Reflective Cracking between Precast Prestressed Box Girders

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16. Abstract

The adjacent precast prestressed concrete box-beam bridge is the bridge of choice for short and short-to-medium span bridges. This choice is because of the ease of construction, favorable span-to-depth ratios, aesthetic appeal, and high torsional stiffness. However, this bridge is losing favor primarily because of persisting durability performance issues resulting from longitudinal deck cracking at the shear key locations. This project was initiated to develop practical recommendations for modifications to current adjacent precast prestressed box-beam bridge details, specifications, and methods used in Wisconsin with a goal of minimizing the potential for developing longitudinal deck cracking over shear keys.

A list of best practices was developed after conducting (i) an extensive review of state-of-the art literature and highway agency manuals and guides and (ii) a survey of selected highway agencies and fabricators. In addition, the impact of using various wearing surface types on adjacent box-beam bridge superstructure durability performance was evaluated using NBI data. Based on the outcome of these activities, shear key detail and material and construction specifications were updated. The revised details and specifications were implemented on three bridges: one with traditional abutments and a 6 in. thick cast-in-place concrete slab, and the other two with Geosynthetic Reinforced Soil (GRS) abutments and 2 in. thick masonry overlays. Deck cracking over the shear keys was documented during the inspection conducted just after construction and after the bridges had been in service for five months. In addition to the cracking over shear keys, randomly dispersed short cracks were observed on these decks. This observation is consistent for all three bridges. Subsequent analysis was conducted using structural details of these bridges, and material property data recorded during construction shows that shrinkage and thermal gradient loads initiate cracking irrespective of the overlay types used on these bridges. Recommendations include continuing with updated details and specifications while exploring the use of crack resistant overlay types and revising details through additional research.

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EXECUTIVE SUMMARY

The adjacent precast concrete box-beam bridge is the bridge of choice for short and short-to-medium span bridges, especially if removing clearance is limited making falsework removal problematic. However, this bridge is losing favor primarily because of persisting durability performance issues such as longitudinal deck cracking over the shear keys. Therefore, this project was initiated to develop practical recommendations for modifications to current adjacent precast prestressed box-beam bridge details, specifications, and methods used in Wisconsin with a goal of minimizing the potential for developing reflective deck cracking.

A list of best design and construction practices was developed and presented after conducting (i) an extensive review of state-of-the art literature and highway agency manuals and guides, (ii) a survey of selected highway agencies and fabricators, and (iii) an evaluation of the impact of wearing surface types on adjacent box-beam bridge superstructure durability performance.

Based on the outcome of these efforts, Wisconsin DOT design details along with material and construction specifications were updated. The use of box-beams with rectangular voids is continued because these sections have the least dead load per unit length and minimize the impact on the fabrication process. Shear key configuration was updated to a full-depth shear key which allows adequate confinement to prevent grout spall even if the shear key material is debonded from the beam. Further, full-depth shear keys have shown an improved performance of bridge superstructure in terms of longitudinal deck cracking. An asphalt wearing surface with a waterproofing membrane has shown better performance compared to a cast-in-place deck slab and has contributed to maintaining adjacent box-beam bridges in good or satisfactory condition over 30 years. However, overlay types used in Wisconsin were not updated because the use of asphalt overlays with a waterproofing membrane has been discontinued in Wisconsin.

The revised details and specifications were implemented on three bridges. One of the bridges was constructed on traditional abutments with a 6 in. thick cast-in-place concrete slab. The other two bridges were constructed on Geosynthetic Reinforced Soil (GRS) abutments with 2 in. thick masonry overlays. The updated specifications and design details of these three bridges are presented in Appendix F. These bridges were inspected just after construction and after being in service for five months, and longitudinal deck cracking over the shear keys was documented.

Randomly dispersed short cracks were also observed on these decks. All three bridges show a similar pattern of cracking. Subsequent analysis was conducted using structural details of these bridges and material property data recorded during construction. The analysis results revealed that the stresses developed due to shrinkage and thermal gradient loads are high enough to initiate cracking in typical concrete mixes. Full-depth cracking due to shrinkage and thermal gradient loads initiates over the supports. Thin concrete overlay used on B-14-216 and B-14-217 undergoes higher stresses than the thick overlay due to smaller volume-to-surface ratio. Cast-in-place end diaphragms are used in B-26-40 bridge to tie the superstructure with the integral abutments. Bridge specifications allow casting the diaphragms with the slab or two weeks prior to placing the slab. Casting the diaphragms and slab together reduces the slab stresses due to concrete shrinkage, a practice recommended to continue. Recommendations include continuing with updated details and specifications while exploring the use of crack resistant overlay types and revising transverse connection details through additional research.

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1.1 OVERVIEW

The adjacent precast concrete box-beam bridge, also known as the side-by-side box-beam bridge, is the bridge of choice for short and short-to-medium span bridges. This choice is because of the ease of construction, favorable span-to-depth ratios, aesthetic appeal, and high torsional stiffness. However, this bridge is losing favor primarily because of persisting durability performance issues related to longitudinal cracking at the shear key locations. This bridge design was first introduced to the U.S. in the 1950's. Design changes have been periodically incorporated since then primarily to abate the cracking. However, performance problems, specifically the longitudinal deck cracking over shear keys, still persist. The deck cracking is identified as the leading cause for triggering other distresses and safety concerns.

1.2 PROJECT OBJECTIVES AND TASKS

The objective of this project is to suggest practical recommendations for modifications to current adjacent precast prestressed box-beam bridge details, specifications, and methods used in Wisconsin with a goal of minimizing the potential for developing longitudinal deck cracking over the shear keys. The investigation includes the following:

- Review past and present efforts documented in literature to solve this problem.
- Survey WisDOT regional bridge maintenance engineers, industry fabricators, and other State DOT's to identify extent and consistency of this problem.
- Review past applications of this structure type in Wisconsin including visiting sites to make observations and measurements.
- Make recommendations for improved policies, design detailing, specifications, and construction inspection.
- Identify a candidate precast box-beam test structure to incorporate revised details and construction practices.
- Monitor the structure and document the performance of updated practices.

1.3 REPORT ORGANIZATION

The report is organized into 7 chapters:

- Chapter 2 - The State-of-the-Art Literature Review
- Chapter 3 - State Department of Transportation Practices
- Chapter 4 - In-service Bridge Performance
- Chapter 5 - Newly Built Bridge Performance
- Chapter 6 - Summary, Conclusions, and Recommendations
- Chapter 7 - Bibliography

The report appendices include the following data:

- Appendix A: Grout and Special Mixes for Box-Beam Shear Keys
- Appendix B: State Department of Transportation Practices
- Appendix C: Wisconsin Department of Transportation Practices
- Appendix D: Survey Questionnaire and Results
- Appendix E: Wisconsin Bridge Performance
- Appendix F: Revised Specifications and Design Details
- Appendix G: Newly Built Bridge Performance

2 STATE-OF-THE-ART LITERATURE REVIEW

2.1 OBJECTIVE AND APPROACH

Recent studies and findings/recommendations related to box-beam bridge performance are documented in this chapter. Specific attention is given to box-beam geometry, shear key configuration, transverse post-tensioning procedures, shear key grout materials, deck overlay or concrete slab, bearing layout, and design and construction recommendations.

2.2 BRIDGE PERFORMANCE

Longitudinal reflective deck cracking along the shear keys of side-by-side box-beam bridge superstructures has been identified as a recurrent problem (Huckelbridge et al. 1995; Lall at el. 1998; Miller et al. 1999; Aktan et al. 2005; Attanayake and Aktan 2013). The reflective cracks allow penetration of surface water along the full-length of the beams. Since the moisture often gets trapped in the shear keys, reflective deck cracking is a durability concern in states where de-icing salt is used for winter maintenance. The surface water is often laced with chloride ions that diffuse into concrete and initiate corrosion of prestressing tendons and reinforcements leading to delamination, cracking, spall and breaking of the prestressing tendon.

The purpose of shear keys and transverse post-tension is for the beams to develop continuity in the transverse direction for developing moment and shear stiffness. Reflective cracking of the bridge with full-depth shear keys and transverse post-tensioning may not lead to differential movement of the beams unless the shear keys are not properly grouted or there is significant deterioration. In the case of partial-depth grouted shear keys, the shear key failure may lead to loss of load transfer capacity between two adjacent box-beams. In this case, overloading of individual box beams may occur (Harries 2006).

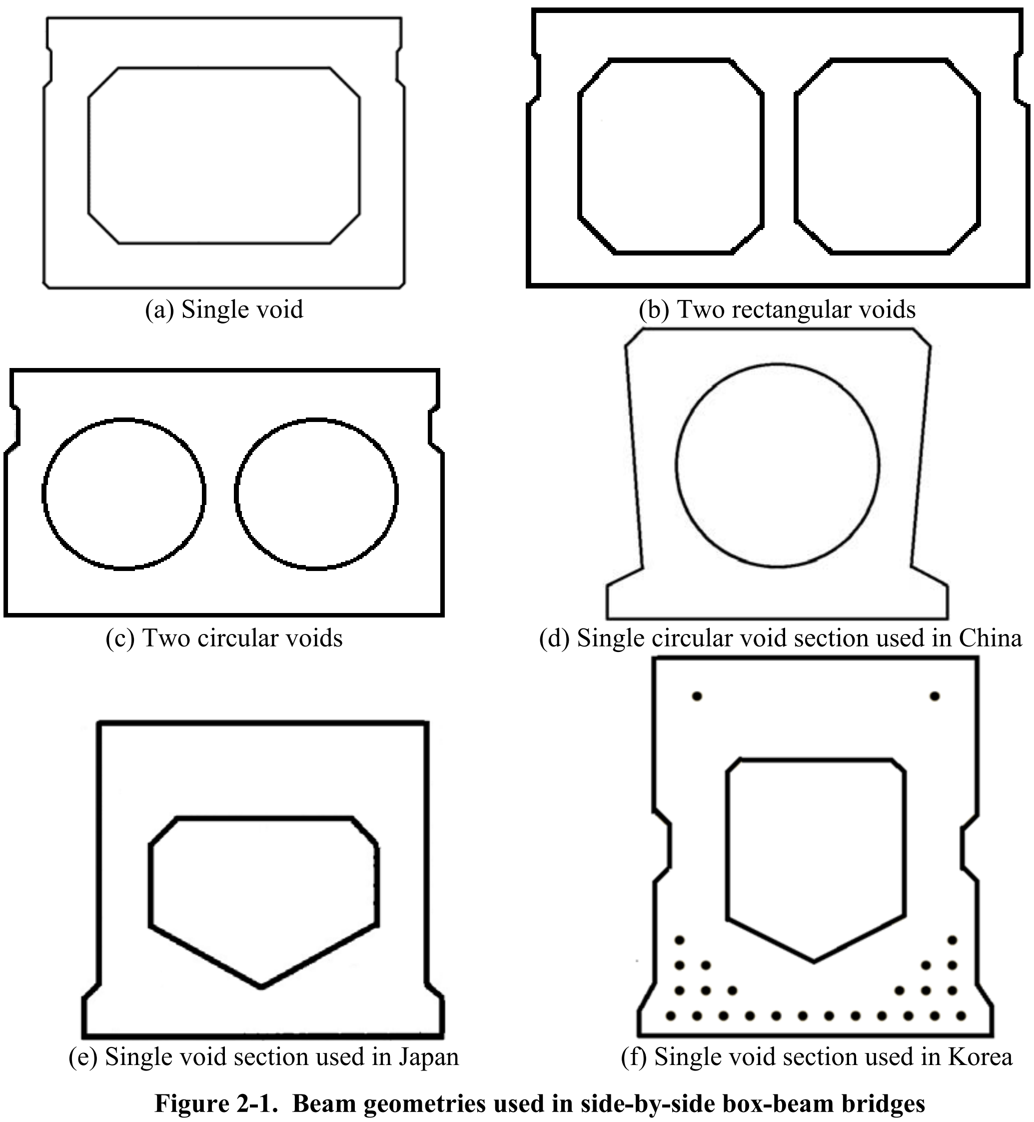
The box beam connection details and construction practices that assure durability of the system are essential. Addressing the critical need, NCHRP Synthesis Report 393 was developed documenting the best practices implemented by the North American Highway Agencies in terms of connection design, details, materials, and construction practices (Russell 2009). At about the same time, the Precast/Prestressed Concrete Institute also published a report on the State-of-the-Art of Precast/Prestressed Adjacent Box Beam Bridges (PCI 2011). In conjunction with these studies, two comprehensive national surveys were conducted to identify the extent and consistency of the reflective cracking problem as well as collecting information on connection design, details, materials, and construction practices (Russell 2009; PCI 2011).

Russell (2009) and PCI (2011) provide a list of recommendations for design, fabrication, and construction. Throughout the years, a few states, including Michigan and New York, have implemented a majority of the design, fabrication, and construction best practices, described in the above two documents. Attanayake and Aktan (2013) described Michigan's evolving design provisions, performance of in-service bridges, and observations during demolitions and construction of new bridges. Michigan's most up-to-date design and construction specifications could not abate reflective cracking prior to opening the bridge for traffic. The design includes full-depth grouted partial depth shear keys, transverse post-tensioning, and a 6-in. thick cast-in-place concrete slab (Attanayake and Aktan 2013). Cracking was also documented at the interface between the grout and box-beam during construction. Several researchers proposed that increasing the magnitude of post-tensioning force can be a potential solution to mitigate reflective cracking (Grace and Jensen 2008). However, Ulku et al. (2010) have demonstrated that increased post-tensioning will not mitigate reflective deck cracking. Lall et al. (1998) acknowledge having reflective deck cracking on the NYSDOT system as well. NYSDOT has implemented full-depth shear keys, a cast-in-place concrete slab, and transverse post-tensioning.

The following sections of this chapter present the efforts and outcome of recent studies to mitigate reflective deck cracking and associated durability performance.

2.3 BOX-BEAM GEOMETRY

The common box-beam standard sections include rectangular or circular voids. The selection of a beam geometry for a specific bridge is based on the availability of formwork, fabricator familiarity, and ease of fabrication. Hence, the standard section shown in Figure 2-1a is commonly used by the North America highway agencies. The standard sections in China, Japan, and Korea are presented in Figure 2-1d, e, and f (Yuan et al. 2013; Yamane et al. 1994; Kim et al. 2008). In earlier bridges, the box-beam void was formed using corrugated cardboard boxes. Fabrication challenges and accumulation of moisture inside the void forced highway agencies to look for alternate materials to form the voids (Aktan el al. 2005). Also, the concrete cover of the top layer of prestressing strands in the box-beam bottom flange and located nearest the void was not adequate (Aktan et al. 2005). At present, the void is formed by placing nonabsorbent material such as Styrofoam blocks. Drains or weep holes are provided at the bottom flange to drain moisture from inside the void. Storm et al. (2013) discuss implications of Styrofoam block deformation on prestressed box-beam camber calculation. Even though the focus of their work was on camber calculation, the Styrofoam block deformation may cause nonuniform thickness and insufficient concrete cover to prestressing strands. The uneven interior concrete surface may also affect the effectiveness of the drains provided at the end of each void. In that regard, Japan and Korean standard sections can provide adequate cover to prestressing strands for enhanced durability.



(a) Single void (b) Two rectangular voids

(c) Two circular voids (d) Single circular void section used in China

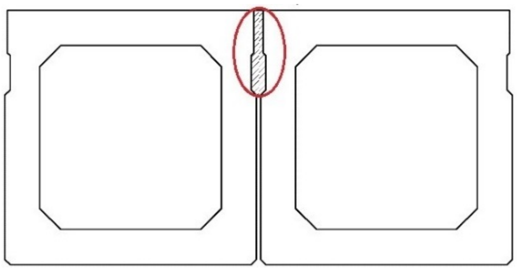
(e) Single void section used in Japan (f) Single void section used in Korea

Figure 2-1. Beam geometries used in side-by-side box-beam bridges

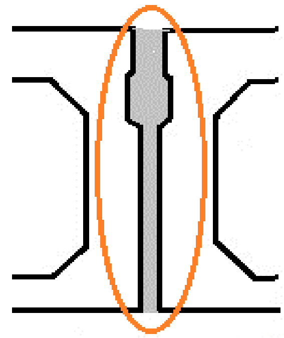
2.4 SHEAR KEY CONFIGURATION

The partial-depth shear key (Figure 2-2a) was the common configuration in earlier bridges. Due to its inability to transfer moment in the transverse direction and observed reflective deck cracking, highway agencies started requiring full-depth grouted shear keys. As an example, since 1985 Michigan has used the full-depth grouted partial depth shear key shown in Figure 2-2b. Field observations documented grout spall (Figure 2-3). The inability to contain the grout within the key region is one of the challenges of using the configuration shown in Figure 2-2b. The NYSDOT uses the shear key configuration shown in Figure 2-2c (Lall et al. 1998). This configuration was developed by the North-East Precast/Prestressed Concrete Institute (NE PCI) and has the ability to contain grout within the shear key. According to Lall et al. (1998), the use of full-depth shear keys, cast-in-place reinforced concrete deck, and transverse post-tensioning reduced reflective deck cracking.

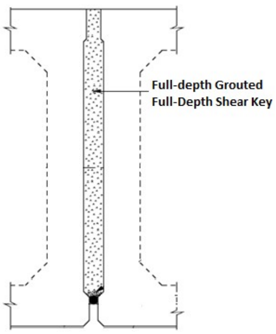
Yamane et al. (1994) presented the shear key configuration used in Japan (Figure 2-2d). The Japanese shear key is formed by filling the space with cast-in-place concrete. Figure 2-2e shows the shear key configuration also with cast-in-place concrete used in Korea (Kim et al. 2008). The first side-by-side box-beam with cast-in-place concrete shear key and transverse post-tensioning was built in Korea in 2005. After conducting static and dynamic load testing on the bridge, Kim et al. (2008) recommend using cast-in-place concrete for the shear keys and transverse post-tensioning. As a result of contractor reluctance to use post-tensioning on site for small span bridges, side-by-side box-beam bridges designed with transverse post-tension did not achieve wide use in the UK in the 1960s. Hence, other forms of transverse connections were designed. One of the designs proposed by the North Western Road Construction Unit (NWRCU) is shown in Figure 2-2f. This design utilized reinforcements within the shear key with cast-in-place concrete (Cusens 1974).



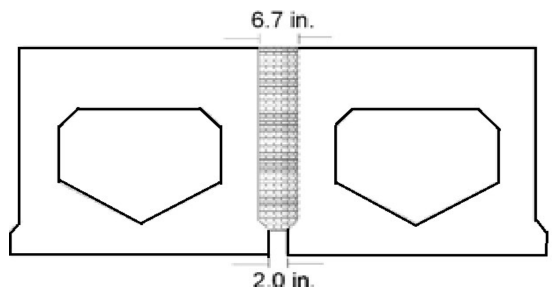
(a) Partial depth shear key



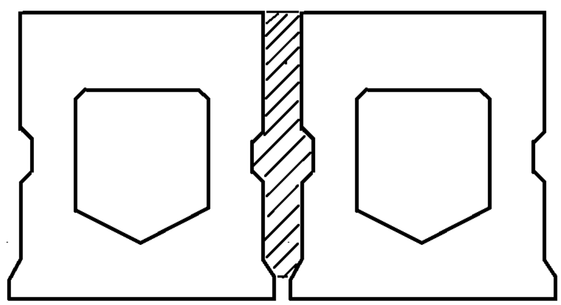
(b) Full-depth grouted partial depth shear key



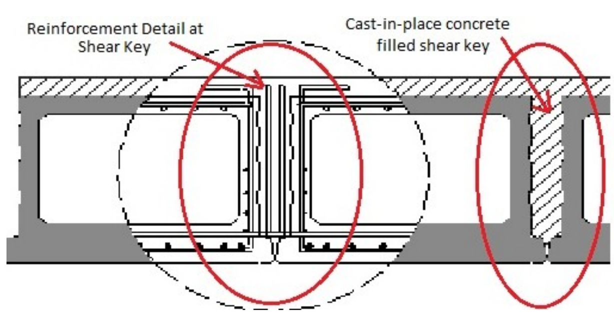
(c) Full-depth shear key



(d) Cast-in-place concrete shear key used in Japan



(e) Cast-in-place concrete shear key used in Korea



(f) Cast-in-place reinforced concrete shear key used in the UK.

Figure 2-2. Shear key configurations used in side-by-side box-beam bridges

Figure 2-3. A shear key with missing grout

Dong (2002) studied the shear key performance by finite element analysis technique under thermal and truck loads. Five shear key configurations were considered in this analysis. Dong (2002) recommended using full-depth shear key to resist the transverse tensile stresses developed near the base of the shear key without exceeding the tensile strength of non-shrink grout material. The bond strength limitation between the grout and beam surface was not considered in this analysis. A perfect bond between the grout and the beam surface was assumed. In general, the bond strength at the beam-grout interface is much weaker than the tensile strength of grout.

2.5 TRANSVERSE POST-TENSIONING

Korea constructed the first side-by-side box-beam bridge in 2005 (Kim et al. 2008). The span of the bridge is 43 ft. Five diaphragms (3 intermediate and two end diaphragms) are placed at equal spacing for transverse post-tensioning. Transverse ducts that were 2 in. in diameter were placed in the upper and lower part of the beam side with a transverse post-tensioning force of 32 kips/strand. End diaphragms included 4 strands while intermediate diaphragms included 6 strands; hence, the force at each end and intermediate diaphragm is 128 kips and 192 kips, respectively. In other words, the post-tensioning force is 19 kips/ft.

Grace et al. (2012) used finite element models to determine the number of diaphragms and amount of transverse post-tensioning force required to mitigate longitudinal deck cracking due to the combined effect of temperature gradient and traffic loads. The analysis recommendations were related to the number of diaphragms and the post-tensioning force to mitigate reflective cracking (Figure 2-4 and Figure 2-5).

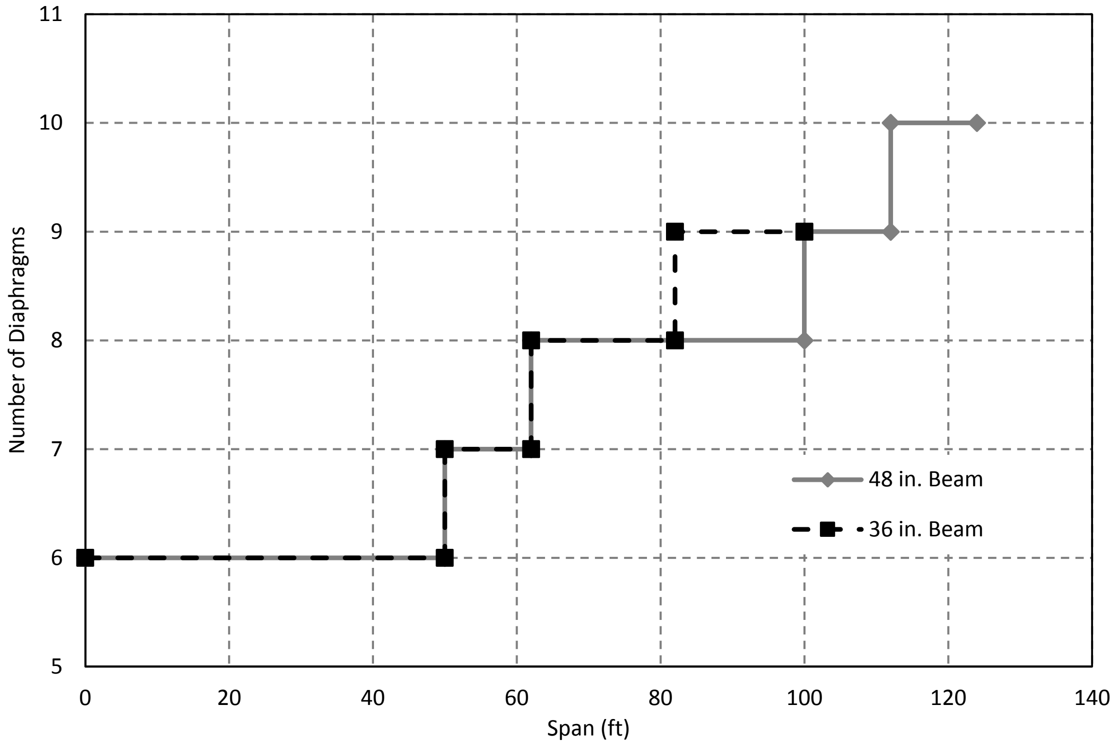


Figure 2-4. Recommended number of diaphragms (source: Grace et al. 2012)

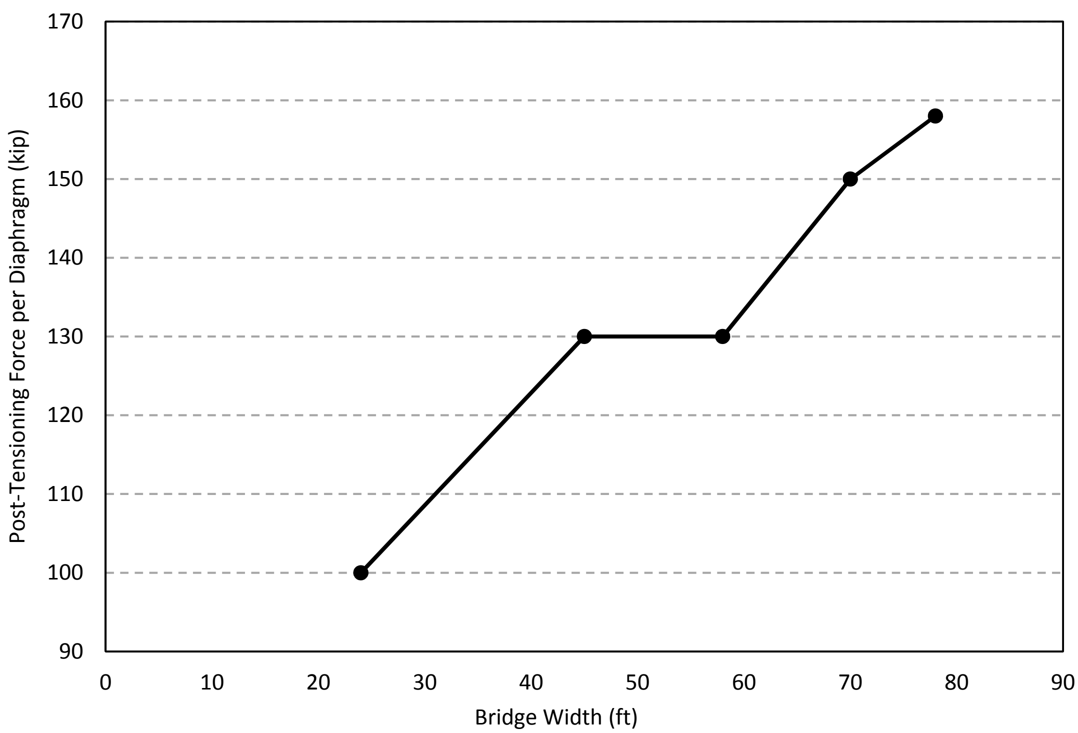


Figure 2-5. Recommended transverse post-tensioning force magnitude for a bridge deck slab with 5000 psi (Source: Grace et al. 2012)

Recommendations from Hanna et al. (2009) are included in the PCI Bridge Design Manual post-tensioning magnitudes to reflect the most recent AASHTO LRFD loads. The outcome of this study is an equation to calculate post-tensioning force per unit length of the bridge. Hence, by knowing the spacing between transverse post-tensioning ducts, the force per diaphragm can be calculated. The proposed equation is given below.

$$P = \left(\frac{0.9W}{D} - 1.0\right) K_l K_s \ \leq \left(\frac{0.2W}{D} + 8.0\right) K_l K_s$$

Where,

D = box depth (in.)
W = bridge width (in.)
K_L = correction factor for span-to-depth ratio
$\quad =1.0 + 0.003(\frac{L}{D} - 30)$
K_S = skew correction factor
$\quad = 1.0 + 0.002\theta$
L = bridge span (in.)
θ = skew angle (deg.)

The recommendations by Grace et al. (2012) and Hanna et al. (2009) are for applying post-tensioning through diaphragms before placing the cast-in-place concrete slab or the wearing surface. Aktan et al. (2009) demonstrated that the shear keys located longitudinally in between the diaphragms are not compressed when post-tensioning is applied through the diaphragms. Hence, transverse post-tensioning and increasing post-tensioning will not prevent cracking at the beam-shear key interface. The cast-in-place concrete slab is expected to provide an added protection in terms of durability. However, the reflective cracks develop in the cast-in-place slab even before the bridge is opened to traffic (Attanayake and Aktan 2013). The solution was to apply post-tensioning in two stages, before and after placing the concrete deck slab. This post-tensioning sequence will compress the cast-in-place concrete slab and close the cracks. Attanayake and Aktan (2009) also developed a rational design process to calculate the post-tensioning required at each diaphragm during each stage of post-tensioning. As an example, Table 2-1 presents the required two-stage post-tensioning for a 25 ft wide, 50 ft long bridge with 5 diaphragms. The effectiveness of applying two-stage post-tensioning was also demonstrated by Ulku et al. (2010). However, Ulku et al. (2010) did not consider the effect of cast-in-place concrete shrinkage and creep effects. Hence, further studies are required to evaluate the impact of the intrinsic loads such as shrinkage and creep effects.

Table 2-1. Post-Tensioning Force Requirement for a 50 ft Long, 25 ft Wide Bridge

	Posttension Force per Diaphragm, kips		
	End Diaphragm	Intermediate Diaphragm	Middle Diaphragm
Before deck placement	7	41	52
After deck placement	63	105	130
Total	70	146	182

Ulku (2009) studied post-tension stress distribution at grouted connections of full-depth deck panels. The study evaluated the effect of grout modulus, the bond between grout and the panel, and post-tension spacing. After studying the post-tension stress distribution at connections with grout of similar modulus to concrete, Ulku (2009) presented two major conclusions: (1) the compression is limited to a width of 1.35 times the precast panel dimension in the direction of post-tensioning; and (2) to achieve a uniform compression, the maximum post-tension spacing needs to be limited to the precast panel dimension measured in the direction of post-tensioning. The compressed zone width can be increased to 2 times the panel dimension in the direction of post-tensioning by using a grout with lower modulus of elasticity. This requires using a grout with an elasticity modulus of less than 50 ksi (i.e. the ratio of panel concrete modulus to grout modulus of 100 or greater). Use of grout with such a low modulus is not practical in prefabricated structures because the grouted connection stiffness is not adequate to transfer load.

With regards to box-beams, post-tensioning can be applied at the top and bottom flanges to develop adequate compression along the shear keys. Attanayake and Aktan (2013) recommended to limit the maximum post-tension spacing equal to the width of the fascia beam. However, it is necessary to perform a cost-benefit analysis, considering the life-cycle cost, and an evaluation of implementation challenges of the above-stated recommendations to justify implementation of such a post-tension configuration.

2.6 GROUT, MORTAR, AND NON-SHRINK MATERIAL

Shear keys are formed with grout, mortar, or cast-in-place concrete. The cast-in-place concrete shear keys are common in Japan and Korea (Yamane et al. 1994; Kim et al. 2008). The common practice in the US is to use mortar (a mix of cement, sand, and water). There is limited bond that develops between the mortar and beam. Hence, the bond between the shear key mortar and the beam surface is lost before grout cracking. The loss of bond was documented during a forensic investigation of a side-by-side box-beam bridge that was being demolished in Michigan

(Attanayake and Aktan 2013). As shown in Figure 2-6, the shear key mortar adherence to the beam was limited. Further, as shown in Figure 2-7, when typical cement mortar is used, grout-beam interface cracking develops shortly after grouting (Attanayake and Aktan 2013).

A few studies evaluated the potential use of commercial grout material. Gulyas et al. (1995) evaluated non-shrink grout using several test configurations including the composite direct tension test. According to Gulyas et al. (1995), the bond failed at a tensile load of 1940 lbs. The equivalent stress magnitude of this load is about 75 psi, which is much lower than the seven-day tensile strength of non-shrink grout (390 psi) used in the test. The use of Set-45 (magnesium ammonium phosphate; $MgNH_4PO_4$) mortar has increased the bond strength by 214% compared to the non-shrink grout. The composite testing of grouted shear key assembly performed by Gulyas et al. (1995) is a practical approach for evaluating the effect of shear key shape, substrate preparation method, grouting procedures, curing methods, and grout properties on the shear key performance.

Sang (2010) evaluated performance of shear key with cementitious grout, epoxy grout, and fiber reinforced cementitious grout. According to his study, epoxy grout and fiber reinforced cementitious grout provided better bond strength as compared to cementitious grout. Hoomes et al. (2014) evaluated bond strength of High Performance Fiber Reinforced Cementitious (HPRFC) materials using the Direct Tension Bond Test (ASTM C 1404), the California Test 551, the Guillotine Shear Test, and the Splitting Prism Test (based on ASTM C 496/496M). The direct tension test provided consistent results. Difficulties in conducting direct tension bond tests were documented. Both the direct tension bond test and the California test 551 data gave the lowest coefficient of variation. Several different mixes were included in the study, and the lowest average bond strength of 366 psi was documented using the direct tension bond test. As can be seen from literature, new material with improved properties have been developed during the last decade. With the use of these materials developing durable prefabricated component connections is a possibility. Hoomes et al. (2014) recommends performing bond strength evaluation tests after identifying the most suitable test configuration to represent the structural response expected from the connection.

Sharpe (2007) summarized the small-scale specimen test results documented in literature. As presented in Sharpe (2007), the weakest failure mode associated with grouted keyways is the beam-shear key interface bond failure. This observation strengthens the discussion presented previously in the chapter.

After evaluating literature on box-beam performance, it can be concluded that a grout material which (i) provides an adequate bond strength for the specific application, (ii) shows non-shrink properties (or a very low shrinkage after maximum expansion), and (iii) has a low permeability or chloride absorption needs to be considered for shear keys to eliminate the weak link in the structural system.

(a) (b)

Figure 2-6. (a) Shear key material attached to a beam and (b) beam surface after grout had fallen off

Figure 2-7. Shear-key interface cracks before deck slab placement

The commercial grouts are of interest to many DOTs. This is especially true for the repair materials which possess high bond strength and compatible properties to concrete. Commercial grouts are considered by several DOTs for prefabricated element connections. Hence, a list of commercially available grout materials and the material properties, application limitations, and possibility of extending the grout for filling larger voids is developed and presented in Appendix A. As discussed in Chapter 3, NYSDOT provides an approved commercial grout list for forming box-beam shear keys.

In addition to commercial grout material properties documented in the manufacturer's data sheets, the laboratory and field data presented in the literature are also summarized in Appendix A. Compressive strength data documented in the literature are compared to the material data sheets.

In the majority of cases, the compressive strength presented in the literature is significantly different than that presented in the material data sheets by the manufacturer. This highlights the importance of conducting mock-up testing in order to evaluate the application procedures and material behavior under anticipated exposure conditions before recommending a specific material for forming shear keys between box-beams.

Ultra high performance concrete (UHPC) is often specified for connecting precast elements. As an example, the Iowa DOT used UHPC to grout dowel pockets at the longitudinal connection between the pi-girders. The exposed surface of the UHPC connection often requires grinding because there is a tendency of steel fibers to protrude out of the surface (Perry et al. 2010). UHPC was also specified to form the longitudinal closure joints between deck modules in the US Highway 6 Bridge over the Keg Creek project in Iowa. The longitudinal connections are 6 in. wide and 8 in. deep and reinforced with #5 hairpin bars at 8 in. spacing. The bridge was constructed in 2011. Seven months later, cracking was documented at the interface between UHPC connections and girder flanges (Phares et al. 2013). According to Royce (2016), NYSDOT has constructed 30 bridges utilizing UHPC connections. Durability performance of these bridges would justify the use of UHPC in the field.

The closure between full-depth deck panels and decked bulb-tee girders is often formed using high performance concrete (HPC). Japan, Korea, and Texas DOT use concrete filled shear keys in box-beam bridges. Appendix A includes the mix designs and material properties of HPC documented in the literature. HPC develops early strength as presented by French et al. (2011) and Freyne et al. (2012). On the other hand, HPC shrinks. Unless adequate bond strength can be developed, HPC may not be suitable for shear keys since it may lead to cracking at the interface between the beam and shear key.

Shrinkage can be managed by using shrinkage reducing admixtures or expansive cement. Chaunsali et al. (2013) investigated the potential use of shrinkage-reducing admixtures (SRA), shrinkage-compensating cements (Type K and G), and mineral admixtures as a means of reducing drying shrinkage cracks in Illinois bridge decks. Mix designs, shrinkage or expansion, and strength values were recorded in two tables in Appendix A, Table A-10 and Table A-11. Table 2-2 provides a summary of the data presented in Appendix A. Type K and G expansive cement showed the lowest shrinkage at the end of 100 days. Furthermore, fly ash and silica fume affect the shrinkage characteristics of expansive cement Type K and G. As shown in Table 2-2, the use of Type K with

Type F fly ash as well as Type G with Type C fly ash resulted in concrete expansion (Chaunsali et al. 2013). Troli and Collepardi (2011) show the potential of reducing shrinkage using *dead burnt lime*, which is CaO processed in temperatures higher than 1000°C. Also, there is a potential of developing expansive concrete using *dead burnt lime* and a shrinkage reducing admixture (SRA). Ramey et al. (1997) showed the impact of mixing and curing conditions on shrinkage/expansion properties of the mixes with Type K cement with or without micro silica. Battaglia et al. (2008) evaluated Eclipse® SRA for Wisconsin concrete and did not recommend its use due to a tendency of destabilizing the air void structure. This product is not included in the Wisconsin DOT approved product list. Eclipse® reduces shrinkage but is not appropriate for developing non-shrink grout or concrete (Table 2-2).

Table 2-2. A Summary of Shrinkage and Strength of Different Admixtures Documented in Literature

Source:	**Admixture**	**Shrinkage**		**Strength**
		με	**%**	**psi**
Chaunsali et al. (2013)	Tetraguard AS20 (4.45 L/m^3)	-450		4700
	Komponent (Type K)	-90		4400
	Conex (Type G) (6%)	-30		4400
	Type K + F Fly Ash	**40**		4250
	Type K + C Fly Ash	-10		4600
	Type K + C Silica Fume	-110		5700
	Type G + F Fly Ash	-70		4250
	Type G + C Fly Ash	**10**		4400
	Type G + C Silica Fume	-100		5500
Freyne et al. (2012)	HPC 1	-260		6976
	HPC 2	-320		7005
Troli and Collepardi (2011)	CaO[1]+SRA	250		7397
	CaO	-220		7832
	Plain concrete	-580		6962
Manufacturer Datasheet	KSC Concrete		0.045	4500
	KSC Komponent Concrete		0.045	4500
	KSC Komponent Grout Mix		0.045	7250
Ramey et al. (1997)	Type K Cement – Cold[2]		0.037	7455
	Type K Cement with MS[3] - Cold		0.027	8369
	Type K Cement - Dry		-0.018	6512
	Type K Cement with MS - Dry		-0.032	7092
	Type K Cement - Hot		0.041	6280
	Type K Cement with MS - Hot		0.041	6701
	Type K Cement - Moist		0.041	6614
	Type K Cement with MS – Moist		0.045	6802
Manufacturer Datasheet	Eclipse 4500		-0.030	

[1] *Dead burnt lime* – CaO subjected to higher temperatures than 1000°C.

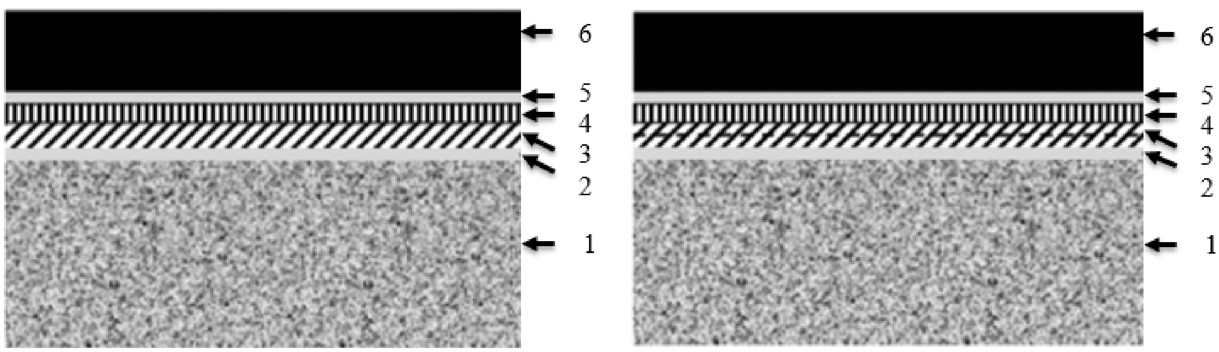
[2] Cold: mixed and moist cured at 40 ^{0}F; Dry: mixed and cured at 40% RH and 72 ^{0}F; Hot: mixed and moist cured at 90 ^{0}F; Moist: mixed and moist cured at 72 ^{0}F.

[3] MS – micro silica

2.7 DECK OVERLAY AND REINFORCED CONCRETE SLAB

2.7.1 Deck Overlay

As shown in Figure 2-8, flexible overlay deck protection systems can be developed with either a preformed sheet or liquid membranes. Each of these deck protection systems consists of five components. These components are placed over the box-beam and shear key assemblage in the following order: primer, membrane, protection board, tack coat, and asphaltic concrete. Recently, the *NCHRP Synthesis 425: Waterproofing Membranes for Concrete Bridge Decks* was developed by Russell (2012) with an objective of documenting information related to materials, specifications requirements, design details, application methods, system performance, and costs of waterproofing membranes used on new and existing bridge decks since 1995. This report section only provides an overview of waterproofing membrane types, advantages of each waterproofing membrane system, installation challenges, and performance issues. Detailed information on waterproofing systems can be found in the NCHRP Synthesis 425 (Russell 2012).



(a) Preformed sheet membrane system

(b) Liquid membrane system

Components:
1. Concrete girders
2. Primer
3. Preformed membrane
4. Protection board
5. Tack coat
6. Asphaltic concrete

Components:
1. Concrete girders
2. Primer
3. Liquid membrane with fabric
4. Protection board
5. Tack coat
6. Asphaltic concrete

Figure 2-8. Components of waterproofing membrane systems (Russell 2012)

According to the survey results presented in Russell (2012), the life of the membrane system is limited by the life of the asphalt. Sixty percent (60%) of respondents acknowledged that 1/4th of the bridge population with waterproofing membranes placed on new bridge decks provided between 16 to 20 years of service life. Moreover, Krauss et al. (2009) present information such as service life, cost, overlay, thickness and installation time for flexible overlay. This information is not specifically for box beams; rather it is for flexible overlay with a waterproofing membrane in general (Table 2-3).

Table 2-3. Flexible Overlay with a Waterproofing Membrane

Rehabilitation Method	Expected Service Life Range (years) [Mean]	Cost Range as of Year 2009 ($/ft^2) [Mean]	Overlay Thickness (in.) [Mean]	Estimated Installation Time
Asphalt overlays with a Membrane	3-40 [12-19]	1.5-23.5 [3.1 - 7.6]	1.5 - 4 [2.4 - 3.1]	> 3 days

2.7.1.1 Membranes

Table 2-4 shows two major waterproofing membrane types (preformed and liquid), advantages of each waterproofing membrane system, installation challenges, and performance issues. The performance of the system is greatly influenced by the installation design, workmanship, and the composition of the waterproofing membrane (Kepler et al. 2000; Price 1989).

Preformed sheets are applied by rolling and using a pressure-sensitive adhesive on the sheet (Russell 2012). The types of preformed sheets are bituminized fabrics, polymer, elastomer based systems, bituminized laminated boards, and mineral dressed bitumen protective sheets (Price 1989). The bituminized laminated board is also used as a protective board in membrane systems.

The liquid membranes are applied using either a spray apparatus or a roller and squeegees. Depending on the manufacturer's recommendations, the membranes are placed as either hot or cold. It may or may not have a reinforcing fabric. If a reinforcing fabric is used, the first layer of liquid is sprayed before the fabric is placed. Then, the fabric is placed over the liquid, and the second layer of liquid is applied (Russell 2012).

Constructed-in-place systems can be subdivided into bituminous and resinous liquid-sprayed systems. Bituminous systems are subdivided into bituminous solutions or compositions and keg mastic asphalt. The keg mastic asphalt requires heat to transform it into liquid. Asphalt-based liquids are various bitumen solutions blended in hydrocarbon solvents, two-part polymer modified compositions or refined natural or elastomer-modified mastic asphalts. Resinous membranes are subdivided into polyurethane, epoxy, and acrylic resin based systems (Price 1989). Polyurethane-based systems are all elastomers with carborundum of coal tar. Other polyurethane systems, known as pitch epoxies, are modified with coal tar. The acrylic systems are based on polymethacrylate resin (Liang et al. 2010).

Table 2-4. Waterproofing Membranes: Advantages, Installation Challenges, and Performance Issues

	Preformed Sheets Membrane	Liquid Membrane
Advantages	• Quality and thickness can be controlled during manufacturing • Elastomer materials provide a moderately satisfactory bond	• Can be applied in one application by a sprayer or a squeegee without laps • Blisters and pinholes are easy to repair in self-sealing materials • Does not depend on the geometry of the deck • Less vulnerable to poor workmanship • Epoxy resin develops an excellent bond with concrete • Acrylic resin develops an adequately strong bond with concrete
Installation Challenges	• Blisters have to be repaired by puncturing and patching • Bitumen protective sheets usually are damaged under base course • Bitumen laminated boards are usually damaged under base coarse asphalt • Difficult to install on curved or rough decks • Installation is labor intensive • More vulnerable to poor workmanship • Require lapses at each edge • Some bitumen fabric material having a thickness less than 1/6 in. (4 mm) are punctured by hot aggregate • Certain elastomer materials soften under asphalt with a risk of puncturing, but can be protected with a sand carpet	• Difficult to ensure consistent quality and thickness • Bituminous solutions are likely to blister during placement • Keg mastic are likely to blister during placement and soften under medium to high ambient temperature • Epoxy resin systems are likely to blister and embrittled with the application of hot asphaltic materials
Performance Issues	• Membrane leakage • Debonding due to water accumulation	

2.7.2 Concrete Slab

Attanayake and Aktan (2013) documented the initiation of reflective deck cracks even before completing the approach pavements of a new Michigan DOT bridge deck with full-depth grouted partial depth shear keys, transverse post-tensioning, and a 6 in. cast-in-place concrete slab. The causes of reflective crack initiation are not identified. However, considering the time of cracking and loading on the bridge, the observation by Attanayake and Aktan (2013) indicates that the reasons for cracking may be due to concrete shrinkage, temperature gradient, time dependent losses of box-beam prestress, or a combination thereof. Also, the reflective cracks were observed over the supports (abutments and piers). Finite element analysis results presented by Sharpe (2007) support the findings.

Sharpe (2007) describes the impact of the concrete shrinkage on reflective cracking and recommends reducing shrinkage effects. As discussed in the previous section and Appendix A, the use of expansive cement Type K and G with mineral admixtures can be used for reducing shrinkage. Lall et al. (1997) recommend a higher amount of steel reinforcement in the concrete slab to reduce the crack width by increasing the number of cracks. A prudent approach is to use non-shrink mixes to reduce concrete cracking potential.

2.8 BEARING LAYOUT

Lall et al. (1997) recommends full-width bearing pads to prevent off-axis tilting. Sang (2010) recommends placing bearing pads underneath the shear keys to prevent differential movement of the beams due to cracking or less stiff shear keys. PCI (2011) recommends providing a 3-point bearing system to minimize the rocking of girders.

The common practice is to use neoprene pads on reinforced concrete abutments and pier/bent caps to support box-beams. More recently, a Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) is introduced as a cost effective solution for small, single span bridges (FHWA 2011). According to FHWA (2011) and NYSDOT (2015), when the GRS-IBS concept is implemented to build adjacent box-beam bridges, beams are directly supported on a bearing bed prepared using reinforced soil (Figure 2-9). Even though FHWA (2011) suggests using a jointless pavement over the abutments, NYSDOT (2015) acknowledges the need of using an expansion joint to accommodate bridge movement due to thermal expansion and contraction. The Alberta Ministry of Transportation (Alberta MOT 2017) detail includes a precast concrete abutment seat and neoprene pads. Since cracking is imminent at the pavement-beam end interface, Alberta MOT (2017) detail includes a waterproofing membrane (Figure 2-10).

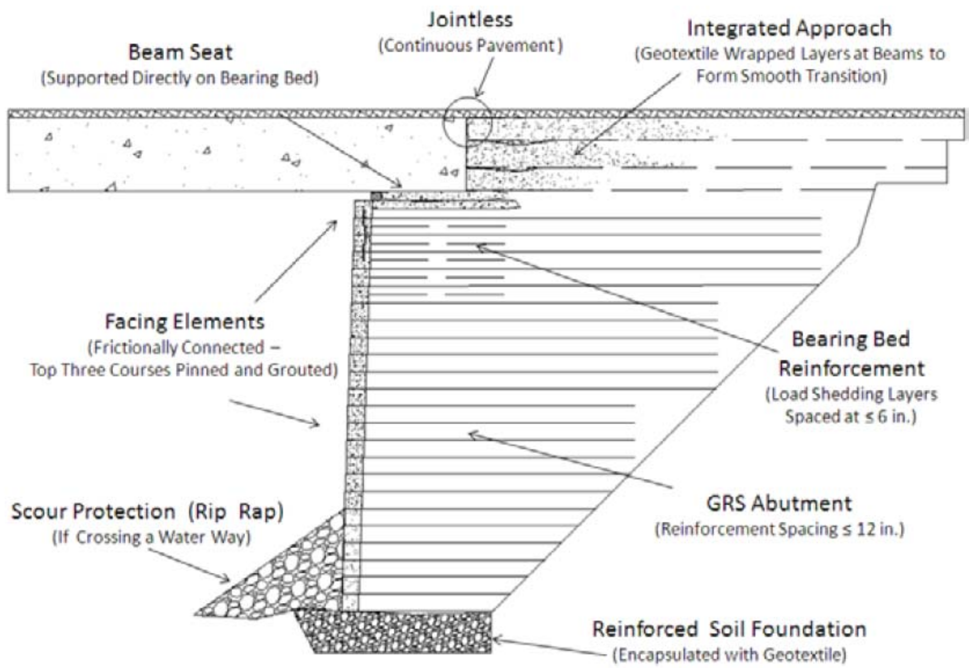


Figure 2-9. Typical GRS-IBS detail (FHWA 2011)

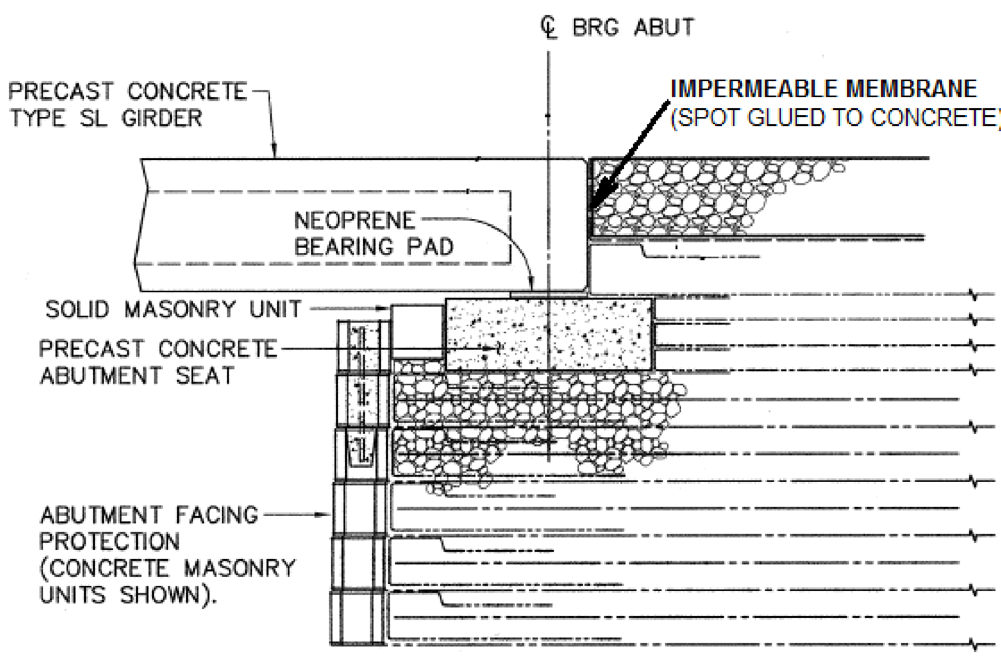


Figure 2-10. Alberta MOT GRS-IBS detail (Alberta MOT 2017)

2.9 DESIGN AND CONSTRUCTION RECOMMENDATIONS

Table 2-5 and Table 2-6 list the design and construction recommendations and concepts documented in the literature.

Table 2-5. Best Design Practices Recommended/Suggested in the Literature

NCHRP Synthesis 393	PCI State-of-the-Art Report	PCI Northeast
▪ Provide full-depth shear keys that can be grouted easily. ▪ Provide transverse post-tensioning so that tensile stresses do not occur across the joint. ▪ Provide a cast-in-place, reinforced concrete, composite deck with a specified concrete compressive strength of 4,000 psi and a minimum thickness of 5 in., to limit the potential for longitudinal deck cracking.	▪ Provide shear key geometries that allow deck concrete to fill the key, or use full depth shear keys. ▪ Utilize high performance or high strength, low permeability concrete in the beams and deck slab. ▪ Provide a minimum of 1.5 in. cover to all reinforcing. Use 2 in. where practical. ▪ Utilize strand patterns which omit the use of prestressing strands in the exterior corners. ▪ Design for composite action with a reinforced concrete deck slab (minimum thickness of 5 in.). ▪ Minimize skews where practical. ▪ Provide lateral restraint at piers and abutments. ▪ Consider 3-point bearing system to minimize rocking of girders. ▪ Utilize corrosion inhibitor in the concrete mix design for the beams. ▪ Provide waterproofing between top of structural member and overlay if a noncomposite overlay is to be used.	Provide full-depth shear keys.

Table 2-6. Best Construction Practices Recommended/Suggested in the Literature

NCHRP Synthesis 393	PCI State-of-the-Art Report	PCI Northeast
▪ Use stay-in-place expanded polystyrene to form the voids. ▪ Sandblast the longitudinal keyway surfaces of the box beams immediately before shipping to provide a better bonding surface for the grout. ▪ Clean the keyway surfaces with compressed air or water before erection of the beams to provide a better bonding surface for the grout. ▪ Grout the keyways before transversely post-tensioning to ensure compression in the grout. ▪ Use a grout with high bond strength to the box beam keyway surfaces to limit cracking. ▪ Provide proper curing for the grout to reduce shrinkage stresses and ensure proper strength development. ▪ Provide wet curing of the concrete deck for at least 7 days to reduce the potential for shrinkage cracking and to provide a durable surface.	▪ Utilize polystyrene material to form voids. ▪ Provide consistent casting conditions to minimize differential camber in beams. ▪ Properly anchor polystyrene forms to prevent floating of forms during casting. ▪ Provide vent holes for beam curing in addition to drainage holes in boxes. ▪ When extending stirrups for shear connection to slab, consider bent shape of bar in relation to placement of void forms. ▪ When extending mild reinforcing steel at the ends of beams, provide straight bars and bend after fabrication. ▪ Provide transverse post-tensioning to compress joints and minimize differential deflections between boxes. ▪ Sandblast shear keys prior to grouting or concreting. ▪ Utilize epoxy grout in keyways when using small shear keys. ▪ Post-tension transverse ties after grouting on square bridges. ▪ Post-tension transverse ties prior to grouting shear keys on skewed bridges. ▪ Grind concrete pier and abutment surfaces if necessary to achieve uniform bearing surface. ▪ Offset longitudinal deck joints a minimum of 1 ft from edge of adjacent box in staged construction. ▪ When differential camber occurs, force beams together or provide smooth transition with joint grout material.	**<u>Bridges with skew < 30°</u>** ▪ Layout working lines. ▪ Verify beam seat elevations and install bearing pads. If seats are high, grind to correct elevations and if seats are low, use shims as required. ▪ Erect beams. Install hardwood wedges between adjacent beams to maintain proper shear key joint opening. ▪ Install polyethylene closed cell backer rod as joint filler at shear key locations. Filler shall be placed below the bottom of the shear key joints. Filler shall be installed sufficiently tight to prevent loss of the shear key grout. ▪ Install transverse ties through ducts. Verify the hardwood wedges are in place. Post-tension transverse ties to approximately 5 kips to remove sag in the tie and to seat the chuck. ▪ Grout shear key. Ensure the structural integrity of the superstructure. Clean it with an oil free air-blast immediately prior to grout placement. Verify the backer rod is still in place. Rod carefully to eliminate voids. ▪ Shear key grout shall attain a minimum compressive strength of 1500 psi before post-tension application. ▪ Post-tension transverse ties to 30 kips beginning with inner most ties and proceeding symmetrically about mid-span towards the member ends. For box beams with top and bottom transverse ties, tension the bottom and top tie to 15 kips. Repeat the sequence once more so that each transverse tie has 30 kips of tension. ▪ Finish work. Remove wedges and patch the deck and fascia beams at transverse tie locations. Place overlay concrete.

2.9.1 Grouting Shear Keys

All grouting operations with cementitious materials require wetting the precast element surfaces to attain a saturated-surface-dry condition before placing the grout or special concrete. Generally, wetting of the component surfaces should start at least 4 hours before the grout placement. However, most grout material datasheets recommend a wetting process to start 24 hours before placement.

Surface preparation is important and is a critical factor for bonding grout to the precast elements. The surface should be clean from any foreign materials, and the joints should be roughened or mechanically abraded to allow forming a mechanical bond between the grout and the precast elements. Cementitious grout with non-shrink properties is often recommended in precast construction due to assumed material compatibility of the grout with precast elements. The material datasheet for magnesium phosphate grouts indicates the need for special surface preparation to enhance bonding at the grout – precast element interface. Once the surface is prepared, the magnesium phosphate grout will provide desired bonding properties as per the manufacturer datasheet.

Ambient vibration, propagating from traffic or other construction operations, is a factor that promotes grout cracking and failure at the grout-precast element interface bond. Grouts, mostly those requiring longer setting time, are sensitive to the structural vibration. The impact of vibration needs to be considered, for example, in the case of staged construction.

Grout placement methods include dry packing, gravity flow (pouring), and pumping (Figure 2-11, Figure 2-12, and Figure 2-13). Dry packing is commonly used for shear keys. Moreover, grout mixed at flowable and fluid consistency can be pumped into tight spaces. The pumping process requires a leak-proof formwork that can withstand the pumping pressure. Joints are commonly sealed with a foam backer rod, which is flexible and may not be sufficient for pressure grouting.

Figure 2-11. Grouting adjacent box-beam shear keys using type R-2 grout (Oakland Drive over I-94, MI)

Figure 2-12. Pumping W.R. Meadows Sealight CG-86 non-shrink grout (Source: Oliva et al. 2007)

Figure 2-13. Grouting of full-depth deck panel connections (Source: Courtesy of MDOT)

2.10 SUMMARY AND CONCLUSIONS

Recent studies and findings/recommendations related to box-beam bridge performance, box-beam geometry, shear key configuration, transverse post-tensioning, shear key grout material, deck overlay or concrete slab, bearing layout, and construction recommendations are summarized in this chapter. The following sections provide the conclusions derived from the information presented in the chapter.

2.10.1 Box-Beam Bridge Performance

Review of literature and Michigan experience show that reflective deck cracking develops irrespective of the changes implemented to abate cracking.

2.10.2 Box-Beam and Shear Key Geometry

The box-beam with rectangular voids is commonly used by the state DOTs. Compared to the sections used in other countries, the box-beam with rectangular voids has the least self-weight. Unless additional investigations are performed to justify the added benefits of using a different section, the use of box-beams with rectangular voids is preferred to minimize the impact on the fabrication process.

Box-beams with full-depth shear keys are used by several state DOTs as well in Japan, Korea, and China. The drawback of using a full-depth grouted partial-depth shear key was discussed. A full-depth shear key provides adequate confinement to prevent grout spall even when the shear key material is cracked. Further, it provides an adequate space for material placement and consolidation.

2.10.3 Transverse Post-Tensioning

Aktan et al. (2009) demonstrated that the shear keys located longitudinally in between the diaphragms are not compressed when post-tensioning is applied through the diaphragms, even under increased post-tensioning. Two-stage post-tensioning is suggested, before placement and after strength gain of the concrete deck slab. Ulku et al. (2010) demonstrated the effectiveness of applying two-stage post-tensioning to mitigate reflective cracking.

2.10.4 Shear Key Grout

Cementitious grout with non-shrink properties is often recommended in precast construction due to assumed material compatibility of the grout with precast elements. Beam surface preparation is important and critical for effective bonding between the grout and precast elements. The surface should be saturated-surface-dry for cementitious grouts, free from any foreign materials, and roughened or mechanically abraded. Following a good surface preparation, an adequately strong mechanical bond can be developed between grout and the beam surface. Use of commercial grouts requires special surface preparation to achieve the desired bonding properties. Hence, manufacturer recommendations need to be strictly followed.

Grout placement methods include dry packing, gravity flow (pouring), and pumping. Grout mixed at flowable and fluid consistency can be pumped into tight spaces to form shear keys. Flowable grout requires a leak-proof formwork.

2.10.5 Cast-in-Place Concrete Slab

The literature showed that the reflective deck cracking develops even before the bridge is opened to traffic. In addition, despite the use of full-depth grouted partial depth shear keys, transverse post-tensioning, and a 6 in. cast-in-place concrete slab, the reflective deck cracking persisted. These observations show that the cracking might be originating due to concrete shrinkage, temperature gradient load, time dependent losses of box-beam prestress, or a combination thereof. Use of non-shrink concrete with staged post-tensioning may decrease the cracking potential.

2.10.6 Flexible Overlay with Waterproofing Membrane

The literature presented that a flexible overlay deck system can be developed with either preformed sheets or liquid membranes. The system can provide a service life between 16 to 20 years. Both membrane systems present advantages, installation challenges, and performance issues. Its performance can be greatly influenced by the installation design, workmanship, and the composition of the membrane.

2.10.7 Bearing Layout

Different bearing configurations are proposed in the literature. Other than transferring load from superstructure to substructure, the prime objective of using a specific bearing configuration is to ensure box-beam stability. The literature findings are inconclusive but recommend using a bearing configuration that can prevent box-beam rocking during construction.

2.10.8 Design Recommendations

Based on the information presented in this chapter, the following design recommendations are presented:

- Specify full-depth shear keys.
- Specify two-stage transverse post-tensioning.
- Specify a cast-in-place reinforced concrete slab with a compressive strength of 4000 psi and a minimum thickness of 5 inches.
- Avoid skew where practical.
- Provide lateral restraint at piers and abutments.
- Provide a minimum of 1.5 in. cover to all reinforcing. Use 2 in. cover where practical.
- Utilize corrosion inhibitor in the concrete mix design for the beams.
- Utilize high performance or high strength, low permeable, non-shrink deck concrete.
- Use non-shrink grout material that possesses a high bond strength.

2.10.9 Construction Recommendations

Based on the information presented in this chapter, the following construction recommendations are presented.

2.10.9.1 General Recommendations

- Use expanded polystyrene blocks to form box-beam void.
- Properly anchor the foam blocks.
- Provide vent holes for beam curing in addition to drainage holes in boxes.
- When extending stirrups for shear connection to slab, consider a bent shape of bar in relation to placement of foam blocks.
- Verify beam seat elevations and install bearing pads. Grind concrete pier and abutment surfaces, if necessary. Consider a 3-point bearing system to minimize rocking of girders.
- Sandblast the longitudinal keyway surfaces immediately before shipping.

- Erect beams. When differential camber occurs, force beams to match the camber or provide smooth transitions with joint grout material.
- Install hardwood wedges between adjacent beams to maintain beam spacing.
- Install a polyethylene closed cell backer rod as the formwork at shear key locations.
- Install formwork around post-tensioning ducts to prevent shear key grout getting into the ducts.
- Install transverse ties through ducts. Verify that the hardwood wedges are in place. Post-tension transverse ties to approximately 5 kips to remove sag in the tie and to seat the chuck.
- Grout the keyways before transverse post-tensioning.
- Use a grout with high bond strength to form shear keys.
- Provide proper curing for the grout.
- Remove wedges.
- Post-tension transverse ties to attain the specified force at each post-tensioning location. When two or more post-tensioning ducts are located along the beam depth, apply half of the force specified at each location along the depth. Repeat the sequence along beam depth to attain the specified force at each location.
- Start posttensioning over the supports and sequentially proceed towards the midspan to prevent tensile stresses developing at the shear keys over the supports.
- Follow the steps given in Section 2.10.9.2 or Section 2.10.9.3 as applicable to complete construction.

2.10.9.2 Recommendations for a Deck with an Overlay

- Post-tension transverse ties to attain the specified force at each post-tensioning location by following the sequence presented in Section 2.10.9.1.
- Grout post-tensioning ducts.
- Place an overlay with a waterproofing on top of structural members. Offset longitudinal construction deck joint a minimum of 1 foot from the closest shear key.
- Fill the stress pockets with a non-shrink grout.

2.10.9.3 Recommendations for a Deck with a Cast-in-place Concrete Slab

- Apply the required first-stage post-tension by following the sequence presented in Section 2.10.9.1.
- Place a cast-in-place concrete slab with a single layer of reinforcement.
- Offset the longitudinal construction deck joint a minimum of 1 foot from the closest shear key.
- Provide wet curing of the concrete deck for at least 7 days.
- Provide consistent casting conditions.
- Apply the second stage post-tensioning to compress the box-beam, shear key, and cast-in-place concrete slab assembly. Follow a post-tensioning sequence similar to first stage post-tensioning.
- Grout post-tensioning ducts.
- Fill the stress pockets with a non-shrink grout.

3 STATE DEPARTMENT OF TRANSPORTATION PRACTICES

3.1 OBJECTIVE AND APPROACH

This chapter includes the documentation of department of transportation (DOT) practices related to box-beam geometry, shear key configuration, transverse post-tensioning, shear key grout material, deck overlay or concrete slab, bearing layout, and construction procedures. Two surveys were previously conducted for the NCHRP Synthesis 393 (Russell 2009) and the PCI State-of-the-Art Report (PCI 2011) (Table 3-1). During this project, the surveys cited above were reviewed, and the states with box-beam bridges were selected. Twenty one out of 35 states affirmed on the NCHRP Synthesis 393 survey that they have side-by-side box-beam bridges. Out of 29 states that affirmed, on the PCI survey, that they have box-beam bridges, only 27 states have side-by-side box-beams. After considering the states with side-by-side box-beam bridges and the exposure conditions in the State of Wisconsin, 17 states were selected for review (Table 3-1).

Table 3-1. States That Participated in Two Previous Surveys and the Ones Selected for Review

No	State	Survey Participation		Remarks	States Selected for Review
		PCI	NCHRP Synthesis 393		
1	CT	Yes	-		√
2	DE	Yes	Yes		√
3	IA	-	-		
4	IL	Yes	-		√
5	IN	Yes	-		√
6	KS	-	-		
7	KY	Yes	-		√
8	MA	Yes	Yes		√
9	MD	Yes	Yes	Solid slab boxes only	
10	ME	-	-		
11	MI	Yes	Yes		√
12	MN	-	-		
13	MO	Yes	Yes		√
14	ND	Yes	-	Spread box beam only	
15	NE	-	-		
16	NH	Yes	Yes		√
17	NJ	Yes	Yes		√
18	NY	Yes	Yes		√
19	OH	Yes	Yes		√
20	PA	Yes	Yes		√
21	RI	Yes	-		√
22	SD	-	-		
23	VA	-	Yes		√
24	VT	Yes	-		√
25	WI	-	Yes		√
26	WV	-	-		

- denotes either negative responses or no response

From the information available in the respective DOT websites, the data related to box-beam bridges is compiled. Appendix B presents the compiled information from all the selected DOTs except the Wisconsin DOT (WisDOT).

In order to compile WisDOT practice, the Wisconsin Highway Structures Information (HSI) system was used to acquire bridge plans, design calculations, and other documents. The HSI database was queried using the following options and yielded 168 bridges.

> STRC_TYPE=B; SpanMaterial=PRECAST CONC; SpanMaterial=PREST CONCRETE; and SpanConfiguration = BOX GIRDER and BOX SECTIONS.

In addition, there were six bridges constructed as continuous for live load. These bridges were identified using the following options:

> STRC_TYPE=B; SpanMaterial=CONT PREST CONC; and SpanConfiguration =BOX GIRDER.

From HSI, a total of 174 bridges were identified. Five bridges had a superstructure other than box-beams; 4 of them were widened using box-beams. These 5 bridges were not considered in the analysis. Also, one of the bridges had box-beams in only one span out of the seven spans. Since the bridge has one complete span with box-beams, it was considered in the analysis. The oldest box-beam bridge that is still in service in Wisconsin was built in 1954. Appendix C presents the compiled information from WisDOT.

In addition to the information acquired through respective DOT websites, a survey was conducted to learn designers', inspectors', and fabricators' experience related to the reflective deck cracking. This chapter presents a summary of compiled information from all 17 states and the survey.

3.2 DOT PRACTICES

3.2.1 Box-Beam Geometry

The geometries of box-beams used by the respective DOTs are shown in Figure 3-1.

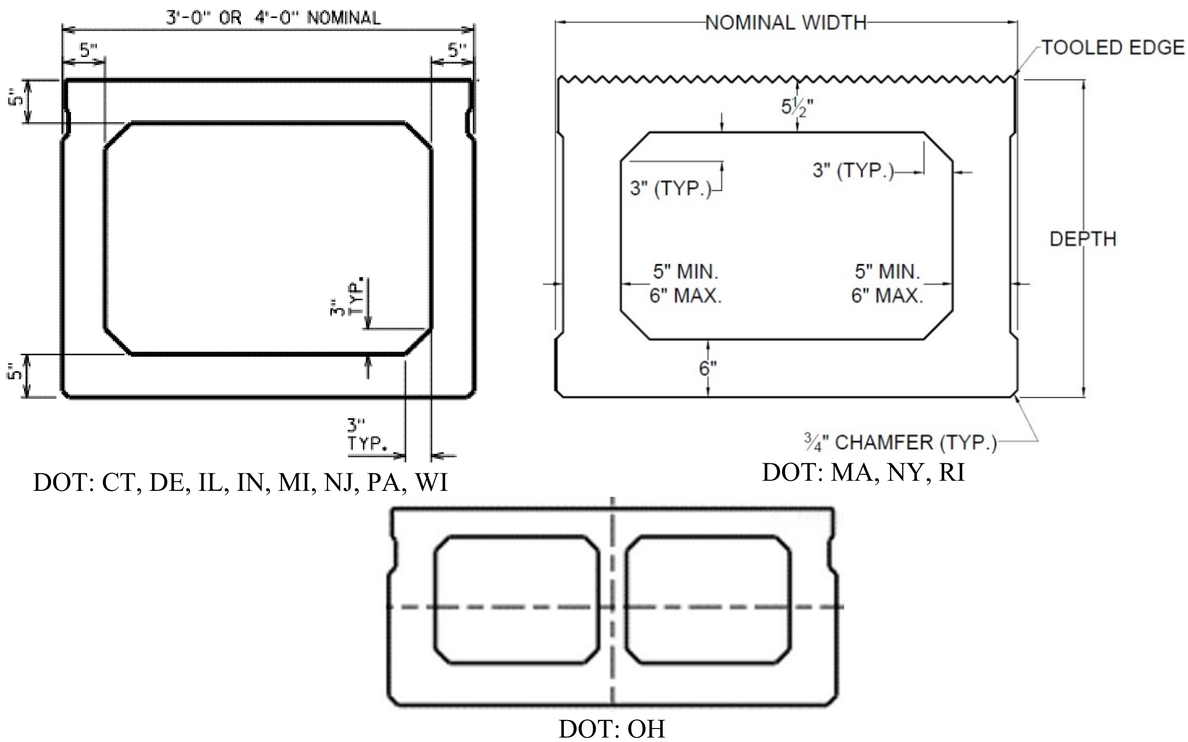
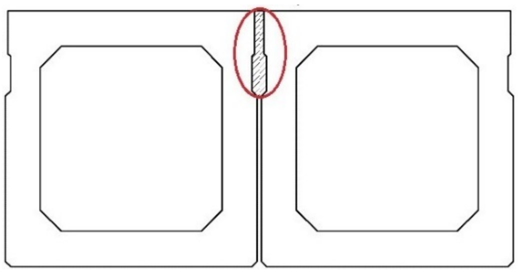


Figure 3-1. Typical box-beam geometries used by the State DOTs

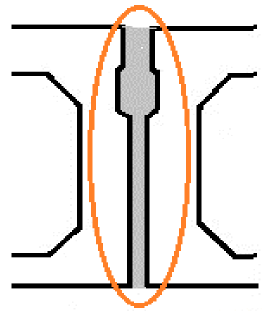
The most recent WisDOT standard details show rectangular sections up to 42 in. in depth. The section depth ranges from a 12 in. (solid section) to 42 in. with a rectangular void. Beam width ranges from 36 in. to 48 in. Typical exterior girders support the curb that varies in width and height. It appears that these variations depend on the width of the beam and the height of the overlay being placed upon the bridge. Use of closed stirrups is the current standard practice. Weep holes of 1 in. in diameter are provided at each end of a void to drain moisture. The number of weep holes at each end of a void ranges from 1 to 2 depending on the width of the section.

3.2.2 Shear Key Configuration

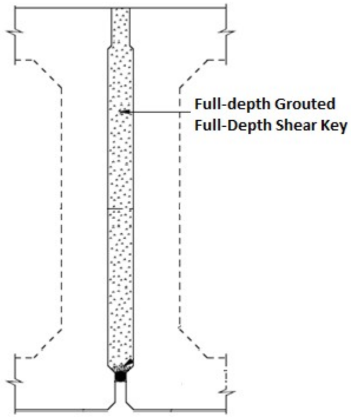
Partial depth, full-depth grouted partial depth, and full-depth shear keys are commonly used (Figure 3-2). Table 3-2 lists the DOTs and the shear key configurations used in the respective jurisdiction.



(a) Partial depth shear key



(b) Full-depth grouted partial depth shear key



(c) Full-depth shear key

Figure 3-2. Shear key configurations used by various state DOTs

Table 3-2. Shear Key Configurations Used by the State DOTs

State DOT	Shear Key Configuration			Remarks
	Partial depth	Full-depth grouted partial depth	Full-depth	
CT, DE	√			
IL, MI, OH, WI		√		
KY, NJ, PA				Partial depth shear key. Grout depth information could not be identified.
NY, MA, RI			√	
VT				Grouted shear key depth is 2/3 of the beam depth.

3.2.3 Transverse Post-Tensioning

Transverse post-tensioning details used by 17 state DOTs are given in Appendix B and C. A summary is provided in Table 3-3. The use of post-tensioning strands or tie-rods is the common practice. The number of post-tensioning duct locations along the span and depth of a girder as well as the tie force calculation methods vary significantly. As an example, Connecticut DOT has tables showing predetermined post-tensioning tie locations. Illinois DOT has a formula to calculate the number of transverse tie locations along the span. As per the formula, a 50 ft span requires only one tie location along the span. In Indiana, transverse post-tensioning is applied at 1/3 and 1/4 points along the span for bridges up to 40 ft and over 40 ft, respectively. Post-tensioning is not applied over the supports (abutments or piers). New Jersey calculates the post-tensioning force based on superstructure dead load. Michigan has tables showing post-tensioning duct locations along the depth and span as well as the force per diaphragm. The duct locations and tie forces were determined based on refined finite element analysis. Vermont provides a graph to calculate midspan post-tensioning force per foot for a given section. The post-tensioning force per duct is calculated by multiplying the force per foot and duct spacing.

As presented in Appendix B, skew policy on box-beam bridges varies significantly. As an example, Michigan allows placing transverse post-tensioning ducts parallel to skew up to 30^0. Whereas, NJ and KY allow placing transverse post-tensioning ducts parallel to skew up to 15^0 and 10^0, respectively.

Table 3-3. Post-Tensioning Force Used by the State DOTs

State DOT	Post-Tension Force	Tie Location
CT	30 kips/ location	At the ends (i.e. over the supports) and multiple locations along the span
DE	-	
IL	-	Uses a formula to calculate number of post-tensioning locations along a span. Transverse ties are not provided over the supports.
IN	20 ksi per rod.	1/3 locations for span ≤ 40 ft ¼ locations for span > 40 ft Ties are not provided over the supports.
KY	20 ksi per rod.	1 location for span ≤ 50 ft 2 locations for span > 50 ft Ties are not provided over the supports.
MA	-	Ends and midspan for span ≤ 50 ft Ends, ¼ locations, and midspan for span > 50 ft
MI	120 kips/diaphragm	4 location for span ≤ 50 ft 5 locations for span 50 ft < span ≤ 62 ft 6 locations for span 62 ft < span ≤ 100 ft 7 locations for span > 100 ft
MO	-	
NH	-	
NJ	Calculated based on superstructure dead load	Ties are provided
NY	-	Ends and midspan for span ≤ 50 ft Ends, ¼ locations, and midspan for span > 50 ft
OH	-	Midspan for span ≤ 50 ft 1/3 locations for span 50 ft < span ≤ 75 ft ¼ locations for span > 75 ft
PA	-	-
RI	44 kips at each duct	Ends and midspan for span ≤ 50 ft Ends, ¼ locations, and midspan for span > 50 ft
VA	-	Only slabs are used. Post-tensioning is placed at ends, ¼ locations, and midspan.
VT	A chart is provided. Force per tendon is calculated based on duct spacing.	-
WI	86.7 kips per duct	1 at center and 1 at each end for span ≤ 50 ft
		2 at center (12 ft apart) and 1 at each end for 50 ft < span ≤ 62 ft
		1 at center, 1 at each quarter point, and 1 at each end for span > 62 ft

3.2.4 Grout, Mortar, and Non-Shrink Material

Non-shrink grout and mortar are commonly specified. Yet, the requirements vary significantly. As an example, AASHTO LRFD (2013) Section 5.14.4.3.2 specifies using grout that can reach 5000 psi in 24 hours. The Michigan Department of Transportation uses R-2 mortar specified in Section 702 of the Standard Specifications for Construction. This mix is primarily developed with cement, water and fine aggregate. A total air content of 14% +/-4% is required in the mix. This mix can only develop about 3500 psi in 3 days (Aktan et al. 2009). The Delaware specification requires grout that can reach 5000 psi in 24 hours. The typical grout mix used in Wisconsin and placement requirements are presented in Table 3-4. The special provisions of a recent bridge detailed the mix and placement requirements shown in Table 3-5. NYSDOT allows commercial grouts and has an approved product list for grouting box-beam shear keys. Even though the Wisconsin DOT is not using commercial grouts to form the shear key, a few products that are listed in the NYSDOT list are also in the Wisconsin approved product list. These are 1107 Advantage Grout, NC Grout, and Sika Grout 212. Only Sika Grout 212 shows freeze/thaw resistance and expansion during hardening, making it a suitable product for forming shear keys in Wisconsin adjacent box-beam bridges. However, none of the technical data sheets provide bond strength with the substrate, a very important parameter for developing durable connections.

Table 3-4. Typical Shear Key Grout Mix and Placement Requirements

Cement (lbs/yd^3)	920 (Type I)
Sand (lbs/yd^3) @ SSD (Saturated Surface Dry) 2.65 specific gravity	2,350
Water	To reach approximately 5 in. slump or to a consistency to insure that the voids are completely filled
Air content	14% +/- 4% by using masonry cement or an air entraining admixture
Placement requirements	Grout shall be rodded to insure that the voids are completely filled.

Table 3-5. Shear Key Grout Mixes Given in a Recent Special Provisions

Mix 1	Type 1 portland cement (lbs/yd^3)	468
	Type 'N' masonry cement (lbs/yd^3)	349
	Fine aggregate (lbs/yd^3)	1,991
	Net water (approximate) (lbs/yd^3)	415
Mix 2	Type 1 portland cement (lbs/yd^3)	930
	Fine aggregate (lbs/yd^3)	1,966
	Net water (approximate) (lbs/yd^3)	415
	Air content	14% +/- 4% by using an air entraining admixture

3.2.4.1 NYSDOT Approved Commercial Grout for Shear Keys

Table 3-6 lists the shear key grout test requirements. The approved cement based grout materials for placement in *shear keys between prestressed concrete box-beams* and hollow slab units are listed in Table 3-7 and Table 3-8. The list was revised on May 2, 2016. The following grout properties are compiled from the manufacturer datasheets and presented in Table 3-7 and Table 3-8:

- Compressive strength at 1, 3, 7, 28 days,
- Initial setting time,
- Grout pocket dimensions,
- Working temperature range,
- Freeze/thaw resistance,
- Non-shrink properties (Change in height/volume as per ASTM C1090), and
- Possibility for extending grout.

The following is an excerpt of NYSDOT specification that details the process of getting an approval for a grout to be included in the department approved product list of *Cement Based Grout Materials for Shear Keys*.

> ***GENERAL.*** *The material must be flowable to fill the shear key with no voids. The Department will test the material in accordance with Test Method NY 701-12P, C following the manufacturer's proportioning and mixing instructions printed on the package. Material meeting the requirements of this specification will be placed on the Approved List. The Approved List titled: Shear Key Grout will state the precise water-grout ratio by weight. This ratio shall not be altered. For field use, follow the manufacturer's mixing and curing recommendations.*
>
> ***MATERIAL REQUIREMENTS.*** *The material shall be a prepackaged dry component: to which water or emulsified compound is added, used for concrete repair, containing no metallic expansion aides, to which no aggregate may be added. The material must meet the shear key pourability test as per Test Method NY 701-12P,C and the requirements of Table 701-06 (*Table 3-6*).*

Table 3-6. NYSDOT Shear Key Grout Test Requirements

TABLE 701-06 SHEAR KEY GROUT		
TEST REQUIREMENT	**Min.**	**Max.**
Initial Set (minutes)	120	-
Expansion (%)	0.02	1.0
Contraction (%)	-	0.0
7 Day Compressive Strength (psi)	6000	-
Freeze-Thaw Loss % (25 cycles)	-	1.0
Total Chloride Content (% by weight)	-	0.05
Total Sulfate Content (% by weight)	-	5.0

Table 3-7. NYSDOT Approved Grout List for Shear Keys (Part 1 of 2)

		1107 Advantage Grout	Certi-Vex Grout 1100 NY DOT	Conset Grout NY	Harris DOT Construction Grout	Kemset Grout	Mapei Planigrout 740	NC Grout
Compressive strength (ksi)	1 day	2.5	4.8	3.15	-	4.4	5.0	-
	3 days	5.0	6.2	-	4.5	5.8	6.0	4.0
	7 days	6.0	7.1	6.0	5.8	7.4	7.4	6.0
	28 days	8.0	8.2	7.48	8.0	8.3	9.4	7.5
Initial setting time (min)		30	200	180	-	42	420	150-210
Fill depth/thickness for neat grout (in.)	Min	-	-	-	1	-	0.5	1
	Max	3	12	2	2	2	2.375	2
Working temperature (ºF)	Min	45	50	40	40	40	41	40
	Max	90	80	90	85	90	95	-
Freeze/thaw resistant		-	YES	-	-	-	YES	-
Change in Height/Volume (%) (as per ASTM C1090 or CRD-C-621)	28 days	-	1.9	0.10	0.03	-	0.15	-
Extend with aggregate		YES	YES	YES	YES	YES	YES	YES

Table 3-8. NYSDOT Approved Grout List for Shear Keys (Part 2 of 2)

		Pro Grout 100	ProSpec C1107 Construction Grout	ProSpec High Strength Precision Grout	SC Multi-Purpose Grout	SikaGrout 212	Symons NY DOT, MultiPurpose Construction Grout+	SureGrout 106
Compressive strength (ksi)	1 day	-	5.5	5.5	3.1	3.5	5.2	2.4
	3 days	5.2	-	7.0	5.0	-	-	-
	7 days	9.2	7.3	8.5	6.9	5.7	6.5	6.4
	28 days	10.0	8.5	10.0	8.4	6.2	8.1	7.6
Initial setting time (min)		-	300 – 420	360	-	300	15-20	240
Fill depth/thickness for neat grout (in.)	Min	-	1	1	-	0.5	-	-
	Max	-	4	4	3	2	2	-
Working temperature (°F)	Min	50	40	40	50	40	40	-
	Max	80	90	90	80	-	-	-
Freeze/thaw resistant		YES	-	YES	YES	YES	YES	YES
Change in Height/Volume (%) (as per ASTM C1090 or CRD-C-621)	28 days	-	-	0.03	0.03	0.06	0.04	0.68
Extend with aggregate		-	YES	YES	YES	YES	YES	-

+ Only three grout types are approved by the NYSDOT from this supplier. They are 1107 Advantage, HD 50, and Pave Patch 3000. The 1107 Advantage is already included in the table. The description of the HD 50 states that it is for vertical applications and bridge decks. Hence, the date from HD 50 specification is included in the table for the Symons NY DOT MultiPurpose Construction Grout.

3.2.5 Deck Overlay or Concrete Slab

The most common practice is to use an overlay or a reinforced concrete cast-in-place slab over the box-beams (Table 3-9).

Table 3-9. Overlay or Concrete Slab Used by the State DOTs

State DOT	Overlay or Concrete Slab
IN	1 in. monolithic concrete 3 in. asphalt concrete- only for non-composite prestressed box beams 6 in. reinforced concrete cast-in-place slab
MI	6 in. reinforced concrete cast-in-place slab Hot mix asphalt where the ADT is less than 500 and/or commercial traffic is less than 3% of ADT
NH	3 in. concrete wearing surface
NJ	5 in reinforced concrete cast-in-place slab
OH	6 in reinforced concrete cast-in-place slab Asphalt wearing of 8 inches maximum for non-composite
PA	5 ½ in reinforced concrete cast-in-place slab A bituminous wearing course of 2 ½ inches minimum and 6 inches maximum for non-composite
RI	A minimum of 5 in. thick reinforced concrete cast-in-place slab A bituminous wearing surface with a waterproof membrane

According to section 19.3.9.2 of Wisconsin Bridge Manual, three types of overlays can be used as the wearing surface of box-beam bridges. They are as follows:

- Concrete overlay-Grade E or C,
- Asphaltic overlay with waterproofing membrane,
- Modified mix asphalt.

Only the STH 76 over the Embarrass River (B440198) Bridge consists of a 5 in. thick cast-in-place concrete slab with a single mat of reinforcement. This bridge has been built following Michigan practice to evaluate the potential for mitigating reflective cracking with a 5 in. thick slab and post-tensioning.

3.2.6 Bearing Layout

Different bearing configurations are used by the state DOTs to prevent box-beam rocking during construction (Figure 3-3).

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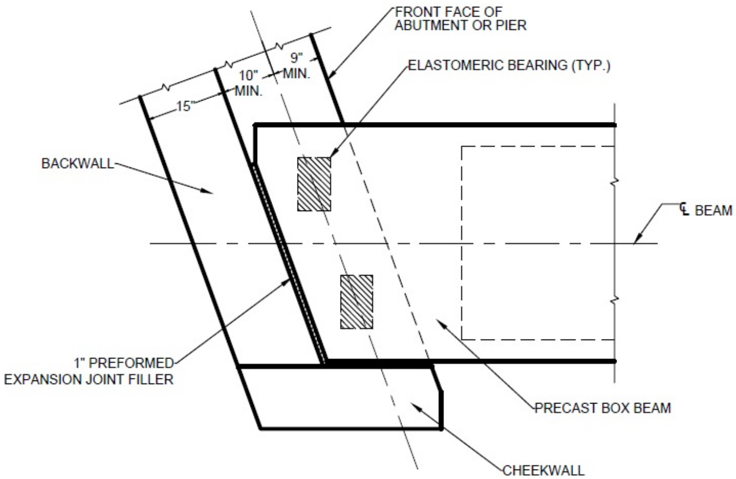
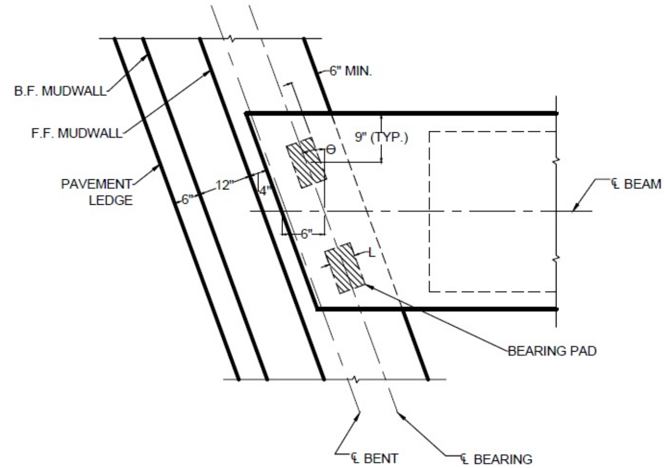
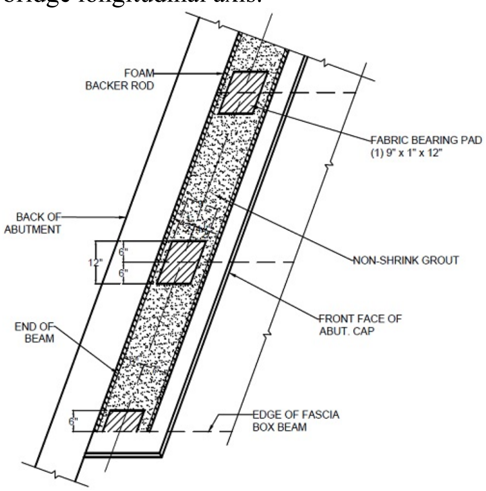


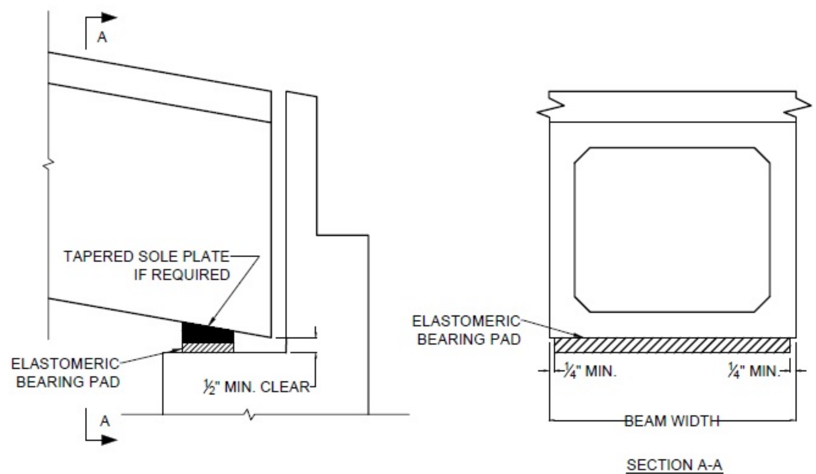
State: CT, KY
Note: Two bearing pads at each end. Bearing axis is perpendicular to the bridge longitudinal axis.



State: IN, MA, OH, PA, VA, VT
Note: Two bearing pads at each end. Bearing axis is parallel to the longitudinal axis of the substructure.



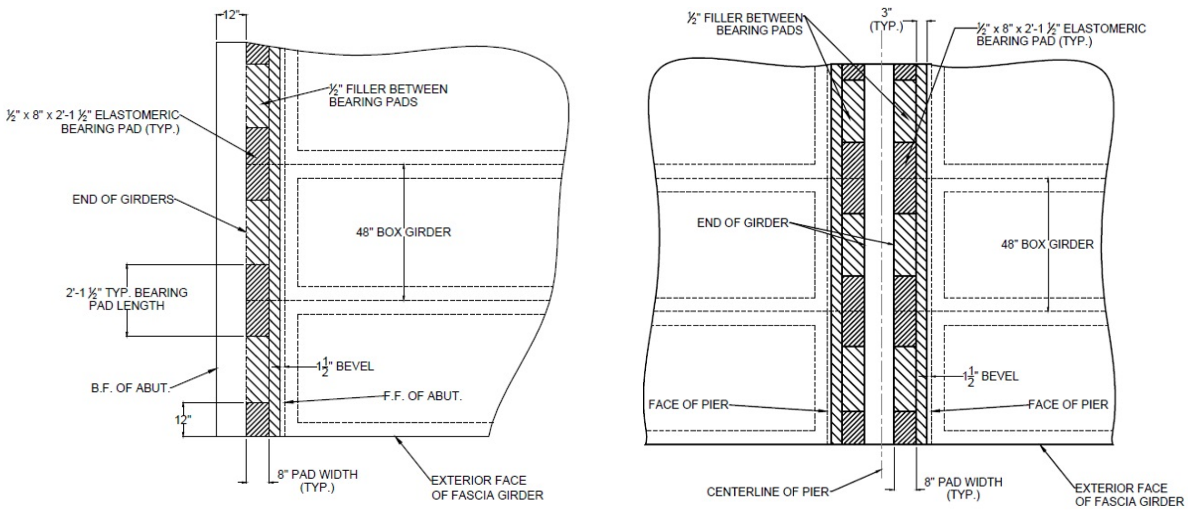
State: IL
Note: 1 in. fabric pad across two beams (*This provides two bearings underneath each beam end*).



State: DE, MI, NJ, NY, PA, RI
Note: A full-width bearing pad at each end.

Figure 3-3. Bearing configurations used by the state DOTs

The standard practice in Wisconsin was to have the bearing pads located at the corners of the girder, with adjacent girders sharing a bearing pad (Figure 3-4). The bearing pads were specified to be ½ in. in thickness. A filler material was placed in between the bearings. The current standard detail is to place 12 in. wide full-width bearing pads over the pier and 8 in. wide full width pads over the abutments (Figure 3-5). It appears that these are the only two bearing pad configurations that have been used.



(a) Over an abutment (b) Over a pier

Figure 3-4. Bearing pad plan

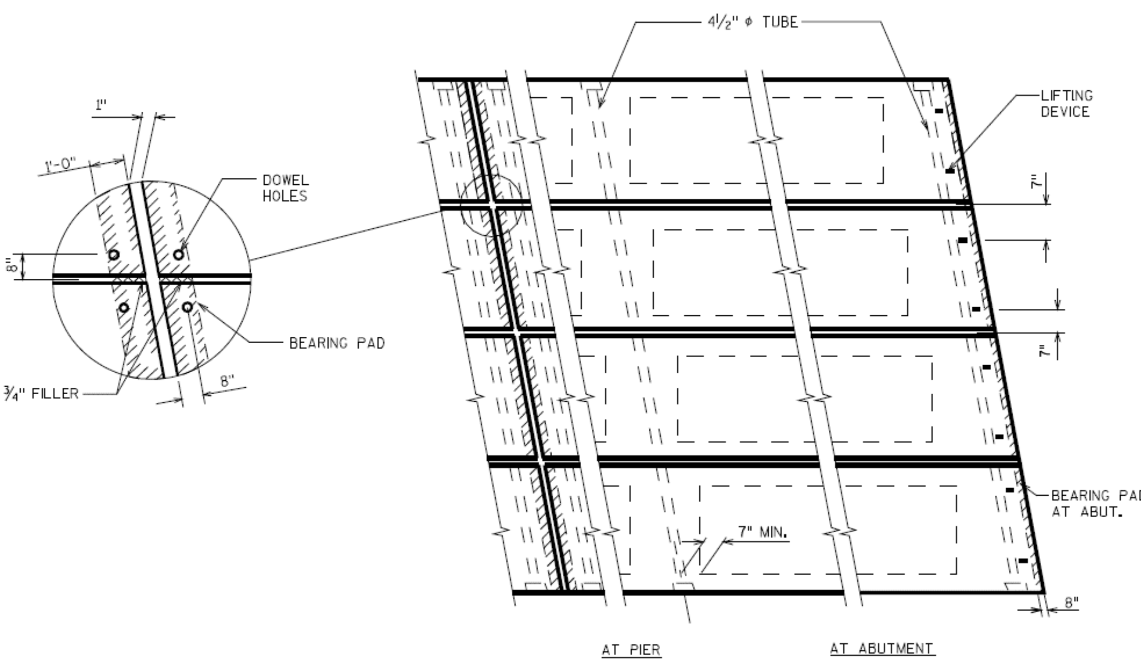


Figure 3-5. The bearing layout given with the most recent standard details

3.2.7 Construction Specifications

The construction procedures specified by each state vary significantly. A few states have specific guidelines for constructing bridges with a skew of $\leq 30^0$ and $> 30^0$ while many DOTs limit bridge skew to 20 or 30 degrees. Additional details are provided in Appendix B. This section provides a few highlights from the specifications related to box-beam bridges with a skew of $\leq 30^0$.

- Use expanded polystyrene blocks to form box-beam voids.
- Properly anchor the foam blocks.
- Provide drains or weep holes at the lowest point in each box-beam void in the finished structure. Puncture weep holes immediately after removal from casting bed. *Note: Some states provide a hole at each end of a void, or two holes at each end if the beam is wide.*
- Rough finish box-beam top surfaces to provide a ¼ inch surface texture unless otherwise required.
- Verify beam seat elevations and install bearing pads. Shim beam bearing pads to minimize the rocking of girders. The use of a tapered sole plate or a tapered grout pad may be required so that the bearing surfaces are set level in the longitudinal direction.
- Drill position dowel holes into bridge seats through holes provided in each beam end. Insert dowels. At the expansion bearings, fill position dowel holes with hot-poured rubber-asphalt type filler to at least 3 inches above the position dowels. Fill the remainder of the hole with a specified grout. Fill holes at fixed bearings with a specified grout.
- Install hardwood wedges between adjacent beams to maintain beam spacing.
- Seal the bottom of the longitudinal shear keys with a closed cell polyethylene foam backer rod. Also, provide gaskets at the post-tensioning duct locations to prevent shear key grout from flowing into the ducts.
- Install transverse ties.
- Tension each transverse tie to 5 kips. *Note: Some states do not require applying this initial tension.*
- Pre-wet all shear key surfaces for a minimum of 24 hours if the surfaces are not protected using penetrating sealants. *Note: When commercial grout is used, the manufacturer recommendations for surface preparation need to be followed.*

- Clean shear key surfaces with high-pressure water. *Note: Some states recommend using oil free air-blast for cleaning. When commercial grout is used, the manufacturer recommendations for surface preparation must be followed.*
- Place non-shrink grout in the longitudinal keyways to form shear keys and rod or vibrate grout to form a solid, tight shear key. *Note: Some states use cement, sand, and water mix with chemical admixtures while a few others use epoxy and other commercial grouts in shear keys. When commercial grout is used, the manufacturer's instructions are strictly to be followed. Also, exposure conditions at the time of grouting shear keys need to be considered.*
- Cure grout for 48 hours. *Note: Some states require a minimum of 24 hours of grout curing before post-tension application. Expected exposure conditions need to be considered when planning grouting and curing operations.*
- Apply post-tensioning after grout has attained the minimum required strength.

  On shallow members with one row of ties, tension each transverse tie to provide the required force.

  On deep members with multiple rows of ties, tension each transverse tie to provide half of the required force. Then, repeat this tensioning sequence once more so that each tie is tensioned to provide the total required force.

  Begin with the tendons at each end, and then work symmetrically towards the midspan from each end.
- Use galvanized end anchorage hardware.
- Grout post-tensioning ducts.
- Grout stress pockets.
- Place reinforced concrete deck slab with adequate cover to protect steel.

3.3 SURVEY OF TRANSPORTATION AGENCIES AND FABRICATORS

3.3.1 Overview

An online survey was developed and administered using QuestionPro website. The survey was sent to the 17 states stated previously, the region offices in Wisconsin, and a few Precast/Prestressed Concrete Institute (PCI) regional managers. Only 10 out of 17 states responded to the survey (Table 3-10). Since the survey was sent to a selected number of PCI regional managers, Montana, West Virginia, and Wyoming also participated. Therefore, all together 13 states participated in the survey.

Table 3-10. States Selected and Participated in the Survey

No	States Selected for the Survey	States Participated in the Survey
1	CT	-
2	DE	Yes
3	IL	Yes
4	IN	-
5	KY	-
6	MA	Yes
7	MI	Yes
8	MO	Yes
9	NH	Yes
10	NJ	Yes
11	NY	-
12	OH	Yes
13	PA	-
14	RI	Yes
15	VA	-
16	VT	-
17	WI	Yes

3.3.2 Survey Results

The survey was attempted by 24 participants. Only 13 completed it while 11 participants provided answers to a limited number of questions in the survey. The participants represented design, inspection, fabrication, and some other roles (Table 3-11). The participants were primarily from the State Departments of Transportation (86.36%) and the precast industry (13.64%).

Table 3-11. Role of the Participant and Participation Percentage

Role of the Participant	Percentage with Respect to the Number of Participants, %
Design	63.64
Inspection	18.18
Construction	0
Fabrication	13.64
Other	4.55

3.3.2.1 Wearing Surface

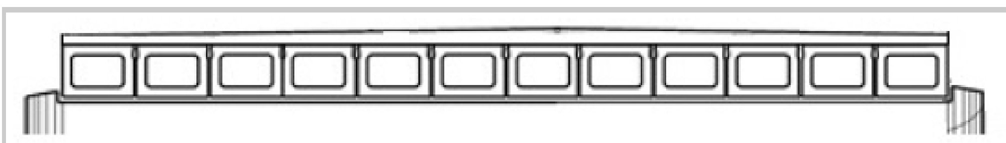
Various wearing surface types have been used on box-beam bridges (Table 3-12). The most common type is the asphalt wearing surface with a waterproofing membrane. Irrespective of the wearing surface type, reflective cracking has been documented. However, none of the respondents has used a cast-in-place slab with a single layer of reinforcing steel and non-shrink concrete.

Table 3-12. Wearing Surface Type, Implementation, and Observation of Longitudinal Reflective Cracking

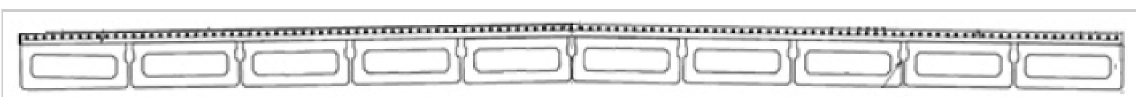
Wearing Surface Type	Response, %	Longitudinal reflective Cracking, %	
		Yes	No
Box-beams with an asphalt wearing surface	15.38	100.00	
Box-beams with a waterproofing membrane and an asphalt wearing surface	34.62	77.78	22.22
Box-beams with a concrete wearing surface	23.08	85.71	14.29
Box-beams with a cast-in-place slab with a single layer of reinforcing steel	23.08	60.00	40.00
Box-beams with a cast-in-place slab with a single layer of reinforcing steel and non-shrink concrete	0.00	-	-
Box-beams without a wearing surface	3.85	50.00	50.00

3.3.2.2 Roadway Crown

As shown in Figure 3-6, a roadway crown is formed primarily using two approaches. The most common practice is to change the girder elevations and place a uniformly thick wearing surface or a concrete slab (Table 3-13). However, the superstructures built with both approaches show reflective cracking (Table 3-13).



(a) Girders are placed at the same elevation, and the wearing surface thickness is varied.



(b) Girder elevations are changed, and a constant wearing surface thickness is maintained.

Figure 3-6. The primary approaches for maintaining roadway crown

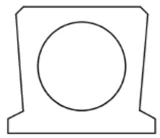
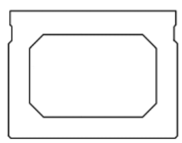
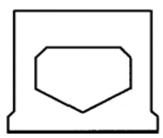
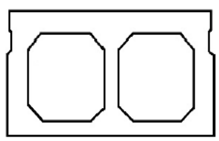
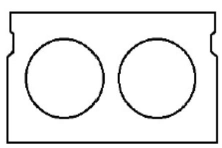
Table 3-13. Superstructure Cross-Section and Longitudinal Reflective Cracking Potential

An approach for maintaining roadway crown	Response, %	Longitudinal reflective Cracking, %	
		Yes	No
Girders are placed at the same elevation and the wearing surface thickness is varied.	16.67	66.67	33.33
Girder elevations are changed and a constant wearing surface thickness is maintained.	83.33	80.00	20.00

3.3.2.3 Box-Beam Geometry

The most commonly used section has a single rectangular cavity (section 2 in Table 3-14). The preference for section 2 and section 3 for future implementation is 60% and 40%, respectively. The void of the beam is formed with Styrofoam. There is a tendency to float the foam during fabrications that alters the beam cross-section. One of the challenges noted in the survey is resizing the Styrofoam to fit the space available during fabrication. Box-beam fabrication is a challenge when non-standard shear keys are used. The survey responses were not consistent on the impact of using Styrofoam on durability and moisture accumulation within the void. However, as a common practice, drains are provided at each end of the void.

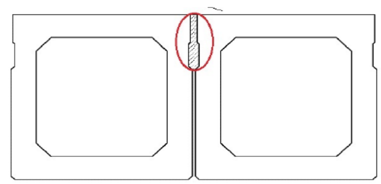
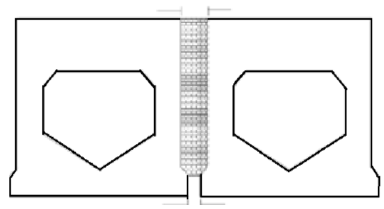
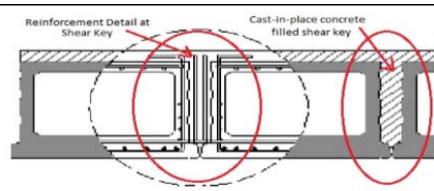
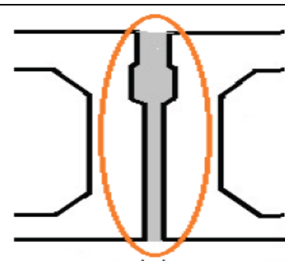
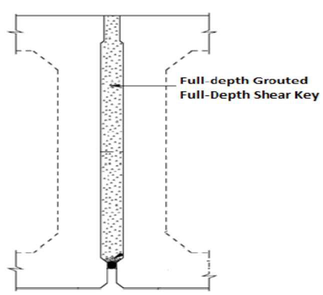
Table 3-14. The Box-Beam Cross-section, Current Implementation, and Preference for Future Adoption

Box-Beam Section		Implementation within US, %	Preference for Future Adoption, %
Section 1		-	-
Section 2		73.33	60.00
Section 3		-	40.00
Section 4		13.33	-
Section 5		13.33	-

3.3.2.4 Shear Key

Five shear key configurations were presented in the survey (Table 3-15). The most commonly used is the partial-depth shear key. The full-depth grouted, partial-depth shear key has the highest preference for future implementation. The other preferred types are full-depth (New England) shear key and full-depth concrete shear key. The maximum space between box-beams at the shear key bottom ranges from ½ in. to 1½ in. The minimum space between box-beams at the shear key bottom ranges from 0 in. to 1 ½ in. This indicates that, in certain cases, box-beams are placed next to each other, and post-tensioning force is allowed to transfer across the sections without compressing shear keys. The flexible foam formwork is used to hold shear key grout. However, the formwork is left in place without removing it. There is a potential for moisture accumulation within shear keys when the formwork is left in place. Only a very few states, such as Michigan, use wooden formwork which is removed at the end of construction. A few other states use wooden formwork only at the staged construction line; then it is removed at the end of construction.

Table 3-15. Shear Key Configuration, Current Implementation, and Preference for Future Adoption

Shear Key Configuration		Implementation within US, %	Preference for Future Adoption, %
Partial-depth shear key		50.0	-
Full-depth concrete shear key		-	25.0
Full-depth reinforced concrete shear key		-	-
Full-depth grouted partial-depth shear key		12.5	50.0
Full-depth shear key		37.5	25.0

3.3.2.5 Shear Key Material

Proprietary material is commonly used to form the shear key (Table 3-16). Pourable grout is the most preferred (Table 3-17). None of the participants uses beam-shear key interface bond enhancing material.

Table 3-16. Shear Key Material and Current Implementation

Shear Key Material	Implementation within US, %	Remarks
Cement mortar (water, cement, fine aggregate)	25.00	
Cement mortar with adequate air content	18.75	
Cement mortar with adequate air content and shrinkage reducing admixtures	25.00	
Concrete	-	
Concrete with adequate air content and shrinkage reducing admixtures	-	
Proprietary material or other, please list	31.25	Epoxy grout; Dayton Superior; Non-shrink grout with minimum compressive strength of 6000 psi.

Table 3-17. Shear Key Material Workability Requirements and Current Implementation

Workability Requirement	Implementation within US, %
Slump < 4 in.	12.5
4 in. < Slump < 6 in.	12.5
Pourable	75.0

3.3.2.6 Transverse Post-Tensioning

The most common practice is to use prestressing tendons. A rational process is not followed for calculation of the number of post-tensioning locations and the force. None of the participants has tried applying post-tensioning in two stages: before placing and after hardening the cast-in-place concrete slab. One of the challenges for applying two-stage post-tensioning might be the duct misalignment. Also, no respondents have tried applying post-tensioning through the top and bottom flanges of the box-beam instead of applying through the rigid diaphragms. The challenges associated with providing post-tensioning through top and bottom flanges include conflict with reinforcement and prestressing steel, along with the weight of the beam if flanges are thickened to accommodate post-tensioning ducts.

3.3.2.7 Suggestions to Eliminate/Minimize Reflective Cracking

Survey participants presented the following suggestions to minimize reflective cracking:

- Use transverse post-tensioning.
- Apply a stress of about 90 psi on gross cross sectional area.
- Use a stronger grout material.
- Modify bearing details to prevent rocking under loads.

4 IN-SERVICE BRIDGE PERFORMANCE

4.1 OBJECTIVE AND APPROACH

One of the objectives of the work covered in this chapter is to document the adjacent box-beam bridge performance in the State of Wisconsin. The inspection scope was limited to longitudinal cracking and its influence on bridge superstructure performance. The inspection was limited to 11 bridges that were built on or after 1983. The bridge performance data was collected by visual inspection.

The second objective is to evaluate the impact of cast-in-place concrete slab or overlay on bridge superstrucure durability performance. Considering similar exposure conditions, 17 states were selected. Even though the best approach for this type of study is to review the inspection forms, it is not practical due to a large volume of documents and challanges with accessing inspection forms from several highway agencies. Hence, the bridge condition rating available in the National Bridge Inventory (NBI) database was used.

4.2 WISCONSIN BRIDGE PERFORMANCE

4.2.1 Selection of Bridges

The Wisconsin Highway Structures Information (HSI) system was used to acquire bridge inventory and condition data. As discussed in Chapter 3, 169 bridges were identified. The oldest bridge that is still in service was built in 1954. Until 1982, use of tie-rods was the practice for transverse connection. In 1983, transverse post-tensioning strands were introduced with a force of 86.7 kips/duct. Hence, inventory data was grouped as pre-1983 and post-1983. As shown in Figure 4-1, 116 bridges were built with transverse post-tensioning. The majority of these bridges are on roads with low traffic. As shown in Table 4-1, 107 bridges are on roads with average daily traffic (ADT) of less than 1,000. Only 9 bridges carry more than 10,000 vehicles a day.

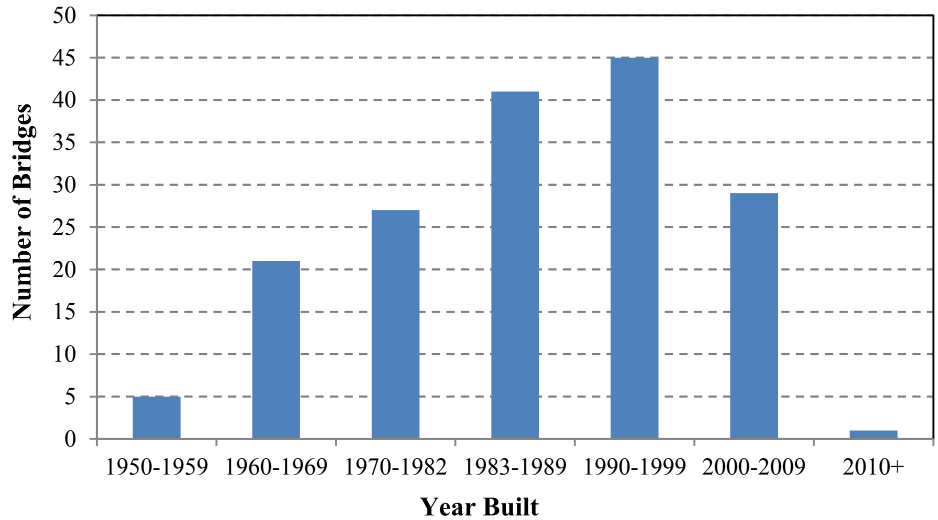


Figure 4-1. Number of bridges vs. year built

Table 4-1. Number of Bridges vs. ADT

Number of Bridges	Average Daily Traffic (ADT)
107	ADT ≤ 999
53	999 < ADT ≤ 9,999
9	ADT ≥ 10,000

The NBI rating of deck and superstructure was reviewed and summarized in Table 4-2. The impact of ADT on bridge deterioration is highlighted with the deck and superstructure condition data shown in Table 4-2. As an example, 15% of the bridges built during 2000 and 2009 (and with 1000≤ ADT≤9999) have a deck rating of either 4 or 5, while none of the bridges built during that era with ADT ≤ 999 has such deck ratings. The NBI rating, with respect to ADT, is shown in Figure 4-2 and Figure 4-3.

The average daily truck traffic (ADTT) volumes were also considered. Out of 169 bridges, 116 bridges appear not to have truck traffic (i.e., 69%) (Table 4-3). The database does not contain ADTT data for 7 bridges. Hence, only 46 bridges carry measurable truck traffic. Moreover, bridge performance with respect to the ADTT was evaluated. Unfortunately, due to a limited number of bridges in each ADTT category, the impact of ADTT on bridge deterioration is inconclusive. The data is graphically presented in Figure 4-4 and Figure 4-5. Additionally, as presented in literature, traffic can be a cause for crack propagation rather than initiation. Hence, the bridges carrying a low volume of traffic are considered for inspection to document longitudinal cracks.

Table 4-2. Percent of Bridges in Each NBI Rating and ADT Categories

			Percent of Bridges in Each NBI Rating Category					
			NBI Deck Rating			NBI Superstructure Rating		
Year Built	ADT	Number of Vehicles	4 - 5	6 - 7	8 - 9	4 - 5	6 - 7	8 - 9
1950-1959	ADT ≤ 999	3	0	100	0	67	33	0
	1000≤ ADT≤9999	2	50	50	0	50	50	0
	ADT ≥ 10000	-*	-	-	-	-	-	-
1960-1969	ADT ≤ 999	10	30	60	10	40	50	10
	1000≤ ADT≤9999	10	20	50	30	30	50	20
	ADT ≥ 10000	1	100	-	-	0	0	100
1970-1982	ADT ≤ 999	18	22	56	22	17	33	50
	1000≤ ADT≤9999	7	29	57	14	43	29	29
	ADT ≥ 10000	2	0	100	0	0	100	0
1983-1989	ADT ≤ 999	30	10	70	20	13	63	23
	1000≤ ADT≤9999	10	30	60	10	0	80	20
	ADT ≥ 10000	1	0	100	0	0	100	0
1990-1999	ADT ≤ 999	34	0	50	50	0	24	76
	1000≤ ADT≤9999	10	20	50	30	0	60	40
	ADT ≥ 10000	1	0	100	0	0	100	0
2000-2009	ADT ≤ 999	12	0	8	92	0	8	92
	1000≤ ADT≤9999	13	15	62	23	0	23	77
	ADT ≥ 10000	4	0	50	50	0	25	75
2010+	ADT ≤ 999	-	-	-	-	-	-	-
	1000≤ ADT≤9999	1	0	0	100	0	0	100
	ADT ≥ 10000	-	-	-	-	-	-	-

* '-' No bridges are in that ADT category

The deck and superstructure rating is also presented in Figure 4-2 and Figure 4-3.

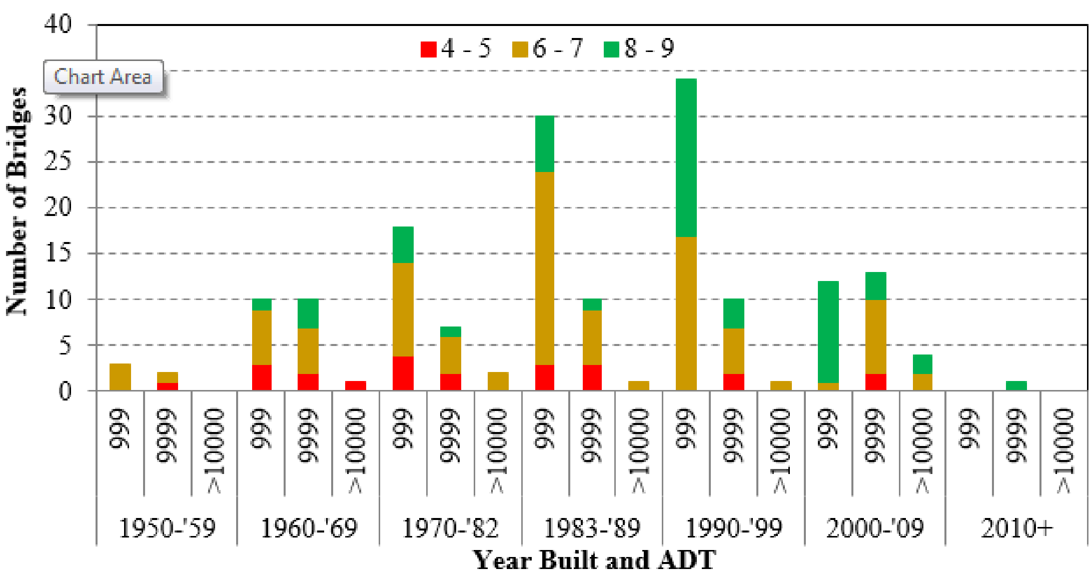


Figure 4-2. NBI deck rating: number of bridges vs. year built and ADT

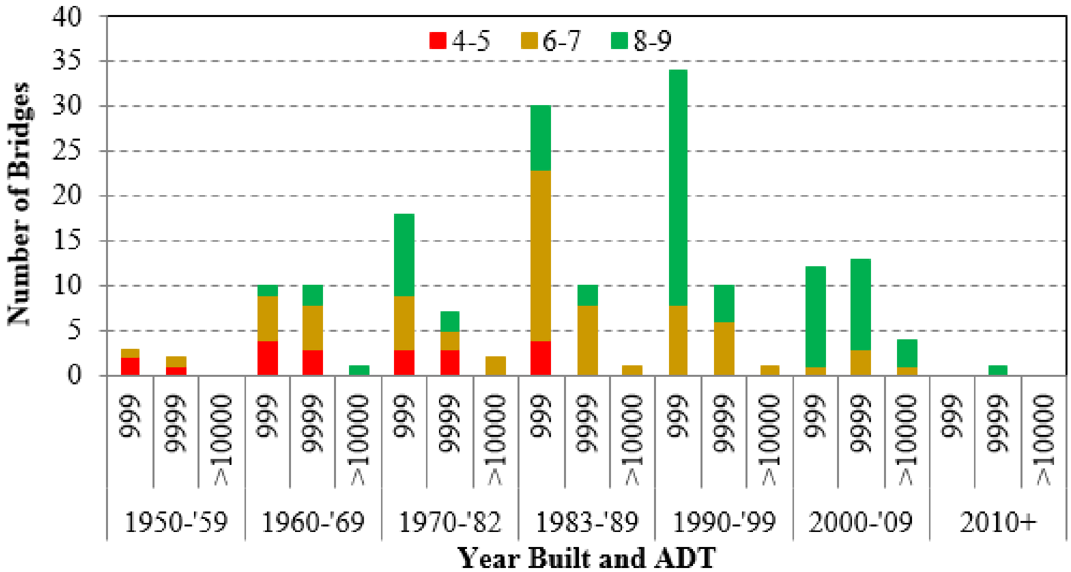


Figure 4-3. NBI superstructure rating: number of bridges vs. year built and ADT

Table 4-3. Percent of Bridges in Each NBI Rating and ADTT Categories

			Percent of Bridges in Each NBI Rating Category					
			NBI Deck Rating			NBI Superstructure Rating		
Year Built	ADT	Number of Bridges	4 - 5	6 - 7	8 - 9	4 - 5	6 - 7	8 - 9
1950-1959	ADTT = 0	5	20	80	0	60	40	0
	ADTT ≤ 100	-*	-	-	-	-	-	-
	ADTT > 100	-	-	-	-	-	-	-
1960-1969	ADTT = 0	17	35	41	24	35	41	24
	ADTT ≤ 100	1	0	100	0	100	0	0
	ADTT > 100	3	0	100	0	0	100	0
1970-1982	ADTT = 0	24	21	62	17	21	33	46
	ADTT ≤ 100	-	-	-	-	-	-	-
	ADTT > 100	3	33	33	33	33	67	0
1983-1989	ADTT = 0	35	14	66	20	8	66	26
	ADTT ≤ 100	-	-	-	-	-	-	-
	ADTT > 100	6	17	83	0	17	83	0
1990-1999	ADTT = 0	33	6	55	39	0	30	70
	ADTT ≤ 100	9	0	33	67	0	22	78
	ADTT > 100	3	0	67	33	0	100	0
2000-2009	ADTT = 0	3	67	0	33	0	33	67
	ADTT ≤ 100	12	0	33	67	0	17	83
	ADTT > 100	8	0	63	37	0	25	75
2010+	ADTT = 0	-	-	-	-	-	-	-
	ADTT ≤ 100	1	0	0	100	0	0	100
	ADTT > 100	-	-	-	-	-	-	-

* '-' No bridges are in that ADTT category

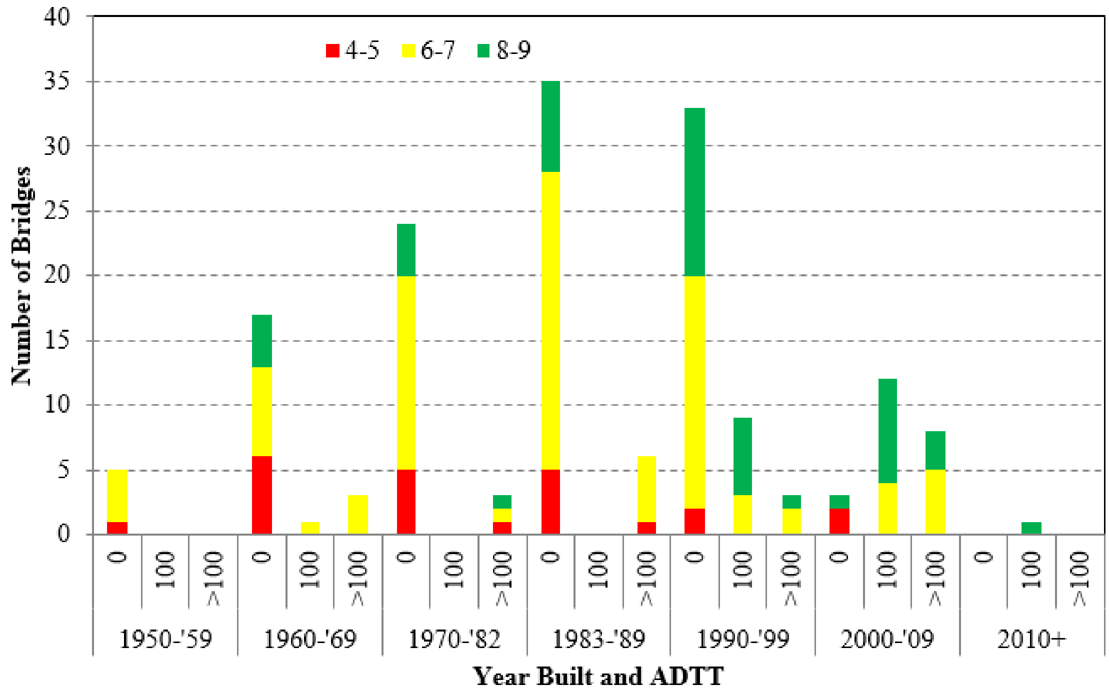


Figure 4-4. NBI deck rating: number of bridges vs. year built and ADTT

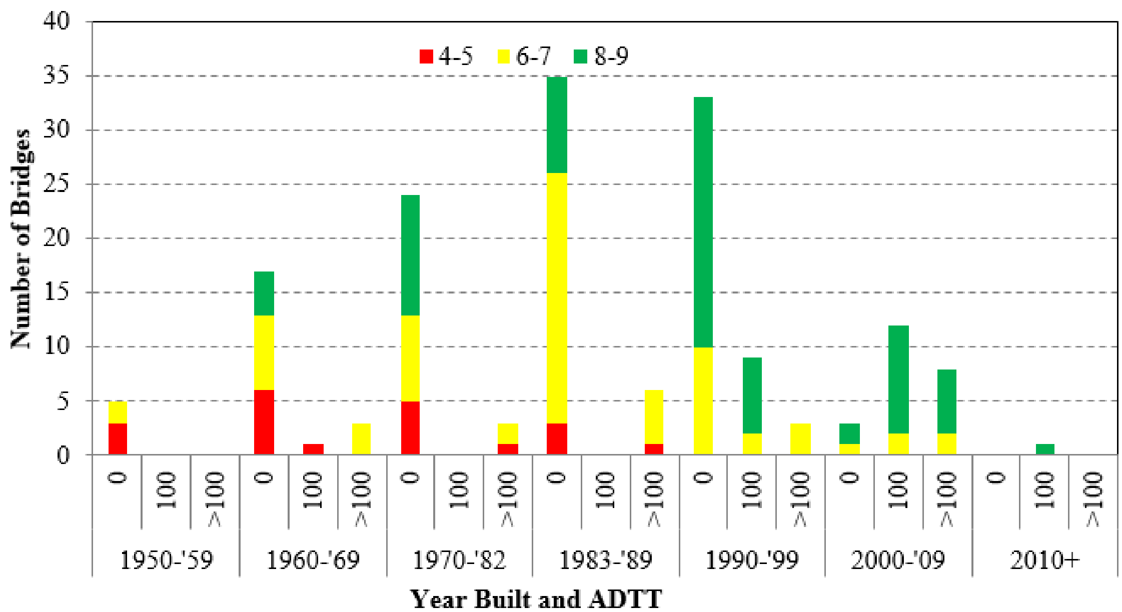


Figure 4-5. NBI superstructure rating: number of bridges vs. year built and ADTT

Figure 4-6 and Figure 4-7 show the NBI rating of deck and superstructure against skew. As shown, 133 bridges out of 169 have a skew less than or equal to 20 degrees. The NBI ratings of the deck and superstructure of the post-1983 bridges, with respect to skew and ADT, are shown in Figure 4-8 and Figure 4-9. As shown, 64 out of 116 bridges have a skew less than or equal to 20 degrees and carry less than 1,000 ADT. In order to eliminate the impact of traffic on reflective crack propagation and complexities arising from skew, only the bridges with a skew up to 20 degrees and ADT less than 1,000 are considered for selecting a pool of 10 bridges for inspection.

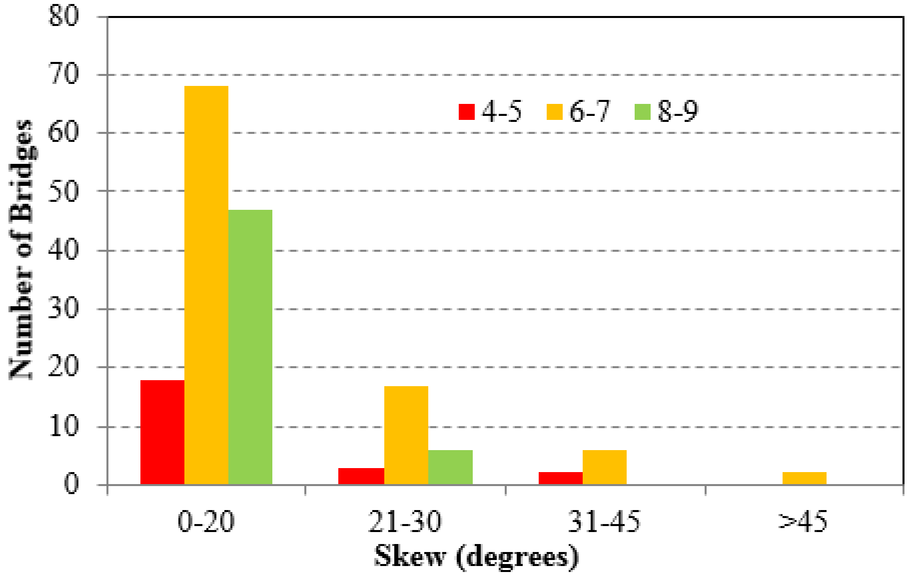


Figure 4-6. NBI deck rating: number of bridges vs. skew

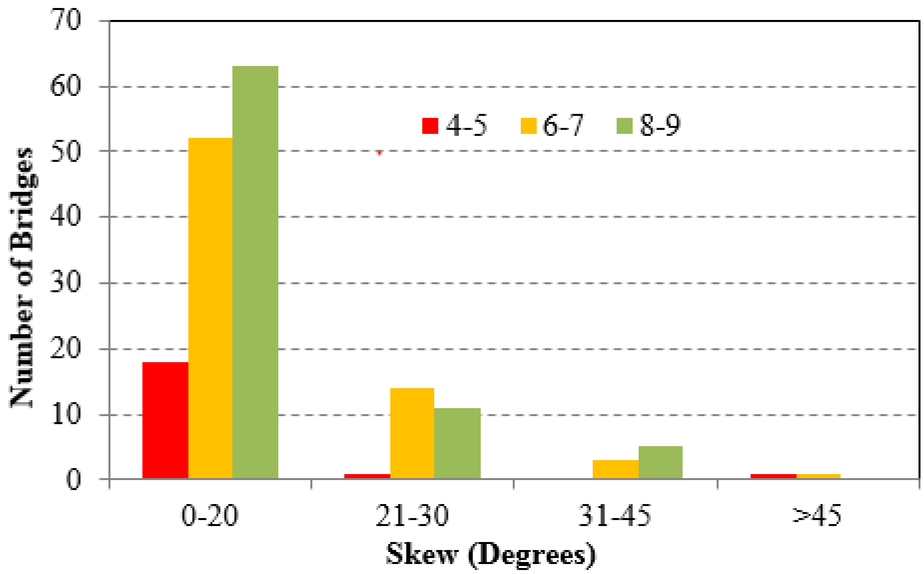


Figure 4-7. NBI superstructure rating: number of bridges vs. skew

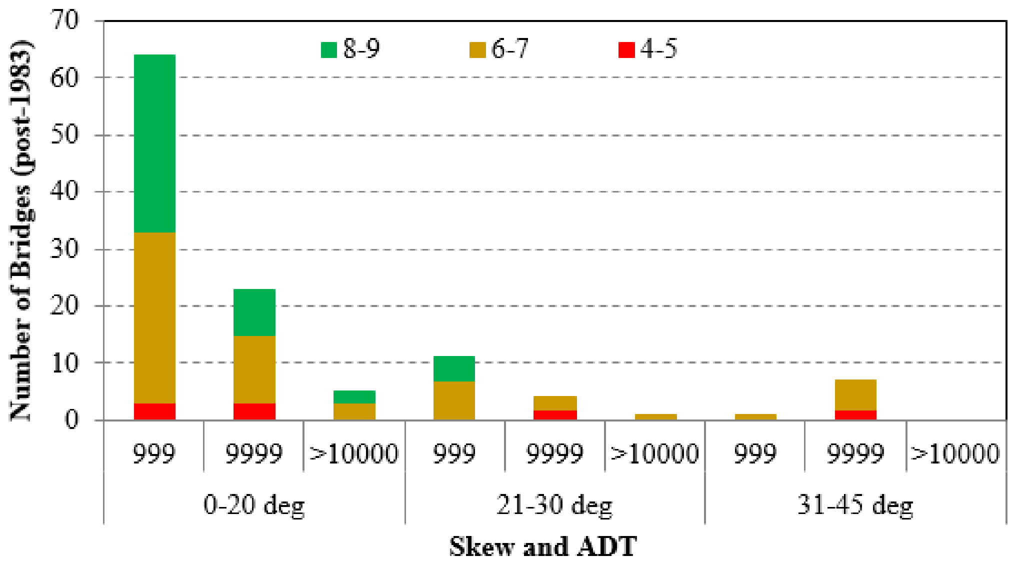


Figure 4-8. NBI deck rating: number of bridges (post-1983) vs. skew and ADT

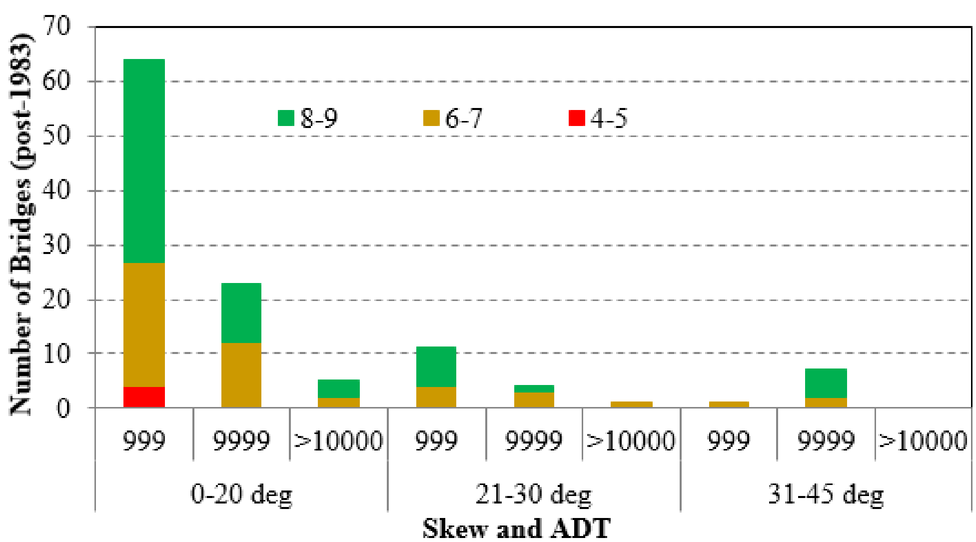


Figure 4-9. NBI superstructure rating: number of bridges (post-1983) vs. skew and ADT

Among the 64 post-1983 bridges with a skew up to 20 degrees and ADT less than 1,000, one bridge was built in 2013. Hence, that bridge was also eliminated, and the remaining 63 bridges were selected for further analysis. Table 4-4 shows the number of bridges constructed at various time periods and the associated numbers identified for field inspection. With this process, 10 bridges were identified for inspection. An additional bridge, which was constructed in 2008 with a 6 in. thick cast-in-place deck, was included as the 11th bridge based on feedback from the research project oversight committee members (Table 4-5). While inventory data is presented in Table 4-5, the NBI condition rating and the inspection date are given in Table 4-6. As shown in Table 4-6, the condition rating of bridges ranges from 6 to 9.

Table 4-4. Number of Post-1983 Bridges with Skew ≤ 20 Deg. and ADT <1,000 and the Numbers Selected for Inspection

Period	Number of bridges with skew ≤ 20 deg., and ADT < 1,000	Number of bridges selected for inspection
2000 – 2010	11	2
1990-1999	28	4
1983-1989	24	4

Table 4-5. Inventory Data of Selected Bridges for Inspection

Bridge ID	Year Built	Feature Intersected	Facility Carried	Spans	Max Span (ft)	Length (ft)	Skew (Deg.)	Deck Width (ft)	ADT
B440198	2008	Embarrass River	STH 76	3	61	185	0	41	1100
B290135	2005	Sprague Mather Flowage	Speedway Road	1	24	24	0	25	100
B290133	2005	Little Yellow River	25th Street West	1	34	34	20	25	100
B370257	1992	Plover River	Esker Road	1	48	48	0	25	40
B290091	1991	Little Yellow River	11th Street	1	30	30	0	25	20
B290096	1991	Rynears Flowage	20th Street West	1	30	30	0	25	100
B370134	1990	Spring Brook	School Road	1	34	34	20	25	50
B050282	1989	BR Kewaunee River	County Line Road	1	50	50	0	25	100
B200094	1987	W Branch of Milwaukee River	River Drive	1	48	48	10	25	100
B360127	1985	E Twin River	Zander Road	2	30	60	20	29	140
B360125	1985	Danmar Road	Branch River	2	44	88	15	34	110

Table 4-6. NBI Condition Rating of the Bridges

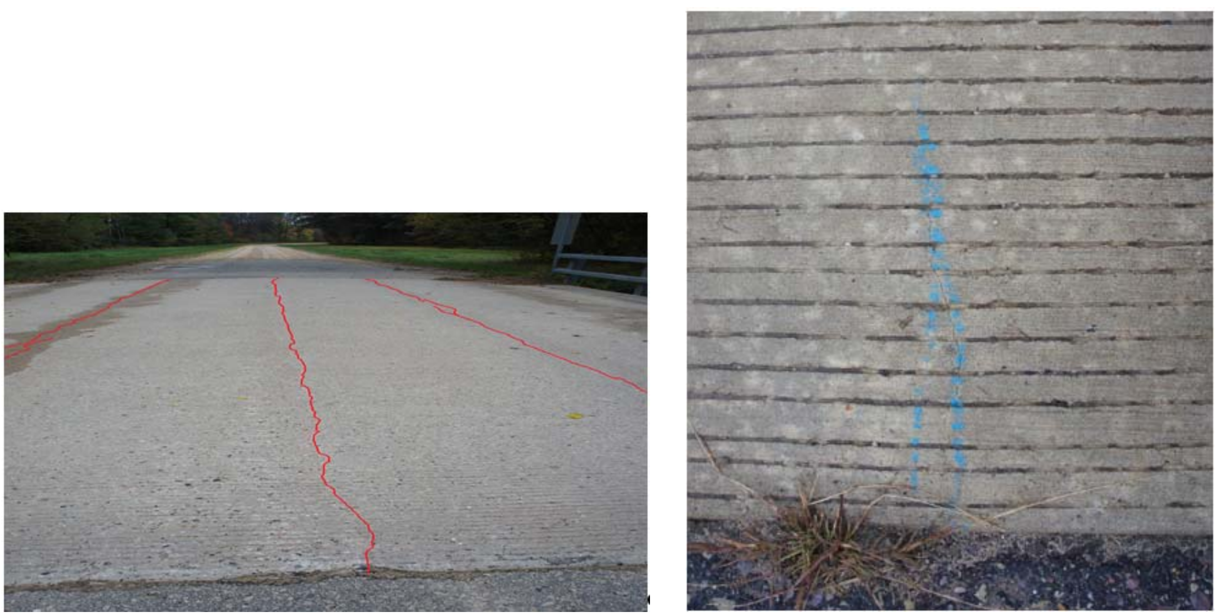
Bridge ID	Year Built	Spans	Max Span (ft)	Length (ft)	Skew (Deg.)	Deck Width (ft)	ADT	NBI Rating		NBI Inspection Date
								Deck	Superstructure	
B440198	2008	3	61	185	0	41	1100	8	8	08/01/2012
B290135	2005	1	24	24	0	25	100	9	9	06/12/2012
B290133	2005	1	34	34	20	25	100	9	9	12/06/2012
B370257	1992	1	48	48	0	25	40	7	7	10/19/2012
B290091	1991	1	30	30	0	25	20	8	8	05/12/2012
B290096	1991	1	30	30	0	25	100	8	8	12/06/2012
B370134	1990	1	34	34	20	25	50	7	7	09/01/2012
B050282	1989	1	50	50	0	25	100	7	8	10/15/2012
B200094	1987	1	48	48	10	25	100	7	7	10/29/2012
B360127	1985	2	30	60	20	29	140	6	6	11/26/2012
B360125	1985	2	44	88	15	34	110	7	7	11/14/2012

4.2.2 Bridge Inspection Summary and Conclusions

The bridges were inspected during the first week of October 2013. During the 3-day inspection period, a 50% - 70% chance of rain was predicted. There was no need for spraying water on the deck since all bridge sites received some rain. Moist deck conditions exposed reflective longitudinal cracks. Due to water levels rising under some of the bridges, it was not possible to document the condition of shear keys on a couple of bridges.

Bridge design details and field observations of each bridge are presented in Appendix E. The following are the summaries and conclusions derived from field observations:

1. Irrespective of the age or the design changes, reflective cracking was documented on all the bridge decks (Figure 4-10). Where full-length reflective cracking was not present, short-length cracks directly over the piers or abutments and above the shear keys were documented. This observation verifies that the reflective cracking initiates over the supports (piers and abutments) and develops to full-length cracks under live loads. Table 4-7 summarizes the inspection data in terms of reflective deck cracking and moisture and efflorescence documented at the shear keys.



(a) Full-length reflective cracks

(b) A short crack over the abutment and above a shear key

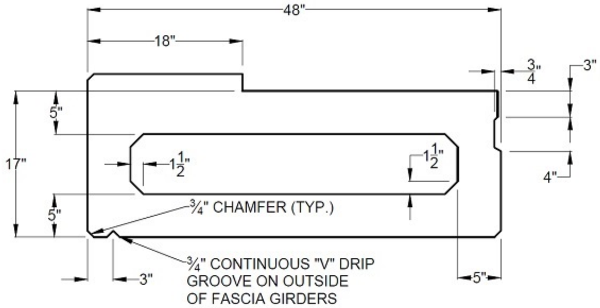
Figure 4-10. Reflective deck cracking

Table 4-7. Summary of Inspection Data Related to Reflective Cracking and Shear Key Condition

Bridge ID	Year Built	Feature Intersected	Facility Carried	Reflective Cracking	Moist Shear Keys	Efflorescence
B440198	2008	Embarrass River	STH 76	Yes	No	No
B290135	2005	Sprague Mather Flowage	Speedway Road	Yes	Yes	No
B290133	2005	Little Yellow River	25th Street West	Yes	-*	-
B370257	1992	Plover River	Esker Road	Yes	Yes	Yes
B290091	1991	Little Yellow River	11th Street	Yes	Yes	No
B290096	1991	Rynears Flowage	20th Street West	Yes	-	-
B370134	1990	Spring Brook	School Road	Yes	Yes	Yes
B050282	1989	BR Kewaunee River	County Line Road	Yes	Yes	Yes
B200094	1987	W Branch of Milwaukee River	River Drive	Yes	Yes	Yes
B360127	1985	E Twin River	Zander Road	Yes	Yes	Yes
B360125	1985	Danmar Road	Branch River	Yes	Yes	Yes

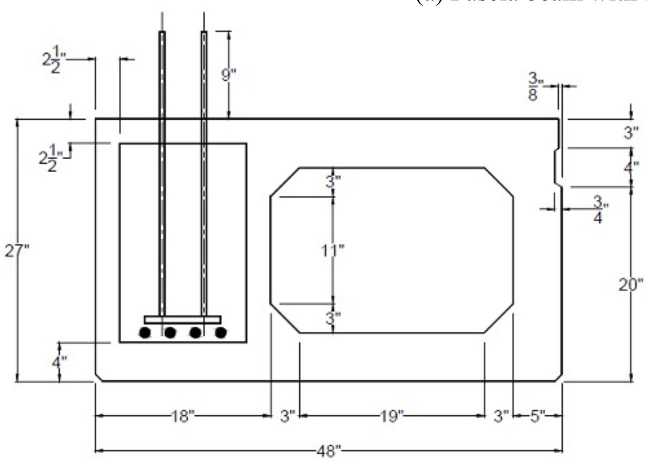
* Beam soffits were not inspected.

2. Two different fascia beam geometries have been used since 1983. In one case, a curb is integrated into the beam geometry (Figure 4-11a). The most recent design incorporates a solid block with reinforcement detail to attach bridge railings (Figure 4-11b). Irrespective of these changes, transverse cracking was documented on the fascia beams.



(i) Fascia beam detail (ii) Transverse cracks over the curb

(a) Fascia beam with an integral curb



(i) Fascia beam detail (ii) Transverse cracks over the fascia beam

(b) Fascia beam without an integral curb but with a solid block

Figure 4-11. Fascia beam detail and transverse cracks

3. The current bridge superstructure configuration permits surface water laced with deicing salts to drain over the fascia beam and into the stress pockets (Figure 4-12).

Figure 4-12. A stress pocket exposed to surface water

4. In some bridges, the construction joint of the deck overlay was placed directly over the shear key located at the bridge centerline (Figure 4-13). This joint has not been sealed. Hence, the surface water can seep through this joint moistening the shear key and the beams.

(a) The deck overlay construction joint

(b) Close-up view of the construction joint

Figure 4-13. The deck overlay construction joint

5. As shown below, the flood level had reached the superstructure of one of the inspected bridges. As a result, flood water percolated through the drains at the beam soffit. The efflorescence at the drains is an indication of the amount of water trapped inside the boxes (Figure 4-14).

(a) Indication of flood water level

(b) Drains with moisture and efflorescence

Figure 4-14. Flood water trapped inside boxes

4.3 IMPACT OF CAST-IN-PLACE CONCRETE SLAB AND OVERLAY ON SUPERSTRUCTURE DURABILITY

Different bridge deck protection systems are used to enhance bridge durability (Russell 2012). In adjacent box-beam bridges, the superstructure is protected with a 5 to 6 in. thick cast-in-place concrete slab, rigid overlay, or a flexible overlay with a waterproofing membrane. Use of a cast-in-place concrete slab requires a minimum of 7-day moist curing. Installation of overlays may take from 1 to 3 days depending on the overlay type (Krauss et al. 2009). Typically, cast-in-place concrete slabs are used on high-volume road or freeway bridges while overlays are used on low-volume roads. According to the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (FHWA 1995), a bridge superstructure with a rating of 7 is in good condition, and a rating of 6 indicates a satisfactory condition. Generally, a bridge superstructure is recommended to be replaced when it is rated at 4 or below. Hence, a service life of a bridge superstructure can be considered as the time it takes to reach the NBI rating of 4 from the year built. In other words, the time it takes for a bridge superstructure to reach a predefined NBI rating can be used as a service life indicator. In this study, the time it takes from year built until the deck superstructure is assigned a rating of 6 or 7 is defined as the service life to avoid the impact of any repairs on bridge durability performance.

4.3.1 Deck and Wearing Surface Types

According to FHWA (1995), deck structure types applicable for box-beam bridges are concrete cast-in-place (Code 1) and concrete precast panels (Code 2). Code 1 is used when a 5 to 6 in. thick cast-in-place concrete slab is placed on top of the box-beams. This is typical for bridges that are on freeways or highways with high ADT. For low-volume roads, box-beam top surface is used with or without a wearing surface. Then, code 2 is used to represent the deck structure type and is referred to as concrete precast panels. Ten (10) different wearing surface types are listed in FHWA (1995). The coding ranges from 0 to 9. Only codes 0 to 6 and 9 are applicable for box-beams. Column 1 and 2 of Table 4-8 present deck structure types, wearing surface types, and the associated codes. Column 3 of the same table lists the most commonly used combinations of deck structure and wearing surface types. The numbers shown within brackets represent the relevant codes.

Table 4-8. Deck Structure Type, Wearing Surface Type, and the Most Common Deck Structure and Wearing Surface Type Combinations for Adjacent Box-Beam Bridges

Deck structure type		Wearing surface type		Mostly commonly used deck structure and wearing surface combinations
Code	**Description**	**Code**	**Description**	
1	Concrete Cast-in-Place	1	Monolithic Concrete (concurrently placed with structural deck)	Concrete cast-in-place (1) Monolithic (1)
2	Concrete Precast Panels	2	Integral Concrete (separate non-modified layer of concrete added to structural deck)	Concrete Precast Panels (2) Monolithic (1)
		3	Latex Concrete or Similar Additive	Concrete Precast Panels (2) Integral(2)
		4	Low Slump Concrete	Concrete Precast Panels (2) Bituminous (6)
		5	Epoxy Overlay	Concrete Precast Panels(2) None(0)
		6	Bituminous	
		9	Other	
		0	None (no additional concrete thickness or wearing surface included in the deck)	

4.3.2 Durability Performance Analysis

Following the approach discussed in Chapter 3, 17 states with side-by-side box-beam bridges, and the exposure conditions similar to the State of Wisconsin, are selected. The NBI database of year 2014 is used for this purpose (NBI 2014). Table 4-9 shows the total number of box-beam bridges in each state. Table 4-10 presents the total number of box-beam bridges under each combination. The last category ("Other") includes all the box-beam bridges with deck structure and wearing surface type combinations that do not belong to the combinations listed in Table 4-10. According to the data shown in Table 4-10, cast-in-place concrete slabs with monolithic concrete wearing surface and bituminous overlay on box-beams are the most common combinations. For further analysis, the states with more than 30 bridges with these two combinations are selected.

Table 4-9. Total Number of Box-Beam Bridges in Each State (NBI 2014)

State	Total number of Box Beam Bridges
Connecticut	286
Delaware	108
Illinois	8796
Indiana	4465
Kentucky	3531
Massachusetts	175
Michigan	2480
Montana	11
New Hampshire	41
New Jersey	528
New York	1939
Ohio	6892
Pennsylvania	3332
Rhode Island	67
Virginia	310
Vermont	27
Wisconsin	88
Total	33,076

Table 4-10. Box-Beam Bridges with Deck Structure and Wearing Surface Types

Combination of Deck Structure and Wearing Surface Type	CT	DE	IL	IN	KY	MA	MI	NH	MO	NJ	NY	OH	PA	RI	VA	VT	WI
Concrete cast-in-place, Monolithic	-	32	-	609	596	5	656	-	1	299	117	146	1341	5	12	2	40
Concrete Precast Panels, Bituminous	5	31	3192	2665	853	125	944	8	-	67	1	-	22	37	78	14	1
Concrete Precast Panels, None	-	-	3371	107	226	-	28	2	-	1	-	-	-	-	1	5	4
Concrete Precast Panels, Monolithic	-	-	1	457	1315	5	62	-	-	2	-	-	4	-	1	-	1
Concrete Precast Panels, Integral	-	-	247	181	43	1	38	1	-	-	2	-	-	-	-	1	1
Other	281	45	1985	446	498	39	752	30	10	159	1819	6746	1965	25	218	5	41

The durability performance of bridge superstructures with (1) cast-in-place concrete deck and monolithic wearing surface types and (2) precast panel deck structure and bituminous wearing surface types are evaluated. The other parameters considered in the analysis include ADT, bridge superstructure age, and bridge superstructure condition rating. ADT is divided into three ranges: ADT ≤ 1,000; 1,000 < ADT ≤ 10,000; and ADT > 10,000. The year of bridge reconstruction is also taken into consideration when calculating the service life (age) of a bridge superstructure. The year reconstructed is used as the year built for the analysis of reconstructed bridges. The superstructure age is categorized into four ranges: 0-10, 11-20, 21-30 and greater than 30 years. The condition rating defined in the NBI is used, and the ranges are defined as greater than 7 (very good to excellent), 6 and 7 (satisfactory to good), and less than 6 (failed to fair).

4.3.2.1 Superstructure with a Cast-in-place Concrete Deck and a Monolithic Wearing Surface

The total number of bridges with a cast-in-place concrete deck and a monolithic wearing surface is 3836. Out of 3836 bridges, 52% (1992), 36% (1382), and 12% (462) carry ADT ≤ 1,000; 1000 < ADT ≤ 10,000; and ADT > 10,000 respectively. Table 4-11 shows the analysis results.

Table 4-11a presents analysis results of 1992 bridges with ADT ≤ 1,000. The following conclusions can be derived from the results:

- A total of 777 (39%) are in very good to excellent, 1039 (52.2%) are in satisfactory to good, and 176 (8.8%) are in failed to fair condition.
- 37.5% (i.e., 747 out of 1992) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 14.2% (i.e., 282 out of 1992) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 5.1% (i.e., 101 out of 1992) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-11b presents analysis results of 1382 bridges with 1000 < ADT ≤ 10,000. The following conclusions can be derived from the results:

- A total of 527 (38.1%) are in very good to excellent, 724 (52.4%) are in satisfactory to good, and 131 (9.5%) are in failed to fair condition.
- 35.6% (i.e., 493 out of 1382) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 11.5% (i.e., 160 out of 1382) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 4.6% (i.e., 64 out of 1382) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-11c presents analysis results of 462 bridges with ADT > 10,000. The following conclusions can be derived from the results:

- A total of 153 (33.1%) are in very good to excellent, 253 (54.8%) are in satisfactory to good, and 56 (12.1%) are in failed to fair condition.
- 30.5% (i.e., 141 out of 462) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 6.2% (i.e., 29 out of 462) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 5.6% (i.e., 26 out of 462) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-11. Durability Performance of Bridge Superstructures with a Cast-in-Place Deck and a Monolithic Wearing Surface

(a) ADT ≤ 1,000						
Age (Years)	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
< 10	413	20.7	129	6.5	36	1.8
11-20	228	11.4	299	15.0	13	0.7
21-30	108	5.4	357	17.9	26	1.3
> 30	28	1.4	254	12.8	101	5.1
Total	**777**	39.0	**1039**	52.2	**176**	8.8
(b) 1,000 < ADT ≤ 10,000						
Age (Years)	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
< 10	281	20.3	95	6.9	19	1.4
11-20	157	11.4	225	16.3	21	1.5
21-30	76	5.5	257	18.6	27	2.0
> 30	13	0.9	147	10.6	64	4.6
Total	**527**	38.1	**724**	52.4	**131**	9.5
(c) ADT > 10,000						
Age (Years)	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
< 10	89	19.3	35	7.6	4	0.9
11-20	50	10.8	91	19.7	13	2.8
21-30	12	2.6	100	21.6	13	2.8
> 30	2	0.4	27	5.8	26	5.6
Total	**153**	33.1	**253**	54.8	**56**	12.1

4.3.2.2 Superstructure with a Bituminous Wearing Surface

The total number of bridges with a bituminous wearing surface is 7991. Out of 7991 bridges, 80.5% (6430), 17.5% (1398), and 2% (163) carry ADT ≤ 1,000; 1000 < ADT ≤ 10,000; and ADT > 10,000 respectively. Table 4-12 shows the analysis results.

Table 4-12a presents analysis results of 6430 bridges with ADT ≤ 1,000. The following conclusions can be derived from the results:

- A total of 1921 (29.9%) are in very good to excellent, 3757 (58.4%) are in satisfactory to good, and 752 (11.7%) are in failed to fair condition.
- 57.5% (i.e., 3698 out of 6430) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 31.1% (i.e., 2003 out of 6430) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 9.3% (i.e., 597 out of 6430) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-12b presents analysis results of 1398 bridges with 1000 < ADT ≤ 10,000. The following conclusions can be derived from the results:

- A total of 274 (19.6%) are in very good to excellent, 764 (54.6%) are in satisfactory to good, and 360 (25.8%) are in failed to fair condition.
- 53.7% (i.e., 751 out of 1398) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 33.3% (i.e., 465 out of 1398) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 20.2% (i.e., 282 out of 1398) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-12c presents analysis results of 163 bridges with ADT > 10,000. The following conclusions can be derived from the results:

- A total of 23 (14.1%) are in very good to excellent, 79 (48.5%) are in satisfactory to good, and 61 (37.4%) are in failed to fair condition.

- 37.5% (i.e., 61 out of 163) of these bridges have been in service for more than 20 years and have maintained a condition rating of more than or equal to 6.
- 23.4% (i.e., 38 out of 163) of these bridges have been in service for more than 30 years and have maintained a condition rating of more than or equal to 6.
- 31.9% (i.e., 52 out of 163) of these bridges have been in service for more than 30 years with a condition rating less than 6.

Table 4-12. Durability Performance of Bridge Superstructures with a Bituminous Wearing Surface

(a) ADT ≤ 1,000						
	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
Age (Years)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
0-10	500	7.8	165	2.6	7	0.1
11-20	623	9.7	692	10.8	29	0.5
21-30	610	9.5	1085	16.9	119	1.9
>30	188	2.9	1815	28.2	597	9.3
Total	**1921**	29.9	**3757**	58.4	**752**	11.7
(b) 1,000 < ADT ≤ 10,000						
	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
Age (Years)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
0-10	82	5.9	27	1.9	3	0.2
11-20	67	4.8	111	7.9	11	0.8
21-30	76	5.4	210	15.0	64	4.6
>30	49	3.5	416	29.8	282	20.2
Total	**274**	19.6	**764**	54.6	**360**	25.8
(c) ADT > 10,000						
	Condition Rating >7		Condition Rating 6 and 7		Condition Rating <6	
Age (Years)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)	Number of Bridges	Percentage (%)
0-10	10	6.1	7	4.3	-	-
11-20	8	4.9	16	9.8	3	1.8
21-30	1	0.6	22	13.5	6	3.7
>30	4	2.5	34	20.9	52	31.9
Total	**23**	14.1	**79**	48.5	**61**	37.4

4.3.2.3 Performance Comparison

Durability performance of bridge superstructures with (1) a cast-in-place concrete deck and a monolithic wearing surface, and (2) box-beams and a bituminous wearing surface is compared. As shown in Figure 4-15, when a cast-in-place concrete deck with a monolithic wearing surface is used, the number of bridge superstructures in service for more than 30 years drops drastically. Performance during first 30 years is not affected by the traffic volume. However, rate of deterioration beyond 30 years is greatly affected by the traffic volume. As shown in Figure 4-16, irrespective of the age or the traffic volume, very few bridge superstructures remain in service once reaching a condition rating of less than 6. This is primarily due to a drastic increase in rate of deterioration after the condition rating drops below 6.

In contrast, a large percentage of bridge superstructures with a bituminous wearing surface have been in service for more than 30 years (Figure 4-17). Similar to the performance of superstructures with a cast-in-place deck and a monolithic wearing surface, the rate of deterioration of superstructures with a bituminous wearing surface increases with the traffic volume (Figure 4-18). As shown in Figure 4-17 and Figure 4-18, a majority of the superstructures with a bituminous wearing surface and a service life of more than 30 years has a condition rating of 6 or greater.

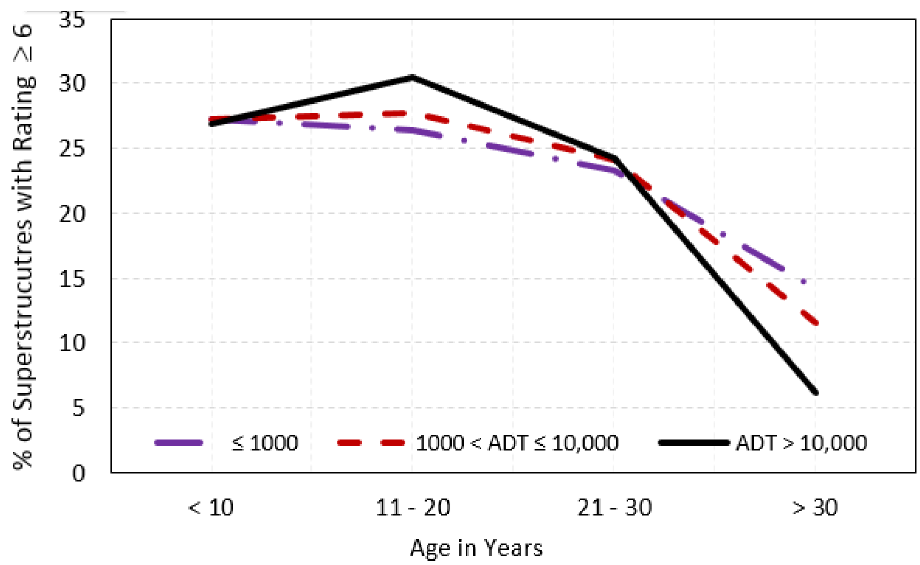


Figure 4-15. Percent of superstructures with a cast-in-place deck and a monolithic wearing surface against age and ADT (Condition Rating ≥ 6)

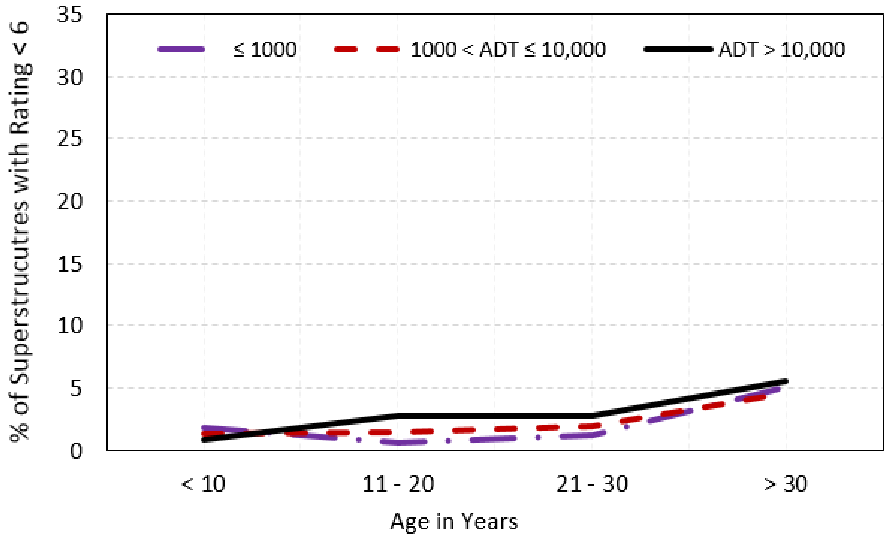


Figure 4-16. Percent of superstructures with a cast-in-place deck and a monolithic wearing surface against age and ADT (Condition Rating < 6)

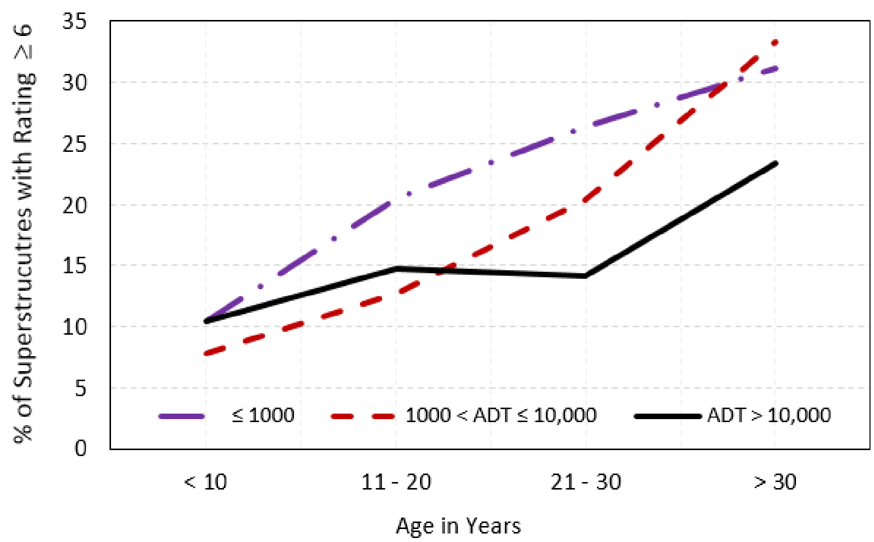


Figure 4-17. Percent of superstructures with a bituminous wearing surface against age and ADT (Condition Rating ≥ 6)

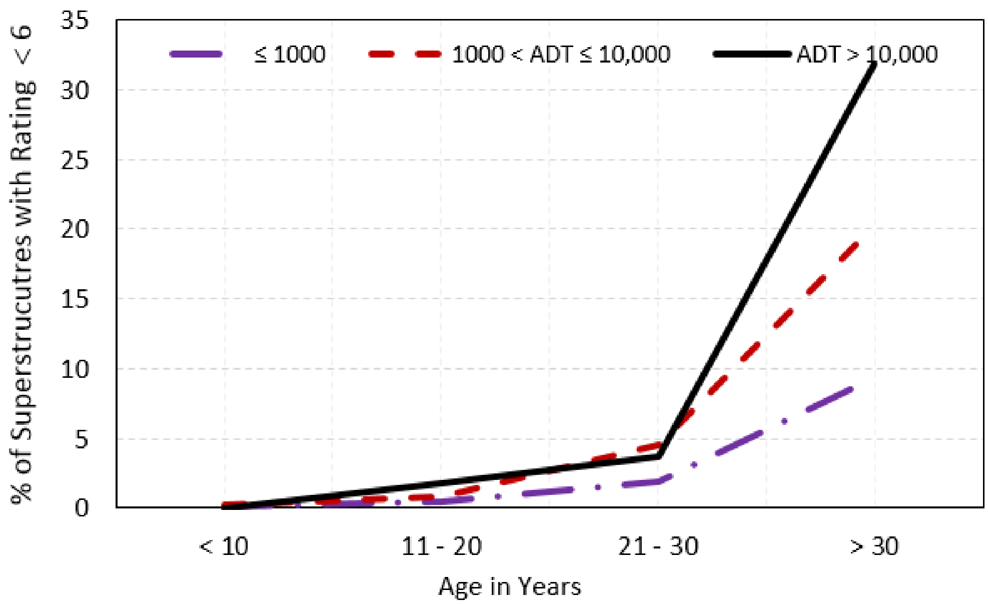


Figure 4-18. Percent of superstructures with a bituminous wearing surface against age and ADT (Condition Rating < 6)

4.3.3 Bridge Durability Performance Evaluation Summary, Conclusions, and Recommendations

The 2014 NBI data is used for evaluating the impact of deck structure and wearing surface types on bridge superstructure durability performance. Data for 17 different states was selected. Typically, waterproofing membranes are used with bituminous wearing surfaces. Due to lack of data, the impact of various waterproofing membranes on durability is not investigated during this study. Based on the analysis results the following conclusions can be derived:

1) Box-beam bridge superstructures with cast-in-place decks and monolithic wearing surfaces show better performance during the first 20 years in service.
2) A large number of box-beam bridge superstructures with bituminous wearing surfaces have been in service for more than 30 years with a superstructure condition rating of 6 or greater.
3) There is a possibility of using a bituminous wearing surface on box-beams to develop durable bridges when ADT ≤ 10,000.

After evaluating the impact of deck structure and wearing surface types on bridge superstructure durability performance, and looking at the national and international practices, it is recommended to consider using a multi-layer bridge deck protective system for improving durability of bridge decks. As an example, the Ministry of Transportation Ontario uses a multi-layer protective system (Lai 2008); Massachusetts DOT places a wearing surface with a waterproofing membrane on top of concrete deck structures (MassDOT 2017); and some countries in Europe use a multi-layer bridge deck protective system (as shown in Figure 4-19) (Attanayake et al. 2002). Use of an exposed deck or a wearing surface depends on the current highway agency policies. In order to propose amendments to current policies, a comprehensive study needs to be conducted with a statistically meaningful data set that evaluates the effectiveness of various bridge deck preservation policies and strategies implemented by the highway agencies.

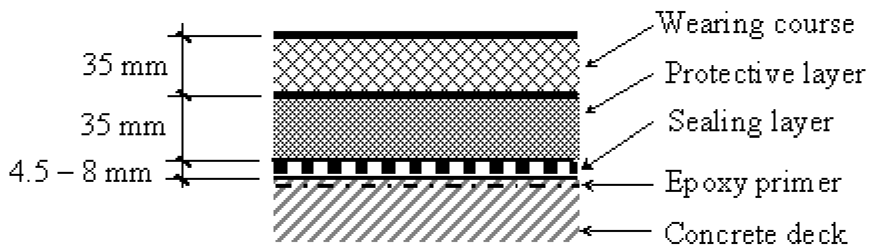


Figure 4-19. Bridge deck protective system used in the Europe (Attanayake et al. 2002)

5 PERFORMANCE EVALUATION OF NEWLY CONSTRUCTED BRIDGES

5.1 OBJECTIVE AND APPROACH

Chapter 2 and 3 of this report document the details and construction practices implemented by various highway agencies for minimizing reflective deck cracking on adjacent box-beam bridges. Chapter 3 also includes a summary of the responses received from a selected number of highway agencies for a survey conducted as part of this study. A representative sample of in-service box-beam bridges in Wisconsin was inspected, and the findings are documented in Chapter 4. Based on the outcome of the efforts described in Chapters 2, 3, and 4, a list of best practices was developed. In 2016, WisDOT built three adjacent box-beam bridges and incorporated a limited number of recommendations.

This chapter presents (a) the list of best practices documented through the efforts documented in Chapter 2, 3, and 4, (b) details and practices implemented in three recently constructed bridges, (c) performance of bridge decks documented during the field visits that were conducted following construction, and (d) structural response to shrinkage and thermal gradient loads. The inspection scope was limited to visual inspection and evaluating bridge performance in terms of longitudinal deck cracking.

5.2 DESIGN AND CONSTRUCTION RECOMMENDATIONS

This section presents a list of best practices identified through the efforts documented in Chapter 2, 3, and 4.

5.2.1 Box-Beam Section

- The box-beam with rectangular voids is the standard used by the state DOTs. Compared to the sections used in other countries, the box-beam with rectangular voids has the least dead load per unit length. Further, fabricators are familiar with this cross-section, and the formwork is readily available. Hence, the use of box-beams with rectangular voids minimizes the impact on the fabrication process.
- The use of a full-depth shear keys allows adequate confinement to prevent grout spall even if the shear key material is debonded from the beam.

- Inspection of the interior condition of the voids is a challenge during routine inspection. Irrespective of the material used to form the voids, historical records have shown that the deterioration starts from the interior of the voids. Providing two drain holes at each end of a void minimizes moisture accumulation and helps inspection of the interior.

5.2.2 Shear Key Grout

- Cementitious grout with expansive or non-shrink properties is often recommended in precast construction due to assumed material compatibility. Surface preparation is important and critical for effective bonding between the grout and box-beam. Some state DOTs use approved commercial grouts instead of traditional mortar due to non-shrink and strong bond properties.
- Use of a cementitious grout with non-shrink properties and adequate bond strength that develops at least 3,000 psi compressive strength before transverse post-tension application is desired. The grout strength needs to reach at least 5,000 psi by the time the cast-in-place concrete slab is poured or an overlay is placed to form adequately stiff connections for load transfer.

5.2.3 Cast-in-Place Concrete Slab or Wearing Surface

Chapter 4 presents the durability performance of box-beam bridge superstructures with an exposed deck slab or a bituminous wearing surface with a waterproofing membrane. The practice is governed by the policies of a highway agency. In order to propose amendments to current policies, a comprehensive study needs to be conducted with a statistically meaningful data set by evaluating the effectiveness of various bridge deck preservation policies and strategies implemented by the highway agencies. Until such amendments are developed, the following practices are proposed:

- Use of an asphalt wearing surface with a waterproofing membrane has contributed to maintaining adjacent box-beam bridges in good or satisfactory condition over 30 years. With a cost effective life-cycle treatment plan, the service life can be extended. With a proven performance record, other wearing surface types can be used on box-beam bridge decks.

- Irrespective of the type of wearing surface used on bridge decks, the longitudinal construction joint needs to be offset a minimum of 1.5 ft but not greater than 2 ft from the closest shear key.

5.2.4 Transverse Post-Tensioning

The shear keys that are located longitudinally in between the diaphragms are not compressed when post-tensioning is applied through the diaphragms, even under increased post-tensioning. Developing recommendations for revising post-tension details and procedures or alternative connection details requires additional research. Hence, the use of adequate post-tensioning at least for maintaining structural redundancy is recommended.

Wisconsin DOT standard detail specifies placing girder end dowel bars and grouting after post-tension application. This construction sequence is encouraged because it allows compressing shear key grout over the abutments and piers, and avoids post-tension force transfer to the substructure.

5.2.5 Construction Process

- Properly anchored expanded polystyrene blocks shall be used to form box-beam voids.
- Voids shall be vented and drained by casting two, one-inch diameter tubes at the bottom edges of the corner fillets. Puncture the holes immediately after removal from casting bed.
- When extending stirrups for shear connection to slab, consider a bent shape of bar in relation to placement of foam blocks.
- Two, 2 in. diameter dowel holes shall be provided at each beam end.
- Box-beam top surfaces shall be rough finished to provide a ¼ inch surface texture unless otherwise required.
- The longitudinal keyway surfaces shall be sandblasted immediately before shipping.
- Beam seats shall be sloped to match the roadway crown.
- Beam seats shall be sloped parallel to grade line if the grade at the bridge is greater than 1%. Elevations shall be placed on plans to meet these requirements. Beam bearing pads shall be shimmed to minimize rocking of girders. The bearing surfaces shall be set level in the longitudinal direction.

- Beams shall be erected. When differential camber occurs, beams shall be forced together, or a smooth transition shall be provided with joint grout material.
- Hardwood wedges shall be installed between adjacent beams to maintain beam spacing.
- Gaskets shall be provided at the post-tensioning duct locations to prevent shear key grout flowing into the ducts. The gasket shall be sponge neoprene of 2 ¾ in. minimum thickness.
- A transition between changing slopes of post-tensioning ducts shall be provided by either a circular or parabolic curve with a minimum length of 3 ft.
- The transverse ties shall be installed. The position of the hardwood wedges shall be verified.
- All shear key surfaces shall be prewetted for a minimum of 24 hours if the surfaces are not protected using penetrating sealants. *Note: When commercial grout is used, the manufacturer's recommendations for surface preparation shall be followed.*
- Shear key surfaces shall be cleaned with high-pressure water or oil free air-blast. *Note: When commercial grout is used, the manufacturer's recommendations for surface preparation shall be followed.*
- A Polyethylene closed cell backer rod shall be installed at shear key locations.
- Transverse ties shall be post-tensioned to approximately 5,000 lbs to remove sag in the tie and to seat the chuck.
- Shear keys shall be grouted with a non-shrink cementitious grout with adequate bond strength and a compressive strength of at least 3,000 psi. Grout shall be rodded or vibrated to form a solid, tight shear key. *Note: When commercial grout is used, the manufacturer's instructions shall strictly be followed. Also, exposure conditions at the time of grouting shear keys shall be considered.*
- Proper curing shall be provided for the grout. *Note: When commercial grout is used, the manufacturer's recommendations shall be followed.*
- Wedges shall be removed before post-tension application.
- Post-tensioning shall be applied to compress box-beam and shear key assembly. Galvanized end anchorage hardware shall be used.

  On shallow members with one row of ties along the depth, each transverse tie shall be tensioned to provide the required force.

On deep members with multiple rows of ties along the depth, each transverse tie shall be tensioned to provide half of the required force. This tensioning sequence shall be repeated once more so that each tie is tensioned to provide the total required force.

Tensioning shall begin with the tendons at each end and then work symmetrically towards the midspan from each end.

- Post-tensioning ducts shall be pressure grouted from one grout pipe until all entrapped air is expelled and grout begins to flow from the open grout pipe. The open grout pipe shall be closed and a pressure of 50 psi maintained for 15 seconds.
- Holes shall be drilled in piers and abutments for dowels. For expansion joints, hot-poured rubber-asphalt type shall be used to fill up to 3 in. in the dowel holes. The remainder of the hole shall be filled with non-shrink grout. For fixed joints, the entire hole shall be filled with non-shrink grout.
- The stress pockets shall be filled with chloride free non-shrink grout after post-tensioning.
- An approved wearing surface type shall be installed. *Note: The grout strength shall reach at least 5,000 psi by the time the wearing surface is installed to form adequately stiff connections.*
- Longitudinal construction joints shall be offset a minimum of 1.5 ft, but not greater than 2 ft from the closest shear key.

5.3 WISCONSIN-47 OVER LOST CREEK (B-26-40)

Wisconsin bridge B-26-40 that carries Wisconsin-47 over Lost Creek is located in the town of Sherman in Iron County, Wisconsin (Figure 5-1), just southeast of the unincorporated community of Manitowish. This single span, straight bridge is aligned in the east-west direction and carries an average daily traffic volume of less than 1,100 vehicles. After demolition of the existing bridge, the new bridge was constructed at the same alignment in September 2016.

(a) Bridge location on a map

(b) Arial view of bridge

Figure 5-1. Bridge location (Source: Google Maps)

A new superstructure with two 12 ft wide traffic lanes and two 5 ft shoulders was constructed using nine 4-ft wide × 17 inch deep prestressed concrete box-beams and a 6-in. thick cast-in-place concrete deck slab with a minimum specified 28-day strength of 4000 psi. Girders are positioned at different elevations to form a road crown. Bridge span and width are 35 ft and 37.5 ft respectively. The bridge superstructure is transversely post-tensioned over the abutments, and at quarter-points and mid-span. The specified post-tensioning force is 86.7 kips per duct (about 12.38 kip/ft), and applied at the mid-depth of the beam through the diaphragms. Full-depth grouted shear keys are used. Post-tensioning was applied after the grout had cured 2 days and had attained a compressive strength of 3,000 psi. Structural details of the bridge and relevant sections of the project special provisions are presented in Appendix F. As per the structural details and the project special provisions, a majority of design and construction recommendations listed in section 5.2 of this report is implemented. However, the cast-in-place slab and end diaphragms used a typical concrete mix. Table 5-1 shows the concrete mix design, fresh and hardened concrete properties, and ambient temperature at the time of concrete placement. Shear keys were formed with BASF MasterFlow® 928 of which the properties are in compliance with the project special provisions. The technical data sheet of the grout is provided in Appendix F.

Table 5-1. Cast-in-Place Concrete Slab - Mix Design and Fresh Properties

Material	Quantity
Course Aggregate 1.5 in. (with 0.75% moisture)	7600.00 lb
Course Aggregate 0.75 in. (with 0.75% moisture)	9000.00 lb
Washed Sand (with 2.78% moisture)	11,780.00 lb
St. Marys Cement Type I/II	4060.00 lb
Class C Flyash	1045.00 lb
Water Added	1467.18 lb
Air Entraining Admixture – Sika Air 260	20.50 oz
Water Reducing Admixture – Sikament 686	152.00 oz
Total Water in the Mix	1910.26 lb
W/C ratio	0.374
Measured Average Air content	5.3%
Measured Average Concrete temperature	65 ^{0}F
Measured Average Slump	3.25 in.
Ambient Temperature	65 ^{0}F
Average Compressive Strength	4815 psi

5.3.1 Bridge Inspection Summary

As shown in Appendix F, bridge details provide two options for casting deck slab and end diaphragms: in a single operation or with two discrete operations executed within two weeks. For this project, the contractor selected the first option, and the slab and end diaphragms were poured on the 6th of October (Figure 5-2). The bridge deck was first inspected on October 10th and 11th for longitudinal cracking. Ambient temperature during these two days ranged from the mid 50's to high 60's in Fahrenheit while there was a rain in the morning of October 11. The slab was inspected prior to pouring the approach slab and allowed inspection of slab cross-section over the abutments. As specified in the plans, the deck was moist cured for seven days by placing a wet burlap over the entire deck and covering the burlap with a large plastic sheet and a tarp. Figure 5-3 shows a general view of the bridge.

Figure 5-2. Monolithically cast end diaphragm and deck slab

(a) Facing east

(b) Facing west

(c) Side view

Figure 5-3. A general view of the bridge

As shown in Figure 5-3c, due to site constraints, only the deck slab was inspected. Two cracks were identified over the west abutment. Both cracks had an average width of less than 0.005 inches. Figure 5-4 shows the crack locations in reference to the bridge cross-section. The first crack was located above the shear key between beam 5 and 6. Figure 5-6 documents the geometry of the crack. As shown, the crack has reached the full depth of the slab; additionally, the crack has propagated approximately 8 inches along the deck. Since the end diaphragm stem length parallel to bridge longitudinal axis is 12 inches, the crack has not propagated to the deck slab over the beams.

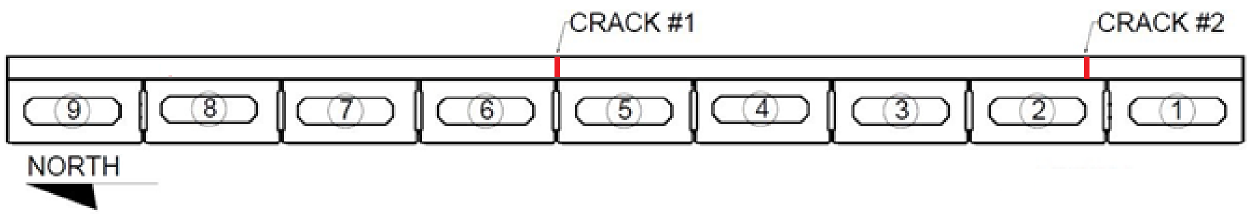


Figure 5-4. Crack locations on west abutment (facing east)

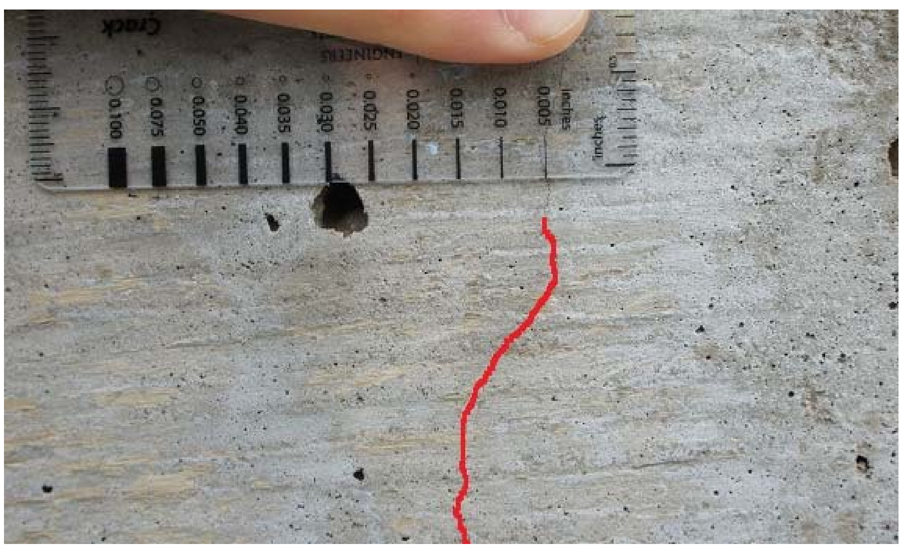


Figure 5-5. Deck slab crack width (the crack located in between beam 5 and 6 over the west abutment)

(a) Full-depth crack (b) Crack propagation along deck

Figure 5-6. Geometry of the crack located in between beam 5 and 6 over the west abutment

The second crack was documented above beam 2. It is located 14 inches towards beam 3 from the shear key between beams 1 and 2. The crack appeared slightly smaller, widthwise, than the other crack. This crack also had reached the full depth of the slab and had propagated approximately 2 inches along the deck (Figure 5-7 and Figure 5-8).

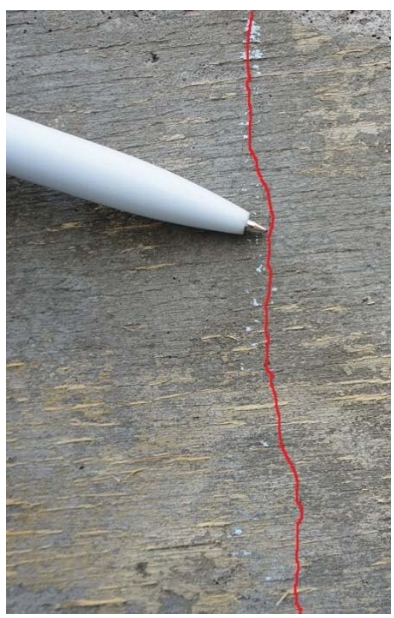


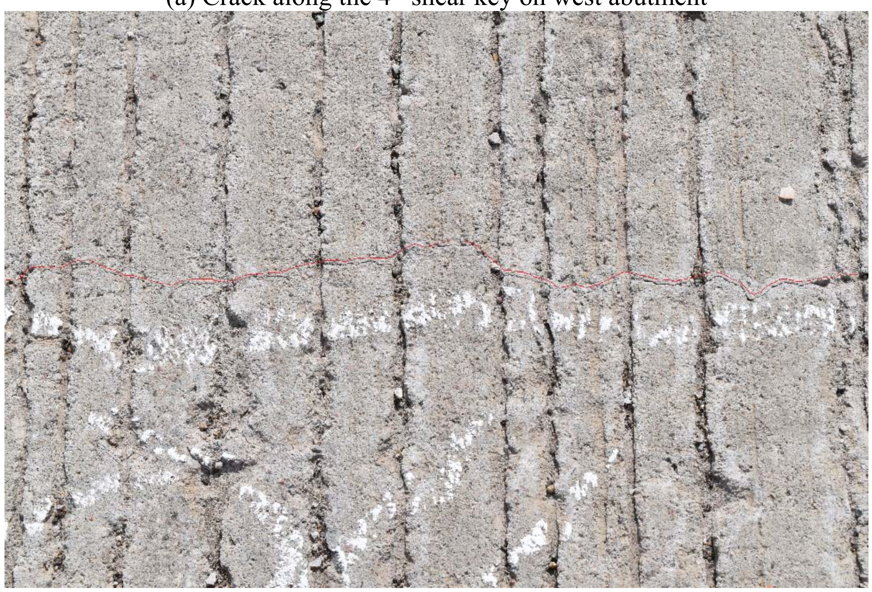
Figure 5-7. Crack over beam 2

(a) Full-depth deck slab crack (b) Crack propagation along deck

Figure 5-8. Geometry of the crack above beam 2 on the west abutment

A follow-up inspection was conducted on March 8, 2017 (five months after initial inspection). By March, the bridge had been subjected to several months of winter weather conditions. On the day of inspection, the ambient temperature was about 20 ^{0}F but feeling much colder with high winds. A detailed summary of inspection data is presented in Appendix G. Several cracks were documented over the shear keys. The length and width of the longest crack over the west abutment was about 28 inches and 0.02 in., and located along the 4th shear key (Figure 5-9). The length and width of the longest crack over the east abutment was about 60 inches and 0.01 in., and located along the 4th shear key (Figure 5-10).

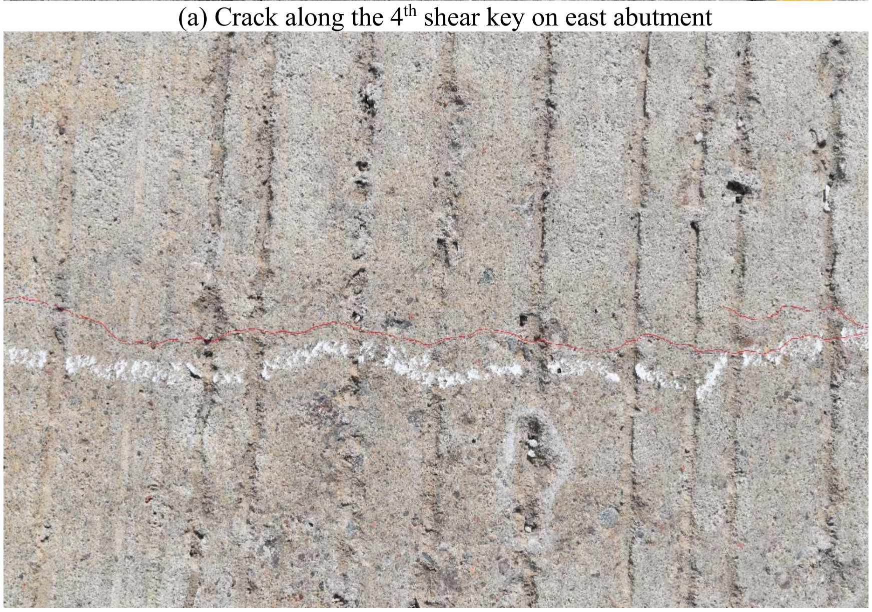
(a) Crack along the 4th shear key on west abutment



(b) A close up view of the crack

Figure 5-9. Longitudinal crack along the 4th shear key over west abutment

(a) Crack along the 4th shear key on east abutment



(b) A close up view of the crack

Figure 5-10. Longitudinal crack along the 4th shear key over east abutment

5.4 SHAW BROOK BRIDGE (B-14-216) AND PRATT CREEK BRIDGE (B-14-217)

Both projects started on July 5, 2016 and completed on September 17, 2016. These two structures represent the first adjacent box beam bridges with Geosynthetic Reinforced Soil (GRS) abutments in Wisconsin. Structural details are presented in Appendix F. As per the structural details, a majority of design and construction recommendations listed in section 5.2 of this report is implemented. As shown in Table 5-2, both bridges have comparable span lengths while the bridge B-14-217 has a skew of 15 degrees.

Table 5-2. Bridge Inventory and Design Data

Inventory and Design Data	Bridge ID	
	B-14-216	B-14-217
Length	43 ft	46 ft
Width	37 ft – 10.75 in.	33 ft – 9.5 in.
Skew	0	15 deg.
AADT (2015)	1300	440
Number of beams	9	8
Number of posttension locations along the span	6	6
Posttension force	12.1 kip/ft	11.3 kip/ft

Girders are supported on GRS abutments at different elevations to form the road crown. Each superstructure is transversely post-tensioned over the abutments and at four locations within the span. The specified posttensioning force is 86.7 kips per duct and applied at mid-depth of the beam through the diaphragms. Full-depth grouted shear keys are used. Shear keys are formed with BASF MasterFlow® 928 of which the properties are in compliance with the project special provisions. The technical data sheet of the grout is provided in Appendix F. Posttensioning was applied after the grout had cured for 2 days. According to the test data, the 3-day and 28-day strengths of grout are 4,745 psi and 6,515 psi, respectively. Grade E concrete overlay is used on both superstructures. Overlay specification, mix design, fresh and hardened concrete properties, and ambient temperature at the time of overlay placement are given in Table 5-3.

Table 5-3. Grade E Overlay Specifications, Mix Design, and QA/QC Data

Grade E Overlay Specifications		Mix Design	
Cement	823 lb	Cement	823 lb
Total Aggregate	2810 lb	Coarse Aggregate (1.3% Moisture)	1454 lb
Fine Aggregate	50% of total aggregate	Fine Aggregate (3.5% Moisture)	1423 lb
Water (Design)	32 gal	Coarse Aggregate Absorption	1.5%
Water (Maximum)	35 gal	Fine Aggregate Absorption	3.3%
Air Content	6% +/- 1%	Water Added to the Mix	242 lb
Slump	< 2 in.	Air Entrainment	8.75 oz
		Water Reducer	35 oz
		Water/Cement Ratio	0.29

Fresh and Hardened Concrete Properties and Ambient Temperature		
	B-14-216	B-14-217
Testing Date	08/30/2016	09/08/2016
Slump	1.4	1.25
Air Content	6%	5.8%
Concrete Temperature	78 ^{0}F	78 ^{0}F
28-day Strength	5567 psi	5815 psi
Ambient Temperature	80 ^{0}F	-

5.4.1 Bridge Inspection Summary

The research team did not inspect this bridge during construction or just after construction. The first inspection was conducted on March 9, 2017. By the time of inspection, both bridges had been subjected to several months of winter weather conditions. On the day of inspection, the ambient temperature was about 20 ^{0}F. A detailed summary of inspection data is presented in Appendix G.

5.4.1.1 Shaw Brook Bridge (B-14-216)

A general view of the bridge is shown in Figure 5-11. Several closely spaced cracks were documented over the abutments (Figure 5-12). The width of those cracks was about 0.01 in. A few cracks with the length ranging from 24 in. to 36 in. were documented over the shear keys. Since the scope of the project is to evaluate the bridge performance in terms of longitudinal deck cracking, the large number of hairline cracks that were observed after spaying water on the bridge deck was not documented. Observation of such hairline cracks is expected with the mix design used for the overlay. Low water levels at the bridge allowed inspecting beam soffits and the shear keys. As shown in Figure 5-13, no shear key grout leak was observed.

Figure 5-11. General view of the bridge

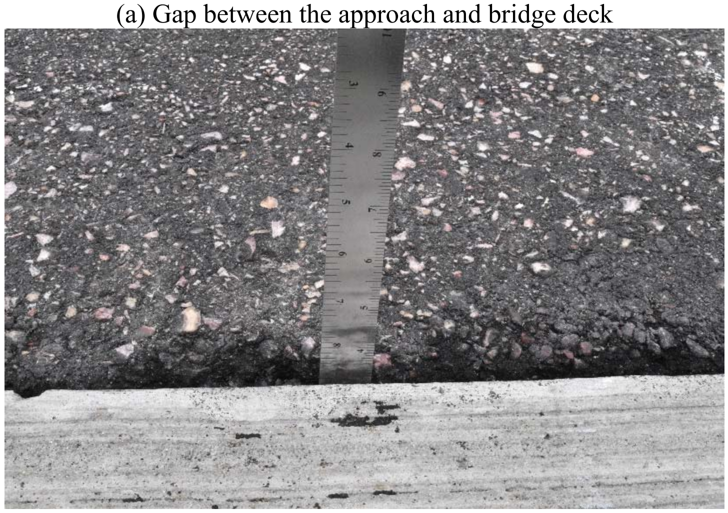
(a) Cracks over the west abutment (b) A close up view of a crack

Figure 5-12. Cracks over the west abutment

Figure 5-13. Self-adhesive compressible sealer at the shear key

The other observations include the opening at the joint between approach and bridge deck (Figure 5-14) and erosion of the backfill material (Figure 5-15). A gap of about 0.25 in. wide was recorded at the joint between the approach and the bridge deck. This is expected due to contraction of bridge superstructure under cold weather conditions.

(a) Gap between the approach and bridge deck



(b) Depth of the gap (3.5 in.) between the approach and bridge deck

Figure 5-14. Condition of the joint between the approach and bridge deck

(a) Erosion of backfill material near the fascia beam

(b) Backfill material underneath the fascia beam

(c) Erosion of backfill materal at other locations

Figure 5-15. Erosion of backfill material

5.4.1.2 Pratt Creek Bridge (B-14-217)

A general view of the bridge is shown in Figure 5-16. A 12 ft long, 0.013 in. wide crack was documented at the east abutment and over the 3rd shear key (Figure 5-17). In addition, several short cracks were documented at both abutments. Similar to B-14-216, erosion of backfill material was observed (Figure 5-18).

Figure 5-16. General view of the bridge

(a) A 12 ft long crack over the east abutment (b) A close up view of a crack

Figure 5-17. Cracks over the east abutment


(a) Erosion of backfill material near the fascia beam


(b) A close up view

Figure 5-18. Erosion of backfill material

5.5 STRUCTURAL SYSTEM RESPONSE TO SHRINKAGE AND THERMAL GRADIENT LOADS

The structural system response under shrinkage and thermal gradient loads was simulated using three dimensional finite element (FE) models. Two FE models were developed representing as built details of B-26-40 and B-14-216 structures. Simulation of concrete elasticity modulus and shrinkage development with respect to time required using user subroutines (USDFLD, UEXPAN, and UMAT) written in FORTRAN. The USDFLD subroutine serves two purposes. First, it is used to obtain instantaneous values of strain throughout the analysis. Second, it is used to change field variables with time. The instantaneous strain is called using the GETVRM command and placed as a state variable in the STATEV array for every element during each increment of the analysis. The UEXPAN subroutine is used to define incremental thermal strains (Abaqus 2013).

The strains can be a function of temperature, field variables, and/or state variables. By excluding thermal effects, the subroutine can be modified to model other forms of expansion and shrinkage (Kasera 2014). The UMAT subroutine was used to calculate the modulus of elasticity and stress at each time increment. Intel Composer XE 2013 SP1 was used to compile the subroutine. A right hand Cartesian coordinate system was used with a z-axis parallel to the bridge longitudinal axis and an x-axis parallel to the width of the bridge. Five shrinkage calculation models, listed in ACI 209.2R-08, were considered: ACI 209R-92, Bazant-Baweja B3, CEB MC90, CEB MC90-99, and GL2000. The CEB MC90-99 model calculates autogeneous as well as drying shrinkage; however, the other models only calculate drying shrinkage. These models were used to calculate the total shrinkage developed in the B-26-40 cast-in-place concrete deck slab. In order to account for lack of curing, a 3-day curing period was considered. Shrinkage was calculated up to 7 days, and presented in Figure 5-19. In addition, a curing period of 7 days was considered, and the total shrinkage up to 7 days was calculated. Since CEM MC90-99 is the only model that calculates autogeneous shrinkage, as shown in Figure 5-19 and Figure 5-20, it represents shrinkage development in the slab during the curing period. With a 3-day effective curing period, a total shrinkage of 45 microstrains develops in 7 days (Figure 5-19). With a 7-day effective curing period, a total shrinkage of 30 microstrains develops in 7 days (Figure 5-20). After evaluating all five models, the CEM MC90-99 model was implemented in FE models to simulate shrinkage.

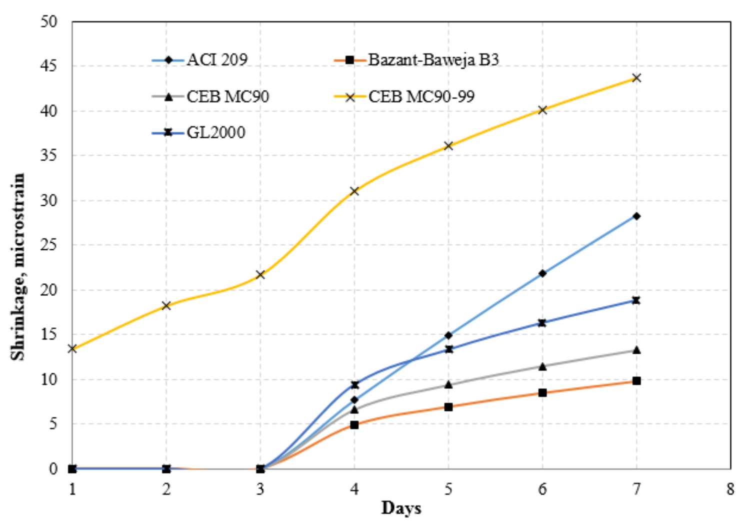


Figure 5-19. Total shrinkage developed in B-26-40 deck slab during a 7-day period with 3-day curing

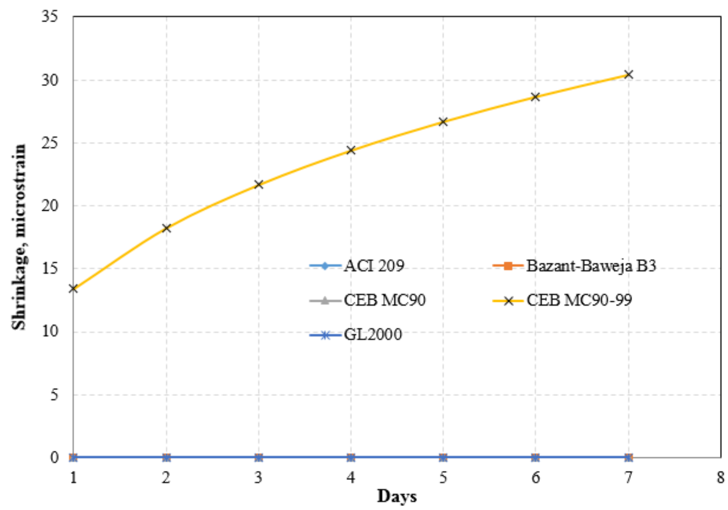


Figure 5-20. Total shrinkage developed in B-26-40 deck slab during a 7-day period with 7-day curing

In order to calculate the stresses developed in the deck as a result of drying and autogeneous shrinkage, the modulus of elasticity development of concrete in the deck slab needs to be modeled. To serve this purpose, the models presented in ACI 209.2R-08 were considered. The results are shown in Figure 5-21. ACI 209R-92 and Bazant-Baweja B3 models are similar. The CEB MC90-99 model gives the highest modulus values and is incorporated in the user subroutine material model (UMAT) in FORTRAN.

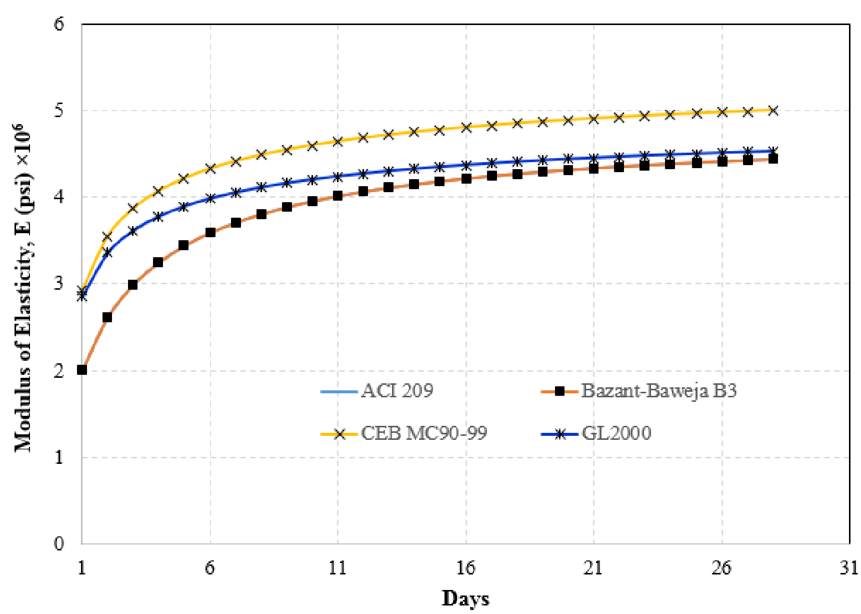
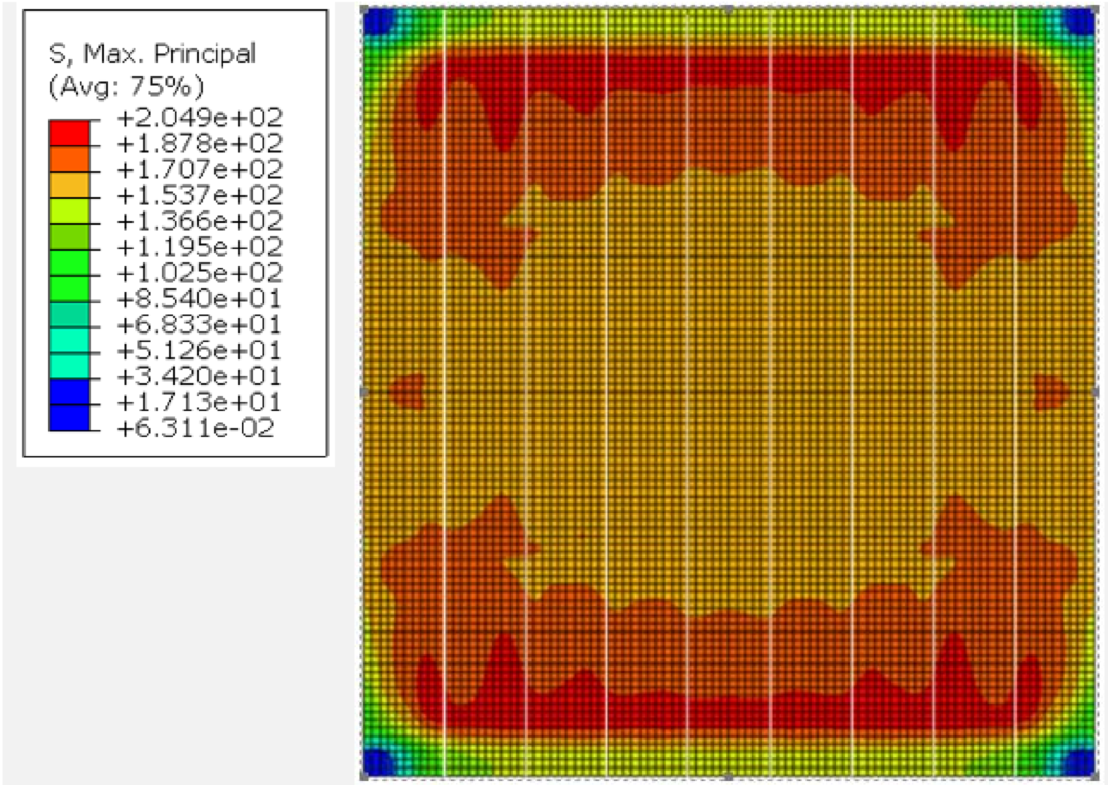


Figure 5-21. Development of modulus of elasticity with time

The maximum principal stress distribution under shrinkage at 7 days with 3-day moist curing of a 6 in. thick cast-in-place concrete slab is shown in Figure 5-22. In addition to concrete shrinkage, temperature gradient develops tensile stresses in the deck. Literature recommended fifth and second-order thermal profiles to represent the thermal gradient profile at noon and 6 p.m. during a summer day (Priestley 1978; Hedegaard et al. 2013). Due to lack of precise models and field data for adjacent box-beam bridges, these two profiles were used to evaluate the stresses developed in bridge B-26-40. The maximum principal stress developed in the deck slab with a 10 ^{0}F temperature difference between heating up and cooling down is shown in Figure 5-23. As shown in the figures, the maximum principal stress over the support is oriented in a direction perpendicular to the longitudinal axis (Figure 5-24). Hence, there is a potential to develop cracks parallel to the longitudinal axis of the bridge. These results support the field observations that the cracking starts over the supports. As shown in Figure 5-25, both top and bottom surfaces of the deck slab over the supports are under tension due to concrete shrinkage. Even though the stresses are less than the cracking strength of concrete at 7 days, concrete shrinkage will eventually develop into such values resulting in stresses that are greater than concrete cracking strength. As shown in Figure 5-23, temperature gradient can result in stresses that are greater than the cracking strength of concrete. Since the tensile stresses are developed at the top surface under the thermal gradient, most of the cracks will initiate from the top surface and propagate through the thickness with increased shrinkage. Also, as shown in Figure 5-25, principal stresses are greater in the proximity of the shear keys or the beam webs. *(Note that the shear key locations are marked with white lines.)* When a 2 in. thick overlay is used, the volume-to-surface ratio is smaller compared to a 6 in thick concrete slab of the same size. Hence, greater shrinkage values result. Bridge B-14-216 has a 2 in. thick overlay, and shrinkage simulation resulted in greater stresses than those observed in B-26-40 (Figure 5-26). When a 6 in. slab is used, the maximum principle stresses are developed over the supports and distributed across bridge width (Figure 5-22). With a 2 in. overlay, the maximum stresses are also developed over the supports, but concentrated around bridge centerline (Figure 5-26).

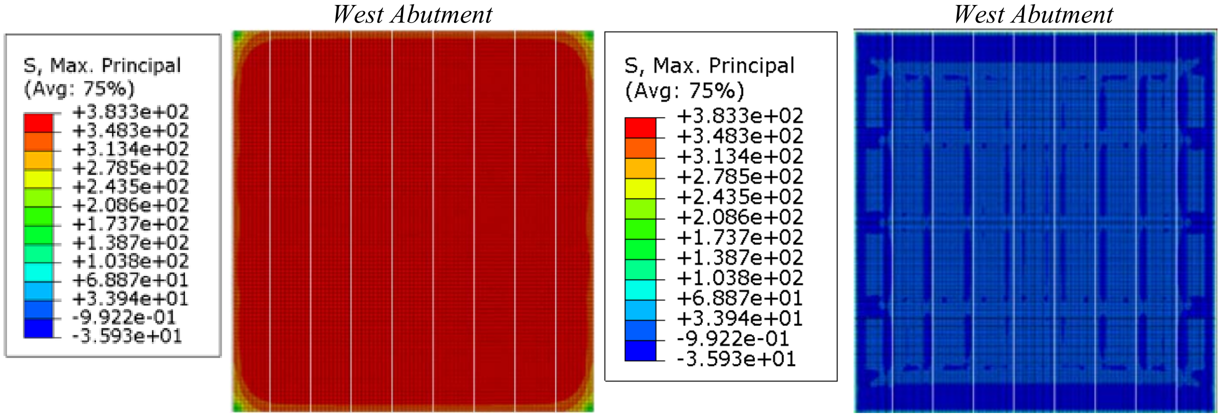
West Abutment



East Abutment

Figure 5-22. The maximum principal stress distribution under shrinkage at 7 days with 3-day moist curing (B-26-40)

West Abutment *West Abutment*

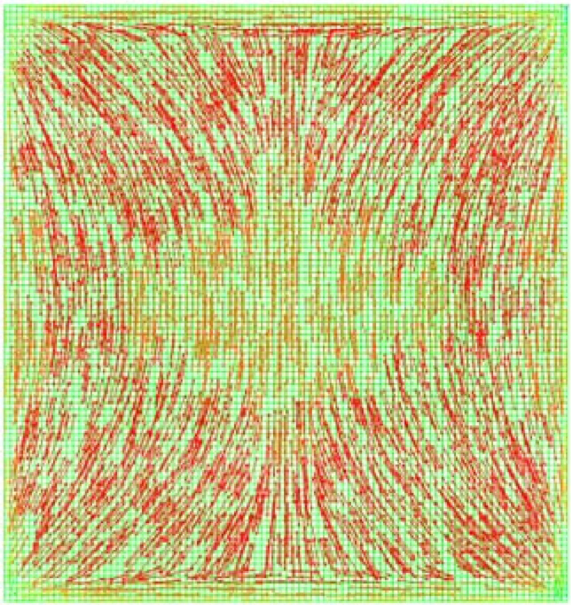


East Abutment *East Abutment*

Top Surface Bottom Surface

Figure 5-23. The maximum principal stress developed in the deck slab with 10 ^{0}F temperature difference between heating up and cooling down (B-26-40)

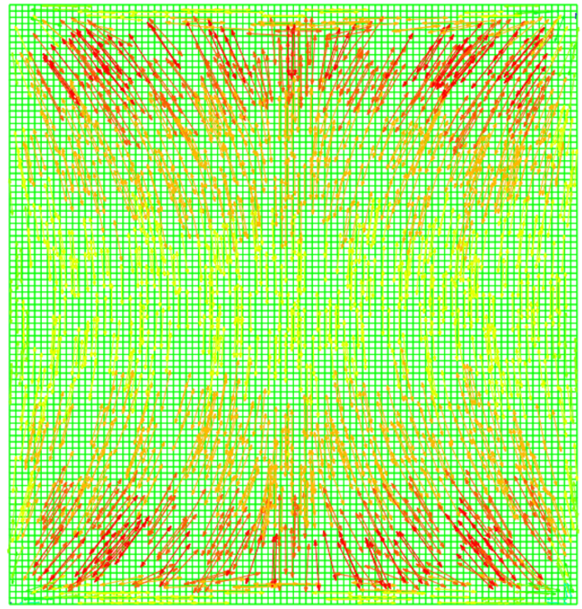
West Abutment



East Abutment

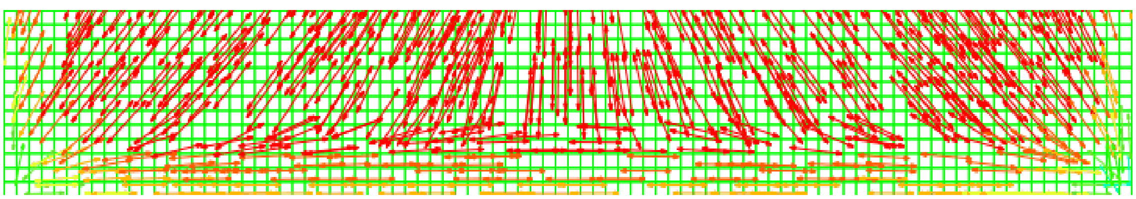
(a) The maximum principal stress due to thermal gradient

West Abutment

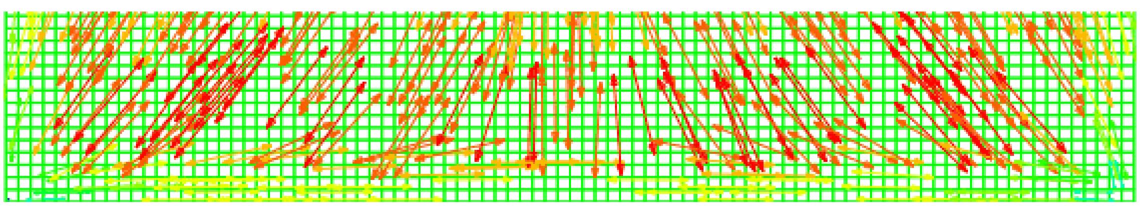


East Abutment

(b) The maximum principal stress due to shrinkage

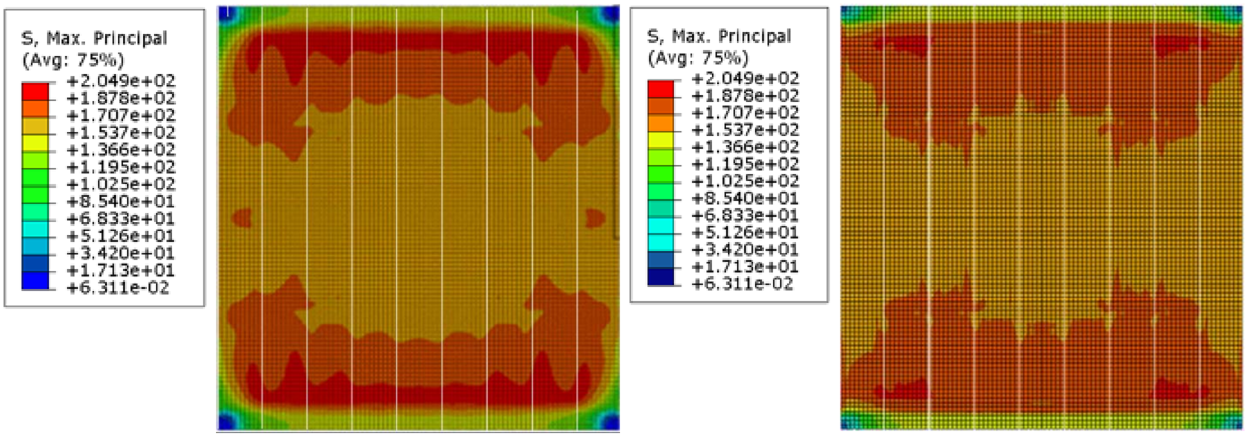


(c) A close up view of the maximum principal stress distribution over the east abutment due to thermal gradient



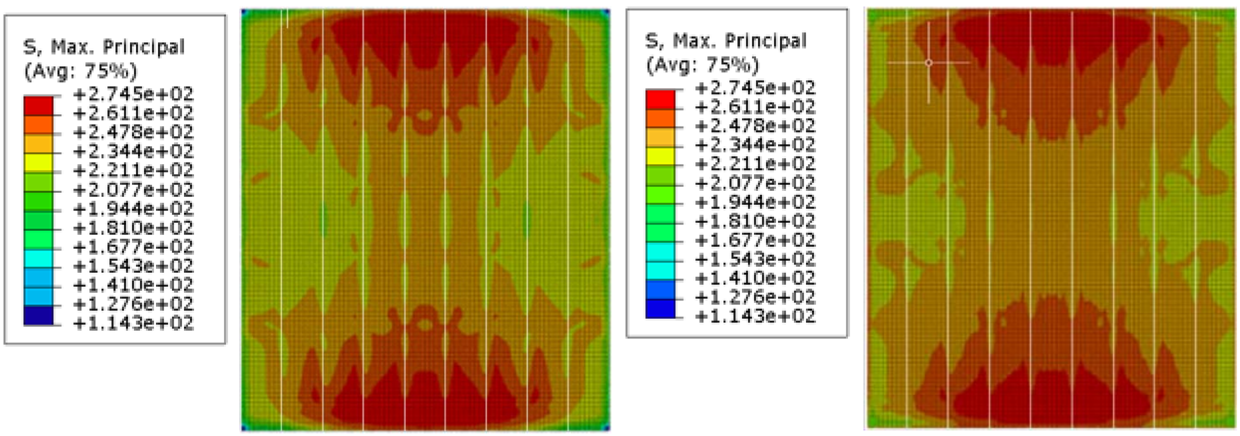
(d) A close up view of the maximum principal stress distribution over the east abutment due to shrinkage

Figure 5-24. Symbol plot showing the maximum principal stress distribution due to thermal gradient loading and shrinkage



Top Surface Bottom Surface

Figure 5-25. The maximum principal stresses at top and bottom surfaces of the deck slab of 7 days with 3 days curing (B-26-40)



Top Surface Bottom Surface

Figure 5-26. The maximum principal stresses at top and bottom surfaces of the deck slab of 7 days with 3 days curing (B-14-216)

Figure 5-27 shows the beam end and abutment details of bridge B-26-40. The beam end diaphragm length is 34 in. In addition, the 18 in. thick end diaphragm is either constructed integrally with the 6 in. thick concrete slab or constructed prior to pouring the slab. A parametric study was conducted to evaluate the impact of the having an end diaphragm and the casting sequence of the end diaphragm on the stresses developed in the deck slab due to concrete shrinkage. Based on the parametric analysis results, the following conclusions can be derived:

- End diaphragm provides additional constraints to the system and increases deck slab stresses compared to a bridge without an end diaphragm.
- Construction of an end diaphragm with the slab reduces deck slab stresses due to shrinkage of the slab and the end diaphragm. However, stresses in the deck with an end diaphragm are slightly greater than the stresses developed in a bridge without an end diaphragm.

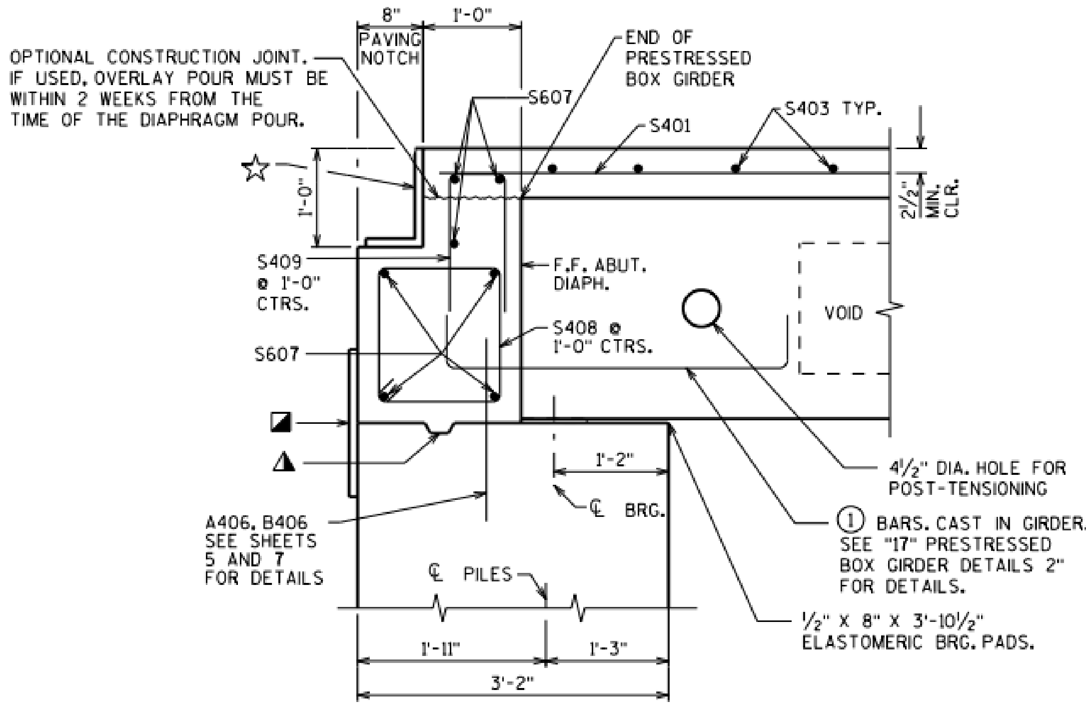


Figure 5-27. Bridge B-26-40 beam and end diaphragm detail

As shown in B-14-216 and B-14-217 bridge plans (see Appendix F for details), a bridge superstructure is supported on a bearing bed prepared using reinforced soil. Vertical stress and strain data presented in Adams et al. (2011) were used, and an equivalent modulus of 7000 psi was calculated. Poisson's ratio of 0.35 was assumed. With these parameters, the impact of reduction in vertical stiffness on deck stresses due to shrinkage was investigated. Even though the reduction in vertical support stiffness reduces deck stresses, the combined effect of shrinkage and thermal gradient load can develop stresses greater than the cracking strength of the overlay.

5.6 SUMMARY OF FINDINGS

Bridge B-26-40 was inspected just five days after pouring the deck and prior to construction of the approaches. Two cracks were documented over the west abutment. During this time, the deck slab was subjected to only barrier loads and volume change loads such as heat of hydration, shrinkage, and temperature gradient due to ambient weather. The early appearance of these cracks indicates that the cracking is initiated primarily due to the aforementioned loads. B-26-40 and two other bridges that were constructed with GRS abutments (B-14-216 and B-14-217) were inspected in March, 2017 (i.e., 5 months after construction). A limited number of long cracks were documented on all three bridge decks. Additionally, randomly dispersed short cracks over the beam ends were documented. On all three bridges, typical concrete mixes are used for the overlays. Apart from deck cracking, drainage problems and cracking (or opening) of the joint between the approach pavement and bridge superstructure were documented.

Refined finite element analysis was conducted by incorporating user subroutines to simulate concrete strength and shrinkage development with time. Further, the temperature gradient effect was also investigated. Based on the analysis results, the following conclusions can be derived:

- Concrete shrinkage develops tensile stresses through the thickness of an overlay and the greatest stresses are located over the supports of the superstructure.
- Thin concrete overlays develop greater stresses compared to thick overlays due to a smaller volume-to-surface ratio.
- Temperature gradient load develops tensile stresses at the top surface and over the entire deck area. A differential temperature of 10 ^{0}F between heating up and cooling down is enough to develop stresses greater than the concrete cracking strength.
- The combined effect of shrinkage and thermal gradient loads is adequate to develop full length longitudinal deck cracking.
- Due to lack of field data for developing temperature gradient profiles for adjacent box-beam bridges, the profiles derived for multi-cell box-beams were used. Hence, field verification of temperature profile and strain development in the bridge superstructure is necessary to validate the findings of this study and develop recommendations for reducing or eliminating longitudinal deck cracking.

6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY AND CONCLUSIONS

The adjacent precast concrete box-beam bridge is the bridge of choice for short and short-to-medium span bridges. This choice is because of the ease of construction, favorable span-to-depth ratios, aesthetic appeal, and high torsional stiffness. However, this bridge is losing favor primarily because of persisting durability performance issues such as longitudinal cracking at the shear key locations. This research project was initiated to identify modifications to the current precast prestressed adjacent box-beam bridge details, specifications, and methods used in Wisconsin with a goal of minimizing the potential for developing longitudinal deck cracking over the shear keys.

This project was conducted in two phases. During the first phase of the project, details and best practices documented in literature and implemented by various highway agencies for minimizing longitudinal deck cracking on adjacent box-beam bridges were reviewed and documented. A selected number of highway agencies were surveyed to document their adjacent box-beam bridge design, construction practices, and performance observations with the implementation of their most recent policies. Also, a representative sample of in-service box-beam bridges in Wisconsin was inspected. Based on the outcome of the aforementioned efforts, a list of best practices was developed, and revised details and specifications were presented. In 2016, WisDOT built three adjacent box-beam bridges and incorporated a limited number of recommendations. One of the bridges (B-26-40) was constructed on traditional abutments and incorporated a 6 in. thick cast-in-place concrete slab. The other two bridges (B-14-216 and B-14-217) were constructed on Geosynthetic Reinforced Soil (GRS) abutments and incorporated a 2 in. thick concrete overlay. The details and special provisions of these projects are presented in Appendix F.

The B-26-40 bridge deck was inspected on the 5th day following deck pour (i.e., during the 7-day moist curing period). Two cracks were identified over the west abutment. One of the cracks was 8 in. long and located above a shear key while the other crack was located above a box-beam. The inspection data shows that the cracking initiates well before the bridge is opened to traffic due to volume change loads. A follow-up inspection was conducted on March 08, 2017 (five months after construction and the bridge had been subjected to several months of winter weather conditions). Several cracks were documented over the shear keys. The length and width of the longest crack over the west abutment was about 28 in. and 0.02 in., respectively, and located along the 4th shear key. The length and width of the longest crack over the east abutment was about 60 in. and 0.01 in., respectively, and located along the 4th shear key. This observation shows that the negative temperature gradient, coupled with concrete shrinkage, could be the leading cause of cracking.

The B-14-216 and B-14-217 bridge decks were inspected on March 09, 2017. On the B-14-216 deck, a few cracks with lengths ranging from 24 in. to 36 in. were documented over the shear keys. These cracks had originated from the edge of the deck and propagated towards the mid span. A large number of hairline cracks were observed after spraying water on the bridge deck. Observation of such hairline cracks is expected with the mix design used for the overlay. Inspection of the beam soffit showed that the use of flexible foam formwork for shear key grouting has worked very well. Similar performance was observed with bridge B-14-217. However, an exceptionally long (12 ft long, 0.013 in. wide) crack was documented at the east abutment and over the 3rd shear key. Additional observations include some drainage issues at the GRS abutments, cracking at the approach pavement-bridge deck joint and, in the case of B-14-216, a significant impact from live load. Observations from these two bridges reveal that, irrespective of the bridge support types, longitudinal deck cracking persists.

Refined finite element analysis was conducted by incorporating user subroutines to simulate concrete strength and shrinkage development with time. Further, the temperature gradient effect was also investigated. Based on the analysis results, the following conclusions can be derived:

- Concrete shrinkage develops tensile stresses through the thickness of an overlay, and the greatest stresses are developed over the abutments.
- Thin concrete overlays develop greater stresses compared to thick overlays due to smaller volume-to-surface ratio.
- The temperature gradient load develops tensile stresses at the top surface and over the entire deck area. A differential temperature of 10 ^{0}F between heating up and cooling down can develop stresses greater than concrete cracking strength.
- The combined effect of shrinkage and thermal gradient loads is adequate to develop full-length longitudinal deck cracking.

- The end diaphragm provides additional constraints to the system and increases deck slab stresses compared to a bridge without an end diaphragm.
- Construction of the end diaphragm with the slab reduces deck slab stresses due to shrinkage of the slab as well as the end diaphragm. However, stresses in the deck with an end diaphragm are slightly greater than the stresses developed in a bridge without an end diaphragm.
- Use of GRS abutments reduces the vertical support stiffness; hence, the deck slab or overlay stresses. However, the combined effect of shrinkage and thermal gradient load can develop stresses greater than concrete cracking strength.

6.2 RECOMMENDATIONS

It is recommended to continue implementing the design and construction best practices presented in Chapter 5 of this report until additional research is conducted to address the following concerns:

- The mix designs used for the 6 in. thick cast-in-place concrete slab and 2 in. thick overlay include ordinary portland cement. These mixes are prone to cracking due to shrinkage. Since temperature gradient load also develops additional tensile stresses in the system, use of an overlay with a proven record of crack resistant properties is desired.
- An asphalt overlay with a waterproofing membrane has a better performance record compared to a concrete overlay or a slab. Hence, adoption of such a deck protection system or a multi-layer protection system, as practiced in Europe, needs to be considered.
- Due to lack of field data for developing temperature gradient profiles for adjacent box-beam bridges, the profiles derived for multi-cell box-beams were used. Hence, field verification of temperature profile and strain development in the bridge superstructure is necessary to validate the findings of this study and develop recommendations for reducing or eliminating longitudinal deck cracking.
- Two adjacent box-beam bridges were constructed with Geosynthetic Reinforced Soil (GRS) abutments. The vertical stiffness of GRS abutments is smaller compared to the stiffness of traditional abutments. This lower stiffness helps reduce the stresses developed in the superstructure due to volume change loads. However, the combined effect of shrinkage and thermal gradient loads can still develop stresses in excess of concrete cracking strength. Therefore, the above stated recommendations, related to deck protective

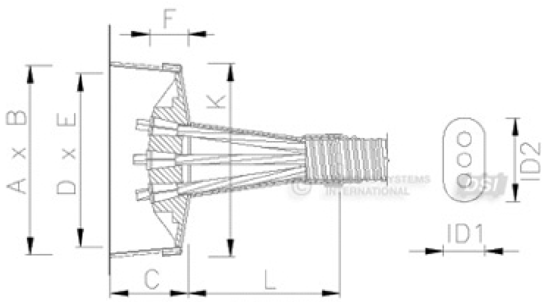
systems, need to be considered for improving bridge durability. In addition to deck cracking, drainage problems, and cracking at approach pavement and deck connection were documented during field inspection. Hence, implementation of an approach slab with a sleeper slab or a water proofing membrane system, as practiced by the Alberta Ministry of Transportation, is recommended.

- When GRS abutments are used, the current practice is to support the girders directly on reinforced soil pads. Significant live load impacts were noticed during the inspection of bridge B-14-216. If differential settlement occurs, an adjacent box-beam bridge superstructure with a single layer of post-tensioning strands through the depth is not adequately detailed to carry transverse moments. Hence, it is recommended to evaluate the use of concrete pads to control potential impacts due to differential settlement.
- It is possible to use 1.5 in. by 3 in. wide flat anchorage duct with unbounded, lubricated, encased strands through top and bottom flanges of the beam. Figure 6-1 shows the technical details of the post-tensioning system. Figure 6-2 shows the arrangement of longitudinal and transverse reinforcement with a flat anchorage duct. The changes can be accommodated by reducing the height of void from 7 in. to 6 in. and increasing the height of top and bottom flanges from 5 in. to 5.5 in. Top reinforcement has deeper cover than the cover required for durability. Alternative details (such as having stirrup and longitudinal steel at the typical location in between the ducts) can be investigated. The current exterior girder provides adequate space for anchorage as shown in Figure 6-3. The duct spacing is limited to the smaller of 4 ft or beam width.

  Post-tensioning force of 175 to 234 kips can be applied. This unbonded system allows girder and deck replacement. As needed, post-tensioning can be applied in two stage with this system to compress the cast-in-place concrete slab to mitigate tensile stresses due to volume change loads. The temperature effect on strands will be minimum due to the presence of a plastic duct, air gap between the duct and strands, and sheathing on the strands, which will be beneficial for maintaining a consistent force in strands during daily heating and cooling cycles of the superstructure. Thus, this system will allow maintaining deck slab in compression to mitigate the tensile stresses develop in the deck slab due to temperature gradient loading. Post-tensioning force magnitudes with two-stage post-tension, cost, stresses at anchor zone, and other design and maintenance requirements need to be further researched.

(a) Flat anchorage system

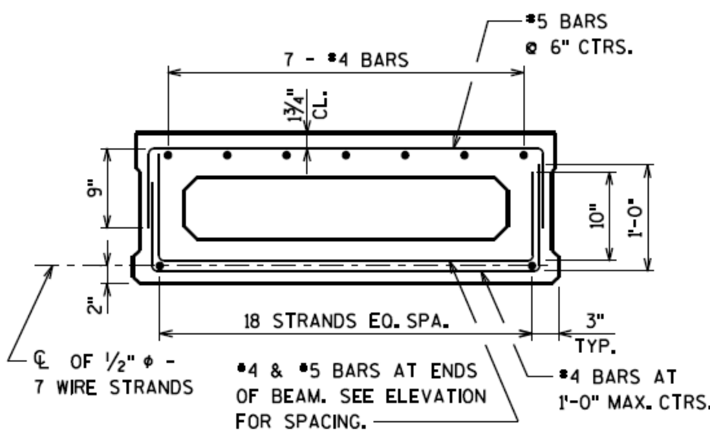
(b) Anchorage with 4 - 0.6 in. strands



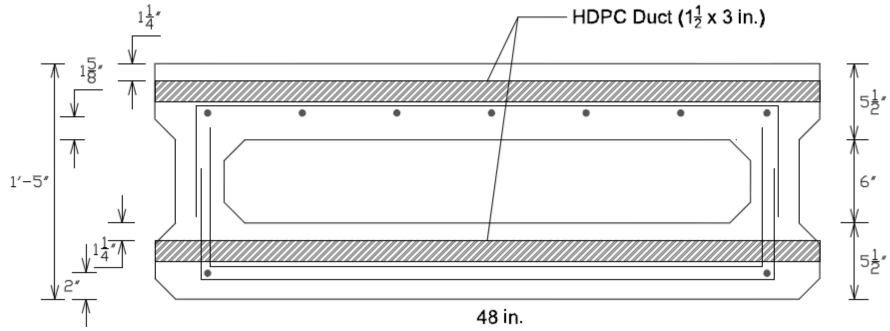
		Tendon configuration	
		3-0.6 in. or 4-0.6 in.	4-0.6 in. or 5-0.5 in.
Dimensions	A (in.)	10.75	13.75
	B (in.)	4.5	4.875
	C (in.)	5.5	5.875
	D (in.)	10	13
	E (in.)	4	4
	F (in.)	2.25	2.25
	ID1 (in.)	1	1
	ID2 (in.)	3	3
	K (in.)	12.25	--
	L (in.)	4.5	8.625

c) Technical detail

Figure 6-1. Flat anchorage geometry and technical detail

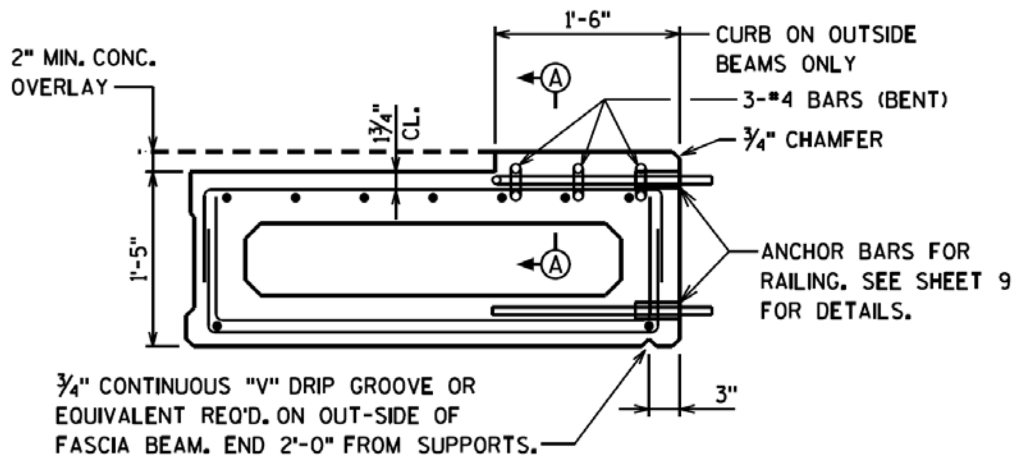


a) Current beam details

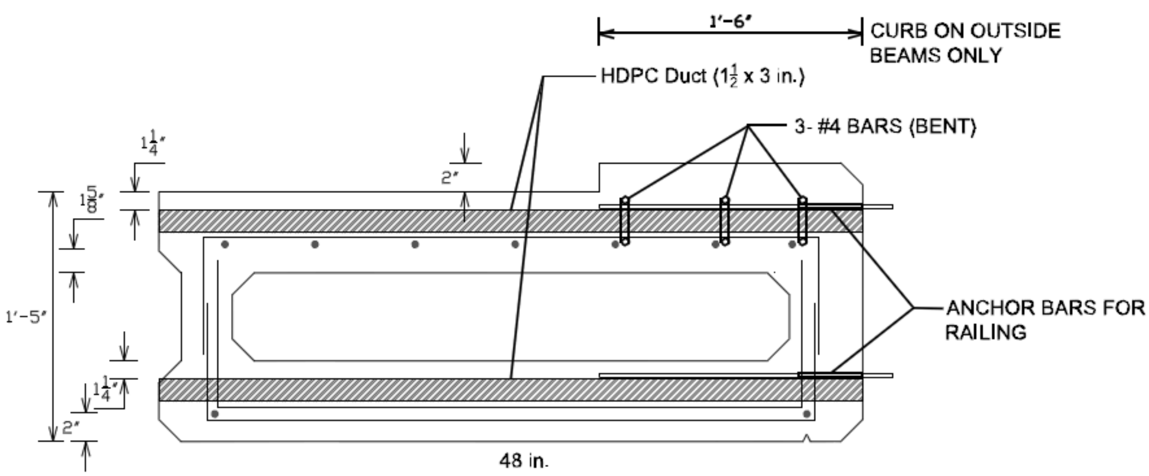


b) Proposed changes to beam details with ducts

Figure 6-2. Interior beam with and without transverse post-tensioning ducts



a) Current beam details



b) Proposed changes to beam details with ducts

Figure 6-3. Exterior beam with and without transverse post-tensioning ducts

7 BIBLIOGRAPHY

1. AASHTO LRFD (2013). Bridge Design Specifications. American Association of State Highway and Transportation Officials, 444 North Capitol Street, NW, Suite 249, Washington, DC 20001.
2. Abaqus (2013). Abaqus Documentation. Dassault Systèmes Americas Corp., 175 Wyman Street, Waltham, MA 02451
3. ACI 209.2R-08 (2008). Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete. American Concrete Institute, Farmington Hills, MI 48331.
4. Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W. and Hartmann, J. (2011). *Geosynthetic Reinforced Soil Integrated Bridge System,Synthesis Report,* FHWA-HRT-11-027, Federal Highway Administration, McLean, VA.
5. Alberta MOT (2017). Low Volume Standard Bridge GRS Abutment Alternative Typical Details. The Ministry of Transportation, Alberta, Canada. http://www.transportation.alberta.ca/Content/docType30/Production/low_volume_bridge_Geosynthetic_Reinforced_Soil_GRS_foundation_typical_details.pdf (Last accessed: 03/26/2017)
6. Aktan, H., Attanayake, U., Ulku, E., Ahlborn, T. M., and Deshpande, Y. (2009). "Condition Assessment and Methods of Abatement of Prestressed Concrete Box-Beam Deterioration – Phase II," *MDOT RC-1527*, Michigan Department of Transportation, Detroit, MI 48226.
7. Aktan, H., Ahlborn, T. M., Attanayake, U., and Gilbertson, C. G. (2005). "Condition Assessment and Methods of Abatement of Prestressed Concrete Box-Beam Deterioration – Phase I," *MDOT RC-1470*, Michigan Department of Transportation, Lansing, MI.
8. Attanayake, U. B. and Aktan, H. (2013). "The First Generation ABC System, Evolving Design, and Half a Century Performance– Michigan Side-by-Side Box-Beam Bridge," ASCE Journal of Performance of Constructed Facilities, Vol. 29(3), pp. 1-14.
9. Attanayake, U. and Aktan, H. M. (2009). "Side-by-side Box-beam Bridge Superstructure: Rational Transverse Posttension Design," 88th *TRB Annual Meeting, No. 09-3420,* January 11-15, 2009.
10. Attanayake, U., Aktan, H.M., and Ng, S. (2002). "Criteria and Benefits of Penetrating Sealants for Concrete Bridge Decks," MDOT RC-1424, CSD 2002-03, Report to the Michigan Department of Transportation, Lansing, MI.

11. Battaglia, I., Whited, G., and Swank, R. (2008). "Eclipse® Shrinkage Reducing Admixture Product Evaluation." *FEP-01-08*, Wisconsin Department of Transportation, WI 53704.
12. Chaunsali, P., Li, S., Mondal, P., Foutch, D., Richardson, D., Tung, Y., and Hindi, R. (2013). "Bridge Decks: Mitigation of Cracking and Increased Durability." *FHWA-ICT-13-023*, Illinois Department of Transportation, Springfield, IL 62704.
13. Culmo, M.P. (2009). *Connection Details for Prefabricated Bridge Elements and Systems,* Technical Report FHWA-IF-09-010, Federal Highway Administration, McLean, VA.
14. Cusens, A. R. (1974). "Load Distribution in Concrete Bridge Decks." *CIRIA Report 53,* Construction Industry Research and Information Association, 6 Storey's Gate, Westminster, London SW1P 3AU.
15. Dong, X. (2002). "Traffic forces and temperature effects on shear key connection for adjacent box girder bridge." *Ph.D Dissertation*, University of Cincinnati, Ohio.
16. FHWA (2011). Geosynthetic Reinforced Soil Integrated Bridge System, Synthesis Report, FHWA-HRT-11-027, Office of Infrastructure Research and Development, the Federal Highway Administration, McLean, VA 22181.
17. FHWA (1995). Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, FHWA-PD-96-001, Office of Engineering, Bridge Division, the Federal Highway Administration, McLean, VA 22181.
18. French, C.E., Shield, C.K., Klaseus, D., Smith, M., Eriksson, W., Ma, J.Z., Zhu, P., Lewis, S., and Chapman, C.E. (2011). Cast-in-Place Concrete Connections for Precast Deck Systems, Contractor's Final Report, NCHRP Project 10-71, Transportation Research Board of the National Academies, Washington, D.C.
19. Freyne, S., Ramseyer, C., and Giebler, J. (2012). "High-Performance Concrete Designed to Enhance Durability of Bridge Decks: Oklahoma Experience." *Journal of Materials in Civil Engineering,* Vol. 24, No. 7, pp. 933-936.
20. Grace, N., Jensen, E., and Bebawy, M. R. (2012). "Transverse post-tensioning arrangement for side-by-side box-beam bridges." *PCI Journal*, Spring 2012, pp. 48-63.
21. Grace, N. and Jensen, E. (2008). "Use of Unbonded CFCC for Transverse Post-Tensioning of Side-by-Side Box-Beam Bridges," MDOT RC-1509, Report to the Michigan Department of Transportation, Lansing, MI 48909.

22. Gulyas, R. J., Wirthlin, G. J., and Champa, J. T. (1995). "Evaluation of keyway grout test methods for precast concrete bridges." *PCI Journal*, Vol. 40, No. 1, pp. 44-57.
23. Hanna, K. E., Morcous, G., and Tadros, M. K. (2009). "Transverse post-tensioning design and detailing of precast, prestressed concrete adjacentbox- girder bridges." *PCI Journal*, Fall 2009, pp. 160-174.
24. Hansen, J. (2012). Transversely Post-Tensioned Box-Beam System for Precast Parking Structures. MS thesis, University of Nebraska-Lincoln.
25. Harries, K. A. (2006). "Full-scale Testing Program on De-commissioned Girders from the LakeView Drive Bridge." FHWA-PA-2006-008-EMG001, University of Pittsburgh, Department of Civil and Environmental Engineering, 949 Benedum Hall, Pittsburgh, PA.
26. Hedegaard, B. D, French, C. E. W., and Shield, C. K. (2013). "Investigation of Thermal Gradient Effects in the I-35W St. Anthony Falls Bridge." The *Journal of Bridge Engineering*, Vol. 18, No. 9, pp. 890–900.
27. Hoomes, L. C., Ozyildirim, C., and Brown, M. C. (2014). "Evaluation of high performance fiber-reinforced concrete for bridge deck connections, closure pours, and joints." 93rd Annual Meeting, Transportation Research Board, Washington, D.C.
28. Huckelbridge, A. A. Jr., El-Esnawi, H., Moses, F. (1995). "Shear Key Performance in Multibeam Box Girder Bridges." *Journal of Performance of Constructed Facilities,* Vol. 9, No. 4, pp. 271-285.
29. Issa, M.A., Ribeiro do Valle, C.L., Islam, S., Abdalla, H.A., and Issa, M.A. (2003). "Performance of Transverse Joint Grout Materials in Full-Depth Precast Concrete Bridge Deck Systems." PCI Journal, July-August, 2-13.
30. Jones, L. (2017). Geosynthetic Reinforced Soil Integrated Bridge System (GBS-IBS) – Concepts and Background. http://www.fdot.gov/design/training/designexpo/2012/Presentations/Jones.Larry.GRS-IBS.pdf (Last accessed: March 30, 2017)
31. Kasera, S.C. (2014) "Simulation of the effect of deck cracking due to creep and shrinkage in single span precast/prestressed concrete bridges." *Master's thesis,* University of Cincinnati, 2600 Clifton Ave, Cincinnati, OH 45220.

32. Kepler, J. L., Darwin, D., and Locke, C. E. (2000). *Evaluation of corrosion protection methods for reinforced concrete highway structures.* Lawrence: The Kansas Department of Transportation.

33. Kim, J. J., Nam, J. W., Kim, H. J., Kim, J. H., Kim, S. B., and Byun, K. J. (2008). "Overview and applications of precast, prestressed concrete adjacent box-beam bridges in South Korea." *PCI Journal*, July – August 2008, pp. 83-107.

34. Krauss, P. D., Lawler, J. S., & Steiner, K. A. (2009). *Guidelines For Selection Of Bridge Deck Overlays, Sealers and Treatments.* NCHRP Project 20-07 Task 234, National Cooperative Highway Research Program, Transportation Research Board.

35. Lai, D. (2008). "The Evolution of Bridge Deck Rehabilitation Strategies in Ontario," the 2008 Annual Conference of the Transportation Association of Canada, Toronto, Ontario.

36. Lall, J., Alampalli, S., and DiCocco, E. F. (1998). "Performance of Full-Depth Shear Keys in Adjacent Prestressed Box Beam Bridges." *PCI Journal*, Vol. 43, No. 2, pp. 72-79.

37. Liang, Y.-c., Zhang, W., and Xi, Y. (2010). *Strategic Evaluation of Different Topical Protection Systems for Bridge Decks and the Associated Life-Cycle Cost Analysis.* Colorado: Colorado Dept. of Transportation

38. MAINROADS, W. A. (2007). *Guidelines for the application of waterproof membranes to bridge decks.* Australia: Government of Western Australia.

39. Miller, R., Hlavacs, G. M., Long, T., and Greuel, A. (1999). "Full-Scale Testing of Shear Keys for Adjacent Box Girder Bridges." *PCI Journal*, Vol. 44, No. 6, pp. 80-90.

40. NBI (2014). The National Bridge Inventory. https://www.fhwa.dot.gov/bridge/nbi/ascii.cfm (Last Accessed: February 6, 2016).

41. NYSDOT (2015). Guidelines for Design and Construction of Geosynthetic Reinforced Soil Integrated Bridge System. The New York State Department of Transportation, Albany, NY.

42. Oesterle, R. G. and El-Remaily, A. F. (2009). Guidelines for Design and Construction of Decked Precast, Prestressed Concrete Girder Bridges, Final Report, NCHRP Project No. 12-69, Transportation Research Board, National Research Council, Washington, D.C.

43. PCI (2011). The State of the Art of Precast/Prestressed Adjacent Box Beam Bridges. Precast/Prestressed Concrete Institute, 209 West Jackson Boulevard, Chicago, IL 60606.

44. Perry, V., Dykstra, D., Murray, P., and Rajlic, B. (2010). "Innovative Field Cast UHPC Joints for Precast Bridge Systems - 3-span Live Load Continuous." Proc., Bridges -

Adjusting to New Realities (A) Session, Transportation Association of Canada, Halifax, Nova Scota.

45. Priestley, M.J.N. (1978). “Design of thermal gradients for concrete bridges,” New Zealand Engineering, 31 (9), 213-19.

46. Phares, B., Rouse, J., and Miksell, J. (2013). *Laboratory and Field Testing of an Accelerated Bridge Construction Demonstration Bridge: US Highway 6 Bridge over Keg Creek,* Report. InTrans Project 11-411. FHWA and Iowa Department of Transportation Ames, IA.

47. Price, A. R. (1989). *A field Trial Of Waterproof Systems For Concrete Bridge Decks.* Bridges Division Structure Group Trasnport and Road Research Laboratory.

48. Ramey, G.E., Pittman, D.W., Webster, G., and Carden, A. (1997). Use of Shrinkage Compensating Cement in Bridge Decks. Auburn University, Department of Civil Engineering, Auburn, AL.

49. Royce, M. (2016). “Utilization of Ultra-High Performance Concrete (UHPC) in New York.” Proceedings of the First International Interactive Symposium on UHPC, July 18-20, Des Moines, Iowa.

50. Russell, H. G. (2009). Adjacent Precast Concrete Box Beam Bridges: Connection Details. NCHRP Synthesis 393, Transportation Research Board, Washington D.C.

51. Russell, H. G. (2012). Waterproofing Membranes for Concrete Bridge Decks. NCHRP Synthesis 425, Transportation Research Board, Washington D.C.

52. Sang, Z. (2010). “A Numerical Analysis of the Shear Key Cracking Problem in Adjacent Box Beam Bridges.” *Master Thesis*, the Pennsylvania State University.

53. Scholz, D. P., Wallenfelsz, J. A., Lijeron, C., and Roberts-Wollmann, C. L. (2007). Recommendations for the Connection Between Full-Depth Precast Bridge Deck Panel Systems and Precast I-Beams, Final Contract Report, FHWA/VTRC 07-CR17, Virginia Transportation Research Council, Charlottesville, VA.

54. Sharpe, G. P. (2007). “Reflective cracking of shear keys in multi-beam bridges.” *Master Thesis*, Texas A & M University.

55. Storm, T. K., Rizkalla, S. H., and Zia, P. Z. (2013). “Effects of production practices on camber of prestressed concrete bridge girders.” *PCI Journal*, Winter 2013, pp. 96-111.

56. Tadros, M. K., Hanna, K., Jaber, N., and Hansen, J. (2017). Transversely posstentioned, pretopped box-slab system for precast concrete parking structures. *PCI Journal*, Vol. 62, No. 2, March-April.
57. Troli, R., and Collepardi, M. (2011). "Shrinkage-compensating concrete for special structures." Proceedings of 4th International Conference on "Non-Traditional Cement & Concrete", Brno, Czech Republic 27-30 June.
58. Ulku, E., Attanayake, U., and Aktan, H. M. (2010). "Rationally Designed Staged Posttension Abates Reflective Cracking on Side-by-Side Box-Beam Bridge Decks," *Transportation Research Record: Journal of the Transportation Research Board*, No. 10-1153, Transportation Research Board of the National Academies, Washington, D.C. pp. 87-95.
59. Yamane, T., Tadros, M. K., and Arumugasamy, P. (1994). "Short to Medium Span Precast Prestressed Concrete Bridges in Japan." *PCI Journal*, Vol. 39, No. 2, pp. 74-100.
60. Yuan, A., Qian, S., He, Y. and Zhu, X. W. (2013). "Capacity Evaluation of a Prestressed Concrete Adjacent Box Girder with Longitudinal Cracks in the Web." ASCE Journal of Performance of Constructed Facilities, Vol. 29, No. 1, pp. 1-9.

DOT DOCUMENTS

1. CONNECTICUT BRIDGE DESING MANUAL (2003 EDITION) http://www.ct.gov/dot/lib/dot/documents/dpublications/bridge/bdm.pdf (Last Accessed: January 9, 2014).
2. DELAWARE BRIDGE DESIGN MANUAL (MAY 2005) http://www.deldot.gov/information/pubs_forms/manuals/bridge_design/ (Last Accessed: January 9, 2014).
3. ILLINOIS DESIGN GUIDES http://www.dot.il.gov/bridges/Design%20Guides/Design_Guides_Web.pdf (Last Accessed: January 9, 2014).
4. INDIANA DESIGN MANUAL 2013 http://www.in.gov/indot/design_manual/design_manual_2013.htm (Last Accessed: January 9, 2014).
5. INDIANA STANDARD DOCUMENT http://www.in.gov/dot/div/contracts/standards/dm-Archived/07%20Metric/Part%206/Ch%2063/Ch63.pdf (Last Accessed: January 9, 2014).

6. KENTUCKY STRUCTURAL DESING MANUAL http://transportation.ky.gov/Structural-Design/Manuals/Chapters.pdf (Last Accessed: January 9, 2014).
7. KENTUCKY STANDARD SPECIFICATION (STRUCTURES AND CONCRETE 2012) http://transportation.ky.gov/Construction/Standard%20amd%20Supplemental%20Specifications/600%20Structures%20and%20Concrete%2012.pdf (Last Accessed: January 9, 2014).
8. KENTUCKY STANDARD DRAWINGS (2012) http://transportation.ky.gov/Highway-Design/Pages/2012-Standard-Drawings.aspx (Last Accessed: January 9, 2014).
9. MASSACHUSETTS LRFD BRIDGE MANUAL PART I (2017) http://www.massdot.state.ma.us/highway/DoingBusinessWithUs/ManualsPublicationsForms/LRFDBridgeManual2013Edition.aspx (Last Accessed: January 9, 2017).
10. MICHIGAN STANDARD SPECIFICATION FOR CONSTRUCTION http://mdotcf.state.mi.us/public/specbook/2012/ (Last Accessed: January 9, 2014).
11. MICHIGAN BRIDGE DESIGN GUIDE http://mdotcf.state.mi.us/public/design/englishbridgeguides/ (Last Accessed: January 9, 2014).
12. MICHIGAN BRIDGE DESIGN MANUAL http://mdotcf.state.mi.us/public/design/englishbridgemanual/ / (Last Accessed: January 9, 2014).
13. MISSOURI LFD AND LRFD BRIDGE DESIGN GUIDELINES http://www.modot.org/business/manuals/bridgedesign.htm (Last Accessed: January 9, 2014).
14. MISSOURI BRIDGE STANDARD DRAWINGS http://www.modot.org/business/consultant_resources/bridgestandards.htm (Last Accessed: January 9, 2014).
15. NEW HAMPSHIRE BRIDGE DESIGN MANUAL (OCT. 2000) http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/documents.htm (Last Accessed: January 9, 2014).
16. NEW JERSEY DESIGN MANUAL BRIDGES AND STRUCTURES (5^{TH} EDITION) http://www.state.nj.us/transportation/eng/documents/BSDM/ (Last Accessed: January 9, 2014).

17. NYSDOT CEMENT BASED GROUT MATERIALS FOR SHEAR KEYS (701-06) https://www.dot.ny.gov/divisions/engineering/technical-services/technical-services-repository/alme/pages/230-1.html (Last Accessed: September 18, 2016).
18. OHIO BRIDGE DESIGN MANUAL (2007) http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/BDM/BDM2007_10-18-13.pdf (Last Accessed: January 9, 2014).
19. PENNSYLVANIA DESIGN MANUAL PART 4- STRUCTURES (MAY 2012 EDITION) ftp://ftp.dot.state.pa.us/public/Bureaus/design/PUB15M/2012_Edition/DM-4_May_2012.pdf (Last Accessed: January 9, 2014).
20. PENNSYLVANIA ARCHIVED BRIDGE DESIGN STANDARDS http://www.dot.state.pa.us/Internet/BQADStandards.nsf/bd-archives?readform (Last Accessed: January 9, 2014).
21. RHODE ISLAND LRFD BRIDGE DESIGN MANUAL (2007 EDITION) http://www.dot.ri.gov/documents/engineering/br/RILRFDBridgeManual.pdf (Last Accessed: January 9, 2014).
22. RHODE ISLAND BRIDGE DESIGN STANDARD DETAILS (2009) http://www.dot.ri.gov/documents/engineering/BlueBook/BRG_STD_PDF_COMB_1-12-09.pdf (Last Accessed: January 9, 2014).
23. VIRGINIA PRESTRESSED CONCRETE BEAM STANDARDS (2013) http://www.extranet.vdot.state.va.us/locdes/electronic%20pubs/Bridge%20Manuals/VolumeV-Part4/VolumeV-Part4.pdf (Last Accessed: January 9, 2014).
24. VIRGINIA STRUCTURE AND BRIDGE MANUALS (2013) http://www.virginiadot.org/business/bridge-manuals.asp (Last Accessed: January 9, 2013).
25. VERMONT STRUCTURES DESIGN MANUAL (2010) http://vtransengineering.vermont.gov/sites/aot_program_development/files/documents/structures/Structures_Design_Manual_0.pdf (Last Accessed: January 9, 2013)