based on the results of the pile test program and subsurface data. In general, there were two main considerations. One was to estimate the necessary penetration to develop the design ultimate capacity and the other was to develop a driving resistance to satisfy the dynamic pile-driving formula that, in this case, was the WEAP program.

The pile penetration was based on the exploration data and the pile load test results. The estimated pile tip elevations were selected to have the tips in sand with a minimum of about 15 ft of sand below the pile tip to reduce the pile group settlement.

The driving criteria were based on the continuous driving resistance, considering the freeze factor. Tests and previous experience in the area indicated that the freeze factor for concrete piles was about 2 to more than 5.

For the steel pipe piles, the specified pile penetrations were about 30 and 50 ft into the glacial deposits for the east channel (site C) and west channel (site D) piers, respectively.

Hammers other than those used in the pile test program were permitted. However, it was required that those hammers be calibrated by using the dynamic pile analyzer.

CONSTRUCTION PILE INSTALLATION

The test pile program was accomplished after the conceptual design of the West Seattle Freeway Bridge replacement project was developed but prior to the final design. Based on the results obtained from the test pile program, the final pile penetrations for the concrete and steel piles were reduced by about 10-25 percent from the preliminary estimated lengths prior to the load test program. This resulted in a reduced cost for the project. During the bidding process, the geotechnical report, which included the pile test data and results, was made available to all prospective contractors. The pile test data eliminated many questions concerning pile hammer selections and pile drivability. This may have contributed, in part, to the fact that the bid prices were less than the engineer's estimated costs for every project contract.

CONCLUSIONS

A comprehensive pile load test program was accomplished during the initial design stage of this very large project. The information thus gained was very beneficial in the final design and the preparation of contract specifications. It is also believed to have had an impact on the four contract bid prices, which were all below the engineer's estimate.

Static empirical methods supplemented with local pile experience proved satisfactory for estimating pile penetrations and pile capacities. However, instrumentation data indicate that the apparent frictional resistances and end-bearing capacities are different from those calculated from empirical formulas.

The WEAP program was very useful in selecting pile-driving hammers that successfully drove the 24-in concrete and 24- and 36-in steel piles to the required penetration and desired capacities with the driving stresses below the ultimate stress of the pile material. The pile analyzer was used extensively and was useful in the overall understanding of pile driving. It was used to evaluate hammer performance and energy delivered to the pile as well as to estimate capacity and provide soil constants for wave-equation analysis for that particular pile and hammer. The pile analyzer was particularly well suited to evaluate the performance of diesel hammers.

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Foundation Design and Evaluation for Winnemuca Viaduct

G. G. GOBLE, DAVID COCHRAN, AND FLOYD MARCUCCI

In preparation for the construction of Interstate 80 through Winnemucca, Nevada, a pile load test program was performed on candidate pile types to assist in the selection of a foundation design. Six pile types were statically tested at each site. Dynamic tests were also conducted, both at the end of driving and on restrike, and Case method static-capacity predictions were made. The results of this program are reported. Treated-timber piles were selected in the design; they had allowable stresses of 1200 psi. The actual stresses in the foundation piles are reviewed and reported. Also, driving records for one foundation group are presented to illustrate the effect of group behavior on driving resistance.

The initial planning for the section of Interstate 80 through Winnemucca, Nevada, called for twin

viaduct structures more than 4000 ft long. The structures were to span three crossings of the Humbolt River and two city streets. Preliminary foundation recommendations called for 55-ton design load steel H-piles. Because of the large amount of piling required to support the structure, a preliminary full-scale pile load test program was conducted to examine combinations of pile length and type in the hope that improved economy would result. Six different types of piles were driven and load tested at three different locations along the proposed route for the structure. As part of the same test program contract, two locations were tested in Lovelock, Nevada, in preparation for future bridge construction. The results of the Lovelock tests will not be discussed here, since bridge construction there has not yet started.

By rechanneling parts of the Humbolt River to avoid intersection with the I-80 alignment, the design length of the Winnemucca structure was reduced to about 900 ft after the load test program was completed. The layout of the structure and surrounding area is shown in Figure 1. With the reduced length of structure, only two of the load test sites were relevant to the design of the foundation piles. The locations of these load test sites are shown in Figure 1.

Based on the results of the preliminary test program, Douglas fir timber piles, which were approximately 50 ft long, were selected with a design load of 70 tons. Alternatively, the contractor was permitted to use 12-in² precast, prestressed concrete; 14-in monotube; or Raymond step taper. The winning contractor chose to use timber piles in the structure. Because of the higher-than-usual design loads, a review of the preliminary test program, the structural design, and the driving experience will be of interest.

SOIL CONDITIONS

The site is located in a large east-west trending intermontane basin that was occupied during the glacial period by an arm of old Lake Lahonton. From the original ground surface to depths that vary from about 5 to 20 ft, the soil along the viaduct alignment consists primarily of clayey silts and silty clays. This material is relatively firm above the water table and soft to very soft below it. The groundwater surface fluctuates seasonally with the surface of the adjacent Humbolt River. Below these layers of silts and clays, to a depth of about 60 to 70 ft, the soil consists chiefly of slightly compact, medium to very coarse sand and gravel.

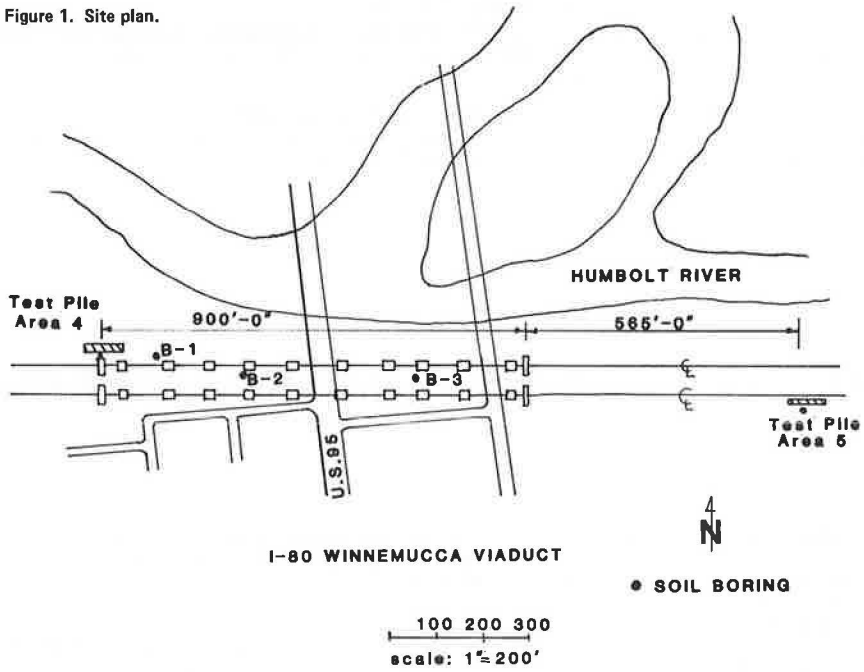
Five borings are available from near the structure and preliminary pile load test sites. The boring locations are shown in Figure 1. The borings were made by both 2.875-in wet-rotary and 8-in hollow-stem auger methods. Split-spoon samples were obtained at regular intervals in each of the borings by using the standard penetration test (SPT), according to ASTM 1586. Two of the boring logs obtained at the pile test sites that are representative of the area are given in Figures 2 and 3.

PRELIMINARY PILE TEST PROGRAM

At the test sites presented here, six pile types were tested. They were steel H-sections, treated timber, 12-in² prestressed concrete, monotube, Raymond step taper, and 12-in-diameter closed-end steel pipe. The details of the piles are given in Table 1, and they are shown in place relative to the soil profile in Figures 2 and 3. Anchor piles were of untreated timber, and the general arrangement of the load test setup is shown in Figure 4. This arrangement was attractive in that it was possible to perform three static load tests at the same time. At each site, the timber anchor piles were driven first and then followed by the static test piles. All of the test piles were driven by a DELMAG D-30 hammer, which is rated by the manufacturer at 54 140 ft-lbf. The driving records for the test piles are shown in Figures 5, 6, and 7. All of the test piles drove quite easily; the maximum blow count for all of the test piles was about 80 blows/ft for the monotube at site 4.

Two static-load tests were performed on each test pile. The first test was the 48-h American Association of State Highway and Transportation Officials (AASHTO) test. In this test, the pile was loaded in 10-ton increments at a rate of one increment every

Figure 1. Site plan.



10 min until a load of twice the anticipated design load was reached. The settlement was read before and after the application of each load increment. The load was then maintained for 48 h and the settlement read once each hour. Unloading proceeded in the reverse of the loading procedure. The second load test was the modified Texas quick test. In this test, the load was applied in increments of 5 tons every 2.5 min. Pile top settlements were read before and after each application of each load increment. Loading was continued until plunging failure or 200 tons, which is the capacity of the testing system.

The test results were evaluated by using two

Figure 2. Soil profile, test pile at site 4.

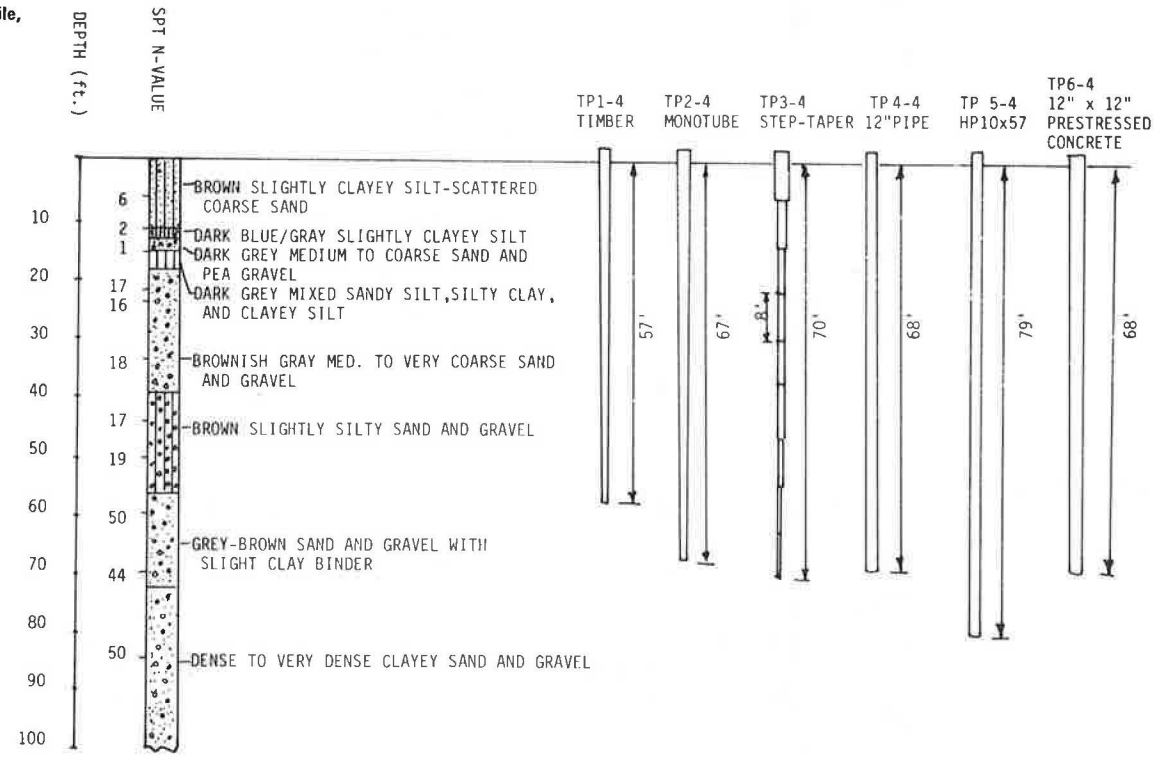
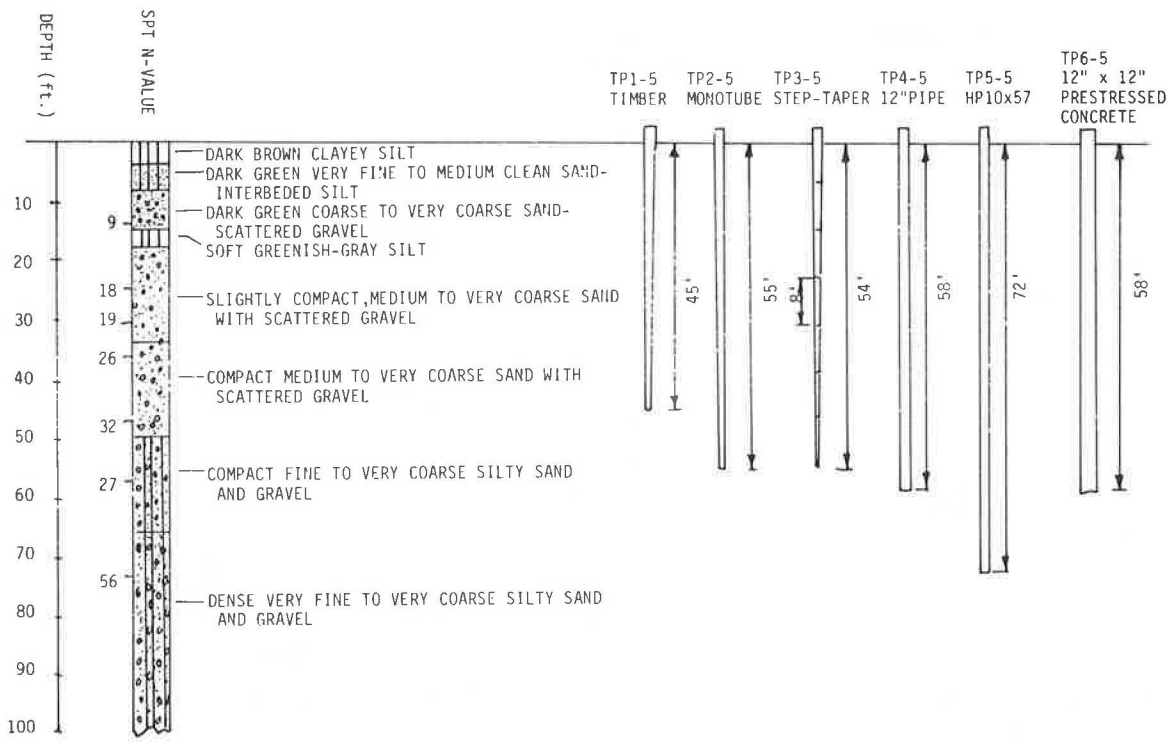


Figure 3. Soil profile, test pile at site 5.



different procedures to define the ultimate capacity. The procedure used by the Nevada State Highway Department is the double tangent method. In this procedure, the ultimate capacity is defined as the load at the intersection of tangents drawn to the load test curve at its beginning and end. The other procedure, which is the Davisson method (1), has been used in evaluating data obtained in the Case piling research project. Static load test curves from site 5 are shown in Figures 8-13. All six piles at site 4 carried the full 200-ton capacity of the static testing system, so the results

Table 1. Test pile details.

Pile No.	Pile Type	Driven Length (ft)	Cross-Section Description
TP1-4	Timber	57	14.7-in-diameter butt 7.3-in-diameter tip
TP1-5	Timber	45	14-in-diameter butt 10-in-diameter tip
TP2-4	Monotube	27 40	18-in ND-type N7 gage top 18-in butt-type J7 gage
TP2-5	Monotube	15 40	18-in ND-type N5 gage top 18-in butt-type J7 gage
TP3-4	Step taper	70	9-in tip, 1-in steps each 8 ft
TP3-5	Step taper	54	11-in tip, 1-in steps each 8 ft
TP4-4	Pipe	11 57	12.75-in outside diameter, 0.5-in wall top 12.75-in outside diameter, 0.4375-in wall
TP4-5	Pipe	45.5 12.5	12.75-in outside diameter, 0.5-in wall top 12.75-in outside diameter, 0.4375-in wall
TP5-4	HP 10x57	73	
TP5-5	HP 10x57	72	
TP6-4	Prestressed concrete	68	12x12 in
TP6-5	Prestressed concrete	58	12x12 in

Figure 4. Load test setup.

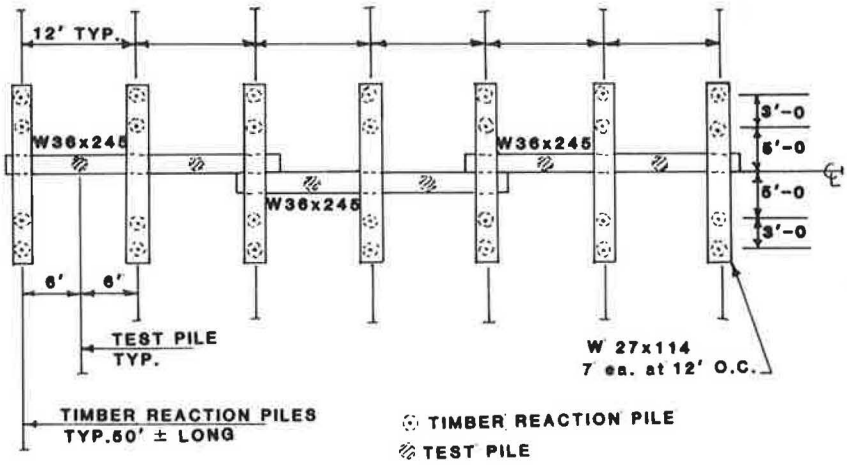


Figure 5. Driving records for timber and monotube piles.

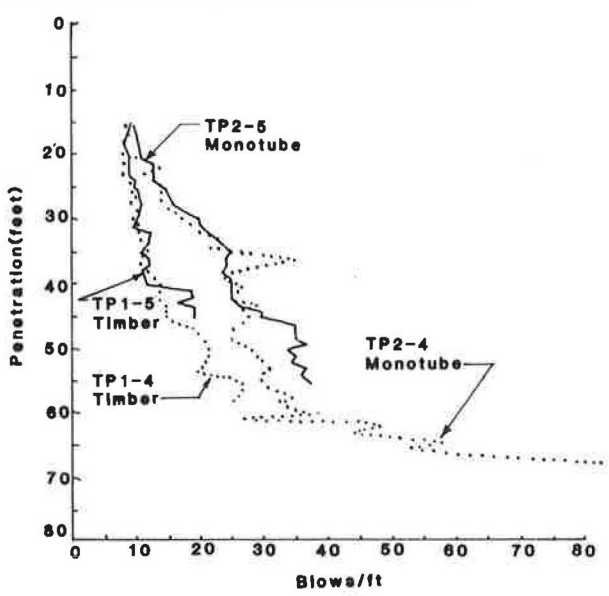


Figure 6. Driving records for pipe and step-taper piles.

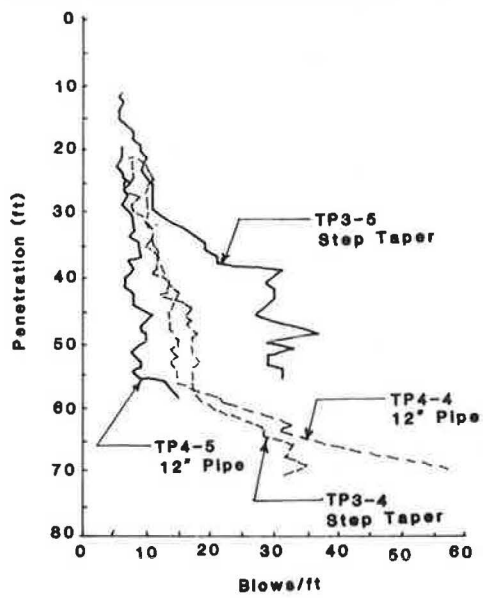


Figure 7. Driving records for prestressed and H-piles.

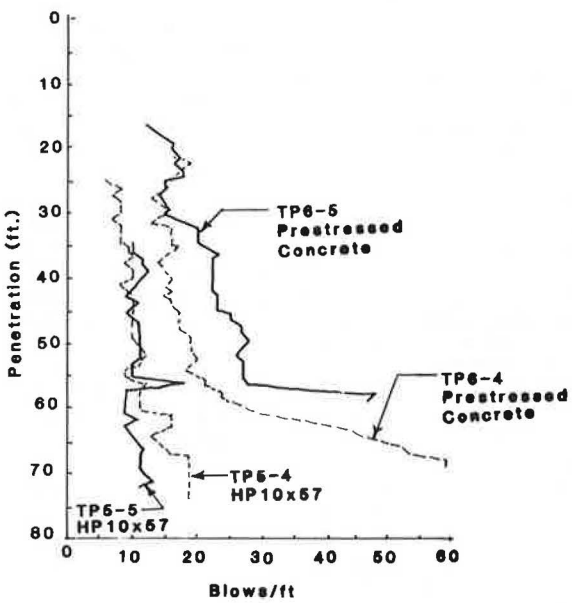


Figure 8. Timber pile static load test.

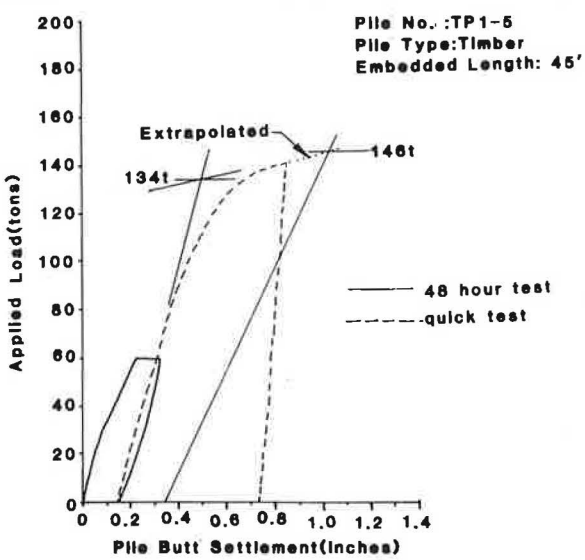


Figure 9. Monotube pile static load test.

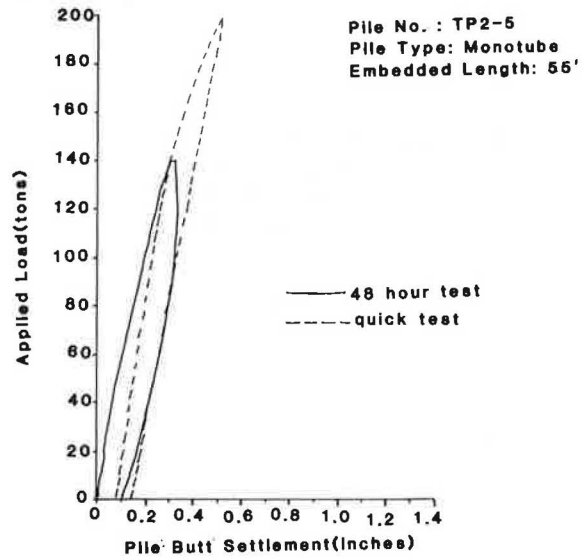
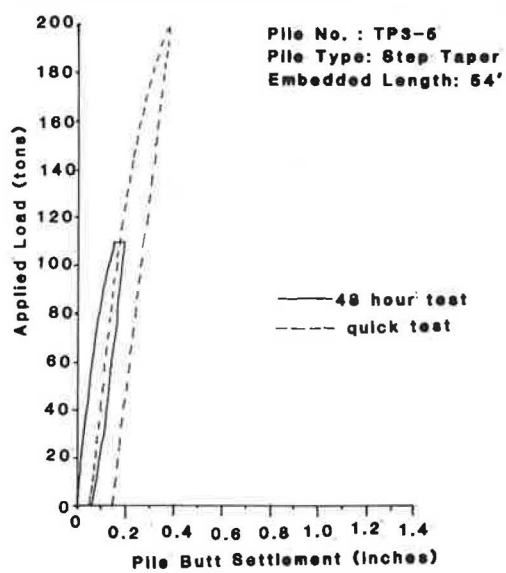


Figure 10. Step-taper pile static load test.



are not tabulated and the load test curves are not presented. The results of the evaluation of the tests at site 5 are given in Table 2.

An extensive dynamic test program was also performed at site 5. Measurements of force and acceleration at the pile top were made during the driving of the test piles. After completion of the static load tests, several of the piles were restruck with several different hammers, and again pile top force and acceleration were measured.

A procedure for determining static pile capacity during impact driving was developed at the Case Institute of Technology beginning in 1964 (2). This procedure, known as the Case method, requires the measurement of force and acceleration at the pile top during the hammer blow. These measurements have become quite routine. The capability to measure impact stresses and hammer energy transferred to the pile have been useful secondary developments. A pile-driving analyzer can perform these computations in the field. One such device was used during the driving of the test piles at Winnemucca site 5. More compact, versatile, and reliable pile analyzers have been constructed since the time of the Nevada tests.

During dynamic testing, the records of force and acceleration were also recorded on analog magnetic tape. This record could then be reanalyzed in the laboratory, and Case method capacity and energy delivered to the pile top could be recalculated. Energy delivered to the pile top is calculated from the following relation:

$$E(t') = \int_0^{t'} F(t)v(t)dt \qquad (1)$$

where F is the measured force at the pile top and v is the associated velocity, both given as a function of time (t). The maximum value of delivered energy

(ENTHRU) is a common measure of the performance of the hammer and driving system.

A second method of capacity determination was developed during the Case project. This method, known as the Case pile wave analysis program (CAPWAP), treats the pile as elastic by using the Smith model (3). Soil resistance parameters are calculated from the measured force and acceleration input. An elastic dynamic analysis is required, so a substantial computational capability must be used. CAPWAP cannot be done easily in the field and real-time results cannot be obtained.

The results of the tests are summarized in Table 2. Piles 2-5 and 3-5 were not tested dynamically either during driving or restriking. Pile 2, the monotube, was driven at a later date, and measurements could not be made. Dynamic measurements were made at the top of the Raymond step-taper mandrel. However, they were very noisy due to the reflections from the mandrel joints and they could not be satisfactorily processed. Both of these piles were filled with concrete prior to static load testing. Restrike testing was not successful due to difficulties in determination of the composite elastic modulus.

It can be seen from Table 2 that the agreement between the Case method capacity and the static load test results was good. It should be evaluated by comparing it with the Davisson capacity for which it was developed. Restrike capacities should be used to compare it with the static load test results, since there was a strength gain with time. The Case method results for the timber pile were 16 percent higher than the Davisson capacity. The other results were in error by smaller amounts. In fact, it

Figure 11. Pile static load test, 12.75-in pipe.

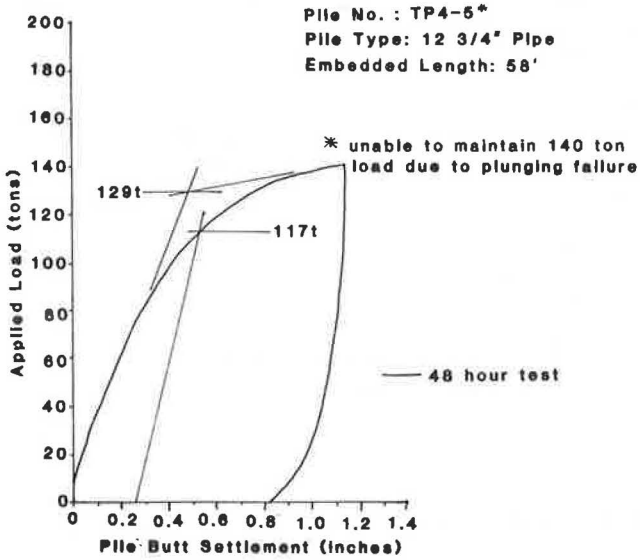


Figure 12. H-pile static load test.

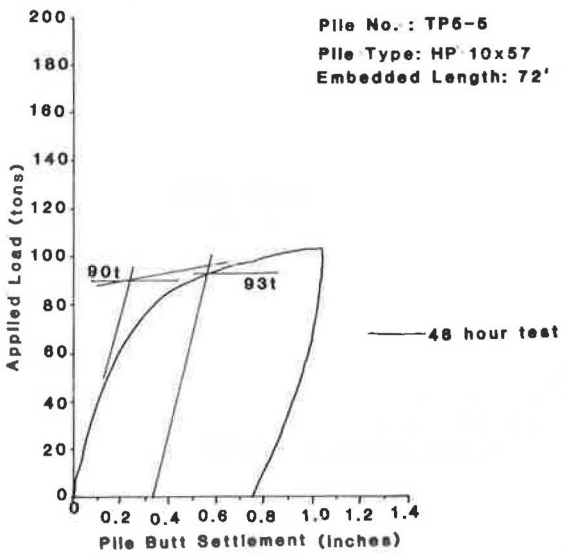


Figure 13. Prestressed concrete pile static load test.

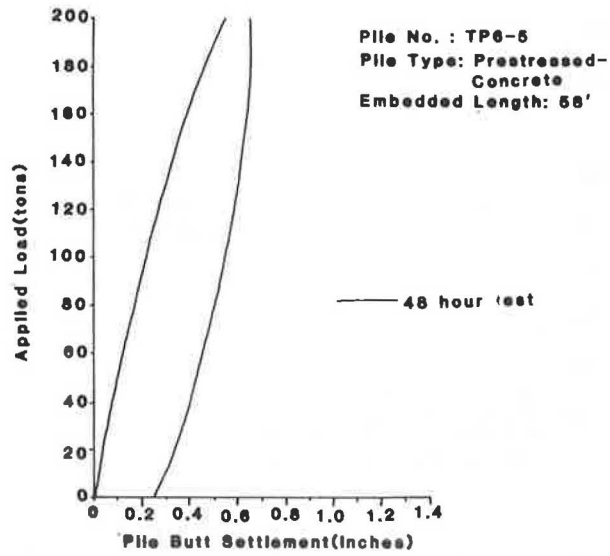


Table 2. Capacity test results for site 5.

Pile	Pile Type	Test Results, Static Capacity (tons)		Time of Dynamic Test	Blow Count (blows/ft)	Dynamic Capacity (tons)		
		Double Tangent	Davisson			Case Method	CAPWAP	WEAP
1-5	Timber			End of driving	19	144	149	160
1-5	Timber	134[a]	146[a]	Restrike	44	170	NA	220
2-5	Monotube			End of driving	38	NA	NA	212
3-5	Step taper			End of driving	32	NA	NA	220
4-5	Pipe			End of driving	20	104	100	190
4-5	Pipe	129[b]	117[b]	Restrike	27	130	151	220
5-5	H			End of driving	12	88	115	110
5-5	H	90[b]	93[b]	Restrike	17	93	96	160
6-5	Prestressed			End of driving	41	139	NA	260
6-5	Prestressed	200+[a]	200+[a]	Restrike	120	301	NA	460

Note: CAPWAP = Case pile wave analysis program.
[a]Texas quick test.
[b]AASHTO test.

Table 3. Restrike test results.

Pile	Pile Type	Hammer	Case Method Capacity (tons)	Blow Count (blows/ft)	ENTHRU (kip-ft) Measured	ENTHRU (kip-ft) WEAP	Apparent Efficiency (%)
4-5	Pipe	DELMAG D-30-10[a]	130	27	22.2	28.7	77
		DELMAG D-30-8	171	46	18.7		
		DELMAG D-30-6	160	51	15.8		
		Kobe K-22	140	70	15.0		
5-5	H	DELMAG D-30-10	143	17	14.1	27.6	51
		Kobe K-22	126	21	13.2	13.2	100
		Link-Belt LB-520	68	23	9.6	12.6	76
		Vulcan No. 1	70	45	6.1	9.4	65
		Raymond R-80CH	83	22	10.6	16.2[b]	65
6-5	Prestressed concrete	DELMAG D-30-10	301→198[c]	120→63[c]	14.1	19.9	71
		DELMAG D-30-8	192	69	15.8		
		DELMAG D-30-6	182	57	14.3		
		Kobe K-22	160	105	9.5		
		Link-Belt LB-520	144	118	7.4		

[a]This notation indicates that the D-30 hammer was operating with the throttle at setting 10, or fully open.
[b]Data on this hammer were not available when the WEAP run was made, so the Vulcan 80C was used as a replacement.
[c]Parameter indicated changed in the range given during the test.

is interesting to note that the difference between the two static test evaluation procedures was of about the same magnitude as the difference between the Davisson method and the Case method capacity.

The prestressed concrete pile could not be tested to failure with the 200-ton capacity test. Of particular interest here is the very large strength gain measured by the dynamic method between driving and restrike. This result is supported by the blow count, which almost tripled. No explanation is offered for the fact that the concrete pile showed so much more strength gain than the other pile types. Analyses by the CAPWAP method (3) produced results similar to the Case method except for the timber pile, where a difference of about 30 percent occurred.

Wave-equation analyses were also made by using the WEAP program. The input data used were considered to be typical for the conditions being analyzed. Most of the results obtained were considerably higher than either the static load test results or the dynamic predictions made by both the Case method and CAPWAP. The most likely reason for this difference is probably hammer performance. If the hammer was not performing properly (as assumed in the wave-equation analysis), then the blow count would be increased. Thus, the capacity predicted would be too high. One other factor is of considerable practical importance. Most of the blow counts recorded were quite low. In this region of the bearing graph, the curve is very steep. Thus, a small change in the blow count indicates a large change in capacity.

During restrike testing, several different hammers were tested, ENTHRU was measured, and a careful measure of the blow count was made. ENTHRU is defined as the maximum value of the energy passing through the pile top, as calculated from Equation 1. The blow count was measured by attaching a piece of paper to the pile and then drawing a pen along a reference attached to the ground so that a record of pile motion is recorded. Thus, the penetration from each hammer blow could be recorded. The results are presented in Table 3.

For TP4-5, the effect of the throttle on the D-30 performance was tested. This hammer had 10 positions on the throttle. At settings below about position 4, the hammer would not run. It is seen that positions 6 and 8 give a measurable reduction in energy output. This is a very useful tool in controlling tension stresses with concrete piles in easy driving.

Extensive testing was done on TP5-5, the steel H-pile. It should be emphasized that in order to obtain the data from all of the various hammers, the pile was driven several feet. Five different hammers were tested: DELMAG D-30, Kobe K-22, Link-Belt 520, Vulcan No. 1, and Raymond 80CH. In the dynamic test, Case method capacity was determined, and it is seen that, as the pile was driven, the capacity broke down from 143 tons to about 70 tons. ENTHRU was also calculated at the pile top and values are recorded for each hammer.

Wave-equation analyses were also made and ENTHRU calculated with WEAP by using the dynamically determined capacity. This ENTHRU calculation assumed the usual hammer and driving-system parameters. The values tabulated in Table 3 represent expected hammer performance. The apparent efficiency is simply the ratio of the measured ENTHRU to the value calculated by WEAP.

The results presented support the conclusion that the D-30 tested was not performing up to its expected operation. However, the K-22 was operating very well. The Link-Belt 520, Vulcan No. 1, and Raymond 80CH did not perform at expected levels. More data of this type would be desirable so that improved recommendations on hammer efficiency values for use in wave-equation analyses could be made.

The above comments only apply to the particular hammers tested on this job. Other hammers of the types tested here are known to have performed better at other sites.

FOUNDATION DESIGN

The pile design load was selected to be 70 tons by using a factor of safety of two against soil failure for the lowest load carried by any of the timber test piles. The design was based on timber piles, but other types were permitted as alternates. The design timber pile was assumed to have an 8-in-diameter tip and a 15-in-diameter top 3 ft below the butt and a length of 50 ft at the 15-in diameter. Due to the pile taper, the critical stress location will depend on the load transfer characteristics of the soil. All of the load was assumed to be carried in granular material on the lower 31 ft of the pile. The force distribution used for design purposes is given in Figure 14. With this force distribution, the stress distribution is also given in Figure 14. The maximum stress in this analysis is 1158 psi with a 70-ton design load. If the same stress-transfer assumptions are made for test pile TP1-4 for the 200-ton applied load and the dimensions given in Table 1, then the critical stress during the static load test was 3300 psi.

The Nevada State Highway Department uses the load factor procedure as defined by AASHTO in bridge structural element design. Thus, the various load

Figure 14. Force and stress distribution in design pile.

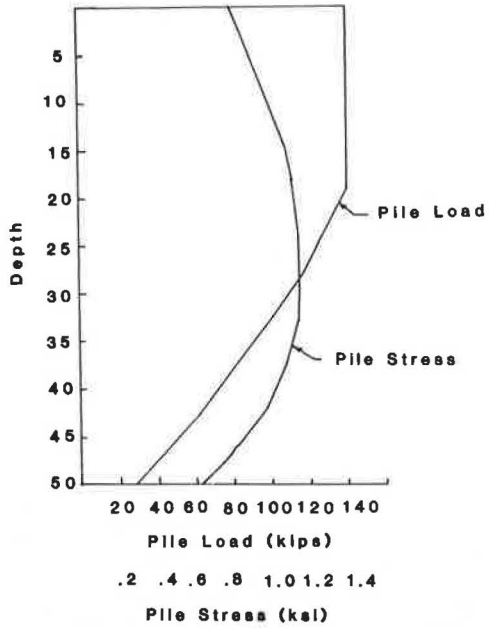
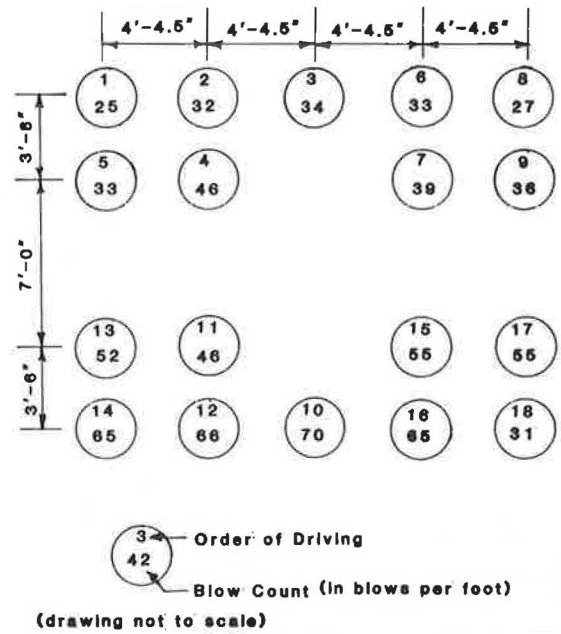


Figure 15. Driving resistances for pier 10W.



contributions for each load condition are carried down the structure and combined according to accepted procedures for the strength design of each element. In foundation design, pile reactions are calculated for the factored loads in order to design the pile cap. However, since piles are designed by using working stresses, working loads must also be carried to the piles. They are also combined according to the AASHTO working stress procedures, and critical conditions are calculated. Because pile design is based on ultimate strength concepts that have a rather arbitrarily selected factor of safety, a great deal of rationality would be added to the procedure if load factor methods were extended to include pile design.

The structure consisted of precast, prestressed box sections erected in a simply supported configuration in spans up to 100 ft. After erection of the precast sections, they were made continuous in three span units by placement of a cast-in-place deck with negative-moment reinforcement over the piers. The structure was analyzed as continuous for live loads. The piers had a wide single column configuration with a single footing under each pier. Thus, eccentric live loads arising from traffic in a single lane induce substantial moments to the footing, and live load effects can be substantial.

Individual pile groups under the piers ranged in size from 18 to 26 piles. All of the pile groups had some piles loaded to very near the design load under some load conditions. Controlling conditions were either the dead plus live load combination (AASHTO group I) or the dead plus live plus temperature combination (AASHTO group IV). The highest dead loads were about 45 tons on one pile group. Because dead load was concentrically applied, all of the piles in the group were equally loaded. This gives a critical pile stress of 740 psi (assuming the same load transfer ratios as given in Figure 14).

CONSTRUCTION

The production piles were driven with a Kobe K-22 open-end diesel hammer, which is rated at 41 300 ft-lbf. Three static test piles were driven and tested. Two were driven to 25 blows/ft and the third to 87 blows/ft. All piles were loaded to 200 tons by using the Texas quick test defined earlier. One of the piles failed by soil failure at a defined failure load of 164 tons, while the other two carried 200 tons without failure. The driving criterion was defined to be 25 blows/ft with a prescribed minimum tip elevation. In the driving of the production piles, blow counts much higher than the specified minimum occurred prior to reaching minimum penetration. The difficulty was probably due to the densification of the granular material due to driving of the pile group. Some preboring was permitted to achieve the specified tip elevation.

It is interesting to examine the driving record for a foundation where no preboring was done. In Figure 15, the final blow count is shown together with the order of driving for one pier. All piles were driven to the same tip elevation. Except for pile 10, where the blow count is somewhat higher than would be expected, the driving resistance is reasonably consistent with what would be anticipated.

About 500 timber piles were driven for the structure. A total of six piles were damaged during driving and had to be replaced. Damage was either visually detected or was noted by a sharp decrease in blow count. The structure (as of August 1981) has been complete for about two years. It is not yet open to traffic, but it has been used by the contractor for trucks hauling base material for the completion of the roadway. These loads are probably equal to the operating loads that the structure will carry. The structure is performing well, based on a visual inspection.

CONCLUSIONS

Based on the data obtained in the preliminary test program and the results of the production driving, the following conclusions are justified:

1. The use of design stresses of 1200 psi in treated Douglas fir has proved to be successful and should be continued. With these design stresses, good quality construction-control procedures must be followed.
2. Timber piles can be successfully driven with

large open-end diesel hammers. Difficulties during production driving were practically nonexistent.

3. Dynamic-capacity predictions agreed well with the static load test results.

4. Measured hammer performance was poorer than predicted in most cases. Thus, construction control by using dynamic measurements or static load tests is necessary as design loads are increased.

5. A preliminary test program of the type conducted here can be expected to save large amounts of money.

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Pile Selection and Design: Lock and Dam No. 26 (Replacement)

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Lock and Dam No. 26 is a major navigation structure on the Mississippi River some 25 miles north of St. Louis, Missouri. At the site there is a large, unbalanced horizontal water load of 24 ft. The soils at the site are sands, gravels, cobbles, boulders, and clay tills that are 80 ft thick. The history of, and logic for, the selection of piling on this project is presented. The soil and foundation information available at each stage of design is outlined. The interrelation of capacity determination by testing or by computational methods is discussed. The design process is analyzed and a critique is furnished. An evaluation of pile test extent and timing by using decision-analysis techniques is recommended.

Large projects generally have long histories. The size and related logistics are principal contributors to this lengthy process. Response to conflicting interests, reviewing agencies, and differing engineering advice also provides interruptions. The intent of this paper is to follow the selection of pile type and design capacity through the intermittent stages of a large project with a view toward improving this selection process.

GENERAL

Existing Locks and Dam No. 26 is located on the Mississippi River at Alton, Illinois (Figure 1). The existing structure consists of semigravity locks 110 ft wide by 600 and 360 ft long; the walls are supported principally on vertical 35-ft-long timber piling. The dam portion includes 32 tainter gate bays that are 40 ft wide and are also supported on short vertical wood piling. The soils at the site consist of alluvial sands and gravels grading coarser with depth to limestone bedrock at 65 ft below the base of the structure. The zone of pile embedment is composed of fine to medium sands with variable density. The riverward lock wall has displaced horizontally more than 10 in, and other lock walls have displaced varying distances up to 6 in. Early construction problems, notably the failure of the third-stage cofferdam, are related by White and Prentis (1). Extensive scouring of the river bottom attended this failure. The construction of the riverward auxiliary lock was done in sands placed in this area by dredging shortly before pile installation. The piles were jetted and then seated by driving an additional 5 ft or to refusal. There has been no observed failure of the piles themselves. Drill cores and diver examination of the piling show strong, firm timbers. The problem appears to be inadequate lateral support from the soil and pile system when subjected to a large number of load repetitions. This deficiency is present even though when this structure was designed in the early 1930s a full-scale pile testing program was instituted. The effects of cyclic loads were evaluated through numerous repetitions of a horizontal load on single- and multiple-pile monoliths. The tests were performed within the main lock area, not the auxiliary lock. The tests were reported by the principal engineer, S.B. Feagin (2).

The replacement lock and dam are located about two miles downstream from the present structure (Figure 1). Foundation support and lock chamber shape and size are the principal features altered for the new structure. The present concept for the replacement structure consists of a single 110x1200-ft U-frame lock and nine 110-ft-wide tainter gate bays for the dam. These configurations are shown in Figure 2 (section) and Figure 3 (plan).

Several stages of design can be recognized in the development of these configurations. The U.S. Army Corps of Engineers labels these as survey, general, and detailed stages. Most engineers use similar labels for steps within their practice. Survey involves the evaluation of several major alternative structures and sites by using limited available information and experience. General and detailed, as the names imply, involve increasing amounts of basic information and refinement of design features.

The following discussions relate the amount of information available and the procedures used to establish pile type and predict capacity at each