The NU Precast/Prestressed Concrete Bridge I-Girder Series

K. Lynn Geren
Graduate Research Assistant
University of Nebraska-Lincoln
Omaha, Nebraska

Maher K. Tadros, Ph.D., P.E.
Cheryl Prewett Professor
of Civil Engineering and
Director, Center for Infrastructure
Research
University of Nebraska-Lincoln
Omaha, Nebraska

Designers have experienced limitations when using existing precast, prestressed concrete I-girders in continuous span bridges. The Nebraska University (NU) girder series was recently developed to overcome these limitations and to take advantage of recent advances in precast concrete production technology. The girder was developed in "hard" metric units for optimal performance in a two-span bridge with full-length continuity post-tensioning. In addition, the girder performs well for pretensioned systems with continuity achieved by mild steel reinforcement in the cast-in-place deck, and in simple span applications. The NU girder has a wide bottom flange to enhance the compressive strength in negative moment regions for continuous span designs, and to allow placement of a large number of strands in the bottom flange for simple span designs. This is particularly helpful when high strength concrete is used. The relatively long span capability and shallow depth of the NU girder makes it an economical alternative in situations previously reserved for structural steel girder systems. This paper presents a short history of precast, prestressed concrete bridge I-girder development, the procedure undertaken to develop the NU girder series, performance comparisons with several existing standard girder shapes, and the steps taken by the Nebraska Department of Roads (NDOR) to implement the research results.

In the United States, precast, prestressed concrete I-girders were first used in bridge applications in the early 1950s. Today, they have become the most widely used girders in the 70 to 120 ft (21 to 36 m) span range.[1] During its early stages of development, a different girder shape was designed and manufactured for each new bridge. However, as precast, prestressed concrete girders were used more frequently in bridges, several states began to adopt their own standard girder section shapes.

In 1956, the Bureau of Public Roads, known today as the Federal Highway Administration (FHWA), adopted its own standards for precast concrete I-girders.[2] These shapes, now known as the AASHTO girders, are illustrated in Fig. 1 with complete dimensions and section properties contained in Appendix A.

Significant savings were realized as a result of standardization. However, standardization has tended to inhibit the advancement of the design of precast, prestressed concrete bridge girders. Since the introduction of the AASHTO V and VI shapes, evolution toward increased structural efficiency has been very slow. This is due, in part, to the fact that precasters, having made an investment in particular forms, are reluctant to purchase additional forms to make newer girder shapes available to local markets.

The financial incentive to optimize the shape is further hampered by the fact that the older, less efficient shapes are still generally less expensive than the competing structural steel shapes. Also, engineers and designers are sometimes reluctant to use new girder shapes when they have had success with older, well established girder shapes. One example that highlights this situation is the Washington bulb tee developed by Arthur R. Anderson in 1959. This shape was not adopted as the AASHTO/PCI bulb tee standard shape until 1988 — nearly 30 years after its introduction.[3,4] This AASHTO/PCI bulb tee is a slightly modified version of the girder shape developed by Anderson (see Fig. 2 and Appendix A).

Some states have continued to rely on the federal standards while other states pioneered their own girder shapes. This uncoordinated evolution has resulted in significant diversity in the girder shapes that are used from state to state throughout the nation. Currently, many states specify the use of AASHTO Types III, IV, V, and VI or similar girders in their standard bridge plans, despite the existence of more efficient girders such as the AASHTO/PCI bulb tee (see Fig. 2 and Appendix A).

Differences in available local materials, as well as in the technology available in different geographic regions, have also brought about diversity from state to state in girder shapes. For example, the thin webs of the Washington and Colorado shapes (see Fig. 3 and Appendix A) can be attributed to the availability of aggregates that allow easy production of higher strength concrete and to the technology developed by the local fabricators that allows excellent consolidation of concrete.

In eastern states such as Pennsylvania, very shallow girders were necessary to compete with structural steel in areas where under-clearance was a critical concern. In addition, the girder was required to withstand large prestressing forces, leading to its relative bulkiness.[5] Pennsylvania's modern standard sections, though deeper, have retained their characteristic bulky flanges.

These and the other factors mentioned above have contributed to the diversity in girder shapes. In Fig. 3,

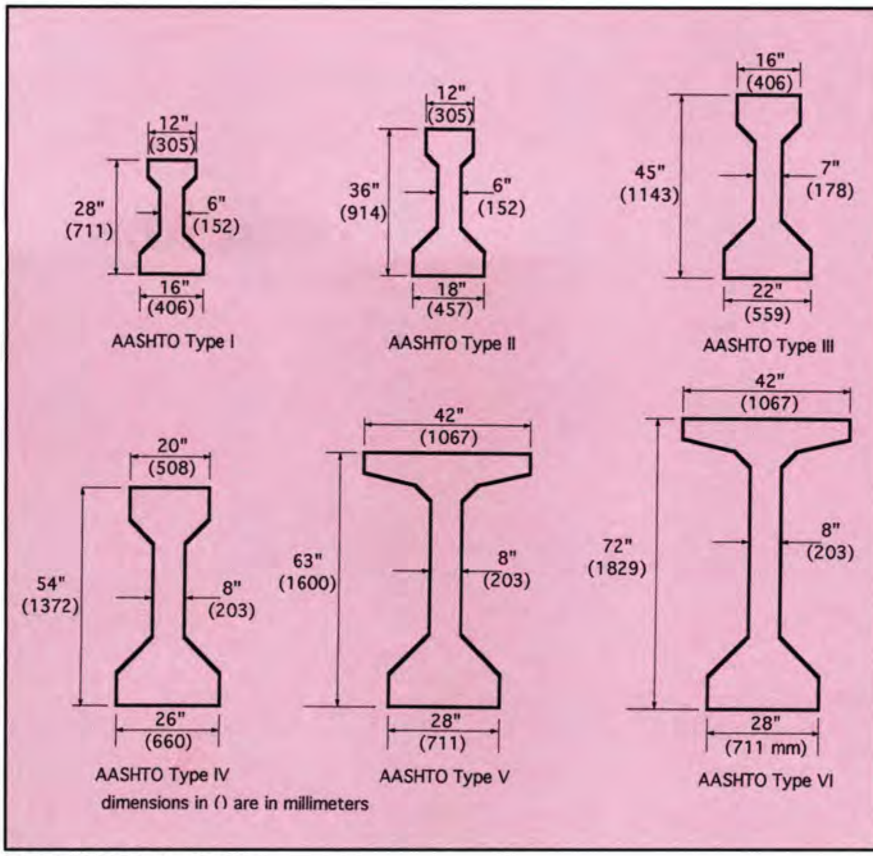

Fig. 1. AASHTO girders.

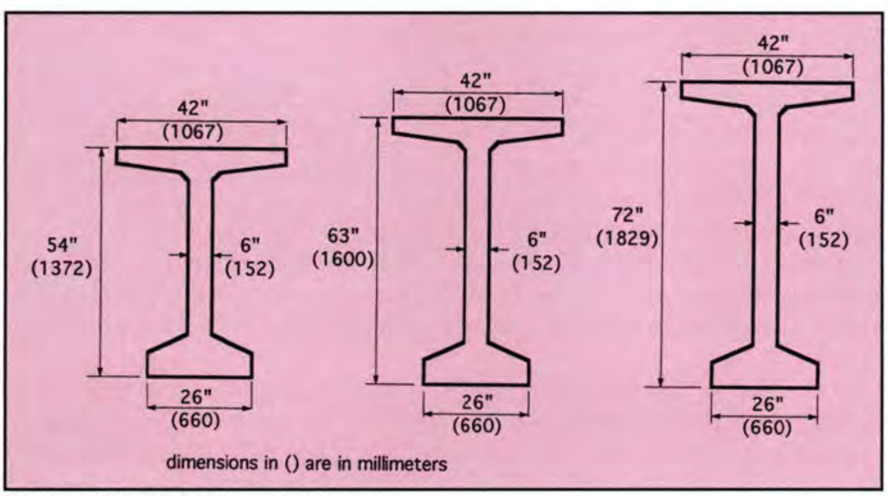

Fig. 2. AASHTO/PCI bulb tees.

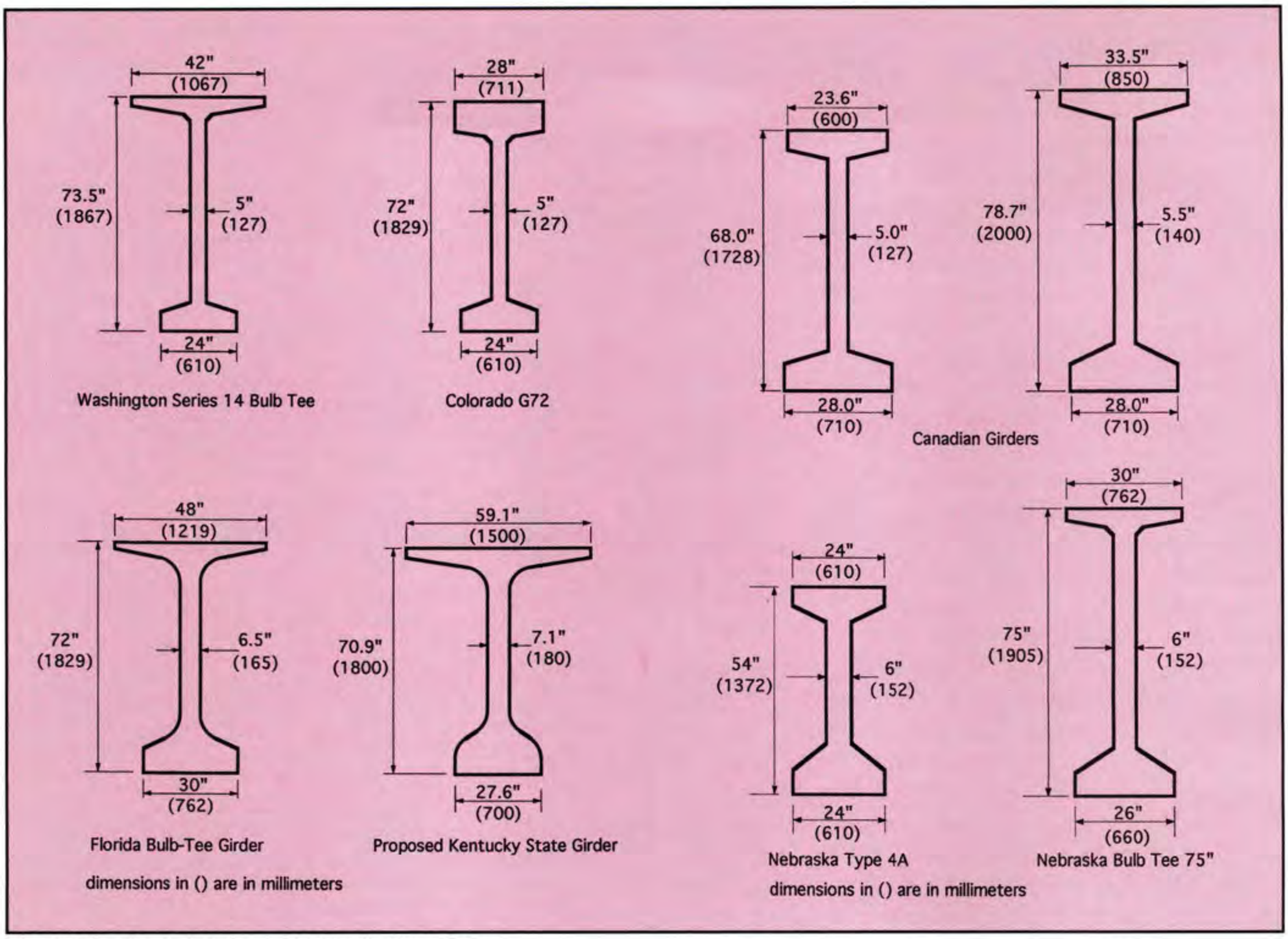

Fig. 3. Standard girders used by individual states.

examples of shapes used by a number of states and Canadian provinces are given. Complete dimensions and section properties of these girders are contained in Appendix A. The Canadian girders are produced by Con-Force Structures Limited for use in British Columbia.

In the near future, all projects involving federal participation will be required to use metric units.[6] Although girders with "soft" metric units will be usable, the girder discussed here was designed in "hard" metric units, where the actual dimensions are in round metric figures since new forms are required anyway.

Response to the survey conducted as part of this project revealed there is now considerable interest, among bridge design professionals and fabricators, in the development of an optimum girder shape for continuous spans. This response leads to optimism regarding expeditious implementation of the NU girder series.

CURRENT AND EMERGING DESIGN PRACTICES

Design for continuity in bridges is increasing for several reasons. The most common reason for introducing continuity is to eliminate expansion joints in the deck. Experience has shown that joints in a bridge deck require costly maintenance. When these joints are eliminated, the bridge deck is made continuous and the girders are connected longitudinally with cast-in-place joints. Thus, they act as continuous beams for loads applied subsequent to the joint gaining adequate strength, such as superimposed dead and live loads.

Another major reason for introducing continuity is that it allows a given girder to span greater lengths than if it were simply supported. Engineers are being challenged to design bridges with longer spans and improved seismic resistance. A good example of the requirement for longer spans is the highway overpass. In the past, these bridges were constructed in four relatively short spans of 60 to 80 ft (18 to 24 m) each. Now, federal highway regulations call for the elimination of the shoulder piers. This change requires that the bridge be constructed of two spans. To continue to gain market share, it is essential that precast, prestressed concrete girders stretch to lengths previously reserved for structural steel.

Although most existing standard girders are being used in continuous bridges, they were initially designed for use in simple span structures. In continuous applications, the girders are subjected to negative bending moments at levels that often control their design, especially when the girders are made continuous before the cast-in-place deck weight is introduced. Insufficient compression area in the bottom of the girder and web widths too small to accommodate continuity post-tensioning are common limitations to increased span length and/or transverse girder spacing. An exception to this is the Florida bulb tee (see Fig. 3). It is the only existing standard girder developed for continuous spans and is believed to be one of the most efficient girders available today.

There are many ways in which continuity has been achieved in precast concrete girder bridges. The most widely used method for achieving continuity is to place conventional mild steel reinforcement in the cast-in-place deck to resist the negative moment over the pier. This is the most convenient splicing method; however, the span is limited to the length of girder that can be transported to the construction site and the continuity is only effective for a small portion of the total dead load plus the live load. State transportation agencies have limitations based on length and/or weight. In most areas of the United States, this limits the maximum span to about 120 ft (37 m). One drawback to this type of construction is that cracks develop near the top surface of the deck in the pier area due to negative continuity moments. Exposure of the cracks to traffic and de-icing chemicals can promote rapid deck deterioration.

The second method that is becoming more widely used is full length post-tensioning (see Fig. 4). Resistance to negative bending moment, as well as to positive bending moment, is provided by post-tensioned tendons that are stressed after placement of the cast-in-place diaphragm and, often, after the deck is placed. This provides greater resistance to loads and allows longer spans than the conventionally reinforced joint. Cracking in the deck over the pier is virtually eliminated.

When full-length post-tensioning is provided, significantly fewer pretensioning strands are required for positive moment resistance. Pretensioning is required only to support the weight of the girder itself and the weight of the cast-in-place deck, if it is placed before post-tensioning. The reduced pretensioning requirement can help reduce excessive camber and the need for high strength of concrete at release.

Several other methods that have been used to make precast concrete girders continuous are described in detail in Refs. 7 and 8.

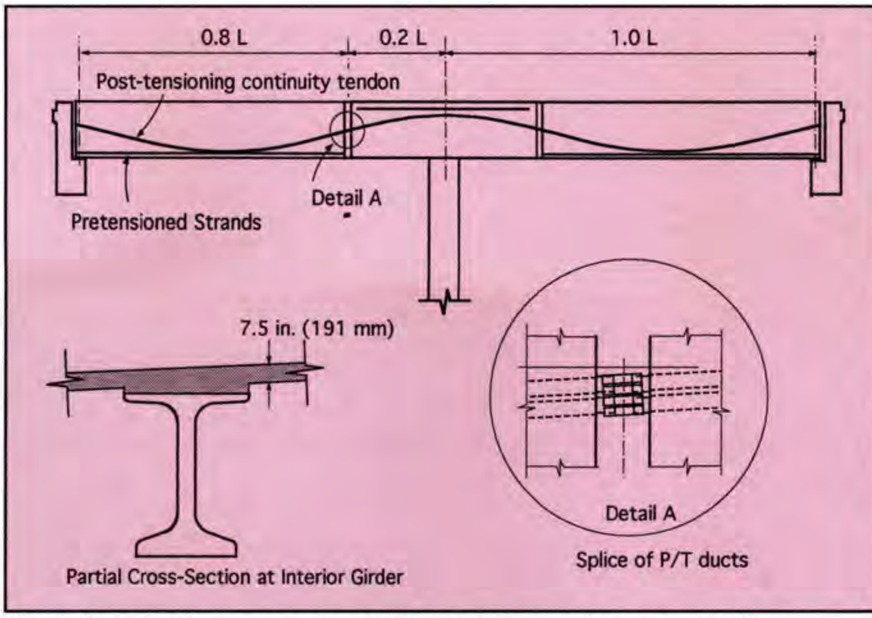

Fig. 4. Span configuration and post-tensioning layout of system used in shape optimization.

RESEARCH OBJECTIVES AND PLAN

The main objective of this project was to develop a metric precast, prestressed concrete girder that is not only structurally efficient, but one that is as easy as possible to manufacture, transport and erect. The girder developed must produce a cost effective bridge system — including girder, slab, substructure and approach.

Previous Studies

A report by Rabbat et al., in 1982,[9] focused on existing standard girder sections and analyzed the structural efficiency and cost effectiveness of these girders when used in simple span structures. The study demonstrated that the AASHTO/PCI bulb tees were the best shapes available for simple span applications and that increased girder spacing provides improved superstructure economy. The Florida bulb tee, developed by the Florida Department of Transportation in the mid-1980s[10] was the result of further development of the AASHTO/PCI bulb tees to account for continuity by post-tensioning. This project expands on the work presented in Refs. 9 and 10.

An approach to mathematical optimization of precast, prestressed concrete bridge girders was developed by Lounis and Cohn.[11] This is a valuable theoretical approach since it rationally produces information on the relative importance of various cross section components for given design and cost criteria. This study was done for simple spans, 10 to 45 m (32.8 to 147.6 ft), with pretensioned concrete girders. It showed that at the given span range the maximum feasible girder spacing is 3.38 m (11.1 ft) with a minimum deck slab thickness of 225 mm (8.9 in.), and that the optimal design does not necessarily require maximum tendon eccentricity. The findings of this study were considered in this project. However, emphasis was placed on aspects relating to continuous span design, multi-stage prestressing (i.e., pretensioning combined with post-tensioning), impact of top flange width on deck slab design, and girder fabrication, transportation and erection.

Parametric Study

A computer spreadsheet was developed to perform a parametric study to determine the most structurally efficient section. Survey responses from nearly 90 state engineers, bridge consultants, precast manufacturers and form makers led the authors to weigh heavily the non-quantifiable issues concerning girder fabrication, trans-

Table 1. Relative costs used for girder comparison.

Item	Cost
Precast girder concrete - including all reinforcing bars, labor, and overhead	\$320 per cu yd
Accessories and hauling - includes PVC, coil ties, coil rod, bearing pads, bearing plates, lifting loops, prestress hold-downs and transportation	\$1075 per girder
Girder prestress - strand, labor and overhead	\$1.14 per lb
Post-tensioning - materials, labor and overhead	\$1.50 per lb
Cast-in-place deck	\$200 per cu yd
Mild steel reinforcing bars (including epoxy coated bars)	\$0.55 per lb

Note: 1 lb = 0.4536 kgf; 1 cu yd = 0.7646 m^3.

portation and erection. For example, limitations were placed on the bottom slope of the top flange to allow ease of form stripping during fabrication, on the edge thickness of the top flange to prevent damage during transportation and handling, and on the maximum width of the bottom flange to accommodate existing precasting beds. Cost considerations were a controlling factor in the identification of the optimum girder shape.

Girder optimization was based on analysis of an interior girder in a continuous bridge of two equal spans. The design adopted the system featured in the appendix to Ref. 7. The system consists of a double cantilever pier segment and two field segments that are initially erected as simply supported. The girders are made continuous by a cast-in-place splice and full length post-tensioning as illustrated in Fig. 4.

For purposes of optimization, the girder shape and span length were made variable with a constant girder spacing of 10 ft (3 m). The length of the field segments was determined such that the midspan of the field segments corresponded with 4/10 of the total span length. In this way, the points of maximum flexure during shipping and erection correspond with the points of maximum flexure in the final structure. This allows the most efficient use of flexural reinforcement.

Precast girder concrete strength was taken as 4000 psi (27.6 MPa) at release and 6000 psi (41.4 MPa) at 28 days. The 7.5 in. (191 mm) cast-in-place deck concrete strength was taken as 3000 psi (20.7 MPa) at 7 days and 3500 psi (24.1 MPa) at 28 days. Both girder and cast-in-place concretes were assumed to be of normal weight.

Concrete stresses specified in Subsection 9.15.2 of the AASHTO *Standard Specifications for Highway Bridges*[12] were used with the exception that the allowable compression stress after losses was taken to be $0.60f'_c$, where f'_c is the compressive strength of the concrete at 28 days.

Superimposed dead load of 23 psf (1.1 kPa) was assumed. HS-25 truck loading was used in the analysis with impact and distribution factors computed pursuant to AASHTO Subsections 3.8.1 and 3.23.2.2, respectively.[12]

Cost analysis was based on average relative values provided by precast concrete manufacturers from across the United States. These costs are tabulated in Table 1. Note that these are costs used for comparison between precast girders in this study. They are not intended to be used to predict the actual cost of construction.

The 0.5 in. (12.7 mm) diameter, 270 ksi (1862 MPa), low-relaxation pretensioning strand was assumed to resist 1.2 times the unfactored girder and deck slab weight to provide a factor of safety for the additional flexural stresses due to transportation and erection and to provide allowance for construction loads. The 0.6 in. (15.2 mm) diameter, 270 ksi (1862 MPa), low-relaxation post-tensioning strand was designed to resist the remainder of the loads. Initial stress of 189 ksi (1303 MPa) was assumed in all prestressing strands, with assumed pretensioning loss of 10 ksi (69 MPa) at erection of the girder and additional loss of 19 ksi (131 MPa) due to time-dependent effects at the final stage. Assumed post-tensioning loss due to time-dependent effects was 29 ksi (200 MPa) at the final stage.

Due to the trend of concrete bridge design experts toward greater emphasis on ultimate strength criteria, strength design was used for optimization. "Working stress" or "service load" allowable concrete stresses were satisfied at all construction stages. With these criteria, structural efficiency is maximized when the girder shape simultaneously reaches ultimate strength capacity in both the positive and negative moment regions under dead load plus maximum live load conditions.

FEATURES OF THE OPTIMUM GIRDER SHAPE

The proposed Nebraska University Girder Series is shown in Figs. 5 and 6. The new shape is designated: NU2000 for Nebraska University, 2000 mm (78.7 in.) deep; NU2000PT designates a section with widened web for post-tensioning applications.

For all the girders examined in the parametric study, the factor limiting the maximum span length achievable at a given girder spacing was the maximum reinforcement index in the negative moment section, as described below. Up to these maximum achievable span lengths, service load stresses were within allowable limits. Therefore, optimization depended heavily on increasing the available compression area in the bottom flange to keep the reinforcement index below the maximum allowed.

Early in the project a non-prismatic shape was considered. However, a prismatic shape was finally selected for ease of fabrication, handling and transportation; difficulty in standardizing shape transition and widespread industry resistance to non-prismatic girders were also factors. This decision prevented a "perfect" solution in which ultimate strength capability is reached simultaneously in both maximum positive and negative moment sections at the maximum achievable span length.

Due to the importance of the depth of the girder and its impact on substructure and approach costs, an effort was made to minimize the girder depth required to achieve a given span length. Also, for the same span length and superstructure depth, the higher load ca-

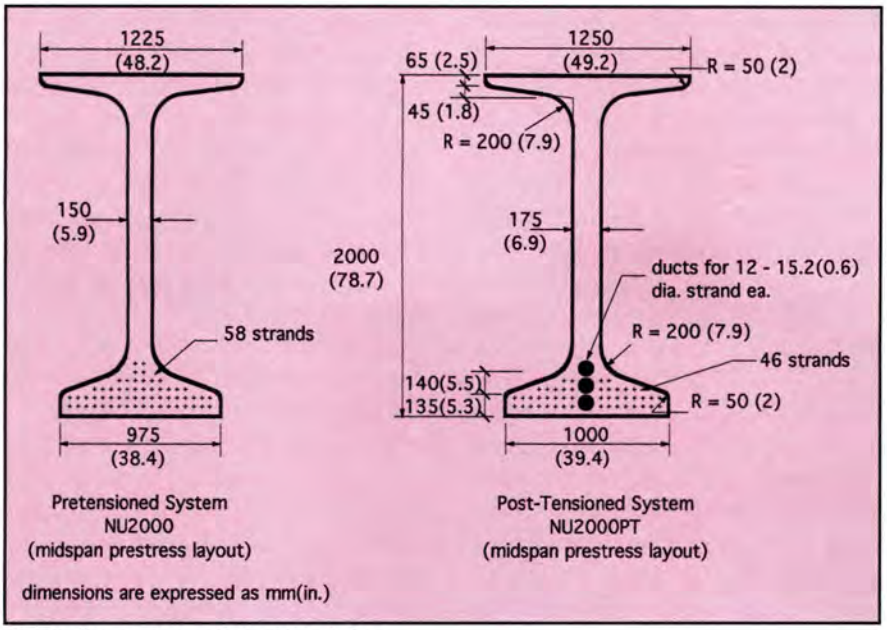

Fig. 5. Dimensions of the NU girders.

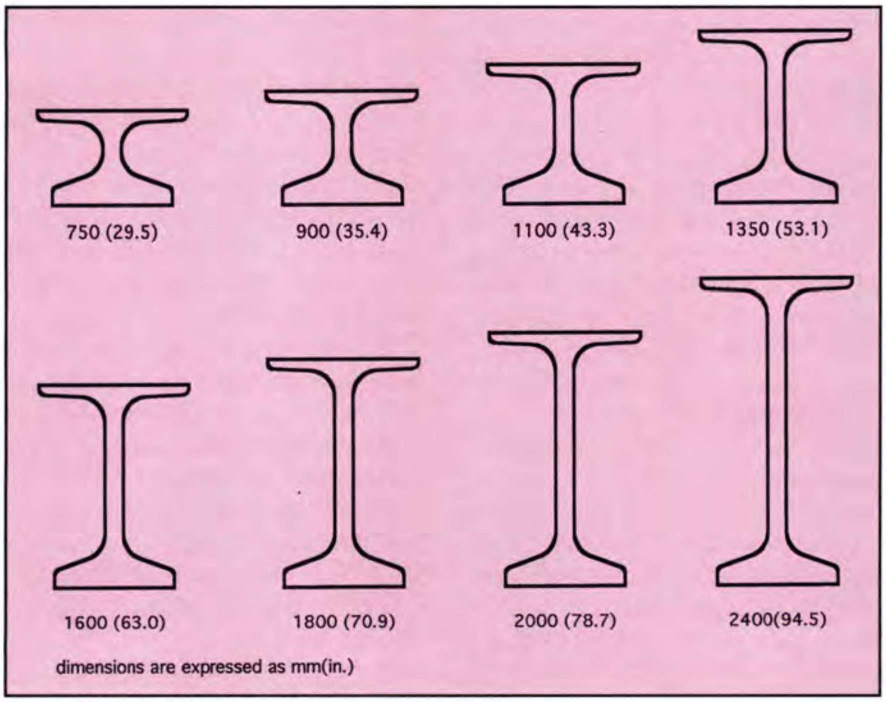

Fig. 6. The NU girder series.

pacity of the NU girder is beneficial in bridge replacement applications.

This study was initially conducted on a series of girder depths beginning at 1600 mm (63 in.) and increasing in 400 mm (15.7 in.) increments up to 2800 mm (110 in.). However, after presentation of the results to several state agencies, it became apparent that a strong need exists in the entire range of existing I-girder depths and that new girder depths should be suitable for bridge replacement and widening projects that were originally constructed with existing girder shapes. Further investigations have resulted in the following series of depths: 750, 900, 1100, 1350, 1600, 1800, 2000, and 2400 mm (29.5, 35.4, 43.3, 53.1, 63, 70.9, 78.7, and 94.5 in.).

The new depths will add flexibility because of the smaller increments between depths, and because of the close match with existing girder depths used by most states. Obviously, each state would adopt as many of the depths as needed for its use. For example, NDOR has adopted for immediate use NU1100, NU1350 and NU2000 girders. It also anticipates eventual use of the NU1600 and NU2400 girders. It should be noted that the top and bottom flange dimensions are identical for the entire series. This allows the use of only one set of forms with web extension panels. The concrete surface "seams" at form panel joints have not been found to be aesthetically objectionable.

Aesthetics is becoming a greater concern, and many designers are of the opinion that I-girders are less attractive than box girders. This is mainly because the sharp angles where flanges join the web and on the outside edge of flanges are considered unsightly, especially when viewed under bright sunlight. In an effort to improve its appearance, the NU girder series was designed with circular curves, rather than sharp angles at the edges of the flanges and at the points where the flanges join the web. The circular curves at the junctures of the flanges and the web have a radius of 200 mm (7.9 in.); at the outside flange edges, the radius is 50 mm (2 in.). Another important benefit of the circular curve between the web and the bottom flange is that it facilitates placement and consolidation of concrete.

The cross-sectional area of the bottom flange is the most important factor in determining the maximum achievable span length of a given girder. In the negative moment area the limiting factor was the maximum reinforcement index, as dictated by AASHTO Subsection 9.18.1[12] and the requirement in Subsection 18.8.1 of the ACI Building Code and Commentary.[13]

The maximum reinforcement index in the ACI Code corresponds to $0.85a/d_p \leq 0.36\beta_1$, where a is the depth of the rectangular compression block, d_p is the distance from the extreme compression fiber to the centroid of the prestressed steel, and β_1 is the ratio of the compression block depth to the neutral axis depth. The National Cooperative Highway Research Project 12-33 resulted in new bridge design specifications known as the LRFD Specifications.[14] In the LRFD Specifications, Section 5.7.3.3, this relationship is $c/d_e \leq 0.42$, where c is the neutral axis depth = a/β_1 and d_e is the effective depth of the combined prestressed and non-prestressed steel areas. Due to difficulty in determining d_e, the authors chose to rely on research performed by Skogman et al.[15]

In their report, Skogman et al. have shown that the maximum reinforcement index can be approximated by $c/h \leq 0.36$; where h is the total girder depth. This limit gives identical results to those of the LRFD Specifications when $d_e = 0.86h$.

It was found that for a given total cross-sectional area and a given bottom flange area, the girder with the wider bottom flange has better structural efficiency. This is because a greater number of pretensioning strands can be placed in the bottom row of the wider flanged section and thus at the greatest eccentricity from the centroid of the concrete section. This allows the wider flanged section a greater positive moment resistance for a given area of pretensioning strand.

The drive toward longer spans and wider girder spacing has caused an increased use of high strength concrete. This increases the required pretensioning force and thus the area of pretensioned steel. To accommodate this, specification of 0.6 in. (15.2 mm) diameter strand instead of 0.5 in. (12.7 mm) diameter strand is likely to increase in the future. This is consistent with the trend in Japan[16] and elsewhere. The additional bulk in the bottom flange of the NU girders allows them to accommodate the increased area of strand and resulting pretensioning forces better than existing girders.

For the greatest structural efficiency, the bottom flange was made as wide as can be accommodated in existing plants. The edge thickness is deep enough to allow two rows of pretensioning strands at 2 in. (50.8 mm) on center with 2 in. (50.8 mm) of vertical and horizontal concrete cover. This allows enough area for sufficient strand to be placed when using the girder in an application in which pretensioning only, with no post-tensioning, is used. This results in a bottom flange width of 1000 mm (39.4 in.) and an edge thickness of 135 mm (5.3 in.).

Caution must be taken when designing bearing pads for wide flanged girders. A pad that is as wide at the bottom flange is not only unnecessary, but also may cause splitting in the flange. It is recommended that the pad width not exceed 24 in. (610 mm). This width is comparable to that used with AASHTO standard girders.

The slope of the top part of the bottom flange was dictated by two conflicting requirements. It had to be as shallow as possible to keep the maximum concrete area low in the girder to gain the greatest structural efficiency. On the other hand, it had to be steep enough to allow good consolidation of concrete. Extensive investigation of current practices convinced the researchers that a slope of about 1 to 3 was practical, i.e., 140 mm (5.5 in.) vertically over a horizontal distance of 412.5 mm (16.2 in.). It should be kept in mind that the widespread use of superplasticizers and improved production techniques make unnecessary the very steep 1 to 1 slope of early girder shapes.

This study has confirmed earlier studies that structural efficiency is increased when the web width is decreased. The web of the proposed girder was thus set to accommodate the required reinforcement: a 75 mm (3 in.) diameter post-tensioning duct, two 12.5 mm (0.5 in.) draped pretensioning strands, 12.5 mm (0.5 in.) diameter stirrups for shear reinforcement and two 25 mm (1 in.) concrete covers — a total of 175 mm (6.9 in.). It is not necessary to use strand draping, especially when post-tensioning is utilized. Debonding (shielding) of some of the strands at girder ends is a cost-effective method of satisfying working stress design requirements. When no draped strands are used, a 50 mm (2 in.) wide space becomes available for larger post-tensioning ducts and/or clear cover.

Some bridge design professionals have expressed concern that the web width of 175 mm (6.9 in.) is too thin. Many designers have not used girders with webs less than 8 in. (203 mm) wide and are uncomfortable with the thinner webs. Others are concerned that precast concrete manufacturers will have difficulty with concrete placement in the thinner webs that contain post-tensioning hardware, mild steel reinforcement and perhaps draped pretensioning strand. Several manufacturers and bridge designers have offered their assurance that, based on experience with a number of recent projects, there would be no difficulty with concrete placement in this situation. In fact, recent bridges in Canada, notably the Esker Overpass,[17] have 150 mm (5.9 in.) webs. It should be noted that a narrower or wider web can be easily manufactured by moving the side forms inward or outward.

The study has shown that the top flange area should be minimized and its width maximized. A large cross-sectional area in the top flange has been found unnecessary. The concrete deck, in composite action with the girder, provides the required compression resistance. By minimizing the area of the top flange, significant savings in costs can be made.

The wider the top flange, the smaller the effective span length of the deck for a given cross-sectional area. This reduces the required deck thickness and material costs. In addition, the wider top flange reduces the formwork cost of the deck slab and provides a convenient worker platform during construction.

The top flange should not be too wide, however. Wide top flanges require additional cast-in-place concrete to accommodate cross-slope and superelevation of the deck. Also, wide top flanges must have increased average thickness if the bottom surface slope is to be maintained for convenience in formwork removal. The NU girder has a top flange width of 1250 mm (49.2 in.). This is approximately the same as the Florida bulb tee, 183 mm (7.2 in.) wider than the AASHTO/PCI bulb tee and 742 mm (29.2 in.) wider than the AASHTO Type IV.

The NU girder top flange edge thickness was initially set at 75 mm (3 in.) and later reduced to 65 mm (2.6 in.). Some bridge designers are uncomfortable with a top flange thickness of less than 3 in. (76.2 mm) despite the record of the Florida bulb tee, which has a top flange edge thickness of only 2 in. (50 mm). A thicker edge, if required, can be easily accommodated by minor adjustments of the top flange edge forms.

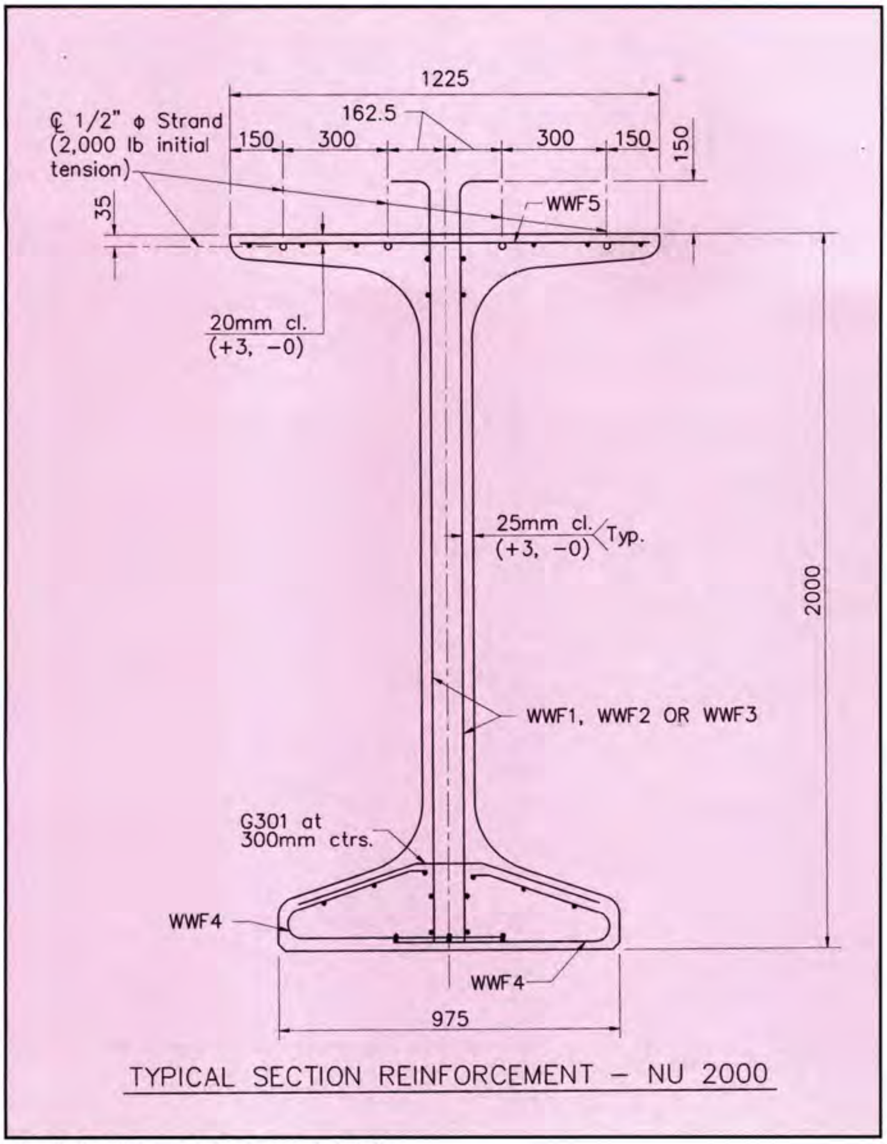

Fig. 7. Auxiliary reinforcement details.

The area of the top flange can be minimized while maintaining the desired width and edge thickness by minimizing the slope of the bottom surface of the top flange. Only a small slope is required to facilitate stripping of the concrete forms from the completed girder. The flange bottom slope was initially set at about 1 to 22. After trial production of full-scale NU2000 girder segments, this slope was increased to 1 to 12.

Lateral stability of the NU girder was evaluated by Mast using a computer model he developed.[18] His analysis for stability during transportation and erection indicated that its performance was superior to girders with narrower bottom flanges.

The girder concrete strength was not varied in this study. Clearly, 28-day concrete strengths higher than the conventional 5000 to 6000 psi (34.5 to 41.4 MPa) will figure prominently in the future of I-girder bridge construction. In fact, NDOR has recently adopted a new standard girder strength of 7500 psi (51.7 MPa) and cast-in-place deck strength of 5000 psi (34.5 MPa), instead of the old 5000 and 3500 psi (34.5 and 24.1 MPa) values. Recent studies[19-21] have concluded that the use of high strength concrete in prestressed girder bridges is not only feasible but also economical, especially when durability and life-cycle costs are considered.

The use of high strength concrete in bridge I-girders has been explored by Castrodale et al.,[22] and a project involving testing of full-sized high strength concrete bridge girders is being performed at the University of Minnesota by Dr. French and her associates. The longer spans achievable by use of high strength concrete require an increased number of prestressing strands. The NU girder is better suited for use with high strength concrete than existing standard girders due to its bulky bottom flange that allows accommodation of a large number of prestressing strands.

Conventional, non-prestressed reinforcement is required for such effects as vertical shear, horizontal composite action shear, end zone stresses, confinement of bottom flange concrete and transverse bending of the top flange. Traditionally, individually tied reinforcing bars have been used for I-girders.

The recent success of the use of welded wire fabric in double tees and some box girder sections has led to consideration of its use in I-girders. In cooperation with HDR Engineering Inc., the authors developed the details shown in Figs. 7 and 8. Most of the reinforcement, with the exception of the #3 "cap" and end zone reinforcement (not shown), consists of prefabricated welded wire fabric. These details have been found to meet all design requirements for the Valley West Bridge, the design of which has recently been completed in Nebraska. The project features NU2000 girders with spans ranging from 135 to 150 ft (41.1 to 45.7 m) and transverse spacing of 9 ft 7 in. (2.92 m).

Welded wire fabric with the same size and spacing of wires has been found to meet the requirements in other bridge designs. When girders with a different depth are used, only the depth of the vertical mesh of the reinforcement need be adjusted. Use of welded wire fabric is expected to greatly simplify production. It is anticipated that standard sizes of welded wire fabric will emerge.

AASHTO requires that all vertical steel reinforcement be extended into the cast-in-place concrete deck. Some states require additional dowel reinforcement near the outside edges of wide flanges. This has been found unnecessary for horizontal shear resistance or for composite action, and may cause complications with future deck removal and replacement.

SPLICED POST-TENSIONED GIRDER PERFORMANCE

The following discussion demonstrates how the proposed girder performs relative to some of the commonly used girders in the United States and Canada. The two-span spliced-girder system shown in Fig. 4 is used in the comparisons in this section. The Florida bulb tee and the girder proposed for the state of Kentucky are believed to be the most modern girders in the United States, specifically developed for continuous post-tensioned bridge applications. Other sections are the Canadian girders, the AASHTO/PCI bulb tee, and the AASHTO Type VI. These girders were used for performance comparisons.

Each of the standard girders examined has a different web width. To provide a meaningful comparison between the girders analyzed, the overall widths were adjusted to produce a constant web width of 175 mm (6.9 in.). The dimensions and section properties of the girders with these modifications are contained in Appendix A.

At a girder spacing of 10 ft (3 m), the minimum deck thickness of 7.5 in. (191 mm) governed for all the girders. Due to this, the advantage of reduced deck thickness associated with wider top flanges has not been accounted for in this analysis. The NU girder would have compared more favorably had a wider girder spacing been used.

In the bridge system analyzed, the maximum reinforcement index in the negative moment section was the factor that limited achievable span length for all girders considered. In Fig. 9, the relationship of the reinforcement index vs. span length for the girders is shown. With the exception of the much deeper Canadian 2000 girder, the NU1800 has the highest achievable span length, 157 ft (47.9 m), even though some of the girders have larger cross-sectional areas.

The girders were also compared for the relative cost of a bridge superstructure constructed using each girder. Fig. 10 shows the superstructure cost at various span lengths and a constant girder spacing of 10 ft (3 m). It can be seen that the AASHTO/PCI BT 72" and the Canadian 1728 girders are economical in the shorter span ranges. However, at approximately the same depth of 72 in. (1829 mm), the NU girder outperforms all other girders in terms of overall span range and economy. The Canadian 2000 girder's economy comes primarily from the fact that it is 200 mm (7.9 in.) deeper than the other girders presented in Fig. 10.

Another way to illustrate the capacity and economy of the NU girder is presented in Table 2. The same as-

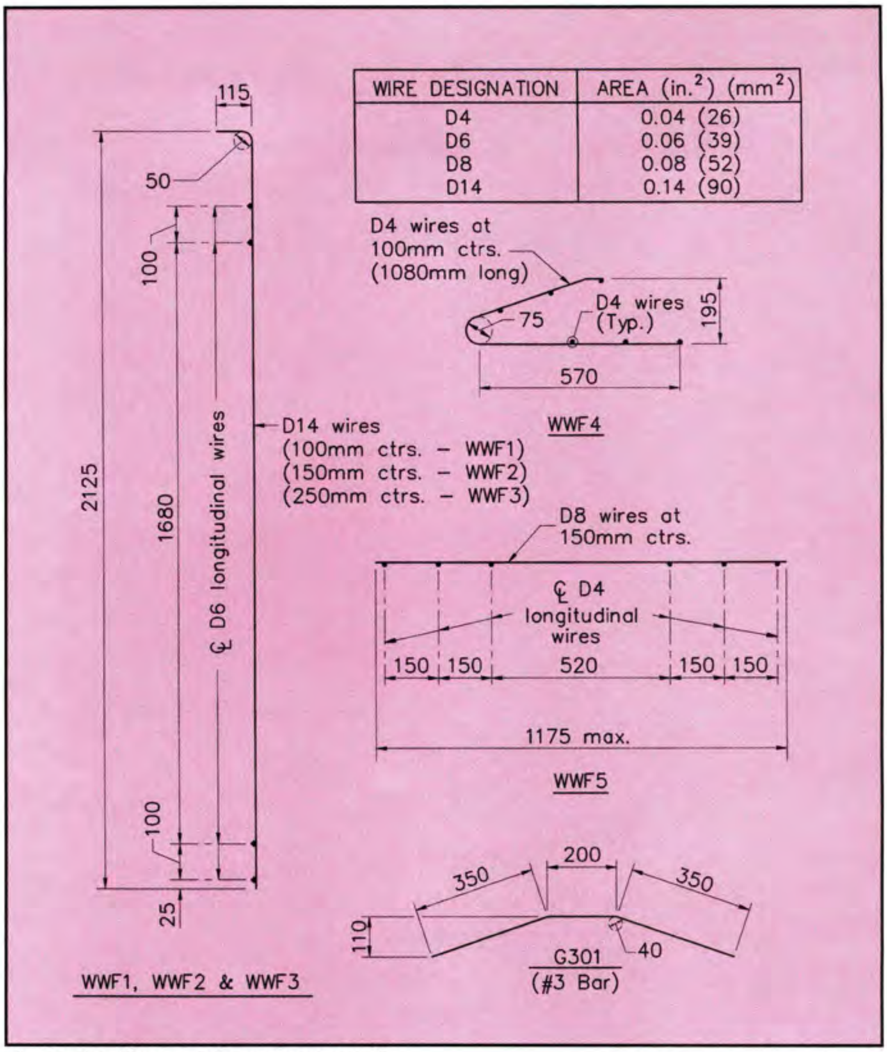

Fig. 8. Auxiliary reinforcement details.

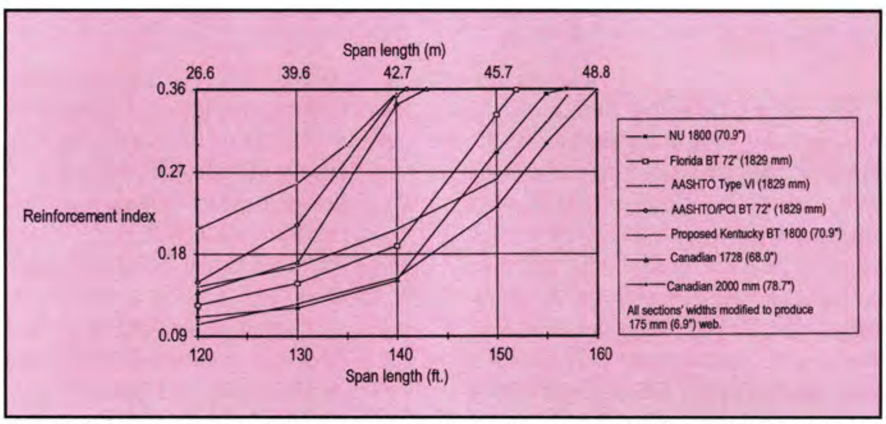

Fig. 9. Reinforcement index vs. span length for the system in Fig. 4 with a girder spacing of 10 ft (3 m).

sumptions used in generating Figs. 9 and 10 are used here, except the girder spacing is variable. Also, the properties of the Canadian 1728 and 2000 girders were interpolated to produce a "Canadian 1800" shape in order to keep the depth of all girders constant. For a given span length, the table contains the maximum spacing possible for each girder and the corresponding unit cost.

The table shows that the NU girder is more economical than other existing shapes when its full capacity is utilized. It also demonstrates the possibility of using 1800 mm (70.9 in.) deep precast concrete girders to economically span 170 ft (51.8 m) at a reasonably wide spacing of 8 ft (2.4 m). Obviously, longer spans are possible. However, feasibility of transporting the field segment, which is 80 percent of the span length, must be considered.

GIRDER PERFORMANCE WITH CONVENTIONAL DECK REINFORCING BAR CONTINUITY

The NU girders outperform existing standard girders, even in systems with pretensioned girders made continuous by adding mild steel reinforcement in the cast-in-place deck. In this system, the majority of the load is applied to a simple span configuration. Only the superimposed dead load and the live load are applied after continuity is achieved.

The figures being referred to in the following discussion make the same assumptions regarding material prop-

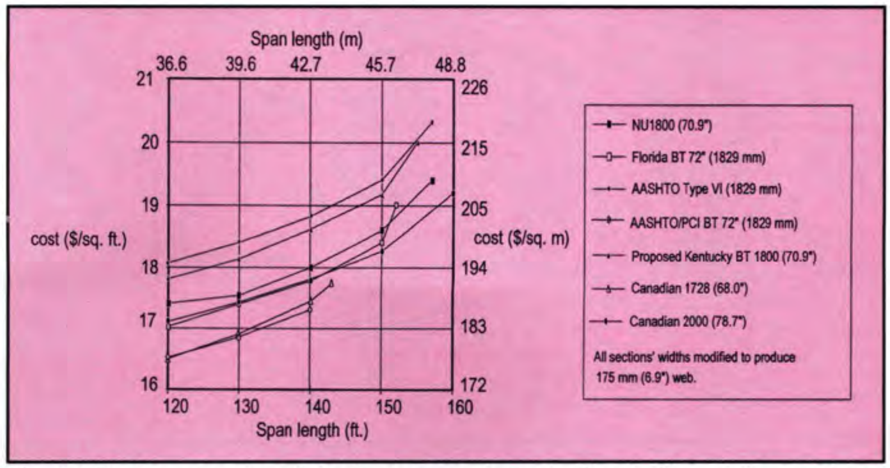

Fig. 10. Cost vs. span length for the system shown in Fig. 4 with a girder spacing of 10 ft (3 m).

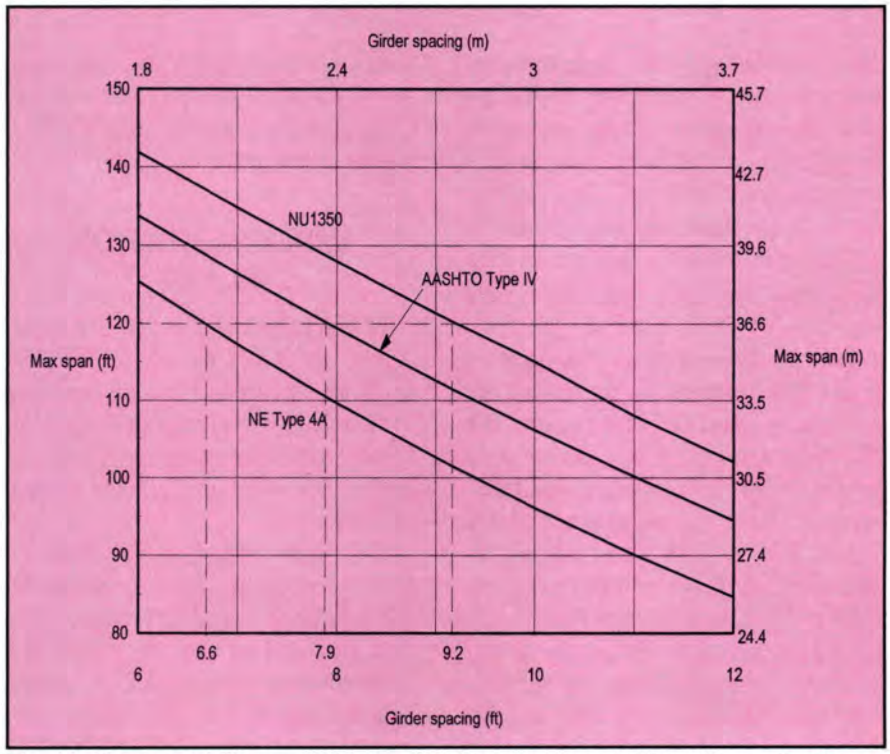

Fig. 11. Span capability vs. girder spacing for a two-span bridge with continuity achieved by deck reinforcing bars.

Table 2. Maximum spacing and corresponding unit cost for various span lengths of a two-span spliced girder bridge.

Span, ft	140		150		160		170	
Girder	Spacing (ft)	Cost ($ per sq ft)	Spacing (ft)	Cost ($ per sq ft)	Spacing (ft)	Cost ($ per sq ft)	Spacing (ft)	Cost ($ per sq ft)
NU1800	13.02	16.85	11.19	18.15	9.57	20.04	8.15	22.39
Florida BT 72"	12.00	16.96	10.30	18.51	8.94	20.53	7.48	23.05
AASHTO Type VI	13.38	17.53	11.42	18.75	9.53	20.96	8.00	23.67
AASHTO/PCI BT 72"	10.00	17.31	8.47	19.25	7.16	21.67	6.11	24.52
Proposed Kentucky BT 1800	12.47	17.31	10.71	19.10	9.08	21.22	7.82	23.40
Canadian 1800	11.53	17.13	9.73	18.73	8.24	20.87	6.91	23.61

Note: 1 ft = 0.3 m; $1.00 per sq ft = $10.75/m^2.

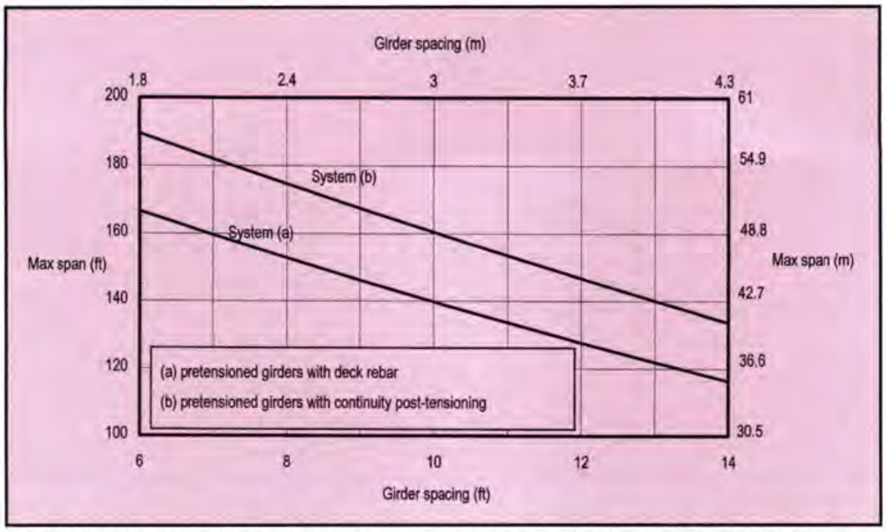

Fig. 12. Span capability vs. girder spacing for two methods used for achieving continuity.

erties and loads as the comparison of Figs. 9 and 10, except that precast concrete strength at release was taken as 5500 psi (37.9 MPa) and 28-day strength was 7500 psi (51.7 MPa); cast-in-place deck concrete strength at 7 days was taken as 4000 psi (27.6 MPa) and 28-day strength was 5000 psi (34.5 MPa).

Fig. 11 demonstrates the relative span capabilities of the Nebraska Type 4A, the AASHTO Type IV, and the NU1350 girders. For a given span length of 120 ft (36.6 m), the three types can have spacing as much as 6.6, 7.9, and 9.2 ft (2, 2.4, and 2.8 m), respectively. A good indicator of girder efficiency is the equivalent solid slab thickness, which is the girder cross-sectional area divided by the spacing. For the 120 ft (36.6 m) span considered, the NU girder equivalent slab thickness of 6.82 in. (173 mm) is significantly less than that of the AASHTO Type IV (8.32 in., 211 mm), and the Nebraska Type 4A (8.13 in., 207 mm). The NU girder can be shown to be similarly superior in simple span applications.

Fig. 12 illustrates the advantages of the post-tensioned system used for optimization compared to the system with mild steel reinforcement in the deck. At all girder spacings, the post-tensioned system increases the span capability by about 15 percent. Note the very large span capacity of the NU1800 girder, which is slightly shallower than the AASHTO/PCI 72 in. (1829 mm) bulb tee. It can span up to 160 ft (48.8 m) at 10 ft (3.0 m). The corresponding bulb tee span is 141 ft (43 m) (see Fig. 9).

IMPLEMENTATION PLANS

The Nebraska Department of Roads is now specifying the use of NU girders in all precast I-girder construction. In addition, it has awarded the University of Nebraska a two-year research project to develop implementation details.

This project will comprise the completion of design details, including shear, deflection and end-zone design. Also required will be the preparation of preliminary design aids that establish the girder size and spacing required for given span and loading conditions. The study will examine the economics and structural efficiency of various methods for achieving continuity.

The Salem West Bridge, in Richardson County, Nebraska, is scheduled for construction in the second half of 1994. It is a three-span bridge, with spans of about 78 ft (23.8 m). The system consists of pretensioned NU1100 girders made continuous by mild steel reinforcement in the cast-in-place deck.

Since the beginning of this project, several states have expressed interest in the study and a desire to update their current standard girder shapes.

CONCLUSIONS

1. Precast concrete I-girder systems are the most widely used methods for constructing bridges in the 70 to 120 ft (21 to 36 m) span range.

2. The main objective of this study was to develop a metric I-girder shape that is highly efficient in continuous span applications and that will extend the span capabilities of the popular I-girder bridge system.

3. This study has produced a new I-girder shape, called the NU girder, that has dimensions in "hard" metric units and has the following main features:

(a) Girder depths of 750, 900, 1100, 1350, 1600, 1800, 2000, and 2400 mm (29.5, 35.4, 43.3, 53.1, 63, 70.9, 78.7, and 94.5 in.) that cover the entire range of depths;

(b) Large bottom flange allowing increased strand placement capacity for simple spans and increased negative moment capacity for continuous spans;

(c) Constant top and bottom flange dimensions for various girder depths to allow the use of only one set of panelized steel forms, where web height can be adjusted with addition or deletion of form panels;

(d) Wide top flange to allow for better worker platform and shorter deck slab span; and

(e) Curves, rather than straight line fillets, to allow for easy placement of concrete and for improved bridge superstructure appearance.

4. The proposed NU girders have been shown to span further than any other standard I-girder shape in existence, which makes it a strong competitor for bridges currently in the exclusive domain of structural steel, i.e., spans in the 200 to 300 ft (61 to 91 m) range, and for bridge replacement projects where higher load capacities are generally required for the same structural depth.

5. The span capability of the NU girders is superior to existing standard girders in all types of I-girder bridge systems including simple span designs, applications where continuity is achieved by mild steel reinforcement in the cast-in-place deck, and in systems that utilize full-length continuity post-tensioning.

6. Regardless of the I-girder shape used, when girder continuity is implemented before the cast-in-place deck is placed, e.g., by means of post-tensioning, the span capability can be increased by as much as 15 percent.

7. To improve fabrication productivity, welded wire fabric, rather than individually tied reinforcing bars, is used to resist vertical shear, horizontal composite action shear, and other effects. With increased use of welded wire fabric, it is anticipated that standard sizes will be manufactured.

8. The Nebraska Department of Roads has already adopted the NU girder series and is now specifying NU girders in all precast I-girder construction. The first bridge using NU girders is scheduled for construction in the second half of 1994.

ACKNOWLEDGMENT

The authors wish to thank the Precast/Prestressed Concrete Institute and the Center for Infrastructure Research at the University of Nebraska for providing the funding for this project.

The Precast Concrete Association of Nebraska, in particular Larry Fischer and Morrie Workman, has been instrumental in making this theoretical development a reality. Their helpful suggestions and willingness to keep an open mind throughout the duration of this project are greatly appreciated.

Special thanks go to Lyman Freemon, Nebraska Department of Roads, whose leadership and encouragement has led Nebraska to implement advancements in precast concrete bridge technology. A note of thanks to Jeff Curren, HDR Engineering Inc., who designed the welded wire fabric reinforcement details.

Many other individuals deserve a mention of appreciation for the information and guidance they provided on several occasions throughout the progress of this project. They include Robert F. Mast, Jagdish C. Nijhawan, Paul Johal, Basile G. Rabbat, Leo Spaans, Mohsen Shahawy, Richard R. Imper, Reid W. Castrodale and Alex Aswad. The authors also wish to thank all those who responded to the survey.

Finally, the authors wish to express their appreciation to Deborah Derrick and David Salmon for their editorial work on this manuscript.

REFERENCES

1. Dunker, Kenneth F., and Rabbat, Basile G., "Performance of Prestressed Concrete Highway Bridges in the United States — The First 40 Years," PCI JOURNAL, V. 37, No. 3, May-June 1992, pp. 48-64.
2. *Standard Plans for Highway Bridges*, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., 1956.
3. *Standard Plans for Highway Bridges, Concrete Superstructures*, V. 1, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., January 1990.
4. Anderson, Arthur R., "Systems Concepts for Precast and Prestressed Concrete Bridge Construction," Special Report 132, Highway Research Board, Washington, D.C., 1972.
5. Koretzky, Heinz P., "Precast Prestressed Bridges," Preliminary Report, Pennsylvania Department of Transportation, Harrisburg, PA, 1980.
6. "Metric Conversion Activities of Federal Government Agencies in Compliance with P.L. 100-418, Section 5164, Metric Usage," Report prepared by the Congressional Research Service, Library of Congress, Washington, D.C., November 1991.
7. Abdel-Karim, Ahmad, and Tadros, Maher K., "State-of-the-Art of Precast-Prestressed Spliced-Girder Bridges," PCI Special Publication Sponsored by PCI Committee on Bridges, Precast/Prestressed Concrete Institute, Chicago, IL, 1993, 133 pp.
8. Abdel-Karim, Ahmad, and Tadros, Maher K., "Design and Construction of Spliced I-Girder Bridges," PCI JOURNAL, V. 37, No. 4, July-August 1992, pp. 114-122.
9. Rabbat, Basile G., and Russell, Henry G., "Optimized Sections for Precast, Prestressed Bridge Girders," PCI JOURNAL, V. 27, No. 4, July-August 1982, pp. 88-104.
10. Garcia, Antonio M., "Florida's Long Span Bridges: New Forms, New Horizons," PCI JOURNAL, V. 38, No. 4, July-August 1993, pp. 34-49.
11. Lounis, Z., and Cohn, M. Z., "Optimization of Precast Prestressed Concrete Bridge Girder Systems," PCI JOURNAL, V. 38, No. 4, July-August 1993, pp. 60-78.
12. AASHTO, *Standard Specifications for Highway Bridges*, Fifteenth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1992.
13. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318R-89)," American Concrete Institute, Detroit, MI, November 1989.
14. "Draft LRFD Specifications and Commentary," Draft No. 4, prepared by Modjeski and Masters Consulting Engineers as part of the National Cooperative Highway Research Project, Transportation Research Board, Washington, D.C., March 1993.
15. Skogman, Brian C., Tadros, Maher K., and Grasmick, Ronald, "Ductility of Reinforced and Prestressed Concrete Flexural Members," PCI JOURNAL, V. 33, No. 6, November-December 1988, pp. 94-107.
16. Yamane, Takashi K., and Arumugasaamy, P., "Short to Medium Span Prestressed Concrete Bridges in Japan," PCI JOURNAL, V. 39, No. 2, March-April 1994, pp. 74-100.
17. Mills, Donald, Chow, Kenneth T., and Marshal, Scott L., "Design-Construction of Esker Overhead," PCI JOURNAL, V. 36, No. 5, September-October 1991, pp. 44-51.
18. Mast, Robert F., "Lateral Stability of Long Prestressed Concrete Beams — Part 2," PCI JOURNAL, V. 38, No. 1, January-February 1993, pp. 70-88.
19. Roller, John J., Martin, Barney T., Russell, Henry G., and Bruce, Robert N., Jr., "Performance of Prestressed High Strength Concrete Bridge Girders," PCI JOURNAL, V. 38, No. 3, May-June 1993, pp. 34-45.
20. Durning, Timothy A., and Rear, Kenneth B., "Braker Lane Bridge — High Strength Concrete in Prestressed Bridge Girders," PCI JOURNAL, V. 38, No. 3, May-June 1993, pp. 46-51.
21. Dolan, Charles W., Ballinger, Craig A., and LaFraugh, Robert W., "High Strength Prestressed Concrete Bridge Girder Performance," PCI JOURNAL, V. 38, No. 3, May-June 1993, pp. 88-97.
22. Castrodale, W., Burns, Ned H., and Kreger, Michael E., "A Study of Pretensioned High Strength Concrete Girders in Composite Highway Bridges," Center for Transportation Research, University of Texas at Austin, Austin, TX, January 1988.

APPENDIX A — SECTION PROPERTIES OF STANDARD AND MODIFIED GIRDERS

Table A1. Section properties of standard girders.

I-sections

Agency	Type	D1	D2	D3	D4	D5	D6	B1	B2	B3	B4	*A*	*I*	y_b
AASHTO	Type I	28	4	3	–	5	5	12	16	6	–	276	22,750	12.6
	Type II	36	6	3	–	6	6	12	18	6	–	369	50,980	15.8
	Type III	45	7	4.5	–	7.5	7	16	22	7	–	560	125,390	20.3
	Type IV	54	8	6	–	9	8	20	26	8	–	786	260,730	24.7
	Type V	63	5	3	4	10	8	42	28	8	4	1013	521,180	32.0
	Type VI	72	5	3	4	10	8	42	28	8	4	1085	733,320	36.4
AASHTO/PCI	BT-54	54	3.5	2	2	4.5	6	42	26	6	2	659	268,077	27.6
	BT-63	63	3.5	2	2	4.5	6	42	26	6	2	713	392,638	32.1
	BT-72	72	3.5	2	2	4.5	6	42	26	6	2	767	545,894	36.6
Washington	Series 14	73.5	2.875	2.625	2	3	6	42	24	5	2	674	514,312	9.2
Colorado	G72	72	9.5	1.5	2	3.5	6.5	28	24	5	2	760	557,552	39.2
Canada	1728 mm	68.0	5.0	2.5	–	3.5	7	23.6	28.0	5.0	–	673	428,127	31.2
	2000 mm	78.7	3.9	3.9	–	5.5	7.1	33.5	28.0	5.5	–	820	701,533	37.4
Nebraska	Type 4A	54	5	4.5	–	7	6	24	24	6.5	–	644	236,105	25.9
	BT-1A	75	3	2	2	6.75	5.75	30	26	6	2	760	564,754	35.6

I-sections with curved surfaces

Agency	Type	D1	D2	D3	D4	D5	B1	B2	B3	R1	R2	R3	R4	*A*	*I*	y_b
Florida	BT-72	72	2	4	5.5	7.5	48	30	6.5	8	8	0	0	901	638,672	34.4
Kentucky*	BT-1800	70.9	3	3.9	6.9	8.9	59.1	27.6	7.1	7.5	7.5	0	7.5	1,010	702,828	36.5
Nebraska University	NU750	29.5	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	614	69,403	13.6
	NU900	35.4	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	649	110,444	16.1
	NU1100	43.3	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	695	182,550	19.6
	NU1350	53.1	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	753	302,743	23.9
	NU1600	63.0	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	812	459,057	28.4
	NU1800	70.9	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	858	612,059	32.0
	NU2000	78.7	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	905	791,498	35.7
	NU2400	94.5	2.6	1.8	5.5	5.3	48.2	38.4	5.9	7.9	7.9	2.0	2.0	998	1,235,547	43.0

*Kentucky girder section properties were computed from a straight line approximation of curved surface.

Note: 1 in. = 25.4 mm.

Table A2. Section properties of modified girders.

I-sections

Agency	Type	D1	D2	D3	D4	D5	D6	B1	B2	B3	B4	*A*	*I*	y_b
AASHTO	Type VI	72	5	3	4	10	8	42.9	26.9	6.9	4	1,006	699,093	36.4
AASHTO/PCI	BT-72	72	3.5	2	2	4.5	6	42.9	26.9	6.9	2	832	573,909	36.6
Canada	1728 mm	68.0	5.0	2.5	–	3.5	7	25.5	29.9	6.9	–	802	478,762	31.7
	2000 mm	78.7	3.9	3.9	–	5.5	7.1	34.9	29.4	6.9	–	930	758,791	37.6

I-sections with curved surfaces

Agency	Type	D1	D2	D3	D4	D5	B1	B2	B3	R1	R2	R3	R4	*A*	*I*	y_b
Florida	BT-72	72	2	4	5.5	7.5	48.4	30.4	6.9	8	8	0	0	930	651,190	34.4
Kentucky*	BT-1800	70.9	3	3.9	6.9	8.9	58.9	27.6	6.9	7.5	7.5	0	7.5	996	696,975	36.5
Nebraska University	NU750	29.5	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	643	71,554	13.6
	NU900	35.4	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	684	114,178	16.2
	NU1100	43.3	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	738	189,390	19.7
	NU1350	53.1	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	806	315,398	24.1
	NU1600	63.0	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	874	480,111	28.6
	NU1800	70.9	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	928	642,003	32.3
	NU2000	78.7	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	982	832,521	36.0
	NU2400	94.5	2.6	1.8	5.5	5.3	49.2	39.4	6.9	7.9	7.9	2.0	2.0	1,091	1,306,244	43.4

*Kentucky girder section properties were computed from a straight line approximation of curved surface.
Note: 1 in. = 25.4 mm.

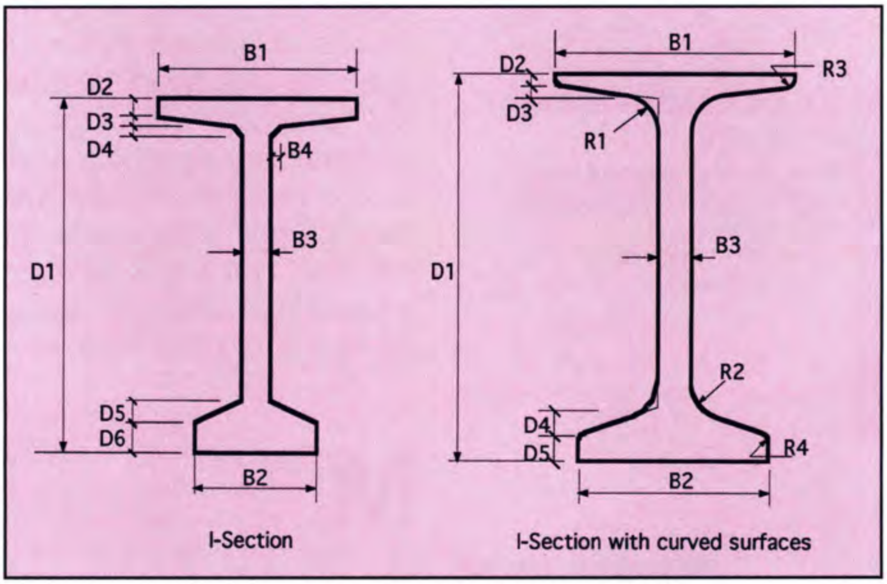

Fig. A1. Notation for dimensions contained in Tables A1 and A2.