Threaded rod continuity for bridge deck weight

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- A threaded rod continuity system was developed and implemented for stringer-type bridges to allow precast concrete girders to be continuous for deck weight using high-strength threaded rods over piers.
- The use of a threaded rod continuity system can effectively eliminate cracking at the bottom of the pier diaphragm due to positive restraint moment because the permanent negative moment created by deck placement offsets the positive restraint moment.
- This paper highlights the development of the threaded rod continuity system, addresses the design criteria and approach, discusses the experimental tests, describes the system implementation, and presents a numerical example.

The use of precast, prestressed concrete girders in bridge construction began in the United States in the early 1950s. Until the 1960s, bridges built with pretensioned I-girders and cast-in-place concrete decks were designed as simply supported spans subjected to dead and live loads. In the early 1960s, a number of state agencies started to build continuous highway bridges with prestressed concrete girders.[1] The deck slab was made continuous without any joints by adding longitudinal reinforcement in the deck slab over the pier. Additional loads from the superimposed dead and live loads were resisted by the continuous composite section when the deck concrete cured and gained strength. In this way, prestressed concrete girders were designed as simple-span beams because of their self-weight and deck weight and as continuous-span beams because of the superimposed loads.

This type of continuity has become a common method for highway bridge construction. However, the superstructure is continuous for only about one-third of the total load, which requires a greater demand of prestressing force compared with the threaded rod continuity system described in this paper. As a result, the conventional continuity system can cause significant creep growth of member camber, particularly when a large number of prestressing strands is required. The negative moment due to superimposed dead load and bridge railing is too small to overcome the positive long-term restraint moment due to creep growth of camber. Lack of permanent negative moment at the pier diaphragms results in a net positive restraint moment due to creep and causes bottom cracking at the pier diaphragms.

A number of studies have been performed to evaluate these restraint positive moments and their proper design.[2–5] Common practice is to provide reinforcement to carry the positive restraint moment, but this causes more reinforcement to attract more positive restraint moment. More important, the positive restraint moment is affected by many factors, including the girder size and length, the amount of prestressing force, the age of the girders at the formation of continuity, and the construction schedule and sequence.

Some of these factors, such as the construction schedule, are not entirely under the control of the designer. Miller et al.[6] concluded that positive restraint moment effects are minimal when the continuity is formed after the girders are over 90 days old. The construction schedule, however, may not allow waiting 90 days on most projects, especially for accelerated bridge construction and for replacement of girders damaged by overheight vehicle impact.

The proposed threaded rod continuity system is intended to make precast concrete girders continuous before the deck concrete is placed. This allows introduction of a permanent negative moment, due to the deck weight, that is likely to overcome the positive restraint moment.[7] As a result, the threaded rod continuity system can effectively eliminate cracking at the bottom of the pier diaphragm, which is a significant benefit over a conventional continuity system. This point will be further illustrated by a numerical example. Additional advantages of the threaded rod continuity system are discussed in the following sections.

Threaded rod continuity system

The threaded rod continuity system has undergone two generations of development and implementation. In the first generation, threaded rods are embedded in the girder top flange and are mechanically coupled prior to deck placement. In the second generation, threaded rods are placed on the girder top flange and are housed in a concrete pour strip over the beam top flange. Concrete is placed with the pier diaphragm prior to deck placement.

First generation

The concept of making precast concrete girders continuous using coupled high-strength threaded rods over the piers was presented by Ma et al. in 1998.[7] **Figure 1** shows a laboratory demonstration model of threaded rod connections between two Nebraska University (NU) I-girder ends.[8] The threaded rods projecting from the girder top flange were painted for illustration purposes. The lengths of the threaded rods are determined by considering the length needed to resist the negative moment diagram along the girder due to the deck weight. A total of four 1⅜ in. (35 mm) diameter, Grade 150 (1030 MPa) threaded rods are embedded in each girder top flange. The rods are coupled using two rectangular steel bars and five loose threaded rods. Heavy nuts and washers are included to couple the threaded rods with the rectangular steel bars.

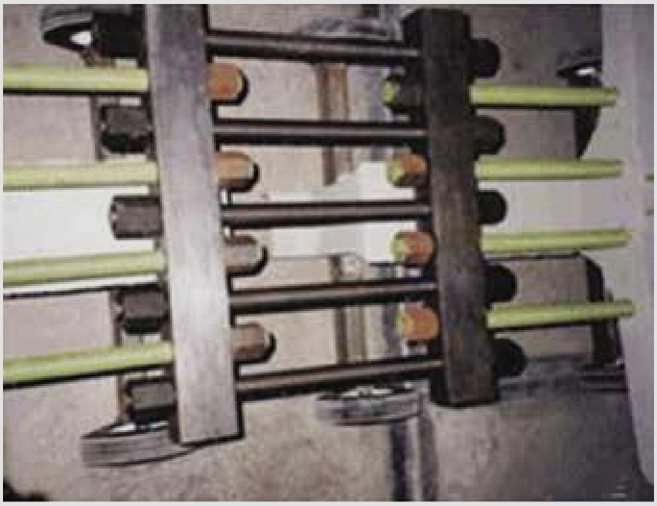

Figure 1. Demonstration model of the first-generation of threaded rod continuity system.

The number and size of the required threaded rods depend on the bridge span, girder size, and girder spacing. However, this detail allows space for only four 1⅜ in. (35 mm) diameter rods unless the girder top flange is thickened. A 25 in. (635 mm) long blockout in each girder top flange allows for adequate space to snug-tighten the nuts in the field.

This system has been successfully implemented in a number of bridges, including the Clarks Viaduct (**Fig. 2**) in central Nebraska[9] and bridges in Illinois and Alberta, Canada. The Clarks Viaduct was the first bridge built in the

Figure 2. First-generation threaded rod continuity system in the Clarks Viaduct in Clarks, Neb. Photo courtesy of Ted Butler.

Figure 3. First-generation threaded rod continuity system in Waverly Eastbound Bridge in Waverly, Neb.

United States that implemented the threaded rod continuity system. It was a value-engineered project that converted a Grade 70 (480 MPa), high-strength steel plate girder design to a precast concrete I-girder design. It generated moderate cost savings while matching the girder depth and spacing of the steel alternative. The first-generation coupled threaded rod continuity system was used in the Clarks Viaduct in Clarks, Neb.; Waverly Eastbound Bridge (**Fig. 3**) in Waverly, Neb.; Platte River East Bridge in Douglas and Saunders Counties, Neb.; and South Omaha Bridge in Omaha, Neb.

Second generation

During development of the second generation of threaded rod continuity system, the researchers focused on simplification of girder production and continuity-joint construction.[10] Having threaded rods embedded in the top flange created congestion in the relatively thin NU I-girder top flange. To keep the splice rods from being skewed, the two ends of the girders meeting at the pier are required to be properly aligned. In addition, the large rectangular steel bars, washers, and nuts were eliminated. For the second generation of threaded rod continuity system, the threaded rods cover the negative moment zones on both spans being connected without splicing over the piers. The threaded rods are laid on top of the girder top flange after the girders have been erected and secured in position.

To ensure development of the rods, they are enclosed with confinement reinforcement (**Fig. 4**). The confinement stirrups are provided in pairs of C-shaped bars. The bottom stirrup, type A, is embedded during the precasting operation, and the top stirrup, type B, is placed in the field, thus forming a continuous loop confinement. Both type A and B are no. 4 (13M) bars spaced at 12 in. (300 mm). The rods and stirrups are encased in a 3½ in. (89 mm) thick concrete strip, and this concrete is placed with the diaphragm concrete (before the deck concrete). This interface strip eventually becomes part of the haunch (or buildup) between the girder and deck slab.

Shear reinforcement in the girder web projects from the top flange by 6 in. (150 mm) to allow for composite action between the girder and the deck. Due to the difference in camber along the girder length, the shear reinforcement near the pier supports may not be sufficiently embedded in the deck. For this reason, additional type C no. 4 (13M) hat bars are provided (Fig. 4). They can be slanted as needed to satisfy the concrete cover requirements at the top of the deck slab. Also shown in Fig. 4 are ten 1⅜ in. (35 mm) diameter, Grade 150 (1030 MPa) threaded rods. This is the maximum number of rods that can be accommodated according to available space and test results.

A minimum ¾ in. (19 mm) gap is used between the threaded rods and the girder top flange. This gap allows concrete to flow underneath the rods for full consolidation of the concrete around the rods. Extensive experimental studies have been performed on this detail[10] to demonstrate that it is capable of fully developing the capacity of ten 1⅜ in. rods. The experimental results are summarized later in the paper.

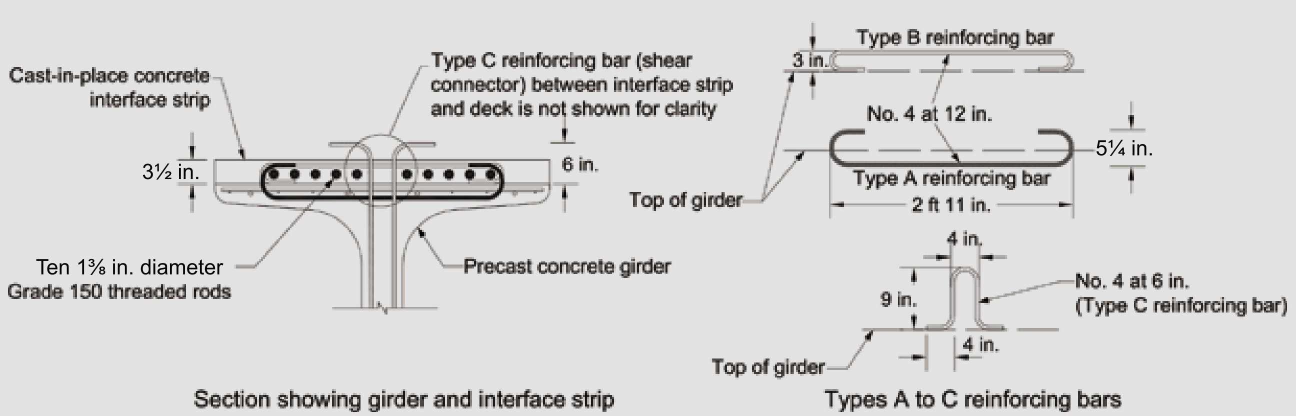

Figure 4. Reinforcement in the second-generation threaded rod continuity system. Note: no. 4 = 13M; Grade 150 = 1030 MPa; 1 in. = 25.4 mm.

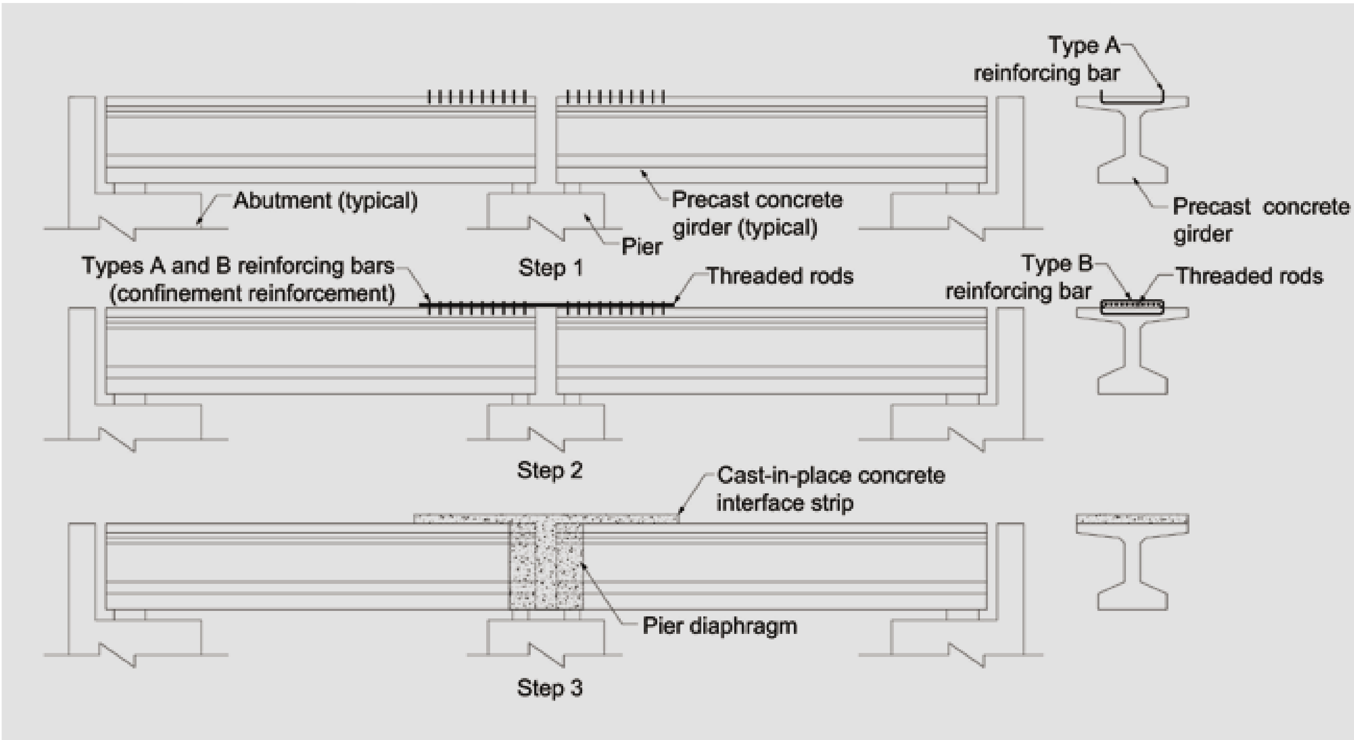

Figure 5. Construction steps of implementing the second-generation threaded rod continuity system prior to deck placement.

Figure 5 illustrates the proposed construction sequence using the second-generation threaded rod continuity system. After the girders are erected (step 1), the threaded rods, type B confinement reinforcement, and type C hat bars are installed (step 2). This is followed by concrete placement for the interface strip and pier diaphragm (step 3). When the required concrete strength is reached, the deck reinforcement and the deck concrete are placed.

Figures 6 and **7** show typical girder/deck cross sections at the midspan and the pier end, respectively. Figure 6 illustrates the recommended minimum haunch thickness of 3 in. (75 mm) at the midspan at the centerline of the girder. Assuming a 2% cross slope, the thickness varies by about 1 in. (25 mm) across a 4 ft ¼ in. (1.225 m) flange width. Assuming that the girder will camber at least 1 in. at the time of deck placement, the haunch thickness would be 4 in. (100 mm) at the girder centerline near the pier end. This corresponds to a 3½ in. (89 mm) thickness at the shallow edge of the girder top flange, which is the minimum desirable thickness of the interface strip (Fig. 7). The second-generation threaded rod continuity system has the following advantages over the first generation:

- All threaded rods are placed on the girder top flange, which eliminates the need for mechanical coupling in the field and results in material saving.
- The number of threaded rods for continuity is not as limited as in the first-generation details, where at most four or five threaded rods could be embedded in the girder top flange.
- In the second-generation details, a maximum of ten 1⅜ in. (35 mm) threaded rods is recommended. This allows for the details to be standardized with a conservative amount of reinforcement. As will be shown, this amount is nearly twice the amount required for

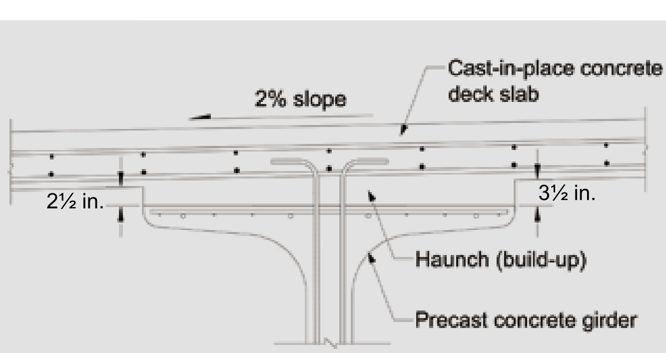

Figure 6. Girder and slab section near midspan in the second-generation threaded rod continuity system. Note: 1 in. = 25.4 mm.

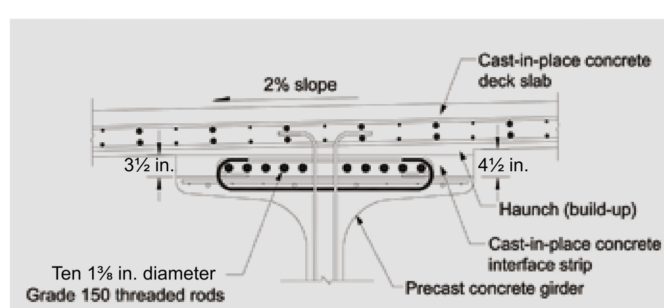

Figure 7. Girder/slab section showing continuity detail over the pier in the second-generation threaded rod continuity system. (Type C hat bars are not shown for clarity.) Note: Grade 150 = 1030 MPa; 1 in. = 25.4 mm.

strength resistance of deck weight for most applications. This standardized quantity is not believed to create a significant cost premium for the total bridge and is advisable until more experience with this system is gained.

- Due to the significant amount of steel area provided in the negative moment zone, negative moment redistribution over the pier is expected to be minimal. Thus the degree of continuity is enhanced and transverse deck cracking is minimized.

Advantages

As discussed, use of the threaded rod continuity system essentially mitigates the possibility of a positive restraint moment large enough to cause cracking at the bottom of the pier diaphragm. This allows the girders to be erected shortly after production, which provides for accelerated bridge construction. The negative moment due to deck weight will counteract the positive restraint moment due to prestress-camber creep. Ma and Tadros[7] performed a detailed time-dependent analysis to confirm that when the threaded rod continuity system is employed, no positive moment restraint reinforcement is necessary.

Other advantages of the threaded rod continuity system include savings in the number of required prestressing strands for the same girder span and spacing because the precast concrete girders are made continuous for deck weight, which is approximately one-third of the total load in highway bridges involving precast, prestressed concrete girders. As a result, the use of the threaded rod continuity system allows a reduction in the magnitude of positive moment near midspan, the prestressing force required, and the concrete strength at prestress release. The same girder size and spacing has been found to gain 10% to 15% additional span capacity with the use of the threaded rod continuity system.[11] The use of the threaded rod continuity system is, therefore, effective in achieving the shallowest superstructure depth, which may be needed at sites where the vertical clearance requirement is critical. Therefore, the threaded rod continuity system may be a cost-effective alternative to steel plate girders and spliced posttensioned concrete girders. Unlike with posttensioned concrete girders, no specialty posttensioning subcontractor is needed.

Design criteria

The following discussion focuses on the design differences between typical precast, prestressed concrete girder systems and the threaded rod continuity system. For flexure, three limit states are considered: strength, service, and fatigue. The design criteria are developed in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[12] while including research findings by Tadros et al. that have been incorporated into the Nebraska Department of Roads' (NDOR's) *Bridge Office Policies and Procedures (BOPP) Manual*.[13] The discussion relates to the following:

- flexural and interface shear design at strength limit state
- stresses in the concrete, threaded rods, and deck reinforcement at service limit state
- stresses in the concrete and the threaded rods and deck reinforcement at fatigue limit state

Vertical shear strengths are designed in the same manner as girders made continuous for superimposed dead load and live load.

Strength limit state

Flexural strength Because the threaded rods are not posttensioned, the negative moment region near the pier is considered to be conventionally reinforced. Two primary loading cases are considered:

- at time of deck placement (Strength IV)
- for the completed bridge due to full loads (Strength I)

When applying the Strength IV load combination, the load factor for the deck weight, including the weight of the haunch, should be taken as 1.5.

To test the limits of this system considering detailing, ten 1⅜ in. (35 mm) diameter rods with a corresponding steel area of 15.8 in.^{2} (10,200 mm^2) are used. Even if the analysis justifies fewer threaded rods, it is still recommended that 10 rods be used to ensure sufficient stiffness of the negative moment zone. This recommendation may be modified as experience is gained with the system. The threaded rods are the only steel used to resist the negative moment due to deck weight. If the provided steel area from the 10 threaded rods is greater than the required area due to deck weight, the remaining threaded rod area can be used to reduce the amount of longitudinal reinforcing bars required in the deck to resist the negative moments due to full load. The critical negative moment sections for the flexural strength design at Strength I are the composite section at the face of the pier diaphragm and the pier diaphragm section at the centerline of the pier.

Based on the study by Tadros et al.,[14,15] the minimum cast-in-place concrete compressive strength in the pier diaphragm and the interface strip shall be at least equal to 50% of the final compressive strength of the girder concrete. For example, for girders that have a final design concrete compressive strength of 10 ksi (69 MPa), the diaphragm design concrete compressive strength shall be at least 5 ksi (34 MPa).

Interface shear Interface shear design is performed in accordance with article 5.8.4 of the AASHTO LRFD specifications, which is based on the shear friction theory. Two interfaces should be checked: at the top and bottom faces of the interface strip. When the interface strip concrete is placed on the girder top flange and gains adequate strength, it acts compositely with the precast concrete girder. The interface shear due to deck and haunch weight is resisted by the interface between the strip and the girder. The corresponding interface shear reinforcement is the bars that project from the top flange (the vertical shear reinforcement and the hooks of type A reinforcement [Fig. 4]). After the deck slab concrete hardens, it works compositely with the previously cast concrete (the precast concrete girder and interface strip). Accordingly, the interface shear due to the full dead and live loads should be resisted by the reinforcement that crosses the planes between the girder/interface strip combination and the underside of the deck. The reinforcement in this case is the steel that crosses the planes just described (the vertical shear reinforcement and the type C hat bars).

Service limit state

Positive moment Working stress design of the girders at the positive moment sections is similar to that employed for conventional continuity systems. Exceptions are as follows.[13] They are recommended at this time as conservative measures until more experience is gained with this system.

- The positive moment region subject to a Service III loading combination must be satisfied with a bottom fiber concrete stress not exceeding zero for all stages of loading. This stress limit is more conservative compared with the AASHTO LRFD specifications, where tensile stress is allowed.

- There are two compression limits in accordance with the AASHTO LRFD specifications. The concrete compressive stress due to effective prestress plus dead loads is limited to 0.45 f_c', and the concrete compressive stress due to effective prestress plus full loads is limited to 0.60 f_c' (where f_c' is the concrete compressive strength). These compression limits are waived by NDOR as long as the flexural strength design requirements are satisfied. This waiver is based on the research work by Tadros et al.[16,17]

Negative moment Crack control of deck reinforcement should be evaluated at the service limit state. In accordance with article 5.7.3.4 of the AASHTO LRFD specifications, the spacing of mild-steel reinforcement in the layer closest to the tension face s shall satisfy the following:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

where

γ_e = exposure factor

= 1.00 for Class 1 exposure condition

= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest to it

f_{ss} = tensile stress in steel reinforcement at the service limit state

β_s = ratio of flexural strain at the extreme tension face to strain at the centroid of the reinforcement layer nearest the tension face

$$= 1 + \frac{d_c}{0.7\left(h - d_c\right)}$$

h = overall thickness or depth of the component

An additional NDOR requirement based on experience with the system is that a sheet of welded-wire reinforcement (WWR) is to be placed in the centerline of the girder web to ensure that if cracking develops in the web due to full loads, it will be controlled. The limit at which the extra WWR sheet is required is 0.24 $\sqrt{f_c'}$ at the top face of the web due to full dead plus live loads. The WWR sheet is 6 ft (1.8 m) wide × (height of girder – 1 ft [0.3 m]). It consists of D20 (MD130) × D20 at 4 × 4 in. (100 × 100 mm). The diameter of D20 is equivalent to a no. 4 (13M) bar.

Fatigue limit state

The moment due to fatigue truck loading is determined in accordance with the AASHTO LRFD specifications. A single-lane live-load distribution factor should be used. A live-load factor of 1.50, representing infinite fatigue life, should be used. The dynamic allowance factor should be 0.15.

Tadros and Wang performed fatigue testing on the threaded rods conforming to ASTM A722.[10] Five sets of threaded rods were tested to five million cycles. Also, fatigue stress limits of precast concrete girders using a threaded rod continuity system were investigated. As a result, the following design criteria are suggested:[14,15]

Concrete compressive stress limit The concrete fatigue limit check stipulates that the concrete compressive stress due to 50% of effective prestress, 50% of dead load, and 100% of fatigue truck live load shall be limited to 0.40 $f_c^{'}$.

$$0.5\left(f_{DL}+f_{Prestress}\right)+1.5f_{FatigueLL}\leq 0.40f_c^{'}$$

where

f_{DL} = concrete stress due to dead loads

$f_{Prestress}$ = concrete stress due to effective prestress

$f_{FatigueLL}$ = concrete stress due to fatigue live loading

Stress limit in threaded rods No fatigue limit is specified for threaded rods in the AASHTO LRFD specifications. Based on the fatigue testing performed by Wang and Tadros,[10] the stress range in the threaded rods f_f (equal to $36 - f_{min}/3$) should not exceed 18 ksi (124 MPa), where f_{min} is minimum live-load stress resulting from the fatigue limit state I load combination plus the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads.

When performing these calculations, tensile stress is positive and compressive stress is negative. Also, it is suggested that $f_{min} \leq 54$ ksi (372 MPa), which is established to mitigate the cracks at the top of the precast concrete girder due to dead loads, particularly at the time of placement of the deck concrete.

Stress limit in deck reinforcement Article 5.5.3.2 of the AASHTO LRFD specifications shall be followed to evaluate the stress in the deck reinforcement:

$$(\Delta F)_{TH} = 24 - 0.33f_{min}$$

where

$(\Delta F)_{TH}$ = allowable stress range in mild reinforcement

Design methodology

Flexural strength methodology for the threaded rod continuity system requires the use of the fundamental strain compatibility method, which uses force equilibrium rather than the approximate closed-form flexural strength formulas. The method can be programmed using a computer spreadsheet and used repeatedly to calculate the required reinforcement.

The moment-curvature relationship is a valuable tool for determining cracked-section properties and moment redistribution. It is used to determine whether moment redistribution due to cracking of the conventionally reinforced negative moment zone would require an increase in the maximum required positive moment over that calculated using the common elastic uncracked-section analysis.

Strain compatibility and force equilibrium method

The strain compatibility and force equilibrium method is based on three well-accepted fundamental assumptions:[18]

- Plain sections remain plain after bending.
- There is compatibility of strains. That is, there is full bond between steel and concrete at the section being considered.
- There is an equilibrium of forces within a section.

The equations on the strain compatibility and force equilibrium are given as Eq. (1) and (2):

$$\varepsilon_{si} = \varepsilon_c\left(\frac{d_i}{c}-1\right) \quad (1)$$

where

ε_{si} = strain of steel row number i

ε_c = strain of the concrete at the extreme compression fiber

d_i = depth from extreme compression fiber of steel row number i

c = distance from the extreme compression fiber to the neutral axis

$$\sum f_{si}A_{si} + \sum f_{cj}A_{cj} = 0 \quad (2)$$

where

f_{si} = stress of steel row number i

A_{si} = area of steel row number i

f_{ci} = stress of each concrete layer

A_{ci} = area of each concrete layer

Steel stress f_{si} in each row can be computed from the total strain ε_{si} using Eq. (3), commonly known as the power formula:[18]

$$f_{si} = \varepsilon_{si} E_{si} \left\{ Q + \frac{1-Q}{\left[1+\left(\frac{\varepsilon_{si} E_{si}}{k f_{yi}}\right)^{R}\right]^{1/R}} \right\} \quad (3)$$

where

E_{si} = modulus of elasticity of the steel used in row i

Q = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

k = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

f_{yi} = yield strength of the steel grade used in row i

R = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

The power formula (Eq. [3]) allows for the use of different steel grades in the cross section. For the threaded rod continuity system, three different steel grades—Grades 60, 150, and 270 (410, 1030, and 1860 MPa)—may be used in the same section. The constants E_{si}, Q, f_{yi}, k, and R for Grade 60 steel are 29,000 ksi (200 GPa), 0, 60 ksi, 1.096, and 100. For Grade 150 steel, used in high-strength threaded rod, the constants are 29,000 ksi, 0.0217, 120 ksi (827 MPa), 1.01, and 4.224. For Grade 270 steel, used in prestressing, the constants are 28,500 ksi (197 GPa), 0.031, 243 ksi (1680 MPa), 1.04, and 7.36. The basis for these constants is explained in a paper by Devalapura and Tadros.[19] A detailed explanation of how to use the strain compatibility method for strength calculations is given in the *PCI Bridge Design Manual*.[18] It is applicable for use in the cracked-section service limit state and the fatigue limit state, as presented in the numerical example.

Moment-curvature relationship

Because the negative moment region is conventionally reinforced, cracked-section analysis is required to check stresses in the threaded rod and deck reinforcement. Cracked-section properties can be determined using the moment-curvature relationship, as given by the classical elastic analysis:[20]

$$I = \frac{M}{E\phi} \quad (4)$$

where

I = moment of inertia of a section

M = bending moment

E_c = modulus of elasticity of concrete

ϕ = curvature in a section

The curvature of a section can be determined by the following equation:

$$\phi = \frac{\varepsilon_c}{c} \quad (5)$$

The curvature in a section corresponding to the bending moment is determined by Eq. (5), where c is obtained from Eq. (1) and (2).

It is recommended that ten 1⅜ in. (35 mm) diameter threaded rods be used on the girder top flange in this example of the threaded rod continuity system. The number of threaded rods is so great that the negative moment redistribution due to deck weight is negligible.[11] If fewer threaded rods are provided, the design should account for possible moment redistribution due to deck weight. The boundary limit is to use zero threaded rods; for this case, the negative moment region over the piers would attract zero negative moment and a 100% distribution into the positive moment. Thus, use of threaded rods may be viewed as a general case with the lower bound as the conventional simple-span-for-deck-weight system.

Sun performed an analysis of the cracked section and stress redistribution.[11] That research shows that redistribution between the negative and positive moments does not exceed the maximum allowed code redistribution limit of 20%. However, NDOR has elected to continue using the traditional elastic uncracked section analysis and to reduce the allowable tensile stress limit for the Service III load combination to zero. This simplification captures the effect of moment redistribution while keeping the analysis the same as that conducted traditionally for all bridge design.

Experimental tests

A series of full-scale tests were conducted by Wang[10] and Hanna[15] to evaluate girder behavior using a threaded rod continuity system:

- two 25 ft (7.6 m) long NU2000 (2000 mm [80 in.] deep) girders involving four threaded rods over the pier
- two 140 ft (43 m) long NU1100 (1100 mm [43 in.] deep) girders involving 10 threaded rods on the girder top flange

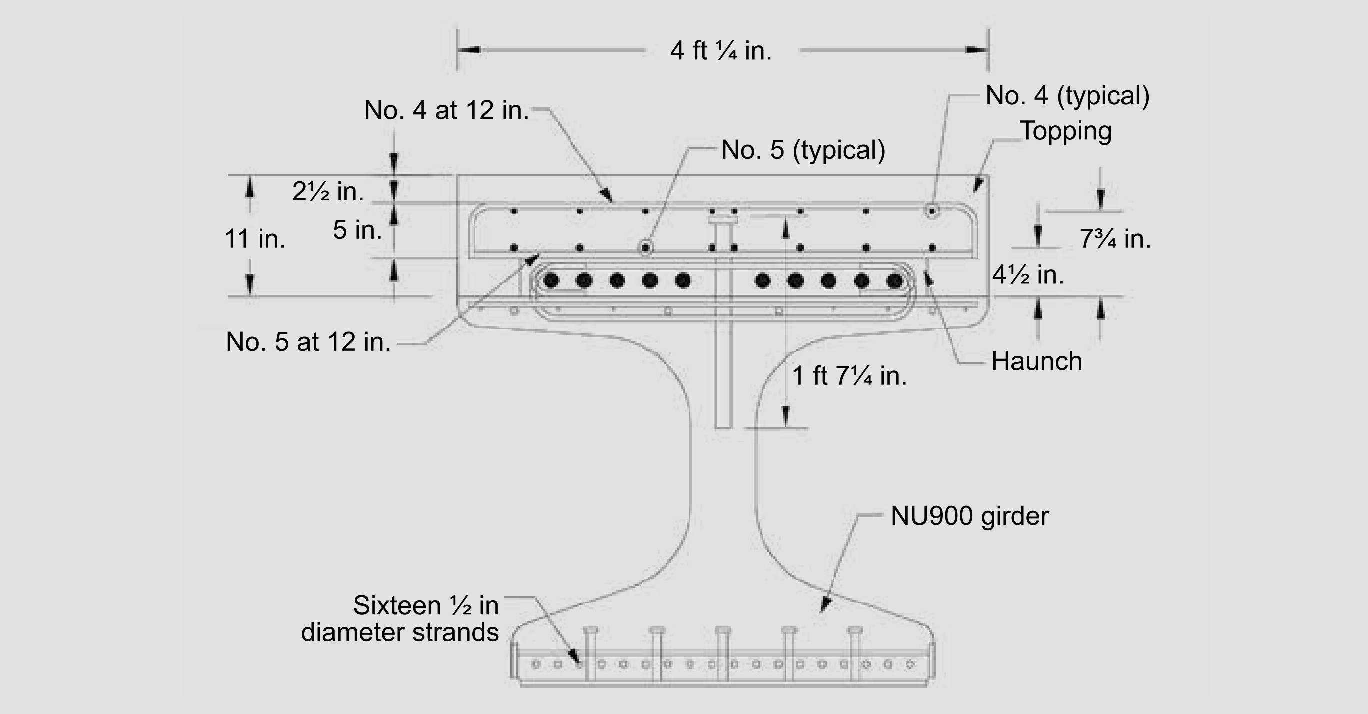

Figure 8. Cross section of the third specimen. Note: NU900 = 900 mm (36 in.) deep. No. 4 = 13M; no. 5 = 16M; Grade 150 = 1030 MPa; 1 in. = 25.4 mm; 1 ft =0.305 m.

- two 25 ft (7.6 m) long NU900 (900 mm [36 in.] deep) girders including 10 threaded rods on the girder top flange, 8 ft (2.4m) long bearing plates at the girder ends, and two vertical side plates at the girder bottom flange
- a specimen identical to the previous one, except that the bearing plate was reduced to the standard 2 ft (0.6 m) long and the two side plates were eliminated to simplify precast concrete production

Grade 150 (1030 MPa), 1⅜ in. (35 mm) diameter threaded rods were used in all tests. Only the third and fourth tests are discussed in this paper.

The third test involved the use of NU900 (36 in. [900 mm]) deep girders. The specimen represented the negative moment zone of a bridge that comprised two 100 ft (30.5 m) spans. Two 25 ft (7.6 m) long pieces were assembled in the laboratory. Cast-in-place concrete was used for the diaphragm with a 4½ in. (114 mm) haunch and for a second-stage 7½ in. (191 mm) thick deck. **Figure 8** shows the cross section of the specimen. The girder section included sixteen ½ in. (13 mm) diameter, Grade 270 (1860 MPa) strands in the bottom flange; a bearing plate assembly at the girder bottom near the pier; ten 1⅜ in. (35 mm) diameter threaded rods in the haunch; and additional reinforcement in the deck slab. The bearing plate assembly, consisting of one bottom plate and two side plates, was provided with the goal of confining the concrete at the girder bottom flange near the pier. **Figure 9** shows the top view of the specimen prior to concrete placement.

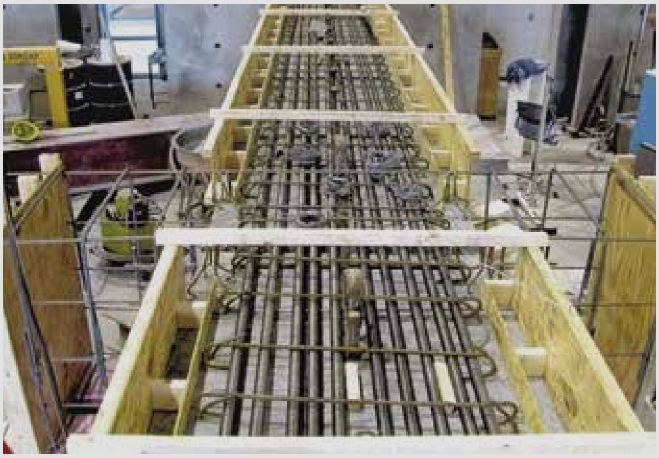

Figure 9. Top view of the third specimen showing threaded rods on the girder top flange.

One end of the specimen was cantilevered and applied with a vertical load through a hydraulic jack (**Fig. 10**). The other end of the specimen was simply supported on a concrete block. The load was applied in two stages. In the first stage, the load was applied once the haunch concrete was hardened. The load was predetermined such that it generated a negative moment corresponding to the moment due to deck weight. Afterward, the hydraulic jack was locked in position to maintain the same loading on the specimen. In the second stage, the load was applied after the deck slab achieved its 28-day concrete compressive strength. The load was increased until the specimen failed. The maximum load can be converted to a negative moment by multiplying the loads with the arm (distance between the loading point and the support centerline), which corresponds to the flexural strength of the specimen over the pier.


Figure 10. Test setup of the third specimen.

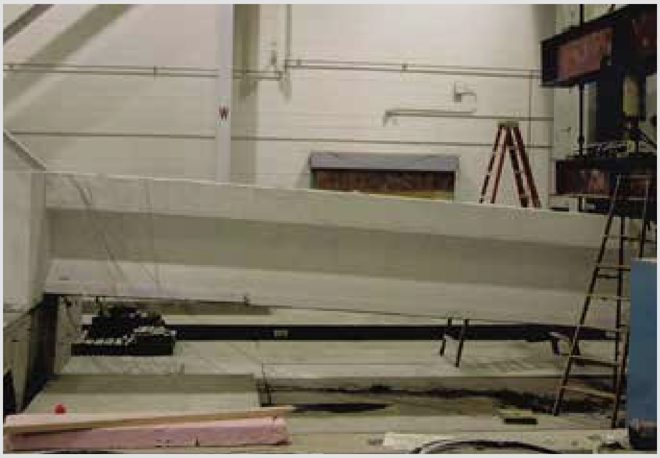
Figure 11. Deflection of the third specimen near failure.

The specimen, as well as all preceding specimens, behaved predictably in terms of cracking and strength requirements. The maximum failure load for this specimen was 382 kip (1700 kN), compared with the predicted load of 332 kip (1480 kN). The specimen showed excellent ductility (**Fig. 11**). None of the specimens had any deficiency in the interface horizontal shear behavior. The latest specimen exhibited similarly superior bond between the threaded rods and the surrounding concrete in the haunch. No signs of slippage were observed between the threaded rods and the concrete during loading or at the failure load. No horizontal shear failure was observed at any of the concrete interfaces between the two interfaces (at the top of the girder and at the top of the haunch). Failure took place when the girder bottom bearing plate buckled, indicating large compressive strains in the bottom fibers (**Fig. 12**). This test is an extreme test for deflections and strains. The free ends of the double cantilever were permitted to deflect without constraint, which is contrary to these points' being within the span of the two-span bridge being modeled.

After the third test, the fourth and final test was conducted. It was found that the side plates were unnecessary and that the predicted strength could be achieved with a single bottom plate. Also, in this test, the diaphragm concrete strength was reduced to 5000 psi (35 MPa), which was half of the girder concrete strength. This test gave similar behavior to that of the third test. It confirmed the validity of the simplified bottom bearing assembly and eliminated the requirement for high-strength concrete in the bulky cast-in-place concrete diaphragm.

Design criteria

To guide designers and consultants, the following steps and criteria were developed for the design of bridges using the threaded rod continuity system:[13]

1. Determine moments due to girder and interface-strip weight from a simple-span analysis. Moments due to deck weight (including haunch or buildup) and all subsequent superimposed dead and live loadings shall be determined using continuous uncracked elastic section analysis.

2. Determine the number of prestressing strands and strand pattern based on the working stress design satisfying the concrete stress limits at Service I and Service III and at prestress release. Verify the number of strands using the flexural strength design at the positive moment sections.

3. Perform the flexural strength design at the negative moment region. The number of threaded rods provided shall be determined to satisfy the Strength IV load combination under loads introduced at the critical section just before the girder becomes composite with the deck slab. Use of the maximum number of threaded rods—ten 1⅜ in. (35 mm) diameter rods—is encouraged because it improves system stiffness and contributes to improved crack control of the girder due to deck weight. The additional capacity required to satisfy the ultimate flexural strength limit state shall be provided through mild reinforcement in the deck. Steel plates or compression reinforcement may be provided at the bottom of the girder near the pier end, if necessary.


Figure 12. Bearing plate buckled in the third specimen at the maximum load.

4. Check the area of provided threaded rods for development length beyond the location where it is required for all strength calculations. Threaded rods may be staggered in the negative moment region as long as the required flexural resistance is satisfied.

5. Check the crack control criteria for the top deck reinforcement.

6. Determine the moment due to fatigue truck loading and perform the fatigue design by checking the stresses in the concrete, threaded rods, and top deck reinforcement. The allowable compressive stress limit in the girder due to the fatigue loading shall be satisfied at the critical positive moment section and also at the face of the pier diaphragm. The fatigue stress ranges in the threaded rods and top deck reinforcement shall be checked as described in the Design Criteria section. Cracked-section analysis shall be performed for fatigue investigation.

7. Satisfy the live load deflection limit in the AASHTO LRFD specifications.

8. Consider the restraint caused by the threaded rod continuity in estimating deflection due to deck weight. Uncracked-section analysis may be used for prediction of deflections for shims.

Implementation

Since 2005, the threaded rod continuity system has been implemented in a number of bridges in Nebraska. It is thought that this technology has also been used in the state of Illinois and the province of Alberta, Canada. For example, the 176th Street Bridge over Interstate 80 in Lancaster County, Neb., has two spans of 126 ft 8 in. (38.6 m) each. It includes four girder lines spaced at 10 ft (3 m).

Figure 13. Detail of girder ends over the pier prior to placement of pier diaphragm in the 176th Street Bridge over Interstate 80. Image courtesy of Robert A. Traudt.

Figure 14. Pacific Street Bridge in Omaha, Neb., open to traffic.

NU1100 (1100 mm [43 in.] deep) girders are used. **Figure 13** shows the high-strength threaded rods placed on the girder top flange over the pier. Instead of using types A and B confinement reinforcement (Fig. 4), the designer eliminated type B reinforcement by allowing the type A bars to be long enough to be bent in the field and to fully overlap the threaded rods. Also shown in Fig. 13 is the sheet of WWR that is placed in the pier diaphragm between the girder ends. The WWR sheet is provided to confine the cast-in-place concrete near the girder ends.

The Pacific Street Bridge is another example. It has two spans of 98 ft (29.9 m) each. Ten girder lines spaced at 10 ft 8 in. (3.25 m) were used. NU900 (900 mm [35 in.] deep) girders were used in this bridge. **Figure 14** illustrates the completed bridge, which opened to traffic in 2008.

Numerical example

A numerical example is presented for two purposes: to show how to perform the design using the threaded rod continuity system, with an emphasis on the unique aspects of design of the system, and to show the benefits of using the threaded rod continuity system compared with other systems. Solutions are developed for three options. The span length is fixed for a two-span bridge. The girder spacing, reinforcement, and precast concrete compressive strengths are determined for each option.

Input data:

- There are two 100 ft (30.5 m) long spans.
- The bridge is wide enough to carry multiple lanes.
- Interior girder design is performed.
- Three options are analyzed:
  - option 1: simple span for all loads

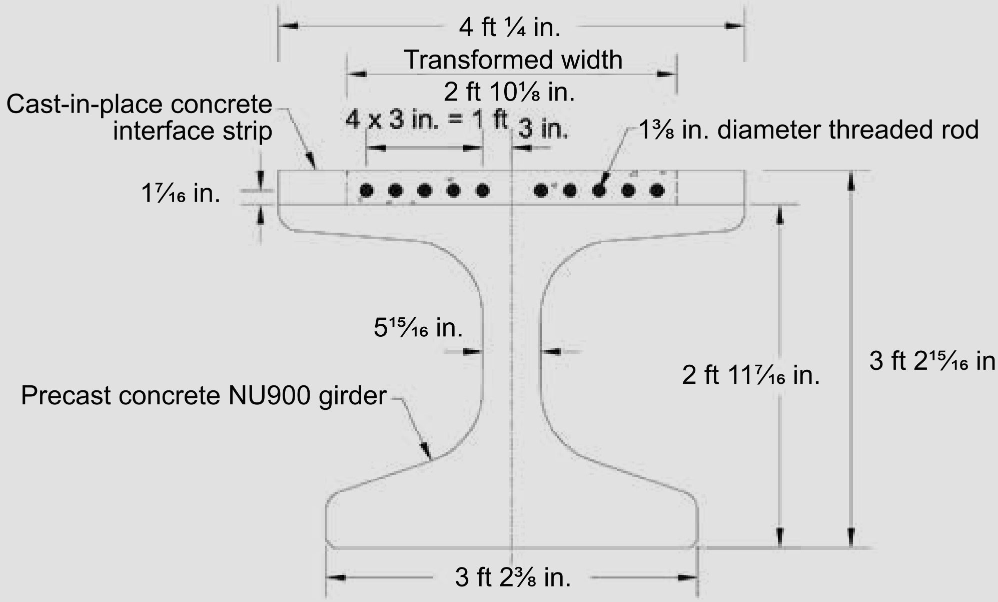

Figure 15. Beam section in option 3 involving the threaded rod continuity system. Note: NU900 = 900 mm (36 in.) deep. 1 in. = 25.4 mm; 1 ft = 0.305 m.

  - option 2: simple span for girder weight and deck weight and continuous for additional loads
  - option 3: simple span for girder weight and continuous for additional loads

- The concrete compressive strength for the precast concrete girder is 10,000 psi (69 MPa) at 28 days and 6500 psi (45 MPa) at prestress release.
- The concrete compressive strength for the concrete in the cast-in-place diaphragm and deck is 5000 psi (35 MPa) at 28 days.
- For a NU900 (36 in. [900 mm]) I-girder with area of 648.3 in.^{2} (418,300 mm^2), moment of inertia I of 110,260 in.^{4} (45,894,000,000 mm^4), and a distance between the girder bottom fiber and centroid y_b of 16.13 in. (408.9 mm), the bottom flange is able to house up to fifty-eight 0.6 in. (15 mm) diameter strands (18 + 18 + 12 + 6 + 2 + 2 strands located at 2, 4, 6, 8, 10, and 12 in. [50, 100, 150, 200, 250, and 300 mm] from the girder bottom fiber, respectively).
- The deck slab is 8 in. (200 mm) thick, including a 0.5 in. (13 mm) thick sacrificial wearing surface.
- Haunch thickness is assumed constant in structural capacity calculations at 1 in. (25 mm) for options 1 and 2 and 3.5 in. (89 mm) corresponding to the interface strip height for system 3 (**Fig. 15**). Actual haunch thickness variation along the span should be considered in a detailed design.
- The deck slab is placed when the girders are 28 days old.
- The pier diaphragm is 3 ft 4 in. (1.0 m) wide. The interface strip is 35 ft 6 in. (10.8 m) long at each side of the pier centerline.
- Loads consist of girder weight, haunch weight, deck weight, a 0.15 kip/ft (2.2 kN/m) allowance for barrier weight per girder line, a 25 lb/ft^2 (1.2 kN/m^2) allowance for future wearing surface, and AASHTO-LRFD HL-93 live loads.
- The strand is 0.6 in. (15 mm) diameter and Grade 270 (1860 MPa). Mild reinforcement is Grade 60 (410 MPa).
- Ten 1⅜ in. (35 mm) diameter, Grade 150 (1030 MPa) threaded rods are placed in the interface strip in option 3. The rods extend 30 ft (9.1 m) beyond the centerline of the pier into each span. Each rod is 50 ft (15 m) long. The rods are staggered such that 50% extend 30 ft (9.1 m) into one span and 20 ft (6.1 m) into the other span.

The design was performed using in-house spreadsheets and checked with PSBeam V4 in accordance with the 2015 edition of the AASHTO LRFD specifications. Typically, options 1 and 2 would allow a certain limit of concrete tensile stress at the bottom fibers of concrete in the positive moment zone near midspan due to the Service III loading combination. However, to simplify comparison between the options, the stress limit is set at zero for all three options. In the analysis, the following items were considered:

- maximum positive moment section, assumed at midspan for option 1 and at 40% of the span for options 2 and 3

Table 1. Design results for example bridge, positive moment

	Option 1: simple span	Option 2: continuous for SIDL + LL	Option 3: continuous for deck + SIDL, LL (threaded rod continuity system)
Girder maximum positive moment, kip-ft	844	810	810
Deck and haunch maximum positive moment, kip-ft	800	940	856
Barrier maximum positive moment, kip-ft	188	105	105
Future wearing surface maximum positive moment, kip-ft	184	129	176
Live load maximum positive moment, kip-ft	1453	1345	1709
Maximum girder spacing, ft	5.90	7.33	10.00
Number of 0.6 in. diameter strands in each girder	38	38	38

Note: LL = live load; SIDL = superimposed dead load. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip-ft = 1.356 kN-m.

- maximum negative moment section, assumed at the face of the pier diaphragm
- maximum shear at face of the pier diaphragm, which was found to be within code limits and no reinforcement was determined

Maximum positive moment section

Table 1 shows the values of the maximum moments for the three different options. These values are functions of the girder spacing. Iterations were conducted to determine the maximum possible spacing for each of the options. The maximum spacing and number of required bottom flange strands are also given in the table. The design was controlled by Service III loading combinations. The concrete strength at release was found to be adequate in all cases.

For a 100 ft (30.5 m) wide bridge, assuming that the deck overhang length is half of the girder spacing, the number of required girder lines is 17, 14, and 10 for options 1, 2, and 3, respectively. Therefore, use of the threaded rod continuity system allows for a significant reduction in the number of beams required for a given span length and girder depth. Another way to state the advantage of the threaded rod continuity system is that it results in a reduction of girder depth for a given span and spacing or an increase in span length for a given depth and girder spacing. For the same bridge span, girder depth, and girder spacing, the threaded rod continuity system will result in improved behavior and will reduce the required prestress and concrete strength at prestress release. It is not likely that bottom fiber cracking at the pier will occur because the permanent negative moment will exceed the positive restraint moments due to volume change effects.

Critical negative moment section

The negative moment zone for options 2 and 3 are treated similarly to a conventionally reinforced zone. Analysis is demonstrated here for option 3 only. The reinforcement is determined based on flexural strength design. Then, crack control and fatigue criteria are satisfied. **Table 2** gives the negative moments at the face of the pier diaphragm for various loading cases.

There are special provisions for the calculation of the fatigue effects of truck loading in the AASHTO LRFD specifications. The live load distribution factor, which is computed to be 0.522, is determined for one lane only. A live load factor of 1.50, representing infinite fatigue life, is used. The dynamic allowance factor is 0.15. The resulting moment is -546 kip-ft (-740 kN-m).

Using strain compatibility, flexural strength analysis of the precast concrete section for a factored load of 2093 ft-kip (2838 kN-m) results in a required area of 6.0 in.^{2} (3900 mm^2) of Grade 150 (1030 MPa) steel, or approximately four 1⅜ in. (35 mm) diameter rods. The provided area is 10 × 1.58 in.^{2}, which equals 15.8 in.^{2} (10,200 mm^2). Having a provided area that is larger than the required area helps with fatigue and crack control performance and still contributes to the overall strength due to full load. By performing the flexural strength analysis again with the full load and with two types of steel, 15.8 in.^{2} of Grade 150 in the interface strip and an unknown amount of Grade 60 (410 MPa) steel in the deck, the required amount of steel in the deck can be determined. Assuming no. 5 (16M) reinforcing bars spaced 12 in. (300 mm) at both the top and bottom layers in the deck throughout the length of the bridge, the flexural strength of the composite section is 5711 kip-ft (7743 kN-m), which slightly exceeds the factored moment of 5098 kip-ft (6912 kN-m). Therefore, the flexural capacity is sufficient without any additional deck reinforcement at the negative moment area. However, additional deck reinforcement over the pier is required to satisfy other design criteria at service limit states, such as crack control. An additional no. 7

Table 2. Design results for example bridge, negative moment

	Option 1: simple span	Option 2: continuous for SIDL + LL	Option 3: continuous for deck + SIDL, LL (threaded rod continuity system)
Girder	0	0	0
Deck and haunch maximum negative moment, kip-ft	0	0	-1288
Barrier maximum negative moment, kip-ft	0	-173	-173
Future wearing surface maximum negative moment, kip-ft	0	-211	-288
Live load (fatigue) maximum negative moment, kip-ft	0	-435	-537
Live load (Service I) maximum negative moment, kip-ft	0	-1277	-1624
Factored load, precast concrete (Strength I) maximum negative moment, kip-ft	0	0	-1932
Total factored load (Strength I) maximum negative moment, kip-ft	0	-2767	-5098
Required additional longitudinal reinforcement over pier	n/a	Two rows of no. 8 at 12 in. spacing in deck	Ten 1⅜ in. rods in interface strip plus one row of no. 7 at 12 in. spacing in deck

Note: LL = live load; n/a = not applicable; SIDL = superimposed dead load. No. 7 = 22M; no. 8 = 25M; 1 in. = 25.4 mm; 1 kip-ft = 1.356 kN-m.

(22M) reinforcing bar spaced at 12 in. is assumed at the top layer of the deck over the pier.

The total negative moment due to dead loads is -1748 kip-ft (-2370 kN-m). The threaded rod stress due to this moment is 33.9 ksi (234 MPa). This is smaller than the allowable stress limit for threaded rods due to dead loads, which is 54.0 ksi (372 MPa).

When performing service load analysis, the multistage construction and multiple layers of steel should be recognized. This numerical example will be used to illustrate how the stresses in the reinforcement in tension develop as the loads are applied incrementally during construction and service of the bridge. This will allow designers to assess the performance of the bridge for crack control and fatigue. The critical section is taken at the face of the pier diaphragm. Prestressing is ignored in the design at the negative moment section to avoid complication in the determination of cracked-section properties. This is a conservative assumption because prestress in the bottom of the section increases the neutral axis depth and reduces the top steel tensile strains and stresses compared with a nonprestressed section.

A spreadsheet program was developed to perform the design and incorporates the strain compatibility method, force equilibrium, and moment-curvature relationship. **Figure 16** illustrates the negative moment section, consisting of precast concrete girders, and cast-in-place concrete interface strip, haunch, and deck slab. Also included are the corresponding strain and stress diagrams in the precast concrete and composite sections.

Figure 16 illustrates the strain and stress diagrams in concrete and threaded rods when the deck weight is applied to a girder that is made continuous through the interface strip and threaded rods. The moment causes the section to be cracked, based on a simple uncracked-section analysis. The standard assumption of ignoring concrete resistance on the tension side of the neutral axis is employed here. Similarly, the strain and stress diagrams are shown for the composite section subjected to the superimposed dead and live loads. Superimposition of the strain diagrams in Fig. 16 results in a total strain diagram and its corresponding stress diagram. Similarly, the steel strain is superimposed before and after the composite section is formulated.

A spreadsheet was used to determine the moment-stress diagrams for the threaded rods and the top layer of the longitudinal deck reinforcement (**Fig. 17**). Two diagrams are shown for the threaded rods: one diagram for the precast concrete section only and the other for the composite section after the deck concrete becomes composite with the precast concrete girder. The diagram of the top deck reinforcement is shown for the composite section only. Also shown in the vertical axis of the figure are the moments due to various loads, including deck (and haunch) weight, superimposed dead load, fatigue live load, and live load (at Service I). **Figure 18** shows the moment-curvature diagrams for both precast concrete and composite sections.

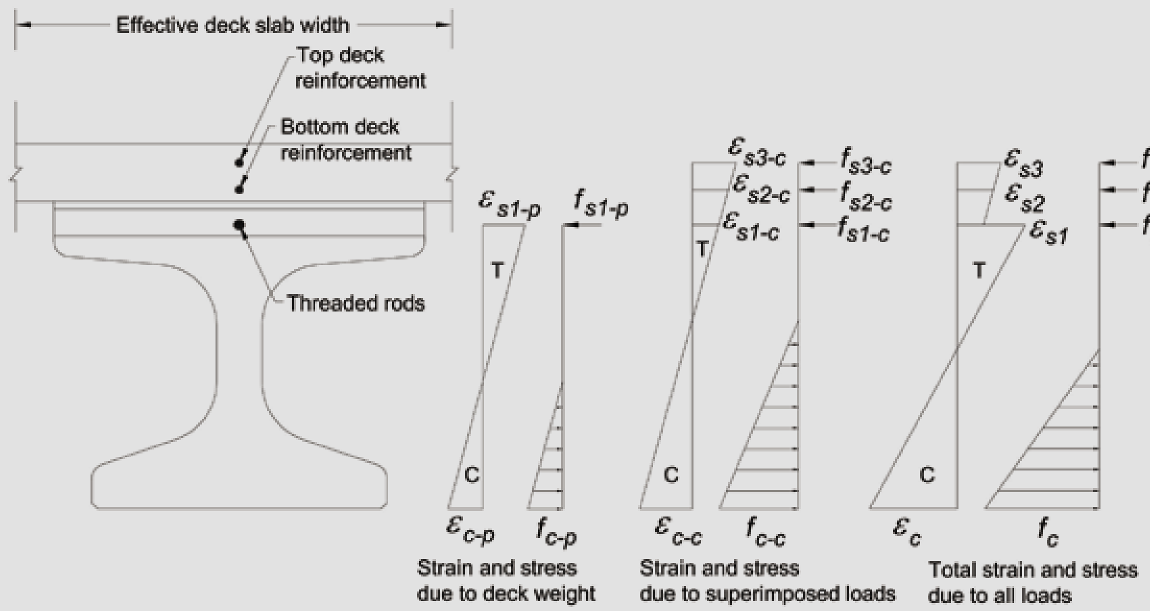

Figure 16. Moment–steel stress diagrams for the threaded rods and top deck reinforcement. Note: f_c = concrete compressive stress due to all loads; $f_{c\text{-}c}$ = concrete compressive stress due to superimposed dead and live loads; $f_{c\text{-}p}$ = concrete compressive stress due to deck weight; f_{s1} = stress in the threaded rod due to all loads; $f_{s1\text{-}c}$ = stress in the threaded rod due to superimposed dead and live loads; $f_{s1\text{-}p}$ = stress in the threaded rod due to deck weight; f_{s2} = stress in the bottom deck reinforcement due to all loads; $f_{s2\text{-}c}$ = stress in the bottom deck reinforcement due to superimposed dead and live loads; f_{s3} = stress in the top deck reinforcement due to all loads; $f_{s3\text{-}c}$ = stress in the top deck reinforcement due to superimposed dead and live loads; ε_c = strain of the concrete at the extreme compression fiber; $\varepsilon_{c\text{-}c}$ = concrete strain at the bottom of the precast beam due to superimposed loads after the composite section is made; $\varepsilon_{c\text{-}p}$ = concrete strain at the bottom of the precast beam due to deck weight before the composite section is made; ε_{s1} = strain of the threaded rods due to all loads; $\varepsilon_{s1\text{-}c}$ = strain of the threaded rods due to superimposed dead and live loads; $\varepsilon_{s1\text{-}p}$ = strain of the threaded rods due to deck weight before the composite section is made; ε_{s2} = strain of the bottom deck reinforcement due to all loads; $\varepsilon_{s2\text{-}c}$ = strain of the bottom deck reinforcement due to superimposed dead and live loads; ε_{s3} = strain of the top deck reinforcement due to all loads; $\varepsilon_{s3\text{-}c}$ = strain of the top deck reinforcement due to superimposed dead and live loads. 1 kip-ft = 1.356 kN-m; 1 ksi = 6.895 MPa.

The stresses in the threaded rods and the top deck reinforcement due to fatigue and service load combinations can be determined using the moment-stress diagrams. Alternatively, a simplified approach to determine the stress of the reinforcing bars may be used:

$$f_s = \frac{M}{A_s jd} \tag{6}$$

where

A_s = area of the steel

j = ratio of lever arm of resisting couple to depth = approximately 0.9

d = distance between the concrete compression fiber and the centroid of the steel

Following is a brief discussion on use of this simplified approach.

Crack control check The negative moment due to the deck weight, superimposed dead loads, and live load is -3372 ft-kip (-4572 kN-m) at the face of the pier diaphragm, which corresponds to a stress of about 28.8 ksi (198.6 MPa) in the top deck reinforcement (Fig. 17).

The allowable reinforcing bar spacing is calculated in Eq. (7).

$$s = \frac{700\gamma_e}{\beta_s f_{ss}} - 2.5d_c \tag{7}$$

where

γ_e = 0.75

β_s = 1.09

d_c = 29.4 in.

Thus, the allowable bar spacing is 9.4 in. (240 mm). The provided top deck reinforcing bars are no. 5 (16M) plus no. 7 (22M) spaced at 12 in. (300 mm), corresponding to bar spacing of 6.0 in. (150 mm). Therefore, crack control criteria are met.

If the simplified approach is used, Eq. (8) determines the stress of the top deck reinforcement f_s.

$$f_s = \frac{M}{A_s jd} = \frac{(3372)(12)}{(45.8)(0.9)(38.9)} = 25.2 \text{ ksi (174 MPa)} \tag{8}$$

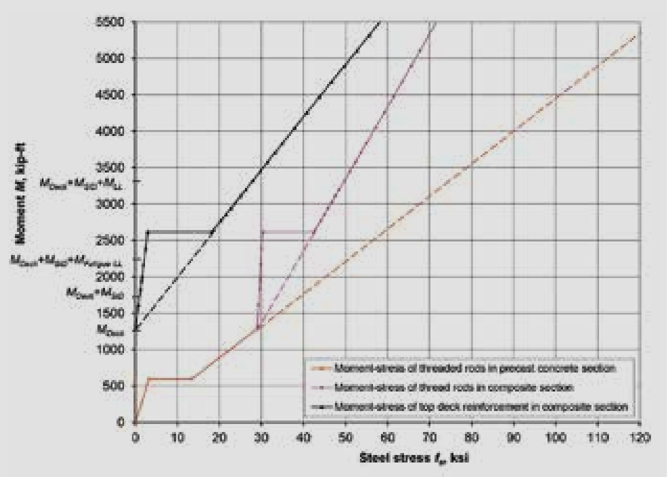

Figure 17. Girder/deck section at the negative moment region and corresponding strain and stress diagrams in precast concrete and composite sections. Note: M_{Deck} = moment due to deck weight; $M_{FatigueLL}$ = moment due to fatigue live load; M_{LL} = moment due to live load; M_{SID} = moment due to superimposed dead load. 1 kip-ft = 1.356 kN-m; 1 ksi = 6.895 MPa.

This stress corresponds to an allowable bar spacing of 11.8 in. (300 mm). This simplified approach gives different results from that of Fig. 17 because it ignores the multistage construction and simplifies the multiple layers of steel.

Fatigue checks The stress of the threaded rods due to dead load plus fatigue live load moment of -2285 kip-ft (-3098 kN-m) is 39.0 ksi (269 MPa) based on the moment-stress diagram of Fig. 17. Thus, the live load stress range is 39.0 − 33.9, which equals 5.1 ksi (35 MPa). This stress range is less than the limit determined by $36 - f_{min}/3 = 36 - 33.9/3 = 24.7$ or 18 ksi (170 or 124 MPa), whichever is smaller.

Thus, the fatigue stress limit is satisfied.

Similarly, the stress of the top deck reinforcement due to fatigue live load is 14.0 − 7.0, which equals 7.0 ksi (48 MPa). The allowable stress limit is $f_f = 24 - f_{min}/3 = 24 - 7.0/3 = 21.7$ ksi (150 MPa).

Thus, the fatigue limit in the deck reinforcement is not exceeded.

The concrete compressive stress limit due to fatigue load shall be satisfied under the following load combination (Eq. [9]):

$$0.5\left(f_{DL} + f_{Prestress}\right) + 1.5 f_{FatigueLL+I} \leq 0.40 f_c' \qquad (9)$$

where

$f_{FatigueLL+I}$ = concrete compressive stress due to fatigue live load

Computing the concrete stresses at the face of the diaphragm, given that of the required 38 strands it was found that 6 needed to be draped and 8 debonded, and substituting in the fatigue stress formula:

$$0.5\,(f_{DL} + f_{Prestress}) + 1.5\,f_{FatigueLL+I} = 0.5(2.709 + 2.890) + 1.5(0.567) \leq 0.40 f_c'$$

3.650 ksi (25.17 MPa) ≤ 4.0 ksi (28 MPa)

The presence of prestress in the end zone could be further reduced, if needed, to improve the compressive stresses, which would increase the margin against the fatigue limit.

Impact of threaded rod continuity on positive volume-change restraint moment

The proposed threaded rod system has significant advantages compared with the conventional system of applying the deck weight to a simple-span beam. One of these advantages is essentially eliminating the need for crack control positive moment reinforcement at intermediate piers. This cracking is caused primarily by positive moment due to creep caused by prestress and thermal gradient. It is somewhat offset by negative moment due to creep caused by beam weight and deck weight and by the elastic moment caused by superimposed dead loads.

To illustrate the value of providing continuity before the deck is placed, two cases applied to the beam example used in preceding sections were considered (**Table 3**). The first case corresponds to the example just described. The second case is the same example with the deck weight applied to a simple span with 46 strands, instead of the 38 strands used in the first case. The increased number of strands satisfies the Service III limit state with the second case. Time-dependent analysis was performed using the age-adjusted effective modulus method.[18] The negative elastic moment due to deck weight in the first

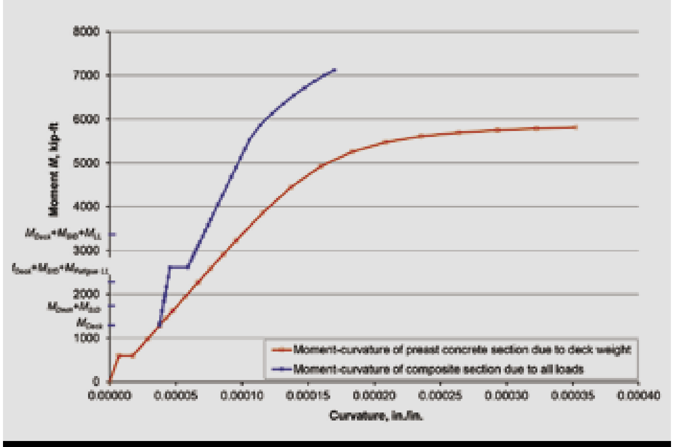

Figure 18. Moment–curvature diagrams in the precast concrete and composite sections. Note: M_{Deck} = moment due to deck weight; $M_{FatigueLL}$= moment due to fatigue live load; M_{LL} = moment due to live load; M_{SID} = moment due to superimposed dead load. 1 in. = 25.4 mm; 1 kip-ft = 1.356 kN-m.

Table 3. Comparison of beam moment at pier centerline between the proposed system and the conventional system

	Proposed system: continuous for deck and superimposed dead and live loads	Conventional system: continuous for superimposed dead and live loads
Creep moment due to girder weight, kip-ft	-459	-459
Creep moment due to prestressing, kip-ft	1059	1282
Moment due to thermal gradient, kip-ft	943	843
Elastic moment due to deck weight, kip-ft	-1395	0
Creep moment due to deck weight, kip-ft	0	-653
Elastic moment due to barrier weight, kip-ft	-188	-188
Total moment, kip-ft	-40	925

Note: 1 kip-ft = 1.356 kN-m.

case is more than the negative creep moment in the second case due to the same load (Table 3). Also, due to the higher prestress, the positive creep moment in the second case is higher than the same moment with the threaded rod continuity. As a result, net moment due to elastic and creep effects is negative for the threaded rod system, with no likelihood of having the joint open in the bottom and need for providing significant continuity reinforcement. Obviously, there is no guarantee that this condition will exist in all bridges, and detailed analysis may be required in some applications.

Conclusion

This paper describes a system for making precast concrete girders continuous for deck weight through the use of high-strength threaded rods. Early development of the threaded rod continuity system included a bolted connection detail in which threaded rods were embedded in the girder top flange and coupled in the field prior to deck placement. The recent development of the threaded rod continuity system involves adding cast-in-place concrete strips over the pier that house the threaded rods and the pertinent confinement reinforcement. The concrete for these strips is placed before the deck concrete is placed, creating continuity for deck weight. A design approach at the negative moment region is presented and involves the strain compatibility method, force equilibrium, and moment–curvature relationship. The stresses in the threaded rods and the deck reinforcement can be analyzed to account for sequentially introduced loading, which allows for the determination of the stresses in the threaded rods and deck reinforcement at fatigue and service limit states. The authors address design criteria and procedures to account for the threaded rod continuity system. Load testing of the threaded rod continuity system and system implementation are discussed. A numerical example compares various systems. Also included in the example is the design of a threaded rod continuity system with a number of selective criteria. The conclusions are as follows:

- Because precast concrete girders are made continuous for approximately two-thirds of the total loads, the threaded rod continuity system results in a reduced demand for prestressing force and for high-strength concrete at release. Using the proposed system, the same girder size can span about 10% to 15% longer than in the conventional system. Alternatively, a wider girder spacing can be used, which may result in fewer girder lines.

- Overall structural performance is improved, as the permanent negative moment at deck placement likely offsets the positive restraint moment, which essentially eliminates possible cracking at the bottom of the pier diaphragm.

- The threaded rod continuity system is believed to be an efficient solution to make deck weight continuous without resorting to posttensioning. Implementation of this system has shown that it can be constructed efficiently without need for a specialty subcontractor.

- The threaded rod continuity system provides a feasible alternative for concrete superstructures to compete against long-span steel highway bridges and results in substantial cost savings.

- The threaded rod continuity system is particularly effective when a shallow structure is mandatory. The proposed system can be also incorporated into other types of precast concrete girders, such as inverted-tee beams and box beams.

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Notation

A_{ci} = area of each concrete layer

A_s = area of steel

A_{si} = area of steel row number i

c = distance from the extreme compression fiber to the neutral axis

d = distance between the concrete compression fiber to the centroid of the steel

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest to it

d_i = depth from extreme compression fiber of steel row number i

E_c = modulus of elasticity of concrete

E_{si} = modulus of elasticity of the steel used in row i

f_c = concrete compressive stress due to all loads

f_c' = specified compressive strength of concrete for use in design

f_{c-c} = concrete compressive stress due to superimposed dead and live loads

f_{ci} = stress of each concrete layer

f_{c-p} = concrete compressive stress due to deck weight

f_{DL} = concrete stress due to dead loads

f_f = stress range in the threaded rods

$f_{FatigueLL}$ = concrete stress due to fatigue live loading

$f_{FatigueLL+I}$ = concrete compressive stress due to fatigue live load

f_{min} = minimum live-load stress resulting from the fatigue limit state I load combination combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads

$f_{Prestress}$ = concrete stress due to effective prestress

f_s = tensile stress in steel reinforcement

f_{si} = stress of steel row number i

f_{ss} = tensile stress in steel reinforcement at the service limit state

f_{s1} = stress in the threaded rod due to all loads

f_{s1-c} = stress in the threaded rod due to superimposed dead and live loads

f_{s1-p} = stress in the threaded rod due to deck weight

f_{s2} = stress in the bottom deck reinforcement due to all loads

f_{s2-c} = stress in the bottom deck reinforcement due to superimposed dead and live loads

f_{s3} = stress in the top deck reinforcement due to all loads

f_{s3-c} = stress in the top deck reinforcement due to superimposed dead and live loads

f_{yi} = yield strength of the steel grade used in row i

h = overall thickness or depth of the component

I = moment of inertia of a section

j = ratio of lever arm of resisting couple to depth

k = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

M = bending moment

M_{Deck} = moment due to deck weight

$M_{FatigueLL}$ = moment due to fatigue live load

M_{LL} = moment due to live load

M_{SID} = moment due to superimposed dead load

Q = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

R = constant produced from curve fitting the power formula to the stress-strain relationship for the steel grade used in row i

s = spacing of mild-steel reinforcement in the layer closest to the tension face

y_b = distance between girder bottom fiber and centroid

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

$(\Delta F)_{TH}$ = allowable stress range in mild reinforcement

ε_c = strain of the concrete at the extreme compression fiber

$\varepsilon_{c\text{-}c}$ = concrete strain at the bottom of the precast beam due to superimposed loads after the composite section is made

$\varepsilon_{c\text{-}p}$ = concrete strain at the bottom of the precast beam due to deck weight before the composite section is made

ε_{s1} = strain of the threaded rods due to all loads

$\varepsilon_{s1\text{-}c}$ = strain of the threaded rods due to superimposed dead and live loads

$\varepsilon_{s1\text{-}p}$ = strain of the threaded rods due to deck weight

ε_{s2} = strain of the bottom deck reinforcement due to all loads

$\varepsilon_{s2\text{-}c}$ = strain of the bottom deck reinforcement due to superimposed dead and live loads

ε_{s3} = strain of the top deck reinforcement due to all loads

$\varepsilon_{s3\text{-}c}$ = strain of the top deck reinforcement due to superimposed dead and live loads

ε_{si} = strain of steel row number i

ϕ = curvature in a section

γ_e = exposure factor

About the authors

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Abstract

Precast, prestressed I-girder bridges are generally designed as simple spans for the girder self-weight and deck weight and continuous spans for superimposed dead and live loads. Because the superimposed loads are only about one-third of the total load, structural efficiency can be further improved if continuity is achieved for the deck weight. This paper presents a threaded rod continuity system to make precast concrete girders continuous for the deck weight without resorting to posttensioning. The threaded rod continuity system can increase bridge span capacity from 10% to 15% and essentially eliminate possible cracking at the bottom fiber of the pier diaphragm. The threaded rod continuity system allows precast concrete to compete favorably with steel in long-span highway bridges. This paper covers the historical development of the threaded rod continuity system. This paper also includes design criteria, experimental tests, design procedures, system implementation, and a numerical example.

Keywords

Bridge; continuity; girder; high-performance concrete; I-girder; threaded rod continuity system.

Review policy

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

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