

Nebraska Precast Record: 204th Street Bridge Spans Single Point Interchange



Fouad Jaber, P.E.

Assistant Bridge Engineer,
Bridge Division
Nebraska Department of Roads
Lincoln, Neb.



Maher K. Tadros, Ph.D., P.E., FPCI

Charles J. Vranek Professor
University of Nebraska–Lincoln
Omaha, Neb.



Sam Fallaha, P.E.

Assistant Bridge Engineer,
Bridge Division
Nebraska Department of Roads
Lincoln, Neb.



Chuanbing Sun, Ph.D.

Project Engineer
Janssen & Spaans Engineering
Indianapolis, Ind.



Amgad F. Morgan Girgis, Ph.D.

Research Assistant Professor
University of Nebraska–Lincoln
Omaha, Neb.



Fig. 1. A view of the 204th Street Bridge in Douglas County, Neb.

The 204th Street Bridge, with a single span of 63 m (207 ft), achieves a span-to-girder depth ratio of 31.5 using Nebraska University, 200-mm-deep (79 in.), post-tensioned (NU2000 PT), spliced I-girders. It is the longest simple-span concrete I-girder bridge in Nebraska and perhaps the United States. Each girder line consists of three precast, pretensioned concrete segments, erected on false work and post-tensioned together after attaining the required strength at the wet joints and before forming the deck slab. High-performance concrete with a 28-day design compressive strength of 68.75 MPa (10 ksi) was used for the girders. A non-bulky NU post-tensioning anchorage block was used to house all the post-tensioning anchorage hardware. The bridge cost \$624/m² (\$58/ft²), which was less than the cost of typical precast, prestressed concrete I-girder bridges in Nebraska (\$646/m² to \$807/m² [\$60/ft² to \$70/ft²]) at the time of bid letting. The efficient NU section and state-of-art design theory contributed to the success of this record-setting bridge. This paper presents the detailed description of the design and construction of the 204th Street Bridge in Douglas County, Neb.

The 204th Street Bridge consists of 14 total girder lines in two structures, an eastbound and a westbound structure. Each girder line consists of two 8.75-m-long (28.7 ft) precast concrete segments and one 45.5-m-long (149 ft) segment. All I-girder segments have Nebraska University, 2000-mm-deep (79 in.) (NU2000) cross sections and are made with precast, prestressed concrete.¹ The precast concrete segments were post-tensioned at the site on temporary supports to achieve the required 63-m-long (207 ft) single span.

A total of 28 short and 14 long NU2000 segments were used in the

bridge. The span length of the bridge makes its precast concrete girder spans the longest ever built in Nebraska. The bridge is also believed to have the longest simple span, with the shallowest girder depth-to-span ratio (31.5) in the United States.

As a 63-m-long (207 ft) simple-span bridge (centerline bearing to centerline bearing), the bridge has an overall length between abutments of 75.44 m (248 ft). The deck area is 2668 m² (28,700 ft²) including the suspended slab spans and 2241 m² (24,100 ft²) excluding the suspended spans. The total bridge cost was \$1,673,797, resulting in a cost per

square meter of \$624 (\$58/ft²).

The project design was completed in early 2002 and was let to contract on August 1, 2002. Construction on the bridge began in fall 2002, and the bridge was opened to traffic in late 2004. The bridge surface is expected to be stained with an attractive color in the near future. In 2005, the Precast/Prestressed Concrete Institute (PCI) named the bridge a design award co-winner in the category of bridges with spans greater than 135 ft (see 2005 PCI Design Awards Jury Comments on p. XX).

The 204th Street Bridge, at the intersection of 204th (Nebraska Highway 31) and Dodge Street (U.S. Highway

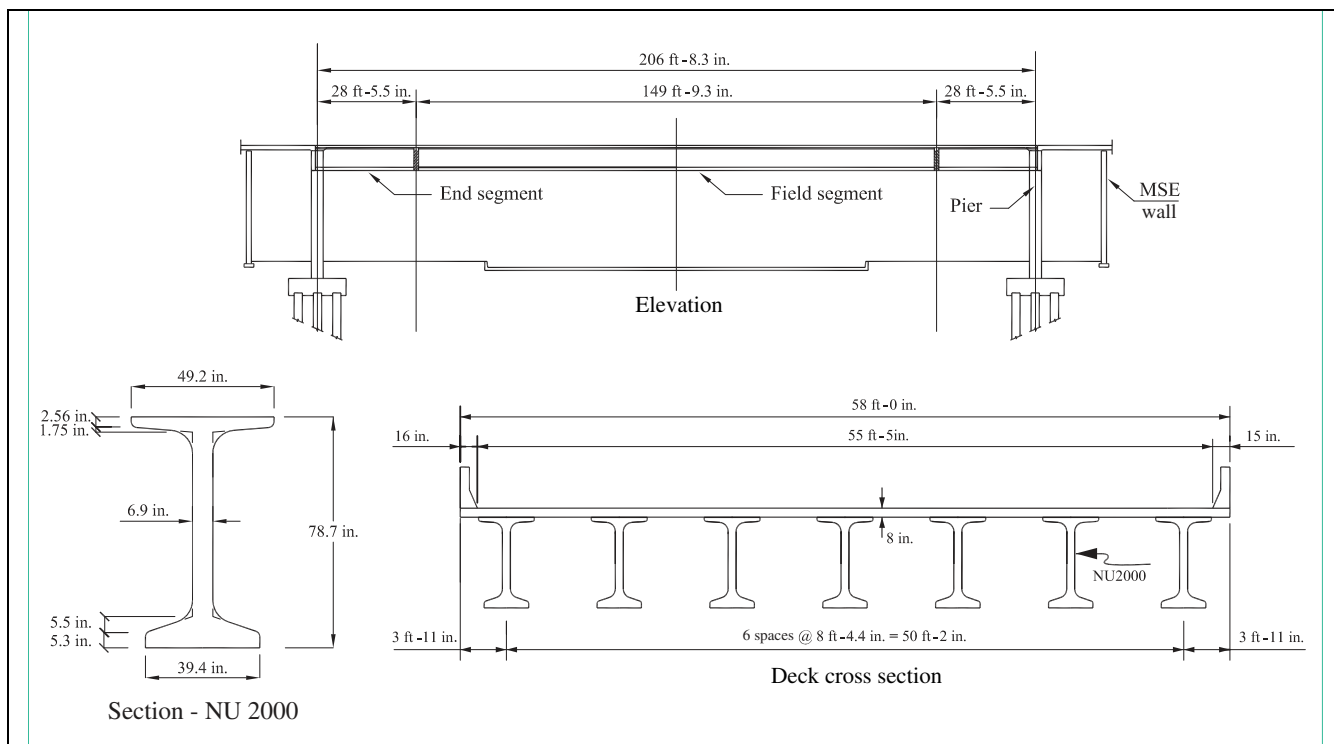


Fig. 2. The general layout of the 204th Street Bridge. Note: 1 in. = 25.4 mm; 1 ft = 12 in.

6) in Douglas County, Neb., provides the structure for a single-point urban interchange (SPUI). Geometric considerations for this SPUI required that the bridge employ a 63-m-long (207 ft) center span.

Due to interference between the mechanically stabilized earth (MSE) wall reinforcement and the foundation piling, as well as sight distance requirements of the SPUI, it was necessary to separate the MSE wall reinforcement and the foundation piling. This was achieved by creating piers and two additional short spans, resulting in the total bridge length of 75.44 m (248 ft). The two additional spans were only about 6.1-m-long (20 ft) each.

In this paper, the total structure is called a single bridge for simplicity. Also, the structural response of the main span was not significantly affected by the two end spans, and the structure may be considered a simple-span bridge. The total bridge width is 35.36 m (116 ft), allowing for three lanes of traffic and two shoulders in each direction (Fig. 1, 2).

The 204th Street Bridge is a grade separation structure that is part of the redevelopment to make Dodge Street an expressway uninterrupted by traffic signals. Dodge Street, at the bridge

location, carried an average daily traffic (ADT) of 12,080 in 2002 and is expected to carry 40,890 ADT by 2022. The Nebraska Department of Roads (NDOR) is the owner and designer of this bridge. Figure 3 shows plan and elevation views of the 204th Street Bridge.

SYSTEM SELECTION

Several structural systems were considered in the preliminary design stage for the 204th Street Bridge. The first system considered was a three-span continuous girder bridge. In this system, short-span, pretensioned, NU I-girders would have been used in place of the slab spans. Due to the significant cost and time (primarily related to the substructure construction), this option was not chosen. A second option in the structural design of the 204th Street Bridge was to increase the spans of the two side spans and to create a “balanced” span design with haunched steel or concrete girders.² This option proved considerably more expensive than the chosen system.

The third option, designing the bridge using simple, 63-m-long (207 ft), NU2000 I-girders, was found to be feasible if 68.75 MPa (10 ksi)

concrete was used and if an additional thickness of concrete was added to the top flange of each I girder. A single-piece, pretensioned-only beam was also investigated in option four.³ However, the prestressing beds of two local producers did not have the required capacity to stress the ninety 0.6-in.-diameter (15 mm) strands required in each beam. Also, the weight of these girders would have exceeded the maximum handling capacity of one of the two producers, and the length would have exceeded the roadway capacity of the other producer.

The fifth option studied used a single-span steel plate girder.^{1,4,5} Deflection and vibration limits would have required deeper sections than those of the NU2000 girders, which would have had a significant impact on the cost of the ramps. It was, therefore, ruled out.

After investigating all of these options, the designers selected an NU2000 spliced post-tensioned girder (NU2000 PT) with a web that is 25 mm (1 in.) wider than the standard NU2000 pretensioned girder (175 mm versus 150 mm [6.9 in. versus 5.9 in.]).^{2,6} The design called for three girder segments per girder line, with a design concrete compressive strength of 68.75 MPa (10 ksi) and two cast-in-place concrete splice joints. The bridge cross section

consists of seven girders with a 2.55 m (8.4 ft) spacing and topped with a 29.47 MPa (4.3 ksi), 200-mm-thick (8 in.) cast-in-place concrete slab. Each girder line includes two 8.75-m-long (29 ft) end segments and a 45.5 m (149 ft) center segment, as mentioned previously.

DESIGN AND ANALYSIS

The bridge was designed for HS-25 live load according to the *AASHTO LRFD Standard Bridge Design Specifications*.⁵ The bridge design was performed by NDOR and independently checked by a design office in Omaha. The pretensioning force was designed to resist the girder weight and any tension in the concrete bottom fibers during handling. The pretensioning force was then progressively increased to minimize the amount of post-tensioning required, without making the required release strength excessive. All of the post-tensioning was applied before the deck was placed. This is consistent with NDOR's general philosophy of not applying post-tensioning after the deck is placed and cured,¹ reasoning that it may remove the deck at any time without special analysis for girder capacity to handle the previously installed post-tensioning. Other reasons relate to economy: Nebraska has few local post-tensioning installers, one stage post-tensioning would require that the post-tensioning subcontractor be on site fewer times, and once the deck slab and approach slab are placed, access to the post-tensioning anchorages at the girder ends is more challenging.

Commercial computer programs were not relied on during design. Designers at NDOR and the independent design office used spreadsheet programs to complete the design. An internally developed time-dependent analysis program (Creep III) was employed to verify prestressing force losses and determine camber and blocking requirements during construction. One of the most challenging issues was determining the elevations of the girder ends at the wet-cast joints to allow for a smooth top-of-road profile with the least haunch concrete (the concrete directly above the girder flange) and with

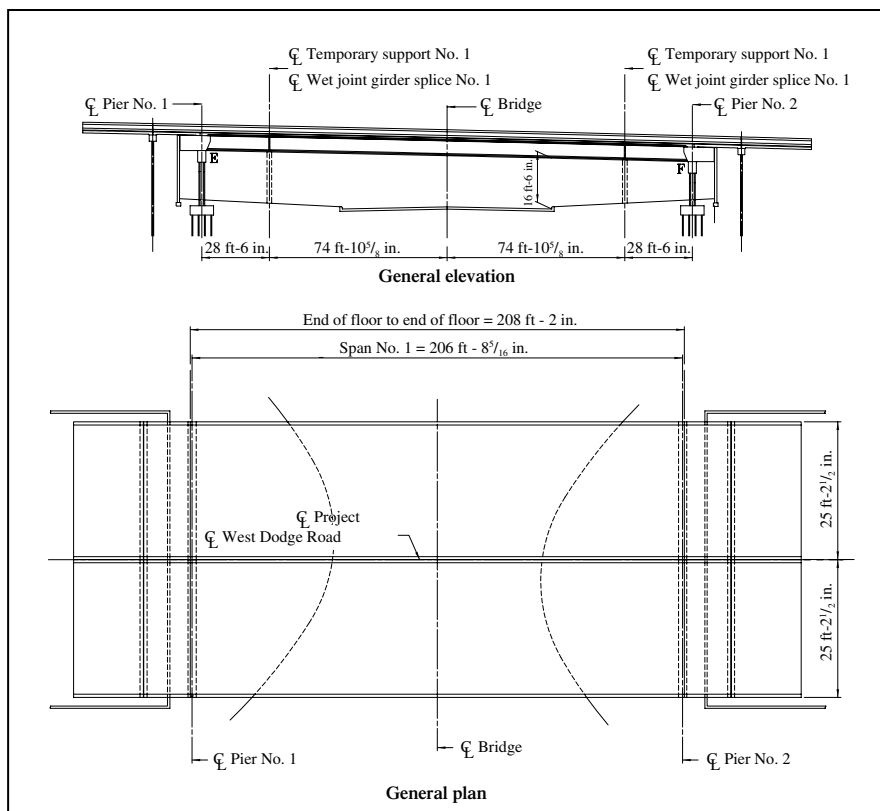


Fig. 3. A general elevation and general plan view. Note: 1 in. = 25.4 mm; 1 ft = 12 in.

no sag in the girder-soffit profile.

During initial checking of the service load stresses, they were found to be acceptable except for the concrete compressive stress caused by the effective prestressing force and permanent loads. For this case, the value of the compressive stress in the top fibers of the concrete girder at mid-span exceeded the $0.4f'_c$ allowed by the AASHTO specifications.⁴ Thus, the designers were required to add 50 mm (2 in.) of concrete thickness to the tops of the girders. However, when the stresses were checked by the Creep III program, which accurately models differential creep and shrinkage and the relieving effects of girder creep in these hypothetical elastic analysis stresses, it was determined that these critical stresses occurred only in the girders' top fibers, only at the girder mid-span, and only at the time of deck placement. The girder stress continues to decrease with time and becomes relatively minor after all prestressing losses. This analysis, along with extensive theoretical and experimental studies previously completed at the University of Nebraska, gave the designers enough confidence to remove the additional 50 mm (2 in.)

of concrete thickness from the tops of the girders, resulting in significant improvement in the economy and applicability of the system.^{7,8} This project was one of the main factors that convinced NDOR to establish a state-wide policy that removed the requirements for two compressive stress limits from design: the limit due to dead load plus effective prestressing force and the limit due to full load plus effective prestressing force. The policy took effect in May 2003.^{1,7}

The strength of the precast concrete girders at the time of transfer of pretensioning and at 28 days were specified as 47.44 MPa (6.9 ksi) and 68.75 MPa (10 ksi), respectively. The strength of the concrete placed in the wet joints between the precast concrete pieces was specified as 41.24 MPa (6.0 ksi) before post-tensioning. A 28-day compressive strength of 29.65 MPa (4.3 ksi) was specified for the cast-in-place concrete deck slab.

The splice joint was designed for zero tension at all times. No special reinforcement was provided in the joint. The flexural strength was determined from the strain capability approach provided in PCI's *Precast Prestressed Concrete Bridge Design Manual*.⁹ This

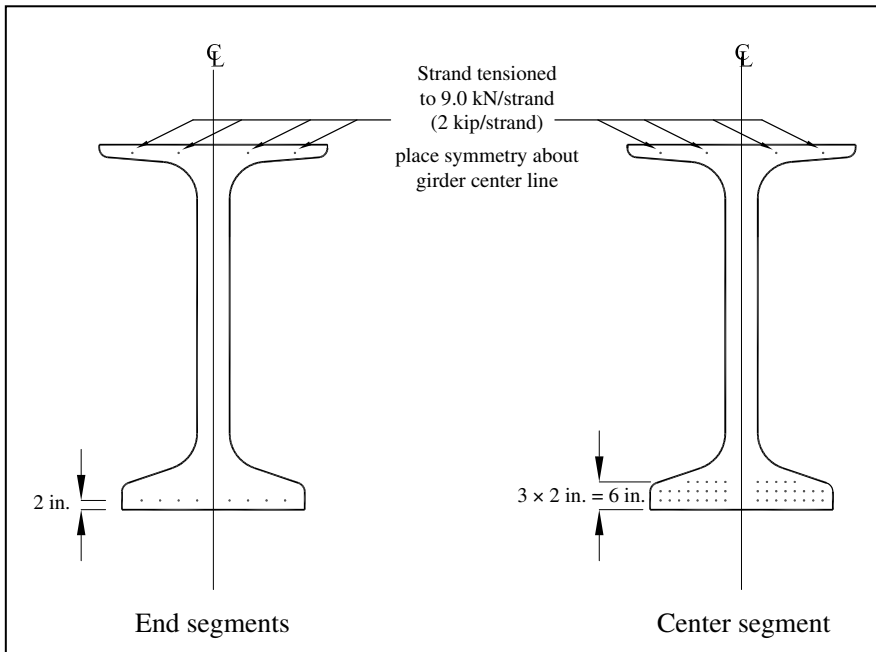


Fig. 4. Pretensioning scheme of the 204th Street Bridge at an end segment and a center segment. Note: 1 in. = 25.4 mm; 1 ft = 12 in.

is the same method that was approved by AASHTO Committee T-10 in April 2005 for inclusion in the 2006 edition of the *AASHTO LRFD Bridge Design Specifications*. This method is more rational and produces more realistic results than the current AASHTO method. Using the current method would have shown the design to be over-reinforced and unacceptable.

Another first in the design of this bridge is its shear capacity. Previous studies and full-scale testing at the University of Nebraska had indicated the NU I-girder's shear capacity to be as great as $0.25f'_c b_v d_v$, where b_v and d_v are the web width and the effective shear depth of the section, respectively.¹⁰ This feature of the NU girder allowed its web width to remain at a standard 175 mm (6.9 in.) without widening, which would have increased the girder's weight. This was possible even with the allowance for the 95-mm-diameter (3.7 in.) post-tensioning duct as required by the code, which reduced the effective width for shear calculations, $b_v = 175 - 0.5 \times 95$, which is only 127.5 mm (5 in.). This maximum shear limit was made possible in the modified compression field theory included in the *AASHTO LRFD Bridge Design Specifications*. Grade 518 (75 ksi) welded wire reinforcement (WWR) was used. The girder design was based

on a typical 414 MPa (60 ksi) WWR strength with the 25% increase in capacity, compared with using Grade 414 (60 ksi) individual mild steel reinforcing bars, considered an added safety margin at no additional cost (Grade 518 [75 ksi] WWR costs the same as Grade 414 [60 ksi] WWR). Extending the girders' vertical shear stirrups into the concrete deck proved adequate for interface (composite action) shear. Shear friction theory was used to check the splice joint shear capacity.

In general, member shear capacity equations are developed through theoretical and experimental research. They are valid regardless of whether the member is being designed for use in a building or bridge and regardless

of whether the code/specifications used for design are ACI 318, AASHTO standard, or AASHTO LRFD. Therefore, in the authors' opinion, it is acceptable to use $0.25f'_c b_v d_v$ as the maximum shear force to be applied to any NU I-girder section as long as that maximum is backed by theory and experiments. The University of Nebraska did extensive full-scale testing to show that this limit is achievable for NU I-girders and with adequate longitudinal reinforcement anchorage as specified by the Modified Compression Field Theory.^{5,10}

The final prestressing in the NU I-girders used in the 204th Street Bridge consists of forty-two 15-mm-diameter (0.6 in.), Grade 1862 MPa (270 ksi) pretensioning strands in the center segment and eight pretensioning strands in the end segments, and three post-tensioning tendons, each comprising fifteen 15-mm-diameter (0.6 in.) strands, run the full length of the bridge. Post-tensioning details are shown in **Fig. 4** and **5**.

Another major innovation in this bridge is use of the optimized post-tensioning end block previously developed by the University of Nebraska using a full-scale experimental program supplemented with finite element and strut-and-tie analyses.^{8,9,11} The end blocks are only 700 mm (28 in.) wide and taper to the standard 175 mm (6.9 in.) web width in a distance of 1000 mm (39 in.), as shown in **Fig. 6**. The weight of this block is only 20% of the weight of a standard *AASHTO LRFD Bridge Design Specifications* post-tensioning end block.⁵ The dimensions of the block were selected to allow housing of the post-tension-

2005 PCI DESIGN AWARDS JURY COMMENTS

"This very innovative project uses spliced-girder technology to create span lengths of more than 200 feet with a span-to-depth ratio of 31.5. It pushes precast concrete girder construction to another level. It is a strong example of how to extend the adaptability of precast, prestressed concrete girders to longer spans in competition with other materials. The cost was only \$58 a square foot."

ing anchorage hardware. It was found that the size of the concrete end block had almost no impact on cracking at the time of post-tensioning. It was the reinforcement that contributed to crack control. A special reinforcement cage is used in the vertical spacing between the three post-tensioning tendons to control vertical splitting. WWR at a narrow 50 mm (2 in.) spacing was used to control horizontal cracking. The details for this custom end block were developed, checked, and full-scale-test verified. The developed end blocks are adequate for up to three post-tensioning tendons comprising fifteen 15-mm-diameter (0.6 in.) strands in a vertical line.

SHIPPING AND HANDLING THE MIDDLE SEGMENTS

Pretensioning of a precast concrete segment is usually provided to sustain a load factor of 1.5 considering the dead load due to shipping and handling. It was found that the factors of safety against cracking and failure due to shipping and handling of the 45.5-m-long (149 ft) middle segments were adequate. No major problems arose with the handling and shipping of the precast concrete girder segments to the project site. Standard lifting loops consisting of strands embedded in the concrete were adequate for handling the end precast concrete segments.

BRIDGE CONSTRUCTION

The 204th Street Bridge was constructed in 2004. The contractor was given the option of assembling the three segments of each girder either on the ground or over the permanent supports and temporary shoring towers. The contractor opted to use the latter. Temporary steel shoring towers were used to support the precast concrete I-girders. After the temporary towers and piers were built, the segments were installed (Fig. 7). A gap of 0.46 m (18 in.) was left between precast concrete girder segment ends for the wet-joint girder splice. Figure 8 shows a side view of the girders after all segments are erected over the supports. Fig-

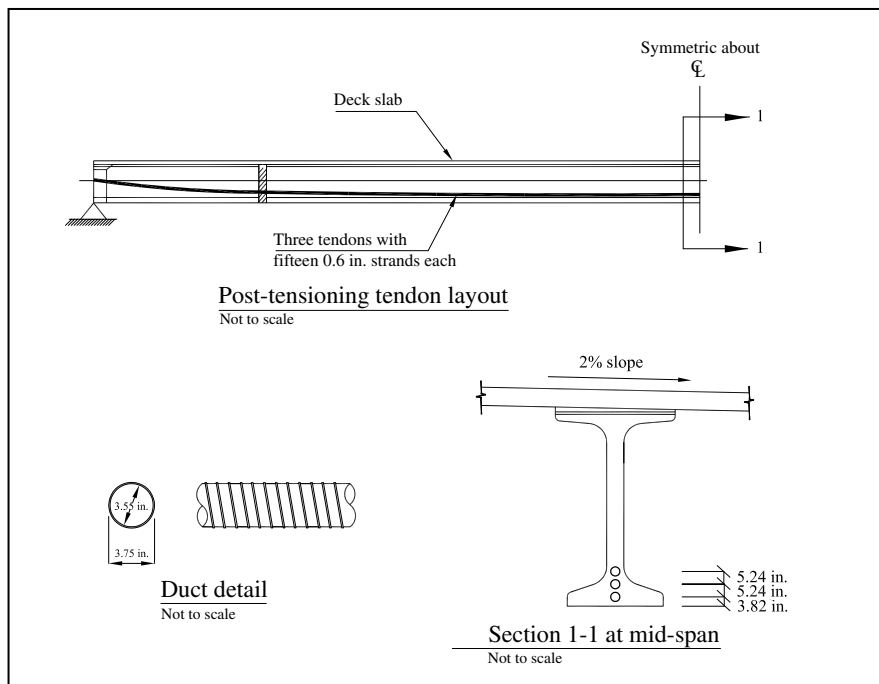


Fig. 5. Girder post-tensioning details. Note: 1 in. = 25.4 mm; 1 ft = 12 in.

ure 9 illustrates the temporary tower and segment joint. Note the screw jacks used to make elevation adjustments to the girders. Intermediate steel diaphragms at the quarter point locations of the center segment are shown in the figures. In addition, the contractor was required to supply temporary diaphragms between the end segments as needed during post-tensioning and before the deck had cured.

Once the segment joints were formed and concrete was placed and had gained

the required strength, the post-tensioning was applied. The applied post-tensioning force was controlled by gauge readings and checked by strand elongation. The strength of the joint was only specified at 41.37 MPa (6000 psi) (not the 68.95 MPa [10 ksi] specified for the girder). This strength was found to be adequate for this location in the span and was believed to be attainable in the field with a conventional concrete. As theoretically calculated, the girders lifted off the temporary towers once the



Fig. 6. Optimized post-tensioning end block.



Fig. 7. Center segment prior to placement over temporary towers.



Fig. 8. All girder segments placed over piers and temporary towers.

post-tensioning was applied. As soon as the post-tensioning was completed, the tendons were grouted to avoid accumulation of moisture and the temporary towers were removed. Finally, concrete for the deck slab and barrier was placed. The bridge was opened to traffic in late 2004. It is already a source of attraction and pride to the community.

BENEFITS OF A SPLICED GIRDER SYSTEM

A number of methods are already available to extend the typical span ranges of prestressed concrete beams. These include the use of high-strength concrete, increased strand size or strength, the lightweight concrete of long-span beams, and splicing of the bridge girders. The simplest and most economical is to use the full-span beam for the bridge. However, limitations are encountered on handling and transportation of long-span beams. The simple-span spliced girder approach adopted for the 204th Street Bridge project presents a feasible alternative to full-



Fig. 9. A close view of a temporary tower and segment joint.

span beam solution. Even though the spliced-beam system is more expensive than a pretensioned-only concrete system, it is generally more cost-effective than a steel plate beam system. Moreover, the spliced-beam system allows for excellent durability and structural performance, low maintenance, and a low-cost bridge structure.

ACKNOWLEDGMENTS

This project, and most other innovative projects in Nebraska in recent years, would not have been possible without the leadership and motivational abilities of Lyman Freemon, state bridge engineer, NDOR. Several others at NDOR worked on the design and detailing of the 204th Street Bridge, including Dennis Boyd and Chad Packard. Mark Traynowicz, Marty Weander, Pat Haeker, and Dennis Vodica of NDOR managed and coordinated the construction of this bridge. Todd Culp and Dennis Drews of Coreslab led the effort in producing the precast concrete girders and in capably responding to challenging field issues. Jim Gregory and Ted Butler of Hawkins Construction Co. were instrumental to the success of constructing this bridge. Emily Lorenz, the *PCI Journal* editor-in-chief, and five anonymous manuscript reviewers provided valuable input relative to contents and method of presentation. The firm formerly known as Tadros Associates LLC had Maher

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REFERENCES

1. Bridge Division. Bridge Operations. 2001. *Bridge Operations, Policies & Procedures* (BOPP). Lincoln, NE: Nebraska Department of Roads.
2. Girgis, A. 2003. *Optimization of Spliced Precast Concrete I-Girder Superstructures*. Ph.D. diss., University of Nebraska-Lincoln.
3. Girgis, A., S. Hennessey, D. Drews, and M. K. Tadros. 2004. Value Engineering of a Spliced Girder Bridge Produces Pretensioned Concrete Records. Concrete Bridge Conference, Charlotte, NC.
4. American Association for State Highway and Transportation Officials (AASHTO). 1996. *Standard Specifications for Highway Bridges*. 16th Ed. Washington, DC: AASHTO.
5. AASHTO. 1998. *AASHTO LRFD Bridge Design Specifications, Second Edition, and 1999 and 2000 Interims*. Washington, DC: AASHTO.
6. Abdel-Karim, A., and M. K. Tadros. 1992. *State-of-the-Art of Precast/Prestressed Concrete Spliced Girder Bridges*. Chicago, IL: PCI.
7. Noppakunwijai, P., M. K. Tadros, and C. Sun. Application of the Strength Design Method for Flexural Members at Prestress Transfer. *PCI Journal*, Vol. 48, No. 5 (September-October): pp. 62-74.
8. Tadros, M. K. 2002. *Compression Limits in Prestressed Concrete Members*. Project No. PR-PL-1(037) P527. Nebraska Department of Roads.
9. Bridge Design Manual Steering Committee. 1997. *Precast Prestressed Concrete Bridge Design Manual*. First Ed. Chicago, IL: PCI.
10. Tadros, M. K. 2001. *Shear Limit of NU I-Girder*. Project No. PR-PL-1(035) P517. Nebraska Department of Roads.
11. Ma, Z., M. Saleh, and M. K. Tadros. 1999. Optimized Post-Tensioning Anchorage in Prestressed Concrete I-Beams. *PCI Journal*, Vol. 44, No. 2 (March-April): pp 56-73.

