Type (A Undercut) (B Expansion) (C Screw) mechanical anchors using a product from the QPL. The minimum pullout strength is lbs with a minimum effective embedment depth (h_{ef}) of _____ in. (The maximum depth of the drilled hole is _____ in.) For the design, the steel anchor yield strength is _____ psi and the tensile strength is _____ psi. Furnish anchor materials with (Type 316 stainless steel) (hot-dip galvanized coating). Install anchors according to the manufacturer's recommendations.

The minimum pullout strength is the controlling factored strength for a single anchor including applicable eccentricity, edge effect, and splitting factors in tension using resistance factors for Extreme Event II Limit State.

1.5.11.2.5 Testing Requirements

Two types of field tests are required during construction to ensure proper installation and to achieve as-specified capacity of post-installed anchors. Demonstration Test includes installation of anchors using the same material and methods as shown on the plan and testing the anchors to a load at minimum pull out strength. Production Test is performed during construction on actual anchors used in final position up to a load level specified in the Special Provisions.

1.5.11.2.6 Construction

DRILL TYPES

See *BDM 1.5.11.2.3* or *SP 00535* for the drill type to be used.

HOLES

Holes for resin bonded anchors are normally 1/8 inch diameter larger than the nominal bolt diameter. Holes should be cleaned with compressed air, a non-metallic brush and water. Concrete dust is one of the most destructive elements to a resin bonded system and water is the best method to remove the dust. Holes for mechanical anchors are dependent on the type and manufacturer. Holes for grouted anchors are normally 1/4 inch diameter larger than the anchor diameter.

TEMPERATURE

Epoxy resin is not allowed for low temperature applications. The set times become quite long at low temperatures. It will normally be better to use a deeper embedment with a non-epoxy product at low temperatures.

TIGHTENING

SP 00535 requires tightening to only 1/6 turn past snug tight. Consider what tightening is appropriate for the application and show on the plans, when different than the specifications. Check when distribution plates are needed to transfer the bearing loads (from the tensioned bolt) to the concrete.

1.5.11.3 Cast-In Anchors

1.5.11.3.1 Generic Anchor Types

Design cast-in anchors, i.e. hex head bolt with washer, L-bolt, J-bolt, and welded headed stud, according to *Section 5 of the AASHTO LRFD design specifications*, which refers to the *ACI design code*. Use resistance factors as shown in *Table 1.5.11.3.1-1* for applicable failure modes.

Table 1.5.11.3.1-1 Strength Reduction Factors ϕ for Design of Cast-in Anchors

Failure Mode	Strength Limit State	Extreme Event II Limit State
Reinforcement in tension	0.75	1.00
Concrete breakout in tension	0.70	0.90
Reinforcement in shear	0.65	1.00
Concrete breakout in shear	0.70	0.90
Concrete pryout in shear	0.70	0.90

1.5.11.3.2 Coil Inserts

Specify inserts for rods or bolts with a minimum size of ¾ inch diameter.

Detail inserts to have a ½ inch setback under the concrete surface.

Use design parameters from *Table 1.5.11.3.2-1* for strength calculation and details for coil inserts.

C 1.5.11.3.2

The smallest coil inserts commercially available are ¾ inch diameter rods or bolts.

Table 1.5.11.3.2-1 Design Requirements for Single Concrete Inserts

Bolt or Rod Diameter (inches)	Min. Insert Length (inches)	Min. Member Thickness (inches)	Nominal Capacity in Tension or Shear (lbs)	Strength Reduction Factor: Strength	Strength Reduction Factor: Extreme Event	Min. Edge Distance (inches)	Min. Spacing to Adjacent Inserts (inches)*
¾	4 ½	8	16900	0.70	0.90	12	8
1	5 ½	8 ¾	24300	0.70	0.90	16	10
1 ¼	7 ½	10 ¾	38900	0.70	0.90	24	12
1 ½	9 ½	12 ¾	55500	0.70	0.90	26	16

Table notes:
*To avoid group effect.

Locate concrete inserts at a distance from the edge of deck according to the minimum edge distance shown in *Table 1.5.11.3.2-1*, otherwise an additional strength reduction factor applies.

When the inserts are not to be used immediately, install short, galvanized bolts in the inserts to prevent rusting of the threads.

The minimum member thickness from *Table 1.5.11.3.2-1* accounts for the insert length, setback, and concrete clear cover.

The nominal capacity from *Table 1.5.11.3.2-1* assumes concrete breakout strength controls.

1.5.12 Diaphragm Beams

1.5.12.1 End Diaphragm

For bridges in SDCs B, C, and D, specify full-depth reinforced concrete diaphragms cast in place at ends of each span to connect bridge deck and girders.

1.5.12.2 Intermediate Diaphragm

Use full depth reinforced concrete intermediate diaphragms.

Full-depth diaphragm: to facilitate construction, bottom of diaphragms may be located higher than connected girders no more than 6 inches.

Use the span length criteria in *Table 1.5.12.2-1* to determine the number of intermediate diaphragms for bridges crossing over major truck routes including Interstate 5, Interstate 84 and routes with 20 year projected ADTT greater than 5000; or waterways where there is a high probability of large debris or vessel collision.

C 1.5.12

C 1.5.12.1

The end diaphragm forms a deck-girder system to resist force effects during a seismic event. The end diaphragm at abutments can also be used for interacting with backfill to mobilize soil passive pressure.

C 1.5.12.2

Intermediate diaphragms distribute collision loads from over-height vehicles, vessels and large debris. Full depth diaphragms are more effective in distributing impact loads. Diaphragms can also be used for maintaining camber profile of precast prestressed decked members after differential camber adjustment and provide stability prior to deck concrete pour.

Table 1.512.2-1 Criteria to Determine Number of Intermediate Diaphragms

Span Length (feet)	Location of Diaphragm
40 – 80	Midspan
81 – 120	1/3 points
Greater than 120	1/4 points

For other bridges, provide at least one set of intermediate diaphragms at midspan. Stagger and place intermediate diaphragms perpendicular to girder centerline, when the skew is greater than 25 degrees. Steel diaphragms may be used. When a bolted connection is used to attach steel diaphragms to concrete girders, specify nut tightening past snug tight by no more than 1/6 turn to avoid unintended concrete damage.

Steel diaphragms may reduce construction time.

1.5.12.3 Temporary Diaphragm

C 1.5.12.3

In the contract plans, indicate the use of temporary diaphragm beams midway between the end and midspan diaphragm beams before pouring the end and midspan diaphragm beams (see *BR350*). Remove beams after deck overhang brackets are removed.

APPENDIX – Section 1.5.11.2.2 – Resin Anchor Design

This appendix contains the legacy equations used by designers prior to 2019, which were replaced by the equations according to *ACI 318 Chapter 17* with ODOT modifications. Use the legacy equations only when it is required.

GENERAL EQUATION FOR RESIN TENSION CAPACITY

Ultimate tension capacity = $R_0 * R_1 * R_2 * \pi * D * E * [U(max) - (35\ lb/in^3 * E)]$

where:

π = Pi = 3.14159
D = anchor diameter (inches)
E = anchor embedment (inches)
U(max) = 1400 psi for "low strength" resin
= 2300 psi for "high strength" resin

R_0 = reduction factor for non-redundant applications. This applies when anchors are used with only two anchors per attachment.
R_0 = 0.85 for non-redundant horizontal applications
R_0 = 1.0 for all other applications

R_1 = reduction factor due to edge distance
R_1 = 1.0 - (1.5-A)/2.5 when edge distance < 1.5 * E
R_1 = 1.0 when edge distance ≥ 1.5 * E
where A = edge distance/E

R_2 = reduction factor due to anchor spacing
R_2 = 1.0 - (1.0-B)/1.7 when anchor spacing < 1.0 * E
R_2 = 1.0 when anchor spacing ≥ 1.0 * E
where B = anchor spacing/E

Specify edge distance and anchor spacing greater than $6 * D$ or $0.5 * E$, whichever is greater.

When rebar is anchor material, add two times the anchor diameter to the required anchor embedment. This extra embedment is necessary for rebar since the exact location of rebar deformations cannot be known. Most of the tension load in a rebar anchor is transferred to the concrete at the deformation location. For this reason fully-threaded anchors are generally preferred for most resin-bonded anchor applications.

For horizontal applications, add 20 percent to the required anchor embedment. This extra embedment is necessary since full resin coverage cannot be assured for horizontal applications.

Horizontal applications angled down a minimum of 15 degrees do not require the additional 20 percent.

RESIN TENSION EQUATION: SERVICE LOADS

Ultimate tension capacity ≥ 3 * design tension load

RESIN TENSION EQUATION: SEISMIC LOADS

Ultimate tension capacity:

≥1.9 * design seismic load for "low strength" resin

≥1.6 * design seismic load for "high strength" resin

Note: for seismic loading, maximum rod loading ≤ 0.9 Fy

Fy = Yield strength of the anchor rod

RESIN TENSION EQUATION: LRFD LOADS

0.5 * Ultimate tension capacity ≥ factored load

GENERAL EQUATION FOR RESIN SHEAR CAPACITY

Ultimate Shear Capacity = $R1 * R2 * \lambda * D * E * f'c$ where:

D = anchor diameter (inches)
E = anchor embedment (inches)
f'c = compressive strength of concrete
λ = 0.75 for "low strength" resin
λ = 1.0 for "high strength" resin
R_1 = reduction factor due to edge distance
- $R1 = 1.0 - (1.5 - A)/2.0$ when edge distance < 1.5 * E
- $R1 = 1.0$ when edge distance ≥ 1.5 * E

R_2 = reduction factor due to anchor spacing
- $R2 = 1.0 - (1.0 - B)/1.7$ when anchor spacing < 1.0 * E
- $R2 = 1.0$ when anchor spacing ≥ 1.0 * E

If concrete for an existing structure appears to be in good condition, use f'c = 1.2 times the concrete strength shown on the existing plans.

RESIN SHEAR EQUATION: SERVICE LOADS

Ultimate shear capacity ≥ 3 * design shear load

RESIN SHEAR EQUATION: SEISMIC LOADS

Ultimate shear capacity ≥ 1.7 * design seismic shear load

RESIN SHEAR EQUATION: LRFD LOADS

0.5 * Ultimate shear capacity ≥ factored load

COMBINED RESIN TENSION AND SHEAR

Combined Stress Ratio (CSR) ≤ 1.0

$CSR = (ft / Ft) + (fv / Fv)2$

f_t , f_v = factored loads (i.e., the right side of service load, seismic, or LRFD equations)

F_t , F_v = capacities (i.e., the left side of service load, seismic, or LRFD equations)