Design and Construction of Clarks Viaduct in Nebraska

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ABSTRACT

Clarks viaduct has 4 spans (100ft, 151ft, 148ft, 128.5ft) and a total width of 38.3ft. A solution was developed that utilized NU 1100 girders with 43.3 in. depth, modified to 50.0 in. to match the original steel girder depth. To meet the challenge of spanning 151ft with a 50.0-in.-deep girder (span to depth ratio equal to 36), cast-in-place haunches and special threaded rod connections for continuity for deck weight, were incorporated. This paper presents the special features and details of this new hybrid system, which is the first high performance concrete application with non-post-tensioned continuity for deck weight.

Keywords: Precast concrete, Prestressed concrete, Value engineering, Continuity connection, NU girder.

INTRODUCTION

Historically, long-span girder bridges have been the domain of steel plate girder superstructures. Concrete girders, except when post-tensioned, have not been able to match, depth for depth, steel plate girder equivalents. In order to match structure depth, something else had to be adjusted, such as reducing girder spacing. These factors lead to concrete structures not being as cost competitive with steel in spans longer than about 150 feet (45 m). A major advantage steel has had over concrete is its ability to make the structure continuous before placement of the deck slab. Steel plate girders also have the advantage of a haunched girder depth in the negative moment zone, resulting in high negative moment capacities and an optimized utilization of materials compared to constant depth, prismatic, members. Value-engineering of 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 to complicated precast concrete girder geometry.

Precast prestressed concrete I-girder bridges represent about one-third of the bridges built in the United States each year. They are generally constructed as simple span for their weight and the weight of the cast-in-place deck. Cast-in-place diaphragms and reinforcement in the deck render the superstructure continuous for superimposed dead loads and live loads. This system has served very well over the past three decades, especially in cold climate states where expansion joints over the piers create maintenance problems. However, the girders are made continuous for about one-third of the total load only and are thus not fully utilized in the negative moment zones. In addition, some of the bridges built using this type of continuity have experienced cracking due to positive time-dependent restraint moments at the piers, especially in highly prestressed girders. With a new continuity system developed by Dr. Maher K. Tadros at the University of Nebraska, the girders are coupled over the pier using four 1-3/8” diameter, Grade 150 ksi threaded rods before the deck weight is applied. Using this system, the girders are made continuous for about two-thirds of the total loads. The threaded rod system resists the negative moments due to deck weight. After the deck concrete has hardened, deck reinforcement, along with the high-strength threaded rods, resist the negative moments due to superimposed dead load and live load. Span capacities are improved by about 10 to 15 percent within a given girder size. More importantly, bridge performance is improved as the negative moments due to deck weight more than offset the positive restraint moments due to time-dependent effects. Reduction in positive restraint moments results in less cracking in the pier diaphragms.

VALUE ENGINEERING

The Clarks Viaduct is a four span replacement bridge with span lengths of 100’-151’-148’-128.5’. The 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 (Hawkins) of Omaha, Nebraska. After the construction drawings were released, Tadros Associates started working with the contractor on a unique concrete alternate that would result in considerable cost savings, a better overall structure, and would incorporate the latest in material technology and structural theory and design. The design was proposed to the contractor, who in turn submitted a value-engineering proposal to the State of Nebraska Department of Roads (NDOR) for approval. The value-engineering proposal was ultimately accepted by NDOR, and Tadros Associates was directed by Hawkins to develop the necessary construction documents.

The first challenge Tadros Associates (TA) faced was to provide a concrete alternate that fit within the profile of the steel structure. The viaduct passes over three railroad tracks and US-30, a major highway in central Nebraska. Modifications to the profile grade line, vertical clearance limits, or bridge width would result in an unacceptable solution. The original design consisted of four lines of varying depth plate girders with a beam spacing of 10'-9". The depth of plate girder at midspan of spans 1, 2, 3, and 4 was 50.25", 51.125", 51.125", and 51", respectively. The maximum depth of haunched plate girder section over piers 1, 2, and 3 was 75.5". Therefore, a 50" deep concrete girder was chosen to match the minimum steel section. The girder was a modified NU-1100 with 7" of depth added to the top of the girder. The top flange was narrowed from 4' to 2' in width in order to save weight. The deck consisted of an 8" uniform slab haunched at the supports with a 28-day strength of 4,000 psi. The concrete used in the modified NU-1100 girders had a release strength of 6,500 psi and 28-day strength of 8,500 psi.

Due to the complicated design and analysis, typical bridge design software could not be used. Instead, the analysis was performed using spreadsheets and a 2d frame analysis program. LEAP software's CONSPAN LA was used to obtain camber data. The first step in girder design was to determine the number of pretensioned strands required based on girder concrete allowable stresses. A spreadsheet was written that allowed the designer to quickly change the number of strands and see how it affected girder stresses. The spreadsheet incorporated AASHTO LRFD lump sum prestressed losses and transformed section analysis for stress computations. Once the number of strands was set for each span, the ultimate flexural capacity was determined by hand using AASHTO Standard ultimate flexure equations. The negative moment capacity of the system was determined using strain compatibility. Strain compatibility was used in order to account for the high strength threaded rods in addition to the mild reinforcement in the deck slab.

Using the modified NU girder and the initial design assumption that the support condition between the superstructure and substructure would be pinned (the superstructure is free to rotate on top of the substructure), large negative moments developed over the piers. In fact, the concrete stress in the bottom girder flanges at the piers exceeded the 28-day compressive strength. Tadros Associates' solution to this problem was to create a unique haunched concrete section over the piers. Creating a haunched concrete section also required TA to make minor modifications to the substructure. The pier consisted of four columns on individual pile supported footings and a pier cap. Because the method of analysis assumed that the substructure and superstructure make a rigid frame, the pier column locations were adjusted so that they were centered directly under each girder line.

The haunched section was created by first casting a cast-in-place (CIP) haunched beam over the top of the pier cap (see Fig. 1). The CIP haunched beam used concrete with 28-day strength of 5,000 psi. The CIP haunched beam was used to give the superstructure extra depth at the piers and move the ends of the girders closer to the structure's dead load inflection points, thus reducing the concrete stress due to negative moment in the bottom flanges of the girders. Each CIP haunched beam is four foot wide, 25'-8" long and tapers in height from a minimum thickness of 2'-0" at the centerline of pier to 6" at the ends of the beam. Once the CIP beam achieves its required strength, the girders are placed, as shown in Fig. 1. The ends of girders overlap 2'-0" onto the CIP haunched beam. The four 1-3/8" diameter high-strength threaded rods embedded in the girders were then coupled using two Grade 50 rectangular steel bars and five Grade 150 ksi threaded rods (see Fig. 2 and Fig. 3). The rectangular steel bars are designed to transfer the ultimate tensile force of the four rods. Once the threaded rods were connected, concrete was cast over the entire area of the CIP haunched beam up to the top of the girders – this concrete section will be referred to as the pier block, as shown in Fig. 4. The pier block also used concrete with 28-day strength of 5,000 psi. Once the concrete pier block achieved 28-day compressive strength, the deck was cast. The bridge construction was completed in July 2003 (see Fig. 5).

The CIP 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 cast and hardened, the CIP haunched beam had to resist the weight of the girder and the wet concrete from the pier block. As a result, 14-#8 bars were used to reinforce the haunched beam during erection.

Fig. 1 Precast girders seated over the CIP haunched beam

Fig. 2 Showing the pier section before placement of the pier blocks.

Fig. 3 Close-up view of the continuity connection

In order to provide stability of the CIP haunched beam during girder placement, a moment connection had to be made between the haunched beam and the pier cap. Creating a moment connection between superstructure and substructure is not usually done in conventional bridge design. Typically, the only loads transferred to the substructure are vertical loads and lateral loads. Due to the magnitude of dead and live load moments, the vertical column reinforcement was increased from 20-#8 bars to 26-#11 bars, while the dimensions of the column (3'-0" x 3'-0") remained the same. Another design issue that had to be resolved, because of the moment connection, was that the maximum column moment occurred at the intersection of the superstructure and substructure (top of the pier cap). As a result, the 26-#11 vertical column bars had to be extended through the pier cap, CIP haunched beam, and into the pier block. Extending the column reinforcement created another challenge in that the vertical column bars had to pass through intersecting bars from the pier cap, as well as the 14-#8 bars in the CIP haunched beam. Because of the tight tolerances, steel template plates were required to help the contractor place the vertical column bars. Three of these steel plates were provided – one was placed in the pier cap, another in the CIP haunched beam, and the third was placed in the pier block. Two of these steel templates (CIP haunched beam and pier block) were welded to the vertical bars in order to anchor the vertical reinforcing.

Complicating the design even further was the issue of bridge skew. The skew on this structure is 52 degrees. In bridges (as with all other structures), bending moment and shear forces tend to follow the stiffest members (which in this case are the girders), where in slab bridges the forces tend to take the "shortest" route to a support, which is the obtuse corners of a span. This behavior does occur in girder bridges, but is not as pronounced as in slab bridges. Often times this leads to a different distribution of load than assumed in design. The longitudinal bending moments tend to be reduced, but shear 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 as well as to 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 they 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 and shear forces to more closely follow the longitudinal girder lines.

Another 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 is different from the common assumption that the superstructure rotation is along the longitudinal axis of the girders. The Clarks Viaduct was designed without the use of transverse pier diaphragms to help minimize the effects of the heavy skew and reduce the dead load on the piers.

One of the main benefits of the continuity connection used on the Clarks Viaduct is its simplicity. Tadros Associates' feedback from local contractors is that they do not like to use post-tensioning. While post-tensioning is more efficient and offers some other benefits, it is significantly more expensive and requires specialized labor during jacking and grouting. In Nebraska, and 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. If you can get the specialized personnel out to the site, the contractor has to pay for travel, room and board, and the time on site. This can be complicated if there are any project delays, or if the weather is bad. With the continuity connection used on this bridge, the contractor is only required to assemble a few pieces and cast a pier block to establish the connection.

CONCLUSIONS

In value engineering the Clarks Viaduct, Tadros Associates' has shown that it is now technically and economically feasible for concrete superstructures to compete with long span steel bridges. Not only will the concrete superstructure result in long-term savings (in terms of lower maintenance cost), but will provide immediate benefits to the owner and the contractor in construction cost savings. The overall construction savings were approximately $100,000, which was split between Tadros Associates', Hawkins Construction Company, and the State of Nebraska.

Fig. 4 Girder continuity by placing the pier block

Fig. 5 Completion of bridge construction