Transverse post-tensioning design and detailing of precast, prestressed concrete adjacent-box-girder bridges

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Editor's quick points

- This paper presents a review of various transverse design and detailing practices for adjacent-box-girder bridges.
- Design charts were developed for various combinations of span length, bridge width, skew angle, and girder depth using the latest loading from *AASHTO LRFD Bridge Design Specifications* to update the information in section 8.9 of the PCI *Precast Prestressed Concrete Bridge Design Manual*, which was based on an earlier version of the AASHTO standard specifications.
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Precast concrete adjacent-box-girder bridges are the most prevalent box-girder system for short- and medium-span bridges (which typically span from 20 ft to 127 ft [6.1 m to 38.7 m]), especially on secondary roadways. These bridges consist of multiple precast concrete box girders that are butted against each other to form the bridge deck and superstructure.

There is new interest in using these bridges for rapid construction under the Federal Highway Administration (FHWA) Highways for Life program. Adjacent box girders are generally connected using partial- or full-depth grouted shear keys along the sides of each box. Transverse ties are usually used in addition to the grouted shear keys, and they may vary from a limited number of threaded rods to several post-tensioned tendons. In some cases, no topping is applied to the structure, while in other cases a noncomposite topping or a composite structural slab is added.

Bridges built with adjacent precast, prestressed concrete box girders have several advantages:

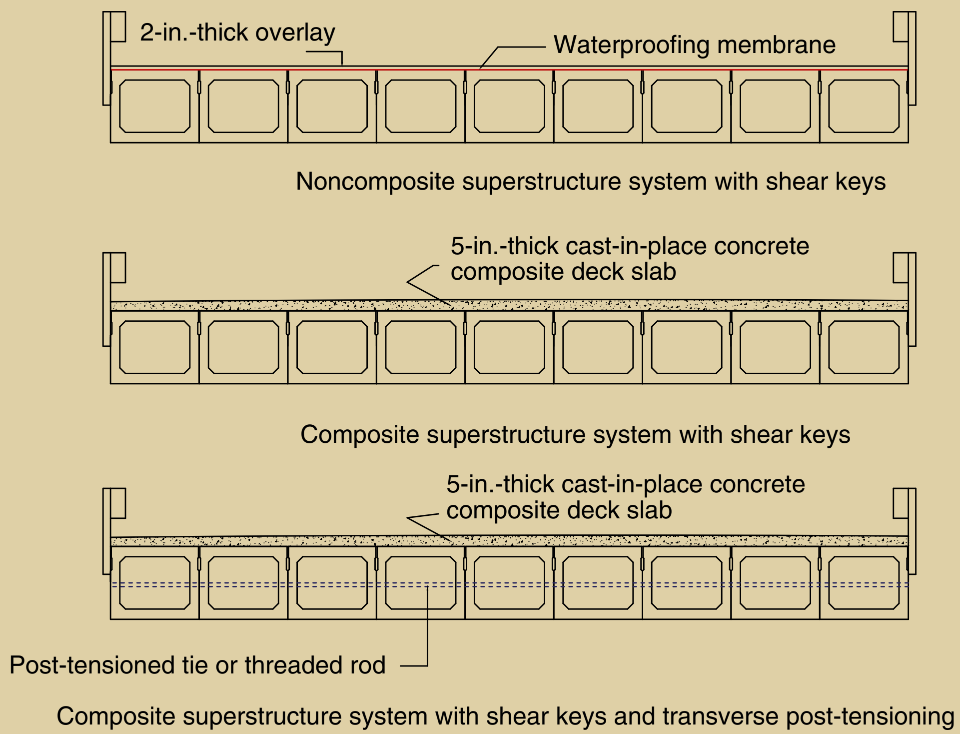

Figure 1. Adjacent-box-girder bridges incorporate various practices in the design and detailing of transverse connecting systems. Note: 1 in. = 25.4 mm.

- ease and speed of construction because of eliminating concrete forming and placing operations (for example, the Arbor Rail Line Bridge in Nebraska City, Neb., was erected and opened to traffic within 72 hr)[1]
- a shallow superstructure depth, which is often necessary to maintain the required vertical clearance (for example, an interstate bridge in Colorado has a span-to-depth ratio of 39)
- low construction cost compared with I-girder bridges and other competing systems
- hollow portions inside the box girders that reduce the self-weight of the girders and provide space for gas lines, water pipes, telephone ducts, storm drains, and other utilities
- improved bridge aesthetics because of the flat soffit and slender superstructure
- high torsional stiffness, which is ideal for curved-bridge construction

Bridges constructed using box girders have been in service for many years and have generally performed well. However, a recurring problem is cracking in the grouted joints between adjacent units, resulting in reflective cracks in the wearing surface.

The development of these longitudinal cracks over the shear keys jeopardizes the durability and structural behavior of adjacent-box-girder bridges.[2,3] In most cases, the cracking leads to leakage, which allows chloride-laden water to penetrate the sides and bottoms of the girders, causing corrosion of the steel reinforcement. In addition, the load distribution among the girders is adversely affected because the loaded girders are required to carry more load than the design load.[3]

These deficiencies have led to severe deterioration and premature replacement of several bridges. On December 27, 2005, the east-side fascia girder of the Lakeview Drive Bridge over Interstate 70 in Washington, Pa., failed near midspan and fell to the highway below. Inspection of the bridge revealed heavy spalling and corrosion of the strands on the bottom flange of the failed noncomposite prestressed concrete box girder. Additional corrosion was revealed on other box girders, and the bridge was subsequently removed from service.[4]

A similar failure occurred in a railroad bridge in Nebraska in 2007. Unfortunately, public attention focuses on the few failed cases and not on the many successful examples.

Current practice

Adjacent-box-girder bridges incorporate various practices in the design and detailing of transverse connecting systems. **Figure 1** shows three configurations. The first is a noncomposite system with a nonstructural overlay as the riding surface applied directly to the top flange of the adjacent box girders. This system depends only on the grouted shear key to provide the shear-transfer mechanism between adjacent box girders.

The second is a thick, reinforced cast-in-place concrete slab anchored to the supporting box girders using shear connectors to act as a composite superstructure system.

The third system is a typical transverse connection made between the adjacent box girders using a post-tensioning tie or a threaded rod. This transverse connection system can be used in conjunction with the composite or noncomposite systems to prevent differential deflection.

According to the Ministry of Transportation of Ontario's (MTO's) *Ontario Highway Bridge Design Code*,[5] the general design philosophy of adjacent-member systems assumes that the entire load between adjacent members is transferred by transverse shear, and the transverse flexural rigidity is completely ignored.

Also, grouted shear keys are considered inadequate to transfer the shear force, and therefore a structural concrete slab of a minimum thickness of 5.9 in. (150 mm) is required. The transverse shear force is determined as a function of the bridge width-to-span ratio, longitudinal flexural rigidity, and longitudinal torsional rigidity.

Some states' departments of transportation (DOTs) combine the use of a structural concrete slab and transverse post-tensioning. This is based on the assumption that both shear and flexure forces must be transversely transferred at the joints between adjacent members to control both translational and rotational deformations.[6]

In Japan, adjacent box girders are designed using sections and design criteria similar to those used in the United States. However, longitudinal joints are detailed differently and transverse post-tensioning is significantly higher. Cast-in-place concrete is placed in full-depth joints that are 6.7 in. (170 mm) wide and 22 in. (560 mm) deep. After grouting, post-tensioning is applied through several ducts located at different elevations. All box girders are covered with a 2-in.-thick to 3-in.-thick (50 mm to 75 mm) asphalt-concrete wearing surface. Using the Japanese practice, longitudinal cracking and concrete deterioration has rarely been reported. El-Remaily et al.[6] give details of the post-tensioning arrangement and joint dimensions used.

In Korea, transverse connection is achieved by using mid-depth shear keys fully filled with cast-in-place concrete in addition to heavy transverse post-tensioning similar to the Japanese practice. The choice of a mid-depth shear key is based on a detailed analysis and full-scale testing.[7]

The state of Oregon has developed empirical transverse design and detailing procedures for adjacent box girders that have demonstrated satisfactory performance over several years. The developed system is based on using transverse threaded ties at several locations according to the span length; grouted, partial-depth shear keys; and recesses as $^1/_4$ in. (6 mm) chamfer at the bottom edges of the girder to prevent spalling due to stress concentration.

The results of research conducted by the West Virginia DOT on several bridges that had joint fracture and topping cracks revealed that vertical shear failure in the key was due to poor grouting and inadequate transverse tie force.[8] As a result of this study, the West Virginia DOT follows certain guidelines:

- Post-tensioned high-strength ties are used.
- A pourable epoxy is used instead of a nonshrink grout in the shear key.
- The surfaces to be grouted are sandblasted.

Before 1992 in New York state, depths of shear keys were about 12 in. (300 mm) from the tops of the precast concrete girders.[9] Transverse tendons applying a compressive force of 30 kip (133 kN) were used across the width of a bridge. Spans up to 50 ft (15 m) long had no transverse tendons, but those from 50 ft to 75 ft (23 m) long had one transverse tendon at the center. For those longer than 75 ft, tendons were used only at the outer quarter points.

The bridge continuity in transverse direction was ensured by using a 6-in.-thick (150 mm), cast-in-place concrete deck slab reinforced with welded-wire reinforcement. A survey in 1990[8] indicated that 54% of such bridges built from 1985 to 1990 had developed longitudinal cracks over the shear keys. In 1992, two major changes were adopted in New York state's design standards:

- Shear keys were placed at almost the full depth of the precast concrete box girders.
- The number of transverse tendons was increased to three for spans less than 50 ft (15 m) and five for longer spans.

Since the changes were adopted, more than 100 bridges have been built statewide. In 1996, a survey was conducted to evaluate the effectiveness of implemented design changes. The survey indicated that only 23% of the bridges built from 1993 to 1996 experienced longitudinal cracks. This indicated the effectiveness of applying transverse post-tensioning to reduce cracking of the deck slab.[9]

The Ohio DOT constructed a high-performance concrete (HPC) adjacent-box-girder bridge.[10] This bridge was constructed as a replacement for a three-span bridge. The bridge used an experimental shear key at mid-depth of the cross section. The girders were tightened together using nonprestressed threaded rods located transversely through diaphragms at the ends and quarter points of the bridge.

The shear keys were grouted after tightening the transverse bars. The area above the shear key was filled with sand and a sealant to further guard against leakage. After constructing the entire bridge width, the bridge was subjected to an eccentric load of 120 kip (534 kN) using four Ohio DOT trucks filled with gravel. The researchers reported that the deflection profiles in the transverse direction showed that all of the girders were working together. In addition, while subjecting the bridge to eccentric load, the deflection on the loaded side was greater than that on the opposite side of the bridge width.

Miller et al.[11] and Hlavacs et al.[12] studied the performance of nonshrink grout and epoxy in shear keys. Nonshrink grout in shear keys close to the top edge of the girder experienced cracks before any load was applied. The researchers reported that these cracks developed because of temperature stresses. The use of epoxy grout made it possible to prevent cracking under either temperature or load effects, but the coefficient of its thermal expansion was two to three times greater than that of concrete.

Gulyas et al.[13] compared the behavior of nonshrink grout with the behavior of magnesium ammonium phosphate mortars in the shear keys. The researchers tested the component material in assemblies using different types of tests, such as vertical shear, direct tension, and longitudinal shear tests. The vertical shear test was intended to simulate the action of a vehicle wheel load on one member and no wheel load on the adjacent member. The direct tension test attempted to simulate the transverse shortening of the precast concrete member due to shrinkage and also to simulate the drying shrinkage that can occur in the keyway grout.

The longitudinal shear tests were performed in the direction parallel to the keyway to simulate the action of the prestressed concrete member shortening because of creep and shrinkage while the grout would not shorten to the same degree. In all tests, the magnesium ammonium phosphate–grouted assembles displayed an exceptionally higher failure load than the nonshrink grout composite assemblies.

In an attempt to overcome the problems associated with the failure of the shear keys, El-Esnawi[14] suggested a new shear-key design. The new design proposed moving the shear key from its position in the upper third of the gap closer to the midheight of the box-girder section. His experimental program included testing the current shear-key design against the proposed one while incorporating different grout materials such as nonshrink grout, magnesium ammonium phosphate mortar, and epoxy resin mortar in both of the designs. The test specimens were loaded gradually until failure.

El-Esnawi observed that the static-load capacity was almost tripled from the current shear-key design with the same grouting material. He reported that magnesium ammonium phosphate was sensitive to carbonated concrete surfaces, and it was difficult to provide a carbonate-free contact surface to prevent the chemical reaction with magnesium ammonium phosphate. The research also revealed that epoxy grout was a strong grouting material that had excellent adhesion with concrete. However, its preparation had many difficulties and its long-term behavior under different temperature changes was questionable.

Annamalai et al.[15] conducted an experimental program to investigate the effect of transverse post-tensioning on the behavior of small assemblies. The parameters studied were the number and thickness of shear keys and the level and distribution of the prestressing. They concluded that post-tensioning significantly improved the shear strength of grouted shear-key connections, and post-tensioned grouted shear-key connections exhibited a high degree of monolithic action.

Assemblies with three keys showed higher rigidity than specimens with two keys. Assemblies with a 1-in.-thick (25 mm) joint were found to have significantly higher shear strengths than specimens with 2-in.-thick (50 mm) joints for the same prestress level. The connections without shear keys and with a prestress of 800 psi (5520 kPa) had nearly the same shear strength as connections with shear keys and no prestress. Prestress in combination with shear keys provided superior performance. The shear strength was not significantly influenced by the distribution of prestress along the height of the specimen.

Stanton and Mattock[16] tested the strength of welded connectors acting alone and with grout keys. The experimental program involved six specimens. Each specimen consisted of two 6-in.-thick (150 mm) reinforced concrete slabs joined together at their edges using a 5-ft-long (1.5 m) shear key.

Stanton and Mattock concluded that the forces from the wheel loads were transferred through the shear key. The steel connectors carried shear forces induced before grouting because of differential camber, and tension forces because of shrinkage. They concluded that the strength of the shear key was affected by inclined cracking in the parts of the member flanges above and below the shear key rather than by failure of the grout itself. They recommended the use of transverse post-tensioning to improve the behavior of transverse connections.

Huckelbridge et al.[3] conducted a total of six field tests on three bridges with noncomposite topping and two field tests with composite cast-in-place concrete decks. One of the bridges was tested before and after repairing a severely deteriorated joint.

They reported that a relative displacement between the girders of more than 0.001 in. (0.025 mm) indicated failure in the shear key. The magnitude of relative displacement experienced by each bridge depended on the actual length of the fracture, stiffness of the girders, and magnitude and proximity of the wheel load to the failed joint. Relative displacements between 0.003 in. and 0.02 in. (0.075 mm and 0.50 mm) were observed at joints that indicated at least partially fractured shear keys. The results also revealed that tie bars had little to no impact on shear-key performance.

Issa et al.[17] tested a total of 36 full-scale specimens for vertical shear, direct tension, and flexural capacity. Four different grout materials were used to construct the shear keys in the specimens. The grout materials were set grout, set 45 for normal temperatures, set 45 for hot weather, and polymer concrete.

Polymer concrete was found to be the best material for transverse joints in terms of strength, bond, and mode of failure. However, they recommended the use of set grout in transverse deck joints due to its ease of use and satisfactory performance and polymer concrete in the joints subjected to excessive stresses or when quick repair is required.

Martin and Osburn[18] tested two precast, prestressed concrete–slab bridge models. Each model consisted of three adjacent precast, prestressed concrete slabs that were connected by two different types of transverse connections. The precast, prestressed concrete slabs had a cross section of 8 in. × 36 in. (200 mm × 910 mm), a length of 18 ft (5.5 m), and a span of 16 ft (4.9 m).

Each bridge model was tested by loading the two outer slabs cyclically at midspan by equal concentrated loads of 16 kip (71 kN), which was intended to simulate American Association of State Highway and Transportation Officials (AASHTO) HS-20 wheel loads.

In the first bridge model, the slabs were joined together by tie rods at the third point of the span, and the tie rods were tensioned to 12 kip (53 kN). The joints were grouted using high-strength, nonshrink grout.

In the second bridge model, the slabs were joined together with three welded connectors located at both supports and midspan. The joints were grouted using relatively low-strength, high-shrinkage grout.

In both bridge models, it was found that the moment resisted by each outer slab was about 33% greater than the moment resisted by the middle slab. Martin and Osburn concluded that a properly grouted shear key and either transverse tie rods or welded connectors are an effective way to transfer shear between adjacent members.

El-Shahawy[19] investigated the behavior of the transverse connections in the double-tee bridges. He tested a half-scale bridge model consisting of three 30-ft-long (8.2 m) double-tees. The transverse connections consisted of V-shaped joints between the girders filled with nonshrink portland cement grout and transverse post-tensioning strands. The developed stresses due to transverse post-tensioning strands were equal to 150 psi (1030 kPa) at the middle portion and 300 psi (2070 kPa) at the ends.

The behavior of the transverse connection was investigated by conducting several punching-shear tests on the deck slab followed by a load-distribution test across the entire bridge width. The results from all tests indicated that the selected level of transverse prestressing satisfied the design requirements. It was also concluded that transverse post-tensioning helped the slab achieve monolithic behavior.

According to the *AASHTO LRFD Bridge Design Specifications*,[20] the use of transverse mild-steel rods secured by nuts is not sufficient to achieve full transverse flexural continuity. Section 5.14.4.3.3d recommends a minimum effective post-tensioning pressure of 250 psi (1720 kPa) through the shear key. There is neither a rational justification for this value nor an adequate explanation regarding the area over which this pressure is applied when different shear keys are used.

The PCI subcommittee on adjacent-member bridges conducted a survey on the current practices in the design and construction of adjacent-box-girder bridges in United States and Canada.[9] This survey indicated that 29 states and 3 provinces are currently using adjacent-box-girder bridges.

Most of these transportation agencies have experienced premature reflective cracks in the wearing surface on the bridges built in the late 1980s and early 1990s. These agencies have emphasized the importance of eliminating these cracks that allow the penetration of water and deicing chemicals and lead to the corrosion of reinforcing steel in the sides and bottoms of concrete box girders. The states and provinces have recommended preventive actions based on the lessons learned in the past two decades:

- Cast-in-place concrete deck on top of the adjacent box girders can prevent water leakage and uniformly distribute the loads on adjacent box girders.

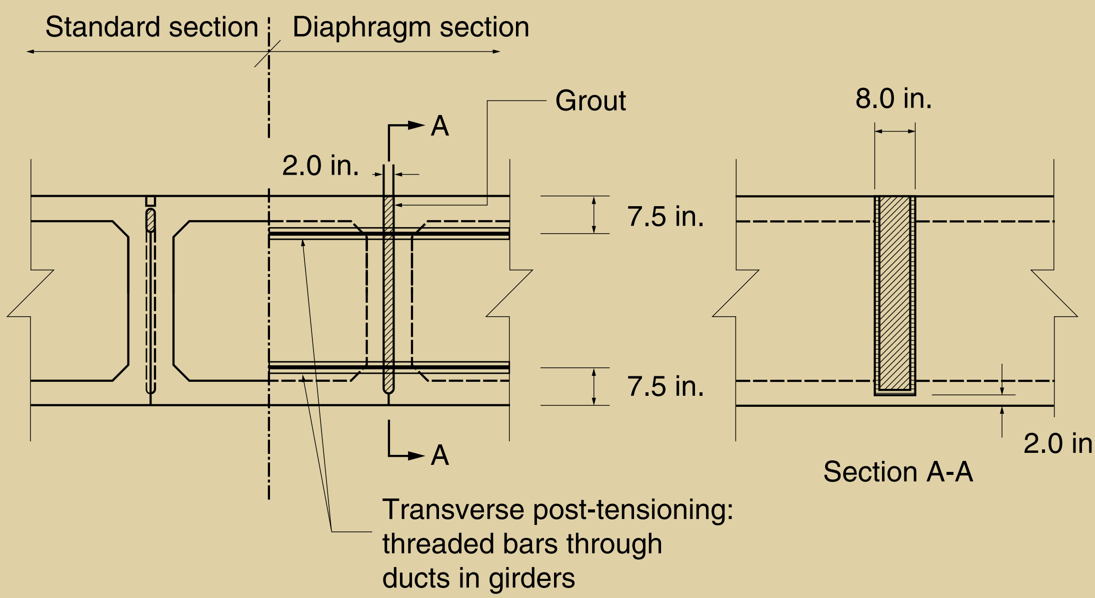

Figure 2. The transverse diaphragms are made continuous across the entire width of the bridge using grouted full-depth shear keys and post-tensioning tendons. Source: Figure 8.9.3-1. PCI Bridge Design Manual Steering Committee, *Precast Prestressed Concrete Bridge Design Manual* (Chicago, IL: PCI, 2003). Note: 1 in. = 25.4 mm.

- Nonshrink grout or the appropriate sealant instead of the conventional sand-cement mortar in the shear keys should be used in addition to blast cleaning of key surfaces prior to grouting. Also, a few states have recommended the use of full-depth shear keys due to their superior performance over the traditional top-flange keys.

- Transverse post-tensioning is recommended to improve load distribution and minimize differential deflections among adjacent box girders. Adequate post-tensioning should be applied after grouting the shear keys to minimize the tensile stresses that cause longitudinal cracking at these joints.

- End diaphragms should be used to ensure proper seating of adjacent box girders, and intermediate diaphragms should be used to provide the necessary stiffness in the transverse direction.

- Wide bearing pads under the middle of the box and sloped bearing seats that match the surface cross slope are recommended to eliminate the rocking of the box while grouting the shear keys.

- Adequate concrete cover and corrosion-inhibiting admixtures should be used in the concrete to resist the chloride-induced corrosion of reinforcing steel.

- It is recommended to eliminate the use of welded connections between adjacent box girders and to avoid dimensional tolerances that result in inadequate sealing of the shear keys.

PCI method

PCI's *Precast Prestressed Concrete Bridge Design Manual*[21] method was developed by El-Remaily et al.[6] and is reported in section 8.9 of the manual. In this method, the post-tensioning force required to achieve adequate stiffness in the transverse direction to keep differential deflection within the acceptable limit (0.02 in. [0.50 mm]) is calculated.

This method assumes that post-tensioned transverse diaphragms are the primary mechanism for the distribution of wheel loads across the bridge. Five diaphragms are provided in each span: one at each end, one at midspan, and one at each quarter point. Without diaphragms, each box girder must be designed to carry a full set of wheel loads without contribution from adjacent box girders. As a result, a large differential deflection between adjacent girders will take place and reflective cracking is generally expected. However, if the box girders are transversely connected using diaphragms, the loads are distributed over the entire bridge width, and the deflected shape becomes a smooth curve. The transverse diaphragms are made continuous across the entire width of the bridge using grouted, full-depth shear keys and post-tensioning tendons (**Fig. 2**).

To determine the required amount of post-tensioning, the bridge is analyzed using a grid model. A series of longitu-

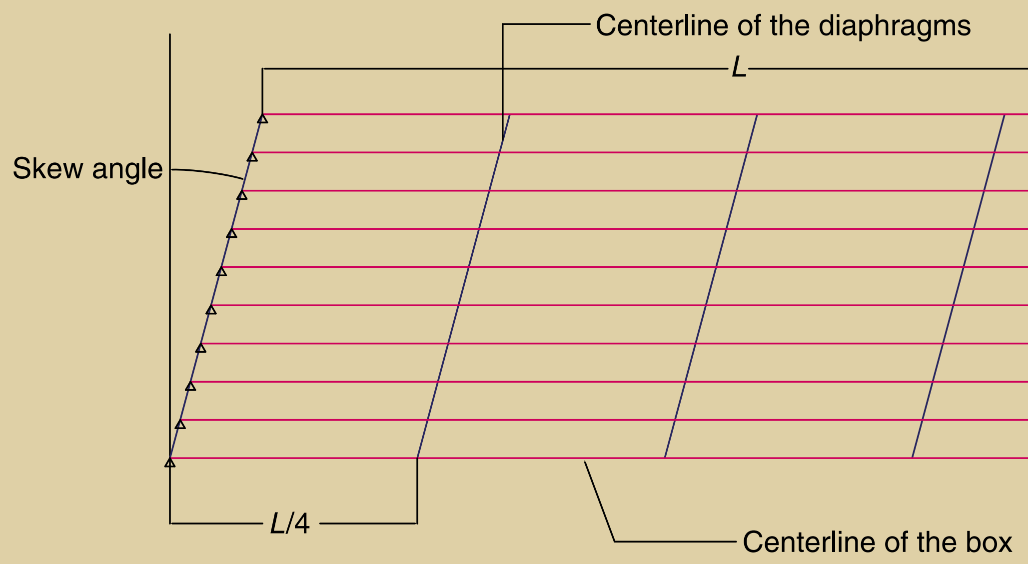

Figure 3. This drawing shows the grid analysis model. Note: L = span length; W = bridge width.

dinal girder elements located at the centerline of each box girder is used to represent the box girders, and a series of transverse girder elements located at the ends and quarter points is used to represent the diaphragms (**Fig. 3**). The joints between elements allow the transmission of shear, bending, and torsion. The weight of barrier rails and live loads are the main source of transverse bending moments generated in the diaphragms. This is because self-weight, deck weight, and wearing-surface weight are considered uniform on all of the elements and therefore do not generate any differential movements.

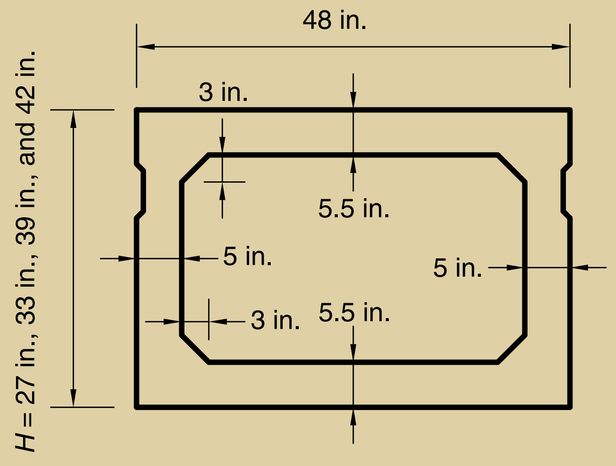

Figure 4. This drawing shows the American Association of State Highway and Transportation Officials' standard box girder used for developing the design charts currently available in the PCI *Precast Prestressed Concrete Bridge Design Manual*. Note: H = height of box girder. 1 in. = 25.4 mm.

Transverse post-tensioning force is calculated so that diaphragm concrete stresses due to both loads and post-tensioning are within the allowable limits (compression = 0.6 f_c' where f_c' is specified compressive strength of concrete and tension is 0). Tensile stresses are not permitted in the diaphragm in order to prevent possible cracking at the interface between precast concrete components and the grout at shear-key locations. Also, post-tensioning force is applied concentrically in the transverse direction because diaphragms experience significant alternating positive and negative bending moments under different loading conditions.

The design chart currently available in the PCI bridge design manual was developed for the AASHTO standard box girders in **Fig. 4** and **Table 1**, assuming mild skew angles (that is, less than 15 deg), average span lengths, and AASHTO HS-25 truck loading with impact.[6] New charts need to be developed to accommodate the cases of highly skewed bridges with different span lengths and using the AASHTO LRFD specifications truck and lane loads in addition to dynamic load allowance.[20]

Updated design charts and design equation

The updated design charts were developed using the same PCI method for the four standard AASHTO box girders in Fig. 4 and Table 1. For each girder, several combinations of bridge width, span length, and skew angle were considered. The AASHTO LRFD specifications for truck and lane live loads (HL-93) and dynamic load allowance (33% for truck load only) were applied in addition to 0.48 kip/ft (7.0 kN/m) for the self-weight of a solid concrete barrier.[20]

Table 1. Box beam properties

Type	Height *H*, in.	Area *A*, in.^{2}	Y_{bottom}	Moment of inertia *I*, in.^{4}
BI-48	27	692.5	13.37	65,941
BII-48	33	752.5	16.33	110,499
BIII-48	39	812.5	19.29	168,367
BIV-48	72	842.5	20.78	203,088

Note: Y_{bottom} = distance from bottom of girder to center of gravity. 1 in. = 25.4 mm.

Figure 5 shows the effective post-tensioning force versus bridge width for the four standard box girders, assuming a 0 deg skew angle and a span-to-depth ratio of 30. This graph indicates that for any girder depth, the wider the bridge, the higher the required post-tensioning force. It also indicates that the required force is higher in shallower girders than in deeper girders for the same bridge width. This is mainly to compensate for the reduction in the transverse stiffness due to the use of shallower diaphragms. Each line in Fig. 5 has two different curvatures. The first curvature represents the relationship when the negative moment controls the design, which occurs in the relatively narrow bridge widths (up to 52.0 ft [15.9 m]). The second curvature represents the relationship when the positive moment controls the design, which occurs in wider bridges.

Figure 6 shows the PCI bridge design manual design chart superimposed over the updated design chart. This graph indicates a significant increase in the required post-tensioning force (up to 40% in some cases) in the updated charts. This is mainly due to the use of the AASHTO LRFD specifications on live-load and dynamic-load allowance. This increase varies depending on the box-girder depth and the bridge width, and it is more noticeable in narrow bridges than in wide bridges. It should be noted that the PCI bridge design manual values correspond to a skew angle of 15 deg and average span length, while the proposed values correspond to a skew angle of 0 deg and a span-to-depth ratio of 30.

Figure 7 shows the required post-tensioning force versus bridge width for a 0 deg skew angle and span-to-depth ratios of 30 and 40. Although the effect of the span-to-depth ratio was evaluated for the four standard box girders, only the lines for the 27-in.-deep and 42-in.-deep (690 mm and 1070 mm) box girders were plotted for clarity. This plot indicates that the span-to-depth ratio has an insignificant and variable effect on the required post-tensioning force per unit length. As the span-to-depth ratio increased, the required prestressing force increased when the design

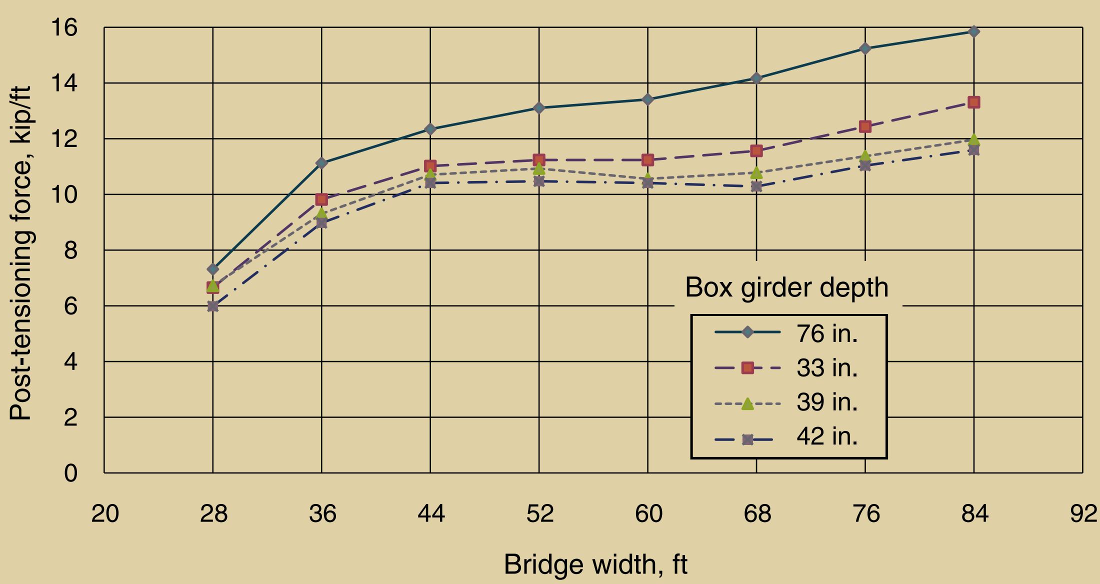

Figure 5. This graph shows the effect of bridge width on post-tensioning force at the midspan diaphragm for the four standard box girders assuming a 0 deg skew angle and a span-to-depth ratio of 30. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

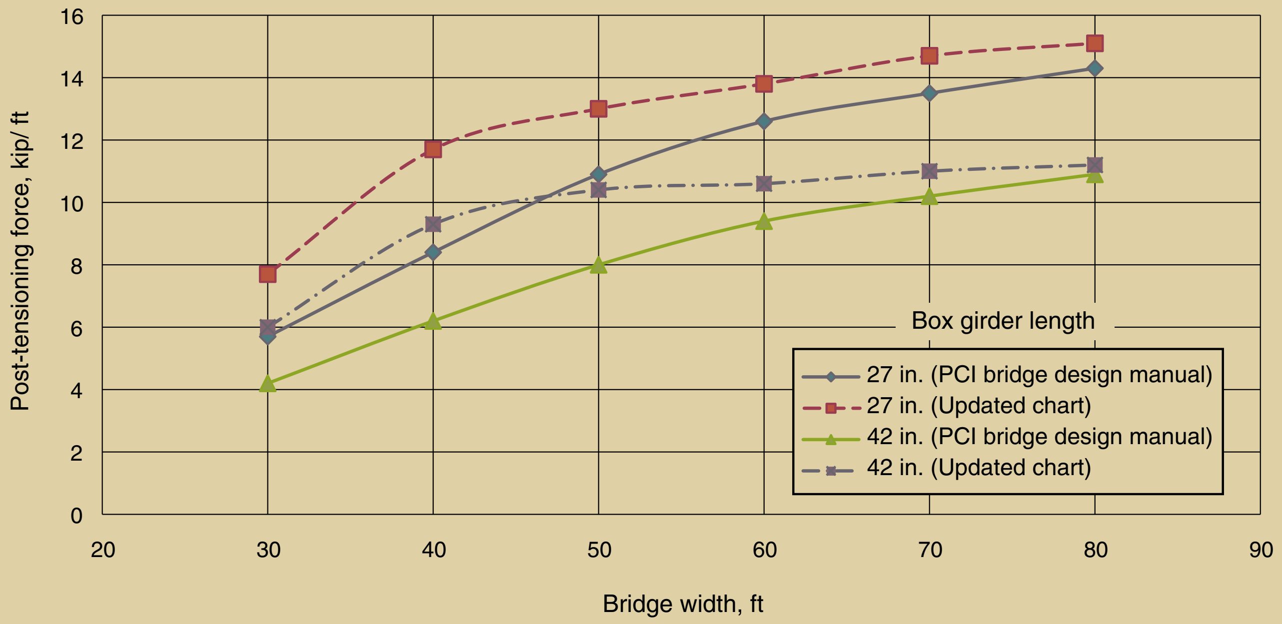

Figure 6. This graph compares the *Precast Prestressed Concrete Bridge Design Manual* design chart with updated charts showing the effect of bridge depth on post-tensioning force. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

was positive-moment controlled, and decreased when the design was negative-moment controlled. This effect was more noticeable in the shallow girders.

Figure 8 shows the effect of skew angle on the required post-tensioning force at the midspan diaphragm for a bridge width of 52 ft (16 m) and a span-to-depth ratio of 30. Figure 8 indicates that the impact of the skew angle on the required post-tensioning force is minimal, especially on deep girders that usually correspond to longer spans. For shallow girders used in short-span bridges, as the skew angle increased the required post-tensioning force also increased.

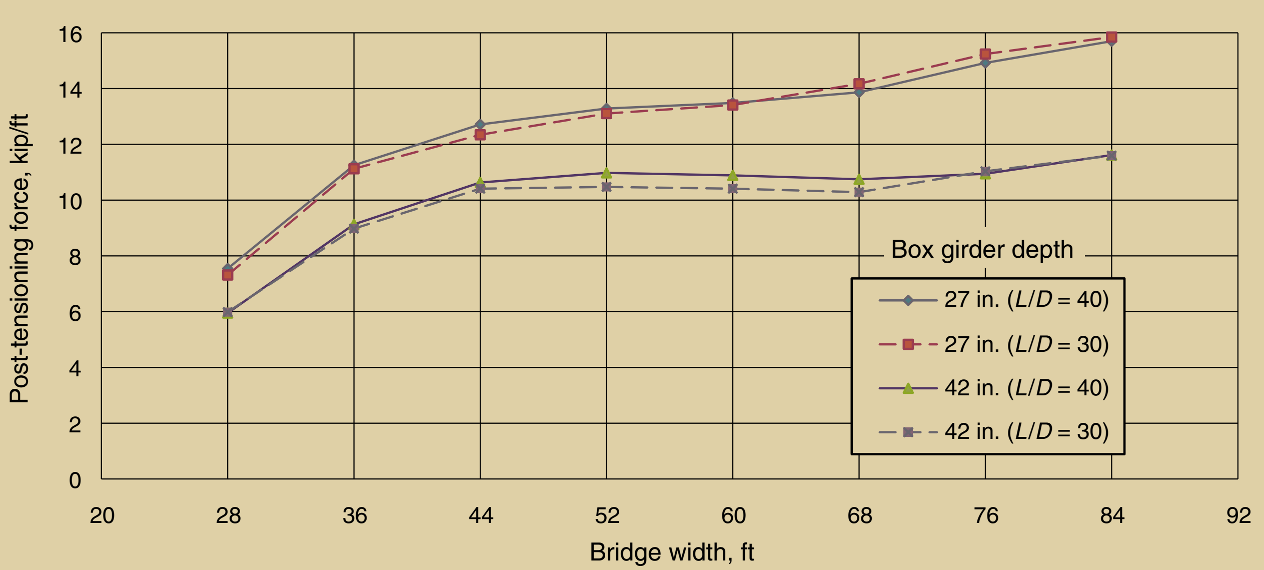

Figure 7. This graph shows the effect of the span-to-depth ratio on post-tensioning force at the midspan diaphragm for a 0 deg skew angle and span-to-depth ratios equal to 30 and 40. Note: D = depth; L = span. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

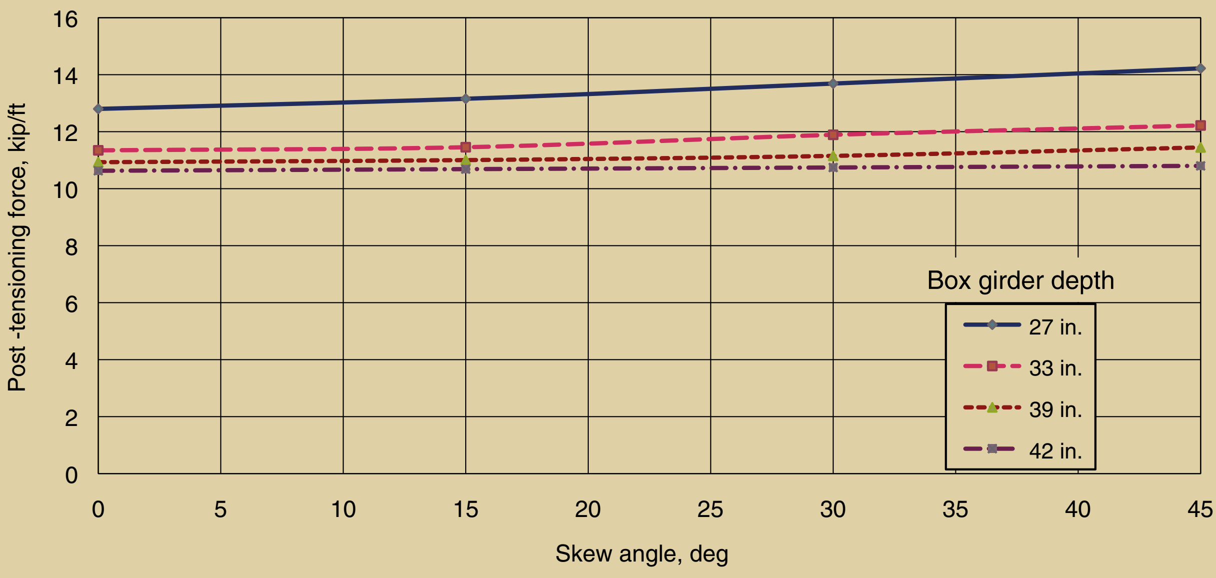

Figure 8. This graph shows the effect of the bridge skew angle on post-tensioning force for the midspan diaphragm for a bridge width of 52 ft and a span-to-depth ratio of 30. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

Figures 5, 7, and 8 indicate that the bridge width and box-girder depth are the most important parameters in determining the required post-tensioning force per unit length of the bridge. Therefore, the designer should first estimate the force based on the bridge width and girder depth using the proposed design chart (Fig. 5). These values correspond to a span-to-depth ratio of 30 and a skew angle of 0 deg and should be corrected using Fig. 7 and 8, respectively, when different span-to-depth ratios or skew angles are used.

Data from the grid analysis were used to develop a simplified design equation for calculating the required post-

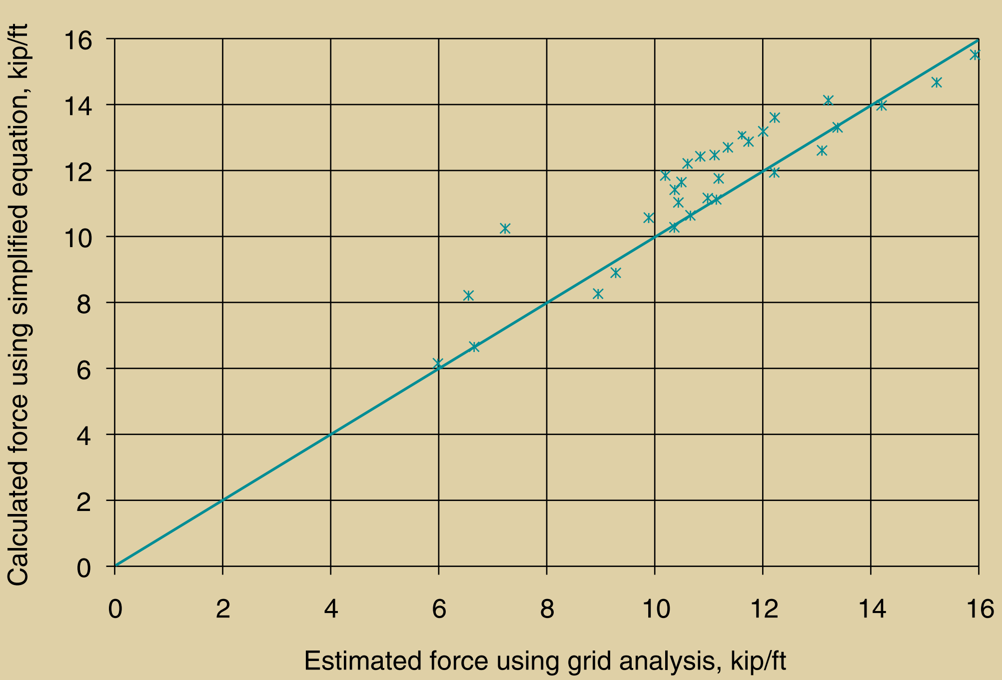

Figure 9. This graph compares the post-tensioning force estimated using grid analysis with the proposed equation. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

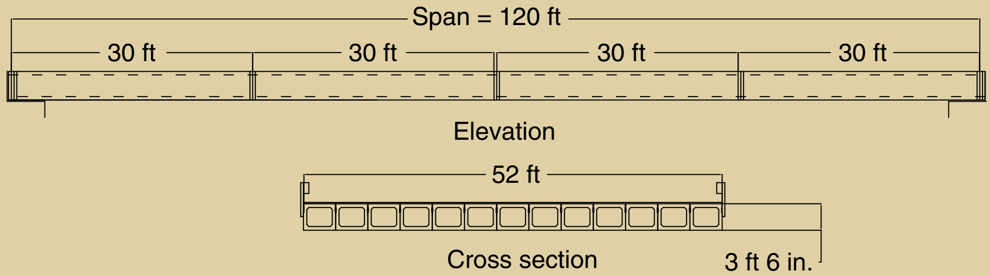

Figure 10. This drawing illustrates the bridge geometry for the design example. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

tensioning force P (kip/ft) for the intermediate diaphragm per unit length of the bridge. The following equation was developed by fitting the data points obtained from the grid analysis of all cases. The first part of the equation represents the relationship when the negative moment controls the design, which occurs in smaller bridge widths (up to 52.0 ft [15.9 m]). The second part of the equation represents the relationship when the positive moment controls the design, which occurs in wider bridges. These relationships were assumed to be linear to eliminate sophisticated formulations.

$$P = \left(\frac{0.9W}{D} - 1.0\right) K_L K_S \le \left(\frac{0.2W}{D} + 8.0\right) K_L K_S$$

where

D = box depth

W = bridge width

K_L = correction factor for span-to-depth ratio

$$= 1.0 + 0.003\left(\frac{L}{D} - 30\right)$$

K_S = correction factor for skew angle more than 0 deg

$= 1.0 + 0.002\theta$

L = bridge span

θ = skew angle

To evaluate the accuracy of the simplified equation in fitting the analysis data, the post-tensioning force values obtained using the equation were compared with those obtained using the grid analysis for several combinations of bridge width and depth. Span-to-depth ratio and skew angle were kept constant to evaluate the accuracy of the basic equation without any correction factors. **Figure 9** shows that the simplified equation provides a conservative estimate of the required transverse post-tensioning force in most of the cases, with an average deviation of 7.7%.

Design example

The provided design example illustrates the design steps of the single-span bridge (**Fig. 10**).

Bridge data

Figure 10 shows the cross section and the elevation of the bridge.

Span = 120 ft (37 m)

Width = 52 ft (16 m)

Depth = 42 in. (1070 mm) (AASHTO LRFD specifications standard box girder)

Skew = 15 deg

Concrete strength

Precast concrete f'_c = 6000 psi (41 MPa)

Grout f'_c = 6000 psi (41 MPa)

Box girder section properties

Area A = 842.5 in.^{2} (543,600 mm^2)

Moment of inertia I = 203,088 in.^{4} (8.453×10^{10} mm^4)

Diaphragm section properties

The cross section of the diaphragms is rectangular. The depth of the diaphragm is equal to the depth of the box girder (42 in. [1070 mm]), and the width is 8 in. (200 mm)

A = 336 in.^{2} (216,800 mm^2)

I = 49,392 in.^{4} (1.68×10^{10} mm^2)

Loading

Dead load: curb and railing w = 0.48 kip/ft (7.0 kN/m)

Live load: HL-93 truck and lane load

Impact factor for truck load = 33%

Calculation of the required transverse post-tensioning force using working stresses analysis

The grid analysis was used to get the member forces. Moments of the midspan diaphragm were used for design calculations. The live-load positions were chosen to give the maximum positive and maximum negative moments. Allowable compressive strength due to effective prestress plus maximum load was calculated using the following equation.

$0.6\,f_c' = 0.6(6000) = 3600$ psi (24,800 kPa)

Tension is not permitted. These stresses must be checked for both the maximum positive and maximum negative load cases:

- Positive-moment load case: the unfactored maximum positive moment is 147 kip-ft (199 kN-m).

$$f_{bot} = -\left(\frac{147(12)(1000)(21)}{49{,}392}\right) + \frac{P(1000)}{336} \geq 0$$

where

$P_{diaphragm} \geq 252$ kip (1121 kN)

f_{bot} = stress in bottom of diaphragm

$$f_{top} = -\left(\frac{147(12)(1000)(21)}{49{,}392}\right) + \frac{P(1000)}{336} \leq 3600$$

$P_{diaphragm} \leq 958$ kip (4381 kN)

f_{top} = stress in top of diaphragm

- Negative-moment load case: the unfactored maximum negative moment is 187 kip-ft (254 kN-m).

$$f_{top} = -\left(\frac{187(12)(1000)(21)}{49{,}392}\right) + \frac{P(1000)}{336} \geq 0$$

$P_{diaphragm} \geq 324$ kip (1441 kN)

$$f_{bot} = -\left(\frac{187(12)(1000)(21)}{49{,}392}\right) + \frac{P(1000)}{336} \leq 3600$$

$P_{diaphragm} \leq 890$ kip (3959 kN)

Based on the previous calculations, the total required transverse post-tensioning force per diaphragm is 324 kip (1440 kN). The total required transverse post-tensioning force per foot of the bridge is 324/30, which equals 10.8 kip/ft (158 kN/m).

According to section 5.14.4.3.3d of the AASHTO LRFD specifications and using the area of the full-depth vertical shear key as the contact area, the minimum required transverse post-tensioning force per diaphragm is equal to 0.25(8)(42 - 2), or 80 kip (360 kN). This is low because it represents only 25% of the force calculated using the updated PCI method. If the entire side of the box is used as the contact area, the minimum required transverse post-tensioning force per diaphragm is 0.25(30)(12)(42), or 3780 kip (16,800 kN). This is extremely high because it is 10 times the force calculated using the updated PCI method.

Calculation of the required transverse post-tensioning force using the developed equation

From the proposed equation, the required transverse post-tensioning force is calculated as follows.

$$K_L = 1.0 + 0.003\left(\frac{L}{D} - 30\right)$$

$$= 1.0 + 0.003\left(\frac{120(12)}{42} - 30\right) = 1.013$$

$K_s = 1.0 + 0.002\theta = 1.0 + 0.002(15) = 1.03$

The required post-tensioning force:

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

$$= \left(\frac{0.9(52)(12)}{42} - 1.0\right)(1.013)(1.03)$$

$$\leq \left(\frac{0.2(52)(12)}{42} + 8.0\right)(1.013)(1.03)$$

$= 12.9 \leq 11.5 \therefore P = 11.5$ kip/ft (168 kN/m)

$P_{diaphragm} = 11.5(30) = 345$ kip (1535 kN)

Required prestressing strands

Two tendons will be used in each diaphragm. The required area of post-tensioning force A_{ps} is calculated by dividing the required force $P_{diaphragm}$ by the effective prestress f_s', which is assumed to be 55% of the ultimate strength of the strands f_{pu}.

A_{ps} = 345/(0.55 × 270) = 2.32 in.^{2} (15.0 mm^2)

Try six 0.6-in.-diameter strands at each tendon. The total area is 2.604 in.^{2} (16.80 mm^2), which is acceptable.

Conclusion

Based on the results of the parametric study and the comparison of the updated design chart with the existing PCI bridge design manual design chart, several conclusions are made:

- The latest AASHTO LRFD specifications for live-load and dynamic-load allowance cause a significant increase (up to 40% in some cases) in the required transverse post-tensioning force for adjacent-box-girder bridges.

- The bridge width and girder depth have the most significant effect on the required transverse post-tensioning force. For any girder depth, an increase in the bridge width is accompanied by a higher post-tensioning force. Also, the required force is higher in shallower girders than in deeper girders for the same bridge width.

- Span-to-depth ratio has a variable effect on the required transverse prestressing force per unit length. As the span-to-depth ratio increases, the required prestressing force also increases when positive moment controls the design, and less prestressing force is required when negative moment controls. This effect is more noticeable in the shallow girders.

- Skew angle has a minimal effect on the required transverse post-tensioning force, especially on deep girders that usually correspond to longer spans. For shallow girders used in short-span bridges, greater skew angles require more transverse post-tensioning force.

- The simplified design equation provides the required transverse post-tensioning force per unit length of the bridge as a function of its width and box-girder depth and accounts for the span-to-depth ratio and skew angle using correction factors. The average deviation of the values calculated using the simplified equation and the grid analysis results is 7.7%.

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References

1. Hennessey, S. A., and K. A. Bexten. 2002. Value Engineering Results in Successful Precast Railroad Bridge Solution. *PCI Journal*, V. 47, No. 4 (July–August): pp. 72–77.

2. Kahl, S. 2005. Box-Beam Concerns Found under the Bridge. *C&T Research Record*, No. 102 (September): pp. 1–4.

3. Huckelbridge, A. A., H. El-Esnawi, and F. Moses. 1995. Shear Key Performance in Multi-Beam Box Girder Bridges. *Journal of Performance of Constructed Facilities*, V. 9, No. 4 (November): pp. 271–285.

4. Naito, C., R. Sause, I. Hodgson, S. Pessiki, and C. Desai. 2006. Forensic Evaluation of Prestressed Box Beams from the Lake View Drive Bridge over I-70. Advanced Technology for Large Structural Systems (ATLSS) report no. 06-13.

5. Ministry of Transportation of Ontario (MTO). 1995. *Ontario Highway Bridge Design Code*. 1995. MTO.

6. El-Remaily, A., M. K. Tadros, T. Yamane, and G. Krause. 1996. Transverse Design of Adjacent Precast Prestressed Concrete Box Girder Bridges. *PCI Journal*, V. 41, No. 4 (July–August): pp. 96–113.

7. Nam, J. W., H. J. Kim, J. H. Kim, S. H. Nam, S. B. Kim, and K. J. Byun. 2008. International Perspective: Overview and Application of Precast Prestressed Box-Beam Bridges in Korea. *PCI Journal*, V. 53, No. 4 (July–August): pp. 83–107.

8. PCI Bridges Committee. 2008. Reflective Cracking in Adjacent Box Girder Bridge Superstructures. *Subcommittee on Adjacent Box Beam Bridges*, October.

9. Lall, J., S. Alampalli, and E. F. Dicocoo. 1998. Performance of Full-Depth Shear Keys in Adjacent Prestressed Box Beam Bridges. *PCI Journal*, V. 43, No. 2 (March–April): pp. 72–79.

10. Greuel, A., T. M. Baseheart, B. T. Rogers, R. A. Miller, and B. M. Shahrooz. 2000. Evaluation of a High Performance Concrete Box Girder Bridge. *PCI Journal*, V. 45, No. 6 (November–December): pp. 60–71.

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

12. Hlavacs, G. M., T. Long, R. A. Miller, and T. M. Baseheart. 1997. Nondestructive Determination of Response of Shear Keys to Environmental and Structural Cyclic Loading. *Transportation Research Record*, No. 1574 (November): pp. 18–24.

13. Gulyas, R. J., G. J. Wirthlin, and J. T. Champa. 1995. Evaluation of Keyway Grout Test Methods for Precast Concrete Bridges. *PCI Journal*, V. 40, No. 1 (January–February): pp. 44–57.

14. El-Esnawi, H. H. 1996. Evaluation of Improved Shear Key Designs for Multi-Beam Prestressed Concrete Box Girder Bridges. PhD thesis. Case Western Reserve University, Cleveland, Ohio.

15. Annamali, G., and R. C. Brown. 1990. Shear Transfer Behavior of Post-tensioned Grouted Shear Key Connections in Precast Concrete-Framed Structures. *ACI Structural Journal*, V. 87, No. 1 (January-February): pp. 53–60

16. Stanton, J. F., and A. H. Mattock. 1986. *Load Distribution and Connection Design for Precast Stemmed Multi-Beam Bridge Superstructures*. TRB report no. 287. Washington, DC: Transportation Research Board.

17. Issa, M. A., C. L. R. Valle, S. Islam, and H. A. Abdalla. 2003. Performance of Transverse Joint Grout Materials in Full-Depth Precast Concrete Bridge Deck Systems. *PCI Journal*, V. 48, No. 4 (July–August): pp. 92–103.

18. Martin, L. D., and A. E. N. Osborn. 1983. Connections for Modular Concrete Bridge Decks. Federal Highway Administration 82/106, National Technical Information Service document PB84-118058, Consulting Engineering Group Inc., Glenview, IL.

19. El-Shahawy, M. 1990. Feasibility Study of Transversely Prestressed Double Tee Bridges. *PCI Journal*, V. 35, No. 5 (September–October): pp. 56–69.

20. American Association of State Highway and Transportation Officials (AASHTO). 2004. *AASHTO LRFD Bridge Design Specifications*. 3rd ed. Washington, DC: AASHTO.

21. PCI Bridge Design Manual Steering Committee. 2003. *Precast Prestressed Concrete Bridge Design Manual*. MNL-133. 2nd ed. Chicago, IL: PCI.

Notation

A = area

A_{ps} = required area of post-tensioning force

D = depth

f_{bot} = stress in bottom of diaphragm

f_c' = specified compressive strength of concrete

f_{pu} = ultimate strength of the strand

f_s' = effective prestress

f_{top} = stress in top of diaphragm

H = height

I = moment of inertia

K_L = correction factor for span-to-depth ratio

K_S = correction factor for skew angle more than 0 deg

L = bridge span

P = post-tensioning force per unit length of the bridge

$P_{diaphragm}$ = post-tensioning force on the diaphragm

w = dead load of curb and railing

W = bridge width

Y_{bottom} = distance from bottom of girder to center of gravity

θ = skew angle

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Synopsis

Precast, prestressed concrete adjacent box girders are widely used in short- and medium-span bridges. Rapid construction and low construction cost are the main attractions of this system. Also, the continuous flat soffit and relatively high span-to-depth ratio make this system aesthetically pleasing.

However, reflective cracking and leakage have been reported along the longitudinal joints between adjacent box girders in a number of bridges. The cracking and leakage are mainly due to inadequate design and detailing of the transverse connection between adjacent box girders, which eventually leads to excessive differential displacement and rotation of adjacent box girders. The reflective cracking and leakage allow chloride-induced corrosion of reinforcing steel and prestressing strand and premature deterioration of the bridge superstructure.

This paper presents a review of the various practices in the transverse design and detailing of adjacent-box-girder bridges. The basis for calculating the transverse post-tensioning force according to PCI's *Precast Prestressed Concrete Bridge Design Manual* is discussed. Design charts and equations were developed for various combinations of span length, bridge width, skew angle, and girder depth using the latest loading from *AASHTO LRFD Bridge Design Specifications*. These aids may be viewed as an update to the information in section 8.9 of the PCI bridge design manual, which was based on an earlier version of the AASHTO standard specifications.

Keywords

Adjacent box girder, bridge deterioration, grid analysis, longitudinal joint, rapid construction, shear key, transverse design.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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