NEBRASKA PRECAST RECORD:

SKY-LINE BRIDGE SPANS SINGLE POINT INTERCHANGE

by

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The Skyline Bridge, with a single-span of 63 m (206.7 ft), achieves a span-to-girder-depth ratio of 31.5 using NU2000 PT (Nebraska University 2000 mm Post-tensioned) spliced I-girders. It the longest simple span concrete I-girder bridge in Nebraska, and perhaps the United States. Each girder line consisted of three precast, pretensioned segments, erected on the false work, and post-tensioned together after attaining the required strength at the wet joints and before forming the deck slab. High-performance concrete (HPC) with a 28-day design compressive strength of 68.75 MPa (10.0 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 cost of the bridge at, $624/m^2 ($58.00/ft^2), which is less than the typical range of the precast prestressed concrete I-girder bridge cost in Nebraska of $646/m^2 to $807/m^2 ($60.00/ft^2 to $70.00/ft^2) at the time of letting, demonstrates the great potential of this spliced girder system. Additionally, the efficient NU section and the state-of-art design theory contributed to this record-setting bridge. This paper presents the detailed description of design and construction of the Skyline Bridge in Douglas County, Nebraska.

INTRODUCTION

The Skyline Bridge, at 204th (Nebraska Highway 31) and Dodge Street (US Highway 6) in Omaha, Nebraska, provides the structure for a single point urban interchange (SPUI). The geometric considerations of the SPUI required that a 63 m (206.7 ft) single span be used. 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 by creating piers and two additional short spans, resulting in a total bridge length of 75.44 m (247.5 ft). The two additional suspended cast-in-place slab spans were only about 6.1 m (20 ft) each. The structure consisted of an eastbound bridge and a westbound bridge, which were very similar. The total structure will be called a single bridge for simplicity of presentation. Also, the main span was not considered to be impacted structurally by the two end spans, and the structure may be considered for all practical purposes to be a simple span girder bridge. The total bridge width is 35.36 m (116 ft), allowing for three traffic lanes and two shoulders in each direction (Fig. 1,2).

Fig. 1. Panoramic View of the Skyline Bridge, Douglas County, Omaha, Nebraska

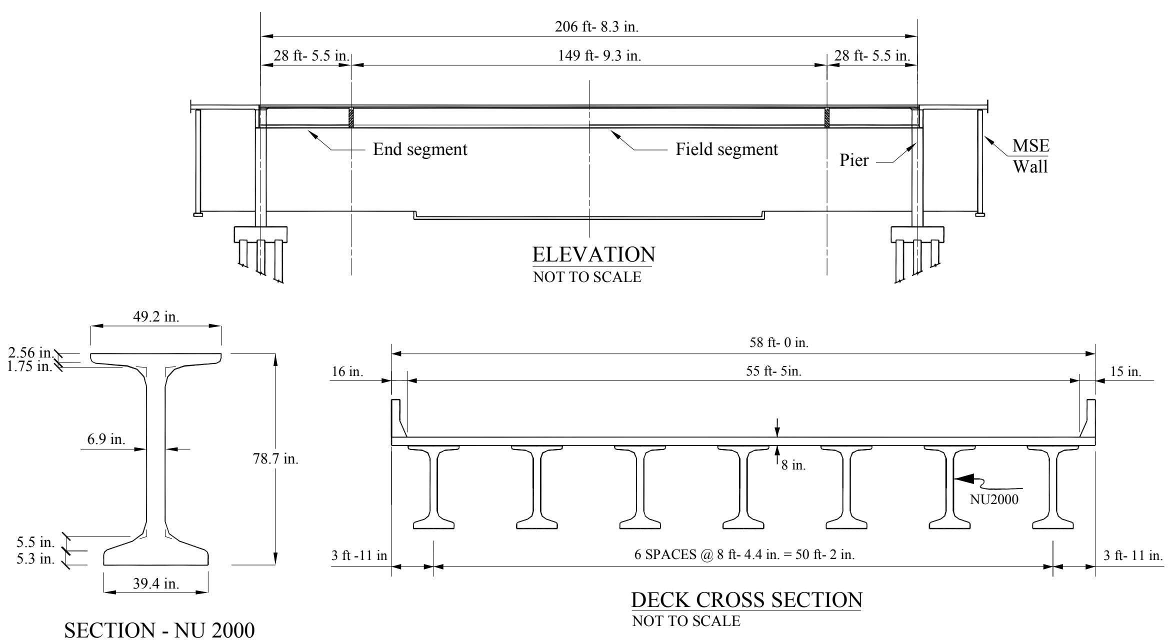


Fig. 2. General Layout of the 198th – Skyline Drive Bridge
Note: 1 in. = 25.4 mm; 1 ft = 12 in.

The Skyline Bridge is a grade separation structure; it is part of redevelopment of the area to make Dodge Street an expressway uninterrupted by traffic signals. Dodge Street at the bridge location carried 12,080 average daily traffic (ADT) in 2002 and is expected to carry 40,890 ADT in 2022. Nebraska Department of Roads is the owner and the designer of this bridge. Figure 3 shows plan and elevation of the Skyline Bridge.

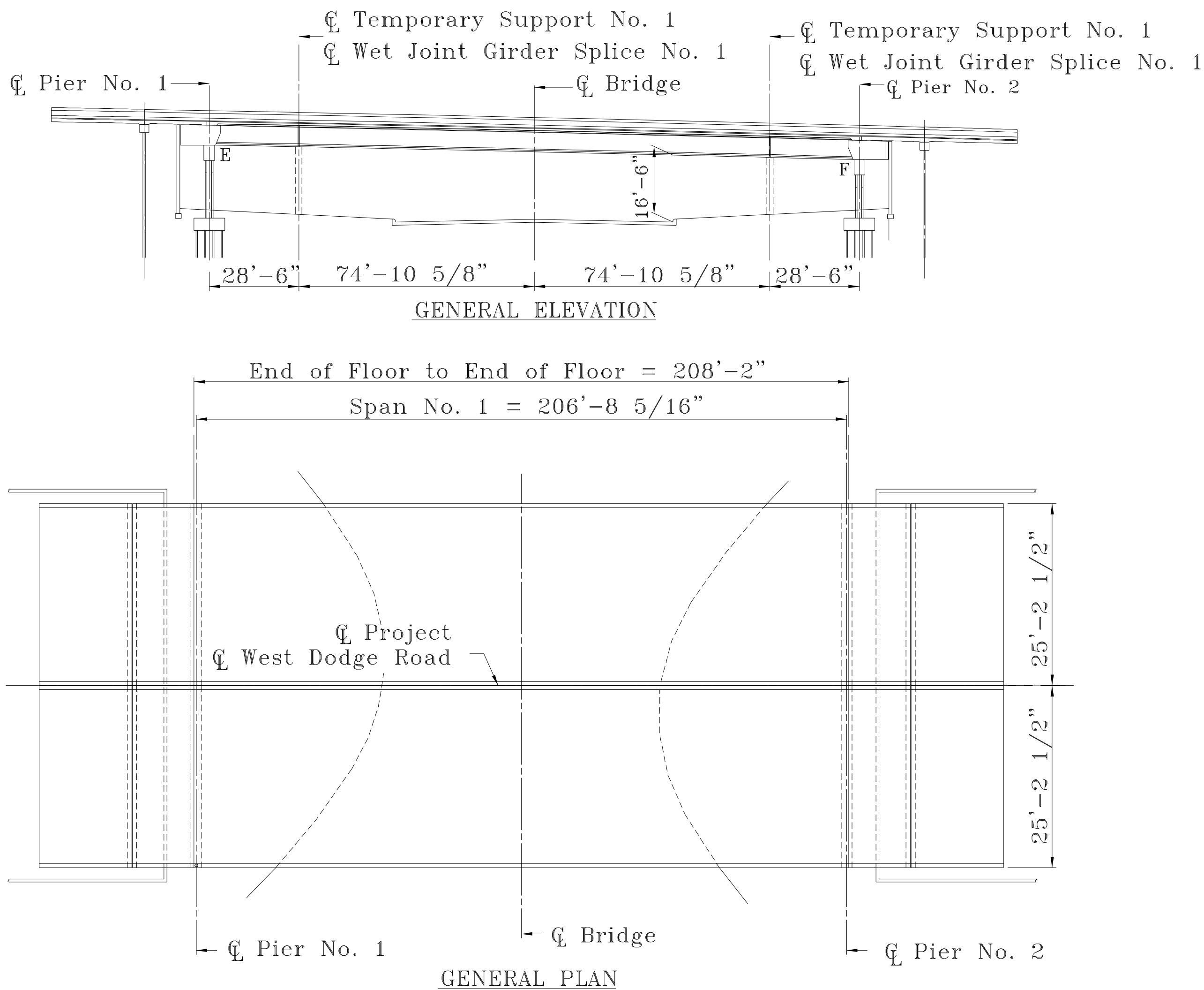


Fig. 3 General Elevation and General Plan View
Note: 1 in. = 25.4 mm; 1 ft = 12 in.

The bridge consists of 14 girder lines, in two structures, an eastbound and a westbound structure. Each girder line consists of two 8.75 m (28.71 ft) segments and one 45.5 m (149.28 ft) segment. All segments are made of precast, prestressed concrete NU 2000[1] (Nebraska University 2 m [78.74 in.] deep) I-girders. The precast concrete segments were then post-tensioned at the site on temporary supports to achieve the required 63 m (206.7 ft) single span. A total of 28 short and 14 long segments were used in the bridge. The span length of the bridge makes it the longest precast concrete girder span ever built in Nebraska. It is also believed to be the longest simple span with the shallowest girder depth-to-span ratio of 31.5 in the United States.

As a simple-span bridge of 63 m (206.7 ft) (centerline bearing to centerline bearing), the bridge overall length between abutments is 75.44 m (247.50 ft). The deck area is 2,668 m^2 (28,722 ft^2) including the suspended slab spans, and 2,241 m^2 (24,118 ft^2)excluding the suspended spans. The total bridge cost is $1,673,797, i.e., a cost per sq ft of $58.

The project design was completed in early 2002, and was let to contract on August 1, 2002. The bridge construction started in the fall of 2002 and the bridge was open to traffic late in 2004. The bridge surface is expected to be stained with an attractive color very shortly. In 2005, the bridge was a design award co-winner in the category of bridges with spans greater than 135 feet by Precast/Prestressed Concrete Institute.

SYSTEM SELECTION

Several systems were considered in the preliminary design stage. The first option was to consider a three-span continuous girder bridge. Short-span pretensioned NU I-girders would be used in place of the slab spans. Due to the significant cost and time in terms of the substructure construction, this option was not favorable. Another option was to increase the spans of the two side spans and to create a "balanced" span design with haunched steel or concrete girders.[2] That concept showed preliminary costs considerably higher than the final system.

Designing the bridge using simple, 63-m- long (206.7 ft), NU2000 I-girders was found to be feasible if 68.75 MPa (10,000 psi) concrete was used and if an additional thickness was added to the top flange. A single piece pretensioned-only beam was investigated.[3] However, the prestressing beds of two local producers did not have the required large capacity of about ninty 0.6-in. (15.2 mm) strands. Also, the weight of the girder 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.

A single-span steel plate girder was studied.[1,4,5] Deflection and vibration limits would have required deeper sections than the NU2000, which would have had a significant impact on the cost of the ramps. It was therefore ruled out.

Accordingly, the designers selected an NU2000 spliced post-tensioned girder[2,6] (NU2000- PT) which has a web 25 mm (1 in.) wider than the standard NU2000 pretensioned girder (175 mm versus 150 mm [6.9 in vessus 5.9 in.]). The design called for three girder segments with a design strength of 68.75 MPa (10 ksi)per girder line with two cast-in-place concrete splice joints. The bridge cross section consists of seven girders spaced at 2.55 m (8.37 ft) topped with a 29.47 MPa (4.3 ksi) strength, 200 mm (8 in.) thick cast-in-place concrete slab. Each girder line includes two 8.75-m-long (28.71 ft) end segments and a 45.5 m (149.28 ft) center segment, as mentioned previously.

DESIGN AND ANALYSIS

The bridge was designed for HS25 live load according to the AASHTO Standard Bridge Design Specifications.[5] The bridge design was performed by the Nebraska Department of Roads (NDOR) and independently checked by Tadros Associates, LLC, Omaha, Nebraska. The pretensioning force was designed to resist the girder weight and provide no bottom tension in the concrete during handling. The pretensinig force, was then progressively increased to minimize the amount of post-tensioning, without making the required release strength excessive. The entire amount of post-tensioning is introduced before the deck is placed. This is consistent with NDOR's general philosophy of not applying any post-tensioning after the deck is placed and cured.[1] The main reason for this philosophy is the desire to be able to remove the deck at any time without special analysis for girder capacity to handle the previously installed post-tensioning. Other reasons relate to economy: (a) Nebraska is a state with little local post-tensioning installation expertise; one stage post-tensioning would require that the post-tensioning subcontractor be on site fewer times, and (b) placement of the deck slab and approach slab makes access to the post-tensioning anchorages at girder ends more challenging.

Commercial computer programs were not relied on during design. The designers at NDOR and Tadros Associates used spreadsheet programs to complete the design. An internally developed time-dependent analysis program (Creep III) was employed to verify prestress losses and to determine camber and blocking requirements during construction. One of the most challenging issues was to determine 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 a final camber (with no sag) in the girder soffit profile.

Service load stresses were initially checked and found to be acceptable except for the concrete compressive stress due to effective prestress plus permanent loads. For this case the value of compressive stress at the top fibers of the girder at mid-span exceeded the $0.4f'_c = 0.4(10) = 4$ ksi allowed by the AASHTO Specifications.[4] That required the designers at that time to add 50 mm (2 in.) of concrete thickness to the top of the girder. However, when the stresses were checked by Creep III program, which accurately models differential creep and shrinkage and also the relieving effects of girder creep in these hypothetical elastic analysis stresses, it was determined that the high stress occurred only at the top fibers, only the mid-span, and only at the time of deck placement. The stress continues to decrease with time, and becomes relatively small at all losses. This analysis, along with extensive theoretical and experimental studies[7,8] previously completed at the University of Nebraska, gave the designers enough confidence to remove the additional 50 mm (2 in.) of concrete, resulting in significant improvement in economy and applicability of the system. This project was one of the main factors in convincing the NDOR to establish a state-wide policy to remove the requirements for two compressive stress limits from design, the limit due to dead load plus effective prestress and the limit due to full load plus effective prestress. The policy took effect in May of 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 (10.0 ksi), respectively. The strength of the wet joints between the precast concrete pieces was specified to be 41.24 MPa (6.0 ksi) before post-tensioning. The concrete deck slab was specified to have a 28-day compressive strength of 29.65 MPa (4.3 ksi).

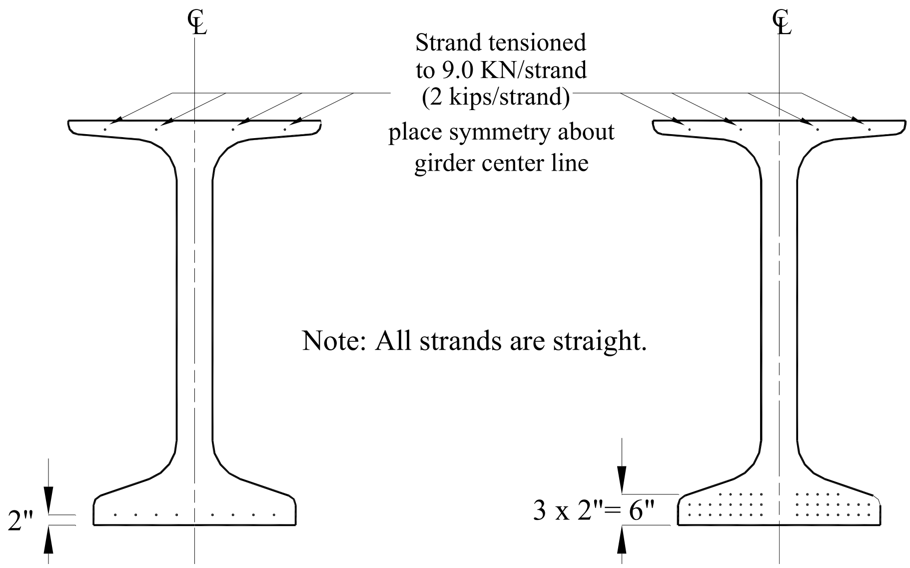
The splice joint was designed for zero tension at all times. No special reinforcement was provided in the joint. The flexural strength was determined using the strain capability approach as given in the PCI Bridge Design Manual.[9] This is the same method that has just recently (April 2005) been approved by AASHTO Committee T10 for inclusion in the AASHTO Bridge Design Specifications in the 2006 Edition. This method is much 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 ability of the NU I-girder to have a shear capacity as high 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.[10] This allowed the web width to remain at the standard 175 mm (6.9 in.) without widening and increasing girder weight. This was possible even with allowance for the 95 mm (3.74 in.) diameter post-tensioning duct as required by the code, i.e. the effective width for shear calculations, b_v = 6.88-0.5*3.74, which is only 5.0 in.. This high maximum shear limit was made possible in the modified compression field theory included in the AASHTO LRFD Bridge Design Specifications. Grade 75 welded wire reinforcement was used. The design was based on the typical 414 MPa (60 ksi) strength, and the 25% increase in capacity, compared to using Grade 60 individual bars, was considered an added safety margin at no additional cost (WWR Gr. 75 costs as much as Gr.60). Extending the vertical shear stirrups into the deck proved adequate for interface (composite action) shear. Shear friction theory was used to check the splice joint shear capacity.

Member shear capacity is developed through theoretical and experimental research. It is valid regardless of whether the member is a building or bridge member and regardless of whether the Code/Specifications used for design is the ACI 318, AASHTO Standard or AASHTO LRFD. It is therefore acceptable, in the authors' opinion, to use $0.25\, f_c'\, b_v\, d_v$ as the maximum limit on shear force to be applied to any section as long as that maximum is backed up by theory and experiments. The University of Nebraska did extensive full scale testing[10] to show that this limit is achievable for NU I-Girders and and with adequate longitudinal reinforcement anchorage as specified by the Modified Compression Field Theory.[5]

The final prestressing consists of 42-0.6 in. (15.2 mm) diameter, Grade 270 ksi (1,862 MPa) pretensioning strands in the center segment, 8 pretensioning strands in the end segments and 3 post-tensioning tendons, 15-0.6 in. -diameter(15.2 mm) in the full length.

Post-tensioning details are shown in Fig. 4 and 5.



(a) End Segment (b) Center Segment

Fig. 4. Pretensioning Scheme of the 198th – Skyline Drive Bridge

Note: 1 in. = 25.4 mm; 1 ft = 12 in.

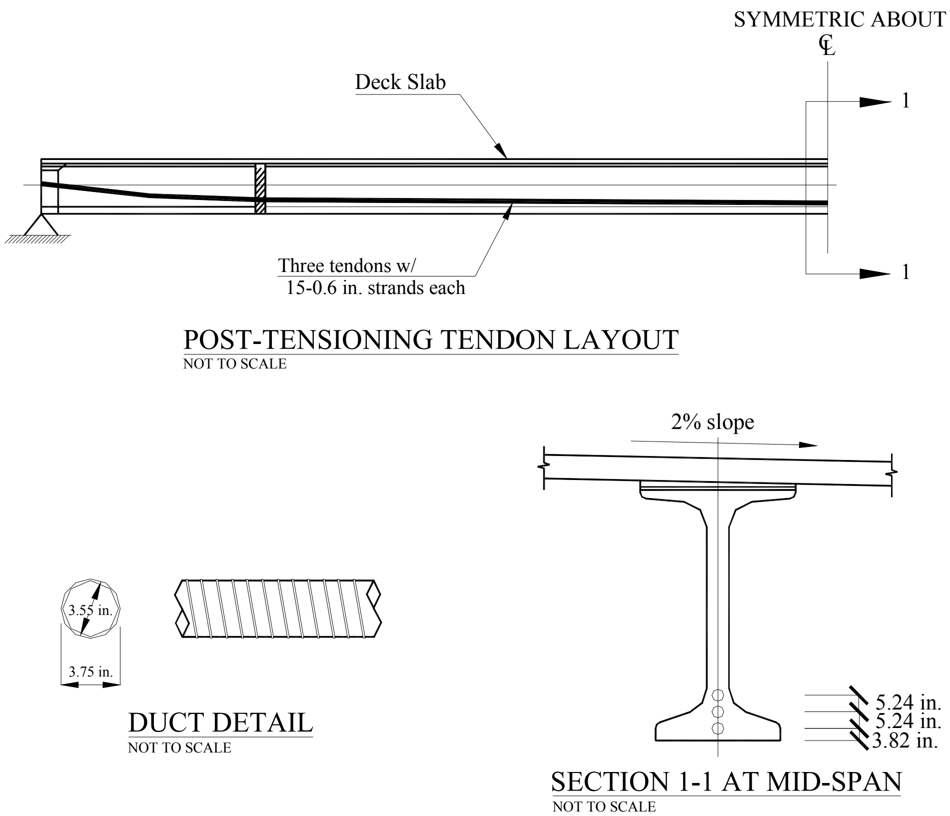


Figure 5. Girder Post-tensioning Details
Note: 1 in. = 25.4 mm; 1 ft = 12 in.

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 blocks are only 700 mm (27.6 in.) wide tapering down to the standard 175 mm (6.9 in.) web width in a distance of 1000 mm (39.4 in.). The weight of this block is only 20% of the weight of a standard AASHTO Standard Bridge Design Specifications[5] post-tensioning end block, see Fig. 6. The dimensions were selected to allow housing of the post-tensioning anchorage hardware. It was found that the size of 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 tendons to control vertical splitting. Welded wire reinforcement at a narrow 50 mm (2 in.) spacing was used to control horizontal cracking. Nebraska designers no longer have to custom-design the post-tensioning end block; the details were developed, checked, and full-scale test verified. The developed end blocks are adequate for up to three tendons with 15-0.6 in. diameter strand aligned in a vertical line.

Fig. 6. Optimized Post-tensioning End Block

I-GIRDER PRODUCTION

As previously mentioned, NU 2000 beams were selected by the designers. Each girder line consisted of two 8.75 m (28.71 ft) end segments and one 45.5 m (149.28 ft) middle segment. The middle segment had 3- 3.75 in. (95 mm) straight post-tensioning ducts. The post-tensioning tendons' profiles in the end segments were parabolic, from the precast concrete girder centroid at the girder line end to match the location of the straight tendons in the middle segment as shown in Fig 5. As mentioned previously, high-performance concrete (HPC) with a 28-day strength of 68.75 MPa (10.0 ksi) was used for the NU I-girders. Grade 270 (1,862 MPa) low-relaxation, 0.6-in.-diameter (15.2 mm) strand, and Grade 75 structural welded wire reinforcement (WWR), were the materials used in the production of the girders.

SHIPPING AND HANDLING THE MIDDLE SEGMENT

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 149.28-ft-long (45.5 m) middle segment were adequate. No major problems arose with regard to shipping and handling the precast concrete girder segment to the project site. Standard lifting loops consisting of strands embedded in the concrete were adequate for handling the two other precast concrete segments.

BRIDGE CONSTRUCTION

The Skyline Bridge was constructed in 2004. The contractor was given the option of either assembling the three segments of each girder on the ground, or over the permanent supports and temporary shoring towers; it opted to use the latter option. The 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. The temporary tower and segment joint are illustrated in Fig. 9. 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 has cured.

Fig. 7. Center Segment Prior to Placement over Temporary Towers

Fig. 8. All Girder Segments Placed Over Piers and Temporary Towers

Fig. 9. A Close View of Temporary Tower and Segment Joint

Once the segment joints were formed, and concrete was placed and gained the required strength, the post-tensioning was applied. The applied post-tensioning force was controlled by gauge reading and checked by strand elongation. The strength of the joint was only specified at 6000 psi (41.37 MPa) (not the 10,000 [68.95 MPa] 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 due to post-tensioning. As soon as 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 source of attraction and pride to the community.

BENEFITS OF SPLICED GIRDER SYSTEM

CONCLUSIONS

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, light weight concrete of long span beams, and splicing 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 Skyline Bridge project presents a feasible alternative to full-span beam solution. Even though the spliced-beam system is more expensive than pretensioned-only concrete system, it is generally more cost-effective than the steel plate beams. Moreover, the spliced-beam system allows for excellent durability and structural performance, low maintenance, and a low cost bridge structure.

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