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)

BRUCE H. MOORE

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

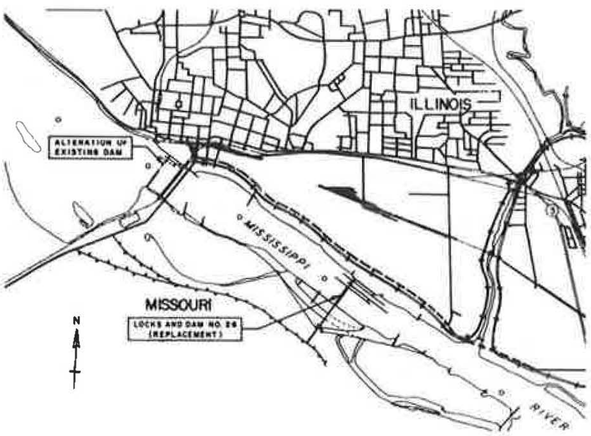
Existing Locks and Dam No. 26 is located on the Mississippi River at Alton, Illinois (Figure 1). The existing structure consists of semigravity locks 110 ft wide by 600 and 360 ft long; the walls are supported principally on vertical 35-ft-long timber piling. The dam portion includes 32 tainter gate 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

Figure 1. Site map.



stage. It is not the intent here to enumerate and compare the values assigned as pile capacities but to compare the general procedures used to develop the capacities. These procedures can be reduced to four basic approaches:

1. The chosen pile can by physically placed on site and tested,
2. The results of tests at a similar yet remote site can be used,
3. Soil parameters can be determined and capacities estimated indirectly through the use of many varied formulations based on either soil shear theory or past testing and performance, and
4. An individual long experienced in an area can simply assign a capacity.

This last method is the least scientific and is normally applied with considerable conservatism. Conservatism also attends the use of formulas and soil parameters. The factor of safety assigned is one measure of conservatism in the formula approach. Extensive exploration and careful testing in parameter selection are other avenues to conservatism. These conservative approaches cost dollars in the selection of numbers and type of pile. Conversely, full-scale testing is also costly. A cost-balancing approach is outlined and recommended.

SURVEY REPORT STAGE

A survey report was accomplished during the period 1964-1968. The purpose of this report was to clearly define the major alternatives available to counter the deterioration and capacity limitations of the existing locks and dam. Rehabilitation of portions of the existing dam and replacement of the lock was one alternative. Relocation of the structure at various locations upstream and downstream was also considered. The chosen alternative was construction of the new lock and dam at the downstream site noted in Figure 1.

Six borings were taken at the new downstream location to establish the foundation conditions. Figure 3 presents the plan location of these borings. Pile type and capacity determinations made at that time were based solely on this limited foundation information and the designers' and reviewers' combined experience and preferences. The poor performance of the existing structure strongly influenced the determinations. Battered H-piles to rock was the clear-cut directive. The reviewers clearly stated their experience-based preference for H-piles to rock. Alternative pile types and lengths

Figure 2. Typical dam and lock sections.

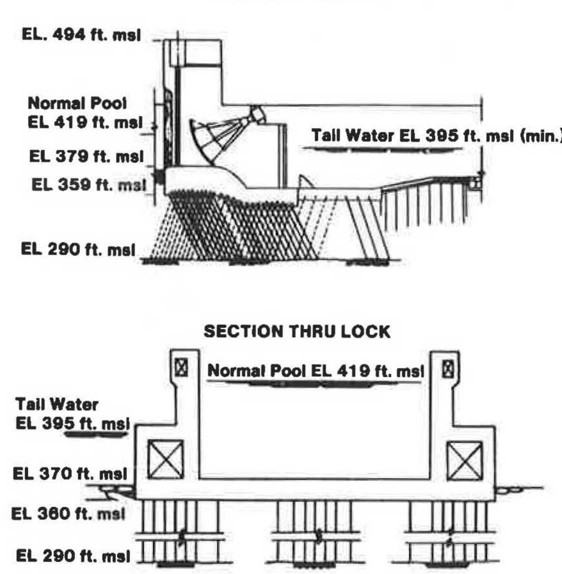
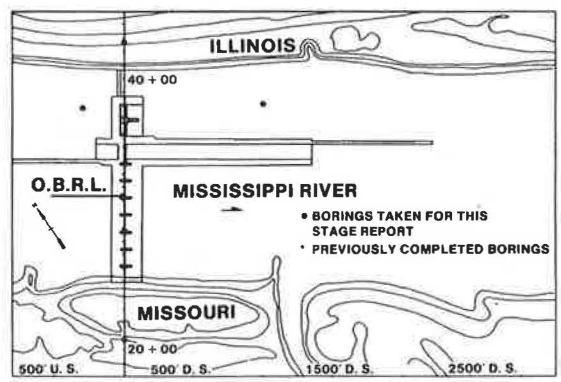


Figure 3. Overwater boring locations: survey report.



could be, and would be, considered in future studies, but a predilection to this type of foundation was clearly established.

Pile capacities for tension, compression, and horizontal loadings were estimated from experience and the results of remote testing. In this case, the remote tests were performed on the Arkansas River Project (3,4). A large pile load test program was envisioned. It was scheduled for accomplishment during the initial construction phases.

GENERAL DESIGN STAGE

The general design memorandum studied during the period June 1968 through July 1977 was intended to establish all major features of the project. Pile type, capacity, and configuration were included in these studies.

In preparation for the general design memorandum studies, approximately 60 additional borings were taken. Figure 4 shows the extent of this exploration program. Figure 5 presents a typical boring profile. Available to the engineer were N-values for 2- and 3-in outside diameter spoons, drillers' evaluations regarding the presence of cobbles and boulders, D_{10} and grain-size information results, some undisturbed densities, and drained direct shear

Figure 4. Overwater boring locations: general design memorandum.

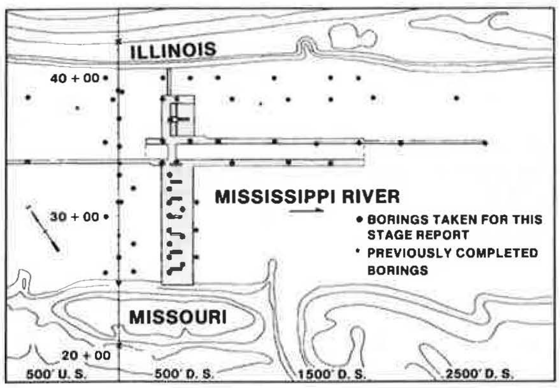
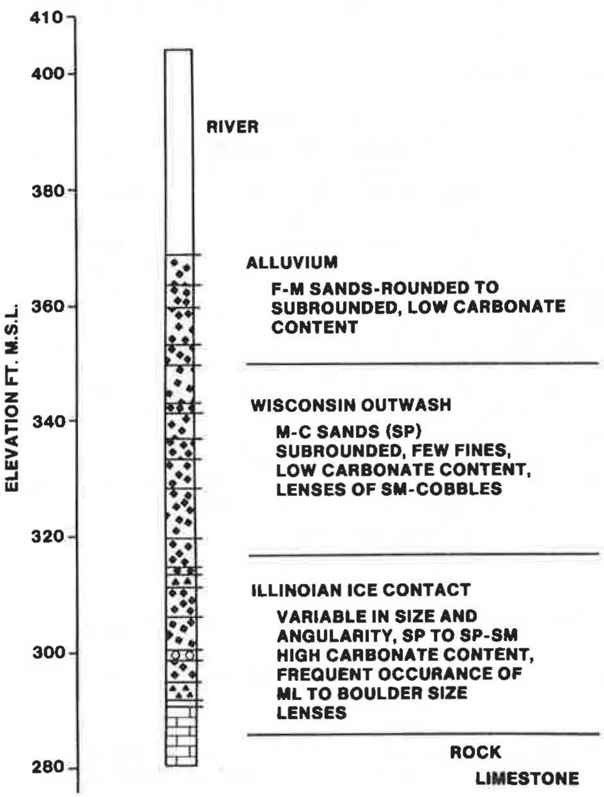


Figure 5. Typical boring profile.



data from testing on reconstituted sand samples. Boulders and cobbles were found scattered across the site. The thickness of these zones varied greatly. Cobble and boulder conditions were labeled heavy when the zone exceeded 15 ft, medium when the zone ranged between 5 and 15 ft, and light where less than 5 ft thick.

The results of the boring program were examined and evaluated in many conferences and informal discussions among the designers and reviewers. Pile types were studied in light of the added exploration information. Experience remained the only tool available to assess the choices. Steel H-piles continued as the selected type. The added information on cobbles led to inclusion of tip reinforcement in the selected foundation.

Design loads were also reevaluated because of increased foundation information. Calculations that used static formulas and the limited soil shear data yielded a new set of capacity estimates. These data were combined with the estimates that used remote (Arkansas) tests to arrive at values for this design stage. A two-phase testing program was developed. The first phase was designed as an overwater, quickly completed driving test to assess the practicality of requiring that the piles penetrate to rock. The second phase, which was for the determination of acceptable loadings, would be accomplished during initial construction of the dam and would follow the format developed during the survey report stage.

The first phase consisted of driving the favored H-piling with a series of three impact hammers and different tip reinforcements. Three locations where heavy, moderate, and light boulders could be expected were chosen for the tests. Tension tests (overwater) were added shortly before driving started. The tension results were not available for this stage of design but were used to evaluate and adjust the allowable loads for later detailed design stages. The driving results supported the original H-pile decision for the dam. Results showed that where medium or light cobbles were present, H-piles protected by tip reinforcement could be driven to rock. Details of this testing program can be found elsewhere (5).

ROLE OF CONSULTANTS

It was during the general design stage that several internationally known consultants were retained to review the progress to date. Among these was one expert in foundation design. This individual provided an invaluable overview. He noted that, while much detailed blow count and shear-testing information was available, no cogent geologic history had been established. He recommended that the focus of the exploration program be adjusted to provide an improved understanding of the overall geology. This improved geologic knowledge would, in turn, yield greater appreciation of detailed engineering requirements.

Geologically oriented profiles were developed for the total area. Multiple exploration methods, including electric logging and overwater seismic evaluation of the overburden, were used. Figure 6 shows one of these profiles. The profile depicts the geologic history of the Mississippi River at this location. At the top of the profile is the recent alluvium. Then the Wisconsin glaciation is represented by an outwash zone. In some areas, the two are intermingled to provide an alluvial-outwash zone. To this point in depth, the normal picture of a river valley filled with stream flow sediments is present. The underlying zone is composed of a heterogeneous mixture of sand, gravel, boulders, and clay till. The till can be found in both the lower and upper portions of this zone. Cobbles and boulders are scattered intermittently throughout. The deposit has both alluvial and morainic characteristics. It has been labeled ice contact material of the Illinoian Age. A patch of older Kansan alluvium and outwash smeared with a lense of the Illinoian clay till completes the picture. Engineering classifications of the soils found in each of these geologic zones are presented on Figure 5.

The added exploration required to develop the profiles yielded a bonus. Penetration resistance graphs were developed that compared 3- and 1.375-in-diameter spoon resistances in geologically similar materials. A discussion of these comparisons is presented by Moore (6).

DETAILED DESIGN STAGE

At Lock and Dam No. 26, the detailed design is

Figure 6. Geologic section (dam).

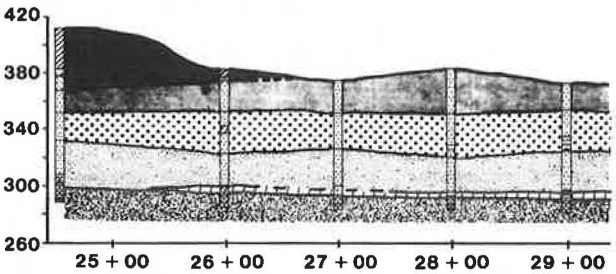
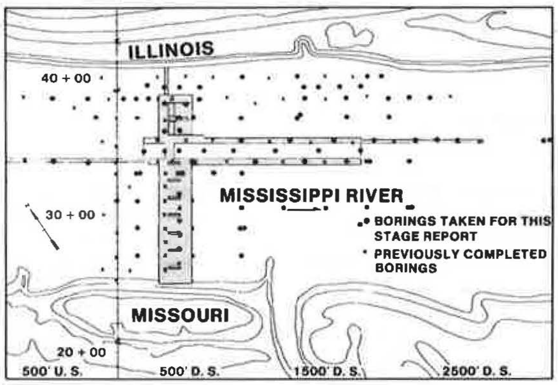


Figure 7. Overwater boring locations: detailed design memorandum.



presented in two reports--one for the dam and one for the lock. The report for the dam is complete. The lock memorandum is currently being prepared. Additional explorations include 85 borings, two large-scale pumping tests, overwater geophysical surveys, and dredge sampling of the river bottom soils. Figure 7 shows the added borings. The added explorations confirm the previously developed geologic sections.

The previous load capacity evaluation tools were static formulas, soil data, and remote test results. For the first time, factual test information was available from the actual site. The load testing accomplished in the overwater pile test program was used in developing the tension capacities for the dam.

Explorations at pile-founded Lock and Dam No. 24, which is about 40 miles upstream of the No. 26 site, revealed extensive voids under this structure. Some voids were also found during drilling for a stability evaluation of existing Dam No. 26. There was concern that earthquake or other vibrations could cause possible soil settlements beneath the new structure and create a similar void. The coefficient of horizontal subgrade reaction was effectively reduced to zero with a resulting increase in the number of piling.

The detailed design memorandum outlined a conceptual pile-testing program similar to that for the preceding stages. It was to be accomplished within the cofferdam during the first phase of construction. The testing was to include an evaluation of the hammer and resistance curves and load results. These relations would control the construction effort for the dam.

At this point, all design for the replacement structure ceased in response to litigation contesting the project. During litigation, there was a major testing program to evaluate methods available to rehabilitate the existing structure. This included chemical-grouting assessments, evaluating rock anchorage, and predicting and measuring lateral movements associated with driving piles near a loaded structure. An overview of this program was prepared by Lacroix, Perez, and Fieldhammer (7). In 1979, the final bars to the design and construction effort were lifted--a hiatus of 5 years.

CONSTRUCTION

After the restriction on design was lifted, the plans and specifications were completed by using the same soil information but also by applying the latest formulas together with the remote and overwater site test results. The pile load test program was expanded to include an evaluation of the accuracy of quick tests similar to those described by Fellenius (8).

ANALYSIS OF DESIGN PROCEDURE

It remains to be established whether any design (numerical) deficiencies or excesses exist. The initial choice (survey report) of pile type and penetration was arbitrary and experienced-based only. The initial capacities were determined through experience and application of remote (Arkansas) test results. The final choice (detailed design) of pile type was again experienced-based but supplemented by extensive explorations and on-site driving tests. The final capacities, as used in the detailed design, were determined through experience, soil testing and calculations, and limited overwater test results. Horizontal capacities were discounted because of unknowns related to the permanence of the soil support. As the construction phase begins, the assurance of capacities and constructibility are still the subject of future testing. The table below summarizes this history:

Stage	Basis for Design
Survey	Experience
General	Experience, remote tests, limited calculations
Detail	Experience, remote tests, driving tests at site, limited tension tests at site
Construction	Experience, driving tests, detailed load tests, production load testing

These procedures can be simplified further into conservative design and testing. Under a conservative design would be found calculations that used soil parameters, testing and performance at other structures, and that facet of engineering labeled experience. Testing would include all the steps from inclusion of results from limited site tests through detailed capacity testing and construction proof testing.

At Lock and Dam No. 26, both testing and conservative design have been used in developing the current plans. The decisions regarding timing and extent of testing were made by using nonquantified evaluations. The decisions tended to follow a direction that said "get as much foundation information as you need but at a minimum cost for the testing." The effect of test extent and timing on construction costs was not given formal consideration. Certainly, the responsible engineers were aware of these relations and included them in their thought processes, but no formal evaluation was developed. This lack of a formalized analysis is

Figure 8. Decision analysis: pile tests.

ACTIONS

a_1 — Capacities chosen from Experience.

a_2 — Capacities chosen from Tests.

a_3 — Negotiate Contract.

STATES

θ_1 — Experience gives valid Capacities.

θ_2 — Tests give valid Capacities.

PAY OFF MATRIX

STATES \ ACTIONS	a_1	a_2	a_3
θ_1	0	-100^k	-50^k
θ_2	-500^k	0	-50^k

considered a serious deficiency and may have allowed testing to be uneconomically delayed or unnecessarily included in the selected options. Had an analysis been developed for Lock and Dam No. 26, the questions regarding need, timing, and extent of pile testing could have been resolved logically and with consideration for total costs.

RECOMMENDATIONS

The choice of whether testing or conservative design is proper for one project is not necessarily valid for another. The design engineer must use his or her experience in early planning as to the best route through these choices. This may not be an engineering decision, but it certainly lies within the professional responsibilities of the engineer. It is to the benefit of the profession that the engineer recognize and translate these actions into a management decision format for his or her client.

The use of decision-analysis techniques is one means of combining available solutions and expected results into such a management decision format. Tummala (9) provides a good description of this method of analysis as applied to engineering decisions. Tummala describes the logical combination of "actions" and "states of nature" with attendant "payoffs". A simplified payoff matrix for pile load testing is shown in Figure 8. This very simple matrix is completely hypothetical. It illustrates three actions combined with two states. Position θ_2, a_1 means that capacities were chosen from experience, but that only tests give correct capacities. The engineer estimates that in such a case it could cost the owner $500 000 to correct the situation during construction of this hypothetical project. Other positions in the matrix are developed in a similar manner. Other states and actions could be introduced with timing of the testing and extent of testing included among them. Various techniques are available to allow logical selection of the most desirable course of action. In one case, risks can be minimized. Conversely, potential savings can be maximized. Other, more complex choices are available. All of these actions are described in Tummala (9).

A formalized evaluation of the need, timing, and economics of pile testing is strongly recommended for any project where such testing appears warranted. In addition, until the driving and testing of production piling are complete, conservatism must govern design thinking. Conservatism does not preclude a design that, after testing, can be adjusted to the higher or lower capacities revealed by the testing. Conservatism does not preclude use of up-to-date procedures for analysis of both pile capacity and load distribution. Whether conservatism is approached in an arbitrary choice of a factor of safety or in a very thorough evaluation of soil and rock parameters that use the most advanced techniques is immaterial. What is essential is that it be present in the design prior to completion of the testing of the driven production pile.

Analysis of the selection and design procedures at Lock and Dam No. 26 (replacement) has revealed a management deficiency, notably the lack of a formal decision analysis relative to pile testing. Four basic approaches to capacity prediction were outlined previously. These were experience, remote testing, calculations by using soils data, and on-site testing. The procedure used in developing capacities at various stages of design for Lock and Dam No. 26 (replacement) were examined and classified. The extensive explorations, iterative design stages, multiple review levels, and the examples provided by existing structures would seem to indicate that a large degree of conservatism has been incorporated into the chosen pile capacities. It will be interesting to ascertain that degree when the planned pile tests are completed.

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