# USING PERFORMANCE SPECIFICATION ILLINOIS TOLLWAY SAVES $8 MILLION WITH PRECAST ALTERNATE

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## ABSTRACT

*The Illinois Tollway offered two plan alternates and one performance specification alternate to construct the I-355 over Des Plaines River Valley bridge on the recently opened I-355 extension located in the Southwest suburbs of Chicago, Illinois. Using the performance based bid specification alternate, the design-build team constructed a $125 million, 6,593 ft long bridge. This alternate beat the Tollway's closest per plan alternate by $8 million. The superstructure design utilized prestressed and post-tensioned precast concrete bulb-tee beams on this 35 span bridge structure. The prestressed spans ranged in length from 114 ft. to 170 ft using bulb-tees 90" in depth. The post-tensioned spans ranged in length from 216 ft. to 270 ft using segmental bulb tees 102" deep haunched to 120" deep for the main spans. All beams where delivered to the project site by truck.*

*The bridge was divided into 8 separate continuous units with the longest one being 1,529 ft in length. This particular unit was post-tensioned. Due to the impractical task of installing strands in a tendon of this length, the designer developed a unique lap detail, completely contained inside the beam, to achieve a continuous post-tensioned unit of this length. The development and detailing of this post-tensioning lap had to be completely performed in 3-D.*

*The design and construction of this bridge took approximately 22 months with the structure being open to traffic on Veteran's Day, November, 2007. The opening was celebrated with a 5K Walk/Run/Roll across the bridge attended by an estimated crowd of 30,000 people.*

**Keywords:** Precast Concrete, Prestressed Concrete, Post-Tensioning, Design-Build

## INTRODUCTION

The 1.3 mile long I-355 bridge over the Des Plaines River Valley located in the Chicago suburb of Lemont, Illinois was first derived, as with any project, because of a purpose and need. This bridge was part of a much larger corridor project. This corridor project was referred to as the south extension of I-355. It was constructed specifically to serve Will County, Illinois. Will County is one of the fastest growing counties in Illinois and the I-355 extension provides a regional connection that improves the north-south mobility between Interstates 55 and 80. This extension is estimated to reduce travel times by 20% and provides a more direct route between residences in Will County and areas where jobs are more plentiful. Even though it was easy to justify the benefit of the I-355 extension to the community, it was also necessary to consider and protect local endangered species present in the project limits. These endangered species actually played a big role in the project, even impacting the overall design of the bridge.

## PROJECT HISTORY AND BACKGROUND

The concept of this route first materialized in 1989 when the Illinois General Assembly authorized the Illinois State Toll Highway Authority (ISTHA) to begin studying the southern extension. After receiving this authorization, ISTHA began its studies and plan development for the project. By 1995 all necessary studies were complete and all construction documents were finalized. After ISTHA received bids for the I-355 bridge, an environmental group filed suit to block the construction of the bridge. The suit was filed because of the discovery of the Hines Emerald Dragonfly. Since the dragonfly was an endangered species a federal judge sided with the environmental group because the no build alternate of the extension had never been investigated in the studies performed by the Illinois Tollway.

In 1999 ISTHA amended its environmental impact study to address the concerns of the environmental group and finally in 2002, the FHWA issued a record of decision that allowed construction of the I-355 extension to proceed. In 2004 ISTHA was able to continue with land acquisition and utility relocations. The I-355 over Des Plaines River Valley Bridge was bid in November of 2005. Construction began in the spring of 2006 and the bridge was opened to traffic in November of 2007.

## PROJECT SCOPE

The scope of the contract was to construct a 6600 ft. long bridge over the Des Plaines River Valley that was to be incorporated into the I-355 extension. The I-355 extension was approximately 14 miles in length and connects Interstates 55 and 80 located in the southwest suburbs of Chicago, Illinois (see Figure 1). The I-355 over Des Plaines River Valley Bridge was the centerpiece of the new I-355 southern extension. The bridge is approximately 1.3

miles long, 125 ft. wide and carries 6 travel lanes, 3 each direction. The bridge crosses over 12 railroad tracks, 2 public roads, 5 private roads, 2 canals and the Des Plaines River. Out of the $730 million total cost of the southern extension, the I-355 bridge was bid at $125 million. The I-355 over Des Plaines River Valley Bridge contract was the largest of 18 contracts let for this corridor.

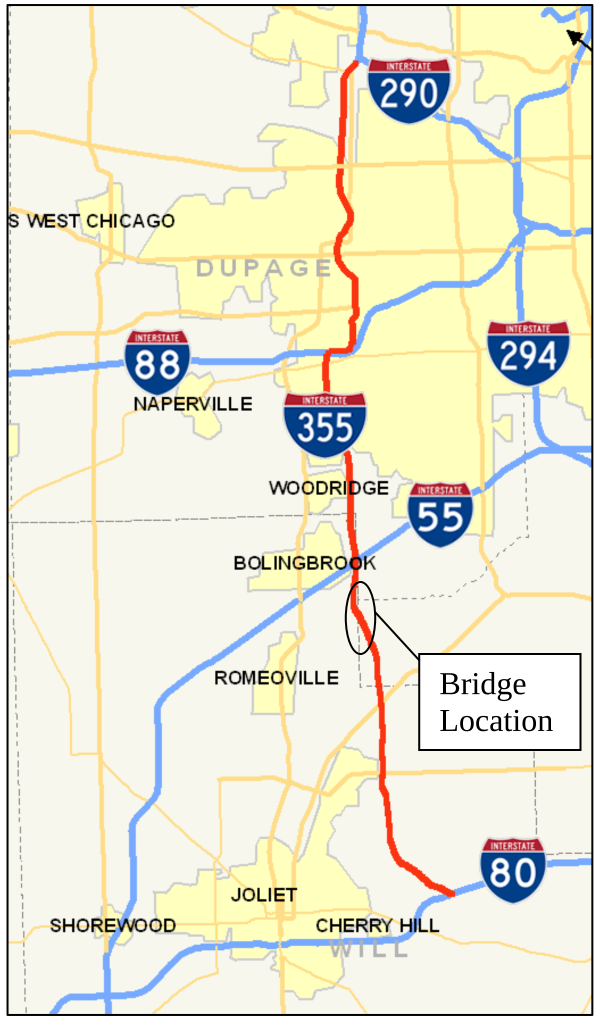

Fig. 1 Project Location

In the given contract documents, prospective bidders had three options for construction of the Des Plaines River Valley Bridge. The first two options were nearly identical to the ones given back in 1995. These options consisted of a steel alternate and concrete alternate. The plans for these two alternates were updated from the 1995 plan sets to meet current design criteria and incorporated some economic efficiencies. The first of these two options, the "As-Designed Steel Alternate" utilized a design consisting of 80" deep plate girders. This design also incorporated 12 steel delta frame piers in order to reduce some of the spans effective lengths. The second option given was the "As-Designed Concrete Segmental Box Alternate" which consisted of twin 11ft. deep post-tensioned box superstructures.

Additionally for the 2005 bid, ISTHA decided to add a performance-based bid specification as the third alternate. This alternate was structured very similar to a design-build alternate where an Engineer and Contractor team up together to develop a design and offer a construction price for this design. The performance-based bid specification did contain stringent design constraints and considerations that had to be adhered to. Additionally, no extra time was given if the performance-based bid was selected. The time required for design and construction had to be equal to the time required for just the construction of the first two alternates.

After evaluating all three alternates for the bridge, the successful contractor elected to bid the performance-based alternate and present ISTHA with their own design. This alternate was the low bid of all bids received and was accepted by ISTHA.

COMPANIES INVOLVED

Walsh Construction has long been a major player in the construction of large public and private projects in the Chicagoland area. Since they are more than capable of tackling such a large project, it came as no surprise that Walsh was quite interested in bidding this job.

Walsh teamed up with Janssen & Spaans Engineering, Inc. (JSE) of Indianapolis to advance a design-build alternate for the bridge far enough so that a bid could be assembled and also serve as the lead design engineer. JSE was a logical choice, as they have pioneered the use of innovative construction technologies throughout the Midwestern United States and have been very successful in obtaining many previous design-build projects.

Since the project specifications called for the team to include an independent QC designer, Walsh chose Bowman, Barrett & Associates, Inc. (BBA) of Chicago to perform these services. BBA not only provided a local presence to the engineering component of the team, having been selected by ISTHA for many traditional design-bid-build projects, but also had teamed up with other area Contractors for the construction of other recent design-build projects on the ISTHA system.

These three primary components gave the team an excellent chance in the competitive bidding phase and a strong design component capable of producing accurate contract documents in accordance with ISTHA requirements in the expeditious manner required for all design-build projects.

## SITE CONDITIONS

The project site, as expected for a 6600-ft. long bridge, was quite complex. The alignment of the bridge cuts through a valley with up to 80-ft. high bluffs on either side. This geometry, combined with the presence of the Des Plaines River, the historic I&M Canal, the Sanitary and Ship Canal, several railroad lines, two local roadways, a bike trail and detention basins made the choice of one, long bridge between the bluffs the most logical. Due to the river valley running between the bluffs on each side, where the abutments were located, the pier heights from the ground surface up to the top of cap were required to be up to 70' high.

In addition to the above site conditions, environmental issues helped shape the bridge geometry. ISTHA required that the bridge provide enough clearance to avoid interference with the flight path of the Hine's Emerald Dragonfly, a species labeled as endangered in 1995. Another environmental consideration that helped limit the total number of piers allowed was the protection of a turtle species, the Blanding's turtle. Blanding's turtles were found near the project site prior to construction. For their protection, the turtles were fitted with electronic tracking devices, so that if one came too close to construction activity it could be relocated.

## STRUCTURE LAYOUT

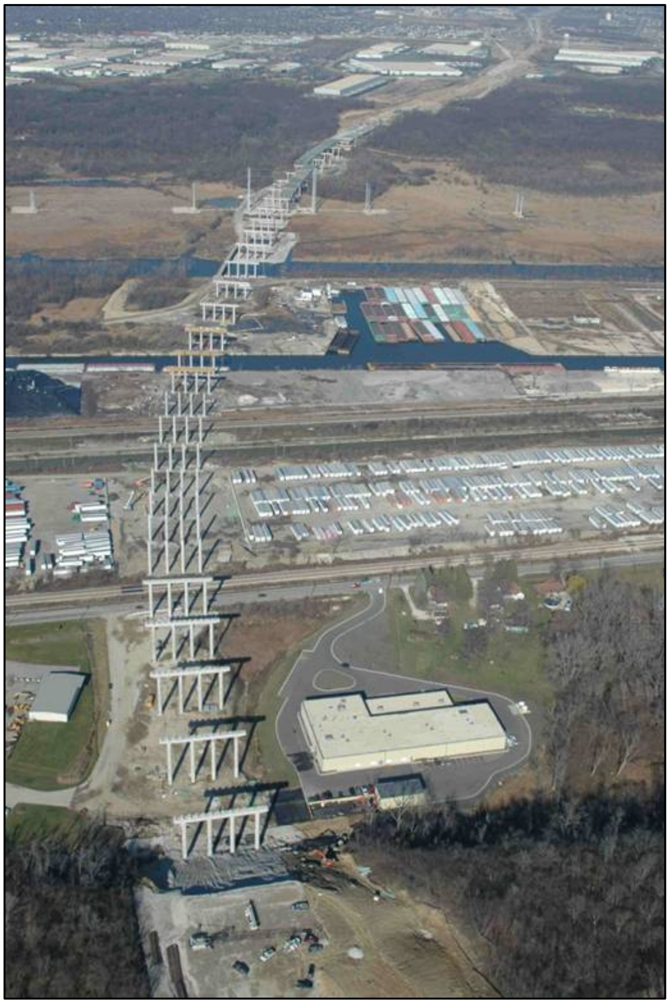

Fig. 2: Project Site

The maximum span length required for the bridge was 270 ft. in order to eliminate any piers within the banks of the Des Plaines River. Following typical design procedures and construction practices in Illinois for this span length, a steel superstructure would be the only stringer-type bridge option available. However, utilizing advances in concrete technology beyond what is typically utilized in Illinois, precast, prestressed concrete (PPC) Bulb-T beams were chosen as the preferred superstructure type. This superstructure type met ISTHA specifications, could be efficiently designed and detailed by the engineer and fit within the realm of Walsh Construction's means and methods.

At a span length of 270 ft., a conventional PPC Bulb-T beam, designed as a simple span for self-weight and slab dead loads and continuous for all superimposed loads, is not feasible. Spliced post-tensioned Bulb-T beams were therefore utilized. These beams were spaced at over 11 ft. centers and were typically 102 in.deep, increased to 120 in. over the piers of the longest spans. The post-tensioning and closure pours near the points of inflection provided the splicing mechanism to connect the beam segments so that they behaved as a continuous girder for all loads, including self-weight. Additional post-tensioning was then performed after the cast-in-place concrete deck was poured, readjusting the stresses in the girders to most efficiently carry superimposed loads.

The original concept developed utilized spliced post-tensioned Bulb-T beams throughout the length of the superstructure. However, the span lengths allowed by the use of these beams could have lowered the total number of piers to less than that allowed by the specifications. In order to decrease superstructure costs, it was concluded that about one-third of the superstructure would consist of conventionally-designed 90 in. deep PPC Bulb-T beams. The span lengths for these beams ranged from 114 ft. to 170 ft. Spliced post-tensioned Bulb-T beams were used in the remaining areas that required the longest span lengths over the many features crossed below.

Reducing the number of transverse deck expansion joints in a bridge deck is the best way to prolong the life of a bridge and decrease future maintenance costs. It also helps reduce initial construction costs by eliminating the joint hardware and providing as many efficient continuous supports as possible. Therefore, a goal of the design-build team was to keep the number of expansion joints required to a minimum. Due to various features crossed below and two horizontal curves within the bridge limits though, no two units between expansion joints could be identical. Through trial and error, the optimum number of units was determined to be eight, with a total of nine expansion joints. Although this averages out to 825 ft. per unit, the actual unit lengths ranged from a minimum of 599 ft. all the way up to 1,529 ft.

## PROJECT DESIGN SPECIFICS

### PRESTRESSED UNITS

While the PPC Bulb-T beams in four of the units were conventionally designed, i.e., simply supported for self-weight and slab dead loads and continuous for all superimposed loads, there were several features of the design that differed from standard practice in Illinois. These features included the depth of the beams, shipping and handling considerations, the strand size, the diaphragm type and continuity detail at the piers.

Fig. 3: Erection of 170 ft Prestressed Beams

### BEAM DEPTH

The maximum Bulb-T beam depth typically used in Illinois is 72 in., and it has a span range of about 150 ft. when spaced at about 4 ft. centers. These beams would not be able to span the 170 ft. required for these units, and the spans where these beams work would be inefficient due to the large number of beams required. Therefore, a larger section used in neighboring Indiana was chosen. The section is 90 in. deep with a 59 in. wide top flange and a 6” thick web. These larger beams were spaced at over 9 ft. centers for all spans, a much more efficient design than could have otherwise been achieved.

### SHIPPING AND HANDLING

A potential with utilizing such long beams though is excessive shipping and handling stresses. Similar beams on other projects have developed significant cracking or even failed while being shipped, erected or moved in the yard. To reduce shipping and handling stresses in the PPC beams on this project, additional "temporary" strands running the length of the beams in the top flange were specified. These strands reduced the tensile stresses that the beam experienced before erection, thereby reducing the chances of significant cracking. In order to eliminate the effects of these strands on the behavior of the in-service beams, the strands were only bonded at the ends of the beams and slowly severed after erection via access pockets cast in the top flange in the unbonded portion.

## STRAND SIZE

These Bulb-T beams also utilized 0.6 in. nominal diameter strands as opposed to the 0.5 in. strands used in Illinois prestressed beams. This increased diameter was requested by the prestressed beam fabricator to reduce the number of strands that had to be tensioned. With proper detailing of mild steel bars in the ends of the beams to resist bursting forces, the fabricator was allowed to use the larger diameter strands. Given the number of beams that required fabrication on this project, a relatively small change resulted in significant savings.

## DIAPHRAGM AND CONTINUITY DETAIL

To provide stability before and during the concrete deck pour, each beam was braced via steel diaphragms to adjacent beams. Some of these diaphragms were left in place permanently to aid in the transfer of design lateral loads from the superstructure to the substructure. Among the steel diaphragms left permanently in place were those installed between adjacent beams directly over the piers. These diaphragms were used in lieu of typical concrete diaphragms that extend between adjacent beams and encase the beams ends, transferring the compressive force that provides continuity between beam ends. Since this compressive force must still be transferred between beams to provide continuity, a closure pour of equivalent dimensions to the cross-section of the beam was specified. This detail significantly reduced forming and concrete material costs in addition to greatly expediting construction of the superstructure.

## POST-TENSIONED UNITS

Units 4 thru 7 of the design incorporated the use of post-tensioned segmental beams. These units crossed over 17 spans. These units ranged in length from 800 ft to 1,529 ft. The design of the post-tensioned units followed the criteria given in the AASHTO Segmental Code and the

Fig. 4: Erection of Segmental Bulb-Tees

AASHTO LFD code. Additionally for long term analysis, losses were calculated based on the criteria listed in the CEB/FIP code.

The design of the post-tensioned units required two different computer models. The first analysis run was a 2-dimensional analysis. This analysis was performed with the computer program *BRUCO* (BRidge Under COnstruction). This program built the entire bridge, one step at a time, from start to finish. This was necessary not only for the construction engineering portion of the project but also needed to determine how the erection procedure impacted the final design of the structure. The program also calculated all the step-by-step beam deflections and camber. This information was critical for setting the geometry of the structure.

The design of the bridge also required 3-dimensional analysis. This analysis was required at some of the more complicated interfaces of the structure. One such place is the anchorage locations of the tendons. The 3-D analysis was used to model the application of the post-tensioning forces to determine the splitting forces and design the necessary confinement reinforcement. Another consideration given to the structure was the horizontal alignment. To accommodate this geometry, the segmental beams were kinked at the splice points. This also was modeled in 3-D in order for the design to properly account for the radial forces developed at these points. Finally, a 3-D analysis was developed for a unique post-tensioning lap detail contained completely inside the beam. This detail will be further discussed below.

All beam segments were precast and initially prestressed to accommodate their selfweight and for handling/erection purposes. These beams were typically 102” deep and for the main spans the beams were haunched to 120” deep over the piers. Each beam line contains 4 tendons with each tendon capable of accepting 15-0.6” strands. The individual piece lengths varied from 113 ft. to 150 ft. in length. The beams used either 7500 psi or 8000 psi concrete mix. Due to the segmental nature of the construction, falsework towers were required for the beam erection. The span lengths for the post-tensioned units ranged from 216 ft to 270 ft.

The unit of most interest was unit 4. This unit was continuous for 1,529 ft. Figure 5 shows the erection sequence and geometric configuration of this unit. Strongbacks were utilized to support the drop-in pieces. After all segments were erected and the closure pours cast, phase I of the post-tensioning was able to commence. After phase I of the post-tensioning was complete, the need for the falsework towers was no longer required. At this stage the deck could be cast and then the final phase of post-tensioning could be completed.

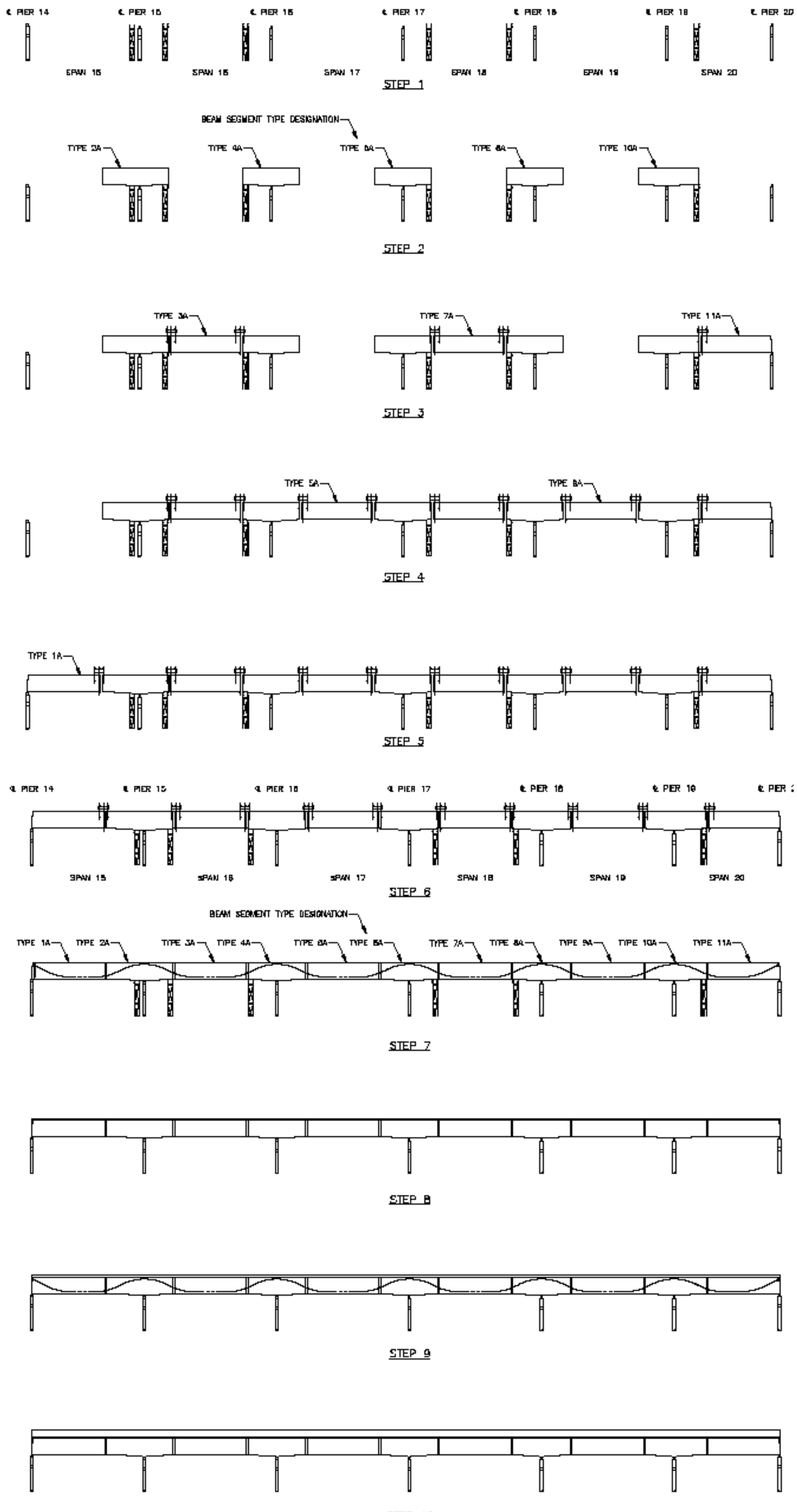


Fig. 5: Erection Sequence

One detail incorporated into the design, that is very unique to beam bridges, is the lapping of post-tensioning ducts. This was required due to the length of this particular unit, 1,529 ft. At this length it would be nearly impossible to install the post-tensioning cable. To get around this physical constraint a detail was developed to lap the tendons over the middle pier of the unit. The development of this detail used three-dimensional finite element analysis to check the splitting force and stresses in the vicinity of the lapped tendons. The geometry of this detail was also developed in three dimensions. Figure 6 shows the calculated stresses inside the beam due to the application of the post-tensioning forces. Figure 7 shows a picture of the beam where the lap was contained.

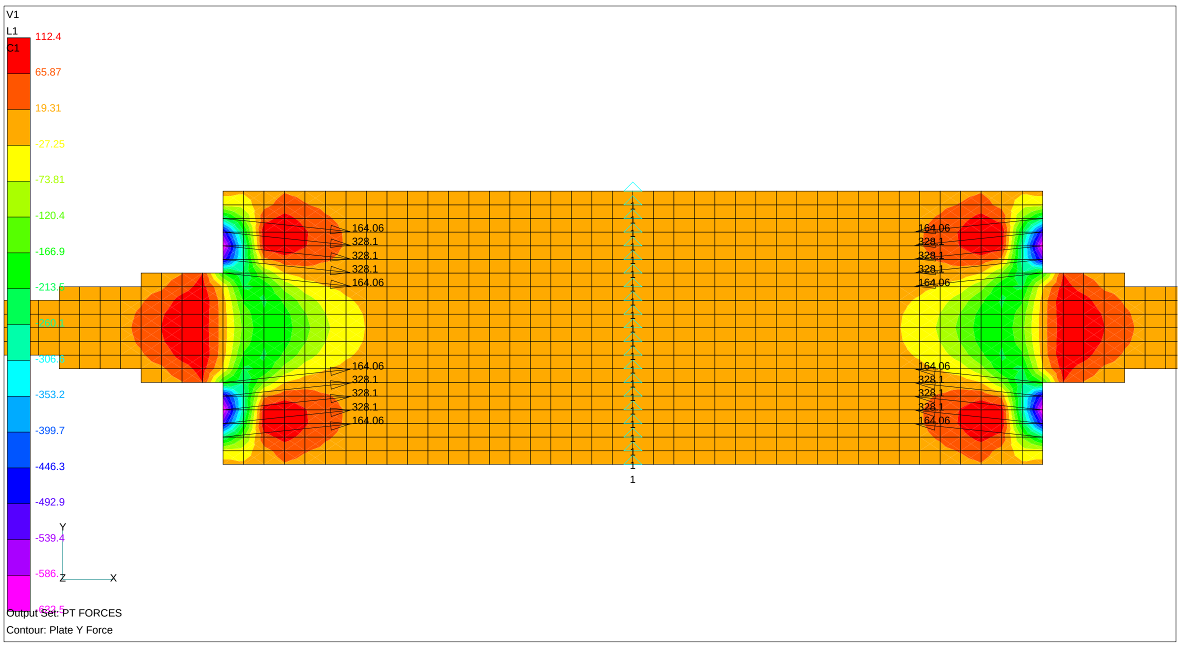


Fig. 6: Beam Stresses at PT Lap

Fig. 7: Beam Containing PT Lap

## DECK AND EXPANSION JOINTS

Although the 8 in. cast-in-place concrete deck was fairly typical, it did provide its own set of challenges. In addition to the transverse and sometimes significant quantity of longitudinal steel required to provide the couple for continuity between beam ends, adjustments had to be made to the vertical geometry of the deck to account for variations in the elevations of the beams after erection and the first stage of post-tensioning. Also, even though stay-in-place forms were allowed and preferred, the additional weight that they would impose on the beams had to be addressed.

## VERTICAL PROFILE ADJUSTMENTS

The camber of prestressed concrete beams is dependent on a number of factors, including the age of the beams and the actual prestressing force present in the beams. Although the computer models used to predict camber take these factors into account, it was impossible to correctly guess in all instances how old the beams would be and what the precise prestressed force in the beams would be, in addition to a number of other minor factors, when the bearing seats were cast.

A few instances occurred where the beams were older than predicted, as might be expected on a 22-month long project. Since camber increases with time, the deck thickness in these instances would have been reduced to as low as 5 in. above the beam flange at midspan. Although it could be proven through refined methods that this deck thickness would have been structurally adequate, the owner requested that the design deck thickness be provided throughout the structure. To accommodate this requirement, gradual profile adjustments were incorporated into the top of deck elevations. These adjustments consisted of a change in the grade of no more than 0.2% from that in the design drawings. Upon driving on the completed bridge at full speed, it is evident that these gradual profile adjustments, a simple solution to a relatively common issue for this structure type, do not affect the quality of the riding surface.

## STAY-IN-PLACE FORMS

Stay-in-place metal forms for construction of the deck were proposed for use throughout, with inserts cast into the top flange of the beams to receive the forms and provide a surface to weld the forms in place. The use of the forms was strongly preferred by Walsh Construction, but the additional dead weight on the beams would have required adjustments to the beam design. To alleviate these concerns, most of this additional weight was eliminated by filling the ribs of the forms with plastic foam pieces conforming to the shape of the rib before pouring the deck (see figure 8). The forms themselves were conservatively assumed to carry no load, so the deck reinforcement was designed to carry the entire design load as for a conventionally formed deck.


Fig. 8: SIP Forms with Styrofoam

Although metal forms are not routinely used in Illinois, the owner's specifications allowed their use on this bridge and the result, considering over 800,000 square ft. of deck area, was an extremely significant savings in cost.

## EXPANSION JOINTS

As mentioned above, one goal of the design-build team was to minimize the number of expansion joints. Of the nine joints incorporated, seven exceeded the range of a simple strip seal joint, so modular joints were used. The remaining two joints, one at each abutment, were standard strip seals.

The cost of a modular joint is significantly greater than that of a strip seal. However, the cost benefits of reducing the number of joints, primarily through the structural efficiencies of continuity over the piers, outweighed the high unit cost of the relatively few expansion joints on the project.

## SUBSTRUCTURE

The design of the substructure of this bridge not only had to accommodate up to 1200 kip bearing reactions but also had to be cost effective and easy to construct. To meet these requirements a multi-column pier system supporting a post-tensioned pier cap was selected. Each pier consisted of four 6 ft diameter columns supported on 6 ft 6 in diameter drilled shafts. The shafts are socketed into competent bedrock for a minimum distance of 15ft. Competent bedrock was encountered anywhere from just below the existing ground surface, to upwards of 55ft. below the ground surface. The tallest column, measured from finished ground to bottom of pier cap, exceeded 70 ft in height.

Supported by the multi-column system was a post-tensioned pier cap. Every pier cap on the bridge was 6ft.-6" wide. The depth of the pier caps varied depending on the amount of load it was subjected to; but at no time did the depth exceed 7 ft. All pier caps were 125 ft long and post-tensioned in two stages. With the full post-tensioning force present, the dead load of the superstructure is required to satisfy the stress requirements in the cap. This necessitated the requirement of the multi stage post-tensioning operations. The pier caps contained either 3 or 4, 19 strand tendons depending on the load the cap was subjected to.

All concrete in the drilled shafts and columns utilized a 4000 psi mix. All concrete in the pier caps utilized a 5000 psi mix. In total there were 34 piers. Fortunately, good bedrock was present at the project site. The allowable service stress on the bedrock for the drilled shafts was 100 kip per square ft.

Fig. 9: Typical Pier

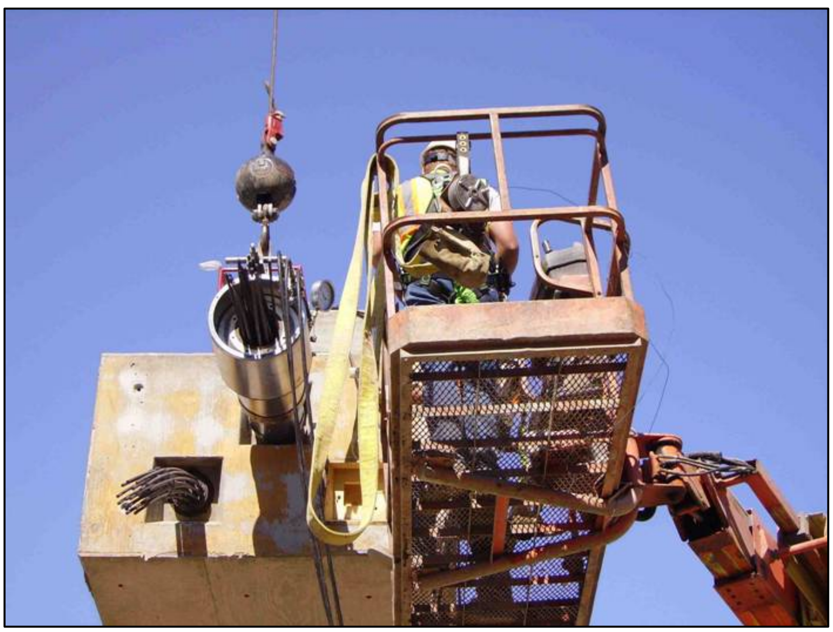

Fig. 10: Post-tensioning of Pier Caps

**PROJECT SUMMARY**

FINAL COST & TIMELINE

The bid for the alternate developed by the design-build team was approximately $125 million. This compared favorably against a $133 million competing bid for the as-designed segmental concrete box option and a $147 million competing bid for a design-build option utilizing a steel superstructure. These bids included the approach roadway on each end and relocation or adjustment of some of the features crossed by the bridge. Also note that all engineering fees associated with the design-build alternates are included in the above cost.

The unit cost of the structure itself was roughly $130 per square ft. Considering the long spans and tall piers, this unit price compares very favorably with other as-bid prices on the ISTHA system.

Construction of the project began in January of 2006 and was completed 22 months later in October of 2007. The project was completed in time for the grand opening of the Veterans Memorial Tollway extension on November 11, 2007. Particularly noteworthy is the fact that both design and construction were completed without delays to the project in the same amount of time it would have taken to construct one of the design-bid-build alternates presented in the contract documents.

OVERALL THOUGHTS

The benefits of precast, prestressed concrete, both pre-and post-tensioned, have been illustrated in detail. Through the use of prestressed concrete, ISTHA saved $8 million for nearly maintenance-free beams in a bridge that comprises the centerpiece of the vital Veterans Memorial Tollway extension in the southwest suburbs of Chicago.

It cannot go unmentioned though that it was ISTHA that had the foresight and fortitude to adopt the design-build concept and allow the use of this relatively new technology to the state of Illinois. It is evident from the success of this project that there is much for an owner to gain, both schedule-wise and cost-wise, from the design-build process. This is particularly evident if precast, prestressed concrete is allowed to compete on a level playing field with other superstructure types.