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Pile Foundation—From Preliminary Borings to Production Driving

GARY C. WHITED AND CLYDE N. LAUGHTER

Foundation design for a major bridge structure requires extensive field and office investigation. The design process undertaken for the Arrowhead Bridge, which carries US-2 between Superior, Wisconsin, and Duluth, Minnesota, over the St. Louis Bay, is presented. Subsurface investigation results, geologic studies, pile load tests, wave-equation analysis, and dynamic pile testing are presented. Results of the geotechnical investigations allowed the use of high-capacity piles in soil for the bridge foundation. Subsurface conditions consisted of soft lacustrine and glacial clay deposits over dense glacial outwash sands. Depths to the underlying dense strata ranged from 130 to 260 ft (39.6-79.2 m) across the site. Six load tests were performed on steel H-piles and cast-in-place type piles. Maximum loads of 344 tons-force (3060 kN) were applied by using both maintained-load (ML) and constant-rate-of-penetration (CRP) methods. Load test results are presented by using five interpretative techniques, and comparisons between ML and CRP methods are shown. Wave-equation analyses were performed by using the WEAP computer program, and results are compared with driving records. Dynamic pile analysis was done by using the Goble-Case Western pile driving analyzer (PDA), and pile capacity predictions are compared with load test results. The PDA was also used on production piling, and experiences while analyzing the very long piles for capacity and damage are discussed.

The design of a foundation for a major bridge project requires a progression through various stages of literature review, field investigations, and office interpretation and evaluation. This paper presents details of the design process and resulting construction experience for the Arrowhead Bridge in Superior, Wisconsin.

The new Arrowhead Bridge is to be some 8400 ft (2.56 km) in length and will carry US-2 between Superior, Wisconsin, and Duluth, Minnesota. Located at the western tip of Lake Superior, the new high-level structure will span St. Louis Bay, the harbor shipping channel, a number of railroad tracks, and Interstate 35 in Duluth. The curved and skewed alignment, as shown in Figure 1, was necessary to provide navigational clearances for a harbor bend down channel yet meet desired connection points in Superior and Duluth. The bridge will provide a horizontal clearance of 500 ft (152.4 m) and a vertical clearance of 120 ft (36.6 m) at the channel.

The geotechnical investigation consisted of a literature review, three separate and progressive phases of subsurface investigation, and a pile load test program. The final foundation design was determined from results of these investigations, and construction was started. Foundation work on three of the four substructure contracts is essentially complete; so far, no major problems have developed.

SITE CONDITIONS

The foundation investigation for the structure began

Figure 1. Soil profile.

BRIDGE ALIGNMENT AND GENERALIZED SOIL PROFILE
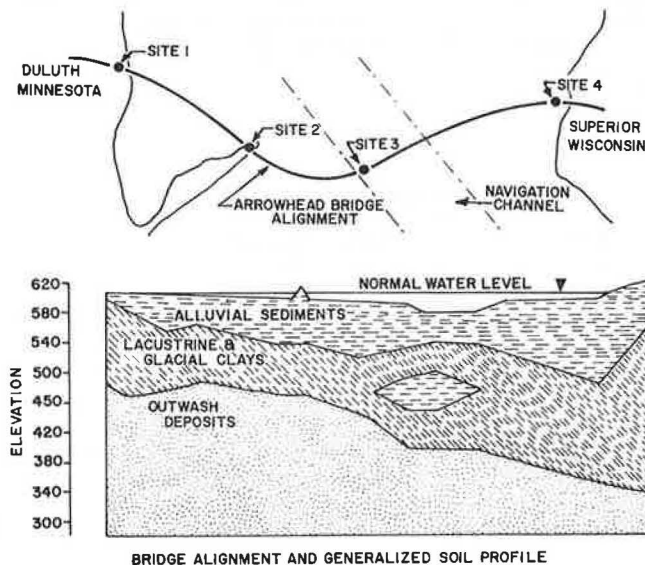
in 1973 with an office review of available U.S. Geologic Survey reports, geologic maps, and well-drilling logs in the area. At the same time, a preliminary subsurface exploration program was initiated, which included making eight widely spaced borings along the proposed alignment. Results of these early investigations indicated a rather complex geologic section, involving alluvial stream deposits, lacustrine clays, glacial drift clays, and glacial outwash deposits. Bedrock was estimated to be at depths in excess of 600 ft (183 m) in what was assumed to be a preglacial bedrock valley of the St. Louis River. Water depths across the bay were generally less than 5 ft (1.5 m); depths in excess of 25 ft (7.6 m) were found in the channel area. A generalized stratigraphic section along the bridge alignment is shown in Figure 1.

The upper alluvial deposits were quite variable and consisted of very loose sands, silts, and often organics. Standard penetration test (SPT) N-values for these materials were generally less than 10. The lacustrine and glacial clays were classified as being soft to stiff; pocket penetrometer values ranged from 0.5 to 2.5 tons force/ft² (47.9-239.4 kN/m²), and SPT N-values varied from 5 to 15. The underlying glacial outwash deposits were very dense silty sands; SPT N-values ranged from 50 to more than 100 for 6-in (15.2-cm) penetration. Depths to this dense foundation layer ranged from 130 ft (39.6 m) on the Minnesota side to 260 ft (79.2 m) on the Wisconsin side.

Based on the results of these preliminary investigations, it was apparent that a deep foundation would be required. The logical foundation choice was piling, based on experience of the local contractors and the type of bridge being proposed. A predesign pile load test program was initiated in 1977 to determine the most economical pile type, maximum pile load-carrying capabilities, probable tip elevations, and pile-driving characteristics (1). Four test locations were selected, as shown in Figure 1, and additional borings were made at each site to determine the soil conditions.

Two pile types were chosen for evaluation: a 16-in (406.4-mm) diameter concrete-filled cast-in-place pipe pile driven closed-end and a HP 14x73 (355.6-mm x 108.6-kg/m) steel H-pile. Selection of these two pile types for testing was based on anticipated loads, Wisconsin Department of Transportation's (DOT) past experience, and the experience of the local contractors. One H-pile was to be tested at each of the four sites; a cast-in-place (CIP) pile was to be tested at sites 1, 2, and 4. Piles were to be driven to minimum bearing of 172 tons-force (1530 kN) as determined by the Wisconsin (modified EN) driving formula (2):

$$P = 2WH/(S + 0.2) \qquad (1)$$

where

    P = bearing values (lb),
    W = ram weight (lb),
    H = height of ram fall (ft), and
    S = penetration per blow (in).

Piles were also to be monitored during driving with the Goble-Case Western pile-driving analyzer (PDA).

PILE LOAD TEST

Contracts for the pile load test program were awarded to Johnson Bros. Corporation of Litchfield, Minnesota. (Johnson Bros. later became the major contractor for substructure construction.) All test pilings were driven with a Menck MRBS-500 single-acting air-steam hammer with an energy rating of 46 500 ft-lbf (63.0 kJ), which obtained the desired bearing when reaching the dense silty sand outwash deposits. Pilings were driven continuously with the exception of stops for splicing. The CIP piles were to have a 0.250-in (6.35-mm) minimum shell thickness, but the contractor opted to drive grade A52, 0.219-in (5.56-mm) shells due to the unavailability of the thicker shell. While driving the CIP piles at sites 2 and 4, the lighter shells buckled. A replacement pile with 0.250-in thickness was then driven at site 4, but this also buckled during driving. A second replacement pile that had a shell thickness of 0.320 in (8.13 mm) was finally driven successfully and load tested. Thus, three of the five shell piles driven for the program were damaged during driving. The H-piles drove with little difficulty at all four sites and essentially had the same driven lengths as the CIP piles at sites 1 and 4. The test pile at site 3, which was located in the deep channel water, was reinforced against buckling by welding 24x1.5-in (60.9x3.81-cm) steel plates across the flanges to a depth of 15 ft (4.6 m) below the mud line.

Six maintained-load (ML) type load tests were conducted on the four H-piles and two remaining CIP piles, essentially in accordance with ASTM D 1143, by using anchor piles for reactions. Four constant-rate-of-penetration (CRP) type load tests were conducted immediately after completion of the ML tests. Test loads were applied in three cycles; the test load was reduced to zero after each cycle to measure net set of the pile head. Test loadings for the respective cycles were cycle 1, 96 tons-force (854 kN); cycle 2, 128 tons-force (1139 kN); and cycle 3, 172 tons-force (1530 kN). These loadings were chosen to correspond to stresses of 9000, 12 000, and 16 000 psi (62, 82.8, and 110.3 MPa), respectively, in the steel for the HP 14x73 H-pile. Loads were applied in increments by using the following percentage of design cycle load and sequence: 0, 50, 100, 125, 150, 175, 200, 150, 100, 50, 0. Each load increment was maintained until the rate of settlement, or rebound, under that increment was less than 0.001 in (0.25 mm) in a 15-min period before proceeding to the next increment. The maximum cycle 3 load of 344 tons-force (3060 kN) was maintained for 24 h.
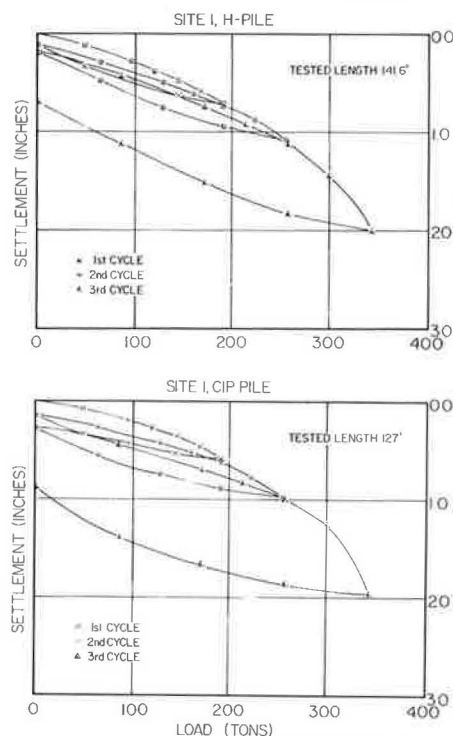
Applied loads were determined from calibrated hydraulic gauges. The instrumentation for measuring pile head movement consisted of three dial gauges spaced equally around the pile and two dial gauges mounted perpendicular to the pile to monitor horizontal movements. Secondary measurement systems of wire line and mirror and surveyor's level were also used. Telltales were not installed on any of the test piles.

LOAD TEST RESULTS

The load test results are summarized in the accompanying load versus displacement plots in Figures 2, 3, and 4. The H-pile at site 2 failed aboveground while placing the last increment of loading. The data point shown for the 344-ton-force loading was estimated from readings made during the loading. A maximum load of only 300 tons-force (2669 kN) could be maintained on a retest of cycle 3 for this pile due to excessive lateral pile movements. The load test for the H-pile at site 3 was terminated after reaching a 300-ton-force load on the third loading cycle as the pile was deflecting with no increase in load.
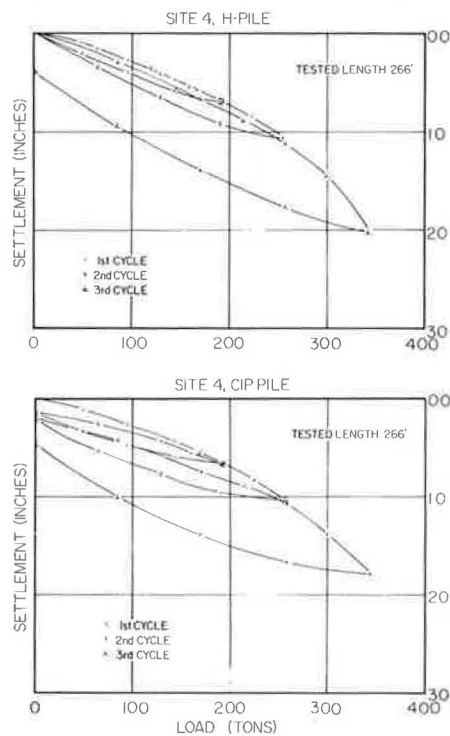
Ultimate pile capacities were predicted by using interpretive techniques suggested by Davisson, Mazurkiewicz, Shin, and the Swedish Pile Commission
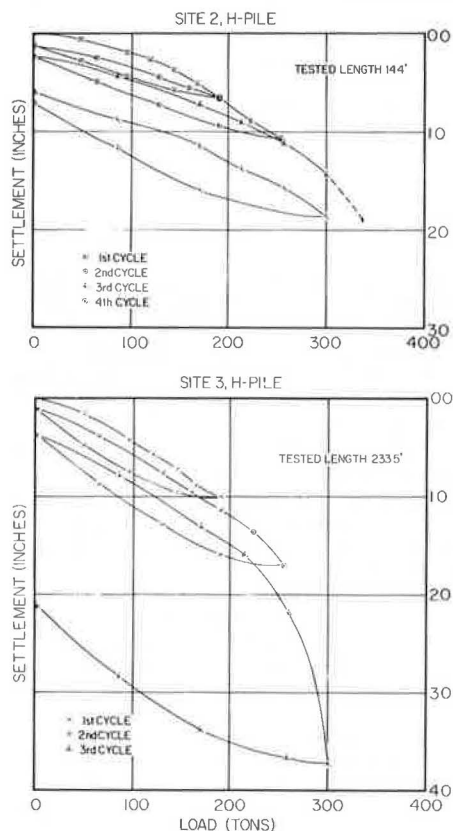
Figure 2. Site 1: load versus displacement plots.

SITE 1, H-PILE

TESTED LENGTH 141 6'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE

SITE 1, CIP PILE

TESTED LENGTH 127'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE

LOAD (TONS)

Note: 1 in = 1.83 cm, 1 ton-force = 8.896 kN, 1 ft = 0.3 m.

Figure 3. Sites 2 and 3: load versus displacement plots.

SITE 2, H-PILE

TESTED LENGTH 144'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE
- 4th CYCLE

SITE 3, H-PILE

TESTED LENGTH 233.5'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE

LOAD (TONS)

Note: 1 in = 1.83 cm, 1 ton-force = 8.896 kN, 1 ft = 0.3 m.

Figure 4. Site 4: load versus displacement plots.

SITE 4, H-PILE

TESTED LENGTH 266'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE

SITE 4, CIP PILE

TESTED LENGTH 266'

SETTLEMENT (INCHES)

- 1st CYCLE
- 2nd CYCLE
- 3rd CYCLE

LOAD (TONS)

Note: 1 in = 1.83 cm, 1 ton-force = 8.896 kN, 1 ft = 0.3 m.

(90 percent criterion) (3). The general interpretation procedures and the American Association of State Highway and Transportation Officials (AASHTO) settlement criteria are shown in Figure 5 and summarized as follows:

1. Davisson: Davisson's method defines the failure load as the load that corresponds to the movement that exceeds the elastic compression of the pile, when considered as a free column, by a value of 0.15 in (3.81 mm) plus a factor depending on the diameter of the pile. For the 16-in (406.4-mm) CIP piles and HP 14x73 H-piles, this factor is approximately 0.1 in (2.54 mm). Thus, the pile reaches failure at a pile-head movement that exceeds the elastic compression by 0.25 in (6.35 mm).

2. Mazurkiewicz: The Mazurkiewicz technique involves arbitrarily choosing a set of equal pile-head movements and constructing from the intersection of these movement lines and the load-deflection curve a set of corresponding load lines. From the intersection of each load line with the load axis, a 45° line is drawn to intersect with the next load line. The intersections fall approximately on a straight line, from which the intersection with the load axis defines failure.

3. Chin: Chin proposes that the load-deflection curve can be approximated by a hyperbola. A plot of pile-head movement divided by load versus an abscissa of head movement gives a straight line, of which the inverse slope is the failure load.

4. Ninety percent criterion: The Swedish Pile Commission's 90 percent criterion defines failure as the load for which the pile-head movement is twice the movement obtained at a 10 percent smaller load.

5. AASHTO: The AASHTO specifications (4) define ultimate pile load as that load that, after a con-

Figure 5. Load test interpretative techniques.
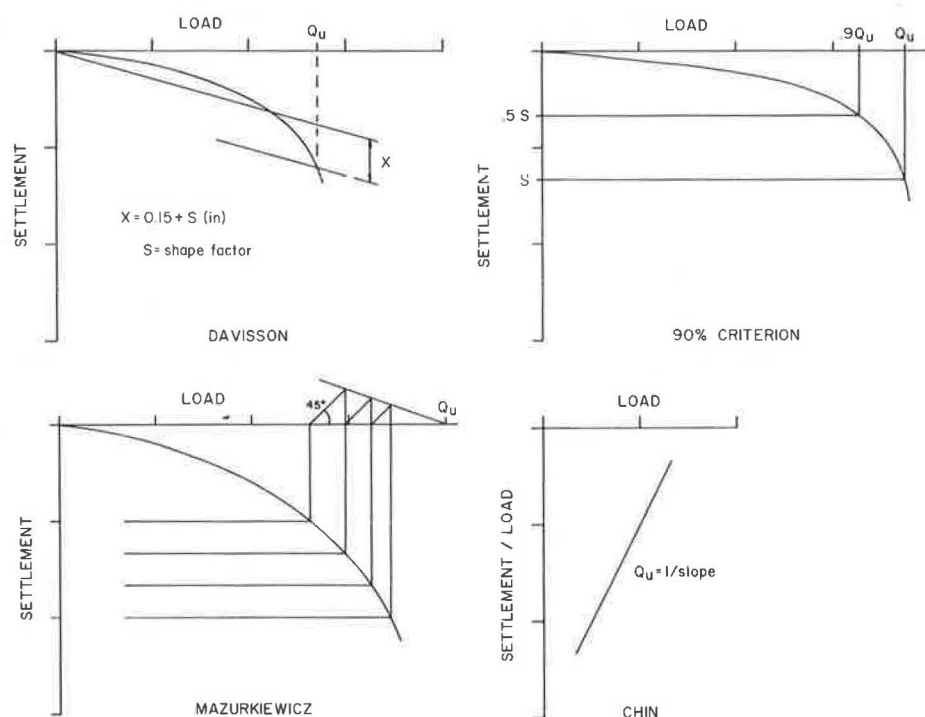

Table 1. Predicted ultimate loads from ML test data.

Item	Ultimate Load (tons-force)					
	Davisson	Mazur-kiewicz	Chin	90 Percent Criterion	AASHTO	Failure Load
Site 1						
H-pile	345	430	590	355	275	375[a]
CIP	300	370	510	360	250	
Site 2, H-pile	380	455	440	400	265	
Site 3, H-pile	295	315	580	300	235	300
Site 4						
H-pile	390	415	620	375+	325	380[a]
CIP	375+	455	630	375+	275	

Note: 1 ton-force = 8.896 kN.

[a]CRP test result.

tinuous application of 48 h, produces a permanent settlement not greater than 0.25 in.

A summary of the predicted ultimate failure loads obtained from the ML load test data is given in Table 1. Although the values obtained from the AASHTO criterion should be defined as a limiting load rather than an ultimate failure load, they are also listed, as this technique is typically used by state highway departments for determining maximum loads (5). Loads obtained by using this method were by far the most conservative capacities obtained. Chin's method, on the other hand, gave the highest values. It should be pointed out, however, that most of the predictions were based on extrapolations of the load-deflection curves. Of the interpretative techniques used, only Davisson's method takes into account the length and diameter of the pile (6). Although other investigators have reported the technique to be conservative, it was felt it represented the most rational approach and thus was used to define the ultimate pile capacity.
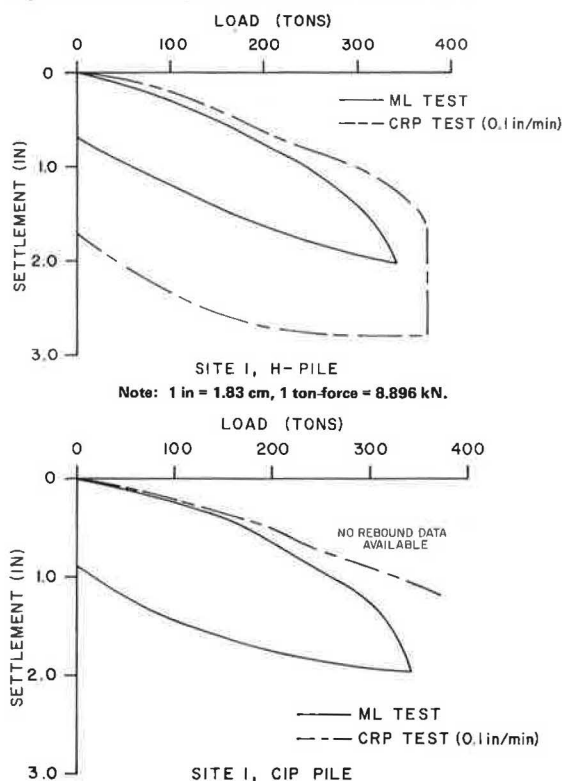
On completion of the ML load tests, CRP-type load tests were run on the piles at sites 1 and 4. Loading rates recommended by the New York State DOT were used (7). Results were very similar to those ob-

tained with the ML tests, although the measured settlements were slightly less with the CRP tests. Figures 6 and 7 show a comparison of the CRP tests with a composite curve of the ML test results. The CRP test was run immediately after the ML test, but the data have been shifted to the zero settlement point to provide a comparison.

The Goble-Case Western PDA was used to monitor driving of the test piles at sites 1, 3, and 4. Testing was performed by the Soil Exploration Company of Minneapolis, Minnesota. Testing with the PDA involves making force and acceleration measurements at the top of the pile during driving. These measurements are then fed into a small field computer, and predictions of the pile's ultimate static capacity are made by using the Case method (8). A comparison of the PDA predictions made at the time of driving with the CRP load test results is given in Table 2.

The Case method of analysis was quite reliable in predicting ultimate capacity of the H-piles. Predictions for the CIP piles were, however, off by a factor of ±2. Reanalysis of the data by using the Case pile wave analysis program (CAPWAP) method of analysis did not improve the predictions. A possible explanation for the poor results on the CIP

Figure 6. Site 1: comparison of ML and CRP test results.

LOAD (TONS)

ML TEST
CRP TEST (0.1 in/min)

SITE 1, H-PILE
Note: 1 in = 1.83 cm, 1 ton-force = 8.896 kN.

LOAD (TONS)

NO REBOUND DATA AVAILABLE

ML TEST
CRP TEST (0.1 in/min)

SITE 1, CIP PILE

Figure 7. Site 4: comparison of ML and CRP test results.

LOAD (TONS)

ML TEST
CRP TEST (.05 in/min)

SITE 4, H-PILE
Note: 1 in = 1.83 cm, 1 ton-force = 8.896 kN.

LOAD (TONS)

ML TEST
CRP TEST (.05 in/min)

SITE 4, CIP PILE

piles is that these piles were driven with an over-sized end plate and probably experienced considerably more setup than did the H-piles during the time delay between driving and load testing. However, it was not possible to obtain restrike data after the load tests to confirm this.

On completion of the load test program, piles at sites 1 and 4 were analyzed by using the wave equation to evaluate the predictive capabilities of the method and how well it could model the pile-driving operation at this site. The analysis was performed independently by Federal Highway Administration (FHWA) personnel who used the WEAP computer program. Results of their analysis indicated that the piles could not have been driven to the capacities measured with the hammer used. A comparison of results from the WEAP analysis, PDA, and Wisconsin standard driving formula is shown in Figure 8. The wave equation gave the poorest correlation of the methods; thus, no further attempts were made to try and establish a driving criteria for field inspection with the wave equation.

Based on the poor driving experience of the CIP piles during the load test program, the HP 14x73 H-pile was selected for use on all substructure units. An ultimate load of 300 tons-force (2669 kN) that had a design load of 150 tons-force (1334 kN) was selected based on the load test results by using Davisson's technique. The piles were expected to drive to the dense granular layer; thus, no group reduction was felt necessary. With the very long piles and high capacities, it was decided to use the Goble-Case Western PDA for quality control on the project. Due to the poor results with the wave equation, driving criteria for production piles were established by using the Wisconson DOT standard driving formula.

FINAL SUBSURFACE INVESTIGATION

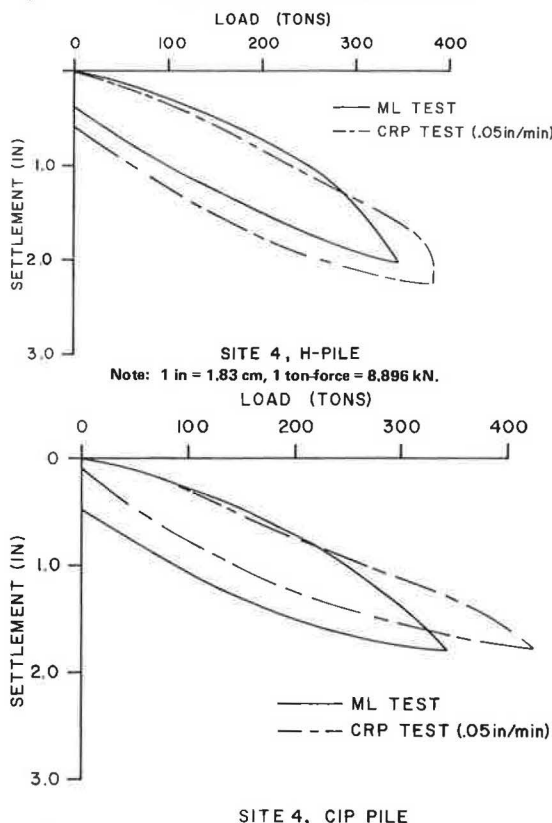On completion of the preliminary bridge plan, a

Table 2. Comparison of PDA predictions with CRP load test results.

| Item | Ultimate Load (tons-force) | |
	PDA (Case method)	CRP Load Test
Site 1		
H-pile	380	375 (F)
CIP	180	360+
Site 3, H-pile	310[a]	300 (F)[b]
Site 4		
H-pile	330	380 (F)
CIP	230	425+

Notes: 1 ton-force = 8.896 kN.
       (F) = actual failure load.

[a] Prediction at driven length of 199 ft (60.7 m).
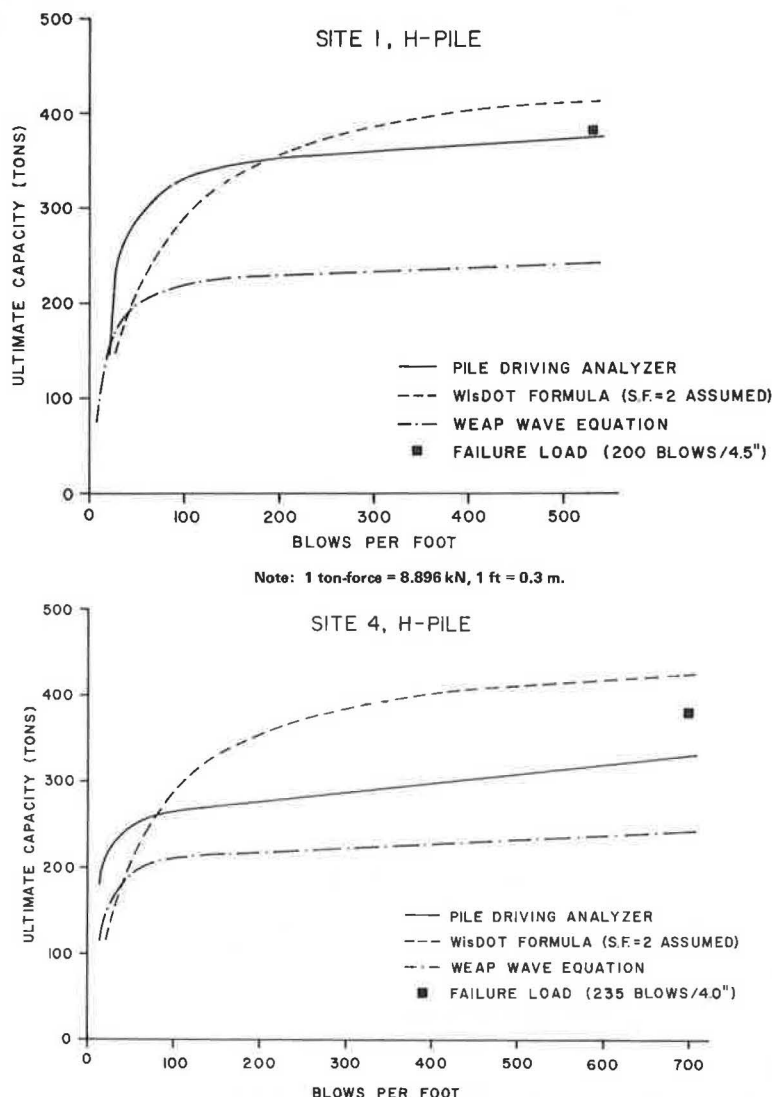[b] Failed under ML test with driven length of 255 ft (77.7 m).

final subsurface investigation program was started in 1978. This program involved taking one SPT boring at each of the 45 substructure unit locations. Results of this investigation confirmed what had been found in the earlier studies, i.e., relatively soft sediments overlying a dense foundation zone. Driving records from the pile load test program indicated that the H-piles would penetrate the dense silty sand layer some 15 ft (4.6 m). Final bridge plans were prepared that required both a minimum tip elevation based on 15 ft of penetration into the dense silty sands and a minimum bearing of 150 tons-force as determined by the Wisconsin DOT standard driving formula.

CONSTRUCTION CONTROL

Bridge construction began in 1979, and the substructure was divided up into four separate construction contracts. Johnson Bros. obtained two of these con-

Figure 8. Wave-equation, PDA, and Wisconsin DOT driving
analysis results.

SITE I, H-PILE

ULTIMATE CAPACITY (TONS)

—— PILE DRIVING ANALYZER
--- WisDOT FORMULA (S.F.=2 ASSUMED)
—·— WEAP WAVE EQUATION
■ FAILURE LOAD (200 BLOWS/4.5")

BLOWS PER FOOT

Note: 1 ton-force = 8.896 kN, 1 ft = 0.3 m.

SITE 4, H-PILE

ULTIMATE CAPACITY (TONS)

—— PILE DRIVING ANALYZER
--- WisDOT FORMULA (S.F.=2 ASSUMED)
—·— WEAP WAVE EQUATION
■ FAILURE LOAD (235 BLOWS/4.0")

BLOWS PER FOOT

tracts, Edward Kramer and Sons one, and the fourth
is yet to be let. Estimates called for some 270 000
linear ft (82 296 m) of piling for the bridge. Pile
driving on the first three contracts is essentially
complete; approximately 185 000 ft (56 388 m) of
piling have been driven.

Construction control for the job consists of
using both the Wisconsin DOT driving formula and the
PDA. A Wisconsin DOT pile inspector remains with
the driver at all times, and blow count, height of
ram fall, and calculated capacity are recorded for
each foot of pile. Piles are required to be driven
into the dense silty sand layer and obtain a minimum
bearing of 150 tons-force by the formula. If pene-
tration into the silty sand is not possible, a 200-
ton-force (1780-kN) capacity by formula is re-
quired. The two impact hammers being used for
production driving are a Delmag D-36 diesel hammer
that has an energy rating of 83 100 ft-lbf (112.6
kJ) and a Conmaco 160 single-acting air-steam hammer
that has a rating of 48 750 ft-lbf (66.1 kJ). These
hammers are not the same as the driver used for the
pile load test program, but the rated energies are
in excess of that of the original hammer. With the
higher energies and the verification with the PDA,
it was felt that a requirement of identical hammers
was not necesssary.

One to two piles in each substructure unit are
also tested with the PDA to verify that the hammer
is performing satisfactorily and that desired bear-
ing is being achieved. Testing with the PDA is
accomplished by attaching reusable strain trans-
ducers and accelerometers to the pile. For each
hammer blow, strain and acceleration signals are fed
into the field computer for processing. An instan-
taneous printout of energy transmitted to the pile
and forces developed in the pile is obtained. Due
to the very long piles and high blow counts, the
ultimate static capacity of the pile must be calcu-
lated by applying corrections for loading and un-
loading. The Conamaco 160 air-steam hammer being
used by Johnson Bros. has produced such high accel-
erations that strain transducers were continually
damaged during periods of prolonged driving. To
minimize the occurrence of damage, piles are tested
with the PDA only on a restrike basis after the pile
has reached bearing by using the Wisconsin DOT for-
mula. A summary of results obtained with the PDA is
given below:

1. Length—maximum = 282 ft (86 m) and minimum =
180 ft (55 m);
2. Capacity—maximum = 500 tons-force (4448 kN)
and minimum = 295 tons-force (2624 kN);

3. Stresses--maximum = 35 000 psi (241.4 MPa) and minimum = 21 700 psi (149.7 MPa); and

4. Energy--Delmag D-36, diesel: maximum = 36 000 ft-lbf (48.8 kJ) (51 percent) and minimum = 21 000 ft-lbf (28.5 kJ) (41 percent); and Conmaco 160, air-steam: maximum = 30 000 ft-lbf (40.7 kJ) (72 percent) and minimum = 21 000 ft-lbf (48 percent).

The energy percentages shown are hammer efficiencies that are based on the ratio of measured energy to the height of ram fall times the ram weight. The ultimate capacities being obtained are near or in excess of the capacities required for the job. Pile stresses and hammer energies were felt to be within an acceptable range.

CONSTRUCTION PROBLEMS

Construction problems have been minimal; the only major problem was associated with pile setup. The initial pile load test program required the test piles to be driven continuously; however, no such provision was included in the construction contracts. The contractor initially began driving in three substructure units, working from one to another. Piles that were in excess of 180 ft (55 m) and left for more than one day experienced difficulty on resumption of driving. Some could not be started again. With the foundation design based on penetrations of ±260 ft (79 m) to the dense granular layer, corrective measures were required or redesign would have been necessary. The problems were eventually resolved after negotiations with the contractor resulted in an agreement where only one substructure unit would be driven at a time, thereby reducing the times for development of setup. Time delays were reduced typically to one or two days, and the remainder of the piles have been driven down to the silty sand layer as planned.

The PDA has proved to be very useful in determining if a pile has been damaged during driving (9). Piles that experienced a dramatic decrease in blow counts or were significantly past plan tip elevation were tested with the PDA. A visual examination of the force and velocity wave traces allows immediate evaluation by the operator as to whether the pile is damaged and the location of the damage. Piles that were determined to be damaged are left in place and replaced by driving an additional pile adjacent to it. No load-carrying capacity is assigned to the damaged pile. To date, the project has experienced a damage rate of 2.3 percent; the majority of the damaged piles are battered.

CONCLUSIONS

From the various investigations, analyses, and later construction experiences, the following conclusions can be made:

1. The sequence of office studies, preliminary exploration program, load tests, and final subsurface investigation was necessary in progressing from initial project conception to final construction. Each step provided input for planning the next phase of the design process, which resulted in a more economical and efficient approach.

2. The pile load test phase proved to be a highly successful predesign program that (a) provided valuable information for selecting the most suitable pile type, (b) evaluated maximum design loads, and (c) determined pile tip elevations for final bridge design. The program also provided insight into the drivability of piles at the site, which aided contractors in preparing construction cost estimates.

3. Davisson's predictive method gave the most reliable results of the interpretative techniques tried. The ultimate capacity predictions from the ML test data were very close to those obtained during the CRP tests that were loaded to failure.

4. The CRP load tests appear to give essentially the same results as the ML tests in a much shorter time. The CRP test does result in less pile settlement and should be used with caution when a net settlement criterion is used for determining allowable load.

5. Results of the Goble-Case Western PDA were very positive in predicting ultimate capacity of the H-piles but were low by a factor of ±2 for the CIP piles. Restrike testing done closer to the time of load testing, though, may have improved the predictions for these piles.

6. The wave-equation analysis did not realistically model the pile driving at this site. However, it remains a powerful tool in the analysis of pile driving. There seems to be great potential for the technique, particularly if it is used in conjunction with PDA testing and CAPWAP analysis to refine the input soil parameters and correlated with static load tests.

7. The HP 14x73 H-pile with a design load of 150 tons-force and a ultimate load of 300 tons-force was selected for the bridge foundation based on the results of the pile load test program. This loading was a significant increase over the 96-ton-force (854-kN) design loading that would normally have been used for this pile in soil following AASHTO standard specifications. With pile driving for three of the substructure contracts essentially complete, construction experience has verified the choice of the pile type for the design loadings imposed. Piles drive to the dense silty sand zone and achieve bearing, as was anticipated. An overall pile damage rate of 2.3 percent has been observed so far.

8. The PDA has shown to be a very useful quality-control device. Due to a high equipment damage rate, however, only restrike data can be taken. Results of the testing verify that required capacities are being obtained and that the hammers are functioning properly. Measurement of hammer energy has been necessary, as the driving criterion is based on the Wisconsin DOT driving formula. Damage detection with the PDA has been an exceptionally useful part of the quality-assurance program.

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Construction Control by Monitored Geotechnical Instrumentation for New Terminal 46, Port of Seattle

BENGT H. FELLENIUS, ARTHUR J. O'BRIEN, AND FRANK W. PITA

Geotechnical instrumentation was used to monitor and control construction pore pressures and soil movement during major modifications to an existing container terminal (old terminal 46) for the Port of Seattle. There was concern that the construction work, which consisted of dredging, filling, and pile driving, might disturb the confined and sloping (5H:1V) 25-ft-thick loose silt layer beneath the fill at the terminal. Construction control by monitored instrumentation was used because the topographic conditions at the site and the Port's economic and marine design parameters precluded conventional methods of preventing slope failure, such as total excavation of the silt and/or flattening the new fill slope. The instrumentation monitored the behavior of the confined silt layer to ensure that excess pore pressures and soil movements induced by the disturbance of the construction work were within acceptable limits. Two warning levels of observed excess pore pressure were established to control the construction sequence and rate. At the yellow level, extra caution and alertness were imposed. At the red level, construction was halted or relocated. The disturbance caused by dredging and filling operations was small. The disturbance from pile driving was limited to a zone that had a radius smaller than 30 ft. The pile-driving contractor was restricted to driving no more than 3 piles/day within 30 ft of each other. This posed little hardship for the contractor, and the construction was completed successfully.

This paper presents the background and results of the construction-control monitoring program implemented during the construction of new terminal 46, Port of Seattle, Washington. The preliminary design for the new terminal specified that an embankment be built on a confined, sloping layer of loose silt and that, afterward, displacement-type piles be driven through the embankment slope and silt layer into an underlying dense, glacial deposit. There was concern that implementation of these two construction procedures might cause embankment instability.

The preliminary design calculations for new terminal 46 indicated an unacceptably low margin of safety against slope failure if construction procedures caused loss of effective strength in the sloping silt layer. Such loss of strength could occur from increased pore pressures caused by rapid dumping of fill or by pile driving. However, the overall topographic conditions of the site and the marine design parameters were such that conventional solutions, such as complete removal of the silt layer or flattening of the new embankment slope, were not practical. Conventional solutions were also not economical because the cost difference between the use of instrumentation to implement the preliminary design concept and the use of conventional solutions was estimated to be more than $1

million. Therefore, the decision was made to implement the preliminary design with some minor modifications and to monitor the stability of the slope during construction by means of piezometers and slope inclinometers. If any excessive pore pressure or soil movements suggesting imminent risk of failure occurred, the construction would be halted until the risk had subsided.

Proper planning and use of the monitoring program would maintain the risk of embankment failure at an acceptably low level; however, too frequent construction halts and/or relocations could cause costly project delays. Nevertheless, the risk of costly delays was preferred over alternative conventional solutions.

SOUTHEAST HARBOR DEVELOPMENT PROJECT

The Port of Seattle implemented the Southeast Harbor Development Project to improve existing waterfront facilities and to provide new facilities for handling the growing volume of containerized cargo. Phases 1 and 2 of this project, which occurred between old pier 37 and old terminal 46, were completed in 1979. Phase 3, which consisted of a modification and lateral extension of old terminal 46, was completed in 1980. The completed facilities include 86 acres of a container storage and handling area; five container cranes will operate on 2740 ft of the pile-supported apron structure (Figure 1).

During the construction of phases 1, 2, and 3, pier 39 and portions of piers 37, 42, and 43 and old terminal 46 were removed (Figures 1 and 2). An earth-fill embankment was built at the outer edge of the old piers. A container storage area was then constructed by filling between the old piers and the new embankment. A pile-supported apron deck was constructed on the outer slope of the new embankment.

A variety of fill materials was used behind the embankment, including fine-grained organic dredge material from the Duwamish River, demolition rubble, riprap, and gravelly sand. The outer fill slope intersects the natural bottom of Elliott Bay, which descends at a slope of approximately 5H:1V at the site. The slopes were built in water at depths up to 90 ft in phase 1 and to 125 ft in phases 2 and 3.

The construction of new terminal 46 (phase 3)