MDOT RC-1602

Improving Bridges with Prefabricated Precast Concrete Systems

APPENDICES

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Western Michigan University
Department of Civil & Construction Engineering
College of Engineering and Applied Sciences

Research

Improving Bridges with Prefabricated Precast Concrete Systems

Appendices

Project Manager: Mr. David Juntunen, P.E.

Submitted to:

MDOT

Submitted by

Dr. Haluk Aktan, P.E.
Professor & Chair
(269) 276 – 3206
haluk.aktan@wmich.edu

Dr. Upul Attanayake, P.E.
Assistant Professor
(269) 276 – 3217
upul.attanayake@wmich.edu

Western Michigan University
Department of Civil & Construction Engineering
College of Engineering and Applied Sciences
Kalamazoo, MI 49008
Fax: (269) 276 – 3211

DISCLAIMER

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APPENDIX A

PREFABRICATED BRIDGE ELEMENTS AND SYSTEMS

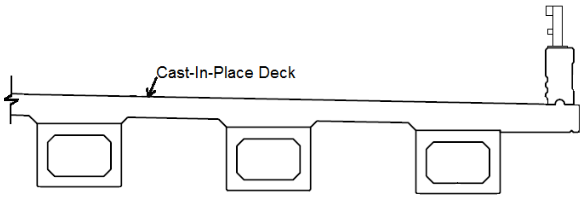
Girders

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast concrete (PC) I-girders  (Source: PCSB 2011)	Standardized as AASHTO type sections. Used in several projects by the State DOT's since 1950's. Recent projects: I-5 Southbound Truck Route Undercrossing, CA. (Superstructure replacement) (2007) Parkview Avenue over U.S. 131, Kalamazoo, MI. (Bridge replacement) (2008)	Following attributes are presented based on the information provided in the listed references. Span range: refer the table beside Depth to span(D/S) ratio: 0.055 Typical girder depth ranges from 28 in. to 54 in. However, there are state specific girders that are much deeper than the standard sections. One such example is the70.9 in. deep MI-1800 girder. Concrete strength: 5000 psi to 7000 psi. MDOT–I beams (AASHTO types I to IV) span up to 114 ft MDOT–70 in. deep I beams span up to 120 ft MDOT–70.9 in. deep I beam (MI-1800girder) spans up to 145 ft	Standard sections. Designers, fabricators, and contractors are familiar with the sections. Forms are available at most of the prefabrication plants. Performance is well documented.	Implementation in ABC is only possible with partial-depth or full-depth deck panels. Girder sweep needs to be controlled when used with full-depth deck panels. Special details and cast-in-place construction is needed to develop continuity over piers. Identified as structurally inefficient compared to bulb-tee, Washington, and Colorado girders in terms of cost effectiveness (Bardow et al. 1997; TFHRC 2006) Cannot extend over long spans without using post-tensioning. Curved spans require use of straight girders. High probability of cracking at transfer with 0.7 in. diameter prestressing strands. (Vadivelu 2009)	Have been used in rapid bridge replacements by using heavy equipment such as SPMT (Ralls 2008) Sources of information: Chung et al. (2008); Abudayyeh (2010); MDOT-BDM (2011); Attanayake et al. (2012).

Girder	Depth (in.)	Span (ft)	28-day concrete strength (psi)
PC - I (Type I – IV)	28 – 54	~114	5,000 – 7,000
PC – I (Wisconsin type)	70	~120	5,000 – 7,000
PC – I (MI 1800)	70.9	~145	5,000 – 7,000

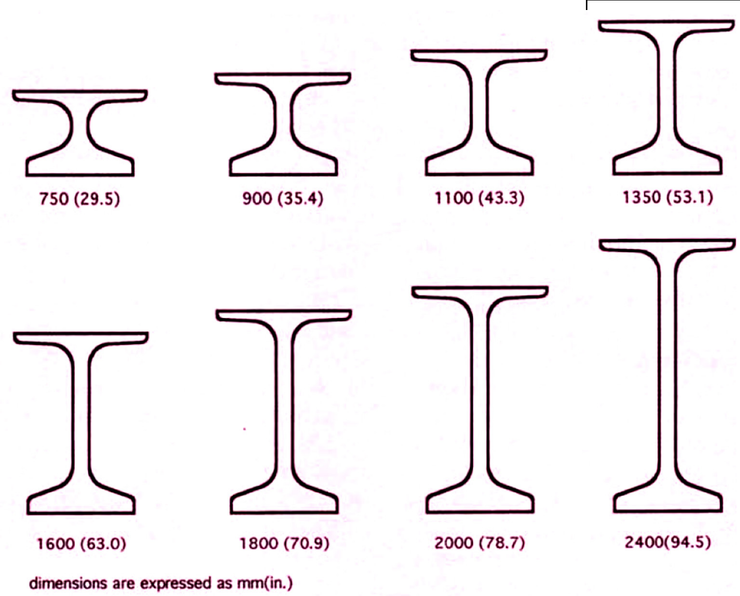
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Steel girders 	Used in several projects by the State DOT's since early 19th century. Recent projects: Oakland Eastbound I-580 Connector, CA (Superstructure replacement) (2007) Route 3 Mosquito bridge over Lake Winnisquam, Sanbornton & Belmont, NH (Deck replacement) (2004)	Following attributes are presented based on the information provided in the listed references. Span range up to 300 ft Depth to span (D/S) ratio: 0.04 to 0.045 which is smaller than D/S ratio of precast prestressed concrete girders.	Could be used on curved bridges. Continuous spans can be developed using the same section. Customized (built-up) sections can be developed to satisfy project requirements Weathering steel is a solution for controlling corrosion provided that there is no accumulation of water, chloride exposure, damages to the girder, etc. Material properties are well known and defined.	Implementation in ABC is only possible with partial-depth or full-depth deck panels. Welding of connections subject to fatigue Require more detailed inspection and maintenance for fatigue and corrosion Costly compared to precast concrete girders Field welding may be required Customized sections are very costly Maintenance requires painting, thus expensive and non-eco-friendly. Use of weathering steel in salt-laden environments is highly discouraged, since the protective layer may not stabilize but rather corrode more rapidly. Moreover, weathering steel is not rustproof in itself; therefore, if water is allowed to accumulate on it, corrosion rate sharply increases.	Has already been used in rapid bridge replacements using heavy equipment such as SPMT (Ralls et al. 2004; Ralls 2008) Sources of information: Chung et al. (2008); Richardson et al. (2009).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast bulb-tee girders  (Source: PCSB 2011)	Used by States such as: New England, Washington, Colorado, Florida, New Mexico, Idaho, Oregon, etc. since several decades. Jetport Interchange, Maine (Bridge replacement) (1999) I 40 Bridge project, CA (Bridge replacement) (2006)	Following attributes are presented based on the information provided in the listed references. Spans range: up to 186 ft (UDOT 2010) Depth to span (D/S) ratio: 0.05 which is smaller than D/S ratio of precast concrete I-girder Depth range: 42 in. to 98 in. Concrete strength range: 6500 psi to 8000 psi Prestressing strands: 0.6 in. dia.	Provides greater capacity than standard precast concrete I-girders. Efficient than AASHTO type V and VI girders (Bardow et al. 1997; TFHRC 2006) Feasible for long spans.	Implementation in ABC is only possible with partial-depth or full-depth deck panels. Girder sweep needs to be controlled when used with full-depth deck panels. Controlling girder sweep is critical due to slenderness of the section compared to standard girders. Special details and cast-in-place construction are needed to develop continuity over piers. Curved spans require use of straight girders.	High Performance Concrete (HPC) with 10,000 psi 28-day strength, could be used to obtain longer spans and more durable structure. Sources of information: Lavallee and Cadman (2001); Fouad et al. (2006); Chung et al. (2008); UDOT (2010).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast spread box girders   (Source: PCSB 2011)	Spread box-girders have not been used in any ABC projects.	Following attributes are presented based on the information provided in the listed references. Spans up to 140 ft. Depth ranges from 12 in. to 60 in. Width: 36 in. and 48 in. Concrete strength: 5,000 psi to 7,000 psi. There are records of using high performance concrete (HPC) with 28-day strength of 8,000 psi.	Shallow depth enables using at sites with tight underclearance. High torsional stiffness of the sections.	Implementation in ABC is only possible with partial-depth or full-depth deck panels Special details and cast-in-place construction are needed to develop continuity over piers. Box-beams are difficult to fabricate as they involve multi-step fabrication process (Culmo & Seraderian 2010) Access to confined space inside the box is not possible because of the Styrofoam blocks used during fabrication (Smith and Hendy 2002) Weep holes are required at the bottom flange. Not possible to detect deterioration inside the concrete box until rust stain is visible at the weep holes or girders crack. Spread box girders require formwork between the girders to form the deck.	Source of information: MDOT-BDM (2011).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast NU I-girders For NU I-girders with 60 - 0.6 in. diameter prestressing strands:	Bow river bridges in Calgary, Alberta, Canada Pacific street bridge over I-680 in Omaha, Nebraska. 14th street bridge over I-80, Lincoln, Nebraska	Following attributes are presented based on the information provided in the listed references. Span: refer the table beside Depths: refer the table beside Concrete strength: refer the table beside Specified 28 day compressive strength of minimum 12000 psi is required if 0.7 in. diameter strands are used. Prestressing strands: 0.5 in., 0.6 in., and 0.7 in. diameter 270 ksi low-relaxation steel. Typically, prestressing strands are spaced 2 in. horizontally and 2.5 in. vertically.	The NU I-girders have sections that can span up to 300 ft with longitudinal post-tensioning Provides shorter deck spans in the transverse direction due to wide top flange. Increased stability during shipping and handling due to virtue of its wide top flange and thick and wide bottom flange compared to AASHTO girders (see figure below for dimensions). The reinforcement details are standardized such that the amount of post-tensioning, girder span, or girder spacing does not affect the reinforcement The large span-to-depth ratio allows for using these sections in lieu of steel plate girders without increasing the superstructure depth	Implementation in ABC is only possible with partial-depth or full-depth deck panels Special details and cast-in-place construction are needed to develop continuity over piers. The lack of readily available hold down devices for depressing 0.7 in. diameter strands is an obstacle These girder sections are not widely implemented; hence, local fabricators may not have the resources and/or expertise because the fabrication requires new forms. Also, devices with adequate capacity to accommodate 0.7 stands.	Flexural capacity of NU 900 I-girder with 28-day concrete strength of 15,000 psi ranges from 5800 kip-ft to 6000 kip-ft. Shear capacity of NU 900 girder with 28-day concrete strength of 15,000 psi ranges from 780 kip to 800 kip The NU 750 I-girder has not been used in any bridge projects. The NU 2400 I-girder has been generally used with post-tensioning.

Depth (in.)	Span (ft)	28 day concrete strength (psi)
94.5	~200	12,000
78.7	~180	8,000 – 12,000
70.9	~172	8,000 – 12,000
63.0	~155	8,000 – 12,000
53.1	~135	8,000 – 12,000
43.3	~118	8,000 – 12,000
35.4	~110	8,000 – 12,000



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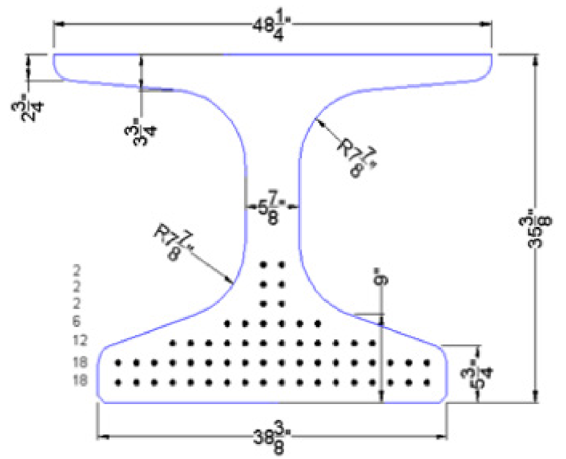
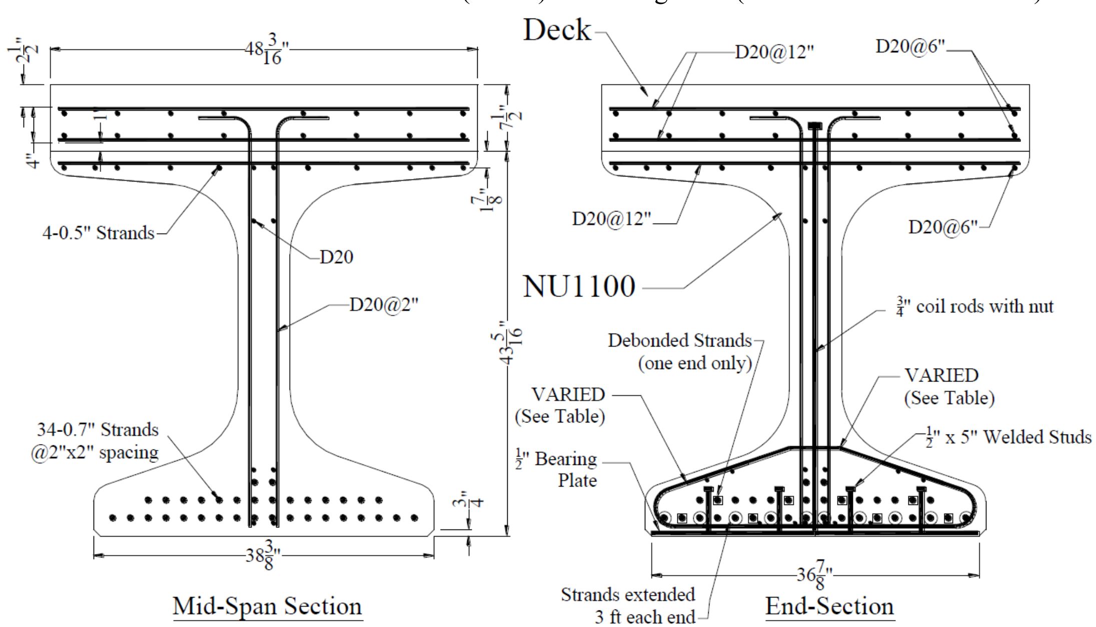
<tr><td>
</td><td></td><td>Concrete deck 7.5 in. thick with 28 day compressive strength of 4000 psi.</td><td></td><td></td><td>Sources of information:
Geren and Tadros (1994); Beacham and Derrick (1999); Hanna et al. (2010b); Morcous et al. (2011).</td></tr>
<tr><td colspan="6">Standardized Welded Wire Reinforcement (WWR) for NU I-girders (Source: Hanna et al. 2010b):

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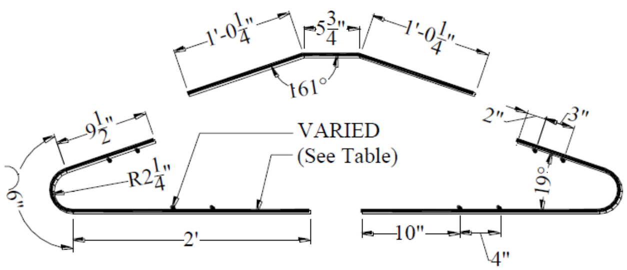
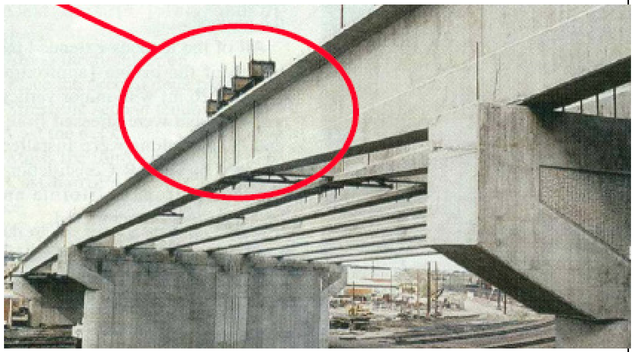
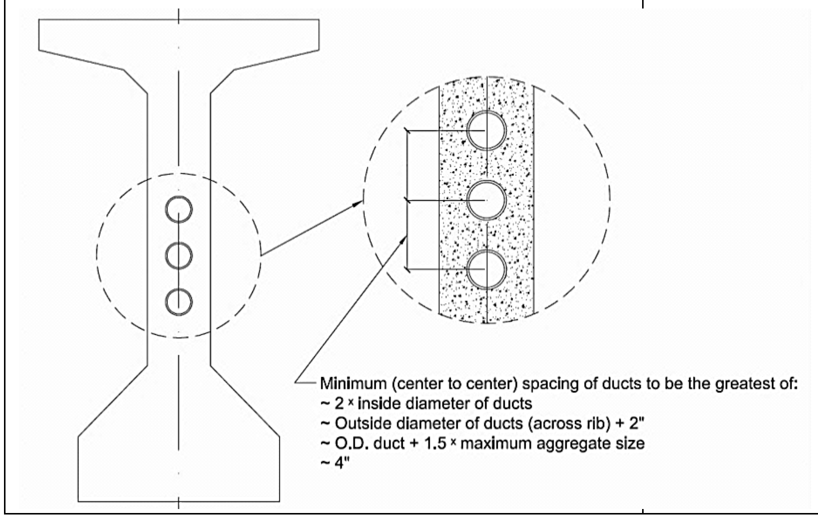
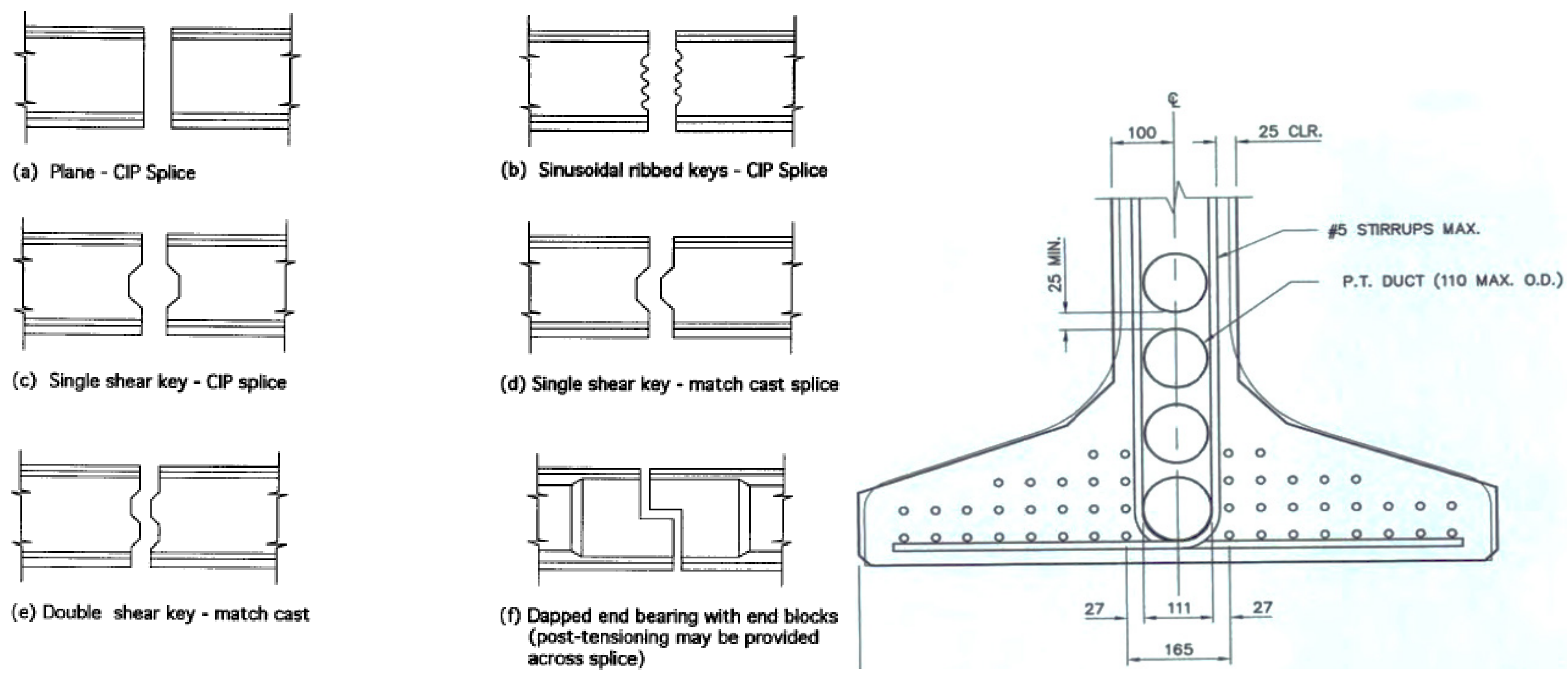
#3 Cap Bar



WWM Confinment Reinforcement
(See Table for size and spacing)

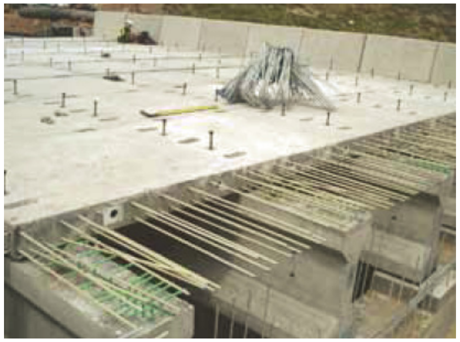
Girder Designation	Specified Confinement	Confinement Reinforcement	
		WWM	Cap Bar
1	2008 NDOR BOPP	D4 @ 4" entire length	#3 @ 12" entire length
2	2004 AASHTO LRFD	D11 @ 6" for 72" each end	#3 @ 6" for 72" each end
3	AASHTO + NDOR	D11 @ 6" for 72" each end D4 @ 4" middle	#3 @ 6" for 72" each end #3 @ 12" middle

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast girders with spliced details  Precast bulb-tee with post-tensioning in the web for splicing operation:  	Esker overhead bridge, British Columbia, Canada (Bridge replacement) (1990) I-15 Bridges, Salt Lake City, Utah (1999) Route 33 bridges at West Point, VA (2007)	Following attributes are presented based on the information provided in the listed references. Span ranges up to 220 ft Precast concrete I-girder, NU I-girder, or precast bulb-tee girder sections can be modified to accommodate post-tensioning for developing spliced details (see figure on the next page) Depth range: overall 6 ft to 9 ft. Over the piers, the depth varies up to 15ft 28-day concrete strength: 9000 psi. to 10,000 psi. Post-tensioning strands: 0.6 in. dia.	An option for developing precast concrete continuous spans Could be used for very high live load requirements Suitable for bridges with restricted pier placement	Implementation in ABC is only possible with partial-depth or full-depth deck panels Special details and analysis are required for spliced connections. Falsework or strongbacks required for splicing operation and could be time consuming Prolonged lane reduction required Large depth of girders and wide web required to accommodate post-tensioning ducts Cast-in-place diaphragm used at the splice locations Requires full-length post-tensioning	Sources of information: Mills et al. (1991); Geren and Tadros (1994); Seguirant (1998); Castrodale and White (2004); Browder (2007).

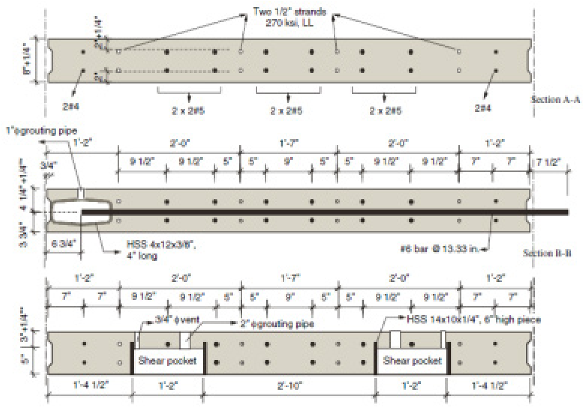
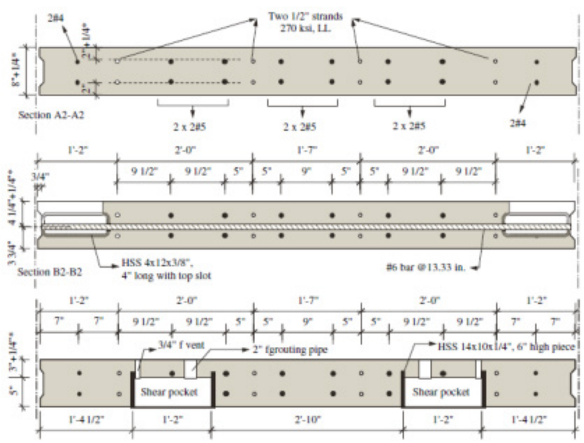


Note: All the dimensions are in mm (1 mm = 0.0394 in.)

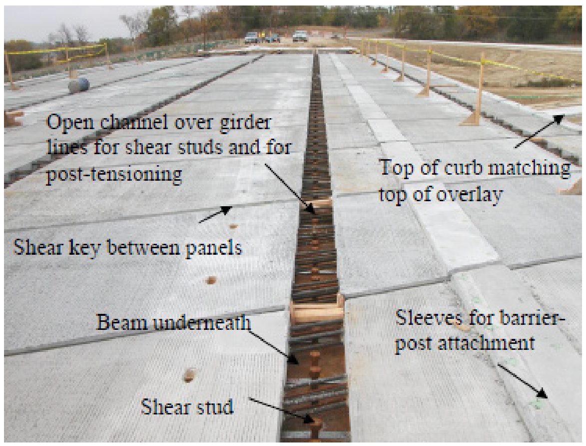
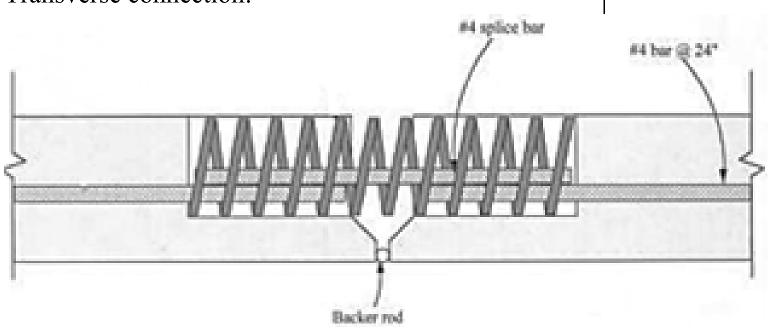
Decks

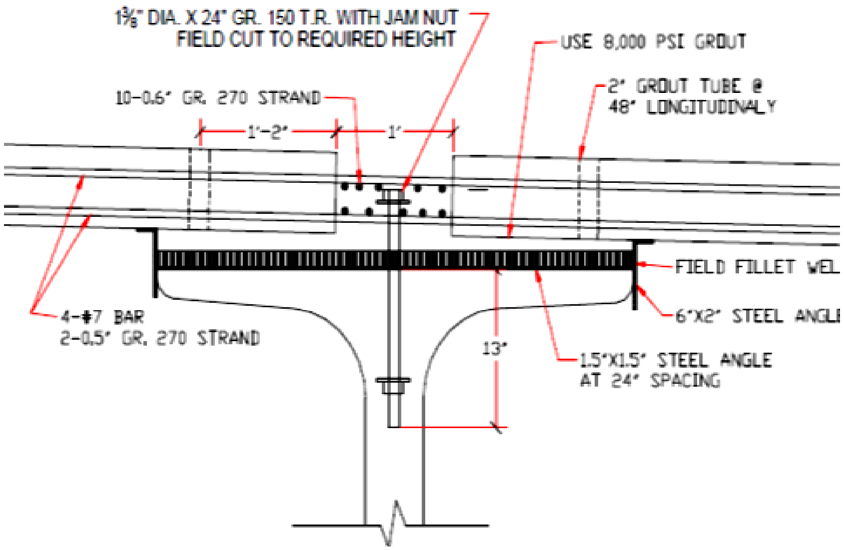
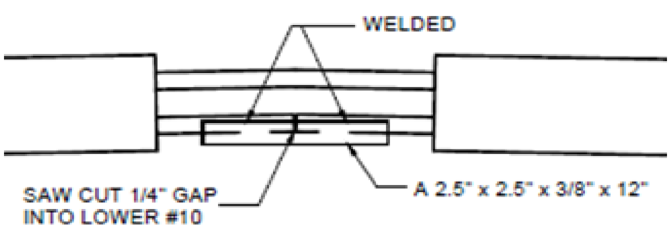
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Full-depth deck panels Two types are commonly used: **Full-depth deck panels with transverse prestressing and longitudinal post-tensioning:**  **Full-depth deck panels with only longitudinal post-tensioning:** 	Lake Koocanusa Bridge, Lincoln County, Montana (Deck replacement) (2001) I-70 Bridge over Eagle Canyon, UT (Deck replacement) (2007) I-215 over 3900 South, UT (Deck replacement) (2007) I-80 Silver creek, UT (Deck replacement) (2010) Parkview Avenue over U.S. route 131, Kalamazoo, MI (Bridge replacement) (2008)	Following attributes are presented based on the information provided in the listed references. Length (in the direction of traffic) varies from 8 ft to 16 ft Width (in the transverse direction to traffic) varies from 24 ft to 40 ft. Nominal thickness: 8.5 in. Deck panel concrete used in the listed projects was required to have the strength of 4,000 psi at release and 5,000 psi in 28 days. Girder spacing for panels with transverse prestressing varies from 8 ft to 12 ft. Girder spacing for panels without transverse prestressing varies up to 10 ft.	Full-depth deck panels have been implemented in several ABC projects and the lessons learned reports are documented. Several states have experience with the system. Full-depth deck panel systems have been implemented long before the ABC concept was introduced and performance of the system is well documented. For skewed bridges, the end panels could be customized to accommodate the skew, while keeping the middle panels rectangular to alleviate fabrication Better workmanship and high quality could be achieved with plant fabrication Transverse	Post-tension is required to achieve durable transverse connections between panels. When repair, retrofit, and demolition are considered, use of post-tensioning is not desirable. Grouting prefabricated Element joints is challenging. The system consists of too many grouted connections thus make the construction challenging. Tighter tolerances and quality assurance are required during the fabrication process. Proper panel support is required until haunch grout achieves required strength. Reinforcing steel	At least 2 leveling devices per girder in each panel is required. Round PT ducts with 2 in. inside diameter are preferred over flat ducts to avoid difficulty in strand placement (Badie et al. 2006) Sources of information: Hieber et al. (2005); Badie et al. (2006); Higgins (2010); UDOT (2010); Attanayake et al. (2012).

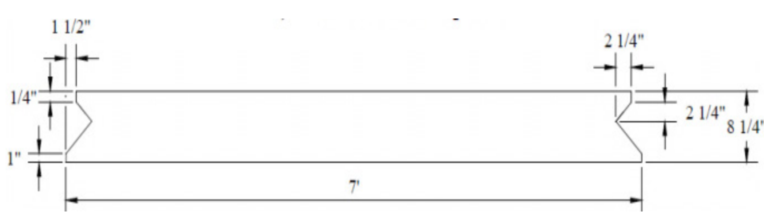
	Emma Park Bridge on U.S. route 6, UT (Bridge replacement) (2008) Trucker Bridge on U.S. route 6, UT (Bridge replacement) (2008) Mile post 200 Bridge on U.S. route 6, UT (Bridge replacement) (2009)	UDOT (2010) has skew allowances as shown below: Up to 15^{o} for skewed panels, and up to 45^{o} for rectangular panels with trapezoidal end panels. Top reinforcing clear cover of 2.5 in. is commonly used after leaving a 0.25 in. sacrificial layer for grinding. The minimum required bottom reinforcing clear cover is 1 in. Prestressing strands: 0.5 in. diameter, 270 ksi low-relaxation steel. AASHTO LRFD (2010) requirement is to have 250 psi after all the losses at the joint. Hence, it is necessary to analyze the continuous span structures to determine the level of prestress required over piers to achieve 250 psi after losses.	prestressing allows for thinner panels, wider girder spacing under the panels, and better crack control. Relatively fast construction, as CIP concrete topping is not required Panels could be used for either stage construction or full-width replacement of the facility.	in closure pour may have overlap issues. Significant tolerance enforcement is required at post-tensioning duct splicing locations and shear pockets in the panels Demolition of the bridge with post-tensioning is a challenge. Impact of vibration on grout bond needs to be considered when the bridge is used in staged construction.	

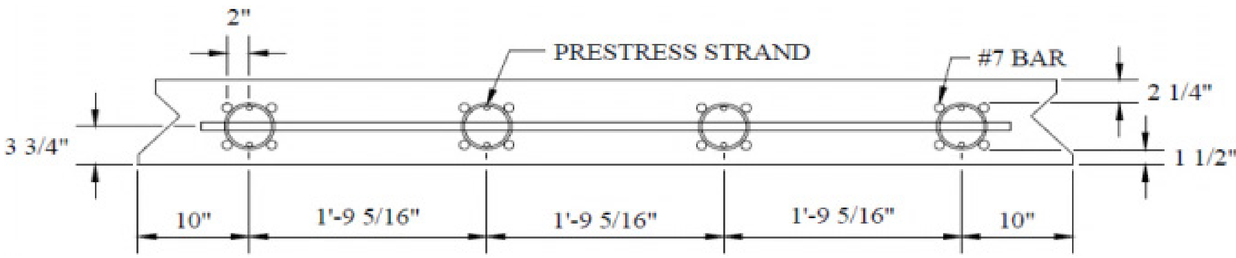
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Full-depth deck panels with only transverse prestressing Galvanized bulged hollow structural steel (HSS) tube – configuration 1 Note: the figures given below shows the cross-section perpendicular to bridge transverse axis.  HSS tube is not bulged and is provided with a 1.5 in. wide top slot – configuration 2  	Three full scale bridge specimens with four steel girders, four NU 1800 girders, and four bulb-tee girders, respectively, were successfully tested at the laboratory of University of Nebraska-Lincoln under the subcontract with George Washington University, Washington, D.C.	Following attributes are presented based on the information provided in the listed references. Length: 8 ft (in the direction of traffic) Width: 44ft (perpendicular to traffic direction) Thickness: 8.25 in., wherein 8 in. is structural slab thickness and 0.25 in. is for a sacrificial layer. Supporting girders spacing: 12 ft Normal weight concrete with unit weight of 150 lb/ft^3 has been used until now. 28-day compressive strength of 6000 psi was used in the project.	Configuration 1 and 2 provides similar details except at the transverse panel connections. Panel with configuration 2 details is vertically placed and a 24.5 in. long splice bar is dropped through the top slot to complete the joint connection. Hence, constructability is enhanced through this detail. Eliminating the post-tension shortens the construction duration, lowers the cost of the deck, and simplifies the construction process. CIP joints between the panels could utilize rapid set concrete mix which will eliminate the limitations	New concept and details; hence, no past performance records. Connections without post-tensioning have proven to be ineffective in terms of durability. Hence, details need to be evaluated before implementing in multiple projects. The deck panels with **bulged HSS** (configuration 1 details) need to be tilted during placement to insert the extended reinforcement into the grouted pocket of the adjacent panel. HSS tubes incur additional cost of fabrication. The 48 in. shear stud cluster spacing is not yet included in LRFD specifications; hence, horizontal shear needs to be evaluated to determine	Following details are exclusively from the bridge specimens that are discussed in Badie and Tadros (2008). Transverse steel: Eight 0.5 in. diameter prestressed strands, 12 No. 5 bars, and 4 No. 4 bars are placed in two layers. A 2 in. top and bottom clear cover is provided. Longitudinal reinforcement: No. 6 bars at 13.3 in. spacing. Clusters of three 1.25 in. diameter double-headed steel studs are used as shear connectors. The clusters are spaced at 48 in. The clusters spaced at 48 in. were found sufficient for bridges with spans from 60 ft to 130

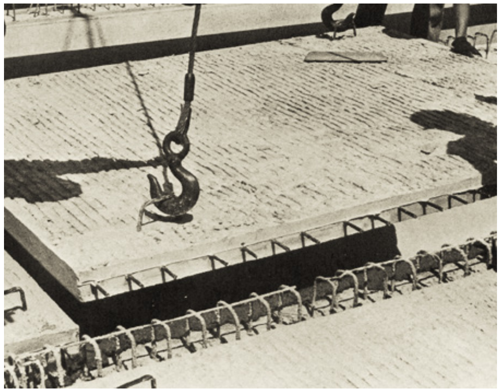
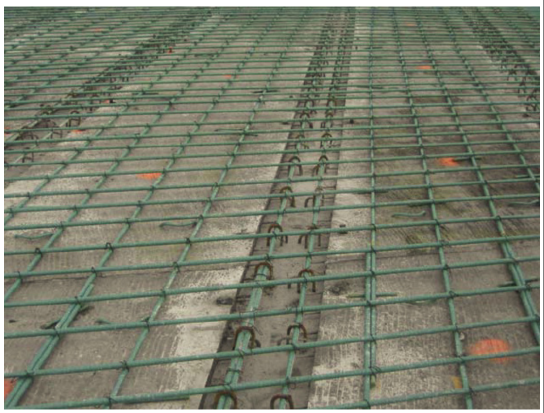
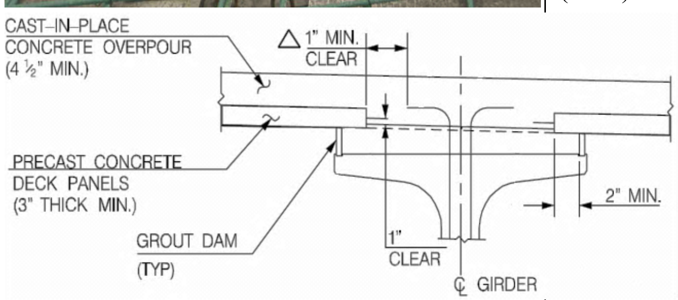
			imposed by grout properties (e.g., depth of fill). Ease of demolition or removal of panels by saw cutting the transverse joints	required number of studs.	ft and with girder spacing up to 11 ft (designed in accordance with the LRFD specifications) Source of information: Badie and Tadros (2008).

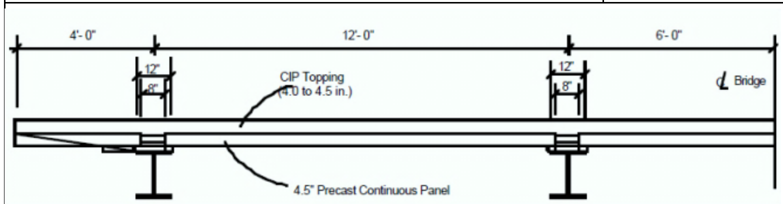
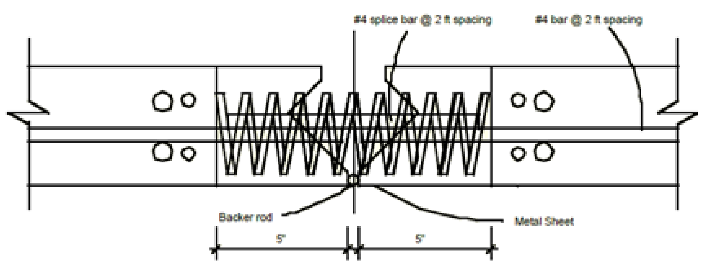
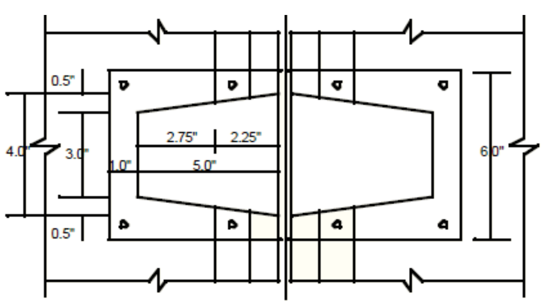
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
NU-deck full-depth panels   Transverse connection:  **Panel-to-panel connection using spiral reinforcement.**	First generation NU-deck: Skyline Bridge, Omaha, NE. Superstructure was replaced in 2003. Second generation NU-deck: 176th Street bridge over I-80, east of Lincoln, NE. Full bridge was replaced in 2009.	Following attributes are presented based on the information provided in the listed references. Length: 12 ft in the direction of traffic. Width: full bridge width Thickness: 7 in. Concrete strength: Release strength of 4500 psi and 28-day strength of 8000 psi Overlay: 1.5 in. CIP topping with 8000 psi Supporting girder spacing: 12 ft Skew: up to 30^{o} Full length channels: 1ft at each beam location Prestressing or post-tensioning strands: Uncoated, 0.6 in. diameter, 7-wire, 270 ksi low relaxation steel. Reinforcing steel: Grade 60	All materials required for fabrication are non-proprietary. The prestressing in panels helps in preventing cracks that may develop during fabrication and handling. Also, helps in reducing the panel thickness. Tolerance issues do not arise because the shear studs are arranged in single row. The 2nd Gen NU-deck has increased construction speed and ease of fabrication, as the crown is moved to a girder line location.	Durability is a major concern as the prestress and post-tensioning strands are placed in cast-in-place concrete joints. New concept and details; hence, past performance data is limited. Girder spacing (i.e., post-tensioning spacing) and post-tensioning sequence (i.e., releasing tendons after grouting the channels over the girders) may not compress the transverse connections which will yield to durability problems.. During fabrication of the 2nd Gen Nu-deck, crown forming in the channel is a challenge because the bars across open channel needs to be cut and welded.	Usually, the panels cover full-width of the bridge. 1st Gen: 1.5 in. CIP concrete overlay with 8000 psi. 2nd Gen: No concrete overlay but the panels are cast with 0.5 in. additional thickness. The deck is finally diamond ground and an asphalt overlay is used as the riding surface. All strands are post-tensioned with final force of 38.9 kips regardless of sequence. Structural steel angles are used to set the panel elevation.

1st generation NU-deck panel: Longitudinal connection:  2nd generation NU-deck panel: Longitudinal connection:  **Crown detail in open channel at girder line.** 		Sources of information: Badie et al. (2006); Wipf et al. (2009b); Hanna et al. (2010a).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Modified NU-deck panel s Partial width, full-depth deck panels developed by the Iowa State University  	In 2006, the Mackey Bridge on 120th Street over Squaw Creek, Boone County, Iowa was replaced with a superstructure comprising of NU-deck panels (partial width, full-depth).	Following attributes are presented based on the information provided in the listed references. Width: Half-width of the bridge. Length: 10 ft Thickness: 8.25 in. Skew: Up to 60°. Concrete release strength of 4000 psi and 28-day strength of 6000 psi. Punching shear capacity of the panel is 135 kips. Flexural capacity of the panel is 263 kip-ft Reinforcement: Grade 60 mild reinforcing bars. Modulus of elasticity of 29,000 ksi is used in the design. Prestressing strands: Uncoated, 0.5 in. diameter, 7-wire, 270 ksi low relaxation steel	The panels are partial width. Hence, it is easy to develop the crown during deck placement. Longitudinal closure allows using this system for staged construction. The panels span from centerline to edge of the bridge, thus eliminate the overhang formwork The open channels provide adequate space for grouting the post-tensioning strands Panel supporting leveling devices are easily accessible from the channels  Partial width panels allow using smaller cranes.	Threading of post-tensioning strands through existing reinforcement is time consuming Durability is a major concern as the prestress and post-tensioning strands are placed in cast-in-place concrete joints. New concept and details; hence, past performance data is limited. Staggering of protruding reinforcement from deck panels at the longitudinal closure is a challenge Effectiveness of post-tensioning for compressing transverse joints needs to be evaluated because the post-tensioning is applied after grouting the	The channel consists of 2-layers of prestressing strands, 2-layers of mild steel reinforcement, 6-No. 2-layers of post-tensioning strands, and the leveling devices. According to the information provided in the literature related to the Mackey Bridge project, concrete mix for the channels contained the maximum aggregate size of 3/8 in. and 35% cement replaced with ground granulated blast furnace slag (GGBFS). The water-cementitious material ratio of the mix was 0.38. After adding a high-

Middle panel reinforcing steel:  End panel reinforcing steel: 	joints which will transfer some of the forces to the deck panel supporting system (i.e., girders).	range water reducer, slump of the mix was 8 in. The minimum concrete temperature at time of placement was 70^{o}F. Sources of information: Wipf et al. (2009b); Wipf et al. (2009c).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Partial-depth deck panels    	SH 249/ Louetta Road Overpass, Houston, TX (Bridge replacement) (1994) I-45/Pierce Elevated, Houston, TX (Bridge replacement) (1997) I-5/South 38th St Interchange, Tacoma, WA (Deck replacement) (2001) SH 66/Lake Ray Hubbard, Dallas, TX (Bridge replacement) (2002) SH 36/Lake Belton, Waco, TX (Bridge replacement) (2004)	Following attributes are presented based on the information provided in the listed references. Uncoated, 0.375 in. diameter transverse prestressing strands are provided at the mid-depth of the panels. Length: 8 ft Width: girder-to-girder span + 3in. to 3.5in. bearing on each girder Thickness: 3.5in. (typical) Thickness of CIP concrete deck on top: 4.5in. (typical) Concrete release strength of 4000 psi and 28-day strength of 6000 psi has been used. Skew: up to 15^{o} has been implemented, based on the information provided in listed references.	Requires no formwork for the CIP deck. Hence, disruption to feature intersected traffic can be minimized. Partial-depth panels can improve work-zone safety and construction speed. Fabrication and handling is simple compared to full-depth deck panels Construction is simple when compared to full-depth deck panels.	Reflective cracks in CIP deck over the transverse and longitudinal joints leads to durability problems and significantly reduce the bridge service life. CIP concrete deck requires extended bridge closure. Panels are typically fragile; therefore moving them frequently during precasting operations may result in a potential damage. The deck overhangs require formwork. The haunches need to be grouted and left intact to achieve required strength, before placing the CIP concrete; hence, there is a slight increase in the construction duration.	Mild steel reinforcement is provided in the cast-in-place concrete deck. The top surface of these panels is roughened to amplitude of 0.06 in. Grouting of haunches can be performed using high density low slump concrete, including high range water reducing admixture. Sources of information: Burkett et al. (2004); Hieber et al. (2005); PCI-NER (2001).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
NU-deck stay-in-place panels (NU-deck SIP panels) (Developed and tested at University of Nebraska in 1998)   	This detail has not been used in any bridge projects.	Following attributes are presented based on the information provided in the listed references. Width: Full-width of the bridge. Length: 4 to 12 ft Thickness: 4.5 in. Thickness of cast-in-place concrete overlay ranges from 3.5 in. to 4.5 in. Self-consolidated concrete was used with the release strength of 4000 psi and 28-day strength of 10,000 psi. Minimum top and bottom clear cover of 1 inch was used. Width of the longitudinal channel over the girders is 8 in. Compressive strength of grout used to fill the channel was 4000	Due to continuity in longitudinal and transverse directions, these panels may eliminate the potential of reflective cracking. These panels eliminate the need of overhang formwork. Wide channels provided over the girders facilitate grouting operation. Since a cast-in-place concrete deck is placed over the partial depth deck panels, use of a high quality grout may not be needed. Deck crown can be formed during cast-in-place concrete placement. Increased load capacity due to continuity and prestressing compared to traditional partial depth deck panel systems.	New concept and details; hence, past performance data is limited. Durability of the system is a concern because the prestressing strands run through cast-in-place concrete. Using CIP concrete requires extended bridge closure.	Panels are prestressed in the transverse direction. Prestressing strand arrangement is similar to the NU deck full depth panels. Prestressing helps the entire panel acts as a transversely continuous member over the girders. Reinforced pockets and shear keys are used to maintain continuity in longitudinal direction. A spiral splice is used to provide full bar yield strength of 60,000 psi.

<table>
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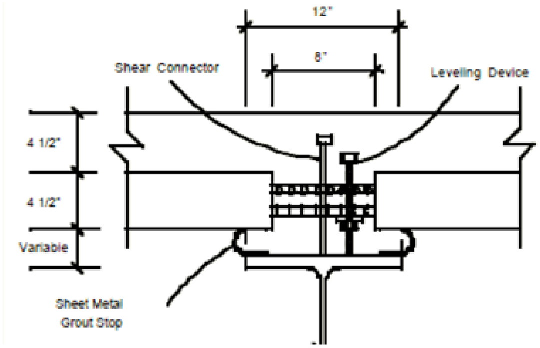
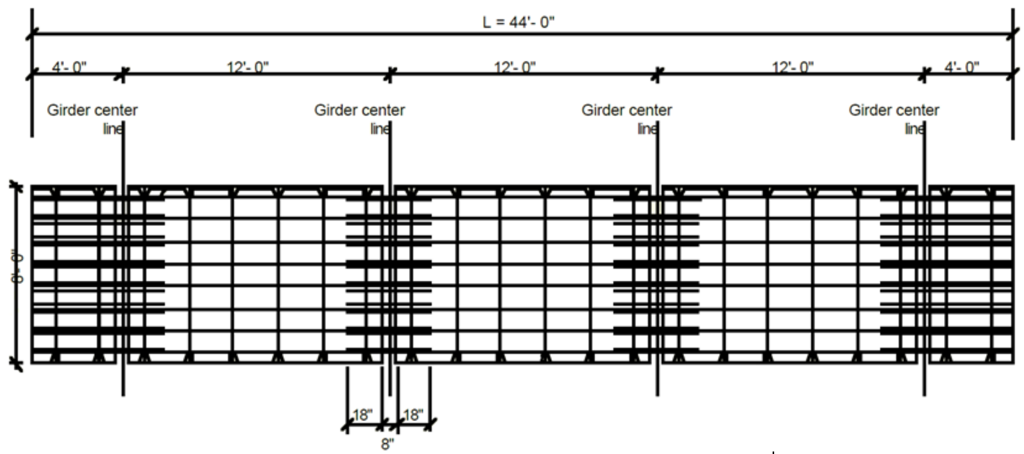
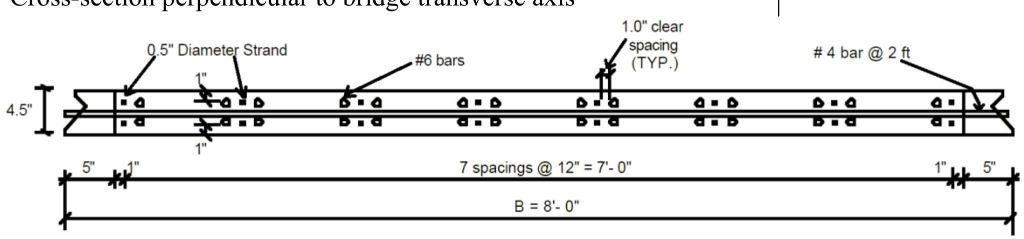
<td>Longitudinal connection:


Typical plan of the NU-deck SIP panel showing the transverse prestressing strands:


Cross-section perpendicular to bridge transverse axis


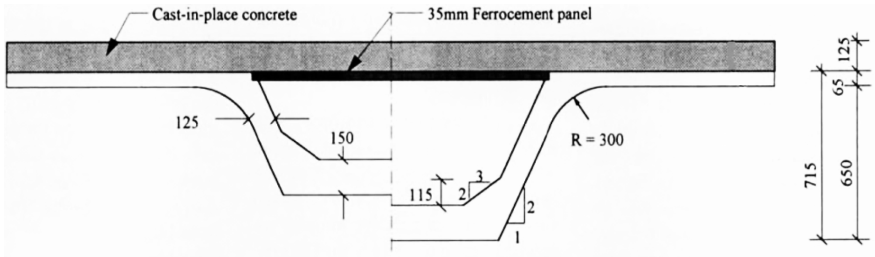
</td>
<td>psi.

Uncoated, 7-wire, 270 ksi low relaxation prestressing strands and grade 60 steel were used in the panels.
Skew: up to 30^{o}</td>
<td></td>
<td></td>
<td>Panels are supported over the girders using adjustable leveling devices that are placed within the channel over the girders.

Sources of information:
Badie et al. (1998); Versace and Ramirez (2004).</td>
</tr>
</table>

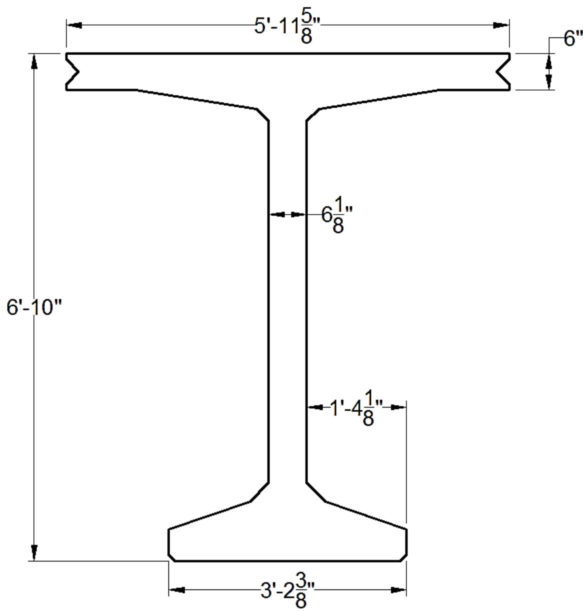
Modular Superstructure Elements and Systems

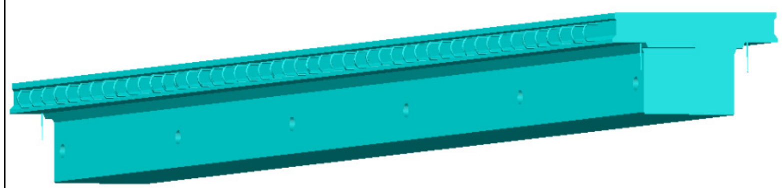
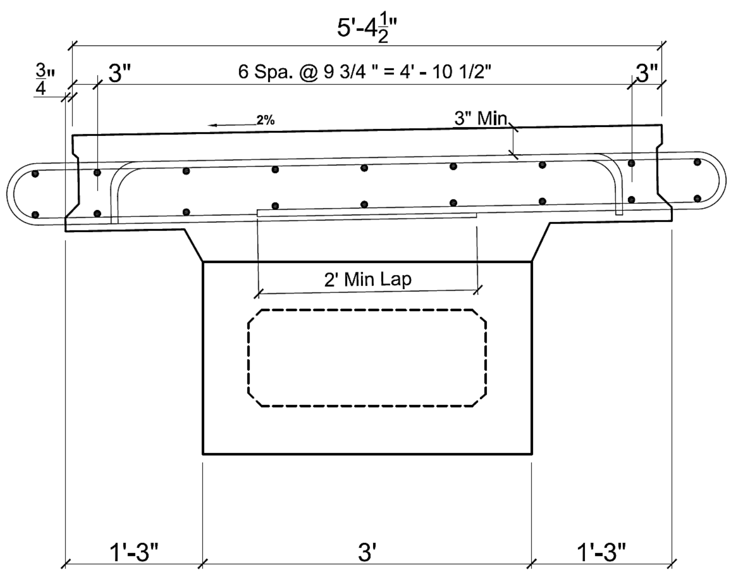
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast adjacent box-beam  (Source: PCSB 2011)  (Source: CPCI 2006)	Baldorioty de Castro Avenue Overpasses, San Juan, Puerto Rico (Bridge replacement) (1992) Mill street crossing in Epping, New Hampshire (Bridge replacement) (2004) Route 99/120 Separation Bridge (Bridge replacement) (2007)	Following attributes are presented based on the information provided in the listed references. Span ranges up to 127 ft. Depth ranges from 12 in. to 60 in. Width: 36 in. and 48 in. Concrete strength: 5000 psi to 7000 psi. There are records of using high performance concrete (HPC) with 28-day strength of 8,000 psi.	Great for sites with tight underclearance. Can accelerate construction by using a wearing surface over the girders. Does not require formwork for the cast-in-place concrete deck. High torsional stiffness. Can be used for constructing aesthetically pleasing structures. The entire bridge superstructure can be prefabricated with adjacent box-beams and kept ready for installation, before closing the traffic.	Reflective cracking is a major concern that leads to deterioration of the bridge superstructure. Not possible to inspect box-beam interior. Special details and cast-in-place construction are needed to develop continuity over piers. Not feasible for carrying utilities underneath Box-beams are difficult to fabricate as they involve multi-step fabrication process (Culmo & Seraderian 2010) Hard to replace a damaged girder when grouted post-tensioning is used in the transverse direction. Complete redesign of the transverse connectivity is essential as none of the existing designs are capable of mitigating reflective deck cracking (Aktan et al. 2009).	Sources of information: Stamnas and Whittemore (2005); Chung et al. (2008); MDOT-BDM (2011).

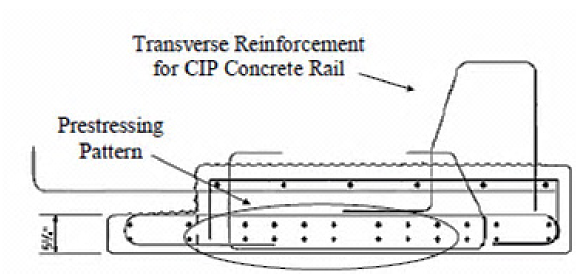
Element	Attributes	Benefits	Limitations	Remarks
Trapezoidal box girder  Note: All dimensions are in mm. **Project(s)** The open-top trapezoidal box girder has been used in several projects in Canada (CPCI 2006). But based on the data currently available, this system has not been implemented in any of the projects in the U.S.	Following attributes are presented based on the information provided in the listed references. Spans range: up to 95 ft Depth range: 20 in. to 28 in. Width range: 6.5 ft to 12 ft Concrete strength of the trapezoidal box section: 7400 psi. at release and 9000 psi at 28-day	Good for up to short-to-medium span bridges. Trapezoidal box girders could cover the entire bridge with relatively few girders compared to AASHTO box girders. Feature intersected is not disturbed during construction of the cast-in-place concrete deck. Transverse post-tensioning is not required. The relatively low weight of the girder (55 tons for a girder with 28 in. depth, 12 ft width, and 95 ft length) makes it feasible to be lifted with conventional lifting equipment.	New concept and details; hence, past performance data is limited. Requires cast-in-place deck which extends the project duration. Access to confined space of the box is limited. Hence, difficult to inspected deterioration that will initiate at the interior walls of the section. Trapezoidal box girders are limited to 95 ft span.	Sources of information: Badie et al. (1999).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast segmental box girder 	Seven mile bridge in Monroe County, Florida (built 1982) Ramp I over I-75 in Florida (built 1984) I-75/SR 826 (5 bridges) in Florida (built 1986)	The sections are standardized as AASHTO-PCI-ASBI segmental box girders. Following attributes are presented based on the information provided in Freyermuth (1997). Span: up to 200 ft Depth: 6 ft to 8 ft Width: 27 ft to 44 ft Specified length is 10 ft for each segment to facilitate shipping. Concrete strength: 5000 psi. Post-tensioning: 7-wire, 0.5 in or 0.6 in. diameter, grade 270 low relaxation strands	A cost effective option for very large projects. Segmental construction techniques are feasible for crossing large waterways Feasible for longitudinal launching applications Optimum for design-build projects A large number of bridges in service. (e.g., by year 2010 there are 68 bridges in Florida. Segmental bridges are widely used in California also.) Hence, data is available to evaluate the performance and improve the design.	Qualified personnel or inspectors required for quality grouting and post-tensioning Durability problems associated with post-tensioning systems. Challenges in inspecting post-tensioning system.	The publication Freyermuth (1997) contains standard section details in metric system. The publication Freyermuth (1997) specifies a span range of 100 ft to 150 ft, for span-by-span construction, and a span range of 100 ft to 200 ft, for the balanced cantilever construction. Sources of information: Freyermuth (1997); Blanchard et al. (2010).

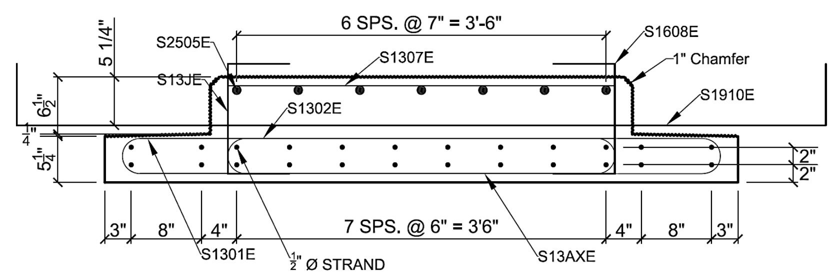
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Double-tee girder  	This section was the earliest development in the precast prestressed concrete sections. Used in several projects since 1950's, but low-volume roads only. Recent project: Russian River Bridge (Superstructure replacement) (2006)	Following attributes are presented based on the information provided in the listed references. Standard span range: 32 ft to 65 ft Depth range: 27 in. to 36 in. Width range: 5 ft to 8ft Concrete strength: 4000 psi. at release and 7000 psi at 28-day Prestressing strands: 0.5 in. or 0.6 in. dia.	Most of the prefabricators are familiar with the section as it is widely used in parking structures. Top flange servers as formwork for CIP concrete deck and working surface for the construction crew Single pour production; hence, it is easy to fabricate compared to box-beams Can accommodate utilities underneath	Requires a CIP concrete deck which extend duration of bridge closure Producers / manufacturers reported vertical and diagonal cracks in the stems of double-tee girder, developed during handling process due to lateral force on the stem. Extreme care should be taken during handling, so that lateral forces are not applied (PCI Committee 1983) The deck slab without transverse post-tension may be a source of durability concern due to potential cracking. Limited for short span bridges with low traffic-volume.	Generally used with CIP concrete deck Sources of information: PCI committee (1983); Bergeron et al. (2005); Chung et al. (2008); Li (2010).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Decked bulb-tee girder   	This section emerged from the bulb-tee girder section. States like Utah and New England utilized this section in several projects. Recent projects: Graves avenue over I-4, Florida (Superstructure replacement) (2006) Route 31 bridge in Lyons, New York State (Bridge replacement) (2009)	The section is standardized. Following attributes are presented based on the information provided in PCI-BDM (2001) and UDOT (2010). Span range: up to 180 ft (UDOT 2010) Depth range: 35 in. to 98 in. Top flange width range: 4 ft to 8ft Concrete strength range: 6500 psi to 8500 psi Prestressing strands: 0.5 in. or 0.6 in. dia.	Can accelerate construction because only a wearing surface is needed over the girders. Section has been used in several projects; hence, structural durability performance data is available. Single pour production; hence, it is easy to fabricate compared to box-beams Can accommodate utilities underneath. More capacity and efficiency than AASHTO type V and VI girders (Bardow et al. 1997) Due to modular nature of the units, the entire bridge superstructure can be prefabricated and kept ready for installation, before closing the traffic	Depth of about 8 ft, not feasible for bridges with underclearance limitations Limited to roadways with ADT up to 30,000 (UDOT 2010) Possibility of flange-to-flange connection failure unless moment transfer connections are used.	Developed in 1969 by Arthur Anderson based on the standard tee girder section details. Standardized as AASHTO/ PCI deck bulb-tee in 1988. Commonly used flange-to-flange connection: Female-to-female grouted shear key or flange-to-flange welded plate connection Sources of information: PCI-BDM (2001); Shah et al. (2006); Graybeal (2010); UDOT (2010); CPMP (2011); Culmo (2011).

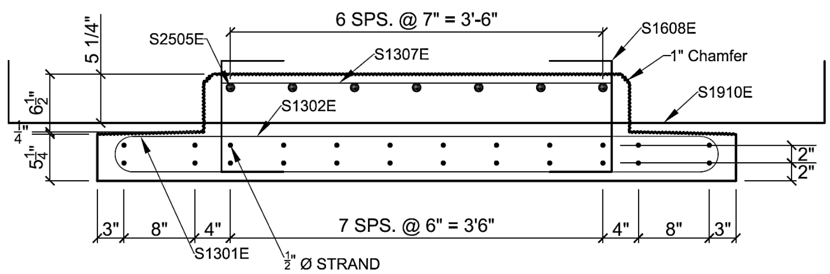
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Decked box-beam  (Source: Michigan M-25 Bridge over White River CAD)  (Source: Michigan M-25 Bridge over White River Bridge plans 2010)	M-25 Bridge over White river, Michigan (Bridge replacement) (2011).	Following attributes are presented based on the information provided in MDOT M-25 Bridge plans. Design is similar to that of a spread-box girder bridge, but with additional connection detailing. Span: 47 ft Depth of the module including the deck is 3 ft Top flange width of the module is 5 ft - 5 in. Specified compressive strength of decked box-beam modules at 28-day is 7000 psi. Post-tensioning: 7-wire, 0.6 in. diameter, grade 270 low relaxation strands	Shallow depth enables using at sites with tight underclearance. Does not require a cast-in-place concrete deck. High torsional stiffness. Can be used for constructing aesthetically pleasing structures. Feasible for carrying utilities underneath.	New concept and connection details; hence, past performance data is limited. Not possible to inspect box-beam interior. Special details and cast-in-place construction are needed to develop continuity over piers. Hard to replace a damaged module when grouted post-tensioning is used through the cast-in-place diaphragms. Box-beams are difficult to fabricate as they involve multi-step fabrication process (Culmo & Seraderian 2010) Not possible to detect deterioration inside the concrete box. Deck reinforcing and casting process should be performed promptly, before the box-beam concrete starts setting.	During the manufacturing process, primarily the box-beam is casted, then the deck reinforcement is placed on top of box-beam, and finally the deck is casted. Source of information: MDOT M-25 Bridge plans (2010); MDOT-BDM (2011).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Inverted-T precast slab Design is based on French Poutre Dalle system  	Projects listed here are Bridge replacements. Truck Highway (T.H.) 8 bridge over Center lake channel, Center City, MN (2005). T.H. 72 bridge over Tamarac river, Waskish, MN (2005). T.H. 65 bridge over Groundhouse river, Kanabec county, MN (2007). T.H. 65 bridge over Ann river, Kanabec county, MN (2007). T.H. 76 bridge over South fork of Root river, Houston county, MN (2007). T.H. 238 bridge over Swan river, Morrison county, MN (2009). T.H. 238 bridge over Pike creek, Morrison county, MN (2009). T.H. 60 bridge over Cannon river, Rice county, MN (2009)	Following attributes are presented based on the information provided in the listed references. Span range: 20 ft to 65 ft Width: 6 ft Structure depth: 30 in. for 65 ft span. Structural depth includes a 24 in. deep precast section and 6 in. thick cast-in-place concrete deck Concrete strength of the precast inverted-T precast slab element is 6,500psi. Cast-in-place concrete deck strength is 4,000psi. Prestressing strands: 0.5 in. dia.	High span-to-depth ratio; hence ideal for projects with underclearance limitations. Does not require formwork for the cast-in-place concrete deck.	New concept and details; hence, past performance data is limited. Requires cast-in-place deck which extends the project duration. Degree of moment continuity provided by the longitudinal connection detail needs to be evaluated. Limited to short span bridges due to individual Element weight.	Composite action between precast section and CIP deck is established through shear reinforcement (#6 bars). The longitudinal reinforcement detail used at the longitudinal joint is expected to alleviate reflective deck cracking. Transverse hooks with 90^{o} angle protruding from webs enables connectivity between reinforcement cage and the girder. Source of information: Bell II et al. (2006); French et al. (2011).

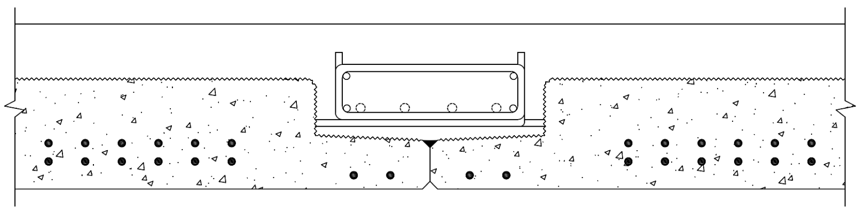
Old detail of the inverted-T precast slab:



(a) End section

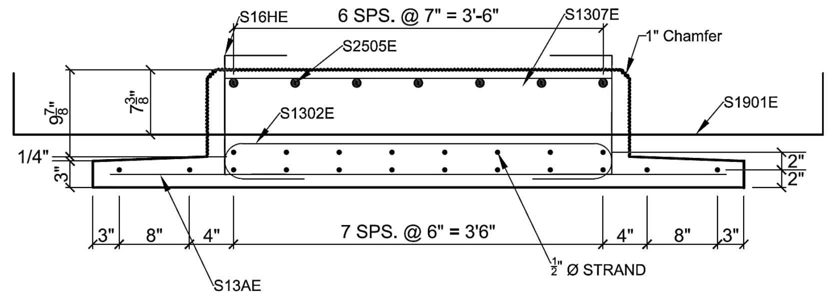


(b) Midspan section

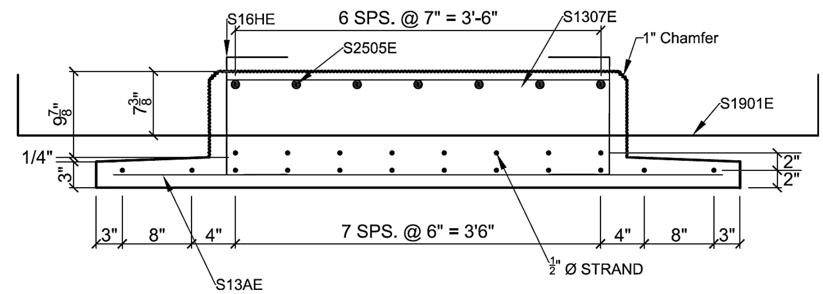


(c) Longitudinal reinforcement cage

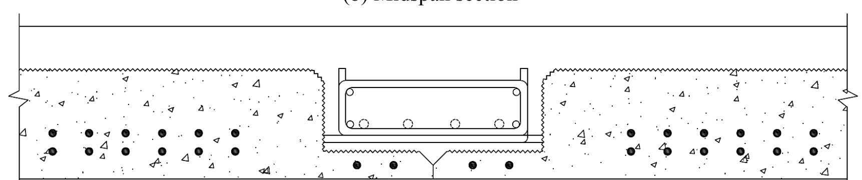
New detail proposed by the NCHRP for the inverted-T precast slab:



(a) End section



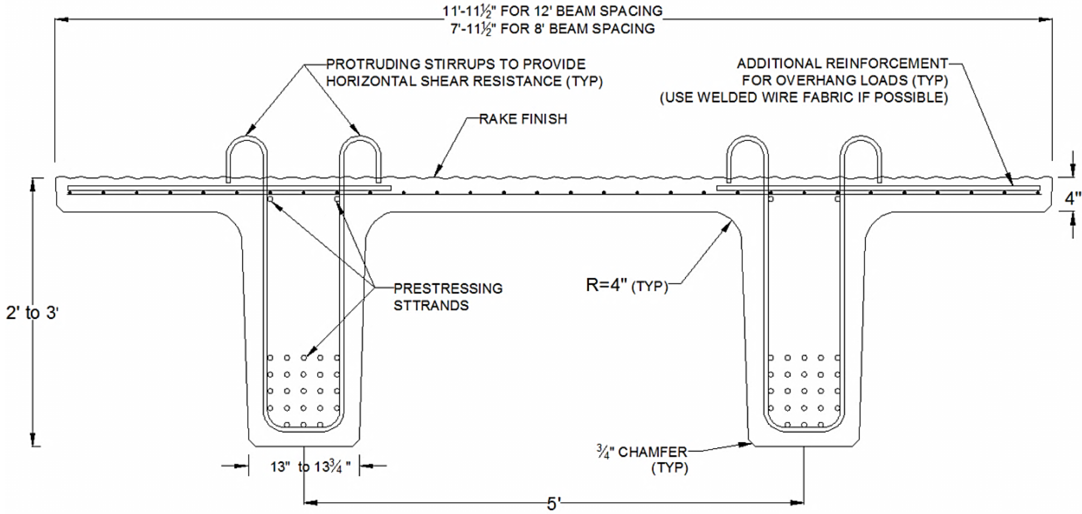
(b) Midspan section



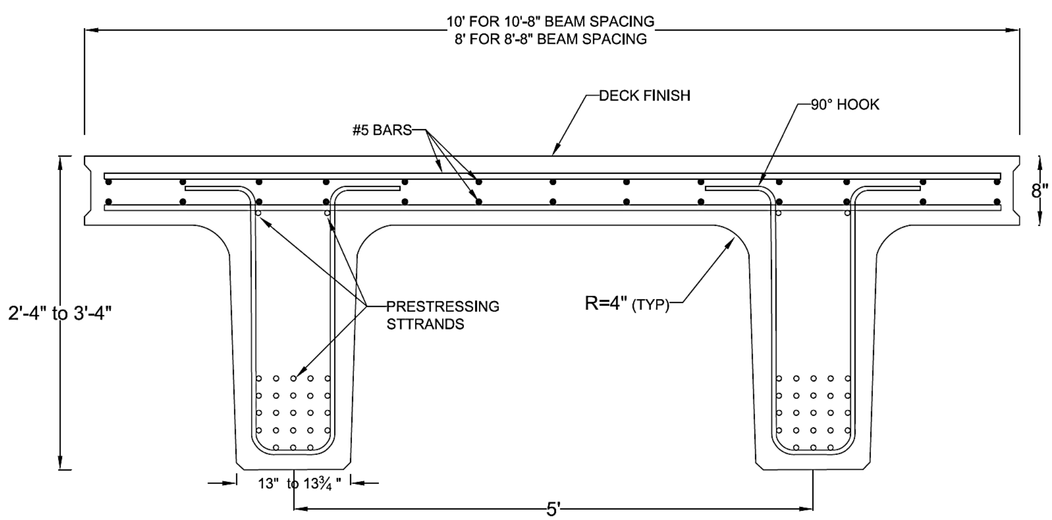
(c) Longitudinal reinforcement cage

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Northeast Extreme Tee (NEXT) beam 	NEXT F –Route 103 bridge over York river in York, Maine (Bridge replacement) (2011) NEXT F – Queen's Blvd over Van Wyck Expressway in New York City (Bridge replacement) (2012) NEXT D – White Boulevard Bridge, Florida (Bridge replacement) (2011)	The cross-section dimensions and span length of both F and D sections shown below are based on the information provided in the listed references. Span ranges from 40 ft to 90 ft Depth of the section ranges from 24 in. to 36 in. with 4 in. increment. Width of the section ranges from 8 ft to 12 ft Stem spacing is 3 ft for 8 ft wide section and 6 ft for 9 ft –12 ft wide sections Stem thickness ranges from 11 in. to 13 in. Prestressing strands: 0.6 in. dia. According to PCI NE (2011) span charts, concrete strength is as follows: 8000 psi at release and 10,000 psi at 28-day. 6000 psi. at release and 8000 psi. at 28-day. 4000 psi. at release and 6000 psi. at 28-day.	Ideal for projects with underclearance limitations. Greater load carrying capacity than standard double tee and box girders. The stem could incorporate more prestressing strands compared to standard double tee girders. Single pour production A range of beam sizes could be produced with one set of formwork. Since the depth, spacing, and size of stems are standardized. No intermediate diaphragms Due to modular nature of the units, the entire bridge superstructure can be prefabricated before closing the traffic. Good for short and up to short-to-	New concept and details; hence, past performance data is limited. NEXT F beam requires 8 in. CIP concrete deck which extends project duration Durability of the longitudinal connections between NEXT F and D beams is a concern. Shipping and handling limitations due to heavy weight.	Approved in CT, MA, ME, NH, RI, VT, DE, MD, NJ NEXT F beam weighs 120 kips for 90 ft length with 4 in. thick flange NEXT D beam weighs 160 kips for 90 ft length Sources of information: Calvert (2010); Culmo and Seraderian (2010); PCI NE (2011); Culmo (2011).

<table>
<tr>
<td>NEXT F beam: flange serves as stay-in-place form for cast-in-place concrete deck



NEXT D beam: flange serves as complete deck


</td>
<td>medium span bridges.</td>
<td></td>
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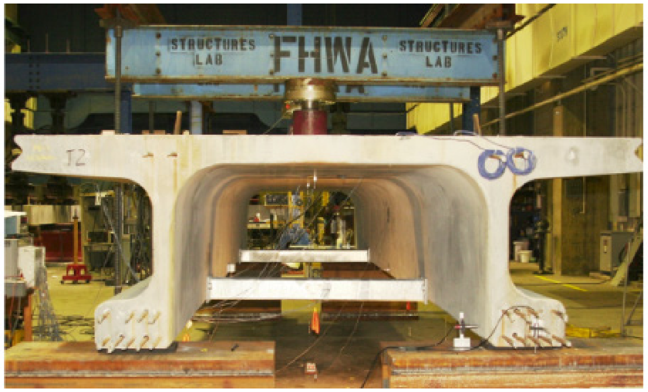
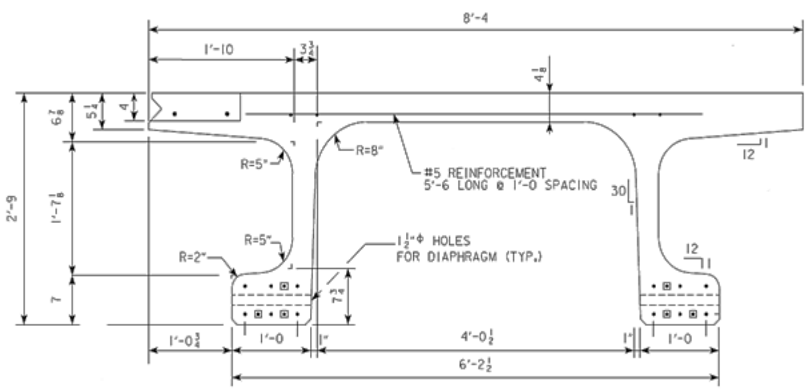
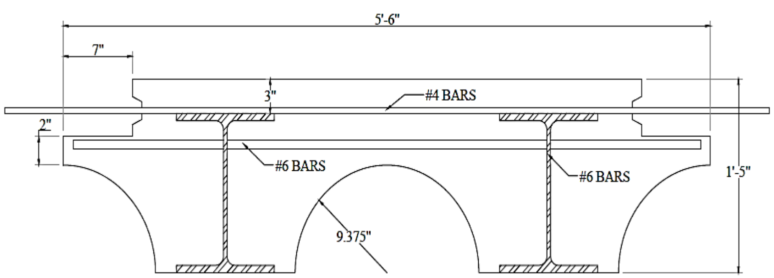
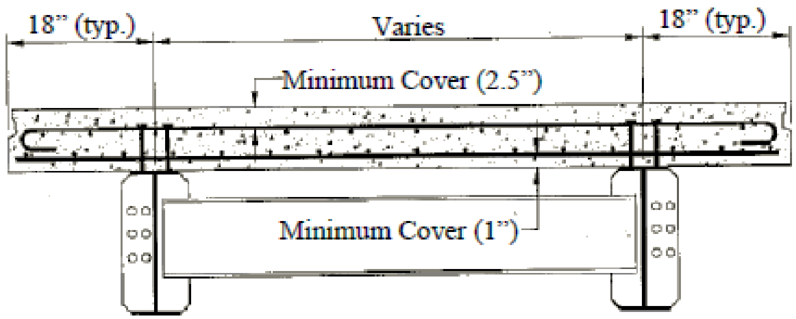
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Pi-girder **2nd generation UHPC pi-girder**  	Jakway Park Bridge, Buchanan County, Iowa (Bridge replacement) (2008)	Following attributes are presented based on the information provided in the listed references. Span: **up to 65 ft** (computed based on limiting tensile stresses to the cracking threshold) Graybeal (2009) estimated **maximum span of 87 ft** with increased prestressing force. Depth: 33 in. Weight: 932 lb/ft Compressive strength of Pi-girder UHPC at release is 12,500 psi and at 28-day is 21,500 psi. Steel tube diaphragms at 1/4rth span and midspan.	Can accelerate construction because only a wearing surface is needed over the girders. The system is good for sites with underclearance limitations. Good for short and up to short-to-medium span bridges. The unhydrated cement content of UHPC would provide crack-sealing capabilities through secondary hydration. Cost savings could be achieved by using partial prestressing in UHPC pi-girder design (i.e., allowing cracking on the bottom of the bulbs under maximum service loads). Transverse mild steel reinforcement could be used in the pi-girder deck, if needed	New concept and details; hence, past performance data is limited. Expensive due to proprietary UHPC. Investigation of torsional properties of 2nd generation pi-girder and its ability to resist eccentric loading, for longer spans is required. Lighter and slender section may amplify dynamic loads on the bridge and need be investigated.	1st generation UHPC pi-girder was developed at Massachusetts Institute of Technology in 2002. For fabricating the UHPC pi-girder, batching of UHPC is performed in the ready-mix concrete trucks In the pilot project, the pi-girder ends were seated on neoprene bearing pads and were encased in CIP concrete diaphragms The girders are steam cured using thermal blankets for 48 hrs at 195°F Sources of information: Graybeal (2009); Matt et al. (2011).

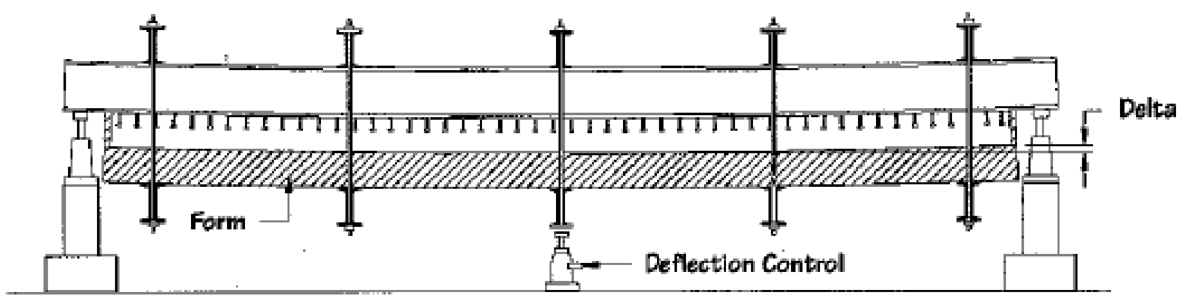
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast modified beam in slab Longitudinal joint:  Complete bridge:  Typical section of the module:  	Mt. Vernon road bridge, Black Hawk County, Iowa (Bridge replacement) (2006) Marquis road bridge, Black Hawk County, Iowa (Bridge replacement) (2007)	Following attributes are presented based on the information provided in the listed references. Span: 40ft to 50ft Width: 4.5ft to 5.5ft Consists of embedded W14 sections spaced at 2 ft-9 in. Depth of the module: 17.25 in. at girders and 7 in. in between Skew: up to 45° Compressive strength at 28-day is 5000 psi Structural steel strength: 50,000 psi Concrete mix was developed with water-cement ratio of 0.43, cement content of 624 lb/cy, and an air entraining admixture.	Can accelerate construction because only a wearing surface is needed over the modules. The system is good for sites with underclearance limitations. The steel girders are embedded in concrete, therefore protected against corrosion and maintenance.	New concept and details; hence, past performance data is limited. Good for short span bridges only.	Original module was developed in 1997 and finally modified in 2004 to formally known as **precast modified beam in slab bridge** module. The module was developed by the Iowa State University Bridge Engineering Center in cooperation with Blackhawk county. Before placing concrete in the longitudinal joints, 14 in. long #4 bars are placed at the center of #4 reinforcing bars protruding from each module. The #4 protruding reinforcing bars are spaced at 15 in. center-to-center for each module. Source of information: Klaiber et al. (2009).

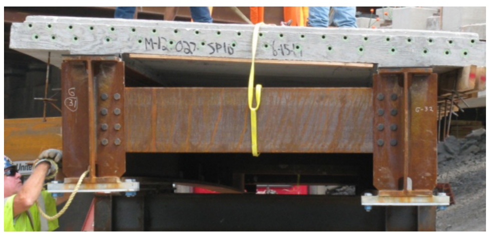
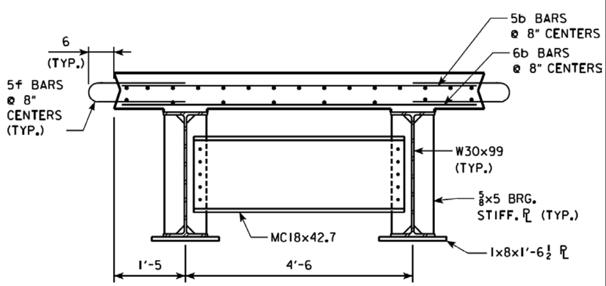
System	Project(s)	Attributes	Benefits	Limitations	Remarks
INVERSET^{TM} system (Proprietary) Developed in Oklahoma in early 1980's and tested in 1997 (PennDOT 1997)  	Used in several projects in New York and Pennsylvania, since 2000. Creek Road over I-295, Burlington county, NJ (Superstructure replacement) (2010) Eastern Ave Bridge over Kenilworth Ave, NE (Bridge replacement) (2010)	It is a standard proprietary module. Following attributes are presented based on the information provided in the listed references. Span: up to 100 ft Steel girder: W 30x99 Depth of the module: girder depth + deck thickness. Deck thickness: 7.5 in. Width: up to 12 ft Skew: up to 60^{o} Deck compressive strength at 28-day is 8500 psi	Had been used in several projects, thus performance data is available. Can accelerate construction because only a wearing surface is needed over the module. The system is good for sites with underclearance limitations Good for short and up to short-to-medium span bridges in non-corrosive environments. The deck of the system will be in compression under its own weight, therefore, prevents transverse deck cracking; hence, and improves the deck durability. Due to modular nature, the entire bridge superstructure can be prefabricated and kept ready for installation, before closing the traffic	Increased cost due to proprietary nature. The pre-compressed deck of the module could not be replaced in the field, thus requires removal of the entire module Steel girders are used in this system; hence, it is expensive to maintain than concrete girders. Weathering steel is useful for corrosion prevention; however, not good for states where deicing salts are used, as it is sensitive to salt-laden environments. Further, there are several durability concerns with regard to fabrication and maintenance, such as: special welding requirements, and maintenance of the nearby structures that develop rust stains due to normal surface weathering of the weathering steel.	The INVERSET system is casted upside-down; hence, the deck is precompressed due to self-weight of the module Sources of information: Versace and Ramirez (2004); Pate (2008); Fort Miller Co. (1998; 2010); NJDOT (2010); Chamberland and Patel (2011).

INVERSET™ module

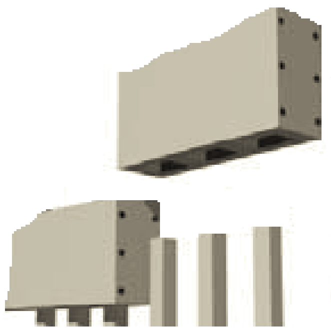
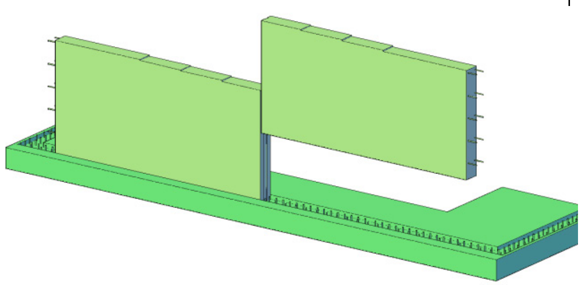


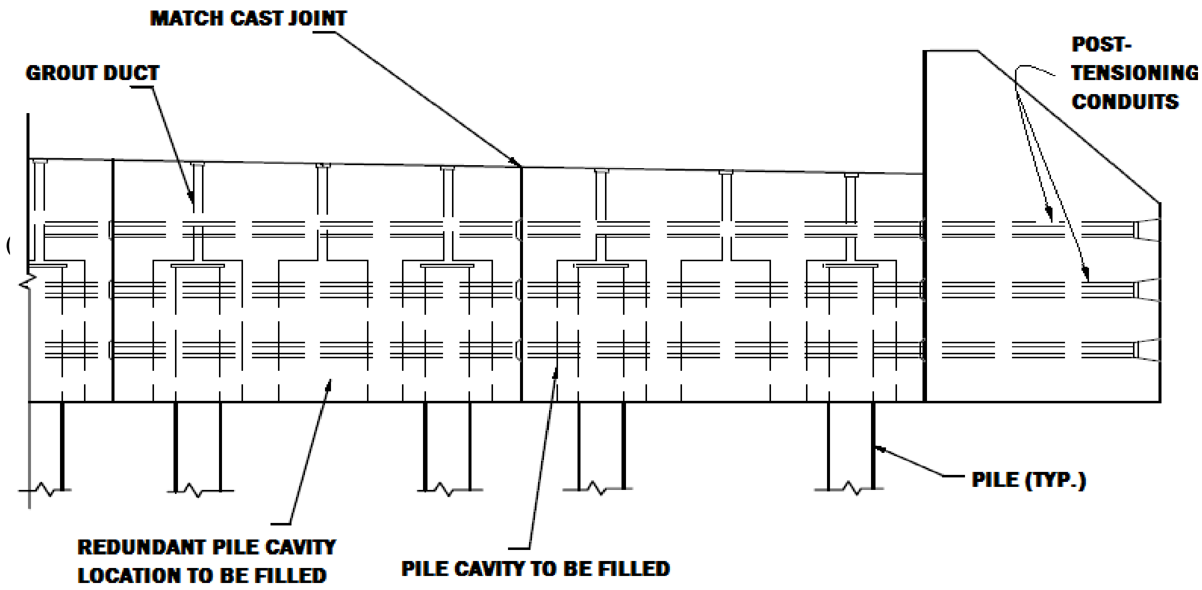
INVERSET™ module casting process

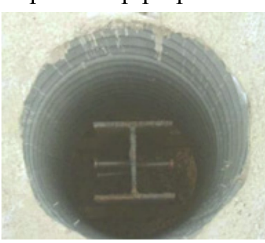


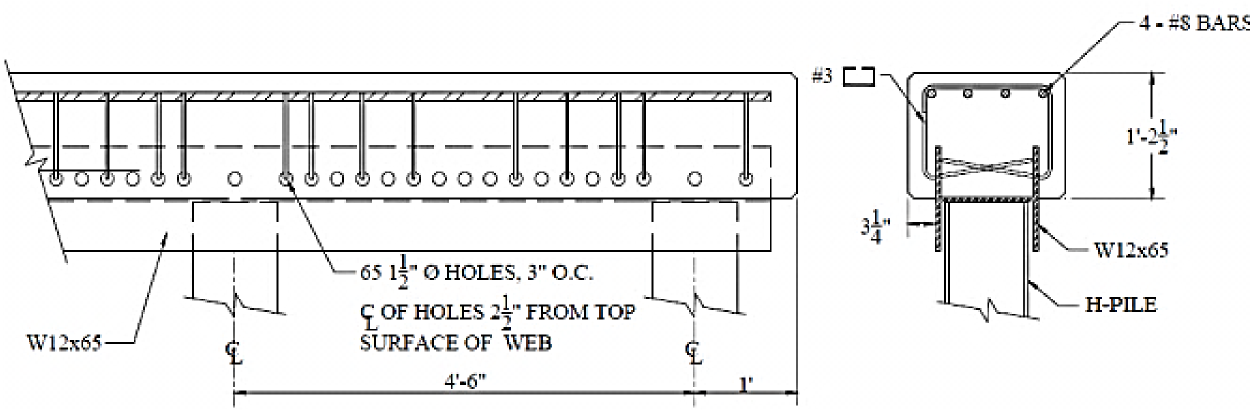
System	Project(s)	Attributes	Benefits	Limitations	Remarks
Decked steel girder system (also referred as decked steel girder module)   Above pictures are from the MassDOT Fast14 project The following detail is used in the Iowa project. 	I-93 Fast 14 Project, Medford, MA. Superstructures were replaced using this system in 2011. Keg Creek Bridge Replacement in Pottawattamie County, IA. Used for the superstructure of a full structure replacement project in 2011.	Following attributes are presented based on the information provided in the listed references. Longest span used until now is 73.2 ft (MassDOT 2011) Steel girder: W 30x99 (depth: 29.7 in.), ASTM A709 grade 50W Width: 8 ft to 9 ft Precast deck: 7.5 in. to 8 in. thick Deck compressive strength at 28-day is 4000 psi to 5000 psi	Can accelerate construction because only a wearing surface is needed over the module. The system is good for sites with underclearance limitations Good for short and up to short-to-medium span bridges in non-corrosive environments. The decked steel girder modules are more biddable by contractors, as they can be prefabricated with conventional designs and processes (non-proprietary). Due to modular nature of the units, the entire bridge superstructure can be prefabricated and kept ready for installation, before closing the traffic	New concept and details; hence, past performance data is limited. Steel girders are used in this system; hence, it is expensive to maintain than concrete girders. Weathering steel is useful for corrosion prevention; however, not good for states where deicing salts are used, as it is sensitive to salt-laden environments. Further, there are several durability concerns with regard to fabrication and maintenance, such as: special welding requirements, and maintenance of the nearby structures that develop rust stains due to normal surface weathering of the weathering steel.	The module was developed under SHPR II project and is non-proprietary. In MassDOT project, the modules were placed adjacently and connected through a reinforced high-early strength concrete closure pour: 2000 psi was achieved within 4 hrs of final set and 4000 psi at 28-day. Iowa project used full, moment-resisting ultra-high performance concrete (UHPC) joints at piers and between deck panels. The bridge deck was diamond grind for profile improvement after UHPC closure pour reached minimum of 14,000 psi. Sources of information: Shutt (2009); LaViolette (2010); MassDOT (2011); IowaDOT (2011); Moyer (2011).

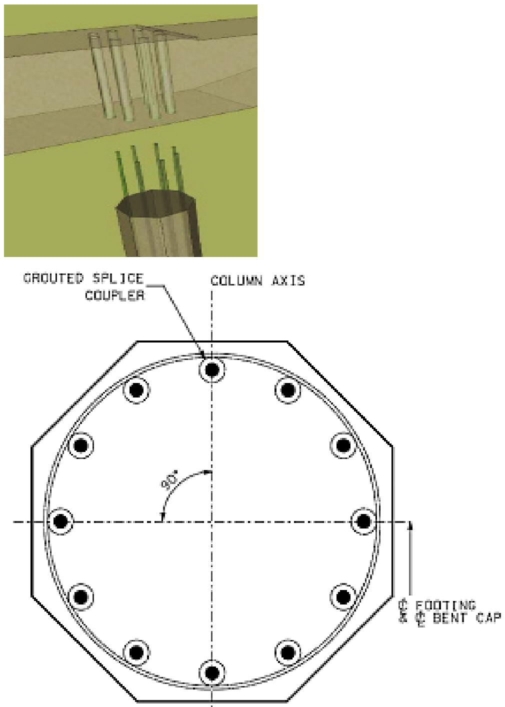
Substructure Elements

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast abutment stem/wall (a) Precast abutment stem segments on piles (Source: Culmo 2009)  (b) Precast abutment wall segments on footing (also known as cantilever abutment) (Source: Michigan M-25 Bridge CAD drawing) 	Precast abutment stem segments on piles – Upton Maine Bridge, Maine (2004) Precast abutment wall segments on footing – Epping, New Hampshire (2005) Precast abutment wall segments on footing – M-25 Bridge over White river, Michigan (2011)	The sections are not standardized. Following attributes are presented based on the information provided in the listed references. Height of abutment stem: 4ft Height of abutment wall: 7ft to 10ft Length of each segment: up to 14 ft Thickness: 2 ft for abutment wall, and 3 ft to 4 ft for abutment stem 28-day compressive strength of precast abutment segments is 5,000 psi.	Abutments precast in segments will alleviate the shipping and handling limitations. Abutment weight can be reduced by creating redundant cavities. **This concept helps to achieve light-weight components for alleviated shipping and handling.** Large prefabricated elements are advantageous for remote locations where access to the ready-mix concrete is difficult.	Abutment segments usually weigh 60 kips or greater; therefore, transportation and mobility of large cranes should be investigated. Grouting large cavities will be challenging because the grout manufacturers may limit the fill depth. A level subbase is required for the abutments on piles. The pile cavity forms makes the fabrication process challenging. Tighter tolerances are required for the pile driving operation. Tighter tolerances are required for proper fit-up between the precast elements while using grouted splice sleeve connections. Proper grouting of the channel in spread footing, at the abutment stem connection is	The abutments could be integral, or semi-integral, based on the design. However, it is encouraged to use semi-integral abutments because it is easy to replace bridge superstructure as needed and also minimize the stresses developed in the system due to thermal loads. The abutment wall segments on spread footing use grouted splice sleeve connections. The redundant pile cavities in an abutment stem can be filled with grout only.

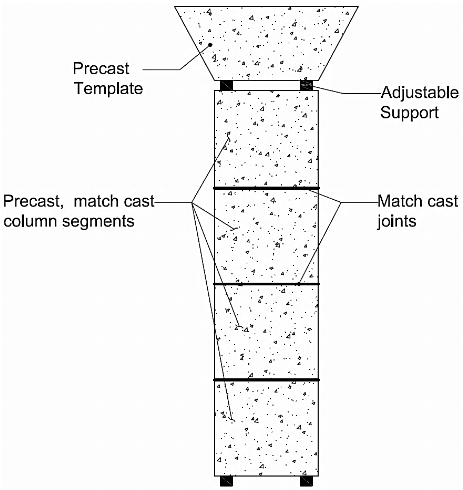
(c) Precast abutment stem segments with redundant pile cavities (Source: Culmo 2009)  	critical. Proper grouting of the splice sleeve connection is critical. Grouting of the vertical shear keys between the abutment segments (figure-b) should be investigated. Projects have reported joint forming and sealing issues under significant pressure head due to height of the abutment.	Sources of information: Stamnas and Whittemore (2005); Culmo (2009); UDOT (2010); PCI-BDM (2001).

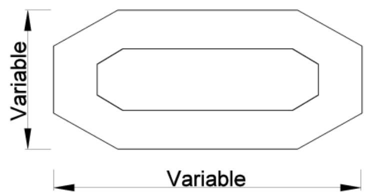
Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast pile cap **with Corrugated Metal Pipe (CMP) cavities** Using as a precast pile cap:  Using as a precast bent cap :  H-pile and pipe-pile in the CMP cavity:  	Mackey Bridge on 120th Street, over Squaw Creek, Boone County, Iowa (2006)	The sections are not standardized. Following attributes are presented based on the information provided in the listed references. Height: 3ft to 3ft-6 in. Length: varies (usually full-width of superstructure) Width: 3ft to 4ft 28-day compressive strength unusually specified is 5000 psi or greater. Yield strength of reinforcing steel is 60,000 psi.	Potential for using as a bent cap as well as a pile cap. Good for bridges with shallow embankments and abutments. Could accommodate large tolerances. Potential of precasting the pile caps at a staging area near the bridge site. The CMP cavities allow easy and effective grouting/ concreting of the connection. **The use of full-depth CMP cavities in a component also provides the benefit of achieving light-weight component, for alleviated shipping and handling.**	Large amount of grout/concrete is required, leading to additional curing and setting time. May face challenges if grout is to be used because manufacturers limit fill depth for neat grouts. Shipping and handling limitations due to wide and heavy section. If used as a bent, formwork is necessary for supporting the section until the grout/concrete achieves required strength. The CMP cavities in the section were observed to create localized tensile stresses on sides; this aspect requires further investigation (Wipf et al. 2009a)	Projects where an integral abutment is desired, a CIP portion is constructed on top of the precast pile cap to form the integral abutment. Mechanical splices are embedded in the pile cap for connecting the reinforcement of the CIP portion. The CMP cavities in a pile cap or bent cap can be filled with either grout or high early strength self-consolidating concrete. Sources of information: Wipf et al. (2009a); Wipf et al. (2009b); IowaDOT (2011).

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast pile cap **with embedded wide flange sections**  	Mt. Vernon Road bridge, Black Hawk County, Iowa (2006) Marquis road bridge, Black Hawk County, Iowa (2007)	The sections are not standardized. Following attributes are presented based on the information provided in the listed references. Height: 1 ft-2.5 in. + ½ flange-width of W 12 section Length: varies (usually full-width of superstructure) Width: 1ft-6in. 28-day compressive strength usually specified is 5000 psi	The pile cap section could be used at abutments and bent caps at the bridge site with shorter substructure (e.g., trestles over streams or bays). The concrete section in composite action with W-section allows for **increased load carrying capacity with reduced section depth (reduced weight),** compared to conventional concrete section.	Suitable for connecting steel sections. Difficult to establish connection with circular sections or concrete sections. Shipping and handling limitations due to wide and heavy section. As the W section is exposed to the environment, corrosion limitations are likely. Field cutting and welding operation of the piles require certified workers. Precision is required in field cutting and grinding of the piles, to obtain required elevation of the pile cap. Overhead field welding is a challenging process.	The pile caps are fabricated by casting concrete around the upper half of W 12 section oriented for weak axis bending. During fabrication, holes are torched in the flange portion which is to be embedded into the concrete. Stirrups are inserted, and concrete is allowed to flow through these holes. After pile driving is completed, the piles are cut off to the desired elevation and then field welded with W section of the pile cap. Sources of information: Klaiber et al. (2009); Wipf et al. (2009a)

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast columns (a) Octagonal section (Source: UDOT 2010)  (b) I-section (Source: Shahawy 2003) 	I-section (Figure-b): U.S.-41/ Edison Bridge over Caloosahatchee River, Fort Myers, FL (1991) Octagonal section (Figure-a): I-287 in Westchester County, NY (1999) Circular section (Figure-c): Parkview Avenue over U.S. route 131, Kalamazoo, MI. (2008) Circular section (Figure-c): I-5, Grand Mount Interchange Bridge, WA (2011) Rectangular/square section (Figure-d): Keg Creek Bridge Replacement in Pottawattamie County, IA (2011) Octagonal section (Figure-a): used by FDOT, TXDOT, UDOT, and PCI-NE in several projects.	The sections are not standardized. Dimensions: vary based on the bridge configuration. Conventional material strengths and design procedures are used. 28-day compressive strength usually specified is 4000 psi.	Octagonal and rectangular columns are easy to fabricate as they can be casted in a horizontal position. Octagonal and rectangular columns are easy to transport. Fabricators could build long forms and cast multiple columns at one time. Octagonal column's seismic performance is identical to a round column. I-section columns are good for tall structures where increased moment of inertia is required, complying the weight limitations. Prestressing could be used for more durable and taller columns. Great durability in corrosive environments. **The I-section precast columns are optimal for supporting inverted-U section**	Shipping and handling may be a limitation depending on the height and weight of column. Fabrication of round column is challenging due to the vertical casting requirement. Rectangular section is not an optimal cross-section due to redundant material. Tighter tolerances are required to avoid tilting of columns and for aligning splice bars with the sleeves. If prefabricated bent cap is used, feasibility of slight tilting of columns during assembly should be considered. This may be considered when specifying bent cap tolerances.	Round columns show better seismic performance compared to other sections. Octagonal shaped columns are preferred instead of round columns due to complex fabrication and vertical casting process of the latter. The columns are connected to the foundation and pier-cap using grouted splice sleeves. Sources of information: LoBuono (1996); Shahawy (2003); UDOT (2010); Khaleghi (2011); Attanayake et al. (2012).

(c) Circular section (Source: Courtesy of MDOT)   (d) Rectangular/square section (Source: IowaDOT 2011)  	**bent cap that is designed to achieve light-weight component.**		

<table>
<tr><th>Element</th><th>Project(s)</th><th>Attributes</th><th>Benefits</th><th>Limitations</th><th>Remarks</th></tr>
<tr>
<td>Precast segmental columns
Elevation:


Section details:


Short span bridge with precast segmental columns:
</td>
<td>Seven mile bridge, FL (1982)
Linn Cove viaduct, NC (1983)
SH-249/ Louetta Road overpass, Houston, TX (1994)
U.S. route 183 elevated Austin, TX (1997)
Victory Bridge, NJ (2005)</td>
<td>The sections are not standardized. Following attributes are presented based on the information provided in the listed references.

Height of each segment: varies from 3 ft to 6 ft

Length of cross-section: varies from 4 ft to 10 ft

Width of cross-section: 4 ft

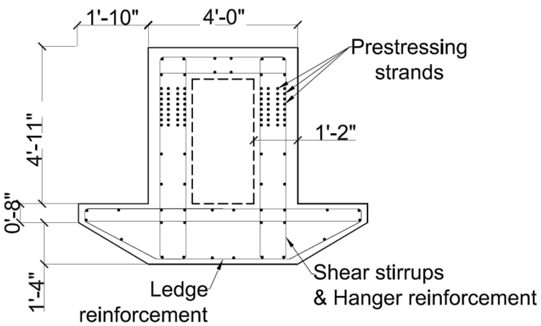
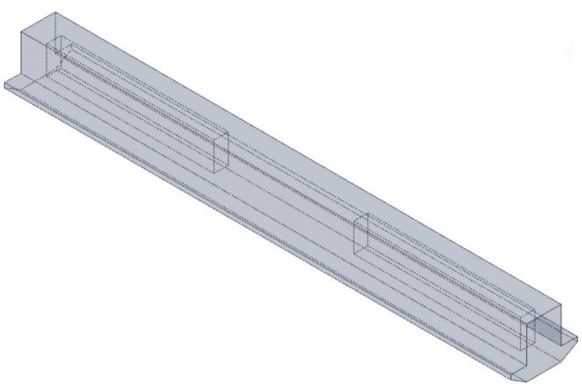
28-day compressive strength usually specified is 5000 psi</td>
<td>Weight of the segments can be limited to match the available resources.
Good for short and short-to-medium span bridges.
Desired column height could be achieved easily by increasing/ decreasing the number of segments and their individual heights.
Potential of eliminating the bent cap beam.
Provides ease in shipping and handling compared to full height precast columns.
The match-cast joints between the segments allow accelerated construction.
The column segments consist of hollow core that leads to reduced weight and thus can be erected using standard construction equipments and alleviates assembling process.
The precast template helps in aligning the pier with bent cap or the girder elevation.
Hollow portion of the segment could accommodate drainage ducts.</td>
<td>Vertical post-tensioning is required to connect all the segments including the foundation.
The footing should be specifically designed to accommodate PT ducts.
Requires match-casting of the segments during fabrication.
If the segments are not match-cast, then they should be connected through a grout layer which requires forms and curing at the site.
Appropriately labeling and delivering the segments is necessary, which may otherwise lead to fit-up limitations and extended project duration.
A challenging construction process.</td>
<td>This is an outcome of FHWA and TxDOT research.
The first column segment is placed and aligned on the adjustable supports on the footing. Post-tensioning (PT) ducts are spliced and the PT bars are tied. The connection is completed by a CIP concrete joint.
The precast segments are coupled together using PT bars and epoxy. The complete column is post-tensioned with PT strands that run through the ducts.
The precast template is aligned with the bent cap or girder elevation using the adjustable supports on the top precast segment. The joint between the top precast segment and precast template is filled with high-strength epoxy grout.
Sources of information:
Billington et al. (2001); Shahawy (2003).</td>
</tr>
</table>

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast pier/bent cap (a) Bent cap with grouted pockets (Source: Ralls et al. 2004)  (b) Bent cap with grouted ducts (Source: Ralls et al. 2004) 	Bent cap with grouted pockets (Figure-a): Red Fish Bay project, TX (1994) Bent cap with grouted ducts (Figure-b): Lake Belton Bridge over SH 66, Bell County, TX (2004) Bent cap with grouted ducts (Figure-b): Mountain Valley Road Bridge over I-40, New Mexico (2004)	Most of the sections are not standardized. Following attributes are presented based on the information provided in the listed references. Height: 3ft to 4ft-6in. Length: varies (usually full-width of superstructure) Width: 3ft to 4ft 28-day compressive strength usually specified is 5000 psi or greater	Bent caps with grouted pockets are easy to align with the column slices. Pier caps are beneficial for bridge sites with features such as power lines, waterways, and a parallel roadway underneath. Use of pier caps reduces number of prefabricated columns and footings. **To retain the bent cap weight within limits for a wide bridge superstructure, multiple bent caps can be utilized** as shown in Figure-d (next page). Further, **tapered cantilever shaped bent caps can be utilized to achieve reduced weight** compared to a rectangular bent cap; thus alleviating shipping and handling process.	The pockets/ ducts in the bent cap should be filled completely with pumped grout. Temporary supports should be used to set the elevation, and must remain in position until the grout achieves required strength. Shipping and handling limitations may arise due to heavy weight. Tighter tolerances are required for the bent caps with grouted ducts.	Most commonly prefabricated substructure element in a bridge. Standardized as single column hammer head bent, two column bent or three column bent (UDOT 2010). Prestressing is used to reduce the height of the segment, thus, reducing the weight. Connection details are available from research projects: Matsumoto et al. (2001); Restrepo et al. (2011). Sources of information: LoBuono (1996); Matsumoto et al. (2001); Ralls et al. (2004); Unlu (2010); UDOT (2010); Restrepo et al. (2011).

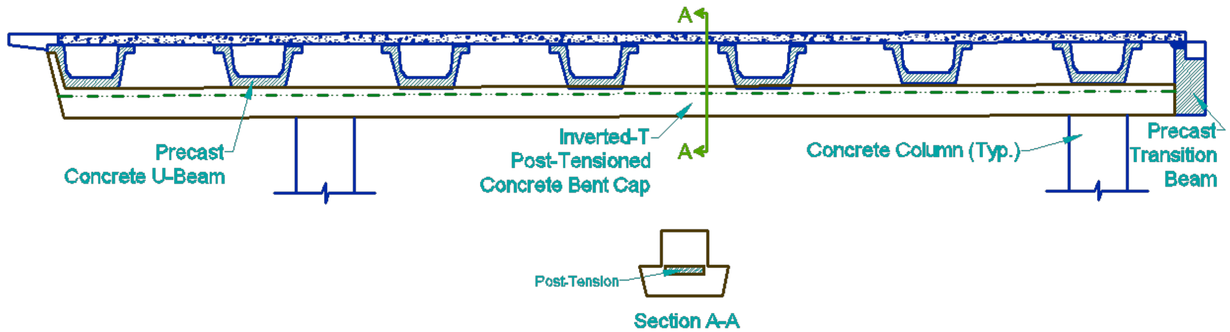
(c) Rectangular bent cap (Source: http://facilities.georgetown.org/2009):

(d) Trapezoidal bent cap (Source: Restrepo et al. 2011):

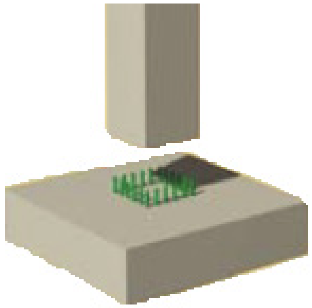
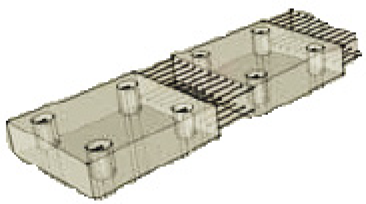
(e) Inverted-U section bent cap (Source: Culmo 2009)

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast pier/bent cap (a) Inverted-T bent cap (Source: Shahawy 2003)  	Inverted-T bent cap (Figure-a): Dallas/Fort Worth International Airport–elevated people mover, TX (2004) Inverted-T bent cap with canted ledges (Figure-b – next page): Austin-Bergstrom International Airport, Austin, TX (2000) Precast bent cap with cavities (Figure-c – next page): Conway Bypass Highway Bridge, Horry County, SC (2001)	Following attributes are presented based on the information provided in Shahawy (2003). Height: 4 ft to 5 ft (height of Inverted-T bent cap is 6ft-10in. including the flange) Length: varies (usually full-width of superstructure) Width: 4ft (width of Inverted-T bent cap is 4 ft at the web and 7ft-6in. at the flange) 28-day compressive strength usually specified is 5000 psi or greater	For the inverted-T bent cap, based on the flow of forces the unnecessary material can be removed to create box void and canted edges at bottom corners (Figure-a), or canted ledges (Figure-b – section A-A). **Thus the bent cap weight and the amount of reinforcement is reduced** compared to conventional bent cap design. **The cavities in the precast bent cap** (Figure-c) are casted to reduce the concrete material in the tension zone and **achieve reduced weight** of the component. Reduced weight of the components alleviates shipping and handling limitations.	New concept and details; hence, past performance data is limited. Temporary supports should be used to set the elevation, and must remain in position until the grout achieves required strength. Fabrication and reinforcement detailing may be challenging because of the hollow core.	Inverted-T bent cap could extend up to a length of 42.7 ft for a single bent. Sources of information: Billington et al. (1999); Powell and Powell (2000); Billington et al. (2001); Shahawy (2003); Culmo (2009).

(b) Inverted-T bent cap with canted ledges (Source: Powell and Powell 2000; Shahawy 2003)



(c) Precast bent cap with cavities (Source: Culmo 2009)

Element	Project(s)	Attributes	Benefits	Limitations	Remarks
Precast footings Footing on subbase:  Spread footing on piles:  Spread footing on subbase: 	Spread footing on subbase: South Maple Street Bridge over Scantic River, Enfield, CT (2011)	Most of the sections are not standardized. Following attributes are presented based on the information provided in the listed references. Conventional material strengths and design procedures are used. Thickness: 3ft Width: 8ft to 10 ft Length: varies	Good for small footings. Large footings could be developed by combining small segments. Good for bridges with shallow footings. Shallow footings have a potential to be supported on piles, in regions where piles are necessary.	A level concrete subbase preparation is necessary, which is an additional operation. Shipping and handling limitations may arise due to heavy weight. The connection between spread footing segments is a CIP closure which extends project duration. Special leveling screws are necessary for aligning the segments of a spread footing. Each individual leveling screw should be capable to withstand entire weight of the segment, without bending. A grout layer is required to ensure full bearing contact with the subbase. Significant amount of grout is required for this operation.	The precast footings serve as shallow foundations for a bridge. Very few projects utilized this element. This is due to requirement of preparing, roughening, and curing the subbase, which is an additional operation while using precast footings. UDOT (2010) developed standard footing designs, but till date, have not implemented on significant projects. Sources of information: UDOT (2010); Unlu (2010); Swanson (2011).

Miscellaneous

Construction technology	Project(s)	Attributes	Benefits	Limitations	Remarks
SPMT move: steel girders with CIP deck  **Slide-in**: CIP adjacent box girders 	SPMT move – Utah I-215 at 4500 South, UT (Bridge replacement) (2007) SPMT move – I-80 State street to 1300 E. multiple structure, Salt Lake city, UT (Bridge replacement) (2008) Slide-in – San Francisco Yerba Buena Island Viaduct (Bridge replacement) (2007)	Design attributes are similar to designing a prestressed concrete girder with a deck, a steel girder with a deck, or a modular superstructure system. But design considerations should also concentrate on the lifting and moving aspects of the structure.	Least disruption to traffic and improved work-zone safety. Bridge can be replaced overnight. Feasibility of maintaining high quality.	A large staging area adjacent to the site is required. Extremely tight tolerances are required. Support points should maintain their relative elevations. SPMT and Roll-in operations' cost are extremely higher than the cost of typical CIP and other ABC construction methods. While the SPMT and roll-in operations can be completed in days or hours, the preparation and construction of new structure still requires extended amount of time. Slide-in is not feasible for, skew or horizontally curved or superelevated structures. Not feasible if utilities are present in the moving path. Require continuous monitoring of carrier beams and deflections.	Light weight concrete is used to reduce the required number of modular transporters. Roll-in operation max. span: 177ft (till date) Sources of information: Baker (2007); Peterson and Ralls (2008); Chung et al. (2008).

Material	Remarks
High Performance Concrete (HPC) Proportioning of High Performance Concrete, Class AA low cement requirements as per Vermont AOT (2011) are shown below:	Central Pre-Mix Prestress (CPMP 2011) utilizes the concrete mixes as shown below to achieve 10,000 psi strength:

HPC Class	Req.** Cem. Mat. (lbs./cy)	Maximum Water-Cem. Mat. Ratio	Max. Slump (in.)	Air Content (%)	Coarse Aggregate Gradation Table	28-Day* Comp. Strength (psi)	28-Day* Modulus of Rupture (psi)
AA Low Cement	611	0.44	6	7.0 ± 1.5	704.02A	4000	650

* The listed 28-day compressive strength or modulus of rupture will serve as the basis of designing or approving the concrete mix.
** See tables located below for required cementitious materials.

Cement (lbs/cy)		Fly Ash (lbs/cy)		Silica Fume Admixture (lbs/cy)		Cementitious Materials (lbs/cy)
449	+	122	+	40	=	611

OR

Cement (lbs/cy)		GGBFS (lbs/cy)		Silica Fume Admixture (lbs/cy)		Cementitious Materials (lbs/cy)
418	+	153	+	40	=	611

OR

Blended Silica Fume Cement (8.0%) (lbs/cy)		Fly Ash (lbs/cy)		Cementitious Materials (lbs/cy)
489	+	122	=	611

OR

Blended Silica Fume Cement (8.0%) (lbs/cy)		GGBFS (lbs/cy)		Cementitious Materials (lbs/cy)
458	+	153	=	611

Materials	Quantity
Cement, type III	685 lbs/cy
Slag cement	65 lbs/cy
Fine aggregate 1	449 lbs/cy
Fine aggregate 2	800 lbs/cy
Coarse aggregate (AASHTO # 67)	548 lbs/cy
Coarse aggregate (AASHTO # 8)	1210 lbs/cy
Water	260 lbs/cy
Air entrainer	As required
High-range water-reducing admixture	As required
Water-cementitious materials ratio	0.35 per cy

Material	Remarks
Ultra High Performance Concrete (UHPC) Typical field-cast UHPC mix composition (Graybeal 2010):	Average material properties of UHPC are shown below (Graybeal 2009; Graybeal 2010):

Typical field-cast UHPC mix composition (Graybeal 2010):

Material	Amount	Percent by weight
Portland cement	1200 lbs/cy	28.5
Fine sand	1720 lbs/cy	40.8
Silica fume	390 lbs/cy	9.3
Ground quartz	355 lbs/cy	8.4
Super plasticizer	51 lbs/cy	1.2
Steel fibers	263 lbs/cy	6.2
Water	218 lbs/cy	5.2

Average material properties of UHPC are shown below (Graybeal 2009; Graybeal 2010):

UHPC material properties (average)	
Property	**Value**
Unit weight	156 lbs/ft^3
Modulus of elasticity	6200 ksi – 8000 ksi
Compressive strength	18 ksi – 35 ksi
Post-cracking tensile strength	1.0 ksi –1.5 ksi
Chloride ion penetrability (ASTM C1202)	Negligible

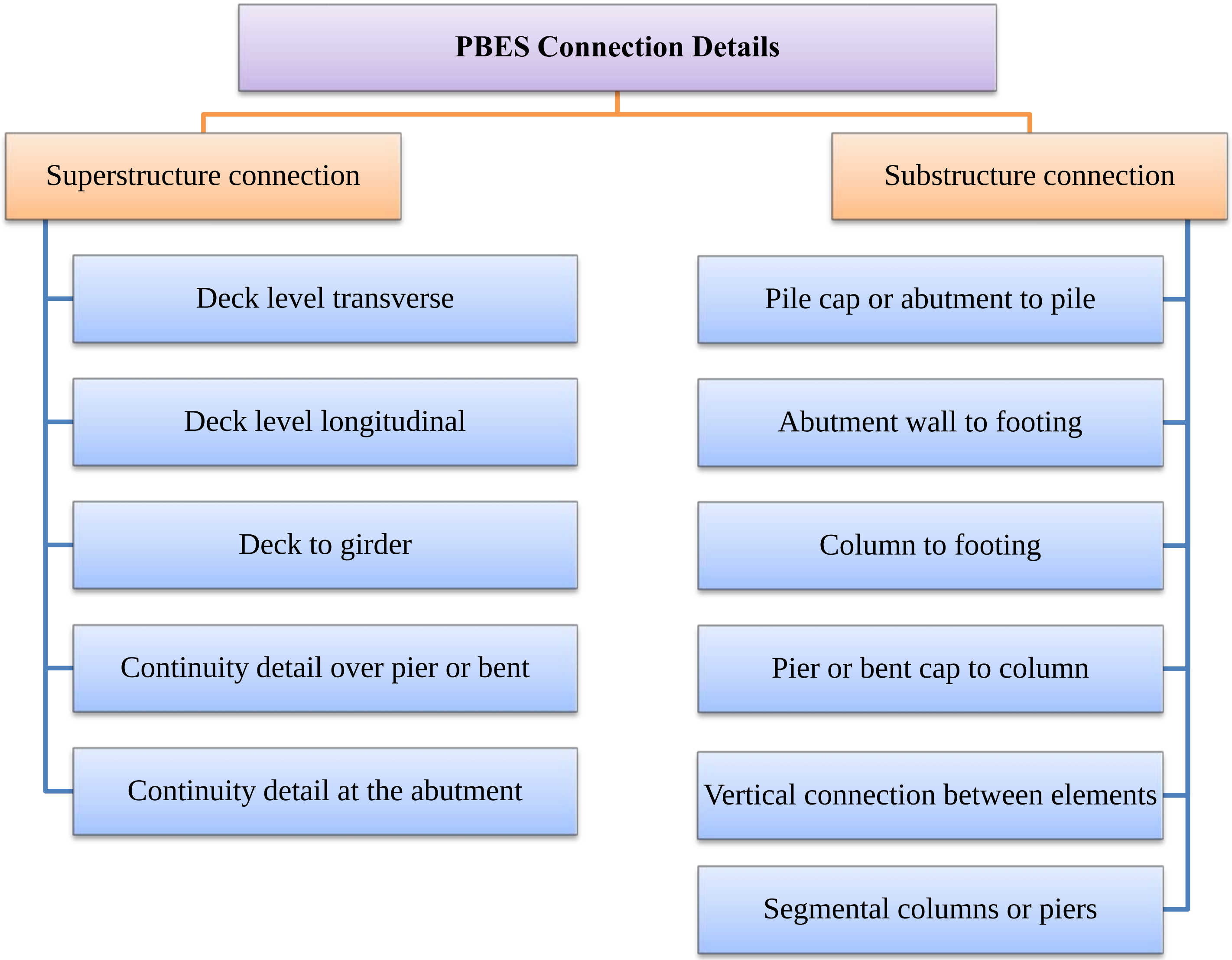
Design values for material properties of UHPC are shown below (Graybeal 2009; Graybeal 2010):

UHPC material properties (design)	
Property	**Value**
Modulus of elasticity at release	5800 ksi
Modulus of elasticity final	7800 ksi
Nominal compressive strength at release	12.5 ksi
Nominal compressive strength final	21.5 ksi
Nominal tensile strength final	1.2 ksi
Allowable compressive release stress 60% of 12.5 ksi	7.5 ksi
Allowable compressive stress at service 60% of 21.5 ksi	12.9 ksi
Allowable tensile stress at service 70% of 1.2 ksi	0.84 ksi

APPENDIX B

CONNECTION DETAILS BETWEEN PREFABRICATED ELEMENTS

COMMONLY USED PBES CONNECTION DETAILS



CONNECTION DETAILS LIBRARY

Rank description

Rank 1 : Represents connection details that have either been used on multiple projects or have become standard practice by at least one owner agency.

Rank 2 : Represents connection details that have been used only once and were found to be practical.

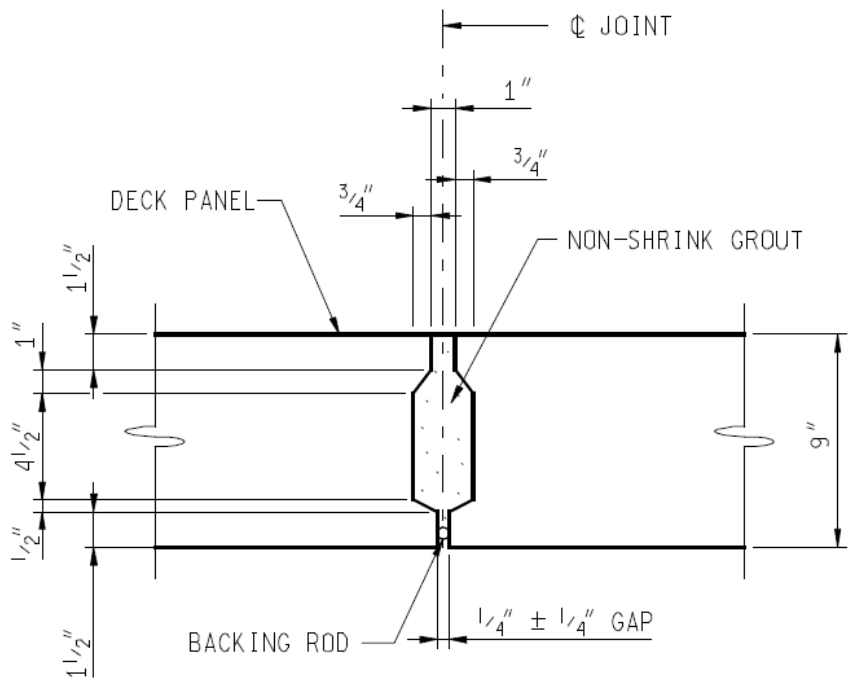
Rank 3 : Represents connection details that are either experimental or conceptual.

FULL-DEPTH DECK PANEL

Deck level transverse connection

FULL-DEPTH DECK PANELS

Panel-to-panel transverse connection

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr>
<td>1</td>
<td>


Implemented projects:
I-84 / Route 8 Interchange, Waterbury, CT
Parkview Ave over US-131, Kalamazoo, MI

Challenges are to:
Ensure a solid and tight grouted connection.
Maintain adequate space between the panels (tolerances).
</td>
<td>
The panels shall be placed at the nominal spacing shown on the plans with a 1/4" wide gap between the panels. The width of the gap can vary due to tolerances of the panels.

Grout for shear keys shall be rodded or vibrated to ensure that all voids in the shear keys are filled.

Use as a transverse joint combined with longitudinal post-tensioning (PT).

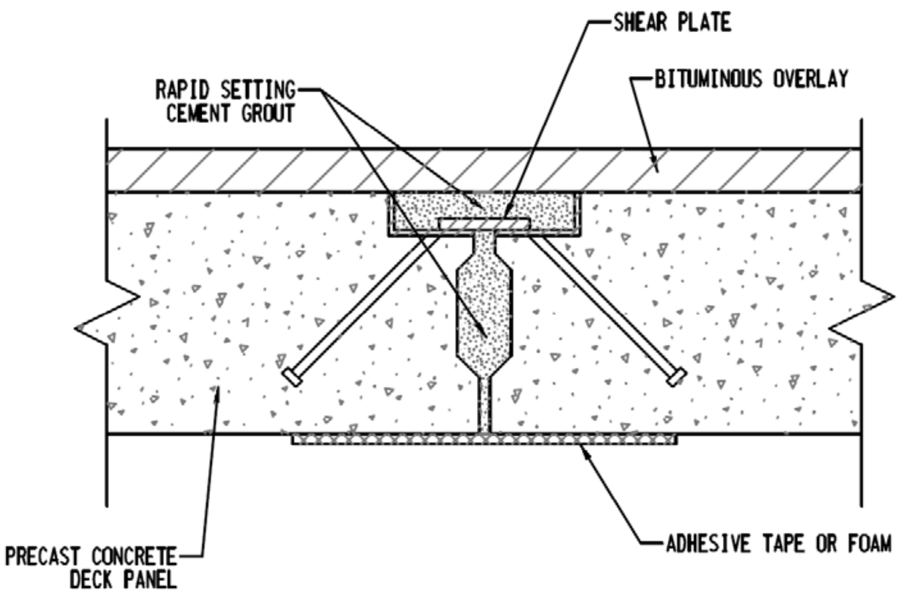
Use of this detail without PT is discouraged due to joint durability issues.

PT stress level of 250 psi (concentric) is recommended (AASHTO LRFD).

Designed to transmit shear and moment with proper PT application.

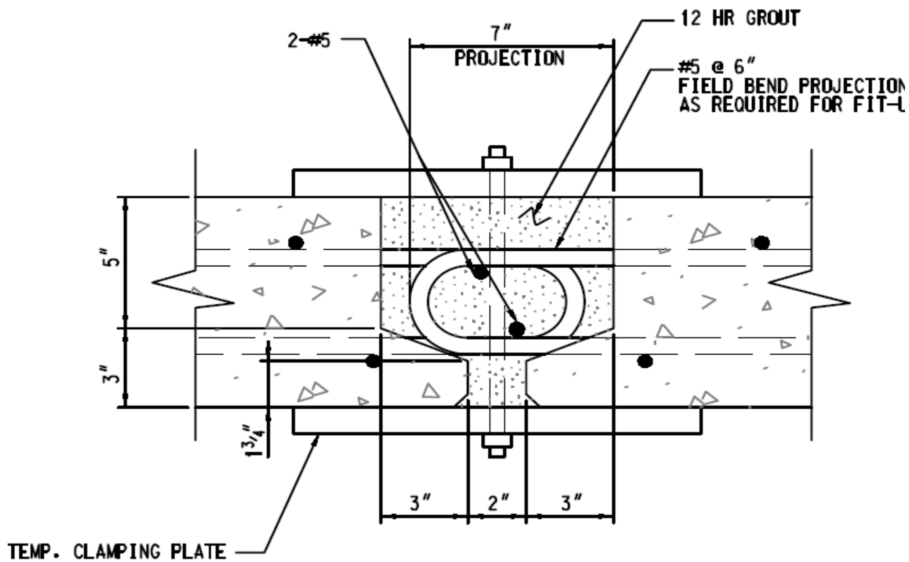
Note: Used in a large number of bridges. Performance data is available.
</td>
</tr>
</table>

FULL-DEPTH DECK PANELS
Panel-to-panel transverse connection

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual Implemented projects: Virginia Department of Transportation, Route 7 over Route 50. Note: New detail. Performance data may be limited.	Continuity is provided using grouted shear key with high-early-strength grout and PT. To further improve shear transfer, welded sliding shear plates are installed across each transverse joint. Note: Grout above the shear plate is filled after PT ; hence, not compressed and may yield to durability problems associated with steel corrosion. **Designed to transmit shear and moment with proper PT application.** **Challenges:** Field welding. Detection and prevention of steel plate corrosion. **Q:** Do we need steel plates? Note: With PT the system becomes very rigid, thermal gradient is the governing load.

FULL-DEPTH DECK PANELS

Panel-to-panel transverse connection

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr>
<td>1</td>
<td>


Source: FHWA Connection Manual

Q: Do we need steel at the connection?

Note: For this specific geometry of the connection, steel is required to transfer shear even with PT. Vertical surfaces can transfer shear if there is a good bond or adequate friction under compression.
</td>
<td>
Implemented projects:
Richmond Road over US285, CO

Self consolidating concrete (SCC) is used.

Designed to transmit moment and shear.

Challenges:
Difficulty feeding the transverse bars between the two panels due to conflicts with the post-tension ducts; hence, prestressing strands were used.

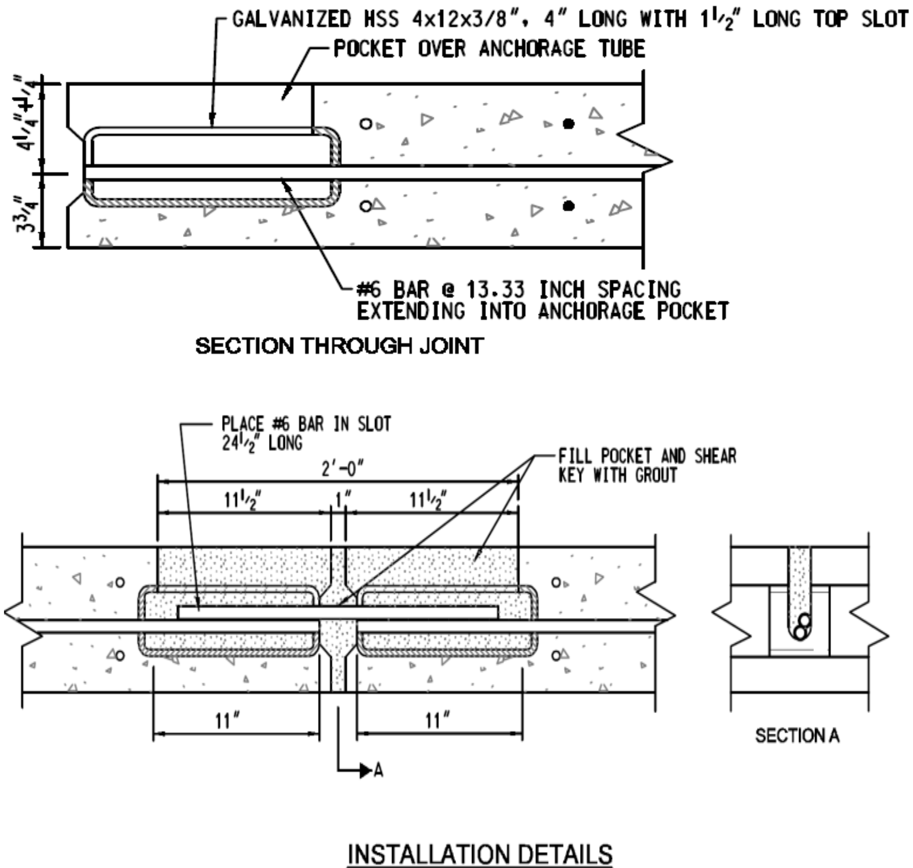
Fit up problem: the projecting tie bars conflicted with the adjacent panels. These bars had to be bent in the field.

Initially the SCC leaked through the forms. Subsequent pours were sealed properly.

Performance data might be available at the respective DOT.
</td>
</tr>
</table>

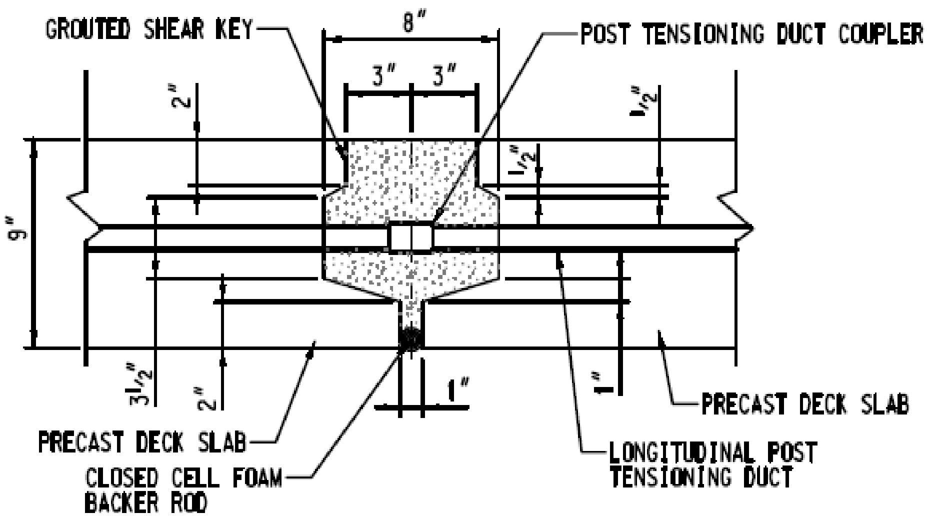
FULL-DEPTH DECK PANELS

Panel-to-panel transverse connection

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented projects: Live Oak Creek Bridge, TX First use of the details developed and tested under NCHRP Project 12-65. Longitudinal **post-tensioning is not required** to connect the deck panels in the field. Short reinforcing bars are placed in reinforced blockouts and grouted in place. **Designed to transmit shear and moment ?** Q: How to achieve moment transfer without precompression or adequate moment arm? **Challenges:** Selecting a non-shrink grout to fill a 2 ft wide pocket.

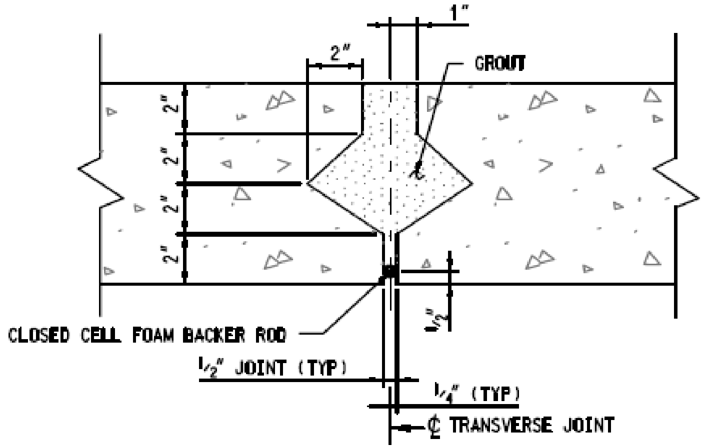
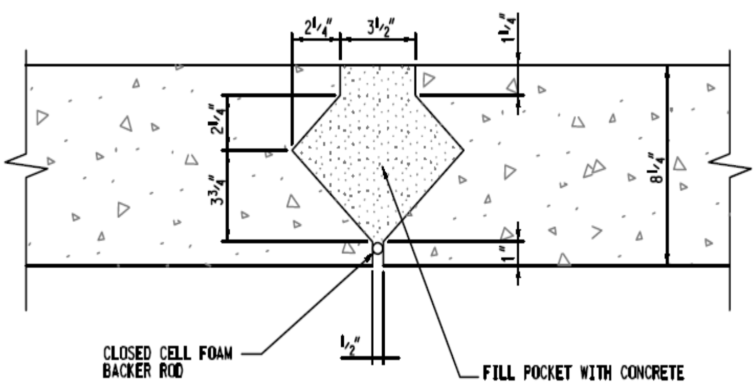
FULL-DEPTH DECK PANELS

Panel-to-panel transverse connection

Rank	Joint details	Description/Comments
1	 TRANSVERSE JOINT DETAILS Source: FHWA Connection Manual	Implemented projects: Replacement of I-287 Viaduct over the Bronx River Parkway, NY Grouted shear key with longitudinal post-tensioning. **Designed to transmit shear and moment with proper PT application.** **Challenges:** Use of non-shrink neat grout to achieve high early strength because, most of the time, fill depth or width is a controlling factor in selecting grout material. Note: Performance data may be available with respective DOTs.

FULL-DEPTH DECK PANELS

Panel-to-panel transverse connection

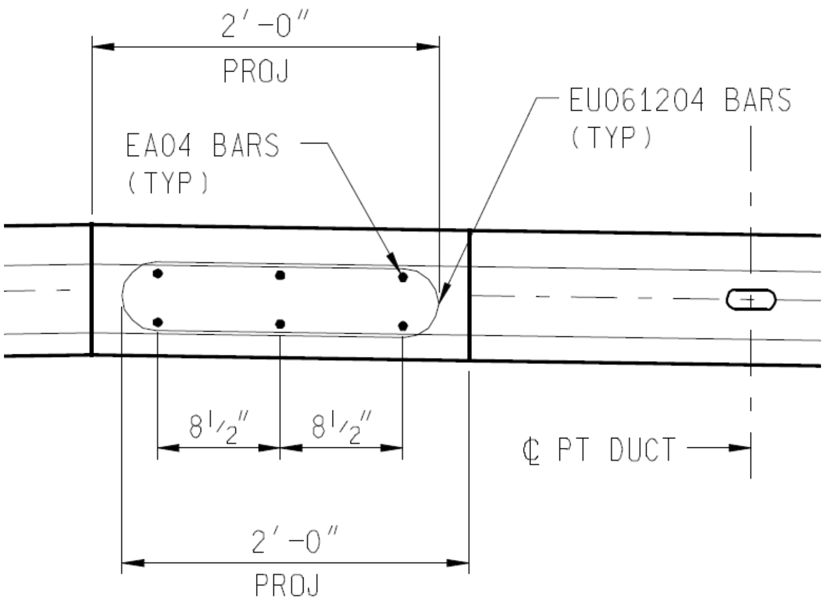
Rank	Joint details	Description/Comments
1	 Detail 1  Detail 2 (**Recommended**) Source: FHWA Connection Manual Note: Performance data may be available with respective DOTs.	Implemented projects: Washington State DOT, US 101 - Nolan Creek Vic. Bridge. Iowa DOT, Boone County IBRC Project over Squaw Creek. Diamond shaped shear key is filled with an early high strength, low shrinkage concrete mix. Extended non-shrink grout can be used to fill the joint. Transverse joints were later compressed using longitudinal post-tensioning of the bridge deck. **Designed to transmit shear and moment with proper PT application.** **Challenges:** Use of non-shrink neat grout to achieve high early strength because, most of the time, fill depth is a controlling factor in selecting grout material.

FULL-DEPTH DECK PANEL

Deck level longitudinal connection (closure)

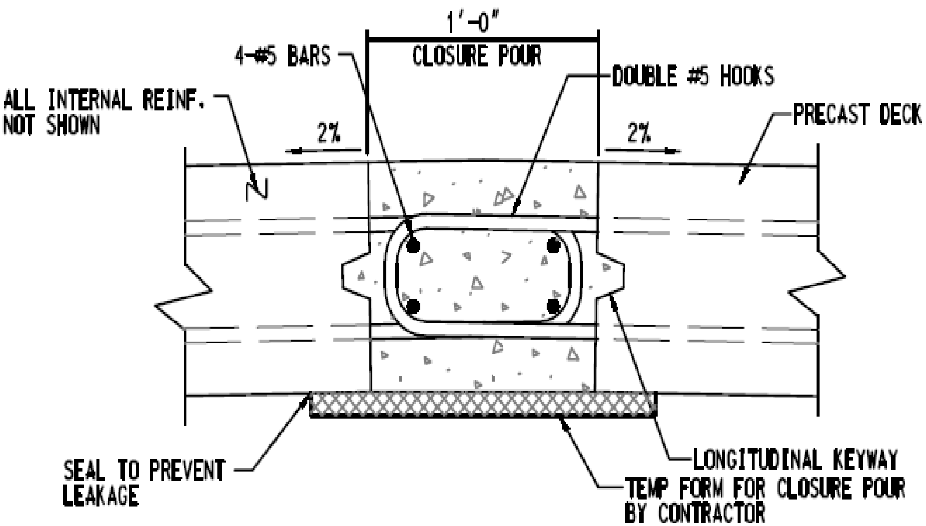
FULL-DEPTH DECK PANELS

Panel-to-panel longitudinal connection (closure)

Rank	Joint details	Description/Comments
1	 Note: The detail has been used for a long period; hence, performance data is available.	Implemented projects: Route 8 Viaduct, Seymour, CT. Parkview Ave over US-131, Kalamazoo, MI. Connection was used to account for the crown of the bridge deck. High early strength concrete is commonly used. Useful for staged construction. Designed to transmit shear and moment. **Challenges:** Spacing issues related to closure details are documented. Minimize the impact of vibration during staged construction *(hydrated cementitious material bond breaks when subjected to vibration after initial setting)*. Minimize shrinkage cracking.

FULL-DEPTH DECK PANELS

Panel-to-panel longitudinal connection (closure)

Rank	Joint details	Description/Comments
1	 Note: Performance data might be available at the respective DOT.	Implemented projects: Boone County IBRC Project over Squaw Creek, IA Connections was established by using a reinforced concrete closure pour and high-early-strength non-shrink concrete. Designed to transmit moment and shear. **Challenges:** Spacing issues related to closure details are documented. Minimize the impact of vibration during staged construction *(hydrated cementitious material bond breaks when subjected to vibration after initial setting)*. Minimize shrinkage cracking.

FULL-DEPTH DECK PANELS

Panel-to-panel longitudinal connection (closure)

Rank	Joint details	Description/Comments
3	 Note: the figure shown is not a full-depth deck panel. It is a prefabricated module of steel girders and a precast panel used in MassDOT Fast 14 project. The figure shown here is only to present the concept.	Use of threaded inserts with straight steel secured at the site is an option to eliminate space issues while placing the precast elements. Benefits should be justified with the time and effort Formwork supports can be attached to the panels to reduce construction time Connection is designed to transmit shear and moment **Detail is <u>not yet implemented</u> with full-depth deck panels.** **Challenges:** Threading a large number of steel bars at the filed. Minimize the impact of vibration during staged construction *(hydrated cementitious material bond breaks when subjected to vibration after initial setting)*. Minimize shrinkage cracking.

FULL-DEPTH DECK PANEL

Deck-to-girder connection
(shear connection and haunch)

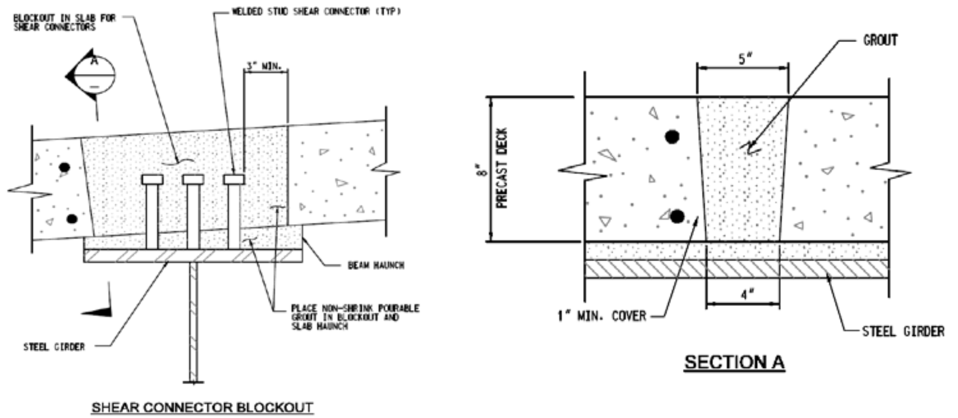
FULL-DEPTH DECK PANELS

Deck panel-to-steel girder connection

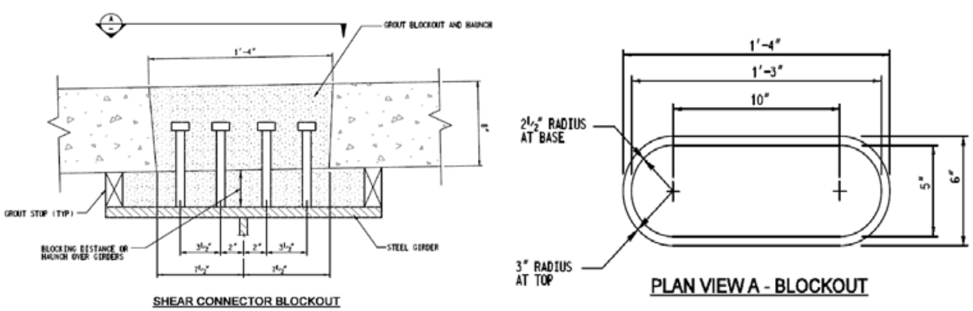
Rank	Joint details	Description/Comments

1

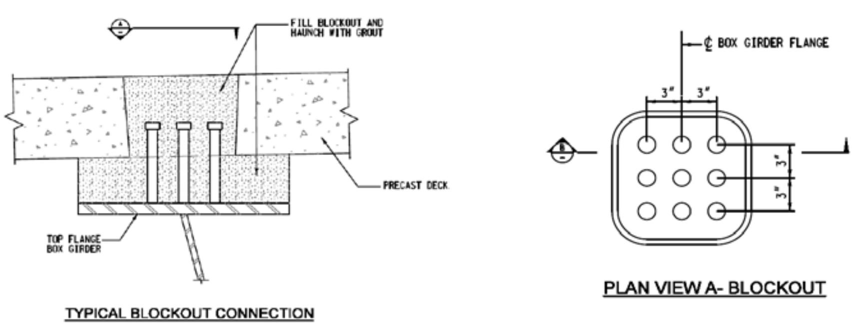
a)



b)



c)



Source: FHWA Connection Manual

Implemented projects:

a) I-84/Route 8 Interchange, Waterbury, CT
b) US 101 - Nolan Creek Vic. Bridge, WA
c) Replacement of I-287 Viaduct over the Bronx River Parkway, NY

Spacing of blockouts is usually kept to 24" to 30" on center.

Blockouts have been built with both rounded and square corners.

Rounded corners are preferred in order to minimize cracking potential.

Rounded corners minimize the potential for grout voids.

Designed to transmit shear.

Challenges:

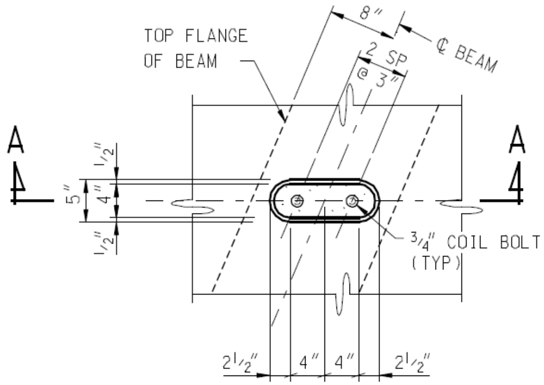
Leak proof formwork.

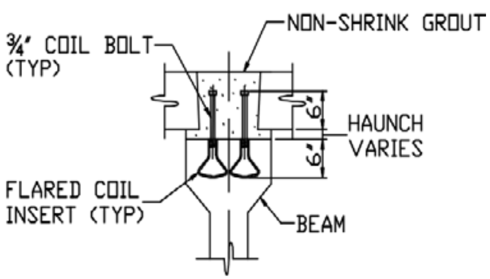
Non-shrink neat grout for large voids.

Performance data might be available at the respective DOTs.

FULL-DEPTH DECK PANELS

Deck panel-to-prestressed concrete girder connection

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr>
<td>2</td>
<td>




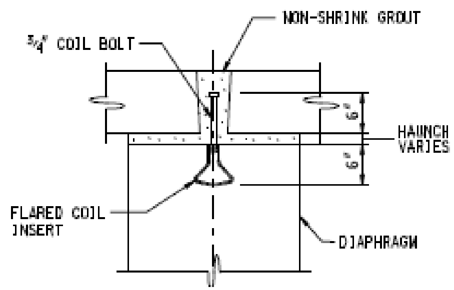



Photo courtesy: MDOT
</td>
<td>
Implemented projects:
Parkview Avenue over US-131, MI

Typical spacing of blockouts is 23” on center.

Designed to transmit shear.

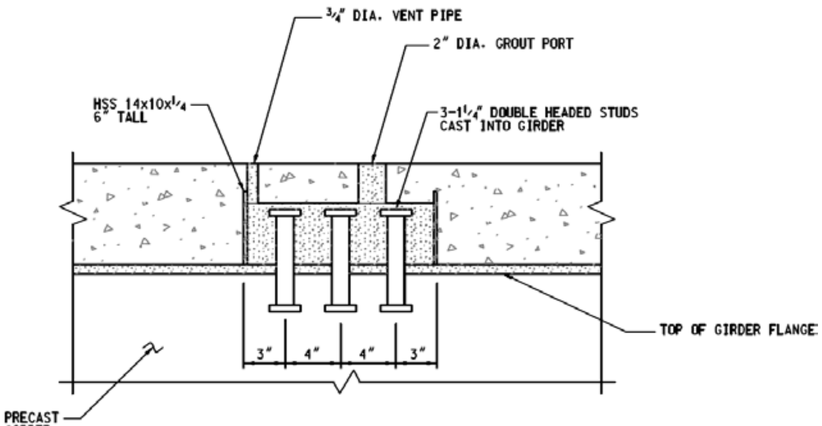
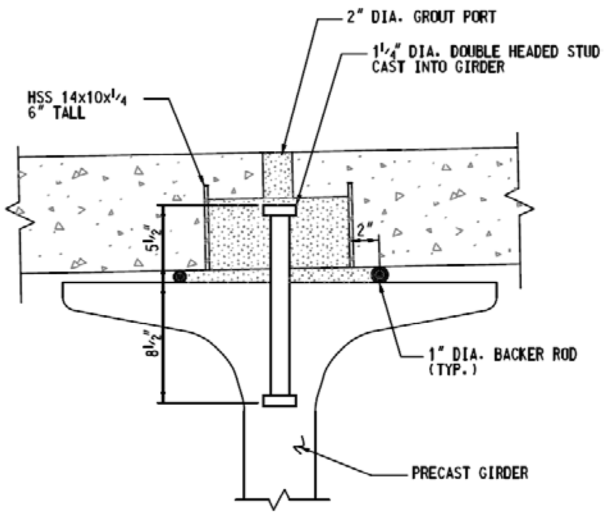
Challenges:
Leak proof formwork.

Tolerance issues for installation of coil bolts due to girder sweep.

Performance data might be available at the respective DOTs.
</td>
</tr>
</table>

FULL-DEPTH DECK PANELS

Deck panel-to-prestressed concrete girder connection

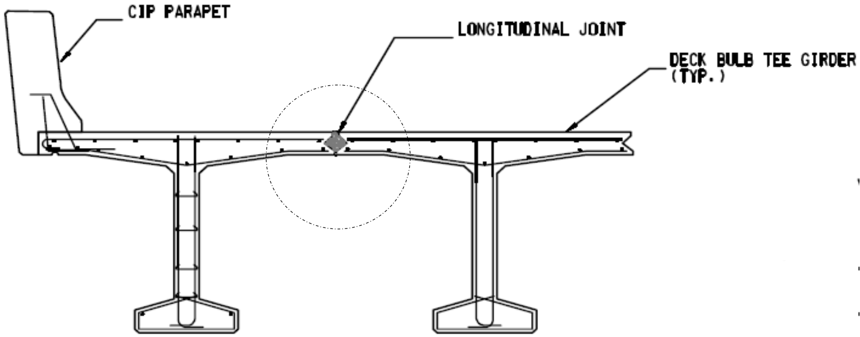
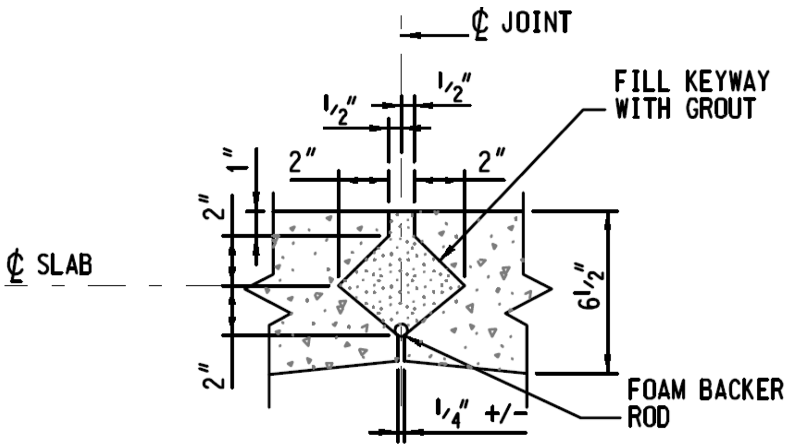
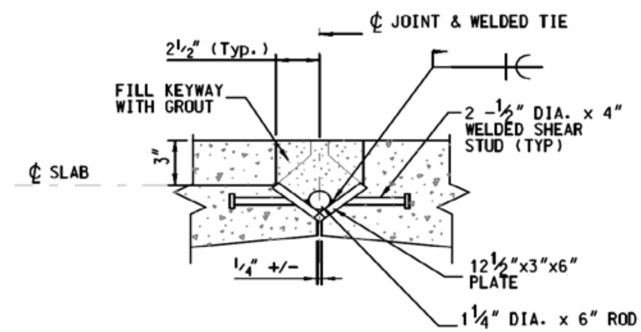
Rank	Joint details	Description/Comments
2	 LONGITUDINAL SECTION THROUGH BLOCKOUT  BLIND BLOCKOUT DECK CONNECTION Source: FHWA Connection Manual	Implemented projects: Live Oak Creek Bridge, TX Spacing of the blockouts is kept at 4 ft on center. This detail in not commonly used; hence, needs performance evaluation. Minimum exposure to ambient conditions. Designed to transmit shear. **Challenges:** Prevent voids in the grouted connection. Leak proof formwork. Limited use. Performance data might be available at the respective DOT.

PREFABRICATED SUPERSTRUCTURE MODULES

Decked Bulb Tee

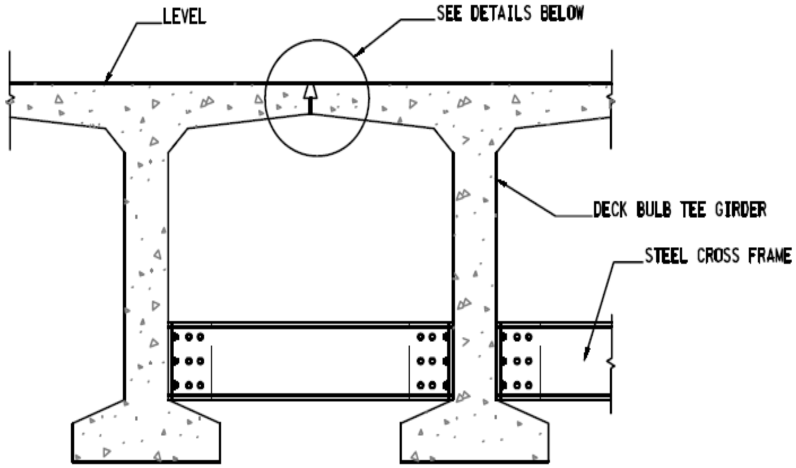
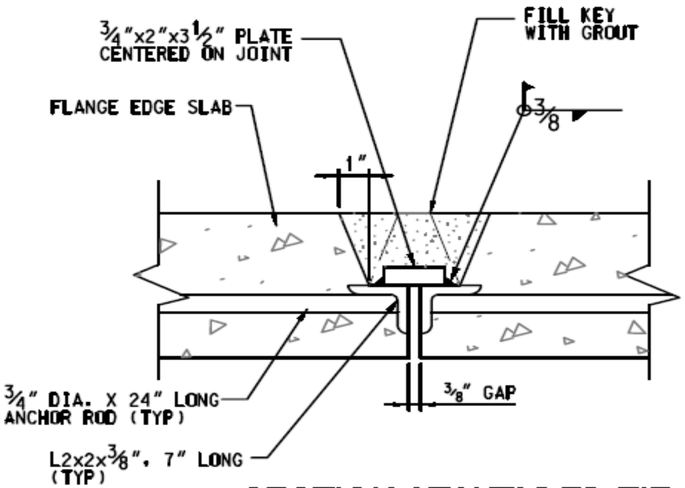
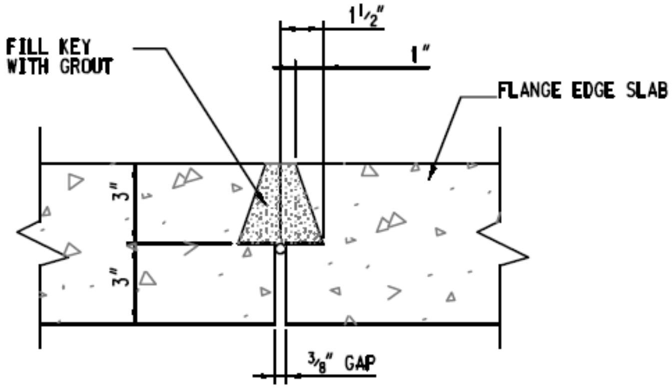
Girder-to-girder connection

PREFABRICATED SUPERSTRUCTURE MODULES
DBT girder-to-girder connection

Rank	Joint details	Description/Comments
1	 TYPICAL SECTION  FLANGE SECTION THROUGH LONGITUDINAL JOINT Source: FHWA Connection Manual	Commonly used in Washington State Girder types include: a) Double tee beams b) Triple Tee (Ribbed) Beams c) Deck Bulb Tees The welded connections are spaced at maximum of 5 feet on center. Durability is a concern because the joint is not designed to transfer moment Designed to transmit shear.  FLANGE SECTION AT WELDED TIE **Any detail that does not carry shear and flexure without cracking under service loads is not suitable for states like MI.**

PREFABRICATED SUPERSTRUCTURE MODULES

DBT girder-to-girder connection

Rank	Joint details	Description/Comments
1	 TYPICAL DECK BULB TEE DETAILS  SECTION AT WELDED TIE	Bridge over middle Fork Crazy Woman Creek, WY The welded connection is spaced at approximately 6 foot intervals. Designed to transmit shear  SECTION ALONG LONGITUDINAL JOINT Source: FHWA Connection Manual

PREFABRICATED SUPERSTRUCTURE MODULES

Tee Girder connections

PREFABRICATED SUPERSTRUCTURE MODULES

Double tee girder-to-girder connection

<table>
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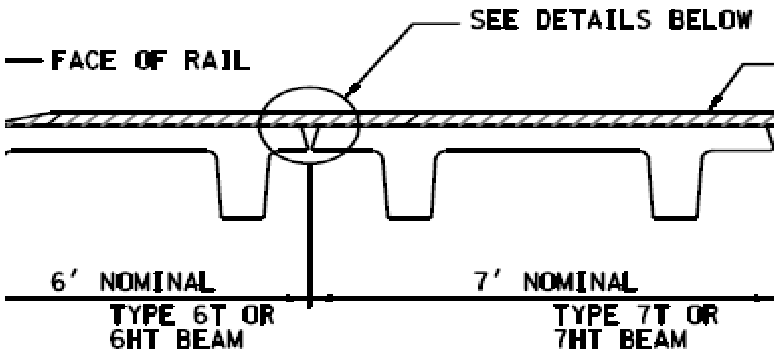
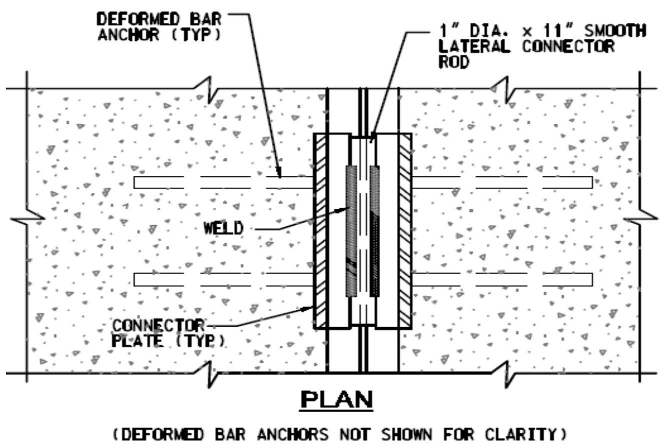
<tr><td>1</td><td>





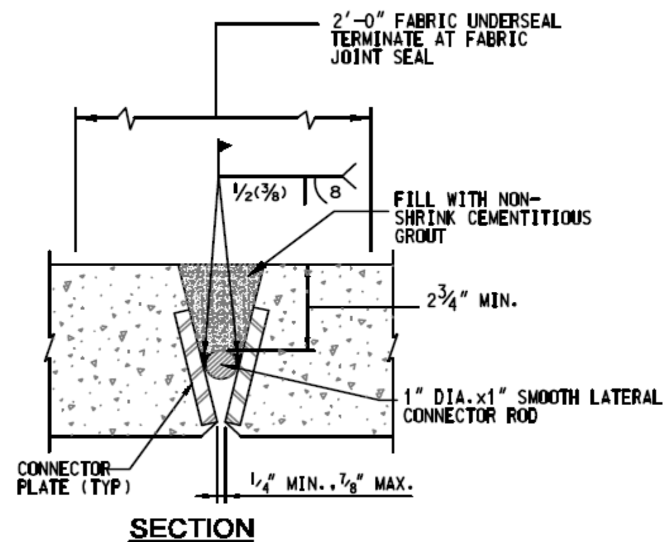
Source: FHWA Connection Manual

</td><td>

Commonly used in Texas State

These connectors are spaced 5 feet on center along the entire length of the beam.

Designed to transmit shear



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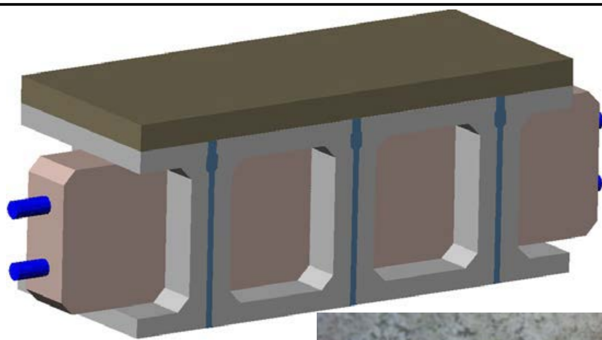
PREFABRICATED SUPERSTRUCTURE MODULES

Side-by-side box-beam

PREFABRICATED SUPERSTRUCTURE MODULES

Side-by-side box-beam: longitudinal connection

Rank	Joint details	Description/Comments
1	  Cracks observed on Oakland over I-94 on June 21, 2007 (before opening to traffic) Analysis and design procedures are given in Attanayake and Aktan (2009) - TRB09-3420. Analysis and design procedures presented in the paper are applicable to most of the modular systems currently used in ABC.	Implemented project: A large number of projects in Michigan and other states. Michigan DOT transverse connection include full-depth grouted shear-keys, transverse post-tensioning, and a 6 in. cast-in-place concrete deck. Deck is placed after post-tensioning the girders. Reflective deck cracking has been observed even before opening to traffic. Durability is a concern in states like Michigan. Staged post-tension through top and bottom flanges with spacing requirements similar to full-depth deck panels need to be considered for enhanced durability.

PREFABRICATED SUPERSTRUCTURE MODULES

Decked steel girder system: longitudinal connection

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>2</td><td>
</td><td>Implemented in the MassDOT Fast 14 project.

Designed to transmit moment and shear.

Once the units were placed, longitudinal joints were formed with field cast high early strength concrete.

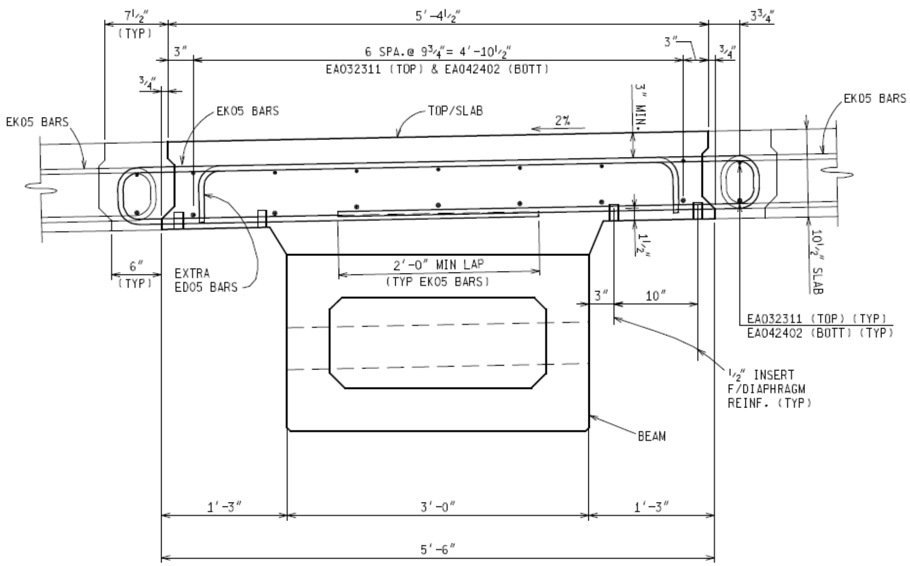
Specialty closure pour concrete needed immediate curing and protection. Polyethylene covering was used to keep the moisture.

A MassDOT requirement was to produce a concrete mix of 2000 psi strength within 4 hours.

Performance data will be available in near future.</td></tr>
</table>

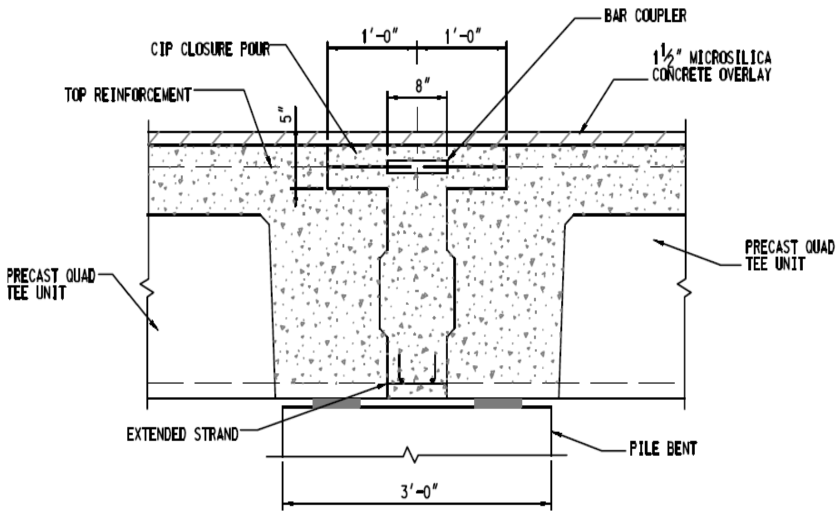
PREFABRICATED SUPERSTRUCTURE MODULES

Decked steel girder system: longitudinal connection

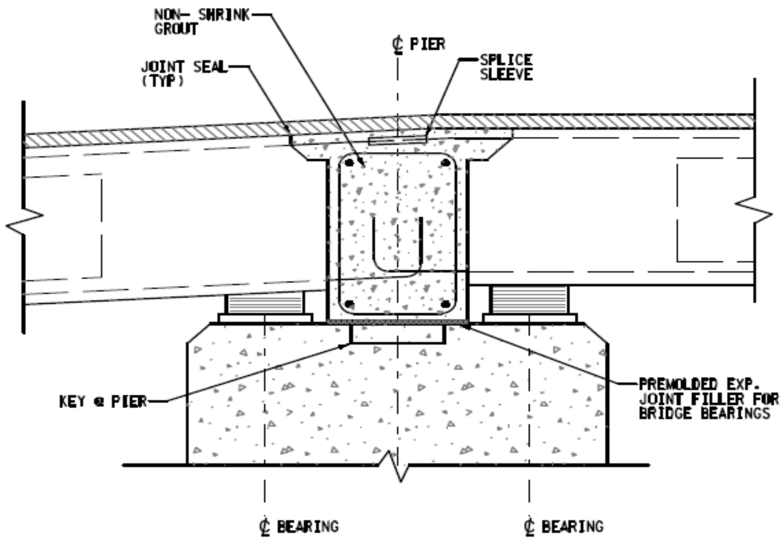
Rank	Joint details	Description/Comments
2		Implemented Project: M-25 crossing over the White River, MI. Designed to transmit moment and shear. Longitudinal closure was cast after post-tensioning the superstructure through the diaphragms. So, the longitudinal joint is not compressed. Performance data will be available in near future.

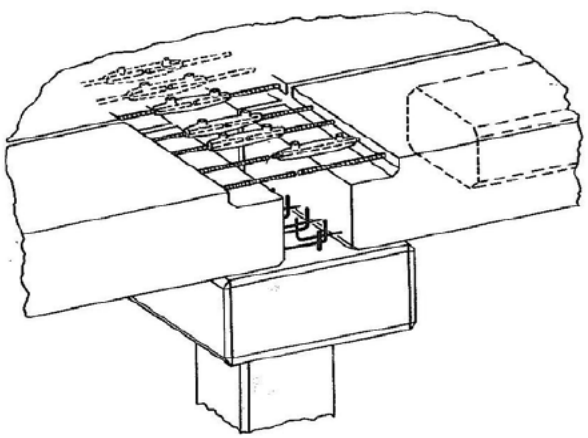
CONTINUITY DETAIL OVER PIER OR BENT

CONTINUITY DETAIL AT PIER

Rank	Joint details	Description/Comments
2	 CONTINUITY CONECTION AT PIER Source: FHWA Connection Manual	Implemented project: NYDOT, Robert Moses Causeway Bridge Rehabilitation over Great South Bay. Precast quad tee beam superstructure is made continuous for live load. Connecting these units together longitudinally is made with a rebar coupler at the top and overlapping mild steel at the bottom (similar to standard AASHTO I-girders). Designed to provide force and moment continuity.

CONTINUITY DETAIL AT PIER

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>2</td><td>


SECTION THROUGH PRECAST BOX BEAM CONNECTION

</td><td>Implemented project:
Unquowa Road, Fairfield, CT.

Splice sleeves were used that could be slid to one side and then slid into position after beam placement.

Designed to provide force and moment continuity.

Source: FHWA Connection Manual</td></tr>
</table>

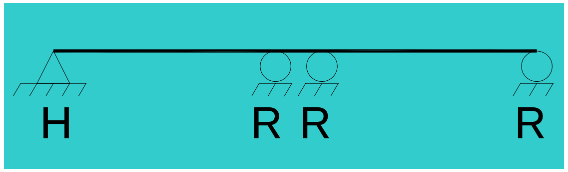
LINK SLABS

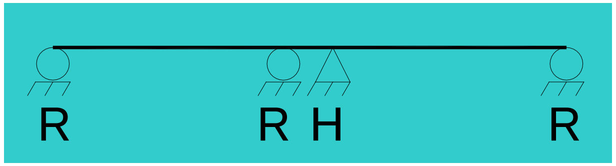
- Detailing over piers (negative moment continuity)

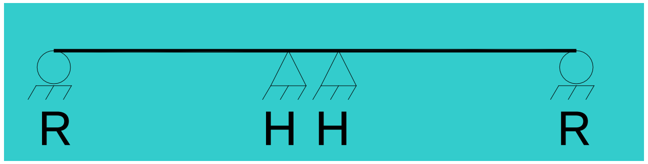
Link slab has already been implemented with ABC

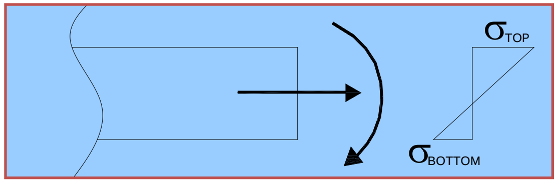
LINK SLABS

- Detailing over piers (negative moment continuity)



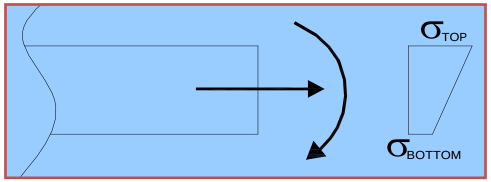






Bottom fiber under compression (resultant force is in tension)

Live, thermal gradient, and uniform thermal loads need to considered



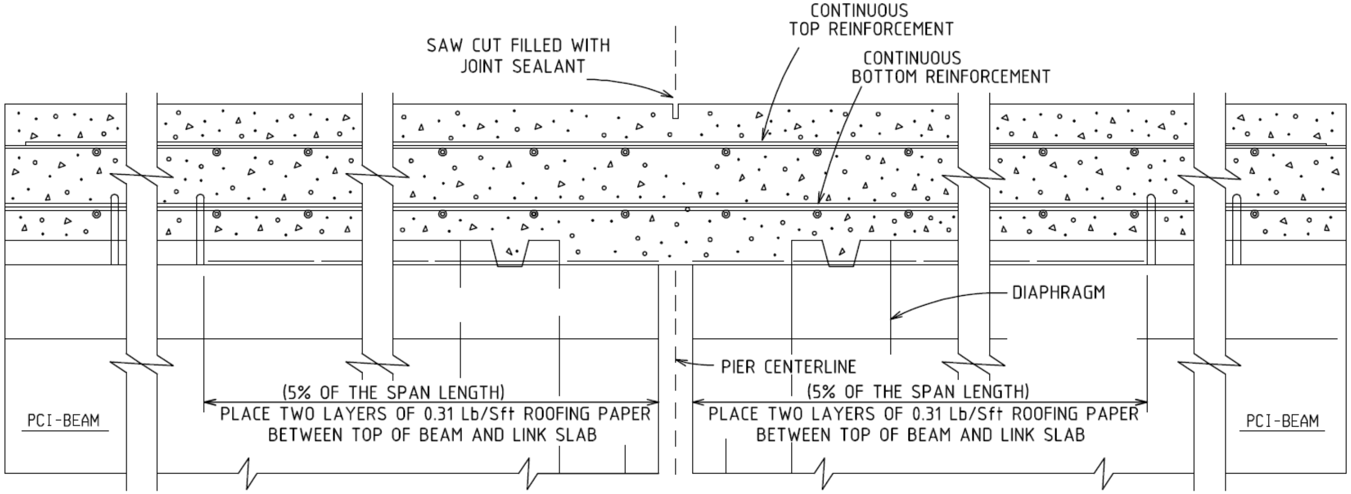
Combined bending and tension

Bearing arrangement should be critically reviewed

LINK SLABS

➢ Detailing over piers (negative moment continuity)

1. Combined effect of live and thermal gradient loads should be considered for link slab design.
2. Link slab should be designed considering flexural interaction with axial loads for RHHR support configuration.
3. Both top and bottom layer reinforcement should be continued.



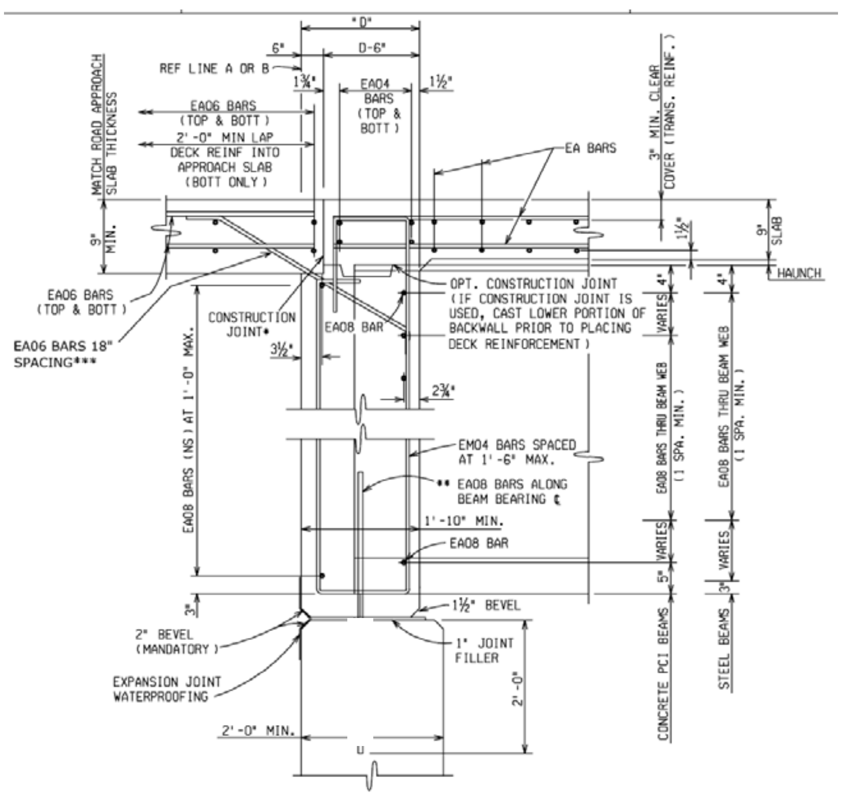
4. Link-slab analysis and design procedures explained in TRB paper 09-3577 is recommended.

CONTINUITY DETAIL AT THE ABUTMENT

CONTINUITY DETAIL AT ABUTMENT

SEMI-INTEGRAL ABUTMENT

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td></td><td>



Source:
Aktan, Attanayake, and Ulku (2008)
(MDOT RC-1514)

</td><td>

Implemented projects:

Friction between backwall and abutment may be a concern.

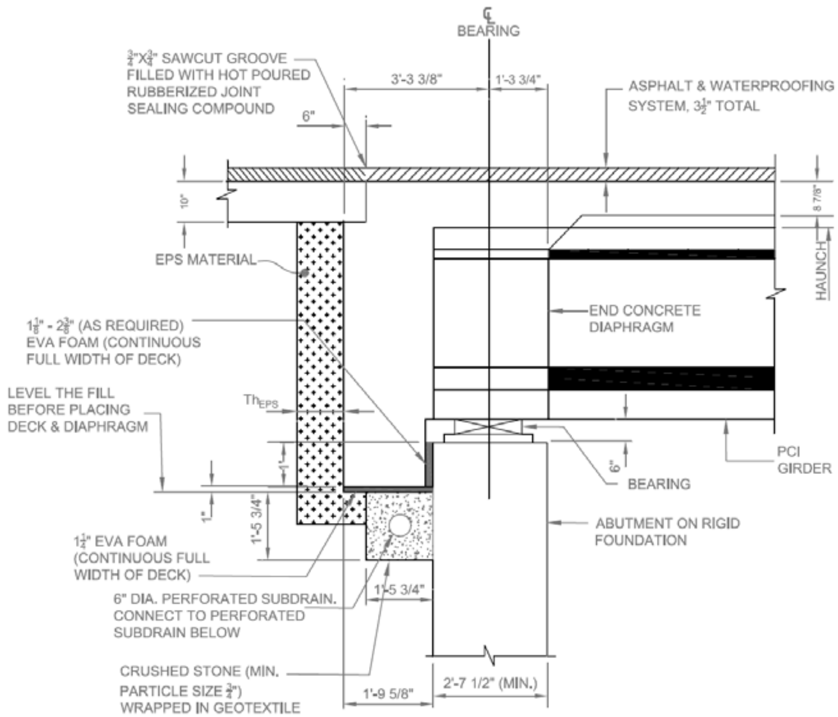
Fit-in tolerance can be an issue if the backwall is prefabricated with the girder.

**Only used with integral abutment bridges

</td></tr>
</table>

CONTINUITY DETAIL AT ABUTMENT

SEMI-INTEGRAL ABUTMENT

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td></td><td>

Source:
Aktan and Attanayake (2011)
MDOT RC-1563</td><td>Implemented projects:

Do not require tighter tolerances

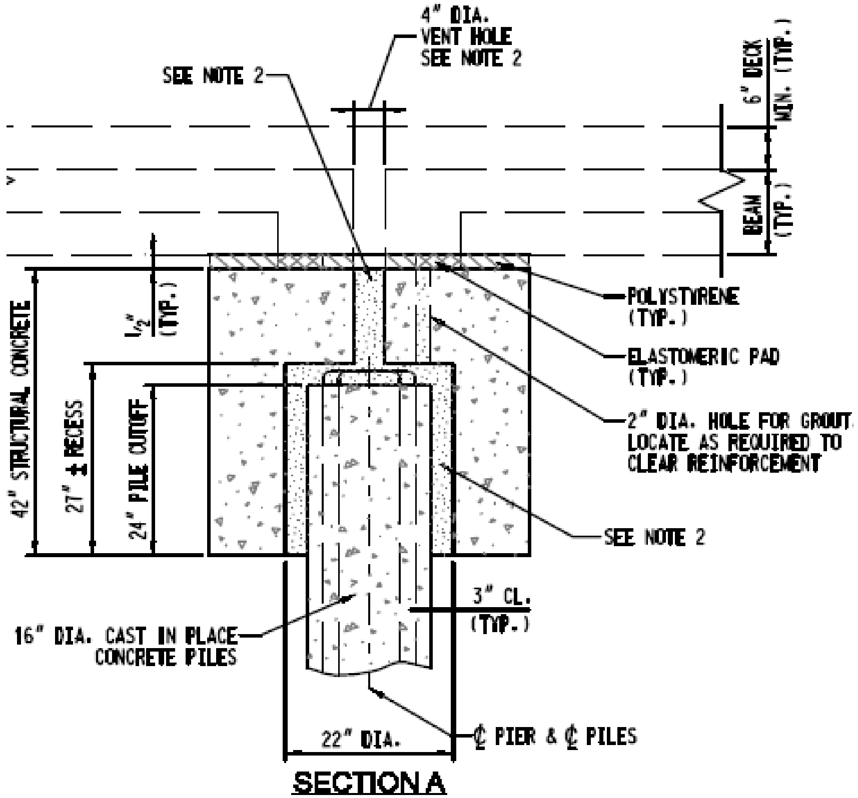
Superstructure is isolated from the substructure

Bearing can be maintained and replaced as needed</td></tr>
</table>

SUBSTRUCTURE CONNECTIONS

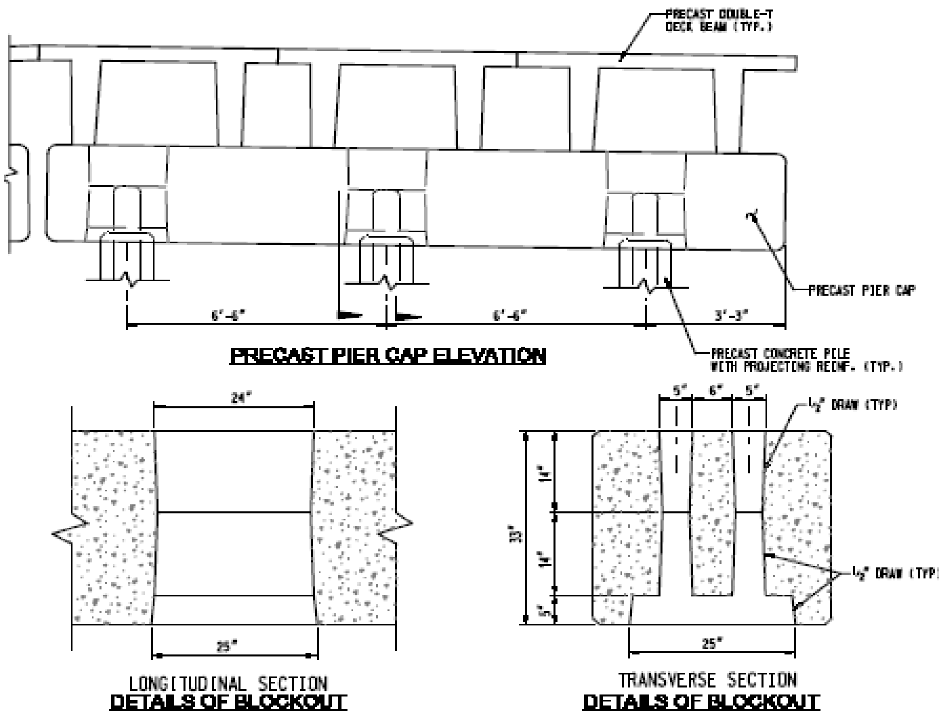
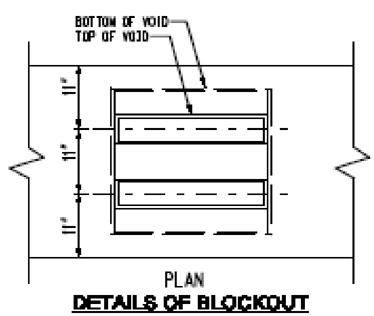
PIER CONNECTIONS

Precast pier cap to round CIP piles

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented projects: Bridge 13004, TH 8 over Center Lake Channel, Center City, MN Cap was precast with oversized holes for piles, and smaller holes between pile blockouts and top of cap were included for grouting After piles were installed, the cap was placed over piles, leveled into position, and high-strength grout injected from top of cap through grouting holes. Rebar inside the CIP piles projected into the pier cap for additional bonding. Designed to transmit shear and compression

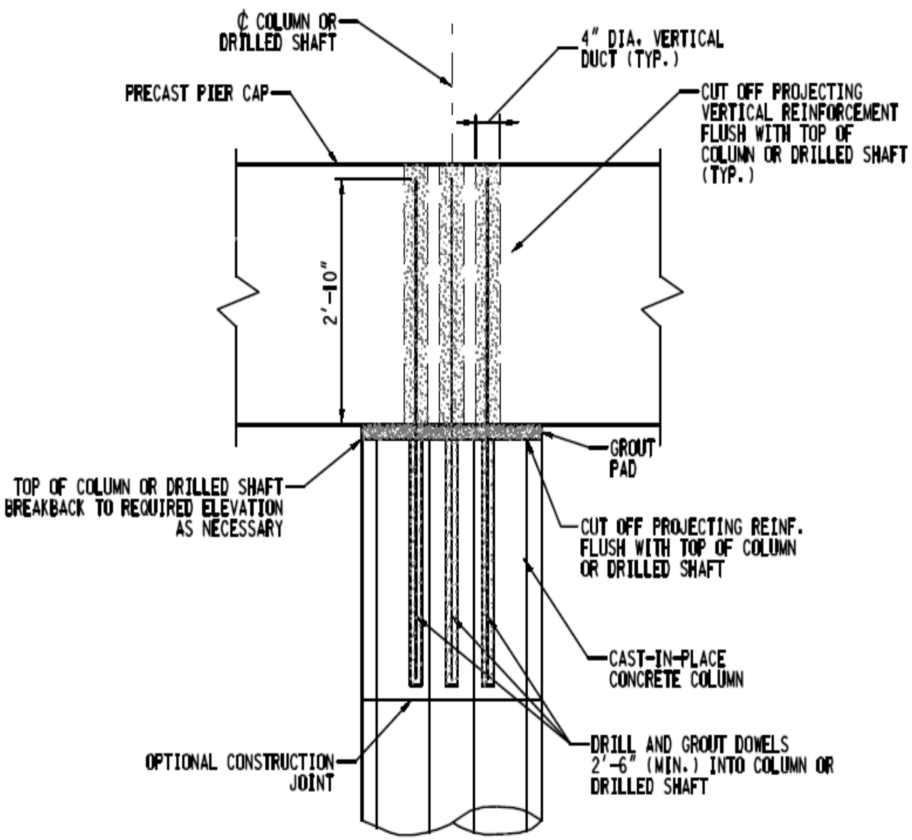
PIER CONNECTIONS

Precast pier cap to precast concrete piles

Rank	Joint details	Description/Comments
1	  Source: FHWA Connection Manual	Implemented project: Redfish Bay, TX Designed to transmit shear, moment, compression, tension and torsion. 

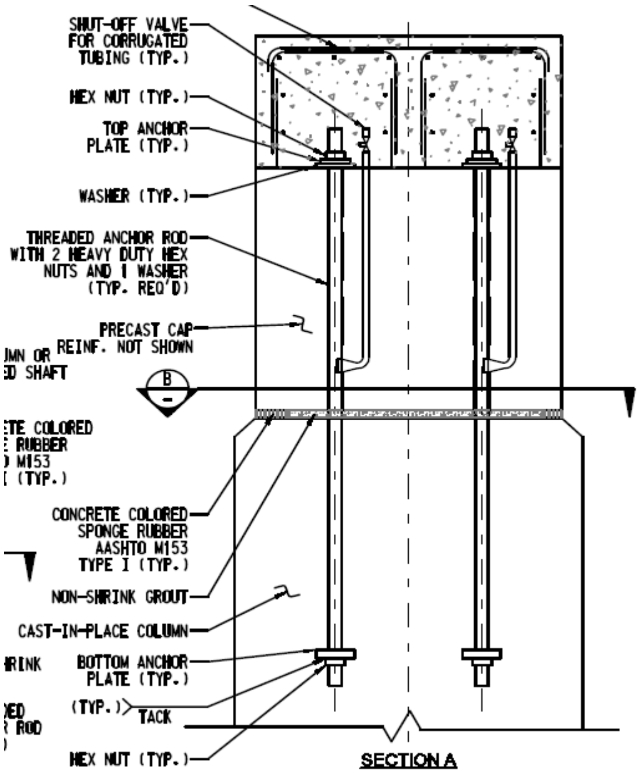
PIER CONNECTIONS

Precast pier cap to CIP concrete columns

Rank	Joint details	Description/Comments
1	 SECTION THROUGH PIER CAP AND COLUMN Source: FHWA Connection Manual	Commonly used in Texas The contractor used shims to set the grades of the caps. The ducts run from the bottom to a point near the top of the cap. This was done to avoid interference with the large amount of top reinforcing in the cap. The ducts are standard post-tensioning ducts. Designed to transmit shear, moment, compression, tension and torsion.

PIER CONNECTIONS

Precast bent cap to CIP concrete column

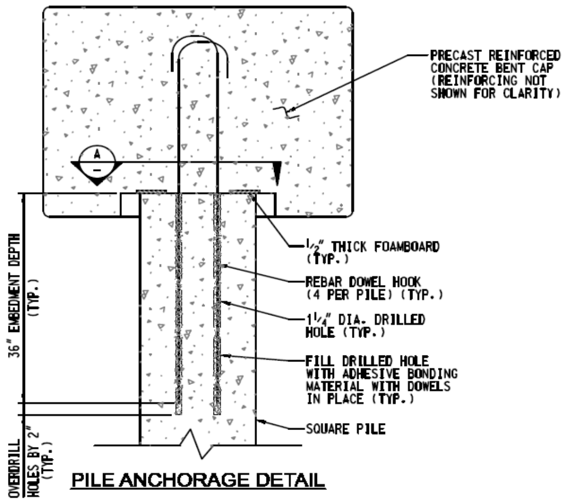
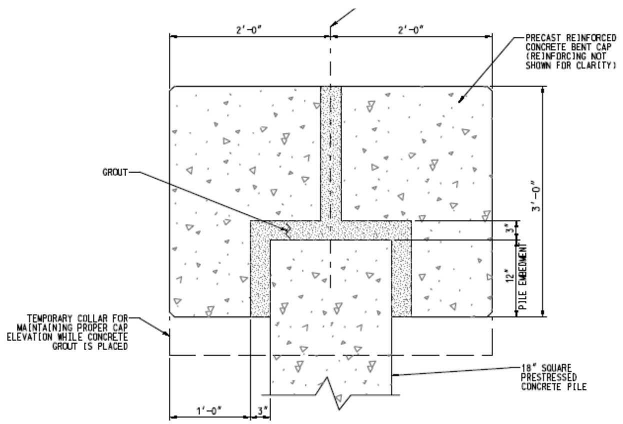
<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr>
<td>2</td>
<td>


Source: FHWA Connection Manual
</td>
<td>
Implemented project:
Bridge over BNSF Railroad (Wyoming Drawing Number 7024), Wyoming

Designed to transmit shear, moment and compression
</td>
</tr>
</table>

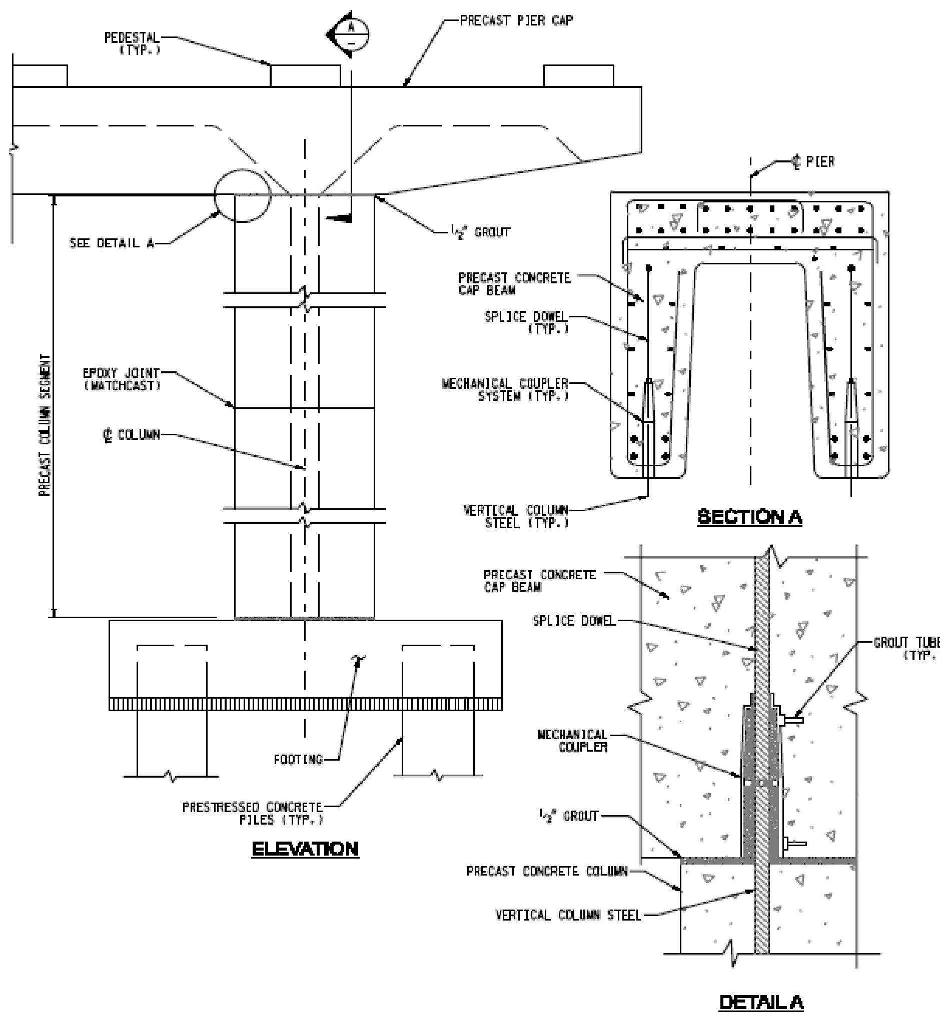
PIER CONNECTIONS

Precast bent cap or pile cap to pre-stressed concrete column or pile

Rank	Joint details	Description/Comments
1	 PILE ANCHORAGE DETAIL  SECTION THRU PILE POCKET Source: FHWA Connection Manual	Implemented projects: Carolina Bays Parkway, South Carolina DOT Conway bypass, South Carolina DOT Designed to transmit shear, moment and compression 

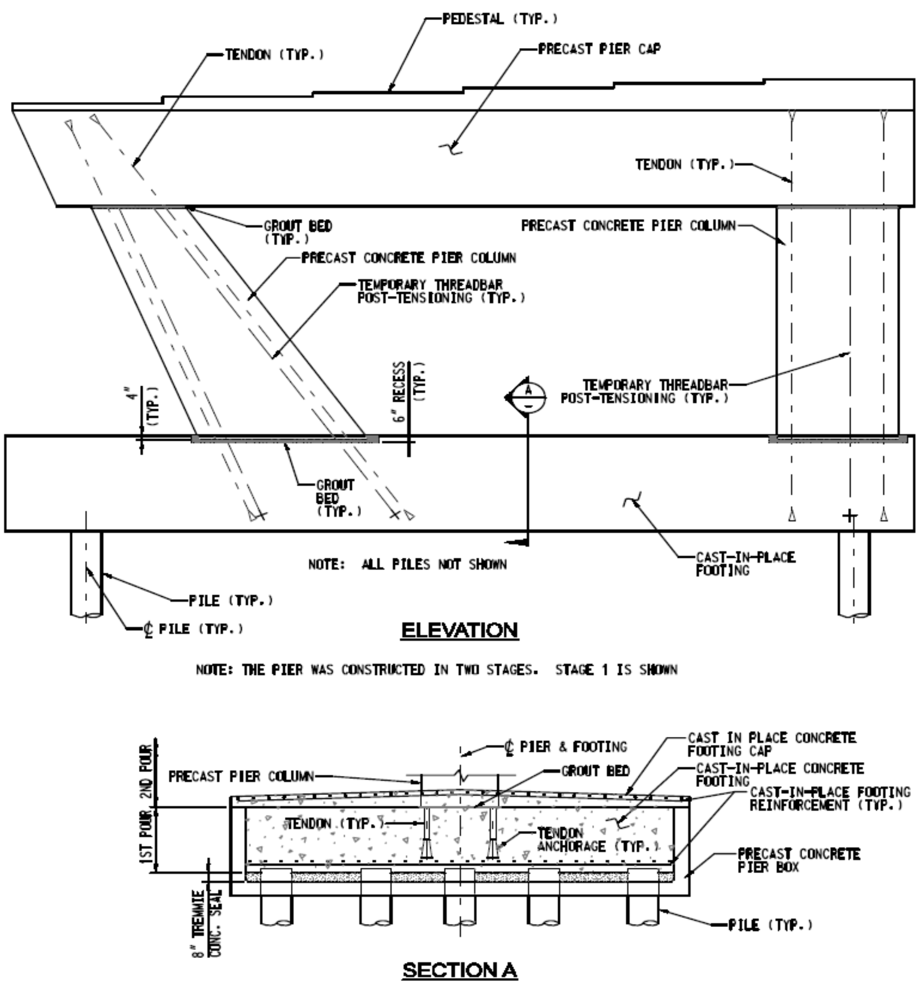
PIER CONNECTIONS

Precast pier column to precast pier cap

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented project: Edison Bridge, Florida Steel grouted reinforcing bar splicer system was used. The splicers can be oversized to accommodate approximately 1/2" of tolerance. The only design effect is that the bars must be moved closer to the center of the members in order to maintain cover around the splicers (approx. 1"). The design incorporates an H-shaped column section and a U shaped cap section to reduce weight. Designed to transmit shear, moment, compression, tension and torsion.

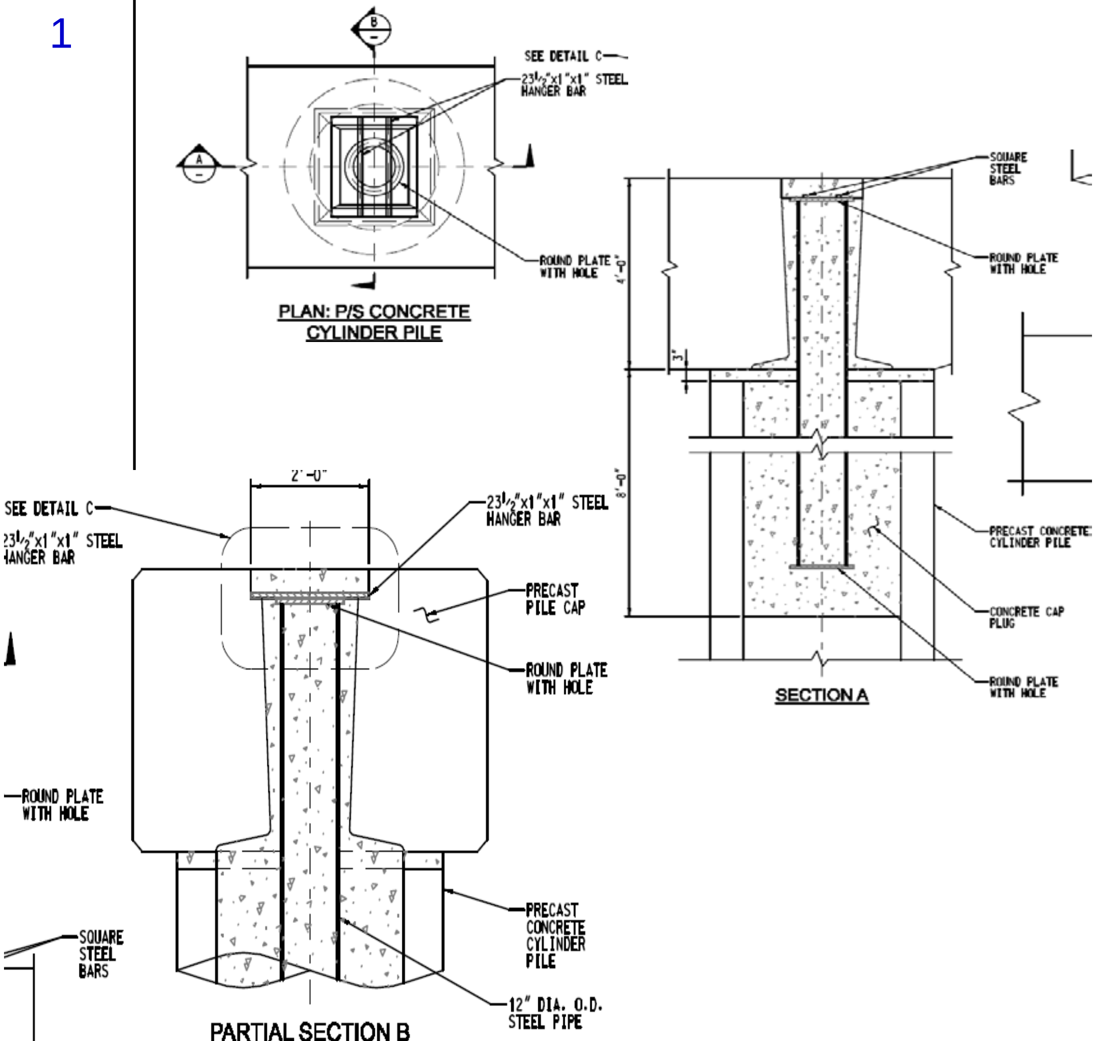
PIER CONNECTIONS

Precast pier column to CIP footing

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented project: Route 70 over Manasquan River, Brielle, NJ The project was designed with Inclined columns with a trumpet shaped anchorages, which allowed strand to be placed and grouted after the column has been erected. All post-tensioning was accomplished with threaded post-tension bars in place of the strand tendons. Once the precast cap beam was installed, the cap was sealed with a cast-in-place concrete cap. Designed to transmit shear, moment, compression, tension and torsion.

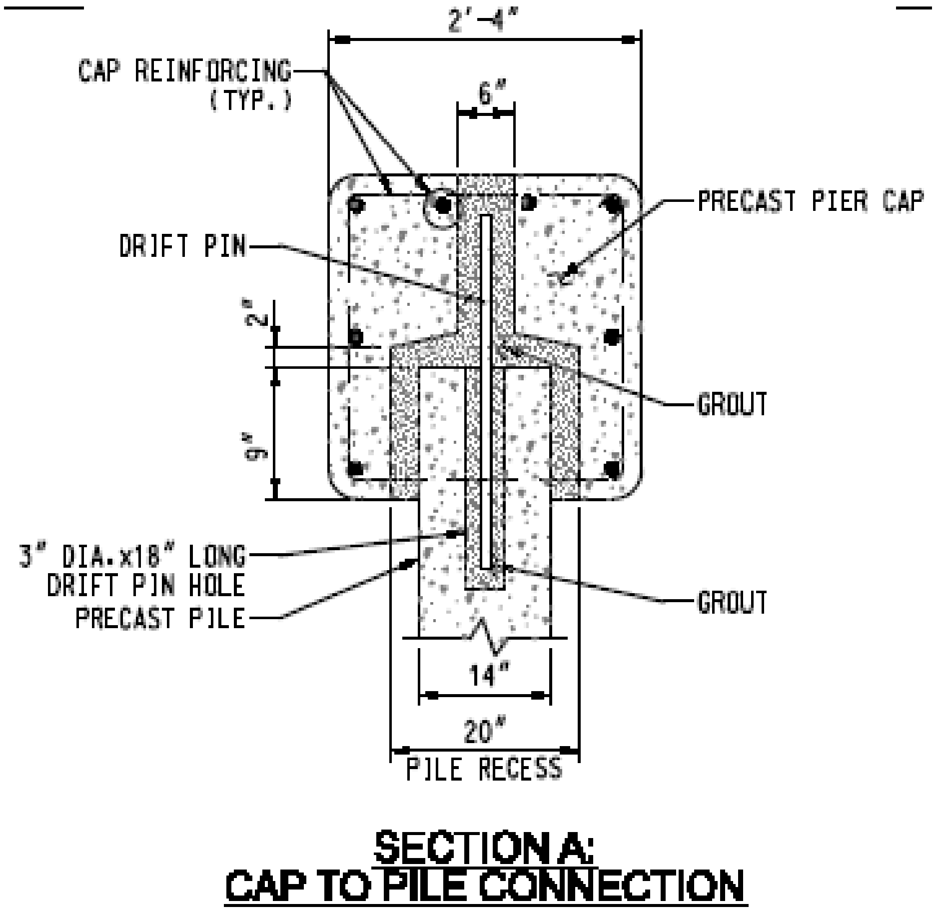
PIER CONNECTIONS

Pre-stressed concrete cylinder pile to Precast pile cap

Rank	Joint details	Description/Comments
1		Implemented project: St George Island Bridge, Florida Water hammer problem on cylinder piles. Concrete spalling at joints between cylinder segments Designed to transmit shear, moment and compression Source: FHWA Connection Manual

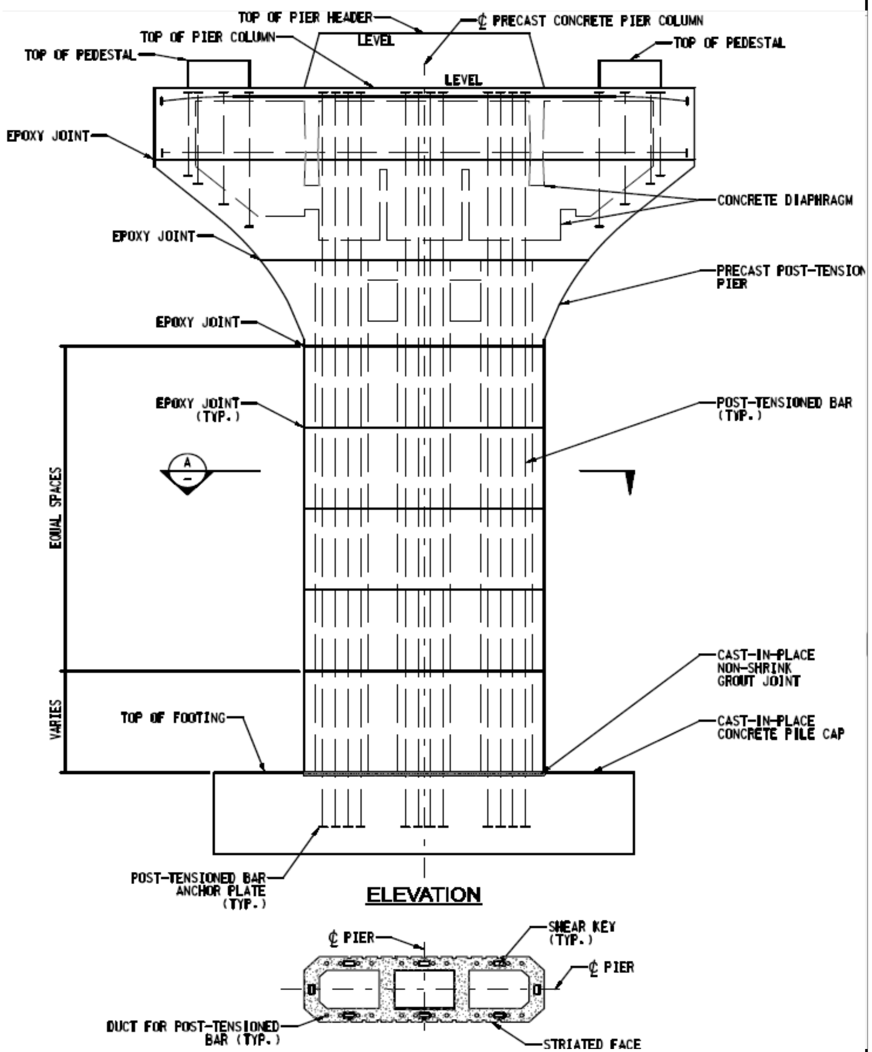
PIER CONNECTIONS

Precast pier cap to precast piles

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented project: Louisiana Forest Highway Kisatchie National Forest Clear Creek Bridge The pier to pile connection is designed as a pinned connection that transfers lateral forces and axial compression.

PIER CONNECTIONS

Precast concrete pier column to precast concrete pier column

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>1</td><td>


</td><td>Implemented project:
Replacement of I-287 Viaduct over the Saw Mill River Parkway, NY

Joints were sealed and bonded with epoxy adhesive. Shear was transferred between pieces by means of shear keys in the precast pieces.

Post-tension rods were embedded in the cast in place footing and spliced with couplers at several levels.

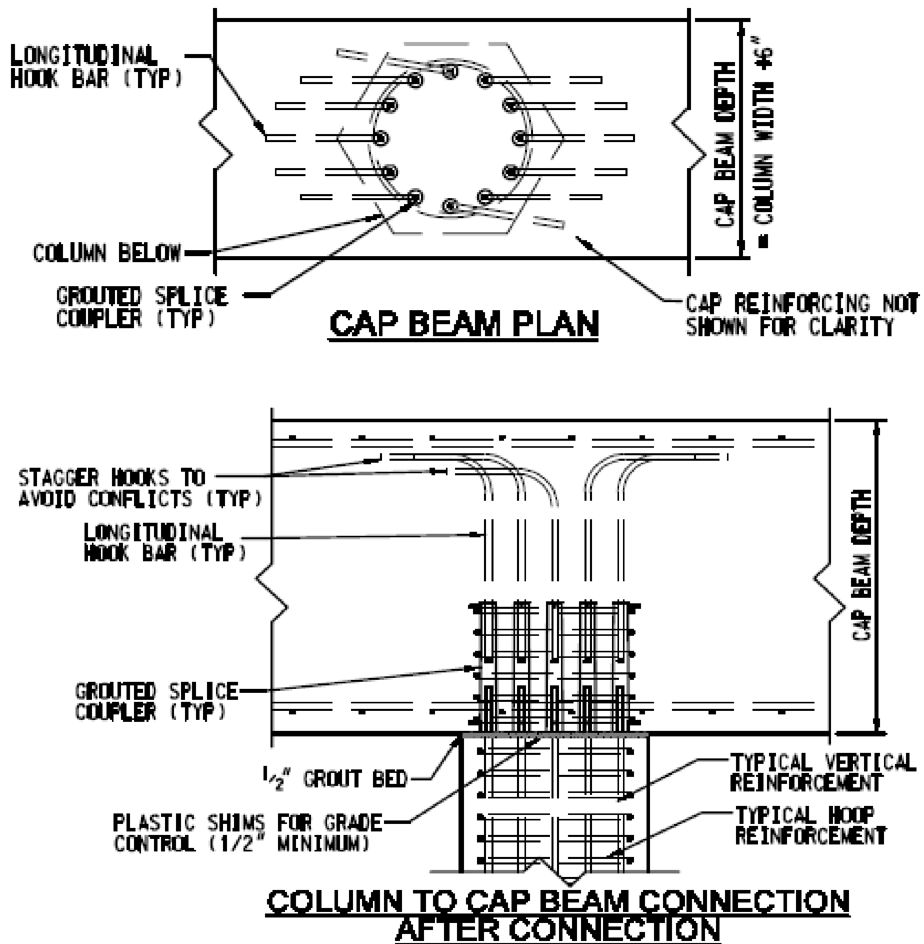
After the installation of all segments, the entire pier was post-tensioned.

Designed to transmit shear, moment and compression

Source: FHWA Connection Manual</td></tr>
</table>

PIER CONNECTIONS

Precast cap beam to precast pier column

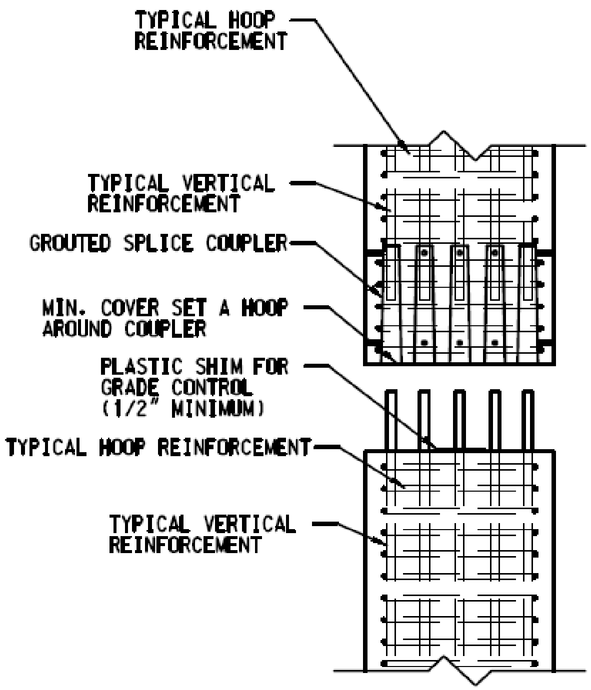
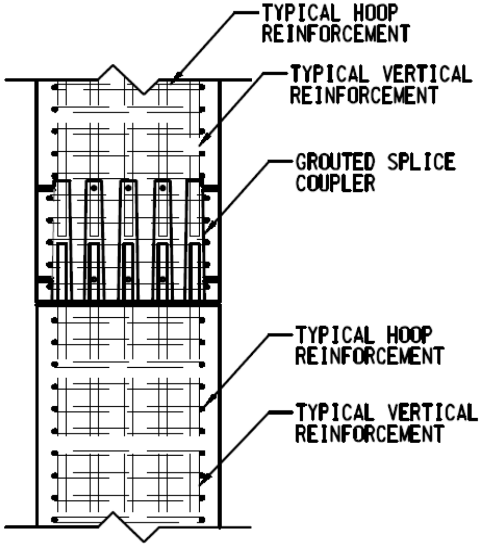
<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>1</td><td>

Source: FHWA Connection Manual</td><td>Implemented in Utah State

Designed to transmit shear, moment, compression, and torsion.</td></tr>
</table>

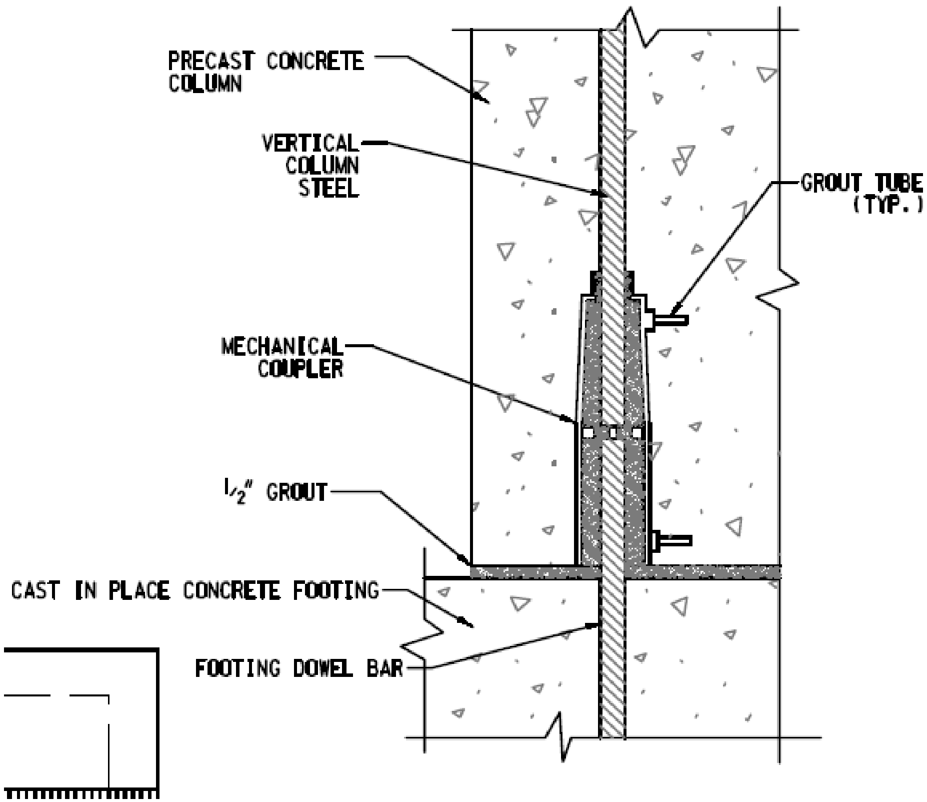
PIER CONNECTIONS

Precast pier column to precast pier column

Rank	Joint details	Description/Comments
1	 COLUMN TO COLUMN CONNECTION PRIOR TO CONNECTION Source: FHWA Connection Manual	Implemented by Utah DOT Designed to transmit shear, moment, compression, tension and torsion.  COLUMN TO COLUMN CONNECTION AFTER CONNECTION

PIER CONNECTIONS

Precast pier column to CIP footing

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr>
<td>2</td>
<td>


Source: FHWA Connection Manual
</td>
<td>Implemented project:
Edison Bridge, Florida

This Connection was made with a steel grouted reinforcing bar splicer system. The grouted splicers can develop over 150% of bar yield.

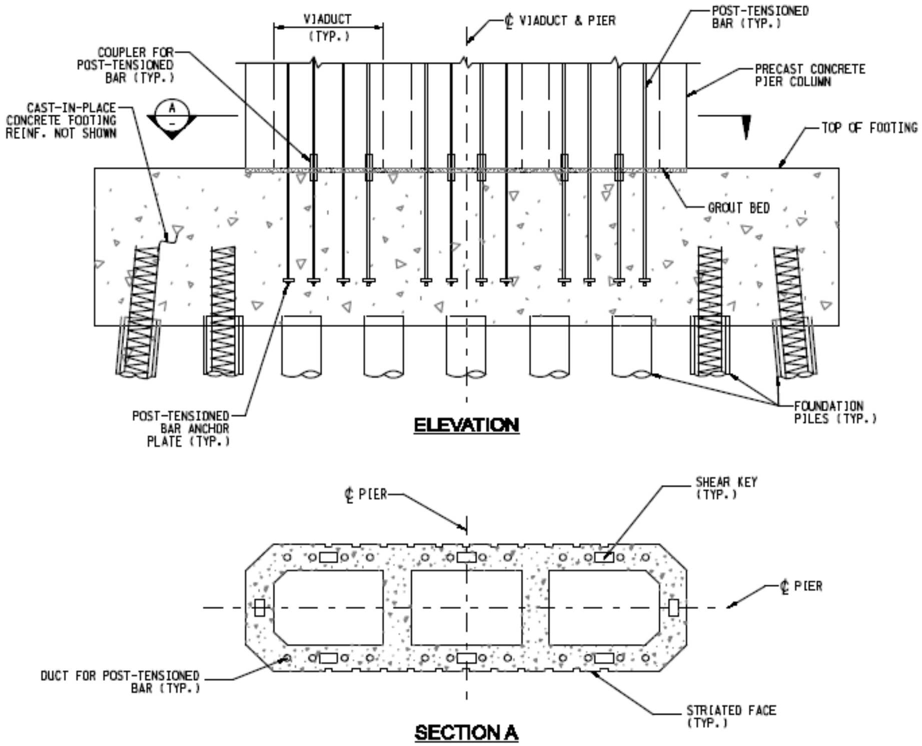
Quality control on bar and splicer locations are critical. The splicers can be oversized to accommodate approximately 1/2" of tolerance.

The only design effect is that the bars must be moved closer to the center of the members in order to maintain cover around the splicers (approx. 1").

Designed to transmit shear, moment, compression, tension and torsion.</td>
</tr>
</table>

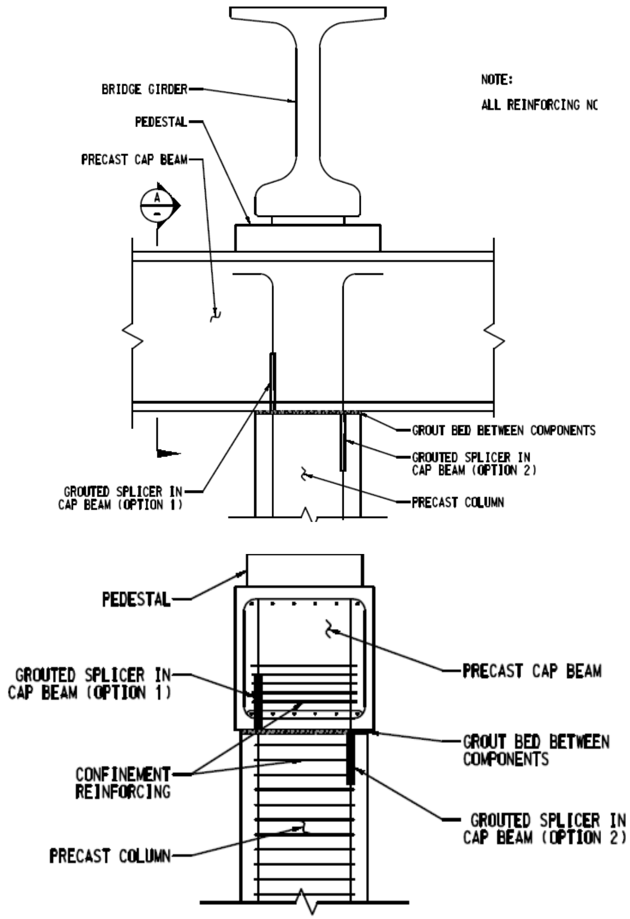
PIER CONNECTIONS

Precast concrete pier column to CIP concrete footing

Rank	Joint details	Description/Comments
1	 Source: FHWA Connection Manual	Implemented project: Replacement of I-287 Viaduct over the Saw Mill River Parkway, NY The detail shown is the connection between the bottom pier segment and the footing. Post-tension rods were embedded in the cast in place footing and spliced with couplers at several levels. Upon the completion of the installation of all segments, the entire pier was post-tensioned. Designed to transmit shear, moment, compression, and tension.

PIER CONNECTIONS

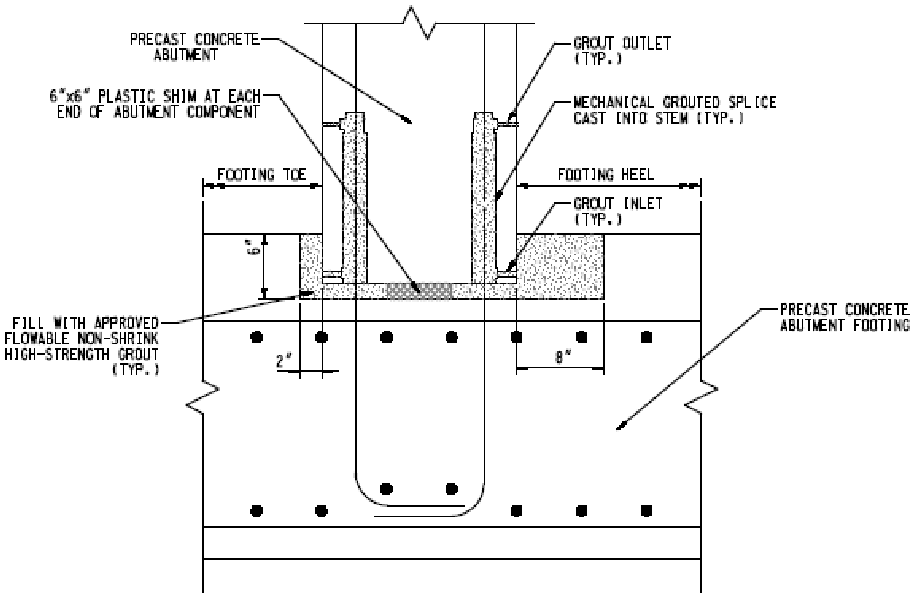
Precast cap beam to precast column

Rank	Joint details	Description/Comments
3	 **SECTION A** Source: FHWA Connection Manual	Conceptual: PCI Northeast Bridge Tech Committee The grouted splicers can be placed in the column or the cap beam. The cap beam is shimmed to grade and grouted. Square, hexagonal and octagonal columns are preferred over round columns because they can be cast in the flat position. Long cap beams may require a joint. This joint can be left open in order to control thermal forces, or can be connected with a small closure pour after placement. Designed to transmit shear, moment, compression, tension and torsion.

SUBSTRUCTURE ABUTMENT CONNECTIONS

ABUTMENT CONNECTIONS

Precast concrete abutment wall to precast concrete footing

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>1</td><td>

DYWIDAG anchor system for connecting the abutments to the footing was suggested to minimize grouting efforts, but do not have specific details available.</td><td>Implemented projects:
NHDOT - Bureau of Bridge Design, Epping, New Hampshire

MDOT, M-25 over the White River, Harbor Beach

The key feature with the connection is the embedded grouted splice sleeve connectors

Care needs to be taken with the casting of the footings and panels within tolerance so that the splice sleeves line up with the extended bars

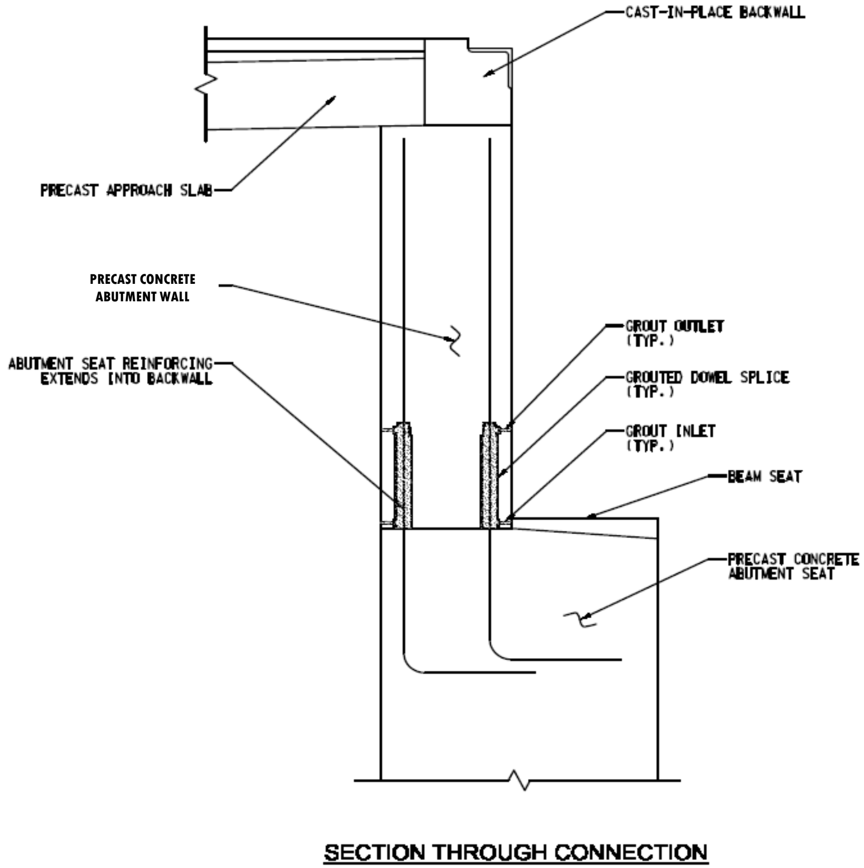
Grouting challenges have been documented

Designed to transmit moment, shear, and bearing

Source: FHWA Connection Manual</td></tr>
</table>

ABUTMENT CONNECTIONS

Precast concrete abutment wall to abutment seat

<table>
<tr><th>Rank</th><th>Joint details</th><th>Description/Comments</th></tr>
<tr><td>2</td><td>

Source: FHWA Connection Manual</td><td>Implemented project:
under NHDOT - Bureau of Bridge Design, Epping, New Hampshire

This detail is similar to the NH wall stem to footing detail. The connection is made with grouted slice sleeves.

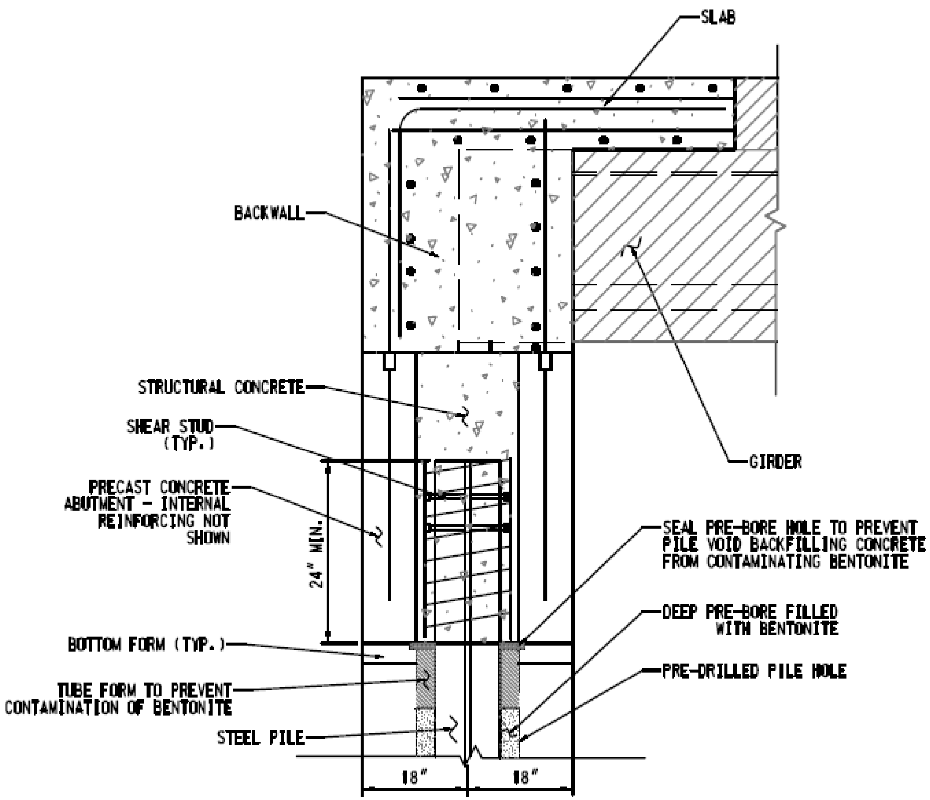
Care needs to be taken with the casting elements within tolerance (about 1/2") so that the splice sleeves line up with the extended bars. This was not a problem during construction.

The design of the connection was according to ACI Manual on Emulation Design. This connection is also used to transfer longitudinal seismic forces from the superstructure to the substructure.

Designed to transmit shear, moment and compression</td></tr>
</table>

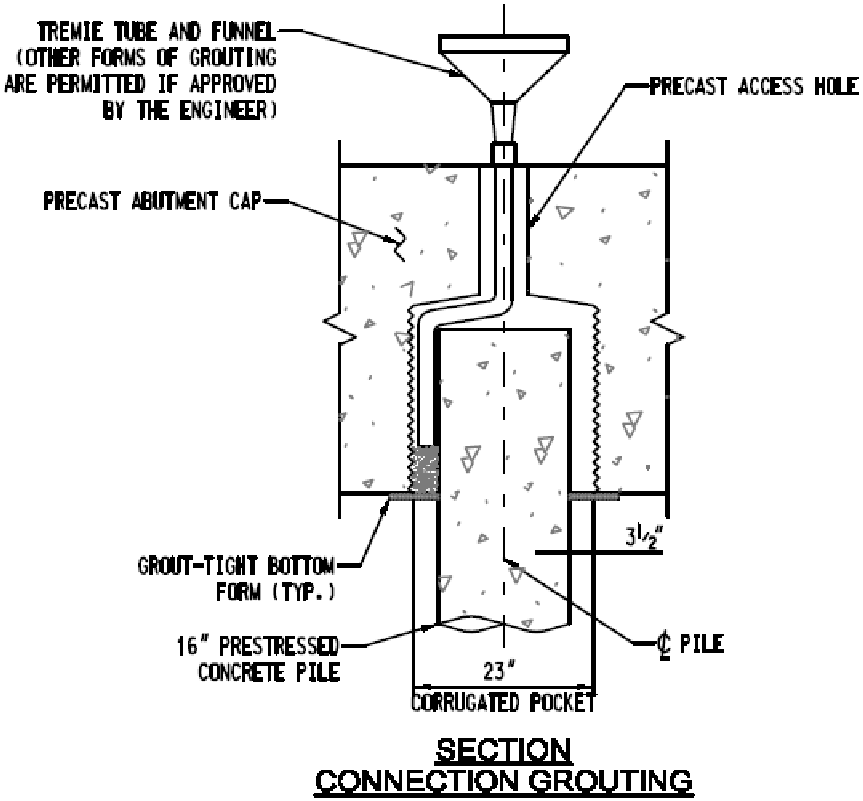
ABUTMENT CONNECTIONS

Precast concrete integral abutment to steel pile

Rank	Joint details	Description/Comments
1	 SECTION THROUGH PRECAST CONCRETE INTEGRAL ABUTMENT Source: FHWA Connection Manual	Implemented projects: Boone County IBRC Project over Squaw Creek, Madison County IBRC Project, Iowa Two 7/8 inch by 5 inch shear studs were welded to each side of the pile web to aid in connection. The footing was placed over the pile and the voids (corrugated metal pipe) were filled with a special concrete mix designed for rapid cure. The void under the precast abutment footing created by this method of construction was filled with flowable mortar. This connection enabled pile to be driven, abutment footing placed, and flowable mortar placed in one day. The concrete in the footing voids were placed the next day. Designed to transmit compression

ABUTMENT CONNECTIONS

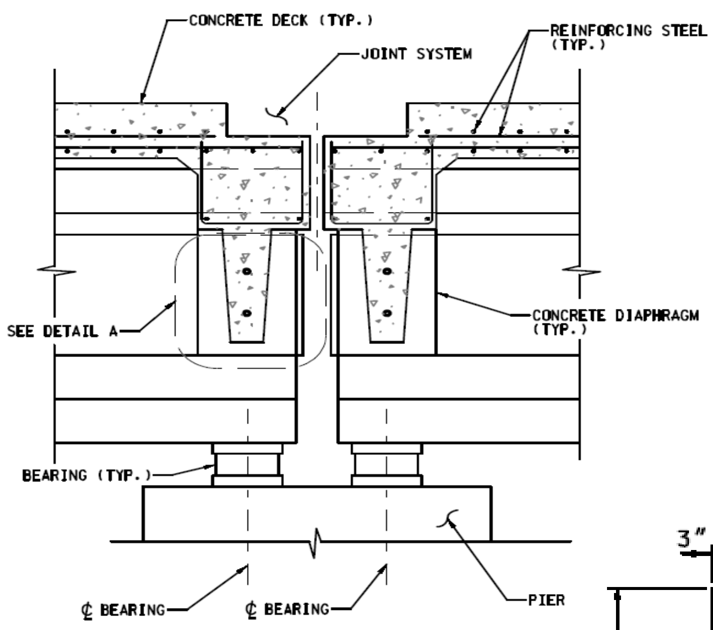
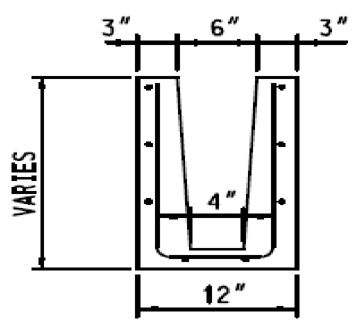
Precast abutment cap to pre-stressed concrete pile

Rank	Joint details	Description/Comments
2	 Source: FHWA Connection Manual	Implemented by Texas Department of Transportation, Tanglewood Designed to transmit shear, moment, compression, tension and torsion.

DECKED STRINGER SYSTEM

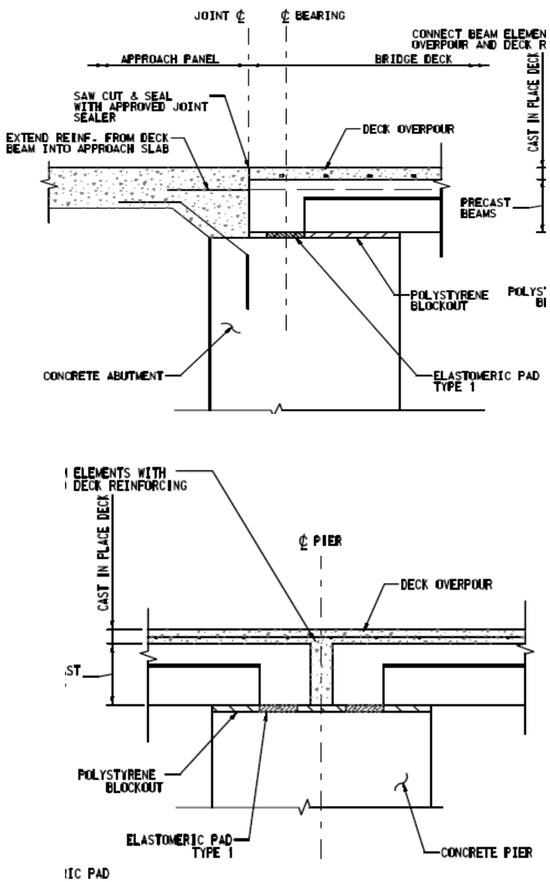
DECKED STRINGER SYSTEM

Diaphragms-to-Precast Spread Girders

Rank	Joint details	Description/Comments
1	 LONGITUNIAL SECTION AT PIER  DETAIL A Source: FHWA Connection Manual	Commonly used in New York State The detail shown is a precast diaphragm that is attached to precast bulb tees and/or AASHTO I-beams. The precast portion of the diaphragm acts as a form. The precast piece is U-shaped and when it is set in place reinforcing runs through the girder webs and into the diaphragm. CIP concrete is then placed in the precast diaphragm to form the connection. Designed to transmit shear, moment and compression

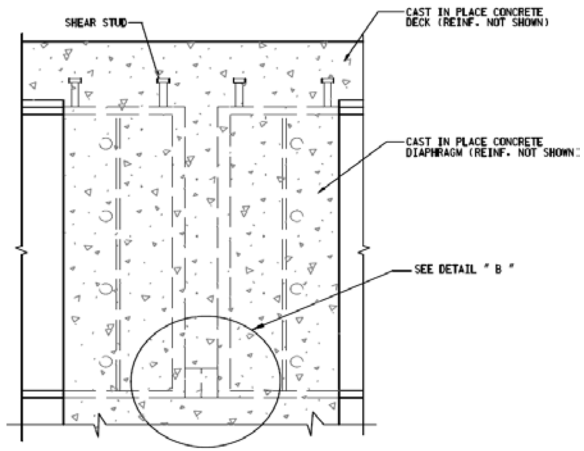
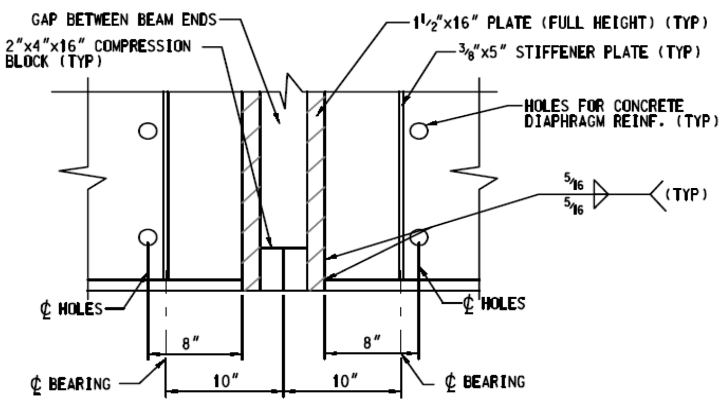
DECKED STRINGER SYSTEM

Inverted Tee beam-to-Inverted Tee beam at pier

Rank	Joint details	Description/Comments
1	 TYPICAL LONG SECTION @ PIERS Source: FHWA Connection Manual	Implemented project: Bridge 13004, TH 8 over Center Lake Channel, Center City, MN Beams rest on elastomeric pads on pier cap with a 4" space between successive spans. Vertical dowels in pier cap, spaced 12" apart over central 50% of bridge width, are between beams. A prefabricated rebar cage spans across pier cap in the channels formed by adjacent Inv-T beams. When deck topping is poured, the space between beams is filled as well, forming a continuous connection. Designed to transmit shear and moment

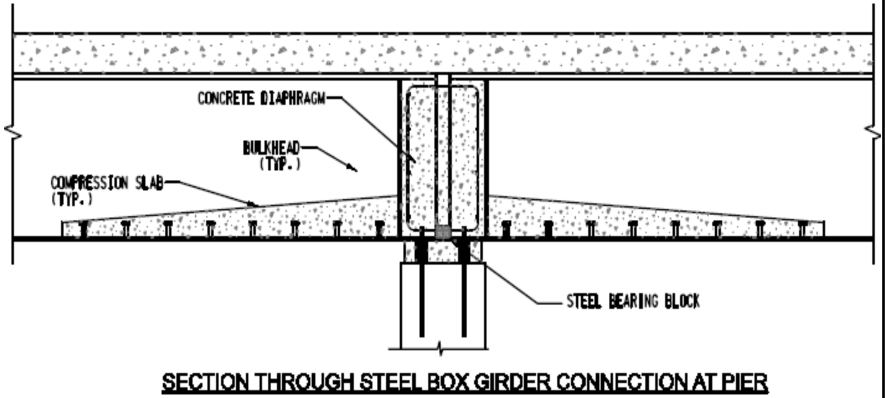
DECKED STRINGER SYSTEM

Steel I-Girder-to-Steel I-Girder connection at pier or abutment

Rank	Joint details	Description/Comments
1	 SECTION AT PIER CONNECTION  DETAIL "B " Source: FHWA Connection Manual	Implemented projects: Sparague Bridge, Lincoln, NE The bottom of girders are in contact through thick plates welded to end bearing plates. The construction sequence proceeds with pouring the concrete diaphragm all the way to bottom of the girder top flange. After about three days after casting concrete diaphragm, the concrete deck could be cast. The continuity for the live loads and superimposed dead loads are provided by reinforcement over the pier, prior to casting deck panel. Designed to transmit shear and compression

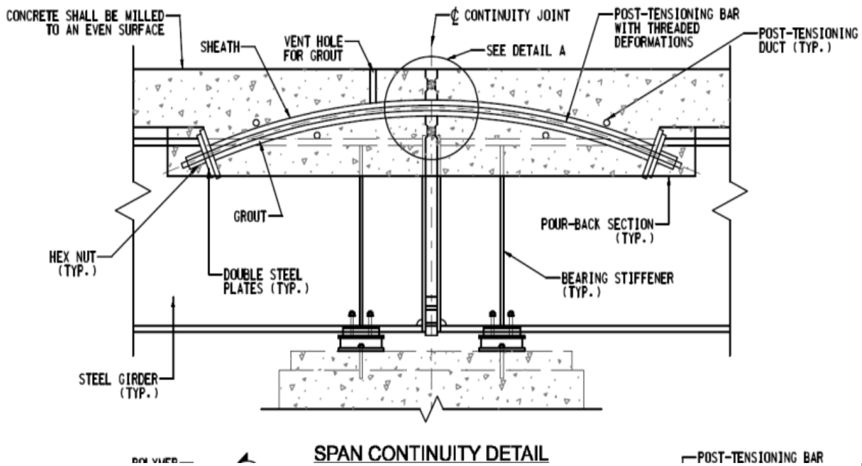
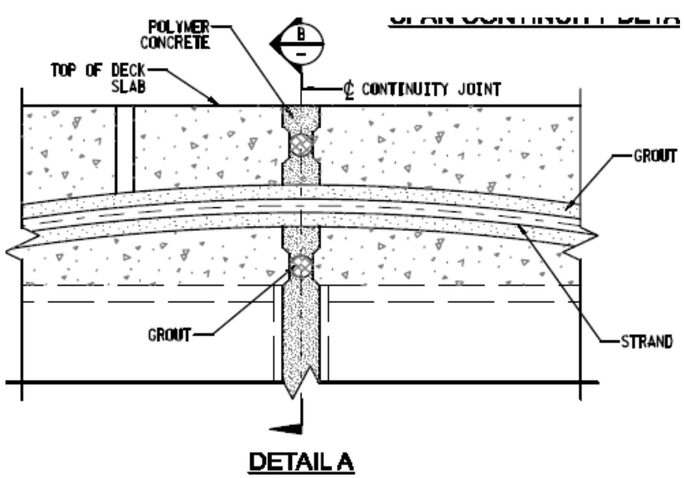
DECKED STRINGER SYSTEM

Steel box girder-to-Steel box girder connection at pier

Rank	Joint details	Description/Comments
1	 Source: FHWA Connection Manual	Implemented projects: N2 Over I-80, Near Grand Island Nebraska The end of girders consists of plates welded to top flanges and thick plates welded to bottom flange of box. The end shown is placed over the pier. The two adjacent box girders contact over the pier via plates welded to the bottom flanges. The steel bulkhead shown serves two purposes. It stabilizes the box ends and provides a formwork for placing the concrete diaphragm over the pier. The continuity for the live loads are achieved through reinforcement placed over the pier prior to casting the deck panel. Designed to transmit shear and compression

DECKED STRINGER SYSTEM

Precast panel unit to precast panel unit over steel girder

Rank	Joint details	Description/Comments
2	  Source: FHWA Connection Manual	Implemented projects: Route I-95 over Lombardy Street and CSX Railroad, VA Designed to transmit shear and compression

APPENDIX C

PERFORMANCE OF BRIDGES CONSTRUCTED USING ABC TECHNIQUES

Full-Depth Deck Panel Systems

Bridge Description	Design Details	Observations
I-84 Bridge, Weber Canyon, Utah, Built 2009		
Deck replacement using precast deck panels.	Three span bridge with a superstructure of prestressed AASTHO I girders with precast deck panels.	Joint Type: Between panels the joint is a welded tie. Observations: Efflorescence and staining was found on the underside of the deck at the transverse joints. Inspected 1- year after construction. (Culmo 2010)

I-15 Bridge, Payson, Utah, Built 2009		
ABC Feature is the full-depth deck panels with post-tensioning. Polymer overlay is used.	Three span bridge with a superstructure of prestressed AASTHO I girders with precast deck panels.	Joint Type: Longitudinal post-tensioning to compress the transverse joints. Transverse dowels spaced at 9" centers between panels develop the longitudinal closure. Polymer overlay is used to increase the water-tightness. Observations: Very little efflorescence found, the joint's performance is good. Less staining than on bridges with a welded tie connection. The deck is in good condition and the post-tensioning and polymer overlay keep the joints water-tight. Inspected 1- year after construction. (Culmo 2010)

I-80 Silver Creek Canyon Bridge, Utah, Built 2007		
Precast full-depth deck panels used as an ABC feature.	Four simple span bridge with steel girders and a continuous deck.	Joint Connection: Welded Tie connection between panels and a bituminous overlay. Observations: Efflorescence staining found at the joints and active water leakage seen through the joints. The joint at the abutment is showing heavy leakage. The continuous deck panels subjects them to negative moment over the piers which could be a cause of the leakage. The deck panels are also integrated into the backwall and does not allow for thermal movement which is cause for the leakage at the abutments. Inspected two years after construction. (Culmo 2010)

I-84; US-89 to SR-167, Weber Canyon, Utah, Built 2008		
ABC feature is the use of full-depth precast deck panels.	Eight span bridge, two continuous spans and six simple spans. The superstructure consists of steel girders and precast deck panels.	Joint Connection: The connection between panels is a welded tie with a polymer overlay. Observations: Efflorescence seen at the transverse joints and the haunches and the closure pours. Staining seen on the underside of the deck panels. Placement of the deck panels during construction was not monitored and the fit of the panels was difficult. The spacing of the panels was increased to accommodate tolerance issues.

		First inspected one year after bridge construction. (Culmo 2010)

I-84; US-89 to SR-167, Weber Canyon, Utah, Built 2008		
ABC feature is the precast full-depth deck panels.	Six span bridge, five continuous spans and one simple span. The superstructure consists of steel girders and precast deck panels.	Joint Connection: The connection between panels is a welded tie and a polymer overlay. Observations: Efflorescence seen at the transverse connections. Staining seen on the underside of the deck panels. The simple support span shows very little leakage and the continuous spans show active leakage at every joint. First inspected one year after bridge construction. (Culmo 2010)

I-215 East over 3760 South and 3900 South, Utah, Built 2006		
ABC feature is the precast post-tensioned deck panels.	The bridge consists of three spans. The superstructure consists of steel girders and precast deck panels.	Joint Connection: The joint connections between panels are post-tensioned deck panels with a polymer overlay. Observations: Connections are performing well. Cracks and leakage is seen in the closure pour. First inspected three years after bridge construction. (Culmo 2010)

I-15 at Parrish Lane Ramp and Interchange Improvement, Utah, Built 2004		
The ABC feature is a widening using precast deck panels and precast bent caps.	The bridge consists of two continuous spans. The superstructure consists of steel girders and precast deck panels.	Joint Connection: The connection was welded tie and has a bituminous overlay. Observations: Efflorescence is seen on the underside of the deck panels at the transverse joints. Active leakage is seen through many joints including the closure pour at the backwall abutment, haunches, transverse panel joint, longitudinal closure pour and the shear connector pockets. Inspected six years after construction. (Culmo 2010)

Riverdale Road, I-15 to Washington Blvd, Utah, Built 2008		
The ABC elements to this bridge were the precast deck panels, approach slab panels, sleeper slab, and abutments.	The bridge superstructure has two spans with steel girders and precast deck panels.	Joint Connection: The panels were connected together using longitudinal post-tensioning. The overlay was a polymer. Observations: A minor amount of leakage occurred prior to the placement of the overlay. Since the overlay was placed no leakage was observed. The overlay is cracking near the backwall and is causing the sleeper slab to deteriorate. The leakage that is present is most likely due to the shrinkage cracks. The bridge was inspected two years after construction. (Culmo 2010)

I-80; Kimball Jct to Silver Creek, Utah, Built 2008		
The ABC technique used was a full-depth deck panel to widen the bridge.	The bridge has one span and uses AASHTO I-Girders in the superstructure of the bridge.	Joint Connection: The connection between panels was a welded tie with a three inch asphalt overlay. Observations: Active leaking was found at the transverse joints. The overlay was cracking at the end of the approach slab. The leaking is being controlled by the bituminous overlay. The shrinkage of the closure pour concrete has caused the deck panels to spread and allow for more leakage. The bridge was inspected two years after construction. (Culmo 2010)

US-6; MP 200 Bridge Replacement, Utah, Built 2008		
The ABC techniques used were precast deck panels, approach slab panels, sleeper slabs, and abutments.	The single span bridge consists of prestressed girders and precast deck panels in the superstructure.	Joint Connections: The panels were connected with closure pours. The overlay has membrane and bituminous. Observations: Minor leakage seen from the underside. Efflorescence seen at the closure pour at abutment to backwall, haunches, transverse panel joints. Shrinkage cracks are seen at the longitudinal closure pour. The bridge was inspected two years after construction. (Culmo 2010)

County Road over I-80, 1.9 Miles East of Wanship, Utah, Built 2003		
The ABC technique was the use of full-depth deck panels.	The bridge has five spans and uses steel girders and precast deck panels for the superstructure.	Joint Connections: The connections between panels were a closure pour with cast concrete. The overlay was a polymer. Observations: Minimal creaking and leakage has been found. The cracks seen on the underside appear to be shrinkage cracks. The bridge was inspected seven years after construction of the bridge. (Culmo 2010)

I-70; Eagle Canyon Bridge, Utah, Built 2009		
The ABC technique was the use of precast deck panels.	The bridge has one span and uses a steel deck arch and precast deck panels for the superstructure.	Joint Connections: The connections between panels were post-tensioned. The overlay was a polymer. Observations: No signs of leakage have been found. The bridge is in good working order. The bridge was inspected one year after construction of the bridge. (Culmo 2010)

Parrish Lane Railroad Bridge Crossing, Utah, Built 2008		
The ABC techniques were the use of precast deck panels and precast bent caps.	The bridge has three spans and uses a steel plate girder and precast deck panels for the superstructure.	Joint Connections: The connections between panels were post-tensioned. The overlay was bituminous asphalt. Observations: The bridge is in good working order. There are minor

		cracks seen at the joints and shear connector pockets. The bridge was inspected two years after construction of the bridge. (Culmo 2010)

I-84; US-89 to SR-167, Weber Canyon, Utah, Built 2009		
The ABC technique was a deck replacement with precast deck panels.	The bridge has three spans and uses prestressed AASHTO I girders and precast deck panels.	Joint Connections: The connections between panels were a welded tie with a polymer overlay. Observations: There is much staining and efflorescence seen on the underside of the deck at the transverse joints, haunches, and closure pours. No sign of leakage is seen at the shear pockets. The welded tie connections do not appear to be performing well. The bridge was inspected one year after construction of the bridge. (Culmo 2010)

I-15; Bridge Deck Replacements; F-127, F-129 & F-130, Payson, Utah, Built 2009		
The bridge used full depth post-tensioned deck panels as the ABC technique.	AASHTO I girders and precast deck panels were used for the superstructure of this bridge. The bridge consisted of three spans.	Joint Connections: The connections between panels were longitudinal post- tensioning. Longitudinal closure had transverse dowels spaced at 9”. The overlay is a polymer. Observations: When compared to the welded tie connection there is less efflorescence and leaking seen. The bridge is in good working condition.

		The bridge was inspected one year after construction of the bridge. (Culmo 2010)

SR-89;State Street Railroad Bridge, Pleasant Grove, Utah, Built 2009		
The bridge used precast deck panels, approach slab, and sleeper slabs.	The single span bridge superstructure consist of AASHTO I girders and precast deck panels.	Joint Connections: The connections between panels were welded tie spaced at 2'centers, perpendicular to beams. Welded ties were used for longitudinal closure pours at quarter points. The overlay was a polymer. Observations: The efflorescence and leakage seen is minimal. Some staining is seen at the closure pour. The sleeper slab is not lined up right with the approach slab. The overlay is cracking at every approach slab joint and over the backwalls. The bridge is in good condition but should be inspected again due to the young age of the bridge. The bridge was inspected in less than a year after construction of the bridge. (Culmo 2010)

US-6; MP 218.7 to Emma Park Road, Utah, Built 2009		
The ABC techniques used were precast post-tensioned deck panels, post-tensioned approach	The single span bridge superstructure consists of AASHTO I girders and precast GRFD deck panels.	Joint Connections: The connections between panels were post-tensioned longitudinally with one longitudinal closure pour at bridge center line. Also, transverse closure pours at the backwall and approach slab joint was used. A polymer overlay is used.

slab panels, and sleeper slab panels.		Observations: Both of the closure pours are leaking and have efflorescence. The minor cracking was documented in the closure. The bridge was inspected a year after the construction. (Culmo 2010)

B-552 Bridge in Bedford county, Everett, Pennsylvania. Built prior to 1973, for a private community.		
One lane simply supported bridge with full-width, full-depth precast concrete panels designed for low volume traffic.	Designed for a private community access. Prestressing or post-tensioning was not used.	Joint type: The panels are held adjacent to each other using slab-girder connection (tie-downs). Width of the joints between deck panels is 0.375 in. typically and 0.44 in. adjacent to the center panel. Observations: Steel girders were extensively corroded but precast concrete panels were in good condition with no leaking, leaching or cracking. However, efflorescence was present along slab and curb joints. The wearing surface had some areas with exposed aggregate. The bridge was inspected during 1993-95. (PTC 2012; Issa *et al.* 1995b)

Dublin 0161 Bridge, Columbus, Ohio. Year built is 1986.		
Two-phase construction was adopted to replace the six-span skew bridge deck. Full-depth	Thickness varied from 9.5 in. - 14 in. Panel width: 28 ft.	Joint type: Epoxy mortar was used as grout for shear key connection between panels in both transverse and longitudinal joints. Panels are post-tensioned longitudinally after erection to secure the joints.

precast reinforced concrete panels with 2.5 in. asphalt concrete wearing surface.	Panel length: 12 ft 1.5 in., 9 ft 10.5 in., 9 ft 6.5 in., 9 ft 5.5 in. and 10 ft 1 in. Panels are not prestressed.	Observations: The bridge superstructure was found to be very rigid but with random cracks on overlay. No signs of leaking, leaching or debonding were present. The bridge was inspected during 1993-95 (i.e., about 7 years after construction). (Issa *et al.* 1995b)

Harriman Interchange Bridge, Orange County, New York. Year built is 1979.		
Three-span, two-lane ramp. Each span is 75 ft long.	Panel thickness: 8 in. Skew precast concrete panels are not leveled on beam flanges to control super-elevation. Thus, epoxy mortar bed, thicker on one edge of flange than other, was used to level it.	Joint type: 3 ft wide cast-in-place longitudinal joint connects adjacent panels. Transverse joints were not post-tensioned. Deck had a membrane and a 6 in. overlay. Observations: Reflective cracks on asphalt near transverse joints, and random cracking and spalling of joints at the bottom side of the panels. The bridge was inspected during 1993-95 (i.e., about 14 years after construction). (Biswas 1986; Issa *et al.* 1995b)

Burlington Bridge over Mississippi river, Illinois – Iowa border. The bridge was built during 1992 – 1993.		
Two cable-stayed spans with 10 in. precast concrete deck covered with 2-in. low slump dense concrete	Panel thickness: 10 in. Panel width: 46 ft 8 in, 37 ft 8 in Panel length: 13 ft 9 in Post-tension spacing: 1 ft 4.75 in.	Joint type: 15 in. gap joints between adjacent precast panels were filled with cast-in-place type-III cement-concrete. Shear pockets and longitudinal joints were filled with class-D concrete. Haunch was formed with high flow, high strength, early load-bearing, non-shrink

overlay. Corrosion inhibitor admixture was incorporated in the concrete of precast concrete deck panels.	Initial post-tension force: 89 and 166 kips	grout. A group of three panels were post-tensioned in transverse and longitudinal directions. Observations: No major joint leaking was observed. The fatigue cracks were observed which were formed as a result of construction equipment vibrations on the deck (note that only three panels were connected at a time). Nevertheless, the bridge was performing well. The bridge was inspected during 1993-95 (i.e., about 1 year after construction). (Issa *et al.* 1995a; Issa *et al.* 1995b)

Bayview Bridge over Mississippi River, Illinois-Missouri border. Year built is 1986. The bridge was opened to traffic in 1987.		
Three-span cable stayed bridge. Fourteen continuous approach spans with 9 in. cast-in-place concrete deck. The main three spans over the river consists of 9 in. precast deck panels covered with a 1.75 in. waterproofed	Panel thickness: 9 in. Panel width: 46.5 ft Panel length: 9 - 11 ft Posttension spacing: 7 in. Initial stress: 105 ksi Three to five panels are post-tensioned to form groups.	Joint type: Butt joints Observations: No significant problems at the joints although some presence of rust, indicating some leakage. The bridge was inspected during 1993-95 (i.e., about 6 years after construction). Major reason for the observed performance is that the cable-stayed precast deck is in compression. (Issa *et al.* 1995b)

bituminous wearing surface.		

B-501 Bridge, Exit ramp for Pennsylvania Turnpike, Somerset County, Pennsylvania. Year built is 1940; Resurfaced in 1970.		
Simple span bridge with full-depth precast concrete panels and an overlay of latex modified concrete.	Panel thickness: 8.5 in. Precast reinforced concrete panels (i.e., no prestressing). No post-tensioning after erection of the panels to compress the joints.	Joint type: Panel-to-panel connection - 65% of panel portion from bottom forms a dry joint. The depth and width of the remaining 35% of the joint are 3 in. and 0.75 in., respectively. A depth of 1.25 in. from the bottom of the 3 in. deep gap is filled with rubberized joint seal material while the rest is covered with neoprene compression joint seal. Observations: Steel girders had extensive surface corrosion. Several tie-downs, which established the deck panel – girder connections, were missing. Excessive leakage was noted at precast deck panel joints. Top surface indicated some active cracks although prior patching was performed on it. The bridge was inspected during 1993-95 (i.e., about 50 years after construction). (Issa *et al.* 1995b; COP 2007)

Waterbury Bridge, Connecticut. Year built is 1965. Entire bridge was reconstructed in 1989.		
Six-span bridge with three continuous spans and a hung span supported by pin and	Panel thickness: 8 in. Panel width: 26 ft 8 in. Panel length: 8 ft	Joint type: Transverse joints are formed by filling female-to-female type joints with high strength non-shrink grout.

hanger connections. The width of the bridge is 27 ft 6 in.	Initial stress: Arbitrary stress of 150 psi for simple spans and 300 psi in the three-span continuous portion was applied. Panels are covered with a Class I waterproofing membrane and a 2.5 in. thick bituminous layer.	Observations: No joint cracking or leaking reported. The bridge was inspected during 1993-95 (i.e., about 4 years after construction). (Issa, *et al.* 1995b)

Route 235 Bridge over Dogue Creek, Fairfax, Virginia. Year built is 1932; Rehabilitated in 1982.		
Information related to structural configuration is not available in literature.	Panel dimensions were not available in literature. Class I waterproofing covers the entire deck Precast concrete panels are neither prestressed nor post-tensioned longitudinally.	Joint type: Standard configuration of female-to-female type grouted shear key joints was provided between panels. Adjacent panels were spaced with 0.25 in. gap at the bottom, which is the recommended practice. Observations: Leakage, cracking and rust stains visible at joints. Transverse deck cracks and efflorescence were noted at construction joints. The bridge was inspected during 1993-95 (i.e., about 11 years after construction). (Issa *et al*. 1995b)

Vischer Ferry Road Bridge, Schenectady County, New York. Year built is 1900, destroyed in 1902 with ice; Rebuilt in 1975; Deck was rehabilitated in 1980.		
Information related to	Haunch was formed using 0.5 in.	Joint type: Panel-to-panel joint details are not available in literature.

structural configuration is not available in literature.	thick stiff grout placed on structural steel members. No longitudinal post-tensioning was provided.	Observations: Minor leakage beneath the deck was noted. It is believed that the low volume of traffic is the cause of better performing deck. The bridge was inspected during 1993-95 (i.e., about 13 years after construction). (Issa *et al*. 1995b)

Krumkill Road Bridge, Albany County, New York. Year built is 1939; Rehabilitated (i.e., deck replaced along with installing welded headed studs to create composite action) in 1977		
Fifty feet long, single span mainline throughway bridge. The bridge consists of two structurally separated three-lane spans supported on common abutments.	Panel thickness: 7.5 in. Panel width: 42 ft and 21 ft. Panel length: 5 ft 2 in. No post-tensioning was applied in the longitudinal direction to compress the transverse joints. A membrane and a 6 in. thick asphalt layer were placed on top of the deck panels.	Joint type: 3 ft wide cast-in-place longitudinal joint. Observations: The bridge was under the category of high traffic volume bridges. Fracture and spalling was observed at transverse joints due to lack of post-tensioning. Moreover, frequent leakage was observed through the transverse joints, which was leading to corrosion of girders. Cracks were also present in the overlay. The bridge was inspected during 1993-95 (i.e., about 17 years after construction). (Biswas 1986; Issa *et al*. 1995b)

Batchellerville Bridge, Saratoga County, New York. Year built is 1930. Deck replaced completely in 1982 along with installation of new floor beams.		
Total length of the bridge is 3,075 ft. Deck width is 28	Panel thickness: 8.5 in. Panel width: 28 ft.	Joint type: Grouted shear keys.

ft.	Panel length: 11 ft 8 in. to 13 ft variable Transverse slab joints are located on top of floor beams. No post-tensioning is present. Two-inch thick asphalt concrete overlay was placed on the full-width precast concrete panels that were mounted on the new floor beams.	Observations: Minor deboning in the joints between precast panels and some cracks in asphalt concrete overlay were noted. Low traffic volume is the cause of the excellent working condition of the structure. The bridge was inspected during 1993-95 (i.e., about 10 years after construction). (NYState DOT 2012; Issa *et al.* 1995b)

Oakland Bay Bridge, San Francisco, California. Rehabilitated during 1960-61.		
Original design was to accommodate trucks and trains on the lower deck and ordinary cars on the upper deck. During 1960-61, the train lines were removed and precast deck panels were placed adding two additional traffic lanes. Due to Loma Prieta	Lightweight concrete deck panel dimensions are not given in literature.	Joint type: 12 in. wide closure pours in between adjacent panels. Observations: Cracking and leaching present in both precast and cast-in-place areas of deck. However, more cracking and leaching was visible on the cast-in-place concrete deck. The bridge was inspected during 1993-95 (i.e., about 33 years after construction). (Issa *et al.* 1995b)

earthquake in 1989, a small section of the lower deck was damaged and the entire width in that section was replaced with precast concrete panels without an overlay.		

Chulitna River Bridge, Over Chulitna River, Alaska. Year built is 1921; Full superstructure rehabilitated in 1992		
The bridge is 790 ft long. Full-depth, full-width precast panels were placed as part of the rehabilitation program. Width of the bridge is 42 ft 2 in. Stage construction was executed to facilitate mobility.	Panel thickness: 10 in. to 7 in. variable. Panel width: 21 ft 0.75 in. Panel length: 12 ft 0.625 in. to 3 ft 0.625 in. variable at edges and 8 ft overall at all other locations. The deck panels were covered with a waterproofing membrane and a 2 in. thick asphalt overlay. Width of the waterproofing membrane was limited to 18 in. but placed over all the joints.	Joint type: Transverse joints are grouted female-to-female connections without post-tensioning. High strength quick set grout (magnesium phosphate concrete) was used for shear pockets, haunches, and joints. Observations: Underneath the bridge minor leaking and leaching was found along with minor debonding at the joints. No overlay cracking was observed and was in good condition. There was a concern on possible rotation at the joints due to partial-depth grouting leading to debonding, leaking and leaching. The bridge was inspected during 1993-95 (i.e., about 1 year after construction). (Issa *et al.* 1995a; Issa *et al.* 1995b)

Cochecton Bridge over Delaware River, Sullivan County, New York. Year built is 1950. Deck panel replacement in 1978.		
The three-span, two-lane, truss bridge is 675 feet long. Staged construction techniques were implemented to facilitate mobility.	Panel thickness: 7.5 in. Panel width: 15ft 4 in. and 13ft 10.5 in. Panel length: 7ft 6in. Deck panels were covered with a waterproofing membrane and a bituminous wearing surface. No post-tension across the joints.	Joint type: Type-II Portland cement with two parts of mortar sand was used as a grout to fill transverse and longitudinal joints. Observations: Reflective cracks were noted along the longitudinal joint. Transverse reflective deck cracking was observed over every other panel joint. Minor spalling was also observed under the deck. The bridge was inspected during 1993-95 (i.e., about 15 years after construction). (Baughn 2012; Issa *et al.* 1995b)

Route 155 Bridge over Normanskill, Albany County, New York. Year built is 1928 and deck replacement after 1972 with full depth precast concrete panels, along with transverse girders held by two trusses.		
Single span bridge. Bridge dimensions are not given in literature.	Panel thickness: 10 in. Panel width: 6 ft 4 in. Panel length: 12 ft 4 in. and 13 ft 4 in. Precast concrete panels were placed on transverse steel girders spaced at 12 ft 6 in. that were held by two trusses on either side.	Joint type: 1.75 in. wide female-to-female longitudinal shear key was filled with non-shrink cement grout. Two-inch wide transverse joints were connected with non-headed 0.75 in. diameter shear studs (4 in. long for intermediate panels and 1 in. long for the end panels) and then filled with grout. Observations: Transverse joints popped out; thus, filled with a foam stopper. Deck was leaking at the transverse joints while asphalt surface showed random cracking. The bridge was inspected during

		1993-95 (i.e., about 21 years after construction). (Issa *et al.* 1995b)

Seneca Bridge, Lasalle County, Illinois. Year built is 1932. The deck was replaced in 1986.		
The bridge consists of 13 spans, of which 9 of them are approach spans while the remaining for spans are interior truss spans. Total length of the bridge is 1510 ft 3 in.	Panel thickness: 6.5 in. Panel width: 23 ft Panel length: 4 ft Post-tension spacing: 2 ft 10 in. One-inch diameter grade 150 deformed bars were used to post-tension the panels. Eight such bars spaced at 2 ft 10 in. were used across the bridge width. Initial stress of 45 ksi was applied. Two-inch thick Class I concrete overlay with a waterproofing membrane cover the panels	Joint type: Match cast with epoxy adhesives Observations: Random cracks at the approach spans, joint leakage, and corrosion on steel supporting system were observed. The bridge was inspected during 1993-95 (i.e., about 7 years after construction). (Issa *et al.* 1995b)

William Preston Jr. Memorial Bridge over Chesapeake Bay, Sandy Point, Maryland. Year built is 1952. In 1987, deck was replaced with precast panels for most of the spans and the remaining with cast-in-place slabs.		
The bridge has two traffic lanes in each direction. Four full-width precast concrete panels are placed on each span.	Panel thickness: 6 in. Panel width: 31 ft 2.5 in. Panel length: 14 to 15 ft Precast concrete panels were not prestressed. The panels were post-tensioned in longitudinal direction to clamp the joints. Six-inch thick deck with two-inch thick layer of Latex Modified Concrete was placed on top of the panels as an overlay.	Joint type: Female-to-female grouted transverse joints between precast panels. Bottom portion of the joints are tight fit; hence, any size irregularities and dimensional growth of panels are not allowed. Observations: Both sides of deck were showing diagonal and map cracking due to absence of prestress in transverse direction. Though latex modified concrete had been used for the haunch, concrete popouts were visible where there was no adhesion between the deck and the supporting structure. Joint leakage under the deck at numerous locations with deposits and stains were observed; including presence of spalling with some steel exposure. The bridge was inspected during 1993-95 (i.e., about 6 years after construction). (Issa *et al.* 1995b; Eastern Roads Org. 2009)

High Street Overhead Separation Bridge, California. Year built is unknown. In 1978, 29 out of 30 spans were replaced with precast concrete deck panels.		
The left and right side of the bridge was widened in 1955 and 1963, respectively.	Panel thickness: Varies from 6.5 in. to 7 in. Panel width: 14 ft 2 in.	Joint type: All the joints including the longitudinal joint and the 9 in. closure pour between every two adjacent panels and the shear stud pockets were grouted with high alumina cement

	Panel length: Varies up to 40 ft. No post-tensioning was provided to clamp the joints.	concrete. Observations: The deck was found with cracking, leaking, leaching and rusting underneath. Transverse cracks spaced 2ft 6 in. apart were observed on the deck along with radial cracks and some shear stud pockets popping out on top. The bridge was inspected during 1993-95 (i.e., about 15 years after construction). (Issa *et al.* 1995b)

Amsterdam Interchange Bridge, Montgomery County, New York. Year built is 1954. Deck was replaced in 1973.		
The bridge has four spans and two lanes. Deck width is 45 ft. In 1973, the deck was replaced with cast-in-place concrete slab and precast concrete panels. Only 7 precast concrete panels were installed on half of span 2. Three panels were connected to the supporting structure using bolted connections	Panel thickness: 8 in. Panel width: 22 ft. Panel length: 4 ft. Longitudinal post-tensioning was not provided to secure the joint tightness. A waterproofed membrane and asphalt concrete overlay was used to protect the bridge deck.	Joint type: Female-to-female transverse joints were filled with low modulus epoxy mortar to form the shear keys. Shear key grout was formed by mixing one part resin and two parts aggregate. Observations: Joint leakage resulting in other deterioration such as spalling, cracking, and corrosion was documented. All spans showed 0.5 in. to 1 in. reflective transverse cracks at both sides of the joints. Width of the reflective longitudinal cracks that were visible in the middle of the bridge deck ranged from 0.125 in. to 0.25 in. The bridge was inspected

while the welded connection was used for the other four.		during 1993-95 (i.e., about 20 years after construction). (Biswas 1986; Issa *et al.* 1995b)

Kingston Bridge on Wurts Street, Over Rondout Creek, Ulster County, New York. Year built is 1921; Deck replaced in 1974 on existing beams.		
Seven hundred feet long three span, two lane suspension bridge deck was replaced with full-depth, full-width precast prestressed panels covering the 24 ft wide bridge.	Panel thickness: Varies from 6 in. to 7 in. from edge to crown. Panel width: 24 ft Panel length: 9 ft Precast panels were transversely prestressed to accommodate transport and handling stresses. Initial prestressing force applied to 0.5 in strands was 28.9 kips.	Joint type: V-shape male-to-female joint, with no grouting or caulking. Panels were tied together in the longitudinal direction using bolted tie rods. Observations: Transverse cracks were present over the transverse joints. Random leaking was found underneath the deck causing the concrete to spall near the leaky joints. The bridge was inspected during 1993-95 (i.e., about 19 years after construction). (Baughn 2012; Issa *et al.* 1995b)

Dalton Highway Bridges, 18 Bridges over Dalton Highway, Alaska. Deck replacement was during 1991-1992.		
Timber decks were replaced with full-depth, full-width precast, prestressed concrete deck panels on new steel stringers during 1991-1992.	Panel thickness: 9.5 in. at midspan and 7.5 in. towards edges. Panel width: 27 ft 5.4 in. Panel length: 4 ft 10 in and 5 ft 7 in. Deck panels were prestressed using 0.5 in. diameter, seven wire, low relaxation strands	Joint type: All bridges had grouted female-to-female transverse joints. Observations: The joint over the supports experienced cracking and loss of material. Most of the joints debonded. Typically all the transverse joints had cracking that were

	spaced at 1 ft 3 in. The effective stress of 149 ksi, after losses, was expected. In all the 18 bridges, there was no post-tensioning to compress the panel joints. There was no overlay on the deck. Traffic was allowed on the bridge with a speed of 3 miles/hour during staged construction. However, traffic was not allowed during grouting.	visible from top of the deck. In general, there were no leaks but specific bridges such as the Minnie creek bridge showed leaking at the joints. The bridge was inspected during 1993-95 (i.e., about 2 year after construction). (Issa *et al.* 1995a)

NB-216 Quakertown Interchange Bridge, Interchange exit for Pennsylvania Turnpike, Bucks County, Pennsylvania. Year built is 1940. Deck was replaced in 1981.		
The suspended cantilever bridge has a composite deck on the suspended span while a non-composite deck is on the cantilever span. Existing deck was removed in 1981 and full-depth deck panels were placed. Existing shear connectors were left undamaged when	Panel thickness: 6.5 in. Panel width: 17 ft 6 in. Panel length: 7 ft 7.5 in. Each panel covers one-half of the width. Longitudinal post-tensioning was provided to secure the transverse joint tightness. However, the force magnitudes and the post-tension spacing are not given in literature.	Joint type: Transverse joints were sealed with latex concrete grout after post-tensioning in the longitudinal direction. Observations: Full-width cracking was observed directly above each joint and reflected through the latex modified concrete overlay. Heavy spalling, water stains, and delaminations were documented. The bridge was inspected during 1993-95 (i.e., about 12 years after construction).

the old deck was removed. The new panels were cast with tight tolerances to accommodate the existing shear connectors.		(Biswas 1986; Issa *et al.* 1995b; COP 2007)

NB-750 Clark Summit Bridge, Lackawanna County, Pennsylvania. Deck was replaced in 1980.		
The two-lane, ten-span, 1627 ft long bridge has two parallel structures. In 1980, the deteriorated deck was replaced on existing superstructure system using full-depth, full-width precast panels.	Panel thickness: 6.75 in. Panel width: 29 ft Panel length: 7 ft Elastomeric strips and epoxy mortar grout was used to form the haunch over the existing girders. Issa et al. (1995) states that the nominal longitudinal post-tensioning was used along with non-shrink grout at transverse joints. Yet, the transverse post-tension design details such as force magnitude and spacing are not provided.	Joint type: Non-shrink cement grout with nominal post-tension was used to form the transverse deck panel joints. Observations: Cracking present in overlay at every panel joint. Underneath the deck, cracking and spalling at every joint was observed. It is believed that the majority of the durability problems arose due to inadequate connection between deck panels and stringers that resulted in significant vibrations under heavy traffic. The bridge was inspected during 1993-95 (i.e., about 13 years after construction). (Biswas 1986; Issa *et al.* 1995b)

Route 229 Bridge over Big Indian Run, Culpeper, Virginia. Year built is 1941. Deck was replaced in 1985.		
In 1985, precast concrete deck panels were installed on the existing steel girders. Ten-inch wide layer of Class II waterproofing membrane covers the joints.	Deck panel design details are not given in literature. There was no longitudinal post-tensioning present to clamp the joints.	Joint type: Female-to-female type grouted shear key joints were used between panels. Panels are in contact within the bottom portion of the joint even though the recommended practice is to leave at least a 0.25 in gap. Observations: No leaking present through the joints, but the overlay was not in a good condition. Observing the bridge deck from the bottom side, uniform transverse cracks were identified approximately 1ft apart near the center portion of the bridge deck. Moreover, some percolation was identified at the ends of the bridge deck. The bridge was inspected during 1993-95 (i.e., about 8 years after construction). (Issa et al. 1995b)

Bridges Constructed Using SPMT or the Slide-In Technique

Bridge Description	Design Details	Observations
I-80; Mountain Dell to Lambs Canyon, Utah, Built 2008		
The superstructure was prefabricated and moved using the SPMT.	The bridge is single span. The superstructure consists of steel girders and a cast-in-place deck.	Observations: Cracking in the span is essentially nonexistent. Cracking is seen in the overlay at the abutment joints. Leakage is seen at the backwall joint. The approach slab has failed and has been repaired. The repairs are also beginning to fail. Inspected two years after bridge construction. (Culmo 2010)

I-215; 4500 South Structure, Utah, Built 2007		
The superstructure was prefabricated and moved into place using the SPMT.	The bridge is single span. The superstructure consists of steel girders and a cast-in-place deck.	Observations: 45 degree cracks are seen starting at each corner of the bridge and continue to the centerline of the bridge. The casting of the bridge deck was not coincided with the pick points. This caused cracking of the deck and parapets. The cracks in the deck are leaking and the parapet cracks show no signs of leakage. The uphill joint is wide open and allowed significant moisture ingress. The bridge was inspected three years after construction. (Culmo 2010)

I-80; State Street to 1300 East, Utah, Built 2008		
The superstructure was precast and moved into place using the SPMT.	The bridge is single span. The superstructure consists of steel girders and a cast-in-place deck.	Observations: 45 degree cracks are seen starting at each corner of the bridge and continue to the centerline of the bridge. The casting of the bridge deck was not coincided with the pick points. The deck cracks are leaking. The cracking in the adjacent span indicates that the leakage is due to shrinkage cracks and not due to the stresses from the SPMT move. The bridge was inspected two years after construction. (Culmo 2010)

I-80; State Street to 1300 East, Utah, Built 2008		
The superstructure was precast and moved into place using the SPMT method.	The bridge's superstructure consists of steel girders and cast-in-place deck. The bridge is single span.	Observations: 45 degree cracks are seen starting at each corner of the bridge and continue to the centerline of the bridge. The casting of the bridge deck was not coincided with the pick points. The deck cracks are leaking. The cracking in the adjacent span indicates that the leakage is due to shrinkage cracks and not due to the stresses from the SPMT move. The bridge was inspected two years after construction. (Culmo 2010)

3300 South over I-215 East, Utah, Built 2008		
The ABC technique used was a precast superstructure with a SPMT move.	The single span bridge superstructure consists of steel girders and a cast-in-place deck.	Observations: Minor 45 degree cracking has occurred at the exterior and interior corner bays of the bridge. Minor leakage has occurred at these cracks but do not seem to be worsening. No overlay was applied; the cast –in-place concrete deck is bare. The pick points did not coincide with the casting support, therefore creaking occurred in the deck and the parapets. Due to the pick points being closer to the ends and the use of light weight concrete in the deck, the cracks were minimal in comparison to other SPMT moved bridges. The cracks that formed are leaking only due to there being no waterproofing system in place. The bridge was inspected two years after construction. (Culmo 2010)

I-80 over Lambs Canyon Road, Utah, Built 2008		
The ABC technique used was a SPMT move.	The single span bridge consists of AASHTO I-Girders and a cast-in-place deck.	Observations: The pick points and the casting supports were coincided. This resulted in nearly nonexistent cracks. The approach slab joints at the abutments are failing and will need repairs. The bridge was inspected two years after construction. (Culmo 2010)

I-15; Widening, 500 North to I-215, Utah, Built 2009		
The ABC techniques used was a precast superstructure and an SPMT move.	The superstructure of the bridge was a bulb tee with a cast-in-place deck. The bridge consists of two spans.	Observations: The pick points were close to the ends of the bridge and resulted in minimal cracking in the deck and parapets. Efflorescence is seen at these cracks. (Culmo 2010)

I-80; Two Bridges near Echo Junction, Utah, Built 2009		
The ABC techniques used was a prefabricated superstructure and a lateral slide-in.	The single span bridge superstructure consists of AASHTO I girder with a cast-in-place deck.	Observations: Minimal cracking seen on the underside of the deck. The approach slabs were placed before the slide-in and minimal cracking is seen in those as well. The bridge is in excellent condition still. The bridge was inspected a year after construction of the bridge. (Culmo 2010)

APPENDIX D

ABC CHALLENGES AND LESSONS LEARNED

Twenty-three bridge projects that utilized ABC concepts were reviewed and summarized. The table given below contains project background, successes, challenges, and lessons learned.

Table D–1. Summary of Accelerated Bridge Construction Case Studies

CASE STUDY	BACKGROUND	SUCCESSES	CHALLENGES	LESSONS-LEARNED
Oakland Eastbound I-580 Connector (Chung et al. 2008)	– Due to the explosion of fuel tanker truck traveling from I-80 to I-880, I-580 collapsed onto the I-880 connector ramp, in the San Francisco Bay Area, CA. – Due to the material procurement difficulties (using steel beams on bent caps), the contractor chose precast, post-tensioned concrete bent caps.	– The designers worked with fabrication experts during the design process to deliver the project fast and safely. – Design team proceeded with built-up sections since the prefabricated rolled shapes were not available. – The flange plates of girders were kept to one size to simplify the fabrication.	– Potential shortage of steel plate stock was a challenge since accelerated reconstruction depended on the availability of materials. – Restricting proprietary rapid strength concrete was a challenge. The addition of shrinkage-reducing admixture made the other methods of concrete construction difficult due to longer stting times.	– Availability of materials will dictate the construction method. – An emergency response plan for decision making authority, communication protocols, and reporting relationship was needed. It needs to address specification limitations for emergency projects to allow for flexibility in the selection of materials.
Russian River Bridge (Chung et al. 2008)	– The bridge is located over the Russian River in Geyserville,CA. – Due to the tight construction schedule and environmental issues, accelerated bridge construction was preferred. – Non-standard double T precast and prestressed concrete girders, cast-in-steel-shell piles, and 6 in. cast-in-place concrete	– Wider precast sections reduced the number of precast girders, resulting in the elimination of deck falsework, and reduction in time and cost of fabrication, delivery, and erection. – Multi-stage transverse post-tensioning of precast sections was used due to the compact span-to-depth ratio demands.	– Calculations of elongations during jacking operation due to staged post-tensioning. – Since a two stage post-tensioning process was used, designing the structures under different stages was challenging. – The shortage of steel shells for piles led to the decision to use State furnished materials in the contract.	– Using wider precast sections was critical in accelerating the construction phase by reducing the number of precast girders, – Multi-stage post-tensioning of precast sections is an alternative solution for bridges with compact span-to-depth ratio demands – Effective communications and partnering are essential elements for frequent exchange of ideas between designers, constructors, and fabricators.

	deck were used.			
San Francisco Yerba Buena Island (YBI) Viaduct (Chung et al. 2008)	– The viaduct carries Route 80 traffic across YBI and links the East Span of the San Francisco-Oakland Bay Bridge with the YBI tunnel. – A Demo-Out-Move-In construction method was used in the replacement of the VBI viaduct. – The new bridge structure was moved with “skid shoes” that ran on oiled steel tracks pushed with hydraulic rams.	– The Demo-Out-Move-In operation was successfully implemented on this project. – The bridge was closed for three days, resulting in an accelerated replacement of the YBI viaduct. – The construction of the bridge structure took place away from live traffic, reducing the risk to the traveling public and minimizing traffic disruptions.	– The moving operation was very expensive. – A staging area adjacent to the existing structure was required. – The designer had to work with a heavy lift contractor during design to facilitate the moving operation. – Falsework had to be cleared out before moving the equipment, requiring the installation of temporary columns.	– Fitting the new span in between the existing structure requires tight tolerances. – A lift test prior to the scheduled move is needed to avoid operational delays. – Support points on the superstructure must line up with the tops of the columns. – The elevations at support points on the superstructure must match the elevations at the respective tops of columns. – The moving operation details must be thoroughly reviewed by the designer. – It is helpful to involve a heavy lift contractor during design to facilitate the construction process.
I-70 Over Eagle Canyon (Reasch et al. 2010)	– The Eagle Canyon Arch Bridge is located on I-70 in southern Utah. – UDOT decided to replace the deck on the arch and approach spans using full-depth precast deck panels that were post-tensioned as batches of 5 panels at a time. – A 600-ton crane with a	– Lightweight concrete was used for the new full-depth deck panels. – To prevent the instability of the arches, the deck panels were removed section by section and replaced with groups of five new full-depth precast concrete panels.	– Due to the load capacity of the existing structure, no heavy equipment was allowed on the bridge, creating a construction challenge. – The remote location made the prospect of trucking-in and operating a concrete batch-plant expensive and impractical.	– Replacement of deck panels can be achieved section-by-section when concerns exist regarding instability of the bridge. – To reduce crane loads and dead loads, lightweight concrete may be used in full-depth deck panels. – Existing structure load capacity is an important factor in selecting the construction method.

	324-foot boom was placed to remove the existing sections of the deck and erect the new panels.			– Complex projects may benefit from the collaboration between owner, designer, and contractor during the design process. – The full-depth precast deck panels that are connected through post-tensioning are a viable ABC technique.
I-215; 4500 South Bridge (Mcminimee et al. 2008)	– The bridge carries 4500 South road over I-215 in Salt Lake City, Utah. – The existing bridge was removed and the new superstructure was moved into its final location using a self-propelled modular transporter system (SPMTs).	– UDOT reported savings over $4 millions in user delay costs by using ABC techniques. – The SPMT was successfully used to move the existing structure. – The removing and replacing operation lasted 58 hours.	– Checking elevations multiple times was required due to the complex geometry of the structure. – Excavating and constructing new abutments beneath the existing structure were challenging as they needed to be quick and cost-effective.	– Communication and coordination between the designer, contractor, and mover are important. – To ensure design requirements are met, it is essential to develop protocols for inspection procedures and site visitations. – Having a contingency plan in place for unforeseen circumstances is necessary. – The staging area for SPMT equipment must be planned properly. – Pre-event meetings help in examining the steps of construction.
Five Bridges on OR 38 (Ardani et al. 2010)	– ODOT decided to replace five bridges on the Oregon Highway 38 between the towns of Drain and Elkton, Oregon. – In order to minimize traffic disruption and maintain traffic flow, the	– Using ABC, the bridge was completed ahead of schedule. – The design-build delivery method further reduced construction time. – HSS made the	– Due to difficult site conditions, replacing of the 3rd and 4th crossings was a challenge. – Due to short-term week-end closures, demolishing the old structure and sliding the new bridge onto the	– Using the design-build delivery approach can add further time reduction for bridge construction. – HSS is a viable accelerated bridge replacement technique that minimize traffic disruption, and improves safety in work zones.

	hydraulic sliding system (HSS) was used.	replacement of crossings 3 and 4 possible over two weekend closures, minimizing traffic disruption and improving safety.	same alignment was a challenge, requiring careful planning of all operations.	– Careful planning of construction operations is an essential pre-requisite to a successful completion of an accelerated bridge replacement.
U.S. 15/29 over Broad Run (Gilley et al. 2009)	– Due to the deteriorated superstructure of southbound U.S. 15/29 bridge over Broad Run in Prince William County, VDOT decided to replace the concrete T-beam superstructure. – Since construction required several stages and many lane closures, prefabricated segments of steel beams made composite with high-performance lightweight concrete deck were used.	– Traffic flow was maintained during the removal and replacement of the bridge. – Abutments were modified by extending wing walls and reconstructing seats. – The obsolete concrete T-beam superstructure was replaced with precast elements using ABC methods over three weekends. – Piers were modified using a corbel secured to the pier via grouted dowels and external post-tensioning.	– The phasing scheme was revised to detour traffic around the bridge over three weekends due to the potential inability to reopen the highway to Monday morning rush-hour traffic. – Due to the high traffic volume on the route, a detailed construction scheme had to be developed. – Since the bridge is adjacent to the historic properties, temporary structure could not be used and the bridge was widened to the median side.	– The designer and contractor worked together to find a new scheme to reduce the closure duration. – Using prefabricated, high performance and lightweight concrete deck that was integrated with steel beams resulted in better performing bridge and in an accelerated construction operation.
The State Highway 86 over Mitchell Gulch	– CDOT decided to replace the severely deterioration timber bridge at State Highway 86 over Mitchell Gulch in Douglas, Colorado. – The new bridge	– The new bridge was successfully completed in 46 hours. – The use of ABC resulted in minimizing traffic disruption and improving	– Vertical alignment between prefabricated components created a problem when precast units were post-tensioned. – The deck unit grouting process resulted in	– Using prefabricated elements minimizes the construction time, and traffic impacts, and improves work zone safety. – Preparing a back-up plan for unforeseen site conditions during construction is useful to ensure on

(Merwin 2003)	substructure was precast concrete elements except for the steel H-pile supports.	work zone safety. – The deck girders were constructed with integrated bridge railing to eliminate the railing installation operation.	unsatisfactory joints. CDOT corrected the problem by devising field modifications.	time project delivery. – Monitoring of casting precast units is required to improve the post-tensioning operations by minimizing tolerance issues.
I-80: State Street to 1300 East (Reasch et al. 2010)	– The seven bridges are located on I-80; 1300 East to State Street in Salt Lake City, Utah were replaced. – The superstructures were transported from the bridge farm to the bridge site using SPMT.	– The project was completed in two years using SPMT, saving one year over conventional construction methods. – A “bridge farm” was used to construct seven bridge superstructures off-site which were then moved to their location.	– Moving the first bridge was cancelled due to the failure of the carrying beam, requiring a revision to the moving operation. – Transporting the next bridge from the bridge farm over a newly constructed bridge was a challenge.	– SPMT technology offers a promising ABC method. – Collaboration between design and construction teams is a key element to mitigating risks and identifying/revising the methods of construction.
Mill Street Bridge (Stamnas and Whittemore 2007)	– The bridge is located on Mill Street in Epping, New Hampshire. – The new bridge comprised of seven precast, prestressed concrete box beams, five precast abutment components, seven wingwalls, and ten precast footing pieces.	– The bridge was constructed in eight days and cost approximately one million dollar.	– The installation of splice sleeves between footings and wing walls was a challenging task. – Careful attention was required for matching the splice sleeves – At times, the curing of the proprietary grout took 12 to 16 hours to reach minimum required strength.	– Standardizing the size of the precast components can improve the efficiency of installation in accelerated construction. – Special attention must be paid during grouting operations to ensure splice sleeves are smoothly connected.
Tucker Bridge (Higgins	– Tucker bridge is located on U.S. 6 at Mile Post 204 in Spanish Fork Canyon, Utah.	– To simplify the fabrication and minimize the number of precast elements, the deck edges were designed to be	– Fabrication tolerances between precast elements had to be considered in design.	– Minimizing the number of unique precast panels saves time and money during fabrication and installation.

2010)	– Girders, abutments, back walls, wing walls, full-depth deck panels, approach slabs, and sleeper slabs were all precast concrete elements.	straight and parallel. – The first abutment was placed in less than a day and the second one took less than 4 hours.		
Lewis and Clark Bridge (Weigel 2011)	– The bridge carries SR 433 over Columbia River between Washington and Oregon States. – An SPMT system was used to install the full-depth lightweight concrete deck panels.	– The accelerated bridge construction system saved over 38 percent of the cost estimated by the engineer. – Incentive and disincentive provisions resulted in early completion of the project.	– Long term detour options were not recommended. – Only allowing off peak hour closures resulted in increased construction time.	– The SPMT system can be used in both removing old deck sections and installing new ones. – Incentive and disincentive provisions can encourage the contractor to expedite the construction process.
Sam White Bridge (Jaynes and Dobmeier 2011)	– The bridge carries Sam White Lane over I-15 in American Fork, Utah. – The 354 feet long and 80 feet wide bridge is the second longest two-span bridge moved in the world with a 3.82 million pounds superstructure.	– The bridge was moved into place in one evening using SPMT. – Interlocking sole and masonry plates were used for the girder-column connections to transfer the seismic loads into the columns. – Relative elevations matched each other and no tolerance issues occurred.	– Grade modifications were required during the move due to the combined cross slope of the length and width of the bridge. – Incorrect flange plates were ordered. An error occurred during the fabrication of cross frames.	– SPMTs is a viable ABC method. – Coordination is needed during the design and move of the bridge. – Due to the high vertical curve on the bridge, SPMT strokes were needed to lift the superstructure. – Due to the combined cross slope of the length and width of a bridge, grade modifications will be required.
Parkview Bridge	– The bridge carries Parkview Avenue over US-131 in Kalamazoo, Michigan.	– Even though some rework took place during construction, the accelerated bridge construction method	– The contractor's hand mixed grout created a challenge for the grouting haunches.	– Properly sizing substructure elements allows efficient installation. – Grout connection details need to be reviewed.

	− It is fully prefabricated bridge using precast piers, abutments, I girders, and full-depth deck panels. − Two precast plants were used on this project, one for the deck and the other for girder and substructure elements.	shortened on-site construction schedule by two months. − Despite a high initial cost, user cost savings more than compensated for this initial added cost.	− Due to potential asphalt cracking along the backwall stem, the construction detail of backwall stem was revised. − The alignment of bars into the ducts in the pier was challenging. − Longitudinal post-tensioning duct misalignment occurred after placing the panels on the girders. − Tolerance issues between the shear connector pockets in the panels and the flared coil inserts on the top flange of the girders.	− The fabrication at the job site or at a nearby location need to be considered to minimize tranportation cost and the impact of load restrictions. − The impact of missing shear connectors needs to be evaluated due to the difficulty of drilling girder flanges when there is a misalignment. − Simple and durable connection details at the abutments are encouraged. − The special provisions need to specify grouting material and procedures to improve workmanship.
120th Street Bridge (Bowers et al. 2007)	− The bridge carries 120th street over Squaw Creek in Boone County, Iowa. − The new bridge was of four girders, three span continuous with full-depth precast deck panels. − The precast deck panels covered the half width of the bridge and were transversely prestressed. − Each deck panel had two	− A high early strength concrete mix was used for filling the substructure blockouts. − The contractor had no problem meeting the end of pile driving tolerance or fitting the precast abutment cap over the H-piling. − The deck panels were provided with leveling device that was designed	− Placement of a precast pier cap or abutment was successful because piles were driven within tolerances. − Due to lack of experience of the contractor, material suppliers, and engineers, the project was delayed 30 working days. − The alignment issue occured during the erection of the first deck panel.	− Using a template for tolerances reduces the time for placing a precast pier cap or an abutment. − The deck panels can be erected in less time − − The experienced contractor is a must to estimate the panel erection duration. − Panel details must be reviewed to prevent alignment issues during the erection of the panels.

	full-depth channels through which the entire bridge deck was longitudinally post-tensioned. Cast-in-place concrete was used to fill the channels.	by the contractor and approved by Iowa DOT.	– The total length of the deck panel portion of the bridge was 9 in. longer than anticipated in the plans. – To modify the beams, additional prestressing strands were added and the concrete release and 28-day stengths were increased.	
Route 99/120 Separation Bridge (Chung et al. 2008)	– The project was located in the City of Manteca, Sacramento, California. – The purpose of the project was to widen the Route 120 from existing 5 lanes to 8 lanes. – The new bridge was comprised of two 105 ft long spans to replace the existing shorter 2-span cast-in-place concrete box bridge structure.	– 50% scale model was designed and tested at University of California for the seismic design. – The precast prestressed box girder with longitudional post-tensioning alternative resulted in less congestion and cost.	– The negative moment occurred over the bent cap during the cast in-place deck pour. – Short of utilizing commercial software created a challenge. – The positive moment associated with seismic loads exceeded the superstructure's dead load moment.	– In order to solve negative problem issue over the bent cap, pre-tensioning strands must be extended into the the bent cap and bar reinforcing must be added. – The positive moment associated with seismic loads must be considered. – In order to provide flexural reinforcement through the bent cap, the bars and extended strands should be bent at a 90° angle in the bent cap as well as staggering the reinforcement in opposing girders.
Skyline Drive Bridge over West Dodge Road (Fallaha et al. 2004)	– The bridge carries Skyline Drive over West Dodge Road in Omaha, Nebraska. – The new superstructure consisted of Steel plate girders with spacing of 10.83 ft and full-depth precast prestressed panel	– The Skyline Bridge was the first implementation of the NUDECK system. – The NUDECk system resulted in reduced construction time. – The use of precast concrete deck panels	– The development of 2% roadway crown in the constant thickness pre-tensioned panel was a challenge during the prefabrication process. – Since increasing the crown thickness towards the center of bridge resulted in	– Non-proprietary materials used in deck panels resulted in easy procurement. – Use of fewer larger diameter studs resulted in cost savings and improved safety in the fabrication of the steel beams. – The channel provision for post-tensioning strands helps in effective

	of NUDECK system.	eliminated formwork for the overhangs.	high dead load, a solid plastic polyvinyl chloride (PVC) rod was placed along the centerline of the crown during the prefabrication process.	grouting around the post-tensioned strands. – The deck is transversely prestressed and longitudinally post-tensioned making it compressed from both direction thus improving durability of bridge
I-215 East Bridge over 3760 South (Miller 2003)	– The bridge carries I-215 East over 3760 South in Salt Lake City, Utah. – The complete removal of the existing bridge was required.	– The project was projected to take 90 working days but the use of prefabrication saved 30 days. – The superstructure was constructed away from the existing bridge and brought to site on a special truck then lifted into position by cranes.	– The pick point lifting devices caused interference with concrete finishing.	– For better constructability the design details must be reviewed thoroughly. – Methods of connecting approach slab to deck panels without closure pour should be investigated in detail. – During the design process the pick points should be accounted and reinforcement provided accordingly. Also the parapet should be considered so as to cast-in-place or slip formed once in place. – Predict the weight of the lifting bridge accurately and determine if counterweights are required at the lift crane. – Steel beams were considered more flexible thus an optimal option for the girders.
MD Route 24 Bridge over Deer Creek (FHWA 2006)	– The deteriorated deck was decided to be replaced on MD Route 24 Bridge over Deer Creek in Harford County in northeastern Maryland.	– The FRP panels offer very light weight, superior corrosion resistance to an anticipated life of 70 years.	– The repairs of the deteriorated steel members increased the construction time.	– FRP panels use in this bridge helped the Maryland State Highway Administration maintain heritage of the region. – Availability of suppliers of FRP deck

	– The accelerated construction was implemented as the bridge was on a school bus route and could be closed for a maximum of 10 weeks in summer. – The bridge was constructed by use of Fiber–Reinforced Polymer (FRP) deck panels.	– Steel angles were welded to the girders and the panels were installed in 3 days using a forklift.		panels need to be considered before deciding the bridge type to be constructed.
I-40 Bridges (Chung et al. 2008)	– Maintenance inspection revealed that twelve bridges on the heavily traveled I-40 corridor in southeastern California were required to be replaced due to severe deck deterioration as well as shear cracking in the bent caps and several girders. – The simple span girders and abutment seats were precast, while the deck, abutment backwalls, wingwalls, and footings were cast-in-place.	– The existing two-span 106 ft long bridge was replaced with a single-span structure designed to reduce substructure construction efforts. – Segmenting precast abutment design facilitated staged construction allowing traffic to be maintained on one completed side while the existing structure is demolished.	– Since the weight of precast abutments were 82-tons, transport permits were required. – Finding a large crane for the abutments and working radius was a challenge.	– The rock slope protection installation below the structure to protect the abutment footings from scour constrained immediate placement of the girders after erection of the abutment. – The limitation of duration of detour at the site location provided in the specifications forced the contractor to shift work forces from other operations as necessary to expedite opening of westbound roadbed. – A 360-ton crane would have been used considering the load and crane position relative to the pick and placement of the abutment. – The site preparation to accept precast abutments can also be accomplished by utilizing leveling screw attachments on the precast abutment

				element with post grouting beneath the abutment footing to ensure uniform load distribution to the soil. – Heavier precast abutments required special transportation permits and larger crane for lifting the same. The working radius for larger crane was a bigger challenge of the project; thus requiring skilled workers. – An abutment cast in segments would eliminate most of the challenges faced during the substructure installation
Western Washington State (Khaleghi 2010)	– The bridge is located in high seismic zone of western Washington. – The research examined the details for a three-span prestressed precast concrete bridge bent substructure system.	– The contractor proposed precasting intermediate piers and bent cap in place of cast-in-place construction.	– Due to pick and shipping weight restrictions, the precast first stage cap was built as two-piece element in lieu of single piece element resulting in longer time required for splicing the segment.	– The pier-to-cap connection was accomplished easily by the use of large duct sizes in the bent cap and large diameter relatively less number of reinforcing bars from the pier. – The pier-to-cap connection was tested under cycle loading in three variations and it behaved identical to cast-in-place bent cap on pier while maintaining safety, rapid construction and long-term performance. – The use of precast bent caps eliminated the need for false work resulting in cost savings.

Fast 14 Projects	– Considering the existing condition of steel girders and failure of few bridge decks, MassDOT decided to replace 14 deteriorated bridge superstructures on freeway I-93 in the city of Medford, MA, within a period of ten weeks, from June 3 to August 14, 2011. – A 3-span bridge superstructure was replaced with modular units. A module consisted of a precast 8 in. concrete deck and two steel plate girders making them similar to double-tee units. – The bridge consists of 18 precast units (6 units per span). Each span was constructed as simple spans and later making only the deck continuous through link slabs using cast-in-place concrete over the piers.	– The bridge design and traffic management design were performed in parallel. – The coordination between the Design-Build team and the MassDOT during construction was very successful. – Weekly progress meetings were held for effective communication between the Design-Build contractor and agency staff. – The incentive/ disincentive clauses were established to complete the bridge replacement on time. – Seven bridges were replaced in 5 weekends. – All the bridges on I-93 highway were completed within the allocated 55-hour timeframe. – The design team, MassDOT, Massachusetts State Police, and the city of Medford Police and Fire departments worked together for successful delivery of the precast components.	– Since this was an emergency project, the concept for the design development had to be formulated in 2 weeks. – The 30% of plans needed to be developed in 2 months. – Facilitating long lane closure periods was a challenge since low traffic volumes on I-93 was limited. – The manageable hydraulic cranes were used since it was difficult to keep up the crane pick. – Putting together the RFP and moving forward to Design-Build Contract procurement in a shortened time frame was a challenge. FHWA worked with MassDOT to complete a RFP and the environmental process in 5-6 weeks. – Allocation of the resources was a challenge. The work kept going 24 hour a day. – Five hundred people were trained for the project.	– The construction staff needs to be involved in the design process from the beginning. – The contractor input needs to be used during design development. – The CM/GC needs to be used to get a contractor in design development as early as possible. – The owner needs to engage with contractors, consultants, and the public. – Project milestones need to be set up to allow reasonable time to the Design-Build team for design activities and to the DOT for design reviews. – Dedicated staff was an essential element for the Design-Build procurement, design review, and construction oversight. – The schedule needs to be more flexible prior to starting construction. This would allow the opportunity to innovate with alternatives and methods of construction and better material procurement. – Sufficient time needs to be allowed to design and plan the intended work in advance of construction. – As the bridges were being constructed,

			– The cross-section and construction sequence of the units needed to be revised. Since the existing approach spans were used to support the crane for the erection of the center span, the center span was built in one weekend and the approach spans were built following two weekends.	the contractor gained the comfort level each weekend and pushed the limit of faster construction. – Contractor needs to be aware of the demand on resources required to keep pace with the project. – The importance of substantial public outreach and coordination with state agencies are essential.
M-25 Bridge over the White River, MI	– The bridge is located in Huron County in Eastern Michigan. – Due to having poor rating of existing bridge deck and columns, the bridge was required to be completely replaced. – The new bridge consists of eight segmental box beams with integrated slab which emulate the precast prestressed concrete spread box beam system with cast-in-place concrete deck. – Except the footings, rest of the bridge is made of precast units.	– The slope walls were placed after the post tensioning of the superstructure segments to avoid any gaps that may have formed as a result of the tensioning. – The inspections took place after concrete placement and before storage of each element, after shipment to the site, and before and after erection of each element.	– Late submittal of shop drawings and fabrication of the precast elements threatened to push back the project completion date. – The supplier for the High Performance Superstructure Concrete provided a binary blend without MDOT approval due to a communication error. This resulted in a compressive strength at 28 days to exceed specifications. – Special equipment was needed to test the compressive strength of the grout. – Screed elevations on the deck varied from one end to the other.	– Consideration should be given to the use of precast elements on projects scheduled for winter construction. – Specifying ternary blended cementitious mix designs in remote areas should be avoided where suppliers are limited and unfamiliar with these mix designs. – In order to obtain the existence of large subsurface obstacles, additional soil borings may be required. – Benchmark location needs to be reviewed with respect to removal work. – The precast elements could be paid as Lump Sum items which would eliminate the need for mathematical calculations to determine final pay quantity. – Pre-approved grouts need to be

			– Grinding of deck surface resulted concerns on friction loss. – The precast superstructure element fabricator did not provide roughened edges on the elements where secondary casting was to be applied for deck closure joints. – Due to the difficulty to use the type HPSC concrete for the concrete end walls and the parapet railing, the contractor used a type D concrete in the end walls that produces a more aesthetic wall. – During construction there were some large rocks found in the underground of the river bed which slowed the process down minutely.	included with the special provision as much research was needed to identify an acceptable grout. – A dywidag system needs to be preferred for connecting the abutments to the footing to reduce the number of openings to be grouted and save considerable time. – Special consideration needs to be given to addressing how lift loop cables will be removed following installation of the precast elements and how the resulting depression will be patched. – The post-tensioning grout tubes needs to be located under the barrier railings for the barrier wall to cover the tubes. – The same concrete mix design needs to be used for both the aesthetic parapet tube bridge railings and the end walls as they are normally cast integrally. – The cure time for the superstructure elements at the fabrication facility should receive at least the minimal cure time prior to incorporation into the project. – The superstructure construction sequence needs to be carefully reviewed, especially with respect to installation and removal of pre-loading.

				– Expectations for compressive strength determinations of post-tensioning grout prior to removal of pre-loading needs to be detailed. – A portion of the joint between the precast slope wall and the underlying abutment wall was not covered by embankment. It may be aesthetically desirable to address this joint differently. – Fabricator needs to maintain the alignment of the reinforcement steel during fabrication. – To reduce cracking in the concrete end walls, a minimum of one expansion slice sleeve needs to be installed in the metal railings of each barrier wall, regardless of length. – Placing vertical curve crests or low points near concrete to HMA transition joints should be avoided to improve ride quality. – A thin epoxy polymer bridge deck overlay needs to be included as part of original contract on projects where pre-cast superstructure elements are used.

APPENDIX E

Mi-ABCD USER MANUAL

MiABCD USERS' MANUAL

Project: Improving Bridges with Prefabricated Precast Concrete Systems

Project Manager: Mr. David Juntunen, P.E.

Submitted to:

MDOT

Submitted by:

Dr. Haluk Aktan, P.E.
Professor
(269) – 276 – 3206
haluk.aktan@wmich.edu

Dr. Upul Attanayake, P.E.
Assistant Professor
(269) – 276 – 3217
upul.attanayake@wmich.edu

Mr. Abdul Wahed Mohammed, E.I.T.
Doctoral Research Assistant
(269) – 276 – 3204
abdulwahed.mohammed@wmich.edu

Western Michigan University
Department of Civil & Construction Engineering
College of Engineering and Applied Sciences
Kalamazoo, MI 49008
Fax: (269) – 276 – 3211

DISCLAIMER

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1 INTRODUCTION

1.1 PURPOSE OF USER MANUAL

The Michigan Accelerated Bridge Construction Decision-making (MiABCD) model was developed to support decision makers with a guided software that can evaluate the Accelerated Bridge Construction (ABC) vs. Conventional Construction (CC) alternatives for a particular project.

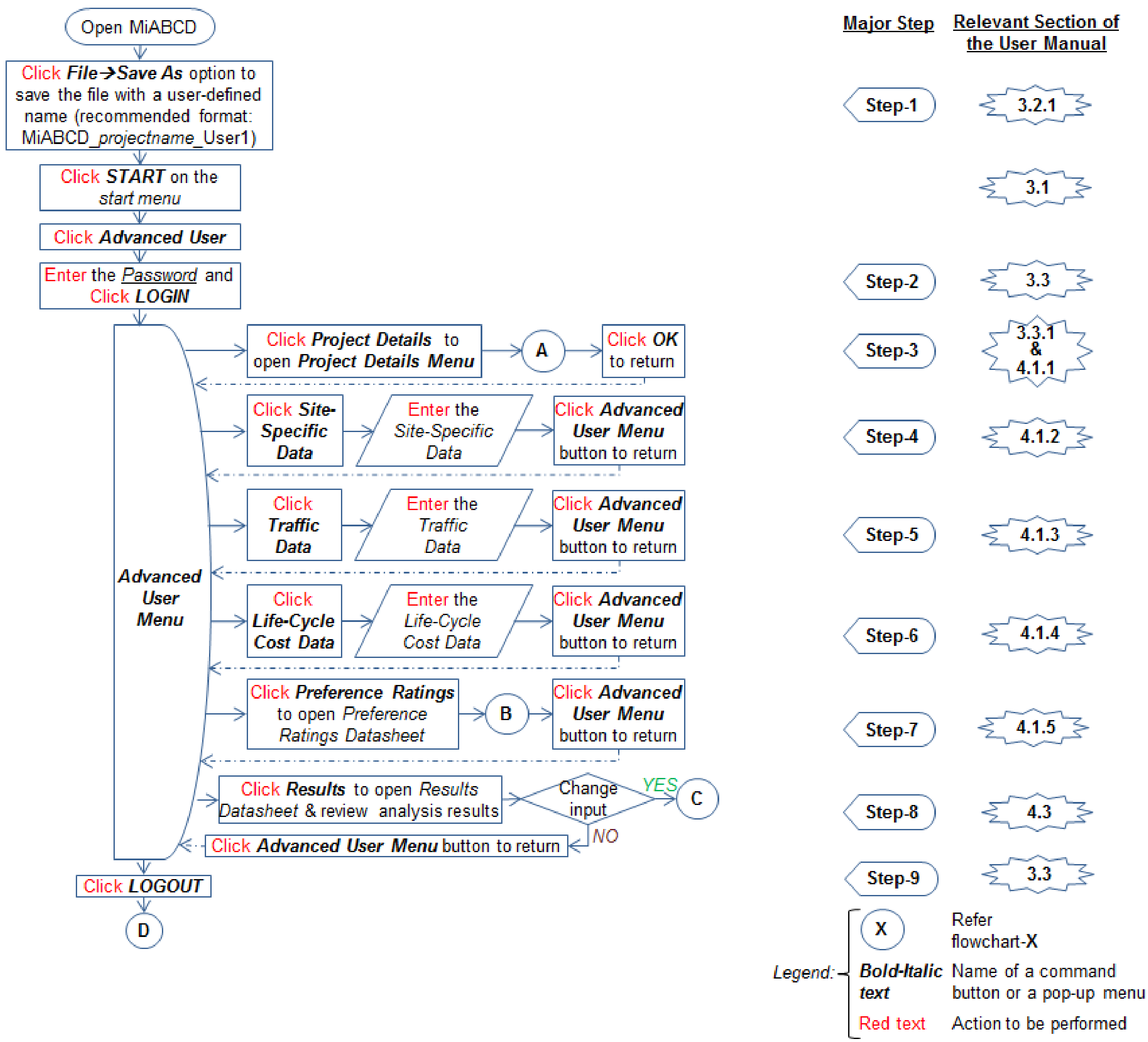
The user manual includes the following:

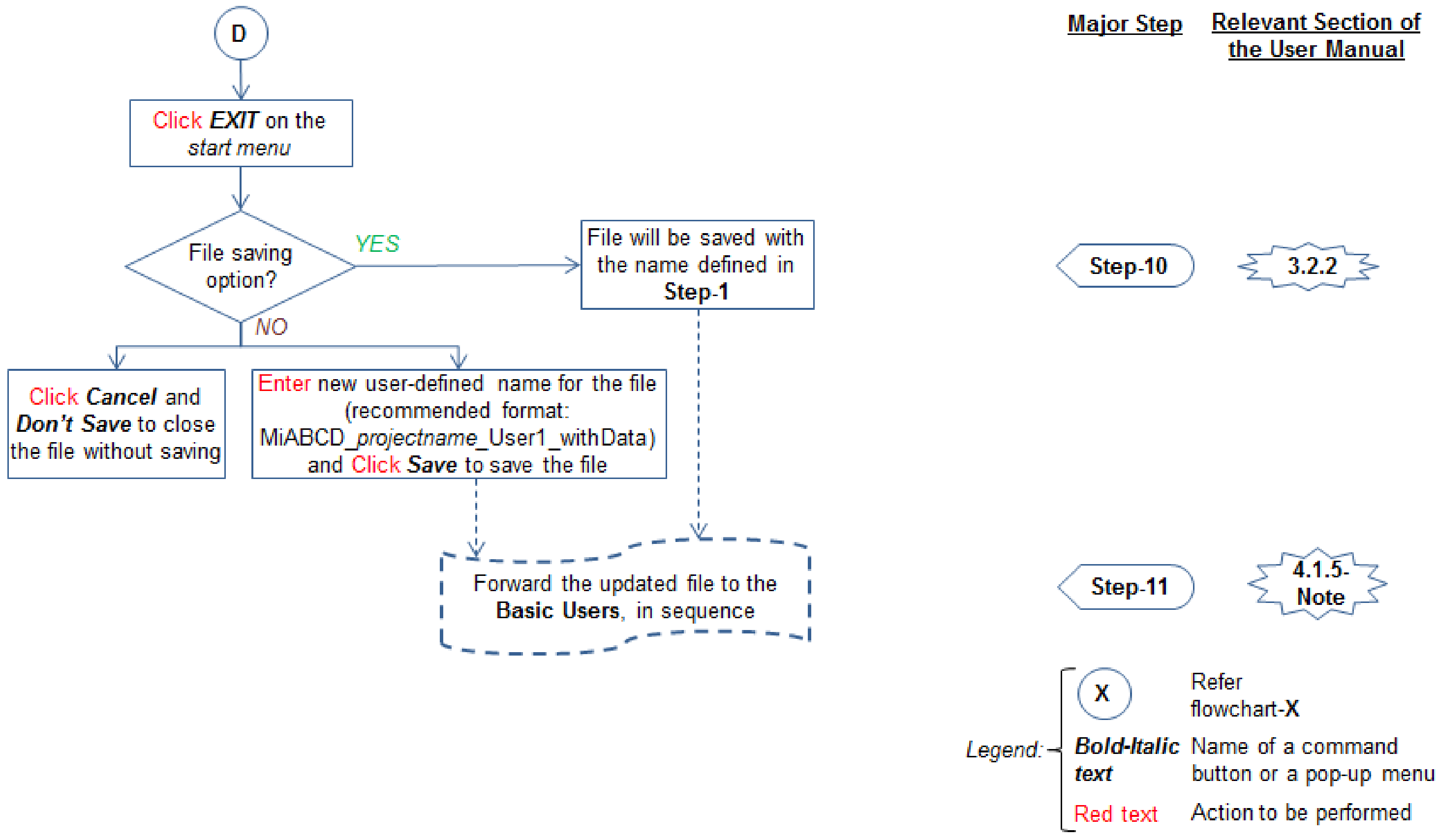
1) Software installation instructions,
2) Description of the *menu* functions, and
3) Instructions for using the MiABCD software.

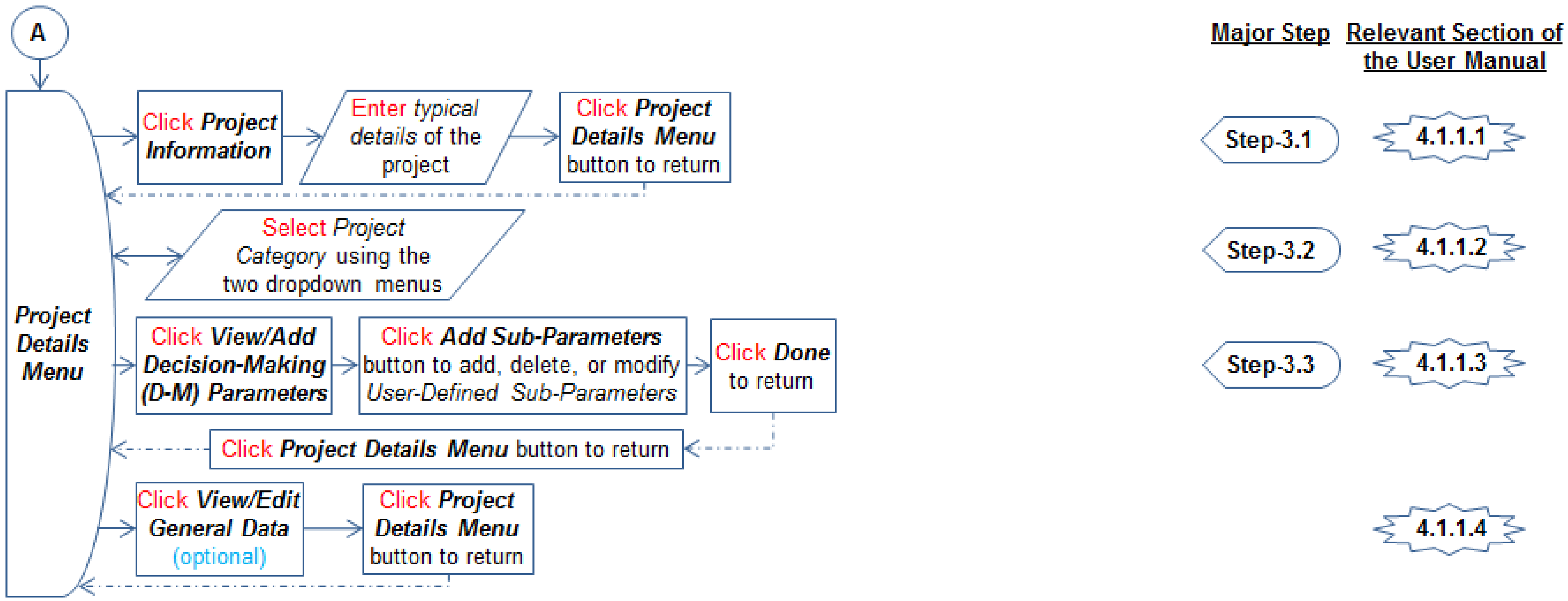
The MiABCD software allows data entry by two types of users: (1) the *Advanced User* and (2) the *Basic User*. Two flowcharts are presented in the following pages to depict the step-by-steps process that needs to be followed by the *Advanced User* and the *Basic User*. With each flowchart, the major steps in completing the decision-making process are defined. In order to execute the decision-making process, the *Advanced User* must complete all the steps defined in *"Advanced User Flowchart"* before any *Basic User* can use the program as described in the *"Basic User Flowchart."*

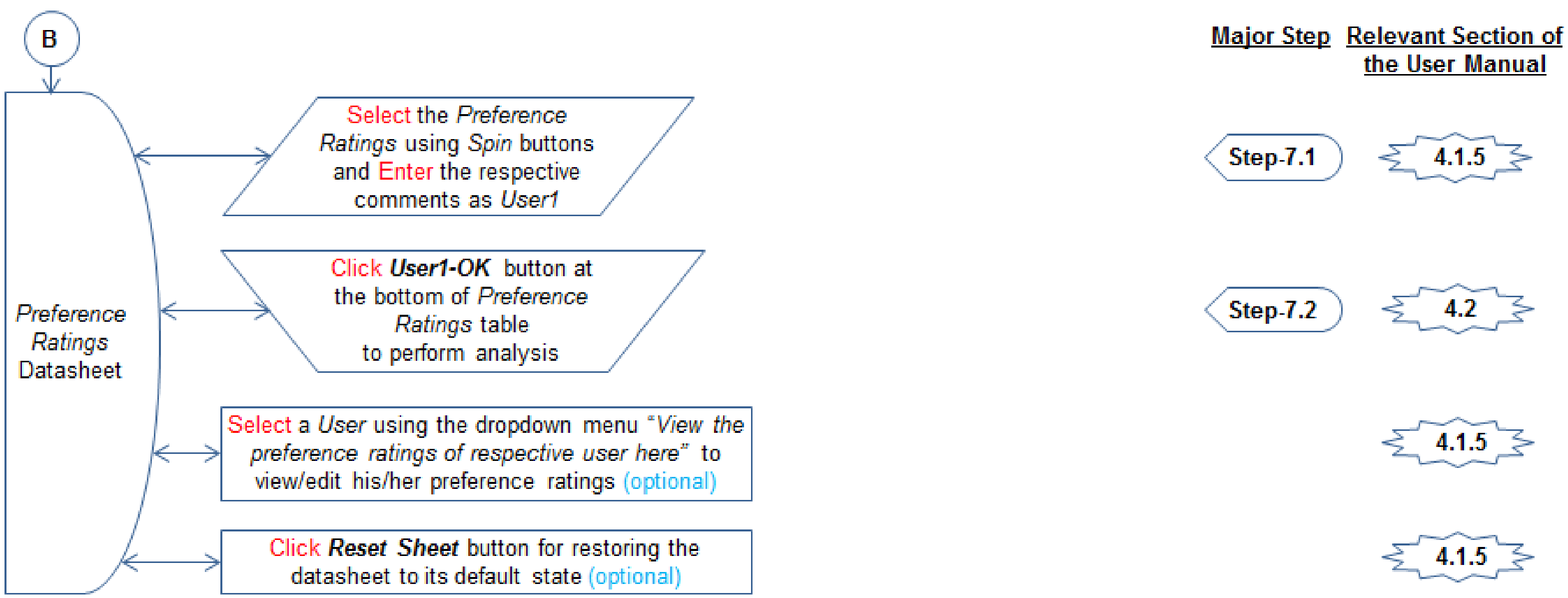
Further, a list of data required for the MiABCD and the sources of information are presented in Appendix-EA. To demonstrate the use of MiABCD, an example, including the mathematical concepts, is presented in Appendix-EB. Appendix-EC, the glossary, provides definitions and commonly used terms/acronyms in MiABCD program and this user manual.

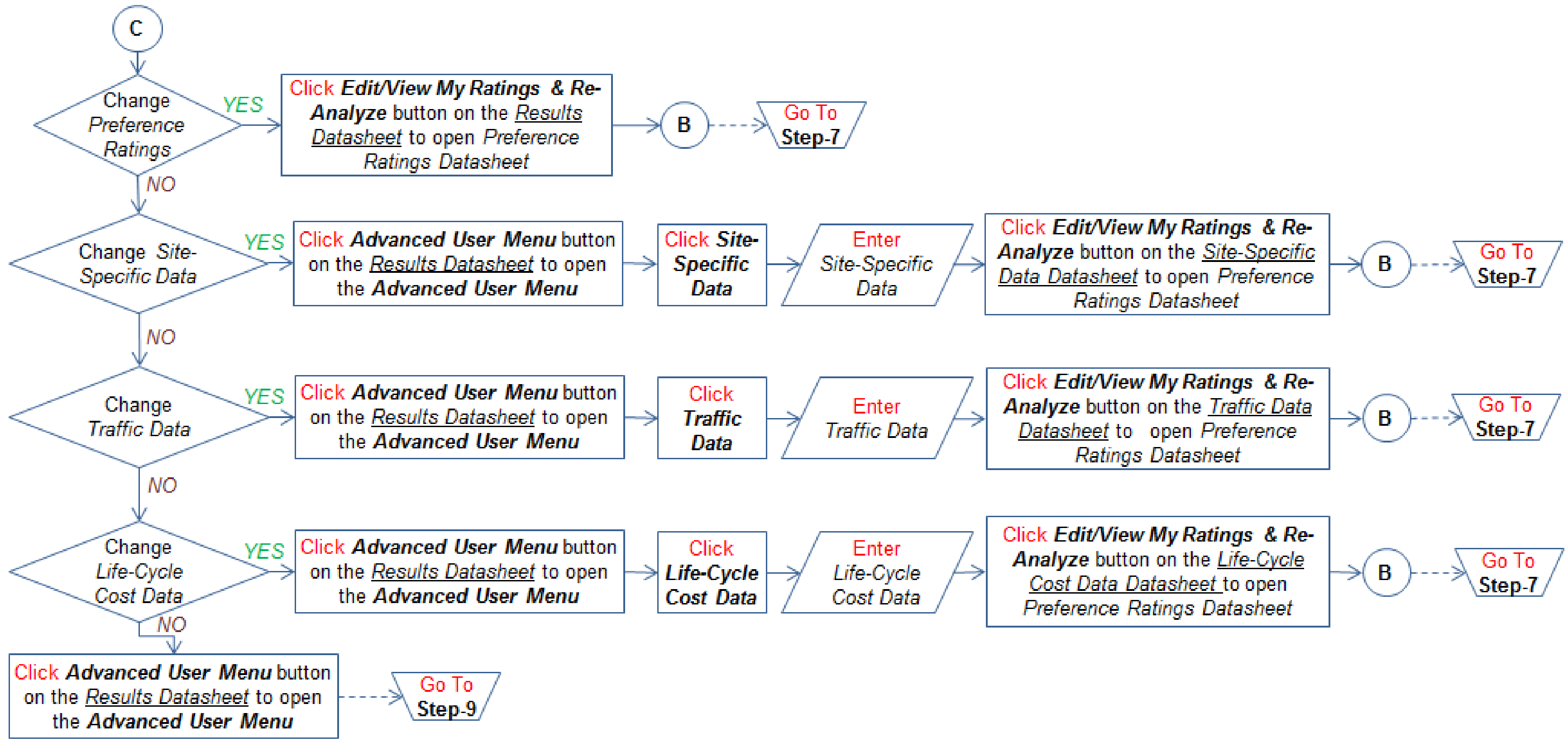
Advanced User Flowchart



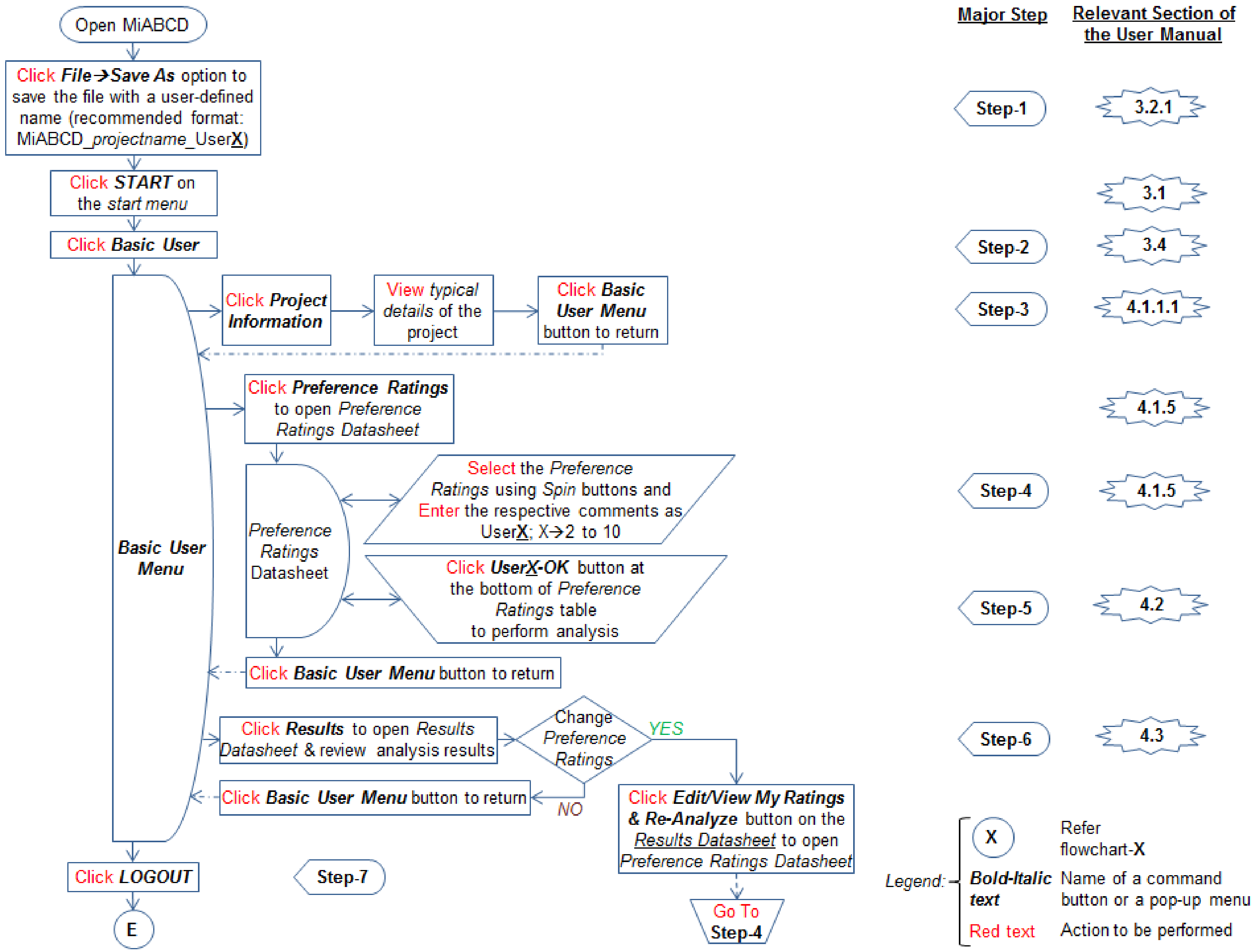

D
Click EXIT on the start menu
File saving option?
YES
NO
File will be saved with the name defined in Step-1
Click Cancel and Don't Save to close the file without saving
Enter new user-defined name for the file (recommended format: MiABCD_projectname_User1_withData) and Click Save to save the file
Forward the updated file to the Basic Users, in sequence
Major Step
Relevant Section of the User Manual
Step-10
3.2.2
Step-11
4.1.5-Note
Legend:
X
Refer flowchart-X
Bold-Italic text
Name of a command button or a pop-up menu
Red text
Action to be performed

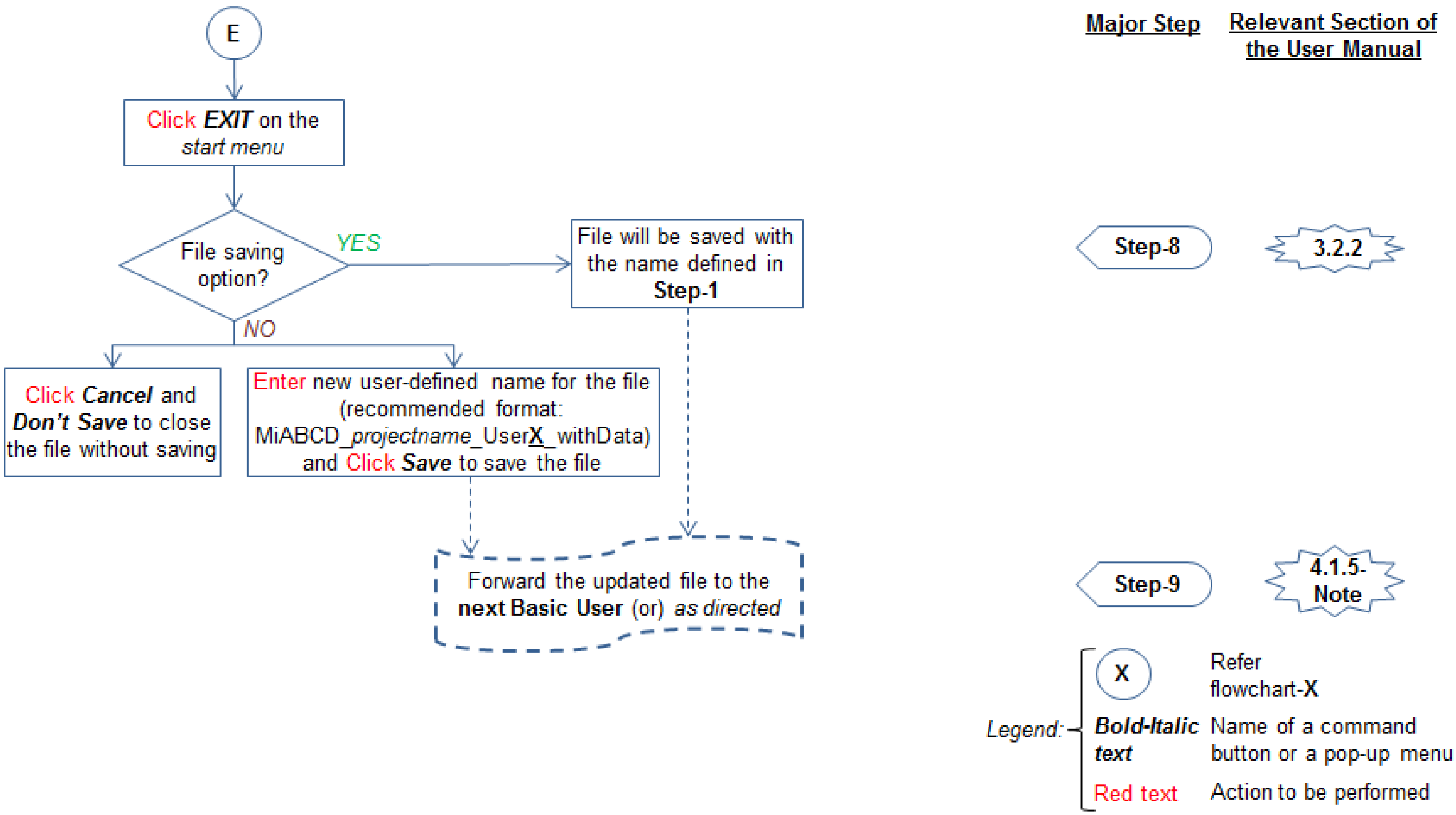

A
Major Step
Relevant Section of the User Manual
Project Details Menu
Click Project Information
Enter typical details of the project
Click Project Details Menu button to return
Select Project Category using the two dropdown menus
Click View/Add Decision-Making (D-M) Parameters
Click Add Sub-Parameters button to add, delete, or modify User-Defined Sub-Parameters
Click Done to return
Click Project Details Menu button to return
Click View/Edit General Data (optional)
Click Project Details Menu button to return
Step-3.1
4.1.1.1
Step-3.2
4.1.1.2
Step-3.3
4.1.1.3
4.1.1.4


B
Major Step
Relevant Section of the User Manual
Preference Ratings Datasheet
Select the Preference Ratings using Spin buttons and Enter the respective comments as User1
Click User1-OK button at the bottom of Preference Ratings table to perform analysis
Select a User using the dropdown menu "View the preference ratings of respective user here" to view/edit his/her preference ratings (optional)
Click Reset Sheet button for restoring the datasheet to its default state (optional)
Step-7.1
4.1.5
Step-7.2
4.2
4.1.5
4.1.5


C
Change Preference Ratings
YES
Click Edit/View My Ratings & Re-Analyze button on the Results Datasheet to open Preference Ratings Datasheet
B
Go To Step-7
NO
Change Site-Specific Data
YES
Click Advanced User Menu button on the Results Datasheet to open the Advanced User Menu
Click Site-Specific Data
Enter Site-Specific Data
Click Edit/View My Ratings & Re-Analyze button on the Site-Specific Data Datasheet to open Preference Ratings Datasheet
B
Go To Step-7
NO
Change Traffic Data
YES
Click Advanced User Menu button on the Results Datasheet to open the Advanced User Menu
Click Traffic Data
Enter Traffic Data
Click Edit/View My Ratings & Re-Analyze button on the Traffic Data Datasheet to open Preference Ratings Datasheet
B
Go To Step-7
NO
Change Life-Cycle Cost Data
YES
Click Advanced User Menu button on the Results Datasheet to open the Advanced User Menu
Click Life-Cycle Cost Data
Enter Life-Cycle Cost Data
Click Edit/View My Ratings & Re-Analyze button on the Life-Cycle Cost Data Datasheet to open Preference Ratings Datasheet
B
Go To Step-7
NO
Click Advanced User Menu button on the Results Datasheet to open the Advanced User Menu
Go To Step-9

Basic User Flowchart





Notes:

(1) This "*Basic User Flowchart*" shall be used only after the MiABCD file is updated with the complete bridge project data, following the steps in "*Advanced User Flowchart*."

(2) In the term "User**X**," **X** can range from 2 to 10 as the preference ratings can be provided by a maximum of 10 users for a particular project.

1.2 ABOUT MiABCD

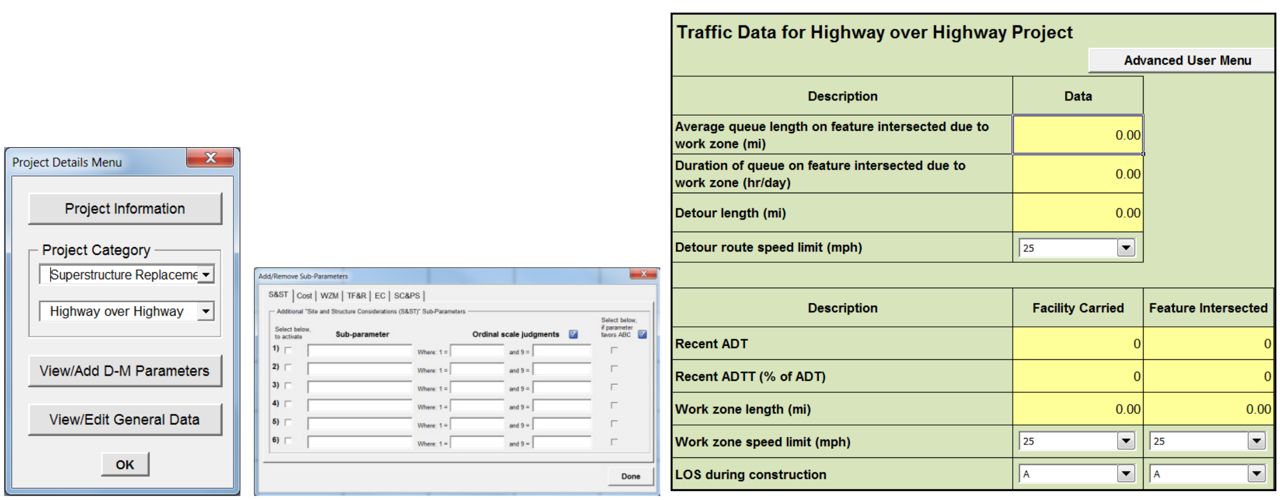
The decision-making model is developed by the Civil and Construction Engineering Department at Western Michigan University, Kalamazoo, MI, under the Michigan Department of Transportation (MDOT) funded project *"Improving Bridges with Prefabricated Precast Concrete Systems."*

The model is developed envisioning the need to incorporate project-specific data and available user-cost and life-cycle cost models to facilitate the decision makers with quantitative data to make informed decisions on bridge construction alternatives. The software is developed using available programming platforms of Microsoft Excel and Visual Basic.

Only the *Superstructure Replacement* decisions can be evaluated using the current version of the software.

1.2.1 MiABCD Graphical User Interface (GUI)

The application is developed using Microsoft Excel and Visual Basic for Applications (VBA) scripts. The Excel worksheets execute the procedures. The VBA's Graphical User Interface (GUI) forms are utilized to interact with the user. These forms are termed as *Pop-up Menus* (Figure 1-a and b), and the Excel sheets that are customized for user input are termed as *Datasheets* (Figure 1-c).



(a) (b) (c)

Figure 1. Sample popup menus and datasheet

1.2.1.1 Pop-up Menus

The main features of the *pop-up menu* are to provide (1) Command buttons, (2) Dropdown menus, (3) Tabs, (4) Text fields, (5) Check boxes, and (6) an Additional information button (?). The most commonly used features are the *command buttons* and *dropdown menus*. A few examples of *pop-up menus,* along with their features, are shown in Figure 2. A description of each key feature is given below:

1) *Command buttons*: A command button is used to execute an embedded VBA script to run an algorithm or direct the user to a pop-up menu or a specific datasheet.
2) *Dropdown menus*: A dropdown menu is used to select a desired option from a predefined list of options.
3) *Tabs*: Tabs are used to switch between options that are predefined on a *pop-up menu.*
4) *Text fields*: Text fields allow incorporating user-defined sub-parameters in the decision-making process. Once defined, the text will be visible in the corresponding *datasheet*.
5) *Check boxes*: A check box allows either activating or deactivating a subroutine.
6) *Additional information button (?)*: This button allows a user to receive additional information to navigate through the pop-up menu or to complete the datasheet.

Note: In subsequent sections of the user manual, ***Bold-Italic*** text is used to represent the names of the *pop-up menus* and *command buttons*.

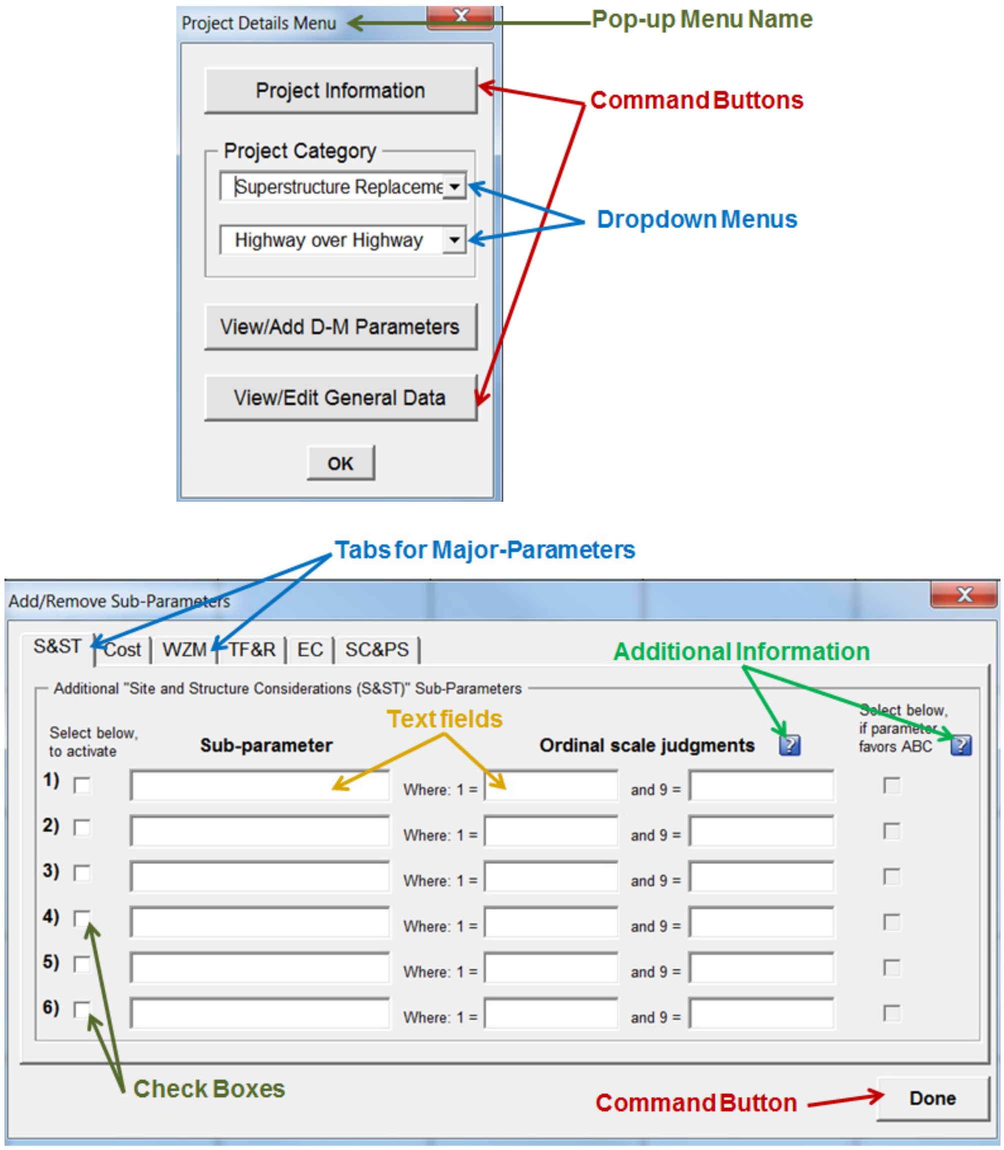


Figure 2. Example pop-up menus and key features

1.2.1.2 Datasheets

An example *datasheet* is shown in Figure 3. The primary features of a *datasheet* are (1) Command buttons, (2) Dropdown menus, and (3) Data input fields.

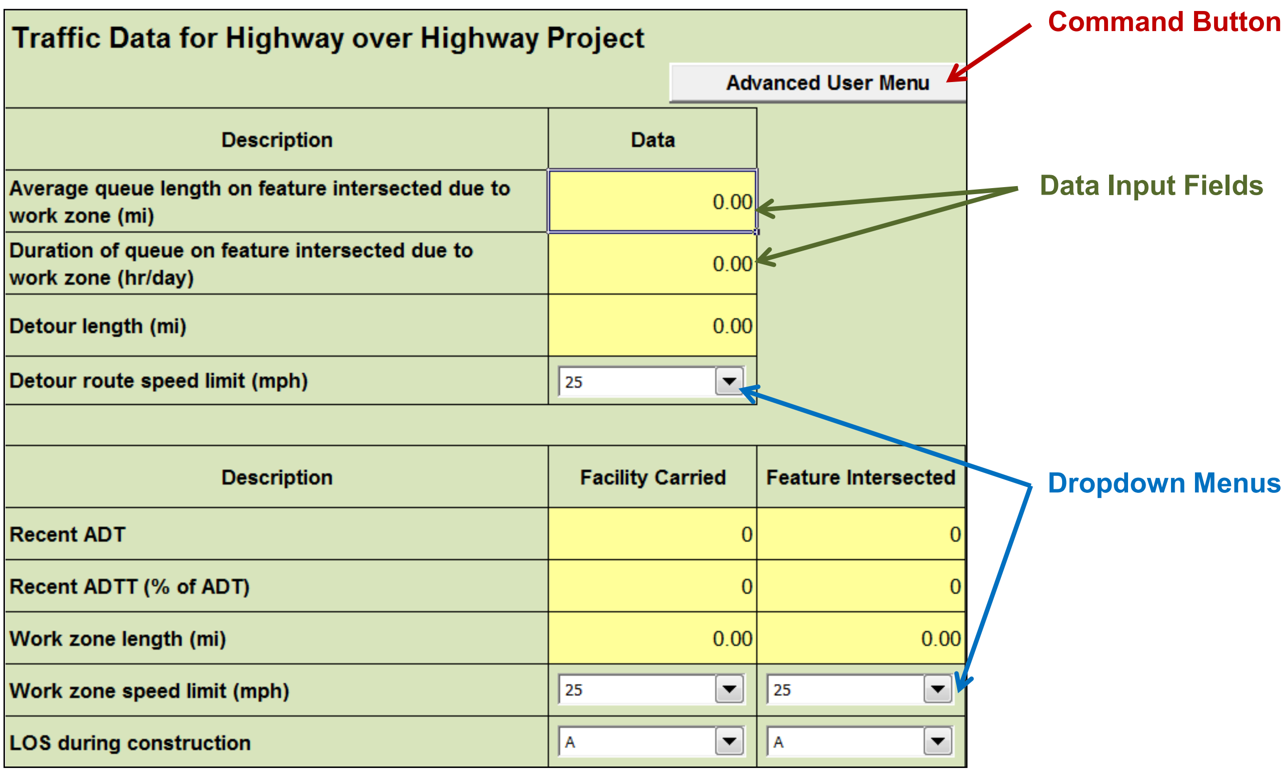


Figure 3. An example datasheet and its features

1.2.2 Default MiABCD File

The default MiABCD file, a Microsoft Excel macro enabled workbook, contains the following:

1) Empty *datasheet* under ***Project Information*** (see Section 4.1.1.1),
2) Default decision-making parameters under ***View/Add D-M Parameters*** (see Section 4.1.1.3),
3) Default "regional data" as well as the "model presets" under the ***View/Edit General Data*** (see Section 4.1.1.4),
4) Default *datasheet* under the ***Site-Specific Data*** (see Section 4.1.2),
5) Default *datasheet* under the ***Traffic Data*** (see Section 4.1.3),
6) Default *datasheet* under the ***Life-Cycle Cost Data*** (see Section 4.1.4), and
7) Default *datasheet* under the ***Preference Ratings*** (see Section 4.1.5).

Note: After running the program for a new project, the data file needs to be saved using the ***Save As*** option with a file name descriptive of the project (see Section 3.2.1). Otherwise, the *default* values may be accidentally edited, and additional steps may be required to restore the default MiABCD file (see Section 2.3.1).

2 INSTALLATION

2.1 SYSTEM REQUIREMENTS

The MiABCD software program is designed to run on the Microsoft Excel platform with up-to-date service packs. The minimum system requirements for installation are as follows:

- Microsoft Excel 2007 (or later)
- Windows XP (or later)
- 500 MHz Processor
- 512 MB Memory (RAM)
- Hard disk 2 GB
- Display 1024 × 768

2.2 PROGRAM INSTALLATION PACKAGE

The program installation package, *Setup MiABCD*, will extract the following files to a specified folder:

1) *MiABCD_v2.0.xlsm*,
2) *MiABCD_v2.0_backup.xlsm*,
3) *MiABCD_User Manual.xlsm*, and
4) *MiABCD_ReadMe.xlsm*.

2.3 SETUP/TROUBLE SHOOTING INSTRUCTIONS

2.3.1 Enabling Full Access to VBA Algorithms *(for Microsoft Excel 2010)*

When the MiABCD software program is executed for the first time, the following steps need to be performed to prevent VBA runtime errors

1) From Excel menu, go to ***File→ Excel Options.***
2) In the *Excel Options* window, go to ***Trust Center*** tab on left.
   - Click on ***Trust Center Settings…*** button on right.
3) In the *Trust Center* window, go to ***ActiveX Settings*** tab on left.
   - Select the option ***Enable all controls without restrictions and without prompting.***
4) In the *Trust Center* window go to ***Macro Settings*** tab on left.
   - Select the option ***Enable all macros.***
   - Check the box for ***Trust access to the VBA project object model.***
5) In the *Trust Center* window, go to ***Protected View*** tab on left.

- Uncheck the box for ***Enable Protected View for files originating from the Internet.***

6) In the *Trust Center* window, go to ***File Block Settings*** tab on left.
   - Uncheck all boxes under the ***open*** column on right.
7) Click ***OK*** until you return to the workbook, and then start using *MiABCD software program*.

Note: If needed, the settings in the *Trust Center* window may be *restored* once *finished* using the MiABCD.

2.3.2 Disabling Acrobat…..COM Addin *(for Microsoft Excel 2010)*

If ***Acrobat…..COM Addin*** is enabled in Excel, additional steps need to be performed to disable this Addin. This Addin may create problems while printing as well as exiting the *MiABCD* software program. For example, a VBA password may be requested while exiting the MiABCD software program.

To check if the ***Acrobat…..COM Addin*** is enabled in the Excel, the following steps need to be performed *(for Microsoft Excel 2010)*,

1) From Excel menu, go to ***File→ Excel Options.***
2) In the *Excel Options* window, go to ***Add-Ins*** tab on left.
   - Look for ***Acrobat….COM Addin***, under the ***Active Application Add-ins*** heading, and the ***Inactive Application Add-ins*** heading in the window.
3) If the ***Acrobat….COM Addin*** is under the ***Active Application Add-ins*** heading, then ***Acrobat….COM Addin*** is enabled in your Excel.

To disable ***Acrobat…..COM Addin***, the following steps need to be performed *(for Microsoft Excel 2010)*,

1) From Excel menu, go to ***File→ Excel Options.***
2) In the *Excel Options* window, go to ***Add-Ins*** tab on left.
   - Look for ***Manage*** drop-down menu at the bottom center of the window.
   - Use the ***Manage*** drop-down menu to select ***COM Add-ins.***
   - Click on ***Go…*** besides the ***Manage*** drop-down menu.
   - Uncheck the box for ***Acrobat….COM Addin.***
3) Click ***OK*** until you return to the workbook, and then start using *MiABCD* software program.

2.3.3 Restoring Default MiABCD File

The *default MiABCD file* may be restored using the backup file, MiABCD_v2.0_backup, from the directory where the MiABCD files were extracted. To restore the *file,* the following steps need to be performed:

1) Delete the file ***MiABCD_v2.0*** from **the respective directory.**
2) Create a copy of the file ***MiABCD_v2.0_backup*** in **the respective directory.**
3) Rename the newly created file to ***MiABCD_v2.0***; this shall be used to run the *MiABCD* software program.

3 MiABCD GRAPHICAL USER INTERFACE (GUI)

3.1 START MENU

The MiABCD *start menu* is shown in Figure 4.

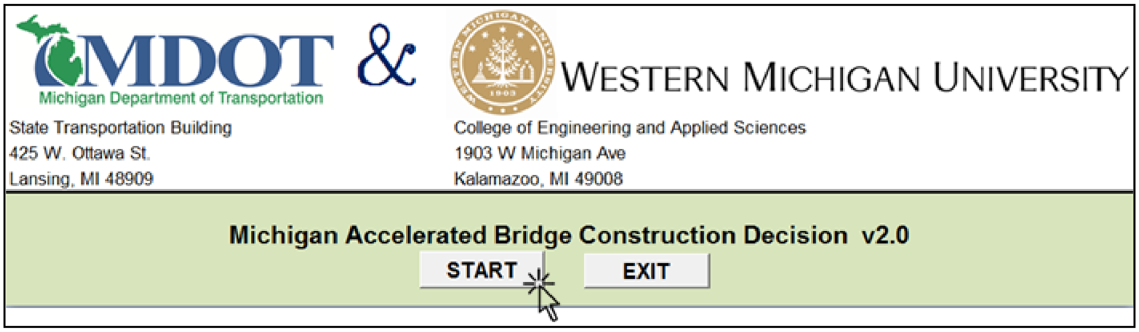


Figure 4. *Start menu* of the decision-making model

The ***START*** command button on the *start menu* opens the ***User Selection Menu*** as shown in Figure 5. This menu provides access to two types of users: *Advanced User* and *Basic User*. The ***EXIT*** command button opens the ***Caution*** pop-up window. This window allows either saving or not saving the data before closing the program (sees Section 3.2.2).

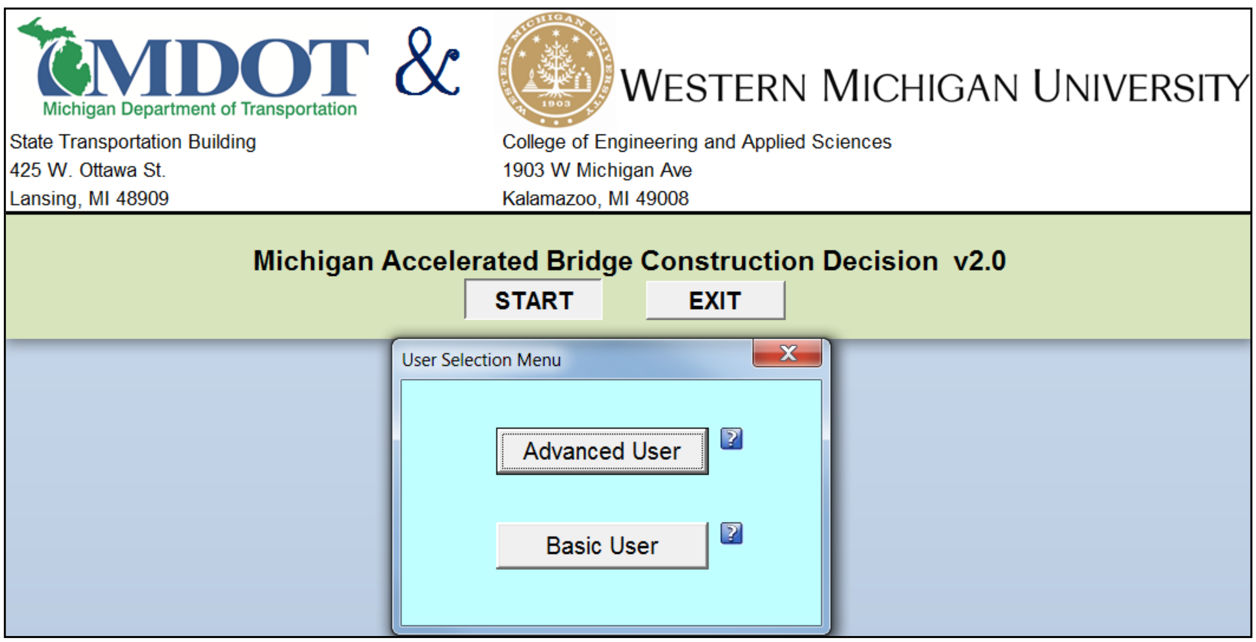


Figure 5. *User Selection Menu* of the decision-making model

The ***Advanced User*** is generally a project manager who is familiar with the project specifics such as site-specific data, cost estimates, traffic data, and construction methodologies. The ***Advanced User*** can perform the tasks listed in Figure 6.

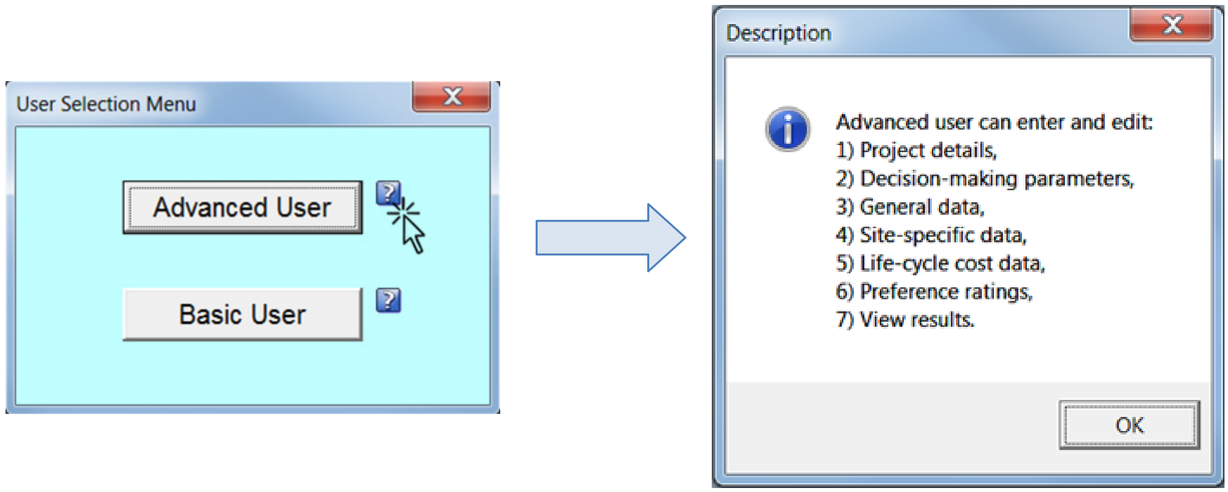


Figure 6. Description of the *Advanced User* option

The ***Basic User*** is generally an expert who will enter the preferences on qualitative parameters based on the experience on recent bridge projects. The ***Basic User*** can perform the tasks listed in Figure 7.

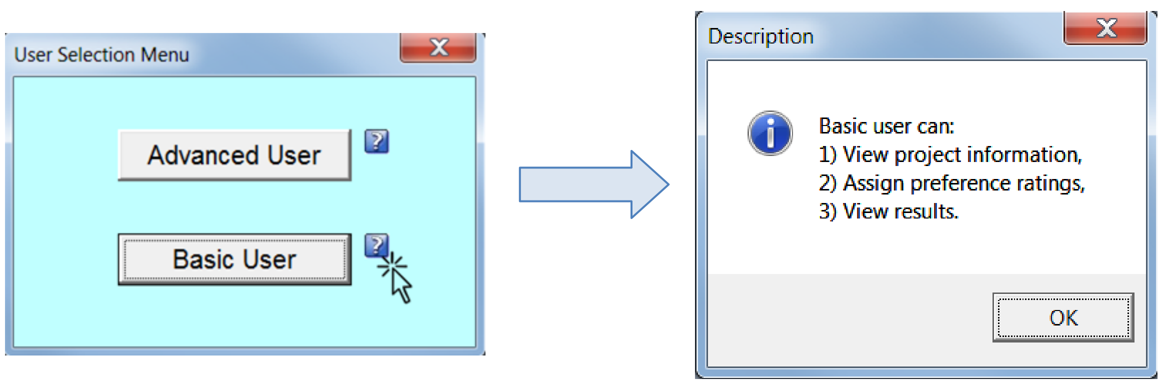


Figure 7. Description of the *Basic User* option

The tasks assigned to the ***Advanced User*** and ***Basic User*** can be accessed by clicking on the icon located next to respective command buttons (Figure 6 and Figure 7).

Note: For every new project, the *Advanced User* needs to first save the *default MiABCD file* with a user-defined name using the *Save As* option from Excel menu (see Section 3.2.1) and complete the data entry process by following the steps defined in the *Advanced User Flowchart.* The *Advanced User*, following the data entry, needs to ***Logout*** from the ***Advanced User Menu*** and ***Exit*** the program using the respective command button on the *start menu* while saving the file using the ***Yes*** option on the ***Caution*** pop-up window that appears subsequently (see Section 3.2.2). For preference rating entry by the *Basic User*, the *Advanced*

User needs to forward the file in a sequential order (i.e., sending the file to expert-1; then after his/her input, sending the file to expert-2, and so on), for obtaining their qualitative judgments. These experts, who are specialized in various aspects of a bridge construction project, are described as *Basic Users* since their task is limited to entering preference ratings of qualitative parameters (see Section 4.1.5).

3.2 FILE SAVING OPTIONS

3.2.1 Using the *Save As* Option after Starting the Program

Immediately after executing the MiABCD program, the file needs to be saved by the *Advanced User* with a file name descriptive of the project using the *Save As* option from Excel menu as shown in Figure 8. This will prevent any accidental changes to the default MiABCD file.

Moreover, immediately after starting the program, each *Basic User* needs to save the file (they received from the *Advanced User*) with their name added to the title using the *Save As* option from Excel menu (Figure 8). This will ensure a backup including the **Project Data**, in case the file is corrupted or saved with incorrect preference ratings.

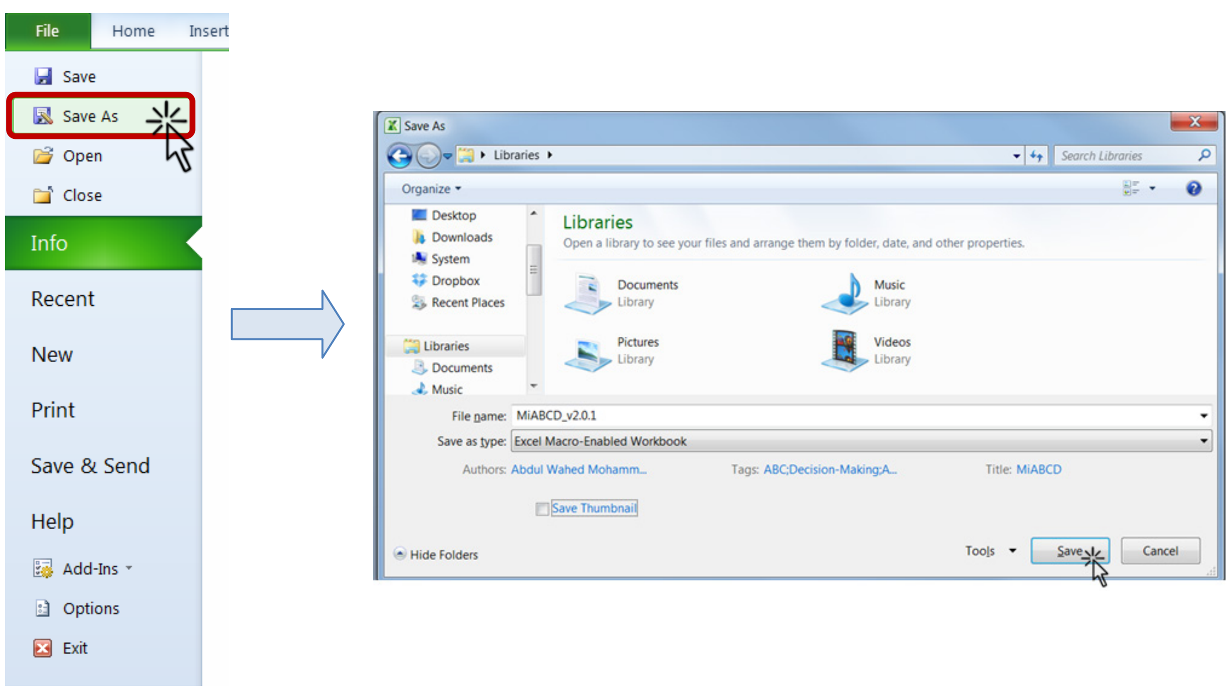
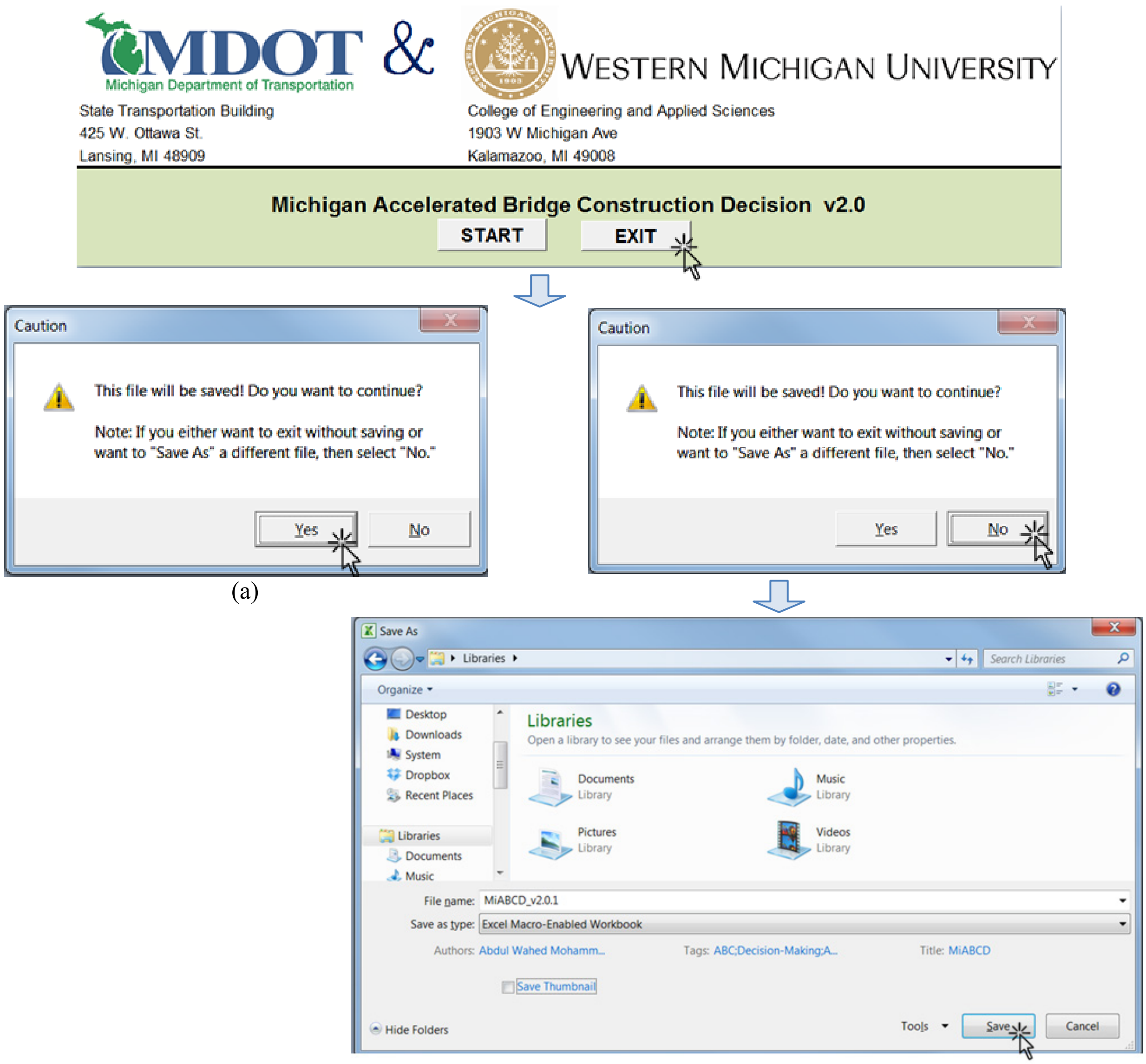


Figure 8. Using *Save As* option from the Excel menu

3.2.2 Exiting the Program with *Save/Save As* Option

The ***EXIT*** command button on the *start menu* of the decision-making model will open the ***Caution*** pop-up window as shown in Figure 9a. The pop-up window options are ***Yes*** and ***No***. If the file was saved earlier using a user-defined name (see Section 3.2.1), select ***Yes*** (Figure 9a). This will save the current data (or any modifications) and exit MiABCD. However, if the file was not saved after starting the program (see Section 3.2.1), select ***No.*** This will open up another pop-up window to use the ***Save As*** option to save the current data to a "new file" with a user-defined name (Figure 9b).



(a) (b)

Figure 9. Exiting the decision-making model by (a) saving on the current file or (b) saving as a different file

3.2.3 Using *Save* Option during Intermediary Steps

The *Advanced User* and *Basic Users* can *Save* the file during the intermediate steps of evaluation using the *Save* option from the Excel menu or the *Save* icon () on the Excel Quick Access Toolbar (Figure 10).

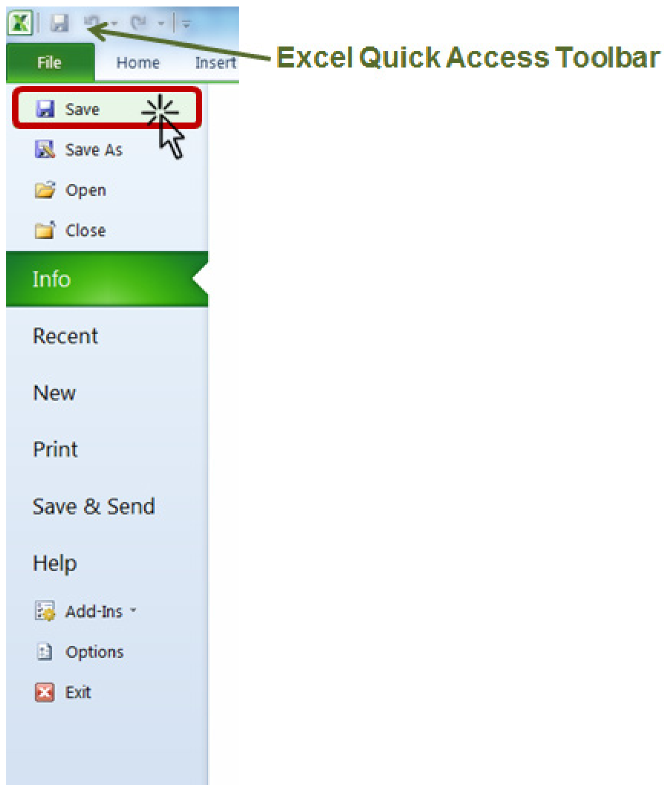


Figure 10. Using *Save* option from the Excel menu

3.3 ADVANCED USER MENU

The ***Advanced User Menu*** is accessed by clicking the ***Advanced User*** command button and then entering the password (Figure 11). This option allows access to all the features of the software program for entering/editing project specific details, changing default general data, and incorporating additional qualitative parameters. For each project, the data entry process should first start with the *Advanced User* to enter the project specific data, and then the ***Basic User Menu*** (see Section 3.4) can be used to enter preference ratings.

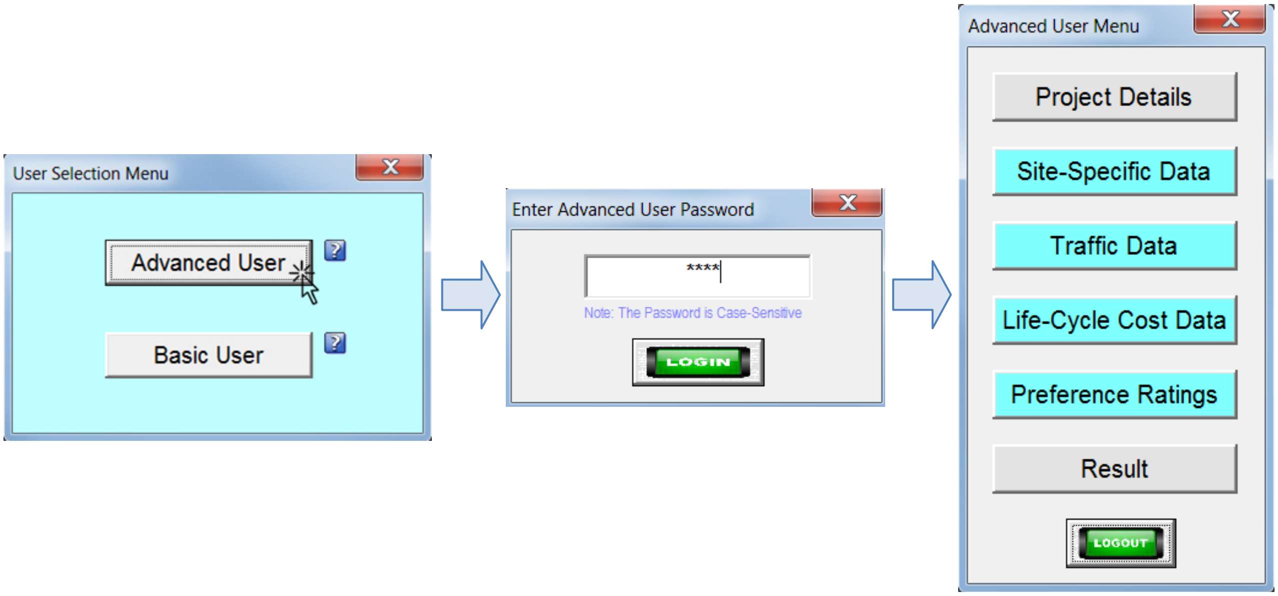


Figure 11. Accessing the *Advanced User Menu*

The ***Advanced User Menu*** consists of seven command buttons (Figure 11); of which four command buttons (viz., ***Site-Specific Data***, ***Traffic Data***, ***Life-Cycle Cost Data***, and ***Preference Ratings***) are used to input the *project specific data* (see Section 4.1). The ***Preference Ratings*** command button is also used to access the data analysis option (see Section 4.2).

The ***Project Details*** command button on the ***Advanced User Menu*** is used to access the ***Project Details Menu*** (see Section 3.3.1). The ***Result*** command button is used to view/print results of the evaluation. The ***Logout*** command button is used to return to the *start menu* after entering the required data (see Section 4.1). After this, the preference ratings from multiple users can be obtained using ***Basic User Menu***.

3.3.1 Project Details Menu

The ***Project Details Menu*** is accessed through the ***Advanced User Menu*** as shown in Figure 12. The command buttons on the ***Project Details Menu*** are as follows:

- ***Project Information***: This command button is used to enter the project related information (see Section 4.1.1.1).
- ***Project Category***: These dropdown menus are used to specify the project type and the feature intersected.
- ***View/Add D-M Parameters***: This command button is used to view the default decision-making major- and sub-parameters: also, to add additional sub-parameters, if required.

- ***View/Edit General Data***: This command button is used to view/edit the default "regional data" as well as the "model presets."

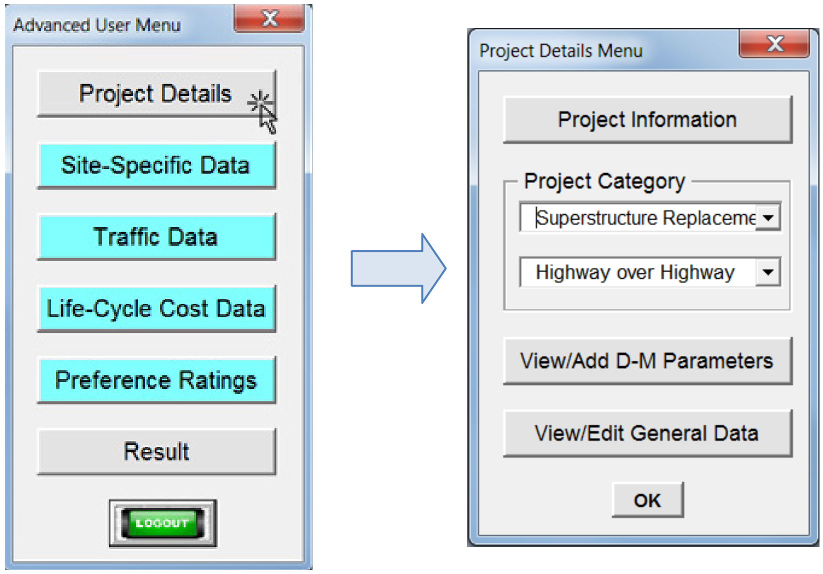


Figure 12. Accessing the *Project Details Menu*

In this menu, generally the ***Project Information*** command button is used for entering the typical details of a project (see Section 4.1.1.1). The other command buttons are used if there is a need to customize the "decision-making parameters", "regional data" and "model presets." The "regional data" includes wage and cost, county jobs multiplier, material procurement distance classification, traffic data classification, and bridge spans classification. The "model presets" include predefined tables that define the relationship among the project data, ordinal scale ratings, and the Analytical Hierarchy Process (AHP) pair-wise comparison ratings. Primarily, this section is the core for AHP automation and ABC vs. CC evaluation.

3.4 BASIC USER MENU

The ***Basic User Menu*** is accessed by clicking the ***Basic User*** command button (Figure 13). This option does not require any password, and it is designed to allow multiple users/experts to enter their preferences for a list of qualitative parameters and also review the analysis result.

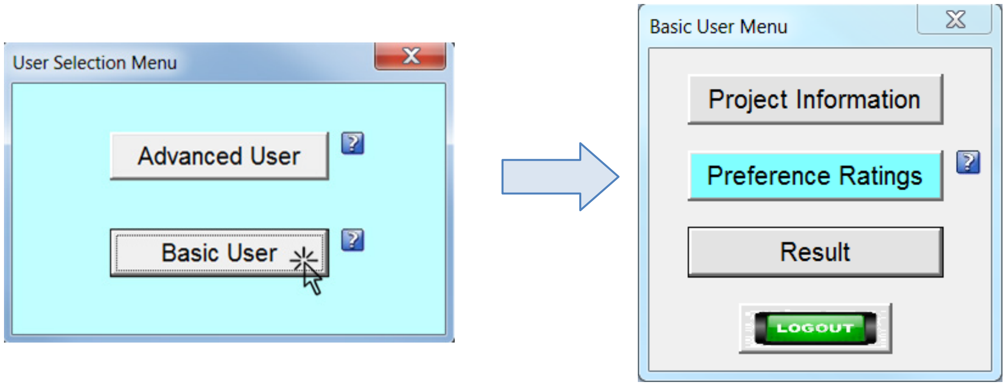


Figure 13. Accessing the *Basic User Menu*

The ***Basic User Menu*** consists of four command buttons that are as follows (Figure 13):

- ***Project Information***: This command button is used to view the project-related information that is entered by an *Advanced User* (see Section 4.1.1.1).
- ***Preference Ratings:*** This command button is used to assign the preference ratings to qualitative parameters using an ordinal scale (1 to 9) (see Section 4.1.5) and to perform the data analysis (see Section 4.2).
- ***Result:*** This command button is used to view/print results of the evaluation.
- ***Logout:*** This command button is used to return to the *Start Menu*.

4 MiABCD BASIC OPERATIONS

The major steps of the MiABCD are (1) data input, (2) data analysis, and (3) reviewing analysis results. First, an *Advanced User* needs to perform these three steps for each new project using the ***Advanced User Menu*** options. Afterwards, ***Basic User Menu*** options are accessed to enter preference ratings, perform data analysis, and review analysis results. Sections 4.1, 4.2, and 4.3 provide detailed explanations of the major steps performed by an *Advanced User*. Also, the steps that need to be followed by a *Basic User* are described in Sections 4.1.1.1, 4.1.5, 4.2, and 4.3.

4.1 DATA ENTERING PROCESS

As shown in Figure 14, the ***Advanced User Menu*** consists of five command buttons that are used for data input. The command buttons are: ***Project Details, Site-Specific Data***, ***Traffic Data***, ***Life-Cycle Cost Data***, and ***Preference Ratings***.

Figure 14. ***Advanced User Manu*** **with five commands buttons used for data input**

4.1.1 Project Details

The ***Project Details*** command button opens the ***Project Details Menu*** (Figure 15). The ***Project Information*** command button is used for entering the basic details about a project (see Section 4.1.1.1). The dropdown menus listed under ***Project Category*** are used to select the scope of the project and feature intersected. The Sections 4.1.1.3 and 4.1.1.4 describe the ***View/Add D-M Parameters*** and ***View/Edit General Data*** command buttons, respectively.

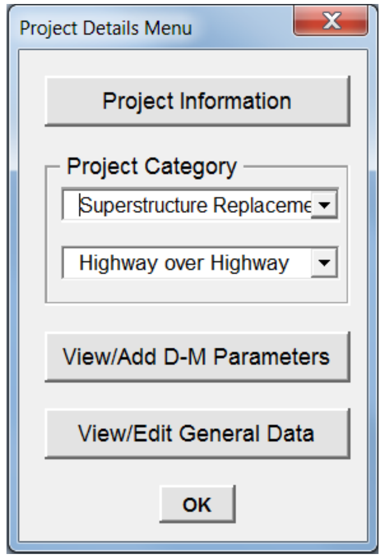


Figure 15. ***Project Details Menu***

4.1.1.1 Project Information

The ***Project Information*** command button will open the datasheet shown in Figure 16. The *Advanced User* needs to provide the information required for completing the datasheet. The datasheet will contain the name of the project, date the decision-making process is initiated, name of the *Advanced User* (typically the project manager), and a description of the project, such as the project location, surrounding businesses and stakeholders, and any critical aspects that the project manager thinks useful to the *Basic User*s (the experts). A reference image of the project site could also be embedded in this *datasheet* as shown in Figure 16. The *Advanced User* may click on the reference image area to open a *Microsoft Paint* window to upload an image. The image needs to be imported using the *"Paste from"* option in the *Microsoft Paint* and saved, so as to be displayed in the ***Project Information*** *datasheet*. Here, the command button on the *Project Information* datasheet (Figure 16) will reopen the ***Project Details Menu***.

The *Basic User* can access the uneditable *Project Information* datasheet using the ***Project Information*** command button on the ***Basic User Menu*** (see Section 3.4). This allows the basic user to review the project information and any critical aspects that are specified by the *Advanced User*. The command button on the *Project Information* datasheet is used to navigate back to the ***Basic User Menu***.

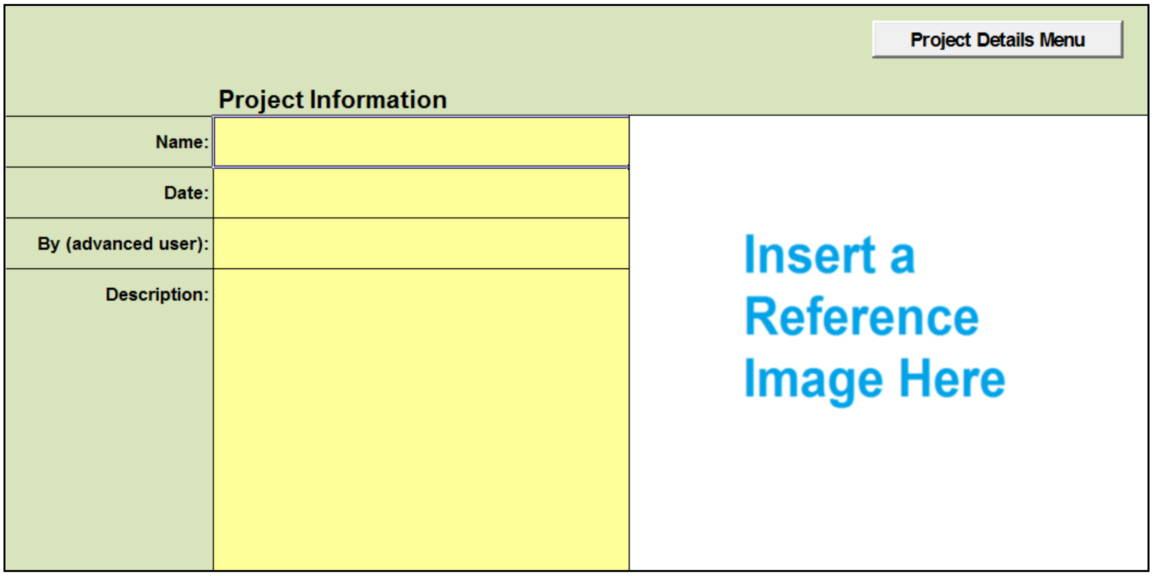
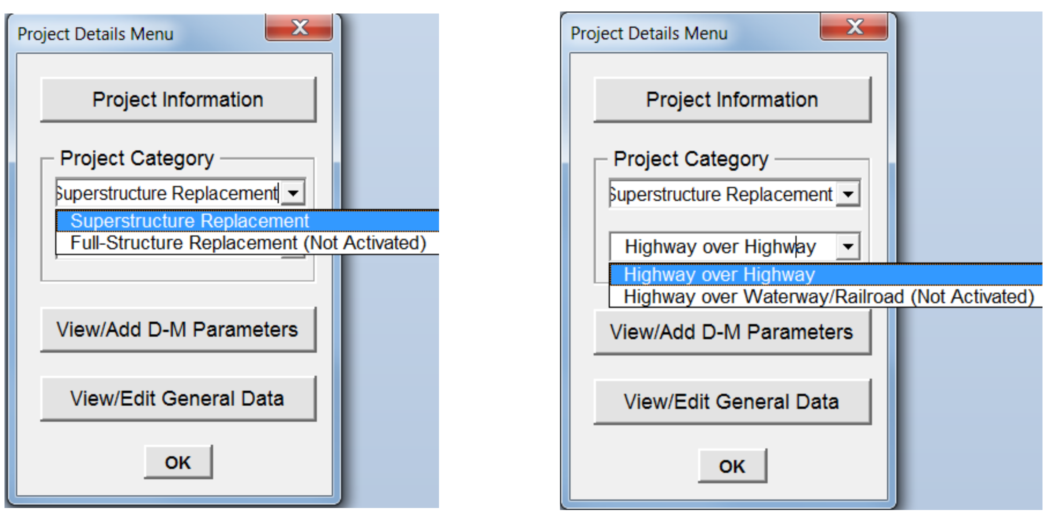


Figure 16. ***Project Information*** **datasheet**

4.1.1.2 Project Category

As shown in Figure 17, two *dropdown menus* (see Section 1.2.1.1) are available under the *Project Category* field. The first *dropdown menu* consists of two options: (1) Superstructure Replacement and (2) Full-Structure Replacement. The second *dropdown menu* consists of two options to select the feature intersected: (1) Highway over Highway and (2) Highway over Waterway/Railroad.



(a) (b)

Figure 17. Selecting project type and feature intersected from *Project Category* field

Select the ***OK*** command button at the bottom of ***Project Details Menu*** to store the *Project Category* changes and go back to the ***Advanced User Menu***.

Warning: Closing the ***Project Details Menu*** using icon will not save the changes made under the *Project Category* field.

Note: The options available under ***Project Category*** dropdown menus in MiABCD v2.0 or its updates, such as, v2.1 etc., are limited to *"Superstructure Replacement"* and *"Highway over Highway."*

4.1.1.3 View/Add Decision-Making Parameters

The ***View/Add D-M Parameters*** command button on the ***Project Details Menu*** opens the *datasheet* with default decision-making parameters (Figure 18). Apart from the default sub-parameters under each major-parameter, an *Advanced User* can add up to six additional sub-parameters under each major parameter. Therefore, a total of 36 (i.e., 6×6) additional sub-parameters can be incorporated in the analysis.

Note: The additional sub-parameters defined by the *Advanced User* can be removed or modified; whereas, the default sub-parameters cannot be changed.

Decision-Making Parameters for Highway over Highway Project

Project Details Menu

Major-Parameters	Site and Structure Considerations (S&ST)	Cost	Work Zone Mobility (WZM)	Technical Feasibility and Risk (TF&R)	Environmental Considerations (EC)	Seasonal Constraints and Project Schedule (SC&PS)
Sub-Parameters	Precaster/Ready-mix supplier proximity	Initial Construction cost	Significance of maintenance of traffic on facility carried	Contractor experience	Environmental protection (e.g., wet land)	Seasonal limitations
	Availability of staging area	Life-cycle cost	Significance of maintenance of traffic on feature intersected	Manufacturer/ Precast plant experience	Aesthetic requirements	Construction duration
	Existing structure type and foundations	User cost	Length of detour	Work zone traffic risk		Stakeholder(s') limitations
Sub-Parameters	Terrain to traverse	Economic impact on surrounding businesses	Significance of level of service on detour route	Construction risks		
	Access and mobility of construction equipment	Economic impact on surrounding communities	Impact on nearby major intersection due to traffic on facility carried			
	Number of similar spans		Impact on nearby major intersection due to traffic on feature intersected			
User-Defined Sub-Parameters		Direct Cost		Scour		

Add Sub-Parameters

Figure 18. Datasheet of decision-making parameters for highway over highway project

The ***Add Sub-parameters*** command button on the datasheet opens the ***Add/Remove Sub-Parameters*** menu as shown in Figure 19.

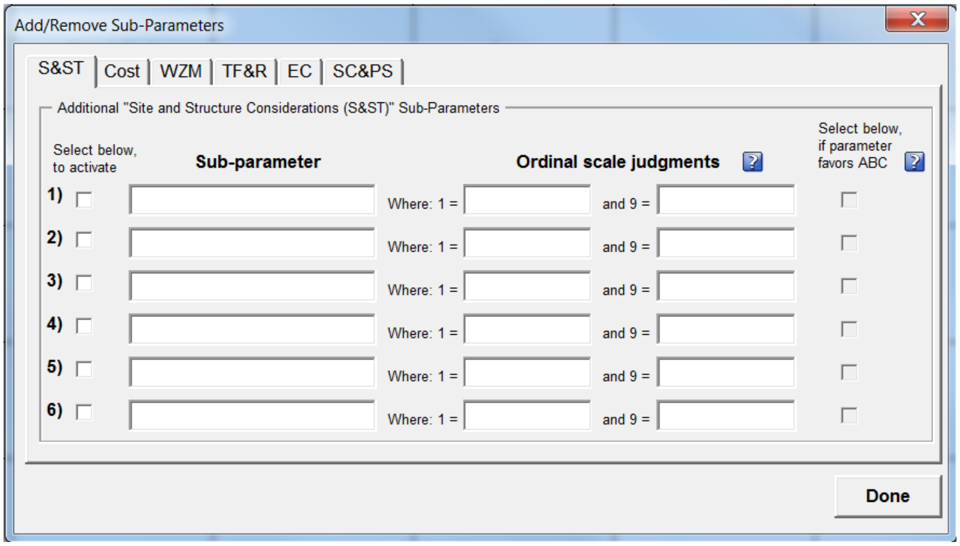


Figure 19. ***Add/Remove Sub-Parameters*** **menu**

The additional sub-parameters can only be qualitative for user preferences entered on an ordinal scale. Further, the additional sub-parameters need to be specified as *favoring ABC* or *favoring* CC with increasing ordinal values. Consider the example of an additional sub-parameter "Scour" under the major-parameter "Technical Feasibility and Risk (TF&R)." In adding "Scour," the tab "*TF&R*" in the ***Add/Remove Sub-Parameters*** menu is selected, and the data is entered as shown in Figure 20.

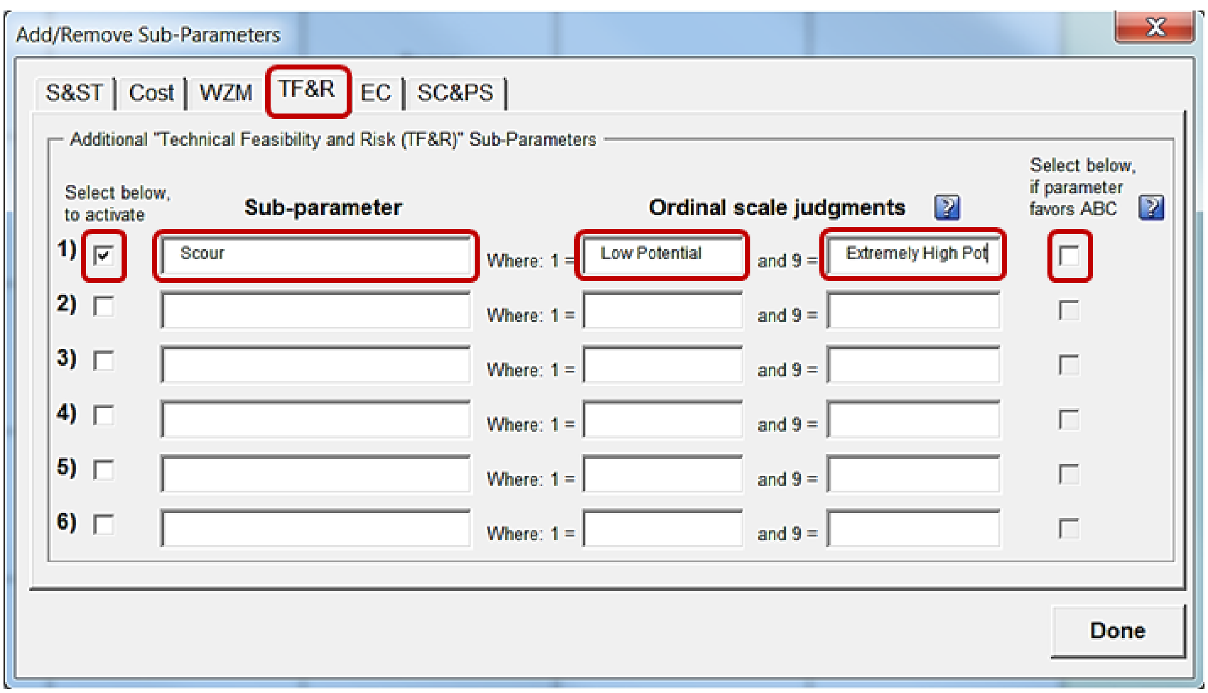


Figure 20. Adding an additional sub-parameter, "Scour," using the ***Add/Remove Sub-Parameter Menu***

The ordinal scale ratings for any sub-parameter range from 1 to 9. The rating of 1 represents low significance, and the rating of 9 represents extreme significance. For "Scour," the rating of 1 implies that there is a *low potential* of scour to occur, and the rating of 9 implies that there is an *extremely high potential* of scour to occur. Therefore, *"Low Potential"* and *"Extremely High Potential"* shall be entered in the *text fields* for 1 and 9, respectively (Figure 20).

As "Scour" potential increases ABC becomes more challenging. Therefore, increased preference for "Scour" favors CC. Then the *check box* for "Scour" (located under the title *Select below)* will be left blank (Figure 20). From this, MiABCD analysis procedure is prompted to consider "Scour" such that an increase in its ordinal scale rating will increase the preference for CC.

The additional sub-parameters, along with their ordinal scale judgment definitions, which are defined in the ***Add/Remove Sub-Parameters*** menu are automatically added to the sub-parameters list in the *Preference Ratings* datasheet (see Section 4.1.5).

4.1.1.4 View/Edit General Data

The ***View/Edit General Data*** command button opens the *"General Data"* datasheet. This datasheet provides a majority of the data needed for Analytical Hierarchy Process (AHP) automation and the evaluation of construction alternatives (ABC vs. CC).

<u>Note:</u> A majority of the data in this datasheet will not change for a region and a time duration.

The following data is included in the *"General Data"* datasheet*:*

1) Wage and cost: This includes the wage rate for personal and commercial vehicle drivers, vehicle operating cost for personal and commercial vehicles, accident cost, and accident rate (Figure 21). The default costs in the decision-making model are the prorated dollar amounts (2012 dollar) of the costs presented in FHWA's pavement division interim technical bulletin (Walls and Smith 1998).

Table 1. Wage and Cost		
Source for dollar amounts: FHWA – Pavement division interim technical bulletin (Walls and Smith 1998) Source for accident rates: Michigan State Police CJIC <http://www.michigan.gov/msp/0,4643,7-123-1593_24055---,00.html> *Note: The 1998 dollar amounts are prorated to current dollar amounts.*		
Description		**Data**
Wage rate of drivers	Personal Wage ($/hr)	16.02
	Commercial Wage ($/hr)	26.70
Vehicle operating cost	Personal vehicle ($/hr)	6.88
	Commercial vehicle ($/hr)	14.15
Accident cost	($/accident)	1024.00
Accident rate	Normal (per million vehicle miles)	215
	During Work Zone (per million vehicle miles)	240

Figure 21. Wage and cost data table

2) County jobs multiplier: The Michigan counties and their respective jobs multiplier are listed under this table. The data is obtained from Montgomery Consulting, Inc. (2011).
3) Material procurement: Distance to the ***prefabrication plant***, ***ready-mix concrete plant*** and a ***staging area*** for prefabrication or concrete batching is required. The distances to all these resource locations are grouped into five ranges to correlate with ordinal scale ratings. The default values are based on the available suppliers in Michigan and reviews of literature (PCC Center 2004, Caltrans 2008).
4) Traffic data classification: This includes the classification of Average Daily Traffic (ADT), the significance of maintenance of traffic (MOT) based on the change in Level of Service (LOS), and detour length. These criteria are classified into five ranges to correlate with the ordinal scale ratings. The default values for ADT and detour length classification are based on the available information from MCGI (2009). However, the default classification of significance of MOT is customized as described below (Figure 22).
   (i) When LOS before and during construction is the same, the ordinal scale rating = 1 (i.e., least significance)
   (ii) When the change in LOS before and during construction = 1 grade, the ordinal scale rating = 5 (i.e., moderate significance)
   (iii) When the change in LOS before and during construction = 2 grades or more, the ordinal scale rating = 9 (i.e., extreme significance)

Table 6. Level of Service Definition	
LOS	Equivalent value
A	1
B	2
C	3
D	4
E	5
F	6

Table 7. Significance of Maintenance of Traffic	
Change in the LOS (i.e., diff. in equivalent values) before & during construction	Ordinal scale rating
0	1
1	5
2	9
3	9
4	9
5	9

Figure 22. Significance of MOT classification based on LOS definition

Further, the default *peak hour factor* for various roadway functional classes is assigned using information from HCM (2000); these *peak hour factors* are used in current LOS calculation in the decision-making model and can be modified.

5) Bridge spans classification: This includes the classification of *number of similar spans* for the proposed bridge configuration. The classification is based on the methodology that if there are several similar spans then prefabrication is preferable; thus a higher number of similar spans indicate preference for ABC.

6) The "model presets" include tables that define the relationships among the project data, ordinal scale ratings, and the AHP pair-wise comparison ratings. These tables cannot be modified and are displayed for information only. The example tables for the "model presets" are shown in Figure 23.

Table 12. Ordinal Scale Rating for Major-Parameter		
Ratio of *SUM* to *RANGE* $\frac{\sum_i^n r}{9 \times n}$ where: i = sub-parameter under a major-parameter, n = no. of sub-parameters under a major-parameter, and r = ordinal scale rating of corresponding sub-parameter.		Ordinal scale rating
0.0	<0.2	1
0.2	<0.3	2
0.3	<0.4	3
0.4	<0.5	4
0.5	<0.6	5
0.6	<0.7	6
0.7	<0.8	7
0.8	<0.9	8
0.9	≤1.0	9

Table 13. Pair-Wise Comparison Rating for *Cost* Sub-Parameters		
Percentage differential of cost $\frac{\lvert C_{ABC} - C_{CC} \rvert}{Max(C_{ABC}, C_{CC})}\%$ where: C_{ABC} = dollar amount of the "cost" sub-parameter for ABC, and C_{CC} = dollar amount of the "cost" sub-parameter for CC.		Pair-wise comparison rating
0	<12	1
12	<23	2
23	<34	3
34	<45	4
45	<56	5
56	<67	6
67	<78	7
78	<89	8
89	≤99	9

Figure 23. Example tables for the model presets

4.1.2 Site-Specific Data

The ***Site-Specific Data*** command button on the ***Advanced User Menu*** opens the *Site-Specific Data* datasheet for a selected *Project Category.* The site-specific datasheet for a *Highway over Highway* project is shown in Figure 24. Data entry to the datasheet is by the *dropdown menus*.

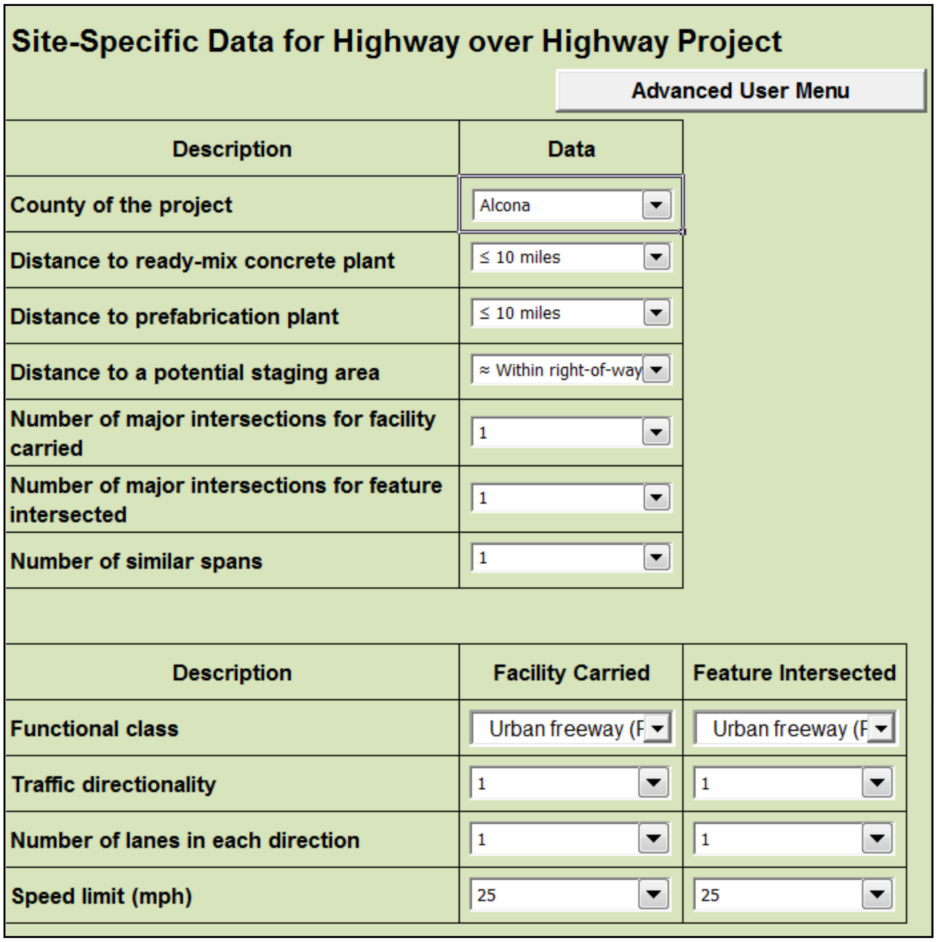


Figure 24. Site-specific datasheet for *Highway over Highway* project

4.1.3 Traffic Data

The ***Traffic Data*** command button on the ***Advanced User Menu*** opens the *Traffic Data* datasheet. For example, the datasheet for a *Highway over Highway* project is shown in Figure 25. The datasheet contains *data input fields* and *dropdown menus* to enter the required traffic data. A traffic study is required before completing the datasheet. The recommendations for obtaining the data required for this datasheet are as follows:

1) The *average queue length on feature intersected due to work zone* (Figure 25 – 1st parameter) and its *duration* (Figure 25 – 2nd parameter) need to be calculated or estimated for the proposed lane closures in the work zone.
2) The *detour length, detour route speed limit, work zone length,* and *work zone speed limit* need to be determined following the corridor planning process is completed.
3) The *recent ADT and ADTT* is available from bridge inventory database, Pontis.

4) The *Level of Service* (*LOS) before and during construction* on the affected routes and intersections can be calculated using traffic simulation or estimated.

Traffic Data for Highway over Highway Project

Advanced User Menu

Description	Data
Average queue length on feature intersected due to work zone (mi)	0.00
Duration of queue on feature intersected due to work zone (hr/day)	0.00
Detour length (mi)	0.00
Detour route speed limit (mph)	25

Description	Facility Carried	Feature Intersected
Recent ADT	0	0
Recent ADTT (% of ADT)	0	0
Work zone length (mi)	0.00	0.00
Work zone speed limit (mph)	25	25
LOS during construction	A	A

Description	Before construction	During construction
LOS on detour route	A	A
LOS on nearby major intersection-1 due to traffic on facility carried	A	A
LOS on nearby major intersection-2 due to traffic on facility carried	N/A	N/A
LOS on nearby major intersection-1 due to traffic on feature intersected	A	A
LOS on nearby major intersection-2 due to traffic on feature intersected	N/A	N/A

Figure 25. *Highway over Highway* project traffic data

4.1.4 Life-Cycle Cost Data

The ***Life-Cycle Cost Data*** command button on the ***Advanced User Menu*** opens the ***Life-Cycle Cost Data*** datasheet (Figure 26). The data in the sheet is independent of the *Project Category* because the data is related to the construction alternatives. The project manager(s) needs to identify the cost estimates for ABC technology prior to entering data in this datasheet.

Life-Cycle Cost Data — Advanced User Menu

Description	Data
Number of years for life-cycle cost analysis	35
Discount factor (%) *Note: A high discount factor will make the life-cycle cost less important than a low discount factor, and vice-versa. Generally, a discount factor around **3%** to **5%** is considered reasonable with average close to **4%** (FHWA 1998; Thoft-Christensen 2009).*	2%

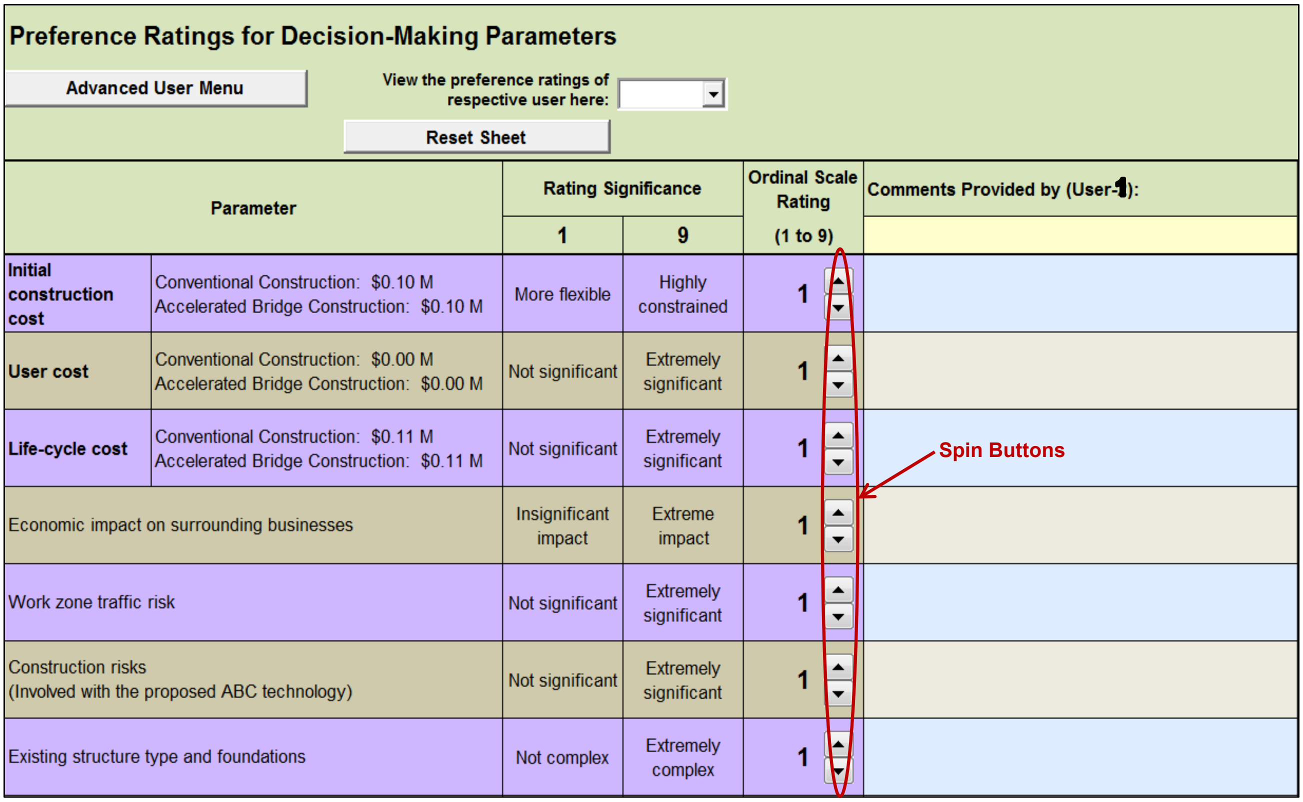
Description	Conventional Construction (CC)	Accelerated Bridge Construction (ABC)
Construction duration (days)	1	1
Initial construction cost ($)	$99,999	$99,999
Cost per each maintenance/repair activity ($)	$1,000	$1,000
Average duration between the maintenance/repair activities (year)	2	2
Disposal cost or salvage value ($) *Note: At the end of life-cycle cost analysis period, if the structure has either a residual life or a slavage value, the input amount should be negative.*	$0	$0

Figure 26. Life-cycle cost data

Note: The data for *Initial construction cost*, *Cost per each maintenance/repair activity*, *Average duration between the maintenance/repair activities*, and *Disposal cost or salvage value*, is available for conventional construction (CC). The access to information related to the ABC is limited. Estimates based on information from the literature are needed. For example, Bonstedt (2010) and Issa et al. (1995) provide information on the full-depth deck panel system. For more information, refer to Attanayake et al. (2012).

4.1.5 Preference Ratings

The ***Preference Ratings*** command button on the ***Advanced User Menu*** or on the ***Basic User Menu*** is used to access the datasheet to assign qualitative judgments in a form of preference ratings on an ordinal scale of 1 to 9 using the *spin buttons* (Figure 27).



Preference Ratings for Decision-Making Parameters

Advanced User Menu | View the preference ratings of respective user here: | Reset Sheet

Parameter		Rating Significance 1	Rating Significance 9	Ordinal Scale Rating (1 to 9)	Comments Provided by (User-1):
Initial construction cost	Conventional Construction: $0.10 M Accelerated Bridge Construction: $0.10 M	More flexible	Highly constrained	1	
User cost	Conventional Construction: $0.00 M Accelerated Bridge Construction: $0.00 M	Not significant	Extremely significant	1	
Life-cycle cost	Conventional Construction: $0.11 M Accelerated Bridge Construction: $0.11 M	Not significant	Extremely significant	1	
Economic impact on surrounding businesses		Insignificant impact	Extreme impact	1	
Work zone traffic risk		Not significant	Extremely significant	1	
Construction risks (Involved with the proposed ABC technology)		Not significant	Extremely significant	1	
Existing structure type and foundations		Not complex	Extremely complex	1	

Figure 27. Datasheet to assign preference ratings

Note:

1) For a particular project, after completing the datasheets for ***Project Details, Site-Specific Data***, ***Traffic Data***, and ***Life-Cycle Cost Data***, the *Advanced User* can also assign preference ratings, perform analysis, and view results. For basic user data entry, *Advanced User* needs to ***Logout*** and forward the saved file to basic users for their entry of preference ratings. The maximum number of basic users is limited to 10. The basic users can also provide comments or any additional information to explain their respective preference ratings.
2) The subsequent *Basic Users* can view the comments provided by the previous users; however, the ratings assigned by the previous user is not visible.
3) *Advanced User* can view all ratings and comments entered by basic users.

The ***Advanced User*** has access to the following from the *preference ratings* datasheet:

1) The ***Advanced User Menu*** command button for reopening the menu,
2) A ***Reset Sheet*** command button for restoring the datasheet to its default state as shown in Figure 27 (i.e., to restore all the preference ratings to 1, delete all comments, and open the ratings and blank comment fields for ***User-1***), and

3) A ***Dropdown Menu* to** view the ratings and comments of any user as shown in Figure 28.

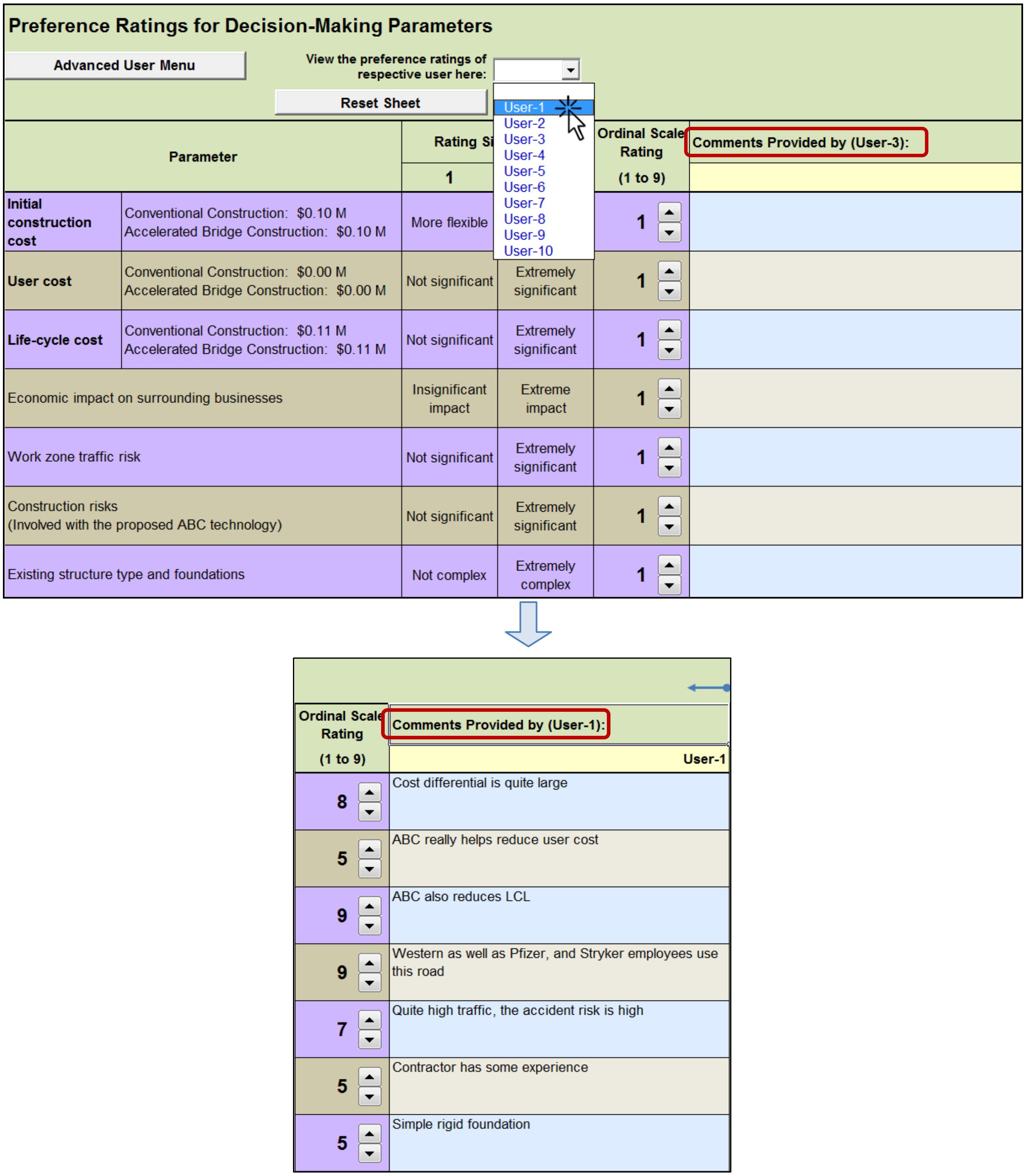
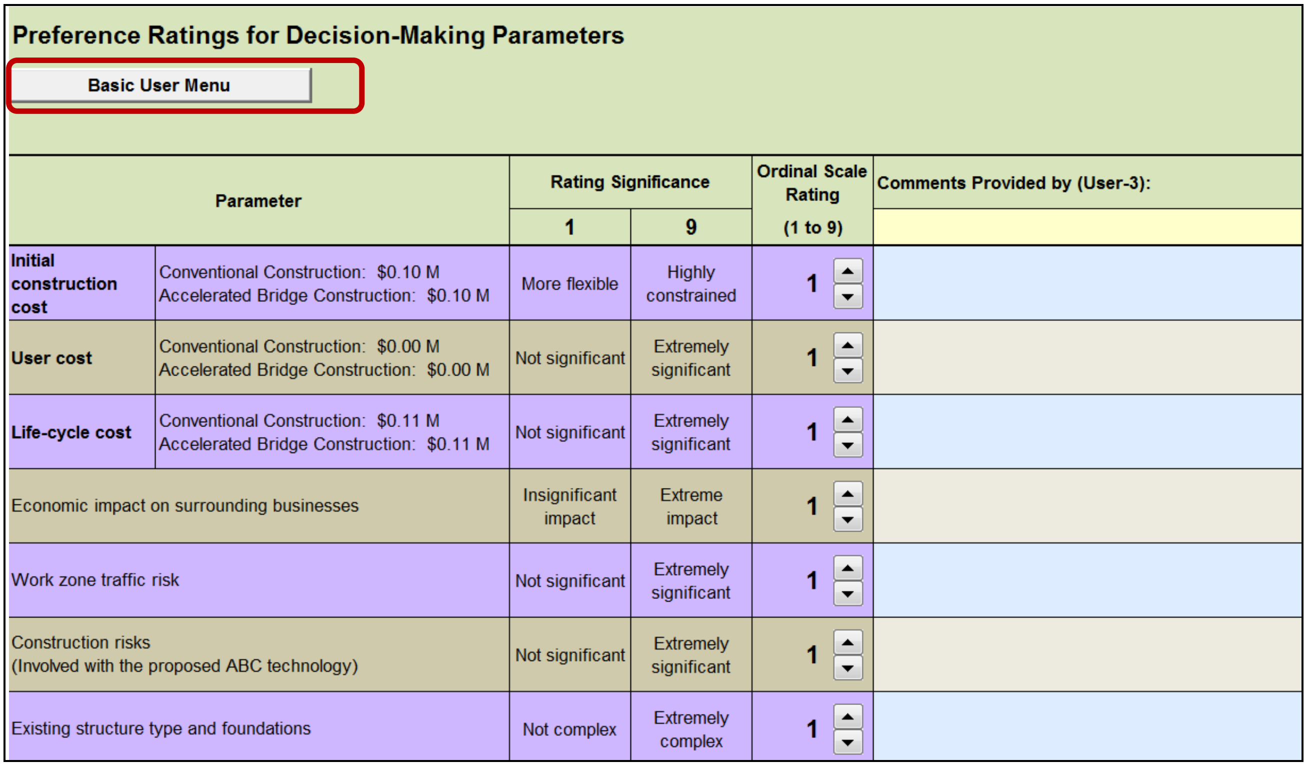


Figure 28. Use of *dropdown* menu on preference ratings datasheet

The ***Basic User*** can access only the command button for reopening ***Basic User Menu*** as shown in Figure 29.



Preference Ratings for Decision-Making Parameters

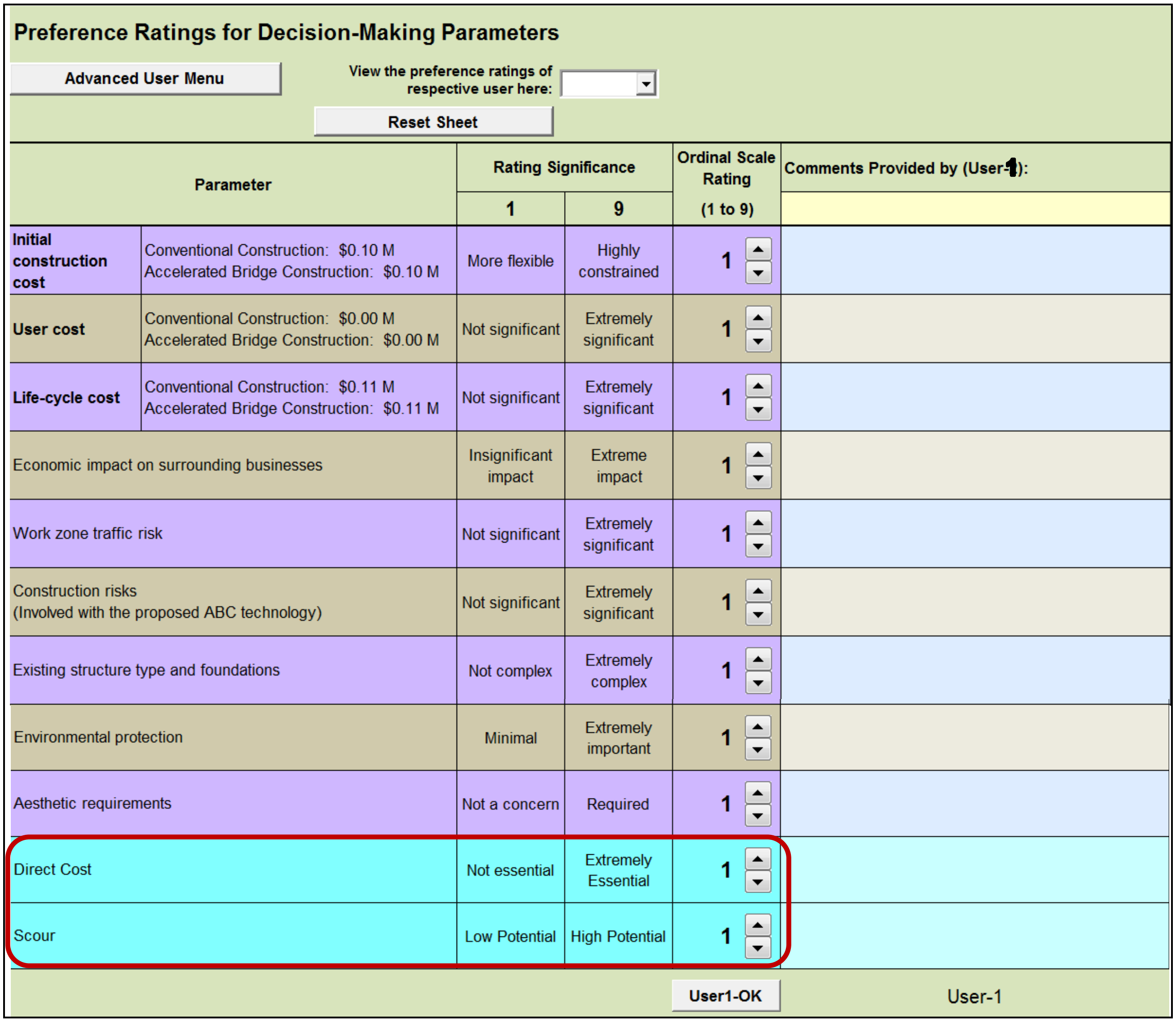
Basic User Menu

Parameter		Rating Significance 1	Rating Significance 9	Ordinal Scale Rating (1 to 9)	Comments Provided by (User-3):
Initial construction cost	Conventional Construction: $0.10 M Accelerated Bridge Construction: $0.10 M	More flexible	Highly constrained	1	
User cost	Conventional Construction: $0.00 M Accelerated Bridge Construction: $0.00 M	Not significant	Extremely significant	1	
Life-cycle cost	Conventional Construction: $0.11 M Accelerated Bridge Construction: $0.11 M	Not significant	Extremely significant	1	
Economic impact on surrounding businesses		Insignificant impact	Extreme impact	1	
Work zone traffic risk		Not significant	Extremely significant	1	
Construction risks (Involved with the proposed ABC technology)		Not significant	Extremely significant	1	
Existing structure type and foundations		Not complex	Extremely complex	1	

Figure 29. Preference ratings datasheet when accessed using *Basic User Menu*

<u>Note:</u>

1) The additional sub-parameters that are added using the ***Add/Remove Sub-Parameters*** menu are automatically added to the sub-parameters list in this datasheet. Further, their ordinal scale preference ratings that are defined in the ***Add/Remove Sub-Parameters*** menu will be listed under the *"Rating Significance"* column in the *preference ratings* datasheet (see Figure 30 bottom two rows, shaded with light Aqua color).
2) In order for a basic user to run the MiABCD, the preceding basic user needs to ***Logout*** following the entry of preference ratings, performing analysis (see Section 4.2), and viewing the results (see Section 4.3).



Preference Ratings for Decision-Making Parameters

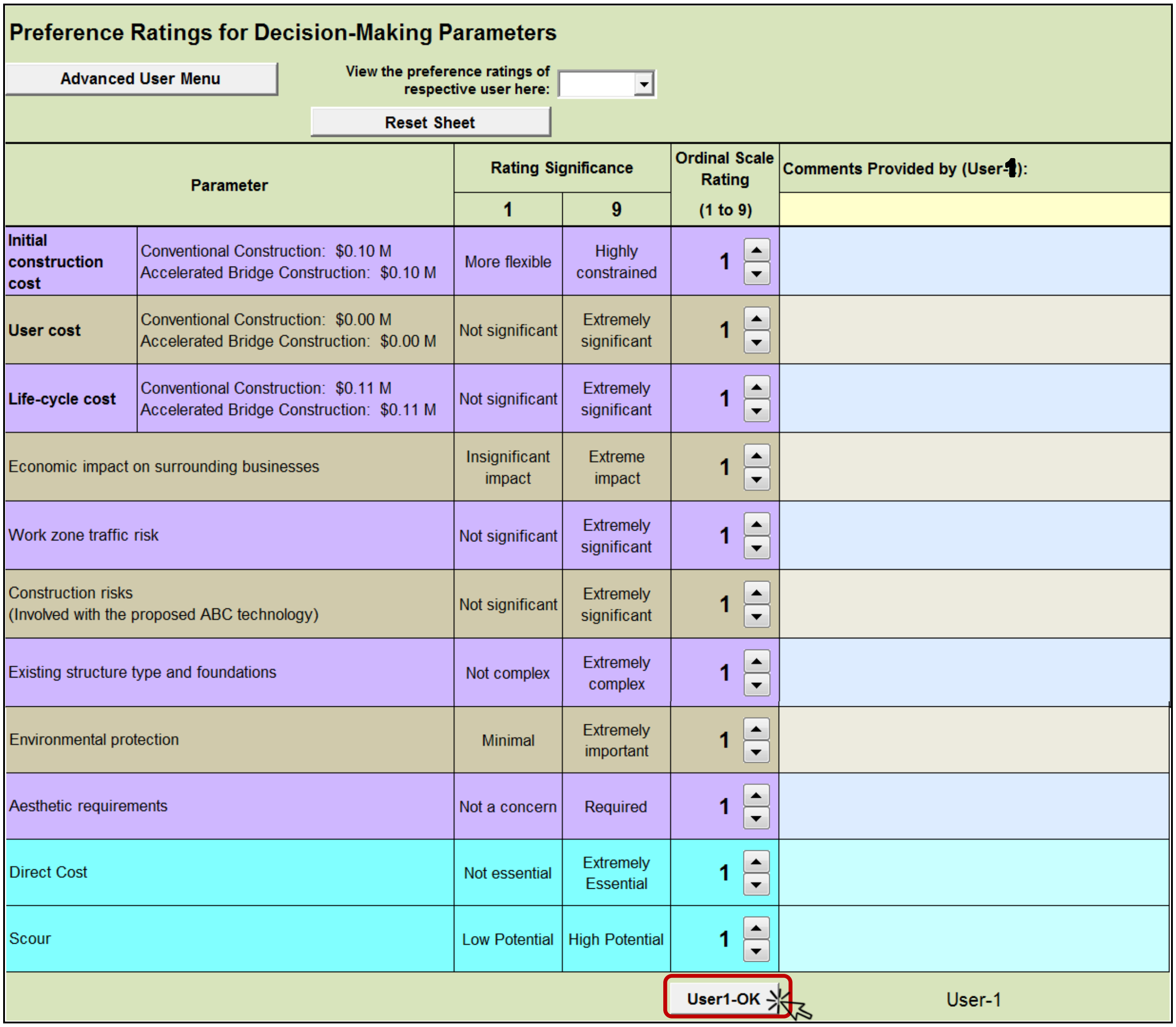
Advanced User Menu | View the preference ratings of respective user here: | Reset Sheet

Parameter		Rating Significance 1	Rating Significance 9	Ordinal Scale Rating (1 to 9)	Comments Provided by (User1):
Initial construction cost	Conventional Construction: $0.10 M Accelerated Bridge Construction: $0.10 M	More flexible	Highly constrained	1	
User cost	Conventional Construction: $0.00 M Accelerated Bridge Construction: $0.00 M	Not significant	Extremely significant	1	
Life-cycle cost	Conventional Construction: $0.11 M Accelerated Bridge Construction: $0.11 M	Not significant	Extremely significant	1	
Economic impact on surrounding businesses		Insignificant impact	Extreme impact	1	
Work zone traffic risk		Not significant	Extremely significant	1	
Construction risks (Involved with the proposed ABC technology)		Not significant	Extremely significant	1	
Existing structure type and foundations		Not complex	Extremely complex	1	
Environmental protection		Minimal	Extremely important	1	
Aesthetic requirements		Not a concern	Required	1	
Direct Cost		Not essential	Extremely Essential	1	
Scour		Low Potential	High Potential	1	
				User1-OK	User-1

Figure 30. Preference ratings datasheet with two sub-parameters added by the *Advanced User*

4.2 AHP ANALYSIS

To perform data analysis, the ***User'X'-OK*** command button on the *preference ratings* datasheet (Figure 31) needs to be used. Here, ***'X'*** can be any number between **1** and **10** to represent the number of users who have entered *preference ratings*.



Preference Ratings for Decision-Making Parameters

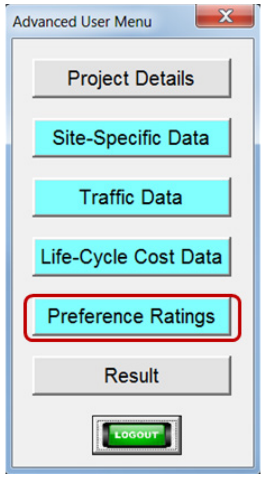
Advanced User Menu | View the preference ratings of respective user here: | Reset Sheet

Parameter		Rating Significance 1	Rating Significance 9	Ordinal Scale Rating (1 to 9)	Comments Provided by (User-1):
Initial construction cost	Conventional Construction: $0.10 M Accelerated Bridge Construction: $0.10 M	More flexible	Highly constrained	1	
User cost	Conventional Construction: $0.00 M Accelerated Bridge Construction: $0.00 M	Not significant	Extremely significant	1	
Life-cycle cost	Conventional Construction: $0.11 M Accelerated Bridge Construction: $0.11 M	Not significant	Extremely significant	1	
Economic impact on surrounding businesses		Insignificant impact	Extreme impact	1	
Work zone traffic risk		Not significant	Extremely significant	1	
Construction risks (Involved with the proposed ABC technology)		Not significant	Extremely significant	1	
Existing structure type and foundations		Not complex	Extremely complex	1	
Environmental protection		Minimal	Extremely important	1	
Aesthetic requirements		Not a concern	Required	1	
Direct Cost		Not essential	Extremely Essential	1	
Scour		Low Potential	High Potential	1	

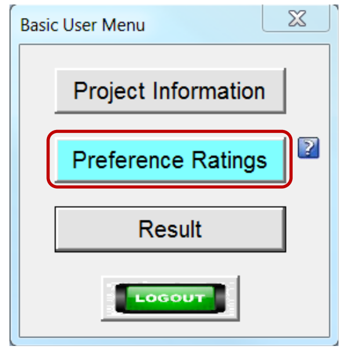
User1-OK | User-1

Figure 31. Preference ratings datasheet showing the analysis command button

The ***Preference Ratings*** command button on the ***Advanced User Menu*** or the ***Basic User Menu*** (Figure 32) is used to access the analysis command button**.**



(a)



(b)

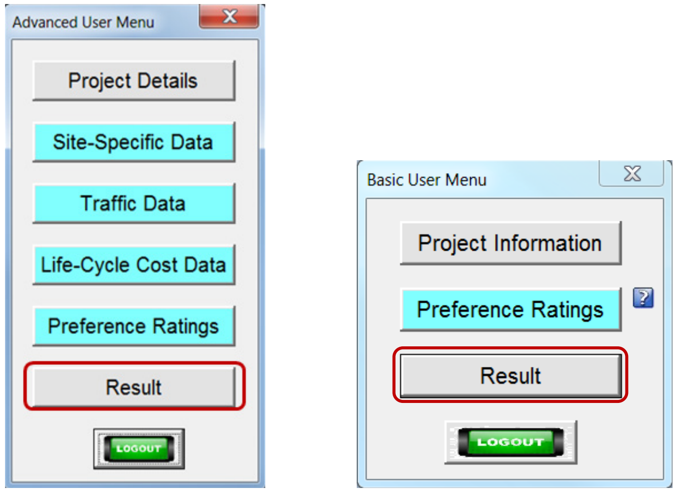
Figure 32. ***Preference Ratings*** **button to access the data analysis command button (a)** ***Advanced User Menu*** **and (b)** ***Basic User Menu***

<u>Note:</u>

1) Once the analysis is completed, the respective user can use the ***Results*** command button on respective ***User Menu*** (i.e., either ***Advanced User Menu*** or ***Basic User Menu***) to view the analysis results.
2) The user needs to select the ***Logout*** command button from the respective ***User Menu*** and then select the ***Exit*** button to close the program.
3) The user can then send the saved project file with the most recent data to the next user to ***Login*** and assign their preference ratings, run analysis, and view results. For the next user, the subsequent *"Ordinal Scale Rating"* column and *"Comments"* field will be displayed, along with ***User'Y'-OK*** command button at bottom of the table (where **'Y' = X+1,** but is limited to 10).

4.3 ANALYSIS RESULTS

The ***Result*** command button from the ***Menu*** (i.e., either ***Advanced User Menu*** or ***Basic User Menu***) (Figure 33) is used to view the results of the ABC vs. CC evaluation.



(a) (b)

Figure 33. *Result* command button to access analysis results (a) *Advance User Menu* and (b) *Basic User Manu*

The results are presented in four formats as follows:

1) Two pie charts showing the *Upper Bound* and *Lower Bound construction alternative preferences* for ABC and CC (Figure 34 – top left). The pie charts show the range calculated from the ratings assigned by multiple users.
2) A chart showing the distribution of *Major-Parameter Preferences from Multiple Users* (Figure 34 – top right). The normalized preferences for each user are plotted using different *lines and colors*.
3) A chart showing the distribution of *Construction Alternative Preferences from Multiple Users* (Figure 34 – bottom). This chart displays the results for each user in *bar* charts separately for ABC and CC. The *bar* is formed with six different colors) representing the contribution from six major-parameters: (i) Site and Structure Considerations (S&ST), (ii) Cost, (iii) Work Zone Mobility (WZM), (iv) Technical Feasibility and Risk (TF&R), (v) Environmental Considerations (EC), and (vi) Seasonal Constraints and Project Schedule (SC&PS).
4) A table showing the contribution (in percentage) of each major-parameter towards the *Overall Preference* for ABC and CC (Figure 35).

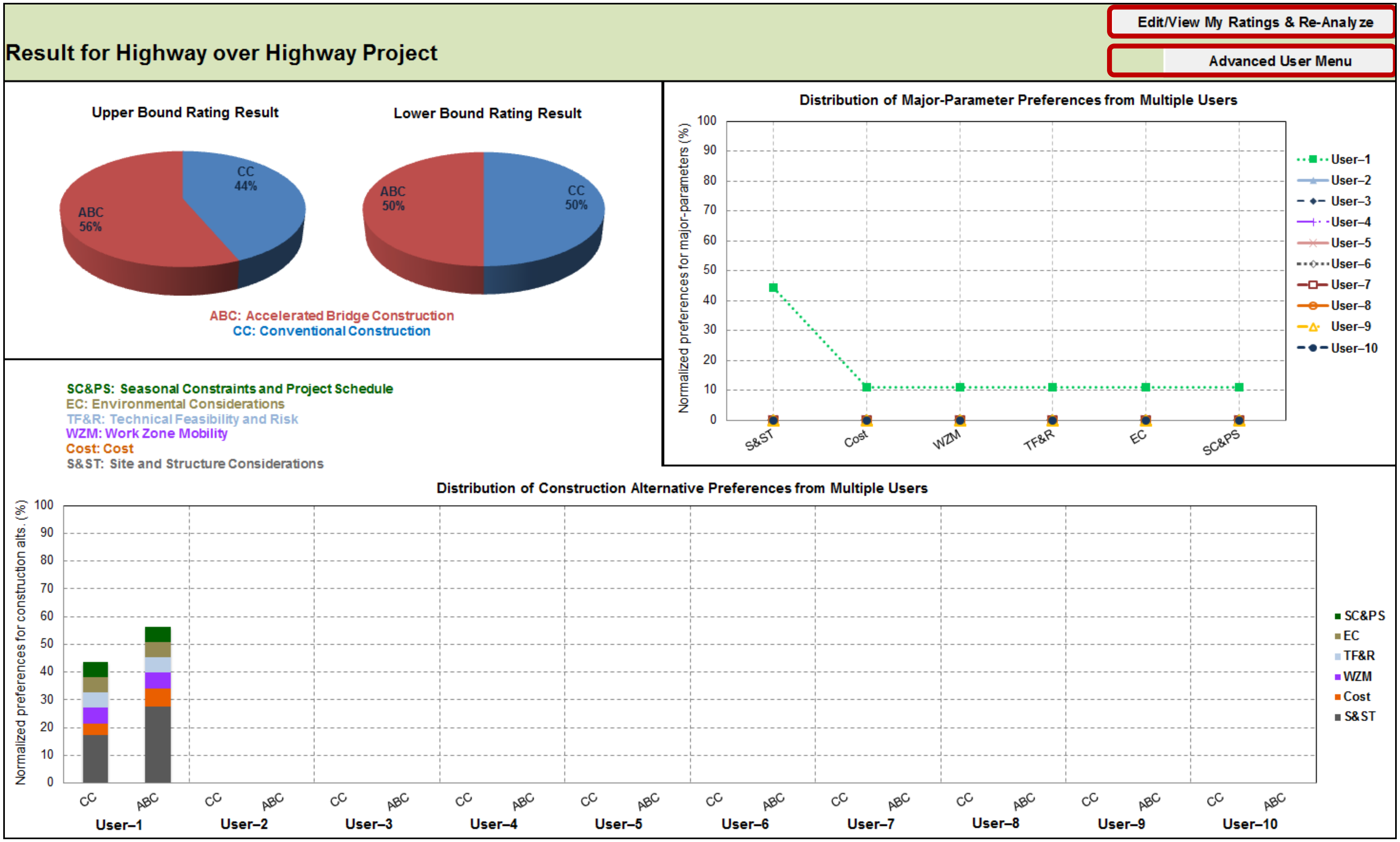


Figure 34. Analysis results presented as charts

Users or Decision Makers	Construction Alternatives	Site and Structure Considerations (S&ST) (%)	Cost (%)	Work Zone Mobility (WZM) (%)	Technical Feasibility and Risk (TF&R) (%)	Environmental Considerations (EC) (%)	Seasonal Constraints and Project Schedule (SC&PS) (%)	Overall Preference (%)
User-1	CC	17	4	6	6	6	6	44
	ABC	27	7	6	6	6	6	56
User-2	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-3	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-4	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-5	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-6	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-7	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-8	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-9	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0
User-10	CC	0	0	0	0	0	0	0
	ABC	0	0	0	0	0	0	0

Edit/View My Ratings & Re-Analyze

Advanced User Menu

Figure 35. Analysis results presented in tabular format

Note:

1) The ***Edit/View My Ratings & Re-Analyze*** command button available with the results (Figure 34 and Figure 35) will direct the user back to his/her own preference ratings, so that the preference ratings can be revised. The user, following the revision of their preference ratings, needs to *run the analysis* before viewing the results with updated ratings.
2) ***Advanced User Menu*** *or* ***Basic User Menu*** command button is available with the analysis results based on the user type. As an example, Figure 34 and Figure 35 show the ***Advanced User Menu*** button.
3) The ***Advanced User*** can change the data in any of the datasheets and rerun the analysis.

5 REFERENCES

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APPENDIX – EA

DATA REQUIRED FOR RUNNING MIABCD

Description	Data
General Data	
Wage rate of drivers	Personal wage = $16.02 /hr, Commercial wage = $26.70 /hr *(Source: FHWA – Pavement division interim technical bulletin (Walls and Smith 1998)) (Prorated to 2012 dollar amount)*
Vehicle operating cost	Personal vehicle = $6.88 /hr, Commercial wage = $14.15 /hr *(Source: FHWA – Pavement division interim technical bulletin (Walls and Smith 1998)) (Prorated to 2012 dollar amount)*
Accident cost	$ 1094 /accident *(Source: FHWA – Pavement division interim technical bulletin (Walls and Smith 1998)) (Prorated to 2012 dollar amount)*
Accident rate	Normal = 215 accidents per million vehicle mile During work zone = 240 accidents per million vehicle miles *(Michigan State Police CJIC <http://www.michigan.gov/msp/0,4643,7-123-1593_24055---,00.html>)*
Site-Specific Data	
County jobs multiplier	*(Source: Montgomery consulting, Inc., Michigan economic developers association <https://montgomeryconsultinginc.com/Resources.html>)*
Distance to prefabrication plant	*(Source: Available suppliers in an area)*
Distance to ready-mix concrete plant	*(Source: Available suppliers in an area)*
Distance to a potential staging area	*(Source: Project details)*
Number of similar spans of the new bridge	*(Source: Bridge configuration)*
Traffic Data	
Speed limit for facility carried, Feature intersected, detour route	*(Source: MDOT - Traffic Monitoring Information System (TMIS))*
Traffic directionality for facility carried, Feature intersected, detour route	*(Source: Project details)*
Functional class for facility carried, Feature intersected, detour route	*(Source: Project details/Pontis database)*

Number of lanes in each direction for facility carried, Feature intersected, detour route	*(Source: Project details)*
ADT/ADTT for facility carried, Feature intersected, detour route	*(Source: MDOT - TMIS)*
Detour length	*(Source: MDOT – TMIS/Pontis database)*
Work zone length on facility carried and feature intersected	*(Source: Traffic study)*
Approximate queue length on feature intersected due to possible intermittent closures/lane closures	*(Source: Traffic study)*
Approximate duration for the queue occurrence	*(Source: Traffic study)*
LOS, before construction, on major intersection(s) near the facility carried	*(Source: Traffic study)*
LOS, during construction, on major intersection(s) near the facility carried	*(Source: Traffic study)*
LOS, before construction, on major intersection(s) near the feature intersected	*(Source: Traffic study)*
LOS, during construction, on major intersection(s) near the feature intersected	*(Source: Traffic study)*
Life-Cycle Cost Data	
Number of years for life-cycle cost analysis	*(Source: Based on the project manager's discretion)*
Initial construction cost of each construction alternative	*(Source: Estimate based on the comparison with similar bridge projects)*
Cost per each maintenance/repair activity for each construction alternative	*(Source: Estimate based on the comparison with similar bridge projects or from available literature)*
Average duration between the maintenance/repair activities for each construction alternative	*(Source: Estimate based on the comparison with similar bridge projects or from available literature)*
Disposal/Salvage cost of each construction alternative	*(Source: Estimate based on the comparison with similar bridge projects or from available literature)*
Construction duration	*(Source: Estimate based on the comparison with similar bridge projects)*

Preference Ratings (Qualitative Data)	
<u>Note:</u> *This data can be obtained from multiple users/experts (up to 10 users/experts). Further, the parameters that are specified here are the default qualitative parameters for a project. Additional qualitative parameters may be present for a project based its requirements and/or project manager's discretion.*	
Initial construction cost (Based on the values of CC and ABC, provided by MiABCD)	Flexibility rating (1 to 9):
User cost (Based on the values of CC and ABC, provided by MiABCD)	Significance rating (1 to 9):
Life-cycle cost (Based on the values of CC and ABC, provided by MiABCD)	Significance rating (1 to 9):
Economic impact on surrounding businesses	Impact rating (1 to 9):
Work zone traffic risk	Significance rating (1 to 9):
Construction risks (involved with the proposed ABC technology)	Significance rating (1 to 9):
Existing structure type and foundations	Complexity rating (1 to 9):
Terrain to traverse	Difficulty rating (1 to 9):
Access and mobility of construction equipment	Difficulty rating (1 to 9):
Contractor experience (Required for the proposed ABC technology)	Experience rating (1 to 9):
Manufacturer/Precast plant experience (Required for the proposed ABC technology)	Experience rating (1 to 9):
Seasonal limitations	Significance rating (1 to 9):
Stakeholder(s') limitation	Significance rating (1 to 9):
Environmental protection	Importance rating (1 to 9):
Aesthetic requirements	Importance rating (1 to 9):

APPENDIX – EB

EXAMPLE DECISION-MAKING EVALUATION USING MIABCD

To demonstrate the decision-making model process, as an example, the Stadium Drive Bridge project in Kalamazoo, Michigan is utilized. The bridge carries Stadium Drive (I-94 BR) over US 131 in Kalamazoo County, Michigan. The authors were familiar with the project site and its details; thus, the decision-making model was implemented to evaluate among CC and ABC, as the construction alternatives.

For the Stadium Drive Bridge, the site-specific data, traffic data, and life-cycle cost data (shown in Table B1) were obtained from the project engineer and other resources. The life-cycle cost data, such as initial construction cost and construction duration for each construction alternative, is estimated based on comparison with similar bridge project reported in the literature and Parkview bridge data. The rehabilitation/repair cost and disposal/salvage cost for each construction alternative is estimated based on information from Bonstedt (2010) and Issa et al. (1995). The ABC disposal/salvage cost is negative because a remaining life of 25 years is expected at the end of life-cycle analysis period (i.e., 75 years Bonstedt (2010). The preference ratings were obtained from three users as shown in Table B2.

Table B1. Site-Specific Data, Traffic Data and Life-Cycle Cost Data for the Stadium Drive Bridge

Description	Data		
County jobs multiplier	Kalamazoo county: multiplier = 1.88 (Source: Montgomery consulting, Inc. 2011)		
Distance to prefabrication plant	~ 10 miles (Source: Available suppliers in an area)		
Distance to ready-mix concrete plant	~ 12 miles (Source: Available suppliers in an area)		
Distance to a potential staging area	1 mile (Source: Project engineer)		
Number of similar spans	2 (Source: Bridge configuration)		
Detour length	1.24 miles (Source: Pontis Database)		
Work zone length on feature intersected	<1 mile (Source: Traffic Study)		
Work zone speed limit on feature intersected	60 miles/hr (Source: Traffic Study)		
Average queue length and its duration, for single lane closure of feature intersected	0.75 to 1.5 miles, 4 hr/day (Source: Traffic Study)		
Impact on the nearby major intersection (M-43 & US-131) due to traffic on feature intersected	LOS before construction = A LOS during construction = C (Source: Traffic Study)		
Impact on the nearby major intersection-1 (Drake Rd & Stadium Drive) due to traffic on facility carried	LOS before construction = C LOS during construction = E (Source: Traffic Study)		
Impact on the nearby major intersection-2 (11th St and Stadium Drive) due to traffic on facility carried	LOS before construction = B LOS during construction = C (Source: Traffic Study)		
	Facility carried	**Feature Intersected**	**Detour route**
Speed limit	45 mph	70 mph	45 mph
Functional class	Urban freeway	Urban freeway	Major arterial
Traffic directionality and no. of lanes in each direction	2-way and 3 lanes	2-way and 3 lanes	2-way and 2 lanes
ADT, and ADTT as a percentage of ADT	41,774 and 3%	52,085 and 12%	40,000 and 3%
Number of years for life-cycle cost analysis	75 years		
Initial construction cost of each construction alternative	CIP construction = $ 6,000,000 ABC = $ 7,500,000		
Cost per each maintenance/repair activity and average duration between those activities for each construction alternative	CIP construction = $ 1,200,000 at every 15 years ABC = $ 1,500,000 at every 35 years		
Disposal/Salvage cost of each construction alternative	CIP construction = $ 600,000 ABC = – $ 750,000		
Construction duration	CIP construction = 152 days ABC = 60 days		

Table B2. Preference Ratings of Three Users

Sub-Parameter		Ordinal Scale Judgment Legend		Ordinal Scale Ratings		
		1 Rating	9 Rating	Expert-1	Expert-2	Expert-3
Initial construction cost	CC: $6.00 M ABC: $7.50 M	More flexible	Highly constrained	8	4	9
User cost	CC: $5.88 M ABC: $2.32 M	Not significant	Extremely significant	5	8	3
Life-cycle cost	CC: $15.65 M ABC: $8.61 M	Not significant	Extremely significant	9	9	7
Economic impact on surrounding businesses		Insignificant impact	Extreme impact	9	9	7
Work zone traffic risk		Not significant	Extremely significant	7	6	5
Construction risks (Involved with the proposed ABC technology)		Not significant	Extremely significant	5	4	3
Existing structure type and foundations		Not complex	Extremely complex	5	2	2
Terrain to traverse (e.g., Viaduct over rapids, deep water, a valley, or restricted access)		Not difficult	Extremely difficult	5	3	2
Access and mobility of construction equipment		Not difficult	Extremely difficult	5	2	2
Contractor experience (Required for the proposed ABC technology)		Limited experience	Experienced	6	2	5
Manufacturer/Precast plant experience (Required for the proposed ABC technology)		Limited experience	Experienced	3	2	5
Seasonal limitations		Not significant	Extremely significant	7	9	5
Stakeholder(s') limitation		Not significant	Extremely significant	7	9	7
Environmental protection		Minimal	Extremely important	3	4	2
Aesthetic requirements		Not a concern	Required	5	1	8

To allow the users to make informed decisions quantitative data in the form of *initial construction cost*, *user cost*, and *life-cycle cost* is provided. Using the data integration methodology of MiABCD (Aktan et al. 2013), the quantitative data (Table B1 and Table B2) is converted to ordinal scale. The pair-wise comparison matrices for the sub-parameters are developed from the ordinal data. As an example, the pair-wise comparison matrix for sub-parameters under *Site and Structure Considerations (S&ST)* is shown in Figure B1-a. Here, the matrix is formed based on the ordinal scale ratings provided by User-1. In that matrix, the lower triangular elements are reciprocals of the upper triangular elements.

To explain the development process of this matrix, consider the sub-parameters *Availability of staging area* and *Terrain to traverse*. In the "model presets" under the ***View/Edit General Data***, the *Site-Specific Numerical Data* is converted to ordinal data as shown in Table B3. Also, the ordinal scale ratings for the sub-parameters are converted to AHP pair-wise comparison ratings using Table B4.

Table B3. Assigning Ordinal Scale Ratings to Site-Specific Data

Ordinal scale rating	Ready-mix supplier proximity	Precaster proximity	Availability of staging area	Number of similar spans	ADT
9	≤ 10 miles	≤ 10 miles	≈ Within right-of-way	>4	100001 ≤ ADT < 300000
7	11–20 miles	11–20 miles	≤ 10 miles	4	50001 ≤ ADT < 100000
5	21–40 miles	21–40 miles	11–20 miles	3	20001 ≤ ADT < 50000
3	> 40 miles	41–60 miles	21–40 miles	2	5001 ≤ ADT < 20000
1	Procuring time > 90 min	> 60 miles	≥ 40 miles	1	1 ≤ ADT < 5000

Table B4. Converting Ordinal Scale Rating to Pair-Wise Comparison Rating

Differential between the ordinal scale ratings (Large – Small)	Pair-wise comparison rating to be assigned to parameter with Large ordinal scale rating
0	1
1	2
2	3
3	4
4	5
5	6
6	7
7	8
8	9

From the site-specific data, the *Availability of staging area* will receive an ordinal scale rating of 7 (refer Table B1 row-4 and Table B3 row-2), and *Terrain to traverse* was assigned an ordinal rating of 5 by User-1. Using Table B4, a pair-wise comparison rating of 3 is assigned to *Availability of staging area* and the reciprocal rating 1/3 is assigned to *Terrain to traverse*. In a similar manner, the pair-wise comparison matrix is compiled for *S&ST*. Similarly, the matrices for sub-parameters under *Cost, Work Zone Mobility (WZM), Technical Feasibility and Risk (TF&R), Environmental Considerations (EC), and Seasonal Constraints and Project Schedule (SC&PS)* are developed. These matrices represent the first set of pair-wise comparison matrices of the AHP.

The second set of AHP matrices are the sub-parameter pair-wise comparison matrices of construction alternatives. These are developed using the ordinal scale ratings of each sub-parameter and considering the relation between sub-parameters and construction alternatives as shown Table B5. As an example, the pair-wise comparison matrix for construction alternatives for *Terrain to traverse* is shown in Figure B1-b. Here, the sub-parameter with increased preference favors CC (Table B5); therefore, the ordinal scale rating of 5 (i.e., assigned by User-1) is the pair-wise comparison rating for CC, and the reciprocal rating 1/5 is for ABC. In a similar fashion, the construction alternative pair-wise comparison matrices are developed for the remainder of the sub-parameters.

	Ready-mix supplier proximity	Precaster proximity	Availability of staging area	Existing structure type and foundations	Terrain to traverse	Access and mobility of construction equipment	Number of similar spans
Ready-mix supplier proximity	1	$\frac{1}{3}$	1	3	3	3	5
Precaster proximity	3	1	3	5	5	5	7
Availability of staging area	1	$\frac{1}{3}$	1	3	3	3	5
Existing structure type and foundations	$\frac{1}{3}$	$\frac{1}{5}$	$\frac{1}{3}$	1	1	1	3
Terrain to traverse	$\frac{1}{3}$	$\frac{1}{5}$	$\frac{1}{3}$	1	1	1	3
Access and mobility of construction equipment	$\frac{1}{3}$	$\frac{1}{5}$	$\frac{1}{3}$	1	1	1	3
Number of similar spans	$\frac{1}{5}$	$\frac{1}{7}$	$\frac{1}{5}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	1

Eigenva analys (EVA →

Normalized Preference Ratings
0.182
0.389
0.182
0.072
0.072
0.072
0.033

(a)

	CC	ABC
CC	1	5
ABC	$\frac{1}{5}$	1

EV. →

Normalized Preference Ratings
0.833
0.167

(b)

	S&ST	Cost	WZM	TF&R	EC	SC&PS
Site and Structure Considerations (S&ST)	1	$\frac{1}{3}$	2	2	3	$\frac{1}{2}$
Cost	3	1	4	4	5	2
Work Zone Mobility (WZM)	$\frac{1}{2}$	$\frac{1}{4}$	1	1	2	$\frac{1}{3}$
Technical Feasibility and Risk (TF&R)	$\frac{1}{2}$	$\frac{1}{4}$	1	1	2	$\frac{1}{3}$
Environmental Considerations (EC)	$\frac{1}{3}$	$\frac{1}{5}$	$\frac{1}{2}$	$\frac{1}{2}$	1	$\frac{1}{4}$
Seasonal Constraints and Project Schedule (SC&PS)	2	$\frac{1}{2}$	3	3	4	1

EVA →

Normalized Preference Ratings
0.150
0.377
0.088
0.088
0.054
0.243

(c)

Figure B1. Pair-wise comparison matrix for (a) sub-parameters under *S&ST*, (b) construction alternatives with respect to *Terrain to traverse* sub-parameter, and (c) major-parameters

Table B5. Sub-Parameters Grouping Based on Their Characteristics

Sub-Parameters that support ABC with increased Preference Rating		**Sub-Parameters that support CC with increased Preference Rating**
Precaster proximity	Significance of level of service on detour route	Ready-mix supplier proximity
Availability of staging area	Impact on nearby major intersection due to traffic on facility carried	Existing structure type and foundation
Number of similar spans	Impact on nearby major intersection due to traffic on feature intersected	Terrain to traverse
Economic impact on surrounding communities	Work zone traffic risk	Access and mobility of construction equipment
Economic impact on surrounding businesses	Environmental protection	Contractor experience
Significance of maintenance of traffic on facility carried	Seasonal limitations	Manufacturer/ Precast plant experience
Significance of maintenance of traffic on feature intersected	Stakeholder(s') limitations	Construction risks
Length of detour		Aesthetic requirements

Note: The sub-parameters related to *Cost* are assigned preference for the *alternative with least value*.

The pair-wise comparison matrices for the major-parameters represent the third set of AHP matrices. As 6 major-parameters are associated with the decision making, one 6×6 matrix will be developed. The pair-wise comparison matrix for major-parameters developed from the preferences of User-1 is presented in Figure B1-c.

Eigenvalue analysis is performed on these three sets of matrices. Following the Eigenvalue analysis principal Eigenvectors for the three sets of matrices are determined. The principal Eigenvectors are normalized to represent the normalized preference ratings of the variables in the respective matrices as shown in Figure B1-a, b, and c. The normalized preference ratings from the three users are processed independently through AHP to independently obtain respective normalized preferences for the CC and ABC (Figure B2-c).

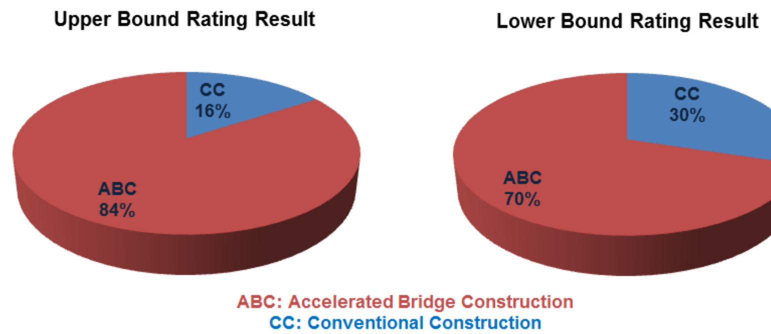
MiABCD Stadium Bridge Analysis Results

The Stadium Drive bridge project data was evaluated using the decision-making model, and the results were obtained in formats shown in Figure B2. Figure B2-a provides results on the upper and lower bound preferences between ABC and CC. Figure B2-a shows that, for the Stadium Drive bridge project, ABC preference of the three users is 84% at the upper bound, and it is 70% at the lower bound.

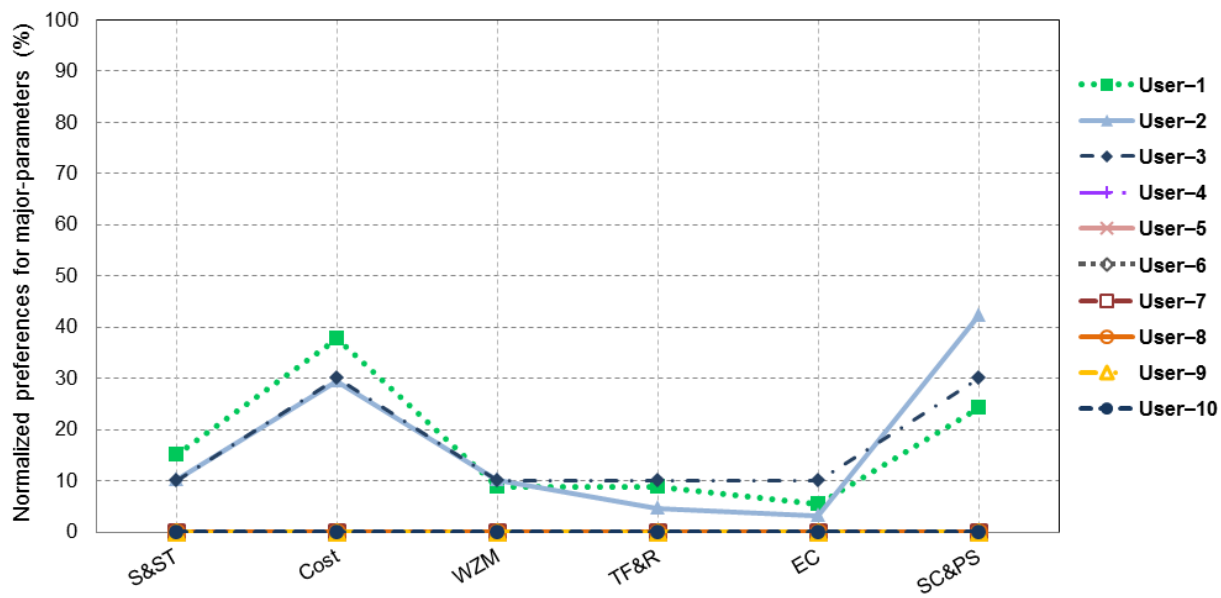
Figure B2-b provides statistics of the *normalized preferences for major-Parameters* from three users. This information is helpful in observing the variability of the user ratings.

Figure B2-c presents the *normalized preferences for the construction alternatives* of the three users. The values are calculated by integrating the normalized preference ratings from the three sets of matrices developed in the AHP. Figure B2-c also shows the contribution of the *construction alternative normalized preference* for each major-parameter. This helps in identifying the contribution of each major-parameter and their underlying sub-parameters towards respective construction alternative preference.

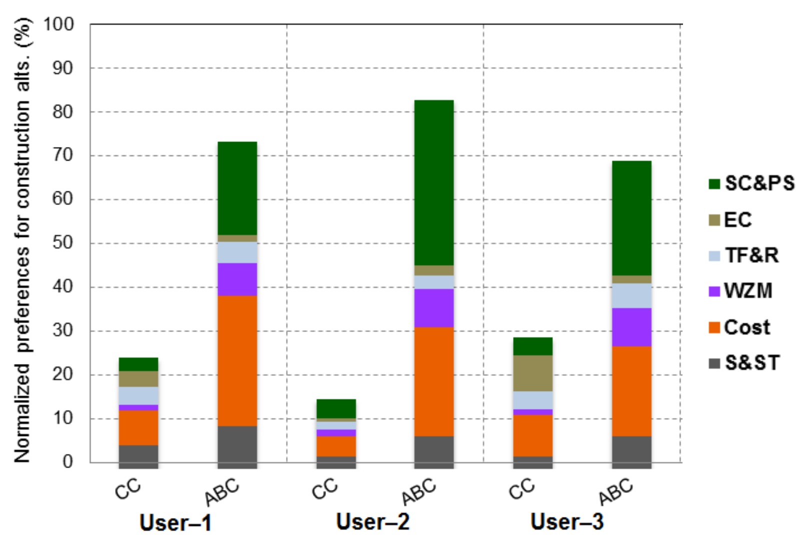
The results show that the preference for ABC is mostly dependent on the *Seasonal Constraints and Project Schedule (SC&PS)* parameter followed by the *Cost* parameter, for the Stadium Drive bridge project.



(a)



(b)



(c)

Figure B2. (a) Upper bound and lower bound rating results, (b) distribution of major-parameter preferences from multiple users, and (c) distribution of construction alternative preferences from multiple users

APPENDIX – EC

GLOSSARY

Term	**Description**
ABC	Accelerated Bridge Construction
ADT	Annual Average Daily Traffic
ADTT	Annual Average Daily Truck Traffic
Advanced User	This is generally the project manager(s) who is (are) familiar with the project details and quantitative data required by the decision-making model.
AHP	Analytical Hierarchy Process
Basic User	This is generally the user/expert that can provide judgment on qualitative parameters as preference ratings based on their bridge design and construction experience on recent projects.
CC	Conventional Construction
Check Box	This is used to declare an option *active* or *inactive* in an algorithm. Generally, an activated option will execute a subroutine.
Command Button	This is used to execute the embedded VBA script that may direct to a pop-up menu/specific datasheet, or may execute an algorithm.
Datasheet	An Excel worksheet that is customized for the MiABCD procedures
Data Input Field	A cell on an Excel worksheet that intakes numeric values in a specific range
Decision-Making Parameters	These include six major-parameters and all sub-parameters in comparing construction alternatives such as ABC vs. CC.
DOT	Department of Transportation
Dropdown Menu	This is used to select the desired option from the available options that are predefined.
Edit/View My Ratings & Re-Analyze Command Button	This is used to take the user back to his/her own preference ratings, in case he/she wants to revise their preference ratings..
Eigenvector	The eigenvector for a tabular set of variables provides the relative strength index among those variables
EC	Environmental Considerations
Facility carried	The bridge to be replaced
Feature intersected	The roadway under the bridge to be replaced or rehabilitated
FHWA	Federal Highway Administration

GUI	Graphical User Interface
LOS	Level of Service
MDOT	Michigan Department of Transportation
MOT	Maintenance of Traffic
MiABCD	Michigan Accelerated Bridge Construction Decision
Model Presets	This include predefined tables that define the relationship among the project data, ordinal scale ratings, and the Analytical Hierarchy Process (AHP) pair-wise comparison ratings
Pop-Up Menu	A GUI form of the VBA
Question Icon	The icon  on any *pop-up menu* or *datasheet* will provide description of the corresponding “item”
Regional Data	This includes wage and cost, county jobs multiplier, material procurement distance classification, traffic data classification, and bridge spans classification
S&ST	Site and Structure Considerations
SC&PS	Seasonal Constraints and Project Schedule
Tab	This is used to switch between different criteria, such as in MiABCD the *tabs* allow switching between the major-parameters on a *pop-up menu*
Text Field	This is used to input text, such as sub-parameter name, etc., that will be transferred to a corresponding *datasheet*
TF&R	Technical Feasibility and Risk
VBA	Visual Basic for Applications
WZM	Work Zone Mobility

APPENDIX F

DRAWINGS OF FULL-DEPTH DECK PANELS FROM UTAH DOT

Note: These are direct extract from <http://www.udot.utah.gov/main/uconowner.gf?n=5440222707642218>
(Last accessed June 25, 2012)

FULL DEPTH PRECAST CONCRETE DECK PANELS GENERAL NOTES

GUIDELINES

USE THESE GUIDELINE DRAWINGS FOR BRIDGES WHICH HAVE ALL OF THE FOLLOWING CHARACTERISTICS:

TANGENTIAL (NO HORIZONTAL CURVATURE) PANELS PLACED ORTHOGONALLY TO THE BEAM/GIRDERS.

SKEW : 0 TO 45 DEGREES

PARALLEL STEEL GIRDERS WITH A MINIMUM TOP FLANGE WIDTH OF 16"; AASHTO GIRDERS (TYPE II, III, IV, V AND VI); OR PRESTRESSED BULB TEE GIRDER.

FOR PRECAST PANELS:
MAX. BEAM/GIRDER SPACING = 10'-0"

FOR PRESTRESSED PANELS:
MAX. BEAM/GIRDER SPACING = 12'-0"

MAX. OVERHANG = 4'-0"

MIN. OVERHANG = 1'-0"

DEAD LOADS:

40 PSF FUTURE OVERLAY

NO MORE THAN 2 EXTERIOR TRAFFIC PARAPETS PER PANEL

MAX. DEAD LOAD PER PARAPET = 569 PLF

PANEL-TO-PANEL CONNECTIONS:

ALL PANELS TO BE CONNECTED WITH LONGITUDINAL POST-TENSIONING COMBINED WITH A TRANSVERSE GROUTED KEYWAY JOINT.

DEFINITIONS

PRECAST PANEL: CONCRETE PANEL REINFORCED WITH DEFORMED STEEL BARS.

PRESTRESSED PANEL: CONCRETE PANEL REINFORCED WITH PRESTRESSING STEEL AND DEFORMED STEEL BARS.

OPTIONAL DETAILS

THESE DRAWINGS ARE BASED ON THE USE OF BLIND BLOCKOUTS FOR SHEAR CONNECTIONS FORMED WITH REMAIN IN PLACE STEEL BOXES.

OPTIONAL BLOCKOUT DETAILS SHOWN ON DRAWING NUMBER PDP-9 ARE ALSO ACCEPTABLE.

IMPLEMENTATION

IT IS THE DESIGNER'S RESPONSIBILITY TO:

DESIGN AND CHECK THE REQUIRED SHEAR STUDS AND/OR REINFORCING STEEL CONNECTING THE GIRDERS/BEAMS TO THE DECK TO ENSURE ADEQUATE COMPOSITE ACTION BETWEEN THE FRAMING MEMBERS AND PANELS IN ACCORDANCE WITH ALL APPLICABLE CODES.

CREATE THE CONCRETE DECK PANEL LAYOUT SHEET SHOWING TYPE AND NUMBER OF PANELS TO BE USED AS WELL AS NUMBER AND SPACING OF SHEAR BLOCKOUTS REQUIRED.

CALCULATE FINAL DECK ELEVATIONS AND CREATE TOP OF PANEL ELEVATIONS SHEET(S).

DESIGN AND CHECK ALL CHARACTERISTICS RELATED TO REQUIRED CLOSURE POURS.

CHECK THE STRUCTURAL CAPACITY OF THE EXISTING GIRDERS/ BEAMS AND/OR NEW GIRDERS/BEAMS FOR THE INSTALLATION OF THE PANELS (INCLUDING EFFECTS OF PANEL INSTALLATION SEQUENCING). USE OF THESE GUIDELINE DRAWINGS IMPLIES NO ASSERTION AS TO THE STRUCTURAL CAPACITY OF ANY GIRDERS OR BEAMS SHOWN. DEVELOP LOAD RATINGS AS DIRECTED BY UDOT. VERIFY ADEQUATE CAPACITY IN GIRDERS FOR THE EFFECTS OF LONG TERM POST-TENSIONING WHEN APPLICABLE.

DESIGN ALL POST-TENSIONING. SHOW SIZE AND LAYOUT OF DUCTS. SPECIFY JACKING FORCES, SEQUENCE OF JACKING, DUCT WOBBLE COEFFICIENTS, AND DUCT COEFFICIENT OF FRICTION.

PROVIDE POSITIVE DRAINAGE DETAILS PER UDOT STANDARD PRACTICE. DRAINAGE HOLES THROUGH THE PANELS ARE PROHIBITED.

DESIGN AND ACCOMMODATE APPLICABLE REINFORCEMENT FOR A HAUNCH GREATER THAN 3".

INCLUDE APPLICABLE GENERAL NOTES IN THE PLAN SET.

VERIFY SIZE AND SPACING OF REINFORCEMENT CONNECTING PARAPET TO PANEL IF PARAPET OTHER THAN TYPE SPECIFICALLY SHOWN IN THESE STANDARD DRAWINGS IS USED.

AT A MINIMUM EXTEND CONTINUOUS REINFORCEMENT FROM PRECAST PANEL, #6 AT 6" SPACING TOP AND BOTTOM, INTO CLOSURE POUR. CLOSURE POUR DETAILS SHOWN FOR MAXIMUM BEAM SPACING OF 10'-0". FOR BEAM SPACINGS GREATER THAN 10'-0", DESIGN AND DETAIL CLOSURE POUR AND APPROPRIATE POST-TENSIONING AS REQUIRED.

INCLUDE A TABLE OF ESTIMATED QUANTITIES OF PRECAST CONCRETE DECK PANELS. TABLE TO INCLUDE THE FOLLOWING:
- PANEL TYPE (BASED ON PANEL LAYOUT)
- NUMBER OF EACH PANEL TYPE REQUIRED
- SQUARE FOOTAGE OF AREA PER PANEL
- TOTAL SQUARE FOOTAGE OF DECK PANELS

GENERAL NOTES

PRECAST CONCRETE PANELS DESIGNED IN ACCORDANCE WITH AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 4TH EDITION WITH ALL INTERIM PROVISIONS.

PANELS DESIGNED FOR AN HL-93 LOAD INCLUDING A 40 PSF LOAD FOR FUTURE OVERLAY.

PRECAST PANEL CONCRETE:	$f'c$ = 4,000 PSI CLASS AA(AE)
PRESTRESSED PANEL CONCRETE:	$f'ci$ = 4,000 PSI $f'c$ = 5,000 PSI
CLOSURE POUR CONCRETE:	$f'c$ = MATCH PRECAST ELEMENTS
NON-SHRINK GROUT:	$f'c$ = 5,000 PSI @ 24 HRS
REINFORCING STEEL (COATED)	fy = 60,000 PSI
PRESTRESSED LOW RELAXATION STRAND:	$fpbt$ = 202.5 KSI fpu = 270.0 KSI
STRUCTURAL STEEL:	fy = 50,000 PSI AASHTO M270 GR 50

USE UTAH DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION (THE LATEST EDITION AND SUPPLEMENTS THERETO WHICH ARE IN EFFECT AT THE DATE OF REQUEST FOR BIDS) ALONG WITH SPECIAL PROVISIONS SECTION 03339S-FULL DEPTH STANDARD PRECAST CONCRETE DECK PANELS FOR MATERIALS, CONSTRUCTION AND WORKMANSHIP.

WELD ACCORDING TO AASHTO/AWS D1.5 BRIDGE WELDING CODE.

USE THE PCI DESIGN HANDBOOK, PRECAST AND PRESTRESSED CONCRETE, FIFTH EDITION WITH ALL INTERIMS AND ERRATA FOR THE DESIGN AND DETAIL OF LIFTING SUPPORTS AND HANDLING CONSIDERATIONS (NO CRACKING CRITERIA).

USE A HEAVY BROOM FINISH FOR TOP SURFACE OF PANELS AND ALL JOINT SURFACES.

CHAMFER ALL EXPOSED CORNERS 3/4". PRECAST PANELS ADJACENT TO CLOSURE POURS OR OTHER PANELS ARE NOT CONSIDERED EXPOSED CORNERS.

THE PRECAST PANELS HAVE A 1/4" CONCRETE GRINDING ALLOWANCE FOR CORRECTING UNEVEN ROADWAY SURFACES AT TRANSVERSE JOINTS BETWEEN PRECAST CONCRETE DECK PANELS AND END OF BRIDGE DECK OR EDGE OF ADJACENT PHASE(S). DECK THICKNESS SHOWN AS NOMINAL OR FINAL THICKNESS AFTER GRINDING. ACCOUNT FOR 1/4" GRINDING ALLOWANCE.

APPLY CONCRETE POLYMER OVERLAY ON BRIDGE DECK AFTER CONCRETE GRINDING OR STEEL SHOT IS COMPLETE. SEE SECTION 03372 IN THE STANDARD SPECIFICATIONS FOR SURFACE PREPARATION REQUIREMENTS.

SEE "GENERAL LAYOUTS" AND "TYPICAL DECK PANEL PLANS AND SECTIONS" SHEETS FOR PANEL TYPES AND LOCATIONS.

SEE TOP OF PANEL ELEVATION SHEETS AND/OR CONCRETE UNIT SHEETS FOR SIZE, TYPE, ORIENTATION, NUMBER AND SPACING OF SHEAR STUDS/BLOCKOUTS.

COAT ALL MILD REINFORCEMENT PER UDOT SPECIFICATIONS UNLESS OTHERWISE NOTED.

USE A CORROSION INHIBITOR ADMIXTURE FOR ALL STRUCTURAL GROUT.

USE A CORROSION INHIBITOR ADMIXTURE FOR ALL PANEL AND CLOSURE POUR CONCRETE.

INDEX OF SHEETS

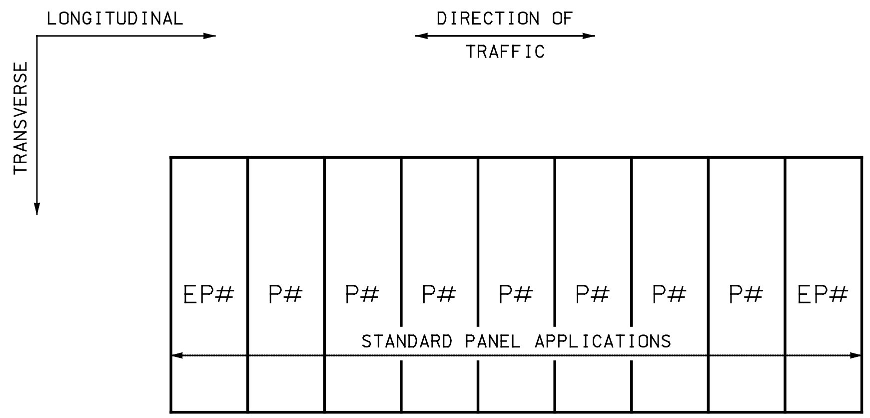
NO. DATE BY REVISION REMARKS CHECK
DESIGN CHECK
DRAWN CHECK
QUANT. CHECK
APPROVAL RECOMM. DATE SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT DATE UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

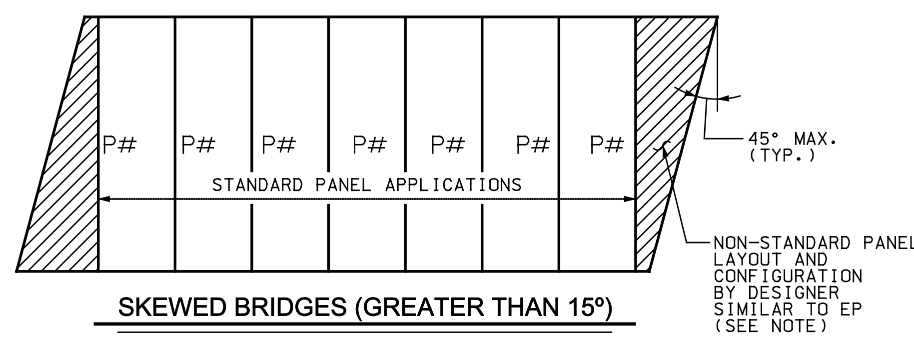
TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
GENERAL NOTES

PIN
PROJECT NUMBER
COUNTY
PDP - 1
DRG. NO.
SHT. OF

PDP-01_PANEL_GEN_NOTES.dgn
7/19/2010



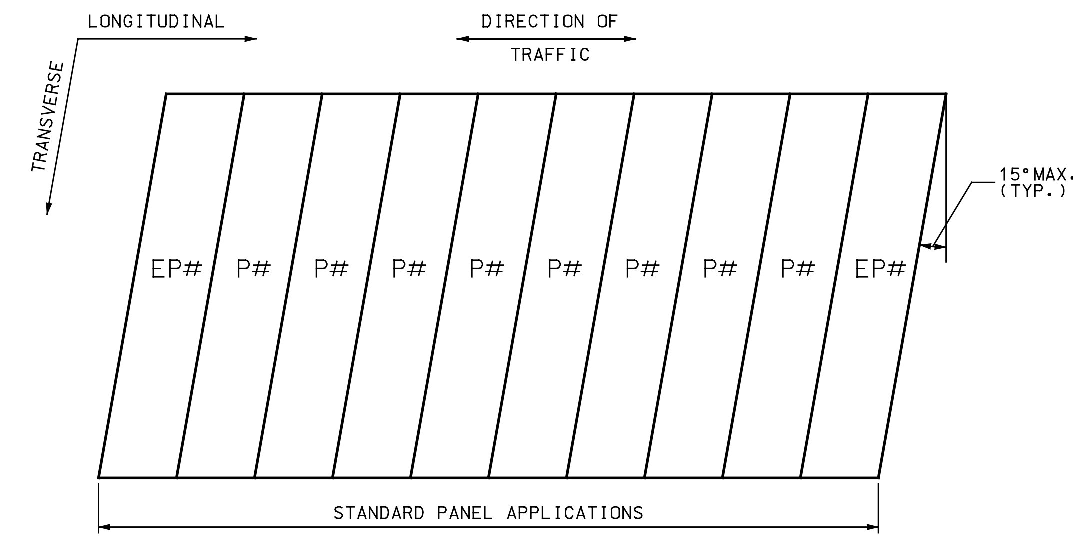
NON - SKEWED BRIDGES



SKEWED BRIDGES (GREATER THAN 15º)

STANDARD PRECAST CONCRETE PANEL APPLICATIONS - NON-SKEWED

NOTE: PARTIAL WIDTH PANELS MAY BE USED IN COMBINATION WITH CLOSURE POURS.



SKEWED BRIDGES (BETWEEN 0° AND 15°)

STANDARD PRECAST CONCRETE PANEL APPLICATIONS - SKEWED

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DESIGN | CHECK | DRAWN | CHECK | QUANT. | CHECK

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER

APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

GENERAL LAYOUTS

PIN

PROJECT NUMBER

COUNTY

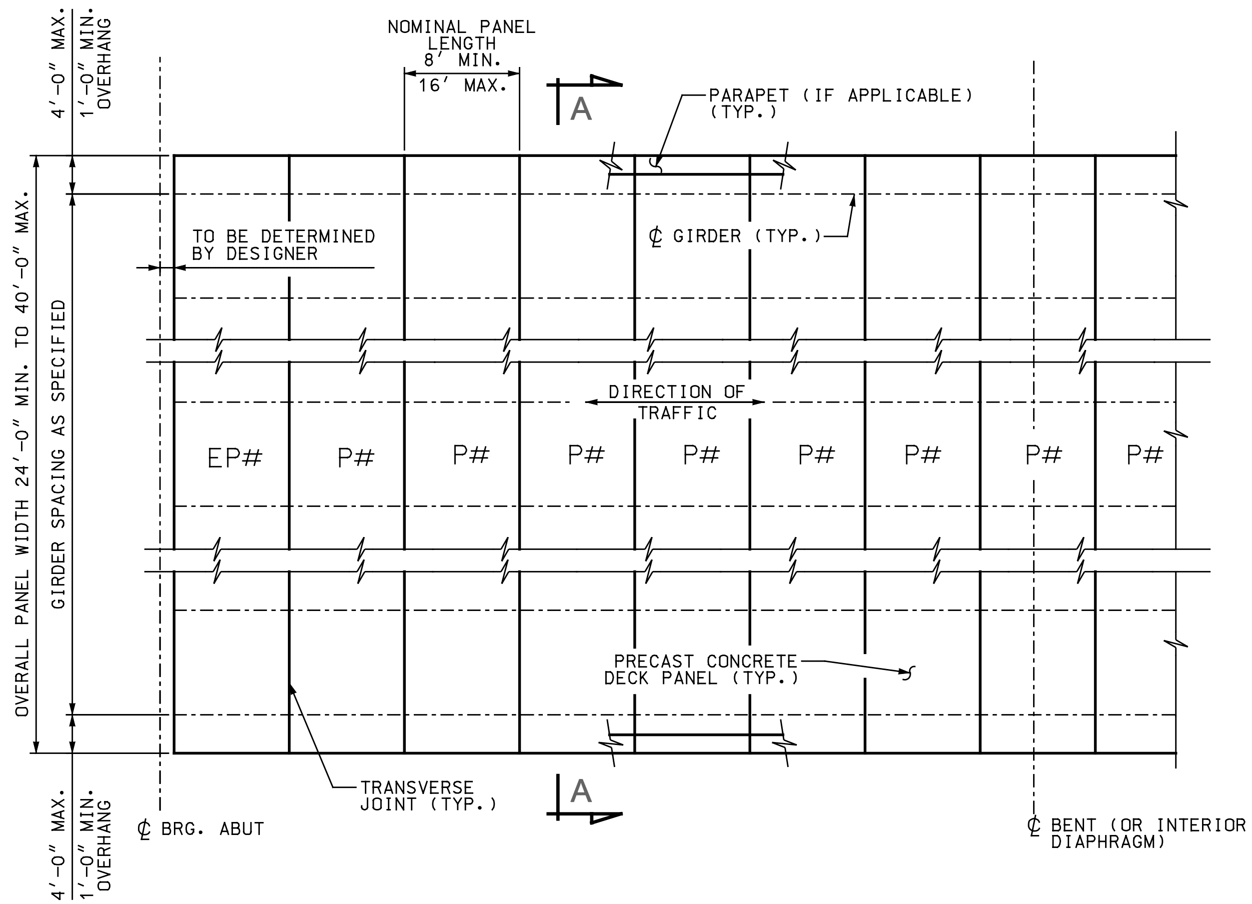
PDP - 2

DRG. NO.

SHT. OF

PDP-02_GENERAL_LAYOUTS.dgn

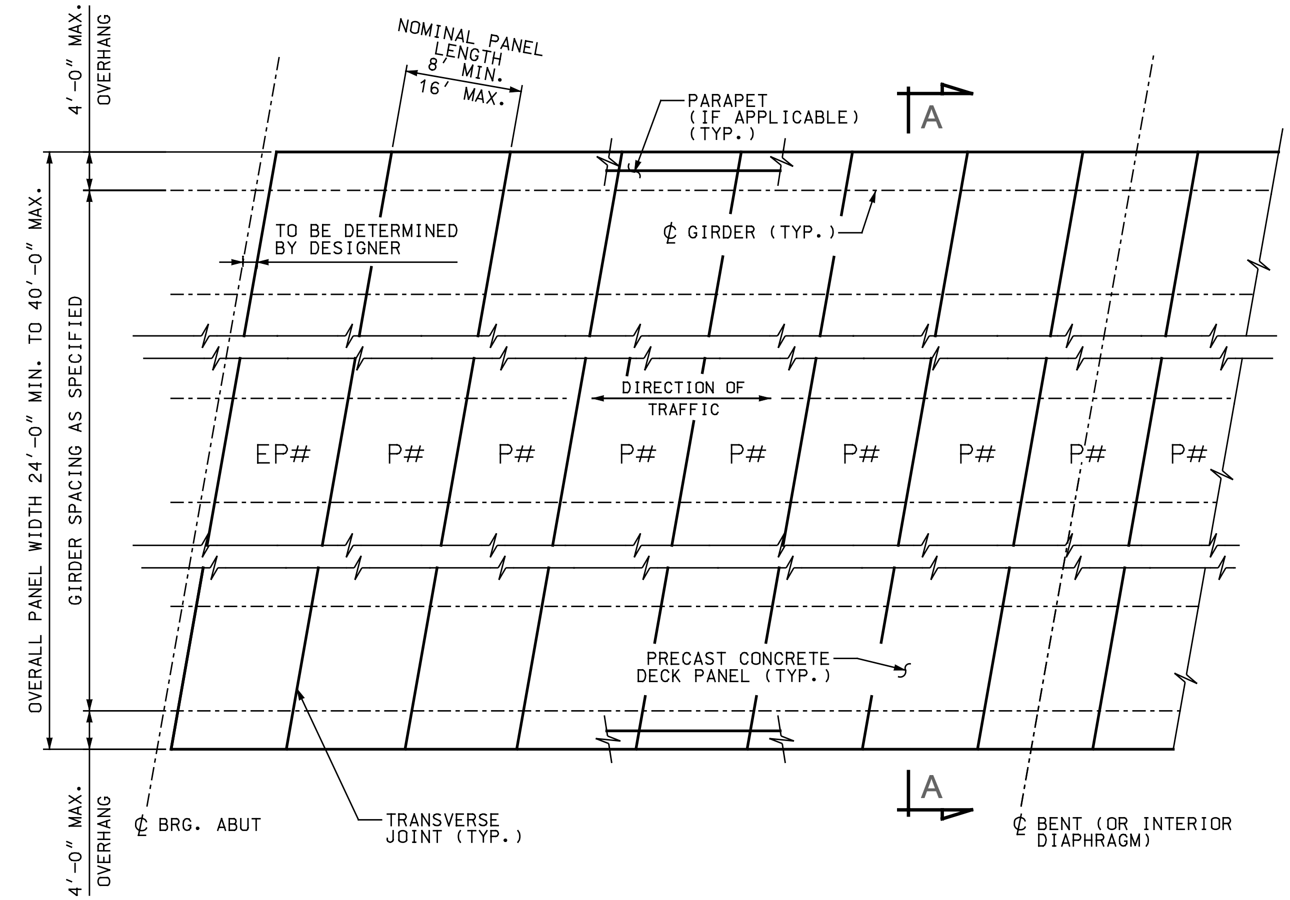
7/19/2010



PRECAST CONCRETE PANEL TYPES

SEE "PANEL PLANS - NON-SKEWED" SHEET FOR ADDITIONAL INFORMATION
NOTE: NOMINAL JOINT SPACING SHOWN

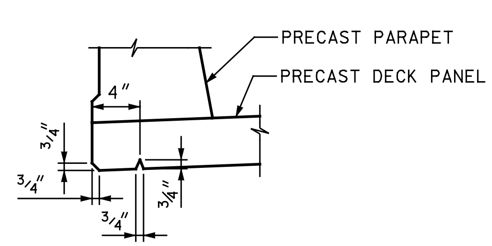
PRECAST PANEL TYPES	
EP#	PANEL ADJACENT TO ABUTMENT
P#	STANDARD INTERIOR PANEL



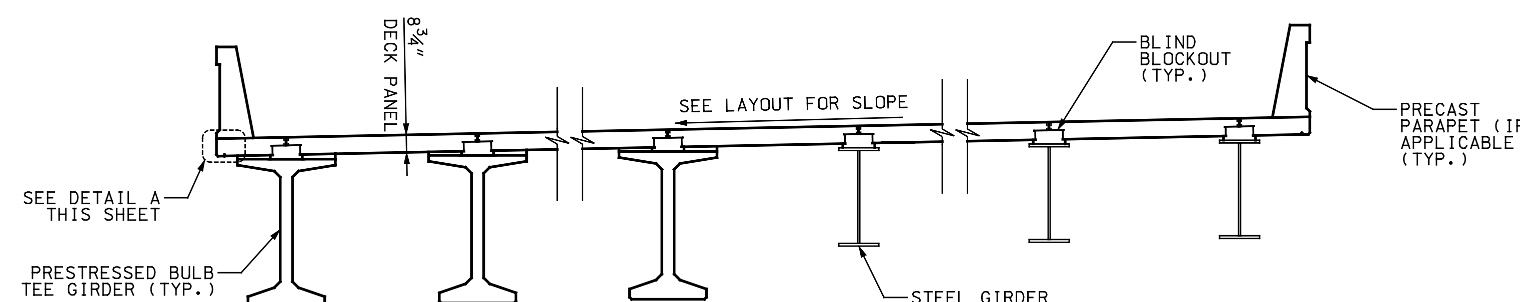
PRECAST CONCRETE PANEL TYPES

SEE "PANEL PLANS - SKEWED" SHEET FOR ADDITIONAL INFORMATION
NOTE: NOMINAL JOINT SPACING SHOWN

PRECAST PANEL TYPES	
EP#	PANEL ADJACENT TO ABUTMENT
P#	STANDARD INTERIOR PANEL



DETAIL A: DRIP GROOVE



SECTION A: PRECAST DECK PANEL

NO.	DATE	BY	REVISION REMARKS

DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

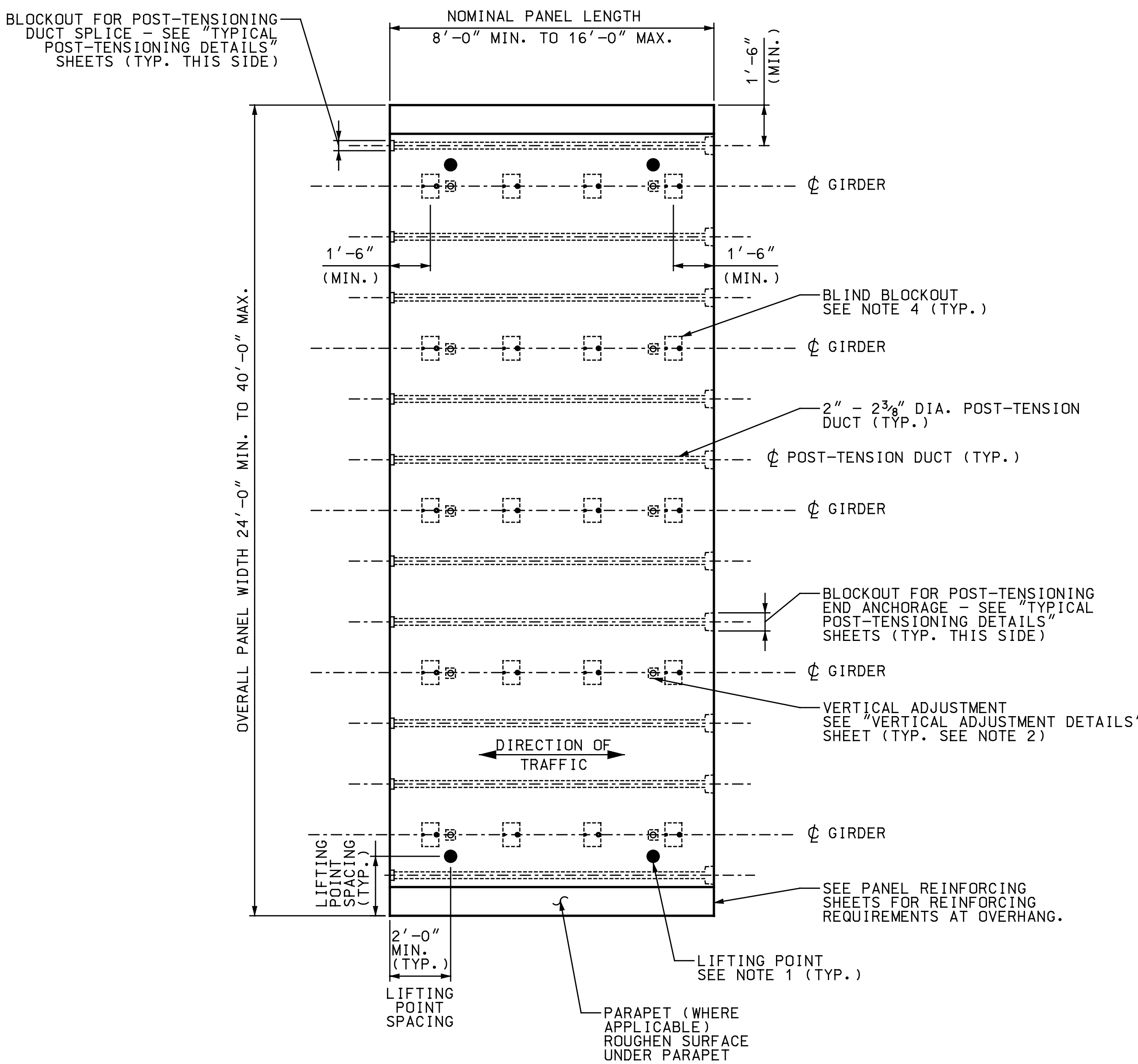
TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
TYPICAL DECK PANEL PLANS AND SECTIONS

PIN
PROJECT NUMBER
COUNTY
PDP - 3
DRG. NO.
SHT. OF

7/19/2010 PDP-03_GENERAL PLANS AND SECTIONS.dgn

NOTES

1. DESIGNER WILL DETERMINE NUMBER AND LOCATION OF LIFTING POINTS.
2. A MINIMUM OF 2 VERTICAL ADJUSTMENT ASSEMBLIES ARE REQUIRED AT EACH ℄ GIRDER.
3. FOR VERTICAL ADJUSTMENT DEVICES SEE "VERTICAL ADJUSTMENT DETAILS" SHEET.
4. FOR DETAILS OF BLIND BLOCKOUT SEE "SHEAR CONNECTOR BLOCKOUT DETAILS" SHEET.
5. SEE "PRECAST PANEL REINFORCING" AND "PRESTRESSED PANEL REINFORCING" SHEETS FOR REQUIRED REINFORCING.
6. SEE "TYPICAL POST-TENSIONING DETAILS 2" SHEET FOR CONNECTION DETAILS.



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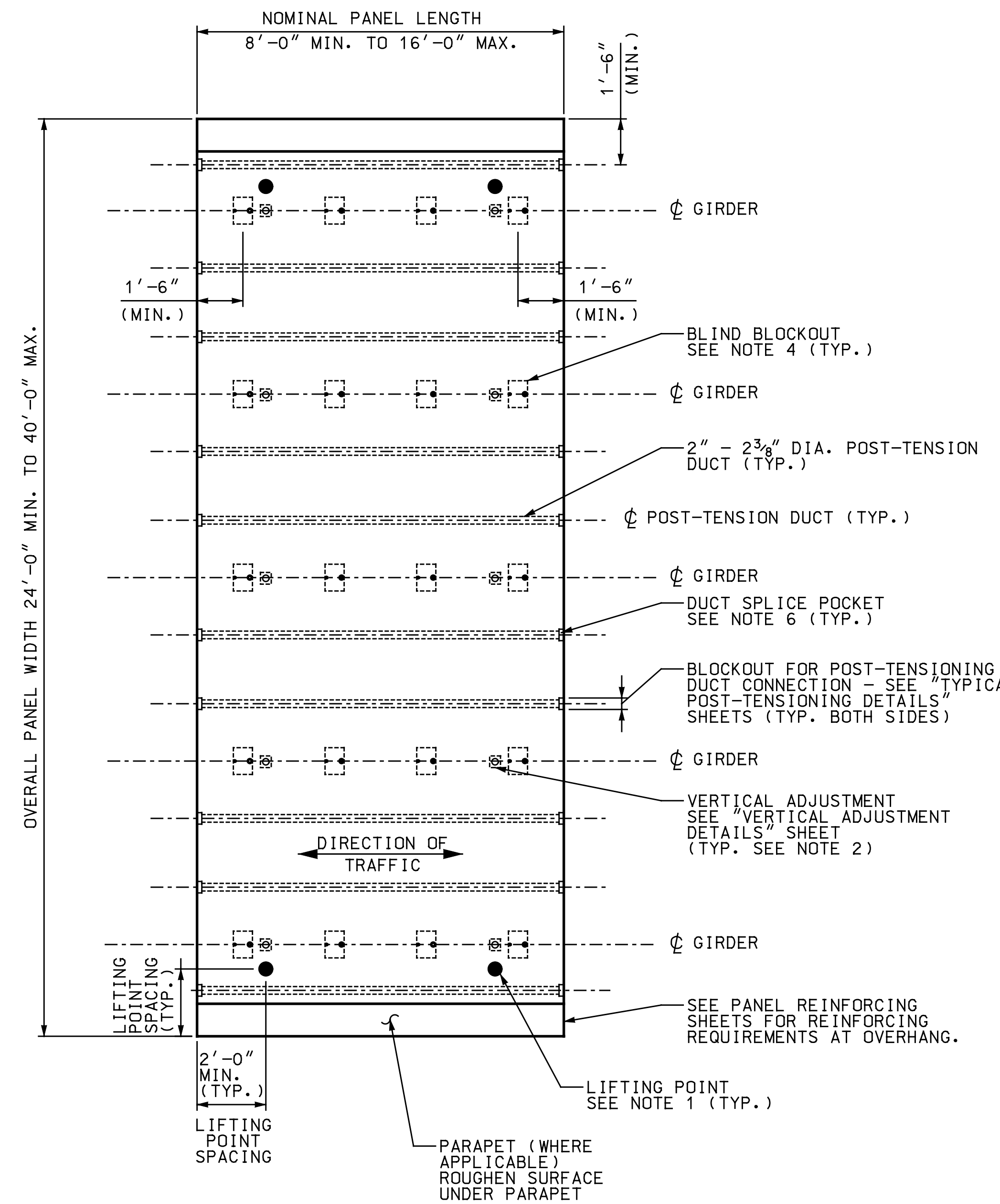

PANEL TYPE P

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DESIGN CHECK
DRAWN CHECK
QUANT. CHECK
APPROVAL RECOMM. DATE SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT DATE UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
PANEL PLANS - NON-SKEWED

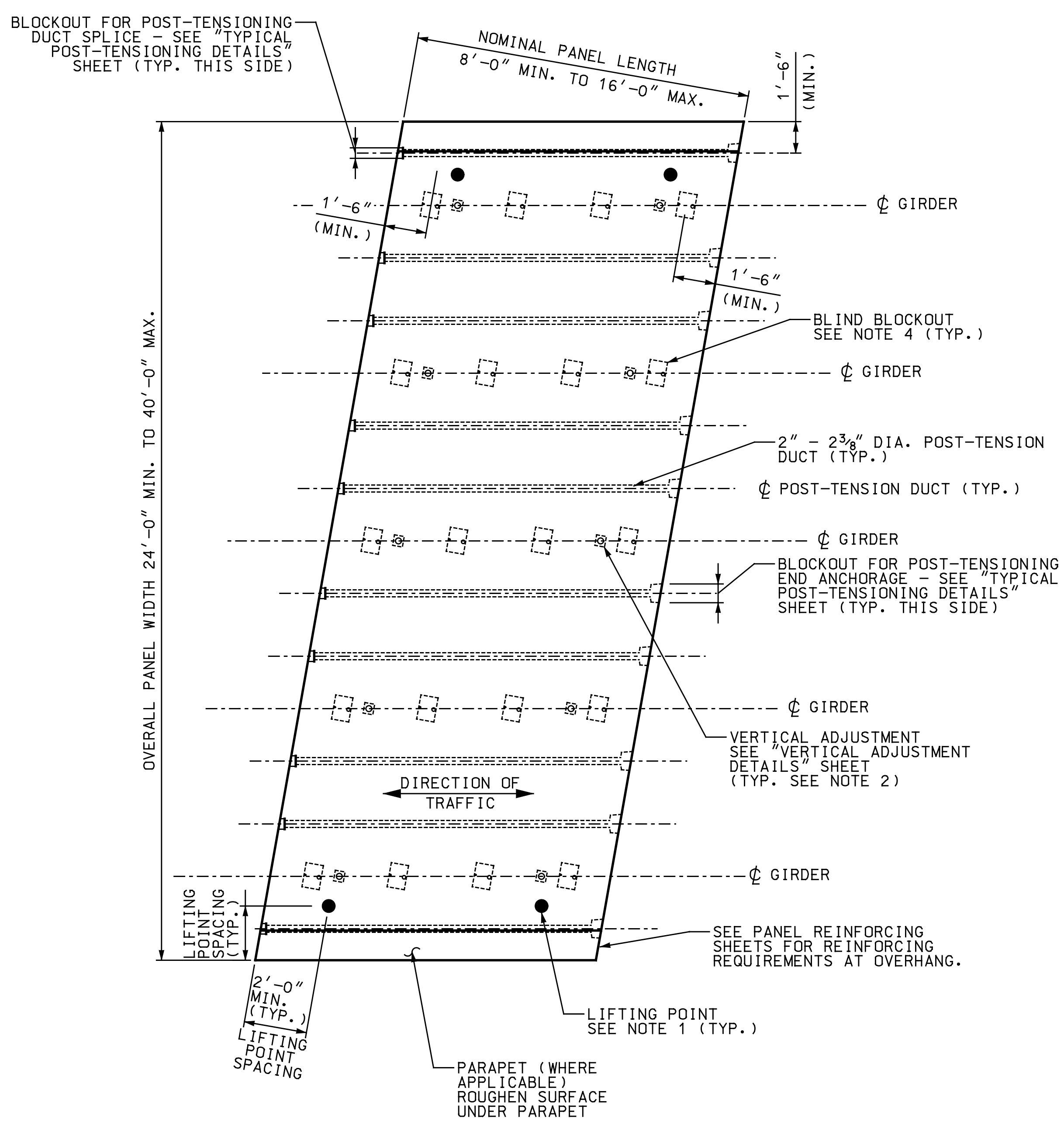
PIN
PROJECT NUMBER

COUNTY
PDP - 4
DRG. NO.
SHT. OF

7/19/2010 PDP-04_TYPICAL_PANEL_PLANS.dgn

NOTES

1. DESIGNER WILL DETERMINE NUMBER AND LOCATION OF LIFTING POINTS.
2. A MINIMUM OF 2 VERTICAL ADJUSTMENT ASSEMBLIES ARE REQUIRED AT EACH ℄ GIRDER.
3. FOR VERTICAL ADJUSTMENT DEVICES SEE "VERTICAL ADJUSTMENT DETAILS" SHEET.
4. FOR DETAILS OF BLIND BLOCKOUT SEE "SHEAR CONNECTOR BLOCKOUT DETAILS" SHEET.
5. SEE "PRECAST PANEL REINFORCING" AND "PRESTRESSED PANEL REINFORCING" SHEETS FOR REQUIRED REINFORCING.
6. SEE "TYPICAL POST-TENSIONING DETAILS 2" SHEET FOR CONNECTION DETAILS.



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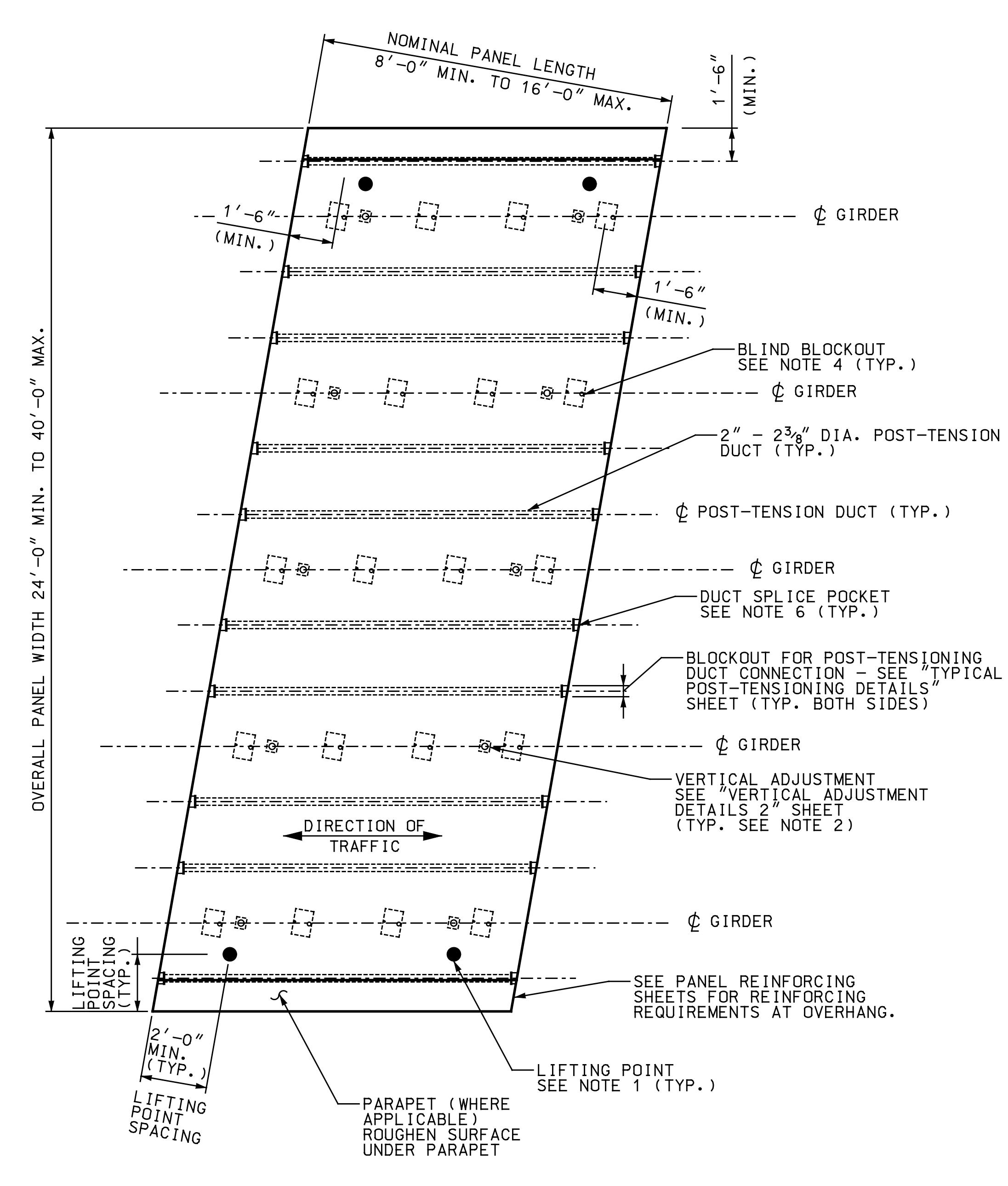


PANEL TYPE P

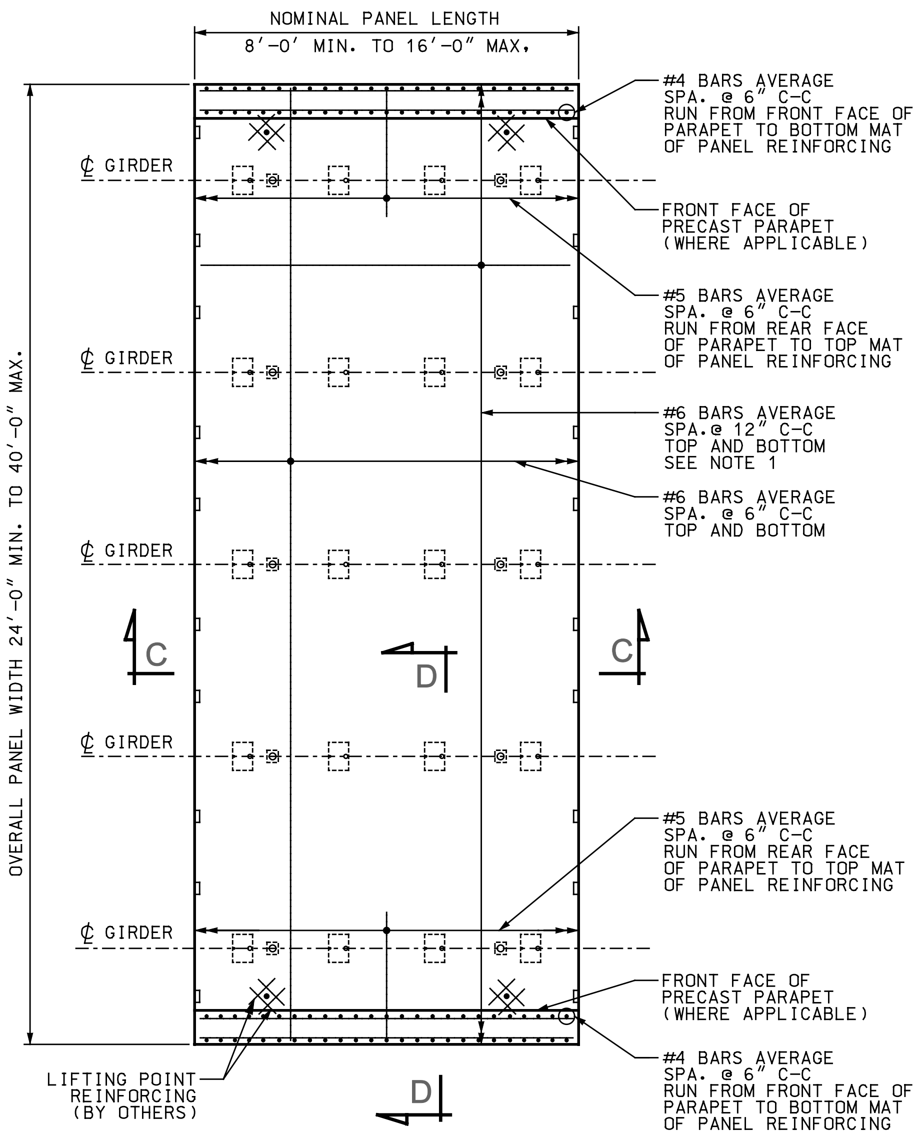
UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

NO. | DATE | BY | REVISION REMARKS
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER
DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK

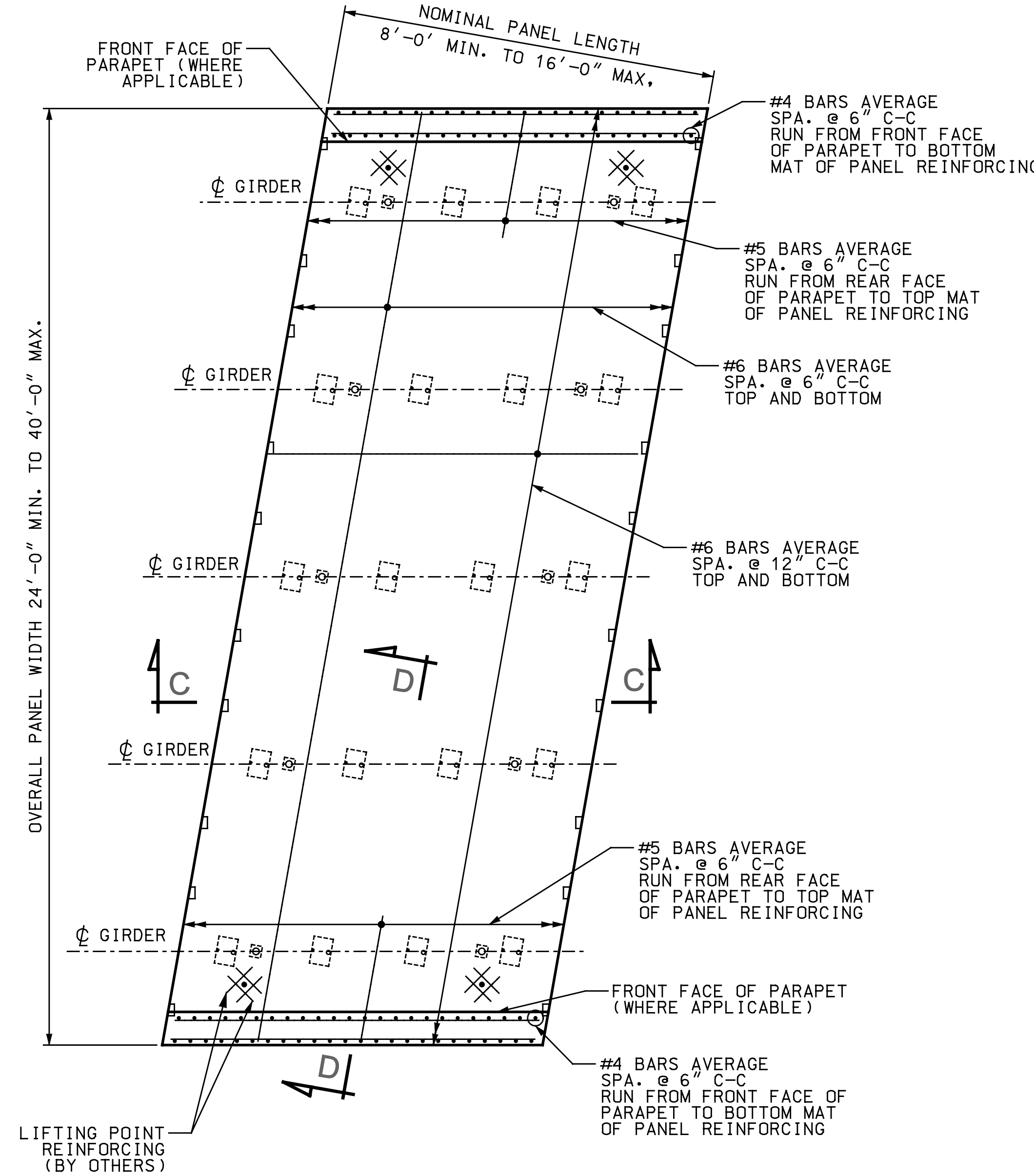
TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
PANEL PLANS - SKEWED

PIN
PROJECT NUMBER
COUNTY
PDP - 5
DRG. NO.
SHT. OF

7/19/2010



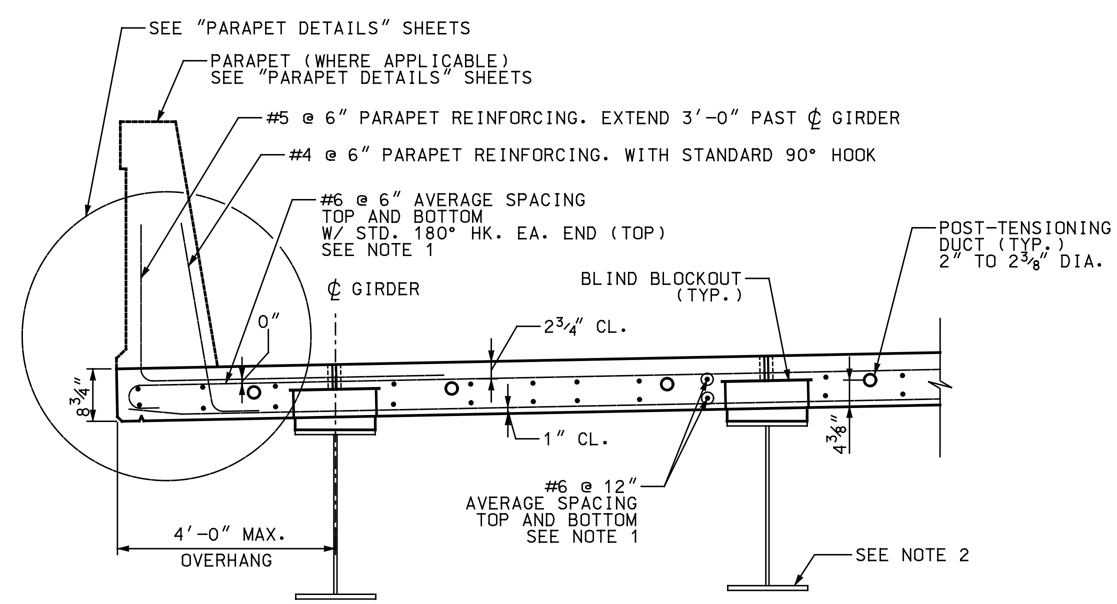
INTERIOR PANEL REINFORCING PLAN



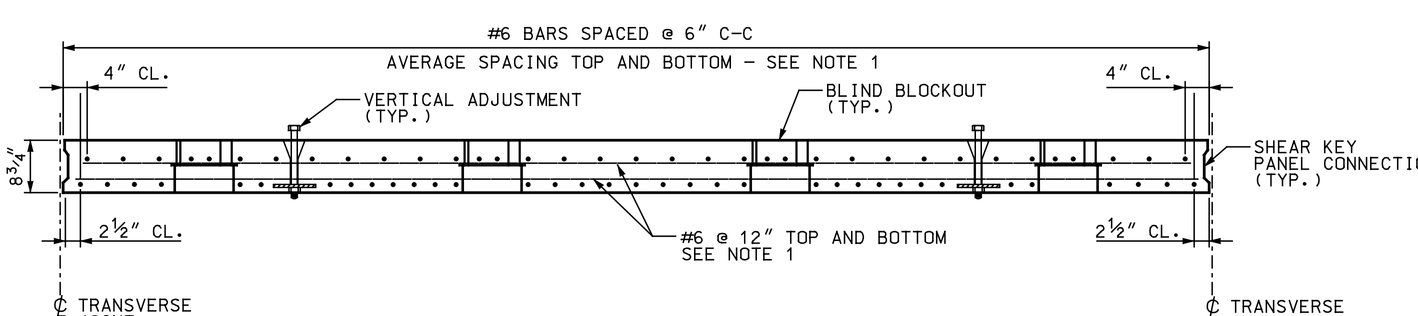
SKEWED INTERIOR
MILD REINFORCED PANEL

NOTES

1. ADJUST LOCATION OF BARS TO AVOID CONFLICTS WITH BLOCKOUTS AS APPROVED BY DESIGNER. ROTATE #6 HOOKS TOP BARS TO PROVIDE ADEQUATE COVER.
2. STEEL GIRDER SHOWN, PRESTRESSED BULB TEE GIRDERS ALLOWED.
3. HOOK TOP BARS STD 180° EACH END.
4. CROWNED ROADWAYS REQUIRE A CLOSURE POUR AT THE CROWN. THIS WILL REQUIRE MULTIPLE PANELS IN A CROSS SECTION. SEE CROWN CLOSURE POUR DETAILS.



SECTION D: PRECAST PANEL



SECTION C: PRECAST PANEL

NO.	DATE	BY	REVISION REMARKS

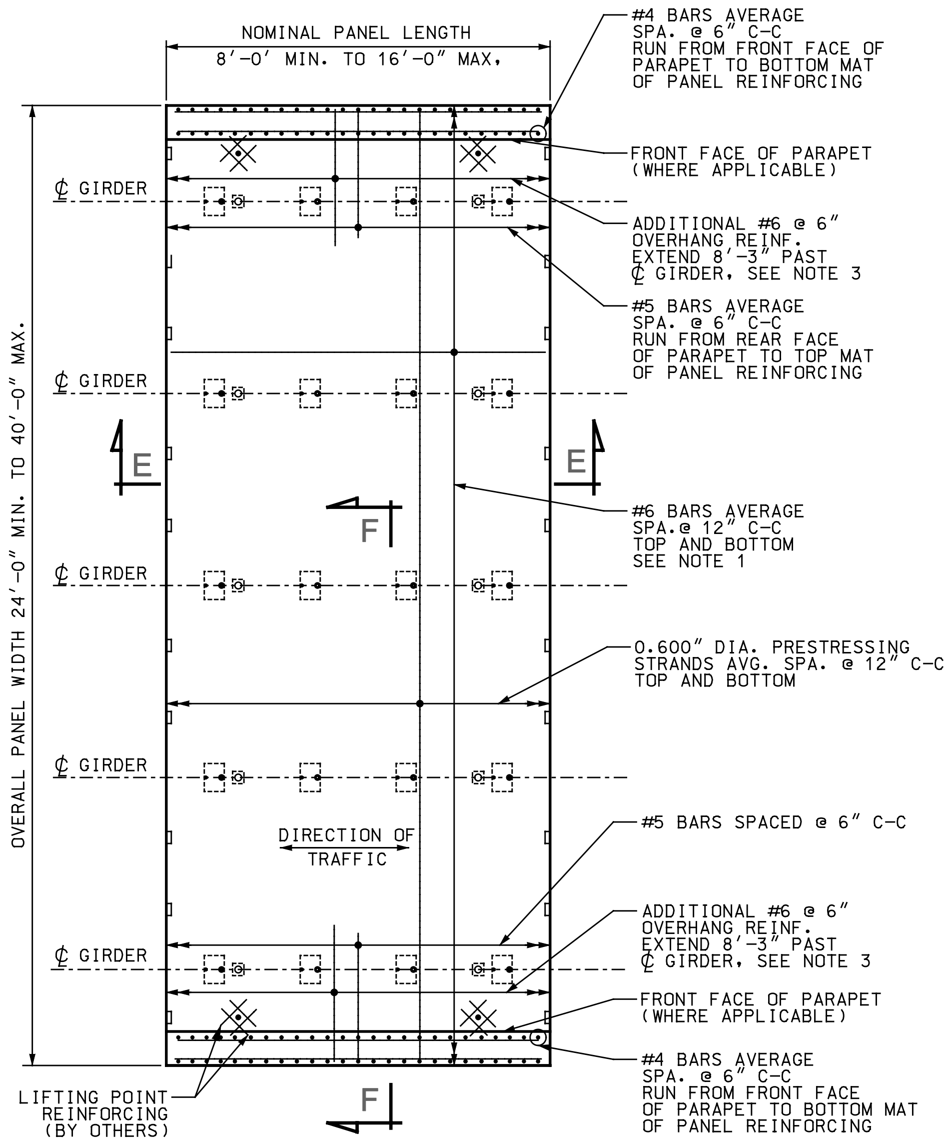
DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

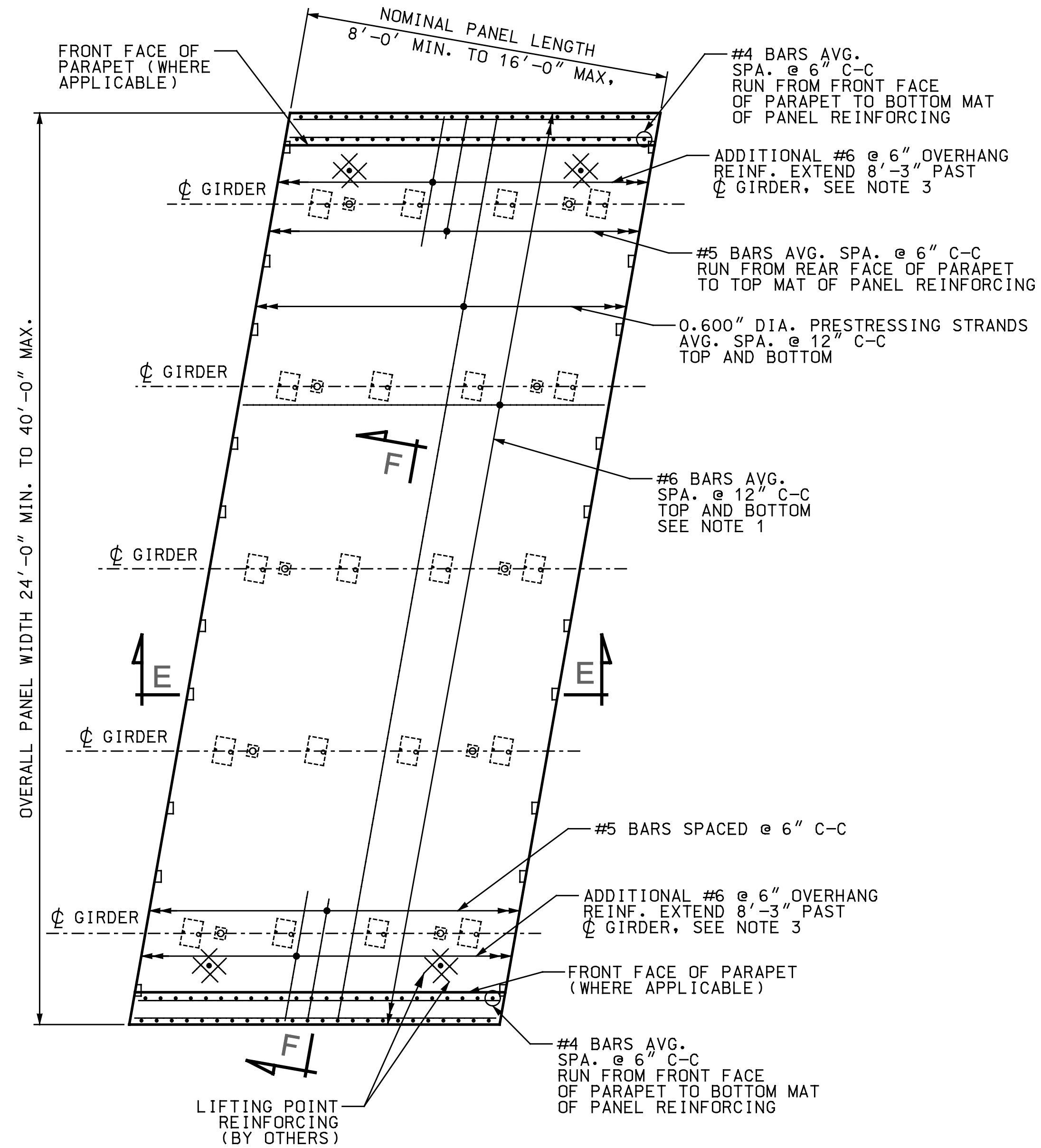
TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
PRECAST PANEL REINFORCING

PIN
PROJECT NUMBER
COUNTY
PDP - 6
DRG. NO.
SHT. OF

PDP-06_PRECAST_PANEL_REINF.dgn 7/19/2010



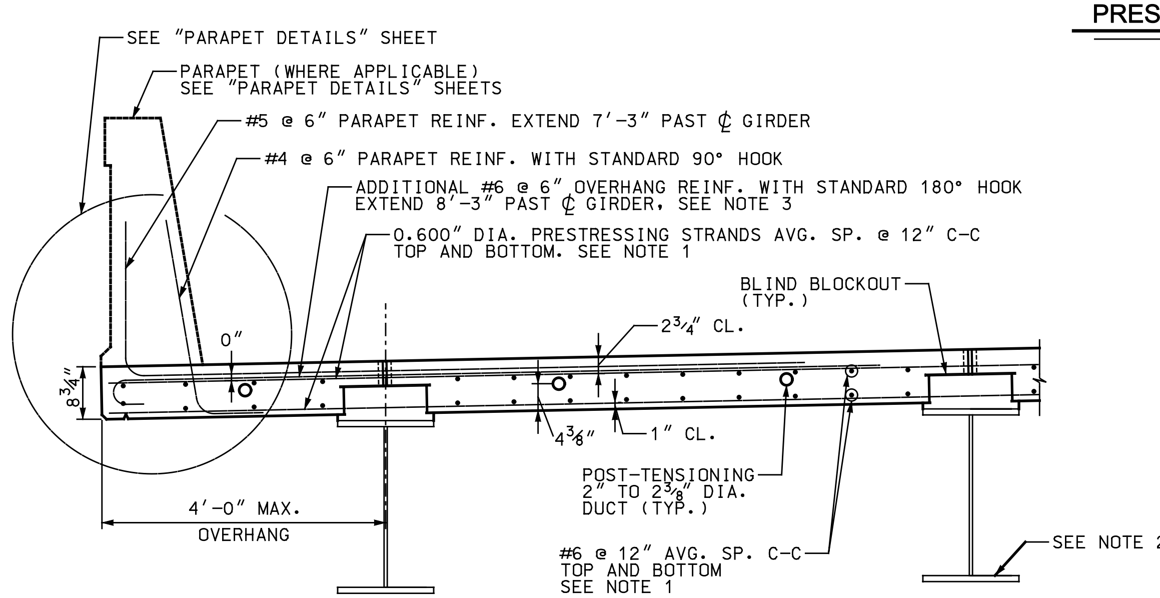
INTERIOR PANEL REINFORCING PLAN NON - SKEWED



SKEWED INTERIOR
PRESTRESSED REINFORCED PANEL

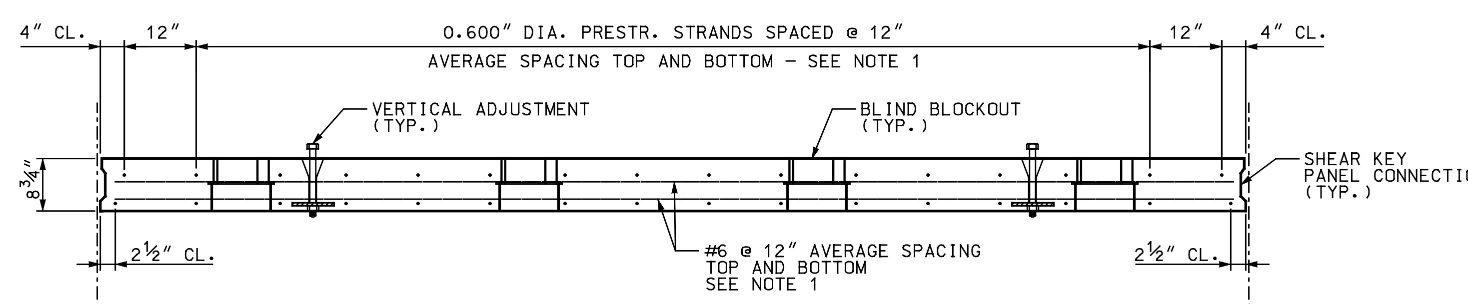
NOTES

1. ADJUST LOCATION OF BARS AND STRANDS TO AVOID CONFLICTS WITH BLOCKOUTS AS APPROVED BY DESIGNER.
2. STEEL GIRDER SHOWN, PRESTRESSED BULB TEE GIRDERS ALLOWED.
3. CROWNED ROADWAYS REQUIRE A CLOSURE POUR AT THE CROWN. THIS WILL REQUIRE MULTIPLE PANELS IN A CROSS SECTION. SEE CROWN CLOSURE POUR DETAILS.



SECTION F: PRESTRESSED PANEL

PORTION OF DECK PANEL SHOWN. REMAINDER SIMILAR.



SECTION E: PRESTRESSED PANEL

NO. | DATE | BY | REVISION REMARKS | CHECK | CHECK | CHECK

DESIGN | DRAWN | QUANT. | APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER | APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

PRESTRESSED PANEL REINFORCING

PIN

PROJECT NUMBER

COUNTY

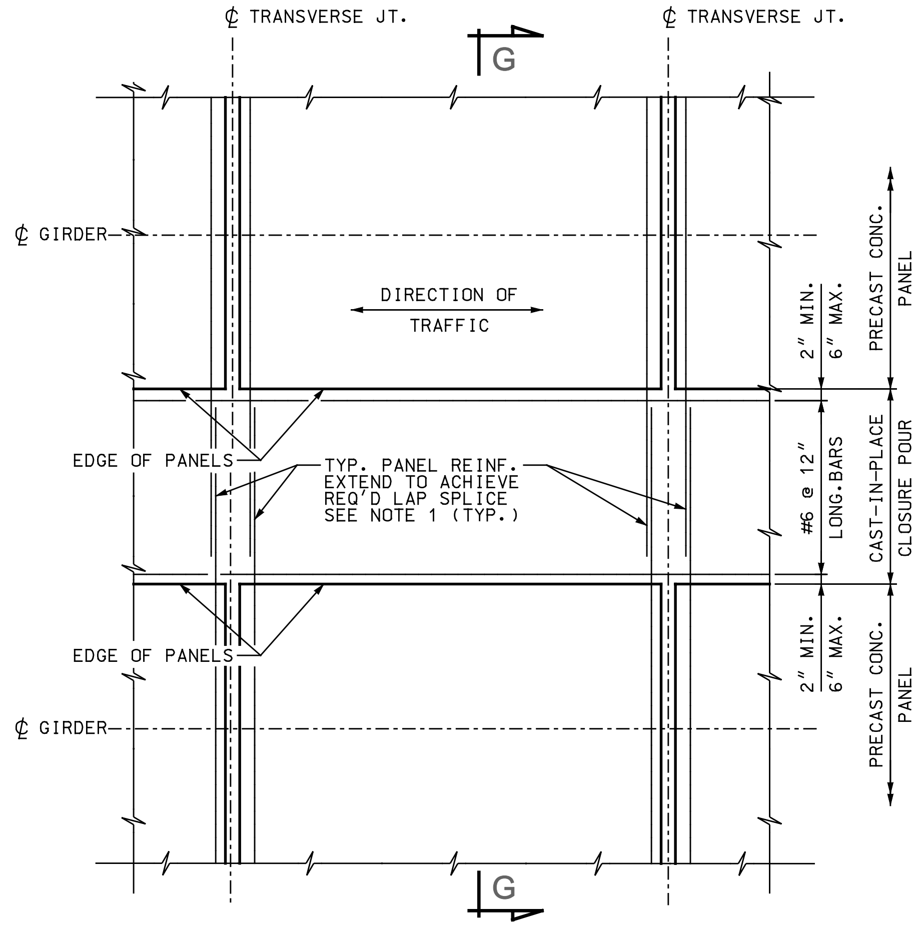
PDP - 7

DRG. NO.

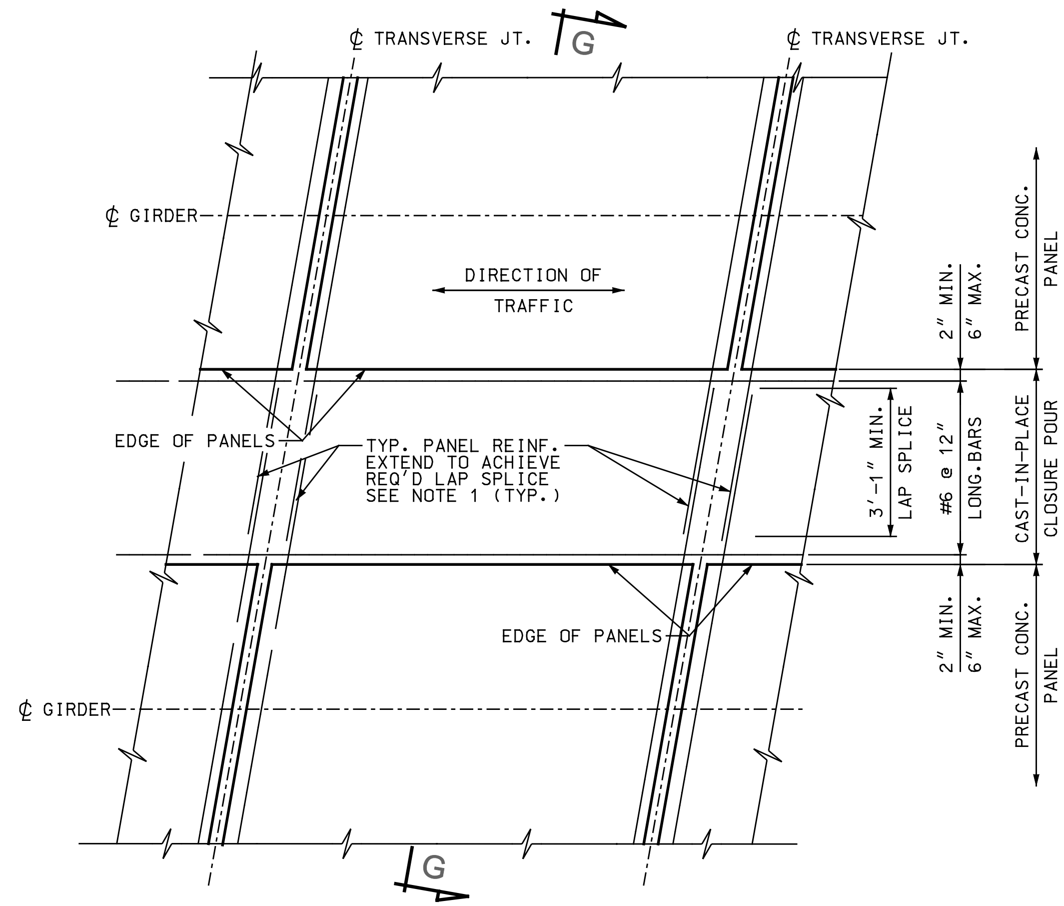
SHT. OF

PDP-07 PRESTR PANEL REINF.dgn

7/19/2010



CLOSURE POUR PLAN

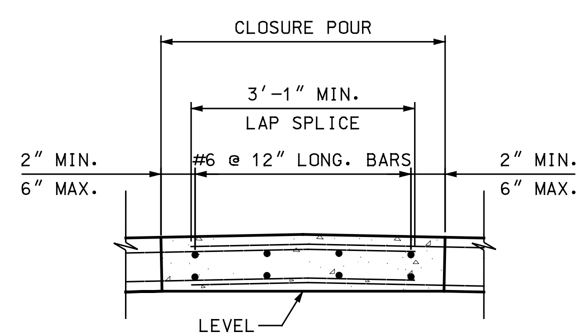


CLOSURE POUR PLAN

NOTES

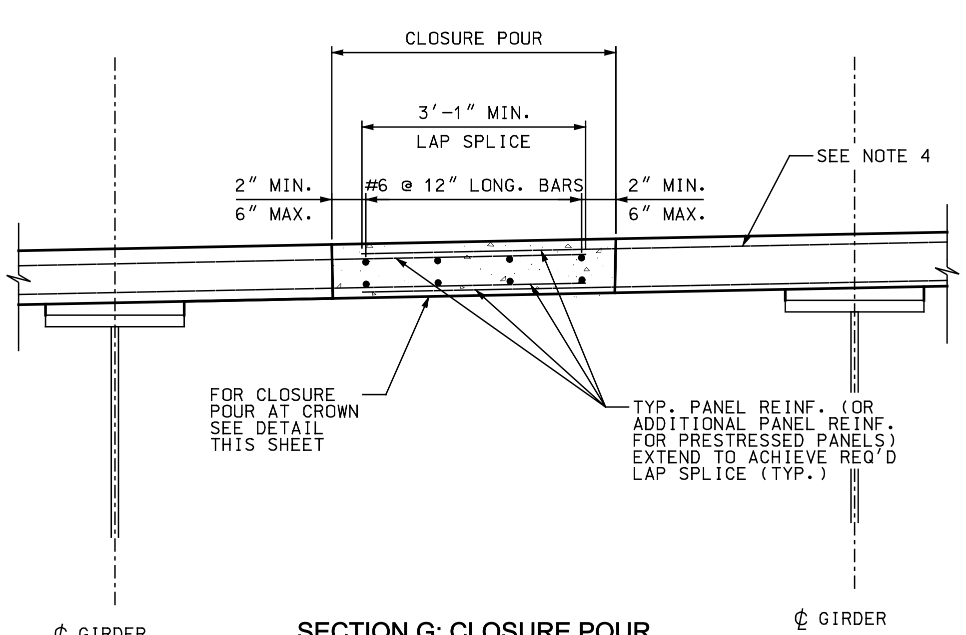
1. MINIMUM REINFORCEMENT SHOWN. IT IS THE RESPONSIBILITY OF THE DESIGNER TO DETERMINE THE ACTUAL REINFORCEMENT REQUIRED. DESIGN AND DETAIL CLOSURE POUR FOR BEAM SPACINGS GREATER THAN 10'-0", AND APPROPRIATE POST-TENSIONING AS REQUIRED.
2. PRECAST PANELS SHOWN, FOR PRESTRESSED PANELS, ADD DOWEL BARS EMBEDDED IN THE PANELS EXTENDING INTO THE CLOSURE POUR. THE LENGTH OF EMBEDMENT AND EXTENSION INTO THE CLOSURE POUR TO BE BASED ON THE LENGTH REQUIRED TO ACHIEVE A LAP SPLICE.
3. AS AN ALTERNATIVE TO EMBEDDING BARS IN PRESTRESSED PANELS, THREADED INSERTS MAY BE ALLOWED AS DIRECTED BY THE DESIGNER. IN PRECAST PANELS WITHOUT PRESTRESSING, PANEL REINFORCEMENT, TOP AND BOTTOM, WILL BE CONTINUED INTO THE CLOSURE POUR.
4. DESIGNER TO CHECK OVERHANG REINFORCING FOR ANTICIPATED LOADS.
5. CLOSURE POURS CAN BE USED FOR:
   - ROADWAY CROWNS
   - STAGE CONSTRUCTION JOINTS
   - BRIDGES GREATER THAN 40 FT WIDE
   - BRIDGE WIDENING PROJECTS
6. THE MINIMUM LAP SPLICE LENGTH SHOWN IS BASED ON THE FOLLOWING PARAMETERS:
   - #6 @ 6"
   - f'_c = 4 KSI
   - 2½" CLEAR COVER
   - TENSION LAP WITH 100% OF BAR SPLICED
   - CLASS C SPLICE
   - AASHTO LRFD SPECIFICATIONS
   - COATED BARS

   FOR BOTTOM BARS WITH 1" CLEAR COVER, INCREASE THIS SPLICE LENGTH TO 3'-10". DESIGNER TO ADJUST LENGTH OF SPLICE FOR DESIGNS BEYOND THESE PARAMETERS.
7. FOR STAGE CONSTRUCTION CLOSURE POURS, PLACE CONCRETE AFTER LONGITUDINAL POST-TENSIONING.
8. FOR NON-STAGE CONSTRUCTION LONGITUDINAL CLOSURE POURS, CONCRETE MAY BE PLACED AFTER POST-TENSIONING.



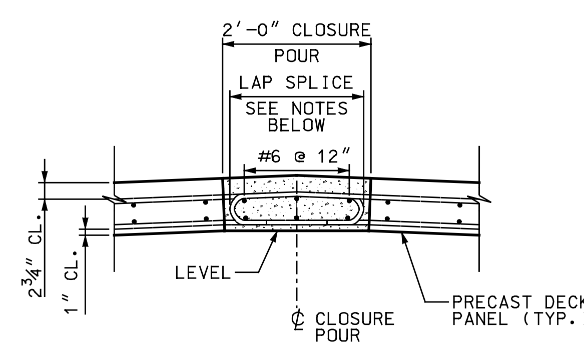
SECTION G: CLOSURE POUR

SECTION AT CROWN



SECTION G: CLOSURE POUR SUPERELEVATED SECTION

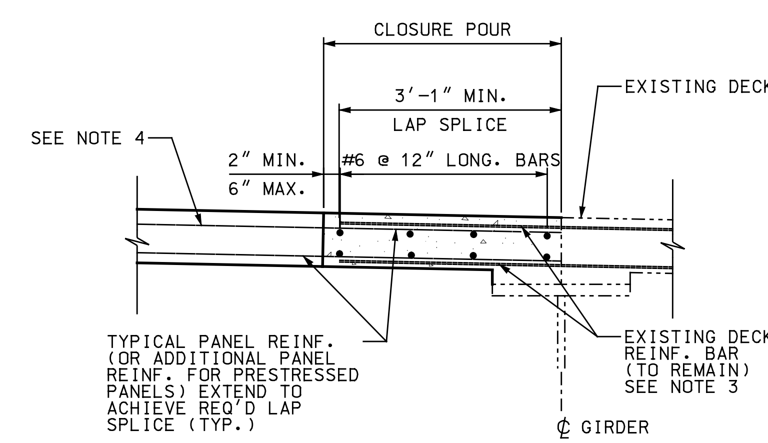
(SHOWING STEEL GIRDERS)



SECTION G: OPTIONAL CLOSURE POUR

SHOWING SECTION AT CROWN - SECTION AT OTHER LOCATIONS SIMILAR

NOTES:
1. HOOKED BARS ARE REQUIRED IN TENSILE ZONES (TOP NEAR GIRDERS, BOTTOM NEAR MID-BAY).
2. HOOKED BARS ARE NOT REQUIRED IN COMPRESSION ZONES (BOTTOM NEAR GIRDERS, TOP NEAR MID-BAY).
3. ROTATE HOOKS TO PROVIDE REQUIRED COVER.
4. LAP SPLICE IN COMPRESSION ZONES TO BE 22½ IN.



CLOSURE POUR TO EXISTING DECK SECTION

(SHOWING STEEL GIRDERS)

PROTECT EXISTING REINFORCING BARS WHEN REMOVING THE PORTION OF EXISTING DECK. BEFORE MAKING THE CLOSURE POUR, ALL EXISTING REINFORCING IS TO BE CLEANED OF RUST AND FOREIGN MATERIAL. RECOAT BAR IF ORIGINAL WAS EPOXY COATED AND DAMAGED. REPAIR OR REPLACE ANY EXISTING REBAR DAMAGED DURING THE REMOVAL OF CONCRETE.

NO. | DATE | BY | REVISION REMARKS
DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

CLOSURE POUR DETAILS

PIN

PROJECT NUMBER

COUNTY

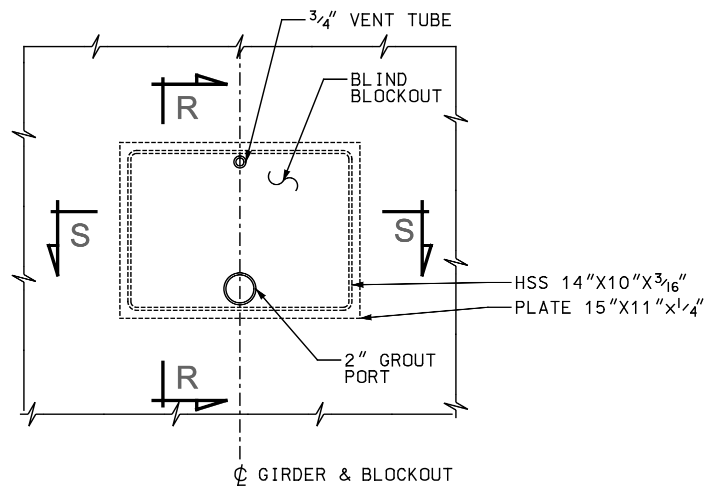
PDP - 8

DRG. NO.

SHT. OF

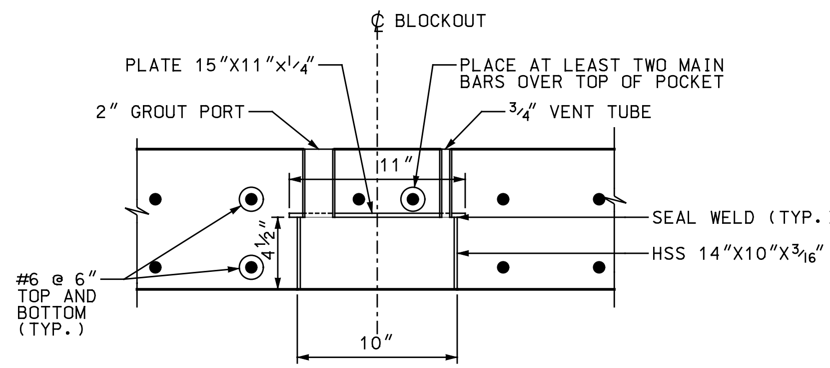
PDP-08_CLOSURE_POUR.dgn

7/19/2010

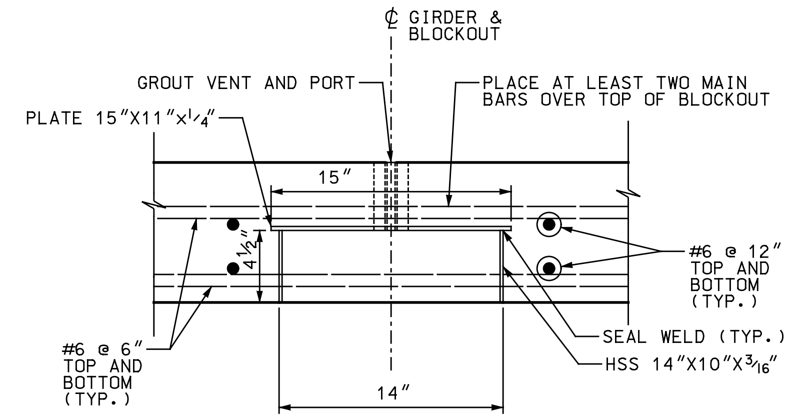


TYPICAL BLIND BLOCKOUT PLAN

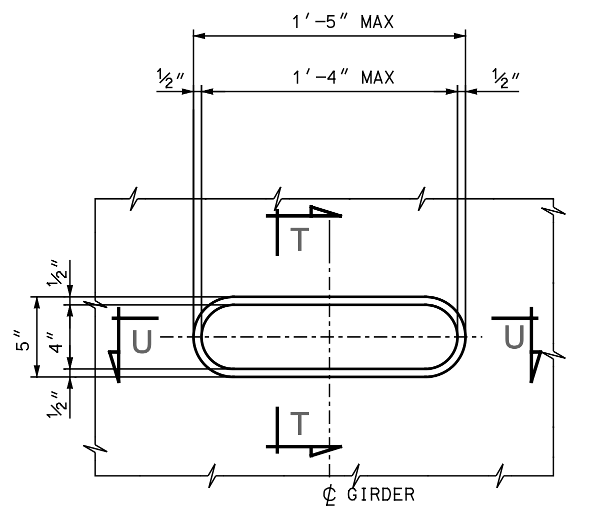
NOTES: 1. REINFORCEMENT NOT SHOWN FOR CLARITY.
2. GALVANIZE STEEL FOR BLOCKOUTS
3. USE PLASTIC PIPE FOR PORTS AND VENT TUBES.



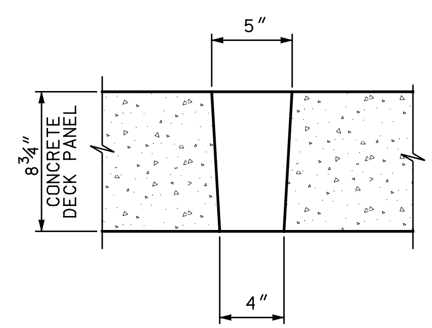
SECTION R



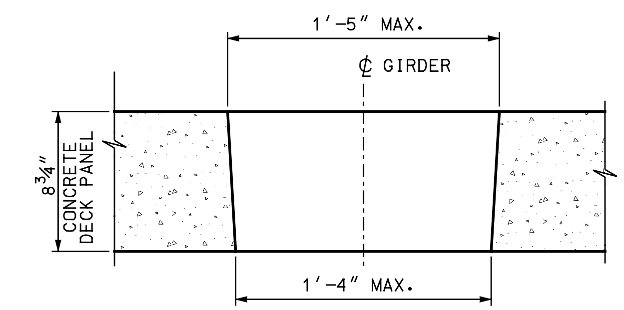
SECTION S



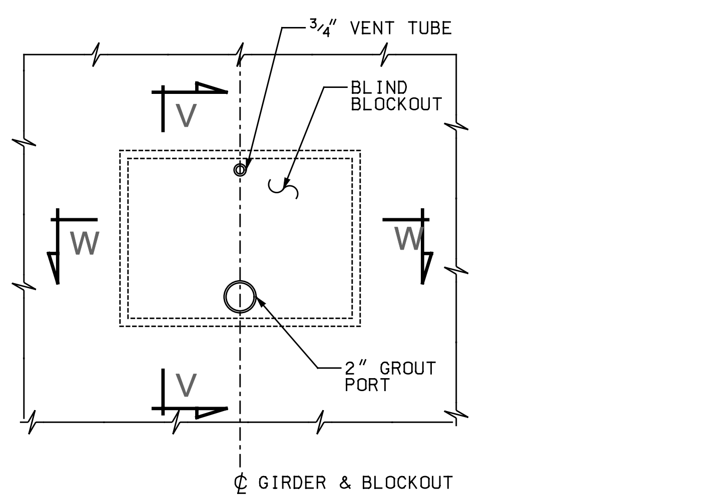
OPTIONAL SHEAR STUD BLOCKOUT PLAN WITH SHEAR STUDS



SECTION T

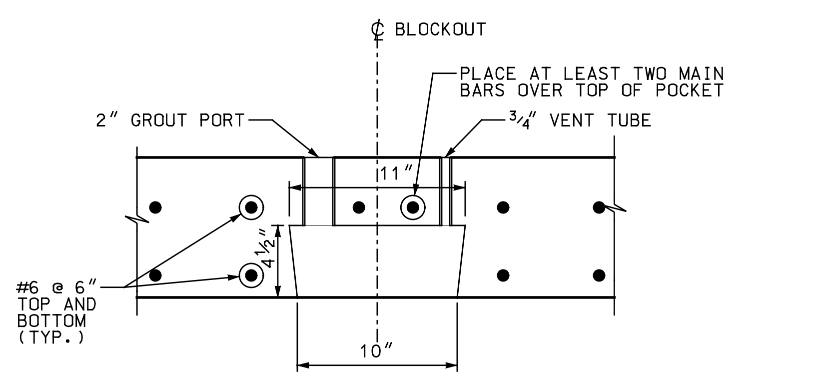


SECTION U

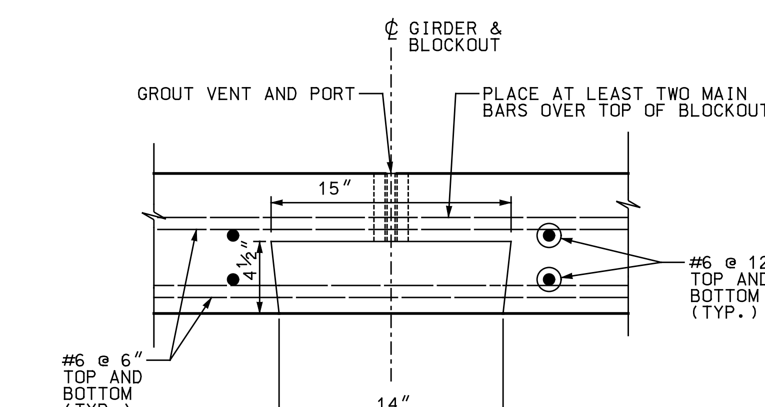


OPTIONAL BLIND BLOCKOUT PLAN

NOTES: 1. REINFORCEMENT NOT SHOWN FOR CLARITY.
2. METHOD OF FORMING BLOCKOUT AND PORTS BY FABRICATOR.
3. USE PLASTIC PIPE FOR PORTS AND VENT TUBES.



SECTION V



SECTION W

NO.	DATE	BY	REVISION REMARKS	CHECK

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER
DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

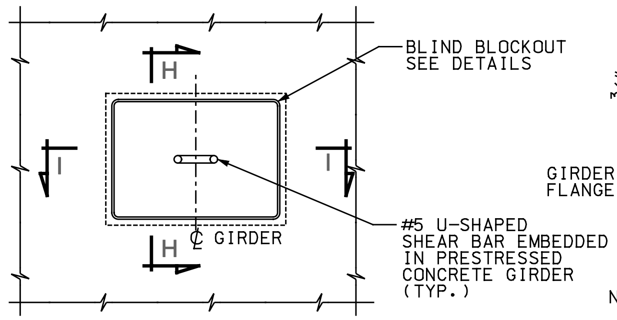
TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
SHEAR CONNECTOR BLOCKOUT DETAILS

PIN
PROJECT NUMBER
COUNTY
PDP - 9
DRG. NO.
SHT. OF

PDP-09 BLOCKOUT DETAILS.dgn
7/19/2010

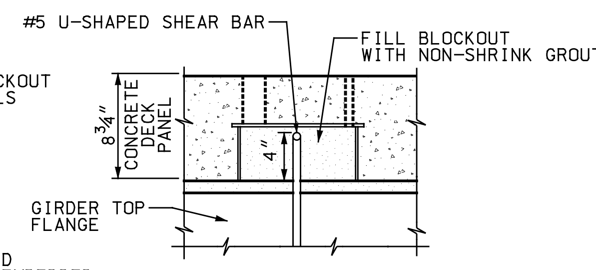
NOTES

1. INCLUDE COST OF WELDED STUD SHEAR CONNECTORS IN THE COST OF THE PRESTRESSED CONCRETE DECK PANEL. INCLUDE COST OF THE U-SHAPED REINFORCING BARS IN THE COST OF THE PRESTRESSED CONCRETE GIRDER.
2. INCLUDE COST OF ALL NON-SHRINK GROUT IN THE COST OF THE PRESTRESSED CONCRETE DECK PANEL.
3. USE A HEAVY BROOM FINISH ON TOP SURFACE OF DECK.
4. DESIGNER TO DETERMINE AND DETAIL THE MINIMUM SPACING OF SHEAR STUDS AND SHEAR BARS.
5. FOR SKEWED PANELS, ROTATE BLOCKOUTS TO BE PARALLEL WITH THE PANEL REINFORCING.
6. BLIND BLOCKOUTS FORMED WITH STEEL BOX SHOWN. OPTIONAL FORMED BLOCKOUTS SIMILAR.



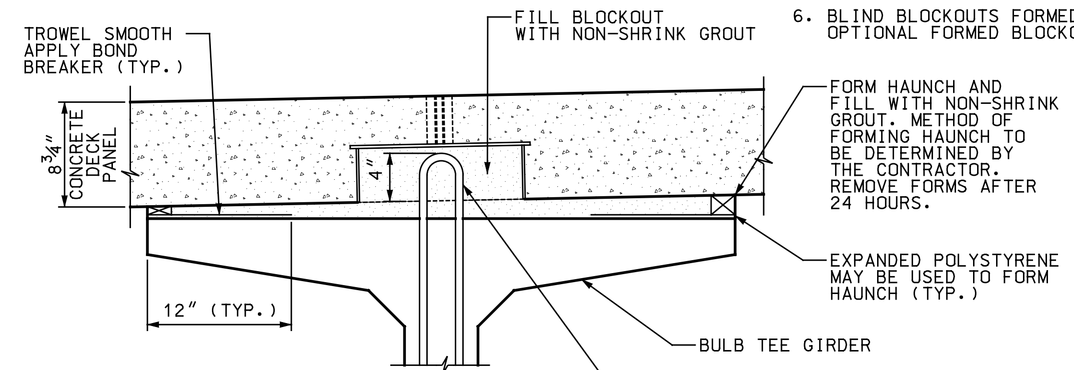
PLAN: U-SHAPED SHEAR BAR BULB TEE GIRDERS: OPTION 1

NOTE: ONE BAR SHOWN. MULTIPLE BARS MAY BE USE AND BUNDLED TO PROVIDE ADEQUATE SHEAR RESISTANCE.

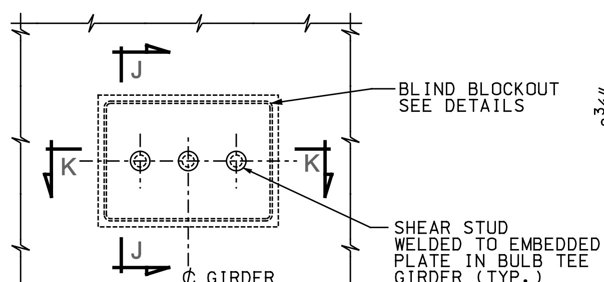


SECTION H: OPTION 1

NOTE: ONE BAR SHOWN. MULTIPLE BARS MAY BE USE AND BUNDLED TO PROVIDE ADEQUATE SHEAR RESISTANCE.

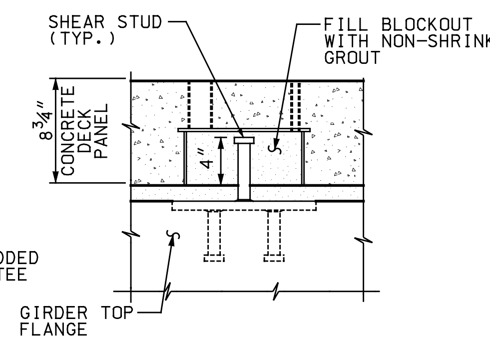


SECTION I: OPTION 1



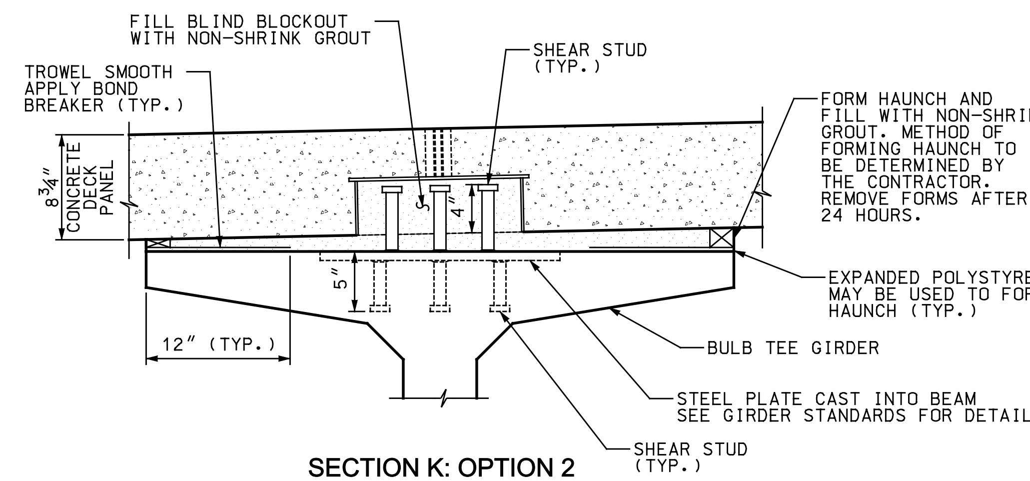
PLAN: SHEAR STUD CONNECTOR BULB TEE GIRDERS: OPTION 2

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED

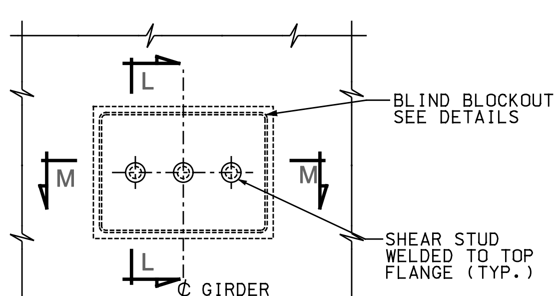


SECTION J: OPTION 2

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED

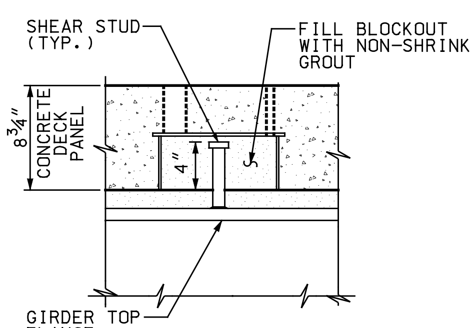


SECTION K: OPTION 2



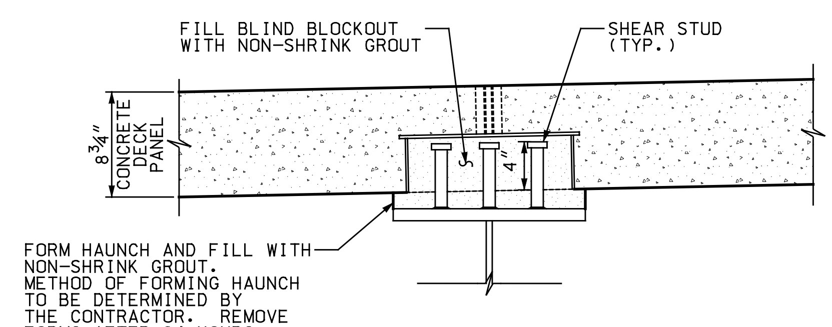
PLAN: STEEL GIRDER DETAILS

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED



SECTION L

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED



SECTION M

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

SHEAR CONNECTOR DETAILS NEW GIRDERS

NO.	DATE	BY	REVISION REMARKS

APPROVAL RECOMM. DATE | SENIOR DESIGN ENGINEER | DESIGN | CHECK | DRAWN | CHECK

APPROVED FOR USE BY UDOT DATE | UDOT DESIGN MANAGER | QUANT. | CHECK

PIN

PROJECT NUMBER

COUNTY

PDP - 10

DRG. NO.

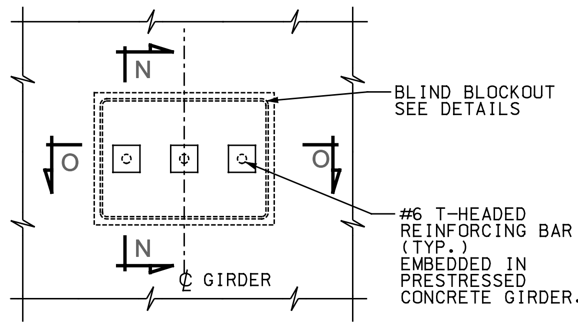
SHT. OF

PDP-10 SHEAR CONNECTOR NEW.dgn

7/19/2010

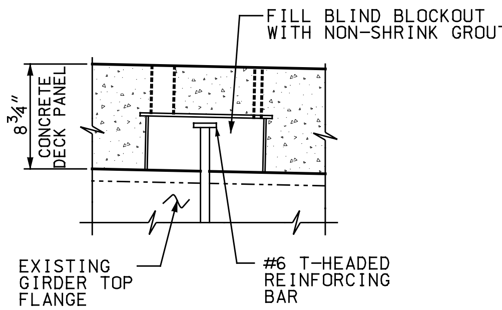
NOTES

1. INCLUDE COST OF WELDED STUD SHEAR CONNECTORS AND T-STUD SHEAR CONNECTORS IN THE COST OF THE PRESTRESSED CONCRETE DECK PANEL.
2. INCLUDE COST OF ALL NON-SHRINK GROUT IN THE COST OF THE PRESTRESSED CONCRETE DECK PANEL.
3. USE A HEAVY BROOM FINISH ON TOP SURFACE OF DECK.
4. FOR SKEWED PANELS, ROTATE BLOCKOUTS TO BE PARALLEL WITH THE PANEL REINFORCING.
5. BLIND BLOCKOUTS FORMED WITH STEEL BOX SHOWN. OPTIONAL FORMED BLOCKOUTS SIMILAR.



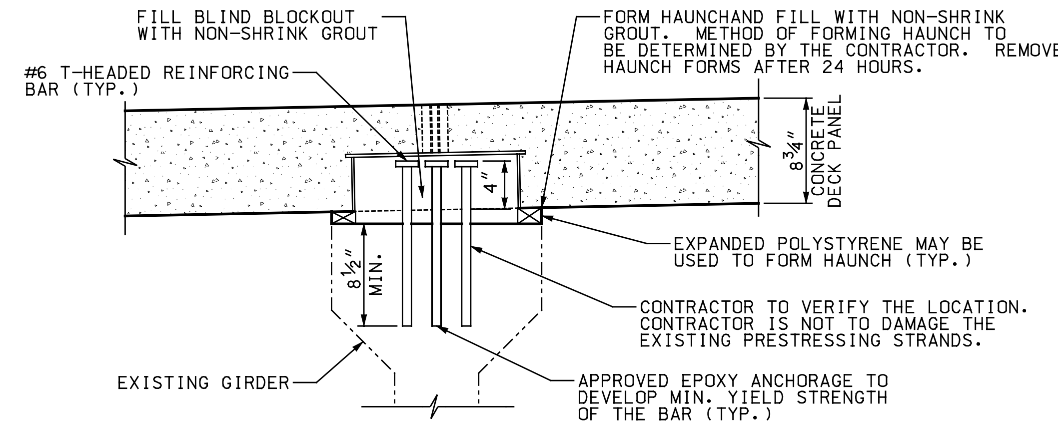
PLAN: T-HEADED SHEAR STUD CONNECTOR ON CONCRETE GIRDERS

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED

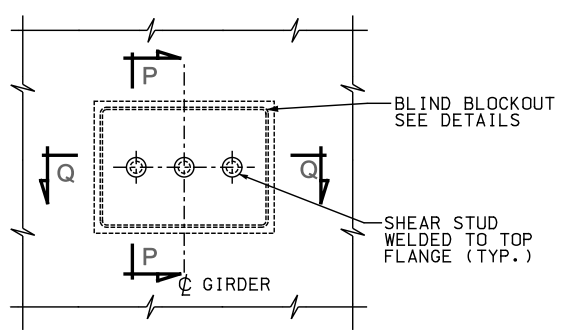


SECTION N

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED

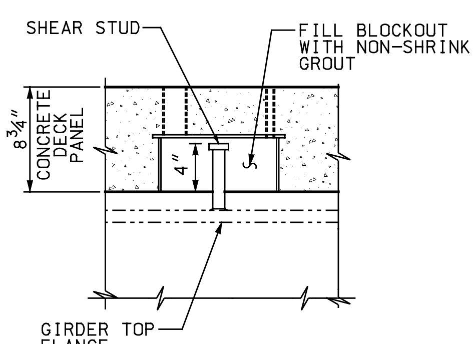


SECTION O



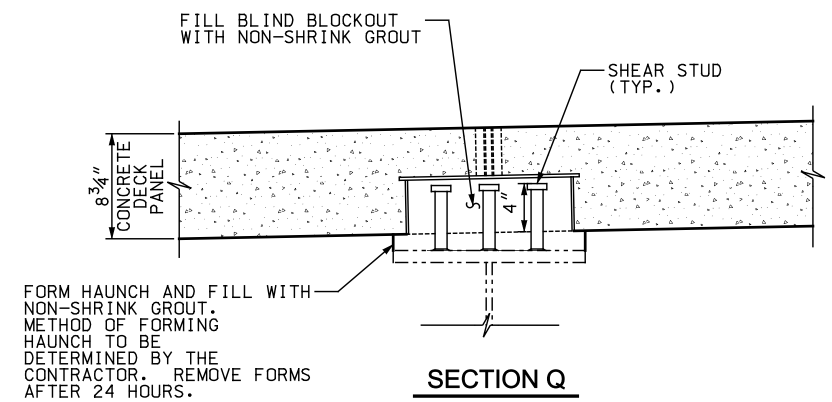
PLAN: STEEL GIRDER DETAILS

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED

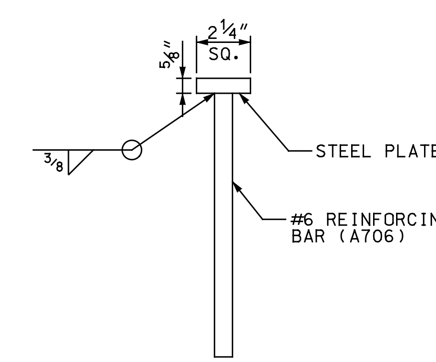


SECTION P

NOTE: ONE ROW OF CONNECTORS SHOWN. MULTIPLE ROWS MAY BE USED



SECTION Q



T-HEADED REINFORCING BAR DETAIL

NO. | DATE | BY | REVISION REMARKS | CHECK | CHECK | CHECK
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER | DESIGN | DRAWN | QUANT.
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

SHEAR CONNECTOR DETAILS EXISTING GIRDERS

PIN

PROJECT NUMBER

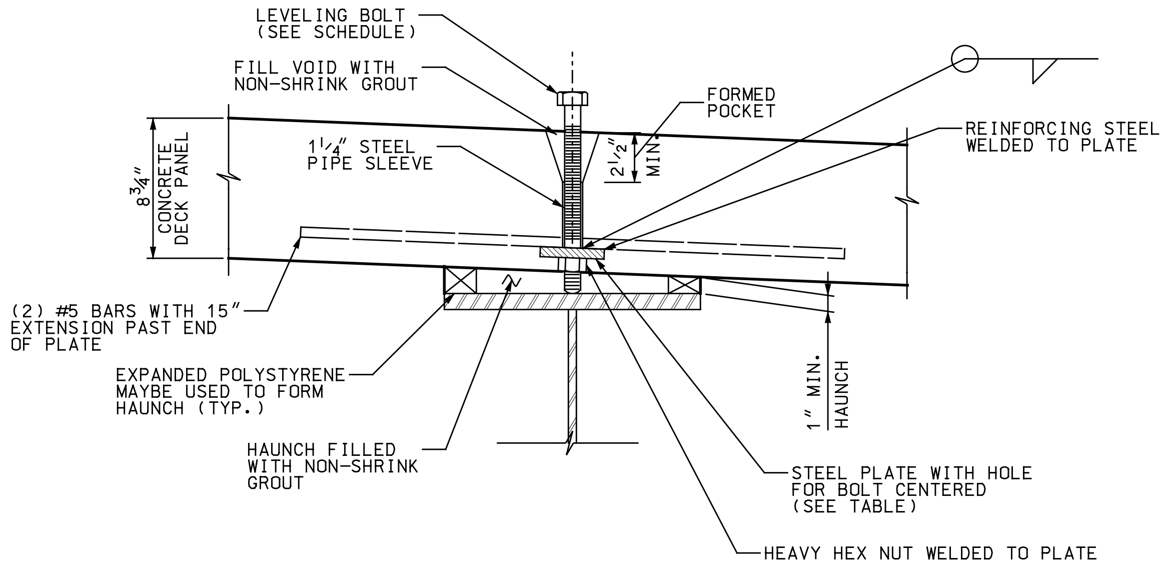
COUNTY

PDP - 11

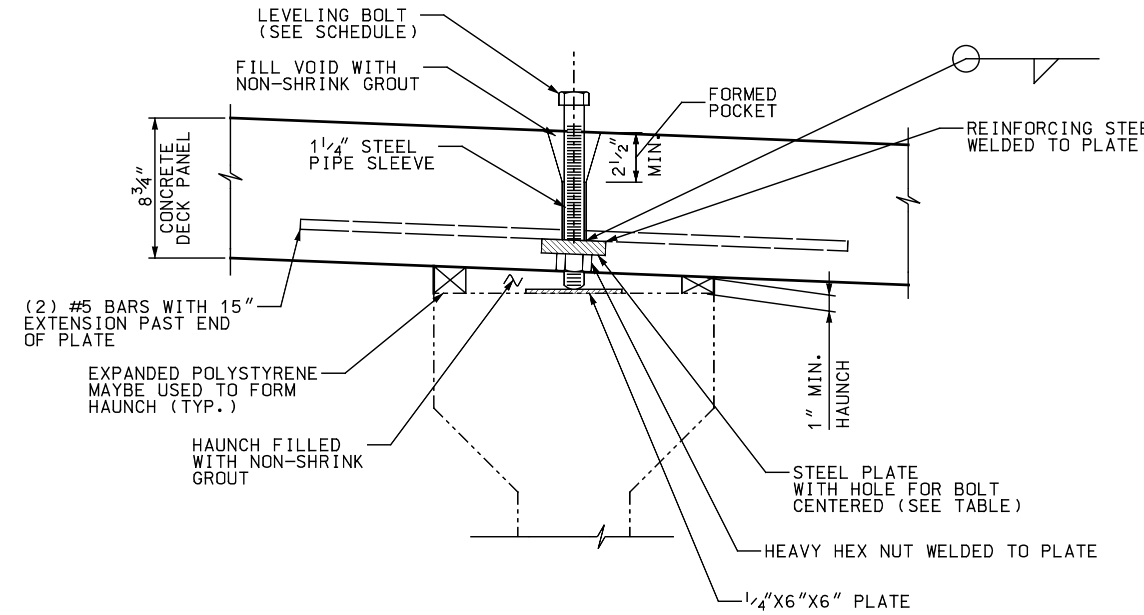
DRG. NO.

SHT. OF

7/19/2010 PDP-11 SHEAR CONNECTOR EXISTING.dgn



VERTICAL ADJUSTMENT DETAIL ON STEEL GIRDER



VERTICAL ADJUSTMENT DETAIL ON CONCRETE GIRDER

VERTICAL ADJUSTMENT SCHEDULE

SERVICE LOAD	BOLT DIA.	STEEL PLATE WITH HOLE FOR BOLT CENTERED
10 K	1"	4"x4"x⅝"
20 K	1¼"	4"x4"x⅞"

NO. | DATE | BY | REVISION REMARKS | CHECK

DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
VERTICAL ADJUSTMENT DETAILS

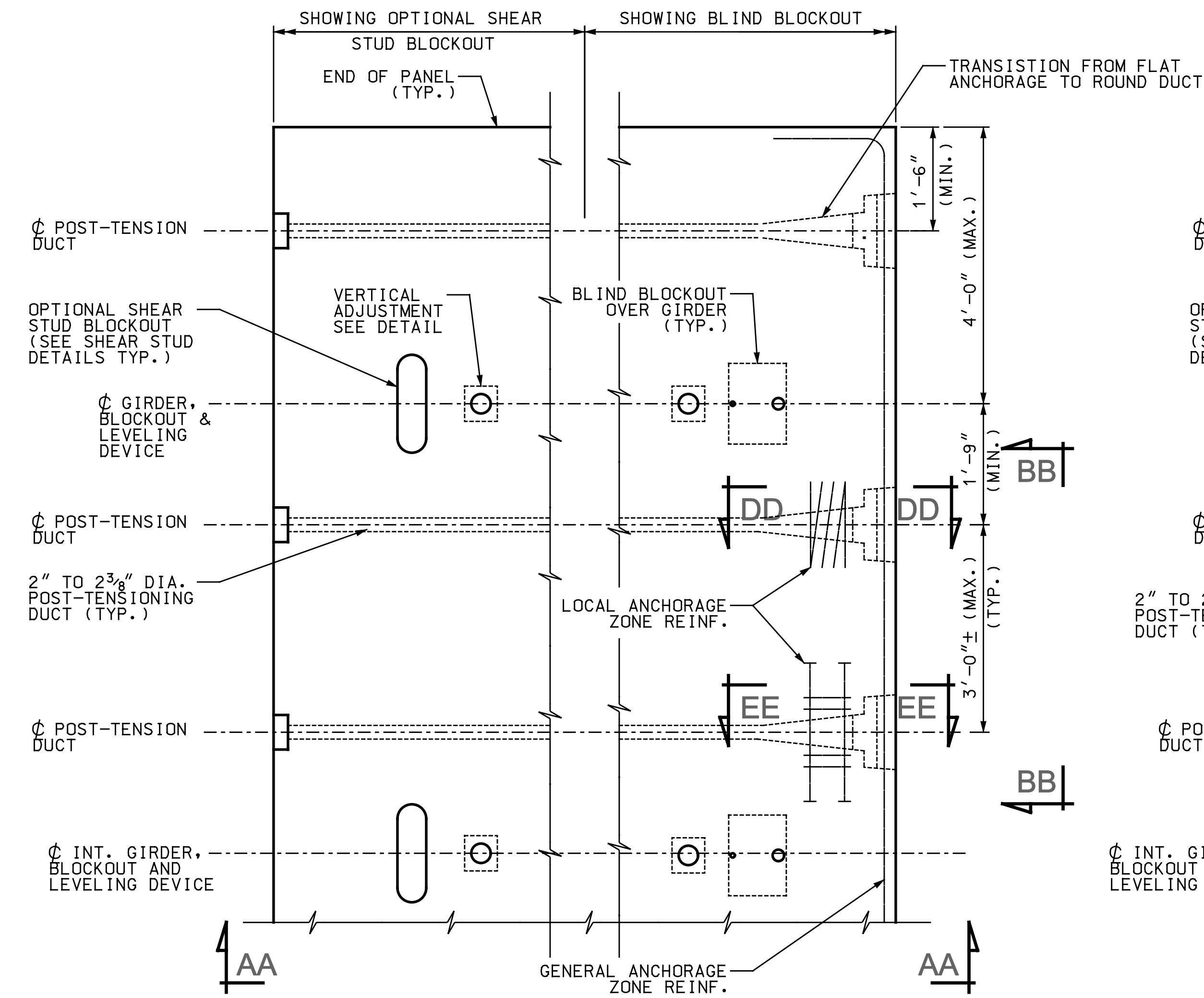
PIN
PROJECT NUMBER

COUNTY

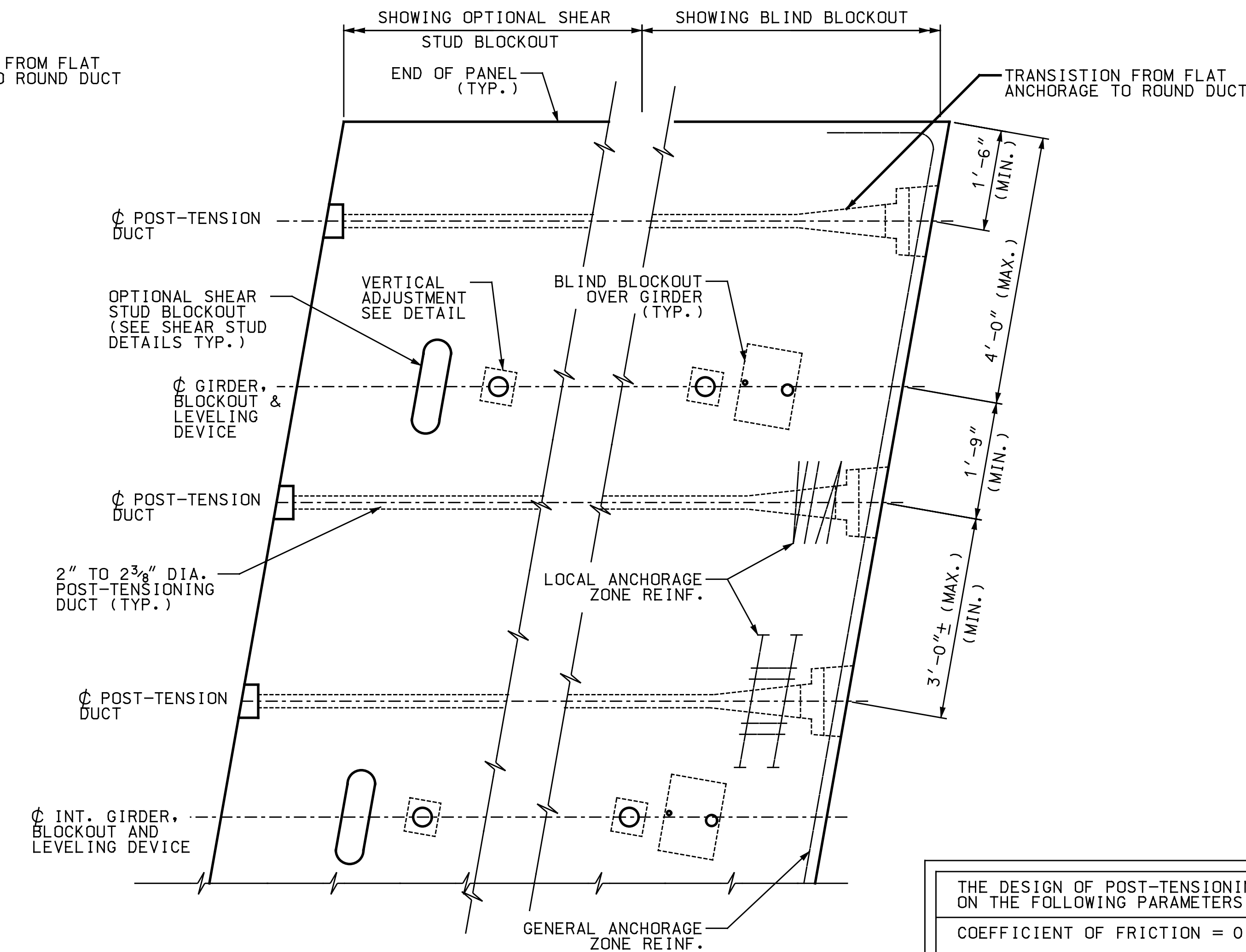
PDP - 12
DRG. NO.

SHT. OF

7/19/2010 PDP-12 VERTICAL ADJUSTMENT.dgn

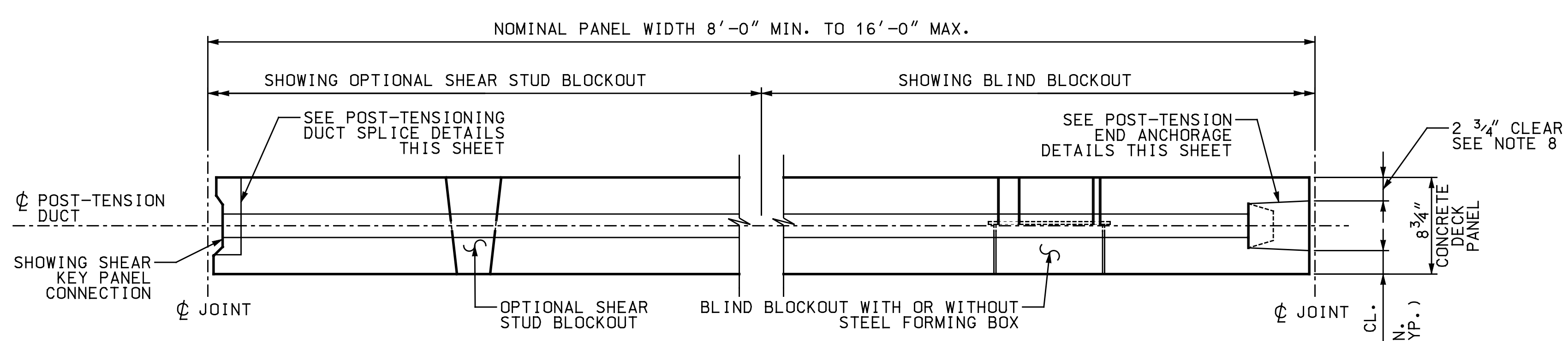


POST-TENSION END ANCHORAGE DETAIL

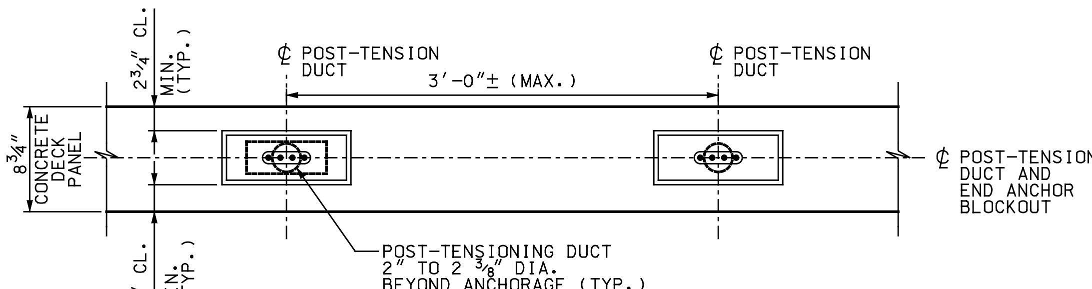


SKEWED POST-TENSION END ANCHORAGE DETAIL

THE DESIGN OF POST-TENSIONING IS BASED ON THE FOLLOWING PARAMETERS:
COEFFICIENT OF FRICTION = 0.XX
WOBBLE FRICTION COEFFICIENT = 0.000X
P-JACKING PER STRAND = XX.X KIPS.
P-FINAL PER STRAND = XX.X KIPS (AFTER LOSSES DUE TO FRICTION, ANCHORAGE SET, AND ELASTIC SHORTENING).
IF THE PROPOSED DUCT DOES NOT MEET THESE VALUES, THEN THE CONTRACTOR TO ADJUST THE JACKING FORCE TO PRODUCE THE FINAL POST-TENSIONING FORCE LISTED.



SECTION AA: TYPICAL TRANSVERSE PANEL SECTION



SECTION BB: TYPICAL LONGITUDINAL PANEL SECTION

POST-TENSIONING NOTES:

1. USE 0.5" DIA. GRADE 270 LOW RELAXATION STRANDS CONFORMING TO ASTM A416.
2. USE 4 STRANDS PER DUCT MAXIMUM.
3. GALVANIZE BEARING PLATE ANCHOR HEADS AND METAL TRUMPETS AT ANCHORAGES. DO NOT GALVANIZE STRAND GRIPPING WEDGES.
4. BEGIN STRESSING AT CENTER OF PANELS. DO NOT ALLOW MORE THAN 12.5% OF THE PRESTRESSING FORCE TO BE ECCENTRIC AT ANY TIME. SUBMIT STRESSING SEQUENCE TO ENGINEER FOR APPROVAL PRIOR TO WORK.
5. DECK PANELS MUST BE ALLOWED TO SLIDE ON GIRDERS DURING POST-TENSIONING.
6. THE CONTRACTOR IS RESPONSIBLE FOR DESIGN OF ALL POST-TENSIONING ELEMENTS AND ANCHORAGE ZONE REINFORCEMENT (REQUIRED FOR SPLITTING, BURSTING, SPALLING, ETC.). DESIGN MUST CONFORM WITH AASHTO LRFD SPECIFICATIONS. TYPICAL REINFORCING FOR TWO DIFFERENT MANUFACTURERS ARE SHOWN.
7. SEQUENCE OF CONSTRUCTION:
   A. ERECT DECK PANELS.
   B. ADJUST PANELS TO GRADE USING LEVELING DEVICES.
   C. INSTALL POST-TENSIONING STRAND LOOSE IN DUCTS.
   D. PLACE GROUT IN TRANSVERSE JOINTS ONLY. CURE TO 500 PSI.
   E. STRESS POST-TENSIONING STRAND AFTER GROUT ATTAINS A STRENGTH OF 500 PSI.
   F. GROUT POST-TENSIONING DUCTS.
   G. GROUT SHEAR CONNECTOR POCKETS AND CAMBER STRIPS.
   H. LEVELING BOLTS MAY BE REMOVED. GROUT BOLT RECESS.
   I. PLACE CLOSURE POURS.
8. ANCHORAGE ASSEMBLY AND DUCTS MAY BE LOWERED UP TO ½" FROM CENTER OF SLAB IN ORDER TO PROVIDE 2¾" TOP COVER FOR REINFORCING ABOVE THE DUCT.

REVISION REMARKS | CHECK | DESIGN | CHECK | DRAWN | CHECK | QUANT.

NO. | DATE | BY

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER

APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

TYPICAL POST-TENSIONING DETAILS 1

PIN

PROJECT NUMBER

COUNTY

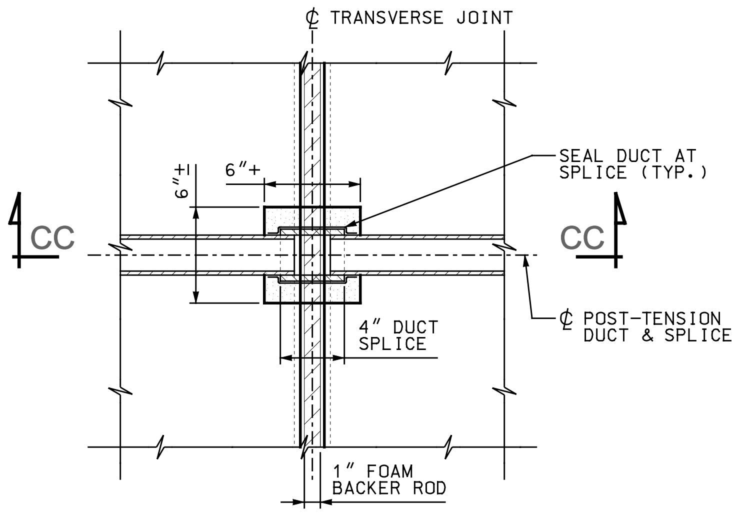
PDP - 13

DRG. NO.

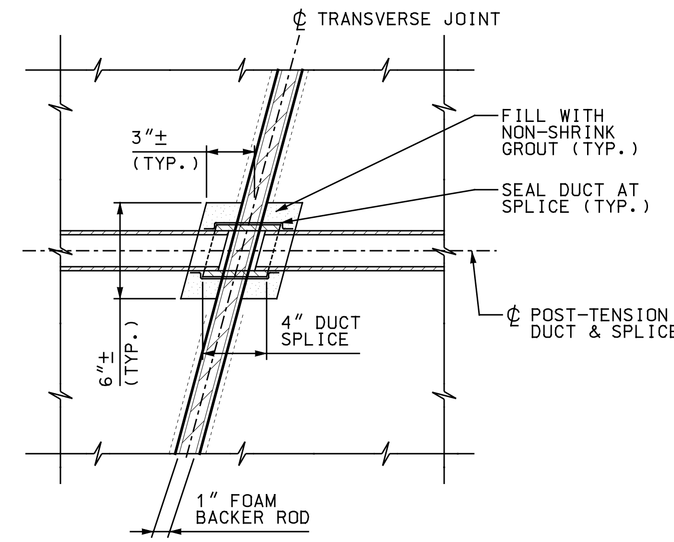
SHT. OF

PDP-13_POST-TENSION DETAILS 1.dgn

7/19/2010



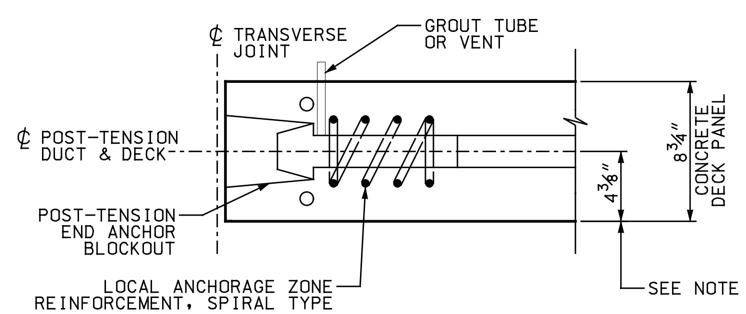
DETAIL AT DUCT CONNECTION



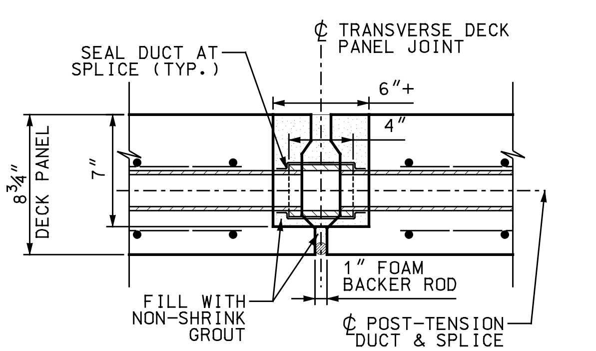
SKEWED DETAIL AT DUCT CONNECTION

POST-TENSIONING NOTES:

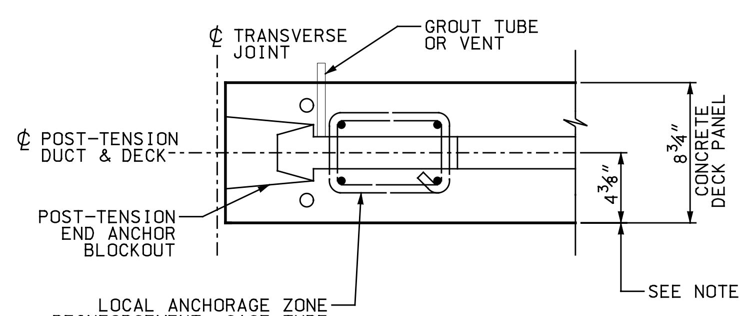
1. ANCHORAGE ASSEMBLY AND DUCTS MAY BE LOWERED UP TO ½" FROM MID-DEPTH OF SLAB IN ORDER TO PROVIDE 2¾" TOP COVER FOR REINFORCING ABOVE THE DUCT.



SECTION DD: LOCAL ANCHORAGE ZONE REINFORCEMENT OPTION 1



SECTION CC: POST-TENSION DUCT SPLICE



SECTION EE: LOCAL ANCHORAGE ZONE REINFORCEMENT OPTION 2

NO.	DATE	BY	REVISION REMARKS

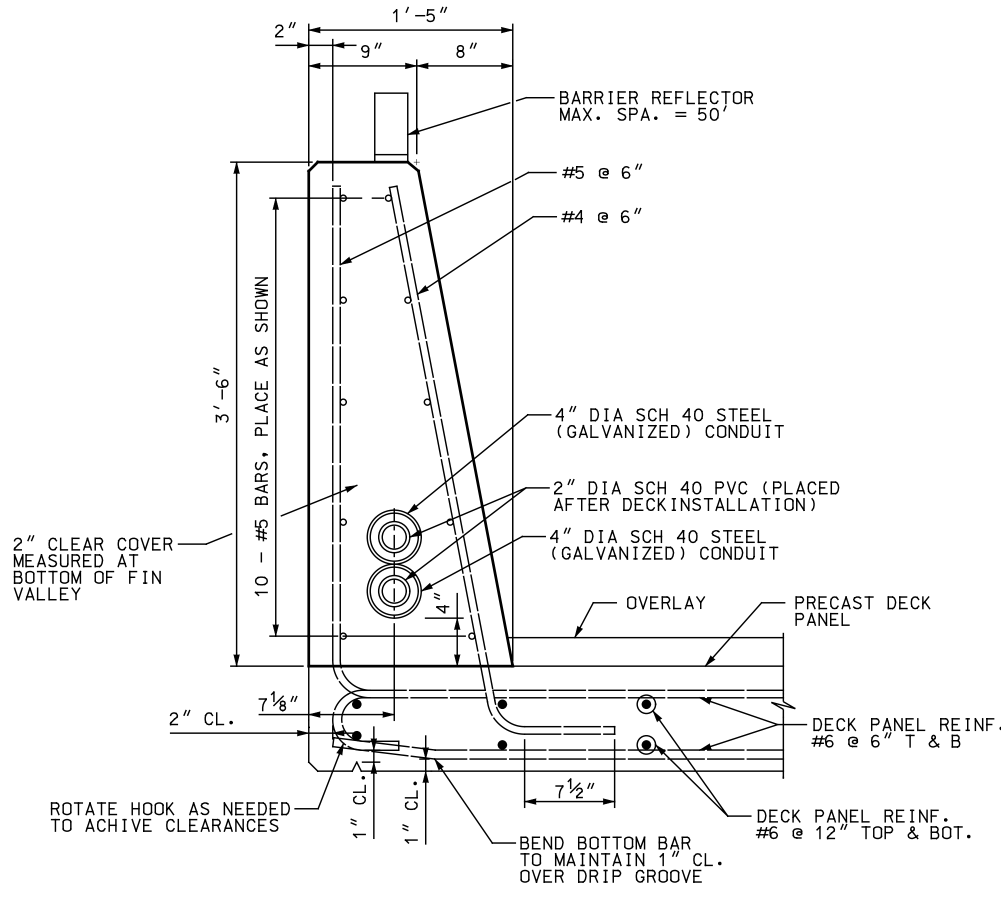
DESIGN ___ CHECK ___
DRAWN ___ CHECK ___
QUANT. ___ CHECK ___
APPROVAL RECOMM. ___ DATE ___ SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT ___ DATE ___ UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
TYPICAL POST-TENSIONING DETAILS 2

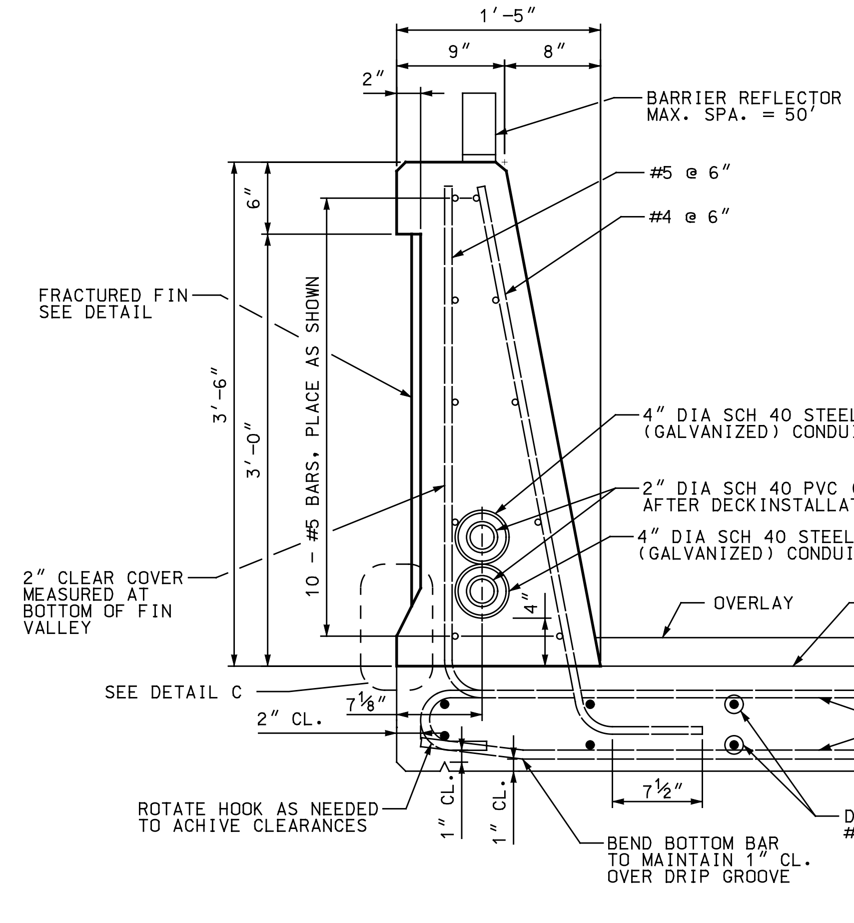
PIN ___
PROJECT NUMBER ___

COUNTY
PDP - 14
DRG. NO.
SHT. OF



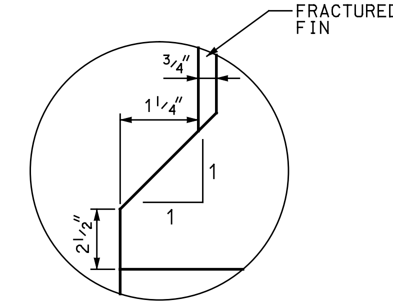
SECTION Y - SMOOTH REAR FACE

NOTE: PRECAST PANEL SHOWN, PRESTRESSED PANEL SIMILAR

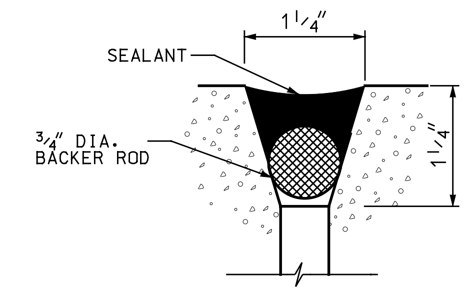


SECTION Y - FRACTURED FIN REAR FACE

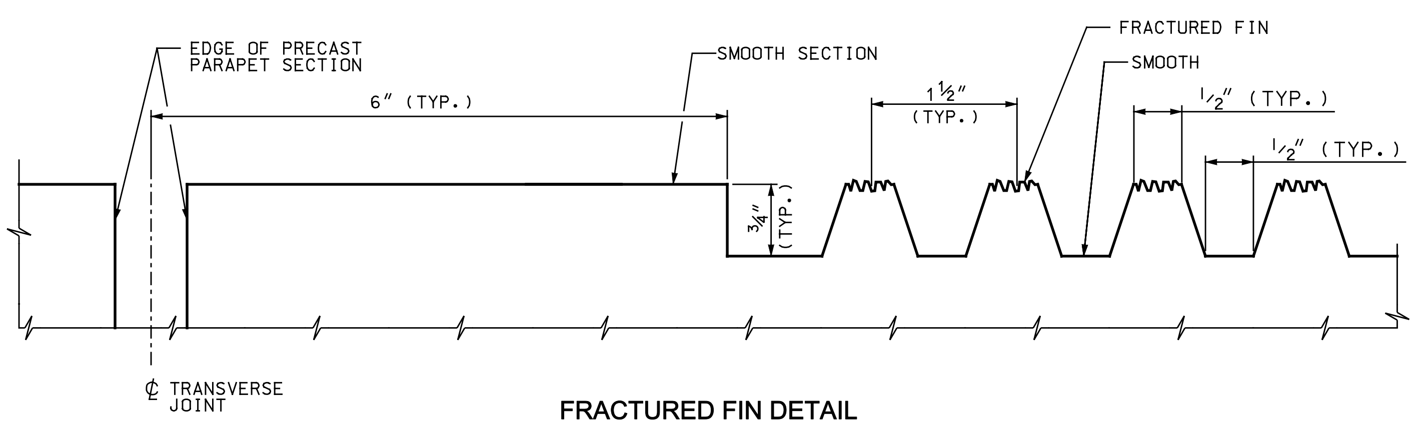
NOTE: PRECAST PANEL SHOWN, PRESTRESSED PANEL SIMILAR



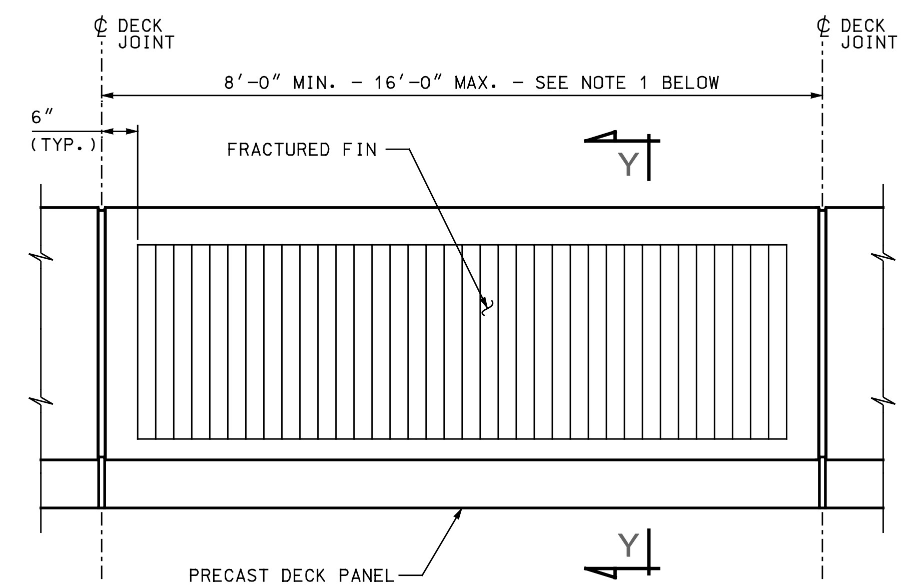
DETAIL C



PARAPET JOINT DETAIL



FRACTURED FIN DETAIL



ELEVATION

NOTES:
1. LENGTH BASED ON PRECAST DECK PANEL.
   LENGTH AS DETERMINED BY DESIGNER.

NOTES

1. EXTEND SEALANT AND FOAM BACKER ROD FROM DECK TOP TO TOP OF PARAPET ON THE INSIDE PARAPET FACE, AND ACROSS TOP OF PARAPET.
2. ADJUST BAR SPACING AS REQUIRED TO NOT EXCEED MAXIMUM SPACING SHOWN.
3. FRACTURED FIN PARAPETS AND CONSTANT SLOPE ARE SHOWN, OTHERS MAY BE USED.
4. VALID FOR TL-4.

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

PARAPET DETAILS 1

NO. | DATE | BY | REVISION REMARKS | CHECK | CHECK | CHECK

DESIGN | DRAWN | QUANT.

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER

APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

PIN

PROJECT NUMBER

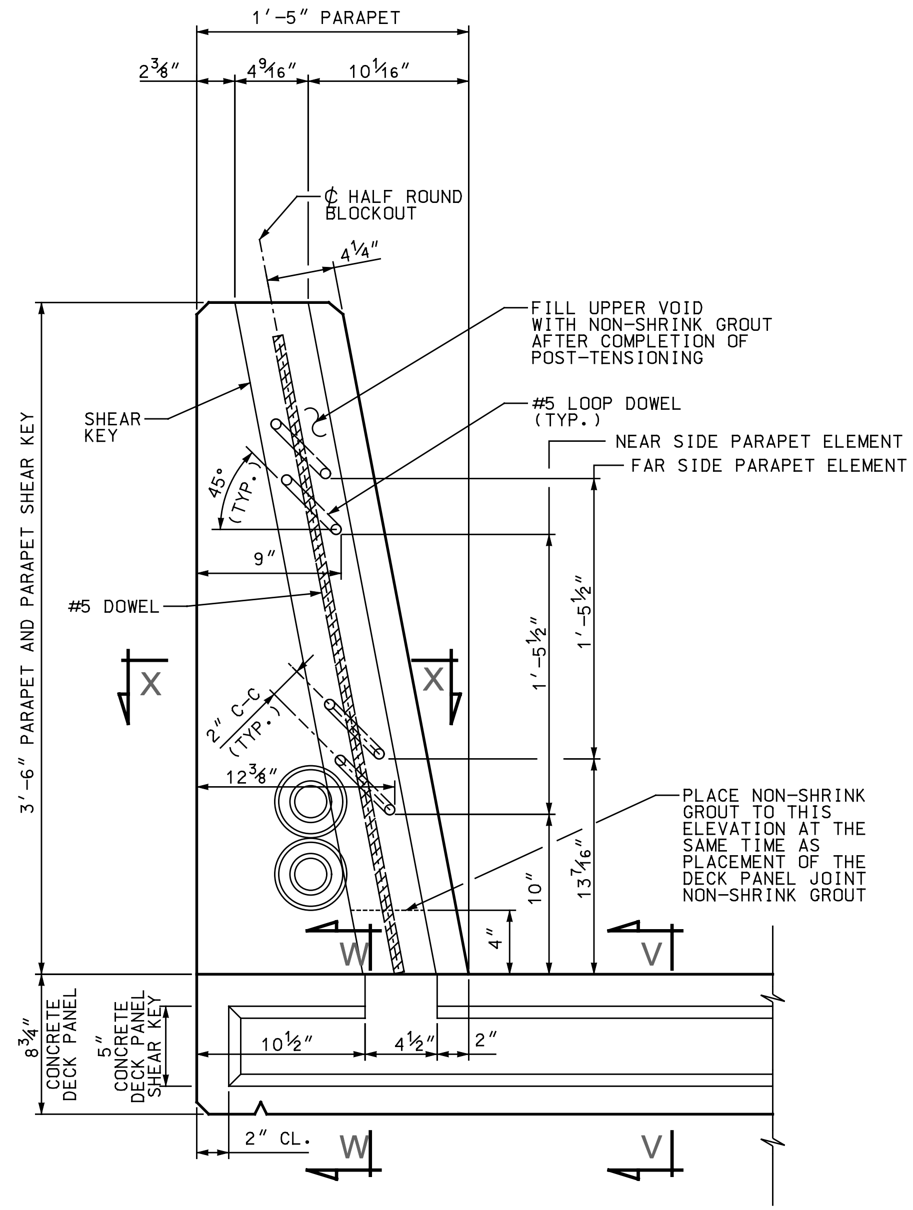
COUNTY

PDP - 15

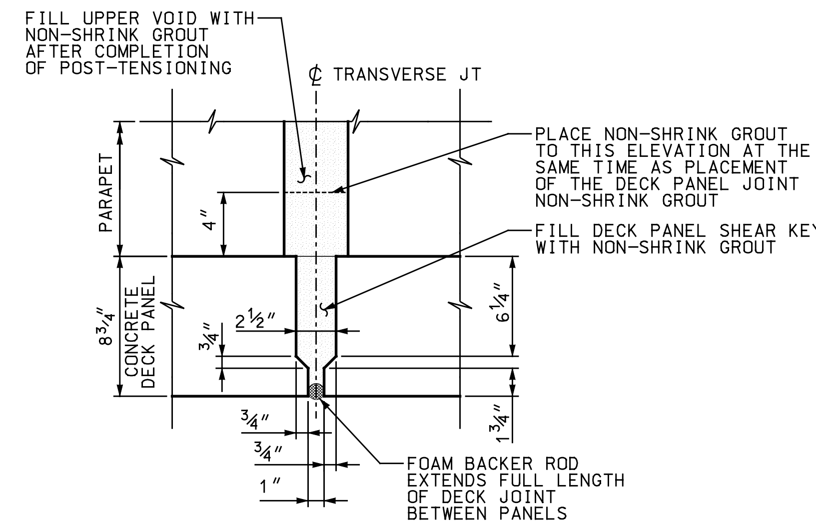
DRG. NO.

SHT. OF

7/19/2010 PDP-15_PARAPET DETAILS 1.dgn

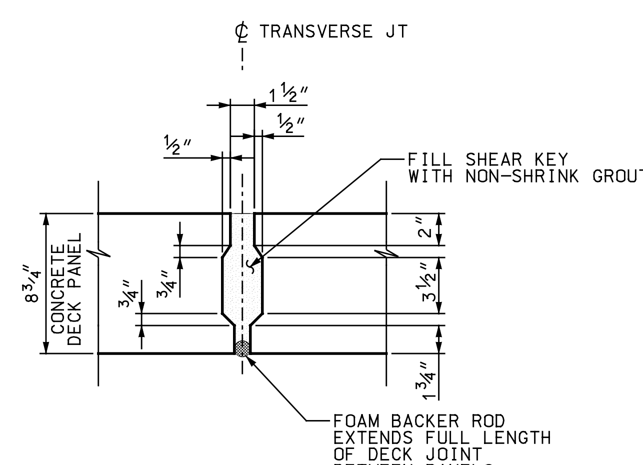


SHEAR KEY SECTION THROUGH PARAPET AND DECK PANEL

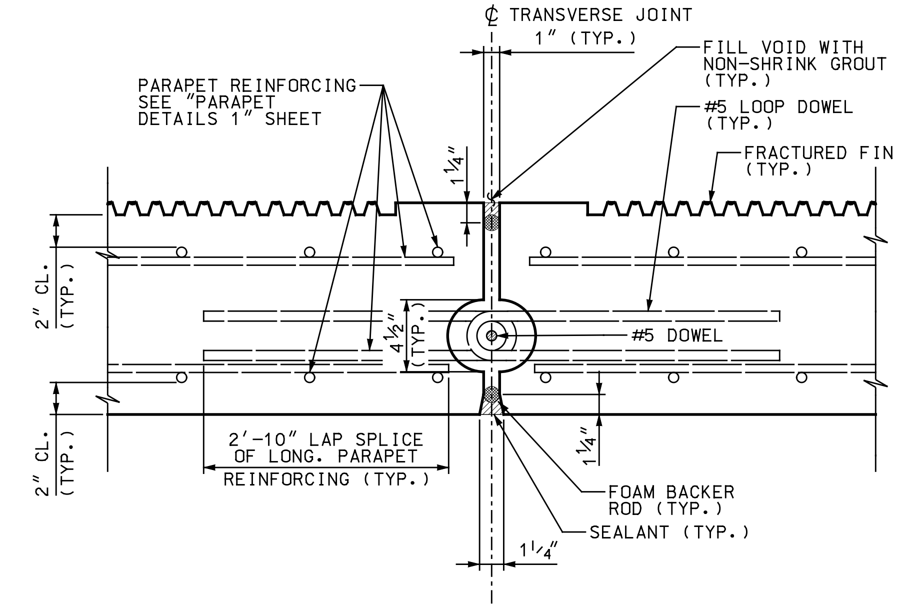


SECTION W: SHEAR KEY THROUGH DECK PANEL AND PARAPET

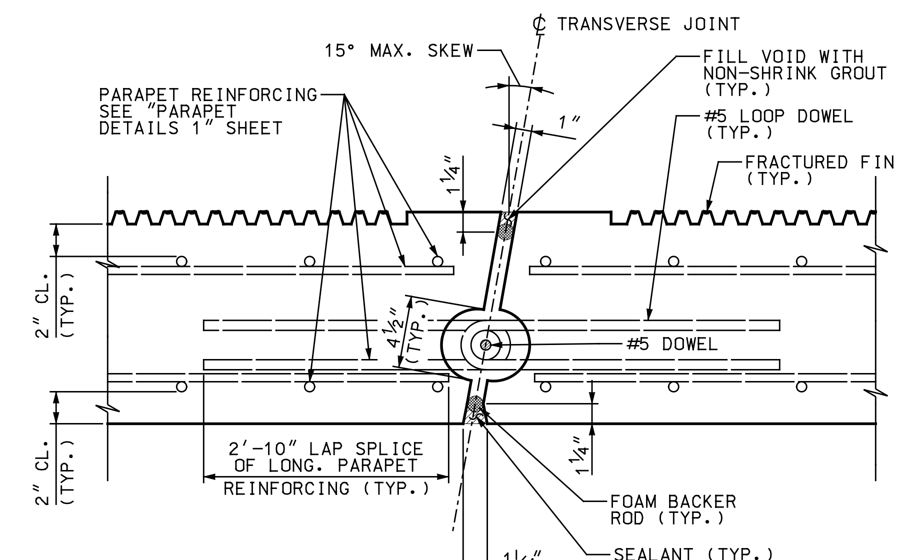
NOTE: PARAPET AND SHEAR KEY REINFORCING NOT SHOWN FOR CLARITY.



SECTION V: SHEAR KEY THROUGH DECK PANEL



SECTION X: PRECAST PARAPET THROUGH SHEAR KEY



SKEWED SECTION X: PRECAST PARAPET THROUGH SHEAR KEY

UTAH DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

TYPICAL DETAIL SHEET
FULL DEPTH PRECAST CONCRETE DECK PANELS
PARAPET DETAILS 2

REVISION REMARKS	CHECK	CHECK	CHECK
NO.	DATE	BY	

DESIGN | DRAWN | QUANT.
APPROVAL RECOMM. DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT DATE | UDOT DESIGN MANAGER

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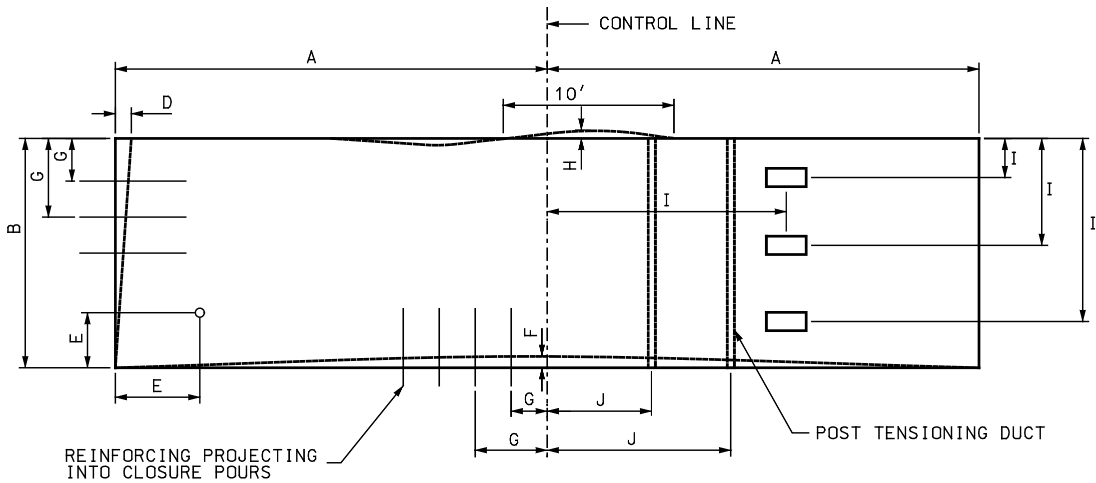
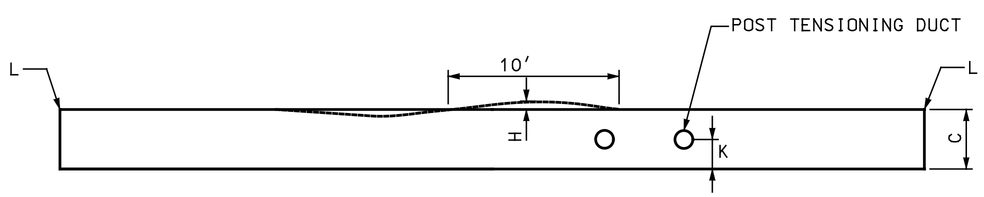
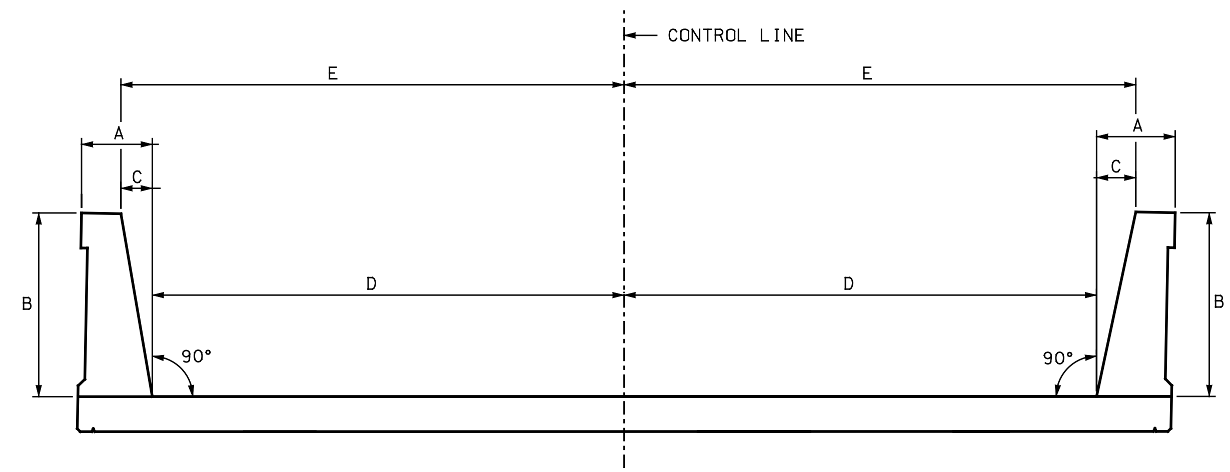
PROJECT NUMBER
COUNTY
PDP - 16
DRG. NO.
SHT. OF



PLAN



ELEVATION



PRECAST PARAPET - TRANSVERSE SECTION

DECK PANEL TOLERANCES

A	LENGTH MEASURED FROM CONTROL LINE	± 3/16″
B	WIDTH (OVERALL)	± 1/4″
C	DEPTH (OVERALL)	± 3/16″
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	± 1/4″
E	LOCATION OF LEVELING BOLTS	± 1″
F	SWEEP OVER MEMBER LENGTH:	± 3/8″
G	LOCATION OF PROJECTING REINFORCING MEASURED FROM A COMMON REFERENCE POINT	± 1/2″
H	LOCAL SMOOTHNESS OF ANY SURFACE	± 1/8″ IN 10 FEET
I	LOCATION OF BLOCKOUT FOR SHEAR CONNECTORS	± 1/2″
J	LOCATION OF POST TENSIONING DUCT MEASURED FROM A COMMON REFERENCE POINT	± 1/8″
K	LOCATION OF POST TENSIONING DUCT MEASURED FROM BOTTOM OF PANEL AT EDGE OF PANEL	± 1/8″
L	ERECTION ELEVATION TOLERANCE	± 1/8″

PRECAST PARAPET TOLERANCES

A	BOTTOM WIDTH OF PARAPET	± 1/4″
B	HEIGHT OF PARAPET	± 1/4″
C	HORIZONTAL DISTANCE OF PARAPET SLOPE	± 1/4″
D	LOCATION OF BOTTOM EDGE OF PARAPET	± 1/8″
E	LOCATION OF TOP EDGE OF PARAPET	± 1/8″

REVISION REMARKS | CHECK | CHECK | CHECK
NO. | DATE | BY
DESIGN | DRAWN | QUANT.
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGINEER
APPROVED FOR USE BY UDOT | DATE | UDOT DESIGN MANAGER

UTAH DEPARTMENT OF TRANSPORTATION

STRUCTURES DIVISION

TYPICAL DETAIL SHEET

FULL DEPTH PRECAST CONCRETE DECK PANELS

DECK PANEL TOLERANCES

PIN

PROJECT NUMBER

COUNTY

PDP - 17

DRG. NO.

SHT. OF

7/19/2010 PDP-17 Deck Panel TOLERANCES.dgn

APPENDIX G

DRAWINGS OF BULB-TEE GIRDERS FROM UTAH DOT

Note: These are direct extract from <http://www.udot.utah.gov/main/uconowner.gf?n=14493404283799689>
(Last accessed June 25, 2012)

PRECAST BULB TEE GIRDERS GENERAL NOTES

GUIDELINES

THESE GUIDELINE DRAWINGS CAN BE USED ONLY FOR BRIDGES WHICH HAVE ALL OF THE FOLLOWING CHARACTERISTICS:

SKEW ANGLE: 0 TO 45 DEGREES

SPECIAL CONSIDERATIONS WITH PRECAST/PRESTRESSED PANELS:

FOR USE WITH PRECAST PANELS - MAXIMUM BEAM/GIRDER SPACING = 10'-0"

FOR USE WITH PRESTRESSED PANELS - MAXIMUM BEAM/GIRDER SPACING = 12'-0"

MAXIMUM OVERHANG = 4'-0"

MINIMUM OVERHANG = 1'-0"

INTERMEDIATE DIAPHRAGMS:
1/5 POINTS OF SPAN FOR SPAN LENGTHS GREATER THAN 160'-0"
1/4 POINTS OF SPAN FOR SPAN LENGTHS 120'-0" TO 160'-0"
1/3 POINTS OF SPAN FOR SPAN LENGTHS 80'-0" TO 120'-0"
1/2 POINTS OF SPAN FOR SPAN LENGTHS LESS THAN 80'-0".

DO NOT USE TRANSFORMED SECTION PROPERTIES FOR GIRDER DESIGN.

IMPLEMENTATION

IT IS THE DESIGNER'S RESPONSIBILITY TO:

FILL IN TABLE FOR EACH GIRDER TYPE ON A PROJECT.

CREATE A FRAMING PLAN OF EACH SPAN. SEE PRECAST BULB TEE GIRDER MANUAL SECTION 4.

CREATE TYPICAL TRANSVERSE SECTIONS AS NEEDED. SEE PRECAST BULB TEE GIRDER MANUAL SECTION 4.

CREATE SPECIAL GIRDER END DETAILS AS NEEDED. SUCH AS, VARYING GEOMETRIC END TREATMENTS, EXTENSIONS OF PRESTRESSING STRAND FOR GIRDER ENDS FOR CONTINUITY OF LIVE LOAD.

DESIGN AND CHECK WEB SHEAR REINFORCEMENT ALONG GIRDER SPAN.

DESIGN AND CHECK THAT END REINFORCEMENT DETAILED SATISFIES ALL APPLICABLE CODE PROVISIONS.

DESIGN AND CHECK ALL CHARACTERISTICS RELATED TO REQUIRED CLOSURE POURS.

ACCOUNT FOR THE HAUNCH LOAD CALCULATIONS,BUT DO NOT CONSIDER IT IN THE COMPOSITE SECTIONS PROPERTIES.

ENSURE APPLICABLE GENERAL NOTES ARE INCLUDED IN THE PLAN SET.

GENERAL NOTES

USE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 4TH EDITION WITH ALL INTERIM PROVISIONS EXCEPT AS NOTED OTHERWISE, FOR ALL BULB TEE GIRDER DESIGNS

USE AN HL-93 LOAD INCLUDING A 35 PSF LOAD FOR FUTURE OVERLAY FOR GIRDER DESIGNS.

PRESTRESSED GIRDER CONCRETE: f'c = 8,500 PSI

CONCRETE STRENGTH UP TO 10,000 PSI MAY BE USED WITH PRIOR APPROVAL FROM THE DEPARTMENT.

SPECIFY CONCRETE RELEASE STRENGTH FOR EACH GIRDER BASED ON DESIGN.

CLOSURE POUR CONCRETE
FOR POST TENSIONED GIRDERS: f'c = MATCH GIRDER STRENGTH

REINFORCING STEEL (COATED) fy = 60,000 PSI
(WELDED WIRE REINFORCEMENT IS NOT ALLOWED)

PRESTRESSED
LOW RELAXATION STRAND: 0.6" DIAMETER
AASHTO M 203 GRADE 270

USE UTAH DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION (THE LATEST EDITION AND SUPPLEMENTS THERETO WHICH ARE IN EFFECT AT THE DATE OF REQUEST FOR BIDS) ALONG WITH SPECIAL PROVISIONS SECTION 03339S - FULL DEPTH STANDARD PRECAST CONCRETE DECK PANELS FOR MATERIALS, CONSTRUCTION AND WORKMANSHIP.

DESIGN AND DETAIL LIFTING SUPPORTS AND HANDLING CONSIDERATIONS IN ACCORDANCE WITH THE PCI DESIGN HANDBOOK, PRECAST AND PRESTRESSED CONCRETE, FIFTH EDITION WITH ALL INTERIMS AND ERRATA (NO CRACKING CRITERIA).

FOR TOP SURFACE OF BULB TEE AND POST TENSIONED BULB TEE, USE A ROUGHENED SURFACE (1/4" AMPLITUDE). FOR DECK BULB TEE USE A HEAVY BROOM FINISH ON RIDING SURFACE OF TOP FLANGE.

DECK BULB TEES HAVE A 1/4" CONCRETE GRINDING ALLOWANCE FOR CORRECTING UNEVEN ROADWAY SURFACES AT LONGITUDINAL JOINTS. LOSS OF 1/4" OF TOP FLANGE TO BE ACCOUNTED FOR IN DESIGN.

APPLY CONCRETE POLYMER OVERLAY ON BRIDGE DECK AFTER CONCRETE GRINDING OR STEEL SHOT IS COMPLETE. SEE SECTION 03372 IN THE STANDARD SPECIFICATIONS FOR SURFACE PREPARATION REQUIREMENTS.

COAT ALL MILD REINFORCEMENT PER UDOT SPECIFICATIONS UNLESS OTHERWISE NOTED.

INDEX OF SHEETS

NO. DATE BY REMARKS
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UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

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QUANT. CHECK
APPROVAL RECOMM. DATE SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT DATE UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
BULB TEE GIRDER
GENERAL NOTES

PROJECT NUMBER

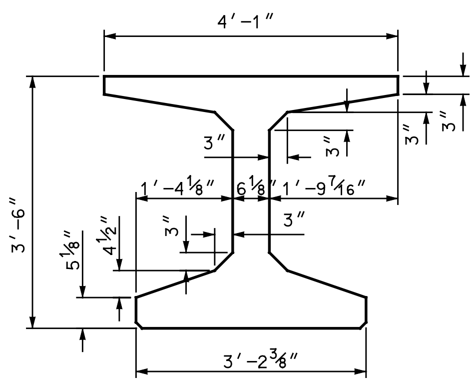
COUNTY

GNBT-1
DRG. NO.

SHT. OF

GNBT-1 BULB TEE NOTES.dgn

7/19/2010



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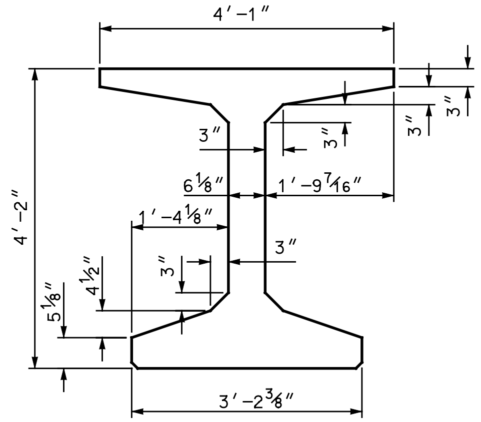

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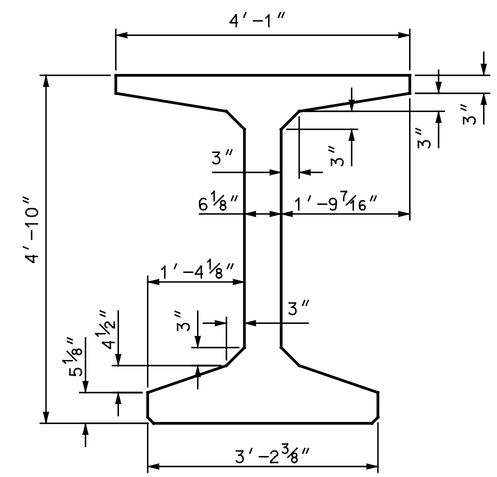

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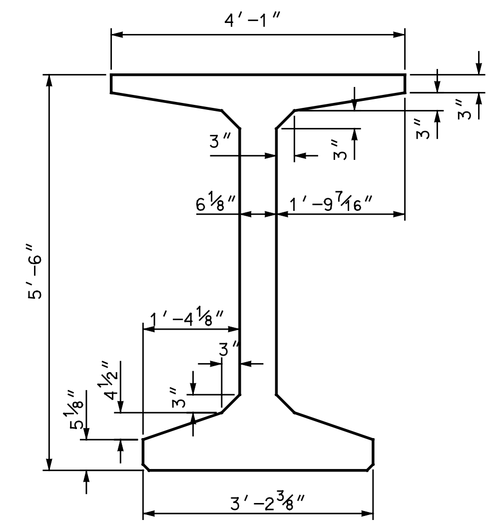

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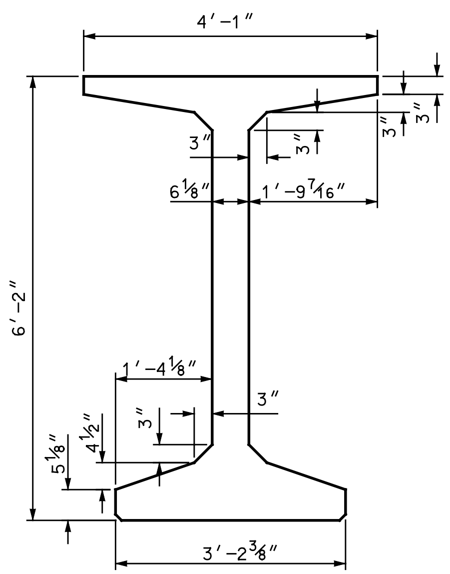

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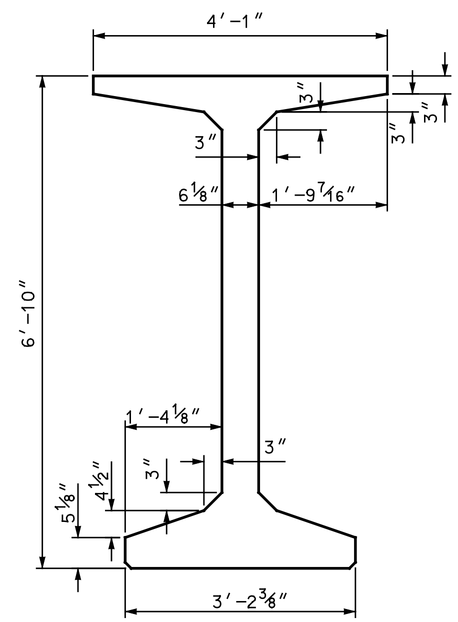

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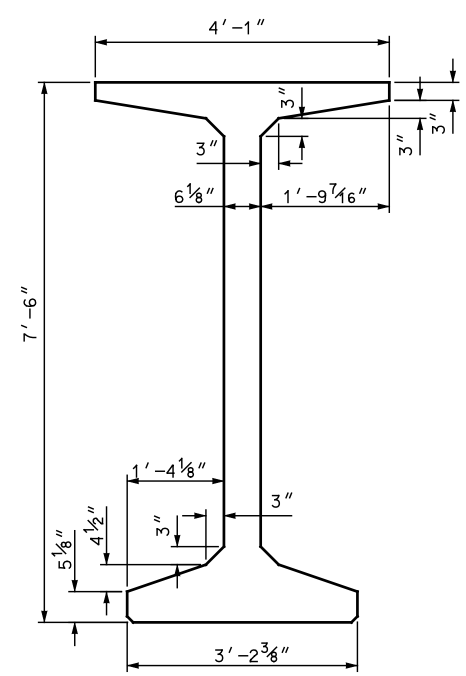

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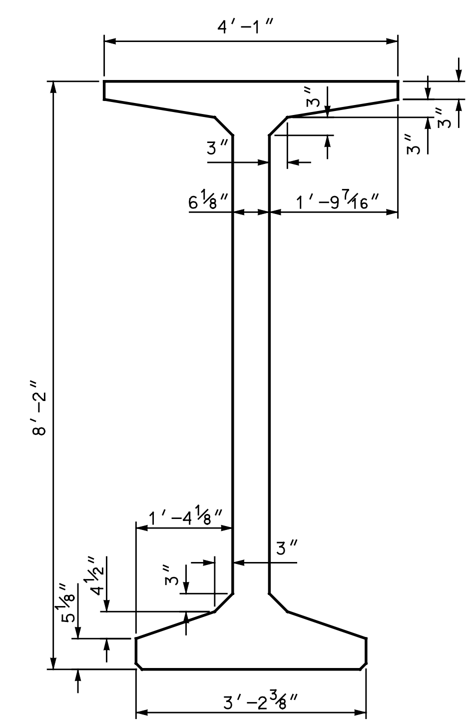

UBT98

GIRDER TYPE	DEPTH (in)	WEIGHT (Lbs/Ft)	AREA (in^2)	Ix c.g. (in^4)	Iy c.g. (in^4)	Yt (in)	Yb (in)	St (in^3)	Sb (in^3)
UBT42	42	759	729	184042	71761	21.67	20.33	8494	9052
UBT50	50	810	778	283124	71914	25.88	24.12	10940	11738
UBT58	58	861	827	407026	72067	30.07	27.93	13538	14571
UBT66	66	912	876	557326	72221	34.23	31.77	16281	17544
UBT74	74	963	925	735599	72374	38.38	35.62	19166	20652
UBT82	82	1014	974	943421	72527	42.51	39.49	22191	23893
UBT90	90	1065	1023	1182360	72680	46.64	43.36	25353	27265
UBT98	98	1116	1072	1454000	72833	50.74	47.26	28653	30769

REVISIONS
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UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK

APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT | DATE | UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
PRECAST BULB TEE GIRDER
STANDARD GIRDER SIZES

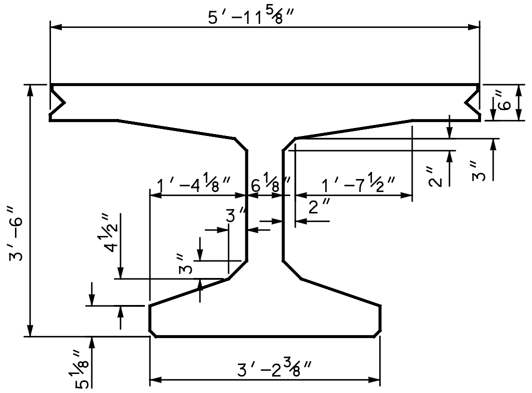
PROJECT NUMBER

COUNTY

BT - 1
DRG. NO.

SHT. OF

BT-1 TO BT-3 STANDARD GIRDER SHAPES.dgn
7/19/2010



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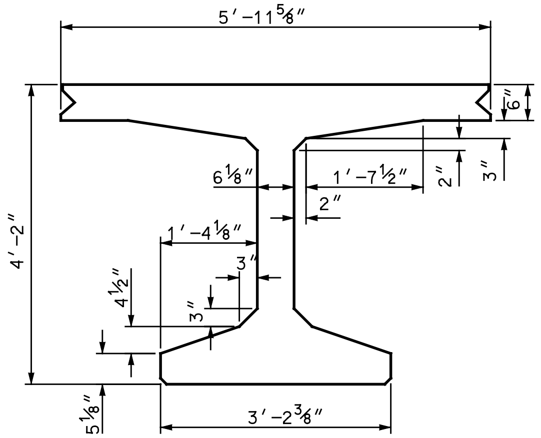

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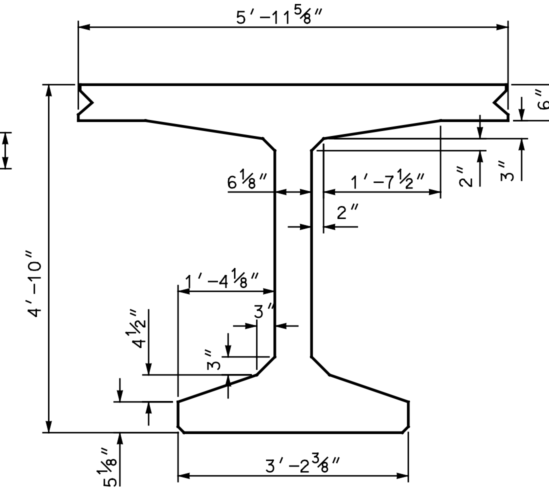

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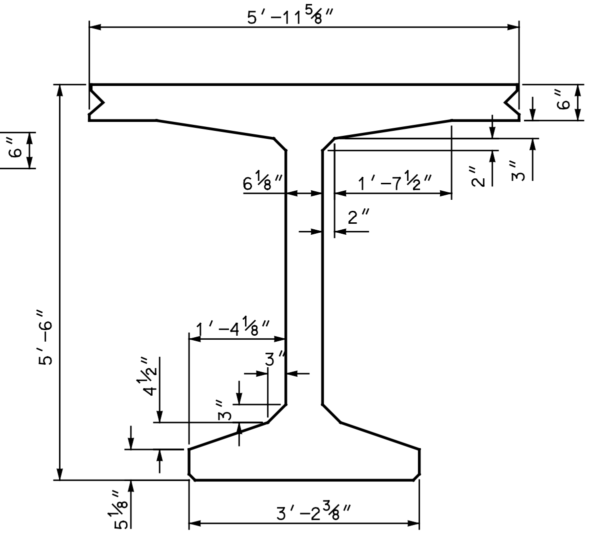

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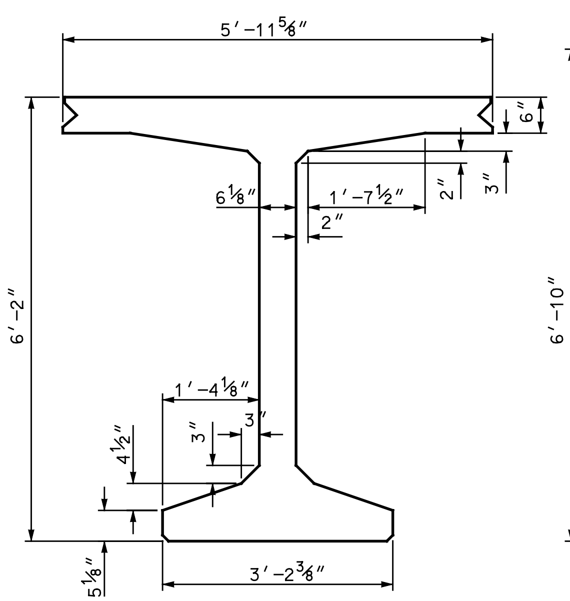

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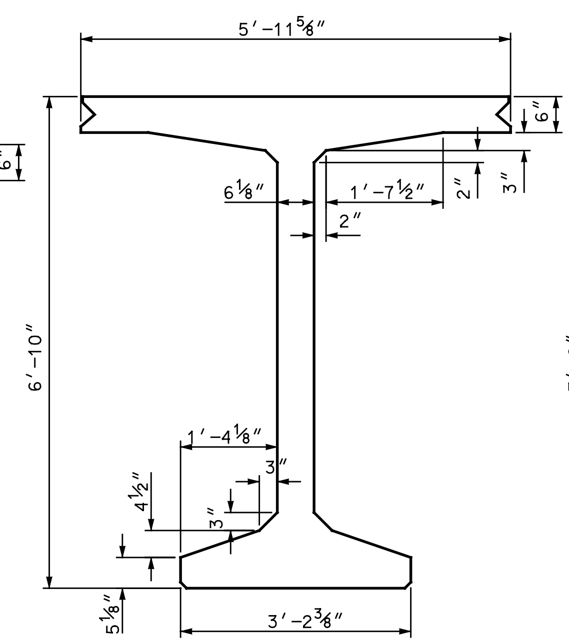

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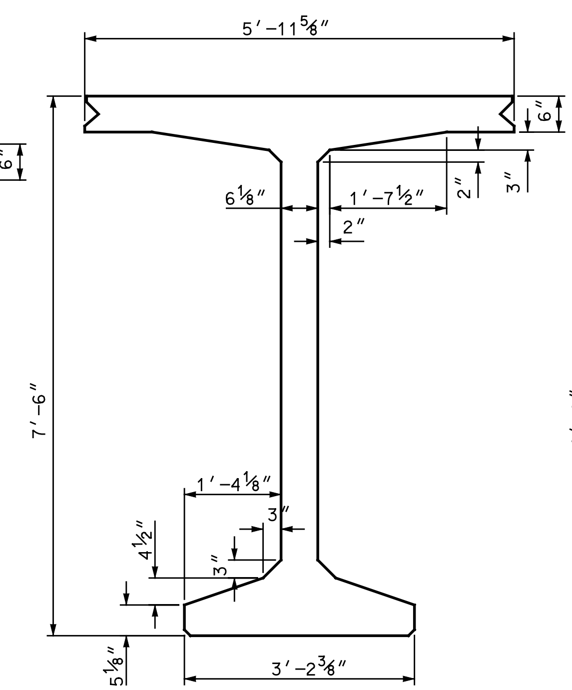

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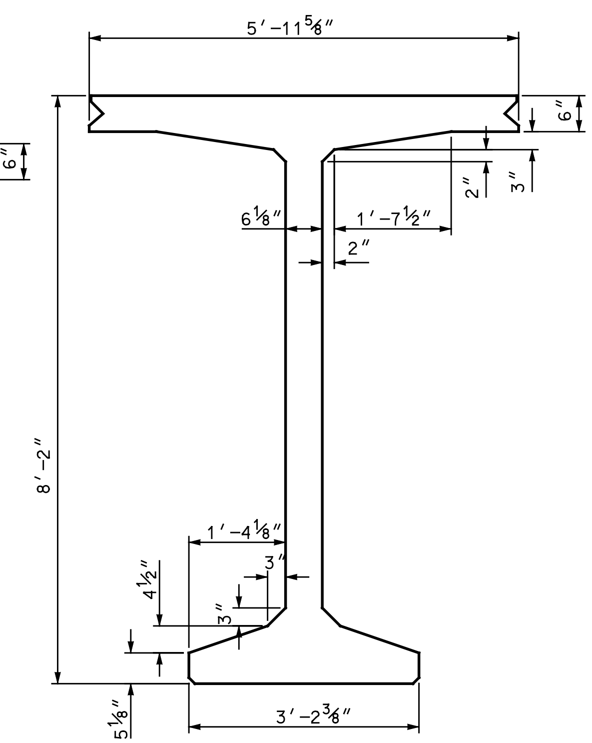


UDBT98

GIRDER TYPE	DEPTH (in)	WEIGHT (Lbs/Ft)	AREA (in²)	Ix c.g. (in⁴)	Iy c.g. (in⁴)	Yt (in)	Yb (in)	St (in³)	Sb (in³)
UDBT42	42	1015	974	238350	212226	17.33	24.67	13754	9662
UDBT50	50	1066	1023	368258	212379	20.74	29.26	17756	12586
UDBT58	58	1117	1072	530226	212532	24.19	33.81	21919	15683
UDBT66	66	1168	1121	725908	212685	27.70	38.30	26206	18953
UDBT74	74	1219	1170	956946	212839	31.24	42.76	30632	22379
UDBT82	82	1270	1219	1224970	212992	34.82	47.18	35180	25964
UDBT90	90	1321	1268	1531590	213148	38.44	51.56	39844	29705
UDBT98	98	1372	1317	1878430	213298	42.08	55.92	44639	33591

REVISIONS

NO.	DATE	BY	REMARKS

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

DESIGN ____ CHECK ____
DRAWN ____ CHECK ____
QUANT. ____ CHECK ____

APPROVAL RECOMM. ____ DATE ____ SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT ____ DATE ____ UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
PRECAST DECK BULB TEE GIRDER
STANDARD GIRDER SIZES

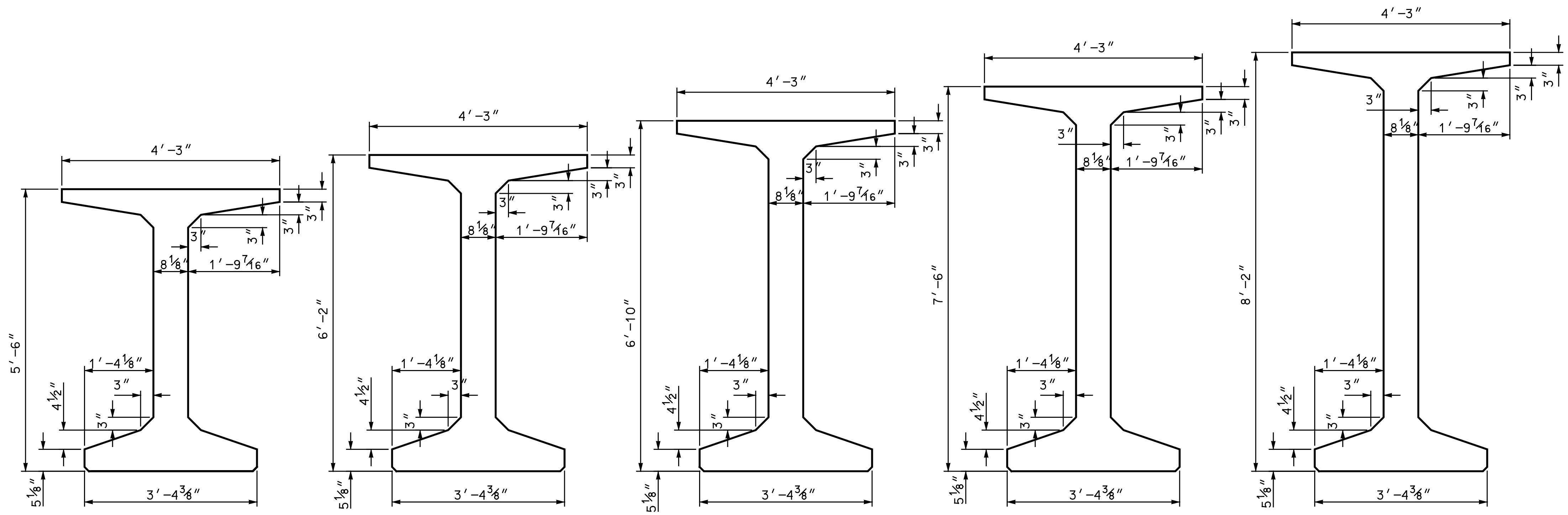
PROJECT NUMBER

COUNTY

BT - 2
DRG. NO.

SHT. OF

BT-1 TO BT-3 STANDARD GIRDER SHAPES.dgn
7/19/2010



UBT 66-PT

UBT 74-PT

UBT 82-PT

UBT 90-PT

UBT 98-PT

GIRDER TYPE	DEPTH (in)	WEIGHT (Lbs/Ft)	AREA (in^2)	Ix c.g. (in^4)	Iy c.g. (in^4)	Yt (in)	Yb (in)	St (in^3)	Sb (in^3)
UBT66-PT	66	1050	1008	605416	84741	34.07	31.93	17769	18961
UBT74-PT	74	1117	1073	803380	85099	38.19	35.81	21036	22435
UBT82-PT	82	1185	1138	1035640	85456	42.30	39.70	24485	26084
UBT90-PT	90	1253	1203	1304270	85814	46.39	43.61	28115	29908
UBT98-PT	98	1320	1268	1611370	86171	50.48	47.53	31924	33906

REVISIONS

NO. | DATE | BY | REMARKS

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

DESIGN ____ CHECK ____
DRAWN ____ CHECK ____
QUANT. ____ CHECK ____

APPROVAL RECOMM. ____ DATE ____ SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT ____ DATE ____ UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
PRECAST POST-TENSIONED BULB TEE GIRDER
STANDARD GIRDER SIZES

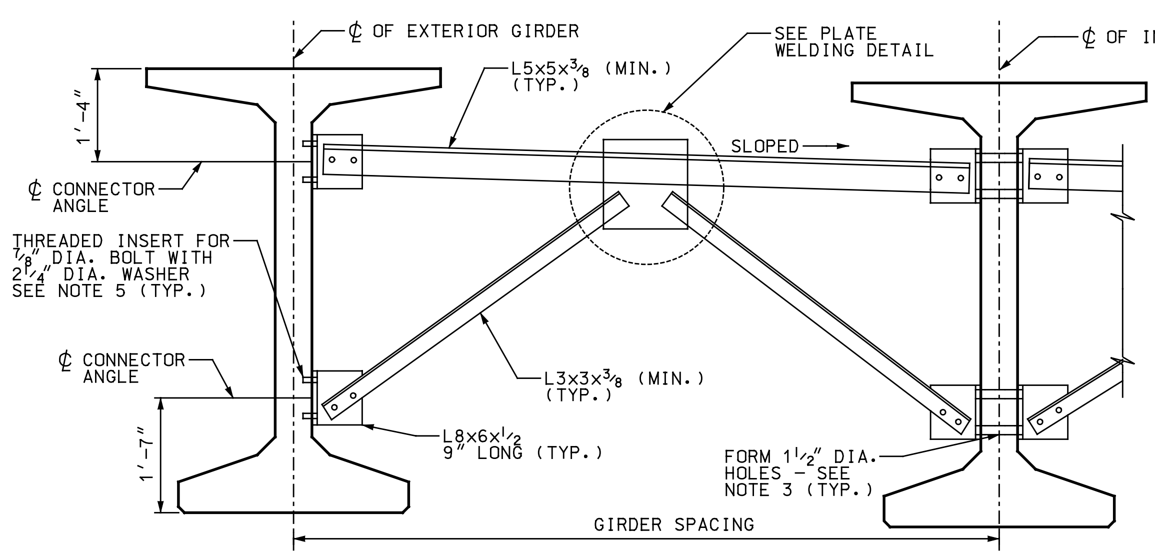
PROJECT NUMBER

COUNTY

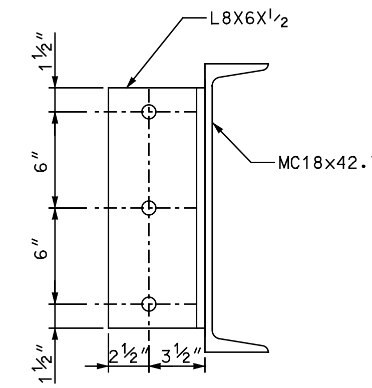
BT - 3
DRG. NO.

SHT. OF

BT-1 TO BT-3 STANDARD GIRDER SHAPES.dgn
7/19/2010

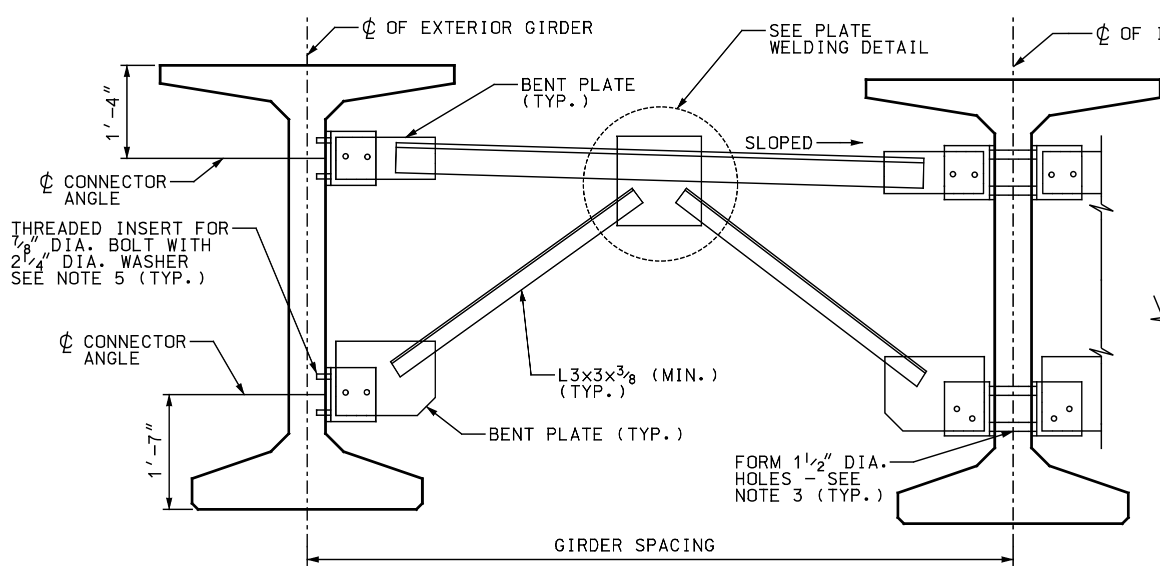


INTERMEDIATE DIAPHRAGM - TYPE II

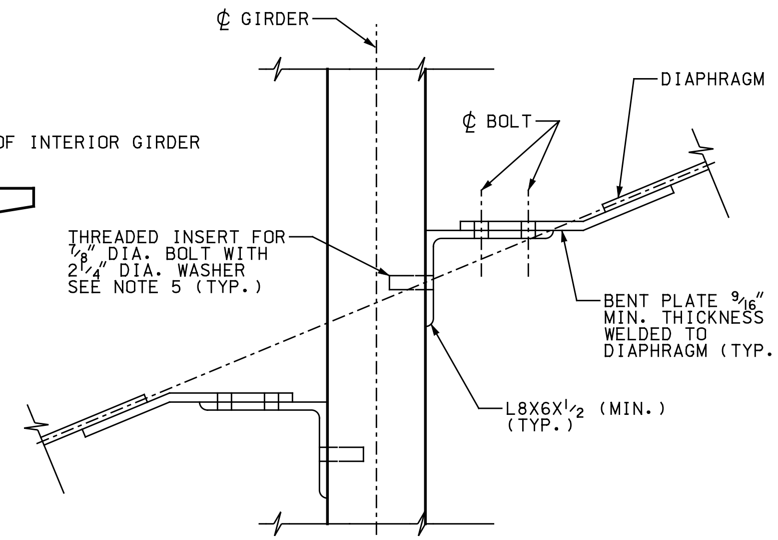


SECTION A-A: INTERMEDIATE DIAPHRAGM

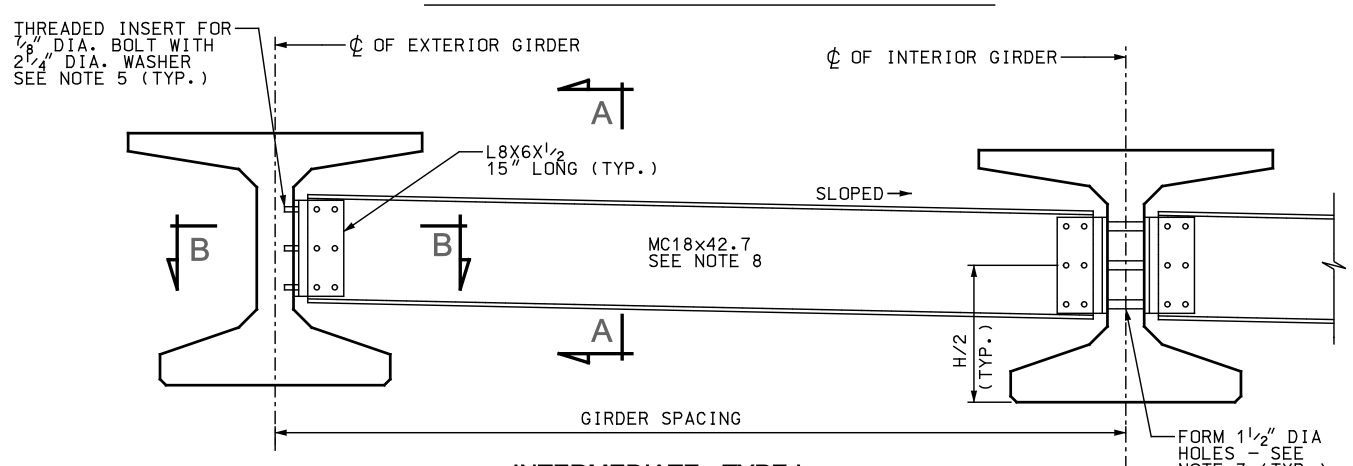
NOTE: CHANNEL SHOWN, K-FRAME DIAPHRAGM SIMILAR.



SKEWED INTERMEDIATE DIAPHRAGM - TYPE II



SECTION B-B: INTERMEDIATE DIAPHRAGMS



INTERMEDIATE - TYPE I

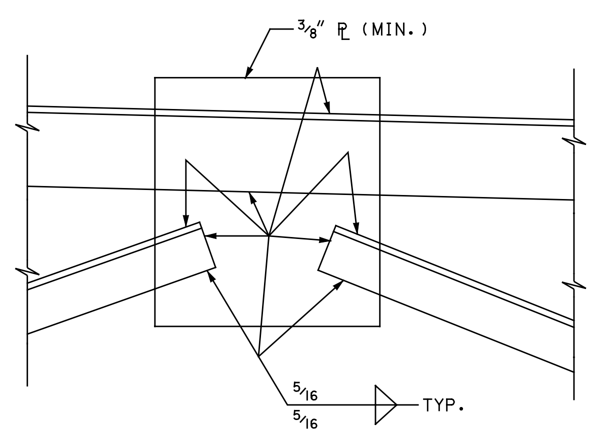


PLATE WELDING DETAIL

NOTE: STOP WELDS 1/4" SHORT OF ALL PLATE ELEMENTS

DIAPHRAGM CONNECTIONS TO BE LOCATED ON THE GIRDER WEB. DESIGNER SHALL VERIFY THREAD INSERT CAPACITIES SPECIFIED AND MODIFY THEM WHERE APPLICABLE.

THE BENT PL SHOWN IN SECTION B IS NOT NECESSARY FOR 0° SKEWS.

NOTES

1. INCLUDE ALL STEEL FOR DIAPHRAGMS IN THE COST OF THE PRESTRESSED CONCRETE GIRDER.
2. ALL STEEL TO BE ASTM A709 GRADE 50 AND GALVANIZED IN IN ACCORDANCE WITH ASTM A 123.
3. FABRICATE INTERMEDIATE AND END DIAPHRAGMS WITH VERTICAL ENDS.
4. STEEL TO STEEL CONNECTIONS TO BE MADE USING 7/8" DIA. GALVANIZED HIGH-STRENGTH BOLTS.
5. STEEL TO CONCRETE CONNECTIONS TO BE MADE USING ASTM A307 GRADE A BOLTS.
6. WASHERS TO BE ASTM F436. ALL BOLTS, NUTS, AND WASHERS TO BE GALVANIZED IN ACCORDANCE WITH ASTM A 153.
7. FIELD DRILLED HOLES IN DIAPHRAGM CONNECTION ANGLES TO BE PERMITTED AT NO ADDITIONAL EXPENSE. ALL OTHER HOLES TO BE SHOP DRILLED.
8. 1/2" BENT PLATE OF EQUAL DIMENSIONS MAY BE SUBSTITUTED FOR MC18X42.7.
9. FOR SQUARE SUPERSTRUCTURES, FORM 1 1/2" DIA. HOLES IN INTERIOR GIRDERS FOR THRU-BOLTING OF DIAPHRAGM ANGLE. FOR FASCIA GIRDERS AND SKEWED STRUCTURES, THREADED INSERTS ARE REQUIRED. PLACE HOLES AND THREADED INSERTS PERPENDICULAR TO GIRDER WEB.
10. SEE GIRDER DRAWINGS FOR GIRDER CONNECTION REQUIREMENTS

INTERMEDIATE DIAPHRAGM SIZING TABLE

GIRDER DEPTH	DIAPHRAGM	CHANNEL SIZE (SEE NOTE 11)
42	TYPE I	MC18x42.7
50	TYPE I	MC18x42.7
58	TYPE I	MC18x42.7
66	TYPE II	N/A
74	TYPE II	N/A
82	TYPE II	N/A
90	TYPE II	N/A
98	TYPE II	N/A

INTERMEDIATE DIAPHRAGM LOCATION TABLE

SPAN LENGTH	DIAPHRAGM LOCATION ALONG GIRDER
< 80 FT	1/2 SPAN PTS.
80 FT TO 120 FT	1/3 SPAN PTS.
120 FT TO 160 FT	1/4 SPAN PTS.
> 160 FT	1/5 SPAN PTS.

NOTE TO DESIGNER:
INSERT SHEET AS NEEDED INTO DESIGN DRAWINGS AND NUMBER AS FOLLOWS: UBTXX-X, UDBTXX-X, OR UBTXXPT-X

REVISIONS

NO.	DATE	BY	REMARKS

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

DESIGN / CHECK — DRAWN / CHECK — QUANT. / CHECK

APPROVAL RECOMM. — DATE — SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT — DATE — UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
PRECAST BULB TEE GIRDER
DIAPHRAGM DETAILS

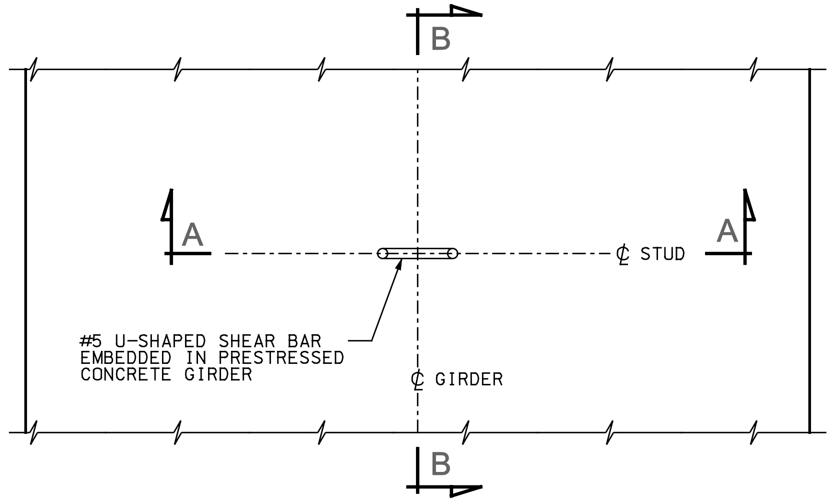
PROJECT NUMBER

COUNTY

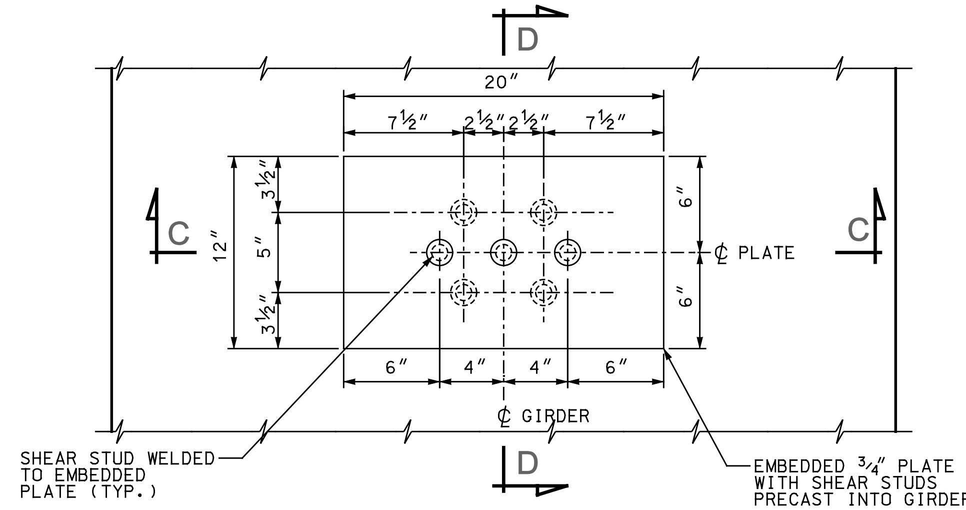
BT - 4
DRG. NO.

SHT. OF

BT-4 DIAPHRAGM DETAILS.dgn
7/19/2010



U-SHAPED SHEAR BAR PLAN ON BULB TEE GIRDER

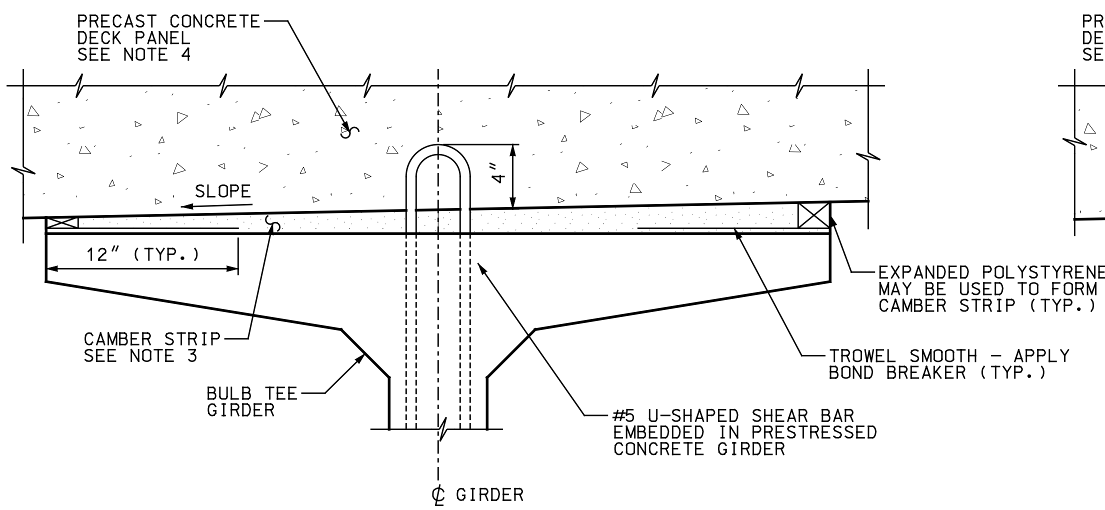


OPTIONAL SHEAR STUD PLAN ON BULB TEE GIRDER

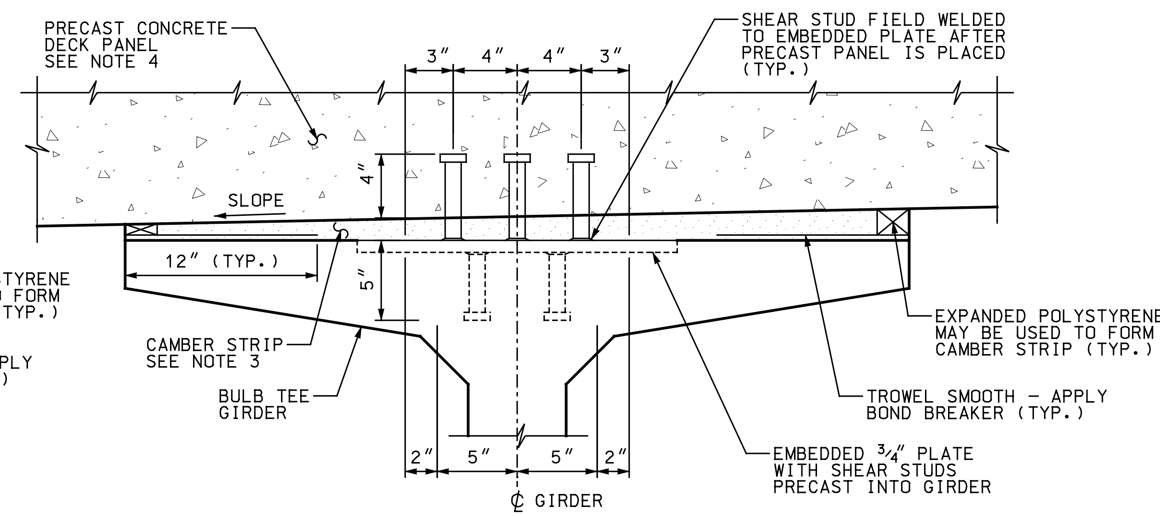
NOTE: ONE ROW OF STUDS SHOWN ON TOP. MULTIPLE ROWS MAY BE USED.

NOTES

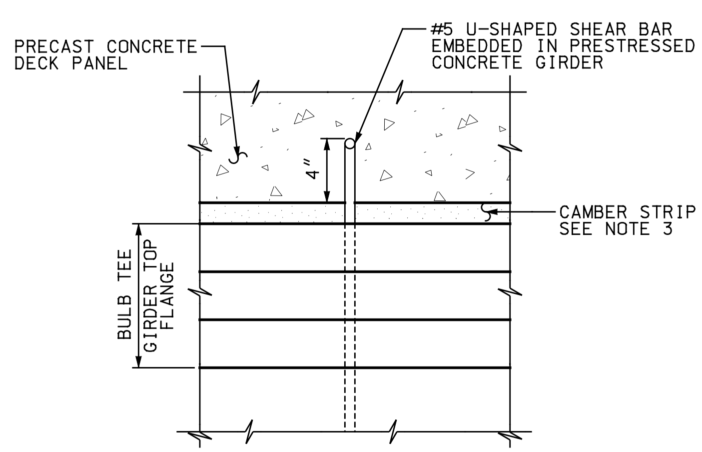
1. DETAILS DRAWN FOR 1″ DIA. SHEAR STUDS.
2. CONTRACTOR TO DETERMINE METHOD FOR ACHIEVING WELDING GROUND FOR ATTACHMENT OF SHEAR STUDS. A WELDED GROUND LUG IS ACCEPTABLE PROVIDED THERE IS NO INTERFERENCE WITH PLACEMENT OF STUDS OR PLACEMENT OF PRECAST PANEL.
3. FORM CAMBER STRIP AND FILL WITH NON-SHRINK GROUT. METHOD OF FORMING CAMBER STRIP TO BE DETERMINED BY THE CONTRACTOR. REMOVE FORMS AFTER NON-SHRINK GROUT OBTAINS A COMPRESSIVE STRENGTH OF 3000 PSI.
4. FOR BLOCKOUT IN PRECAST PANELS SEE OPTIONS IN PRECAST PANEL DRAWINGS.



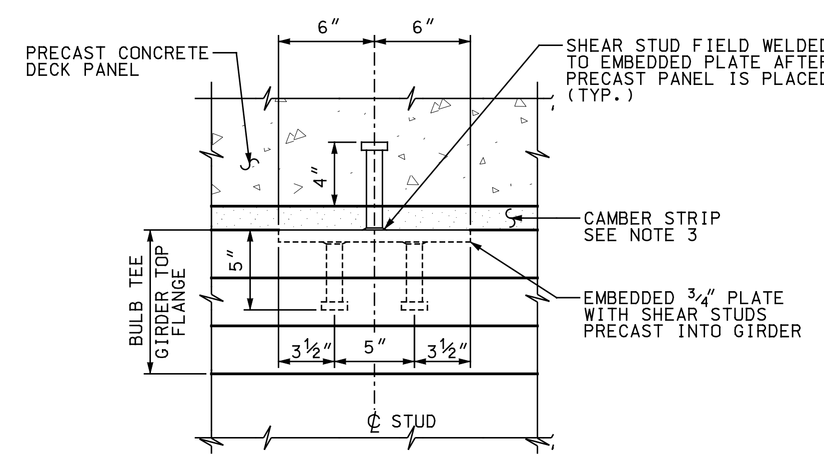
SECTION A-A: U-SHAPED SHEAR BAR



SECTION C-C: OPTIONAL SHEAR STUDS



SECTION B-B: U-SHAPED SHEAR BAR



SECTION D-D: OPTIONAL SHEAR STUDS

NOTE: ONE ROW OF STUDS SHOWN ON TOP. MULTIPLE ROWS MAY BE USED.

NOTE TO DESIGNER:
INSERT SHEET AS NEEDED INTO DESIGN DRAWINGS AND NUMBER AS FOLLOWS:
UBTXX-X, UDBTXX-X, OR UBTXXPT-X

REVISIONS
NO. | DATE | BY | REMARKS

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
STRUCTURES DIVISION

DESIGN | CHECK
DRAWN | CHECK
QUANT. | CHECK
APPROVAL RECOMM. | DATE | SENIOR DESIGN ENGR.
APPROVED FOR USE BY UDOT | DATE | UDOT BRIDGE ENGR.

TYPICAL DETAIL SHEET
PRECAST DECKS ON BULB TEE
GIRDER WELDED STUD OPTION

PROJECT NUMBER

COUNTY

BT - 5
DRG. NO.

SHT. OF

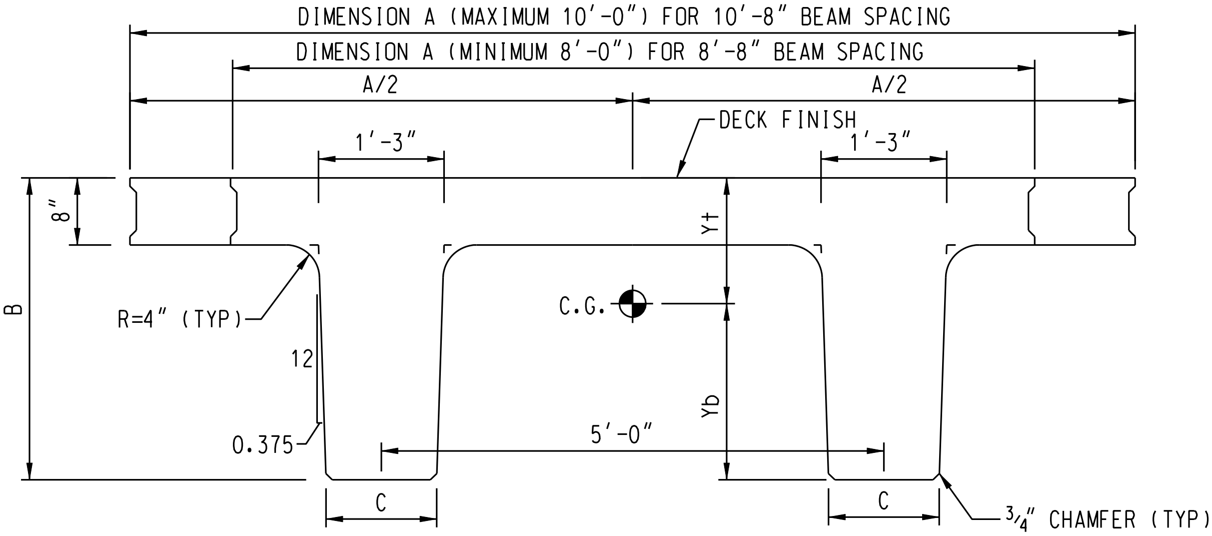
BT-5 WELDED STUD DETAILS.dgn

7/19/2010

APPENDIX H

DETAILS OF NEXT D BEAM FROM PCI-NE

Note: These are direct extract from
<http://www.pcine.org/index.cfm/resources/bridge>
(Last accessed June 25, 2012)



NEXT BEAM - SECTION PROPERTIES

BEAM DESIGNATION	BEAM WIDTH INCHES	BEAM DEPTH INCHES	BASE STEM WIDTH INCHES	AREA IN 2	I IN4	Yb INCHES	Yt INCHES	St IN 3	Sb IN3	WEIGHT PLF
	A	B	C			D	E			
MINIMUM WIDTH BEAMS										
NEXT 40 D	96.00	40.00	13.00	1666	238059	25.47	14.54	16378	9348	1735
NEXT 36 D	96.00	36.00	13.25	1562	176674	23.03	12.97	13624	7671	1627
NEXT 32 D	96.00	32.00	13.50	1455	126111	20.57	11.43	11033	6131	1516
NEXT 28 D	96.00	28.00	13.75	1346	85651	18.06	9.94	8620	4742	1402
MAXIMUM WIDTH BEAMS										
NEXT 40 D	120.00	40.00	13.00	1858	258171	26.55	13.45	19201	9722	1935
NEXT 36 D	120.00	36.00	13.25	1754	191453	24.01	11.99	15973	7973	1827
NEXT 32 D	120.00	32.00	13.50	1647	136502	21.44	10.57	12920	6368	1716
NEXT 28 D	120.00	28.00	13.75	1538	92597	18.80	9.20	10069	4924	1602

NOTES:

1. THE WIDTH OF BEAMS SHOWN ARE THE MINIMUM AND MAXIMUM WIDTH BEAMS. VARIATION BETWEEN THESE LIMITS IS ALLOWED IN ORDER TO CONSTRUCT A BRIDGE TO THE REQUIRED WIDTH. THE VARIATION IN WIDTH IS ACCOMPLISHED BY VARYING THE OVERHANG DIMENSIONS. THE DESIGNER WILL NEED TO CALCULATE BEAM PROPERTIES FOR BEAMS THAT ARE NOT EQUAL TO THE WIDTHS LISTED.

2. THE SPACING OF BEAMS ON A TYPICAL BRIDGE SHALL BE THE WIDTH OF THE BEAM PLUS 8" (EX.: BEAM SPACING = 10'-8" FOR THE 10'-0" SECTION).

3. BRIDGES WITH SMALL CURVATURE CAN BE BUILT USING THESE SECTIONS BY VARYING THE OVERHANG OF THE FASCIA BEAMS ALONG THE LENGTH. INTERIOR BEAMS SHOULD ALWAYS BE SYMMETRICAL ABOUT THE VERTICAL AXIS. NON-SYMMETRICAL SECTIONS ARE POSSIBLE, HOWEVER THE BEAM MAY REQUIRE A SPECIAL DESIGN WITH A NON-SYMMETRICAL STRAND PATTERN.

4. MODIFY THE FASCIA BEAM TO MATCH STATE STANDARDS.

5. THE STEM WIDTH AND SPACING ARE FIXED.

6. THE ENDS OF THE BEAMS SHOULD BE SKEWED FOR SKEWED BRIDGES. THE ACUTE CORNERS OF THE FLANGE OVERHANGS SHOULD BE CHAMFERED 6"x6" IN ORDER TO MINIMIZE CASTING AND HANDLING DAMAGE.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

BEAM PROPERTIES

ISSUE DATE: 01-04-10

SHEET: NEXT D -01

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

PCI

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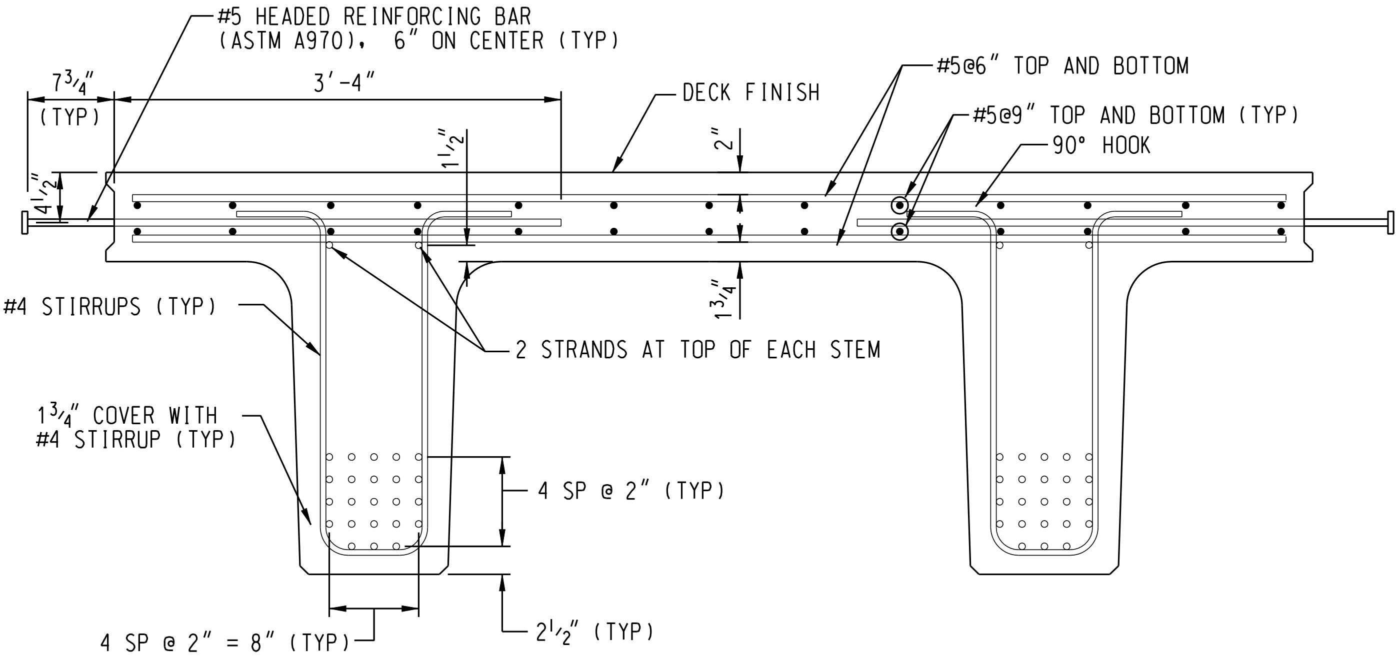

TYPICAL BEAM REINFORCING

DESIGN NOTES:

1. THE REINFORCING SHOWN IS BASED ON A PRELIMINARY DESIGN OF A 10 FOOT WIDE NEXT BEAM. DESIGNERS SHOULD VERIFY THIS REINFORCING FOR EACH DESIGN BASED ON THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS OR STATE STANDARDS.

2. THE STRIP METHOD SPECIFIED IN AASHTO LRFD ARTICLE 4.6.2.1 IS RECOMMENDED FOR THE DESIGN.

3. THE HEADED REINFORCING BARS SHOWN SHOULD BE DESIGNED TO RESIST THE POSITIVE BENDING MOMENT AT THE CENTER OF THE JOINT AS SPECIFIED BY AASHTO. THE NESTED HEADED BARS CAN BE CONSIDERED A LAP SPLICE WITH THE BARS FULLY DEVELOPED.

5. THE CRACK CONTROL PROVISIONS OF AASHTO ARTICLE 5.7.3.4. SHOULD ALSO BE CHECKED.

6. ADDITIONAL REINFORCEMENT MAY BE REQUIRED FOR DECK OVERHANGS AND BARRIERS.

NOTES:

1. THE TOP FLANGE IS INTENDED TO ACT AS A STRUCTURAL DECK.

2. SHEAR REINFORCING SHOULD BE KEPT TO #4 BARS IN ORDER TO MAXIMIZE THE COVER ON THE SIDE OF THE STEM.

3. SEE SHEET D-11 FOR UTILITY SUPPORT DETAILS.

4. MINOR ADJUSTMENT OF THE SPACING OF THE TOP LONGITUDINAL REINFORCEMENT IS ALLOWABLE TO FACILITATE THE INSTALLATION OF THE STIRRUPS.

REVISIONS

NO.	DATE	DESCRIPTION
1	4/11	ADDED DESIGN NOTES

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

TYPICAL BEAM SECTION

ISSUE DATE: 01-04-10

SHEET: NEXT D -02

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

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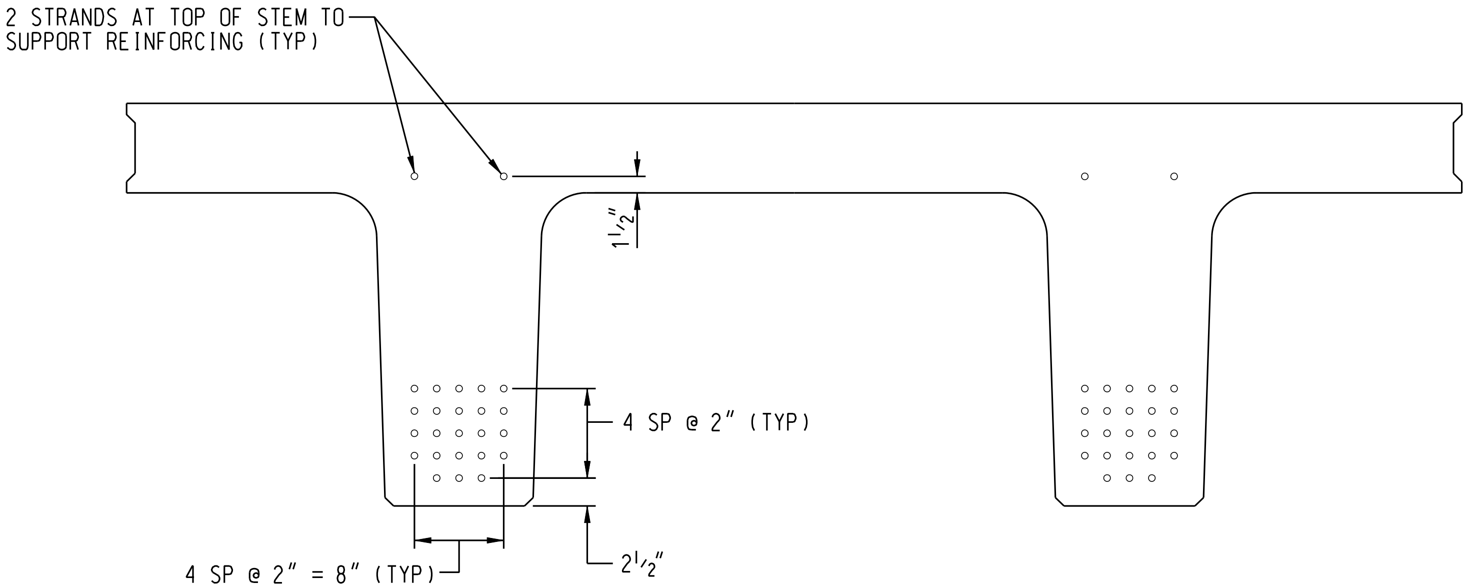

TYPICAL STRAND LOCATIONS

(ENDS AND ALONG THE SPAN)

NOTES:

1. ◦ DENOTES STRAIGHT STRAND. DRAPED STRANDS ARE NOT PERMITTED.

2. DEBONDING OF STRAND IS ALLOWED. NO MORE THAN 25% OF THE TOTAL NUMBER OF STRANDS SHALL BE DEBONDED. THE SPACING BETWEEN DEBONDED STRANDS SHALL BE AT LEAST 2.5 INCHES IN ANY DIRECTION. THE RESTRICTIONS OUTLINED IN THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS SHALL ALSO BE FOLLOWED.

3. IT IS RECOMMENDED THAT APPROXIMATELY 50% OF ALL STRAND BE DEBONDED FOR THE FIRST 6" FROM THE END OF THE BEAM IN ORDER TO CONTROL END CRACKING. SPACING RESTRICTIONS OUTLINED IN NOTE 2 DO NOT APPLY TO THIS 6" AREA, BUT DO APPLY BEYOND THIS 6" AREA.

4. STRANDS SHALL BE PLACED WITHIN THE 2"x2" GRID. THE PATTERN MAY BE RAISED IN 2" INCREMENTS FOR DESIGNS THAT REQUIRE PRESTRESS AT A HIGHER ELEVATION. THE NUMBER AND LOCATION OF STRANDS SHALL BE AS REQUIRED BY DESIGN.

5. THE PATTERN SHOWN DEPICTS THE MAXIMUM NUMBER OF STRANDS ALLOWED (50 STRAND INCLUDING THE TOP STRAND). THIS IS BASED ON THE CAPACITY OF TYPICAL CASTING BEDS.

6. THE TWO BOTTOM CORNER STRAND IN EACH STEM ARE OMITTED TO PROVIDE ROOM FOR THE SHEAR REINFORCEMENT BAR BENDS.

7. ALL PRESTRESSING STRAND SHALL BE 0.6" DIAMETER, UNCOATED SEVEN WIRE, LOW RELAXATION STRANDS CONFORMING TO AASHTO M203. THE ULTIMATE STRENGTH OF THE STRANDS SHALL BE 270 KSI.

8. ADDITIONAL STRAND TENSIONED TO A NOMINAL VALUE MAY BE ADDED TO THE TOP FLANGE TO SUPPORT THE TOP FLANGE REINFORCING.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

TYPICAL STRAND LOCATIONS

ISSUE DATE: 01-04-10

SHEET: NEXT D – 03

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

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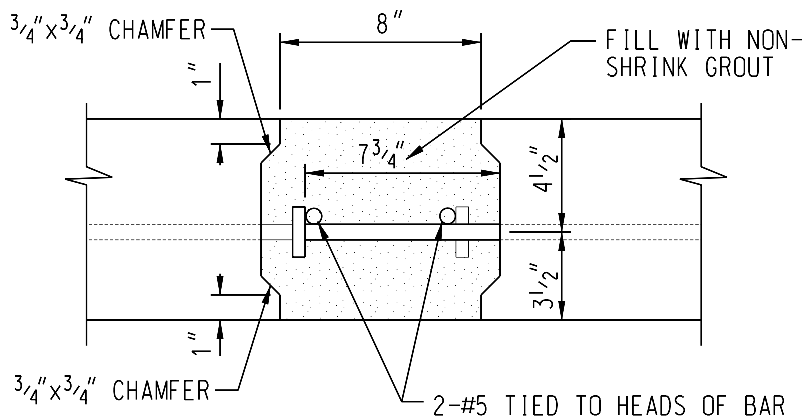
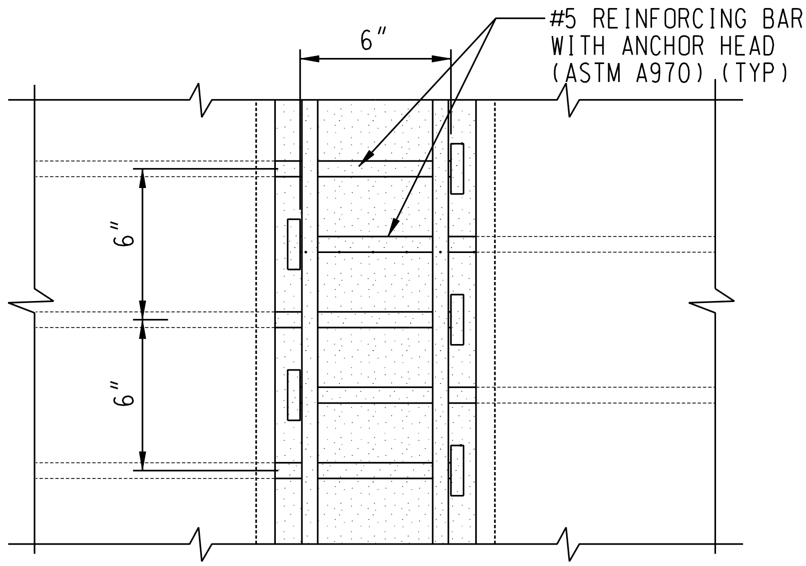


SECTION



PLAN

FLANGE CONNECTOR DETAILS

NOTES:
1. CONNECTOR REINFORCING TO BE PLACED ALONG THE ENTIRE SPAN WITH 6" SPACING.
2. FOR SKEWED BRIDGES, PLACE CONNECTOR REINFORCING PERPENDICULAR TO BEAM EDGE. BEND CONNECTOR REINFORCING WITHIN THE FLANGE IN ACUTE CORNERS TO PRODUCE A SQUARE PROJECTION.
3. METHOD OF FORMING CLOSURE POUR TO BE DETERMINED BY THE CONTRACTOR. THE FORMS NEEDS TO BE REMOVABLE AND ABLE TO ACCOMMODATE DIFFERENTIAL CAMBER. FORM SUPPORTS SHOULD NOT PENETRATE THROUGH TOP OF POUR UNLESS APPROVED BY THE ENGINEER.
4. CLOSURE POUR MATERIAL TO BE A NON SHRINK MIX THAT HAS A MINIMUM COMPRESSIVE STRENGTH OF 7000 PSI. THE GROUT MAY BE EXTENDED WITH AGGREGATE.
5. SAND BLASTING OF THE FACES OF THE KEYS JUST PRIOR TO INSTALLATION IS RECOMMENDED TO IMPROVE GROUT BOND.
6. DESIGNERS ARE RESPONSIBLE FOR THE VERIFICATION OF THE DESIGN OF THIS JOINT. THIS DETAIL CAN BE CONSIDERED EQUIVALENT TO A TENSION LAP SPILCE. IF MORE MOMENT CAPACITY IS REQUIRED, THE LOCATION OF THE BAR MAY BE LOWERED. SEE SHEET 02 FOR INFORMATION ON THE DESIGN OF THIS JOINT.

REVISIONS

NO.	DATE	DESCRIPTION
1	4 /11	ADDED NOTE 6

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

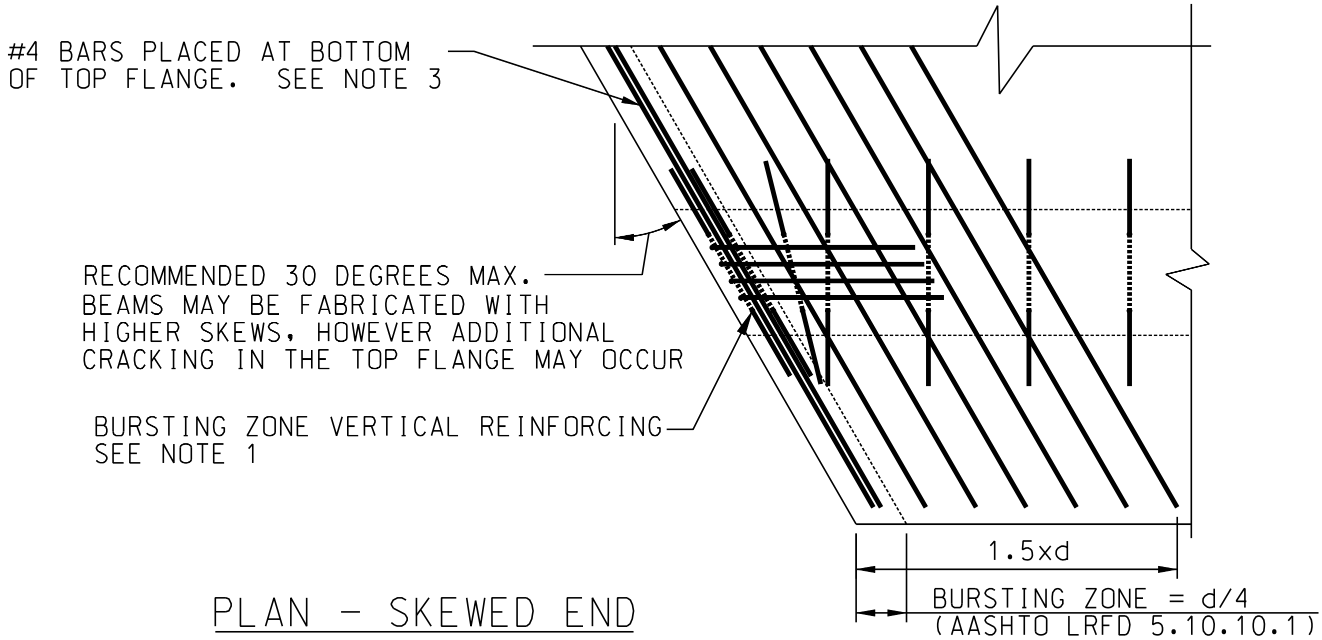
BEAM CONNECTOR DETAILS

ISSUE DATE: 01-04-10

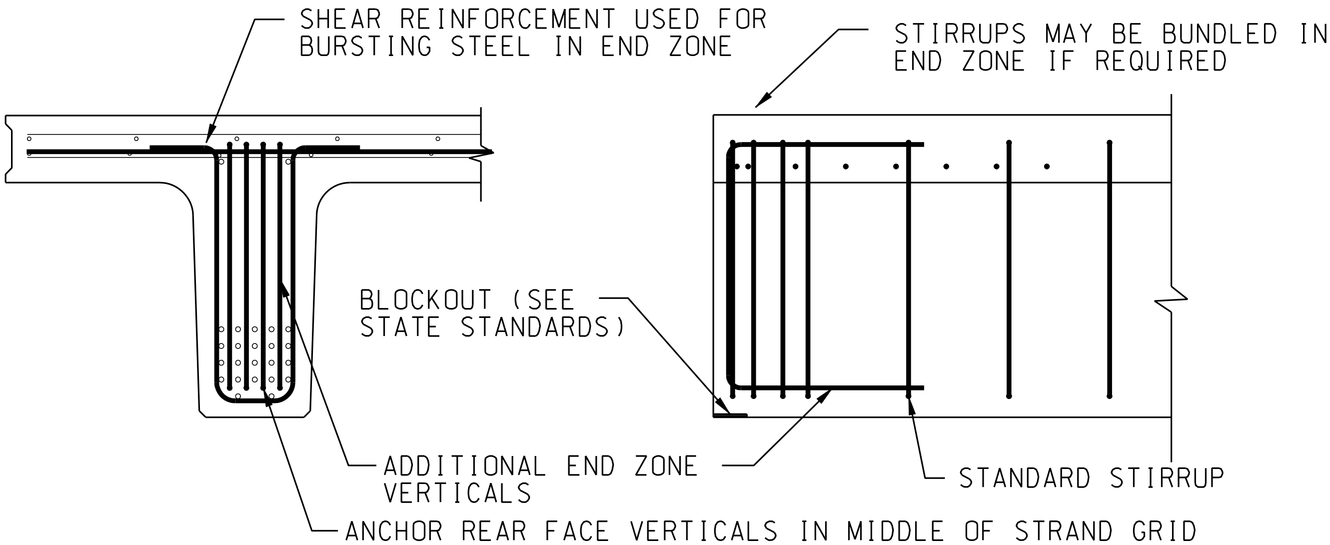
SHEET: NEXT D - 04

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

PCI WWW.PCINE.ORG



PLAN - SKEWED END



END ELEVATION - STEM END

ELEVATION - STEM END

BEAM END REINFORCING DETAILS

NOTES:

1. THE BARS SHOWN ARE APPROXIMATELY THE MAXIMUM NUMBER THAT CAN BE FIT WITHIN THE NEXT 28 D BEAM. SOME OR ALL OF THESE ADDITIONAL END VERTICAL BARS MAY NOT BE NECESSARY DEPENDING ON THE DESIGN.

2. THE AMOUNT OF SPLITTING REINFORCING MAY BE REDUCED BY DEBONDING STRAND IN THIS AREA. ADDITIONAL SPLITTING REINFORCING SHOULD BE PLACED IN AREAS WHERE DEBONDING IS TERMINATED.

3. PLACE 2-#4 BARS AT THE BEAM END, THEN #4 @ 6 INCHES IN THE TOP FLANGE TO MINIMIZE THE POTENTIAL FOR TOP FLANGE END CRACKING DURING RELEASE AND HANDLING. THE MOST COMMON FORM OF POTENTIAL CRACKING IN THIS AREA IS A SERIES OF VERTICAL HAIRLINE CRACKS THROUGH THE INSIDE RADIUS OF THE TOP FLANGE / BEAM STEM INTERFACE RUNNING PARALLEL TO THE STEM.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

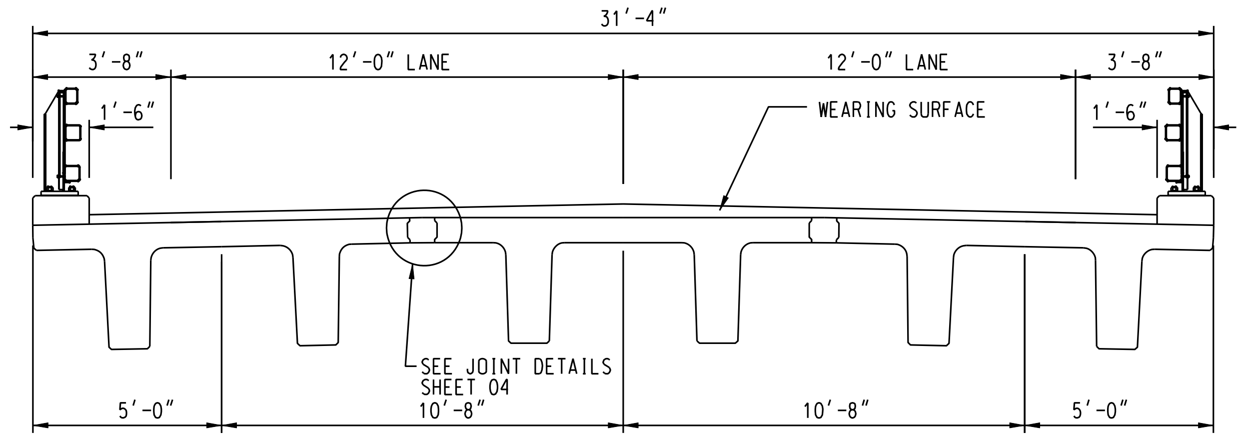
BEAM END DETAILS

ISSUE DATE: 01-04-10

SHEET: NEXT D – 05

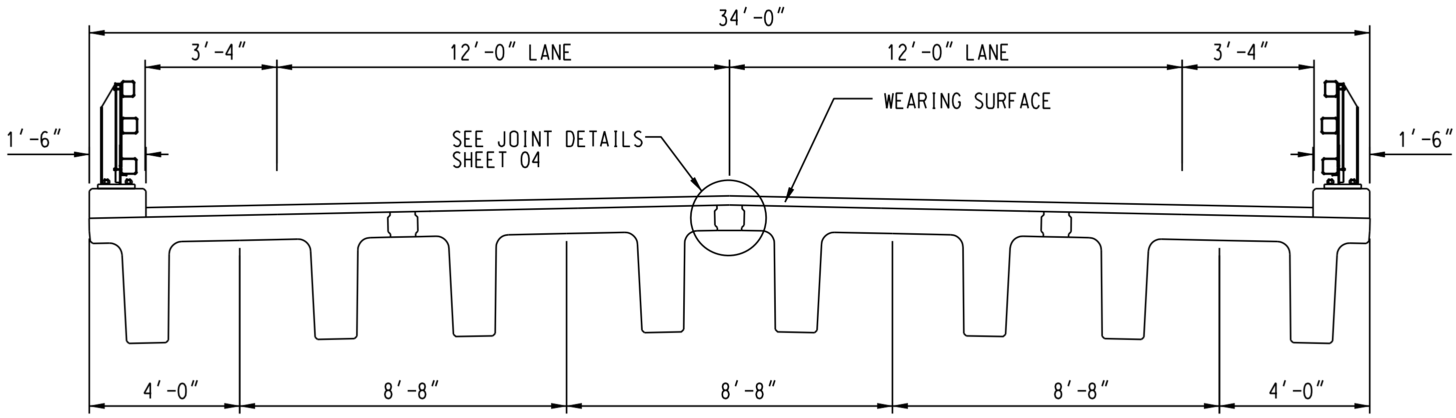
PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

PCI WWW.PCINE.ORG



BRIDGE SECTION WITH MAXIMUM WIDTH BEAMS

TRIAL MAXIMUM SPAN DESIGN - NEXT 40 D x120"
MAXIMUM SPAN = APPROX. XX FEET (f'c = 8 KSI)



BRIDGE SECTION WITH MINIMUM WIDTH BEAMS

TRIAL MAXIMUM SPAN DESIGN - NEXT 40 D x 96"
MAXIMUM SPAN = APPROX. XX FEET (f'c = 8 KSI)

NOTES:

1. THE TWO BRIDGE SECTIONS DEPICTED REPRESENT THE TYPICAL USE OF THE MINIMUM WIDTH AND MAXIMUM WIDTH NEXT BEAMS.

2. THESE SECTIONS WERE USED TO DEVELOP THE BEAM SPAN TABLES DEPICTED ON SHEETS 08 THROUGH 10. THE SPAN TABLES ARE FOR THREE DIFFERENT CONCRETE STRENGTHS; f'c = 10 KSI, 8 KSI, AND 6 KSI. THE SPAN TABLES ARE FOR REFERENCE ONLY. ALTERNATE BRIDGE CONFIGURATIONS WITH DIFFERENT PARAPETS AND OVERLAYS WILL RESULT IN DIFFERENT MAXIMUM SPAN LENGTHS AND STRANDS.

3. THE ABUTMENT SEATS SHOULD BE CAST TO FOLLOW THE PROFILE OF THE BOTTOM STEM OF THE NEXT BEAMS. SKEWED SUBSTRUCTURES MAY REQUIRE ADJUSTABLE BEAM SEATS UNDER THE STEM BEARINGS.

REVISIONS

NO.	DATE	DESCRIPTION
1	05/01/08	MODIFIED NOTE 1 TEXT TO CLARIFY USE OF SECTIONS

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

TYPICAL BRIDGE SECTIONS

ISSUE DATE: 01-04-10

SHEET: NEXT D - 06

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

PCI

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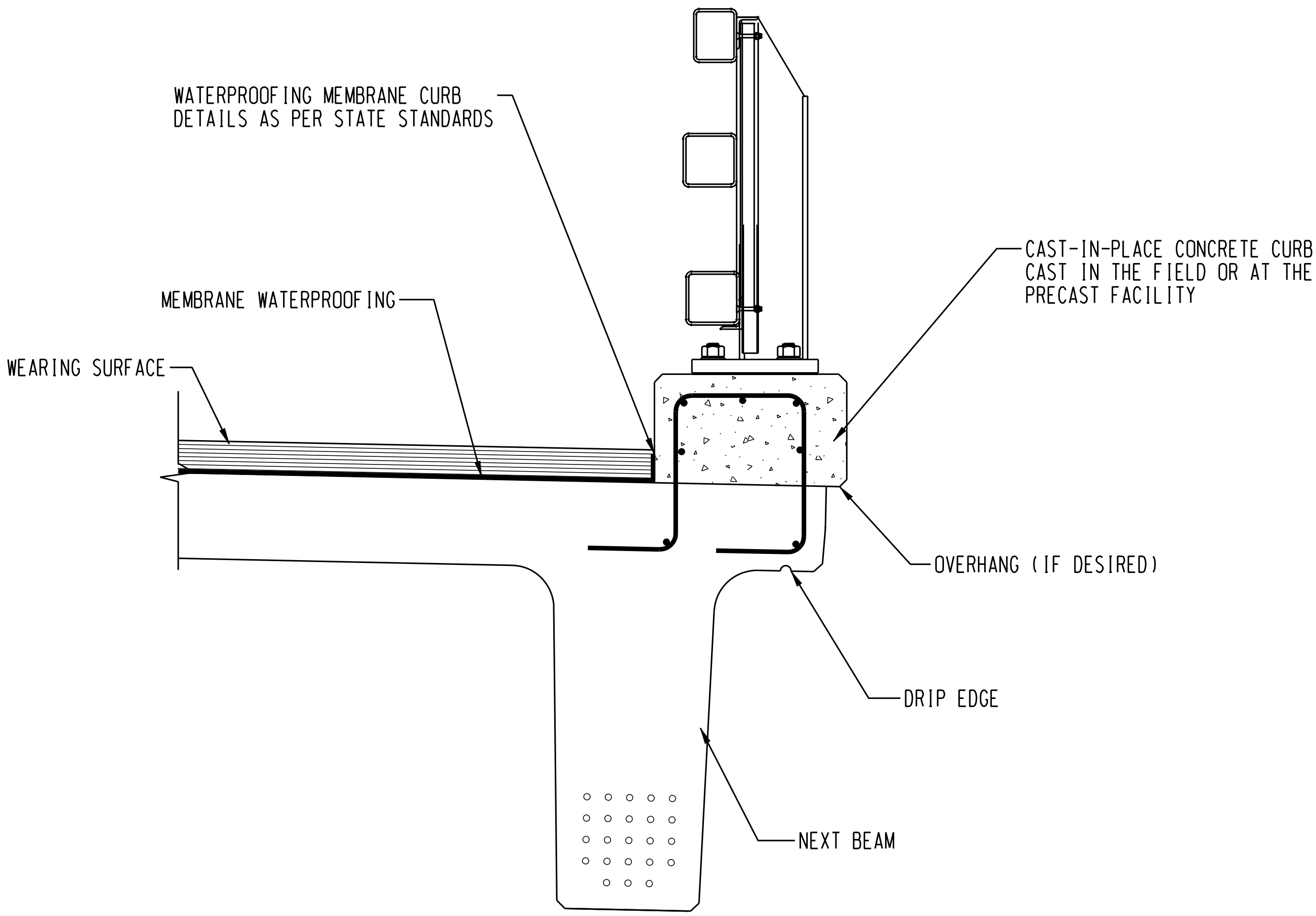


NOTES: THIS DETAIL IS SCHEMATIC. ACTUAL DETAIL WOULD NEED TO BE FULLY DESIGNED.
STEEL RAIL IN CURB SHOWN, OTHER PARAPETS SIMILAR.
THIS DETAIL CAN BE MODIFIED FOR ANY TYPICAL PARAPET SHAPE INCLUDING F SHAPE PARAPETS.
ANCHOR BOLTS NOT SHOWN IN CURB.
ALL REINFORCING IN BEAM NOT SHOWN.

TYPICAL SECTION
PARAPET ATTACHMENT

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

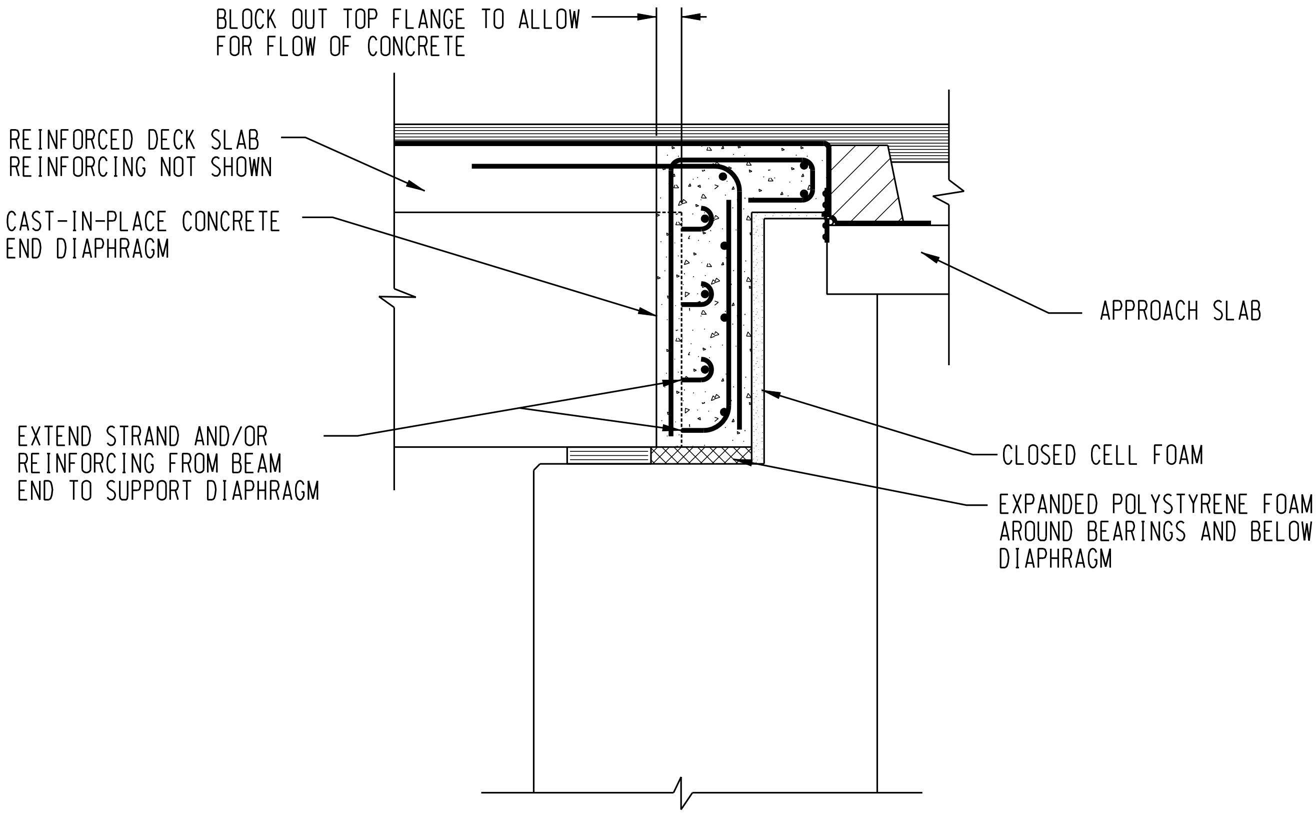
PRECAST PARAPET OPTION

ISSUE DATE: 01-04-10

SHEET: NEXT D – 07

PRECAST/PRESTRESSED CONCRETE INSTITUTE NORTHEAST

PCI WWW.PCINE.ORG



SECTION THROUGH DIAPHRAGM

SAMPLE END DIAPHRAGM DETAIL
CANTILEVER ABUTMENT

NOTES:

1. THE DETAILS SHOWN DO NOT REQUIRE THE USE OF INSERTS OR HOLES IN THE BEAM STEMS. THIS METHOD FACILITATE FABRICATION AND IS PREFERRED.

2. THESE DETAILS ARE SIMILAR TO MASSACHUSETTS DEPARTMENT OF TRANSPORTATION STANDARDS. DETAILS FOR OTHER STATES WILL VARY.

3. THE INSERTS FOR THE THREADED DOWELS IN THE STEMS SHALL BE PLACED SO THAT THEY DO NOT INTERFERE WITH THE PRESTRESSING STRAND PATTERN AND ARE LOCATED A MINIMUM OF 8″ FROM THE ENDS OF THE BEAMS.

4. INTERMEDIATE DIAPHRAGMS ARE NOT REQUIRED.

5. IF THE TOP FLANGE IS BLOCKED OUT AS SHOWN, IT IS RECOMMENDED THAT THE SOME OR ALL OF THE STRANDS BE DEBONDED OVER THE SAME LENGTH TO MINIMIZE CRACKING AT RELEASE.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SAMPLE DIAPHRAGM DETAILS

ISSUE DATE: 01-04-10

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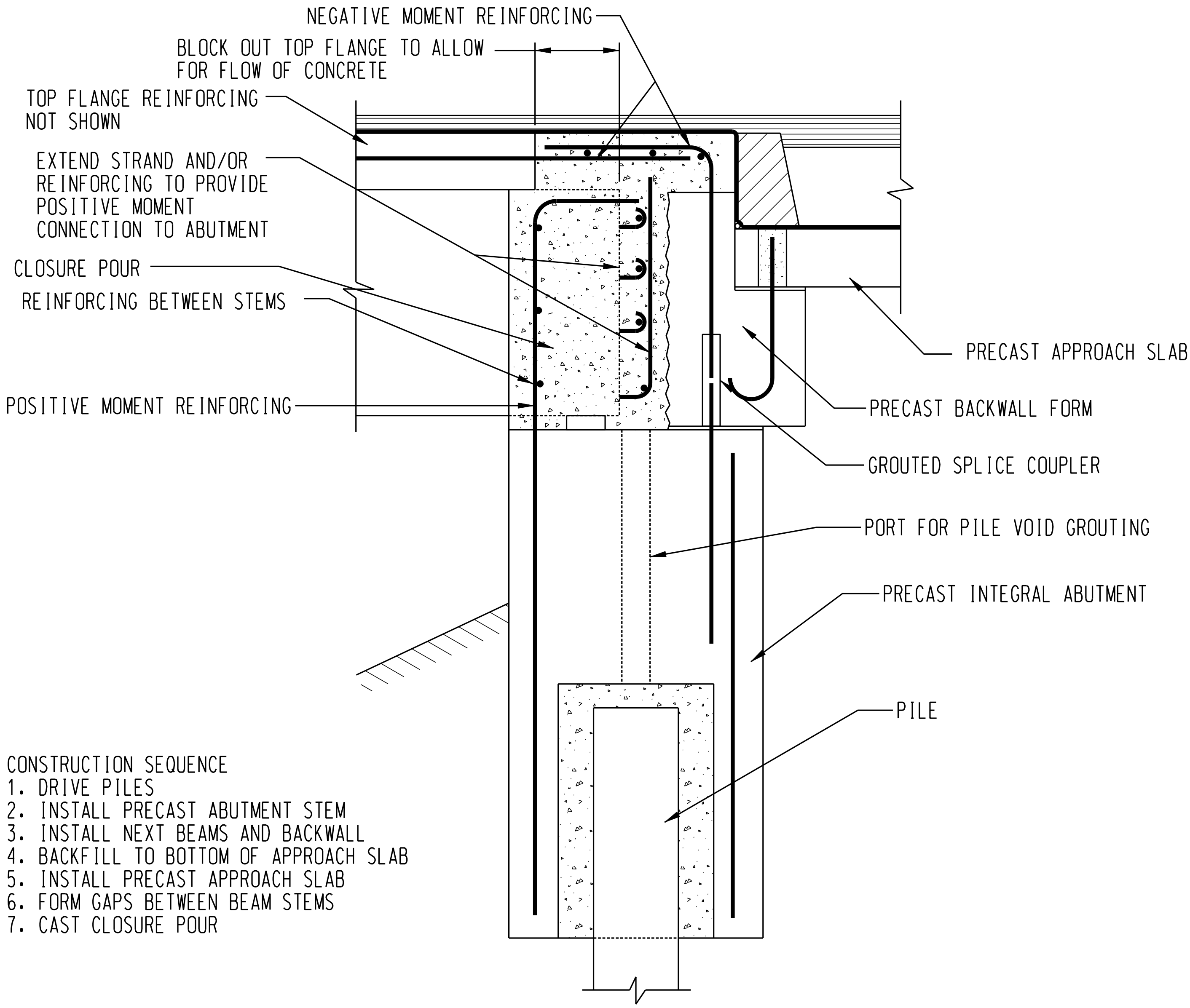

CONSTRUCTION SEQUENCE
1. DRIVE PILES
2. INSTALL PRECAST ABUTMENT STEM
3. INSTALL NEXT BEAMS AND BACKWALL
4. BACKFILL TO BOTTOM OF APPROACH SLAB
5. INSTALL PRECAST APPROACH SLAB
6. FORM GAPS BETWEEN BEAM STEMS
7. CAST CLOSURE POUR

CONCEPTUAL INTEGRAL ABUTMENT SECTION

NOTES:

1. THESE DETAILS ARE BASED ON MASSACHUSETTS DEPARTMENT OF TRANSPORTAION STANDARDS. DETAILS FOR OTHER STATES WILL VARY.

2. A PRECAST PIECE SIMILAR TO THE BACKWALL PIECE CAN BE USED AT THE ENDS OF THE ABUTMENT ALSO.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SAMPLE DIAPHRAGM DETAILS

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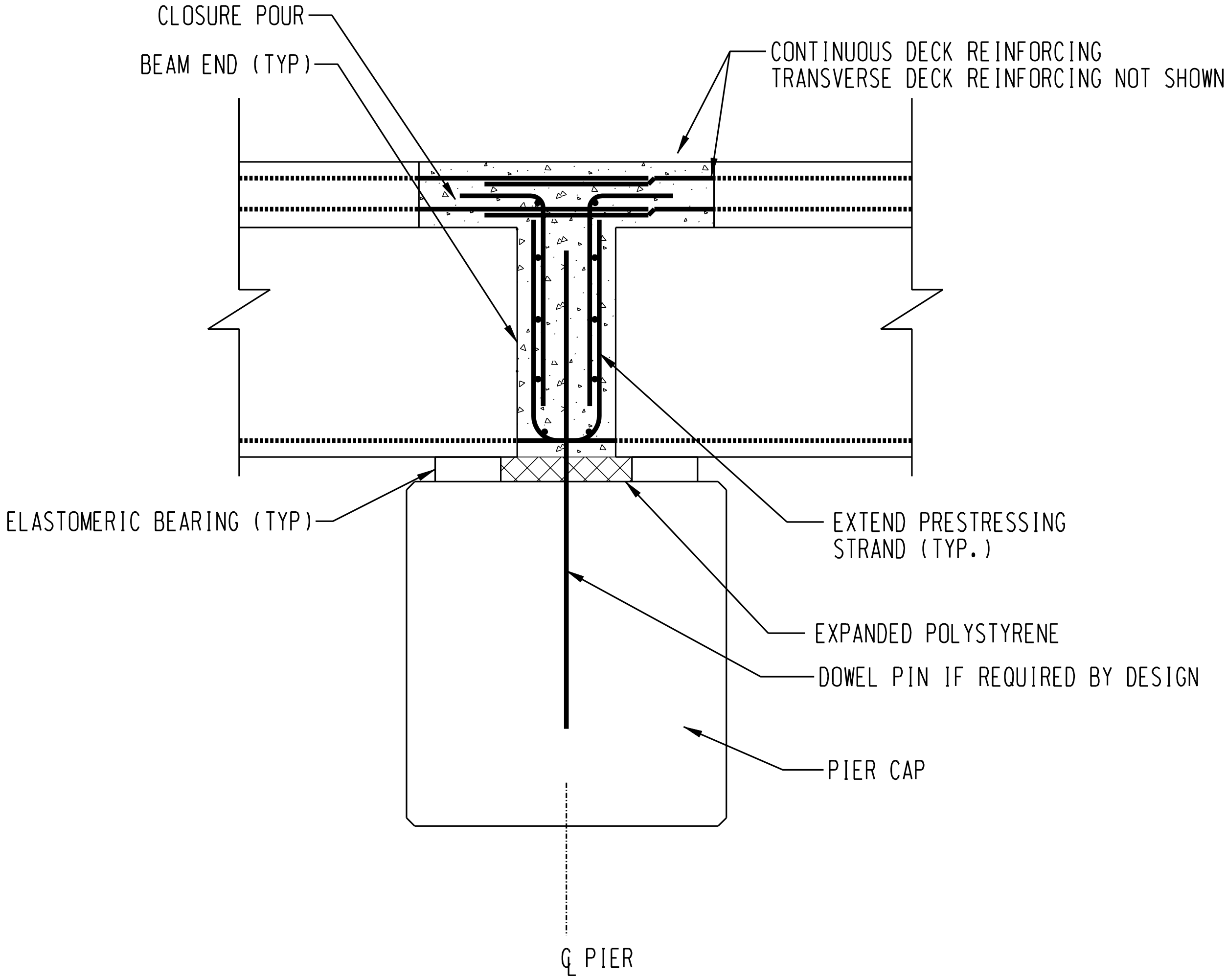

SAMPLE PIER CONTINUITY DETAIL

NOTES:

1. THE DETAILS SHOWN ARE SCHEMATIC. REFER TO STATE STANDARDS FOR SPECIFI DETAILS.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

PIER CONTINUITY DETAIL

ISSUE DATE: 01-04-10

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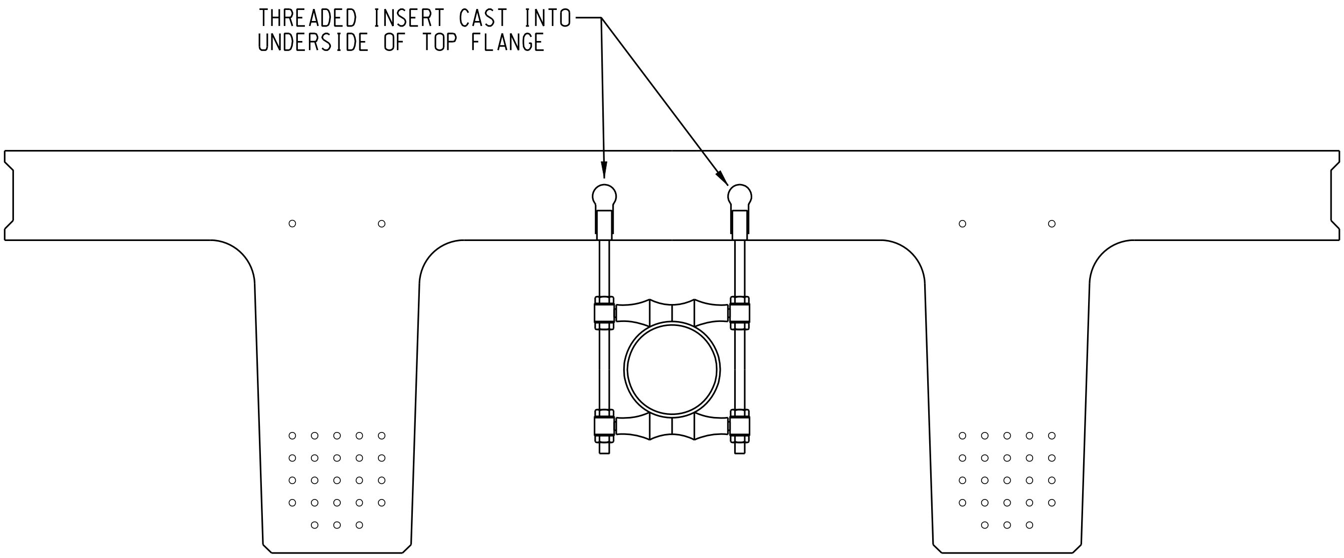

SAMPLE UTILITY SUPPORT DETAILS

NOTES:

1. HANGER RODS FOR UTILITIES SHOULD BE ATTACHED TO THE BEAM BY MEANS OF CAST-IN-PLACE INSERTS. OVERHEAD DRILLED-IN ANCHORS SHOULD NOT BE USED. REFER TO STATE POLICIES FOR OVERHEAD ANCHORING.

2. PLACEMENT OF THE ANCHORS IN THE FLANGE IS PREFERRED. PLACEMENT OF ANCHORS IN THE STEM MAY BE CONSIDERED, HOWEVER THE POTENTIAL FOR INTERFERENCE WITH THE STEM REINFORCING AND STRAND SHOULD BE INVESTIGATED.

3. ONE TYPE OF UTILITY SHOWN, OTHER UTILITIES SIMILAR. REFER TO INDIVIDUAL UTILITY COMPANY DETAILS.

REVISIONS

NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

UTILITY SUPPORT DETAIL

ISSUE DATE: 01-04-10

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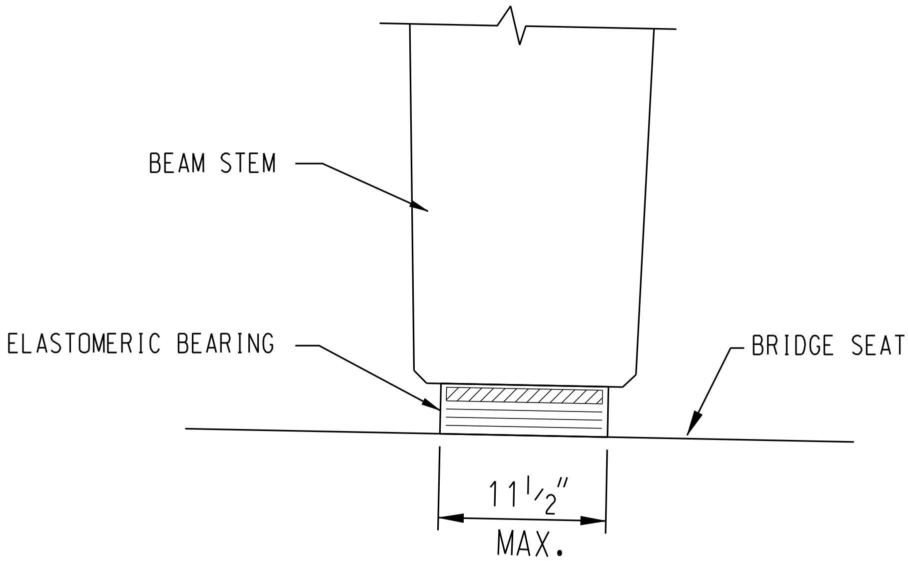
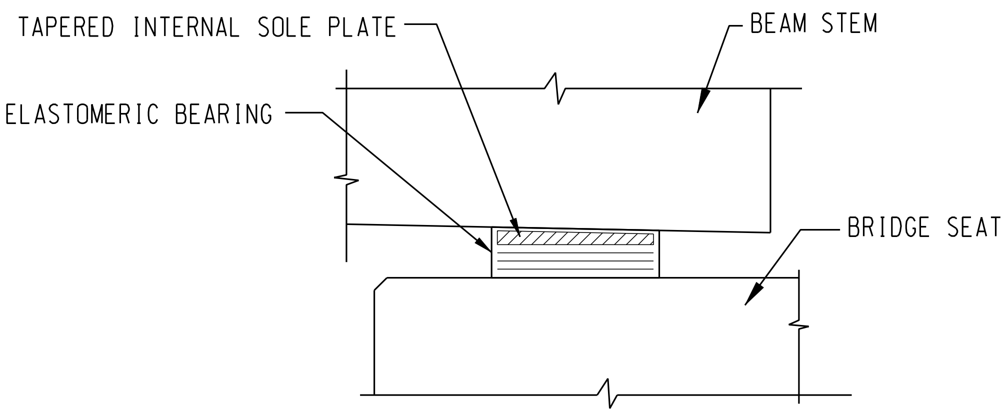


FRONT ELEVATION



SIDE ELEVATION

TAPERED ELASTOMERIC BEARING DETAILS

NOTES:

1. THESE DETAILS ARE SIMILAR TO MASSACHUSETTS DEPARTMENT OF TRANSPORTATION STANDARDS INCLUDING THE USE OF AN EMBEDDED TAPERED STEEL SOLE PLATE. DETAILS FOR OTHER STATES WILL VARY.

2. BRIDGE SEAT AND BEARING MAY BE SLOPED TO MATCH THE CROSS SLOPE OF THE ROADWAY ABOVE.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

BEARING DETAILS 1

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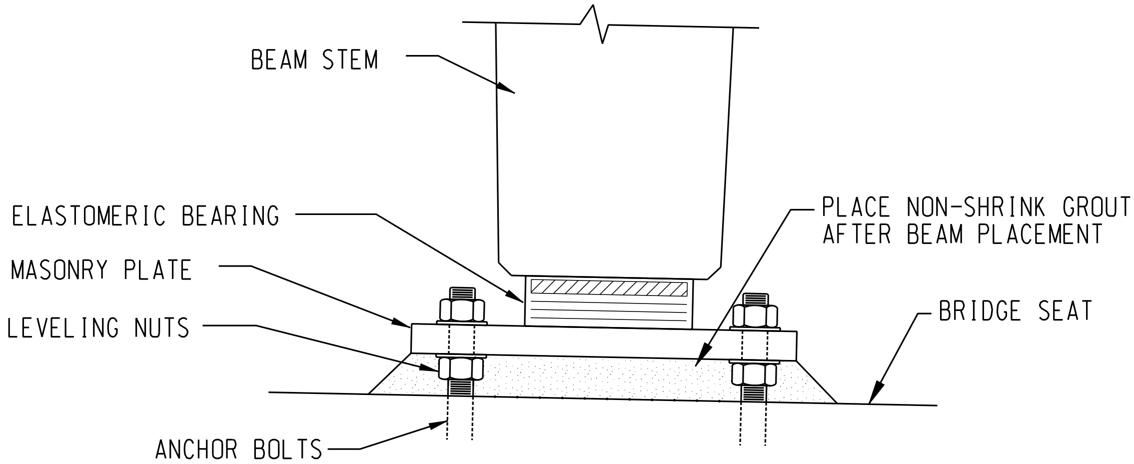
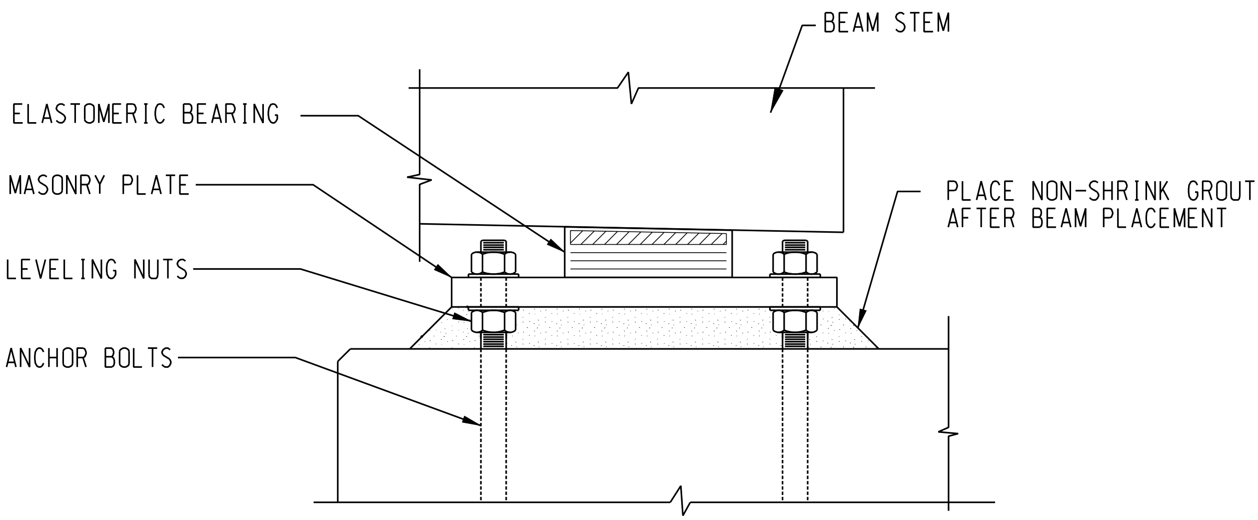


FRONT ELEVATION



SIDE ELEVATION

OPTIONAL ADJUSTABLE MASONRY PLATE DETAILS

NOTES:

1. THESE DETAILS ARE ONLY REQUIRED FOR NON-INTEGRAL SUBSTRUCTURES.
2. GRADE ADJUSTMENT PLATES CAN BE USED WITH NARROW OR WIDE BEARING DETAILS.
3. SIZE THE SOLE PLATE TO SUPPORT THE SELF WEIGHT OF THE BEAMS. PLACE GROUT PRIOR TO PLACING ADDITIONAL LOAD ON THE BEARING.
4. BRIDGE SEAT AND MASONRY PLATE MAY BE SLOPED TO MATCH THE CROSS SLOPE OF THE ROADWAY ABOVE.
5. IT IS RECOMMENDED THAT THE LEVELING BOLTS BE SET PRIOR TO RELEASE OF THE BEAM FROM THE CRANE.

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

BEARING DETAILS 2

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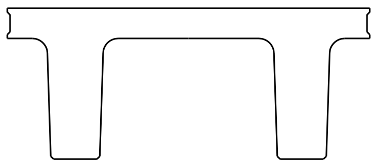
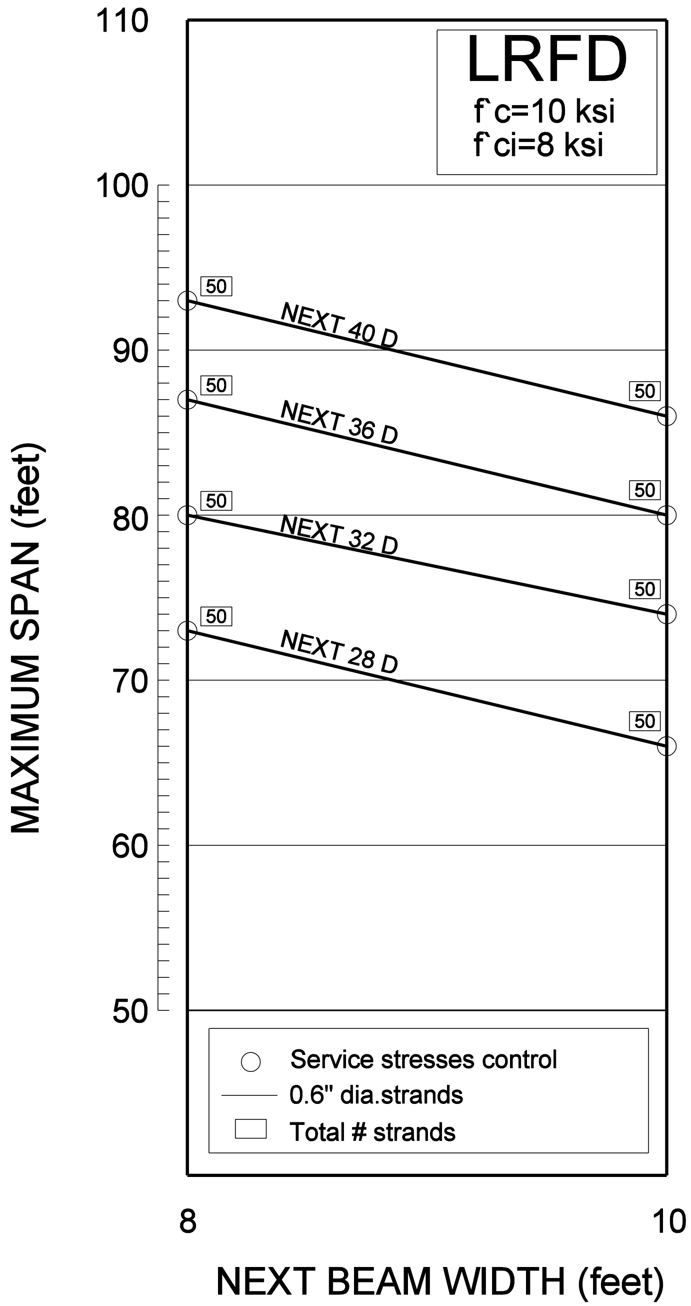


Chart NEXT-1
Northeast Extreme Tee - NEXT



PRELIMINARY

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

DESIGN PARAMETERS

1. 18 inch wide concrete curbs with steel rail
2. 3 inch thick bituminous concrete overlay
3. Beam f'c = 10000 psi
4. Beam f'ci = 8000 psi
5. Debond up to 25% of strand
6. AASHTO LRFD design with allowable tensile stresses for extreme exposure
7. Straight strand only
8. No utility loads
9. Design for interior beam
10. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c=10 KSI)

ISSUE DATE: 01–04–10

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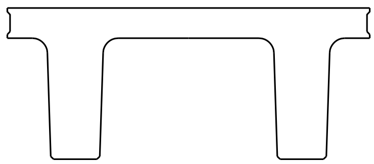
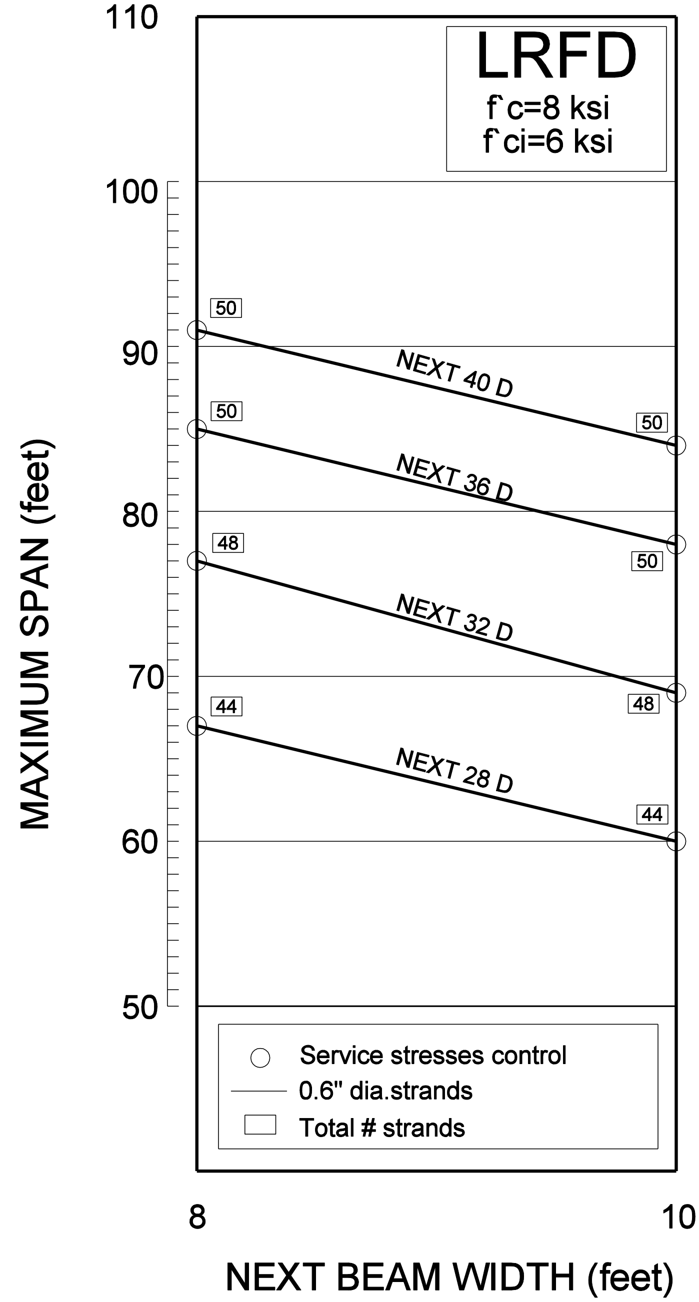
Chart NEXT-2
Northeast Extreme Tee - NEXT





PRELIMINARY

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

DESIGN PARAMETERS

1. 18 inch wide concrete curbs with steel rail
2. 3 inch thick bituminous concrete overlay
3. Beam f'c = 8000 psi
4. Beam f'ci = 6000 psi
5. Debond up to 25% of strand
6. AASHTO LRFD design with allowable tensile stresses for extreme exposure
7. Straight strand only
8. No utility loads
9. Design for interior beam
10. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

REVISIONS

NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c=8 KSI)

ISSUE DATE: 01-04-10

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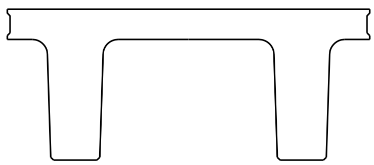
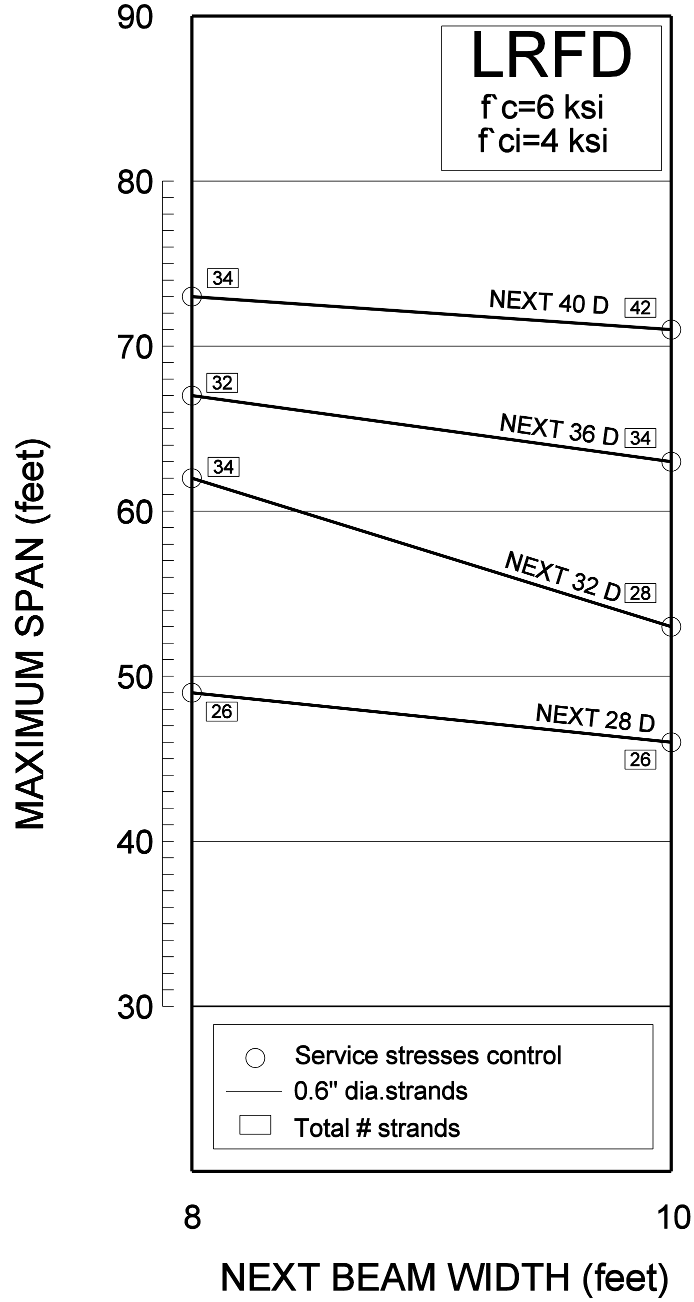


Chart NEXT-3
Northeast Extreme Tee - NEXT



PRELIMINARY

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

DESIGN PARAMETERS

1. 18 inch wide concrete curbs with steel rail
2. 3 inch thick bituminous concrete overlay
3. Beam f'c = 6000 psi
4. Beam f'ci = 4000 psi
5. Debond up to 25% of strand
6. AASHTO LRFD design with allowable tensile stresses for extreme exposure
7. Straight strand only
8. No utility loads
9. Design for interior beam
10. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c = 6 KSI)

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LIGHTWEIGHT CONCRETE

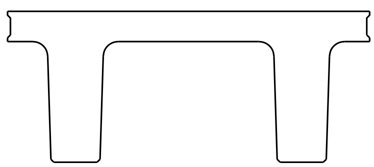
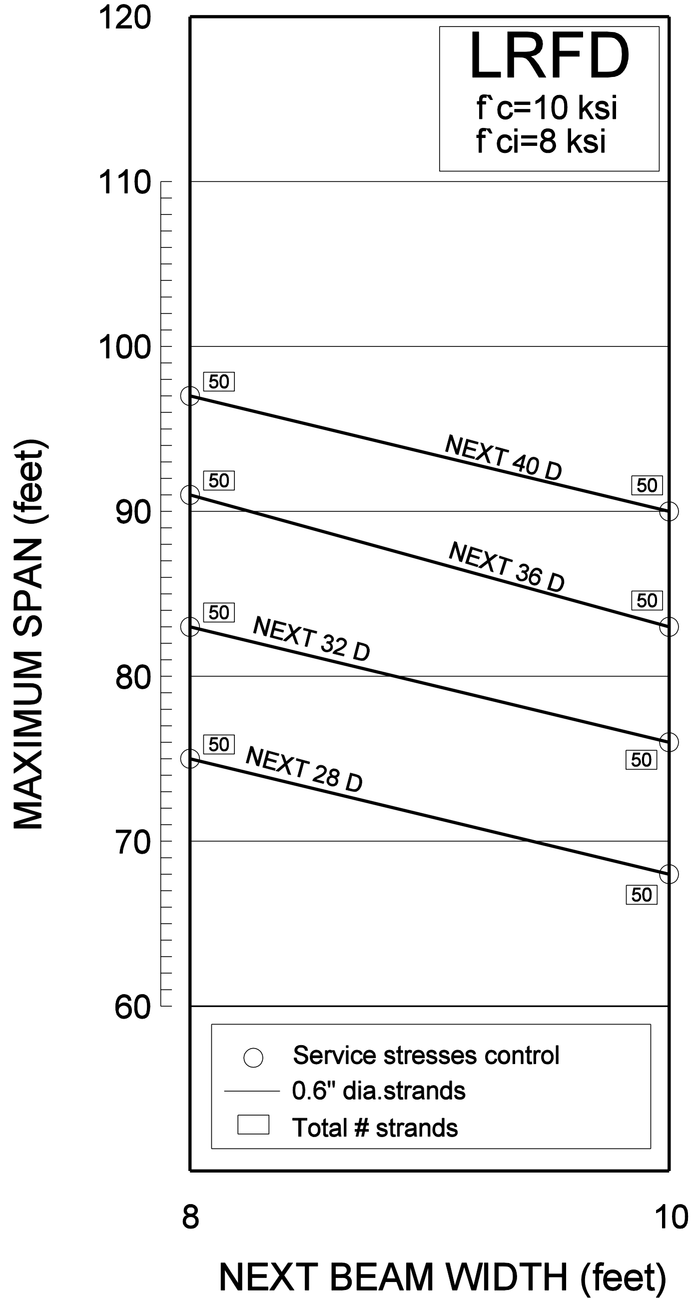


Chart NEXT-1-LW
Northeast Extreme Tee - NEXT



DESIGN PARAMETERS

1. 18 inch wide concrete curbs with steel rail
2. 3 inch thick bituminous concrete overlay
3. Lightweight concrete beams (120 pcf)
4. Beam f'c = 10000 psi
5. Beam f'ci = 8000 psi
6. Debond up to 25% of strand
7. AASHTO LRFD design with allowable tensile stresses for extreme exposure
8. Straight strand only
9. No utility loads
10. Design for interior beam
11. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c = 10 KSI)

ISSUE DATE: 01–04–10

SHEET: NEXT D – 17

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LIGHTWEIGHT CONCRETE

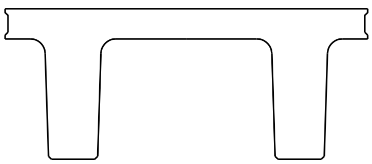
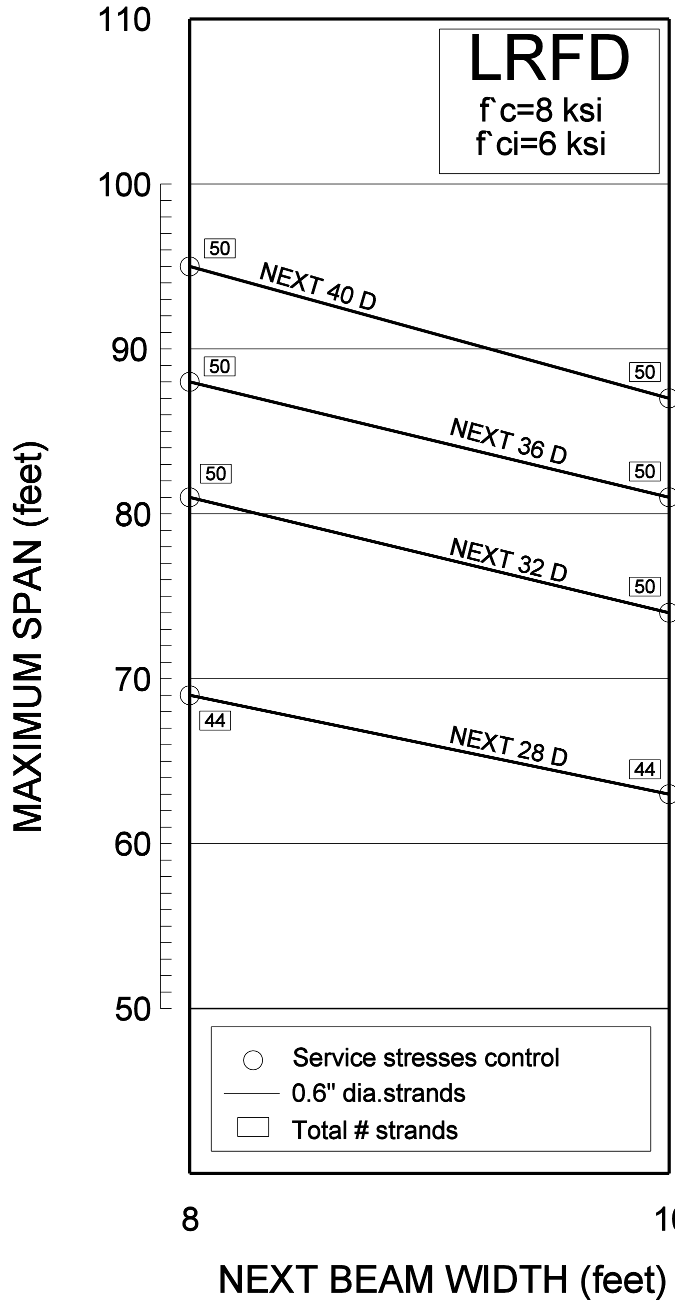


Chart NEXT-2-LW
Northeast Extreme Tee - NEXT



DESIGN PARAMETERS

1. 18 inch wide concrete curbs with steel rail
2. 3 inch thick bituminous concrete overlay
3. Lightweight concrete beams (120 pcf)
4. Beam f'c = 8000 psi
5. Beam f'ci = 6000 psi
6. Debond up to 25% of strand
7. AASHTO LRFD design with allowable tensile stresses for extreme exposure
8. Straight strand only
9. No utility loads
10. Design for interior beam
11. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c=8 KSI)

ISSUE DATE: 01-04-10

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LIGHTWEIGHT CONCRETE

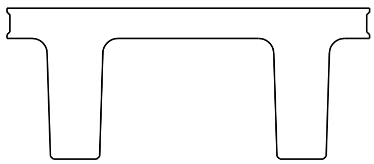
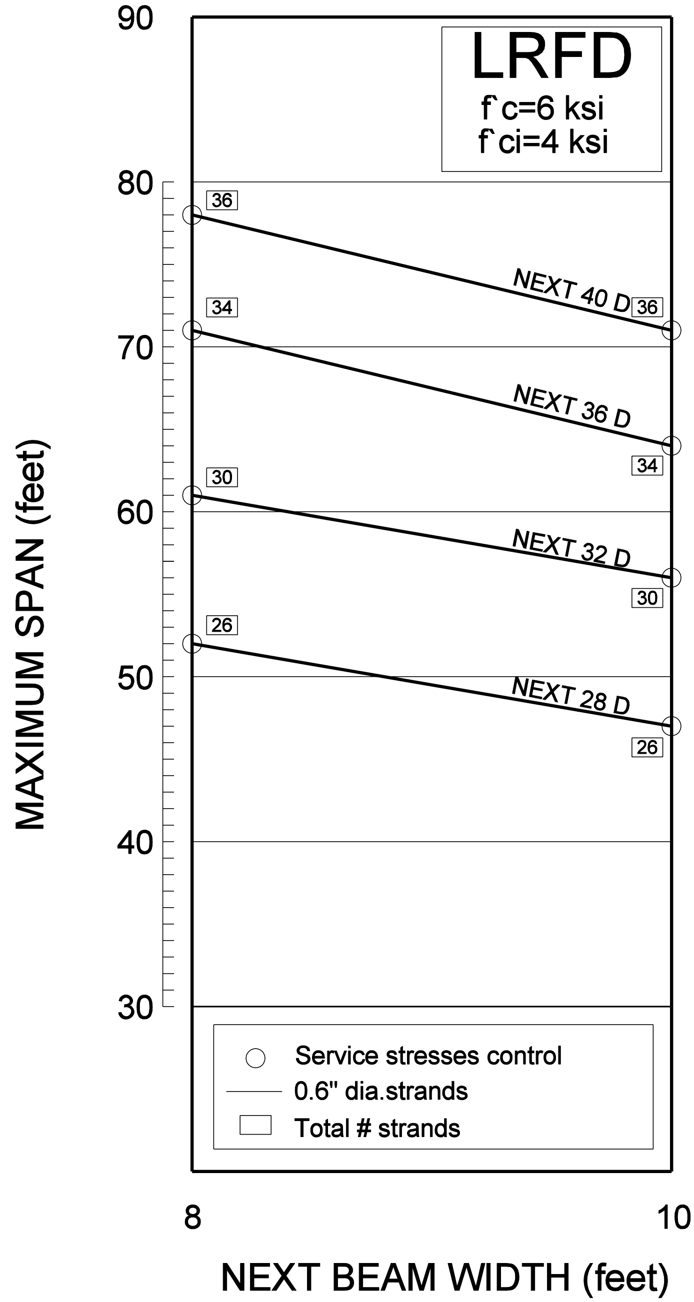


Chart NEXT-3-LW
Northeast Extreme Tee - NEXT



DESIGN PARAMETERS
1. 18 inch wide concrete curbs with steel rail 2. 3 inch thick bituminous concrete overlay 3. Lightweight concrete beams (120 pcf) 4. Beam f'c = 6000 psi 5. Beam f'ci = 4000 psi 6. Debond up to 25% of strand 7. AASHTO LRFD design with allowable tensile stresses for extreme exposure 8. Straight strand only 9. No utility loads 10. Design for interior beam 11. Live load distribution factor based on composite deck stringer bridge, AASHTO cross section Type I

NOTE: EACH BEAM HAS 4 FULLY TENSIONED STRANDS LOCATED 7.5 INCHES FROM THE TOP OF THE BEAM

REVISIONS		
NO.	DATE	DESCRIPTION

NORTHEAST EXTREME BRIDGE TEE
NEXT D BEAMS

SPAN CHART (f'c=6 KSI)

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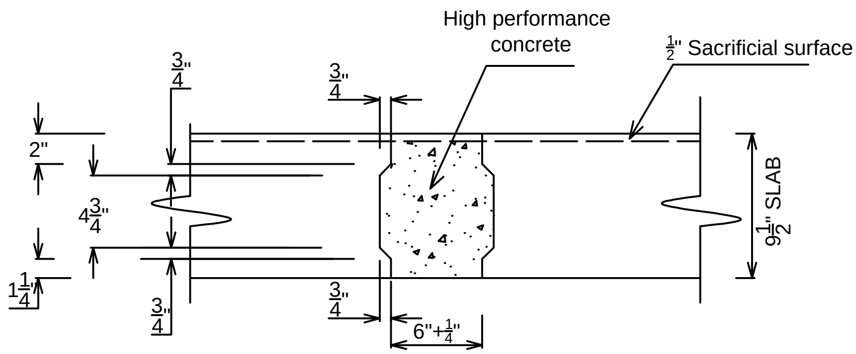
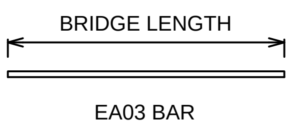
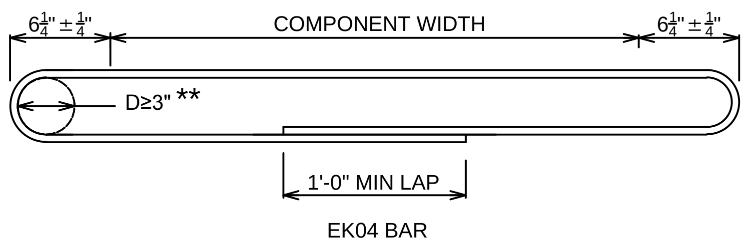
APPENDIX I

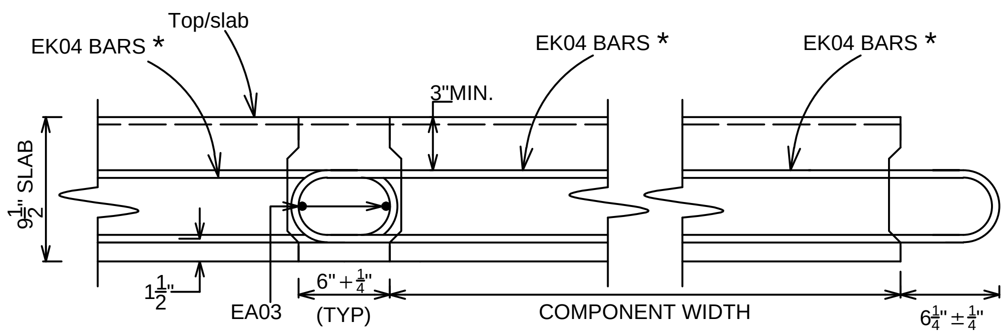
STANDARD LONGITUDINAL CONNECTION DETAILS

DRAWN BY: CHECKED BY: APPROVED BY:	MICHIGAN DEPARTMENT OF TRANSPORTATION BUREAU OF HIGHWAY DEVELOPMENT STANDARDIZED LONGITUDINAL CONNECTION DETAILS	ISSUED: SUPERSEDES:









NOTES:

HIGH PERFORMANCE CONCRETE COMPRESSIVE STRENGTH IS NOT LESS THAN 7 KSI

YIELD STRENGTH OF THE STEEL IS 60 KSI

TEMPERATURE GRADIENT ZONE 3 AND HL-93 MOD LOADS WERE CONSIDERED.

* EK04 BAR SPACING IS : 11" FOR DECKED BOX-BEAM.
7" FOR OTHER DECKED PRECAST PRESTRESSED CONCRETE GIRDERS.

** DIAMETER OF BEND (D) SHALL NOT BE LESS THAN 3 IN.

PREPARED BY DESIGN DIVISION	6.41.xx

APPENDIX J
SPECIAL PROVISION FOR GROUTING PBES CONNECTIONS

(Template)

.... DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION FOR GROUTING PBES CONNECTIONS

1. **General.** This work shall consist of furnishing material, equipment, and manpower for grouting prefabricated component connections (or referred as joints in this section) in accordance with the details shown on the plans and the requirements of these Specification.

   The work shall also include the furnishing and installing of any appurtenant items necessary for completing the grouting operations, including but not limited to, inlets, vents, outlets, and grout and any material used for mixing and curing and protecting grout during the required period.

2. **Contractor Proposed Options:** The contractor may propose for consideration certain changes to the connection details (including but not limited to, the shape, size, reinforcement details), material for filling the voids, application procedures, and curing and protection methods than what is shown in the plans and given in this Specification.

3. **Restrictions to Contractor Proposed Options:** Any changes proposed by the contractor shall comply with the following:

   a. Any changes proposed to the connection details to enhance the grout application procedures shall be demonstrated through mock-up testing or contractors own experience with a previous project.
   b. The ultimate strength of the structure with the proposed changes to the connection details shall meet the requirement of Section xx of the AASHTO XXXX, YY edition, 20XX, and all applicable interims and shall be equivalent or greater than the ultimate strength provided by the original design.
   c. The contractor fully redesigns and details of all the connections and associated components where the alternate details are proposed, as required.
   d. The contractor submits complete shop drawings indicating the locations of the connections and including revised connection and component details, design calculations, and a summary of the specific changes and justification for the changes for Engineer's review.

4. Working Drawings: The contractor shall submit detailed working drawings in accordance with Section XX of the …….. Standard Specification for Construction that include, but are not limited to:

1. Connection detail with multiple views (a minimum of two cross-sections with respect to two perpendicular axes and a plan view)
2. Name (if manufactured grout) or the mix design for each connection in a format similar to Table E–1.
3. Equipment for mixing and placement
4. Formwork, if needed (process of forming and removal; potential challenges such as grout leakage and remedial measures)
5. Surface preparation procedures
6. Grouting procedure and sequence
7. Grout curing, if applicable, and/or protection methods
8. Mock-up testing plan (void if contractor demonstrates prior experience with the specific detail, material, and equipment)
9. QA/QC plan based on the requirements listed in Table E–2

Table E–1. Connection and the grout/special mix

No	***Connection***	***Grout/special mix***
1	Pier column to pier cap	ABC grout extended
2	Transverse connection between deck panels	ABC grout
3	Longitudinal closure	Mix 1
4		
5		
6		

Mix 1: *(example)*

Cement
Supplementary cementitious material
Aggregate
Water
Admixtures

5. Material: The materials to be incorporated into work covered by this section shall conform to the requirements set out herein.

a. Grout/Special mixes

Contractor shall identify a non-shrink grout/concrete mixes based on size, shape, and detailing of the connection, and exposure conditions during mixing, placing, and in-service. Contractor shall submit laboratory test results obtained from an independent testing lab on the following properties as per the specifications listed;

Table E–2. Grout properties, requirements and test methods

Property		**Requirement**	**Test Method**
Strength	1 day		
	3 days		
	7 days		
	28 days		
Slump/flow			
Setting time			
Early age height change			ASTM C827/C1107
Height change of hardened grout			ASTM C1090/C1107
Shrinkage			
Air content			
Freeze/thaw durability			
Modulus of elasticity			
Thermal expansion coefficient			

b. Curing

The contractor shall furnish required grout curing material as per the manufacturer requirement or the Section XX of the ……………Standard Specification for Construction.

c. Grout protection

The contractor shall furnish required grout protection material as per the manufacturer requirement or the Section XX of the ………….Standard Specification for Construction.

6. **Grouting plan and qualifications:** At least XX weeks before grouting commences, the contractor shall submit to the Engineer for review and approval a "Grouting Operation Plan for Precast Component Connections". Written approval of the plan is required before grouting occurs.

    - Names of grouting crew and supervisor
    - Experience of crewmembers and supervisor
    - Training to be provided or undertaken prior to operations
    - Type of equipment to be used, including capacity in relation to demand
    - Working condition of equipment, back-up and spare parts
    - Types, brands, and certifications of materials
    - Identity of independent testing laboratory for certification of materials
    - General grouting procedure
    - Production of grout, on-site testing, adjustments and controls
    - Estimate of grout required amount of each type of grout/special mixes
    - Method of controlling consistency of grout
    - Grout mixing and placement procedures
    - Procedure for controlling w/c ratio, and for ensuring that the water used is acceptable
    - Contractor's QC forms that are to be signed daily by grout supervisor

    The contractor shall, throughout the duration of the grouting, coordinate his work and cooperate with the engineer. The contractor shall also provide at least one person who shall be present at the all times during formwork installation and grouting who is familiar with the operations involved and will direct the work.

7. **Contribution to knowledge base (Report):** The engineer will determine the locations to sample grout and the number and type of samples collected for field and laboratory testing based on the test methods and applicable standards listed in Table E–2. Report should include at least the followings: (the following includes extracts from ASTM C1107 and presented in *italics*)

    - *Source, type and name of grout tested.*
    - *Details of any variations and options practiced by the tester that are recommended or allowed by the manufacturer or others. Also, designate by whom exceptions are allowed or recommended.*
    - *Number and size of each kind of grout specimen and the date molded.*

- *Consistency at the time the specimens were molded and the water to dry solids ratio.*
- *Mixing temperature and curing temperature.*
- *Identity of specimens as being from (a) freshly mixed grout or (b) grout from end of maximum allowed usable working time. State the mixing age of grout when the specimens were prepared.*
- *Height change from placement to time of final setting, %.*
- *Height change of hardened, moist-cured grout at specimen age of 1, 3, 14, and 28 days, %.*
- *Height change of hardened grout at 56 days of age when exposed to air drying for 28 days after 28 days of moist-curing, %.*
- *Compressive strength of cubes at 1, 3, 7, and 28 days.*
- *Yield of the grout.*
- Equipment used for grouting or method of grouting.
- Challenges and lessons learned.
- Recommendations for enhancing performance and construction practices of similar details.