Transverse Design of Adjacent Precast Prestressed Concrete Box Girder Bridges

Ahmed El-Remaily
Research Assistant
University of Nebraska-Lincoln
Omaha, Nebraska

Maher K. Tadros, Ph.D., P.E.
Cheryl Prewett Professor
of Civil Engineering
University of Nebraska-Lincoln
Omaha, Nebraska

Takashi Yamane
Chief Engineer
Kyokuto Corporation
Hiroshima, Japan

Gary Krause, Ph.D., P.E.
Associate Professor
of Civil Engineering
University of Nebraska-Lincoln
Omaha, Nebraska

American and Japanese design and detailing practices of prestressed concrete box girder bridges are compared. A new precast, prestressed box girder bridge design suitable to U.S. practice is proposed. The design is based on requiring the deck to act as a rigid assembly of longitudinal and transverse members. This is consistent with Japanese practice and fulfills the intent of several state DOT initiatives. The proposal advocates quarter-point diaphragms with relatively large amounts of transverse post-tensioning. A design chart and recommended details are provided for bridges up to 80 ft (24 m) long. The amount of post-tensioning was found to be unaffected by bridge span length. Also, a comparison with the requirements of the AASHTO LRFD provisions is given. A fully worked numerical design example is included to demonstrate the proposed design procedure.

Precast, prestressed concrete box girders are widely used in short and medium span bridges in North America. Based on the National Bridge Inventory, Dunker and Rabbat showed the change in percentage of the eight most common prestressed concrete bridge types built in the United States during the period 1950 to 1989 (see Fig. 1).[1] Stringer and multiple box sections are the most prevalent types of prestressed concrete bridges. Each system accounted for about one-third of all prestressed concrete bridges constructed in the United States during 1979 to 1989.

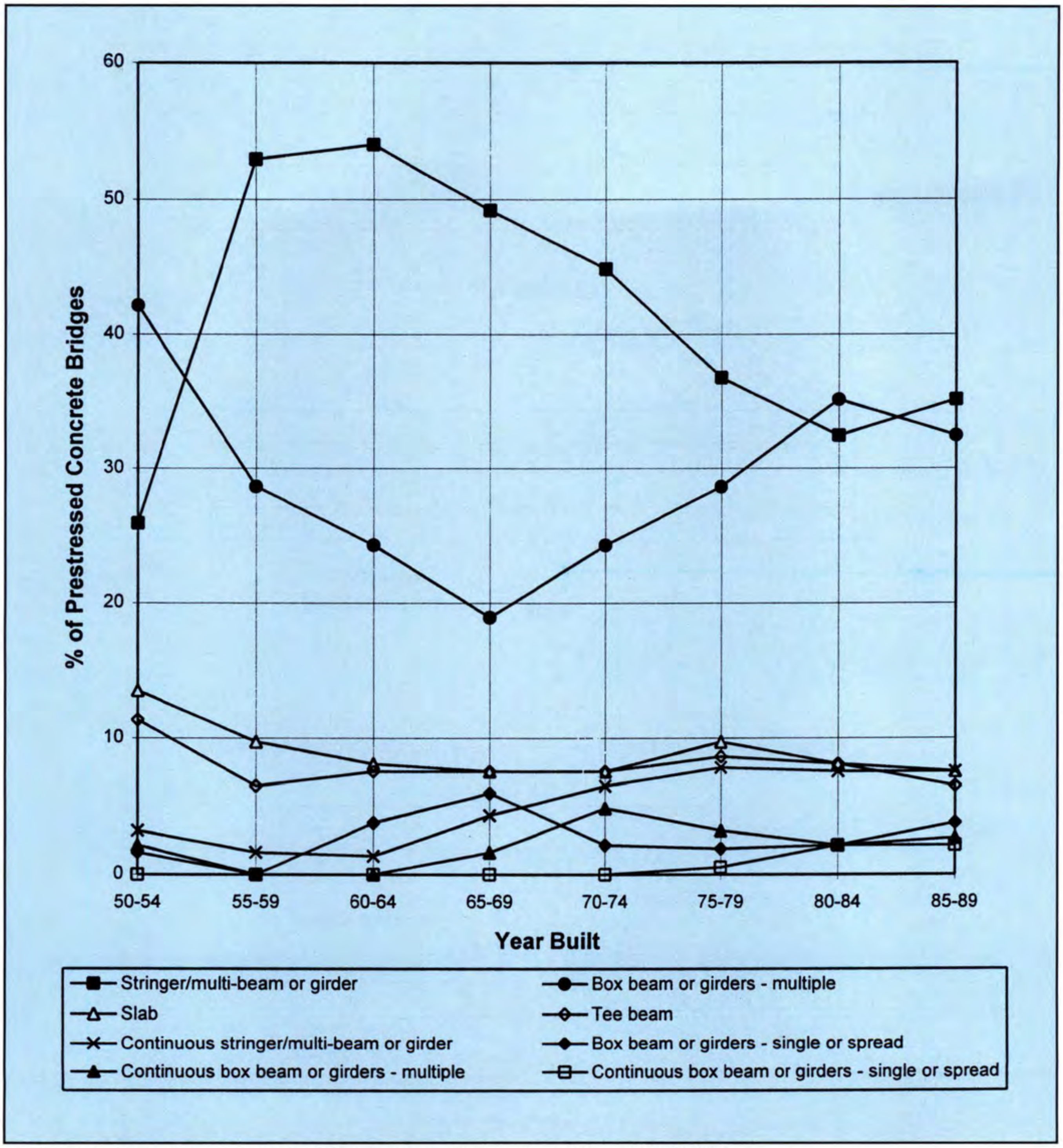


Fig. 1. Percentages of prestressed concrete bridge types built during 1950 to 1989 (Ref.1).

Compared with other types of prestressed concrete highway bridges, tee and single/spread box structures have the highest deficiency percentages. However, since their introduction, no major structural improvements have been made to the system. Therefore, there is definitely good reason to improve these bridge types and thus continue and let grow the already excellent reputation and performance of this category of prestressed concrete bridge.[1]

In adjacent box girder bridges, boxes are placed butted against each other as shown in Fig. 2. Adjacent box girder bridges are widely used in most parts of the United States for spans up to 100 ft (30.5 m) due to ease of erection, shallow superstructure depth, and aesthetic appeal. The girders are generally connected at their interfaces by grouted shear keys and, in some states, are provided with a nominal amount of full-width transverse post-tensioning as shown in Fig. 2. In most applications, a 2 in. (50.8 mm) non-structural wearing surface is added. In a few cases, however, a 5 to 6 in. (127 to 152 mm) structurally composite concrete overlay is used.

Recent surveys of adjacent box girder bridges have revealed frequent

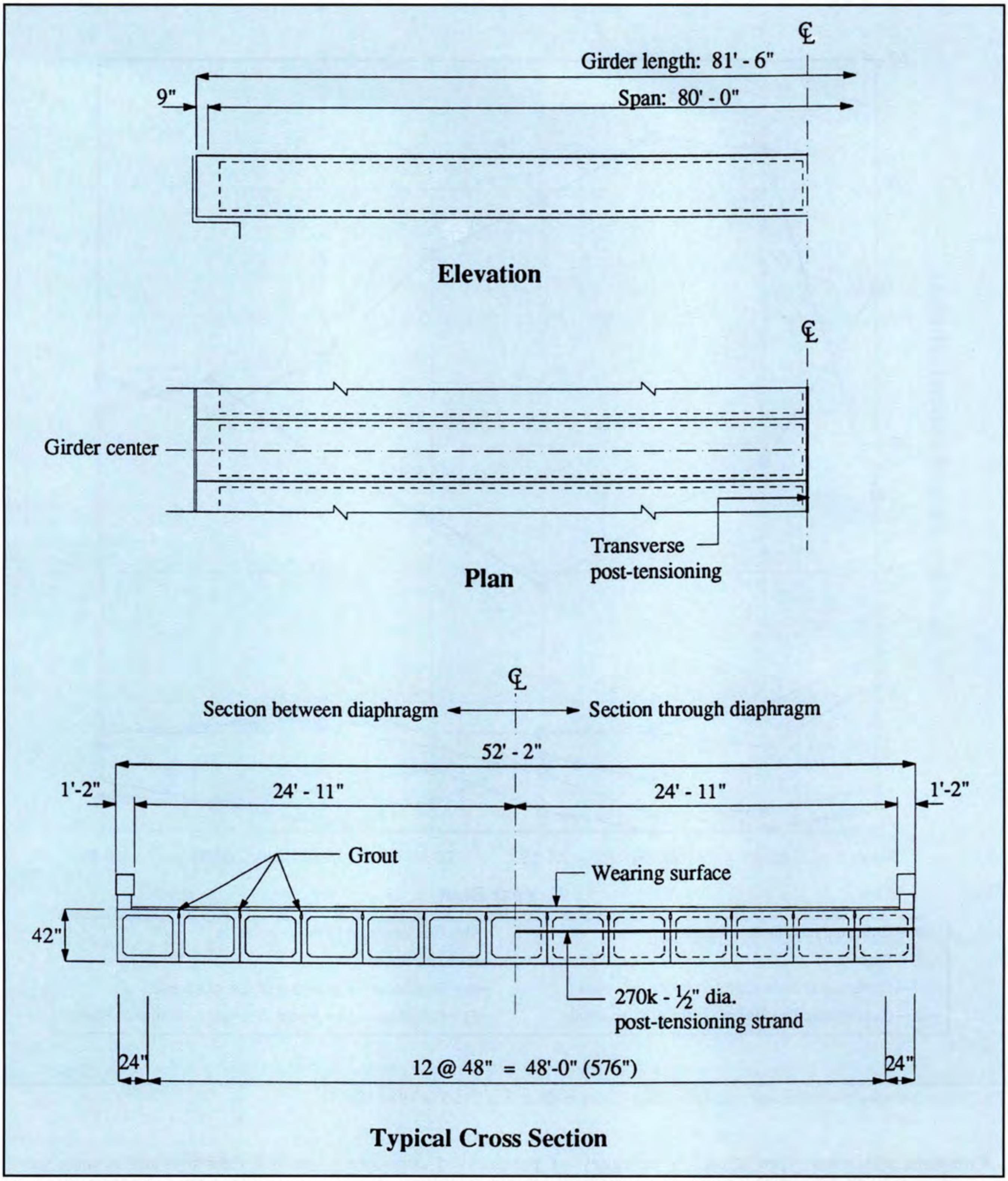


Fig. 2. Elevation, plan and typical cross section of precast box girder bridges in the United States. **Note:** 1 ft = 0.305 m; 1 in. = 25.4 mm.

longitudinal cracking in the grout keys and reflective cracking in the overlay over these keys. In some cases, water and deicing chemicals have penetrated through the cracks, causing concrete staining and spalling and reinforcement corrosion. This is particularly prevalent in bridges without relatively thick composite concrete overlays or inadequate transverse post-tensioning.

Martin and Osborn related the problem of reflective cracking to insufficient consideration of the structural behavior of a bridge.[2] Both shear and bending must be transferred at the transverse joint between girders in order to control both translational and rotational deformation.

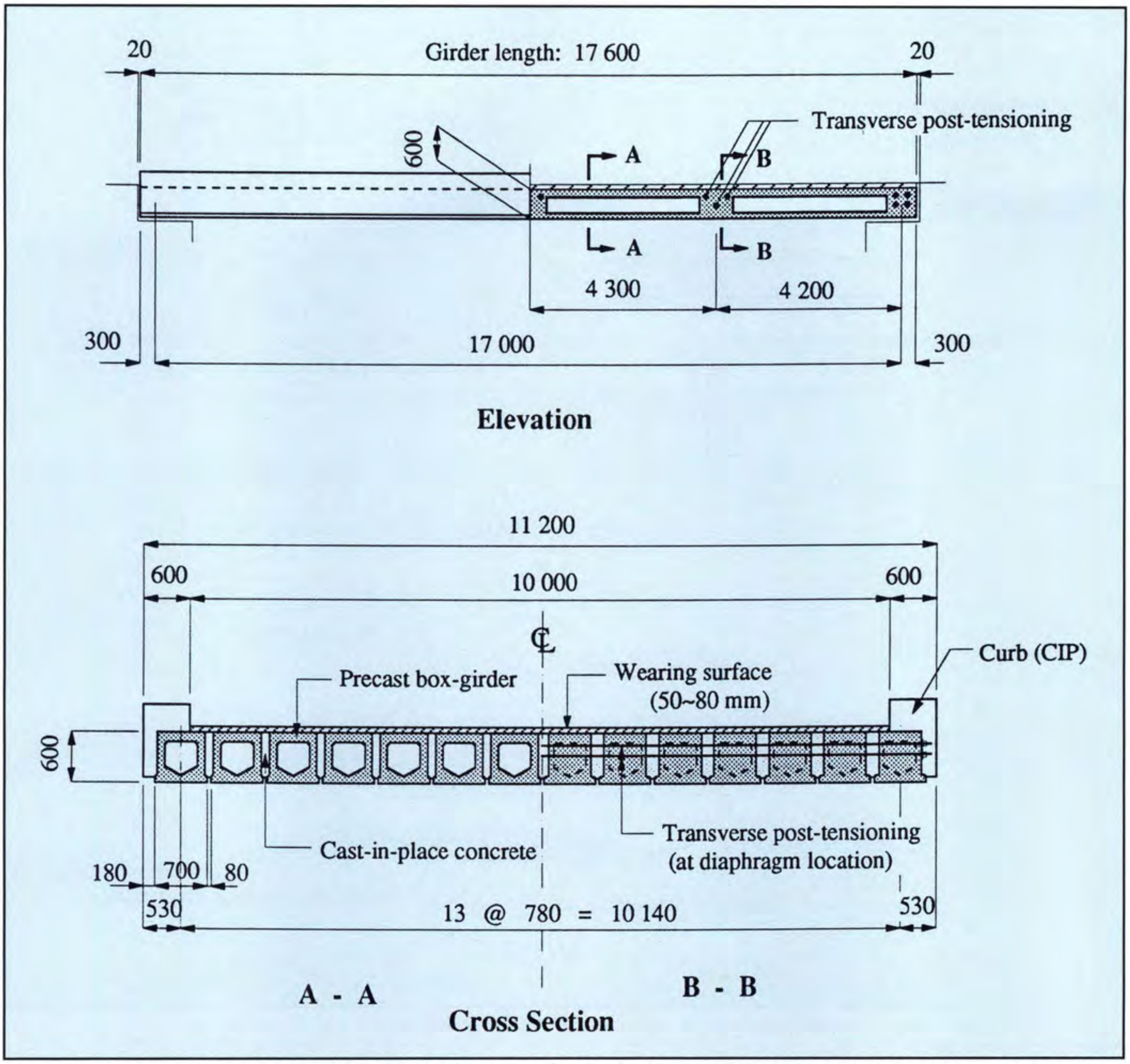


Fig. 3. Precast, pretensioned concrete box girder bridge system for simple short span. **Note:** 1 m = 3.28 ft; 100 mm = 3.94 in.

A few state DOTs, e.g., Michigan, use a combination of heavy structurally composite topping and a large amount of transverse post-tensioning. Composite topping is not a structurally efficient solution because it does not control differential rotation of the box, nor is it an economical solution because a composite concrete topping costs about four times as much as a thin layer of bituminous concrete.

In reviewing the practices in other countries, it was found that cases of longitudinal cracking are seldom reported in Japanese adjacent box girder bridges. Cross-sectional shapes and design criteria for box girders in Japan are similar to those in the United States, except for size and shape of the longitudinal joint between the girders and the amount of transverse post-tensioning.[3] Cast-in-place (CIP) concrete is placed in relatively wide and deep joints between girders, as shown in Figs. 3 and 4, as opposed to narrow mortar-grouted joints in the United States. Higher levels of post-tensioning are used in Japan than is the general practice in the United States.

In the following sections, various design approaches of typical precast, prestressed concrete box girder bridges are discussed. A proposed design is also presented. The design combines the performance requirements for a Japanese bridge with the simplicity of American construction practices. The proposed design involves provision of post-tensioned transverse diaphragms at quarter points of the bridge span.

The diaphragms would be made continuous in the space between the boxes through deep blockouts filled with grout. Post-tensioning is provided based on bridge width and loading, assuming the bridge consists of an assembly of rigidly connected stringers and diaphragms. A preliminary design chart has been developed for simple span bridges of common width and

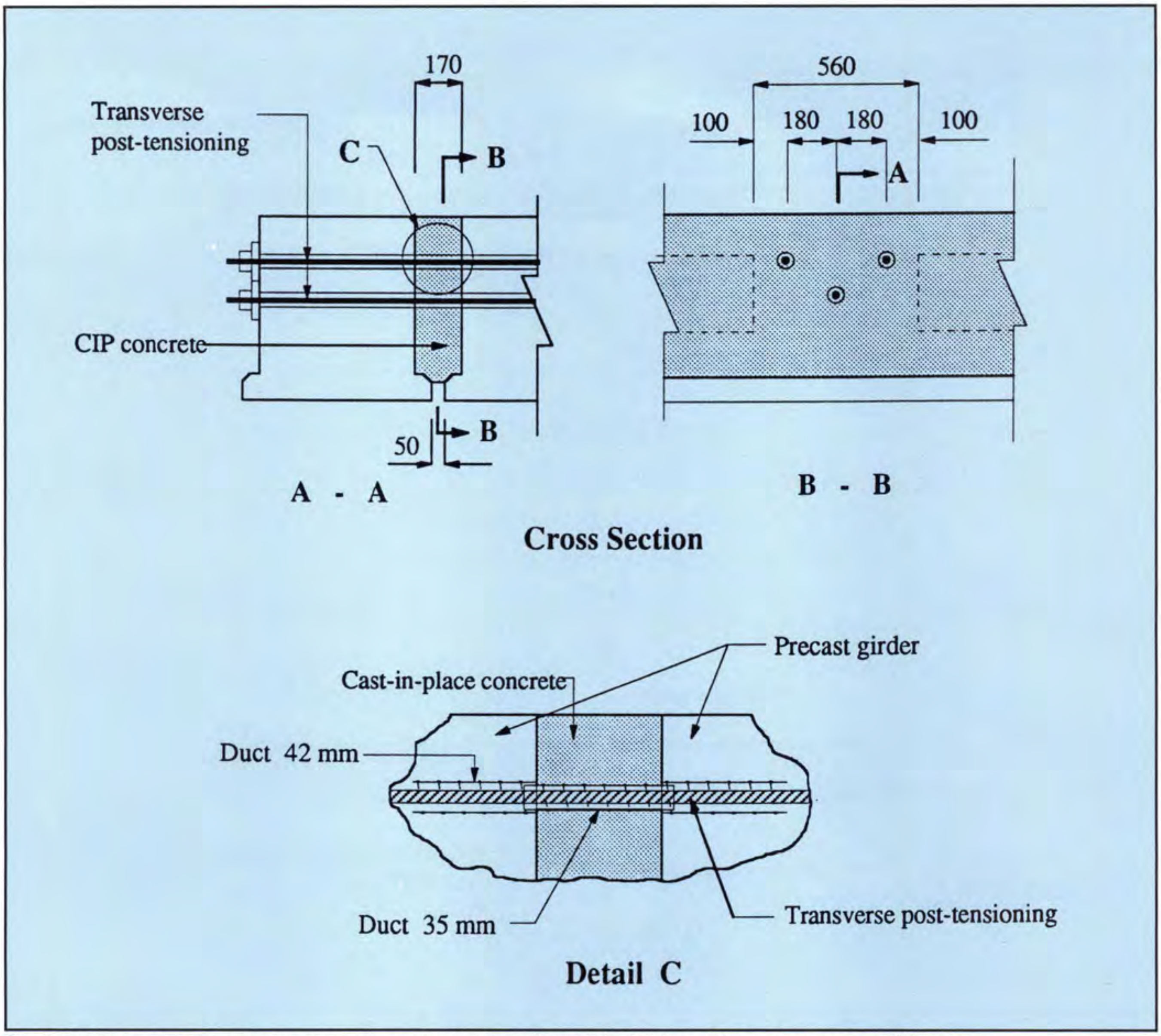


Fig. 4. Transverse post-tensioning arrangement for Japanese box girder bridges. Dimensions are in millimeters.
Note: 100 mm = 3.94 in.

material properties using AASHTO HS-25 live loading.

RECENT STUDIES

In 1992, the PCI Committee on Bridges formed a Subcommittee on Reflective Cracking in Adjacent Box Beam Bridges to study the problem. The subcommittee's report (1995) indicated that at least two national surveys had been conducted with the goal of isolating the causes of reflective cracking.[4] The following are specific questions the committee identified as most pertinent to its investigation:

1. Are there any problems with leakage at the joints between beams?

2. Is a waterproofing material or membrane used over the longitudinal joints?

3. Is skew limited for the use of prestressed box beam bridges?

4. Are shear keys grouted after tensioning the ties?

5. Is there a problem with differential camber between adjacent bands?

6. Are there any problems with uneven sealing of the beam ends for skewed bridges?

7. What material is used for transverse ties?

8. What spacing is specified for transverse ties?

The results of the survey for the first six questions are shown in Fig. 5 in terms of percentage of respondents. The survey data also indicated that 62 percent of the respondents used strands as the transverse tie material while 38 percent used rods. The number of transverse ties varied largely from state to state. Fig. 6 reveals that the number of transverse ties is selected quite arbitrarily.

Case Western Reserve University investigated the performance of shear keys in adjacent box beam bridges in 1993.[5] The five test bridges showed differential deflections between 0.08 and 0.8 in. (2 and 20 mm), which indicated shear key fracture along part or all of the bridge length. The large differential deflection resulted in leakage.

The test structures also showed a satisfactory load distribution among beams after the shear key partially fractured. Mild steel lateral ties were found to be ineffective in resisting differential deflections. The study suggested either moving the shear key down to the neutral axis of the beam or using a stronger epoxy grout in the existing shear key.

West Virginia DOT investigated several high volume, heavily loaded bridges that had joint failure and topping cracking.[4] The investigators concluded that vertical shear failure in the keys was most likely the result of inadequate grout installation and transverse tie force. The ties used for the failed joints were 1 in. (25.4 mm) diameter A36 rods spaced at the third points along the span with an approximately 400 ft-lb (542.3 N-m) torque.

As a result of this investigation, the West Virginia DOT changed its practice as follows:

1. A pourable epoxy is used instead of a non-shrink grout in the shear key.

2. The surfaces to be grouted are sand-blasted.

3. Post-tensioned high strength ties are used.

In Oregon, the practice for adjacent box beams is to begin erection at either one of the exterior beams or at the center of the bridge.[4] After the first two adjacent beams are in place, the transverse tie rods are installed and the nuts are tightened. The sequence continues by placing a beam, installing the appropriate number of tie rods, and tightening the nuts each time.

After all the beams in a span are installed, the bottoms of the shear keys are sealed with a backer rod and the keys are filled with grout. The keys are sandblasted in the precasting plant to remove laitance and enhance bond. The area around the shear keys should be kept moist for 24 hours prior to installing the grout. The grout should be kept moist for a minimum of 72 hours following the installation.

Gulyas, Wirthlin, and Champa undertook a laboratory study to compare non-shrink grouts and magnesium ammonium phosphate mortars.[6] Composite grouted keyway specimens were tested in vertical shear, longitudinal shear, and direct tension. The magnesium ammonium phosphate grouted specimens displayed an exceptionally higher failure load than the non-shrink grout specimens. This result encouraged the use of magnesium ammonium phosphate for keyway grouting applications to eliminate some of the problems encountered with keyways.[6,7]

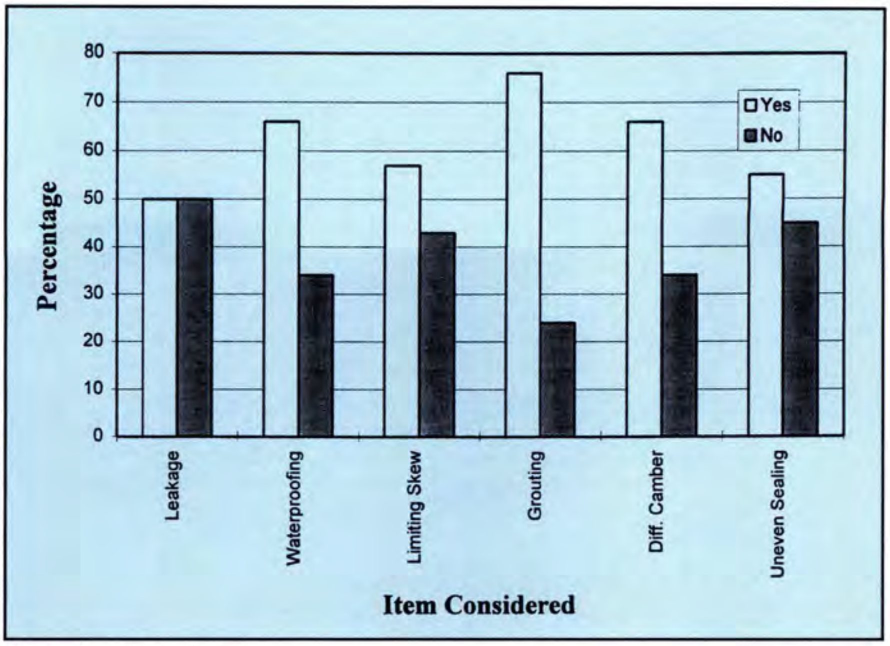


Fig. 5. Survey summary.

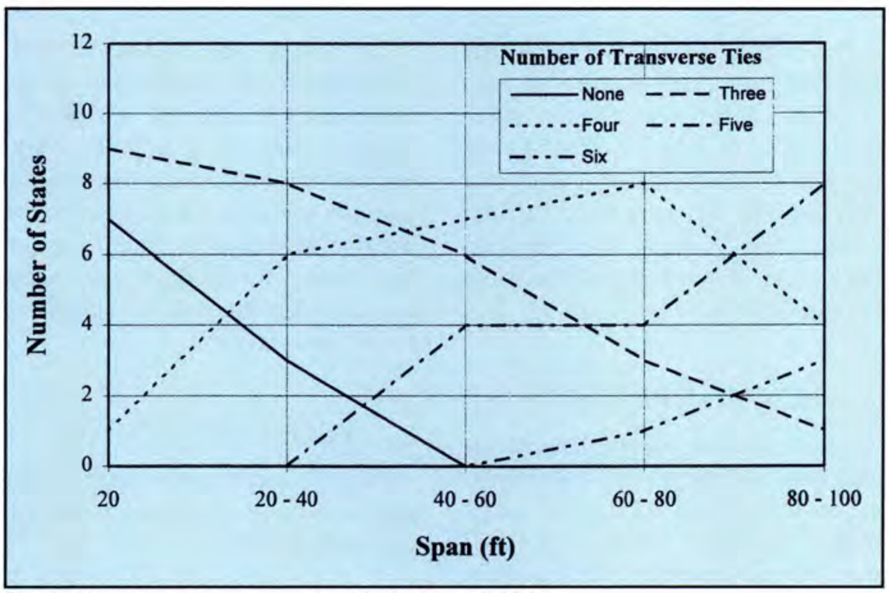


Fig. 6. Number of transverse ties. **Note:** 1 ft = 0.305 m.

The Ontario Bridge Design Code assumes that the transfer of load from one beam to another takes place mainly through transverse shear and that the transverse flexural rigidity is equal to zero.[4,8] Charts such as those shown in Fig. 7 are used to determine the transverse shear force to be resisted by the joint for the appropriate values of span and β, where β is given by the following formula:

$$\beta = \pi \left(\frac{2b}{L} \frac{D_x}{D_{xy}} \right)^{0.5}$$

in which

b = half width of bridge

L = span of bridge

D_x = longitudinal flexural rigidity per unit width

D_{xy} = longitudinal torsional rigidity per unit width

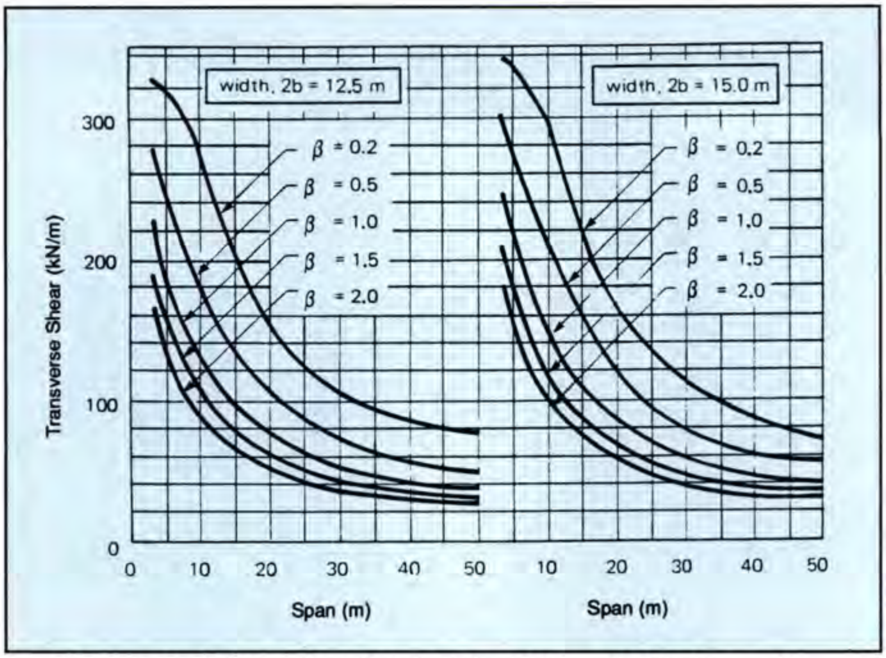


Fig. 7. Ontario Code design chart (Ref. 4). **Note:** 1 kN/m = 0.0685 kips per ft; 1 m = 3.28 ft.

The Ontario Code also requires that adjacent box beam bridges be provided with a reinforced concrete structural slab of at least 5.9 in. (150 mm) thickness capable of providing the shear transfer between the units. Because of this provision, the bridges do not have to rely on a grouted shear key to transfer loads.

JAPANESE PRACTICE

Four to seven equally spaced diaphragms, including end diaphragms, are commonly provided for box girder bridges in Japan.[9] About 6.7 in. (170 mm) of clear spacing in the longitudinal joint between girders is used in Japan to provide for adequate tolerance of differential camber between girders. Box girders and diaphragms are integrated by cast-in-place concrete between adjacent girders and post-tensioning. All highway bridge decks in Japan are covered with a 2 to 3 in. (50 to 80 mm) concrete or asphaltic concrete wearing surface.

Because of the built-in integrity, superimposed dead and live loads are distributed over the entire bridge system in both the longitudinal and transverse directions. The member forces in the box girders and the diaphragms are computed by modeling the bridge deck as a slab or a gridwork of beam elements.

The amount and location of post-tensioning for the diaphragms are determined by the flexural design. The design is primarily based on the working stress method with the flexural strength checked. A shear check is usually waived in standard design. Fig. 4 shows a typical transverse post-tensioning arrangement for a Japanese box girder bridge.

PROPOSED DESIGN

Prior to describing the details of the proposed design, the design methodology is discussed.

Design Methodology

The proposed design involves provision of rigid post-tensioned transverse diaphragms. The diaphragms serve as the primary wheel load transfer mechanism between adjacent boxes. Without diaphragms, each box must be designed to carry a full set of wheel loads without contribution from adjacent boxes. As a result, large differential deflections between girders will take place and reflective cracking could be expected [see Fig. 8(a)]. However, if the box girders are fully connected, the load is distributed over the entire bridge width and the deflected shape becomes a smooth curve [see Fig. 8(b)].

For this design, five diaphragms are provided at a spacing equal to one-fourth of the span. This number was chosen based on a parametric study that was done to determine the appropriate number of diaphragms.

For spans up to 100 ft (30.5 m), by using five diaphragms, two at the ends and three at quarter points, differential deflection is limited to less than 0.02 in. (0.5 mm), which is an acceptable amount. The use of three diaphragms, two at the ends and one at the middle, requires less transverse post-tensioning than a five-diaphragm arrangement. However, the corresponding differential deflection may become unacceptable, which defeats the main purpose of transverse post-tensioning.

Although the proposed five-diaphragm system provides a good balance between performance and economy, an additional parametric study may produce a more optimum solution. Parameters to be considered should include speed of construction, tolerance to differential deflection between girders, and maximum acceptable post-tensioning force.

Precast Section Modification

A minor change in concrete dimensions is recommended to allow for the placement of grout between girders at the diaphragm locations. The modified precast section is shown in Fig. 9. The 1 in. (25 mm) side pockets are provided with internal blockouts on the forms, as shown in Figs. 9 and A1. The proposed change thus does not require any modification of existing steel forms.

Member Force Analysis

Grid analysis is used to determine the member forces. The bridge deck is modeled as a series of beam elements, representing the girders, connected with another series of crossing beam elements, representing the diaphragms. The joints between elements allow for transmission of shear, bending, and torsion.

It is important to select realistic properties of the girder elements to obtain valid results. For example, it may be important to consider shear deformations if the girders are relatively deep and the diaphragm spacing is relatively small. The geometric properties needed for the grid analysis are summarized in Tables 1(a) and 1(b).

Side rails and live loads are the main causes of transverse bending moments generated in the diaphragms. The live load positions are chosen so that maximum positive and maximum negative moments are produced. The live load is placed over the center of the deck for maximum transverse positive moment at midspan. For maximum transverse negative moment at the same location, the load is placed as close to the bridge railing as is allowed.[10] This concept is in general agreement with the results of Gallt's analytical study at the University of Kentucky.[11]

Design and Detailing

It is suggested that the diaphragms be post-tensioned. Designing the diaphragm as a non-prestressed reinforced concrete member is impractical because of the difficulty of projecting reinforcement and splicing it between girders. Also, the absence of precompression would result in possible cracking and leakage. The post-tensioning force should have no eccentricity because the diaphragm experiences significant alternating positive and negative moments.

For working stress design, concrete stresses due to loads and post-tensioning forces are calculated and the total stresses are checked against allowable stress limits. Tensile stress is not permitted because the diaphragm is a composite of both precast and grout components, and cracking of the interface should be avoided. Flexural strength should also be checked.

Construction

The construction process after erecting the girders consists of the following steps:

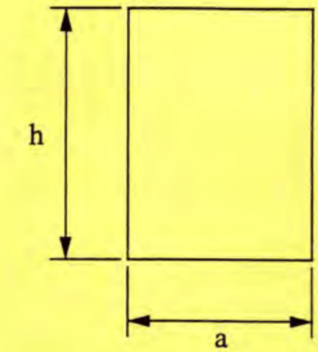
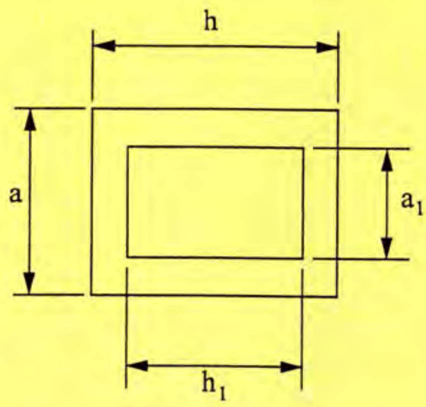
1. Girders are placed and post-tensioning ducts are aligned.

2. Tendons are inserted through the diaphragms.

3. Grout is poured between the girders.

4. When the specified grout strength is reached, transverse post-tensioning is applied.

Table 1(a). Geometric properties needed for grid analysis.

Section type	Geometric properties
Diaphragm 	$A = ah$ $I = ah^3/12$ $J = kha^3$
Box section 	A and I are obtained from Ref. 14. To obtain J, the section dimensions are simplified as shown in the opposite figure. $J = kha^3 - k_1 h_1 a_1^3$

Note: h and h_1 are the longer sides.
A = cross-sectional area; I = moment of inertia; J = torsional inertia; k and k_1 are given in Table 1(b).

Table 1(b). Torsional parameter k for rectangular cross section.

h/a or h_1/a_1	1	1.5	2	2.5	3	4	6	10
k or k_1	0.141	0.196	0.229	0.249	0.263	0.281	0.299	0.312

Design Charts

The four standard AASHTO-PCI box depths, 27, 33, 39 and 42 in. (686, 838, 991 and 1067 mm), were analyzed for three bridge widths of 28, 52 and 84 ft (8.53, 15.85 and 25.6 m). For each combination of section depth and bridge width, three different spans were considered. Appropriate span ranges were considered for each section depth. The required transverse post-tensioning force was found to be almost linearly proportional to the span length.

The design chart shown in Fig. 10 is obtained by dividing the required effective post-tensioning force for the midspan diaphragm by the spacing between diaphragms and then taking the average of the spans analyzed. The transverse post-tensioning should consist of one tendon near the top and another near the bottom in order to provide sufficient flexural strength.

The required post-tensioning force for the quarter-point diaphragms was found to be similar to the midspan diaphragm. It is recommended, therefore, to use the same post-tensioning

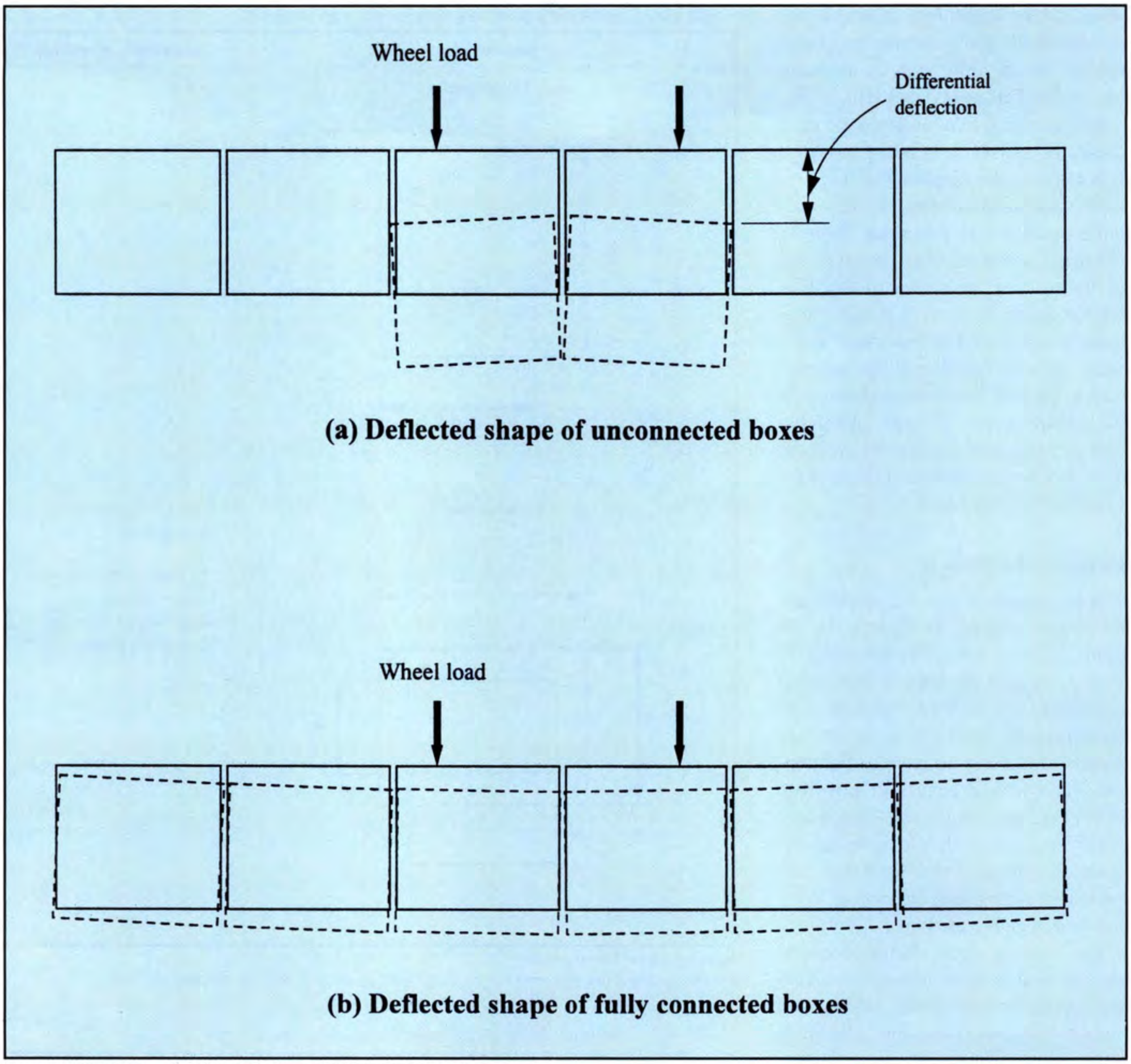


Fig. 8. Schematic of deflected shape of box girder bridge.

force for the quarter-point diaphragms as for the midspan diaphragm. The end diaphragms, however, have almost no bending moments because they are continuously supported over the abutments. For the end diaphragms, it is recommended to use a minimum effective post-tensioning stress of 250 psi (1.72 MPa), applied to the diaphragm cross-sectional area, in order to maintain adequate stiffness at the ends of the bridge.

Table 2. Incremental cost analysis.

Post-tensioning and grout	Materials	Unit cost	Cost
Subtracting post-tensioning (270k $^1/_2$ in. diameter strand)	130 lbs	\$2	– \$260
Adding grout	22.2 cu ft	\$50	+ \$1110
Adding post-tensioning (150k $^1/_4$ in. diameter bar)	1700 lbs	\$2	+ \$3400
Total	—	—	+ \$4250
Incremental construction cost of proposed design = \$4250/(80 × 52) = \$1.02 per sq ft			

Note: 1 ft = 0.305 m; 1 lb = 0.4536 kg.

Cost Analysis

To determine the impact on construction cost, a comparison between a conventional American design and the proposed design is made in Table 2. The proposed design requires grout and increased transverse post-tensioning.

The longitudinal joint between the diaphragms now no longer serves any structural purpose. To make this comparison, five $^1/_2$ in. (12.7 mm) diameter strands, one per diaphragm, are assumed to be used for transverse "ties" in the standard design. They are replaced with the much heavier transverse post-tensioning in the proposed design.

The calculation is made by subtracting the amount of tranverse ties from the proposed transverse post-tensioning to determine the incremental cost. The additional cost of the proposed design is only $1.02 per sq ft. This is a very modest increase in cost considering the projected substantial improvement in durability.

COMPARISON WITH AASHTO LRFD

The AASHTO LRFD Specification states that the use of transverse mild steel rods secured by nuts should not be considered sufficient to achieve full transverse flexural continuity unless demonstrated by test or experience.[12] To make the boxes act together, AASHTO LRFD recommends a minimum average effective post-tensioning pressure of 250 psi (1.72 MPa). However, it does not specify the contact area over which this prestressing force should be introduced. In double-tee applications, for example, it is obvious that the flange area is to be used. In adjacent box applications, it is not clear whether it should be the top shear key area, the diaphragm-to-diaphragm contact area or the full girder side face. Also, it does not specify the spacing between diaphragms nor the diaphragm size. Three different interpretations are compared below for the box type used in the design example.

If the post-tensioning is to be applied over the whole contact area between boxes, the required post-tensioning force per diaphragm is equal to $42 \times 20 \times 12 \times 0.25 = 2520$ kips (11.2 MN). If only the shear key area is to be considered, the required post-tensioning force is $6 \times 20 \times 12 \times 0.25 = 360$ kips (1600 kN) per diaphragm.

Note that if the diaphragm area is used in the calculation, the required post-tensioning force is $8 \times 42 \times 0.25 = 84$ kips (374 kN) per diaphragm. The value obtained by analysis, as shown in Appendix A, is 202 kips

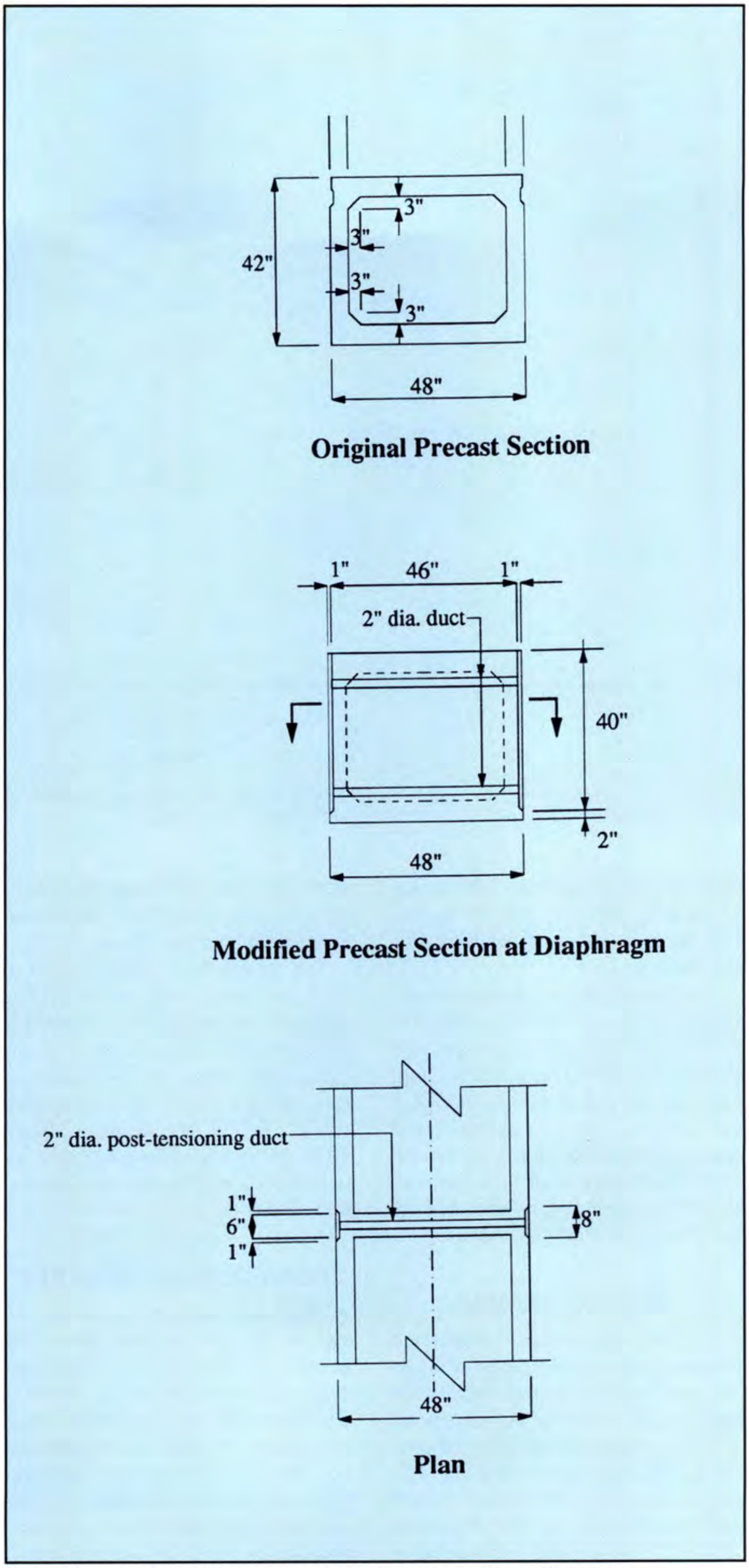


Fig. 9. Comparison of precast concrete sections. **Note:** 1 in. = 25.4 mm.

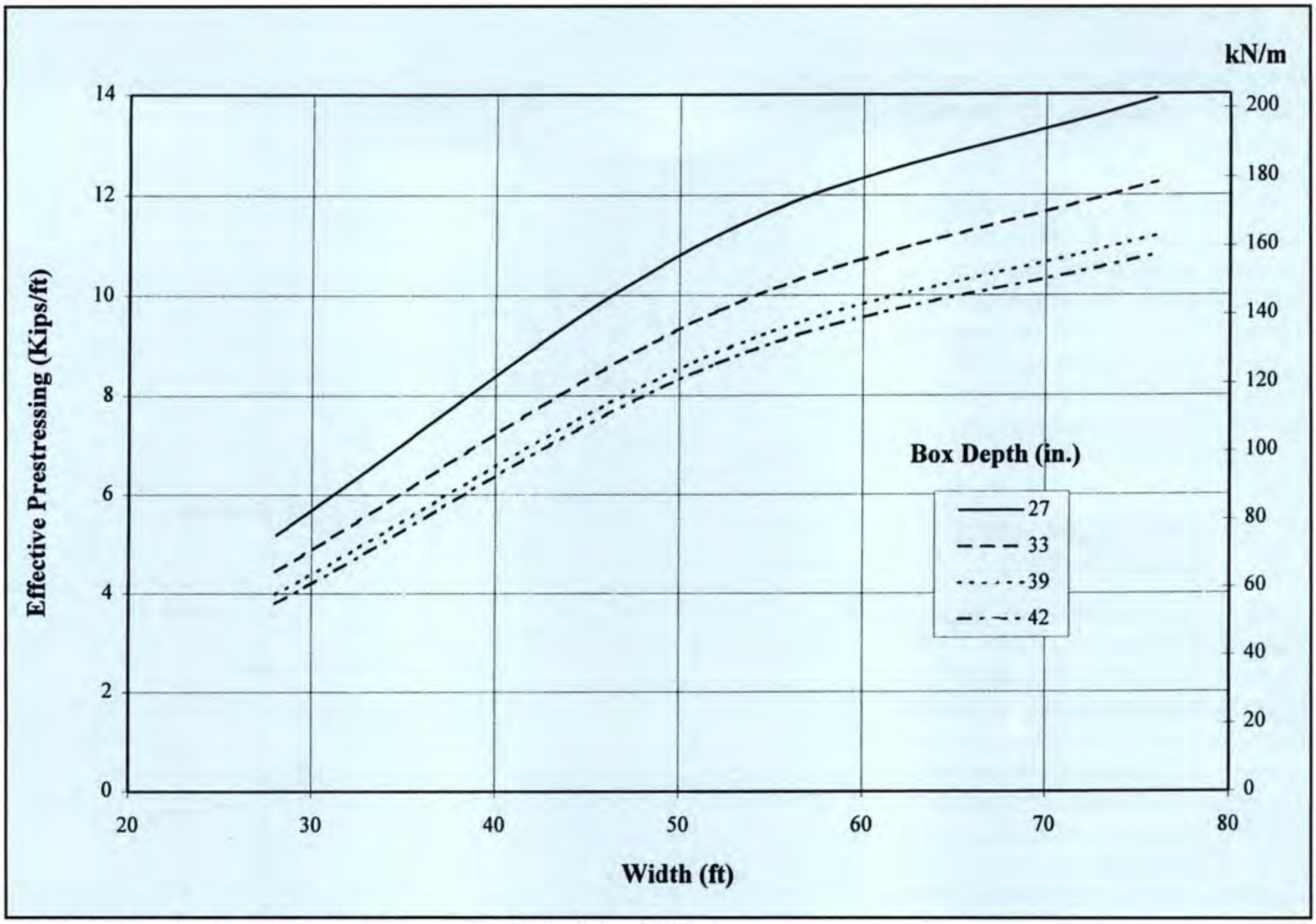


Fig. 10. Prestressing force for midspan diaphragm. **Note:** 1 ft = 0.305 m.

(898 kN) per diaphragm. The design chart in Fig. 10 shows that the applied post-tensioning force depends also on the bridge width.

The above figures show that the recommendation given by AASHTO LRFD is not precise. The authors suggest that the LRFD Specification indicates that the bridge deck be designed as a rigid assembly of gridwork and that post-tensioning acting on the transverse members of that grid, i.e., the diaphragm lines, be designed for not less than 250 psi (1.72 MPa).

DESIGN EXAMPLE

The proposed design is illustrated through a numerical example of a single span bridge. A general view of the bridge is shown in Fig. A1 (Appendix A). The loading arrangement and flexural design criteria are consistent with the provisions of the *AASHTO Standard Specifications for Highway Bridges*.[13] A discussion of the applicability of the AASHTO LRFD provisions[12] has been discussed previously. Step-by-step numerical calculations are provided in Appendix A.

The bridge has a simple span of 80 ft (24.38 m), a width of 52 ft (15.85 m) and is assumed to be subjected to HS-25 truck loading. As the appendix calculations show, the post-tensioning required per diaphragm consists of two 1¼ in. (32 mm) diameter 150 ksi (1034 MPa) post-tensioning bars, one near the top and another near the bottom of the box.

CONCLUDING REMARKS

This paper discusses the current design practices of precast concrete adjacent box girder bridges in the transverse direction. Most state DOTs in the United States use relatively small grout keys and little or no transverse post-tensioning. Some states use large transverse post-tensioning, without any theoretical justification. Japanese practice, on the other hand, requires a detailed analysis of each bridge, use of a very large grout key filled with cast-in-place concrete, and a heavy concentration of transverse post-tensioning. Cases of longitudinal cracking are seldom reported in bridges with heavy full-depth transverse post-tensioning.

This paper provides a methodology for the transverse design of precast concrete box girders without composite topping. It is shown that the transverse post-tensioning needed is almost constant per unit length of the bridge span, and varies significantly with the bridge width. A design chart is offered for a preliminary determination of the post-tensioning required for standard girder depths and common bridge widths. For situations where there is a large skew and where accurate results are required, the detailed grid analysis shown in this paper is recommended.

The construction procedure dictates that post-tensioning be applied after, not before, the shear keys are grouted. Another important feature of the proposed design is to have a full-depth vertical shear key at each diaphragm and the post-tensioning equally divided between the top and bottom of the diaphragm.

The proposed procedure adds about one dollar per square foot to the total cost, which is approximately 2 percent of the bridge cost. The added cost of grouting and post-tensioning is a small price to pay for significantly improved structural behavior and durability.

ACKNOWLEDGMENT

The authors appreciate the review comments received, in particular the detailed review and significant contributions by Jim Gallt, Dick Imper, and George Nasser. This work was supported by the Center for Infrastructure Research of the University of Nebraska and Kyokuto Corporation, Hiroshima, Japan. The authors are indebted to Sameh Badie, Ph. D. student for his considerable assistance in preparing this paper.

REFERENCES

1. Dunker, K. F., and Rabbat, B. G., "Performance of Prestressed Concrete Highway Bridges in the United States — The First 40 Years," PCI JOURNAL, V. 37, No. 3, May-June 1992, pp. 48-64.

2. Martin, L. D., and Osborn, A. E. N., "Connections for Modular Concrete Bridge Decks," FHWA-82/106, NTIS Document PB84-118058, Consulting Engineers Group, Inc., Glenview, IL, August 1983.

3. Yamane, T., Tadros, M. K., and Arumugasaamy, P., "Short to Medium Span Precast Prestressed Concrete Bridges in Japan," PCI JOURNAL, V. 39, No. 2, March-April 1994, pp. 74-100.

4. Subcommittee on Adjacent Box Beam Bridges, "Reflective Cracking in Adjacent Box Beam Bridge Superstructures," PCI Bridges Committee, Second Draft, October 1995.

5. Huckelbridge, A. A. Jr., El-esnawi, H., and Moses, F., "An Investigation of Load Transfer in Multi-Beam Prestressed Box Girder Bridges," prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration, Department of Civil Engineering, Case Western Reserve University, 1993.

6. Gulyas, R. J., Wirthlin, G. J., and Champa, J. T., "Evaluation of Keyway Grout Test Methods for Precast Concrete Bridges," PCI JOURNAL, V. 40, No. 1, January-February 1995, pp. 44-57.

7. Gulyas, R. J., Wirthlin, G. J., and Champa, J. T., Discussion of "Evaluation of Keyway Grout Test Methods for Precast Concrete Bridges," PCI JOURNAL, V. 40, No. 4, July-August 1995, pp. 99-103.

8. Bakht, B., Jaeger, L. G., and Cheung, M. S., "Transverse Shear in Multibeam Bridges," *Journal of Structural Engineering*, American Society of Civil Engineers, V. 109, No. 4, April 1983, pp. 936-949.

9. Japanese Industrial Standard, *Prestressed Concrete Girders for Slab Bridges*, JIS A 5313, Japanese Standards Association, Tokyo, Japan, 1991.

10. *Specification for Highway Bridge*, Japan Road Association, Tokyo, Japan, February 1990 (in Japanese).

11. Gallt, J. G., "Application of the Analytic Strip Method to Adjacent Box Beam Bridge Superstructures," *Lecture Note*, University of Kentucky, obtained by private correspondence with James Gallt, Palmer Engineering, Lexington, Kentucky, November 1994.

12. *AASHTO LRFD Bridge Design Specifications*, First Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1994.

13. *AASHTO Standard Specifications for Highway Bridges*, Fifteenth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1992.

14. *Precast Prestressed Concrete Short Span Bridges*, Precast/Prestressed Concrete Institute, Chicago, IL, 1988.

APPENDIX A — DETAILED CALCULATIONS FOR DESIGN EXAMPLE

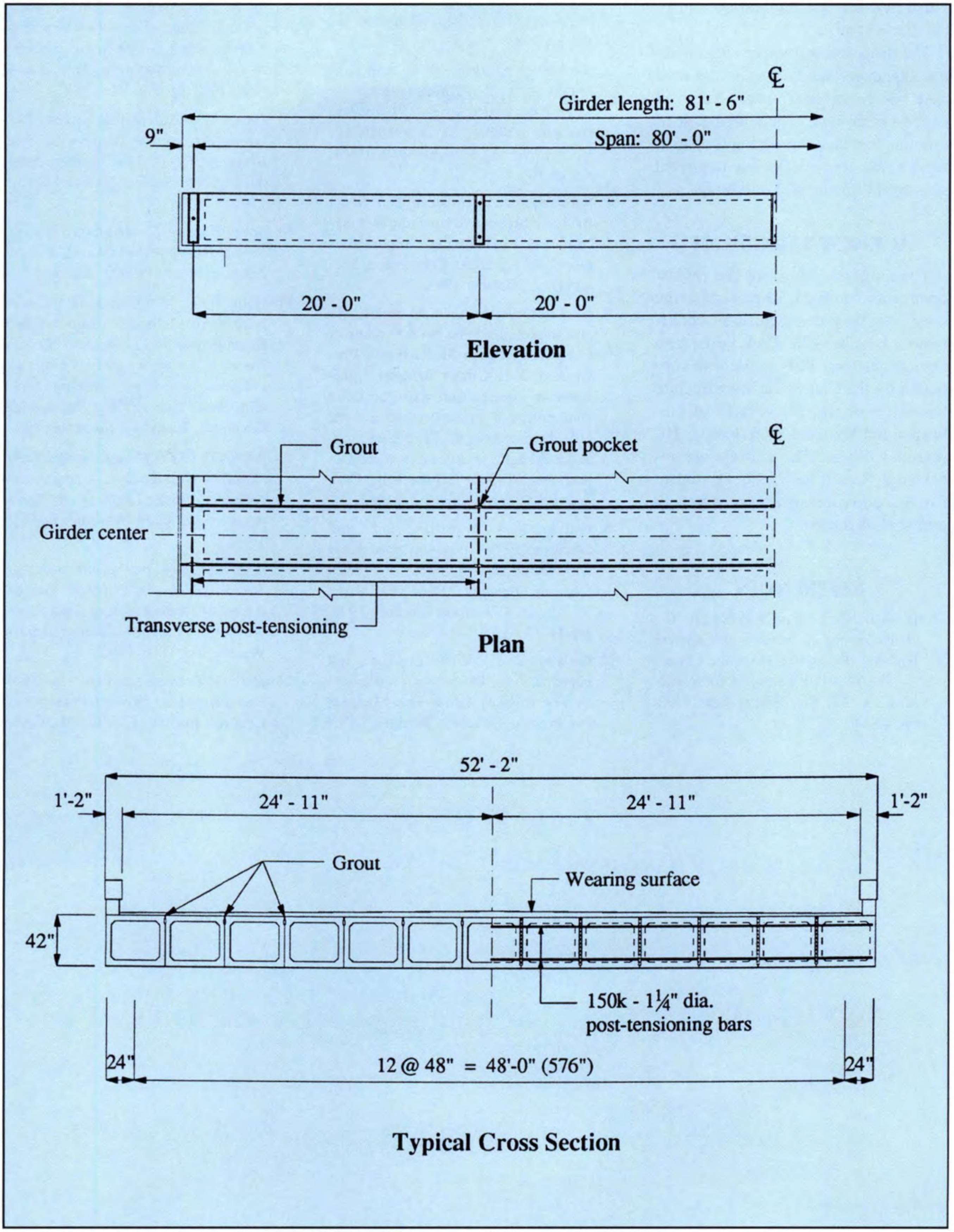


Fig. A1. General view of bridge for proposed design. **Note:** 1 in. = 25.4 mm; 1 ft = 0.305 m.

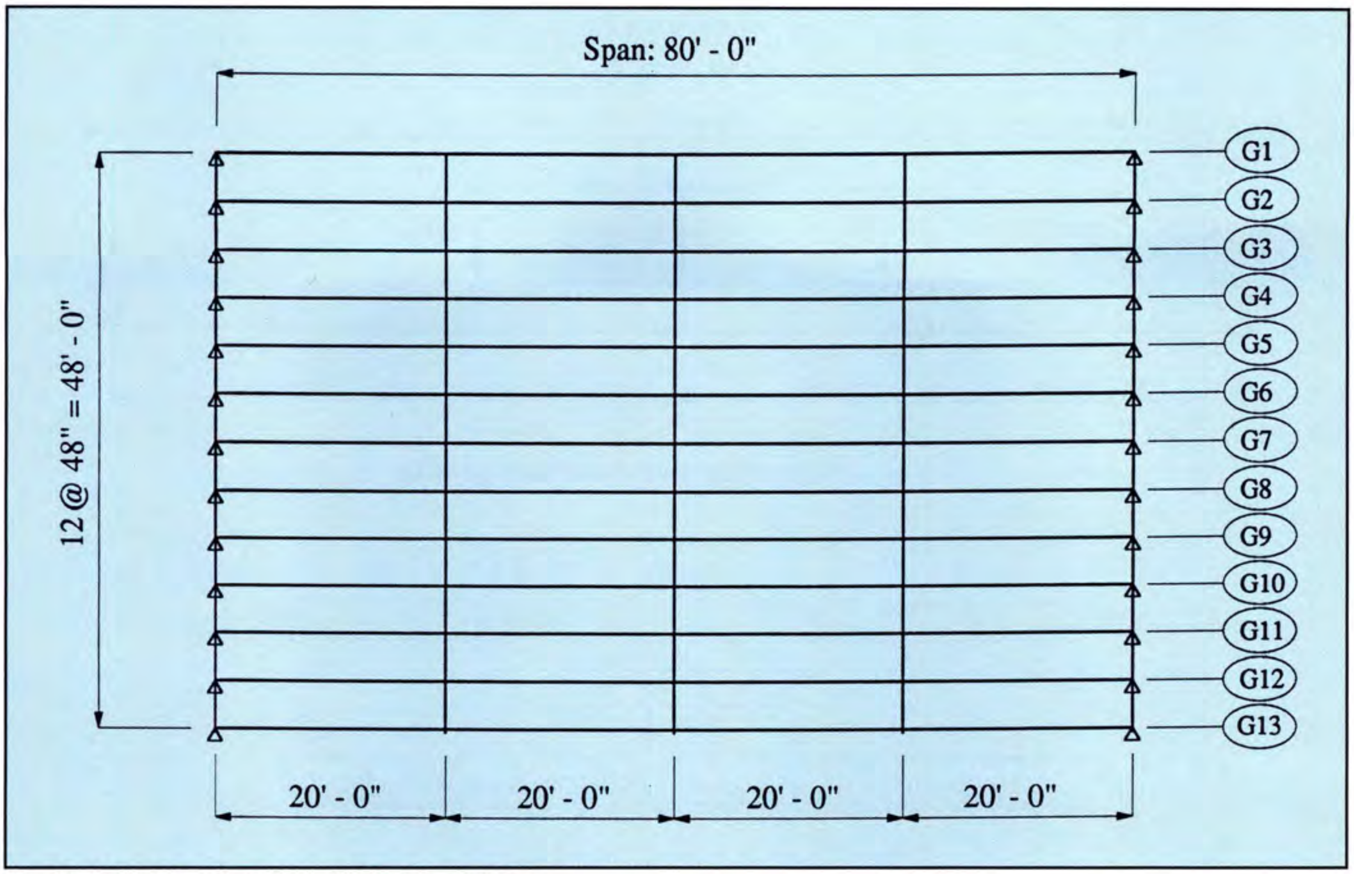


Fig. A2. Model of grid analysis. **Note:** 1 ft = 0.305 m.

The calculations below follow the *AASHTO Standard Specifications for Highway Bridges.*[13] All provisions and equations referenced below correspond to the AASHTO Specifications. The general view of the bridge is shown in Fig. A1.

Design Description

Span: 80 ft (24.38 m)
Total width: 52 ft (15.85 m)
Live load: HS-25
Girder spacing: 48 in. (1219 mm)
Concrete strength:

$f'_{c\ precast}$ = 7500 psi (52 MPa)
$f'_{c\ grout}$ = 7500 psi (52 MPa)

Impact:

$$I = \frac{50}{80+125} = 0.244$$

Section Properties

Box girder:[14]

A = 842.5 sq in. (544×10^3 mm^2)
I = 203,088 in.^{4} (845×10^8 mm^4)
J = 366,849 in.^{4} (153×10^9 mm^4)

Diaphragm: The cross section of the diaphragm is rectangular. The depth of the diaphragm is equal to the depth of the box, in this example 42 in. (1067 mm), while the width is 8 in. (203 mm). Thus, the properties of the diaphragm are as follows:

A = 336 sq in. (217×10^3 mm^2)
I = 49,392 in.^{4} (206×10^8 mm^4)
J = 6279 in.^{4} (261×10^7 mm^4)

Grid Analysis

The grid analysis was performed to compute bending moments in the diaphragms. Fig. A2 shows the structural model for the bridge. Horizontal lines represent box girders and vertical lines represent diaphragms.

Loadings

Dead load: Assume solid concrete curb and railing:

w = 0.48 kips per ft (7 kN/m)

Only a concrete rail weight was applied to Girders G1 and G13 as superimposed dead load because the almost uniformly distributed wearing surface does not produce significant bending moments in the diaphragms.

Live load: Lane live loads and truck loads were applied separately and the larger moments produced were used for design.

Truck loading: Truck loads were imposed according to AASHTO Specifications. The truck loading positions that produce maximum moments in the midspan diaphragm are shown in Fig. A3. Point loads that are not located at girder centerlines are converted to equivalent point loads at the girder centers through straight-line proportions as shown in Figs. A3(b) and A3(c). For example, referring to Fig. A3(b), the first point load is assumed to act entirely on Girder G5 while the second point load is equally divided between Girders G6 and G7. For the case of positive moment, the cases of loading one lane and three lanes were also checked.

Lane loading: The equivalent lane loading positions that produce maximum moments are similar to those for truck loading. Lane loading consists of a distributed load of 0.08 kips per sq ft (3.83 kN/m^2) and a line load of 2.25 kips per ft (32.83 kN/m), over a lane

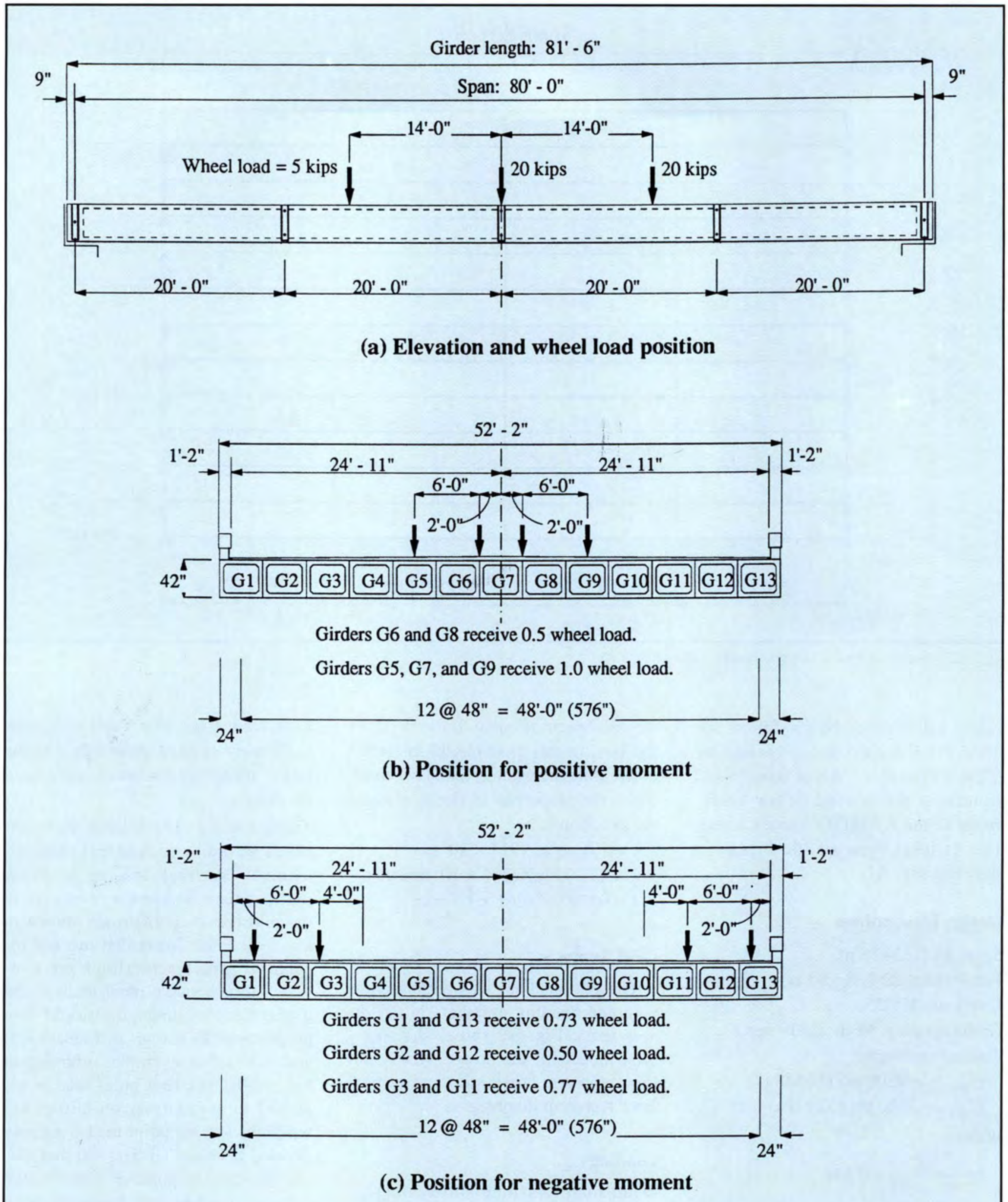


Fig. A3. Loading positions for truck loading. **Note:** 1 ft = 0.305 m; 1 in. = 25.4 mm.

width of 10 ft (30.5 m). Each girder is assumed to carry the part of the load that lies within a width equal to half the spacing between girders on each side. Accordingly, Girders G5 through G9 receive equal shares of the two-lane load for positive moments, i.e., each girder receives 40 percent of the lane load. Similarly, for negative moments, Girders G1 and G13 receive 29 percent of the lane load each, while Girders G2 and G12 receive 40 percent of the lane load each and Girders G3 and G11 receive 31 percent of the lane load, for a total of two lanes.

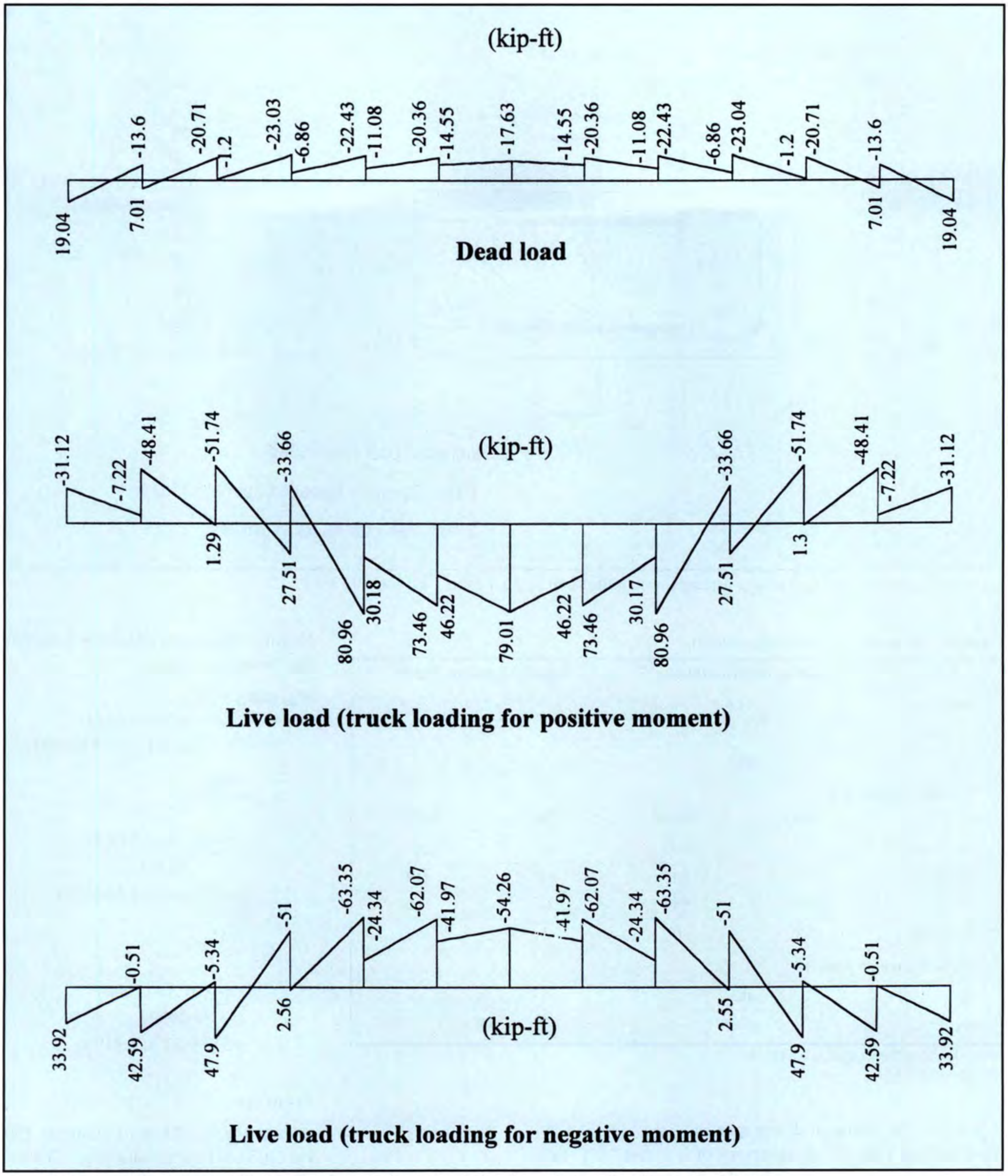


Fig. A4. Moment diagram in midspan diaphragm due to truck loading. **Note:** 1 kip-ft = 1.356 kN-m.

Member Force Analysis

Moments of the span center diaphragm are used for design. In this example, the truck loading rather than the equivalent lane loading controlled the design. The resulting bending moment diagrams are shown in Fig. A4.

Working Stress Design

Concrete stresses at prestress transfer are satisfactory, because the prestress eccentricity is zero and no moment exists at that time. Allowable compressive stress due to effective prestress plus maximum load is 0.6 × 7500 = 4500 psi (31 MPa) according to AASHTO Specifications, 1995 Interim.

Tension is not allowed as explained earlier. Design calculations show that two $1^1/_4$ in. (32 mm) diameter, 150 ksi (1034 MPa) post-tensioning bars are

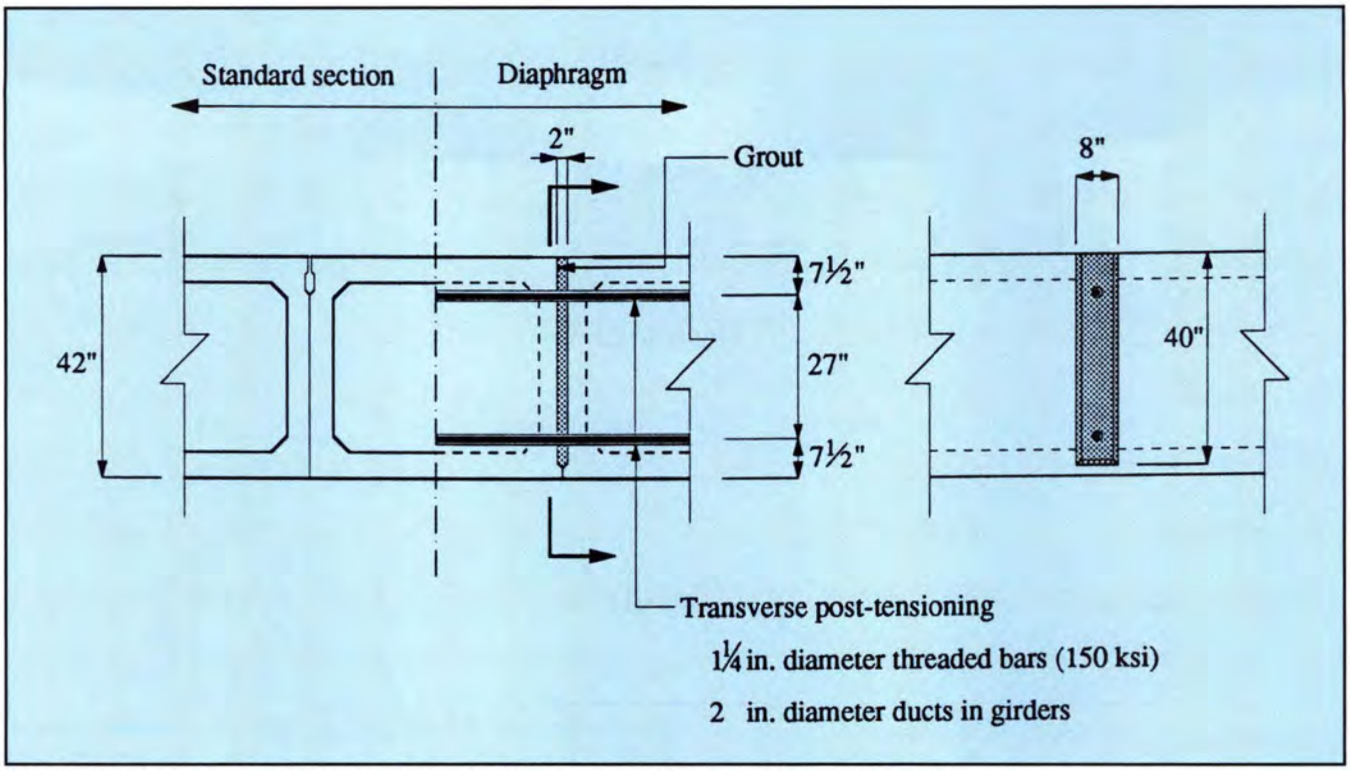


Fig. A5. Transverse post-tensioning arrangement. **Note:** 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

Table A1. Summary of diaphragm design.

Load	Positive moments (kip-ft)		Negative moments (kip-ft)	
Dead load	–17.6		–22.4	
Live load *	98.3		–78.9	
Total	80.7		–101.3	
Working stresses (psi)				
	Top	**Bottom**	**Top**	**Bottom**
Dead + live	412	–412	–517	517
Prestress	601	601	601	601
Total	1013	189	84	1118
Allowable	4500	0	0	4500
Flexural strength (kip-ft)				
ϕM_n	483		483	
M_u	190.5		200.3	

Note: 1 kip-ft = 1.356 kN-m; 1 psi = 6.895 kPa.
* Includes impact.

required at the midspan diaphragm as illustrated in Fig. A5. A summary of stress analysis is shown in Table A1.

Stresses due to service loads:

Positive moment: Design moment due to service load:

$$M_{tot} = M_D + M_{L+I}$$
$$= -17.6 + (79.0 \times 1.244)$$
$$= 80.7 \text{ kip-ft } (109.4 \text{ kN-m})$$

$$f_{top} = \frac{M_{tot}}{I} y_{top}$$
$$= \frac{(80.7 \times 12000)}{49392} \times 21$$
$$= 412 \text{ psi } (2.84 \text{ MPa})$$

$$f_{bot} = \frac{M_{tot}}{I} y_{bot}$$
$$= \frac{(80.7 \times 12000)}{49392} \times (-21)$$
$$= -412 \text{ psi } (-2.84 \text{ MPa})$$

Negative moment: Design moment due to service load:

$$M_{tot} = M_D + M_{L+I}$$
$$= -22.4 - (63.4 \times 1.244)$$
$$= -101.3 \text{ kip-ft } (-137.3 \text{ kN-m})$$

$$f_{top} = \frac{M_{tot}}{I} y_{top}$$
$$= \frac{(-101.3 \times 12000)}{49392} \times 21$$
$$= -517 \text{ psi } (-3.56 \text{ MPa})$$

$$f_{bot} = \frac{M_{tot}}{I} y_{bot}$$
$$= \frac{(-101.3 \times 12000)}{49392} \times (-21)$$
$$= 517 \text{ psi } (3.56 \text{ MPa})$$

Prestress

Two 1¼ in. (32 mm) diameter 150 ksi (1034 MPa) bars are used. The arrangement is shown in Fig. A5.

Prestressing force: The effective prestress is assumed to be 55 percent of the ultimate strength of the bar:

$$P_e = 0.55 f_s' A_{ps}$$
$$= 0.55(150)(1.23)$$
$$= 101 \text{ kips per bar } (449 \text{ kN per bar})$$

$$\Sigma P_e = 101 \times 2 \text{ bars}$$
$$= 202 \text{ kips } (898 \text{ kN})$$

Concrete stresses due to effective prestressing force:

$$f_{top} = f_{bot} = \frac{202000}{336}$$
$$= 601 \text{ psi } (4.14 \text{ MPa})$$

Total stresses

For positive moment:

$\Sigma f_{top} = 412 + 601$
$= 1013$ psi (6.98 MPa)
< 4500 psi (31.0 MPa)(ok)

$\Sigma f_{bot} = -412 + 601$
$= 189$ psi (1.30 MPa)
> 0 psi (0 MPa)(ok)

For negative moment:

$\Sigma f_{top} = -517 + 601$
$= 84$ psi (0.58 MPa)
> 0 psi (0 MPa)(ok)

$\Sigma f_{bot} = 517 + 601$
$= 1118$ psi (7.71 MPa)
< 4500 psi (31.0 MPa)(ok)

Flexural strength check:

The results of the flexural strength checks are shown in Table A1.

Ultimate positive moment:

$M_u = 1.3(M_D + 1.67M_{L+I})$
$= 1.3(-17.6 + 1.67 \times 79.0 \times 1.244)$
$= 190.5$ kip-ft (258.3 kN-m)

Ultimate negative moment:

$M_u = 1.3(M_D + 1.67M_{L+I})$
$= -1.3(22.4 + 1.67 \times 63.4 \times 1.244)$
$= -200.3$ kip-ft (−271.6 kN-m)

Stress in prestressing steel at ultimate load:

$$f^*_{su} = f'_s\left[1 - \left(\frac{\gamma}{\beta_1}\right)\left(\rho\frac{f'_s}{f'_c}\right)\right]$$

AASHTO Eq. (9-17)

$A_s = 1.23$ sq in. (793.6 mm²)
$d = 34.5$ in. (876.3 mm)

$$\rho = \frac{A_s}{bd} = \frac{1.23}{(8)(34.5)} \approx 0.00446$$

$f'_c = 7500$ psi (52 MPa)
$f'_s = 150$ ksi (1034 MPa)
$\gamma = 0.28$ (AASHTO Article 9.1.2)
$\beta_1 = 0.85 - 0.05(7.5 - 4.0) = 0.675$ (AASHTO Article 8.16.2.7)

$$f^*_{su} = (150)\left[1 - \left(\frac{0.28}{0.675}\right) \times \left(0.00446 \times \frac{150}{7.5}\right)\right]$$
$$= 144 \text{ ksi } (993 \text{ MPa})$$

Nominal strength for both positive and negative moments:

$$M_n = A_{ps}f^*_{su}d\left(1 - 0.6\frac{\rho f_{su}}{f'_c}\right)$$

AASHTO Eq. (9-13)

$$= (1.23)(144)(34.5) \times \left[1 - 0.6\frac{(0.00446)(144)}{7.5}\right]$$
$$= 5797 \text{ kip-in. } (654.9 \text{ kN-m})$$

$$\phi M_n = (1.0)(5797) \times \frac{1}{12}$$
$= 483$ kip-ft (654.9 kN-m)
$> M_u = 190.5$ kip-ft (258.3 kN-m)
$> M_u = 200.3$ kip-ft (271.6 kN-m)

Maximum prestressing steel (AASHTO Article 9.18.1):

$$\frac{\rho f^*_{su}}{f'_c} = \frac{(0.00446)(144)}{(7.5)} = 0.086$$
$$< 0.36\beta_1 = 0.243$$

APPENDIX B — NOTATION

a, a_1 = concrete dimensions defined in Table 1(a)
A = area of concrete section
A_{ps} = area of prestressed tension reinforcement
b = half width of bridge
d = distance from extreme compression fiber to centroid of tension reinforcement
D_x = longitudinal flexural rigidity per unit width
D_{xy} = longitudinal torsional rigidity per unit width
f_{bot} = concrete stress at extreme bottom fiber
f'_c = specified compressive strength of concrete
f'_s = ultimate strength of prestressing steel
f^*_{su} = stress in prestressing steel at ultimate load
f_{top} = concrete stress at extreme top fiber
h, h_1 = concrete dimensions defined in Table 1(a)
I = impact fraction
I = moment of inertia about centroid of cross section
J = torsional inertia
k, k_1 = torsional parameters
L = span of bridge
M_D = moment due to dead load
M_{L+I} = moment due to live load including impact
M_n = nominal moment strength of a section
M_{tot} = total moment due to service load
M_u = factored moment at section
P_e = effective prestressing force
y_{top} = distance from centroidal axis of section to top fiber
y_{bot} = distance from centroidal axis of section to bottom fiber
w = uniform load
β_1 = factor for concrete strength
ϕ = strength reduction factor
ρ = tension reinforcement ratio
γ = factor for type of prestressing steel