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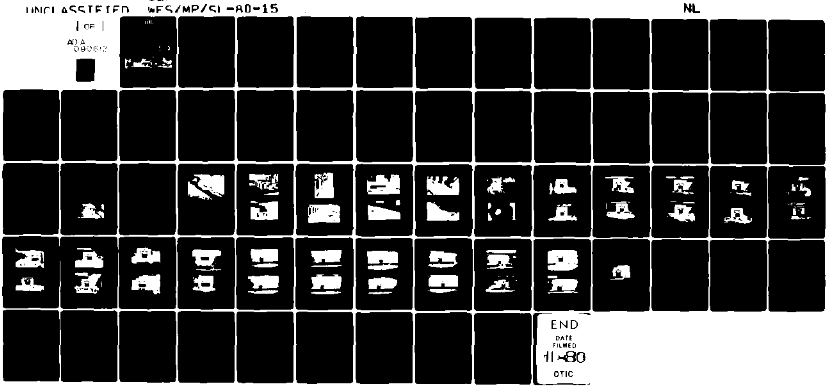

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STRUCTURAL STABILITY EVALUATION POKEGAMA DAM

by

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The 77-year old surface concrete of Pokegama Dam is generally in excellent condition. The average unconfined compressive strength of the concrete above approximately el 1264 is 4900 psi. The remainder of the concrete has a compressive strength at least as low as 1360 psi.

The monoliths of Pokegama Dam are adequate in their resistance to overturning, sliding, and base pressures if the clay seams in the foundation are investigated and found to be of such an extent and nature that stability and

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underseepage at the seams are not a problem. Except for the clay seams, the foundation material under Pokegama Dam appears to be competent and adequate.

The small amount of deteriorated concrete, mainly at the downstream ends of the piers should be repaired in order to prevent the entrance of water into cracks which will stop the freezing and thawing deterioration and eliminate the need for more costly repairs in the future.

The foundation is variable with clay seams present in core P-P4. It is recommended to:

a. Determine the extent of clay seams beneath the dam in the area of pier 4 by drilling upstream, between, and downstream of the piers.
b. Evaluate the effect of the clay seams on the structural stability of the dam piers.
c. Take remedial action necessary to insure that the possibility of any seams washing out is reduced to an acceptable level.

The right embankment of the dam should be investigated for voids and imperviousness.

PREFACE

The evaluation of the stability of Pokegama Dam was conducted for the U. S. Army Engineer District, St. Paul, Corps of Engineers, by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES). Authorization for this investigation was given in Intra-Army Order for reimbursable services No. NCS-1A-78-75, dated 7 July 1978.

The contract was monitored by the U. S. Army Engineer District, St. Paul, with main assistance from Messrs. Roger Ronning and Jerry Blomker. Their cooperation and assistance were greatly appreciated.

The study was performed under the direction of Messrs. Bryant Mather, William J. Flathau, and John Scanlon, SL. The structural stability was performed by Dr. Carl Pace and Mr. Roy Campbell. The core logging was performed by Mr. Sam Wong and Mr. J. Rhoderick under the technical supervision of Mr. Alan Buck. The testing was performed by Mr. Mike Lloyd. The core drilling was under the direction of Mr. Mark Vispi. Dr. Pace and Messrs. Campbell and Wong prepared the report.

Commanders and Directors during the conduct of the program and the preparation and publication of the report were COL John L. Cannon and COL Nelson P. Conover, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To Obtain
feet	0.3048	metres
cubic feet per second	0.0283168	cubic metres per second
inches	0.0254	metres
kips (force)	4448.222	newtons
kips (force) per square foot	47.88026	kilopascals
miles (U. S. statute)	1,609.347	metres
pounds (force)	4.448222	newtons
pounds per square foot	4.882428	kilograms per square metre
pounds (force) per square inch	6.894757	kilopascals
square miles (U. S. statute)	2,589,988.0	square metres

STRUCTURAL STABILITY EVALUATION

POKEGAMA DAM

PART I: INTRODUCTION

Background

1. Pokegama Dam (Figure 1) produces a reservoir which is one of the six Mississippi River headwater reservoirs maintained and operated by the U. S. Army Engineer District, St. Paul. The project map of the reservoir system is presented in Figure 2. The headwater reservoirs were authorized by the River and Harbor Act of 1880 to provide supplemental flow during periods of low flow in the interest of navigation on the Mississippi River at, and below, the Twin Cities. However, with the canalization of the Mississippi River below Minneapolis, Minn., the demands for storage releases from the reservoir system for navigation have been greatly reduced. Thus, in recent years the reservoirs have been operated primarily for other purposes, including flood control, recreation, fish and wildlife conservation, water supply, water quality improvement, and other related uses.

2. The drainage area above Pokegama is 3265 square miles, of which 1442 square miles are above Winnibigoshish Dam, 1163 square miles are above Leech Lake Dam, and 660 square miles are located between these two dams and Pokegama Dam.

3. The headwaters reservoir system is one of the oldest projects in the St. Paul District. The initial surveys and investigations were begun in 1867, at a time when the country was being opened up for development and settlement. Pokegama Dam was placed in operation in the 1880's. It is located 3 miles upstream of Grand Rapids, Minn., 175 highway miles (340 river miles) north of Minneapolis, Minn., and 80 miles west of Duluth, Minn. Documentation prior to construction was limited

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

to the amount required to develop the engineering feasibility of the project, reporting quantities, costs and justification for additional work or study, and requirements for construction. The original construction was almost entirely a practical field application in engineering with only basic theory to rely on; physical design was done in the field and only a minimum of documentation was maintained.

4. The original construction of the dam was of timber and rock-filled timber crib construction; the original structure was essentially replaced in 1903 by a new timber and concrete structure. Construction after 1903 has generally been repairs or rehabilitation of the existing structure. In 1956, a contract was awarded for reconstruction of sills and aprons. In 1968, a contract was let for the addition of sluice gates and operating machinery.

5. Additional background information for Pokegama Dam can be obtained from U. S. Army Engineer District, St. Paul (1965, 1972, and 1977).

Project Features

6. Principal project features of Pokegama Dam include the gated concrete structure, the earth-fill embankment, and four perimeter dikes around Pokegama Lake. This study concentrates on the stability of the concrete control structure.

Control structure

7. The present structure has concrete abutments and piers constructed on a highly variable foundation (see Parts III and V). The general construction layout of Pokegama Dam is presented in Figure 3. Some views of the dam in its present condition are presented in Figure 4. Pictures taken in 1949 of Pokegama Dam are presented in Figure 5. There are thirteen sluiceways 8.0 ft wide and one 12-ft-wide log sluice. Six of the sluiceways are controlled with sluice gates and eight have stop logs. A 3-ft walkway over the top of the structure provides access to the sluiceways. The total length of the structure between the abutments is 225 ft. The concrete top of the piers and abutments is at elevation 1279.27. Pertinent dam data are given in Table 1.

Structure evolution

8. Reconstruction in 1903. Because the original structure was of timber and rock-filled timber crib construction, it was replaced in 1903 by a new timber and concrete structure. The only portions of the original structure retained were the timber sluiceway floor aprons and the rock-filled timber cribs in the embankment at each end of the dam. The left and right cribs are 50 and 84 ft long, respectively. The sluiceway floor aprons were replaced with concrete in subsequent contracts. Because of its completeness and clarity, the following has been abstracted from page 2239 of the 1904 Annual Report of the Chief of Engineers to describe the 1903 reconstruction:

> "The new dam is built of concrete - in proportions, volumes loose, 1 of cement, 3 of sand, and 6 of stone - on the site of the old timber crib and stone structure, whose floor (elevation 1265.27 ft above sea level) was repaired and excavated for the purpose.
>
> "The dam is 247 ft long from outside to outside of abutments. It has twelve 8-ft sluices, one 12-ft log sluice, and one 8-ft fishway. The abutment piers are 54 ft long and 11 ft thick in the middle. The two log sluice piers are each 72 ft long, but one is 10 ft and the other is 11 ft thick, this extra thickness being necessary to compensate the loss in bulk of masonry due to space occupied by the valve flumes. The other eleven piers are each 34 ft long and 8 ft thick. The tops of all piers are level and 14 ft above the floor of the dam.
>
> "The log sluice is closed by a reversed Parker bear-trap gate, built of Oregon fir, with steel hinges and fastenings. The gate has a total rise of 12.75 ft. The least head of water under which the gate was tried was about six-tenths of a foot, and with this it worked smoothly and quickly. The minimum head required to raise the gate to its full height is not known, but under a head of 4.5 ft it rises from lowest to highest point in fifty-one seconds, and it falls the same distance in forty-two seconds without jar or jerk. For the purpose for which this gate was designed it can not be bettered.
>
> "The 8-ft sluices are closed by tainter gates having steel frames and oak-plank facings, and are manipulated by screw-geared winding spools and chains. No counterpoises are employed, and so the handling of the gates is necessarily slow.
>
> "Stop logs are used for closing all sluices during the winter, when, on account of ice, the regular gates can not be operated. For the placing of the stop logs and their transportation to and from the shore, an elevated trolley way with tackle hoists is provided.

"The wings of the dam have the old rock-ballasted cribs for cores, and are backed above and below with earthwork embankments whose surfaces are riprapped and sodded to prevent erosion.

"Above the dam are five timber crib piers, ballasted with stone, connected together with the log sluice of the dam with suitable booms for the control of logs and other floating bodies."

9. Structure modifications. In 1936, a stop-log sill repair was required to stop leakage through the stop-log closure. The timber sills were replaced with reinforced concrete sill plates. In 1941, a 3-ft-wide trench was excavated at the upstream face of the timber cribs at the backside of the abutments. New timbers were placed vertically at the upstream face of crib. A 3-in.-wide sand-bentonite seal was placed at the upstream face of the timbers, and the rest of the trench was backfilled with a clay-sand mixture. In 1956, a contract was awarded for reconstruction of sills and aprons. This contract provided for removal of the four remaining tainter gates, the Parker bear-trap gate in the log sluiceway, and removal of the valve operating machinery from the right log sluiceway pier. The timber and concrete aprons were replaced with a 1-ft-thick reinforced concrete slab. The bed of the Parker bear-trap gate was filled and capped with reinforced concrete. The valve recess and flume were covered and filled with concrete. The top of the timber cribs at the abutments was filled with impervious fill to elevation 1280.72, and the riprap was dumped on the upstream face of the rock crib and the right abutment. Since all the sluice gates, the four tainter gates, and operating machinery were removed by this contract, operation between 1956 and 1968 was by means of stop logs only. In 1968, a contract was let for sluice gate additions. This contract provided for installation of 9-ft-wide by 12-ft-6-in.-high built-up steel sluice gates and operating machinery in 6 of the 14 sluiceways. The top of the gate sills was raised to elevation 1266.77 by placing a new rein-reinforced concrete sill section in those bays. A new bridge, crane rail, and other structural modifications were required to accommodate the new slide gates. An electrical powered portable operator running on the crane rails operates the gates. Also, closure barriers utilizing aluminum stop logs were provided for the six bays having the sluice gates.

Objective

10. The objective of this study is to evaluate the stability of the concrete control structure. For this evaluation two cores were drilled through the dam (Figure 3) and into the foundation. The cores were examined and the structural stability of the dam was evaluated.

Scope

11. The study is limited to a structural stability evaluation of the concrete control structure with consideration given to concrete properties and foundation condition.

PART II: CORING PROGRAM

12. Limited coring was performed to obtain properties of concrete, foundation, and concrete-foundation interface. Four-inch and NX cores were obtained (Part III) with a skid-mounted rotary drill rig using diamond core bits and 5-ft-long double-tube swivel-head core barrels.

13. A typical drilling operation is shown in Figure 6. The core holes were located at approximately third points of the dam in pier 4 (P-P4) and pier 8 (P-P8) to obtain cross-sectional material properties. The locations of the two core holes are presented in Figure 3. The cores being obtained were changed from 4-in. cores to NX cores to eliminate the wear and replacement costs of an excessive number of diamond bits.

14. The coring program was oriented toward determining:

- a. Depth of deteriorated concrete.
- b. Uniformity of concrete with depth.
- c. Unconfined compressive strength of the concrete.
- d. Concrete-foundation interface properties.
- e. Geological characteristics of foundation material (discontinuities, bedding planes, weak seams, etc.).
- f. Uniformity of foundation material.
- g. Unconfined compressive strength of foundation material.

15. The coring program was a minimum for obtaining information on the concrete and foundation material. Since the dam falls into the classification of a low-head dam, it was initially thought that two cores would be adequate for this preliminary investigation. Clay seams were encountered in core P-P4, which will require additional coring to define the extent and nature of the seams. The extent of the clay seams was not investigated at the time of initial coring because winter was approaching, and it could have precluded finishing the coring and testing at the other five Upper Mississippi River Headwater dam sites. The integrity of the concrete and foundation material is presented in Part IV.

16. The core holes were not grouted, and a capped pipe was used to seal the opening in order that they could be used for obtaining piezometric data (Figure 7).

17. Pictures of the cores are presented in Figures 8 and 9.

PART III: PETROGRAPHIC REPORT AND CORE LOGS

Samples

18. Selected samples from two cores were delivered to the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES) for tests and examination. The concrete was placed in 1903. Four-inch-diameter cores were drilled through the concrete and into the upper portion of the foundation rock; an NX size core was then used to complete each hole.

19. Some of the concrete cores and all of the foundation rock cores, mainly NX size, were received at WES on 29 October 1979. Identification is as follows:

Hole No.	Location	Material	Size	Interval, ft
P-P4	Pier 4, 119 in. from the downstream edge, in the center of the 8-ft-wide pier	Concrete	4 in.	0.4 to 1-1/2; 9.6 to 11.0; 17.4 to 18.3; 19.8 to 20.9
		Foundation rock	4 in. and NX	20.9 to end of core at 36.8
P-P8	Pier 8, 119 in. from the downstream edge, in the center of the 8-ft-wide pier	Concrete	4 in.	0.0 to 0.7; 5.8 to 7.8; 14.4 to 16.5
		Foundation rock	4 in. and NX	16.5 to end of core at 25.5

Test Procedure

20. Each length of core was examined to determine the uniformity of the concrete and foundation rock, respectively. Pieces representing concrete of different appearance were selected for petrographic examination.

21. The pieces of concrete selected for examination were sawed longitudinally, ground smooth, and examined with a stereomicroscope.

22. Portions of the foundation rock were also examined with a stereomicroscope. A clayey portion of it was slurried onto a glass slide and allowed to dry. The resulting clayey film was then examined by X-ray diffraction, both air-dry and after saturation with glycerol. The X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

Results

23. All of the concrete was intact; there was no evidence of frost damage or deteriorative chemical reactions even though the concrete was nonair-entrained. The upper 15 ft of concrete was different from the concrete below it (Figures 10 and 11). This upper concrete was believed to have been placed in 1903. The concrete from the upper portion of both holes contained a well graded coarse aggregate composed of 1-1/2-in. maximum size igneous and sandstone rock particles. This nonair-entrained concrete was relatively dense and hard.

24. The concrete at depths greater than about 15 ft contained 2-in. maximum size coarse aggregate that was all sandstone. The nonair-entrained concrete was less dense with some void space created by point-to-point contact of aggregate particles and poorer consolidation. The coarse aggregate appeared to be gap graded with no intermediate size pieces of aggregate.

25. The paste in the deeper concrete was more friable and was very pale orange (10 YR 8/2),* while the paste of the concrete in the upper 15 ft of the holes was light gray (N 7).*

26. The concrete above the 15-ft depth had smaller coarse aggregate of different grading and composition. The different colors of the paste suggested the cement was different. The appearance and the

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* Standards set by 1975 "Rock-Color Chart," by the Geological Society of America.

condition indicated two concrete mixtures were used, and that the upper mixture was better. Both concretes contained a natural sand of mixed composition like the mixed igneous and sandstone coarse aggregate. The coarse aggregate in all of the concrete appeared to be natural gravel that had received some crushing. All of the breaks in the concrete of both cores were judged to be new ones that were due to the coring operation.

27. The concrete to foundation rock contact was loose in both cores. The foundation rock was fractured near the base of the concrete. The fractures were all iron-stained joint planes.

28. The foundation rock consisted of silica bonded sandstone grading to quartzite and hard intact sandstone grading to friable sandstone mixed with reddish clay (Figure 10). A sample of the clay consisted of clay-mica with some smectite* and kaolinite.

29. Clay content ranged from thin seams about one-sixteenth in. thick to beds 4 ft thick (Figure 10). Core P-P8 contained thin clay seams randomly spaced and inclined at about 70 deg.

30. A large amount of the major clay seam shown in Figure 10 was found in core P-P4, which was a longer core. Very little of the clay was recovered; it was located approximately as shown in Figure 10. The evidence for this was the change in drill water color, drill rig response to the material, and inability to recover the core.** The major clay seam was located at a depth of 25.2 to 28.6 ft in boring P-P4. The material below this depth consisted of alternate beds of sandstone and clay with some intermixing of fissile rock.

Summary

31. Examination of concrete core and foundation rock core from the two holes indicated the following:

- a. There was no deterioration of the concrete core due to frost damage or to deteriorative chemical reactions.

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* Swelling clay; the montmorillonite-saponite group.
** Information for site logs.

Nonair-entrained concrete will normally be damaged if exposed to freezing and thawing while critically saturated; the lack of damage to interior concrete showed this did not happen.

_b_. The upper 15 ft or so of concrete placed in 1903 was a denser mixture and appeared to be of better quality than the older concrete below this level. The difference is attributed to differences in composition and consolidation.

The contact of concrete to the foundation rock was loose in both cores.

32. The foundation rock was essentially sandstone with some clay seams. The clay seams ranged from 4 ft to one-sixteenth in. thick. The recognition and positions of the clay seams present in the foundation rock were largely based on interpolation, since much of the clay was missing. More cores of larger diameter than NX would be needed to locate the clay seams more accurately.

PART IV: STABILITY ANALYSIS

Introduction

33. Even though the monoliths of Pokegama Dam have been in service for long periods of time, it is important that they be examined in view of present-day criteria and in relation to deterioration experienced to assure continued structural adequacy. If the design or the deterioration makes the structures fail to satisfy current criteria, thereby producing unsafe or doubtful conditions of safety, the structure must be modified to conform to good engineering practice.

34. One of the main considerations for structural adequacy of a dam is the stability of the various monoliths when subjected to possible loading conditions. The stability study involves analyzing selected monoliths to determine if they have adequate resistance against overturning, sliding, and base pressures. Only one pier had to be analyzed for Pokegama Dam because:

- a. The most critical monoliths are the smaller ones without the weight of the operating gate superstructure.
- b. The smaller piers without the superstructure are stable; therefore, the other piers are stable unless the examination of the extent and nature of clay seams under the dam reveal that they affect the stability of the dam.

35. In general, the stability study was done in accordance with the applicable portions of the following Engineer Manuals and Engineer Technical Letters.

- a. EM 1110-2-2200, Gravity Dam Design, 1958.
- b. EM 1110-2-2607, Navigation Dam Masonry, 1958.
- c. ETL 1110-2-184, Gravity Dam Design Stability, 1974.
- d. ETL 1110-2-22, Design of Navigation Lock Gravity Walls, 1967.
- e. EM 1110-2-2602, Planning and Design of Navigation Lock Walls and Appurtenances, 1960.
- f. EM 1110-2-2502, Retaining Walls, 1961.

36. The adequacy of the structure to resist overturning can be judged by the location of the resultant with respect to the base of the

section where stability is being considered, within the monolith, or at the base-foundation interface. In general, the gravity monoliths where stability against overturning is being considered are required to have the resultant of applied loads fall within the kern of the base of the section being analyzed when subjected to active earth pressures or for monoliths not subjected to earth pressures. For operating conditions with earthquake, the resultant only has to fall within the base, but the allowable foundation stresses should not be exceeded.

37. The percent effective base (percent of the base which is in compression) is a good way to present where the resultant falls in a rectangular base section. It is a good guide for representing overturning resistance for any shape base. For example, for a rectangular base:

Percent Effective Base	Resultant Location Within Base
100	Within middle 1/3 or in kern area
75	At 1/4 points of base
50	At 1/6 points of base

38. Sliding resistance of a monolith is calculated by choosing a trial failure plane or combination of planes and calculating the resistance along this path. The critical section for sliding must be determined. It may be within the monolith, at the base-foundation interface, or at a plane or combination of planes below the base.

39. The resistance may be composed of several types. The sliding resistance due to friction and cohesion for a horizontal surface between the monolith and its foundation is calculated by the formula given in ETL 1110-2-184. The safety factor is obtained by dividing the horizontal resistance by the horizontal driving force. These formulas are inadequate for evaluating structural sliding on inclined planes. For inclined planes, the safety factor is obtained by dividing the resistance along the plane by the driving force along the plane with any passive resistance taken into consideration. The sliding resistance due to all or any part of the failure plane extending through either the concrete monolith or the foundation is calculated from the shearing strength of

the material acting over the length in which shearing occurs. If other restraints exist, such as strut action, they must also be considered in the evaluation. An allowable safety factor against sliding is 4 for normal operation, flood condition and dewatered condition. For normal operation with earthquake, the factor of safety is 2-2/3.

40. The base pressures are the sum of the contact and uplift pressures on the concrete-foundation interface.

41. The dam monolith was investigated for the following case loadings:

a. Normal operation.

b. Normal operation with earthquake.

c. Flood condition.

d. Dewatered condition.

Results

42. The stability analysis of the dam was accomplished using the most critical monolith when not considering clay seams in the foundation. The most critical monolith is the smaller pier without the mass of the superstructure for the gate-operating machinery. It was concluded that the monoliths of Pokegama Dam are adequate in stability if the foundation is investigated and found to be satisfactory in stability at clay seams and against underseepage and washout of the seams. A summary of the stability results are presented in Tables 2, 3, and 4. The stability analysis is presented in Tables 5-8. These results do not include strut resistance or the effect of clay seams.

43. Since no intact concrete foundation samples could be obtained, the interface material properties were estimated by using previous experience with such materials. Conservative values of angle of internal friction (ϕ) and cohesion (c) were used as 30 degrees and 0 psf, respectively. Overturning is adequate. Base pressures are adequate when not considering the clay seams, because the minimum base pressure is less than 100 psi, which is only about 8 percent of the unconfined compressive strength at the monolith base. The unconfined compressive strength of

the silica bonded sandstone and quartzite foundation is much higher (Table 9) than the concrete and does not govern the allowable concrete-foundation interface pressures. The factors of safety against sliding for the normal operation plus earthquake and flood cases are 2.78 and 2.75, respectively. Even though these safety factors are below 4, they are considered adequate because Pokegama Dam is a low-head dam and these safety factors are substantial and are for extreme loading conditions. The extent of clay seams as reflected in core P-P4 needs to be determined and checked for adequacy in stability requirements.

44. There are some properties and concepts used in the analysis and conclusions which should be discussed as follows.

45. First, consideration should be given to the variability and characteristics of the concrete and foundation as reflected in the cores. The upper concrete, which was placed in 1903, is adequate and has an unconfined compressive strength of 4900 psi. The compressive strength of the concrete and foundation are presented in Table 9. The deeper and weaker concrete is adequate and has a compressive strength as low as 1360 psi. No intact structure-foundation interface samples could be obtained for cores P-P4 or P-P8 during the coring program. This was mainly due to the closely spaced bedding planes of this interface.

46. The foundation material in core P-P4 exhibited closely spaced bedding planes with the inclusion of clay seams. The first 4.3 ft was sandstone without clay seams. The next 4 ft was clay, of which 2.1 ft was completely washed away by the drill water. After the 4 ft of clay, core P-P4 exhibited thin clay seams and sandstone foundation material. The clay seams were less predominant with depth.

47. The first 2.25 ft of core P-P8 was fractured sandstone. The rest of the core was hard sandstone and silica bonded sandstone.

48. The foundation material in core P-P4 varied significantly from that in P-P8. From the analysis of the situation, it is concluded that in all probability the clay found in core P-P4 is an isolated pocket of some extent and does not exist under the entire dam. In an area glaciated as extensively as was northern Minnesota, it is very

possible that the coring penetrated a glacial erratic (boulder) of sandstone and then penetrated the substrata of clay.

49. The extent of the clay seam could be under a pier and must be considered. The first consideration of the clay seam is that it begins approximately 4.3 ft below the concrete-foundation interface. This causes the mechanism of sliding failure to be such that

- a. The pier, apron, and 4 ft of foundation must slide monolithicly to fail at the clay seam.
- b. Any strut action in front of this monolithic slide would be added resistance. Consider this strut action zero because from Figure 5 erosion can be seen downstream of the piers. At the time the clay seam is investigated, the available strut action can be accurately determined.
- c. The governing clay seam is the 4-ft thick one 4.3 ft below the concrete-foundation interface. The thinner clay seams were random and dipped at steep angles (approximately 70 degrees). The steepness of the thinner seams eliminates the possibility of them controlling the sliding analysis.

50. Considering the fact that the lowest safety factor is 2.75 without strut resistance, the piers which are not located on clay seams are adequate in stability. The extent of the clay seam must be investigated by coring upstream, between, and downstream of the piers. After the extent and more detailed characteristics of the clay seams are known, the stability of any pier over a clay seam and the evaluation of washout of the clay seam can be determined.

PART V: CONCRETE AND FOUNDATION INTEGRITY

Concrete

51. In general, the 77-year old surface concrete of Pokegama Dam is in excellent condition. The core did not indicate any depth of deteriorated concrete. Minor freezing and thawing deterioration of the surface concrete exists mainly at the corners of the downstream ends of piers. The deteriorated surface concrete is not significant structurally, but it should be economically repaired in order to prevent the entrance of water into cracks, thus reducing the present freezing and thawing, which produces deterioration at an accelerated rate.

52. The cores revealed significant facts about the concrete and foundation. Approximately the first 15 ft of concrete is of good quality and strength. Its average unconfined compressive strength is 4900 psi (Table 9). The concrete which is greater than 15 ft below the top pier surface has lower compressive strength; its average strength was 1930 psi, and the minimum determined was 1360 psi. Some of the low strength concrete broke into small pieces during the drilling operation. It is probable that this weak concrete is under the entire dam, because it was encountered at approximately the same elevation in each of the core holes (P-P4 and P-P8). The reason for the low strength concrete can be speculated but not determined specifically from available data. The important fact is that the weaker concrete exists at the base of Pokegama Dam.

53. The maximum base pressure is about 100 psi (14.4 ksf), which is much less than the determined minimum compressive strength of the concrete (1360 psi).

54. Since the dam has not shown any sign of settlement, it is recommended that no remedial action be taken to correct the deficiencies of the weaker concrete. It is recommended that survey points be positioned on the dam with a reference point located significantly far from the dam so as to be independent of any movement from local disturbances at the dam. This way, if any changes are thought to occur, they can be denied or verified.

55. The boreholes will have piezometers installed, and the uplift pressures can be monitored. Any significant change in uplift presure would be a signal to investigate the cause.

56. With the limited surface deterioration repaired and the movement monitoring plugs installed, the concrete portion of the dam is in excellent condition for service and monitoring.

Foundation

57. The coring in the foundation showed a potential for problems. Core P-P4 revealed a sandstone foundation with closely spaced bedding planes. The sandstone was competent for compressive loads but has clay seams embedded within its layering. One clay seam between coring depths of 23.1 and 25.2 ft washed away with the drill water. The clay seams are indicated on the log of core P-P4, Figure 10. The main consideration here is the extent of the clay seam and the possibility of upstream water entering the seam and washing material out downstream. This is potentially a very critical situation and should be investigated to be sure that either

- a. There is no practicable possibility of a washout.
- b. Remedial measures should be performed to make the possibility of occurrence acceptable.

58. The investigation of the thick clay seam can be accomplished by coring downstream, between, and upstream of the piers, starting at pier 4. The 4-ft-thick clay seam could be critical, but the thinner seams dip at approximately 70 degrees and are not considered critical for sliding stability. After the extent of the thick clay seam is known, the likelihood of it being washed out can be determined. If a possibility exists, remedial measures can be accomplished. A cut-off wall of proper depth and extent upstream of the dam could eliminate the possibility of a washout.

59. The extent and nature of the clay seam should be used to evaluate the stability of monoliths which it can affect. If the clay seam causes any part of the dam to be inadequate in stability, it should be made adequate.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

60. The monoliths of Pokegama Dam are adequate in their resistance to overturning, sliding, and base pressure if the clay seam does not cause a stability or underseepage problem. It is recommended that additional drilling be performed in the area of pier 4 to determine the extent and nature of the clay seam in order to evaluate its effect on stability and underseepage. If after additional drilling, the clay seam is determined to be extensive enough that a stability or underseepage problem exists, remedial measures must then be accomplished.

61. Because of voids in the right embankment of the dam (U. S. Army Engineer District, St. Paul 1972), this embankment should be investigated.

62. The small amount of deteriorated concrete, mainly at the ends of the piers, should be repaired in order to prevent the entrance of water into cracks, which produces deterioration at an accelerated rate.

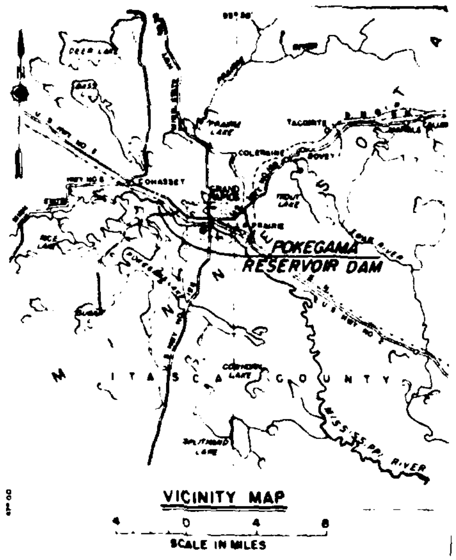
63. Monitoring points should be put on the dam in order that any movement can be detected and appropriate action taken.

REFERENCES

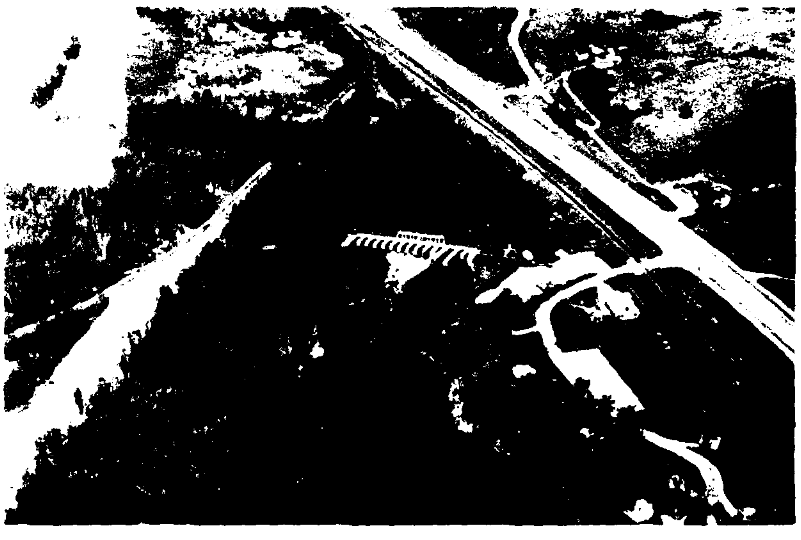
U. S. Army Engineer District, St. Paul. 1965. "Mississippi River, Headwaters Reservoirs, Minnesota, Master Plan for Reservoir Development, Pokegama Lake Reservoir," Corps of Engineers, St. Paul, Minn.

U. S. Army Engineer District, St. Paul. 1972. "Reservoirs at Headwaters, Mississippi River, Minnesota, Pokegama Dam, Periodic Inspection Report No. 1," Corps of Engineers, St. Paul, Minn.

U. S. Army Engineer District, St. Paul. 1977. "Mississippi River, Headwaters Reservoirs, Master Plan for Public Use Development and Resource Management," Corps of Engineers, St. Paul, Minn.



a. Location map of Pokegama Dam



b. Aerial view of Pokegama Dam, 1972

Figure 1. Pokegama Dam

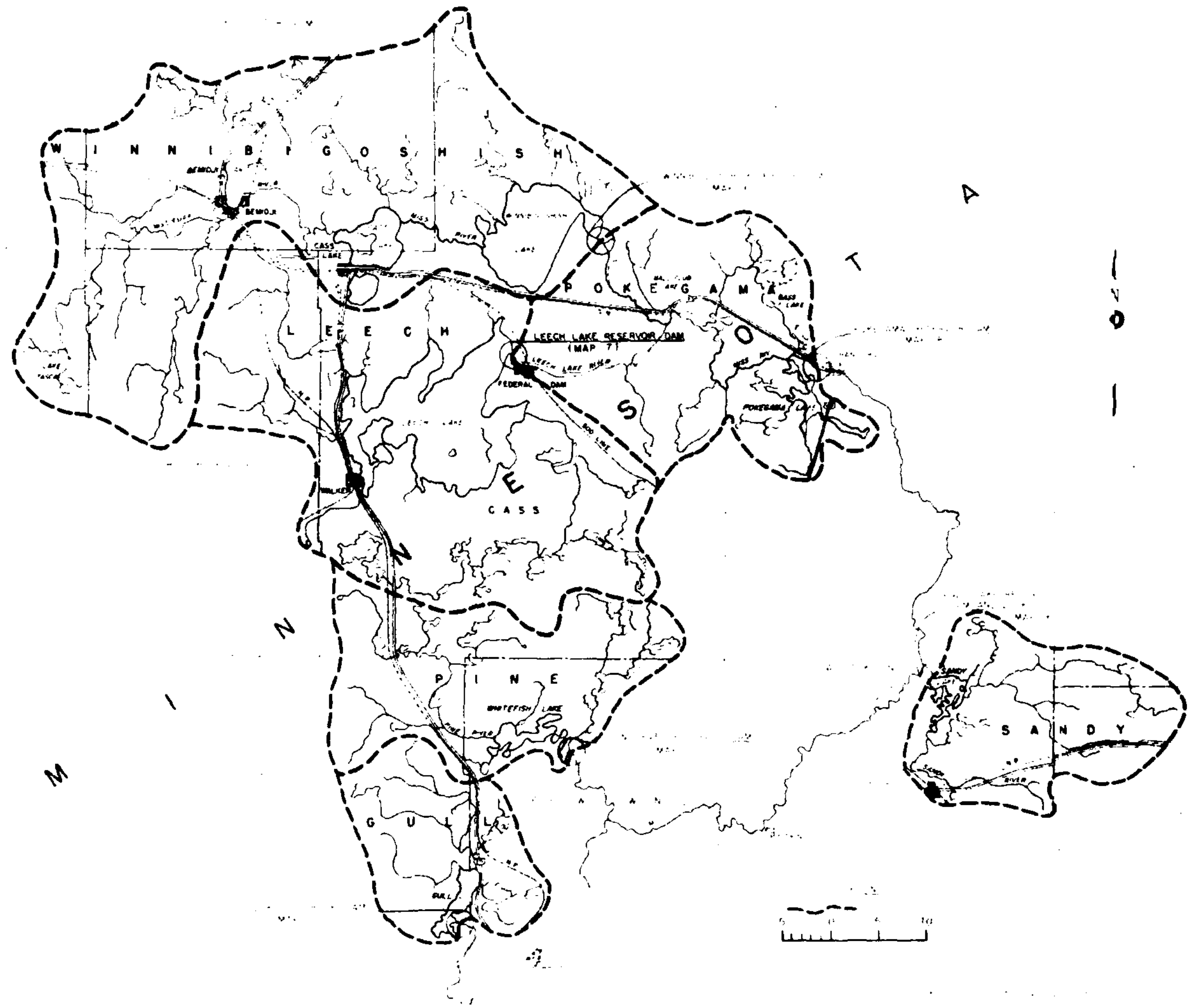


Figure 2. Project map of Mississippi River headwaters reservoirs

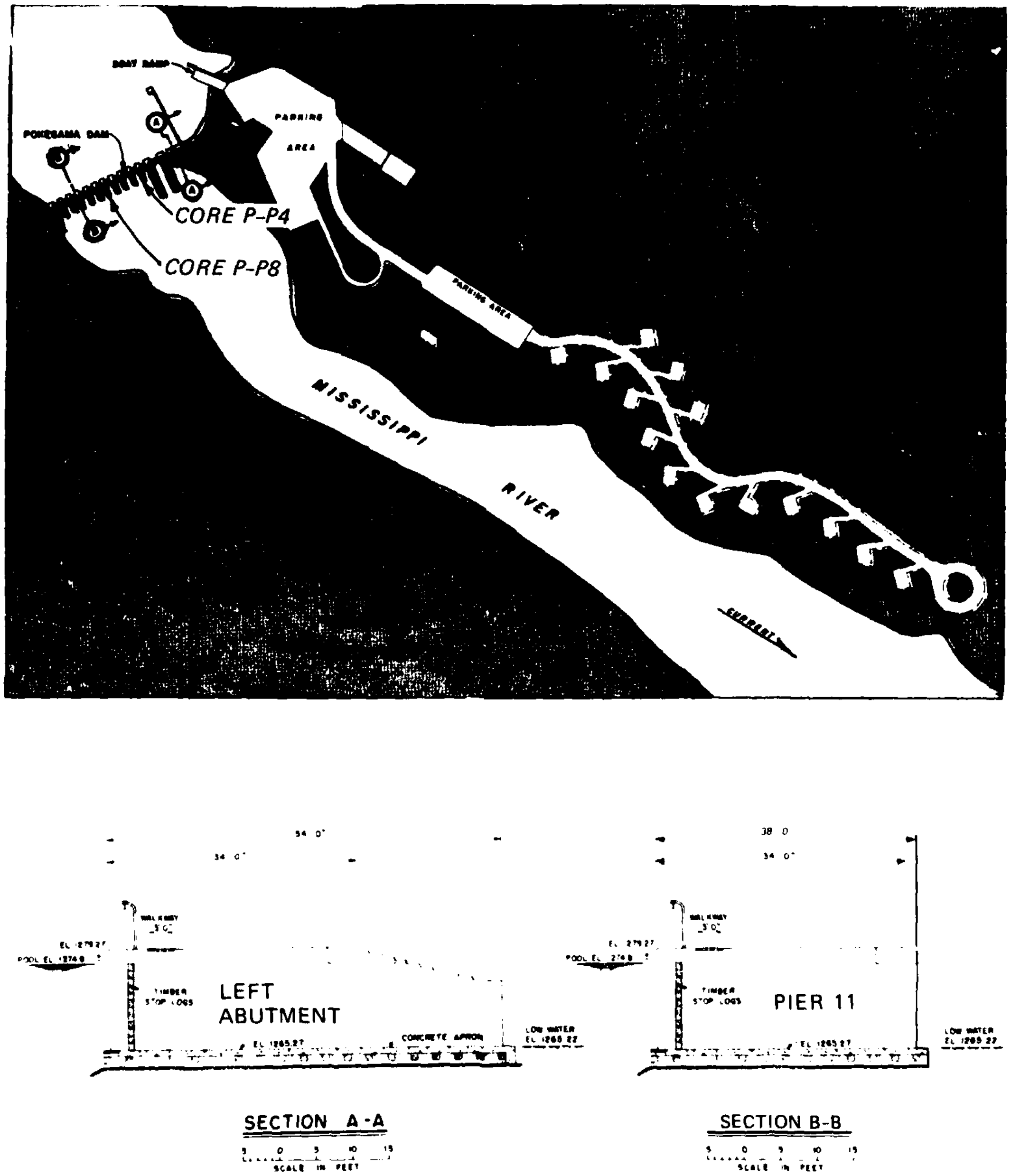
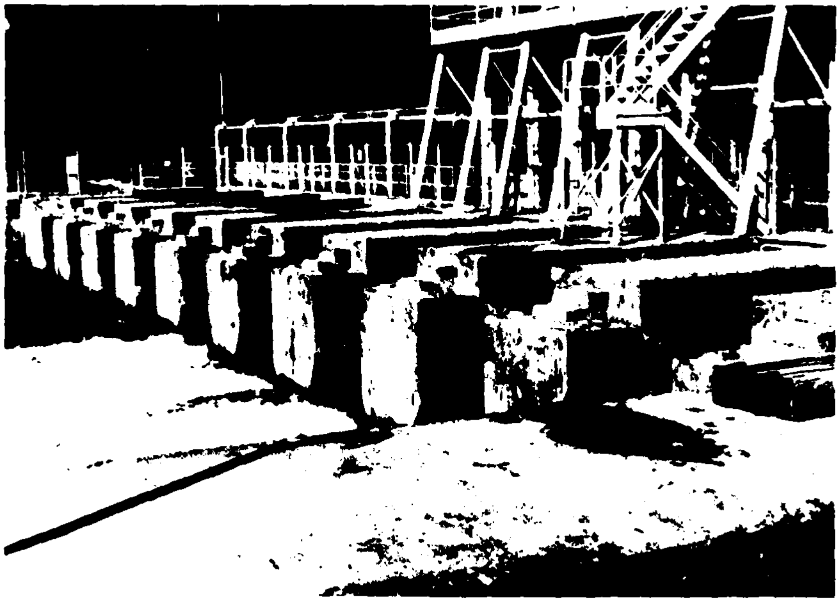
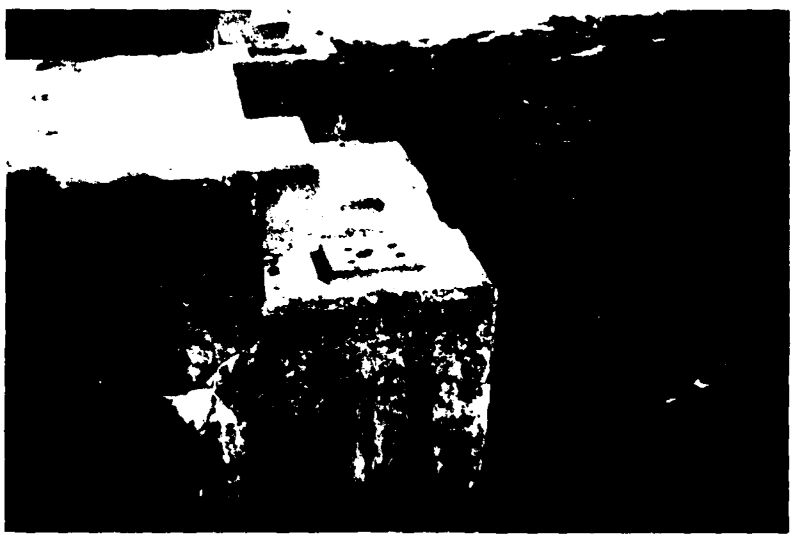


Figure 3. General construction layout, Pokegama Dam



a. Downstream end of piers



b. Closer view, downstream end of piers

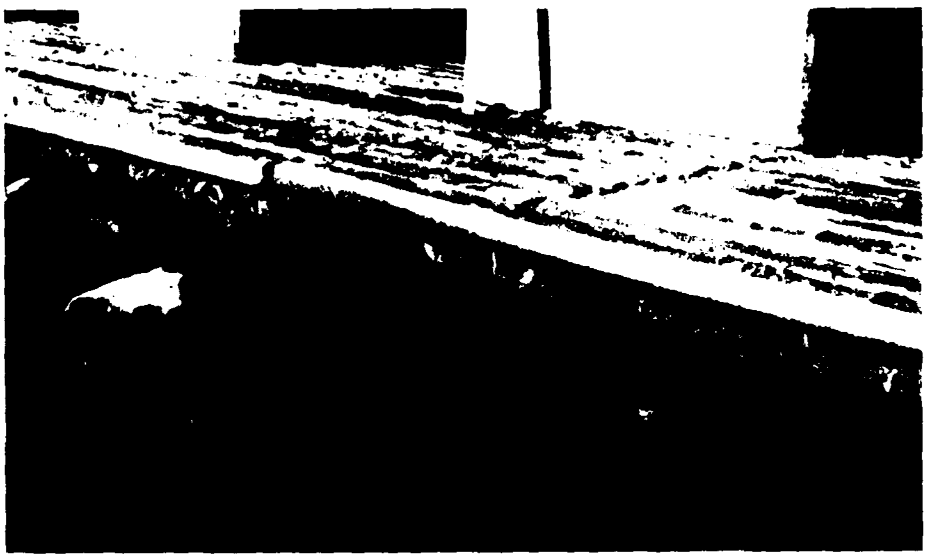
Figure 4. Views of Pokegama Dam in its present condition

a. Downstream view, Pokegama Dam

b. Downstream view, sluice No. 4, near middle of photo

Figure 5. Pokegama Dam, 1949 (Sheet 1 of 3)

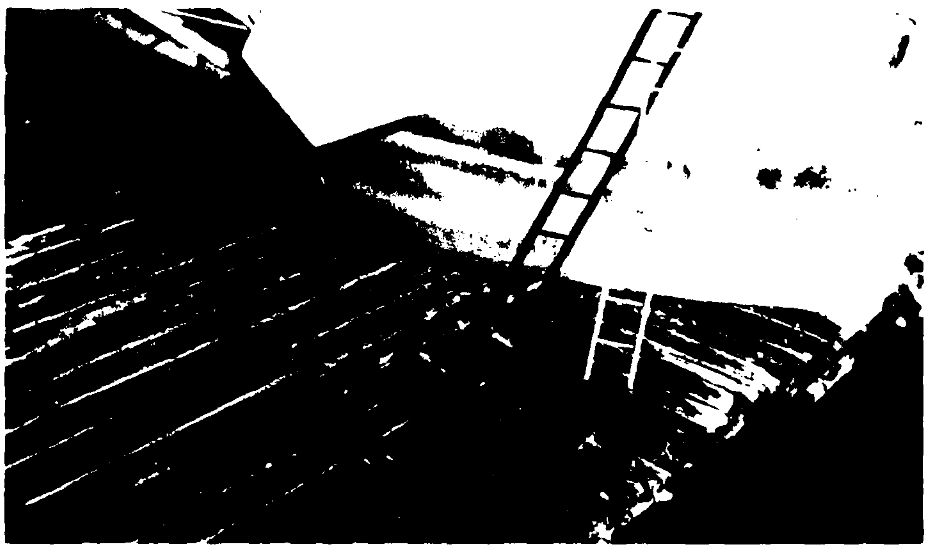
c. Downstream view, sluice No. 4, near middle of photo



d. Downstream view, sluice No. 7, near middle of photo

Figure 5. (Sheet 2 of 3)

e. Downstream view, sluice No. 4, to right of photo



f. Downsteam view, left end

Figure 5. (Sheet 3 of 3)

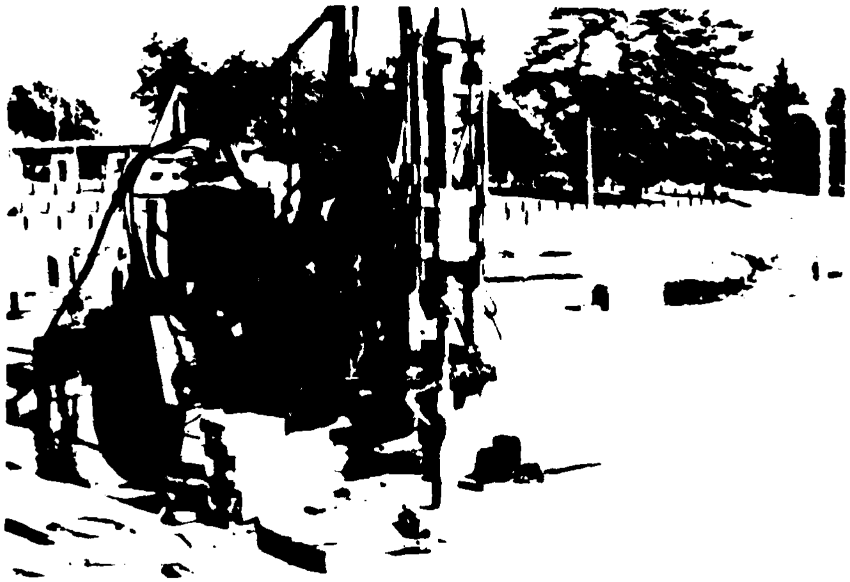


Figure 6. Typical drilling setup

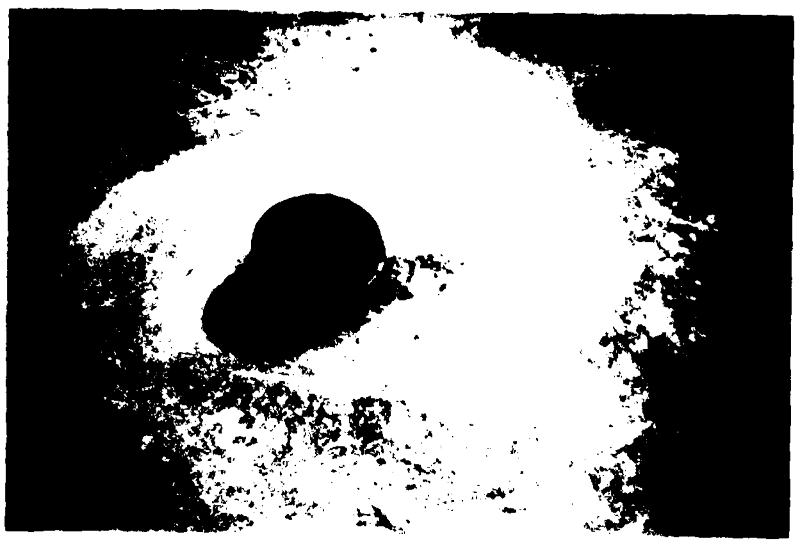
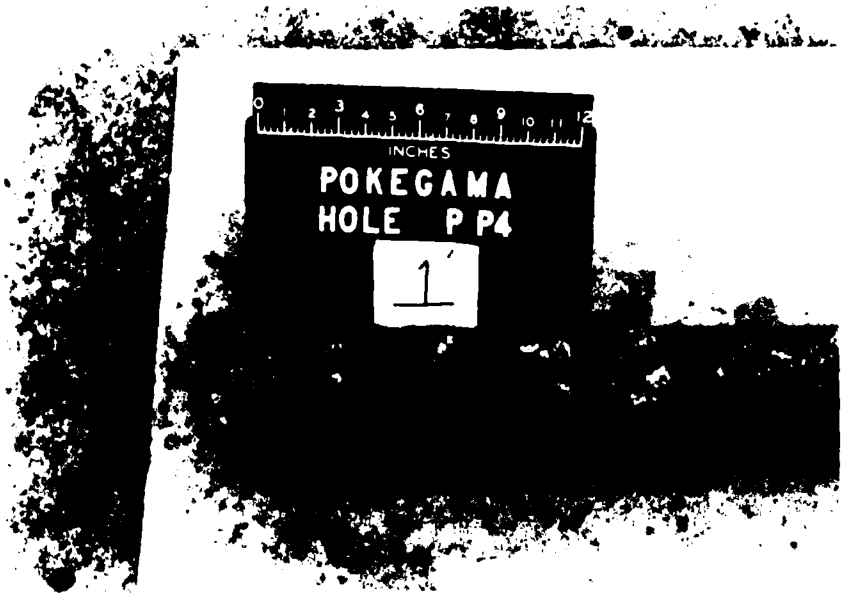


Figure 7. Capped core hole



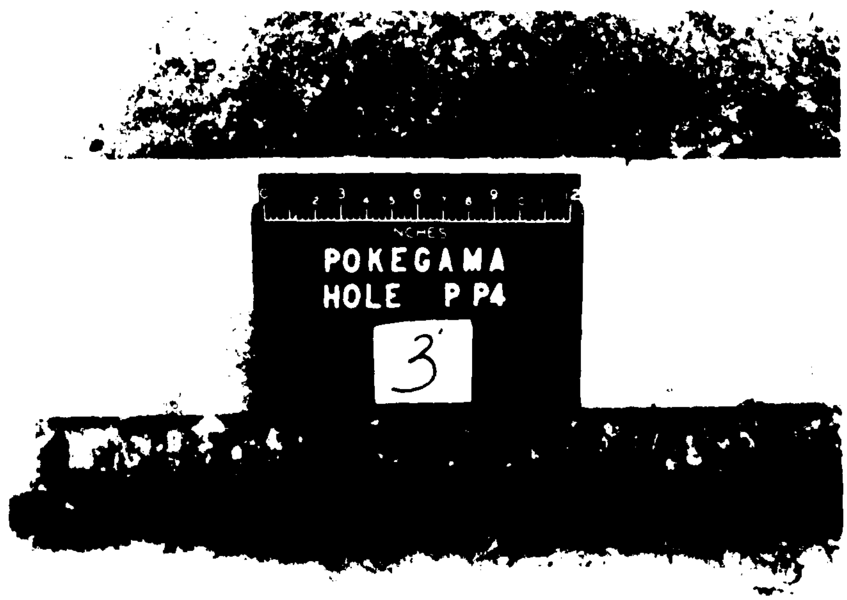
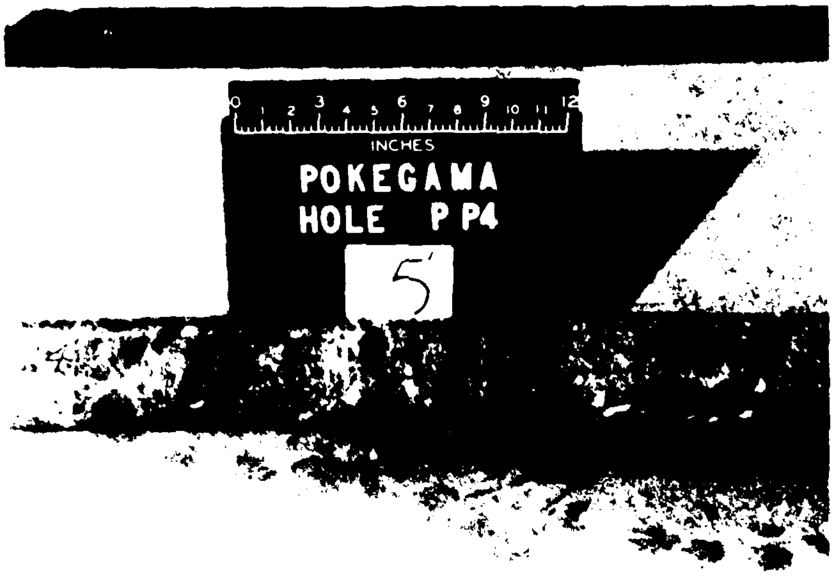


Figure 5. Core from pier 4 (P-P4) (sheet [illegible])



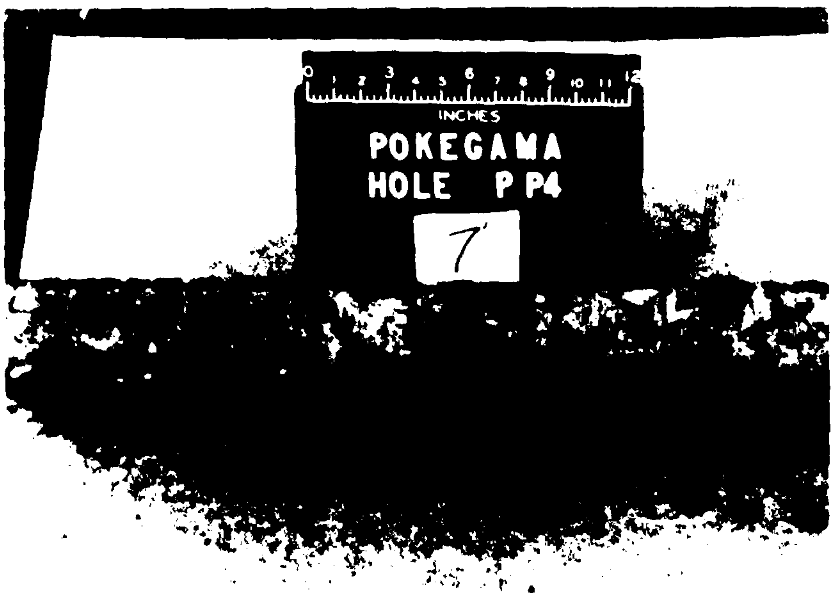
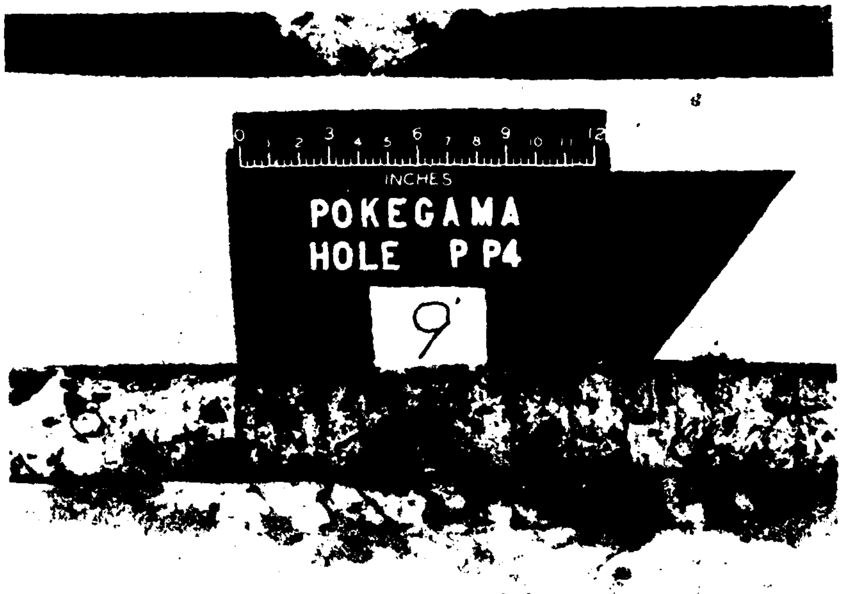


Figure 8. (Sheet 2 of 9)



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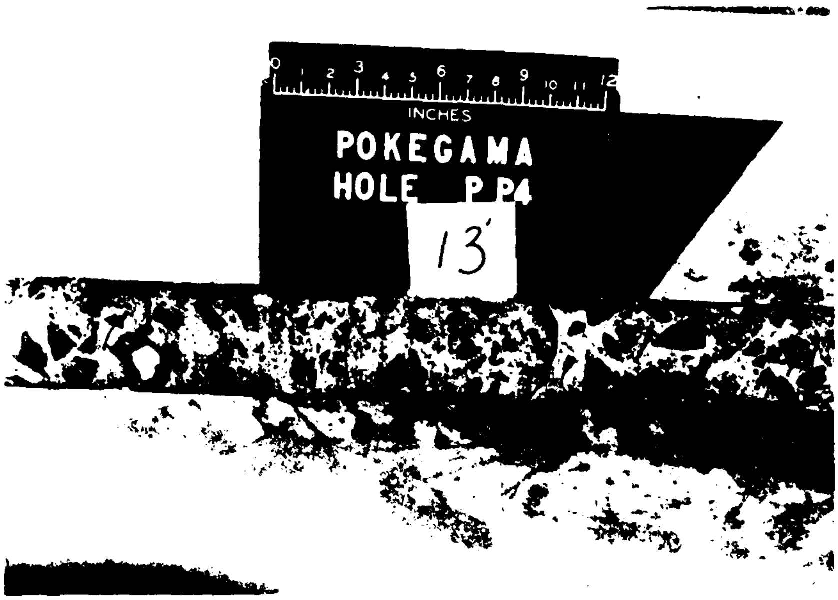

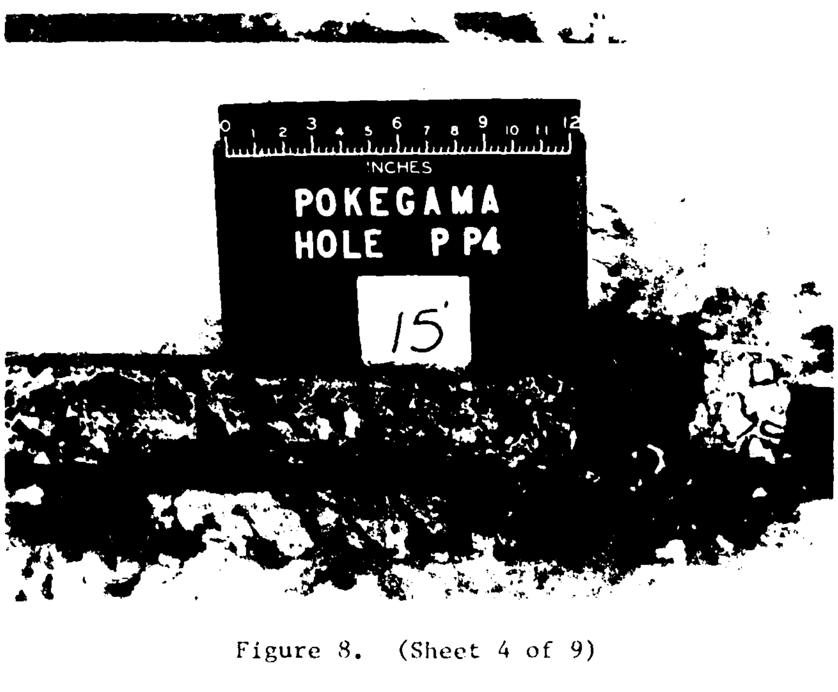

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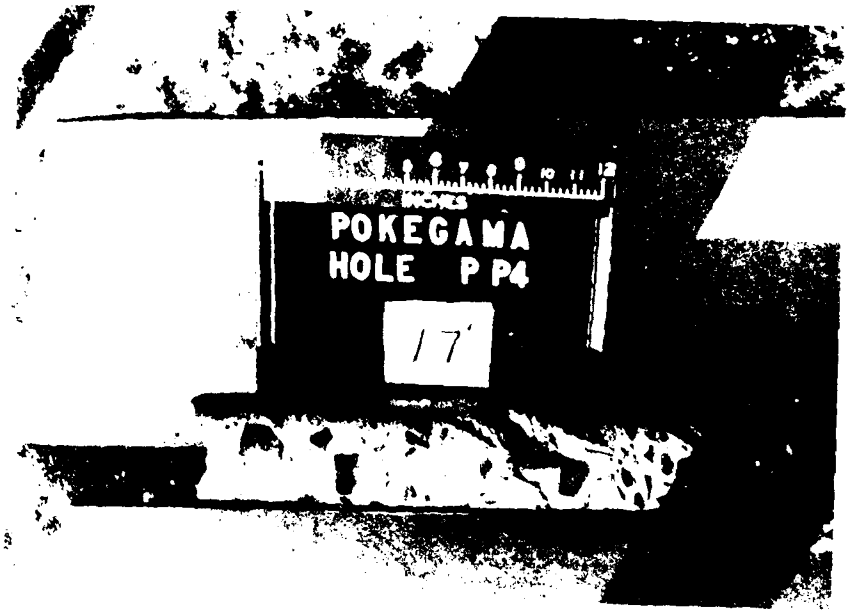

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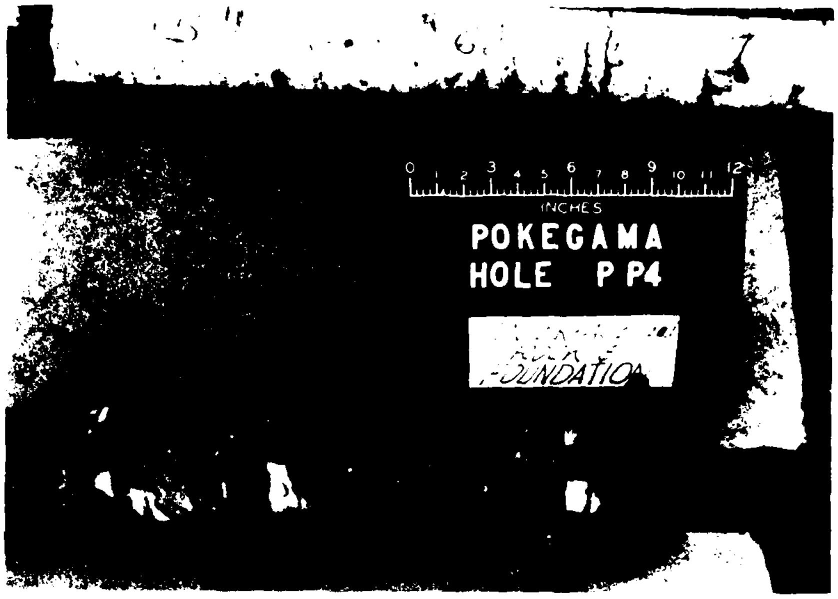

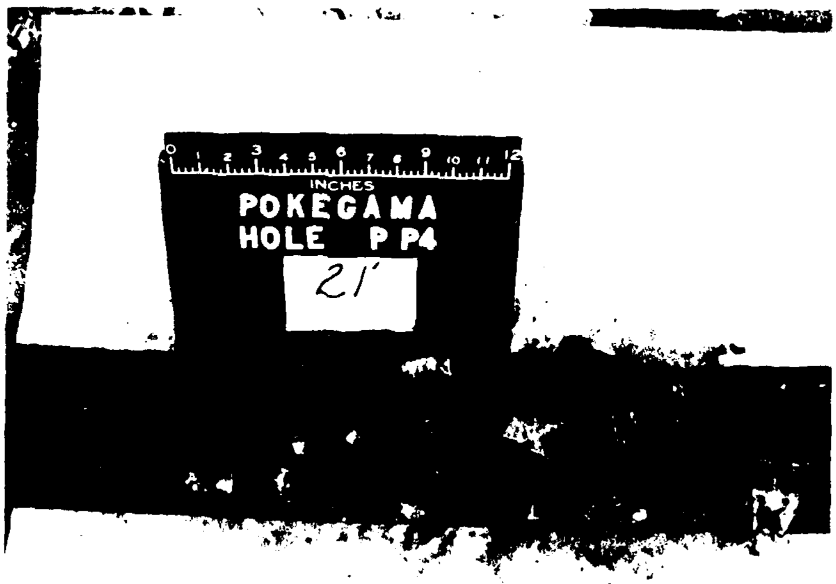

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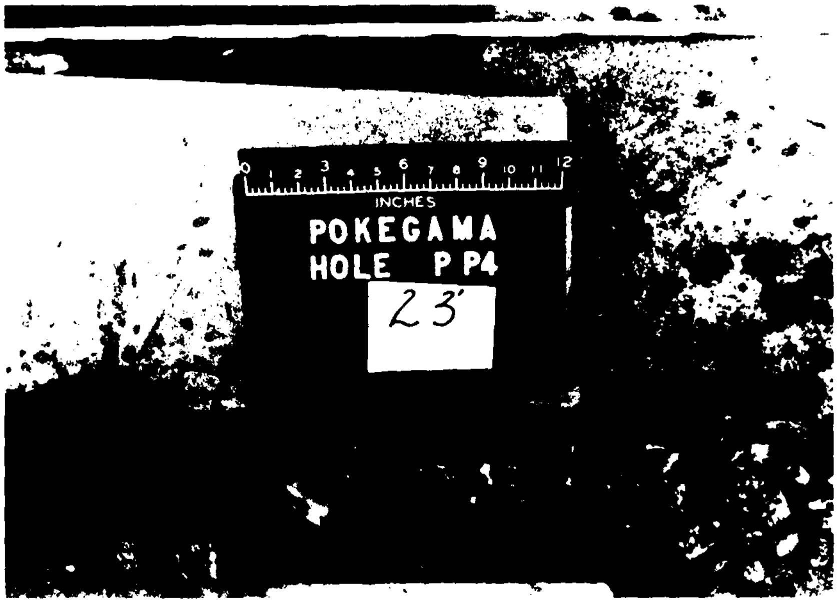

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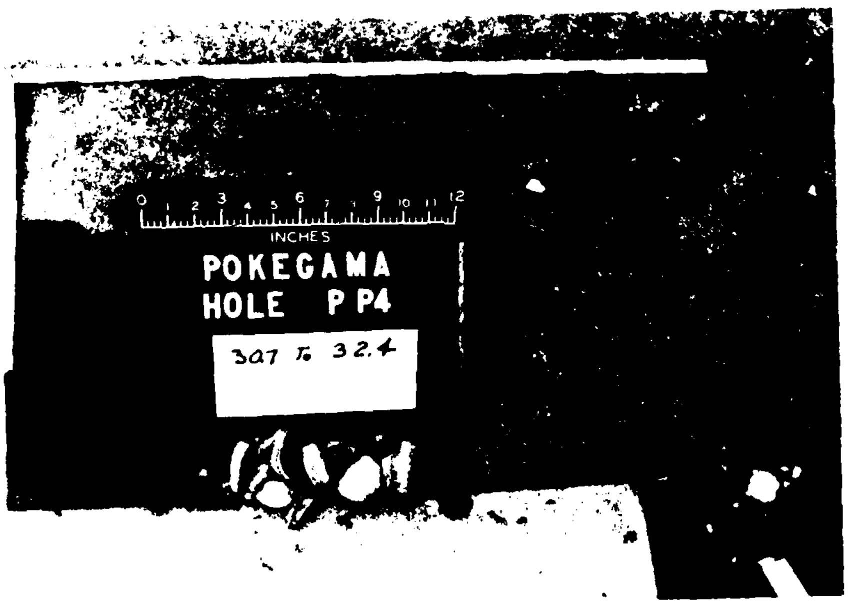

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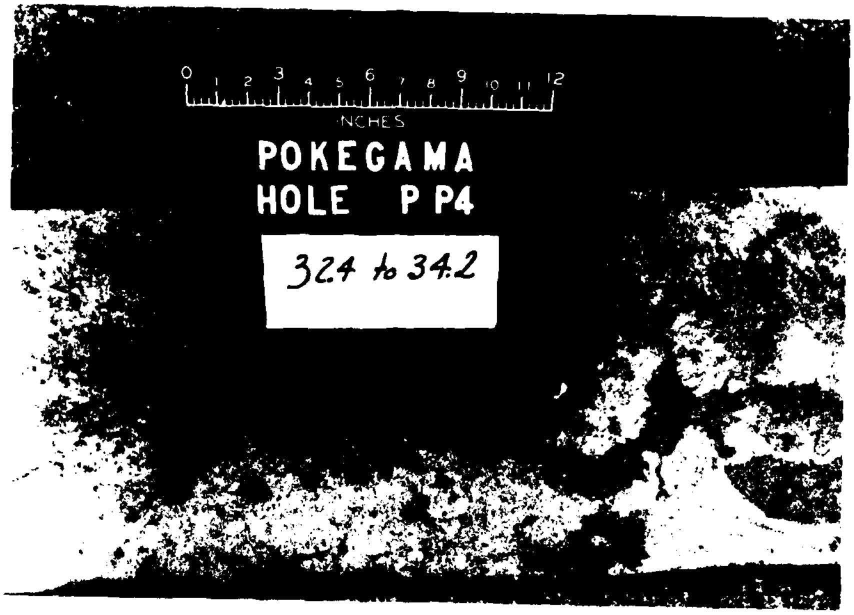

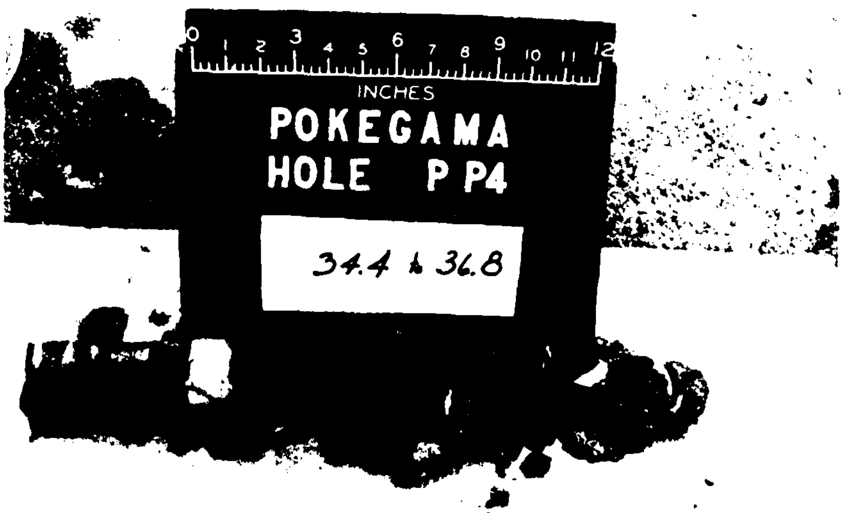

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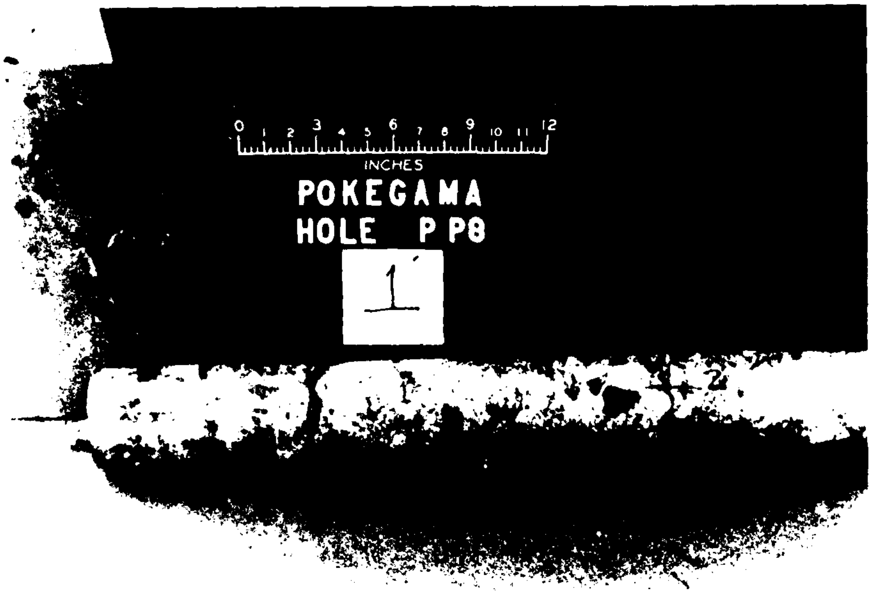


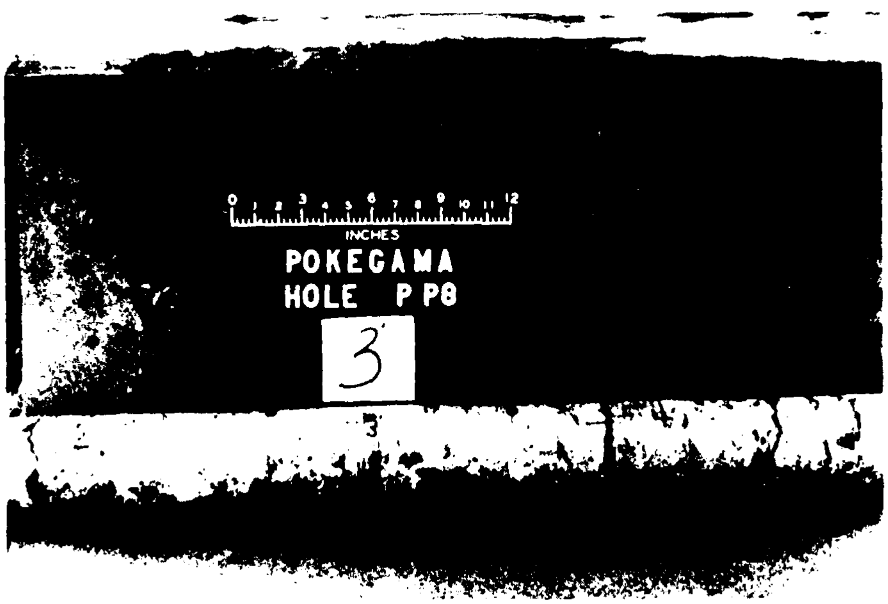
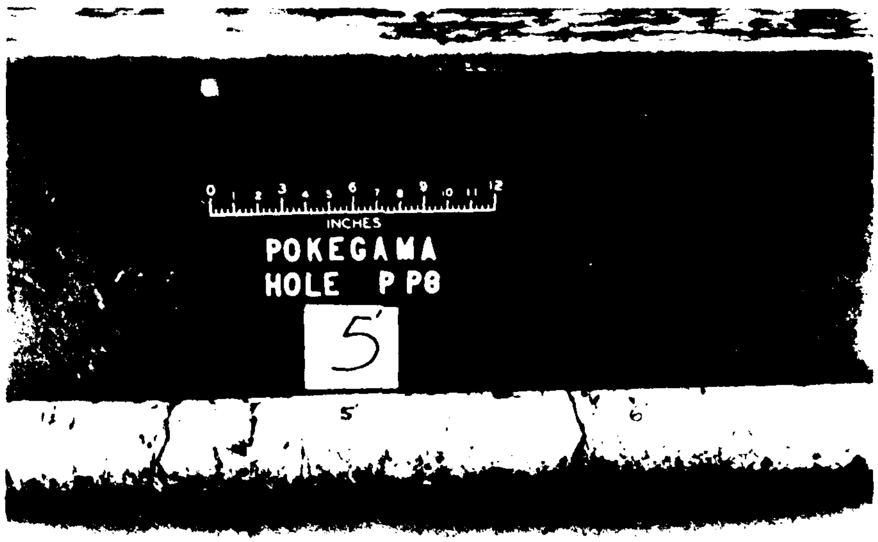


Figure 9. Core from pier 8 (P-P8) (Sheet 1 of 7)



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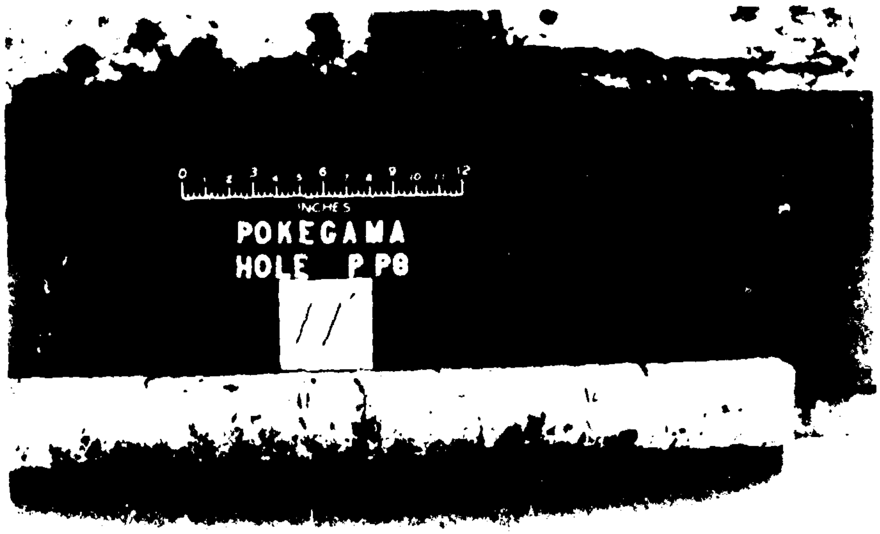

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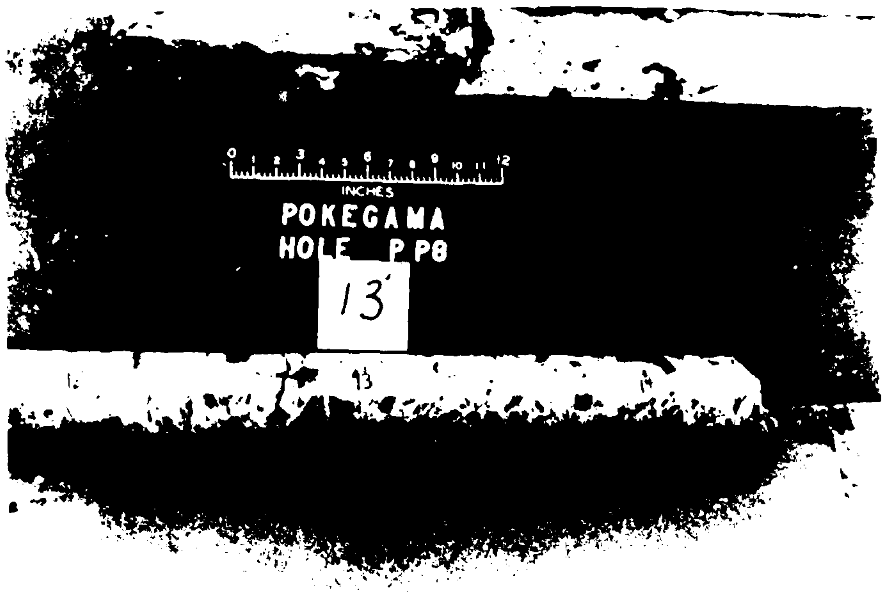


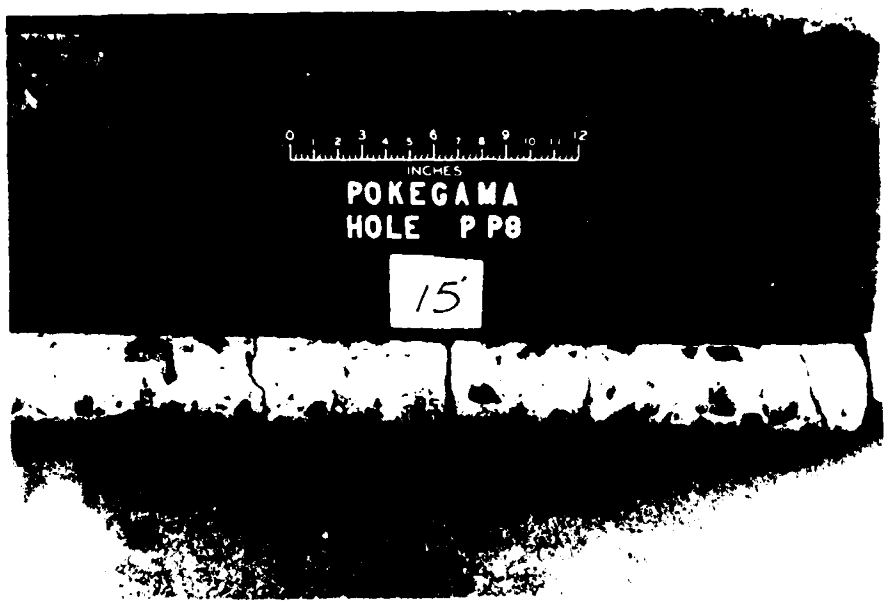
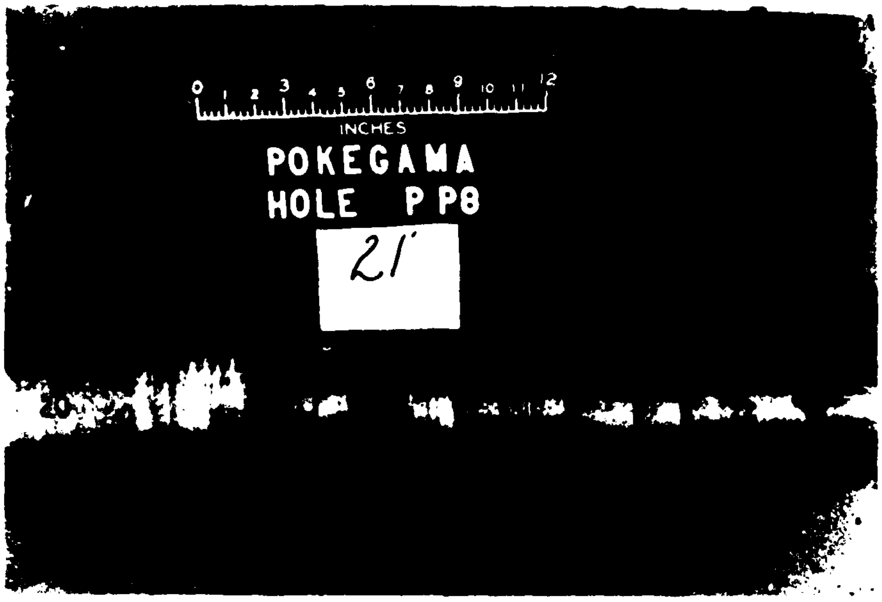
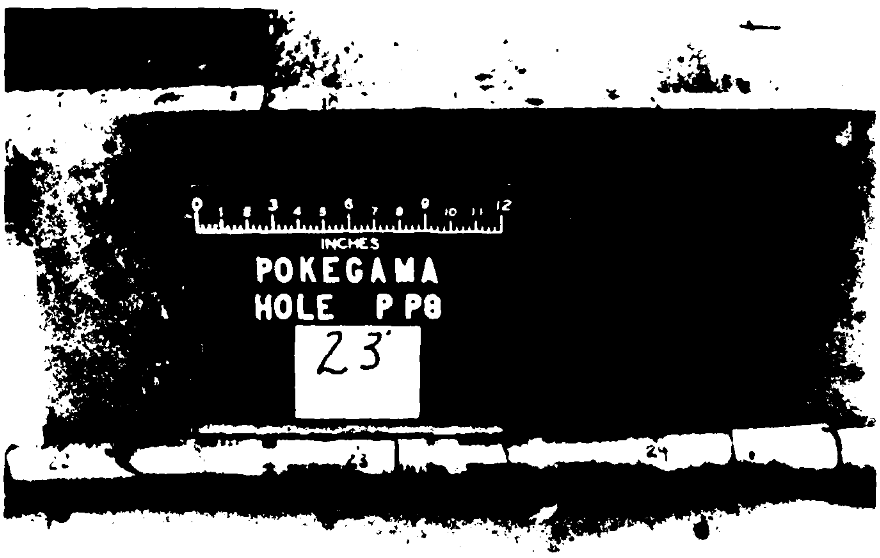


Figure 9. (Sheet 4 of 7)

Figure 9. (Sheet 5 of 7)





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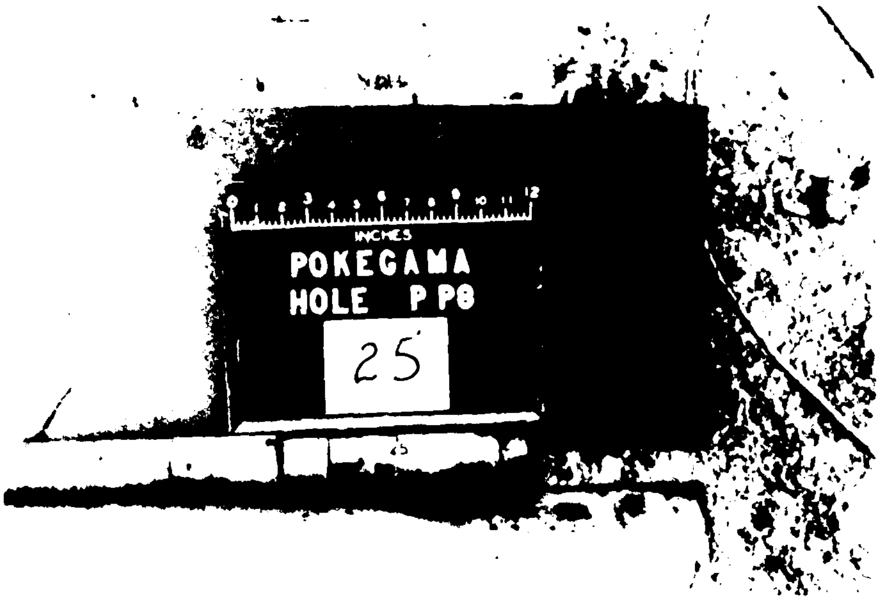

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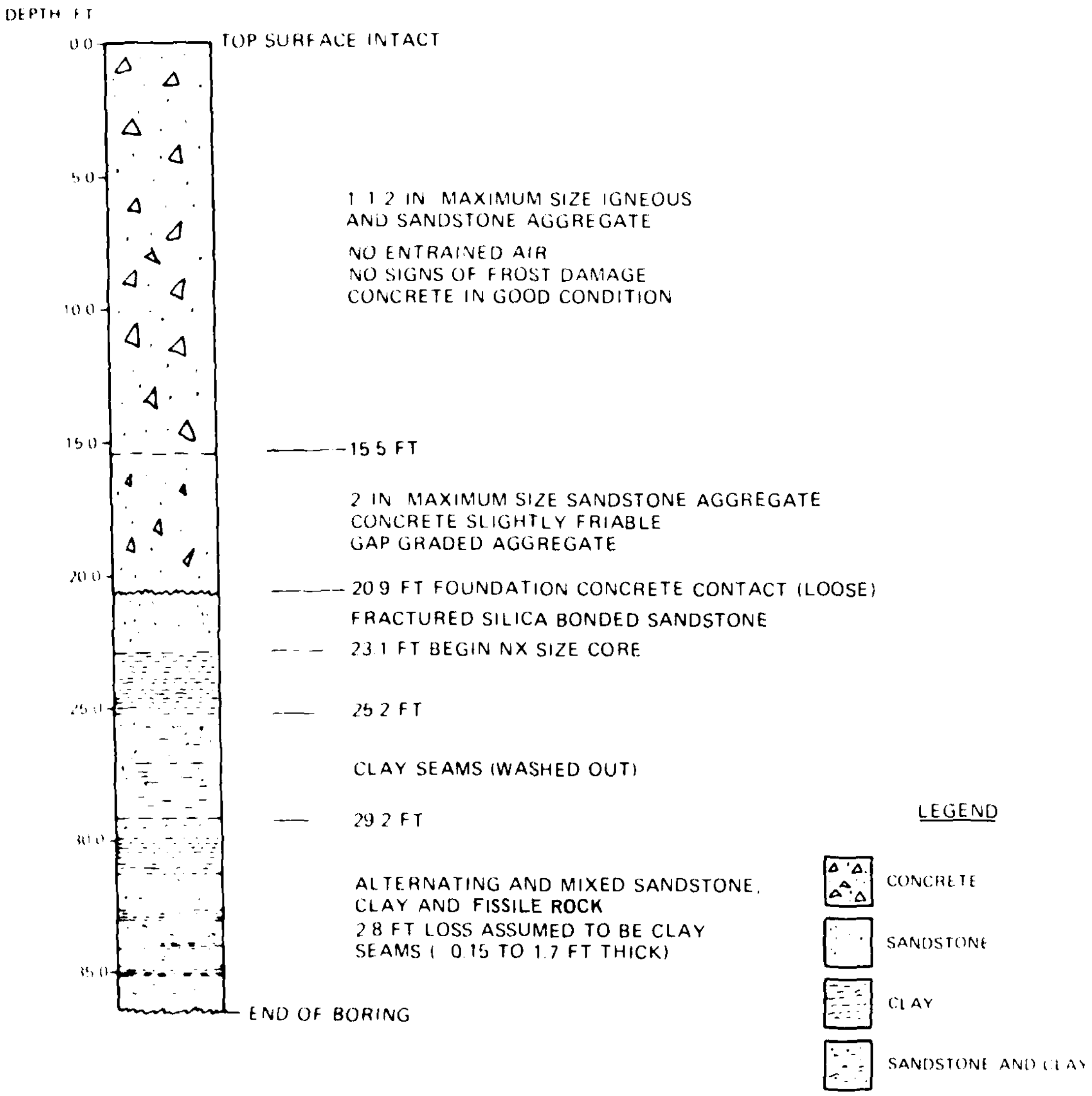

Figure 10. Core log for P-P4

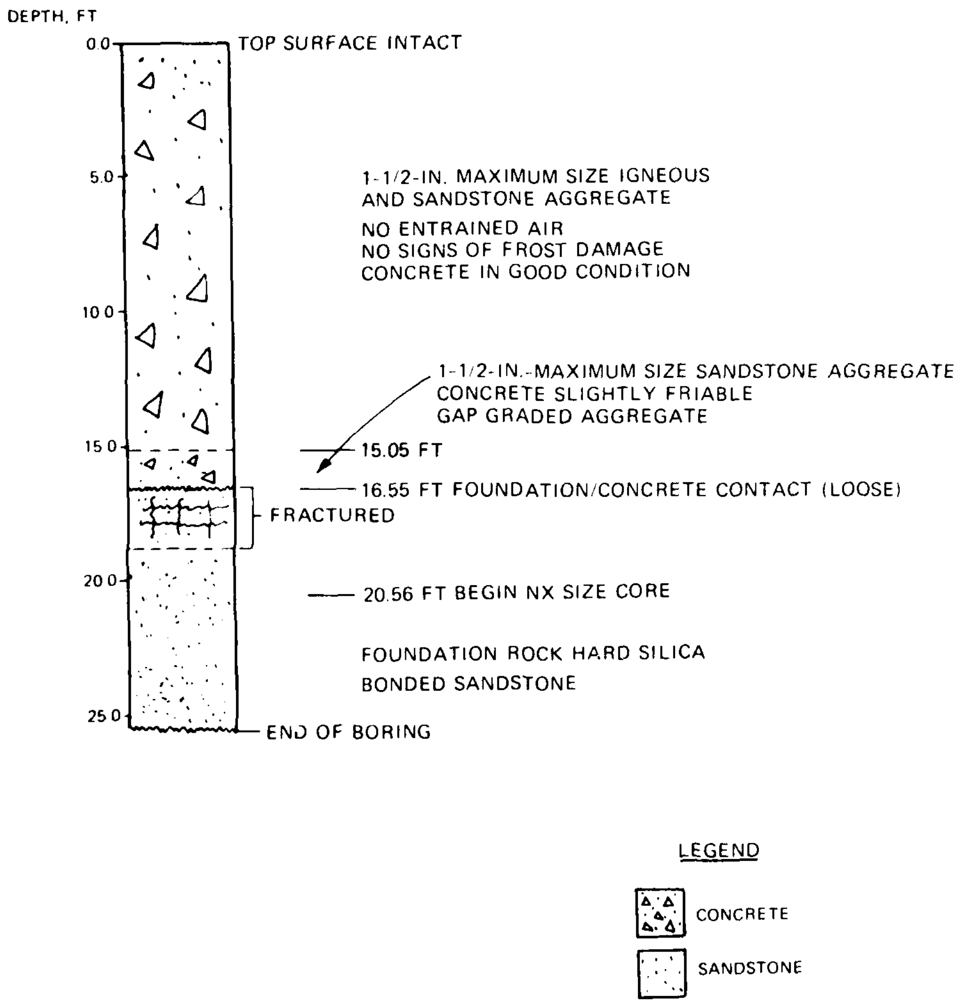

Figure 11. Core log for P-P8

Table 1

Pertinent Data, Pokegama Dam

Dam	
Type	Earth fill with timber diaphragm core, filled with puddled clay
Crest height	14.00 ft
Length	160 ft
Height (maximum)	14.0 ft
Freeboard above maximum stage	0.5 ft
Control structure	
Type	Concrete
Sill stage (8 ungated stop log bays)	0.00 ft
Sill stage (6 bays with slide gates)	1.50 ft
Net length of spillway crest (14 bays)	116 ft
Height of piers	14.0 ft
Sluiceways	
Number of bays	13
Width	8 ft
Log sluice width	12.0 ft
Number of sluice gates	6
Size of sluice gates	9 ft wide by 12.5 ft high
Number of stop log sections	8
Height of stop logs at normal pool	9.0±
Discharge channel capacity	6000 (ft^3/sec)
Spillway apron	
Type	Concrete
Length	52 ft
Width	225 ft
Floor height	0.00 ft

Table 2

Pier, Pokegama Dam, Overturning Results

Load Case	Base Width b (ft)	Sum of Vertical Forces F_V (kips)	Sum of Moments M (ft-kips)	Resultant Moment Arm $e = \frac{M}{F_V}$ (ft)	Percent Effective Base $\frac{3e}{b} \times 100$ (%)
Normal operation	34	438.0	7032	16.05	100.0
Normal operation with earthquake	34	438.0	6826	15.58	100.0
Flood condition	34	397.3	6171	15.53	100.0
Dewatered condition	34	462.0	7302	15.81	100.0

Table 3

Pier, Pokegama Dam, Factor of Safety Against Sliding

Load Case	Sum of Vertical Forces F_v (kips)	Sum of Horizontal Forces F_H (kips)	Friction Angle ϕ (Degrees)	Cohesive Strength C (ksf)	Base Area A (sq ft)	Strut Resistance R_S (kips)	Shear Resistance $R_1 = F_v \tan \phi$ (kips)	Cohesive Resistance $R_2 = CA$ (kips)	Total Sliding Resistance $R = R_S + R_1 + R_2$ (kips)	Factor of Safety Against Sliding $F.S. = \frac{R}{F_H}$
Normal operation	438.0	59.8	30	0	272	0	252.9	0	252.9	4.23
Normal operation with earthquake	438.0	91.0	30	0	272	0	252.9	0	252.9	2.78
Flood condition	397.3	83.3	30	0	272	0	229.4	0	229.4	2.75
Dewatered condition	462.0	62.2	30	0	272	0	266.7	0	266.7	4.29

Table 4

Pier, Pokegama Dam, Base Pressures

Load Case	Sum of Vertical Forces F_V (kips)	Resultant Moment Arm e (ft)	Effective Base Width b (ft)	Axial Stress $f_a = \frac{F_V}{b}$ (ksf)	Bending Stress $f_b = \pm \frac{F_V\left(\frac{b}{2} - e\right)\frac{b}{2}}{(8)b^3/12}$ (ksf)	Inter-granular Stress At Heel $f_h = f_a - f_b$ (ksf)	Inter-granular Stress At Toe $f_t = f_a + f_b$ (ksf)	Hydrostatic Pressure At Heel $f_{uh} = \gamma_w f_{uh}$ (ksf)	Hydrostatic Pressure At Toe $f_{ut} = \gamma_w h_{ut}$ (ksf)	Total Pressure At Heel $f_{Th} = f_h + f_{uh}$ (ksf)	Total Pressure At Toe $f_{Tt} = f_t + f_{ut}$ (ksf)
Normal operation	438.0	16.05	34	12.88	0.27	12.61	13.15	0.70	0.18	13.31	13.33
Normal operation with earthquake	438.0	15.58	34	12.88	0.40	12.48	13.28	0.70	0.18	13.18	13.46
Flood condition	397.3	15.53	34	11.69	0.38	11.31	12.07	0.85	0.33	12.16	12.40
Dewatered condition	462.0	15.81	34	13.59	0.36	13.23	13.95	0.70	0.00	13.93	13.95

Table 5

Pier, Pokegama Dam - Normal Operation

Item	Factors	F_V (kips)	F_H (kips)	Arm (ft)	Moment (ft-k)
W_{conc}	(0.15)(1279.27-1265.27)(30)(8)	504.0		19.00	9576
	(0.15)(1276.27-1265.27)(4)(8)	52.8		2.00	106
P_{hdwtr}	$(0.0625)(1/2)(1276.42-1265.27)^2(16)$		-62.2	3.72	-231
P_{tlwtr}	$(0.0625)(1/2)(1268.10-1265.27)^2(16)(0.6)$		2.4	0.94	2
Uplift	(.0625)(1268.10-1265.27)(34)(8)	-48.1		17.00	-818
	(.0625)(1/2)(1276.42-1268.10)(34)(8)	-70.7		22.67	-1603
		-118.8			-2421
Total		438.0	-59.8		7032

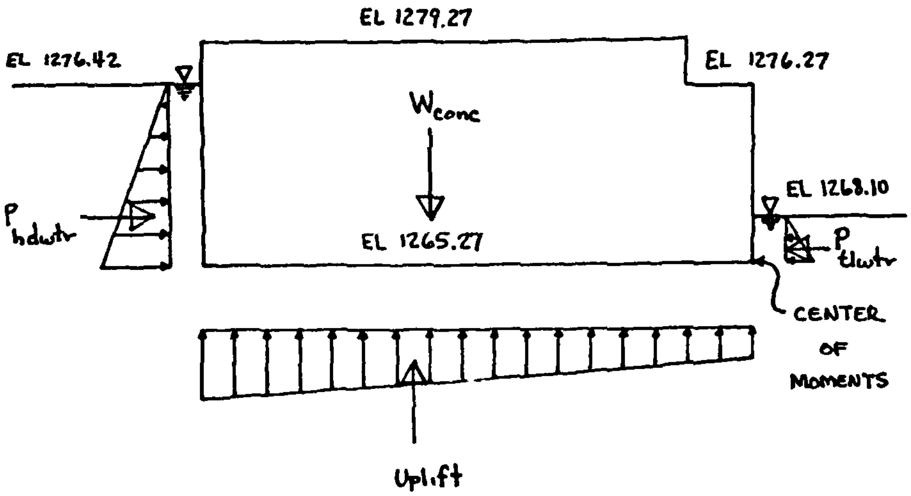

Table 6

Pier, Pokegama Dam - Normal Operation with Earthquake

Item	Factors	F_V (kips)	F_H (kips)	Arm (ft)	Moment (ft-k)
Sum of Normal Operation Loadings		438.0	-59.8		7032
Earthquake (Concrete)	$Pe_1 = (.05)(556.8)$		-27.8	6.86	-191
(Headwater)	$Pe_2 = (2/3)(51)(.05)(11.15)^2(1/1000)(16)$		-3.4	4.46	-15
			-31.2		-206
Total		438.0	-91.0		6826

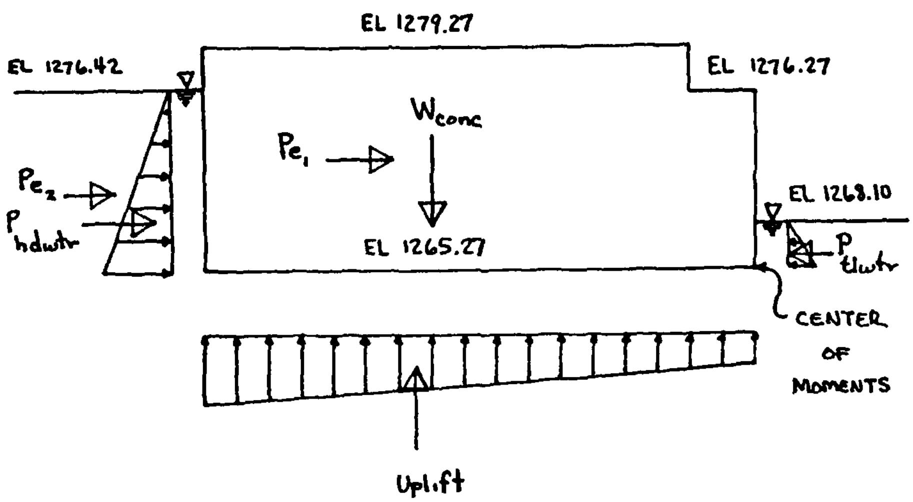

Table 7

Pier, Pokegama Dam - Flood Condition

Item	Factors	F_V (kips)	F_H (kips)	Arm (ft)	Moment (ft-k)
W_{conc}	(0.15)(1279.27-1265.27)(30)(8)	504.0		19.00	9576
	(0.15)(1276.27-1265.27)(4)(8)	52.8		2.00	106
		556.8			9682
P_{hdwtr}	$(.0625)(1/2)(1278.8-1265.27)^2(16)$		-91.5	4.51	-413
P_{tlwtr}	$(.0625)(1/2)(1270.5-1265.27)^2)(16)(.6)$		8.2	1.74	14
Uplift	(.0625)(1270.5-1265.27)(34)(8)	-88.9		17.00	-1511
	(.0625)(1278.8-1270.5)(34)(8)(1/2)	-70.6		22.67	-1601
		-159.5			-3112
Total		397.3	-83.3		6171

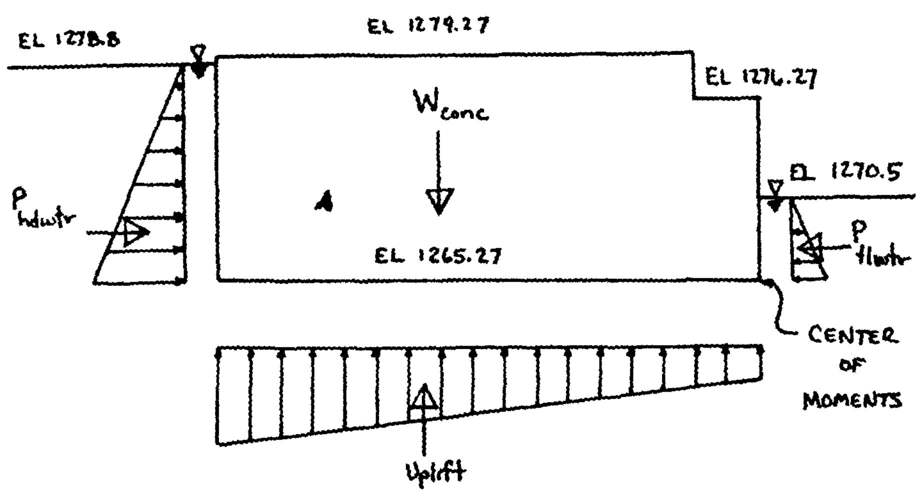

Table 8

Pier, Pokegama Dam - Dewatered Condition

Item	Factors	F_V (kips)	F_H (kips)	Arm (ft)	Moment (ft-k)
W_{conc}	(0.15)(1279.27-1265.27)(30)(8)	504.0		19.00	9576
	(0.15)(1276.27-1265.27)(4)(8)	52.8		2.00	106
		556.8			9682
P_{hdwtr}	$(.0625)(1/2)(1276.42-1265.27)^2(16)$		-62.2	3.72	-231
Uplift	(.0625)(1/2)(1276.42-1265.27)(34)(8)	-94.8		22.67	-2149
Total		462.0	-62.2		7302

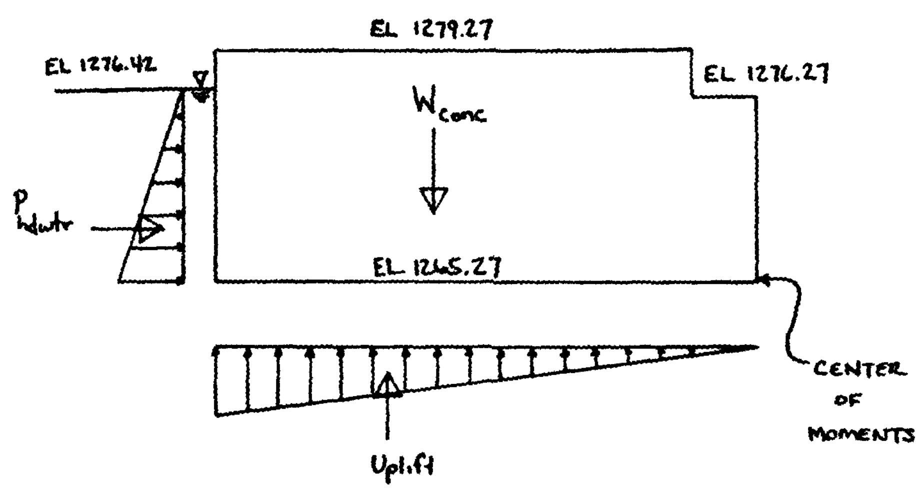

Table 9

Unconfined Compressive Strength

Specimen No.	Material	Specimen Diameter (in.)	Location Below Top of Pier (ft)	Compressive Strength (psi)
P-P41	concrete	3.965	0.85	3,890
P-P42	concrete	3.961	17.9	1,360
P-P81	concrete	3.955	6.5	5,930
P-P82	concrete	3.965	16.0	2,500
P-P83	foundation	3.970	18.85	>35,540

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Pace, Carl Eugene
Structural stability evaluation Pokegama Dam / by Carl E. Pace, Roy L. Campbell, G. Sam Wong. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.
23, [35] p. : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; SL-80-15)
Prepared for U. S. Army Engineer District, St. Paul, St. Paul, Minnesota, under Intra-Army Order No. NCS-IA-78-75.
References: p. 23.

1. Concrete dams. 2. Core drilling. 3. Dam foundations. 4. Dam stability. 5. Deterioration. 6. Pokegama Dam. 7. Structural stability. 8. Underseepage. I. Campbell, Roy L., joint author. II. Wong, Ging Sam, joint author. III. United States. Army. Corps of Engineers. St. Paul District. IV. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; SL-80-15.
TA7.W34m no.SL-80-15