Restraint Moments in Precast/Prestressed Concrete Continuous Bridges

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To eliminate deck joints and shoulder piers, continuous span (jointless) bridges are becoming an attractive option. Different continuity methods and construction sequences have different time-dependent effects on the behavior of the bridge system. These effects are illustrated in this paper. To achieve a favorable performance of bridges made continuous for deck loads, recommendations for achieving continuity and for desirable construction sequence are presented. A new continuity detail using high strength threaded rods is proposed. Based on a cost comparison with currently available continuity methods and a full-scale NU1100 I-beam experiment, this continuity detail is shown to eliminate the potential for bottom concrete creep restraint cracking at pier locations and to increase the span capacity by as much as 20 percent.

Bridge engineers are continually being challenged to design bridges with longer spans. This trend is partly due to safety requirements that dictate the elimination of piers adjacent to roadway shoulders in overpass bridges and the need to reduce the number of piers in water crossings. The desire to eliminate deck joints and their potential for long-term maintenance costs has also contributed to the need for developing techniques to create continuity in bridge superstructures.

More factors need to be considered in the design and construction of jointless bridges as compared to simply supported bridges. One such factor is design for time-dependent restraint positive moment at pier locations. A major objective of this paper is to present a parametric study of the effects of creep and shrinkage of concrete in a continuity analysis of I-beam bridges.

The restraint positive moment can cause cracking near the bottom of the piers if certain conditions exist. This can occur in situations where the depth of the precast beam is relatively small, the prestressing force in the beam is large, the beam is installed at a young age, and the deck is cast much later than the diaphragm. However, this last parameter may become negligible if an appropriate construction sequence is followed and a nominal amount of prestressing strands are extended into the end diaphragms. Extending the bottom strands into the end diaphragms serves the dual purpose of controlling time-dependent restraint moment cracking and improving shear resistance.

Another important issue is the design criterion for which these time-dependent effects should be considered. For example, should they be considered strictly as a serviceability check or reviewed at both serviceability and strength limit states? According to the provisions of the current AASHTO LRFD Specifications,[1] these time-dependent effects should be considered at both serviceability and strength limit states.

Nevertheless, Mattock[2] and many others have indicated that the influence of creep and shrinkage is restricted to deformations and the possibility of cracking at the service load level, and that deformations due to creep and differential shrinkage do not influence the ultimate load-carrying capacity of continuous beams. The authors of this paper believe this issue deserves further investigation.

The common method of achieving continuity for live loads is to provide negative moment mild steel reinforcement in the composite deck over the interior supports. In this conventional continuity method, about two-thirds of the load, beam weight and deck weight are introduced to a simple beam span. Only about one-third of the total load, i.e., the live load and future superimposed dead loads, is applied to the continuous structure.

A new continuity detail using high strength threaded rods is presented in this paper. In this continuity method, live load and future superimposed dead loads as well as the deck weight are applied to the continuous structure. By doing so, the span capacity can be increased by as much as 20 percent. Other features of this continuity method include increased composite action between the deck and the precast beam as well as elimination of the potential for bottom time-dependent restraint moment cracking problems at the piers.

Based on a cost comparison with currently available continuity methods and a full-scale NU1100 I-beam experiment, the newly developed continuity detail has shown much promise not only for I-beam bridge applications but also for inverted-tee and other bridge systems.

TIME-DEPENDENT ANALYSIS

Under the combined effects of creep and shrinkage of concrete and relaxation of prestressing steel, prestressed concrete members gradually change with time depending on the type of structure and construction methods used. For non-composite simple span bridges (such as deck bulb-tee or AASHTO box beam bridges), these time-dependent changes manifest themselves in the form of a loss in effective levels of prestressing and changes in camber or deflection. For composite simple span bridges (such as I-beam or inverted-tee bridges), these time-dependent effects result not only in loss of prestress and changes in camber or deflection, but also in changes of stresses in the composite sections.

Because the beams are not restrained in the case of simple span construction, they can freely rotate and will not have additional moments. However, when beams are set on piers and abutments and made continuous through the diaphragm over the pier and/or cast-in-place deck, the prestressed concrete beams are restrained at their ends and the bridge becomes a continuous bridge. As a result, any time-dependent deformations that occur after the time that the deck is cured will generally induce forces and moments in the beams.

Approximate methods as well as more rigorous methods are available to the designer to account for these effects. The more rigorous step-by-step computer method[3] is used to study the impact of the time-dependent effects in this paper. However, the rigorous and approximate methods are all essentially the same as a conventional elastic analysis of a prestressed concrete cross section using transformed section properties. In place of a conventional modulus of elasticity, however, the age adjusted effective modulus is used for all concrete elements in the section.

Initial strains must be considered in the step-by-step method. An initial strain is defined as a strain that is not caused directly by an applied stress. Outside of a time-dependent analysis, temperature strain may be the most familiar example of an initial strain. In the time-dependent analysis of concrete members, the initial strains normally considered are:

1. Free shrinkage of concrete occurring during the interval being considered.

2. Creep strains of concrete, occurring during the interval being considered, that are due to previously applied loads.

3. The apparent steel strain due to relaxation of prestressing steel during the time interval being considered.

To incorporate initial strains into cross section analysis, it is convenient to calculate a fictitious restraint "load" equal to that which will restrain the initial strains described above. The restraint initial load is then subtracted from any real loads applied to the section. Next, using the net load, an analysis is performed in a manner similar to conventional transformed section analysis. Finally, internal forces are calculated using two components. First, the internal forces associated with the net load applied to the entire composite section are calculated. These are then added to the individual element restraint forces to give the total actual forces on an individual element of the cross section.

The procedure described above is too time consuming to perform by hand calculation. Therefore, a computer program (CREEP3), developed in the 1970s,[4] is used in this paper. In this step-by-step computer procedure, the time is divided into intervals. The stresses and deformations at the end of each time interval are calculated in terms of the stress applied in the first interval and the stress increments that occurred in preceding intervals.

Linear creep growth is adopted. With the usual assumption that plane cross sections remain plane, the axial strain, ε, at any cross section can be related to the axial force, N. During interval i, an increment axial strain, $\Delta\varepsilon(i)$, occurs:

$$\Delta\varepsilon(i) = \frac{\Delta N(i)}{AE_{ce}(i)} + \Delta\varepsilon'(i) \quad (1)$$

where

$E_{ce}(i)$ = effective modulus of elasticity of concrete at middle of interval i

$\Delta\varepsilon'(i)$ = initial strain in ith interval and defined by Eq.(2)

$$\Delta\varepsilon'(i) = \sum_{j=1}^{i-1} \frac{\Delta N(j)}{AE_c(j)}\left[C\left(i+\frac{1}{2},j\right) - C\left(i-\frac{1}{2},j\right)\right] + \Delta\varepsilon_{sh}(i) \quad (2)$$

where

j = time at middle of jth interval

$(i - 1/2)$ = time at beginning of ith interval

$(i + 1/2)$ = time at end of ith interval

C = creep coefficient

$\Delta\varepsilon_{sh}(i)$ = free shrinkage strain during interval i

$\Delta N(j)$ = axial force increment at middle of interval j

$E_c(j)$ = modulus of elasticity of concrete at middle of interval j

A = cross-sectional area of concrete

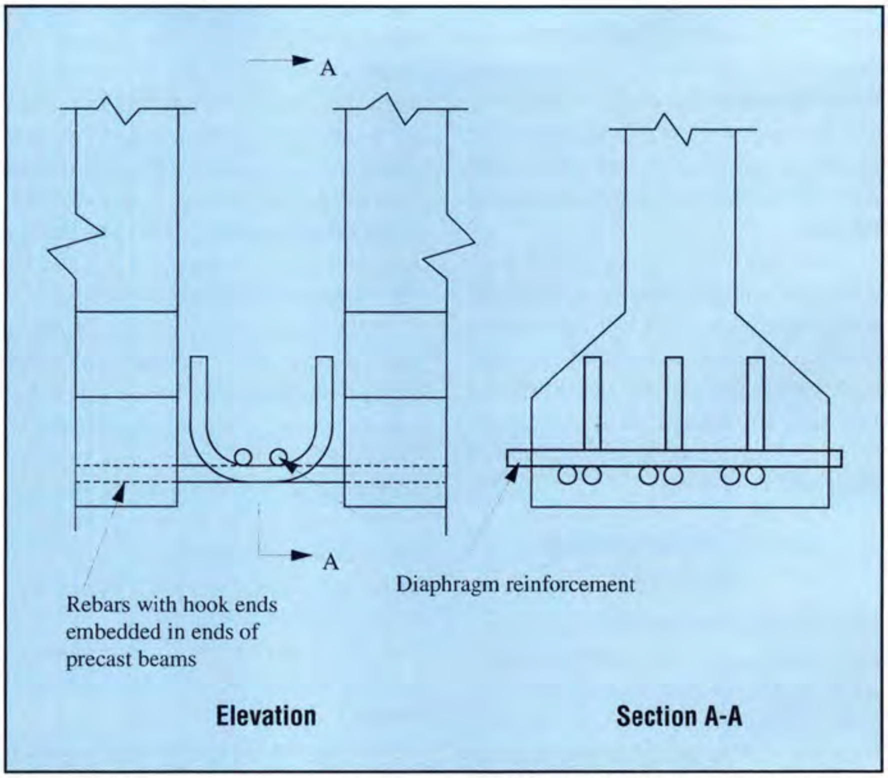


Fig. 1. Positive restraint moment connection details by embedded reinforcing bars.

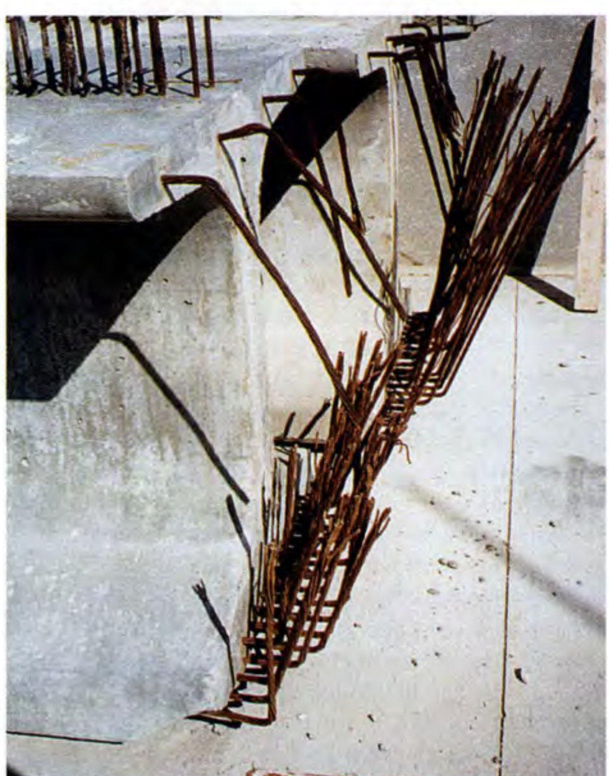


Fig. 2. Prestressing strands being extended and bent.

CONNECTION DETAILS FOR RESTRAINT POSITIVE MOMENT

Depending on the timing and sequence of the construction steps, the time-dependent effects could develop positive restraint moments over the pier, after the beams are restrained by the diaphragm. The amount of the positive moment will depend on the age of the precast members when the diaphragm or deck is cast, the properties of the precast beams and cast-in-place deck concrete, and the bridge characteristics such as beam dimensions, number of strands and beam spacing.

These positive moments induced by creep of concrete require designers to have an understanding of the structural behavior of the bridge in order to properly design the continuity reinforcement over the pier. Over the years, standard reinforcement details for resisting positive moments have been developed. In the late 1960s, researchers at the Portland Cement Association (PCA) developed various connection details to resist the positive moments.[5] Fig. 1 shows one of these connection details.

In the PCA tests,[5] the hooked bars were bent essentially at right angles and a relatively short horizontal bar projection was used to shorten the width of the diaphragm. Although the connection developed substantial moments under static test loads, under fatigue testing most of the connection bars failed after about 670,000 applications of the load. The bars fractured in a brittle manner at the knee of the hook. Another disadvantage of this detail is that it can cause reinforcement congestion at member ends.

In order to form a connection with a mechanism to resist the positive moment and to avoid additional reinforcing bars, it is common practice in Nebraska to extend the bottom prestressing strands of a precast, prestressed beam into the end diaphragms. Fig. 2 shows the bottom strands of the precast beam being extended and bent at the plant. This detail serves the dual purpose of controlling time-dependent restraint moment cracking and improving shear resistance.[6] Based on the excellent performance of this detail in both the laboratory and in practice, the authors recommend that this connection be adopted as a standard detail.

IMPACT OF CONSTRUCTION SEQUENCE

One of the objectives of this paper is to present a parametric study of the time-dependent effects of creep and shrinkage of concrete in continuity analysis of Nebraska University (NU) I-beam bridges.[7] Based on these calculations, the best construction sequence will be recommended later in this paper.

The bridges studied are two-span bridges with NU I-beams of equal length. The beam sizes are NU1100, NU1600, and NU2000. Their depths are 43.3, 63.0, and 78.7 in. (1.1, 1.6, and 2.0 m), respectively. For each beam size, the beam spacing is varied from 6 to 12 ft (1.83 to 3.66 m).

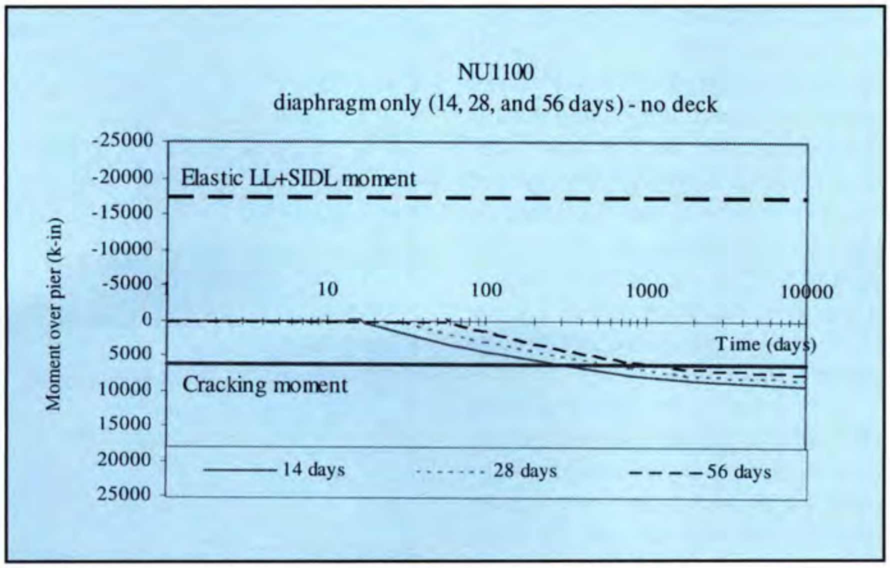


Fig. 3. Time-dependent moment due to diaphragm only (NU1100).

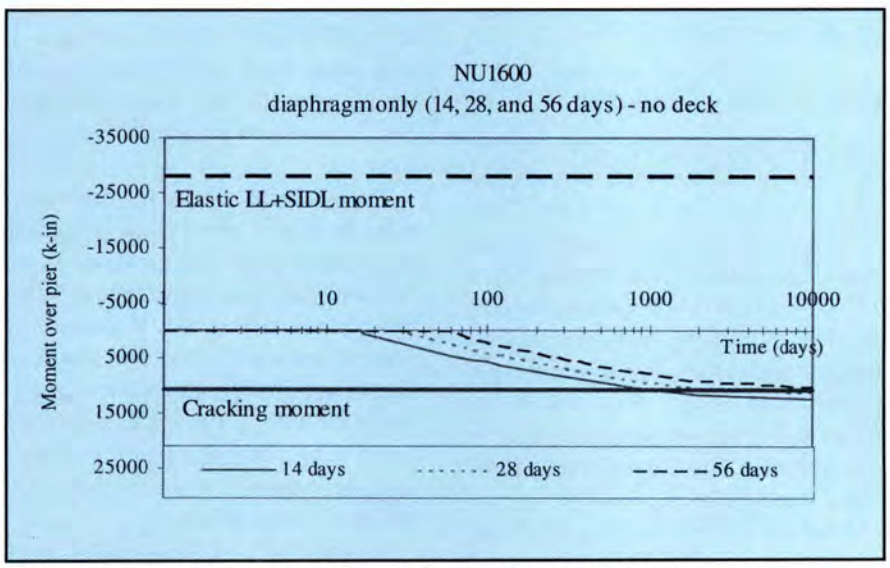


Fig. 4. Time-dependent moment due to diaphragm only (NU1600).

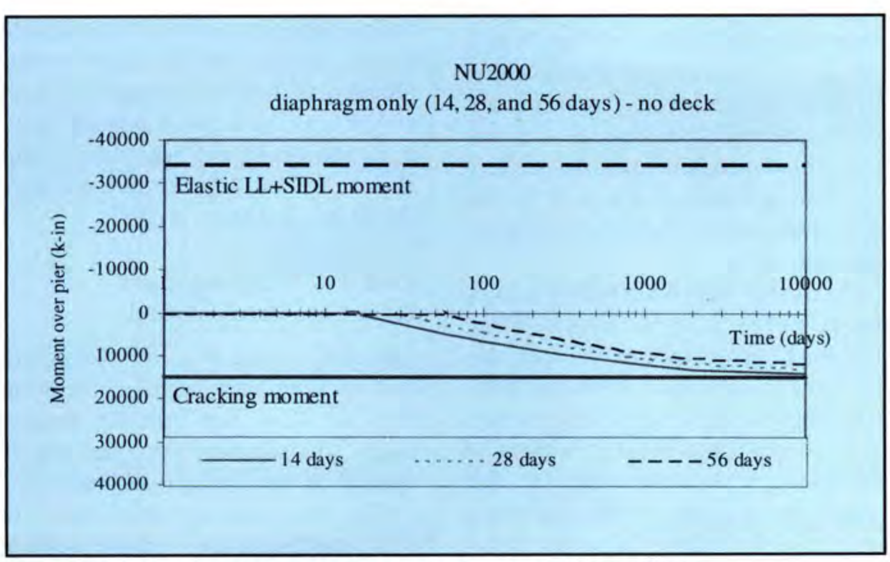


Fig. 5. Time-dependent moment due to diaphragm only (NU2000).

For a specific beam size and spacing, the maximum span and corresponding number of prestressing strands are chosen in the analysis. The thickness of the deck is 8 in. (203 mm). The compressive strengths of concrete at service are $f_c' = 8000$ psi (55.2 MPa) for the beam and $f_c' = 5000$ psi (34.5 MPa) for the deck, respectively. Low relaxation strands [0.5 in. (12.7 mm) in diameter] are used to prestress the beam.

To investigate the time-dependent effect due to different construction sequence, three cases are considered depending on the time when the diaphragm and deck are cast. However, in all three cases the first two construction steps are the same, i.e., (1) release of prestressing force on the first day and (2) application of beam weight on the first day.

Case 1 — Diaphragm Only

In this case, after the beams are set on piers and abutments, only the diaphragms are cast on the 14th, 28th, 56th and 120th day of prestressed beam age, respectively. In practice, a 120-day period is not common or economically desirable. However, it may happen sometimes due to unforeseen construction delays. Also, the development of time-dependent positive moment with time can be clearly illustrated through this case study.

After the diaphragm is cast, it is assumed that a rigid joint is formed and the structure becomes a continuous beam system. The positive moment developed at the joint over the pier is determined and shown in Figs. 3 to 5 for NU1100, NU1600 and NU2000 beams, respectively. It can be observed clearly from Figs. 3 to 5 that the time-dependent restraint positive moment is largely influenced by the age of the precast beam when the diaphragm is cast.

Examine the NU1100 I-beam, shown in Fig. 3, as an example. When the diaphragm is cast on the 14th day of beam age, the time-dependent positive moment developed over the pier will exceed the cracking moment of the section at about the 230th day after casting of the beam. If the diaphragm is cast at 28 days of age, the time-dependent positive moment will be larger than the cracking moment after about 500 days. If the diaphragm is cast even later, it will take a much longer time for the time-dependent positive moment to reach the cracking moment.

The analysis results for other sections in Figs. 4 and 5 show that when the section height of the beam is increased, the time-dependent positive moment is increased. Because the corresponding cracking moment of the beam increases more significantly, it will take a longer time for a beam with a deeper section to have a time-dependent positive moment over the cracking moment.

In summary, to avoid positive restraint moment cracking, the time period after diaphragm casting should be limited to 230 days for the NU1100 beam. Deeper beams will tolerate longer time periods. It is not recommended that the diaphragm be cast earlier than 14 days of precast beam age, and preferably the diaphragm should be cast at least 28 days after the prestressed beam is cast.

Case 2 — Diaphragm and Deck Cast Simultaneously

In this second case, the diaphragm and deck are cast simultaneously on the 28th, 42nd, and 56th day, respectively. The superimposed dead load is applied on the 120th day of beam age.

After the deck and diaphragm are cast simultaneously, negative moment will be developed due to creep and differential shrinkage, as shown in a typical NU1600 section in Fig. 6. The developed negative moment will be increased depending on the time of concrete casting. The later the concrete is cast, the higher the amount of negative moment that will be developed.

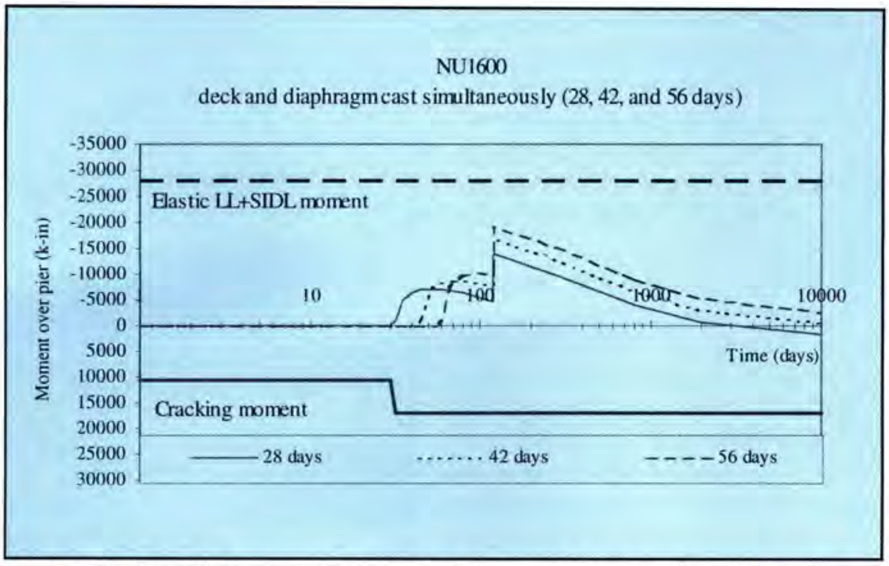


Fig. 6. Time-dependent moment due to diaphragm and deck simultaneous casting.

In general, the magnitude of the negative moment is relatively small compared with the elastic moment due to the live load and superimposed dead load. It is possible to have some positive moment after a very long time, even as much as 30 years. This positive moment will not be larger than the cracking moment of the composite section.

The analysis results for this construction sequence indicate that the time-dependent effects will not have a large impact on the behavior of the beams. However, this construction sequence may be inconvenient to the contractor.

One compromise solution would be to first cast the diaphragm with unbonded joints to provide for lateral stability. These unbonded joints will allow the beams to rotate freely and, therefore, cause no restraint of the ends of the beams. Of course, time-dependent effects will induce moments at the section over the pier after the deck is cast.

Case 3 — Deck Cast After Diaphragm

In this last case, the diaphragm is cast on the 28th, 42nd, and 56th day,

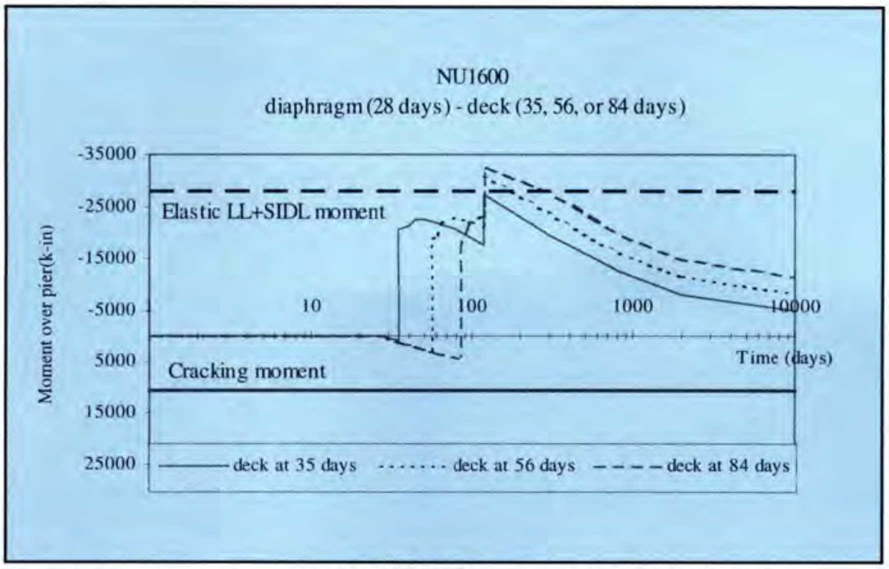


Fig. 7. Time-dependent moment due to diaphragm and deck separate casting.

respectively. Then the cast-in-place deck is placed at 7, 28 and 56 days after the diaphragm is cast. Finally, the superimposed dead load is applied at 30 to 60 days after deck placement.

After the diaphragm is cast, as in Case 1, a rigid joint is assumed to form over the pier. The time-dependent restraint positive moment is developed with time as the structure becomes a continuous structure, as shown in Fig. 7. However, this positive moment is very small. When the deck is cast, the deck weight will be applied to a continuous beam and cause negative moment at the section over the pier. The time-dependent effect will increase the negative moment somewhat. The negative moment due to deck weight and time-dependent effects is more than two-thirds of the negative moment due to elastic live load and superimposed dead load. The total negative moment at the section over the pier is thus much higher than that assumed in design, i.e., reinforcement for live load and superimposed dead load.

The main challenge for this case is how to resist the negative moment over the pier due to the deck weight and time-dependent effects. The only resistance of the continuous beam-diaphragm joint to negative moments is the mechanical interlock due to beam embedment into the diaphragm. This resistance is not quantifiable or reliable. As a result, deck placement and the associated beam deformation can cause premature cracking of the beam-diaphragm joint. To solve this problem, adequate top continuity reinforcement of the beams should be made to resist the negative moment before the deck placement, which will be discussed in the following sections.

COMPARISON OF CONTINUITY METHODS FOR NEGATIVE MOMENT

In the early stages of this research, a detailed study was performed of different continuity methods for negative moment. A brief summary of the study is given below.

Existing Continuity Methods

Method 1: Reinforcement in the deck slab — In this method, the beam ends are embedded in cast-in-place diaphragms. Reinforcing bars in the negative moment zones are placed within the deck thickness and the deck is cast.

This method is the simplest of the existing methods. It requires no specialty contractors. When using this method, however, the beam essentially acts as a simple span under its own weight, the deck slab weight and the construction loads. This loading combination is generally the majority of the total load. The beam is then only continuous under the effects of a relatively small superimposed dead load (traffic barriers and wearing surface) and live load.

Method 2: Full Length Post-Tensioning — This method requires full length ducts and usually necessitates widening of the beam webs. It also requires anchorage blocks to resist stress concentrations at the anchorage zones. These anchorage blocks are specified to be as wide as one of the two flanges and as long as three-fourths of the beam depth,[8] unless an optimized anchorage block is used.[9] The blocks not only add to the weight of the beam, but also require alteration of the beam formwork. In addition, a specialty contractor is often needed to perform the post-tensioning and grouting.

Nonetheless, this continuity method provides higher resistance to stresses and allows longer spans for a given beam size than the conventional deck reinforcement continuity Method 1. Cracking in the deck over the piers is virtually eliminated when multi-stage post-tensioning is used. In addition, beams designed with this method of continuity require fewer pretensioning strands for resisting positive moments. Pretensioning is required only to support the self-weight of the beam. This reduced pretensioning results in less camber and less demand for high early age concrete strength.[10]

Method 3: Top End Strand Extensions Coupling — This method utilizes pretensioned precast members with strand profiles optimized to "balance" external loads. Top strands at adjacent, interior member ends are coupled to provide continuity prior to deck placement. Also, precompression of interior cast-in-place concrete joints occurs. This method has the structural performance offered by full-length post-tensioning, but does not require the expensive post-tensioning anchorage hardware at tendon ends and the post-tensioning duct.

The top strands used as such can either be straight, fully prestressed partially debonded at positive moment regions and cut after release through pockets in the top flange, or a harped continuation of some of the positive moment strands. A combination of these methods was used in the construction of the pedestrian/bicycle overpass in Lincoln, Nebraska.[11] However, this method requires the use of a special coupler, strict alignment of the beams, and field jacking operations.

Newly Developed High Strength Threaded Rod Continuity Method

In this method, two types of non-prestressed high strength threaded rods were considered: Grades 92 and 150 ksi (634 and 1034 MPa) rods. The nominal diameter of these rods is 1 in. (25.4 mm). They are placed in the top flange of the beam to provide resistance to negative moments at the piers. This continuity reinforcement is designed for negative moment due to the weight of the deck slab and construction loads. Conventional longitudinal reinforcement is placed in the deck in the negative moment regions to provide additional negative moment capacity for resistance to superimposed dead and live loads.

The following similar construction steps as Method 1 are recommended:

1. Fabricate the precast concrete beams with high strength threaded rods placed in the top flange of the beam as required by design. The connection can be done in the field by two steel bars. The gap needed for the connection is about 10 to 12 in. (254 to 305 mm). Therefore, if the width of the diaphragm is within this range, no block-out of the beam is needed.

2. Erect and align the beams.

3. Connect the threaded rods of the two adjacent beams in the field.

4. Form and place the concrete diaphragms to the underside of the beam's top flange.

5. Place the deck reinforcement.

6. Place the deck concrete.

Although this method is presented here mainly for use with I-beams, it is equally applicable to other types of precast beams. This continuity method is relatively easy to construct, as was experienced in the full-scale experiment described below.

Comparison of Continuity Methods

For comparison, a two-span NU1100 I-beam bridge example is used with the following assumptions. The overall width of the bridge is 50 ft (15.2 m). The number of beam lines is five. Beam spacing is 10 ft (3.1 m). Deck thickness is $7^1/_2$ in. (191.0 mm). Compressive strength of beam concrete at release f'_{ci} is 6000 psi (41.4 MPa) and at service f'_c is 8000 psi (55 MPa). Compressive strength of deck concrete at service f'_c is 5000 psi (34.5 MPa). Prestressing strands are $^1/_2$ in. (12.7 mm) diameter with f_{pu} = 270 ksi (1860 MPa). Post-tensioning strands are 0.6 in. (15.2 mm) diameter with f_{pu} = 270 ksi (1860 MPa). Mild steel reinforcement is Grade 60.0 ksi (414 MPa) deformed bars.

Two types of high strength threaded rods are used. The first type is 1 in. (25.4 mm) diameter ASTM A449 threaded rod with a yield strength f_y of 92 ksi (634 MPa) and an ultimate strength of 132 ksi (910 MPa). The second type is 1 in. (25.4 mm) diameter high strength threaded rod with an ultimate strength f_{pu} of 150 ksi (1034 MPa). The weight of the wearing surface and parapet is 370 lbs per ft per beam (5.4 N/mm/beam). Traffic load is HS25.

To obtain a realistic cost comparison of the four methods described above, a 95.0 ft (28.96 m) beam span is used. This span is close to the maximum for Method 1. To show the cost difference between the above mentioned continuity methods, the average cost of this type of bridge is taken as approximately \$50 per sq ft of bridge deck. The comparison below is presented as a cost difference per square foot of bridge deck.

The prices used have been obtained from precast concrete producers, bridge contractors, and material suppliers. These prices may vary from one market to another, but they should be valid for comparison purposes. Only the items affected by the different methods of continuity are included in the comparison.

Based on a study conducted at the University of Nebraska,[12] if Method 1 is considered as the baseline for comparison, then the total incremental cost for Method 2, Method 3, and the

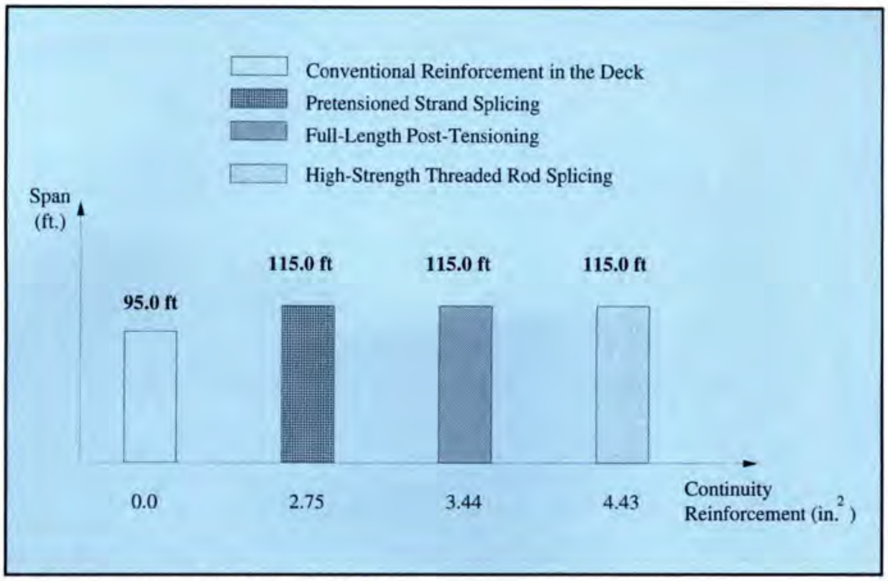


Fig. 8. Span capacity of NU 1100 I-beam.

Table 1. Design parameters of two-span NU 1100 I-beams.

Beam spacing ft (m)	Span length ft (m)	Case	Required reinforcing bar (Grade 60) in deck sq in. (mm²)	Required reinforcing bar (Grade 150) in top flange sq in. (mm²)
8.0 (2.4)	110.0 (33.5)	1	19.0 (12258.0)	0.0 (0.0)
		2	5.6 (3600.0)	9.9 (6387.1)
		3	20.0 (12903.0)	3.1 (2025.8)
12.0 (3.7)	90.0 (27.4)	1	20.4 (13161.0)	0.0 (0.0)
		2	8.1 (5200.0)	9.5 (6129.0)
		3	21.6 (13935.5)	3.1 (2025.8)

threaded rod continuity method is +$0.90 per sq ft, +$0.10 per sq ft and -$0.15 per sq ft, respectively.

Methods 1 and 2 for creating continuity in precast, prestressed concrete beam bridges are presently being used throughout the United States.

The top strand coupling method offers the same advantages of full-length post-tensioning at a lower cost. The most economically attractive method in terms of economy is the threaded rod coupling method. It corresponds to improved behavior at a net savings. The reason for the cost reduction is that the savings in positive moment reinforcement exceed the cost of the threaded rod and couplers.

Methods 2 and 3 offer the advantages of precompression of negative moment zones. All continuity for deck weight methods offer longer span capacity for a given beam size than the conventional continuity for live load method, as shown in Fig. 8. From this figure, it is apparent that the span capacity increases about 20 percent with the added small amount of continuity reinforcement in the top flange of the beam.

FULL-SCALE EXPERIMENT

When the beams are made continuous before casting the deck, a very significant negative moment is developed over the pier as discussed earlier. The bulky NU I-beam's bottom flange offers the advantage of higher compression capacity to resist the negative moment than conventional AASHTO I-beams.

The objectives of the full-scale experiments were as follows:

1. The constructibility of the newly developed high strength threaded rod continuity method needed to be tested.

2. A combination of large moments and large shears exists in the negative moment region. The full-scale testing of the negative moment region in this paper can serve as a starting point for further research of shear capacity in this zone.

Specimen Design

A bridge design utilizing NU1100 I-beams is presented for the purpose of developing representative beam details for the specimens.

The representative bridges are two-span bridges with NU 1100 I-beams of equal length. The compressive strength of the concrete at service f_c' is 8000 psi (55.2 MPa) for the beam and 5000 psi (34.5 MPa) for the deck. Low relaxation strands with a $^1/_2$ in. (12.7 mm) diameter are used.

Details of the design parameters are listed in Table 1. From this table, it can be observed that three cases are considered for each span and beam spacing. In Case 1, the designed threaded rod area for deck weight continuity is zero, which corresponds to Method 1 discussed in the previous section. In Case 2, a minimum amount of reinforcing steel is placed in the deck slab and the rest of the required reinforcement is provided in the top flange of the beam. Case 3 represents a situation in which the required reinforcement for resistance of the deck weight negative moment is provided in the top flange of the beam and the remainder is put in the deck slab.

Based on the design parameters listed in Table 1, three NU1100 I-beams were fabricated for this study. The first specimen, CA, corresponding to Case 3 of Table 1, has six threaded rods with a yield strength of 92.0 ksi (634 MPa) in the top flange, as shown in Figs. 9 and 10. The second specimen, CB, has four threaded rods with an ultimate strength of 150.0 ksi (1034 MPa) in the top flange also representing Case 3 of Table 1, as shown in Figs. 11 and 12. The third specimen, CC, representing Case 1 of Table 1, has no continuity reinforcement in the top flange, as shown in Fig. 13.

The specimens were designed for an 8000 psi (55.2 MPa) concrete strength. The presence of any strands in the bottom flange of the specimens was judged to have no significant impact on the behavior under the effect of negative moment. They were intentionally omitted in the three specimens to simplify specimen production. To save deck forming costs, the $7^1/_2$ in. (190 mm) deck slab width was kept the same as the beam top flange width.

For all three specimens, the same deck slab dimensions and reinforcement were chosen. The specified concrete compressive strength for the deck slab was 5000 psi (34.5 MPa). All the required continuity reinforcement for superimposed dead load and live load was provided in the deck using 22 #9 bars within the width of the top flange of the beam, as shown in Fig. 14.

Considering the high moment and high shear in the negative moment region and the extremely good performance of the orthogonal welded wire reinforcement (WWR),[6] the orthogonal WWR was used as the shear reinforcement, as shown in Fig. 15. The shear reinforcement for the specimens had a specified yield strength of 80 ksi (551.6 MPa). The reader should be aware that in this series of specimens, the horizontal shear reinforcement was separated from the vertical shear reinforcement. Horizontal shear connectors were provided, as shown in Fig. 16.

Connection Design

As shown in Figs. 10 and 12, two rectangular steel bars serve to transfer the tensile forces between rods of adjacent girders. The design of the rectangular steel bars can be done based on the assumption of the loading distribution given in Fig. 17.

The maximum tensile capacity of the Grade 150 ksi (1034 MPa) threaded rod, T_u, is calculated as:

$$T_u = A_s f_{pu}$$

where

A_s = effective cross-sectional area of threaded rod

f_{pu} = ultimate stress of threaded rod

The shear force and bending moment diagrams in the rectangular clamping steel bars are also shown in Fig. 17. The stress intensity, p, is calculated assuming that the maximum tensile capacity in the rod is evenly distributed over a width equal to the width of the nut plus the thickness of washer. The shear force and bending moment diagrams that result from assuming the steel bar to be unrestrained by the surrounding diaphragm concrete are very conservative.

Based on the shear force and moment diagrams in Fig. 17, two rectangular Grade 36 ksi (248 MPa) steel bars with width, b, of 3 in. (76 mm) and thickness, t, of 2 in. (51 mm) were chosen, as shown in Figs. 10 and 12.

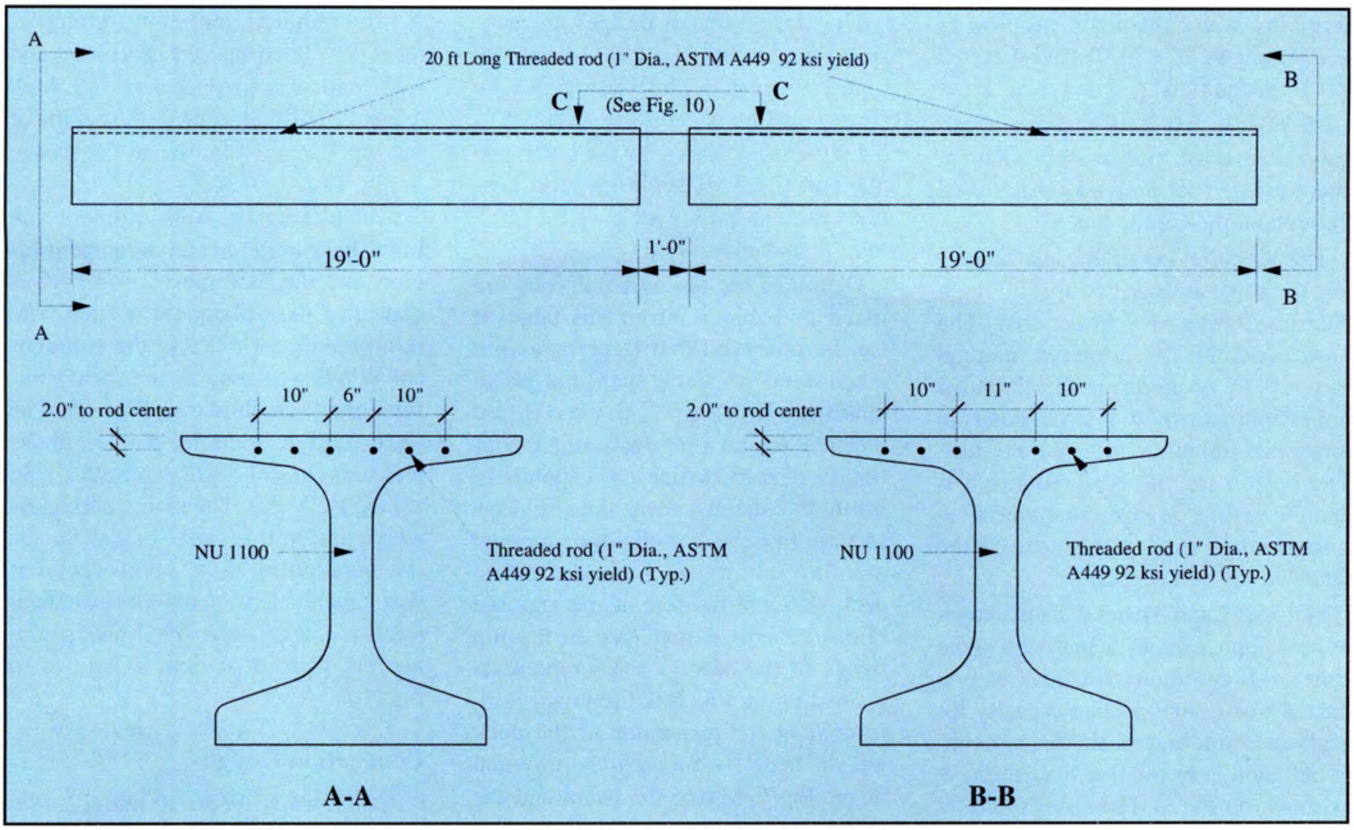
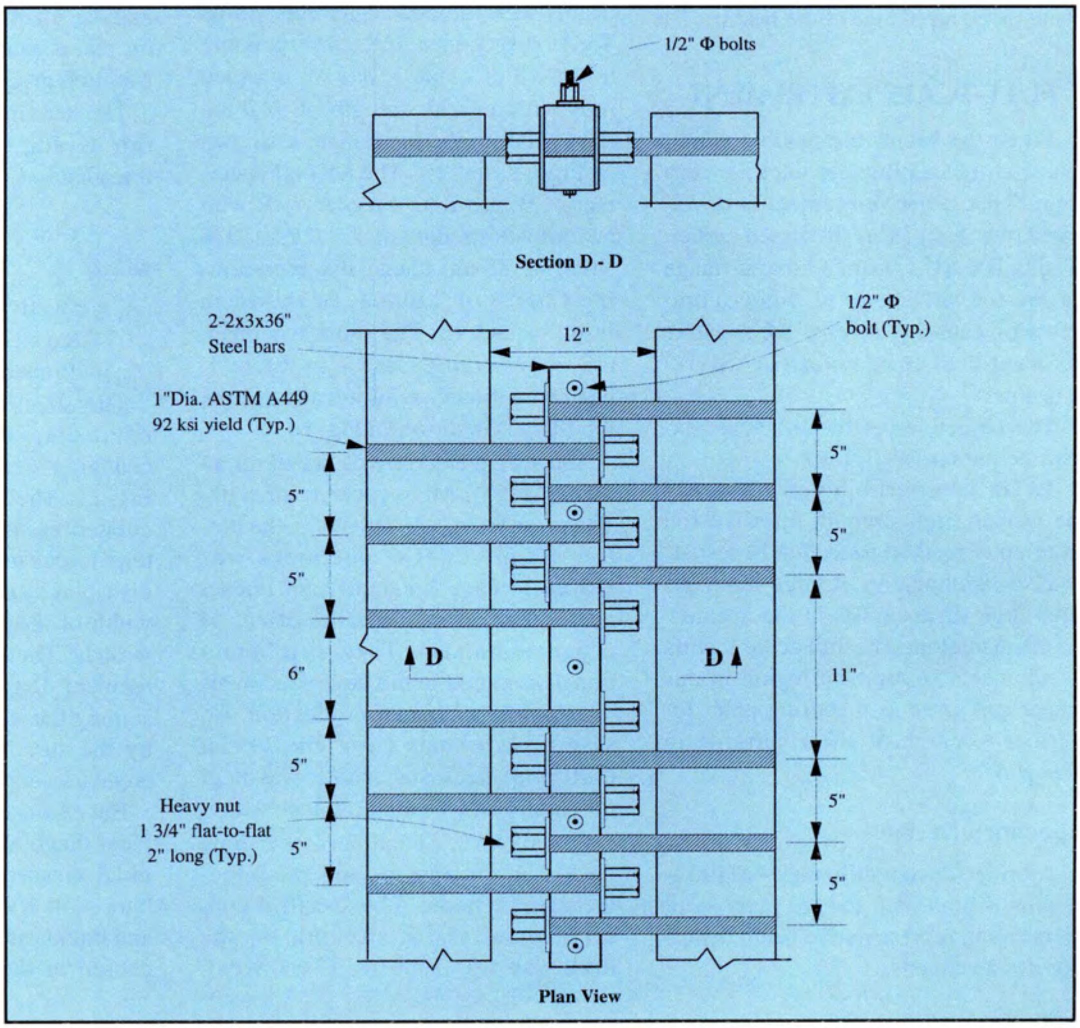


Fig. 9. Continuity for deck weight by Grade 92 ksi (634 MPa) threaded rods (Specimen CA).

Fig. 10.
Top view of Section C-C
(see Fig. 9).



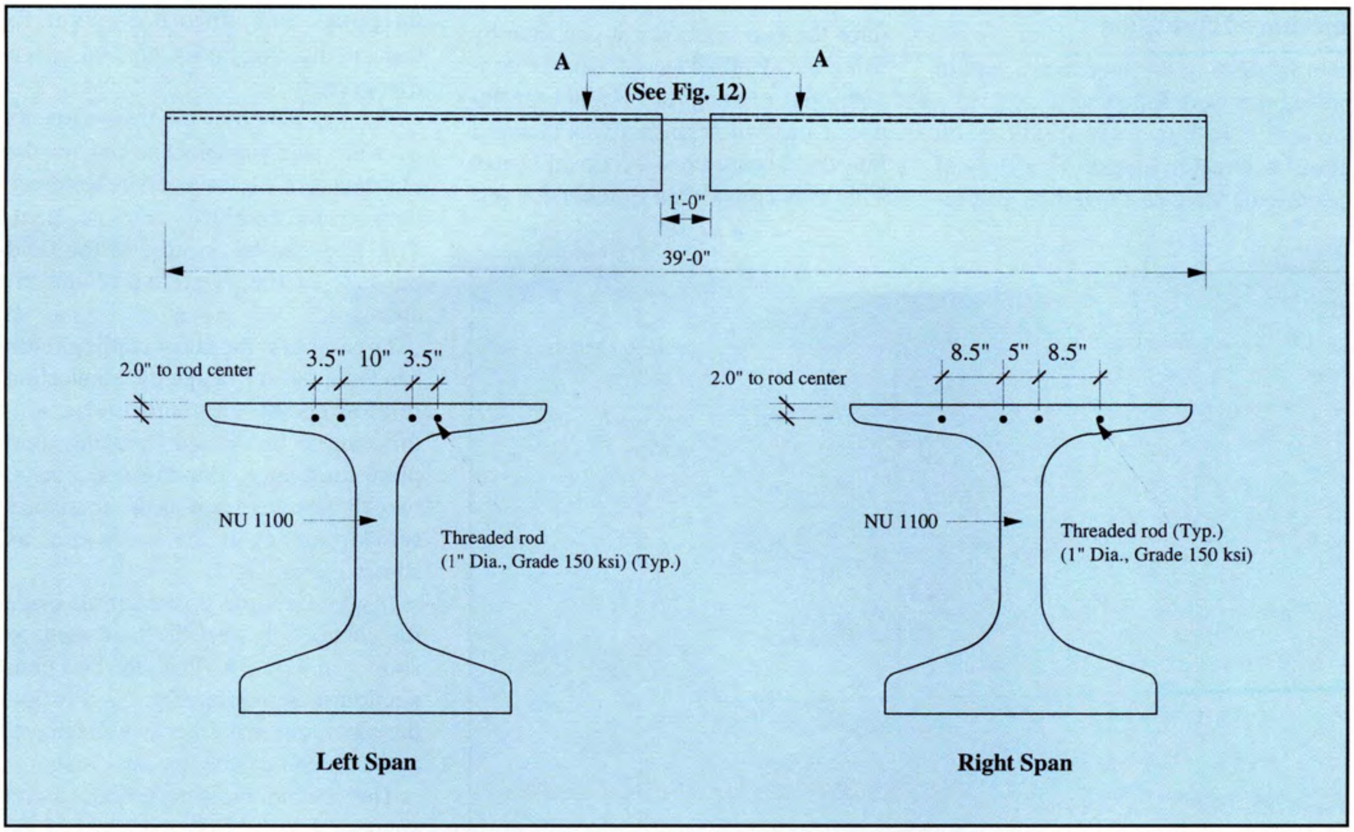


Fig. 11. Continuity for deck weight by Grade 150 ksi (1034 MPa) threaded rods (Specimen CB).

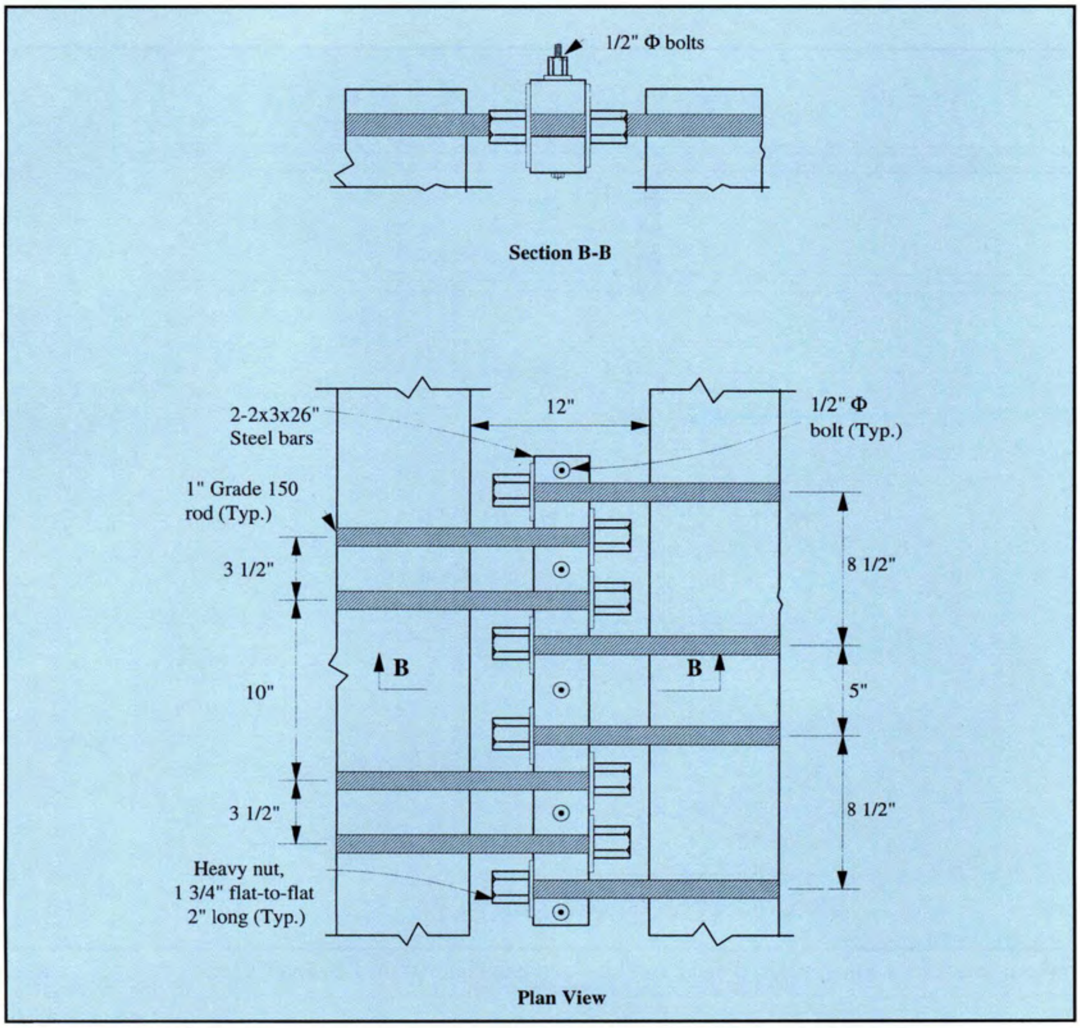


Fig. 12.
Top view of Section A-A (see Fig. 11).

Specimen Fabrication

In fabricating the specimens, certain procedures were followed.

First, two beam segments, each about a quarter length of a typical beam span, were cast together. To produce the two segments at one time by using the standard steel form, the conventional rectangular diaphragm between the two segments was changed into the I-beam cross-sectional shape. This was considered quicker for test purposes and more conservative than using the bulky cast-in-place diaphragms.

The top flange of the two segments over the pier was blocked out for the placement of the connection hardware between the continuity reinforcement. This step can be omitted in the field because of the existence of the diaphragm.

To simplify the rod coupling, care has to be taken to align the connection reinforcement. In plant production, this can be facilitated by using steel plate templates. For the specimens, two #8 bars were tied to the continuity reinforcement at the block-out, as shown in Fig. 18.

It was a simple procedure to place the connection steel bar hardware, as shown in Fig. 19. The finished connection is shown in Fig. 20. Finally, the block-out was filled with concrete at the time of casting the deck slab.

The specimens were produced and connected at the Bellevue plant of the Wilson Concrete Company at Omaha, Nebraska. Cylinders were prepared in

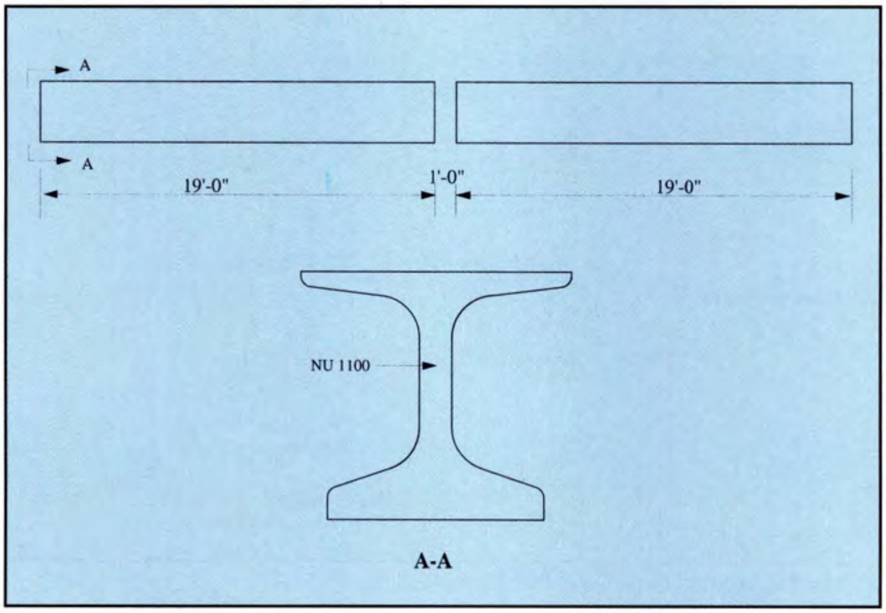


Fig. 13. No continuity reinforcement in beam for Specimen CC.

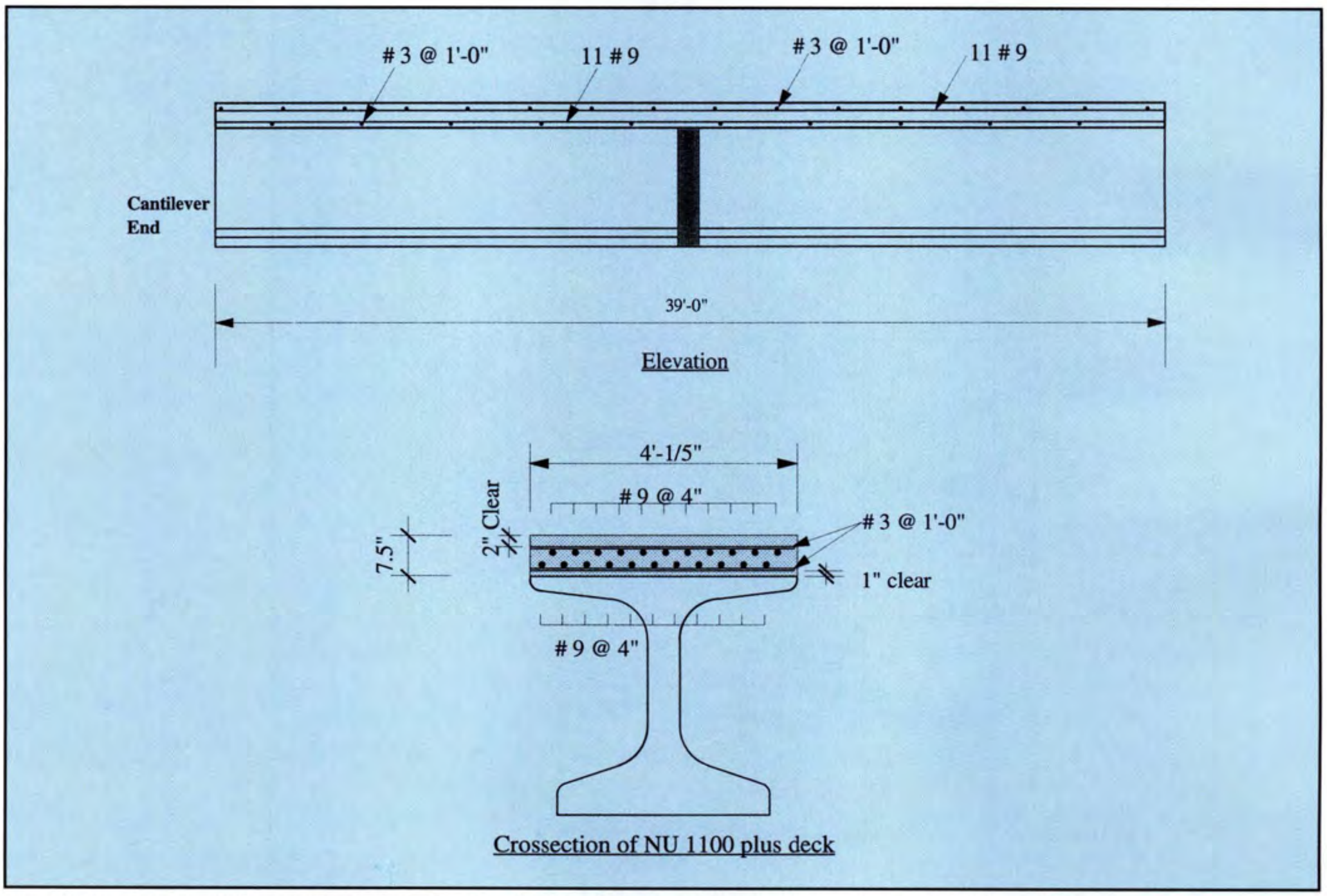


Fig. 14. Continuity reinforcement in deck for superimposed dead and live load (Specimens CA, CB, and CC).

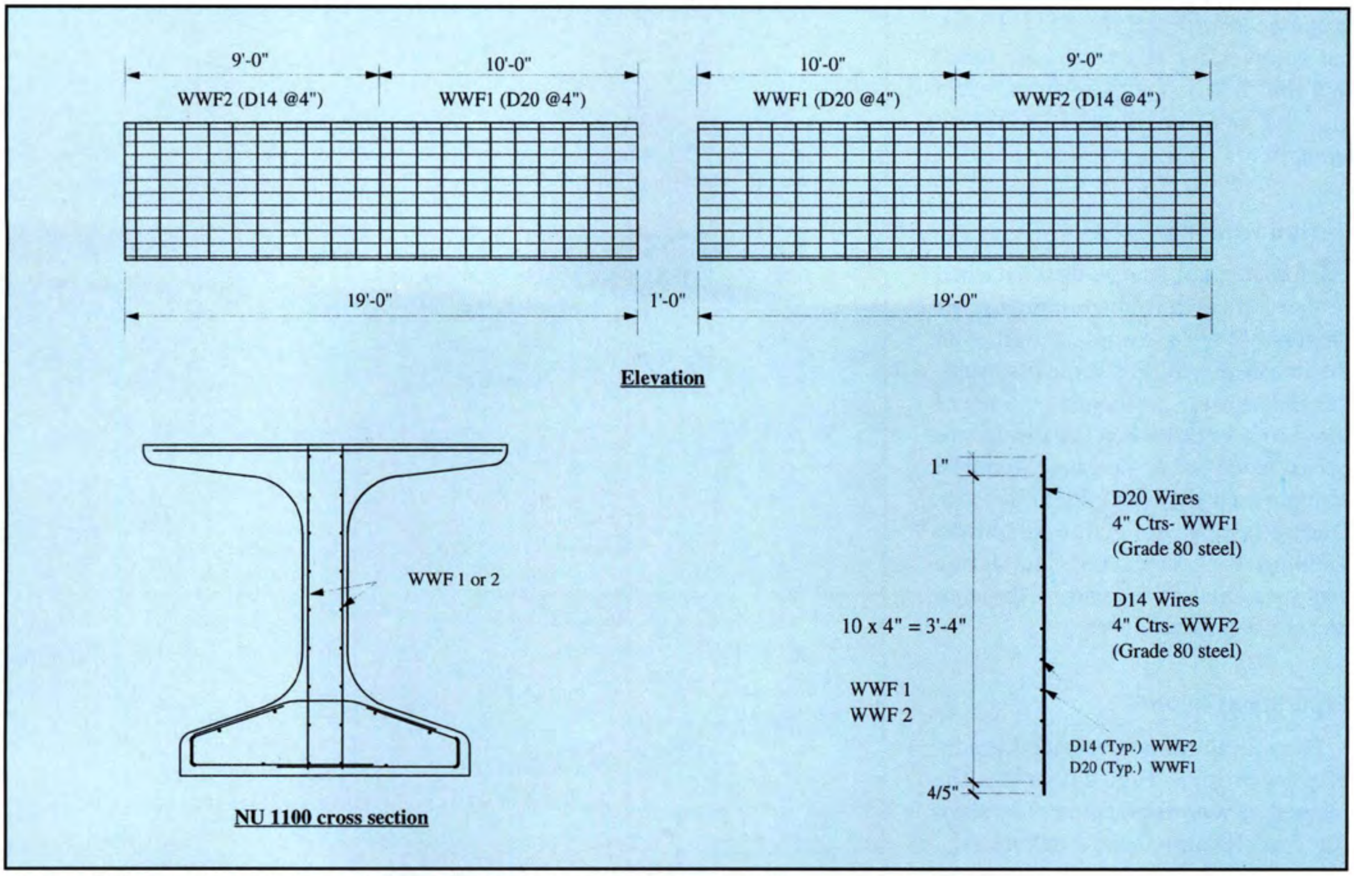


Fig. 15. Orthogonal WWR shear reinforcement (Specimens CA, CB, and CC).

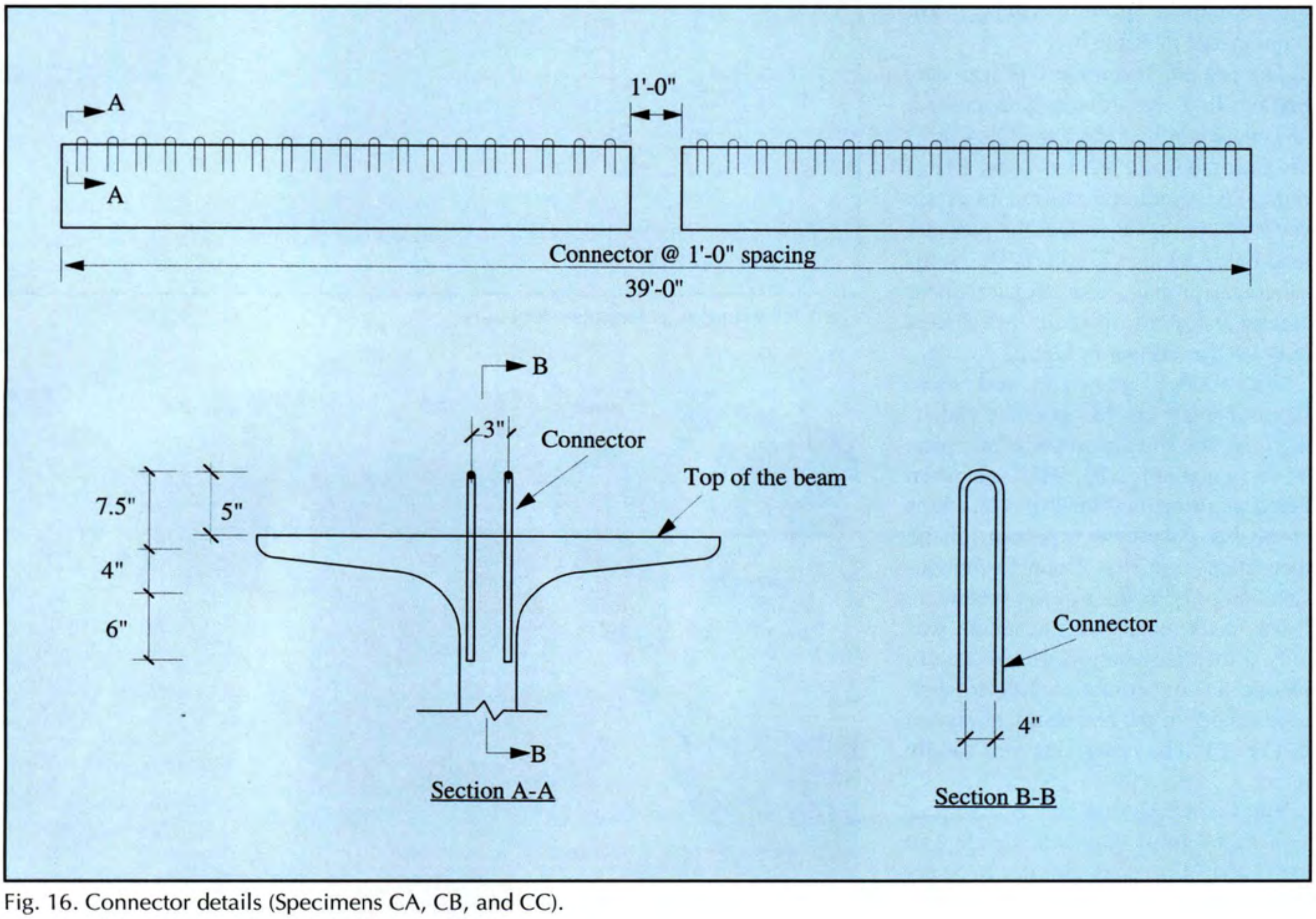


Fig. 16. Connector details (Specimens CA, CB, and CC).

accordance with ASTM C 31. Cylinder compressive test results are listed in Table 2. Also included in the table are the results of deck concrete strength.

Instrumentation

Internal strain gauges were attached to the vertical and horizontal (for orthogonal WWR) stirrups as well as to the threaded rods. For vertical stirrups, the gauges were positioned on a leg of the stirrup so that when the stirrup was placed in the beam, the gauge would be at approximately mid-depth of the web. During testing, deflection and strain readings were measured. Deflection was measured at the end of the cantilever using a transducer.

Experiment Results

The specimens were loaded gradually to failure. Fig. 21 shows the method of load application. The shear force at flexural-shear cracking ($V_{cr\text{-}test}$), at ultimate strength ($V_{u\text{-}test}$), at required ultimate negative moment ($M_{required}$), and at the maximum applied negative moment ($M_{u\text{-}test}$) are summarized in Table 3.

The test of Specimen CA was carried out first. As previously discussed, this specimen had six 1 in. (25.4 mm) diameter, Grade 92 ksi (634 MPa) rods. The specimen started to crack vertically in the deck over the pier at a load of 62.5 kips (278 kN). The vertical cracks propagated as flexural-shear cracks at a load of about 142.5 kips (634 kN), as shown in Fig. 22.

With further increase in load, more flexural-shear cracks appeared radiating from the support of the center pier, as shown in Fig. 23. The specimen failed at an applied load of 328.8 kips (1463 kN), about 99.6 percent of the theoretical capacity. Upon further inspection of both sides of the specimen, it was determined that the failure was only a localized bottom flange failure on one side only due to accidental load eccentricity in the test setup, as shown in Fig. 24. The other side was totally intact.

The second specimen, CB, had four 1 in. (25.4 mm) diameter, Grade 150 ksi (1034 MPa) rods. Similar to Specimen CA, the flexural-shear cracks

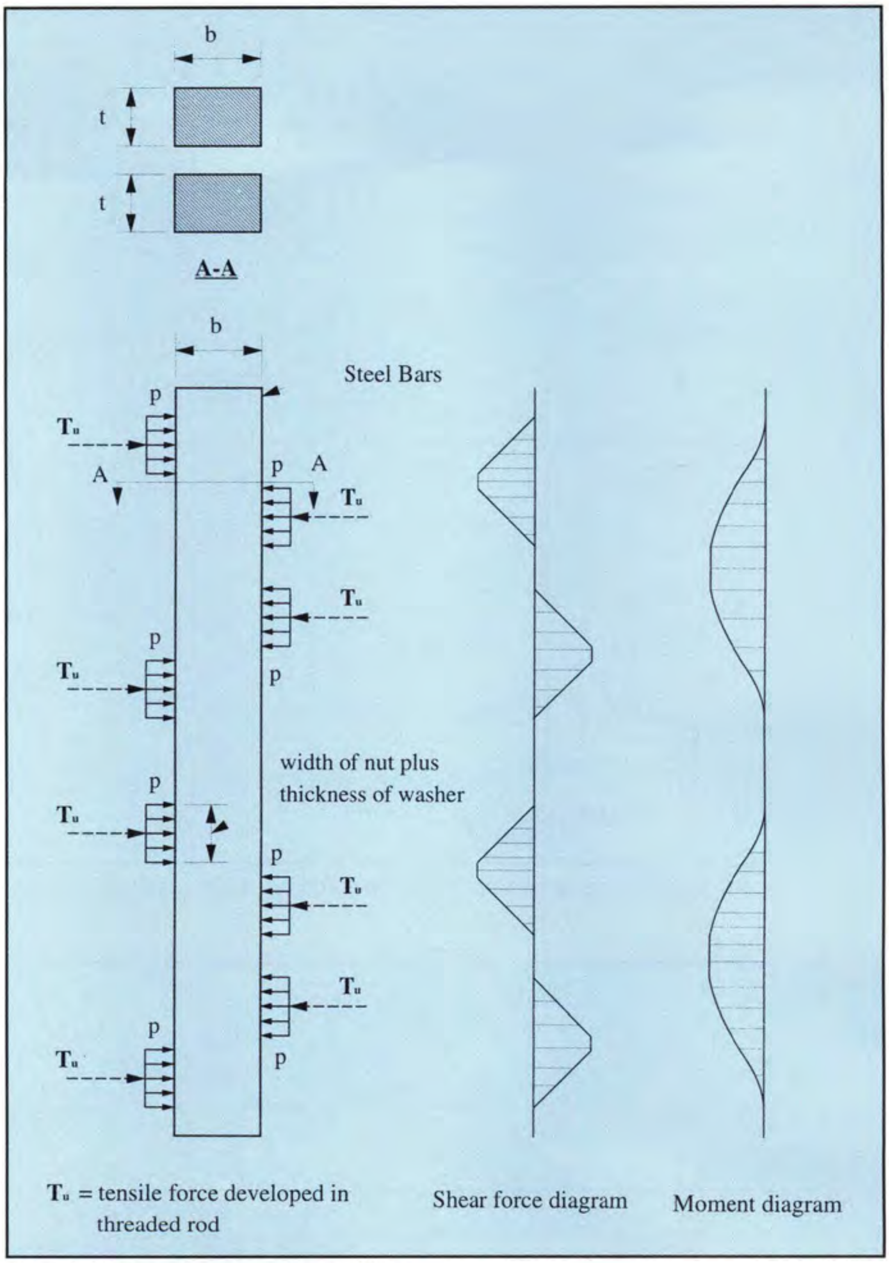


Fig. 17. Design of rectangular steel bars.

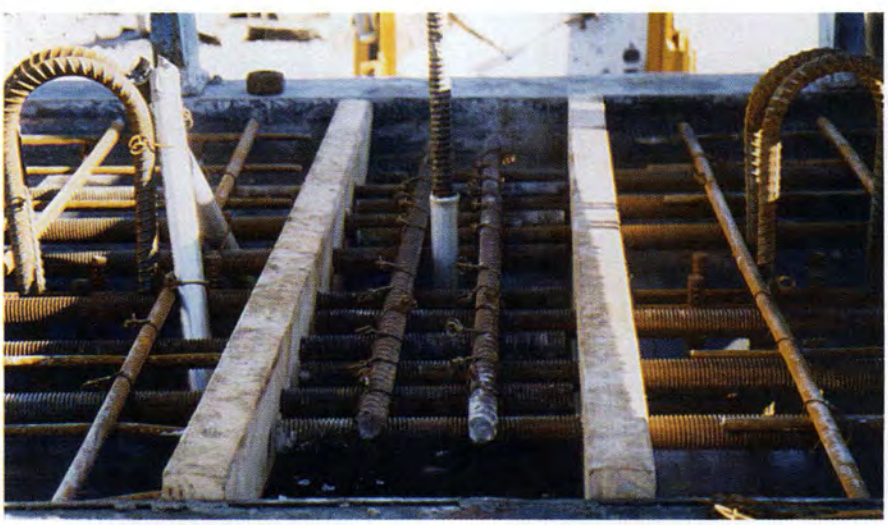


Fig. 18. Method for aligning continuity reinforcement.

Fig. 19. Placement of rectangular steel bar.

Fig. 20. Finished connection.

Table 2. Measured concrete strength.

Specimen	Strength at time of testing (psi)		Deck strength at time of testing (psi)	
	Specified	Actual	Specified	Actual
CA	8000	9120	5000	7880
CB	8000	9120	5000	7880
CC	8000	9120	5000	7880

Note: 1 psi = 0.006895 MPa.

Fig. 21. Load application.

were observed at a load of about 142.5 kips (634 kN). The maximum load of 505.5 kips (2250 kN) was applied and no failure was found in the specimen. A very large deflection at the loading points was observed, as shown in Fig. 25a.

Also, at the maximum applied loading stage, many fine closely-spaced flexural-shear cracks were found, as shown in Fig. 25b. At this point, the test was stopped because of concern regarding failure of the supports. The crack distribution was excellent even at the maximum applied load. It was clear that the specimen showed a very ductile behavior.

Specimen CC had no negative moment reinforcement in the top flange of the beam, only reinforcing bars in the deck. The flexural-shear crack appeared at about 138.5 kips (616 kN) and fine flexural-shear cracks were found at the maximum loading stage. However, delamination of the deck slab from the beam was found over the entire length of the specimen at a load of about 400 kips (1780 kN), as shown in Fig. 26.

The following conclusions can be made from this series of experiments:

1. The new continuity connection details exhibited excellent performance.

2. All three continuity specimens displayed adequate strength and ductility behavior for the intended design forces. However, Specimen CC experienced delamination of the entire deck while the other two specimens did not.

3. The cross-sectional area of the bottom flange is the most important factor in determining the maximum achievable span length of a given I-beam. In the negative moment area, over the pier, the limiting factor is the maximum reinforcement limit. The bulky bottom flange of the NU I-beam has been found to have 1.51 times the required ultimate negative moment capacity, as shown in Table 3.

4. For I-beams with continuity reinforcement in the top flange, there exists a relatively high horizontal shear (composite action) resistance.

5. Orthogonal WWR can be used in the negative moment region to resist high shear combined with high

Table 3. Summary of test results.

Specimen	r_v	r_h (percent)	f_y (ksi)	Continuity in beam	Reinforcing bar in deck	$V_{cr\text{-}test}$ (kips)	$V_{u\text{-}test}$ (kips)	$M_{required}$ (kip-ft)	$M_{u\text{-}test}$ (kip-ft)	$\frac{M_{u\text{-}test}}{M_{required}}$
CA	1.695	1.695	80	6 – ϕ 1 in. Grade 92 ksi rods	22 #9 bars	142.5	351.3	5510.0	5486.0	0.996
CB	1.695	1.695	80	4 – ϕ 1 in. Grade 150 ksi rods	22 #9 bars	142.5	528.1	5510.0	8313.8	1.510
CC	1.695	1.695	80	No reinforcing bar	22 #9 bars	138.5	422.5	4030.0	6625.2	1.644

Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa; 1 kip = 4.45 kN; 1 kip-ft = 1.356 kN-m.
r_v = vertical shear reinforcement percentage = $A_v/b_w s_v$ percent
r_h = horizontal shear percentage = $A_h/b_w s_h$ percent

moment. Tests have shown that the shear cracks are very fine and well distributed at ultimate loading.

Based on the experimental results, either the Grade 150 ksi (1034 MPa) or Grade 92 ksi (634 MPa) threaded rods would give superior structural performance to the Method 1 procedure. Table 4 shows the cost comparison between the Grade 150 and 92 ksi (1034 and 634 MPa) threaded rod details for a typical 110 ft + 110 ft (33.5 m + 33.5 m) two-span bridge, as shown in Table 1. The width of the bridge is assumed to be 48 ft (14.6 m).

The total area of the bridge deck is 10,560 sq ft (978 m^2). Based on the cost listed in Table 4, the unit prices for using the Grade 92 ksi (634 MPa) rod option and the Grade 150 ksi (1034 MPa) rod option are \$0.40 per sq ft (\$4.31/m^2) and \$0.38 per sq ft (\$4.05/m^2), respectively. Therefore, the cost for the two details is about the same. As indicated earlier, this cost is more than offset by the reduction in required positive moment rein-

Fig. 22. First stage of cracks of Specimen CA.

Fig. 23. Fan shape of flexural-shear cracking.

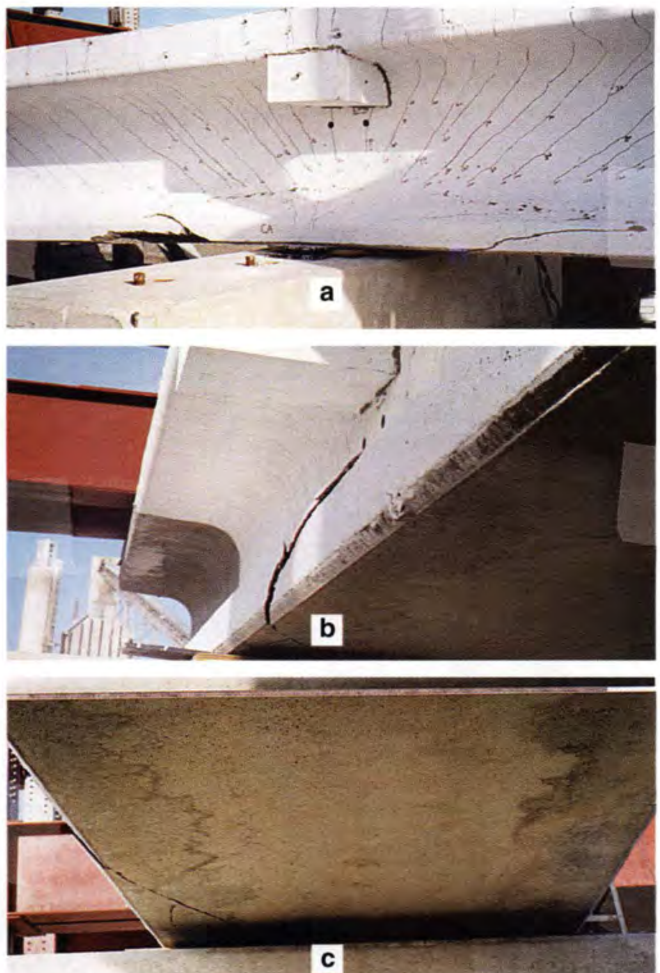


Fig. 24. Unsymmetrical failure of Specimen CA.

Fig. 25. Maximum loading stage of Specimen CB. (a) Large deflection (b) Closely spaced cracks

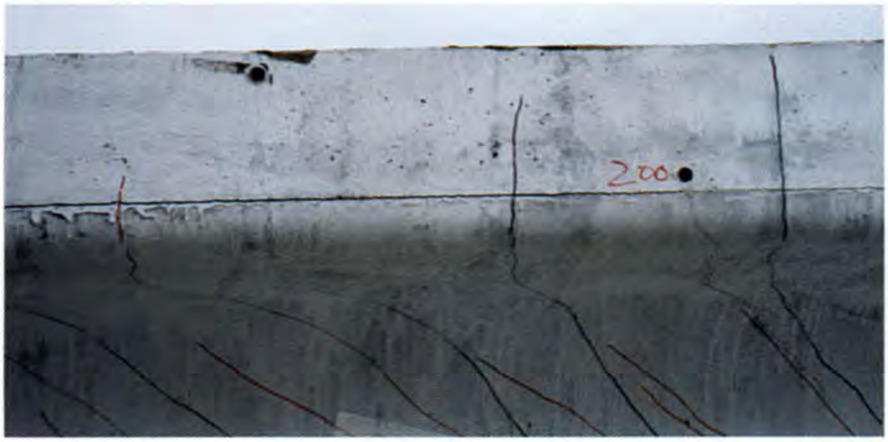


Fig. 26. Delamination of deck slab of Specimen CC.

Table 4. Cost comparison of connection details.

	Grade 92 ksi rods details			Grade 150 ksi rods details		
Items	**Quantity**	**Unit cost**	**Price**	**Quantity**	**Unit cost**	**Price**
Rods	1980 ft	\$1.75 per ft	\$3465	1320 ft	\$2.30 per ft	\$3036
Nuts	72	\$0.55 each	\$40	48	\$7.72 each	\$370
Washer	72	\$1.85 each	\$133	48	\$1.85 each	\$89
Plates	12	\$48 each	\$576	12	\$39 each	\$468
Total	—	—	\$4214	—	—	\$3963

Note: 1 ft = 0.3048 m; 1 ksi = 6.895 MPa.

forcement. Both details offer promising prospects not only for I-beams made continuous over piers but also for the inverted tee and other bridge systems. The Grade 150 ksi (1034 MPa) rod option appears to be slightly more favorable in terms of economy and reinforcement congestion.

CONCLUSIONS

Based on the analysis and experimental results, the following conclusions may be drawn:

1. According to the rigorous time-dependent analysis, different continuity methods and/or construction sequences will affect the behavior of the bridge system differently.

2. For the conventional continuity system for live load only, the time-dependent restraint positive moment, which will develop after a rigid diaphragm is cast, may cause section cracking. The time period after a diaphragm casting should be limited to 230 days for the NU I-beam series. It is not recommended that the diaphragm be cast earlier than 14 days of precast beam age, and preferably at least 28 days.

3. For the conventional continuity system for live load only, when the diaphragm and deck are cast simultaneously, the bridge is expected to have acceptable structural performance. However, the beams would have to be constructed as designed, namely, simple spans whose ends are free to rotate at the time of deck placement. To accomplish this construction sequence, the diaphragm can be cast with unbonded joints and minimal beam embedment into the diaphragm. The unbonded joints will allow beams to rotate freely while the deck concrete is being placed and cause no restraints at the ends of the beams. When the deck is placed, a rigid joint is formed over the piers and continuity of the bridges is effected.

4. For the conventional continuity system for live load only, if a rigid diaphragm is cast ahead of the deck, continuity of the beams for negative moment due to deck weight must be assured over the piers. The concrete beam/diaphragm joint, without negative moment resisting reinforcement, may crack and spall due to the deck weight.

5. Three methods of making continuity for deck weight are presented. The high strength threaded rod continuity method is recommended. Both the Grade 92 and 150 ksi (634 and 1034 MPa) rods are recommended for use in this continuity construction based on cost and a full-scale experimental performance comparison.

6. Placing all continuity reinforcement in the deck slab only is not recommended. On the other hand, placing some continuity reinforcement in the top flange of the I-beam can increase not only the composite action between the deck slab and the precast I-beam but also lengthen the span capacity by as much as 20 percent.

6. The continuity method using threaded rods is considered to be the most cost-effective option. The proposed connection details are relatively simple to construct without the need for specialty contractors. They have been proven to give excellent performance and are therefore highly recommended.

7. The cross-sectional area of the bottom flange is very important in determining the maximum achievable span length of a given I-beam. The bulky bottom flange of the NU I-beam has been found to have at least 1.51 times the required ultimate negative moment capacity.

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APPENDIX — NOTATION

$(i - {}^1\!/_2)$ = time at beginning of ith interval
$(i + {}^1\!/_2)$ = time at end of ith interval
A = cross-sectional area of concrete
A_h = area of horizontal shear reinforcement
A_s = effective cross-sectional area of threaded rod
A_v = area of vertical shear reinforcement
b = width of steel bar
b_w = width of web
$E_c(j)$ = modulus of elasticity of concrete at middle of interval j
$E_{ce}(i)$ = effective modulus of elasticity of concrete at middle of interval i
f_c' = compressive strength (in psi) of concrete at service
f_{ci}' = compressive strength of concrete at time of initial prestress
f_{pu} = ultimate stress of threaded rod
f_y = yield strength of nonprestressed mild steel reinforcement in tension
j = time at middle of jth interval
$M_{required}$ = required ultimate negative moment
$M_{u\text{-}test}$ = maximum applied negative moment
N = axial force
p = stress intensity
r_h = horizontal shear reinforcement percentage
r_v = vertical shear reinforcement percentage
s_h = horizontal spacing of web reinforcement
s_v = longitudinal spacing of web reinforcement
t = thickness of steel bar
T_u = maximum tensile capacity of threaded rod
$V_{cr\text{-}test}$ = maximum shear force at diagonal tension (flexural-shear) cracking
$V_{u\text{-}test}$ = maximum shear force at ultimate strength
$\Delta N(j)$ = axial force increment at middle of interval j
$\Delta\varepsilon(i)$ = increment axial strain occurring during interval i
$\Delta\varepsilon'(i)$ = initial strain in ith interval
$\Delta\varepsilon_{sh}(i)$ = free shrinkage strain during interval i
ε = axial strain