

# Chapter 5 — Concrete

## 5.1—OVERVIEW

This chapter applies to the design of both cast-in-place and precast concrete structures, both traditionally reinforced as well as prestressed.

## 5.2—MATERIALS

### 5.2.1—Concrete

Standard concrete classes and compressive strengths are summarized in Table 5.1.

Table 5.1—Standard Concrete Classes and Compressive Strengths

Structural Element	Concrete Class	Minimum 28 day Compressive Strength
Abutments, Grade Beams, Piers, Bents, and Footings	47B	3.0 ksi
Drilled Shafts	47B	4.0 ksi
Bridge Decks and Rails	47BD	4.0 ksi
CIP Concrete Box Culverts	47B	4.0 ksi
Precast Concrete Box Culverts	See Special Provisions	5.0 ksi
Prestressed Concrete Girders	See Standard Specifications	See §5.2.1.1.3

See following sections for additional details.

#### 5.2.1.1—Strength of Concrete

Specification of  $f'_c$  shall be in 0.1 ksi increments for precast elements and 0.5 ksi increments for cast-in-place applications.

##### 5.2.1.1.1—CIP Concrete Superstructures

Since conditions for placing and curing concrete for CIP components are not as controlled as they are for precast bridge components, 4.0 ksi concrete is typically used. Where significant economy can be gained or structural requirements dictate, 5.0 ksi concrete may be used with the approval of the Bridge Design Section.

##### 5.2.1.1.2—CIP Concrete Substructures

3.0 ksi for abutments, grade beams, bents, piers, and footings

4.0 ksi for drilled shafts

Where significant economy can be gained or structural requirements dictate, up to 5.0 ksi concrete may be used with the approval of the Bridge Division.

### 5.2.1.1.3—Prestressed Concrete Girders

Nominal 28-day concrete strength,  $f'_c$ , for prestressed concrete girders is 8.0 ksi (with concrete strength at release,  $f'_{cr}$ , of 6.0 ksi). Where higher strengths are required,  $f'_c$  of 10.0 ksi can be specified (with concrete strength at release,  $f'_{cr}$ , of 7.5 ksi). Variance from these two strengths requires Bridge Division approval. The specified concrete strengths for prestressed concrete girders shall be shown in the plans on the girder data sheet. Designers should specify the lowest concrete strengths (8.0 or 10.0 ksi) necessary to meet allowable stress limits, strength requirements, and these provisions.

### 5.2.1.1.4—CIP Concrete Box Culverts

4.0 ksi concrete will be used for the design of barrels, headwalls, and wingwalls.

### 5.2.1.1.5—Precast Concrete Box Culverts

5.0 ksi concrete will be used for the design of barrels

## 5.2.1.2—Classes of Concrete

See Standard Specifications for specific mix design.

Class 47B: Used in CIP bridge substructure and culverts.

Class 47BD: Used in CIP bridge superstructure and concrete box culverts when the top slab is used as the driving surface.

For prestressed concrete girders, the producers are responsible for the mix design, which means there is no concrete class associated with this material.

For precast concrete box culverts, the producers are responsible for the mix design, which means there is no concrete class associated with this material. See "PRECAST CONCRETE BOX CULVERT" Special Provision for more details.

## 5.2.1.3—Unit Weight

Unreinforced concrete unit weight shall be calculated based on the specified minimum compressive strength and the formula shown in Table 3.5.1-1 of the LRFDBDS.

For plant produced precast girders 0.01 kcf shall be added to the unreinforced concrete density to account for the weight of reinforcement.

For other mildly reinforced concrete 0.005 kcf shall be added to the unreinforced concrete density to account for the weight of reinforcement.

## 5.2.1.4—Modulus of Elasticity

The modulus of elasticity shall be determined as specified in Chapter 5 of the LRFDBDS. The correction factor  $K_1$  shall be taken as 0.975.

The concrete strength limits have been chosen to minimize costs associated with high strength mixes and extended curing times. Designers should note that higher release strengths may require extended cure times in addition to providing less predictable camber and strength values. The  $0.75f'_c$  release strength limit is based on industry recommendations from PCAN as well as information from PCI. The intent is to have a practical ratio between  $f'_{cr}$  and  $f'_c$  and to provide more a more reliable prediction of final compressive strength.

Where strand patterns vary significantly along the length of a structure, using differing concrete strengths span to span should be investigated.

Designers should also note that the strength design at release method, as described in Hanna et al. (2010), has been found to create cracking in the girders and is therefore not allowed anymore.

In NCHRP Report 496 the authors calibrated  $K_1$  based on aggregates from four different states. For Nebraska aggregates,  $K_1$  was found to be 0.975 (Tadros et al., 2003a).

### 5.2.1.5—Shrinkage and Creep

Losses due to shrinkage and creep shall be calculated in accordance with the refined method provided in Chapter 5 of the LRFDBDS. The relative humidity,  $H$  may be taken as 70% for standard conditions. The age at time of deck placement,  $t_d$ , may be taken as 56 days. The maturity of concrete,  $t$ , may be taken as 27,000 days for standard conditions. In determining the maturity of concrete at initial loading,  $t_i$ , one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.

### 5.2.1.6—Mass Concrete

Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement to minimize cracking. Temperature related cracking may be experienced in thick-section concrete structures, including spread footings, pile caps, bridge piers, thick walls, and other structures as applicable. Concrete placements with the least dimension greater than 5 feet shall be considered mass concrete. Drilled shafts need not be considered mass concrete. Designers should be aware of the cost of monitoring these temperature effects and try to keep the least concrete dimension below these limits. Bridges with components that meet these criteria shall include the Special Provision "MASS CONCRETE PLACEMENT" in the contract documents.

### 5.2.1.7—Self-Consolidating Concrete

Self-consolidating concrete may be used in structural members such as precast prestressed concrete girders as described in the Standard Specifications.

SCC may be specified for cast-in-place applications where the use of conventional concrete could be challenging and problematic. Examples are where new concrete is being cast up against an existing soffit, or in members with very dense/congested reinforcing steel. See "SELF-CONSOLIDATING CONCRETE" Special Provision for more details.

### 5.2.1.8—Ultra-High Performance Concrete

Design shall be performed based on *Guide Specifications for Structural Design with Ultra-High Performance Concrete* (AASHTO, 2024).

Possible applications of UHPC on projects can be discussed with Bridge Division.

Designers shall specify NDOT npUHPC for precast, CIP, or overlay applications. Proprietary mix alternates will be allowed with Bridge Division approval.

70% humidity was chosen as girders are produced on the east end of the state. 27,000 days is based on a 75 year design life for the structure.

Since the early 2000s Nebraska's prestressed concrete girders have been produced almost exclusively with self-consolidating concrete (Lafferty, 2008). Any concrete mixtures for prestressed girders are qualified by NDOT Materials And Research Division. Refer to Section 705 of the Standard Specifications for Mix Design Approval and Changes.

Some examples of work where UHPC is a good material candidate are:

- Joint headers (curing duration needs to be considered for joint headers)
- Link slabs
- Girder end repairs
- Precast connections
- Bearing pedestal retrofits
- Column Encasements

Non-proprietary UHPC mixtures from local material sources have been developed by NDOT in conjunction with UNL. These mixes greatly reduce the cost of UHPC when compared to proprietary mixes, for more information, see Hu et al. (2023) and Morcoux et al. (2023). Research is ongoing for overlay application.

### 5.2.2—Mild Reinforcement

Reinforcing bars shall be deformed and conform to the Standard Specifications. Standard reinforcement for CIP concrete shall be mild steel with a yield strength of 60 ksi produced in accordance with ASTM A615. Higher yield strength material may be used with the approval of the Bridge Division, where justified.

All steel reinforcement used in drilled shafts, prestressed girders, and concrete pile shall be uncoated. Box culverts shall have epoxy coated reinforcing bars when the top surface is used as a driving surface, otherwise they shall use uncoated steel. All other steel reinforcing bars shall be epoxy coated.

Deformed welded wire reinforcement (WWR) used in prestressed concrete girders shall have a minimum yield strength of 70 ksi and be produced in accordance with ASTM A1064 or AASHTO M336. ASTM A615 Gr. 60 reinforcing substitution may be permitted when requested by the fabricator on a per case basis.

Smooth welded wire reinforcement used in precast concrete culverts and concrete slope protection shall have a minimum yield strength of 65 ksi and be produced in accordance with ASTM A1064 or AASHTO M 336

### 5.2.3—Prestressing Strands

Prestressing strands shall be ASTM A416 uncoated, seven-wire low-relaxation steel with an ultimate strength of 270 ksi. Diameters of strand depend on the type of prestressed girder being designed. See §§5.5.2 and 5.5.4 for more information.

Stress relieved strands are not permitted.

### 5.3—CLEAR COVER

The clear cover for reinforcement in CIP concrete is mostly covered by standard note #046, cases not covered by that note are listed below and shall be detailed on the plans as such.

- CIP Slab Bridge Superstructures and Decks
  - Bottom — 1 in. (+ $\frac{1}{4}$  in./-0 in.)
  - Top — 2  $\frac{1}{2}$  in. ( $\pm\frac{1}{4}$  in.)
  - Sides — 3 in.
- CIP Approach Slabs
  - Top — 2  $\frac{1}{2}$  in. (+ $\frac{1}{4}$  in./-0 in.)
  - Bottom and Sides — 3 in.
- Reinforcement projecting from slabs/decks into rails shall be sized for 3 in. clear cover from the top of the rail.
- Drilled Shaft faces cast against soil or rock
  - 3 in. for shafts diameters not greater than 5 ft.
  - 6 in. for shafts diameters 5 ft. or greater

All plan sets containing CIP concrete work shall include standard note #046.

For precast concrete clear covers see the base sheets for each girder template.

High strength reinforcement requires special provisions added to the project.

Preferred wire sizes of WWR are listed in the sections for each girder shape. This does not preclude the use of other sizes, but verification should be done to ensure fabricators can procure desired wire sizes in a timely and cost effective manner. Fabricators may desire to use a larger cross wire than specified in the base sheets in some cases due to ease of fabrication, this is acceptable.

When determining hat bar size in concrete haunches, the bottom of deck clear cover may be utilized at the haunch to girder interface.

## 5.4—CAST-IN-PLACE CONCRETE

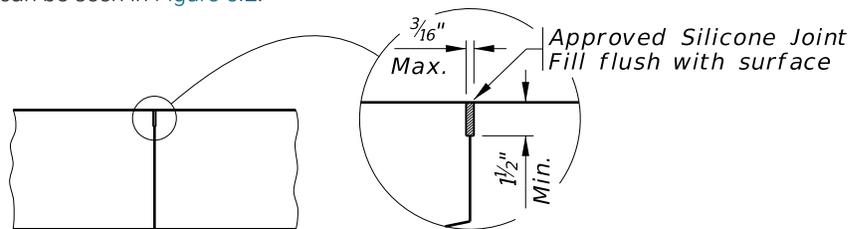
### 5.4.1—General Information

Cast-in-place concrete is currently the predominant material used in Nebraska for substructures and decks. This section provides information and guidance on detailing and design practices.

### 5.4.2—Construction Joints

Slab construction joints less than or equal to 8 1/2 in. thick shall be indicated as a vertical line in the plans. A cell is available in the bridge deck library and can be seen in Figure 5.1.

Slab construction joints greater than 8 1/2 in. thick shall provide a shear key. A cell is available in the bridge deck library and can be seen in Figure 5.2.



**NOTES:**

*The Contractor shall prepare and seal the joint according to the manufacturer's recommendations.*

*Before sealing, the joint wall surfaces shall be sandblasted to remove any loose particles and concrete dust.*

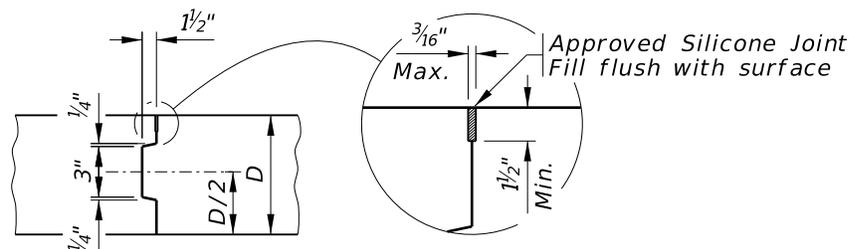
*After sandblasting, the entire joint shall be cleaned with compressed air having a minimum pressure of 90 psi. The compressed air shall be free of any contaminants.*

*The joint shall be dry at the time of sealing.*

#### SLAB CONSTRUCTION JOINT

*Not to Scale*

Figure 5.1—Slab Construction Joint for Slab Thickness ≤ 8.5 in.  
Available as Cell "JOINT"



**NOTES:**

*The Contractor shall prepare and seal the joint according to the manufacturer's recommendations.*

*Before sealing, the joint wall surfaces shall be sandblasted to remove any loose particles and concrete dust.*

*After sandblasting, the entire joint shall be cleaned with compressed air having a minimum pressure of 90 psi. The compressed air shall be free of any contaminants.*

*The joint shall be dry at the time of sealing.*

#### SLAB CONSTRUCTION JOINT

*Not to Scale*

Figure 5.2—Slab Construction Joint for Slab Thickness > 8.5 in.  
Available as Cell "JOINT2"

All joints listed in §§5.4.2.1 and 5.4.2.2 shall be detailed on the plans and listed as mandatory or optional as necessary.

All construction joints shown in the plans are mandatory unless explicitly stated as optional.

#### 5.4.2.1—Mandatory Joints

The following is a list of locations where mandatory construction joints are required and shall be detailed in the plans:

- Horizontally:
  - Semi-integral abutments: In the deck turndown at the abutment, horizontally between the elevation of the approach slab seat and the bottom of the chamfer of the deck.
  - Concrete diaphragms at intermediate substructure elements: at  $\frac{2}{3}$  of the girder height
  - Intermediate concrete diaphragms: Top of diaphragm (bottom of deck)
  - Interior median curbs: Top of deck (bottom of curb)
  - Permanent casing on drilled shaft: At the top
  - Box culvert walls: At the top of the slab (bottom of walls)
- Vertically:
  - End of floor to approach slab. May be replaced with optional non-corroding plate to force bond break if poured continuously
  - Longitudinally from End of Paving to End of Paving for phased construction.

The first two joints are to provide girders necessary lateral restraint prior to deck placement.

Joint at EOF is to allow some rotation of the approach slab seat

#### 5.4.2.2—Optional Joints

The following list of locations where optional construction joints are to be detailed in the plans:

- Horizontally:
  - Box culvert walls: 2 in. below the bottom of the top slab
  - Box culvert turndowns (barrel and wing footings): at the top, or slightly below the top, of the turndown (bottom of bottom slab elevation)
- Vertically:
  - Transverse deck joints: per pouring diagram and parallel to the supports

This limits the allowable locations of the transverse joints in the slab if the Contractor needs to stop the pour for any reason, or if skip pouring is used. Refer to §9.2.5 for more details.

### 5.4.3—Concrete Slab Bridges

Concrete slab bridges are a very long lasting and robust structural system, they often last long enough that they are replaced due to functional obsolescence rather than structural deficiency. For the lengths of bridges shown in Table 5.2 it is common for a three-span slab bridge to be approximately 6 in. shallower than an equivalent three-span prestressed concrete girder bridge.

Refer to §9.2.8 for the concrete finish to be shown on the plans.

Designers should note that concrete slab spans over 50 ft. require the shoring to be designed by a Professional Engineer. Therefore, using spans of 50 ft. and shorter is preferred in order to reduce costs. Slab bridges with span lengths and counts varying from the tables shown herein are acceptable, they just required additional work on the part of the designer. The previous BOPP manual contained information for structure lengths down to 40 ft. A three span slab bridge shorter than 60 ft is rarely economically justified and an alternative structural system should be chosen.

### 5.4.3.1—Slab Bridge Design Assumptions

The slab bridge design information contained in Table 5.2, Table 5.3, and Table 5.4 may be used to provide steel reinforcement for the bridge lengths and spans specified. The table can also be used for bridges of the same length and span with lesser width or loads. These tables were developed in accordance with the LRFDBDS, 9th Ed. The following criteria were assumed in developing the slab design table:

- Superimposed Dead Loads
    - 42 in. NU Open Concrete Rail = 441 plf (DC)
    - Future Surfacing = 35 psf (DW).
  - HL93 Live Load without interstate or expressway traffic. See §3.2 for guidance on load combinations.
  - Edge beam distribution factor is used across the entire width of the bridge.
  - The effective design depth determined by assuming 1/2 in. for a sacrificial wearing surface.
  - $f'_c = 4$  ksi concrete
  - $f_y = 60$  ksi reinforcement, epoxy coated
  - Design Clear Roadway Width = 44 ft.
  - Concrete clear cover per §5.3.
  - Skew Angle: 0°
- 
- Haunch Depth: 6 in. @ abutments.
  - J4 bars are continuous through Span 2 for bridge lengths 60 and 65 ft.

The pairs of design tandems required in §3.4.1 were not used in the slab bridge analysis.

Skews decrease the magnitude of the applied moment so the most conservative case was used.

### 5.4.3.2—Concrete Slab Base Sheet

The Concrete Slab Base Sheets provides typical reinforcement layouts for zero,  $\leq 30^\circ$  and  $> 30^\circ$  skewed bridges. The base sheets are to be coordinated with Table 5.2, Table 5.3, and Table 5.4. Designers must replace the reinforcement designations shown on the base sheet (J2 through J8) with standard bar marks and the haunch information from §5.4.3.3.

Transverse distribution steel shall be as listed here and as shown on the base sheet:

- Top reinforcement shall be #4 bars at 12 in. centers.
- Bottom reinforcement shall be #5 bars at 12 in. centers.
- For skews not greater than 30°, place transverse distribution steel parallel to the CL of the substructure elements.
- For skews greater than 30°, place transverse distribution steel perpendicular to the CL of roadway in the following manner:
  - Transverse top and bottom reinforcement will require bar sets only at the End of Floor.
  - Additional longitudinal steel in the bottom layer (#4 bars @ 24 in. centers) will extend through the haunch to allow the transverse steel in the bottom layer to be placed continuously over the haunch without bar sets at the intermediate supports.

Table 5.2 shall be used to determine individual span lengths, slab thickness at midspan, and the location of the optional construction joint each side of the intermediate supports based on overall bridge length.

Table 5.2—CSB Dimensions

Bridge Length	Span No. 1 & 3	Span No. 2	Slab Thickness	X
60'	18'-0"	24'-0"	11 1/2"	6'-3"
65'	19'-6"	26'-0"	11 1/2"	6'-8"
70'	21'-0"	28'-0"	12"	7'-1"
75'	22'-6"	30'-0"	12 1/2"	7'-6"
80'	24'-0"	32'-0"	13"	7'-11"
85'	25'-6"	34'-0"	13 1/2"	8'-3"
90'	27'-0"	36'-0"	14"	8'-8"
95'	28'-6"	38'-0"	14 1/2"	9'-6"
100'	30'-0"	40'-0"	15"	10'-0"
105'	31'-6"	42'-0"	15 1/2"	10'-4"
110'	33'-0"	44'-0"	16"	11'-0"
115'	34'-6"	46'-0"	16 1/2"	11'-4"
120'	36'-0"	48'-0"	17"	11'-9"
125'	37'-6"	50'-0"	17 1/2"	12'-2"
130'	39'-0"	52'-0"	18"	12'-6"
135'	40'-6"	54'-0"	19"	12'-10"
140'	42'-0"	56'-0"	19 1/2"	13'-2"

Table 5.3 shall be used to determine reactions for substructure design based on overall bridge length.

Table 5.3—CSB Reactions at Abutments and Intermediate Supports\*

Table 5.3a—Service I Reactions (Unfactored)

Bridge Length	DC Reaction k/ft. of width		DW Reaction k/ft. of width		LL Reaction kips per lane No Dynamic Allowance	
	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports
60'	1.08	4.44	0.24	0.87	47.47	74.21
65'	1.17	4.77	0.26	0.94	48.47	77.46
70'	1.30	5.27	0.28	1.01	49.42	80.35
75'	1.44	5.80	0.30	1.08	50.29	82.97
80'	1.59	6.34	0.32	1.15	51.11	85.44
85'	1.75	6.92	0.34	1.22	51.88	87.68
90'	1.84	8.02	0.34	1.30	52.38	91.22
95'	2.01	8.64	0.36	1.37	53.30	93.14
100'	2.19	9.29	0.38	1.44	54.92	94.98
105'	2.37	9.97	0.40	1.51	56.44	96.77
110'	2.55	10.66	0.42	1.58	57.88	98.49
115'	2.75	11.39	0.44	1.65	59.24	100.16
120'	2.95	12.13	0.46	1.72	60.56	101.82
125'	3.16	12.90	0.48	1.79	61.84	103.43
130'	3.37	13.70	0.50	1.86	63.06	105.00
135'	3.68	14.85	0.52	1.93	64.24	106.48
140'	3.91	15.71	0.54	2.00	65.37	107.99

Table 5.3b—Strength I Reactions (Factored)

Bridge Length	DC Reaction k/ft. of width		DW Reaction k/ft. of width		LL Reaction† kips per lane No Dynamic Allowance		LL Reaction† kips per lane With Dynamic Allowance	
	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports
60'	1.35	5.55	0.36	1.30	83.08	129.86	107.52	163.19
65'	1.46	5.96	0.39	1.40	84.83	135.55	109.60	169.98
70'	1.63	6.59	0.41	1.51	86.48	140.61	111.55	175.95
75'	1.81	7.24	0.44	1.62	88.01	145.19	113.33	181.28
80'	1.99	7.93	0.47	1.72	89.44	149.52	114.98	186.29
85'	2.18	8.65	0.50	1.83	90.78	153.44	116.50	190.75
90'	2.30	10.02	0.52	1.95	91.67	159.64	117.46	197.98
95'	2.52	10.80	0.54	2.06	93.28	163.00	119.35	201.69
100'	2.73	11.62	0.57	2.16	96.12	166.22	122.87	205.21
105'	2.96	12.46	0.60	2.27	98.78	169.35	126.16	208.62
110'	3.19	13.33	0.63	2.37	101.28	172.35	129.24	211.86
115'	3.44	14.23	0.66	2.48	103.66	175.28	132.14	215.01
120'	3.69	15.17	0.69	2.58	105.98	178.18	134.96	218.10
125'	3.95	16.13	0.72	2.69	108.23	181.00	137.71	221.10
130'	4.21	17.12	0.75	2.79	110.36	183.75	140.30	224.00
135'	4.60	18.56	0.78	2.90	112.43	186.33	142.78	226.70
140'	4.89	19.63	0.81	3.00	114.40	188.97	145.15	229.44

\* Reaction values given are for the bridge only and do not include any contribution from approach slabs.

† These reactions are intended for use in substructure design therefore they utilize 1.75 for  $\gamma_{LL}$  when factoring.

Table 5.4 shall be used to determine longitudinal reinforcement size and spacing based on overall bridge length.

Table 5.4—CSB Reinforcement

Table 5.4a—Top of Slab - Negative Steel Reinforcement

Bridge Length	At Intermediate Supports									Span 1 & 3	Span 2
	Longitudinal		J5 Bars			J4 Bars			S510	S405	
	Size	Spacing	Length	L1	L3	Length	L2	L4	Length	Length	
60'	#7	11"	17'-9"	9'-4"	8'-5"	51'-10"*	13'-11"		8'-7"		
65'	#7	11"	18'-8"	9'-4"	9'-4"	55'-4"*	14'-8"		9'-7"		
70'	#7	11"	19'-8"	9'-10"	9'-10"	31'-2"	15'-7"	15'-7"	9'-10"	8'-8"	
75'	#7	11"	21'-0"	10'-9"	10'-3"	32'-2"	16'-2"	16'-0"	20'-9"	8'-8"	
80'	#7	10"	21'-9"	11'-11"	9'-10"	32'-10"	16'-11"	15'-11"	11'-3"	8'-2"	
85'	#7	10"	22'-7"	12'-4"	10'-3"	34'-0"	17'-8"	16'-4"	13'-0"	7'-9"	
90'	#7	10"	24'-1"	11'-11"	12'-2"	37'-1"	18'-8"	18'-5"	12'-3"	6'-9"	
95'	#7	10"	26'-7"	14'-0"	12'-7"	38'-6"	19'-11"	18'-7"	13'-2"	6'-1"	
100'	#8	12"	26'-8"	13'-10"	12'-10"	41'-6"	21'-1"	20'-5"	12'-7"	6'-11"	
105'	#8	11"	27'-9"	14'-8"	13'-1"	42'-9"	21'-11"	20'-10"	14'-0"	7'-8"	
110'	#8	11"	29'-6"	15'-10"	13'-8"	44'-6"	23'-2"	21'-4"	15'-3"	7'-11"	
115'	#8	10"	29'-8"	16'-4"	13'-4"	45'-2"	23'-9"	21'-5"	14'-9"	9'-1"	
120'	#8	10"	31'-1"	16'-11"	14'-2"	46'-7"	24'-9"	21'-10"	16'-1"	12'-2"	
125'	#9	11"	35'-7"	19'-8"	15'-11"	50'-8"	27'-0"	23'-8"	17'-4"	12'-5"	
130'	#9	10"	28'-6"	14'-3"	14'-3"	48'-6"	24'-10"	23'-8"	18'-6"	13'-2"	
135'	#9	10"	29'-2"	14'-7"	14'-7"	48'-11"	25'-1"	23'-10"	20'-11"	13'-5"	
140'	#9	10"	30'-3"	15'-4"	14'-11"	49'-9"	25'-7"	24'-2"	22'-4"	14'-2"	

Table 5.4b—Bottom of Slab - Positive Steel Reinforcement

Bridge Length	Longitudinal		Span 1 & 3		Span 2		Haunches
	Size	Spacing	J3 Bars Length	J2 Bars Length	J6 Bars Length	J7 Bars Length	J8 Bars Size
	60'	#7	11"	18'-4"	13'-5"	23'-10"	14'-2"
65'	#8	12"	20'-4"	14'-4"	26'-10"	14'-10"	#6
70'	#8	12"	21'-9"	15'-9"	28'-6"	16'-6"	#6
75'	#8	11"	22'-8"	16'-8"	29'-4"	17'-4"	#6
80'	#8	11"	24'-0"	18'-0"	31'-0"	19'-0"	#6
85'	#8	10"	24'-11"	18'-11"	31'-8"	19'-8"	#6
90'	#8	10"	25'-6"	19'-6"	31'-9"	19'-9"	#6
95'	#8	10"	26'-10"	20'-10"	33'-3"	21'-3"	#6
100'	#9	12"	29'-2"	21'-10"	36'-11"	22'-3"	#7
105'	#9	12"	30'-5"	23'-1"	38'-6"	23'-10"	#7
110'	#9	11"	31'-2"	23'-10"	39'-1"	24'-5"	#7
115'	#9	11"	32'-5"	25'-1"	40'-8"	25'-11"	#7
120'	#9	10"	33'-0"	25'-8"	41'-1"	26'-6"	#7
125'	#9	11"	34'-11"	27'-8"	43'-8"	29'-0"	#7
130'	#9	10"	35'-6"	28'-2"	44'-0"	29'-4"	#7
135'	#9	10"	36'-7"	29'-3"	45'-3"	30'-7"	#7
140'	#9	10"	37'-10"	30'-6"	46'-10"	32'-2"	#7

\* Bar is continuous through Span 2

### 5.4.3.3—Haunch Detail Over Intermediate Supports

#### 5.4.3.3.1—Bridge Lengths of 60 ft. to 85 ft.

- Haunch Depth = 6 in.
- Haunch Length = 4 ft.
- Place five (5) S501 bars at the face of each haunch.

Table 5.5—4 ft. Haunch Bill of Bars

Location	Mark	No.	Length	Type	A	B	C	D	E
J8		Varies	14'-0"	106	6'-0"	2'-0"	9"	9"	6'-0"
Haunch	S404	8	14'-0"	STR					

#### 5.4.3.3.2—Bridge Lengths of 90 ft. to 140 ft.

- Haunch Depth = 9 in.
- Haunch Length = 6 ft.
- Place seven (7) S501 bars at the face of each haunch.

Table 5.6—6 ft. Haunch Bill of Bars

Location	Mark	No.	Length	Type	A	B	C	D	E
J8		Varies	18'-0"	106	8'-0"	2'-0"	1'-0"	1'-0"	8'-0"
Haunch	S404	8	18'-0"	STR					

## 5.5—PRECAST PRESTRESSED CONCRETE GIRDERS

### 5.5.1—General Information

#### 5.5.1.1—Girder Templates

NU or IT girders shall be the preferred shapes. For widenings, designers shall check deflections when widening with a different shape. The use of AASHTO Sections requires approval from Bridge Division.

The precast concrete girder fabricators in the State of Nebraska have forms that match the dimensions of the NU and IT girders described later in this chapter. Due to increased fabrication cost, transportation cost, or both stemming from using different shapes, NU and IT shapes are the most cost-effective sections for Bridge Division.

#### 5.5.1.2—Composite Section Properties

Haunch depth shall be ignored when determining composite section properties during design.

### 5.5.1.3—Service III Load Combination

Stress analysis based on transformed section properties is discouraged and should be avoided.

Using approximate prestress losses is permitted for preliminary design only. Refined analysis for prestress losses shall be used for final design.

The live load factor for the Service III Load Combination shall be:

- $\gamma_{LL} = 1.0$  when one or more of the following are true:
  - transformed section properties are utilized for stress analysis
  - elastic gains are taken into account for stress analysis
  - specified concrete compressive strength is greater than 10 ksi
- $\gamma_{LL} = 0.8$  when none of the above limitations apply

Tensile stresses in the bottom of the girder at the maximum positive moment location shall not exceed  $0.095Vf'_c$  nor 0.3 ksi without Bridge Division approval.

### 5.5.1.4—Strands

Strands at the ends of girders that will not be encased in a concrete diaphragm or turndown shall be recessed, burnt back, and patched with cementitious grout. Notes to the fabricator shall be added to the girder data sheet detailing this condition.

Strands that are extended into the concrete diaphragm shall not be debonded.

The standardized position of the strands in the each girder template is given in their respective sections.

#### 5.5.1.4.1—Extended Strands

Restraint moments (LRFDBDS-9, Article 5.12.3.3.2) do not need to be calculated if prestressing strands are extended and anchored into the turndown/diaphragm following the base sheet details.

Extended strands shall be projected 6 in. beyond the end of the girder and bent using two 45° bends. See base sheets for details.

#### 5.5.1.4.2—Deflected Strands

The strands in Inverted Tee girders cannot be deflected. The strands in NU girders may be deflected see §5.5.2.2 for more information.

The 2016 Interim Revisions to the LRFDBDS and the AASHTO Manual for Bridge Evaluation have made the Service III load factor for live load dependent upon the method by which prestress losses are estimated. The purpose of this change was to maintain a uniform level of reliability against cracking of prestressed concrete components as explained in LRFDBDS C3.4.1.

The conservative nature of the design requirements defined in the BDM provide sufficient protection against cracking and adequate long-term performance of prestressed concrete superstructure components.

Designers should note that when changing from approximate to refined analysis of prestress losses the Precast/Prestressed Girder module of LEAP Bridge Concrete typically overwrites the  $\gamma_{LL}$  load factor for the Service III load combination to 1.0.

As Nebraska is a state with significant road salt application in the winter, tensile stresses in girders should be controlled to prevent corrosion and increase service life of prestressed structures. There is also concern of service limit states and their calibration to a uniform probability of exceedance (National Academies of Sciences, Engineering, and Medicine et al., 2014). Therefore Bridge Division has chosen to limit service limit state stresses until further research is completed.

The 6 in. strand extension is in violation of LRFDBDS Article 5.12.3.3.9c. This number is used based on real world observations, behavior, performance and surrounding state DOTs policies (Tadros & Jongpitakssee, 2003).

#### 5.5.1.4.3—Partially Debonded strands

Where it is necessary to prevent a strand from actively supplying prestress force near the end of a girder, it shall be debonded. This can be accomplished by taping a close-fitting tube to the stressed strand from the end of the girder to some point where the strand can be allowed to develop its load. Debonding limits and strands shall be carefully indicated on the plans according to the design standard sheet. Permitted debonding locations can be seen in the section for each girder template.

#### 5.5.1.4.4—Temporary Strands

Temporary strands in the top flanges of prestressed concrete girders may be required for shipping at the discretion of the precaster and the consent of the design engineer. These strands may be pretensioned and bonded only for the end 10 feet of the girder or may be post-tensioned prior to lifting the girder from the form. These strands can be considered in design to reduce the required transfer strength and to provide stability during shipping. These strands must be cut before the CIP end diaphragms are placed or a maximum of 10 days after stressing, whichever comes first. It is the responsibility of the precaster to state on the shop plans the method and sequence of post-tensioning, and the method and sequence of cutting the temporary strands.

### 5.5.1.5—Non-Prestressed Reinforcement

#### 5.5.1.5.1—Transverse and End Zone Reinforcement

Vertical web shear reinforcement shall be uncoated steel welded wire reinforcement, deformed for concrete, as shown on the base sheets for each girder template. ASTM A615 Gr. 60 reinforcing substitution may be permitted when requested by the fabricator.

The area of the longitudinal wire shall be at least 40% of the area of the transverse wire.

It is important when this method is used in construction that the taping of the tube is done in such a manner that concrete cannot leak into the tube and provide an undesirable bond of the strand

The designer shall verify  $A_v f_y / s$  of proposed steel is not less than the reinforcement necessary per design with the reduced yield strength of rebar compared to WWR.

Where roadway profile and estimated camber combine to require significant supplemental reinforcement near supports in the haunch per §9.4 each of the different sheets of transverse reinforcement may have their projections varied (do not vary projection within a sheet). SWWR overall dimension and the distance from the hook to the top longitudinal wire shall be modified to account for the different projection. This approach is not recommended near midspan due to camber variability.

If designers vary wire sizes of SWWR5 through SWWR7 on the base sheets longitudinal wire sizes may need to vary.

**5.5.1.5.2—Negative Moment Reinforcing Steel in Bridge Deck at Intermediate Supports**

Negative moment reinforcement shall be provided in the bridge deck, taking into account deck longitudinal distribution reinforcement, by assuming 100% live load continuity at supports.

When designing for continuity designers may choose to only check negative moment capacity at the centerline of bearings on each side of the intermediate support. Satisfying negative moment capacity at the centerline of the intermediate support is not required.

When providing additional negative moment reinforcement precautions should be taken to prevent stress concentrations at the termination of the reinforcement. Typically this should be achieved by providing at least two different termination points for the supplemental reinforcement each side of the intermediate supports.

Additional information can be found in §9.2.2.2 and associated commentary.

**5.5.1.6—Vent Holes**

Where the bottom flange of the girder is designed as submerged during a flood event, vent holes shall be provided in the web to reduce buoyant forces on the structure. Vent hole locations shall meet both of the distances:

- 1 ft. from the face of the turndown/diaphragm
- H from the end of the girder

See Figure 5.3 for a graphical example.

Where vent holes are required see §3.5 for lateral and uplift loading requirements.

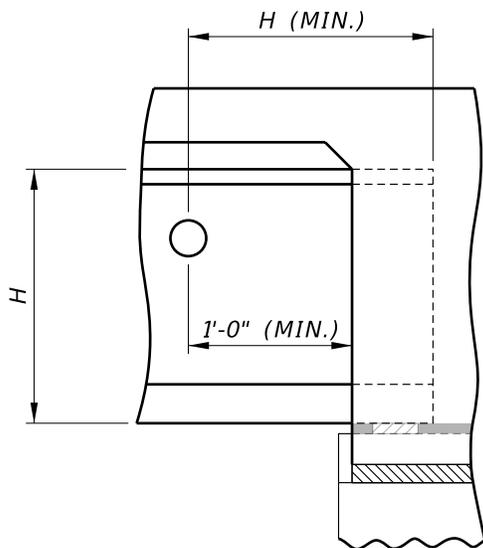


Figure 5.3—Precast Girder Vent Hole Location

### 5.5.1.7—Stay-in-Place Forms

Stay-in-Place metal forms are permitted. Designers shall apply the load over the width between girders. See §3.3 for load magnitude.

### 5.5.1.8—Camber

Camber is defined as the net result of upward deflection due to prestressing and downward deflection due to all dead loads. Camber and any correction for grade vertical curvature must be considered when determining girder seat elevations and concrete quantities. Bridge plans shall indicate typical vertical dimensions from the top of the girder flange to grade at all supports.

During design, calculated camber at the completion of construction shall be upward or straight. Time-dependent prestressing losses shall be calculated using  $t_t = 56$  days during this check.

Research has shown that after the composite deck is poured dead load deflections do not increase, therefore long term deflections need not be checked.

Camber shall not be considered in the vertical clearance determination under a bridge. For the purposes of determining the vertical clearances, the bottom of the girder shall be considered a straight line between the bearings. LL+IM deflection shall be considered.

### 5.5.1.9—Dead Load Deflections

All girder bridge plans shall have deflections calculated at the span tenth points and labeled, "Deflections for Shims."

Deflections for Shims Table in Prestressed Girder Information Sheet shall include instantaneous deflection due to dead loads only (including future wearing surface only if it is placed at time of initial construction).

When haunch thickness at midspan is greater than 3 in., consideration should be given to also indicating the vertical dimensions from the top of the girder flange to grade at midspan as well.

Research has shown that the multipliers commonly used by in the precast industry for long-term deflection calculations are not applicable to today's large high strength precast bridge members with a composite deck rather than a thin topping (Binard & Patel, 2022; Martin, 1977).

At this point in time the erection stage multipliers from Martin's 1977 paper are the best we have. Bridge Division is currently researching adjustments to the erection multipliers for use on future projects.

Girder camber at erection is a random variable that can vary by as much as  $\pm 50\%$  (Tadros et al., 2011). Therefore, camber should not be relied upon to provide vertical clearance.

### 5.5.2—Prestressed I-Girders

NU girders are a very common system on state and local structures. For girder dimensions see Figure 5.4.

Designers to note that the girder shape is not the exact height of the girders in inches. The NU series of girders were originally developed when FHWA required the use of metric units on any Federal-aid highway program project. When highway projects switched back to the United States customary units, the girder dimensions were not modified.

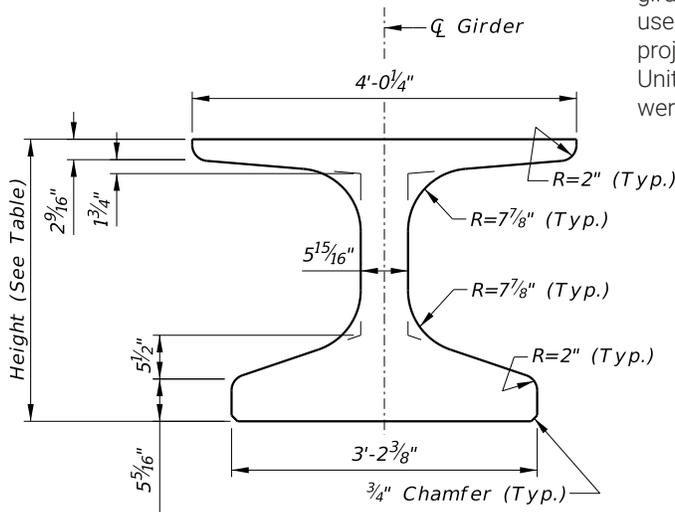


Figure 5.4—NU Girder Dimensions

#### 5.5.2.1—Section Properties and Layout

Girders shall be spaced at a maximum of 15 ft. center to center. For deck overhang limitations see §9.2.3.

After accounting for roadway grade, prestressing, and instantaneous dead load deflections; NU girders shall be detailed with minimum 1 in. haunch at the edges of the girder flanges. During construction and redecking, concrete girders (including AASHTO shapes) may be embedded up to 1 in. into the decks at a minimum number of locations.

Table 5.7—NU Girder Section Properties

Shape	Height in.	Area in. <sup>2</sup>	Perimeter in.	$Y_{nc}$ in.	$I_g$ in. <sup>4</sup>	Weight (8 ksi concrete) klf	Weight (10 ksi concrete) klf
NU35	35.44	648.3	208.1	16.13	110,218	0.711	0.720
NU43	43.31	695.1	223.8	19.57	182,262	0.763	0.772
NU53	53.13	753.3	243.5	23.95	301,949	0.827	0.837
NU63	63.00	812.0	263.2	28.43	458,653	0.891	0.902
NU70	70.88	858.7	279.0	32.05	611,646	0.942	0.954
NU78	78.75	905.5	294.7	35.70	791,107	0.994	1.006

### 5.5.2.2—Strand Pattern

NU Girder bottom flanges can accommodate up to 60 strands. An additional four strands tensioned to 5 kips are permitted in the top flange for the precaster's use. Permitted locations for strands are shown in [Figure 5.5](#).

Standard strand extension into diaphragms and abutments is eight per girder, the extended strand locations are shown in [Figure 5.5](#) and in the NU Girder base sheets.

Standard strand diameter for use on NU Girders is 0.6 in. Other diameters require Bridge Division approval for their use.

Span charts for use in preliminary design are available on NDOT's website (Hanna et al., 2010).

For all NU girders, additional U-shaped bars (G501) shall be added to girder ends to resist the stresses due to lifting and handling of girders (see base sheets). Designers shall hold strands low enough to avoid conflict with G501 bars. When no other solution is possible the G501 bars may be removed and top deflected strand placed 2 in. below the top of the girder.

The 5 kip strands are typically neglected during final design calculations.

[Figure 5.5](#) is a visual representation of a fully populated strand pattern for an NU Girder. Precasters are given leeway on which exact slots to populate in the form for partially full rows based on what they feel will be the most efficient for their production. Deflected strands, extended strands, and strands used to tie reinforcement cages to are filled first and after that there is not a set sequence.

C3/R4 and C6/R3 should typically remain bonded when populated because they may have bottom flange confinement reinforcement tied to them. Per LRFDBDS Article C5.9.4.3.3 tying of mild reinforcement to debonding material is not desirable and bonded strands will help minimize cracking due to the Hoyer effect.

0.7 in. strand has been successfully used on experimental bridges in the past, but has not been adopted by the industry or designers.

Engineering judgment should be taken when using these preliminary design table as they have been found to be slightly unconservative. Designers should note to use the preliminary design tables for Working Stress Method at release, not for Strength Design Method at release. Bridge Division is currently revising these preliminary design tables for future use.

Allowable tensile stresses in the girder at release may be increased by providing additional mild reinforcement to resist tensile load, this may necessitate increasing the length or bar size of the G501 bar. See LRFDBDS Article C5.9.2.3.1b and Table 5.9.2.3.1b-1.

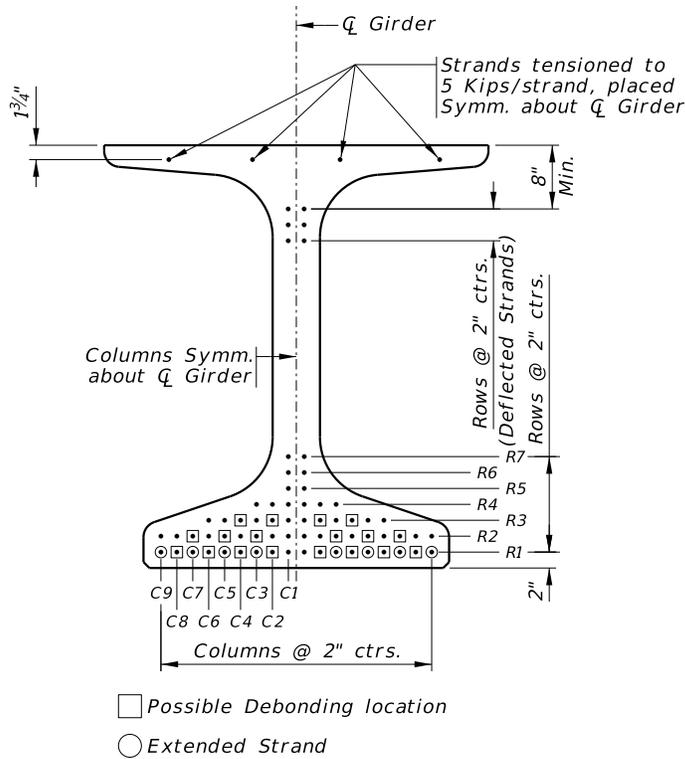


Figure 5.5—NU Girder Strand Pattern

**5.5.2.2.1—Cost-Optimized Strand Pattern**

For fabrication, the most cost-effective girder design will have up to 8 strands harped within a single hold-down device (limited to 25 kips) at each of the hold down points. Debonding can then be used to meet the permitted stress limits.

For girders that fall outside the cost-optimized strand patterns, all limits in §§5.5.2.2.2 and 5.5.2.2.3 must be met.

**5.5.2.2.2—Limitations to Debonding**

Strands may be debonded to help limit stresses in the girders up to the limits listed in LRFDBDS Article 5.9.4.3.3.

**5.5.2.2.3—Limitations to Harping**

Up to 14 strands in Column 1 may be deflected. At the girder ends, the strands are deflected to a normal pattern (meaning 2 in. vertical spacing between strand rows). The centroid of strands at both the girder end and the deflection point may be varied to suit girder stress requirements. When the centroid is varied, the placement of the deflected strands must still be as a group.

The harping of strands is beneficial to the stability of girders during shipping. This has a more pronounced effect on longer spans.

The strand hold-down points are normally located at the 0.4 and 0.6 tenth points of the prestressed girder. The hold-down location may be adjusted if it improves the stresses in the girder or at the request of the girder fabricator. If the fabricator desires a change from what is shown in the plans, they shall submit for approval a strand deflection configuration that meets plan centroid requirements.

The maximum hold-down force that can be accommodated by Nebraska producers is 25 kips per device and a maximum of two devices can be used at each harp point. Each hold-down device can accommodate up to 8 strands but does not need to contain all 8 strands before a second one is used. Deflected strand patterns must be designed to accommodate this limitation. When hold-down forces are a concern, the deflected strand exit location at the girder ends needs to be held as low as possible while maintaining the concrete stresses within allowable limits.

#### **5.5.2.3—Transverse Reinforcement**

The preferred reinforcement in the bursting zone is two mats of D25 wires at 2 in. spacing. Reinforcement may be increased to

- two mats of D31 wires at 2 in. spacing or
- two mats of D25 wires at 2 in. spacing with a third mat of D18 wires at 2 in. spacing in the center of the web.

D25 and D31 wires shall only be used in the end zones for containing bursting forces, D18 wires shall be used elsewhere. D18 wires at 12 in. spacing is the minimum permitted reinforcement in I-shaped girders.

#### **5.5.2.4—Intermediate Diaphragms**

Spans greater than 160 ft. shall have an intermediate diaphragm at midspan. Standard bent plate diaphragm details are provided in the sheet cell library for NU63 and deeper templates.

#### **5.5.2.5—Vent Holes**

When vent holes are required by design, one 3 in. diameter hole shall be provided on the high end of each girder. The centerline of the hole shall be placed 1 ft. 1 in. from the top of the girder. Designer shall ensure deflected prestressing strands are held low enough or are above the vent hole to provide a minimum  $\frac{3}{4}$  in. clear cover.

### **5.5.3—Threaded Rod Connected I-Girders**

Reserved for future use.

For more information see Sun et al. (2016).

Typically girders less than 50 ft. long are fabricated using only straight strands due to high hold down forces required.

When calculating forces per hold-down device, strands in the same row must be in the same hold-down device.

Due to the conservative nature of the hold-down force limitation, no additional friction factor is necessary when calculating hold-down forces.

Where strand debonding is utilized, designers may choose to only count strands bonded at the end of the girder when determining the prestress force at transfer for calculating pretensioned anchorage zone splitting resistance at girder ends in LRFDBDS Art. 5.9.4.4.1. Reference design example 9.6 in the [PCI Bridge Design Manual](#) (Precast/Prestressed Concrete Institute, 2023).

D31 wires are the largest size that can be used while still maintaining minimum cover on the transverse reinforcement.

Designers should consider placing additional intermediate diaphragms when designing submerged girders.

### 5.5.4—Prestressed Inverted Tee Girders

Inverted tee girders are a commonly used system for shorter span lengths. For girder dimensions see Figure 5.6.

Designers to note that the girder shape is not the exact height of the girders in inches. The IT series of girders were originally developed when FHWA required the use of metric units on any Federal-aid highway program project. When highway projects switched back to the United States customary units, the girder dimensions were not modified.

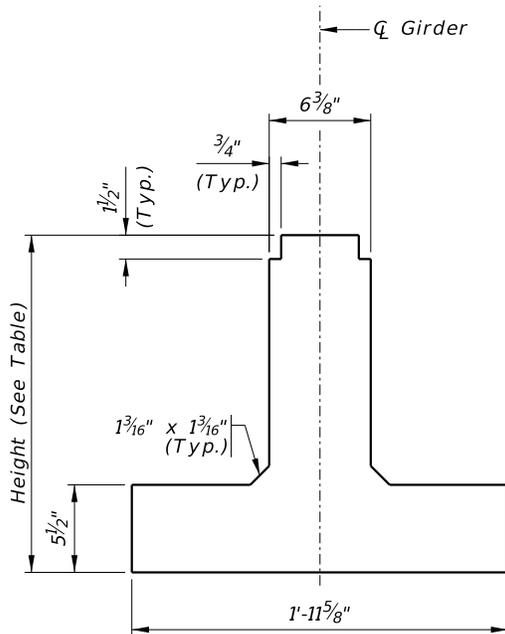


Figure 5.6—IT Girder Dimensions

#### 5.5.4.1—Section Properties and Layout

Girders shall be spaced at a maximum of 37 in. center to center. For maximum deck overhang see §9.2.4.

The use of 8 in. thick decks on shallow inverted tee sections should be approached with caution. Construction issues with excessive deflection due to deck weight are not uncommon.

Table 5.8—IT Girder Section Properties\*

Shape	Height in.	Area in. <sup>2</sup>	$Y_{nc}$ in.	$I_g$ in. <sup>4</sup>	Weight (8 ksi concrete) klf	Weight (10 ksi concrete) klf
IT13	13.31	178.9	4.50	2,034	0.196	0.199
IT17	17.25	204.0	5.79	4,472	0.224	0.227
IT21	21.19	229.1	7.22	8,334	0.251	0.255
IT25	25.13	254.2	8.76	13,871	0.279	0.282
IT29	29.06	279.3	10.37	21,300	0.306	0.310
IT33	33.00	304.4	12.05	30,837	0.334	0.338
IT36†	36.94	329.5	13.77	42,688	0.362	0.366

\* These section properties are for the gross section as specified on the base sheets. Designers shall consider that the girders are embedded  $\frac{3}{4}$  in. into the deck in addition to the mandatory sacrificial wearing thickness specified in §9.2.1 when calculating composite section properties.

† In most cases a NU35 girder would be preferable to a IT36.

### 5.5.4.2—Strand Pattern

IT Girder bottom flanges can accommodate up to 22 strands. An additional two strands located at the top of the web are tensioned to 5 kips/strand and may be increased to help control stresses if required. Permitted locations for strands are shown in Figure 5.7. Top strands are not permitted to be cut in midspan. Draping strands is not permitted on IT girders, the only options to control end stresses is to debond strands or adjust the tension in the top strands.

Standard strand extension into diaphragms and abutments is four per girder, the extended strand locations are shown in Figure 5.7 and in the IT Girder base sheets.

Standard strand diameter for use on IT Girders is 0.5 in. Other diameters require Bridge Division approval for their use.

Allowable tensile stresses in the girder at release may be increased by providing additional mild reinforcement to resist tensile load. See Article C5.9.2.3.1b and Table 5.9.2.3.1b-1 in the LRFDBDS.

The 5 kip strands are typically neglected during final design calculations.

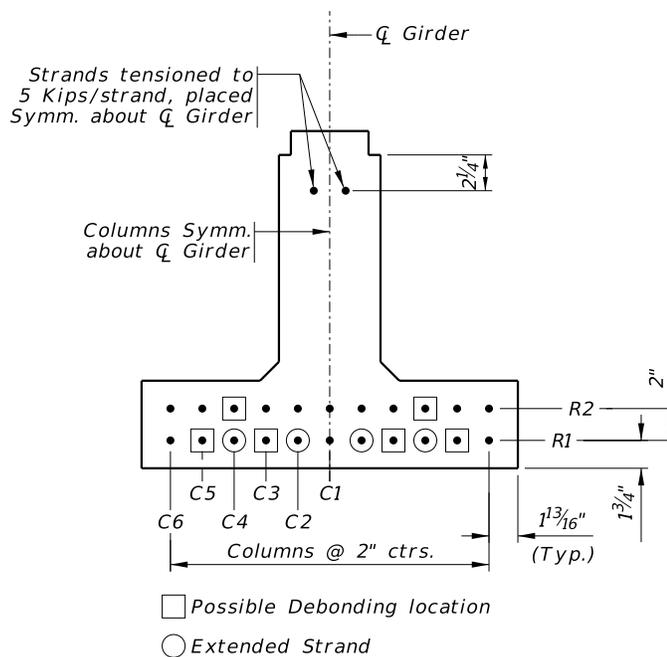


Figure 5.7—IT Girder Strand Pattern

### 5.5.4.3—Transverse Reinforcement

The preferred reinforcement in the bursting zone shall two mats of D20 wires at 2 in. spacing. Beyond the anchorage zone D14 wires shall be used. D14 wires at 8 in. spacing is the minimum permitted reinforcement in inverted tee girders.

Where strand debonding is utilized, designers may choose to only count strands bonded at the end of the girder when determining the prestress force at transfer for calculating pretensioned anchorage zone splitting resistance at girder ends in LRFDBDS Art. 5.9.4.4.1. Reference design example 9.6 in the [PCI Bridge Design Manual](#) (Precast/Prestressed Concrete Institute, 2023).

### 5.5.4.4—Intermediate Diaphragms

Intermediate diaphragms shall be required for all Inverted Tee girder bridges. A minimum of one diaphragm at midspan between the three outside girders is required. Where structures are phased during construction diaphragms shall be provided at the edges of the structure as well as at the edge of Phase I, see Figure 5.8 for an example.

Designers should consider placing additional intermediate diaphragms when designing submerged girders.

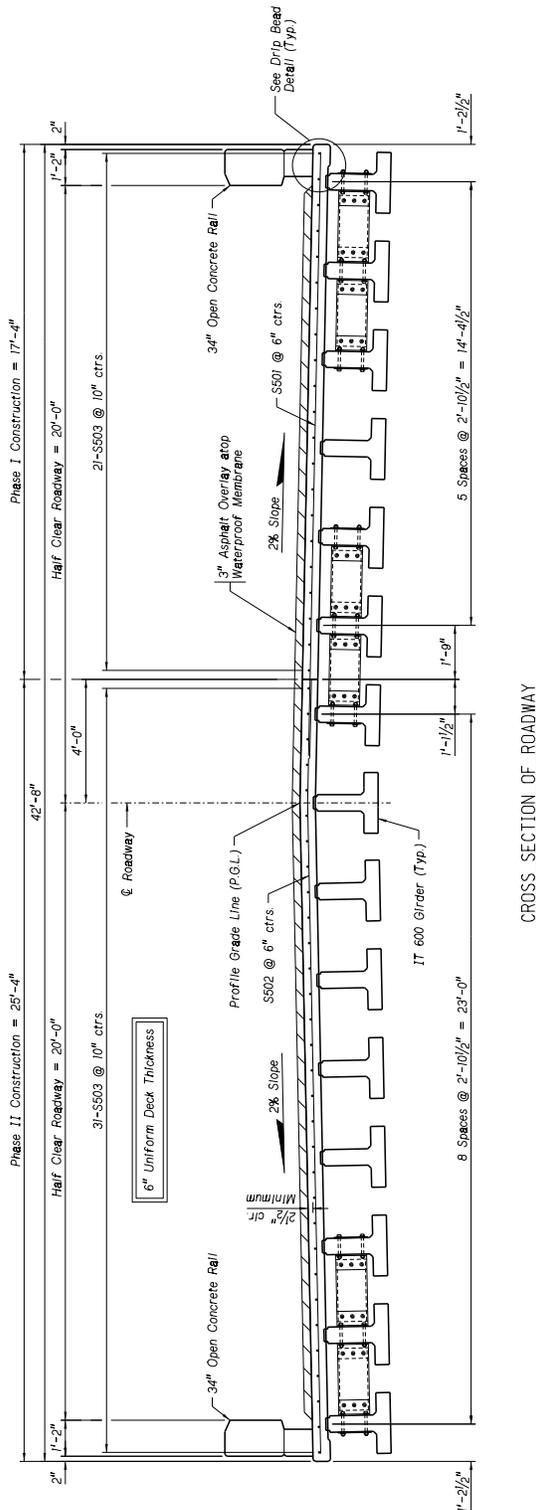


Figure 5.8—Location of intermediate diaphragm for phased construction of IT girder bridge (example taken from S081 08543, CN 42785, 2020)

Standard channel section diaphragm details are provided in the sheet cell library. A sample cross section for concrete intermediate diaphragms is provided in Figure 5.9.

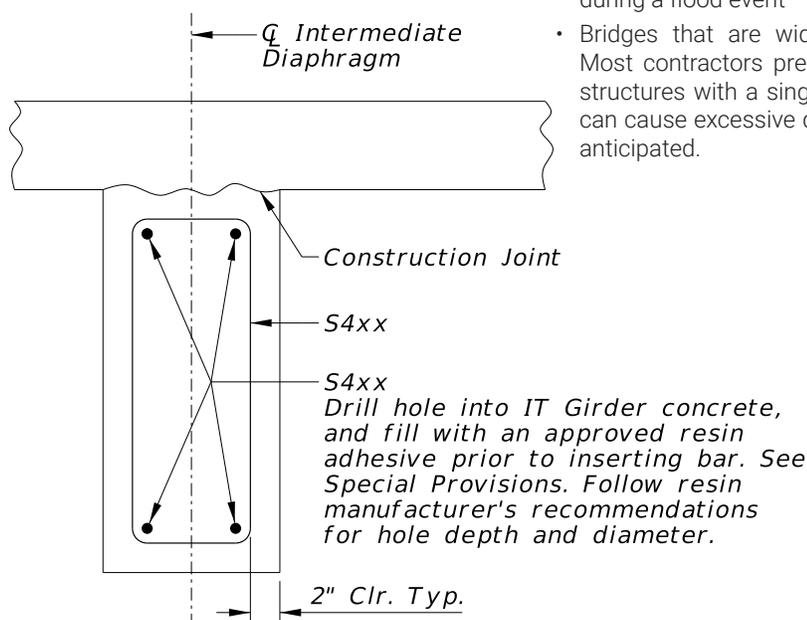


Figure 5.9—IT Girder Intermediate Concrete Diaphragm

Concrete or full width intermediate diaphragms may be of use when the diaphragm is expected to see more load than average such as:

- Bridges expected to be at least partially submerged during a flood event
- Bridges that are wide and built in single phase. Most contractors prefer to construct these type of structures with a single wide finishing machine that can cause excessive deflection in edge girders if not anticipated.

#### 5.5.4.5—Vent Holes

When vent holes are required by design, one 2 in. diameter hole shall be provided on the high end of each girder. The hole shall be placed a minimum  $\frac{3}{4}$  in. clear below the top strands and any additional top reinforcement added for release stresses.

#### 5.5.4.6—Live Load Distribution

The distribution factors of live load per lane for Inverted Tee girders shall be  $S/11$  for moment and  $S/5.5$  for shear, where  $S$  is the beam spacing in feet. These factors are based on research done by the University of Nebraska (Kamel & Tadros, 1996; Martindale et al., 2019).

## 5.6—OTHER PRECAST ELEMENTS

### 5.6.1—Non-Prestressed Elements

Reserved for future use.

Precast concrete elements (non-prestressed) have been used by Bridge Division for:

- Abutment Caps
- Wings
- Grade Beams
- Approach and Paving Sections with rails

See Belden - Laurel (S020 38968) and Murray - US-34 and 75 (S001 02613) for details. Designers are encouraged to use precast concrete elements for ABC projects.

### 5.6.2—Prestressed Deck Panels

Reserved for future use. See §9.3 for details.

Full depth precast deck panels (NU Deck) have been used on Belden - Laurel (S020 38968), Kearney East Bypass (S010 05463R), and 198<sup>th</sup> - Skyline Drive (SL28B00216).

### 5.6.3—Post Tensioned Elements

Reserved for future use.

#### 5.6.3.1—Post Tensioned I-Girders

Reserved for future use.

For more information see Jaber et al. (2006) and Tadros et al., (2003b) as well as the 2016 NDOR Bridge Office Policies and Procedures Manual.

### 5.6.4—UHPC Decked I-Girders

Reserved for future use.

For preliminary information see Morcoux & Tadros (2023).

## 5.7—REFERENCES

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