Spliced Bulb-Tee Girders Bring Strength and Grace to Pueblo’s Main Street Viaduct

The new $7.6 million Main Street Viaduct in Pueblo, Colorado, is a six-span precast, prestressed concrete bulb-tee girder bridge, spliced together with post-tensioning. The overall length of the structure is 807 ft (246 m) with a maximum central span of 174 ft (53 m). There are eight girder lines with girders varying in depth from 72 in. (1829 mm) at the ends to 96 in. (2438 mm) at the center, creating an attractive haunched effect. The project was constructed within the designated 12-month time frame and was under budget. This article discusses the design alternatives and structural design considerations. Production, transportation and erection of the precast concrete components are also highlighted.

The city of Pueblo, Colorado, has a rich heritage going back to the era when mining, railroads and steel mills provided the main livelihood for its people. With a current population of 100,000, Pueblo is well known for its inherent natural beauty, vibrant history, cultural diversity and classic buildings.

In the last decade, however, with the downsizing of the steel industry, the citizens of Pueblo have been forced to search for alternate avenues of economic development. Fortunately, in the past several years, tourism has been growing and retirement communities are on the rise. Beyond that, however, there is a spirit of revitalization in the city and a civic renaissance.

This refocus in the city’s outlook has raised the awareness of the livability of the urban area, and projects that enhance and beautify the town are given the highest priority. As a symbol of a new future for this revitalized city, a new bridge was to be built to replace an old deteriorated steel truss structure, which did not meet the high standards of the community.

Named the Main Street Viaduct, the bridge crosses over 12 railroad tracks, the Arkansas River and “B” Street. Because of the bridge’s significance, function, aesthetics and cost became extremely important requirements.

Several challenges were presented to the planners in the preliminary design phase:
Fig. 1. Main Street Viaduct, Pueblo, Colorado. This award winning spliced precast, prestressed concrete bulb-tee girder bridge is only the second structure of its kind to be built in the state of Colorado.

- The old bridge would have to be demolished without disrupting existing facilities or polluting the environment.
- The new bridge would need to be in harmony and complement the adjacent structures.
- Delicate negotiations were to be conducted with various railroad officials and other agencies with regard to right-of-way and financial participation.
- The bridge would need to meet strength requirements set by AASHTO and the Colorado Department of Transportation (CDOT).
- The long-term durability of the structure and maintenance costs were an important issue.
- The bridge had to be built on a fast construction schedule.
- Lastly, but most importantly, the structure was to be constructed within a tight budget.

LONCO, Inc. (the Engineer of Record), Denver, Colorado, was commissioned to assess potential structural systems for the replacement bridge. This included a steel girder option, a precast I-girder option and a third option, namely, precast, prestressed concrete bulb-tee girders spliced together with post-tensioning.

The advantages of this latter system are structural efficiency, guaranteed quality control, a fast track construction schedule, long-term durability and economy.

Comparative costs of the various structural systems were prepared and the advantages of the precast bulb-tee system soon became apparent. Because of the overall geometry and alignment, the railroad right-of-way limits, and the complex and skewed configuration of railroad tracks beneath the structure, the opportunity to place a pier in this location was restricted to only one spot. This created the need to span long distances without shoring and to skew the railroad pier, resulting in two spans with varying length girders.

With this substructure arrangement, a traditional system of precast I-girders spanning pier-to-pier was investigated. However, the girders would have to have been spaced so closely [20 girder lines in an 80 ft (24.4 m) wide structure] that the scheme was considered unreasonable from a cost standpoint.

A steel structure was also investigated, but following a life cycle cost study, the city found the precast concrete structure to be a more durable system with a lower long-term maintenance cost.

The new Main Street Viaduct (see Fig. 1) is a six span, bulb-tee girder bridge spliced together with post-tensioning. There are eight girder lines with girders varying in depth from 72 in. (1829 mm) at midspan to 96 in. (2438 mm) at the piers, thus creating an attractive haunched effect. The overall length of the structure is 807 ft (246 m) and the largest span at the centerline of the bridge is 174 ft (53 m). Fig. 2 shows the plan and elevation of the bridge.

In addition to the bulb-tee girders, permanent precast, prestressed deck panels were used instead of conventional forming. The stressing of the post-tensioning tendons in the girders was done in three phases: first, to connect the precast girder segments; next, to apply additional force to the girders after placing the deck panels; and finally to stress the composite section after the final deck topping and diaphragms were cast. Thus, the deck...
Fig. 2. General layout plan and elevation of bridge.
Fig. 3. Typical section and slab plan of bridge.
Fig. 4. Precast bulb-tee girder details.
panels played an important role in supplying dead load to overcome stresses from the second phase of post-tensioning.

The preliminary and final design phases of the bridge occurred from May 1993 to August 1994. The project was let out for bids on September 20, 1994, and the bridge was completed in December 1995.

The total cost of the bridge project (including ancillaries) was $7.6 million. The new bridge is only the second of its kind to be built in the state of Colorado and the first in the Pueblo area.

**DESIGN ALTERNATIVES**

The Structural Engineer, Finley McNary Engineers, Inc., Denver, Colorado, performed the analysis of the structure and assisted in the detailed structural design of the precast, prestressed concrete girders.

Preliminary design work for the project began in May 1993, with the final design beginning in December of that year. A number of different structure types and span arrangements were considered in the conceptual phase of the project. Due to the restrictions of the river dike and the numerous rail tracks, there were not many options for span layouts. The two basic choices were a long span in the 250 to 300 ft (76.2 to 91.4 m) range over the Arkansas River with shorter spans in the railyard area (due to the limited structure depth from the vertical clearance available) and the chosen layout.

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**Fig. 5. Typical prestressing strand layout of bulb-tee girder.**

**Fig. 6. Girder segment layout showing stressing sequence for Girders D, E, F, C, B, G, H, A.**
Fig. 7. Bridge erection Sequence 1.
Fig. 8. Bridge erection Sequence 2.
The longer span layout was not economical because it saved only one pier and because of the constructability impacts of erecting a long span in a difficult access area. Once the approximate span layout was determined, the only feasible structure types were steel plate girders, standard precast concrete girders, and spliced and post-tensioned precast bulb-tee girders. Due to the very close spacing required of the standard girders, they were not as economical as the other two options and were discarded.

The steel plate girders were estimated to be slightly higher in initial cost than the spliced bulb-tee girders. However, once the life-cycle cost analysis including maintenance was done, and the aesthetic preferences of the client were taken into account, spliced concrete girders were chosen as the recommended alternate for final design.

The first spliced and post-tensioned girder bridge in Colorado was designed by the Colorado Department of Transportation (CDOT) Staff Bridge section. The spans were 164, 183 and 164 ft (50, 55.8 and 50 m). The piers were skewed at approximately 45 degrees and it spanned over I-76 and a railroad track just north of Denver. Therefore, accurate price information was available on a bulb-tee system.
and the cost estimate was thought to be on target. The owner felt comfortable with this bridge type because a similar structure had recently been built successfully.

**DESIGN CONSIDERATIONS**

The following considerations had a major impact on the design and layout of the spliced bulb-tee girder design:

**River Pier Locations**

The piers near the Arkansas River were important in determining the structural type. It was desirable to keep the piers out of the river as much as possible and also to minimize the impact to the dike on the north side of the river for pier construction. Therefore, a main span of about 175 ft (53.3 m) was selected, which put the south pier at the edge of the river during spring and summer (while out of the water during the winter months when construction would take place) and the north pier at the north toe of the dike. This span length fit well with a spliced bulb-tee girder design using 72 in. (1829 mm) girders and haunching to 96 in. (2438 mm) at the piers.

**Railroad Restrictions**

The railroad owners had several restrictions concerning the new bridge and its construction. First, the standard horizontal clearance to the pier walls and the temporary clearance to the foundation excavation limited the pier locations for the northern half of the bridge. The alignment of the tracks (curving with a tangent at about a 66-degree skew) further limited the placement of the piers. Due to the limited vertical height available, spanning the railroad was not even an option. There was basically only one location for Pier 4 within the sets of tracks that met the criteria and Pier 5 was located just beyond the last track.

Another critical factor was the restriction against falsework. Again, due to the closeness of the track spacing, there was no space available to place falsework for girder support prior to completing the splices. Therefore, the girder layout and erection sequence were designed to allow for overhead

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![Fig. 12. Side view of newly fabricated bulb-tee girder.](image1)

![Fig. 13. Bulb-tee girders were hauled using precaster's standard tandem axle units.](image2)

![Fig. 14. Special precautions were taken by the precaster when hauling the lengthy bulb-tee girders in congested city streets.](image3)
strong-back beams to be used. The final factor was the impact of construction on train scheduling. The tracks are fairly active and the design allowed the contractor to place the girders in the time windows allowed by the train schedules and still be able to move the cranes out of the way when trains passed.

**Vertical Clearance at Railyard**

The vertical profile of the bridge had been set in the preliminary phase and due to the existing touchdown points of the bridge, a fairly steep grade was required at the north end of the bridge to obtain the minimum vertical clearance over the tracks for a structure depth using the 72 in. (1829 mm) deep girders. When the last survey was done for final design and the railroad made its final decision on pier locations, the span layout had changed from 150, 175, 152.5, 152.5 and 91 ft (45.7, 53.3, 46.5, 46.5, and 27.7 m) in preliminary design, to the final layout of 152, 174, 164.5, 155.5 and 88 ft (46.3, 53, 50.1, 47.4, and 26.8 m).

Given the skew angle of Pier 4, Span 3 actually varied from 180 to 149 ft (54.9 to 43.7 m) and Span 4 varied from 171.3 to 140 ft (52.2 to 42.7 m). Due to the vertical clearance restriction over the tracks discussed above, the increase in Span 3 became critical because additional structure depth was not possible. The problem was solved by using the standard bulb-tee end block section [2 ft (0.61 m) web width] over Piers 4 and 5. This provided the required flexural and shear capacity without requiring an increase in structure depth or new forms for the precaster.

**Girder Lengths and Weights**

During the design of the bridge, it was apparent that the girders would probably be cast in Denver, approximately 120 miles (193 km) north of Pueblo. Therefore, the shipping length and weight of the girders were important. This also played an important role in the layout and design of the girders, as the assumed limits for design were 175,000 lbs (79380 kg) and 160 ft (48.8 m).
STRUCTURAL SCHEME

The basic structural scheme consisted of precast bulb-tee girders that were spliced in place and post-tensioned. Various structural details of the bridge are shown in Figs. 3, 4 and 5.

The girders were 72 in. (1829 mm) deep and haunched to 96 in. (2438 mm) deep at Piers 2 and 3. The girders contained pretensioned strands in the top and bottom of the girders and three continuous post-tensioned tendons with 19 0.6 in. (15.2 mm) diameter strands in each. The erection scheme is shown in detail in Figs. 6, 7 and 8.

Some of the key features of the design include:

1. Only two temporary bents were required for erection. All other supports were provided by overhead strong-back beams. The beams were attached to one girder at the precasting yard and then placed onto the previous girder for support.

2. At two critical areas of the structure, the standard end block section was used over Piers 4 and 5 for the required additional flexural and shear capacity. As stated above, no height was available for additional structure depth. At Pier 4 (Girder Segment 5), the section extended 15 ft (4.57 m) to each side of the pier and 17.5 and 7.5 ft (5.33 and 2.29 m) at Pier 5 (Girder Segment 7). In each case, the girder cantilevered beyond the pier and the closure joint consisted of the end block section, while the drop-in girder (Girder Segment 6) contained the end block transition section.

3. The closure joints also consisted of 1 ft 4 in. (406 mm) wide, full depth diaphragms between the girders. All joints, except that near Pier 4, were normal to the bridge centerline. At the skewed Pier 4, the joint was a constant 15.5 ft (4.72 m) from the pier and the diaphragm consisted of two walls in each bay and stair-stepped across the bridge. The joints and diaphragms were reinforced with mild reinforcing steel. The closure joint also consisted of conventional mild reinforcing steel, which provided moment capacity across the joint prior to stressing the tendons. This was critical due to the erection sequence, which minimized the number of falsework bents and thus required some continuity across the joint prior to placing the next girder.

4. The erection sequence consisted of three phases of stressing the longitudinal post-tensioning. This was required in order not to overstress the
Fig. 21. After stressing the deck panels, a cast-in-place topping was placed.

LONGITUDINAL GIRDER DESIGN

The longitudinal girder analysis and design was done using a two-dimensional frame program with time-dependent analysis capabilities. The program’s name is Bridge Designer II and it is also capable of easily analyzing a structure that is constructed in many phases. In addition to the normal frame program input of nodal coordinates, section properties, element definitions and applied loads, this program also allows time-dependent material properties, prestressing tendons, and multiple erection phases by date.

The details of the erection sequence are given in Figs. 6, 7 and 8 but the sequence is basically as follows:

1. Erect the precast girder sections in pieces.

To be stressed without overstressing the girders.

All of the tendons extended the full length of the bridge [approximately 740 ft (226 m) with major vertical deviations] and were stressed from each end. The tendons were installed and stressed by AVAR Construction Systems, Inc. from California and no problems were encountered.

girders during construction. Each tendon duct was coupled across the closure joint between the girder pieces. The first phase consisted of stressing the first tendon after the closure joint had been cast. The second tendon was stressed after the precast deck panels had been placed on the girders. The last tendon was stressed on the composite girder and deck section after the cast-in-place deck slab had cured.

The precast deck panels served two purposes for this project. First, they allowed the deck to be placed with a minimum amount of work occurring over the rail tracks (i.e., minimum labor in setting the form panels and no labor required to remove the permanent forms, as would be required for standard forms). This was very desirable to the railroad because it also minimized the possibility of dropping anything on the tracks below. Secondly, the panels provided an intermediate step of adding the weight of the deck slab, allowing the second tendon
2. Cast closure joints and diaphragms.
3. Stress the first tendon.
4. Place the precast deck panels.
5. Stress the second tendon.
6. Cast the deck.
7. Stress the third tendon.
8. Cast barriers and place wearing surface.

Each of the above steps is modeled in the analysis and the time schedule is also taken into account. The girders are made composite with the deck by connecting the girders to the deck elements using vertical rigid links. The centroids of both elements are offset by the correct amount to result in the proper composite section property.

Due to the skew of Pier 4, the span lengths of all the girders vary in Spans 3 and 4. Therefore, a separate analysis was made of the girders furthest north and south and a hypothetical girder at the structure centerline. The information was then interpolated to design all of the girders.

The General Contractor for the project was Lawrence Construction Co., Littleton, Colorado.

The total contract amount between the owner and Lawrence Construction was $6,106,555. The total contract amount to fabricate, deliver and erect the precast concrete products was $1,076,830.

**PRODUCTION HIGHLIGHTS**

The precast, prestressed concrete components were fabricated at Rocky Mountain Prestress, Inc., at their plant in Denver, Colorado. The company, a PCI Certified Producer Member, has had a long history for furnishing high quality products and superior delivery service in Colorado and other states.

A total of 731 precast, prestressed concrete components were fabricated for this project. A breakdown of the number and types of components is as follows:
- 665 components [37,953 sq ft (3530 m²)] of 3 1/2 in. (89 mm) thick precast, prestressed concrete deck panels.
- 16 components [1280 lineal ft (390 m)] of haunched precast, prestressed concrete bulb-tee girders, varying from 72 in. (1829 mm) at the ends to 96 in. (2438 mm) at the center.
- 40 components [4553 lineal ft (1388 m)] of precast, prestressed concrete BT-72 bulb-tee girders.
- 9 components [616 lineal ft (188 m)] of 72 x 24 in. (1829 x 610 mm) precast, prestressed concrete box girders. These girders were used in Span 6 but were not post-tensioned.

The concrete girder compressive strength was 7500 psi (52 MPa). The maximum girder length was 154 ft 4 1/4 in. (47 m) and the maximum girder weight was 176 kips (782 kN.).

The BT-72 girders were cast during the winter of 1994-95. Special care was taken to accurately and firmly position the three continuous post-tensioning ducts and anchor plate assemblies. After casting, the clearance through the ducts was checked by pulling a “rabbit” through the ducts. Any obstructions in the ducts were removed before shipping the girders to the jobsite.

The haunched bulb-tee girders were cast in an adjustable soffit form that allowed for the girder depth to vary from 72 to 96 in. (1829 to 2438 mm). Similar procedures were followed with the post-tensioning duct.

Figs. 9, 10, 11 and 12 show the various sequences in the fabrication of the bulb-tee girders.

**HAULING PROCESS**

Special measures were taken by Rocky Mountain Prestress to ensure that the precast components would be transported safely from the plant to the project site (see Figs. 13, 14 and 15).
The 120 mile (193 km) journey south to Pueblo proved to be uneventful. Most of the girders were hauled with the precaster's standard tandem axle jeep units in front and back. The four heaviest girders, ranging in weight from 164 to 176 kips (729 to 782 kN), required the use of a three-axle jeep unit at the front of the girder attached to the tractor and a tandem steering/dolly tandem jeep unit at the rear of the girder to limit wheel loads. Moreover, because the total loaded weight of the tractor trailer assembly exceeded 200 kips (889 kN), the Colorado Department of Transportation was involved in verifying that the route used to transport the precast components to the bridge site would not overload any bridge structures along the way.

ERECTION SEQUENCE

Various construction stages of the bridge are shown in Figs. 16 through 21. Erection was accomplished using two 4100 Manitowac Track cranes beginning with the first two spans (Spans 1 and 2) over the Arkansas River. The erection was performed by Stone River Inc. of Sandia Park, New Mexico. Temporary support towers were erected by Lawrence Construction in Span 1 to support the ends of Segments 1 and 2 and in Span 2 to support the ends of Segments 3 and 4. Strong-back assemblies were used to support the free end of Segment 3 from the cantilevered end of Segment 2 and the free end of Segment 5 from the cantilevered end of Segment 4.

Erection of Segments 1 through 4 was accomplished by positioning Crane 1 behind Abutment 1. Crane 2 was positioned in the river bed between Piers 2 and 3. Trucks accessed the site to a picking position behind Abutment 1. Segment 1 was erected by picking directly from the truck and placing in position with Crane 1. Erection of Segments 2, 3 and 4 was accomplished by passing girders in mid-air from Crane 1 to Crane 2.

Erection of Segments 5, 6 and 7 was completed after repositioning both cranes to the railroad side of the structure. Erection of these segments required setting times to be closely coordinated with train schedules of two railroad lines running beneath Spans 5 and 6. The free end of Segment 5 was supported by a strong-back assembly from the cantilevered end of Segment 4. Segment 6 was supported by embedded corbel assemblies cast in the cantilevered ends of Segments 5 and 7. Segment 8 (box girders) was erected after all post-tensioning was completed on Segments 1 through 7.

The erection of the precast components, together with the various post-tensioning operations took less than 5 months to accomplish. Figs. 22 and 23 show the finished bridge.

AWARD WINNER

In June 1996, the bridge was given a design award for “Best Bridge With a Span Greater than 135 ft (41 m)” in the 1996 PCI Design Awards Program. The jury’s citation read:

“...the use of precast concrete allowed the designers to minimize the amount of falsework necessary, especially compared to that needed if cast-in-place hammerheads had been used. The erection process respected a complicated site. The finished project blends well with the adjacent architecture and the haunched girders, which are difficult to make aesthetically pleasing."

CONCLUDING REMARKS

This award winning bridge was completed on schedule in December 1995 and the facility is now fully operational. The finished structure met all of the city’s objectives:

- The bridge is aesthetically pleasing and is in harmony with the surrounding buildings.
- The project was finished within the allotted 12-month construction time frame.
- It met budget.

In retrospect, the owner and all parties involved in the design and construction of the bridge are very pleased with the end result. This was made possible because of the cooperative team effort.

The Main Street Viaduct is already a popular landmark in the City of Pueblo. Indeed, it is one more symbol reflecting the resurgence of the city.

CREDITS

Owner: City of Pueblo, Pueblo, Colorado
Engineer of Record: LONCO, Inc., Denver, Colorado
Structural Engineer: Finley McNary Engineers, Inc., Denver, Colorado
General Contractor: Lawrence Construction Co., Littleton, Colorado
Precast Concrete Manufacturer: Rocky Mountain Prestress, Inc., Denver, Colorado
Illustrations: The line drawings for this article were supplied by LONCO, Inc. The photographs were furnished by LONCO, Inc., Finley McNary Engineers, Inc., and Rocky Mountain Prestress, Inc.