ACCELERATED REPLACEMENT OF A THREE-SPAN BRIDGE IN PENNSYLVANIA

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ABSTRACT

The Dillerville Road (S.R. 4009) project is a three-span bridge replacement over Amtrak Railroad. Due to heavy traffic volumes, limited detour routes, Amtrak involvement, and context sensitive issues associated with the project stakeholders, the Pennsylvania Department of Transportation chose to implement accelerated construction technologies for this project.

The proposed bridge is designed as a prestressed concrete adjacent box beam bridge with precast post-tensioned hammerhead piers, and precast post-tensioned stub abutments and wingwalls. A hydraulic gantry lifting system for demolition of the existing bridge and construction of the new bridge is envisioned by the owner and the designer.

The use of precast technology and innovative construction techniques is anticipated to reduce the original 18 month construction schedule to less than eight months. The cost savings associated with the reduced duration of the required railroad protection services offsets the higher initial direct costs of accelerated construction.

This paper discusses the Dillerville Road project's background and the unique solution proposed by the design team. In addition, some details and construction methods for the precast substructures are discussed and suggestions for considering the use of similar techniques on future projects are highlighted.

INTRODUCTION

In 2007, Pennsylvania Department of Transportation (PennDOT) District 8 will replace a deteriorating and structurally deficient bridge that carries Dillerville Road (S.R. 4009) over Amtrak in Lancaster, Pennsylvania. The existing bridge is a three-span, concrete-encased rolled steel girder on reinforced concrete substructures, which have deteriorated substantially, requiring temporary beam supports at one of the piers. The existing underclearance, horizontal clearances, structure width, and safety features are all substandard.

Discussions with project stakeholders early in the project development initiated the desire to replace the bridge as rapidly as possible. The typical project schedule necessary for replacement of a structure over a railroad was undesirable, especially with the additional consideration that railroad coordination efforts and costs multiply with longer construction duration.

The proposed solution is to implement rapid construction to reduce the anticipated project duration significantly when compared to a staged, conventional construction alternative. In addition to the benefits this provides to PennDOT, the contractor, the public, and stakeholders in the city of Lancaster, the reduction of railroad involvement during construction will benefit Amtrak, due to the railroad's limited resources. The project will also provide an evaluation of the proposed techniques for their use in similar projects.

BACKGROUND

Initially, a half-width staged bridge replacement was considered as the best option for this project, to avoid closure of Dillerville Road during construction. However, this option presented some problems. Due to the existing structure width and the traffic volume, only a single lane of the current two-way traffic could be maintained in one direction across the structure during the staged construction. Also, although this option permits traffic in one direction at all times during construction, staged construction would create a lengthy, two season construction schedule.

In addition to the construction duration, the Department was hesitant to plan for a partial width reconstruction after recently replacing Fruitville Pike over Amtrak, a similar bridge just east of the Dillerville Road crossing. This bridge was initially designed to be reconstructed in half widths; however, the poor condition of the existing structure resulted in construction under unplanned full closure of the bridge. The similar poor condition of the existing Dillerville Road bridge did not bode well for attempting partial-width staged construction.

For these reasons, PennDOT desired to replace the bridge under full closure. Garnering support would be challenging, however, due to the impact of closing Dillerville Road. Dillerville Road is an urban arterial carrying traffic of approximately 20,000 vehicles per day. The road serves as the principal connector between major thoroughfares leading into the city of Lancaster from the north. Closure of Dillerville Road will increase traffic on these adjoining roads and along the chosen detour route.

The use of a long detour and the additional congestion will inconvenience the traveling public. Additionally, local stakeholders, including several businesses and industries on Dillerville Road just to the south of the bridge site, will be seriously impacted while the road is closed. Although full closure will save time over the original staged replacement, the closure time needed to be further reduced. This initiated the drive for rapid replacement.

PennDOT also identified another reason for limiting the duration of the bridge replacement: potentially decreased railroad costs. Railroad costs for engineering, inspection, safety, and construction services associated with a bridge replacement can be a significant portion of the project cost. In fact, railroad fees can approach the actual cost of the bridge construction itself, depending on the duration of the project. This is evident from several recent bridge projects over Amtrak railroad in Pennsylvania, including the Fruitville Pike bridge mentioned above (see Railroad Costs section).

RAPID REPLACEMENT POTENTIAL

Based on potential time and cost savings, PennDOT decided that rapid replacement of the bridge under full closure of Dillerville Road was the best scenario. Although project stakeholders and the public were initially hesitant concerning full closure, they also generally agreed that closure was the best option, if the duration was as short as possible. PennDOT and the designer began to look at techniques to enable an accelerated replacement. The investigation included not only looking at ways to speed up the proposed bridge construction, but also how to demolish the existing bridge more quickly.

During preliminary engineering, a prestressed concrete adjacent box beam superstructure was chosen for its limited beam depth as well as its simplicity and speed of construction. Precast superstructures of this type have a documented history of success in Pennsylvania. On the other hand, precast substructures have only been used in limited applications, and with varying agreement as to their success. Primarily, the applications have included demonstration projects where partially precast abutments were used for single-span applications. Nonetheless, as the substructure construction was anticipated to consume over 60% of the time required to construct the Dillerville Road bridge, precast substructure elements were deemed necessary to the successful shortening of the construction. There is simply too much time involved in constructing standard cast-in-place piers and abutments.

Demolition of the existing structure was also identified as a significant time expenditure. Traditionally, the bridge would be demolished into manageable pieces to be removed from the site. Adding the complication of working over a railroad, where certain procedures can only be done within an acceptable window, the demolition of the existing structure alone could be expected to take up to two months. In addition, the temporary shielding required for demolition over a railroad would be challenging due to the limited vertical clearance. Thus, the demolition of the existing structure was quickly identified as a potential area of the project where time savings might be realized.

RAILROAD COSTS

Project	*Description*	*Duration**	*Bridge Cost**	*Railroad Cost**
Fruitville Pike over Amtrak *Lancaster, PA* *2003*	Two-Span Bridge Replacement	11 months	$7.7 million	$1.8 million
Foreman Road over Amtrak *Lancaster County, PA* *2006*	Single Span Bridge Replacement	7 months	$1.2 million	$1.0 million
Burd Street over Amtrak *Royalton, PA* *2007*	Single Span Bridge Replacement	10 months	$1.8 million	$1.3 million
Church Road and Merion Avenue over Amtrak *near Philadelphia, PA* *2006*	Two Bridge Replacements	24 months	$7.5 million	$3.5 million

**Estimate provided by designer or railroad where construction is not completed.*

Table 1. Cost of Railroad Services on Bridge Projects

The railroad costs shown in Table 1 depict the total paid to Amtrak by the bridge owner. On these projects, the majority of the Amtrak cost is for railroad protection and inspection while work is being performed on or over Amtrak property, and is directly proportional to the construction time. A smaller portion, typically around ¼ or less of the total fee, is cost for electric traction or signal work and is independent of the time spent on the project. As an example, PennDOT estimates that by shortening the construction duration on the Fruitville Pike bridge replacement from 24 months as originally planned to 11 months with total closure, approximately $1.7 million in Amtrak protection service fees was saved. Amtrak estimates showed the railroad costs for the Dillerville bridge replacement project to be similar, so savings in construction time of one month is estimated to save up to $200,000 in direct project costs to the owner. Also, reduced construction time over Amtrak provides significant benefit to Amtrak due to their limited manpower.

THE SOLUTION

PRECAST SUPERSTRUCTURE

The superstructure of the proposed bridge consists of three spans of 41'-0", 58'-4", and 58'-4". The 58'-4" spans approach the maximum length that can be accommodated while providing for improved vertical clearance desired by Amtrak. The span arrangement places piers near the permissible lateral clearances from the railroad, with new stub abutments constructed behind the existing full height abutments, and enables the use of a non-skewed bridge. Figures 1 and 2 show the proposed bridge layout.

The cross-section is 11-composite 48" x 21" adjacent box beams, supporting two-12' travel lanes, two-5' shoulders, and a 5' sidewalk. A 5½" minimum thick deck slab, sidewalk, and barriers will be cast on top of the prestressed beams, made composite by stirrups extending from the precast members. Grouted shear keys and transverse post-tensioning further connect the precast beams.

Other precast superstructure options using spread beams encroached on the vertical clearance desired by Amtrak. A steel beam superstructure was also considered, but required as many beams as the precast adjacent box beam superstructure, in order to limit superstructure depth, provide sufficient horizontal clearance, and limit deflection. A cost comparison showed the precast adjacent box beam superstructure to be the most cost effective, while providing a relatively simple, rapid construction.

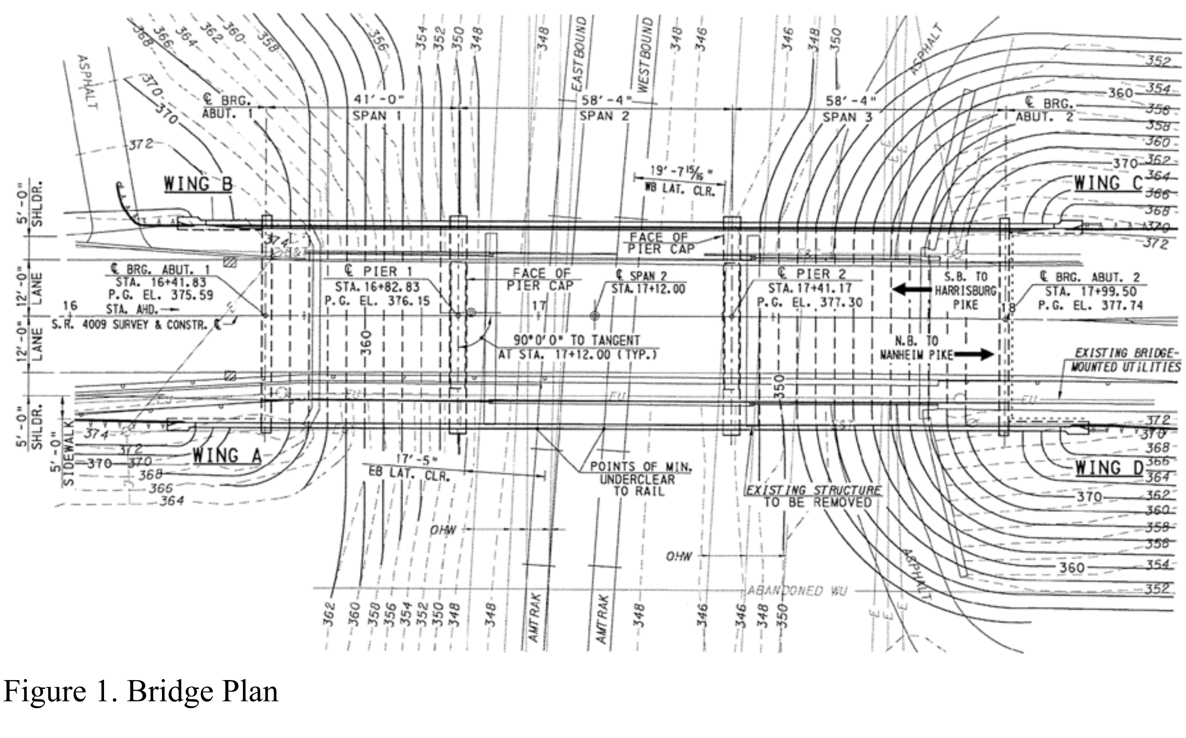

Figure 1. Bridge Plan

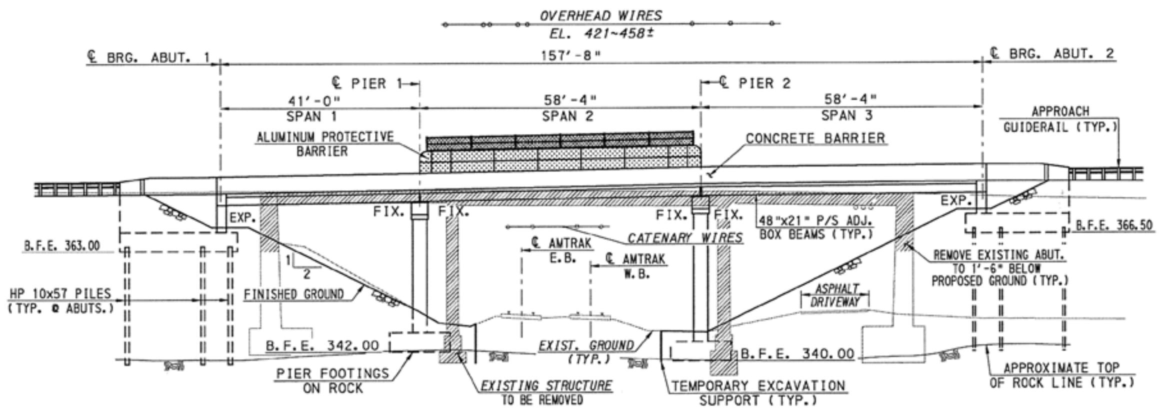

Figure 2. Bridge Elevation

PRECAST PIERS

Since the piers are somewhat more detailed and more time consuming than the stub abutments, the development of a precast substructure design began by investigating options for the piers. The investigation concentrated on simple, easy-to-fabricate details, while at the same time meeting both Amtrak and PennDOT requirements. Input was solicited from local fabricators. Early in the investigation, the footings for both the piers and abutments were chosen to be cast-in-place concrete rather than precast, since the abutments require driven piles, and the piers will be founded directly on weathered dolomitic limestone, for which PennDOT is more comfortable casting the footings directly against the rock.

The piers will be on railroad right-of-way and require Amtrak coordination to construct. Railroad requirements dictate that the piers on this structure are to be protected by either a separate or integral crashwall, at least 2'-6" thick and 12 feet long. An integral crashwall was chosen to expedite construction. Thus, if the pier were fabricated in hollow precast sections, as is often done to minimize weight, the pier would need to be filled with concrete to at least six feet above the top of rail. In order to avoid this, the choices of pier configuration were limited to a wall-type or hammerhead-type single column pier. A hammerhead-type single column pier configuration, made up of solid segments, was chosen, to provide a less imposing final product. The maximum piece weight is the pier caps, which approached 50 tons, a weight that was deemed acceptable after consultation with local precasters. Column segments are close to 40 tons. As discussed later in the paper, the proposed hydraulic gantry rigging system is capable of lifting and maneuvering these segments.

After some consideration, it was decided to use high strength (ASTM A722) threaded post-tensioning bars, rather than post-tensioning strands, to attach the pier segments together and to the cast-in-place footing. Although the bars are not capable of the high post-tensioning forces of the strands, they are less-susceptible to corrosion, and can be epoxy coated for additional protection. The configuration of the pier, necessitated by railroad requirements, is such that the pier sees relatively small loads for its size, so large post-tensioning forces are not needed. Sixteen 1-3/8" diameter post-tensioning bars are required per pier (eight along each long face of the column) to meet stress requirements in addition to ultimate strength requirements, particularly for seismic forces per Seismic Zone 2. The bars are embedded in the footing and extend through the pier cap for final post-tensioning. Several intermediate post-tensioning stages will be needed to stabilize the piers during construction and to squeeze the epoxy in the joints. Temporary bracing of the pier will not be required because of these intermediate post-tensioning stages. Bar couplers are provided at each joint to provide for intermediate post-tensioning and to facilitate placement of the precast segments over the bars. Intermediate groutable anchorage is specified at each intermediate post-tensioning location, so that grouting can be done after final post-tensioning only. The proposed pier configuration is illustrated in Figure 3.

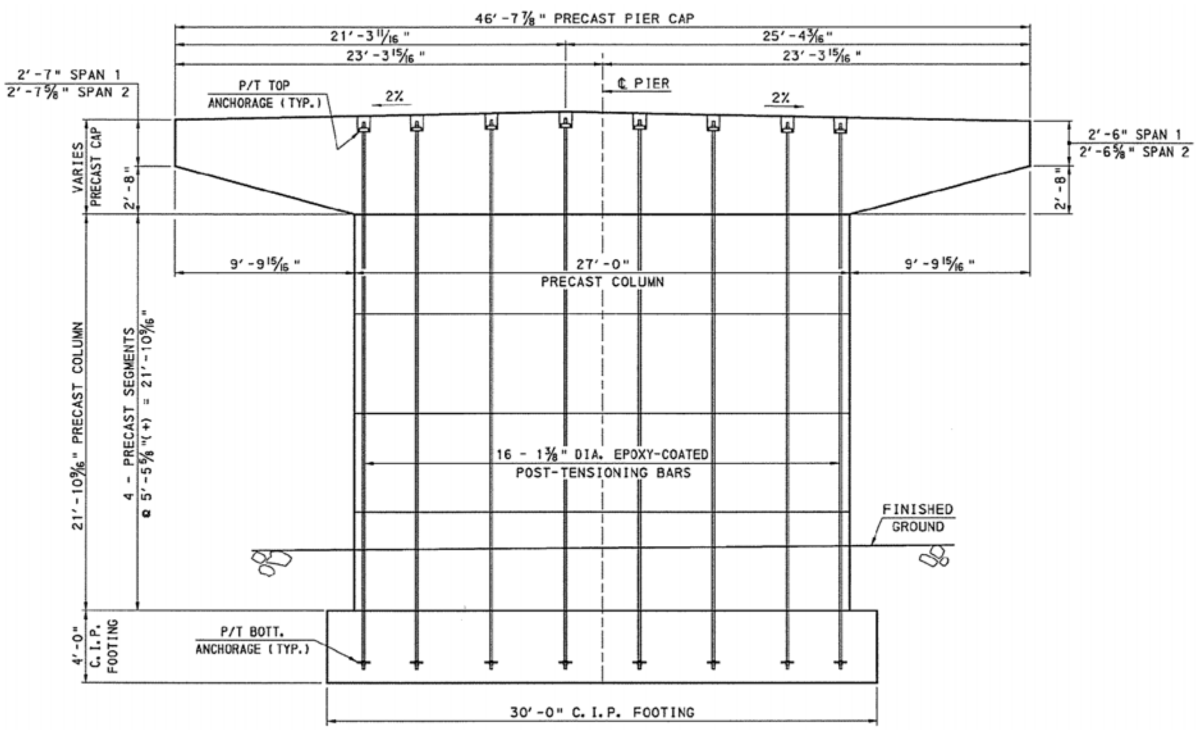

Figure 3. Pier 1 Elevation (Pier 2 similar) *Note: Intermediate anchorage and couplers not shown.*

Some unique details developed for the piers are as follows:

Precast Pier Column and Pier Cap Segments: Precast elements are proportioned and reinforced similarly to a c.i.p. option, with the exception of including ducts and anchorage reinforcing for the post-tensioning bars. As previously noted, each segment is solid. Joints between segments are to be match-cast.

Oversize P/T Ducts: Ducts are typically sized to be at least ¼” larger than a single bar. However, for these 1-3/8” diameter bars, a 3” inside diameter duct is specified, which offers additional room for maneuvering the segments over the vertical bars, adds additional grout cover protection over the bar tendons, and enables placement of bars and couplers inside the same duct (no larger coupler duct required at coupler locations). There is no appreciable increase in grouting time due to over-sizing these ducts.

Grouted Voids at Joints between Precast Segments: The piers’ precast column segments are 3’-0” wide x 27’-0” long. While the segments needed to be solid, a full joint between the segments creates far too much area to develop the required compression on each joint. For this reason, a 1’-0” wide x 26’-0” long void is detailed at the bottom of each segment, creating much less joint area and increasing compression. The void is only 1” to 2” nominal depth and can be grouted with the post-tensioning ducts after final post-tensioning, using grout tubes similar to those for the post-tensioning ducts, with an outlet near the 2” maximum depth. The grouted voids are not to be confused with the shear keys that are match-cast into the joints to provide shear resistance and to ensure proper placement of the precast segments. The shear keys are 12” x 3” and are spaced around the shaft between post-tensioning bar locations. The sole purpose of the grouting the voids is to mimic a solid shaft in the final pier condition. See Figure 4.

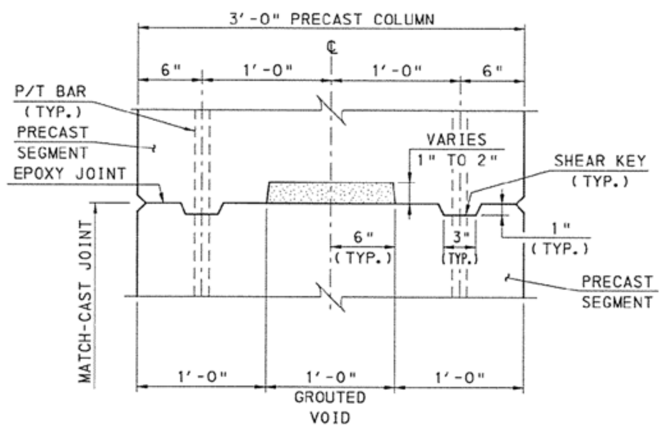

Figure 4. Grouted Void at Column Joint

PRECAST ABUTMENTS

While not as critical as the piers, additional time savings are realized by providing precast abutments and wingwalls. For simplicity, the abutments use similar detailing principles as the piers: precast stem segments detailed similar to a conventional stub abutment, post-tensioned to the c.i.p. footing with high strength bars. The abutments are not detailed with the grouted void noted above since less prestressing force is required than the piers. Segment weights are much less than the pier, so the stem for each abutment is split into two pieces with a vertical grouted joint between. There are no horizontal joints detailed except the joint between the c.i.p. footing and the precast stem, since the maximum height of the stem is only slightly less than six feet. U-type wingwalls are designed for each abutment corner, consisting of single precast segments for each wing with a maximum length equal to 20 feet, and a maximum height of 8 feet. The wingwalls will simply butt against the abutment stem and be post-tensioned to the same cast-in-place footing. Cast-in-place barriers will be added on top of the wingwalls at the same time the superstructure is constructed. Cheekwalls on the abutments will also be cast-in-place. Details of the abutments and wingwalls are shown in Figures 5 and 6.

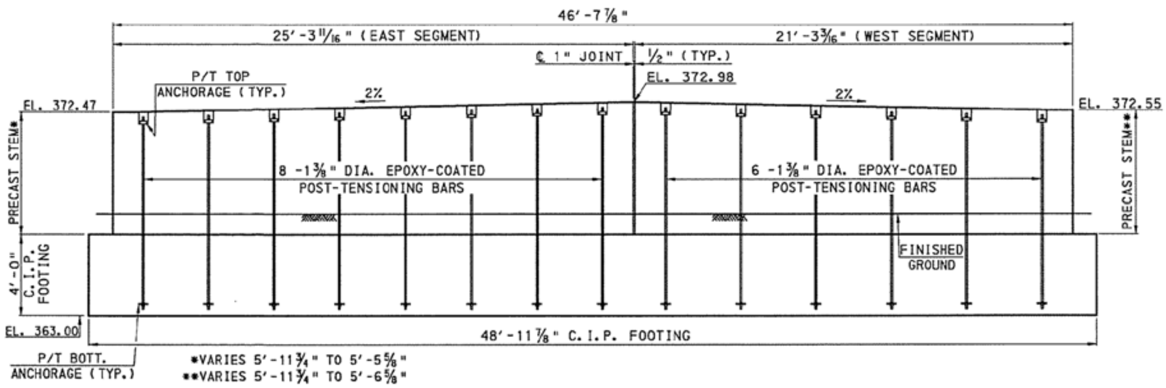

Figure 5. Abutment 1 Elevation (Abutment 2 similar)

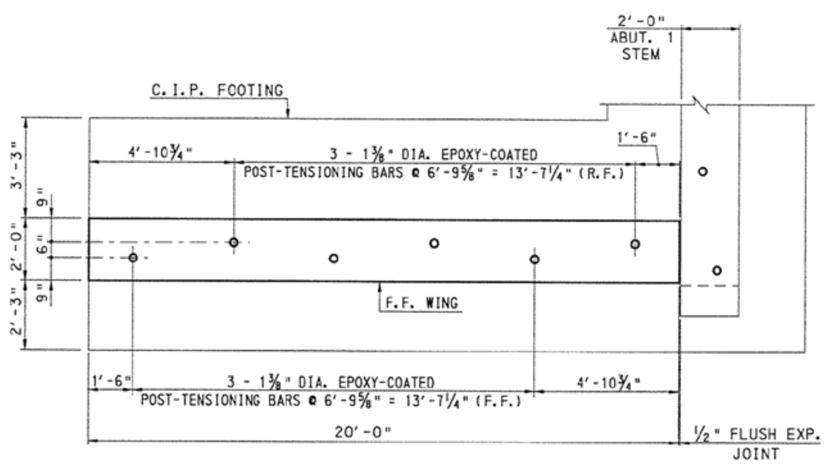

Figure 6. Wingwall/Partial Abutment Plan

The post-tensioning bars for the abutment and wingwalls are staggered so that every other bar is 9” from either the front face or rear face of the stem. This enables the stem to be 2 feet wide; additional stem thickness would have been required had two bars been placed at each face of the stem at the same location. The bars can all be post-tensioned at a final stage only.

The wingwall details are very similar except that mild reinforcement bar couplers will be provided in the top face for attaching bars for the c.i.p. safety barrier.

Integral abutments were not considered as a means of simplifying construction since PennDOT does not permit the use of integral abutments with adjacent beams. If steel beams were used, the overall bridge length becomes an issue, in addition to the large number of beams.

HYDRAULIC GANTRY RIGGING SYSTEM

An integral part of this rapid bridge replacement is the proposed use of a hydraulic gantry rigging system for both demolition of the existing bridge and construction of the new bridge. The use of such a system was considered due to the difficulties working over the railroad and around numerous overhead utilities at the site. The system consists of temporary support towers constructed to each side of the existing bridge which support gantry track beams running longitudinally with the bridge. The support towers can be constructed before the bridge is closed for demolition, using relatively small setup equipment and cranes. A motorized rolling hydraulic gantry capable of lifting and transporting heavy loads along the full length of the bridge runs on the track beams. At one end of the bridge site, the removed portions of the existing bridge can be transported using self-propelled heavy-load platform trailers. This system offers numerous advantages over using standard cranes:

- enables removal of each existing span *in entirety* and offloading to a temporary demolition area;
- eliminates need for railroad protective shielding during demolition;
- facilitates construction of new bridge, particularly the precast substructure segments;
- eliminates conflicts with overhead utilities, including railroad catenaries (no removal or relocations necessary, deenergizing only).

All of these advantages greatly reduce demolition and construction time, and increase ease and safety working around the railroad and numerous overhead power lines. It is estimated that use of this system will save over a month of road closure time for demolition of the existing structure alone. The concept of the rigging system is shown in Figure 7.

The hydraulic gantry rigging system was proposed after discussion with national crane and rigging companies. Similar custom solutions have been used successfully on numerous heavy/civil and industrial projects to enable off-site demolition and construction of large portions of a structure. The modular support towers, motorized hydraulic lifting system, and rigging are typically pre-engineered and reused for many projects. The entire system will be provided and constructed by the rigging contractor primarily before closure of the bridge for demolition. Preliminary estimates show the cost of the system for this project to be around $400,000. Much of this cost can be recouped in the reduced duration of railroad protective services during the existing bridge demolition alone.

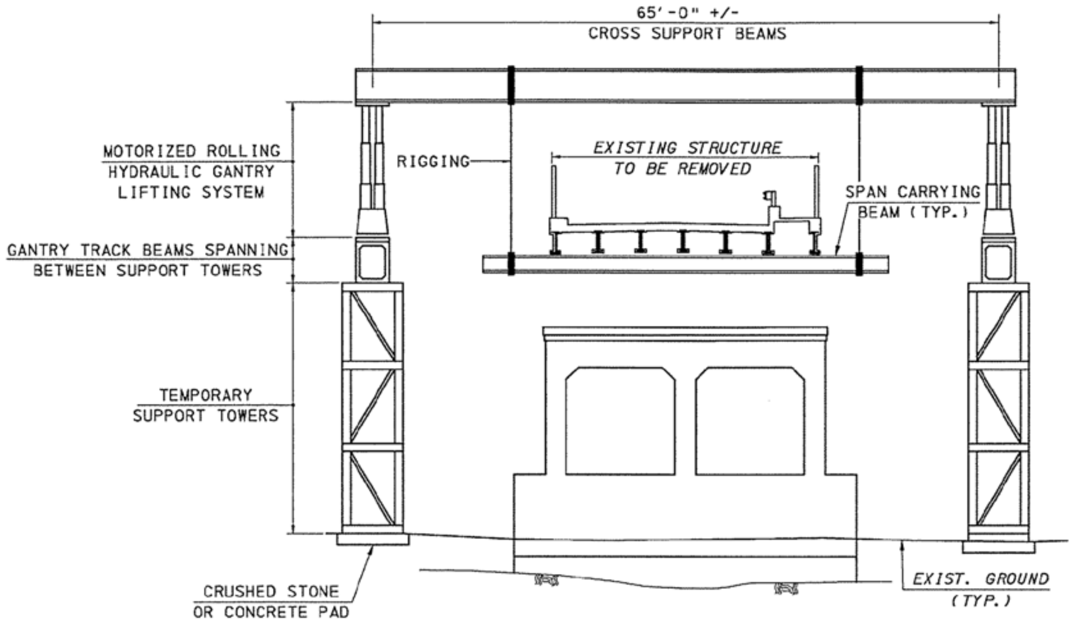

Figure 7. Hydraulic Gantry Rigging System

CONCLUSIONS

The precast substructures and simplified construction techniques outlined in this paper will enable a rapid replacement of the Dillerville Road bridge. The total road closure time is anticipated to be 6-8 months compared with:

- 18 months using the original half-width staged construction schedule
- 10-12 months under full closure, without rapid construction techniques

A general comparison of the major time savings due to the proposed techniques is presented in Table 2. Times are as compared with conventional construction under full closure of Dillerville Road.

Task	*Conventional Construction Duration*	*Rapid Construction Duration*
Existing Structure Demolition	8 weeks	2 weeks
Pier Construction	6 weeks	1.5 weeks
Abutment and Wingwall Construction	6 weeks	2 weeks

Table 2. Time Savings with Rapid Replacement

It should be noted that the bridge replacement itself is expected to last only three to four months maximum under rapid replacement, leaving several months to complete roadway work, both before and after the bridge work, within the same construction season.

The benefits to the project stakeholders, the traveling public and the city of Lancaster have been noted previously. Businesses near the bridge site will lose less business and money with the shortened closure of Dillerville Road. Major industry and the public will need to use the proposed detour for much less time. The city of Lancaster did not believe that full closure was the best option without the added time reduction of rapid replacement.

Perhaps the most interesting benefit, however, is the cost savings associated with the reduced duration of railroad protective services, which benefit the bridge owner and help offset the expected higher cost of the accelerated construction. With the anticipated reduction of about ten months in railroad coordination time versus a staged bridge replacement, these savings approach $2.0 million. Railroad coordination savings from the use of the hydraulic gantry system and precast components alone, about four months time, will be over half a million dollars. These savings will offset a good portion of the expected higher cost of accelerated construction. Still, the total cost of the rapid replacement is expected to be more than its cast-in-place counterpart in direct project costs, depending on the final railroad costs. This does not consider the cost savings to the surrounding community, however.

The use of precast substructures and a hydraulic gantry rigging system increases work zone safety for the contractor, the railroad, and automobile traffic. The precast piers can be assembled within the available construction windows with less disruption to the railroad. The hydraulic gantry rigging system is an integral part of this bridge replacement project, as working around the railroad and numerous on-site utilities would be very difficult for the large cranes that would typically be needed to handle the precast components.

PennDOT will likely try to use rapid replacement techniques on future projects requiring significant railroad coordination, and is anxious to see this project successfully completed due to its anticipated benefits. The techniques and details used in this project show promise in actually decreasing the overall cost of the project for the bridge owner and for the railroad. Railroad costs can be expected to increase in the future, increasing the benefit of rapid replacement of many of these railroad crossings. Bridge replacement projects over railroad crossings are promising areas to implement rapid construction techniques, especially where there is additional need to justify the solution as cost-effective.