The comments and opinions expressed herein are those of the contributing authors and do not necessarily reflect official PCI policy. Some of the provided answers may have alternate solutions. Reader comments are invited.

Flexural Strength of Continuous Bridge Girders – Avoiding the Penalty in the AASHTO LRFD Specifications

Q1: What methods of creating continuity of precast, prestressed concrete I-girders are available, and what are the benefits of each?

A1: Precast, prestressed concrete I-girders can be efficiently designed and constructed in simple- or continuous-span arrangements. There are basically four systems that are used to achieve continuity:

System A: This is the most common system used at this time. Mild reinforcement is placed in the deck over the piers. When the deck concrete cures and gains strength, additional loads from traffic barriers, wearing surface, and live load can be resisted by the continuous composite girder/slab system. Construction of this system is simple. But the superstructure is continuous only for a small portion of the total load. If one assumes that the girder self-weight, the deck weight, and the live load each contribute about one-third of the total load, then this system is potentially continuous for only one-third of the total load.

The relatively high pretension force, compared to that required in the other systems described below, causes creep growth of member camber, which is restrained by the pier diaphragms. The lack of permanent negative moment over the piers may create a net positive restraint moment due to creep and cause bottom cracking at the piers. This "softening" of the negative moment region over the piers further reduces the continuity of the system.[1]

System B: This system is relatively new. Girders are coupled using high strength threaded rods that are embedded in the top flange during fabrication. The threaded rods are coupled in the field at the diaphragms over the piers. The diaphragm concrete is then placed. Deck placement begins when the diaphragm concrete gains adequate strength. See Ma et al.[2] for more details. The first bridge using this system has been designed and is scheduled for construction near Clarks, Nebraska, in the fall of 2002.

In contrast to System A, this system allows the superstructure to be continuous for about two-thirds of the total load. The threaded rod system resists the negative moments due to deck weight. After the deck concrete has hardened, reinforcing bars in the deck help to resist the negative moments due to superimposed dead and live load. Accordingly, System B can increase the span capacity of a given girder size by 10 to 15 percent over System A. Unlike System A, this method of construction creates a significant permanent negative moment that generally exceeds the positive restraint moment due to creep and eliminates any need for bottom reinforcement (for crack control) over the piers.

System C: In this system, continuity is created through longitudinal post-tensioning of the full length of the bridge. The structural design is optimized when post-tensioning is partially introduced before deck placement. A second stage of post-tensioning after the deck has hardened helps prestress the deck and extend its life. (It is the policy of the Nebraska Department of Roads and other state highway agencies to fully apply post-tensioning before the deck is placed. This is done to avoid calling in specialty post-tensioning contractors more than one time and to guarantee that no special requirements are needed when the decks require removal and replacement.)

This is an effective system, especially if spliced segmental I-beams are needed for spans longer than shipping capabilities of single-piece spans. Several references are available on this system, including a number of recent PCI JOURNAL papers, such as Ronald,[3] and the PCI Bridge Design Manual.[4]

System D: This system creates continuity through coupling the ends of the top strands. The method of coupling may involve jacking of these strand ends to restore the prestressing that had existed before prestress release in the precasting operation. This system offers the same structural benefits as full-length post-tensioning, but it does not require the expensive post-tensioning anchorage hardware at the tendon ends nor the post-tensioning ducts and grout. Rather, it requires a special coupling device and possible jacking in the field.

It was demonstrated successfully for a pedestrian bridge near Memorial Stadium in Lincoln, Nebraska.[5] Despite the structural efficiency of the resulting system, it has not been widely used in practice.

Q2: What are the advantages and disadvantages of superstructure continuity?

A2: The principal advantages include the following:

1. The maintenance cost is lower than that of jointed bridges, where moisture penetration can cause deterioration of the member ends and supports.
2. Girder span capacity is increased compared to simple span construction, especially when Systems B, C, and D are used.
3. With System B, C, or D, positive moment reinforcement for resisting creep is not needed, and bottom cracking is avoided near the pier.

Opponents of continuity contend that simple-span bridges are easier to build, do not induce forces due to support settlement, and do not cause negative moment transverse cracking in the deck.

Q3: What controls the design of bridge girders made continuous?

A3: System B is considered here for illustration purposes. The span capacity of the Nebraska I-girder known as NU1100 [1100 mm (43.3 in.) deep] at various girder spacings is shown in Fig. 1.

Four criteria were considered in developing the chart: maximum positive moment section (working stress design), maximum positive moment section (strength design by AASHTO Standard Specifications), maximum negative moment section (strength design by AASHTO Standard Specifications), and maximum shear section near the pier.

A fifth criterion, used for illustrative purposes but not recommended in design for the reasons given in Answers A4 and A5, is maximum negative moment section (strength design by AASHTO LRFD Specifications). Also indicated in Fig. 1 is the maximum transportable length in Nebraska at this time, which is 160 ft (48.7 m). The figure shows that the design is primarily controlled by negative moment capacity. For this chart, the concrete strength of the girders is assumed to be 8000 psi (55 MPa).

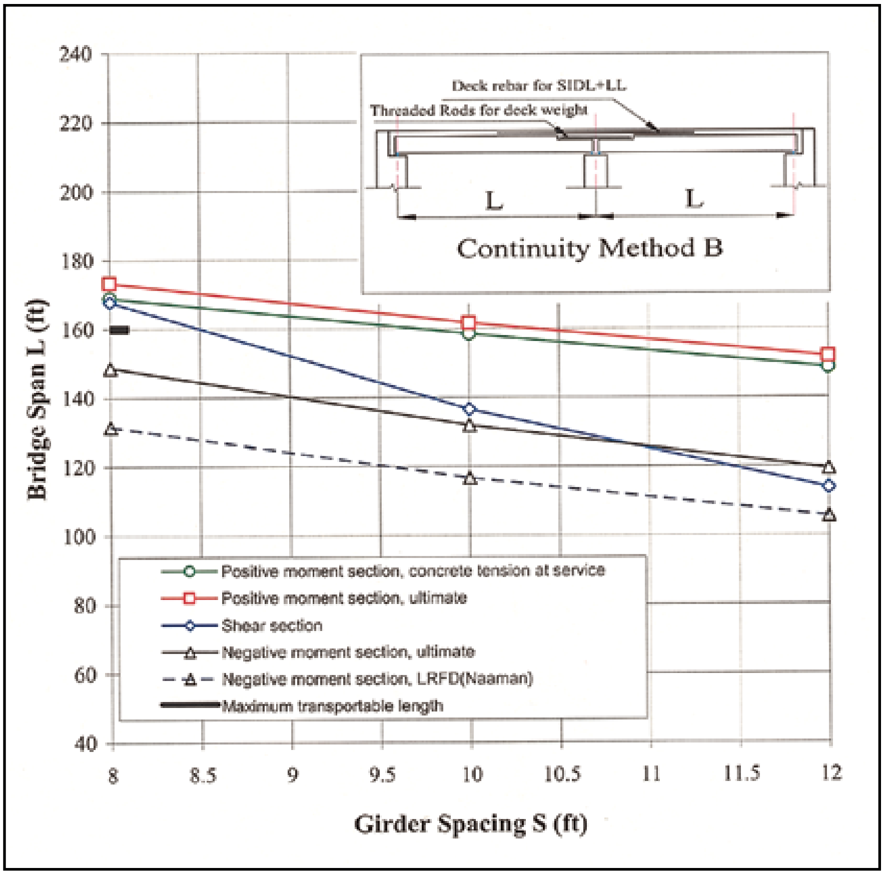

Fig. 1. NU1100 span capacities, System B.

Q4: How is the flexural strength of the negative moment section calculated according to the AASHTO Standard Specifications[6] and AASHTO LRFD Specifications?[7]

A4: The response to this question is best illustrated by an example. An NU1100 I-girder with high strength threaded rods embedded in the top flange is made composite with a 7.5 in. (191 mm) deck as shown in Fig. 2. For simplicity, the cast-in-place concrete haunch over the girder top flange due to girder camber is ignored in this example.

The effective deck slab width for strength calculations is taken as 9.5 ft (2.9 m). The longitudinal reinforcement in the deck consists of one No. 4 plus two No. 6 top bars at 12 in. (305 mm) spacing and one No. 5 plus two No. 6 bottom bars at 12 in. (305 mm) spacing. The effective depth of the top deck bars is 47.97 in. (1218 mm) and of the bottom bars is 45.28 in. (1150 mm).

The threaded rods used to couple the girders are four $1^3/_8$ in. (35 mm) diameter, Grade 150 ksi rods placed at an effective depth of 41.1 in. (1044 mm) from the beam bottom fibers. The areas of the three steel layers are, respectively, 10.26, 11.31, and 6.32 sq in. (6619, 7296, and 4077 mm^2). Other section dimensions are given in the figure. The concrete strength used in the analysis is 8000 psi (55 MPa).

The calculations are presented as two separate items: the nominal flexural capacity and the design capacity.

Nominal Strength, M_n

Because of the different steel types and locations and the use of non-prestressed high strength steel, the AASHTO approximate equations for steel

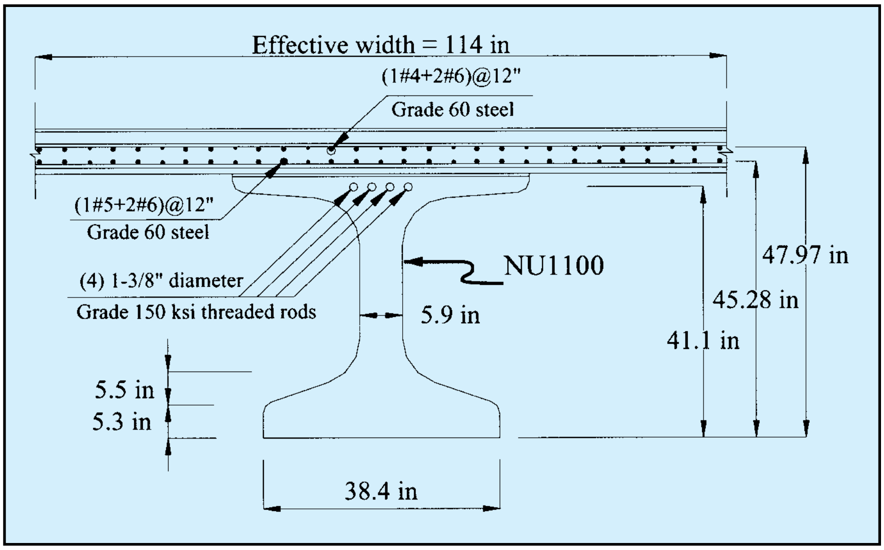

Fig. 2. Negative moment section.

stress and flexural strength cannot be used. Both AASHTO Standard (STD) and LRFD Specifications would direct designers to use a general strain compatibility analysis such as that given in the PCI Bridge Design Manual, Section 8.2.2.5. In this analysis, the only difference between the STD and LRFD procedures is in the relationship between the compression area, A_c, bounded by the neutral axis, and the area A_a subjected to an equivalent compressive stress of $0.85f'_c$.

The LRFD method requires that $A_a = \beta_1 A_c$. This is an additional requirement to the traditional requirement that the depth of the compression block a be set equal to $\beta_1 c$, where c is the neutral axis depth. The two sets of specifications give identical results for rectangular sections. However, the results can be very different if the shape of the compression zone is non-rectangular.

The LRFD provisions are based solely on work by Naaman.[8] Several authors, including Badie and Tadros,[9] and Seguirant,[10,13] have demonstrated that the recommendations are unnecessarily conservative and are not founded on experimental evidence, whereas the well-established ACI 318 and AASHTO Standard provisions have been confirmed by thousands of tests conducted over several decades.

The results of the analysis are summarized in Table 1. The steel stresses are lower according to the LRFD method than the values according to STD. On the other hand, the neutral axis depth is much greater than the corresponding STD neutral axis depth. The theoretical nominal moment using LRFD is 85 percent of the moment using the STD provisions.

Fig. 3 illustrates how the STD and the LRFD provisions differ in defining the size and shape of the compression block area for this example. It is difficult to justify having unstressed areas of the cross section along the same fibers as highly stressed areas. The authors know of no theory or experiments, independent of Naaman's work, that justify this position.

A significant disadvantage of the additional area ratio requirement in the LRFD method is that a relatively small steel area could cause a section to be over-reinforced. This is the case for this example. Because the maximum reinforcement limit is exceeded, the designer is directed to limit the flexural capacity, M_n, to that calculated according to Eq. (C5.7.3.3.1-2), rather than the theoretical strain compatibility value. Thus, when LRFD is used, M_n drops to 82 percent of the STD value.

Design Strength, ϕM_n

The resistance factor, ϕ, for under-reinforced precast concrete members is generally taken as 1.0 in both Standard and LRFD Specifications. When the section is over-reinforced, section capacity is compression-controlled and designers conservatively use ϕ = 0.7 (see Naaman[11]). Accordingly, designers would assign a design strength to this section of 6999.1 ft-kips (65932 N-m) according to STD and 4028.8 ft-kips (37951 N-m) according to LRFD, a 42 percent drop in capacity.

Even if one isolates the additional condition imposed by the LRFD on A_a/A_c by disregarding Eq. (C5.7.3.3.1-

Table 1. Flexural strength analysis by AASHTO Standard Specifications and AASHTO LRFD Specifications.

Property	AASHTO Standard Specifications	AASHTO LRFD Specifications
β_1	0.65	0.65
Neutral axis depth, c (in.)	13.881	22.508
Top rebar in deck, f_{s1}(ksi)	60.00	60.00
Bottom rebar in deck, f_{s2} (ksi)	60.00	60.00
Threaded rod in girder, f_{s3} (ksi)	123.99	71.12
Effective depth to tension force (in.)	44.50	45.15
Strain compatibility M_n (ft-kips)	6999.1	5939.9
Percent	100 percent	85 percent
Max. reinforcement limit ($c/d_e \le 0.42$)	0.312, O.K.	0.499, N.G.
Modified M_n according to AASHTO Eq. (C5.7.3.3.1-2) for over-reinforced sections	6999.1	5755.4
Percent	100 percent	82 percent
ϕ for over-reinforced section	NA	0.7
Design strength ϕM_n (ft-kips)	6999.1	4028.8
Percent	100 percent	58 percent
Extreme tensile strain, ε_{s1}	0.0074	0.0034
Bridge Design Manual (Mast) ϕ	1.000	0.839
ϕM_n (ft-kips)	6999.1	4983.6
Percent	100 percent	71 percent

Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa; 1 ft-kip = 9.42 N-m.

2) and ϕ = 0.7, and by using more realistic values of M_n and ϕ, the error is still significant. Let us assume that we can use the M_n value of 5939.9 ft-kips (55954 N-m) obtained from the strain compatibility analysis and that we can use a resistance factor ϕ equal to 0.839 based on the value of the extreme fiber steel strain, as recommended by Mast.[12] The design moment is still considerably lower (by 29 percent) than the correct one.

Q5: What is your recommendation for the design of highly reinforced non-rectangular cross sections?

A5: It is strongly recommended that the LRFD provision of $A_a/A_c = \beta_1$ not be used in this situation. Use of the strain compatibility approach with STD provisions of $a/c = \beta_1$ is a well-established and safe approach. It is further recommended that Mast's interpolation function for the resistance factor ϕ be used when the maximum reinforcement limit is exceeded. Fig. 4 illustrates the value of using the rigorous strain-compatibility analysis for relatively high steel area contents.

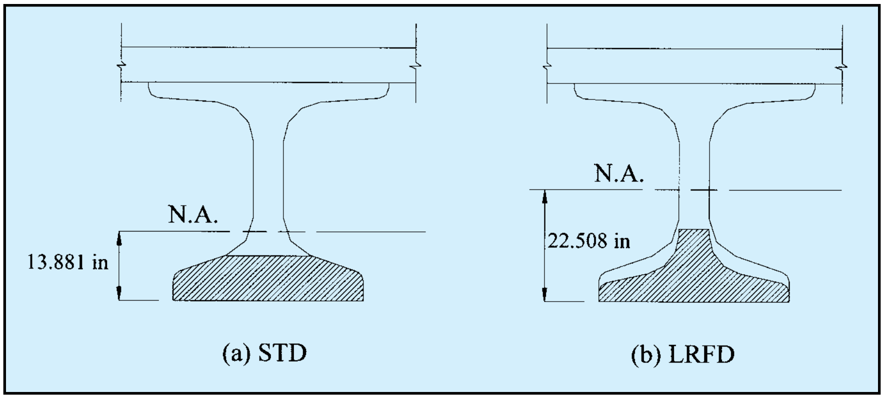

Fig. 3. Compression block area for the example in Fig. 2 according to AASHTO Standard (STD) and LRFD Specifications.

REFERENCES

1. Mirmiran, A., Kulkarni, S., Castrodale, R., Miller, R., and Hastak, M., "Nonlinear Continuity Analysis of Precast, Prestressed Concrete Girders with Cast-in-Place Decks and Diaphragms," PCI JOURNAL, V. 46, No. 5, September-October 2001, pp. 60-80.
2. Ma, Z., Huo, X., Tadros, M. K., and Baishya, M., "Restraint Moments in Precast/Prestressed Concrete Continuous Bridges," PCI JOURNAL, V. 43, No. 6, November-December 1998, pp. 40-57.
3. Ronald, H. D., "Design and Construction Considerations for Continuous Post-Tensioned Bulb-Tee Girder Bridges," PCI JOURNAL, V. 46, No. 3, May-June 2001, pp. 44-66.
4. *PCI Bridge Design Manual*, Precast/Prestressed Concrete Institute, Chicago, IL, 1997.
5. Ficenec, J. A., Kneip, S. D., Tadros, M. K., and Fischer, L. G., "Prestressed Spliced I-Girders: Tenth Street Viaduct Project, Lincoln, Nebraska," PCI JOURNAL, V. 38, No. 5, September-October 1993, pp. 38-48.
6. AASHTO, *Standard Specifications for Highway Bridges*, 16th Edition, American Association for State Highway and Transportation Officials, Washington, DC, 1996.
7. AASHTO, *LRFD Bridge Design Specifications*, Second Edition 1998 and Interims 1999 and 2000, American Association for State Highway and Transportation Officials, Washington, DC.
8. Naaman, A. E., "Unified Design Recommendations for Reinforced, Prestressed and Partially Prestressed Concrete Bending and Compression Members," *ACI Structural Journal*, V. 89, No. 2, March-April 1992, pp. 200-210.
9. Badie, S. S., and Tadros, M. K., "Flexural Strength According to AASHTO LRFD Specifications," PCI JOURNAL, V. 44, No. 4, July-August 1999, pp. 122-127.
10. Seguirant, S. J., "Effective Compression Depth of T-Sections at Nominal Flexural Strength," PCI JOURNAL, V. 47, No. 1, January-February 2002, pp. 100-105.
11. Naaman, A. E., *Prestressed Concrete Analysis and Design: Fundamentals,* McGraw-Hill Book Company, New York, NY, 1982, p. 170.
12. Mast, R. F., "Unified Design Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members," *ACI Structural Journal,* V. 89, No. 2, March-April 1992, pp. 185-199.
13. Naaman, A. E., Reader Comments, and Seguirant, S. J., Author's Closure, PCI JOURNAL, V. 47, No. 3, May-June 2002, pp. 107-113.

*[Contributed by **Amgad Girgis** and **Chuanbing Sun**, Ph.D. Students, and **Maher K. Tadros**, Professor of Civil Engineering, University of Nebraska-Lincoln, Omaha, Nebraska.]*

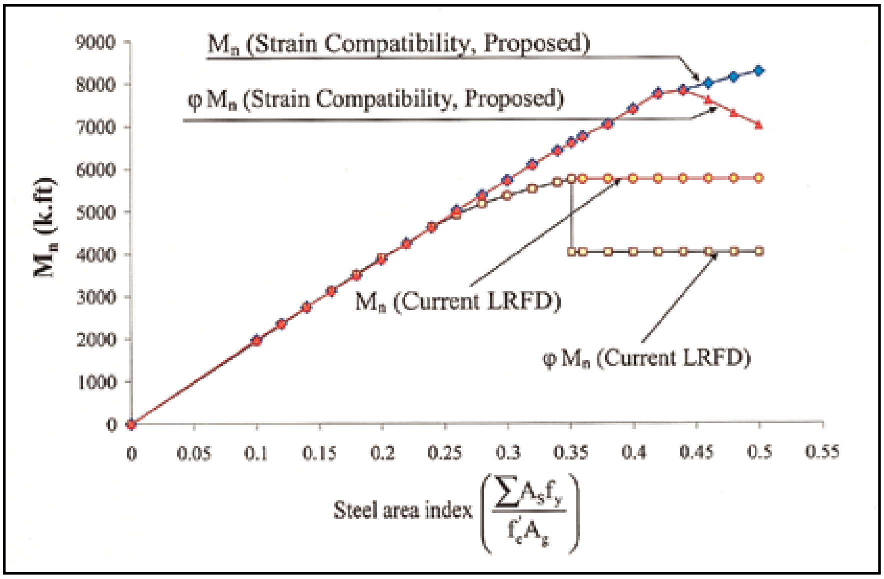

Fig. 4. Proposed strain compatibility design moment for section in Answer A4, versus current AASHTO-LRFD solution.