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CSX Railroad Bridge Replacements

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ABSTRACT

This paper will discuss the geotechnical aspects of foundation design, construction support, and the team work necessary to replace and upgrade 14 bridges along an existing, trafficked rail line through Central Illinois. Seven bridges were constructed using previously completed designs and seven bridges were constructed using a multi-disciplinary design-build approach. The geotechnical team provided construction support for the Phase I bridges while concurrently conducting geotechnical investigations and developing design criteria for the Phase II bridges. The project also involved many logistical considerations including minimizing bridge "out of service time" to less than ten hours for the construction of each bridge, a very tight design and construction schedule, and challenging soil conditions.

The geotechnical engineering tasks associated with each Phase II bridge included: conducting subsurface investigations; performing deep foundation analyses and developing site specific design criteria; and supporting preparation of the foundation design drawings, specifications, and estimated material quantities. Design development was an iterative teamwork exercise involving the geotechnical, structural, and hydraulic engineers working jointly with the construction team. As the hydraulic and structural analyses progressed, the input parameters for the foundation design changed requiring modifications to the geotechnical design. The design process was also influenced by construction observations during Phase I of the project. These observations provided valuable installation data for the geotechnical design to provide a more cost effective and efficient design for the Phase II bridges.

INTRODUCTION

Need for Bridge Replacement

CSX Transportation, Inc. (CSX) required replacement of 14 existing open-deck timber bridges along its rail line between Decatur, Illinois and the Illinois/Indiana State Line in central Illinois. The bridges span small streams and drainage ditches in a primarily agricultural area. The existing bridges had reached or exceeded their design life, and CSX needed to upgrade the line for use by faster and heavier trains to meet customer demands.

Phase I of the project included construction of seven bridges previously designed by others, and Phase II included design and construction of seven additional bridges. CSX had initially planned to replace all 14 bridges using the traditional "design-bid-build" delivery method, but due to schedule and other reasons, they switched to the design-build approach. Construction on the Phase I bridges was accomplished concurrently with the start the detailed design on the Phase II bridges.

Project Constraints

The CSX Decatur Subdivision is an active section of track and long disruptions to freight traffic were not allowed. As a result, all bridge replacements had to be finished within a 10-hour outage window.

The project as a whole had a very aggressive schedule to finish design of the Phase II bridges and construction of all Phase I and Phase II bridges. Construction had to be conducted so that all bridges were open for traffic September 1, 2012, approximately 10 months after award of the contract.

To meet the project schedule, construction of the seven already designed Phase I bridges began immediately upon award of contract, while concurrently starting work on the seven fast track design-build Phase II bridges. Items requiring a long lead time for procurement (steel pipe piles, precast concrete box beams, and other prefabricated pile components) were released for fabrication prior to completion of the Phase II design. This allowed for transition from Phase I to Phase II bridge construction with no lost time. The compressed schedule required the design-build team to focus on critical path elements, value engineering, and close coordination with CSX.

Construction of the bridge foundations and substructures, as well as general site improvements (grading, drainage, erosion control, etc.) was accomplished by subcontractors, while replacement of the open-deck bridge structures was accomplished by CSX forces working in concert with the contractor.

An additional project constraint was that the basic design of the bridges had been selected by the client.

Project Setting

Fourteen open-deck timber bridges were replaced along the CSX Decatur Subdivision track in Central Illinois to upgrade the level of service on the line. The seven 'design-build' bridges were constructed using a multi-disciplinary team approach that included survey, geotechnical engineering, hydraulic engineering, railroad engineering, structural engineering and construction management.

The geotechnical engineering tasks associated with each Phase II bridges included: conducting subsurface investigations at each bridge location; performing deep foundation analyses and developing site specific design criteria; and supporting preparation of the foundation design drawings, specifications, and estimated material quantities. Design development was an iterative teamwork exercise involving the geotechnical, structural, and hydraulic engineers working jointly with the construction team. As the hydraulic and structural analyses progressed, the input parameters for the foundation design required modifications to the geotechnical design. For example, a scour analysis was performed for each bridge to determine the potential exposed height of the foundation piles. The exposed height of the piles was used as an input parameter for the lateral analysis to determine appropriate pile lengths at each bridge bent.

The geotechnical team also provided construction support for the Phase I bridges while concurrently conducting geotechnical investigations and developing design criteria for the Phase II bridges.

PHASE II SITE INVESTIGATION

All site investigation work was performed in compliance with CSX and Patrick safety procedures. All site personnel (including drillers) were required to have completed E-Railsafe training and CSX-approved FRA Roadway Worker Protection training.

A total of fourteen soil borings were drilled during the Phase II subsurface exploration program, with two borings drilled at each of the seven bridge locations. The borings were drilled along the track centerline, approximately 15 feet behind the existing bridge abutments. Site exploration activities required daily coordination with CSX personnel to limit the 'out of service time' and avoid interference with train schedules during the time the drilling equipment was working on the tracks. Drilling work on the tracks was limited to a maximum of eight hours each day, and in some cases the crew and drill rig were required to vacate the site during this window due to oncoming train traffic. Figure 1 illustrates the typical arrangement of the drill rig adjacent to one of the bridges scheduled for replacement.

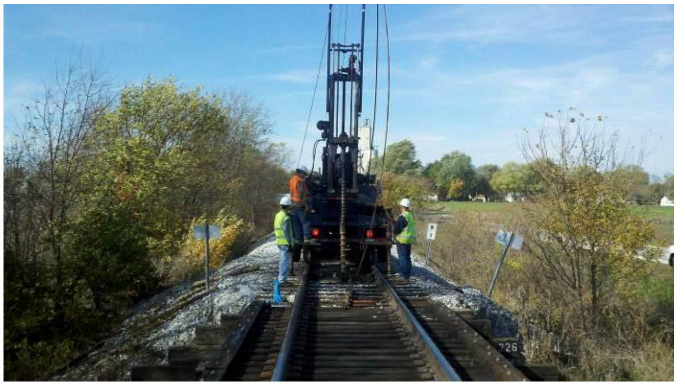

Fig. 1 Geotechnical investigation drilling operation

The soil borings ranged in depth from 79 to 85 feet below ground surface. Drilling began at the westernmost bridge location (BD 263.89) on the Decatur Subdivision and the borings were drilled in sequence working from west to east.

The driller was selected based on their successful experience with the Phase 1 borings as well as having the necessary safety training and proper equipment for the work. All borings were advanced with a rotary CME-55 hi-rail mounted drill rig equipped with 3.25-inch I.D. hollow-stemmed augers and a manual Standard Penetration Test (SPT) hammer raised using a cathead. Soil samples were collected at 2.5-foot intervals beginning at a depth of 3.5 feet below the ground surface and extending to a depth of 15 feet, and at 5-foot intervals thereafter to the terminal depths of the borings.

In several boring locations, hard, dry soils and difficult drilling conditions required the addition of water to annular space between the drill string and the borehole wall to lubricate the augers. In many of the borings, confined sand and gravel layers were also encountered at depth. On several occasions, water was introduced into the borehole during the drilling process to prevent the augers from locking up in the granular deposits.

In some cases, the addition of water during the drilling process prevented accurate groundwater observations in those borings.

However, other methods were used to estimate depth to groundwater such as changes in soil color.

Pocket penetrometer readings and RIMAC tests were performed in the field to estimate unconfined compressive strength on cohesive samples.

PHASE II BRIDGES - PRELIMINARY DESIGN

The Phase I bridge foundations were designed using a 30-inch, 0.625-inch thick wall, open-ended steel pipe filled with reinforced concrete. CSX requested the Phase II bridges incorporate the same foundation design.

Preliminary pile depths for the Phase II bridge piles were determined based on ultimate design capacity (i.e., vertical load-carrying capacity). Bridge loads were estimated using preliminary static loads (i.e., preliminary weight of the structure, span lengths, etc.) and projected live loads (Cooper E-80 railroad loading). Based on site soil conditions and the preliminary loads, the selected pile types generally carried the vertical loads in skin friction, and minimum tip elevations were initially established. The preliminary depths did not include an embedment depth for lateral stability and fixity of the piles.

An investigation report was prepared with preliminary design recommendations and assumptions for installation of the pipe piles, including pile lengths and estimated pile tip elevations. The preliminary recommendations were also based on several design assumptions:

1. Skin friction would be based on the steel to soil interaction
2. Plugging of piles would occur during driving and contribute to the tip resistance
3. Cobbles may interfere with driving

In addition, lateral analysis was not performed during preliminary design and could ultimately be the governing criteria regarding embedment depth. Each of these design assumptions would be later confirmed or refuted during the construction support and observation during the Phase I pile installation which would then in turn help to refine the final Phase II design.

LESSONS LEARNED DURING PHASE I BRIDGE INSTALLATION

During construction of the Phase I bridges, piles at several bridge locations were terminated in saturated sand zones and subsequently experienced “blow in” or heave (condition where saturated sands below the water table mobilize and flow into the bottom of the pipe pile until reaching equilibrium). Based on this observation, Patrick recommended that Phase II bridge piles should not be terminated in any loose saturated sand zones. At the Phase II bridge locations where loose saturated sands were anticipated at the approximate pile bearing depths, pile lengths were extended to terminate in the deeper, silty clay layers that are not susceptible to “blow-in”.

A set of Pile Installation Notes was developed by the geotechnical engineering and structural engineering teams for use during construction. These notes were incorporated into the Project General Notes for construction of Phase II.

If during the pile driving process the driving criteria indicates that the axial pile capacity has been achieved before the pile reaches the recommended pile tip elevation, the piles can be accepted if they meet the minimum depth criteria for lateral resistance, are substantially close to recommended minimum tip elevation (within 3-5 feet), and they meet minimum load-carrying capacity after testing using a pile driving analyzer (PDA). If the pile reaches the recommended pile tip elevation prior to reaching the driving criteria required for axial capacity, the pile was driven deeper until the final acceptance criteria were met. Figure 2 illustrates the driving equipment required to advance the 30-inch diameter pipe piles.

Fig. 2 Pile driving operations next to existing bridge

PHASE II BRIDGES - FINAL DESIGN

<u>Axial Design</u>

Steel pipe-pile foundation designs for each bridge location were based on the final structural axial loads and the calculated ultimate load-carrying capacities. Recommended pile lengths were calculated for each location based on the established design criteria.

The “blow in” observed during the Phase I pile installation coupled with the anticipated sand layers observed during the Phase II Site Investigation, were factored into the final pile lengths at each bridge. At the locations where these conditions were anticipated, the recommended pile lengths were extended to deeper clay layers that would not be as susceptible to the “blow-in”. The depth and location of unsuitable soils, such as the organic layers or sand zones identified in the soil borings, was also considered and minimum pile depths were extended below this strata.

With the anticipated length of piles between 30 to 55 feet, in addition to the structural loads (i.e. dead and live loads), the weight of the steel pipe and the reinforced concrete were added to the maximum structural axial loads to determine the total axial load on the pile.

The design depths were estimated based on the pipes reaching twice the calculated ultimate capacity required for loading. This design process produced a minimum Factor of Safety of 2.0 for the static loading case.

Design depths for the Phase II bridge piles were also based on the assumption that plugging of the steel pipes would occur during driving as was observed during the Phase I bridge installation.

An additional design recommendation included performing a pile driving analysis at each bridge location during the installation in order to meet the final acceptance criteria.

Lateral Design

Piles for each bridge were analyzed using the computer program L-Pile Version 6.0. The L-Pile lateral analysis calculation method solves nonlinear differential equations that model the behavior of the pile-soil system using Reese's p-y method of analysis. L-pile was used to determine the "point of fixity" (depth at which the pile is no longer in bending). The minimum tip elevation was established as 10 feet below the point of fixity. The appropriate soil parameters, pile loads (axial and moments) and geometry were entered in the program for each pile location.

The pile moment arm or pile stick-up was calculated based on the distance from the predicted scour to the top of the pile cap.

For each bridge, a minimum of three loading scenarios were evaluated:

1. End Bent with axial loading
2. Intermediate Bents with axial loading and moment
3. Intermediate Bents for seismic condition

Cases 1 and 2 considered the maximum longitudinal loading applied to the free head condition of the bents as the "worst-case" scenario for lateral loading. These cases were used in conjunction with the required load-carrying capacity to determine the minimum tip elevations for each set of bents at each bridge location.

Seismic Design Considerations

In addition to the soil parameters required for the structural design and lateral analysis, seismic design information was also provided. The site conditions were evaluated using the 2009 International Building Code (IBC 2009) and resulted in a Site Classification of D and the resulting site coefficients. In addition to the IBC, the AREMA 2007 Seismic Design for Railway Structures was reviewed to provide site coefficients for structural design.

Design Optimization

As the Phase II bridges were to be delivered to CSX using the design-build delivery method, it was important to optimize the both the design approach and construction procedures. Design optimization was intended to streamline and expedite the material procurement process so the bridge construction could proceed on schedule and within budget. Considerable effort was expended analyzing soil conditions, lessons learned during Phase I bridge construction, and coordinating between the geotechnical, structural, and construction management teams.

PHASE II CONSTRUCTION OBSERVATIONS AND DESIGN SUPPORT

Construction Observation

Patrick's geotechnical design team provided onsite technical support during installation of the pile foundations. This included counting pile hammer blows during pile driving, verifying proper dynamic pile testing procedures, troubleshooting installation issues, and acting as liaison with the design team in the office.

Figure 3 illustrates the use of an auger to clear soil from inside the pipe pile in preparation for setting the reinforcement cage and concrete.

Fig. 3 Clean out operations of soil plug within driven pile

Wave Equation Analysis of Pile Driving (WEAP)

Bridge-specific Wave Equation Analysis of Pile Driving (WEAP) was performed by others prior to mobilizing to each bridge. The purpose of the WEAP analysis was to determine the general suitability of the proposed driving system to install the piles to the required ultimate pile capacity within the typical driving stress limits and with a reasonable driving resistance.

WEAP analyses were performed for both a plugged and unplugged soil model using GRLWEAP internal static analysis. For the plugged case, it was assumed that only external shaft resistance developed and the toe resistance occurred over the full toe area enclosed by the pile outside diameter. For the unplugged case, it was assumed that the internal shaft resistance was 1/3 the external shaft resistance and that the end bearing developed only on the steel area at the pile toe. The WEAP provided initial pile driving criteria and confirmed the appropriate hammer size and energy.

Pile Driving Analyzer (PDA).

The geotechnical investigation, design and WEAP analyses are predictions of how the soils will behave during the actual pile driving and provide the data with which to estimate pile lengths. However, it is during construction that the capacity of the piles is proven using a Pile Driving Analyzer (PDA) to establish the official driving criteria and confirm the results of the WEAP and static analyses.

Fig. 4 PDA measurements during pile driving operations

Using the real-time data from the PDA, the actual tip elevations are determined for the individual piles based on the required capacity. The real-time data is collected via a calibrated gage physically attached to the pile approximately 6 feet below the head of the pile. Two strain transducers and two accelerometers are bolted to the opposite sides of the pipe pile to monitor strain and acceleration. The signals are converted to forces and velocities using the PDA. The PDA calculates the maximum transferred hammer energy, the maximum compression stress at the gage location, and estimates the capacity using the case method. Force and velocity records from the PDA are viewed during driving to evaluate data quality, soil resistance distribution, and pile integrity. The data are stored for subsequent analysis. Figure 4 shows the PDA sensors attached to the pipe pile.

If the piles are not driving as predicted, the installation contractor must make adjustments to insure adequate capacity while meeting the project schedule.

In most cases for the Phase II bridges, the results of the PDA analyses resulted in the piles being driven deeper than the design elevations.

After piles were driven to capacity and the required depth for fixity, the piles were rough cut approximately 6 inches above the final cutoff elevation.

Figure 5 shows the newly installed pipe piles with the cone-shaped pile caps which will accept the horizontal bent cap after the existing bridge is removed.

Fig. 5 Existing wooden bridge with new piles and pile caps in foreground

Design Adjustments

Installation of the Phase II bridge foundations was complicated by challenging soil conditions. Each Phase II bridge site is located at or very near the southern extent of Ice Age glaciation, and therefore the soils generally include very stiff glacial tills but were subject to localized variations in bedding layers and large granular deposits. These variations in soils made it difficult to accurately predict the pile termination depths from bridge to bridge. At some bridges where additional pile length was required, field splices were performed to add additional pile, and the piles were driven deeper in order to reach the design criteria.

The costs involved with providing additional pile lengths that were spliced in the field generally outweighed the costs of splicing together longer pile lengths prior to delivery to the site. Patrick’s geotechnical staff worked closely with the pile driving crew and supplier to minimize the cost and schedule impact of these items.

Throughout the construction process, Patrick continually evaluated data collected from the PDA as each bridge was constructed in order to make modifications to the predicted pile lengths for the subsequent bridges. This process enabled Patrick’s design team to work with the construction team to confirm the field analyses and order additional pile where necessary to minimize delays and costs that could affect the project schedule and budgets. As a result, Patrick was able to maintain adequate pile material on site for each bridge to accommodate deeper piles as needed.

At one bridge location, BD 256.4, the coordination between the team members and the construction schedule allowed Patrick to take advantage of the favorable properties of the local clay soils. A phenomenon called "pile set" can increase the actual bearing capacity of glacial soils due to the dissipation of pore pressures in the immediate vicinity of the driven pile.

Fig. 6. Removal of existing bridge and placement of new concrete structure

In general, as piles are driven, the groundwater pore pressures increase immediately around the pile. These pressures then reduce the bearing capacity of the pile temporarily. However, once pile driving ceases, pore pressures start to dissipate and the bearing capacity increases.

The increase in bearing capacity can be demonstrated during construction by 'restriking' the piles after several days and monitoring through the PDA the increase in resistance of the driving effort. At one bridge, the piles were allowed to set for several days, and then were restruck. It was determined that several of the piles had attained the necessary capacity at the predicted elevation and no further driving was required.

The project schedule demanded that the pile driving crew keep moving, and the cost of remobilizing the pile driving equipment from bridge to bridge to restrike piles was cost-prohibitive relative to splicing and driving additional pile length. However, in this particular case, the crew was not ready to demobilize until the following week. Patrick's engineers seized this opportunity to verify the pile capacity from set-up and avoid the additional work associated with splicing piles and driving them deeper.

CONCLUSIONS

Installation of the Phase II bridge foundations was complicated by challenging soil conditions. Each Phase II bridge site is located at or very near the southern extent of Ice Age glaciation, and therefore the soils generally include very stiff glacial tills but were subject to localized variations in bedding layers and large granular deposits.

One of the key differences noted between the Phase I piles and Phase II piles was that the soil plug that developed during the Phase I pile driving did not similarly develop during the Phase II pile driving. As a result, the Phase II piles tended to cut through the soil and the pile did not develop as much load-carrying capacity in the stiff glacial soils. These piles subsequently had to be driven deeper to reach harder, stiff material and satisfy the PDA acceptance criteria.

The relatively subtle variations in soil conditions required longer piles to carry the design loads. The relatively soft clay above the hard till did not appear to plug in the bottom of the piles as the piles were advanced. The Phase 2 piles cut this soil creating a cylinder of soil inside the pile like a cookie cutter slicing through the softer clay until encountering the harder till soil.

The larger diameter piles used for this project increased the potential variations in soil plugging type and depth, and therefore had a significant impact on the depth required to reach pile capacity.

Fig. 7 Installation of rail onto new concrete bridge structure

Figures 6, 7, and 8 show preparation and installation of the bridge bent caps and bridge deck and an aerial view of a completed bridge.

RECOMMENDATIONS

Patrick learned several valuable lessons from its involvement in construction of the Phase I bridges and subsequent involvement in the Phase II bridges using the design-build delivery method:

1. Perform WEAP as soon as possible to utilize estimates to verify pile lengths
2. Increase FOS requirement to allow for variation between static design and actual dynamic proofing of the piles
3. Take advantage of restrike whenever possible
4. Use appropriate construction procedures to prepare for blow-in if that is a possibility
5. Avoid estimates for pile length that are overly refined. Instead, allow for some additional pile during ordering for variations. Delays in procurement as well as splicing costs will likely outweigh additional pile length.

Fig. 8 Aerial view of final Bridge 256.4

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