Historically, long-span bridges have been the domain of steel plate girder superstructures. Precast concrete girders, except when post-tensioned, have not been able to match, depth for depth, their steel plate girder counterparts. In order to compete with the comparable structural depth of steel solutions, precast concrete design parameters must usually be adjusted by reducing either girder spacing or span lengths. Until the advent of the new precast concrete design presented in this article, concrete superstructure systems were not as cost-competitive as steel alternatives in bridge spans longer than about 150 ft (46 m).

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The successful completion of a replacement bridge at the intersection of U.S. 30 and N-92, near the town of Clarks in central Nebraska, represents the significant engineering benefits and cost savings that value engineering can offer owners and taxpayers. A value engineering study for the Clarks Viaduct bridge replacement resulted in an innovative precast concrete design solution that was chosen over a haunched steel plate girder superstructure, saving $100,000 in project costs. The precast, prestressed concrete alternate comprised a modified University of Nebraska NU1100 I-girder with an innovative high strength threaded rod connection system to create girder continuity for deck weight. This article describes the design considerations, production, and construction of the first high-performance precast concrete bridge to be constructed using non-post-tensioned continuity for deck weight.
One major advantage that steel bridges have had over pre-cast concrete is its ability to make the superstructure continuous before deck slab placement. Steel plate girders also have the advantage of a haunched girder depth in the negative moment region, resulting in high flexural and shear capacities as well as an optimized utilization of materials compared to prismatic (constant depth) members. The value engineering alternate for the Clarks Viaduct in Nebraska represents an attempt to incorporate improvements in precast concrete prismatic beams that are not fully continuous at the piers — without having to resort to post-tensioning or complicated girder geometry (see Fig. 1).

**CONVENTIONAL PRECAST GIRDER SYSTEMS**

Precast, prestressed concrete I-girder bridges represent about one-third of all the bridges built in the United States each year and are generally constructed as simple spans to support the weight of the girder and the cast-in-place deck. Cast-in-place diaphragms and reinforcement in the deck make the superstructure continuous for superimposed dead loads and live loads.

This precast, prestressed bridge system has served the highway program very well over the past three decades. Precast, prestressed concrete bridges are particularly advantageous in states with cold climates, where steel girder expansion joints over concrete piers can create maintenance problems by allowing chloride-contaminated water from deicing chemicals into the joint, causing steel girder bearings to rust and the concrete to crack and spall.

Conventional precast, prestressed concrete girders have limitations in that they are made continuous for only about one-third of the total load, resulting in a high prestressing force and concrete strength. These limitations mean that precast, prestressed girders are not fully used in the negative moment regions, and smaller girder spans and spacings are required.

Smaller span lengths translate into additional substructure support piers, adding to project costs. Pier diaphragms, normally cast prior to the deck, may undergo distress because no negative moment resistance is available as the deck is being formed.
placed. Some bridges built using this type of construction have experienced cracking due to positive time-dependent restraint moments at the piers, especially in girders with high prestress forces.

**PROPOSED THREADED ROD CONTINUITY SYSTEM**

With the use of a new continuity system developed by Maher K. Tadros,\textsuperscript{3,4} presented here, precast concrete girders are coupled over the piers using $1\frac{1}{8}$ in. (35 mm) diameter, Grade 150 ksi (1035 MPa) threaded steel rods before the deck weight is applied. The major advantages of this system are that the precast concrete girders are made continuous for about two-thirds of the total load, while the threaded rod system establishes continuity over the piers and resists the negative moment due to deck slab weight (see Fig. 2).

After the deck concrete has hardened, the deck reinforcement and the high strength threaded rods resist the negative moment due to superimposed dead load and live load. Continuity reinforcement in the slab is still provided for superimposed dead load and live loads. Spans can be increased by 10 to 15 percent within a given precast concrete girder size. More importantly, bridge performance is improved as the negative moment due to deck weight more than offsets the positive restraint moments due to time-dependent effects. This reduction in positive restraint moments results in less cracking in the pier diaphragms, longer service life, and lower maintenance costs.

**VALUE ENGINEERING**

The Clarks Viaduct passes over Union Pacific railroad tracks (UPRR) and U.S. 30, a major highway in central Nebraska (see Fig. 3). Located outside Clarks, Nebraska, about 80 miles (130 km) west of Omaha, the Clarks Viaduct is a four-span replacement bridge with span lengths of 100.0, 151.0, 148.0, and 128.5 ft (30.5, 46.0, 45.1, and 39.2 m); total bridge length is 527.5 ft (160.7 m) (see Fig. 4). The replacement bridge was originally designed as a haunched steel plate girder bridge and let for bid with the construction contract being awarded to Hawkins Construction Company of Omaha, Nebraska.

After the construction drawings were released, Tadros Associates started working with the contractor on a unique precast concrete alternate that would result in considerable cost savings and better overall structural performance. The design was proposed to the contractor, who in turn submitted a value-engineering proposal to the Nebraska Department of Roads (NDOR) for approval. After the proposal was accepted by NDOR, Hawkins Construction directed Tadros Associates to develop the necessary construction documents for the precast concrete alternative.

**PROJECT CONSTRAINTS**

The primary project constraint from the NDOR was that no time extension of the construction schedule would be allowed; this meant that the redesign and production of construction documents had to be completed in about eight months. The design was submitted to the contractor and accepted by NDOR, and the construction documents were prepared in a timely manner.

Table 1. Project schedule.

| Project let for construction | February 2002 |
| Design of precast concrete alternate | February 2002 |
| Acceptance of value engineering proposal by NDOR | March 21, 2002 |
| Start of production | August 12, 2002 |
| Completion of production | September 5, 2002 |
| Start of construction | October 2002 |
| Completion of project | Summer 2003 |

Fig. 3. The Clarks Viaduct passes over Union Pacific railroad tracks and U.S. 30, a major highway in central Nebraska.

Fig. 4. General sectional elevation of the Clarks Viaduct.
Fig. 5. Elevation of viaduct. Note non-negotiable minimum clearance over railroad tracks of 23 ft 6 in. (7.2 m).
weeks (see Table 1). Other project constraints were that the new design could not change the profile grade line, minimum vertical clearance limits of 23 ft 6 in. (7.2 m) over the UPRR tracks, or bridge width from the original steel design. Furthermore, the NDOR would not allow the bridge piers to be relocated (see Fig. 5).

Despite these constraints, Tadros Associates was able to provide a precast concrete solution that fit within the profile of the original steel design (see Fig. 6). The original steel design consisted of four lines of varying depth steel plate girders with beam spacings of 10.75 ft (3.3 m). The depths of the plate girders at midspan of Spans 1, 2, 3, and 4 were 50.25, 51.13, 51.13, and 51.00 in. (1276, 1299, 1299, and 1295 mm), respectively. The maximum depth of the haunched plate girder section over the piers was 75.50 in. (1918 mm).

A 50 in. (1270 mm) deep precast concrete girder was chosen to match the minimum steel section (see Table 2). The concrete girder was a modified [Nebraska University (NU)] NU1100 43.3 in. (1100 mm) I-girder with 7 in. (178 mm) of depth added to the top of the girder (see Fig. 7). The girder top flange width was narrowed from 4.0 to 2.0 ft (1.2 to 0.6 m) in order to reduce girder weight. The deck consisted of an 8 in. (203 mm) slab with a 28-day strength of 4000 psi (27.6 MPa). The concrete of the modified NU1100 girders had a release strength of 6500 psi (44.8 MPa) and a 28-day strength of 8500 psi (58.6 MPa).

### Table 2. Precast concrete inventory for the Clarks Viaduct project.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Number</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified NU 1100 precast prestressed I-girders</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Span 1</td>
<td></td>
<td>36.7 tons</td>
</tr>
<tr>
<td>Span 2</td>
<td></td>
<td>52.9 tons</td>
</tr>
<tr>
<td>Span 3</td>
<td></td>
<td>51.7 tons</td>
</tr>
<tr>
<td>Span 4</td>
<td></td>
<td>48.4 tons</td>
</tr>
<tr>
<td>Longest high strength threaded rod</td>
<td></td>
<td>230 lb</td>
</tr>
<tr>
<td>Longest precast concrete girder</td>
<td></td>
<td>105,800 lb</td>
</tr>
<tr>
<td>Volume of self-consolidating concrete</td>
<td></td>
<td>376 cu yd</td>
</tr>
<tr>
<td>Total lineal feet of precast girders</td>
<td></td>
<td>1854 ft</td>
</tr>
</tbody>
</table>

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 cu yd = 0.7646 m³; 1 ton = 0.907 Mg; 1 lb = 0.4536 kg.
DESIGN ISSUES

Because the precast concrete girders were designed to be continuous for deck weight, typical available bridge design software could not be used. Instead, the analysis was performed using spreadsheets and RISA 2D, a continuous frame analysis program. An Excel spreadsheet was developed to determine the number of pretensioning strands required to meet the allowable concrete tension stresses. This analysis incorporated the AASHTO LRFD Bridge Design Specifications lump sum prestress losses and transformed section analysis for stress computations.5

Once the number of strands was determined for each span, the ultimate flexural capacity in the positive moment region was calculated using the ultimate flexural strength equations in the AASHTO Standard Bridge Design Specifications.6 The negative moment capacity of the system was determined using the strain compatibility approach. Strain compatibility was used because it accounts for the variable material properties of the Grade 150 (1035 MPa) threaded rods and the Grade 60 (415 MPa) mild steel reinforcement in the deck slab.7 The software program CONSPAN LA, from LEAP, was used to calculate camber.

Further complicating the design was the bridge skew of 52 degrees (see Fig. 8). In girder bridges, as with all other structures, bending moments and shear forces tend to follow the stiffest members—which, in the case of the Clarks Viaduct, are the girders. In slab bridges, the moment and shear forces tend to take the “shortest” route to supports, which are the obtuse corners of a span; this structural behavior also occurs in girder bridges, but is not as pronounced as in slab bridges.

Frequently, this structural behavior leads to a distribution of load that is different from that assumed in the design. The longitudinal bending moments tend to be reduced, but the shear force is increased in the obtuse corners. One way to minimize the effect of skew in the middle spans is to remove the transverse pier diaphragms. Pier diaphragms are typically used with concrete superstructures to tie adjacent spans together, connect the girders to the deck, and connect the superstructure to the substructure.

One problem with using pier diaphragms on bridges with heavy skews is that pier diaphragms create rigid supports, acting to “lock down” the deck slab and prevent transverse deck rotations at piers. By removing the transverse diaphragms, the deck slab can rotate more freely at the piers, allowing the bending moment and shear forces to more closely follow the longitudinal girder lines.

A second problem with using transverse pier diaphragms is that the stiffness of the diaphragm forces the superstructure to rotate about the major axis of the pier. This assumption conflicts with the common design assumption that the superstructure rotation occurs along the longitudinal axis of the girders. To eliminate these problems, the Clarks Viaduct was designed without transverse pier diaphragms.

A particular design consideration was that of shear. With this new superstructure system, shear is more critical than in conventional precast girder systems because of the increased span lengths and girder spacings. The AASHTO LFRD shear limit of $V_s / \phi \leq (0.25 f'_c bd_v + V_p)$ was used in calculations.

Fig. 8. Support piers with haunched girders reflect the 52-degree skew in the bridge orientation.

The tension tie provided by the negative moment reinforcement allows for this limit to be reached.

Using a prismatic concrete girder section along the span and assuming pinned supports over the piers would result in significantly high negative moments. Accordingly, the concrete stress in the girder bottom flanges at the piers would require unacceptably high concrete compressive strength. To resolve this design challenge, a unique haunched concrete section was created over the pier (see Fig. 9).

Fig. 9. The haunched concrete beam was designed to reduce concrete stresses due to negative moments at the piers.
The proposed system carries with it the following advantages:

1. For the same span and spacing:
   - Prestressing force is reduced by about 15 percent.
   - Required concrete strength is reduced.
   - Girder size can be reduced.
2. For the same prestressing force and concrete strength level:
   - Span capacity can be increased by about 15 percent.

3. Improved structural performance:
   - Negative moment due to slab weight exceeds any possible positive creep restraint moment.
   - Distress at pier diaphragm is eliminated.

**PRODUCTION**

The prestressed concrete girders were fabricated at Concrete Industries, Inc., in Lincoln, Nebraska, about 50 miles (80 km) southeast of the construction site. At the beginning of the value engineering process, Concrete Industries was contacted about the possibility of modifying a standard NU1100 girder. Tadros Associates wanted to know what girder configuration was possible using existing plant equipment.

**Modified NU Girder**

The first design consideration for the precaster was to use the standard NU girder shape and modify the top flange by making it narrower and thicker. The production objective was to keep the same weight in the modified girder as that of the standard girder and to use existing form yokes for economy. After analyzing the modified design, the precaster informed Tadros Associates that it was possible to use their existing 10 in. (254 mm) magnetic side forms.

The side forms could be placed on the standard girder form so that the top yokes would just clear the side form. In order to accurately locate the top tension rods, the precaster fabricated new one-piece header plates using a computer operated pattern torch.

**Threaded Rods and Self-Consolidating Concrete**

To provide top continuity, the precaster needed to consider the availability and lengths of \( \frac{1}{4} \) in. (35 mm) high strength...
threaded rods (see Figs. 10 and 11). At 45 ft (13.7 m), the longest threaded rod would weigh about 230 lb (104 kg). Because the designer specified 6500 to 7000 psi (45 to 48.3 MPa) for concrete release strength, self-consolidating concrete (SCC) was selected.

Concrete Industries has had previous experience using SCC on NDOR projects, and only recently two more projects employing SCC have been approved by NDOR and Tadros Associates. However, it should be mentioned that great care must be exercised in producing SCC.

First, it is important to monitor the moisture content of the aggregates, as small changes in total water content in the SCC mixture can cause adverse variations in the final product. Second, the fresh concrete must be closely monitored to ensure it does not segregate. Notwithstanding these issues, the advantages of using SCC far outweigh any potential problems.

SCC allows for easy, uniform concrete placement in the form without mechanical vibration and facilitates screeding operations. More importantly for precast operations, concrete casting time can easily be cut by 50 percent with SCC. Finally, the concrete’s surface finishes are exceptionally smooth. Average strength results for SCC at the precast plant were 7320 psi (50.5 MPa) at release and 10,140 psi (70 MPa) at 28 days.

Specifically, the SCC mixture incorporated 632 lb (287 kg) of Type III cement from Lafarge Cement Company, 100 lb (45 kg) of fly ash from Nebraska Ash Company, 1480 lb (671 kg) of limestone from Kerfords Limestone Company, and 1339 lb (607 kg) of sand from Western Sand and Gravel. The water-cement ratio was 0.393, and the air content was 5.5 percent. Additives from Degussa Admixtures, Inc., included an air entrainer, a water reducing retarder, an ultra-high-range water reducer, and a viscosity modifying agent.

Production began in August 2002 and was completed less than one month later (see Table 1). Sixteen components were cast in eight pours, casting every other day. Over 1850 ft (560 m) of precast girders were fabricated, consuming over 370 cu yd (75 m$^3$) of self-consolidating concrete.

Once production started, it took little time to accurately place the $\frac{3}{8}$ in. (35 mm) diameter, 45.0 ft (13.7 m) long rods. Other than the precise rod placement, production on all the other elements proceeded very efficiently. By the end of this job, the amount of labor per foot for the modified NU girder was the same as that required for the standard NU1100.

**CONSTRUCTION OF NEW HAUCHED SECTION**

Hawkins Construction Company, of Omaha, Nebraska, was able to use its existing equipment for the erection of the Clarks Viaduct. Two cranes were used to handle the long prestressed concrete girders. The haunched section was produced by first casting a haunched beam over the top of the pier cap (see Fig. 12).

The haunched beam provided extra superstructure depth at the piers and moved the girder ends closer to the structure’s dead load inflection points, thus reducing the concrete stress due to negative moment at the girder bottom flanges. The 28-day strength of the haunched beam concrete was 5000 psi (34.5 MPa). Each haunched beam was 4.0 ft (1.2 m) wide, 25 ft 8¼ in. (7.8 m) long, and tapered in height from a minimum thickness of 2.0 ft (0.6 m) at the centerline of pier to 6 in. (152 mm) at the beam ends.

Once the haunched beam achieved its required strength,
the girders were placed as shown in Fig. 13. The ends of girders overlap 2 ft (0.6 m) onto the haunched beam. The four 1\(\frac{3}{4}\) in. (35 mm) diameter high strength steel threaded rods embedded in the girders were then coupled using two Grade 50 (345 MPa) rectangular steel bars and five Grade 150 ksi (1034 MPa) threaded rods (see Figs. 2 and 12). The rectangular steel bars were designed to carry the ultimate tensile force of the four rods embedded in the girder.

After the threaded rods were connected, concrete was placed over the entire area of the haunched beam up to the top of the girders. This concrete section is called the pier block (see Fig. 14). Its 28-day strength compressive strength was 5000 psi (34.5 MPa). After the pier block concrete achieved the required strength, the deck was cast. The bridge construction was completed in July 2003 (see Fig. 15).

The haunched beam was designed to act compositely with the pier block as well as the deck slab to resist the negative moments due to deck slab weight, superimposed dead load and live load. Before the pier block concrete was placed, the haunched beam had to resist the weight of the girder and the wet concrete from the pier block. For this reason, 14 No. 8 (25 M) bars were used to reinforce the haunched beam during erection.

To ensure the haunched beam’s stability during girder placement, a moment connection had to be made between the haunched beam and pier cap. A moment connection between superstructure and substructure is not common in conventional bridge design. Typically, the only loads transferred to the substructure are vertical loads and lateral loads. Since the haunch section was rigidly connected to the substructure, a frame analysis was performed to determine the reactions in the pier.

Due to the magnitude of dead and live load moments, the vertical column reinforcement was increased from 20 No. 8 (25 M) bars in the original steel bridge design to 26 No. 11 (36 M) bars, while the 3.0 × 3.0 ft (0.9 × 0.9 m) dimensions of the column remained the same. To help support the haunch during girder placement, No. 5 (16 M) steel bars were placed around the pier cap.

Because of the moment connection, the maximum column moment occurred at the intersection of the superstructure and substructure at the top of the pier cap. As a result, 26 No. 11 (36 M) vertical column steel bars were extended through the pier cap, cast-in-place haunched beam, and into the pier block (see Fig. 16). Extending the column reinforcement created another challenge in that these bars had to pass through intersecting bars from the pier cap, as well as through the 14 No. 8 (25 M) bars in the haunched beam and the No. 5 (16 M) longitudinal steel bars in the pier cap itself.

Because of the tight tolerances, steel template plates were used to help the contractor in the accurate placement of the vertical column bars. Three of these steel plates were used in construction: one was placed in the pier cap, another in the haunched beam, and the third placed in the pier block. The steel templates in the haunched beam and pier block were welded to the vertical bars in order to anchor the vertical reinforcement.

**Contractor Benefits**

One of the main benefits of the continuity connection used on the Clarks Viaduct is its simplicity. Tadros Associates has received feedback from local contractors indicating they face disadvantages in projects requiring post-tensioning. Although post-tensioning is structurally efficient and offers some other benefits, it is significantly more expensive and requires specialized labor during jacking and grouting. In Nebraska and in many Midwestern states, it is difficult to get specialized personnel and equipment out to the job site. Many job sites are in rural areas, sometimes hundreds of miles from major airports.

To arrange for specialized personnel to go to remote job sites, the contractor must pay travel expenses, room and board, and extra labor hours. These costs can be further inflated when there are project delays or inclement weather. The advantage to the contractor in the continuity connection used on the Clarks Viaduct was that special personnel were not required; the contractor only had to assemble a few precast components and cast a pier block to establish the connection using existing local labor.

Hawkins Construction reported that the new modified girder design accelerated the schedule. Construction went smoothly with the exception of one aspect: the contractor did not anticipate some problems encountered at the thickened haunches over the piers with excessive girder pockets. Because of the girder pocket configuration, conventional
decking required that different hangers be used in the deck-
ning operation.

In hindsight, the contractor would have opted for a stay-in-
place deck over a conventional deck to facilitate placement
of the deck concrete over the girder pockets (see Fig. 17).

Hawkins Construction was pleased to be part of this develop-
ing technology and believes its future use will be successful
in advancing the precast concrete industry because it allows
the spans of precast concrete bridge girders to be extended
further than before, saving on pier construction costs.

CONCLUSIONS

In the value engineering alternate of the Clarks Viaduct,
Tadros Associates has demonstrated that it is now technically
and economically feasible for precast concrete superstructure
solutions to compete with structural steel in long-span bridge
design. The concept described in this article can be used with
I-girders, inverted tee beams, and box beams.

The design eliminates concerns about positive creep re-
straint moments at the piers. Not only will the precast con-
crete superstructure result in long-term savings—in terms of
lower maintenance cost—but also will provide immediate
benefits to the owner and contractor in terms of lower con-
struction costs.

The overall construction savings in the Clarks Viaduct
project was approximately $100,000 over that of the initial
steel girder alternative (see Fig. 18). This savings was divid-
ed between Tadros Associates, LLC, Hawkins Construction
Company, and the State of Nebraska (NDOR).

In recognition of the design-construction team’s inventive-
ness, originality, and forward-thinking in developing its solu-
tion, the Clarks Viaduct project won a 2004 PCI Design Award
in the category of “Best Bridge with Spans Greater than 135
Feet.” At the judging, the bridge jury commented: “Value
engineering resulted in the development of an extremely in-
novative design for this project. The use of haunched beams
at the piers and the implementation of full continuity of the
girders for both deck and live loads make for a very efficient
solution. We believe many more structures will be built like
this in the future. This concept extends the applicability of
prestressed concrete girders. Other designers will be able to
apply this technique in a number of other ways. This solution
pushes the limit on slenderness and girder spacing.”

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Fig. 17. Placement of 8 in. (203 mm) thick concrete deck on
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Fig. 18. A traveler’s view of the Clarks Viaduct in service.

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