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Carlos M. Sevilla
Sverdrup Civil, Inc., St. Louis, Missouri

Joseph M. Gilroy
TAMS Consultants, Inc., Belleville, Illinois

John Jenkins
Hanson Engineers, Inc., Springfield, Illinois

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Carlos M. Sevilla
Sverdup Civil, Inc.
St. Louis, Missouri

Joseph M. Gilroy
TAMS Consultants, Inc.
Belleville, Illinois

John Jenkins
Hanson Engineers, Inc.
Springfield, Illinois

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ABSTRACT

A case history on the design and installation of pile foundations for support two taxiway bridges at the MidAmerica Airport is presented. The pile foundations included the use of heavy HP steel piles designed as friction piles embedded in a stiff clayey silt till. Typical pile foundations in the area consist of H-piles or closed-ended pipe piles driven to bedrock. The use of slightly shorter friction piles allowed substantial savings due to the large number of piles required to support the heavily-loaded bridges. An extensive dynamic testing program was performed to measure allowable pile capacities and soil setup. The taxiway was constructed across a wetlands area with soft, compressible soils where embankment loads caused up to 1-1/2 feet of settlement and negative skin friction on the piles. Longer piles with the same cross sectional area were used to offset the negative skin friction.

KEYWORDS

Friction H-piles, glacial till, PDA, CAPWAP, soil setup, restrikes, negative skin friction

INTRODUCTION

Site Description

The St. Louis Metropolitan area is rapidly growing with the areas east of the Mississippi River, including St. Clair County, Illinois, showing a good portion of this growth. Since the east metropolitan area depends on Lambert International Airport, located to the west of the metropolitan area, supplemental east-side airport locations had been considered over the years. Scott Air Force Base, located approximately 24 miles east of St. Louis, Missouri, was proposed for construction of a new east-side commercial reliever airport. The project included expanding the existing military infrastructure by constructing an adjacent civilian airport. This joint-use airport, named MidAmerica Airport, is currently under construction and will be opened for operation in October 1997.

The airport construction included a new 10,000-foot runway, a parallel taxiway east of the base and a cross-over taxiway to connect the two runways. This crossover taxiway was constructed to allow movement of fully loaded airplanes between the base and the civilian airport. Construction of the cross-over taxiway across the Silver Creek floodplain required 20-foot to 25-foot high, 200-foot wide embankments and two taxiway bridges each including a 110-foot main span and two 80-foot long relief spans. The bridges are 149 feet wide and were designed to support a 1.25 million pound aircraft. Deck elevations are approximately 18 the floodplain below. The bridge superstructures consist of post-tensioned box girders supported by concrete closed abutments and solid piers with steel H-pile foundations. Other structures constructed near the cross-over taxiway included a 217-foot high control tower and a double barrel elliptical underpass to carry the taxiway over the existing base perimeter road.

The design of the taxiway bridges was performed by Sverdrup Civil, Inc. of St. Louis, Missouri in the conceptual and preliminary design stages, and Hanson Engineers, Inc. of Springfield, Illinois during final design. Both consulting firms worked under a subconsultant agreement with TAMS Consultants, Inc. of New York who acted as general design consultant and program manager.

Regional Geology

The regional geology reflects different periods of soil deposition and scouring by glacial action. The site lies to the south of Wisconsinan-aged drift and, therefore, Wisconsinan deposits at the site consist of loessial deposits. Quaternary deposits include re-deposited loess in the form of alluvium along the Silver Creek floodplain. Glacial deposits underlying the loess and/or alluvium consist of heavily overconsolidated glacial till with outwash deposits of sand and gravel above. Isolated lenses of glaciolacustrine dark organic silt and/or sand outwash deposits also occur on top of the bedrock. The upper bedrock consists of interbedded shale and limestone.

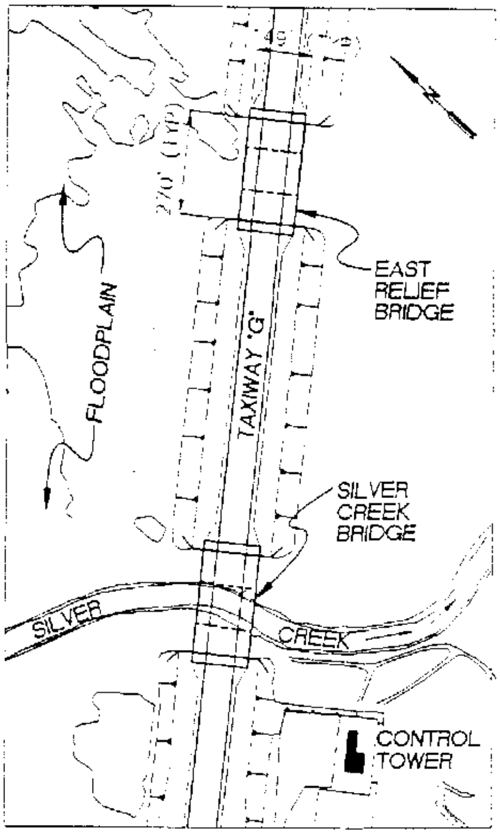

Fig. 1 Bridge locations

<u>Subsurface Conditions</u>

The thickness of each soil stratum varied between explorations, but in general alluvium with a thickness of approximately 20 feet was encountered overlaying lacustrine deposits. The lacustrine deposits have average thicknesses of 10 feet and overlie glacial deposits with thicknesses of approximately 50 feet. Pockets of organic materials consisting of black peat, organic silt, buried trees and stumps were encountered within the lacustrine deposits. Bedrock was encountered at depths of about 80 feet.

The alluvium consists of a medium stiff light brown clayey silt. The lacustrine deposits consist of stratified soft clay and loose fine sand with lenses of black organic silt and other organic materials. The glacial deposits included an upper 10-foot thick layer of medium dense coarse to fine sand with trace of fine to medium gravel underlain by a 40-foot thick layer of medium stiff to very hard gray clayey silt till with trace of organic materials and shell fragments. The glacial till has typical shear strengths of about 4.5 tons per square foot (tsf), but shear strengths around 2.5 tsf were also obtained from some of the samples. These numbers reflect pocket penetrometer measurements on split-spoon samples and unconsolidated undrained triaxial tests performed on Denison tube samples. The upper bedrock surface consists of interbedded limestone and shale. The limestone consists of a hard, fine-grained fresh limestone and the shale consists of a soft clayey shale.

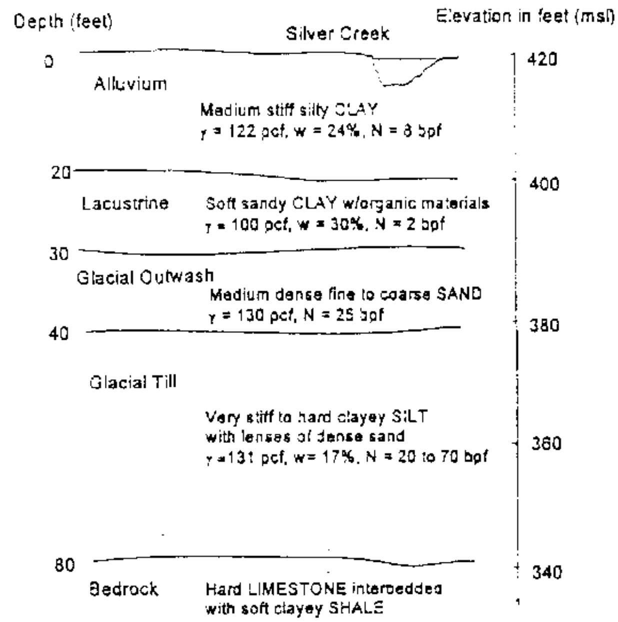

Fig. 2 Generalized stratigraphy and average soil properties

Groundwater conditions along the crossover taxiway reflect a slow time-dependant recharge from Silver Creek's surface water and seasonal near, or above, surface groundwater levels. The markedly different permeabilities between the alluvium and the underlying sand, that may be locally recharged from the uplands, showed temporary excess hydrostatic pressures in the sand. Slight artesian conditions were observed in some of the test borings and cone penetrometer tests.

FOUNDATION DESIGN

<u>Preliminary Analysis and Design</u>

During preliminary design, foundation systems were evaluated based on the following known bridge characteristics: (1) wide, heavy bridges would require a large number of foundation elements over large pier or abutment footprints; (2) heavy live loads on the bridge piers and abutments would make high capacity piles or drilled shafts economical; and (3) relatively high abutments and seismic loading would lead to high lateral loads. A number of deep foundation alternatives were evaluated including drilled shafts bearing on glacial till, open-ended large-diameter pipe piles, precast concrete piles, closed-ended small diameter pipe piles and steel H piles.

Drilled shafts bearing on the upper portion of the glacial till, proposed in the conceptual design stage, were eliminated from consideration because water-bearing layers of sand were encountered within the glacial deposits. In addition, it was presumed that high groundwater levels in the Silver Creek wetlands would make slurry excavation support difficult during part of the year.

A drivability study was performed as part of the pile selection process during preliminary design. The Wave Equation Analysis of Pile Driving (GRLWEAP, Pile Dynamics, Inc., Cleveland, Ohio) computer program was used to complete the drivability study undertaken to evaluate pile drivability, stresses and hammer requirements and possible pile penetration depths for the various pile sections. This effort was intended to primarily evaluate pile drivability in the till, but it also allowed a comparison between the axial load carrying capacities estimated using static methods and the ultimate capacities generated by the program. The pile sections evaluated included an HP14x89, an open-ended 24-inch diameter pipe and a closed-ended 14-inch diameter pipe. Both pipe piles were evaluated using a wall thickness of 0.438 inches. The pipe piles were modeled open ended and closed-ended. In the open-ended case, the piles were checked for plugged and unplugged conditions. Based on Illinois Department of Transportation (IDOT) Standard Specifications, the required hammer energy to drive a HP14x89 would be 36 kip-feet (k-ft). To analyze the drivability of the pile sections, however, open-ended diesel hammers with 39, 51 and 61 k-ft of total energy output were selected to determine which hammer-pile combination would produce greater penetration depths without over stressing the piles.

The static capacity was determined using drained shear strength parameters obtained from triaxial tests performed on Denison and Shelby tube samples of the glacial till and the analysis methods described in Ref.7. The values of skin friction and end bearing along unit sections of pile for a generalized stratigraphy were used as input in the drivability study performed using the drivability option in GRLWEAP. This option allows the direct input of soil parameters used to estimate static pile capacities coupled with input of parameters that reference the static capacity, and associated driving resistance, to the dynamic capacity computed by the program. This static-dynamic correlation factor, labeled sensitivity, was iterated upon to generate sufficient runs to obtain an idea of the probable site soils response to pile driving. This parametric study based on the sensitivity factor indicated that an increase in pile capacity, or pile setup, would need to occur for the piles to be driven to bedrock without exceeding blowcounts of 100 blows per foot. Results using the static parameters (sensitivity=1) indicated that piles could not be driven to refusal on bedrock with the investigated hammer/pile combinations, and that a practical refusal of 20 blows per inch would be obtained for all the piles after penetrating between 20 to 40 feet into the glacial till. A hammer with an energy output of at least 51 k-ft was estimated required for the 14-inch diameter pipe and 14x89 H-piles; the 24-inch diameter piles required an energy output of at least 61 k-ft. Induced pile stresses were approximately 30 kips per square inch (ksi), which are within limits for acceptable driving stresses in steel piles. Conversely, records of pile driving in the general area showed that the piles could be driven to bedrock, but that high blowcounts would be required to drive piles through the layer of glacial till.

The HP sections were selected over the other sections based on the results of the drivability study. The drivability study showed that it would be difficult to drive a 24-inch open-ended steel pipe piles to depths beyond 55 feet, or deeper than 25 feet into the glacial till, if plugged. Since it appeared that driving the open-ended piles would also require washing the soil plug a few times before reaching an adequate bearing depth and increase the cost of pile driving, the 24-inch diameter pipe sections were dropped. The H-piles were selected in favor of the 14-inch closed-ended pipe piles because it was concluded that a low-displacement section would be quicker to drive and could benefit from soil freeze. If the piles were not driven to rock and freeze occurred, the HP sections would show equal or greater capacity increase than the pipe piles. Conversely, if the piles were driven to refusal on bedrock it was likely that the driving resistance of the HP sections would be lower than the resistance generated by a closed-ended pipe pile.

The preliminary analyses considered a HP14x89 section, which was eventually selected. HP sections smaller than a HP14x89 were not considered due to the high estimated foundation loads. Larger HP sections were considered but eventually were not recommended. The recommended foundation alternative included 70-foot long HP14x89 piles, driven to about 15 feet above the top of rock. The axial load carrying capacity was conservatively estimated at 110 tons. This capacity reflected the presumptive capacity based on a tip stress of 9 ksi over the net cross sectional pile area, the depth for which the 110-ton axial capacity was obtained as determined by static methods, and the lowest probable penetration depth from the WEAP runs for a sensitivity equal to unity.

Settlement estimates and a settlement monitoring program, that included near-surface settlement points, deep settlement plates, pneumatic piezometers and inclinometers, showed that settlement of the taxiway embankment could continue to occur after installation of piles at the Silver Creek Bridge. This settlement, induced by consolidation of the soft compressible alluvial soils above the glacial till, was estimated to induce a negative skin friction of 32 tons for the abutment piles; the piles supporting the intermediate piers were sufficiently away from the embankment to not be affected by negative skin friction. The embankment settlement did occur rapidly due to the site's stratigraphy with total settlements of up to 18 inches measured at the end of the surcharging period estimated to range from 7 months at the East Relief Bridge to 12 months at the Silver Creek Bridge. The need to design piles to withstand downdrag due to negative skin friction, therefore, was based primarily on strict schedule constraints.

Final Foundation Design

Pile lengths were estimated during final design using static pile capacity estimating methods, supplemented with engineering judgement and experience with H-piles driven in glacial till. The side friction/adhesion resistance of the soils was estimated using design charts adopted by the IDOT for different soil types. The frictional resistance developed in sandy soils is a function of the soil type and the N-value from the Standard Penetration Test (SPT). The side resistance for cohesive soils is a function of the undrained shear strength of the soil. The end bearing resistance was estimated using the bearing capacity equation for deep foundations by Peck, Hanson, and Thornburn (Reference 5, Chapter 19). The estimated pile tip elevations ranged from El. 353 to El. 357, which corresponds to the lower portion of the glacial till, approximately 10 to 20 feet above bedrock .

Due to the expected variability in the strength and composition of the glacial till, some variation in pile length was expected. The estimated pile lengths were selected using what were considered to be reasonable, yet somewhat conservative, soil properties based on the results of the field investigation and laboratory testing. Two static pile load tests were specified, one at each bridge, to verify the pile capacity. In addition, two test piles for each substructure unit were specified. The test piles were driven in advance of the production piles and were used to establish the furnished pile lengths to be provided to the contractor.

CONSTRUCTION

Pile Load Testing

The pile test program included a combination of static and dynamic load tests. Initially, a WEAP model prepared for the proposed pile driving equipment was submitted followed by driving of the piles for the two static load tests. These piles were monitored with a Pile Driving Analyzer (PDA). Selected stress wave data from the PDA was then analyzed with the CAse Pile Wave Equation Analysis Program (CAPWAP). This combination of PDA/CAPWAP dynamic load tests was also used on two piles per bent that were designated test piles (indicator piles) and specified ten feet longer than the design pile length shown on the plans. Following the IDOT Standard Specifications, the Contractor was required to obtain a furnish pile length, prior to the start of production pile driving, that was based on the results of the test piles.

The first test pile, Pile No. 22 at the East Relief Bridge East Abutment, was driven using a Delmag 19-32 single acting diesel hammer with a rated energy of 42 kips-foot. Based on the submitted WEAP model, the D19-32 required 165 blows per foot to obtain a capacity of twice the design load (440 kips). *During driving of the first test pile*, however, average blowcounts up to 116 blows per foot for a PDA capacity of 280 kips were observed in the till. The pile was driven to a depth of 79 feet, spliced and driven to bedrock at a depth of 83 feet the following day without reaching the blowcount of 165 blows per foot. The PDA capacity showed an increase of about 100 kips overnight, but a capacity of 440 kips was not observed until refusal on rock at a depth of 79 feet.

The second test pile, Pile No. 9 at the East Relief Bridge East Abutment, was driven to a depth of 60 feet for an end-of-initial-driving (EOD) PDA capacity of 350 kips at blowcounts of about 55 blows per foot. After the initial drive, a Delmag 30-32 open-ended diesel hammer with a rated energy of 74 kips-foot had been proposed, and approved, instead of the D19-32. Since the WEAP-generated blowcount associated with the 440-kip ultimate capacity was 49 blows per foot, obtaining 350 kips at 55 blows per foot created some concern for both the validity of the assumptions made in setting up the model and the hammer's efficiency. These two concerns were later clarified, but not resolved, when the CAPWAP showed that higher damping parameters were appropriate at the site, and that the hammer's injection system had been inadvertedly set up at a lower fuel pressure setting. During the restrike performed 14 days later, Pile No.9 beginning-of-restrike (BOR) capacity was measured with the PDA at 550 kips, but the CAPWAP capacity estimated using the higher damping parameters was only 395 kips.

The third test pile driven at the site, Pile No. 6 at the Silver Creek Bridge West Abutment and proposed for a static load test, showed a PDA capacity of 430 kips at 73 blows per foot and a depth of 57 feet and was not restruck. The CAPWAP capacity estimated for Pile No. 6 was only 381 kips, but the static load test later verified the required 440 kips of ultimate capacity . Results from the static load test, performed four days later, showed a vertical deflection of 0.35 inches under twice the design capacity as compared to an estimated elastic shortening of 0.43 inches. The total permanent deflection upon unloading was 0.08 inches.

It is interesting to compare the results from this load test with the results from the load test performed on a 12-3/4 inch OD closed-ended pipe pile driven to support the Air Traffic Control Tower located approximately 400 feet away from the Silver Creek Bridge. This pipe pile was driven to refusal on rock at a depth of 75.9 feet with a similar hammer at a final blowcount of about 20 blows per inch. The load test, performed 10 days after initial drive, showed a vertical deflection of 0.47 inches under a load of 400 kips as compared to an estimated elastic deformation of 0.63 inches. The total permanent deflection upon unloading was 0.18 inches. Therefore, as a percentage of the vertical deflection, the permanent deformation of the friction/end-bearing pipe piles was almost 1.75 times greater than the permanent deformation of the friction HP piles.

Production Pile Driving

Subsequently, a restrike program to correlate EOD blowcounts with ultimate capacity was performed. The BOR blowcounts were also recorded and used to implement a pile driving criteria for field inspection. Since the required ultimate capacity of 520 kips associated with piles subjected to negative skin friction was to be obtained by driving and additional length of pile, the restrike program included evaluation of two sets of pile lengths. The dynamic testing data, collected with the PDA and later analyzed with CAPWAP, showed larger soils' damping than initially assumed. Even though this increased damping led to lower PDA readings, the measured capacities were still within range of the required capacity. In addition, it was expected that the CAPWAP capacities would be somewhat lower than those measured directly by the static load tests. The restrike program continued by driving the remaining 13 test piles to the estimated plan lengths and to a driving resistance of 50 blows per foot for the 110-ton piles and of 70 blows per foot for the 130-ton piles followed by restriking the piles after a minimum waiting period of 48 hours. The driving resistances of 50 and 70 blows per foot were based on the WEAP results using the higher damping parameters.

Pile capacities during both the initial drive and the restrike were documented with the PDA and CAPWAPs performed using one of the initial restrike blows. As additional data was collected, greater penetration resistances were observed at some locations and a number of test piles were driven to lengths shorter than the estimated plan lengths and restruck to determine if the specified capacity was obtainable with shorter-than-estimated pile lengths. Review of the CAPWAP runs for EOD and BOR blows showed that, typically, an increase of approximately 25 percent in the shaft resistance along the bottom half of the pile penetration into the glacial till occurred between the EOD and BOR. An increase in the toe resistance was observed in some instances but could not be quantified based on the available data.

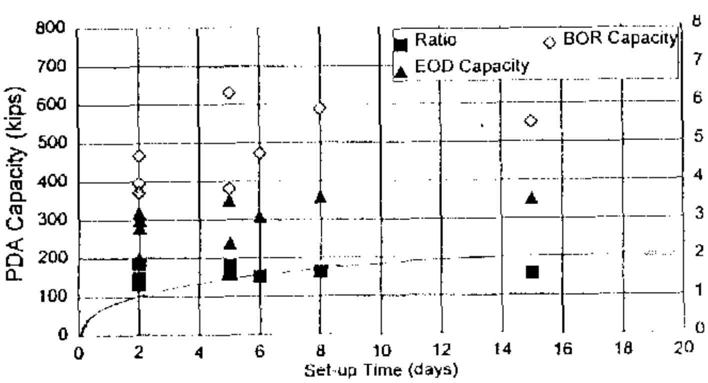

Fig. 3 Plot of pile capacity vs. time

In summary, the restrike program showed that: (1) soil setups ranging from 132 to 180 percent could be obtained at the site; (2) the required ultimate capacities of 440 kips and 520 kips could be easily obtained by a combination of skin friction and end bearing in the glacial till; (3) about 8 feet of additional penetration into the till was required to increase the ultimate capacity from 440 kips to 520 kips; (3) a large variability in pile driving resistance could be obtained at similar penetration depths; and (4) the driving resistances of 50 and 70 blows per foot during EOD would eventually lead to the final static capacity within range of the required 440 and 520 kips, respectively.

The pile driving criteria were prepared to allow the field inspection team to stop the pile short if sufficient penetration resistance was observed within 5 feet of the furnish length. The field inspection was performed using upper-lower bound blowcounts based on the PDA results whereby the piles could be stopped up to five feet short of the estimated depth if the upper limit was observed, but had to be restruck if the lower limit was not observed when the estimated length was reached. The restrike verification, based on the IDOT Standard Specifications, allowed restrike 48 hours after EOD and required a minimum blowcount per inch over three inches and proved very successful in dealing with the site's variability in penetration resistance allowing relatively few splices. The minimum blowcounts used in the restrike verification were also based on the PDA results.

The use of the dynamic testing proved to be a valuable inspection tool. Instances of equipment malfunctioning, that otherwise may have been detected too late, were noted early. Conditions such as the inadvertent use of a low fuel injection pressure setting, slow degradation of a hammer leading to the need for hammer overhaul, and the use of a follower to drive piles underwater and inside a cofferdam could be handled efficiently with the PDA. The use of different hammer types and new equipment brought on site due to equipment malfunctioning could be handled quickly by requesting additional PDA testing and analyzing the data. Because of these changes in the equipment and hammer's performance, the inspection team had to rely heavily on in-office supervision and implement updated driving criteria several times.

A total of 924 HP14x89 piles were driven to support the taxiway bridges. Average pile penetration depths were approximately 20 feet above top of rock. Among the production piles, a total of 79 restrikes were performed and 18 of the piles had to be spliced and driven to depths greater than the corresponding furnish lengths. Among these, four piles did not develop sufficient capacity in the till and were driven to refusal on rock. Post-construction comparison of actual pay lengths to those that would have resulted from driving to rock indicates a savings in pile furnishing and driving costs on the order of $536,000. This averages $580 per pile. By comparison, the costs of using PDA and CAPWAP, including both contract payments and field supervision, were under $30,000. Thus, the approach realized a net savings to the project on the order of a half million dollars. In addition, since driven pile lengths were reduced by some 28%, the calendar time required for completion of the bridges was substantially lessened. This factor was critical at the MidAmerica Airport site for a number of reasons. First, the overall bridge construction schedule benefited immensely from the faster completion of the foundations. Silver Creek, which drains 400 square miles upstream of the site, overtopped its banks several times during the airport construction with heavy flows that at times destroyed haul roads and temporary creek crossings. It was essential to complete the bridge substructure work as quickly as possible so that the bridge work (and work in other contracts) could proceed without interruption by flooding. Secondly, completion of the taxiway bridges was essential to the completion of other project elements, including adjacent paving work and the connection of power, lighting control and communications lines to the new control tower. Third, since MidAmerica is a new commercial airport, the process of certification by the Federal Aviation Administration was in progress at the same time as the bridge construction. A delay in completing the taxiway bridges would have forced the airport to be certified initially without a taxiway link to Scott AFB, with the crossover taxiway receiving certification later. This two-step certification process would have increased program cost and would have adversely affected civilian and Air Force operations and the airport's publicity and business development efforts.

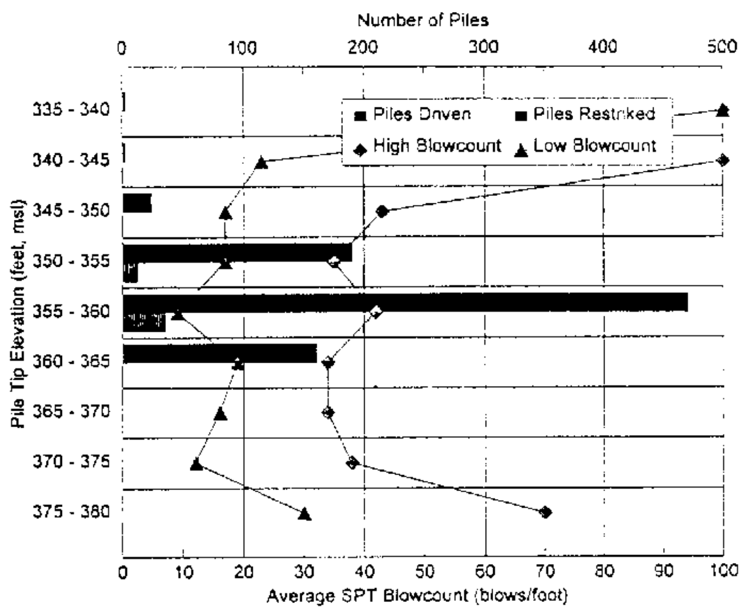

Fig. 4 Production pile histogram

SUMMARY AND CONCLUSIONS

The use of large number of steel H piles to support two heavily loaded taxiway bridges is described. The piles were driven to obtain their capacity by a combination of skin friction and end bearing in glacial till 5 to 20 feet above bedrock. The results of the static and dynamic load testing program are compared with the static pile capacity predictions based on the IDOT method. Sixteen piles were driven during an initial test program to determine final capacities and soil setup at the various piers and abutments of the bridges. Restrikes performed on the piles showed that final capacities ranging from 132 to 180 percent greater than the capacities measured during initial driving can be obtained due to soil setup. It was observed that the soil setup occurred relatively fast after the end of initial driving, and a waiting period of only 48 hours was required to obtain the necessary soil setup. Further increases in the soil setup over time were not investigated. The following conclusions are made based on our evaluation of the data and installation results:

The use of friction piles bearing in a stiff glacial till is practical and can provide significant cost and time savings as compared to end-bearing piles. HP sections may be particularly suited for use as friction piles in stiff clays because of lower resistance during driving and high soil setups. It is possible that soil setup could lead to a "plugged" condition of the H section that may develop shortly after the end of initial driving.

The use of SPT n-values in the IDOT method of pile capacity prediction agreed well with the final penetration depths and capacities after soil setup. The use of a combination of static and dynamic testing, including PDA and CAPWAP, proved to be a reliable and flexible tool in verifying pile capacity predictions and performing field inspection. Results of dynamic test on piles allows flexibility in setting up pile driving criteria and updating it to reflect changes in the driving equipment.

The use of restrikes to verify pile capacity in conditions where soil setup can occur allows flexibility and can reduce the need for pile splicing. The restrike criteria should be determined using actual PDA data from restrikes. The use of test piles should be considered at locations where large variability of penetration resistance is expected. Test piles provide flexibility in managing pile driving and can lead to a lower-cost design.

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