MISCELLANEOUS PAPER C-77-5

AN INVESTIGATION OF CONCRETE CONDITION, WILLIAM BACON OLIVER LOCK AND SPILLWAY

by

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June 1977

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for U. S. Army Engineer District, Mobile
Mobile, Alabama 36628

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IN REPLY REFER TO: WESCV

3 November 1977

Errata Sheet

No. 1

AN INVESTIGATION OF CONCRETE CONDITION,
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1. Replace Table 1 in Appendix F, page F7, with corrected and expanded Table 1.

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: Miscellaneous Paper C-77-5	2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER	
4. TITLE (and Subtitle): AN INVESTIGATION OF CONCRETE CONDITION, WILLIAM BACON OLIVER LOCK AND SPILLWAY	5. TYPE OF REPORT & PERIOD COVERED: Final Report
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): James E. McDonald; Roy L. Campbell	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS: U. S. Army Engineer Waterways Experiment Station, Concrete Laboratory, P. O. Box 631, Vicksburg MS 39180	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS: U. S. Army Engineer District, Mobile, P. O. Box 2288, Mobile, AL 36628	12. REPORT DATE: June 1977
	13. NUMBER OF PAGES: 258
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)	15. SECURITY CLASS. (of this report): Unclassified
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Condition survey
Navigation locks
Overflow spillway
Concrete deterioration
Alkali-aggregate reaction
Concrete cracking
Repair and rehabilitation

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

This report presents the results of an investigation to assess the condition of the concrete in William Bacon Oliver Lock and Spillway on the Warrior-Tombigbee Waterway near Tuscaloosa, AL. The investigation included (1)crack survey of lock walls, (2) soniscope investigation of lock and spillway, (3) examination and tests of concrete and foundation cores to determine material properties, (4) stress analysis of the upper land wall gate monolith, and (Continued)

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(5) comparison of results with previous work on the structure.

Results indicate the concrete, despite extensive cracking in some monoliths containing the higher-alkali cement, is of generally good quality. Although the concrete still has the potential for internal growth and expansion due to alkali-silica reaction, any increases in cracking in recent years is more likely attributed to physical deterioration, such as freezing and thawing, than to the direct effects of continued alkali-silica reaction.

In situ pulse velicity data obtained during the period 1948-1976 indicate that, of the monoliths tested, only the concrete in Monolith Nos. 16 and 20 would be classified as questionable. However, the same data indicate that the concrete in these monoliths is not experiencing progressive deterioration; in fact, the trend is for increased pulse velocities since tests were initiated. Similarly, a comparison between current surface cracking and monolith displacements and that present in 1948 indicates that present conditions are not significantly different from those at the time of the initial investigation.

Results of the material property tests indicate the current concrete quality to be generally good and substantially unchanged from the initial investigation in 1948. This tends to alleviate the concern regarding the effect of reduced concrete strengths on the magnitude and location of stress concentrations within gate monoliths.

Extensive repairs and/or rehabilitation of the structure do not appear necessary at present. For specific areas identified through continuing periodic inspections as requiring maintenance, removal of approximately 1-3 ft of surface concrete and replacement with new concrete is recommended.

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PREFACE

An investigation to assess the condition of the concrete in William Bacon Oliver Lock and Spillway was conducted for the U. S. Army Engineer District, Mobile, by the Concrete Laboratory (CL), U. S. Army Engineer Waterways Experiment Station (WES). This investigation was authorized by Intra-Army Order for Reimbursable Services No. 77-013, dated 27 October 1976.

The contract was monitored by the Mobile District Office under the direction of Mr. Bobby Felder, whose cooperation is greatly appreciated. Mr. Bill Kling coordinated District support to CL during the field work. His assistance and that of the lock personnel was outstanding.

The investigation was conducted under the direction of Messrs. B. Mather and J. M. Scanlon. Active participants in the condition survey included J. E. McDonald, R. L. Campbell, J. T. Peatross, Z. N. Ok, A. Muller, H. Thornton, and D. Glass. The petrographic examination was under the direction of Mr. A. Buck. The stress analysis was directed by Mr. Campbell, with assistance from Mr. A. M. Alexander. The report was prepared by Messrs. McDonald and Campbell.

Col J. L. Cannon, CE, was WES Commander and Director during the conduct of this investigation. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units, as follows:

Multiply	By	To Obtain
inches	2.540000 E-02	meters
feet	3.048000 E-01	meters
pounds (mass)	4.535924 E-01	kilograms
pounds (mass) per cubic foot	1.601846 E+01	kilograms per cubic meter
pounds (force) per square inch	6.894757 E+03	pascals
kips (force) per square inch	6.894757 E+06	pascals
feet per second	3.048000 E-01	meter per second
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following equation: $C + (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

AN INVESTIGATION OF CONCRETE CONDITION, WILLIAM BACON OLIVER LOCK AND SPILLWAY

PART I: INTRODUCTION

Background

William Bacon Oliver Lock and Dam was constructed on the Warrior River near Tuscaloosa, Alabama, between 1937 and 1939. Historical data pertaining to the concrete placed in this structure is included in Appendix A. This record states that several years (the exact time is not known) after completion of the structure, small cracks appeared in the top surfaces and faces of the lock wall. This cracking increased progressively and by 1947 had reached such serious proportions, that it was decided a special investigation should be made to determine its cause and any action necessary to prevent further deterioration. Consequently, a Board of Consultants was appointed to study and report on the condition of the concrete and to recommend remedial measures.

The Board examined the structure in November, 1947, reviewed available data, and concluded that: (1) cracking in monoliths built with Alpha Portland cement was more advanced than in the monoliths built with Penn-Dixie Portland cement; (2) there were no indications that workmanship or inspection was responsible for the condition of the lock; and (3) additional data on the condition of the structure should be obtained by drilling cores, examining concrete specimens and materials, and by making a detailed crack survey. The complete report by the Board is contained in Appendix B.

A series of cores was drilled including a 36-in diameter core from Monolith No. 5, the upstream gate monolith in the land wall. The Board met again in January, 1948, examined the additional data, the cores, the 36-in core hole, and concluded that: (1) the cracking in Monolith No. 5 was not as serious as it had appeared from the surface, and (2) laboratory investigations should be conducted to determine the causes of the cracking. The complete report of the Board is included as Appendix C.

In February, 1948, the Concrete Laboratory at Waterways Experiment Station (WES) was requested by the Mobile District to conduct tests to determine the cause of cracking and disintegration of the concrete in the lock walls. Two sections of the 36-in core, 2 ft and 5 ft in length, and a total of 91 ft of 4 3/4-in core from three other monoliths (Nos. 3, 20, and 60) were sent to the laboratory for study. Detailed results of this investigation are given in Appendix D, and the conclusions are summarized in the following:

1. The primary cause of concrete cracking and disintegration is a deleterious chemical reaction between the alkalies in the cement and unstable silica in the aggregate.

2. The study of the concrete specimens confirmed the indications developed from examinations and physical tests of the structure that, (a) the cracking is largely confined to near-surface zones, and (b) is more pronounced in those portions of the structure in which reportedly Alpha cement was used.

The Board of Consultants met again in October, 1949, to review the field data obtained since the last board meeting, inspect the condition of cracking in the lock walls, and discuss the alkali-aggregate problem involved in this and similar structures. On this basis the Board concluded that: (1) internal expansion and external cracking were continuing throughout the various lock-wall monoliths but at a rate generally decreasing compared to the preceding two years; (2) the cracking in Monolith No. 51 had increased appreciably since the last meeting; (3) the Concrete Laboratory report was sufficiently exhaustive to serve the purpose of this investigation, and no further study of alkali-aggregate reaction was necessary for the maintenance and operation of the lock in the future; (4) internal concrete growth would likely continue for an undetermined period; and, until it ceased, extensive repairs would appear impractical except to specific points, such as Monolith No. 51 where concrete around the mooring bit had deteriorated to such an extent to make it dangerous for use; and (5) no further meetings of the Board were contemplated. The Board recommended that the concrete in the top of Monolith No. 51 be removed, and that a reinforced concrete cap block be cast on top of this monolith. The complete report by the Board is contained in Appendix E.

Purpose and Scope

The purpose of this investigation is to assess the condition of the concrete in Oliver Lock and Spillway through an engineering condition survey and stress analysis. The following is included in this study:

1. Crack survey of lock walls.
2. Soniscope investigation of lock and spillway.

3. Examination of concrete and foundation cores and tests to determine material properties.
4. Finite element stress analysis of the upper land wall gate block.
5. Report of results including comparisons with previous work on this structure.

PART II: CONDITION SURVEY

The engineering survey to assess the condition of concrete in Oliver Lock and Spillway consisted primarily of mapping significant cracking, soniscope investigation of selected monoliths, and an examination and testing of concrete and foundation cores to determine material properties. Field work associated with the condition survey was accomplished during the Fall of 1976.

Concrete Cracking

During the course of the condition survey, a comprehensive examination of concrete cracking was made. Based on these visual and photographic records, maps of surface cracking were prepared for the lock structure, as shown in Figs. 1-10. Excluding the pipe gallery, where no delineation as to size was attempted, the following surface delineation was used:

Designation	Surface Crack Width, in.
....	Maximum width < 1/16
———————	Maximum width ≥ 1/16

In general, monoliths cast entirely with concrete containing the higher alkali cement exhibited the most severe cracking. Of these, Monolith Nos. 8, 16, and 20 had the most extensive surface cracking Figs. 11-18). Monolith No. 5 also had extensive cracking but slightly less severe than the other three monoliths (Figs. 19-20). While the number of cracks for a given surface area was comparable for all four monoliths, the width of the cracks in Monolith No. 20 was generally larger than in the three remaining monoliths. Restraint due to the backfill, operating machinery, gate anchorages, and size of section may have contributed to the generally smaller crack widths in the other three monoliths.

The intensity of surface cracking in the lock wall monoliths generally decreased with distance from the surface, and, for the most part, was limited to the upper 20 ft of the monoliths. Moisture and temperature conditions in these areas were probably more conducive to alkali-silica reaction. It should be noted that significant cracking was located in some instances at greater depths, particularly in the bulkhead recess, Monolith No. 16 (Fig. 13).

The upstream land wall gate monolith (No. 5) received particular attention during the condition survey because it had the most extensive surface cracking of the four gate monoliths and was built entirely with the higher-alkali cement. The current condition of surface cracking in selected areas is compared to the condition of the same areas in 1948 (Figs. 21-31). A number of the cracks located in 1948 are not evident currently; however, the major cracks appear to be slightly wider at present than they were in 1948. Also, there is surface spalling along the edges of the current cracks which was not apparent in 1948. It is suspected that most of these changes occurred during the period shortly after 1948, since the current inspection indicated almost all of the cracks have been inactive for some time. While differences in camera positions, focal distances, etc., make exact comparisons of crack conditions impossible, crack patterns and widths do not appear to have changed significantly during the past 28 years.

Overall, the chamber faces of both lock walls appear to be in relatively good condition. The river wall face, in particular, showed little evidence of deterioration. With the exception of some small areas which have experienced abrasion and gouging from tows during locking operations, the river chamber face remains essentially unchanged with time (Fig. 32). A number of the monoliths in the land wall have a significant horizontal crack in their chamber faces, coinciding approximately with the upper pool water level (Fig. 33). This cracking is generally confined to those monoliths with thin upper sections and coincides approximately with the change in cross section of the monoliths. A number of these monoliths exhibit areas of gel leaching in their upper portions (Fig. 34).

In general, cracking was less extensive in the inspection galleries than on the monolith surfaces. Examples of this type of cracking are shown in Fig. 35. Deposits of gel resulting from the alkali-silica reaction were evident on gallery surfaces within a number of monoliths (Fig. 36). There were only a very few instances, such as Monolith No. 13, where leaching appeared to be a current process.

A number of monoliths, particularly in the river wall, contained the higher alkali cement only in their lower portions. While the upper portions of these monoliths exhibit no significant deterioration, the lower portions have cracked as a result of the alkali-silica reaction (Figs. 37-40). Consequently, the internal growth and cracking of this concrete has caused significant displacements, both horizontal and vertical, of the top surface of some monoliths, particularly in the lower guard wall (Figs. 41-43). Relative displacements between adjacent monoliths of more than 2 in. were measured, and these large displacements have contributed to joint deterioration (Fig. 44). Maximum relative displacements between the joints of other lock-wall monoliths were approximately 1 in., and the majority were due to internal growth of high-alkali concrete in the upper portions of these monoliths (Fig. 45). A comparison of current photographs of joint displacements with similar photographs obtained in 1948 and 1954 (Figs. 41, 42, and 45) indicates the major part of these displacements occurred relatively early in the life of the structure. Some system of periodic measurements to monitor these displacements would appear desirable.

Monolith No. 54 would also appear to merit periodic inspection. What appears to be a transverse settlement crack is located immediately upstream of the operations building. This crack crosses the top of the monolith and continues down both the lock and river faces (Fig. 46) to near the water line. In addition, there is some vertical displacement at the joint between Monolith Nos. 54 and 55 on the river side. This is also evidenced in the displacement of piping at this joint within the operations building.

Soniscope Investigation

The equipment used in a soniscope investigation is similar to that described in Corps of Engineers test method CRD-C 51-72[1]. The apparatus transmits pulses of ultrasonic sound through a material and measures electronically the time required for their transmission. The three principal components of the equipment are: a control unit, a transmitting transducer, and a receiving transducer.

The transmitting and receiving heads consist essentially of stacks of piezoelectric crystals mounted in a metal housing which is covered with a rubber diaphragm and filled with castor oil under slight pressure. The transmitting head transforms electrical pulses into mechanical waves to produce bursts of sound waves lasting a few hundred microseconds. These sound waves travel through the concrete and are picked up by the receiving head. Both the transmitting and receiving heads are connected to the control unit by coaxial cables. The control unit contains the electronic circuits necessary to generate the pulses, and a cathode ray tube upon which both the transmitted and received pulses are displayed. A time-measuring circuit provides for the accurate determination of the pulse transmission time. Velocities through the material can be computed by using the following formula:

$$\text{Pulse velocity, fps} = \frac{\text{Path length, ft}}{\text{Transmission time, sec}}$$

Experience in ultrasonic testing indicates that the relation between velocity and quality of concrete of normal density is approximately as shown in the following tabulation. It should be noted, however, that these values are only typical, and cannot be expected to apply in all instances.

Pulse Velocity, fps	Condition
Above 15,000	Excellent
12,000-15,000	Generally good
10,000-12,000	Questionable
7,000-10,000	Generally poor
Below 7,000	Very poor

Previous Tests

The initial soniscope tests were conducted in June, 1948, by the Portland Cement Association. Subsequent tests have been conducted periodically by the Concrete Laboratory, WES.[2,3,4] Initial results indicated the pulse velocity of the concrete in Monolith No. 5 between the calyx hole and the lock chamber face (7.62-ft path length) progressively increased from 12,200 fps at a depth of 5 ft below the top surface, to 15,110 fps at a depth of 35 ft (Plate 1). Subsequent tests, while limited to the upper 12 ft (with the exception of 1952 and 1954), gave similar results. The variation in pulse velocity, with time for the upper portion of Monolith No. 5, is shown in Fig.47. The pulse velocity of concrete at a depth of 12 ft was essentially constant during this period, averaging approximately 14,500 fps. At the 8½-ft depth, the trend was for concrete pulse velocity to decrease slightly, with an average of approximately 13,700 fps. In comparison the concrete at 5-ft depth exhibited an increase in pulse velocity from approximately 12,000 fps to more that 13,000 fps during this period.

During the period 1948-1975, soniscope tests were conducted on five other monoliths, in addition to Monolith No. 5, with results as shown in Table 1. In this group of tests, the soniscope test path was vertical from the roof of the inspection tunnel to the top of the lock. The variation in average pulse velocity with time for each of these monoliths is shown in Fig.48. In addition the average of the three tests on the upper portion of Monolith No. 5 is included for comparison. Pulse velocities range from approximately 11,00 fps for Monolith No. 20, which exhibits significant cracking, up to approximately 15,000 fps for Monolith Nos. 21 and 60, which are essentially free of cracking in the upper portions tested. In all monoliths a line of best fit determined by least squares analysis indicates an increase in pulse velocity, hence concrete quality, with time.

Current Investigation

During the fall of 1976, soniscope tests were conducted on several areas of the structure not previously investigated. Tests were conducted on Monolith Nos. 5, 8, 16, 18, 68, and 72, upper miter gate sill, and fixed-crest spillway. The nature of this investigation dictated the use of both standard and borehole transducers. Borehole transducers are essentially the same as the standard type previously described, but are waterproofed and are omni-directional. They are lowered into boreholes filled with water, and measurements are made through various elevations of a structure between the boreholes. Water in the hole acts as a couplant.

Test results are presented in Plates 2-9. Of the 72 individual results, only two pulse velocities were less than 13,000 fps, and one of these was 12,990 fps. The other result, 8850 fps, obtained in a test on Monolith No. 8 (Plate 3) is attributed to some local condition on the lock chamber face and is not considered indicative of overall concrete quality for that test path. For this test the transducer inside the lock chamber was at a depth of 40 ft, or approximately 6 ft above lower pool elevation. This is an area of frequent gouges and surface abrasions created during normal locking operations. Since tests at depths both above and below this level gave pulse velocities in excess of 14,000 fps, this 8840-fps test result is not considered representative of the concrete in this general area.

Test results for Monolith No. 5 (Plate 2) were essentially constant for the various depths within the 16- to 35-ft zone, with an overall average of 14,295 fps. It should be noted that the slight decrease in velocity with depth is attributed to the transducer drifting away from the side of the calyx hole which was not drilled exactly vertical.

Excluding the one test from Monolith No. 8 previously discussed, the remaining 42 tests on the various lockwall monoliths gave pulse velocities ranging from 13,360 to 15,210 fps, with an average of 14,575 fps.. Results of tests on the upper miter sill appear even better, ranging from 14,00 to 15,895 fps, with an average of 15,060 fps. Similarly, test results for the fixed-crest spillway ranged from 12,990 to 15,635 fps, averaging 14,510 fps.

With the exception of Monolith Nos. 16 and 20, both of which exhibit significant cracking, soniscope tests indicate concrete pulse velocities generally in excess of 13,000 fps. By comparison pulse velocities in the range of 12,000 to 15,000 fps indicate generally good-quality concrete. Data obtained during the period 1948-1975 indicate the concrete in Monolith Nos. 16 and 20 would be classified as questionable. However, it should be noted that the same data indicate the concrete in these monoliths is not experiencing progressive deterioration; in fact, the trend is for increased pulse velocities since tests were initiated in the early 1950's.

Material Properties

Two shipments of concrete and foundation cores were furnished WES by the Mobile District. The first shipment consisted of concrete and foundation core from Monolith No. 5 and concrete core from Monolith No. 16. Complete laboratory logs of those cores are included in Appendix F. The second shipment of cores consisted of concrete core from Monolith Nos. 8, 16, 18, 68, and 72. In addition, concrete and foundation core from Monolith No. 100 of the fixed-crest spillway was included in this shipment. The foundation portion of core from the spillway was logged in the laboratory, with results included in Appendix F. Field logs of all cores are included in Appendix G. Based on an examination of all logs, portions of the core were selected for testing to determine compressive strength, modulus of elasticity, Poisson's ratio, ultrasonic pulse velocity, and potential for alkali-aggregate reaction. Also, triaxial tests and petrographic examinations were conducted.

Concrete Core Tests

Fifteen 4- by 8-in. specimens from the first shipment of core were tested to determine unconfined compressive strength, modulus of elasticity, and Poisson's ratio, with results as shown in Table 2. The ultrasonic pulse velocity of each specimen was determined prior to mounting four surface strain gages, two each lateral and longitudinal, for destructive testing. Results of tests on 28 similar specimens from the second shipment of core and 3 additional specimens from the first shipment to determine pulse velocity and unconfined compressive strength are shown in Table 3.

The ultrasonic pulse velocity of the concrete generally increased with increased compressive strength (Figs. 49 and 50). For a given strength, the pulse velocity was generally lower for the second series of tests. The time between drilling and testing was longer for the second series and, although the cores were either waxed or wrapped in plastic bags, small losses in moisture content would result in lower pulse velocities.

Compressive strengths of specimens from Monolith No. 5 ranged from 3970 to 6580 psi, with an overall average of 5300 psi. Two specimens from this monolith were tested in 1948, with one 4.75-in. core from a depth of 19.0-23.0 ft having a compressive strength and modulus of elasticity of 5140 psi and 4.52×10^6 psi, respectively. In comparison, the average of four current tests on core from 16.5- to 24.5-ft depth indicates essentially the same compressive strength, 5180 psi, and a somewhat lower modulus of elasticity, 3.38×10^6 psi. The other specimen previously tested was a 6-in. diameter core drilled from the lower section of the 36-in. core which indicated a compressive strength and modulus of elasticity of 5200 psi and 2.68×10^6 psi, respectively. The average modulus of elasticity from the two previous tests, 3.60×10^6 psi, is essentially the same as that determined in the current tests.

Although there were significant variations between individual test results, a least squares curve of best fit (Fig. 51) indicates that the compressive strength of concrete in Monolith No. 5 increases with depth. A similar trend is observed for both modulus of elasticity (Fig. 52), and ultrasonic pulse velocity (Fig, 53). When the results of all strength tests on concrete containing the higher-alkali cement are examined, it appears that compressive strength increases with depth down to approximately 20 ft, after which the strength level is relatively stable (Fig. 54). In comparison, results of tests on concrete containing the lower-alkali cement indicate no systematic variation in compressive strength with changes in depth (Fig. 55). The compressive strength of this concrete ranged from 4960 to 9150 psi, with an overall average of 6630 psi for the 18 specimens tested. Excluding the two test results greater than 9000 psi, the remaining results ranged from approximately 5000 to 7500 psi, with an average of 6320 psi. In comparison, results of tests on the 25 specimens containing the higher-alkali cement ranged from 2580 to 6760 psi, with an overall average of 5070 psi. The differences in pulse velocity for the two concretes were not so pronounced; however, the higher-alkali concrete did seem to be more affected by increased depth (Figs. 56 and 57).

Stress-strain relations for the two concrete specimens tested under triaxial conditions are presented in Fig. 58. The modulus of elasticity and Poisson's ratio for each specimen was calculated as shown in Table 4. The confined modulus of elasticity was somewhat higher than that determined in unconfined tests on comparable specimens.

Excluding the in situ test at 5-ft depth, the current ultrasonic pulse velocities for concrete core from Monolith No. 5 are essentially the same as those determined in situ in 1948 (Fig. 59). The in situ velocity at 5-ft depth has varied considerably over the years (Plate 1). In the two most recent tests, velocities ranged from 12,155 fps in 1969, to 13,535 fps in 1975. The latter result is more consistent with core tests.

Foundation Core Tests

Upon arrival in the laboratory, foundation core from Monolith No. 5 was examined and logged as shown in Appendix F. Based on this examination, four zones were selected for testing to determine material properties for input to the stress analysis as follows:

Zone	Approx. Depth, ft	Description
D	63.1-68.5	Black shale with siltstone
E	68.5-73.7	Siltstone with tan clay, shale
F	73.7-79.9	Black shale with small amounts of tan clay and siltstone stringers
G	79.9-91.4	Black shale with coal stringers

The material descriptions are based on the core log; however the petrographic report states the siltstone is more properly called sandstone.

Eight 4- by 8-in specimens, two from each zone, were tested to determine unconfined compressive strength, modulus of elasticity, and Poisson's ratio, with results as shown in Table 5. In addition two specimens were tested under triaxial conditions. Stress-strain relations for these two tests are shown in Fig. 60. The modulus of elasticity and Poisson's ratio was calculated for each specimen as shown in Table 4.

Petrographic Examination

Samples of both concrete and foundation were taken for petrographic examination. Evidence of alkali-silica reaction and the potential for additional expansion were of particular interest during concrete core examination. The complete petrographic report is included in Appendix F.

Results and conclusions are summarized in the following:

1. The full lengths of concrete core from both monoliths (Nos. 5 and 16) examined show evidence of alkali-silica reaction. Evidence of this reaction decreases with depth, and the major effects of the reaction appear to be concentrated in the upper few feet of each core.

2. Length-changes of concrete specimens from both monoliths stored at 100 percent RH and 100°F show that the concrete still has expansive potential. In general, these length-changes increased with time and with depth.

Figure 1. Surface cracking, land wall, and upper end

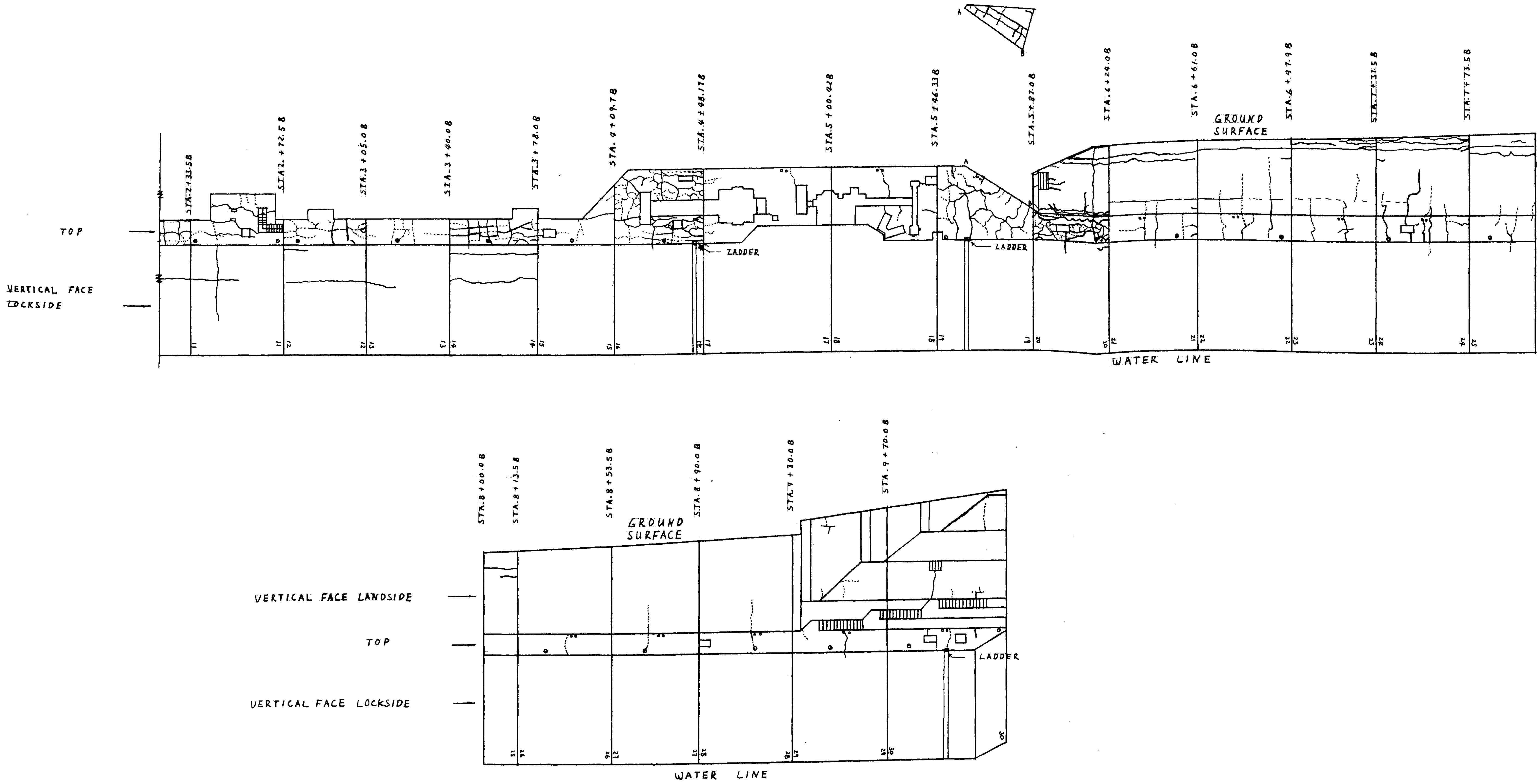


Figure 2. Surface cracking, land wall, and lower end

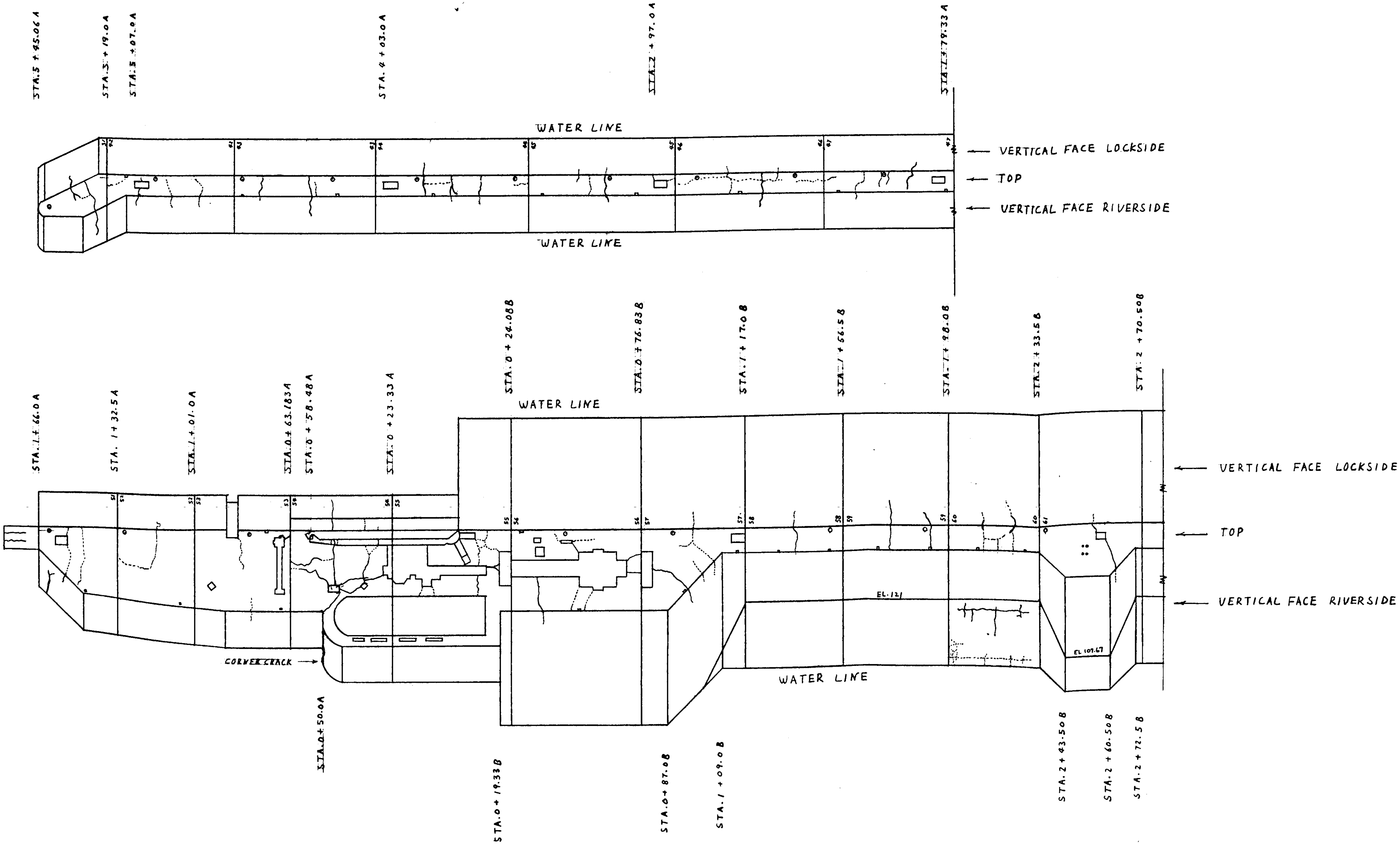


Figure 3. Surface cracking, river wall, and upper end

Figure 4. Surface cracking, river wall, and lower end

Figure 5. Surface cracking, pipe gallery, land wall, and upper end

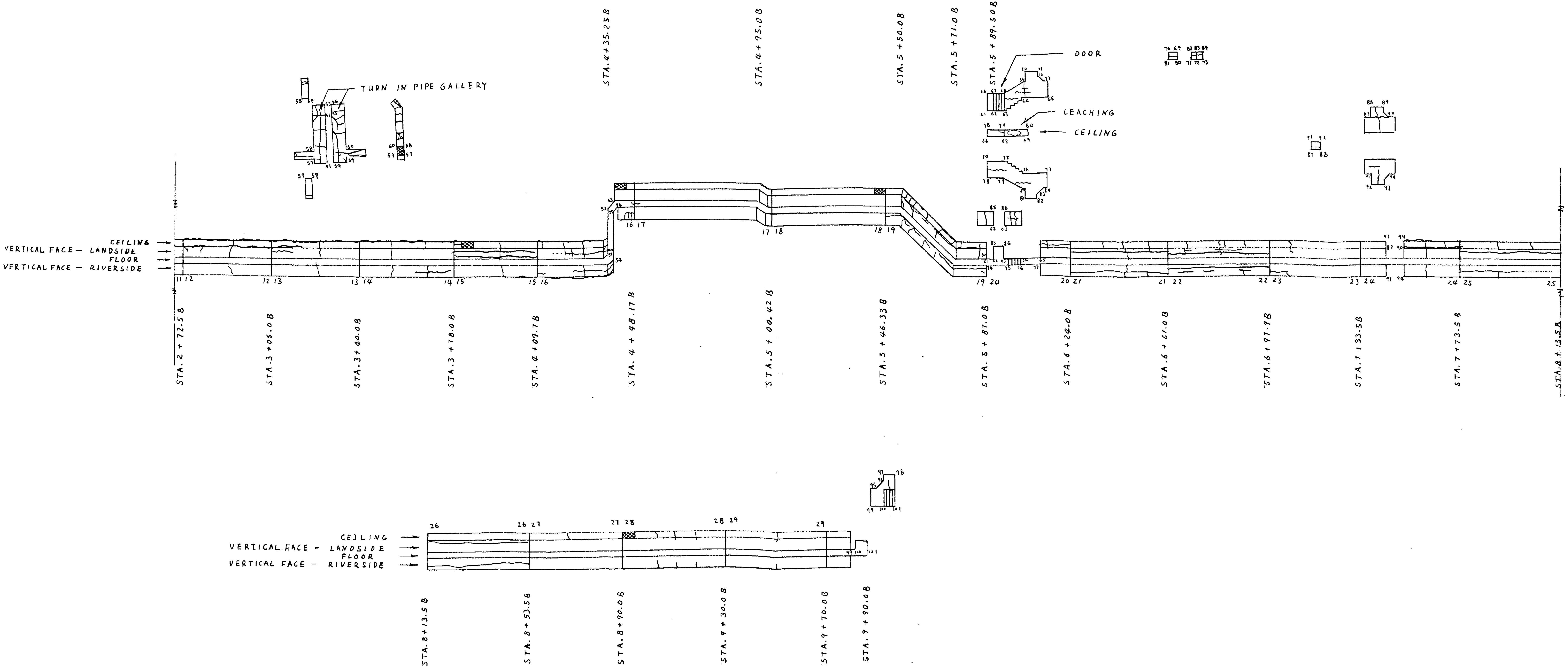


Figure 6. Surface cracking, pipe gallery, land wall, and lower end

Figure 7. Surface cracking, pipe gallery, river wall, and upper end

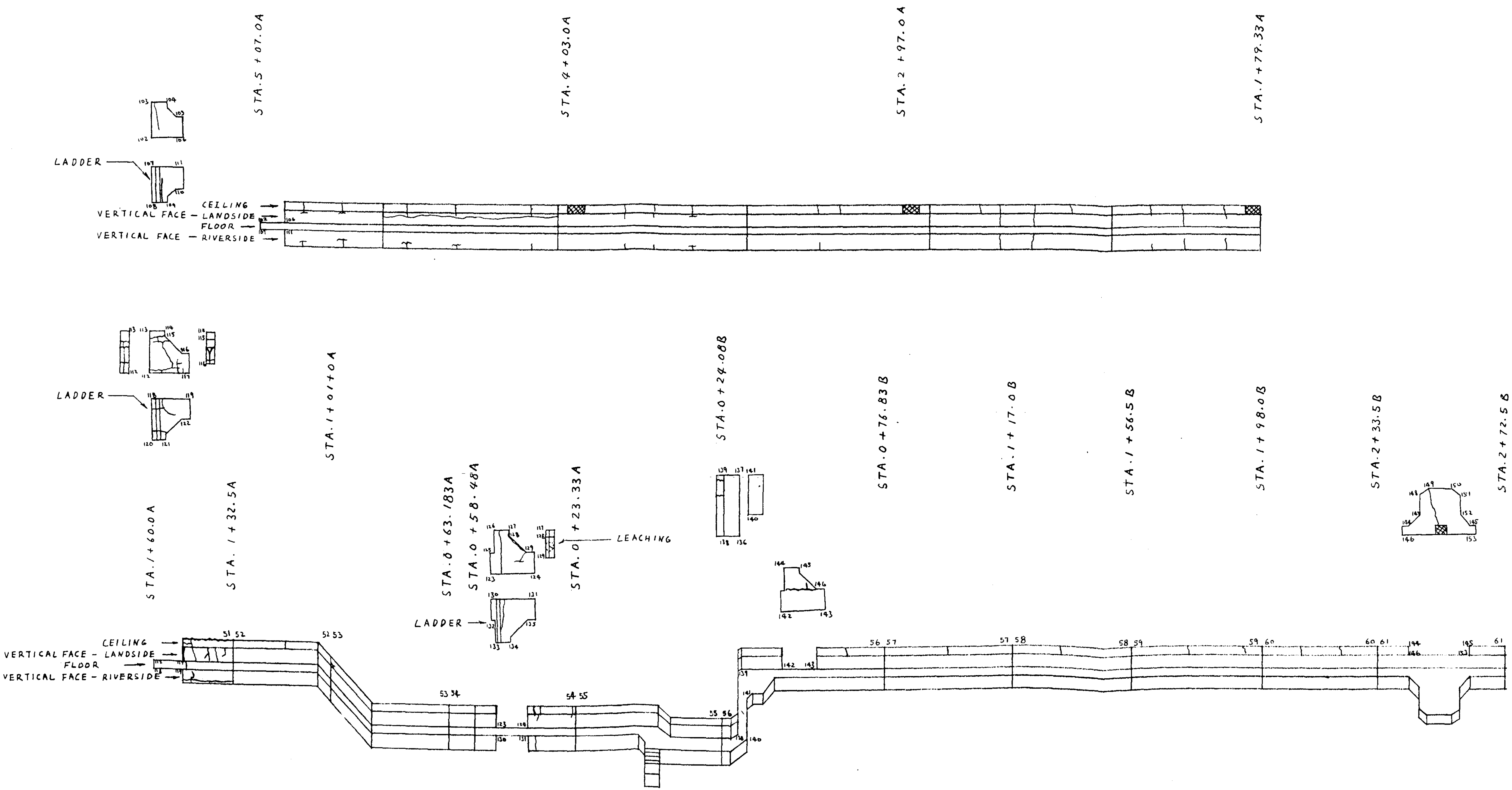


Figure 8. Surface cracking, pipe gallery, river wall, and lower end

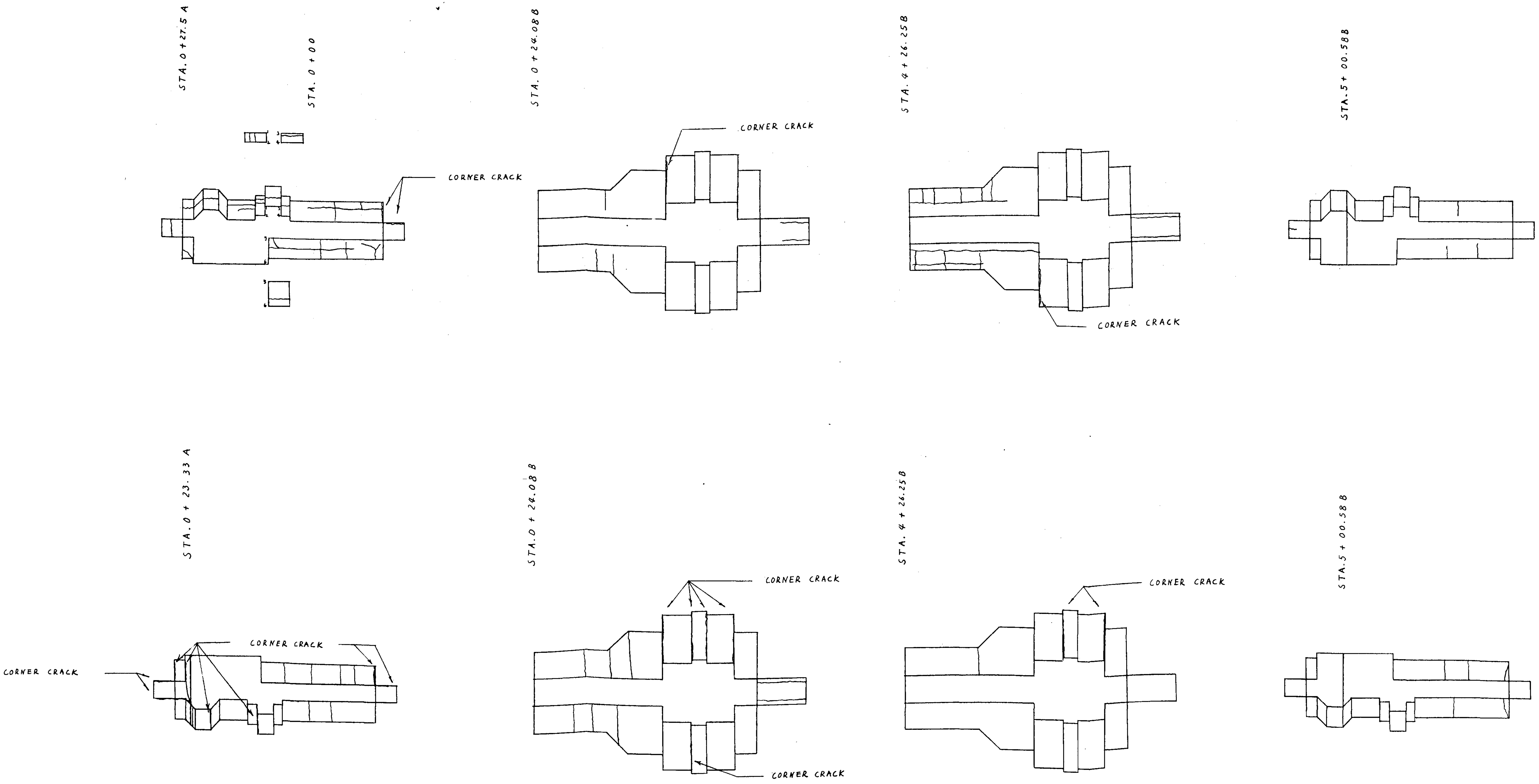


Figure 9. Surface cracking, machinery recesses, and land and river walls

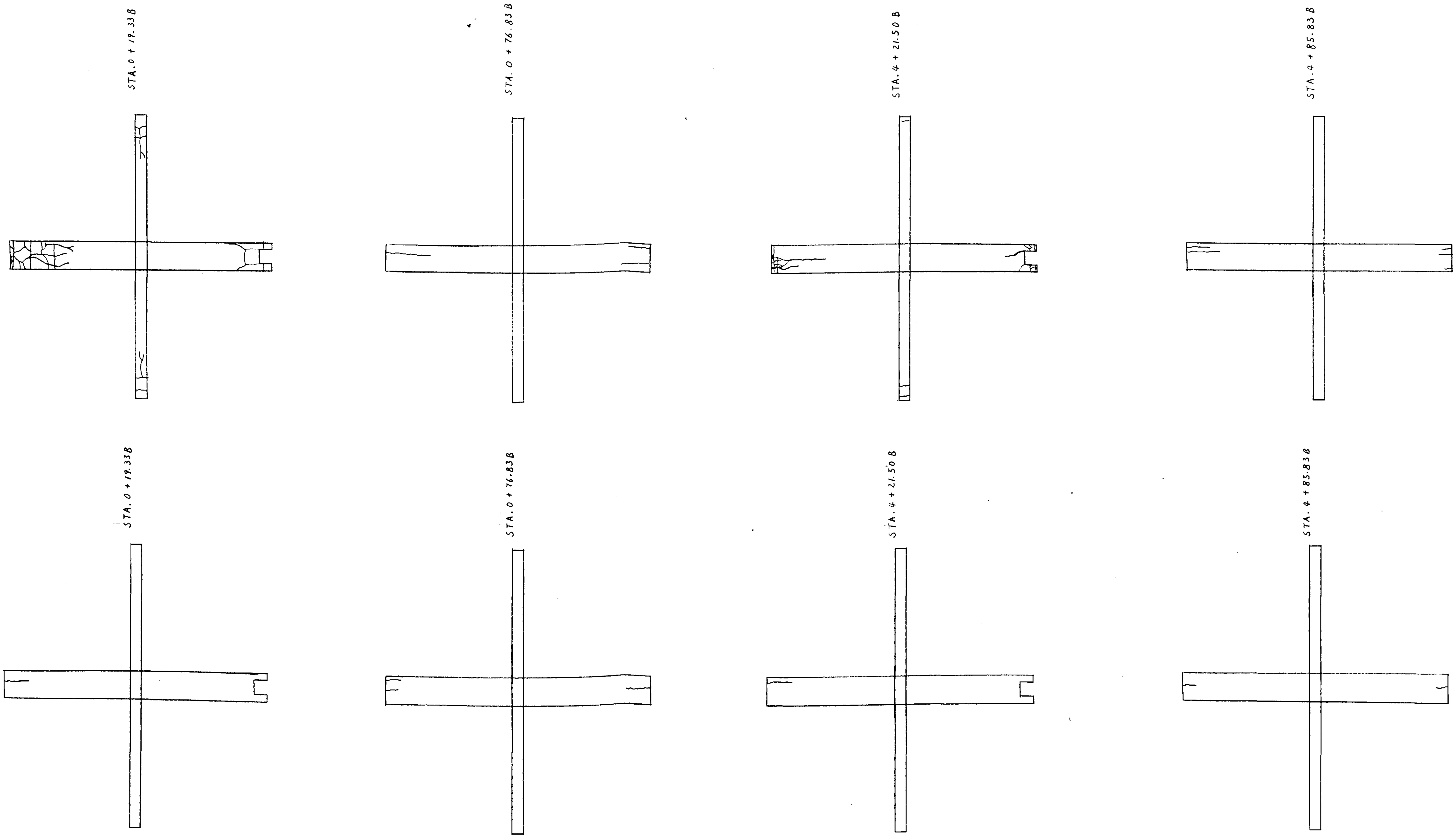


Figure 10. Surface cracking, bulkhead recesses, and land and river walls

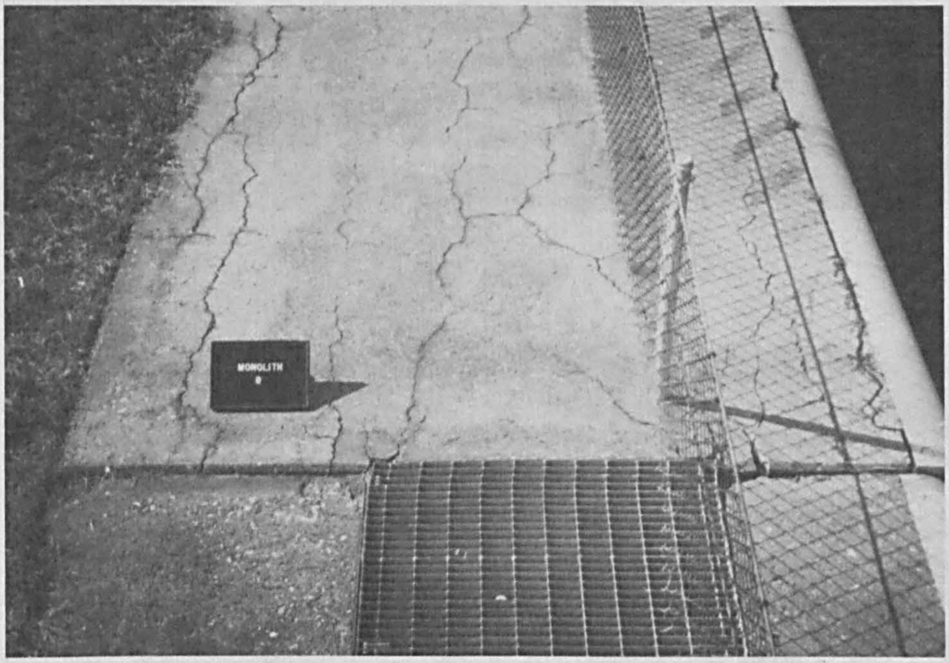
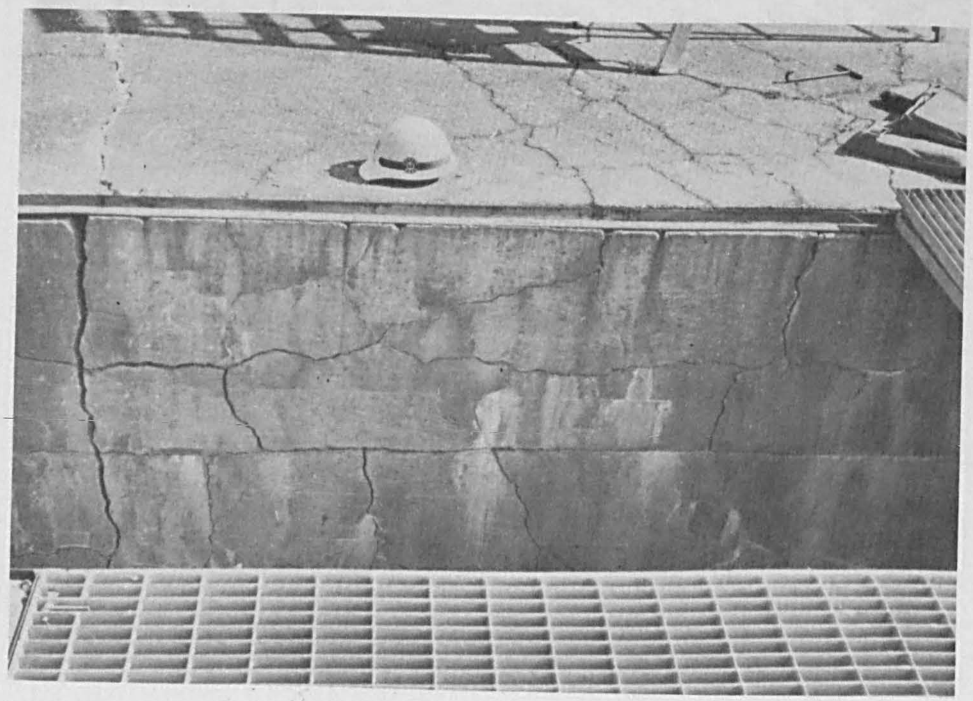


Fig. 11 Expansive force sufficient to warp metal grate.

a. 1948



b. 1976

Fig. 12 Cracking, Monolith No. 16.

Fig. 13 Cracking in upstream face of
bulkhead recess, Monolith No. 16.

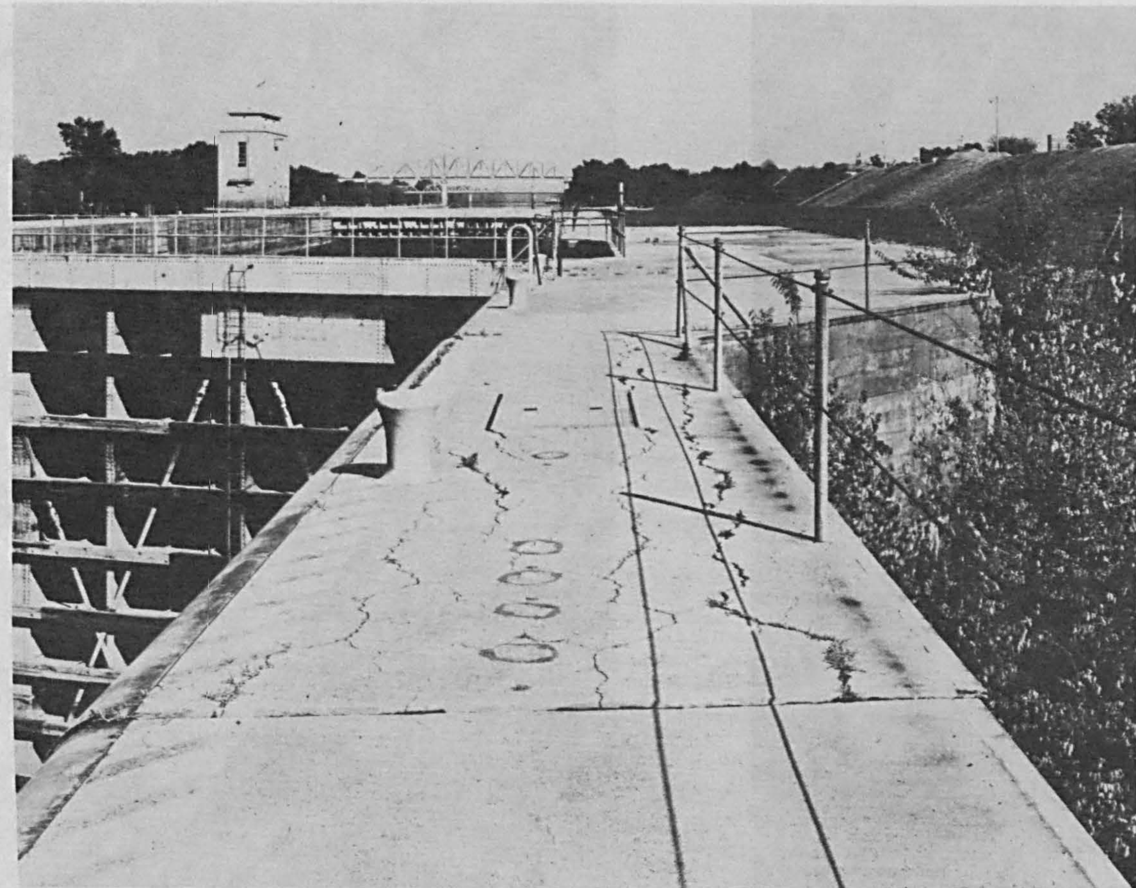
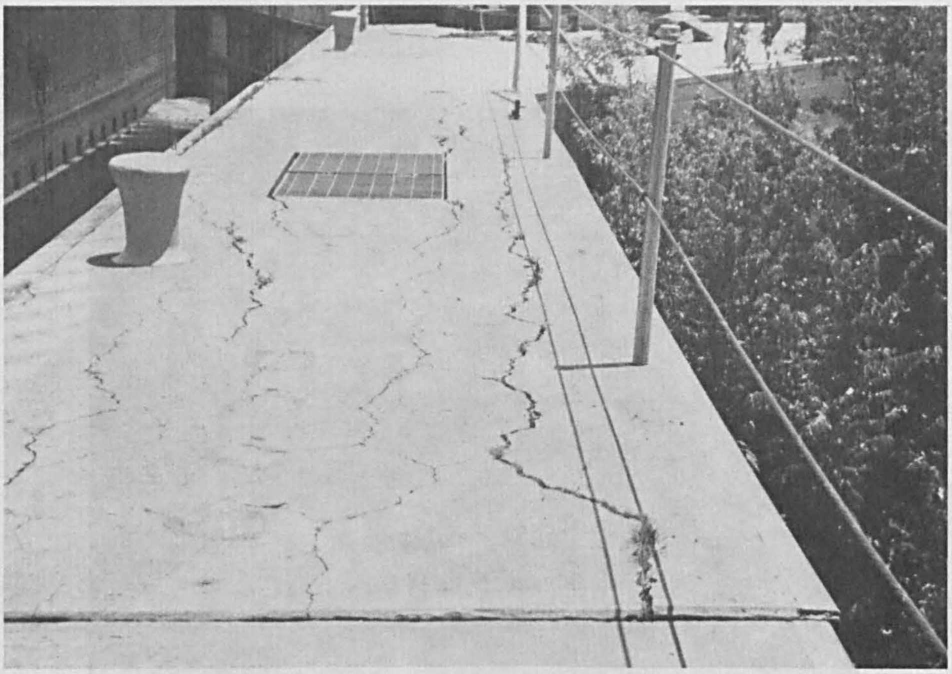


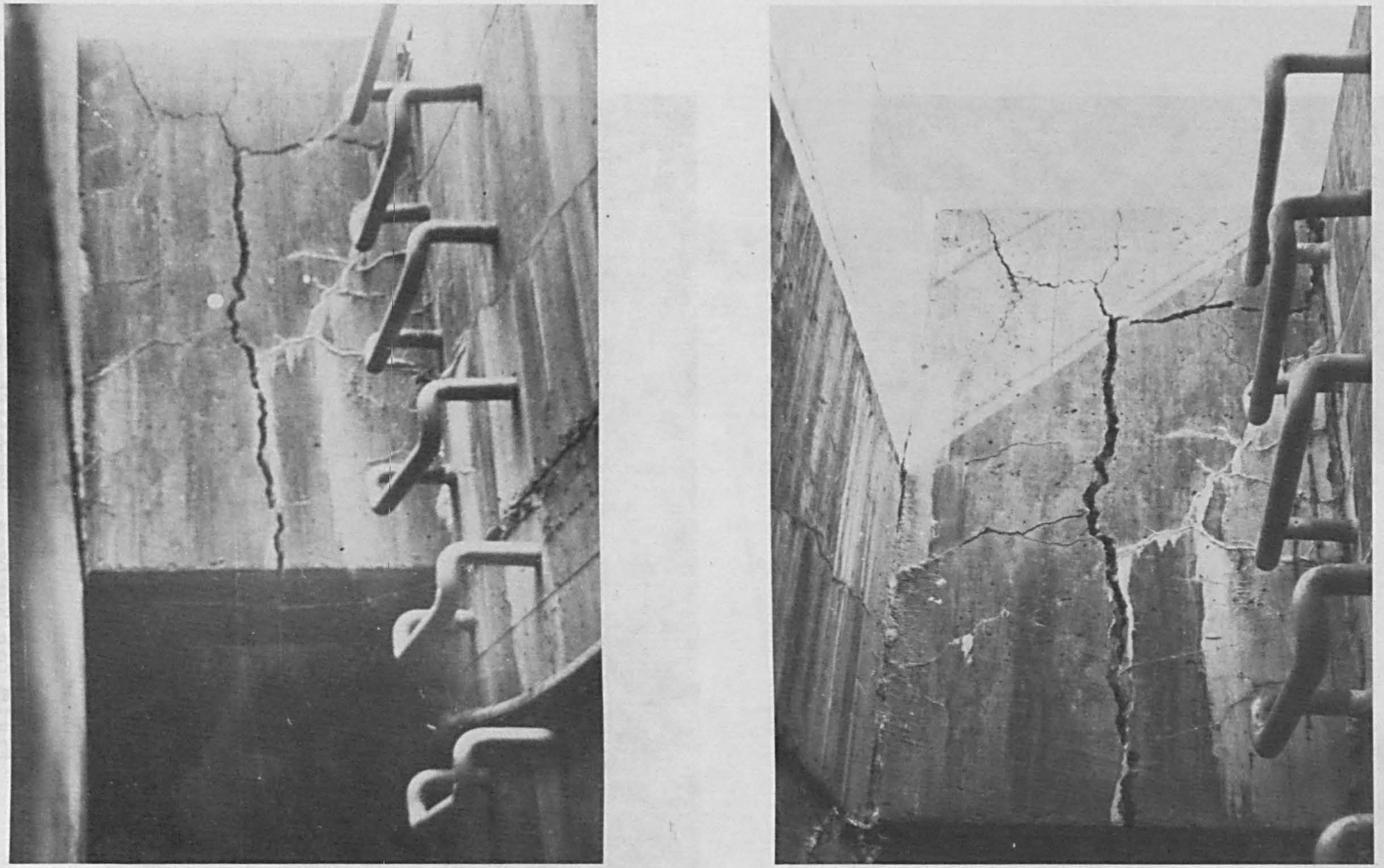
Fig 14 Top Surface, Monolith No. 20, 1948.



a. 1954

b. 1976

Fig 15 Top Surface, Monolith No. 20.

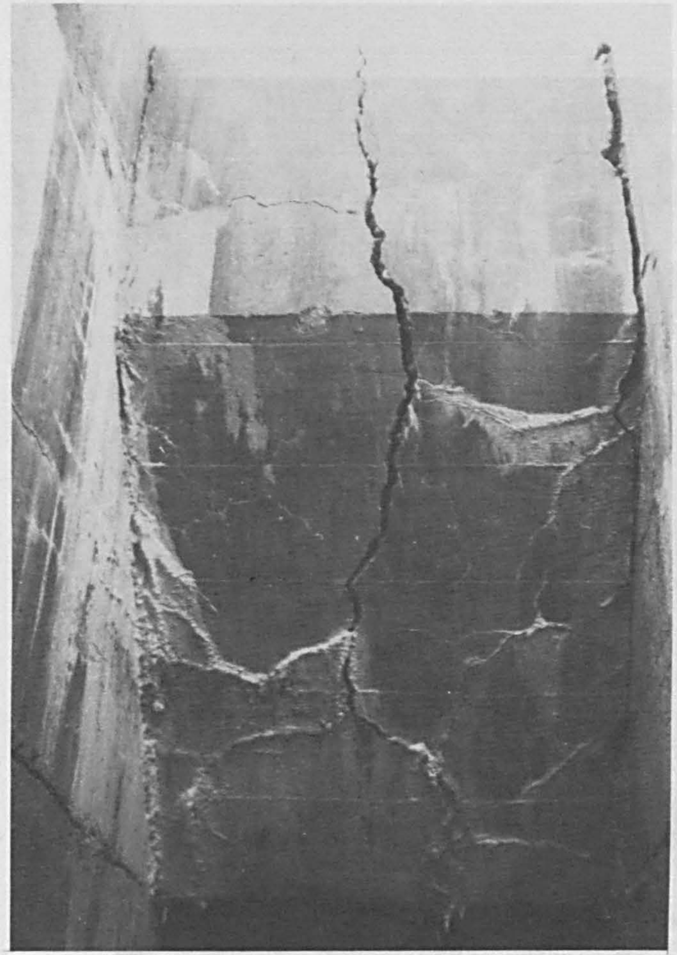


a. 1954

b. 1976

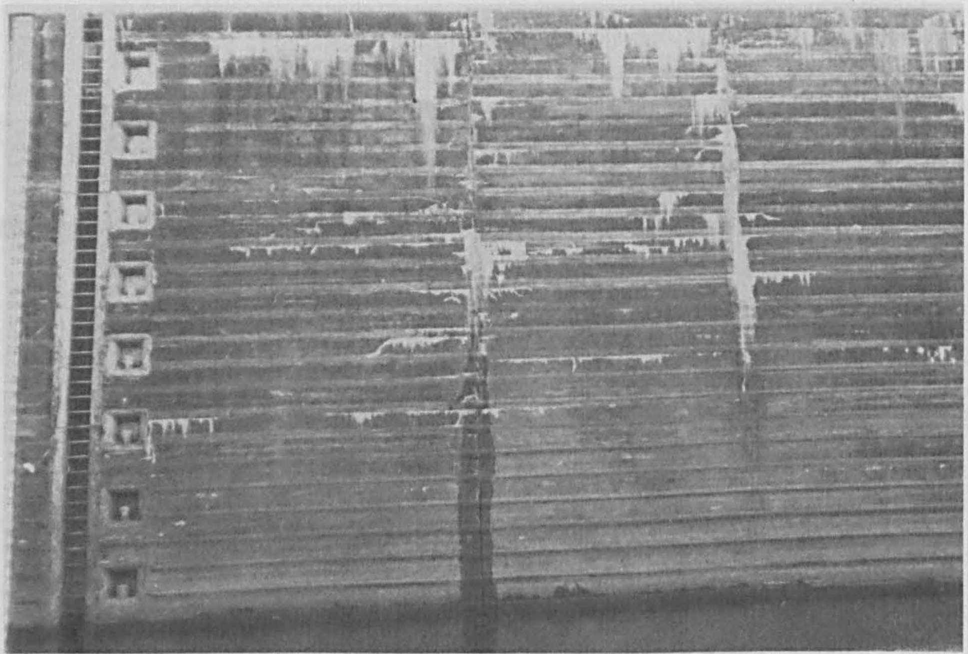
Fig 16 Cracking in Monolith No. 20 as seen from gallery looking upstream.

a. 1954



b. 1976

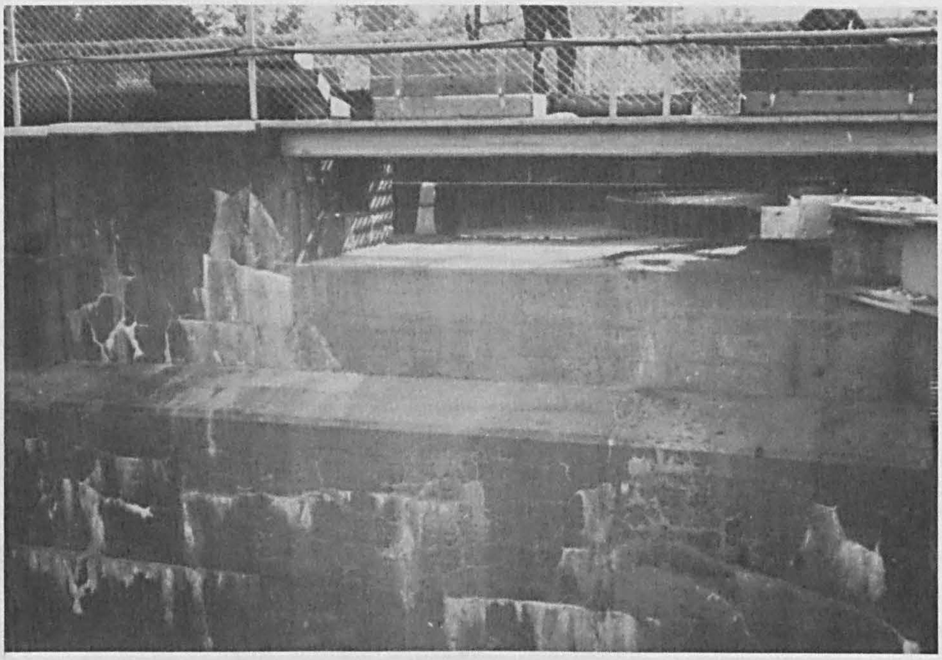
Fig 17 Cracking in Monolith No. 20 as seen from gallery looking downstream.



a. River face, Monolith Nos. 19 and 20

b. Land face, Monolith No. 20

Fig. 18 Lower guide wall deterioration.



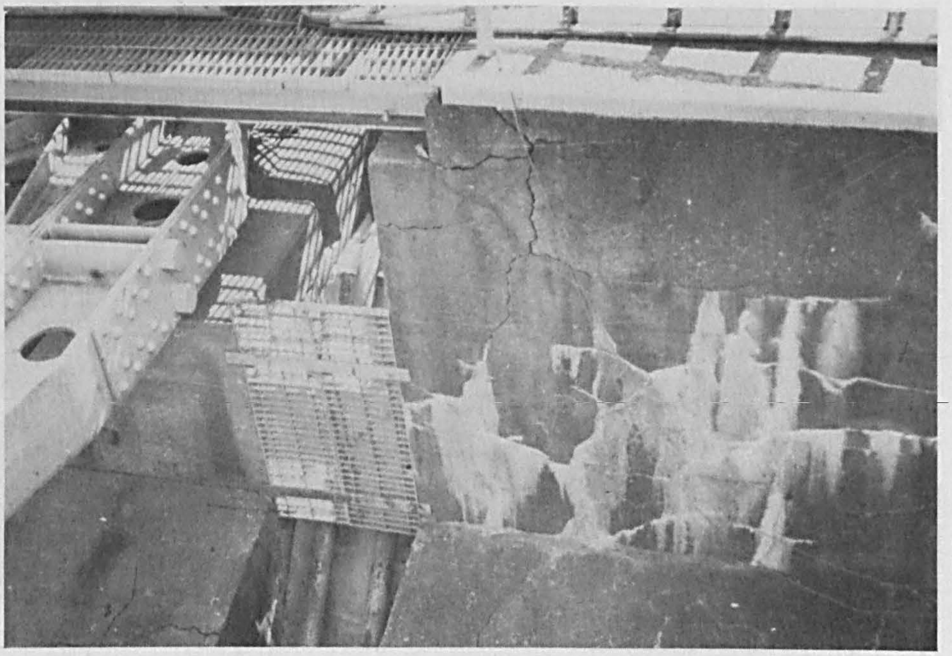


Fig. 19 Cracking in upper gate block,
land wall (Monolith No. 5).

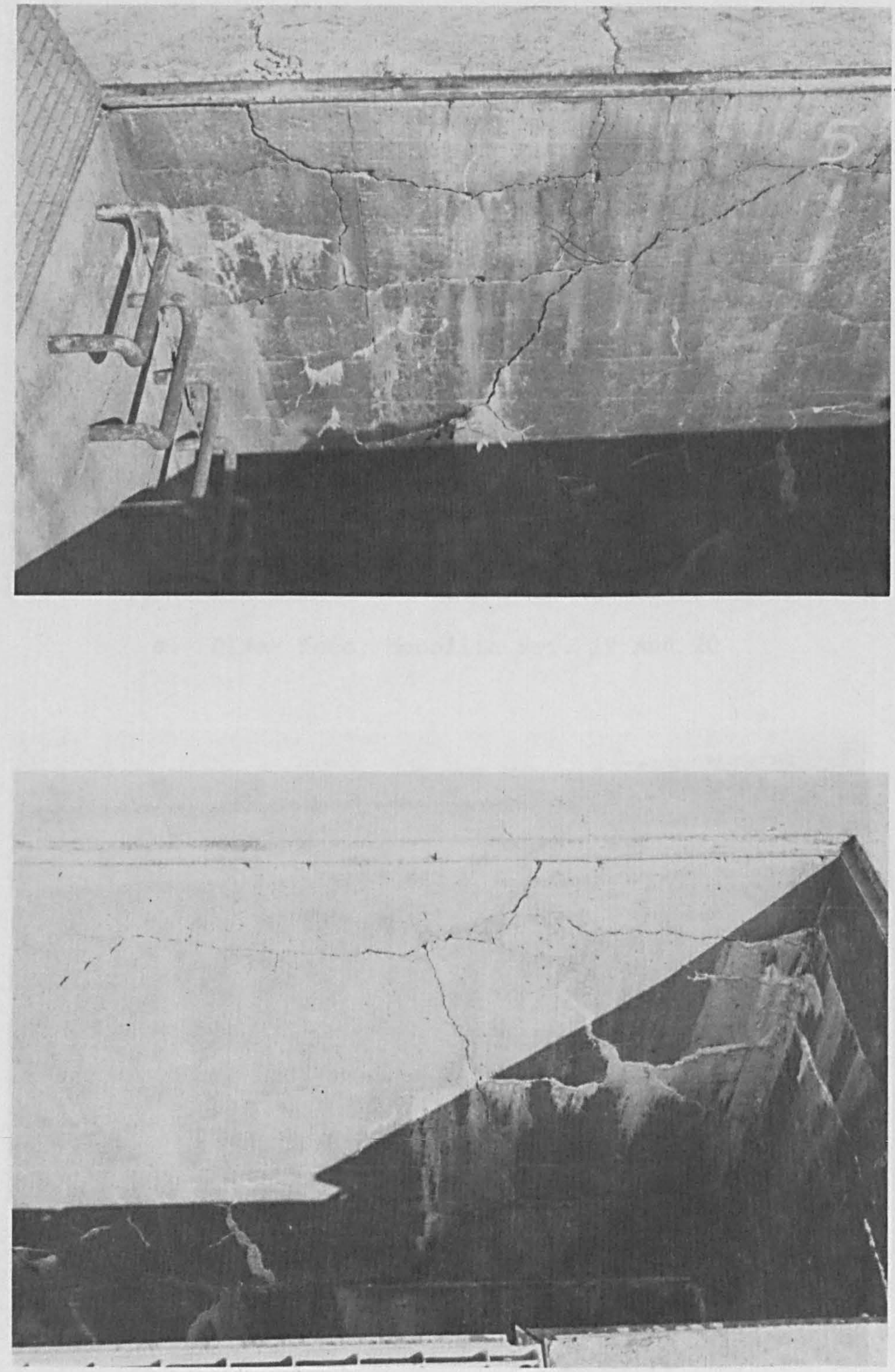


Fig. 20 Cracking in upstream face at bulkhead recess, Monolith No. 5.

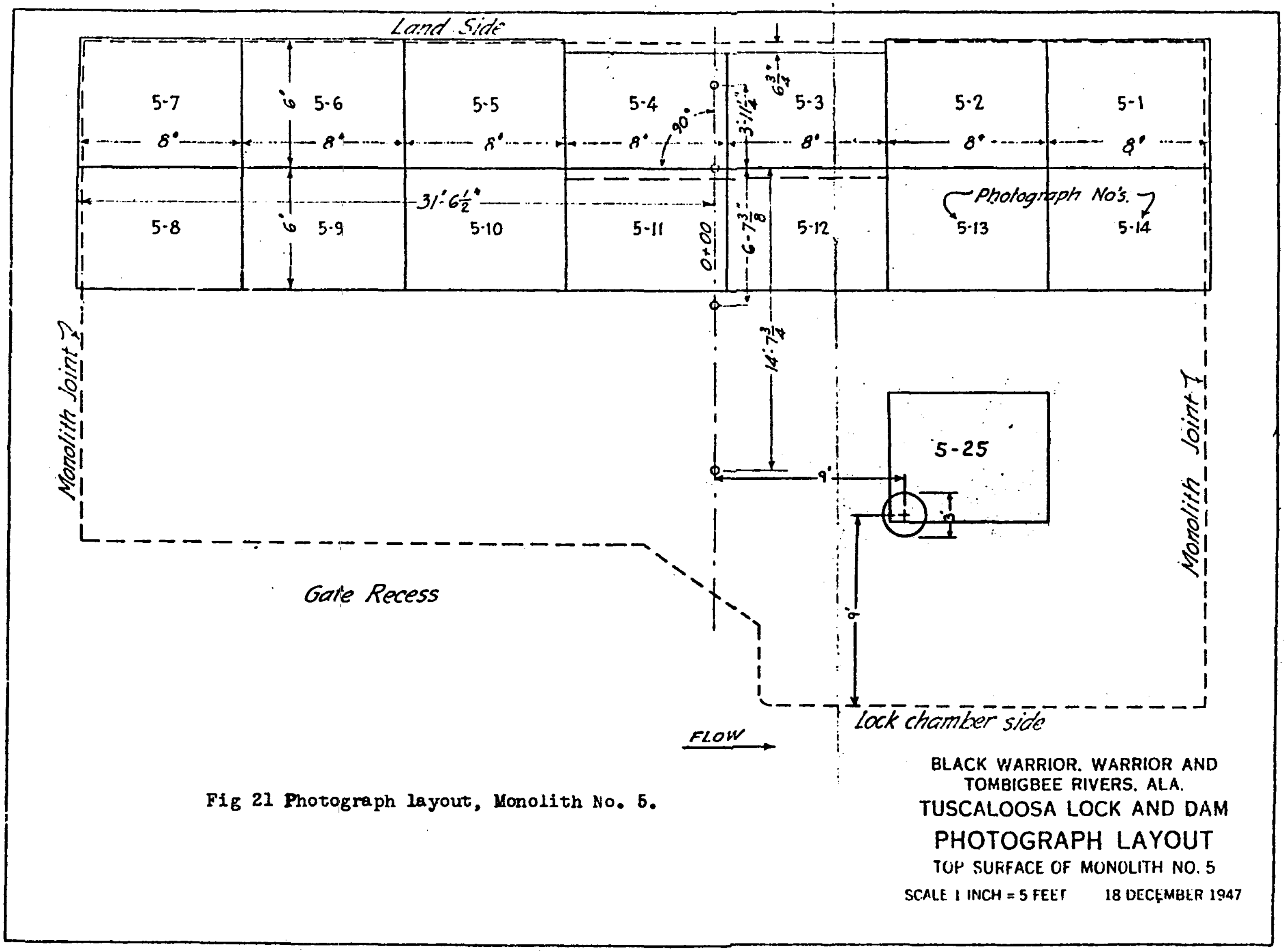
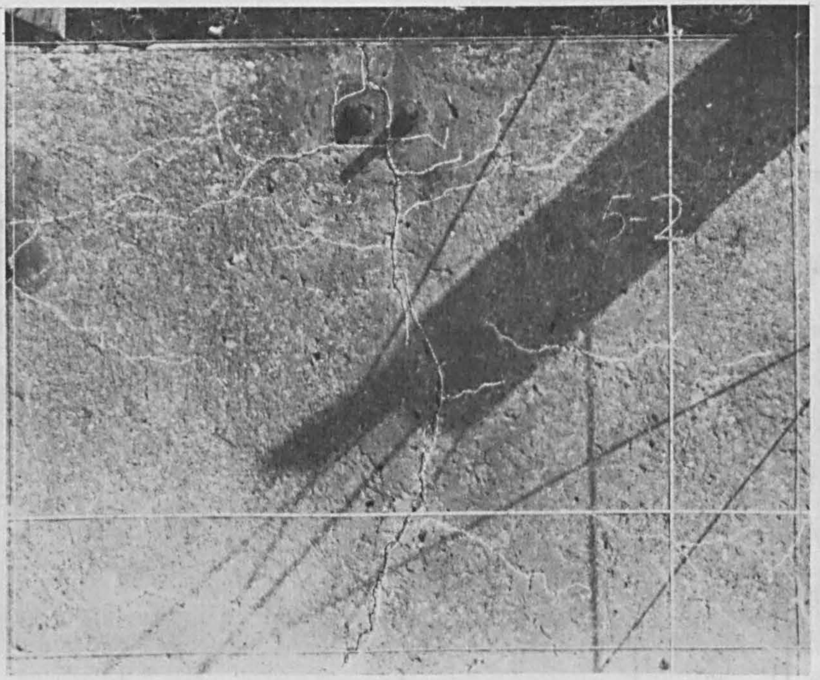


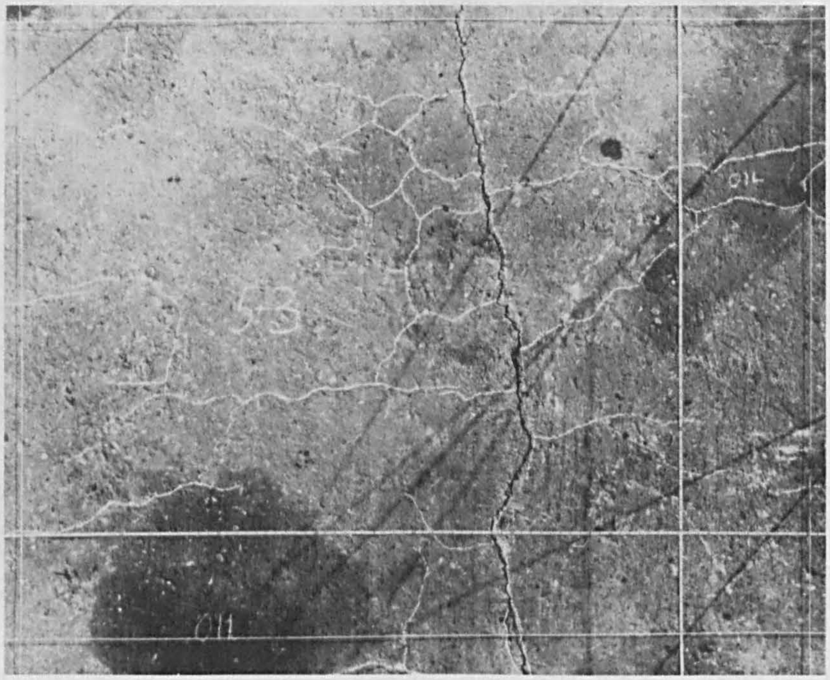
Fig 21 Photograph layout, Monolith No. 5.



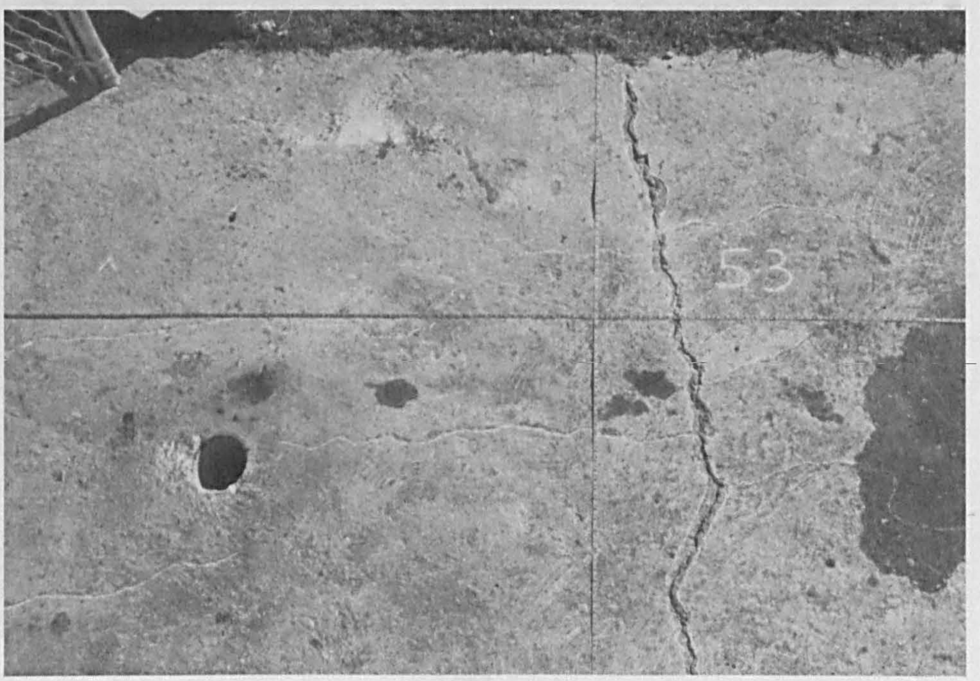
a. 1948

b. 1976

Fig. 22 Top surface, Monolith No. 5.

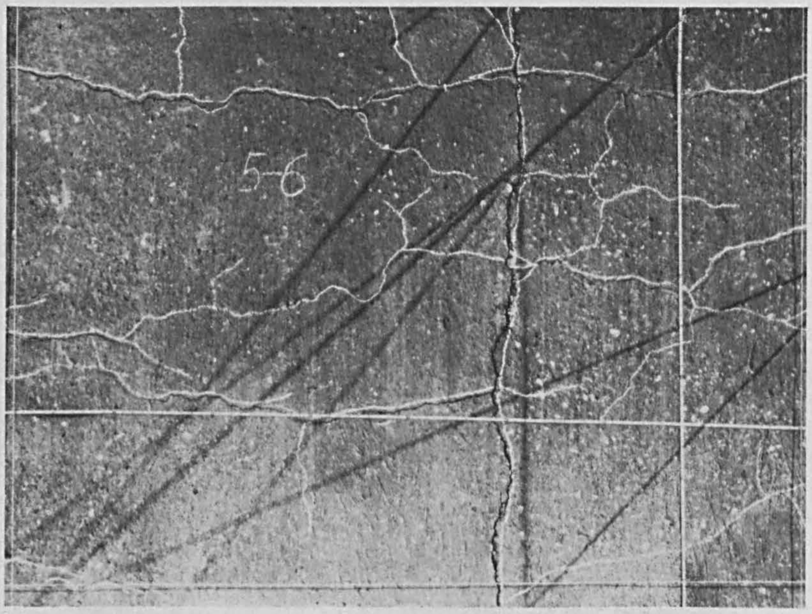


a. 1948

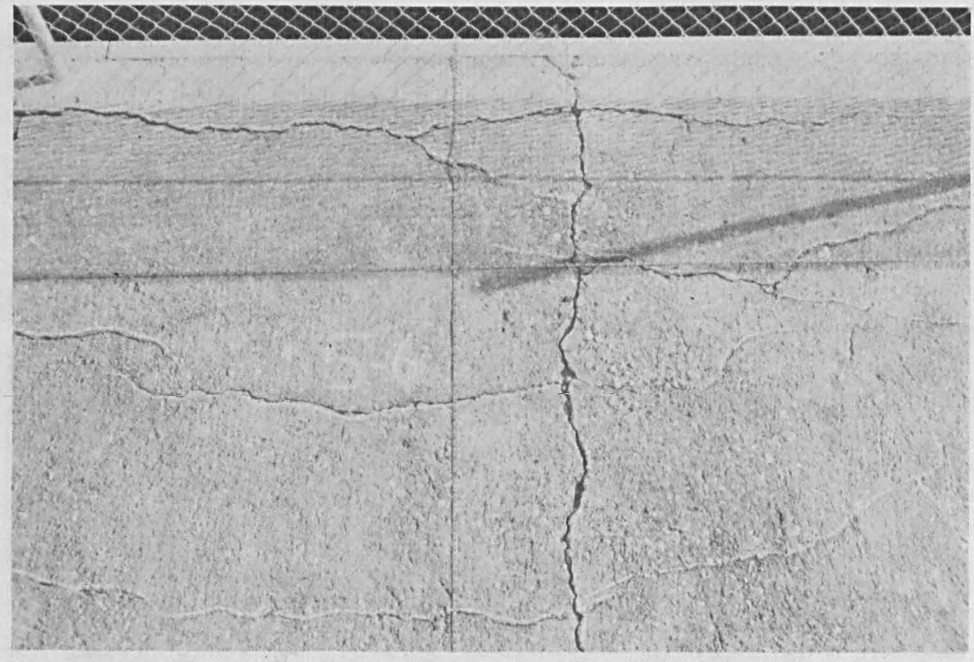


b. 1976

Fig. 23 Top surface, Monolith No. 5.



a. 1948



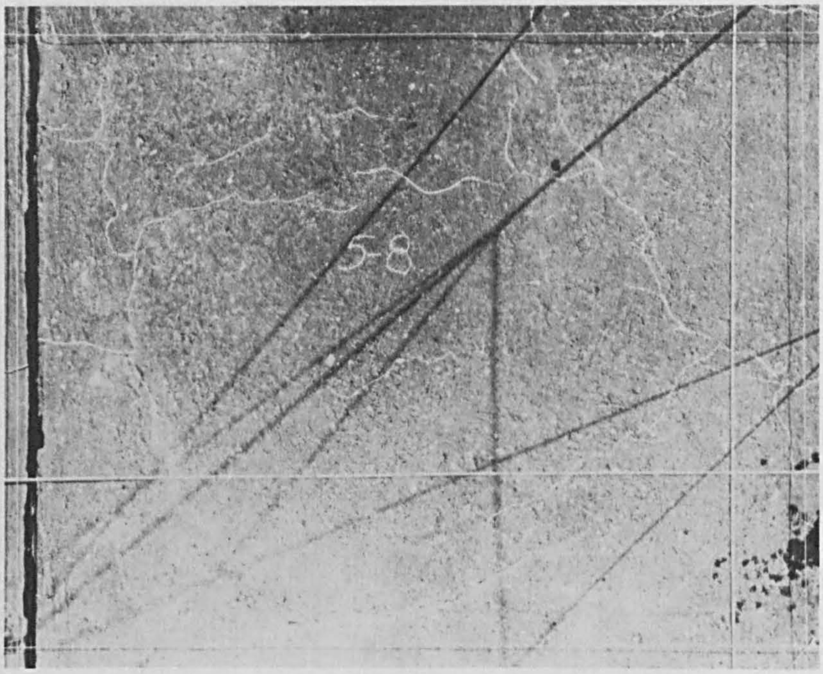
b. 1976

Fig. 24 Top surface, Monolith No. 5.

a. 1948

b. 1976

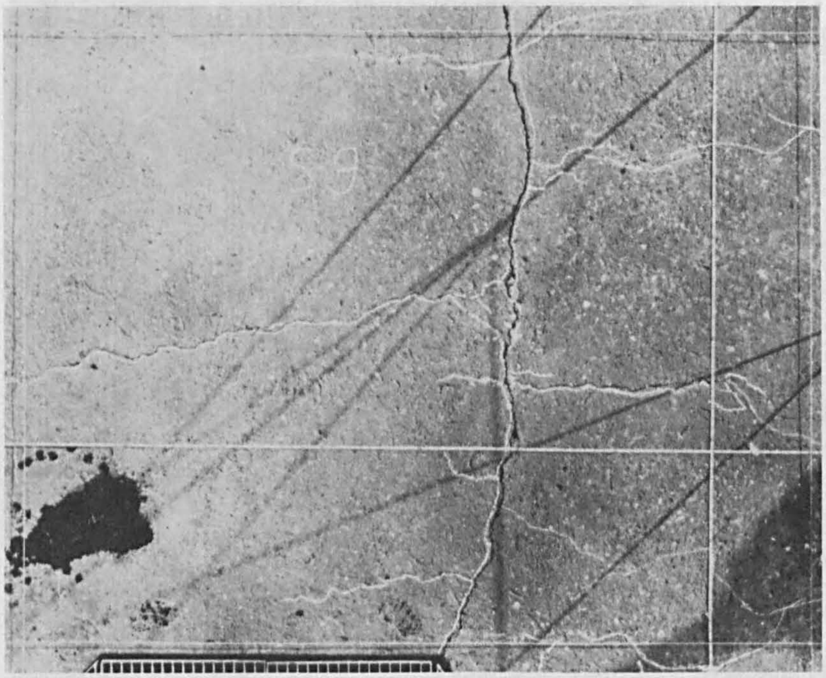
Fig. 25 Top surface, Monolith No. 5.



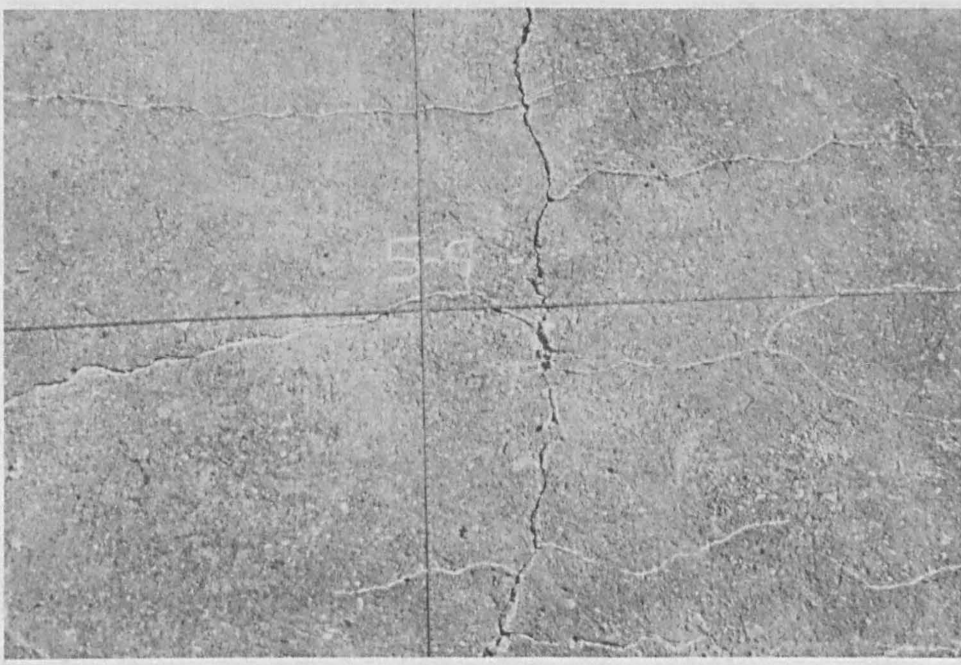
a. 1948

b. 1976

Fig. 26 Top surface, Monolith No. 5.

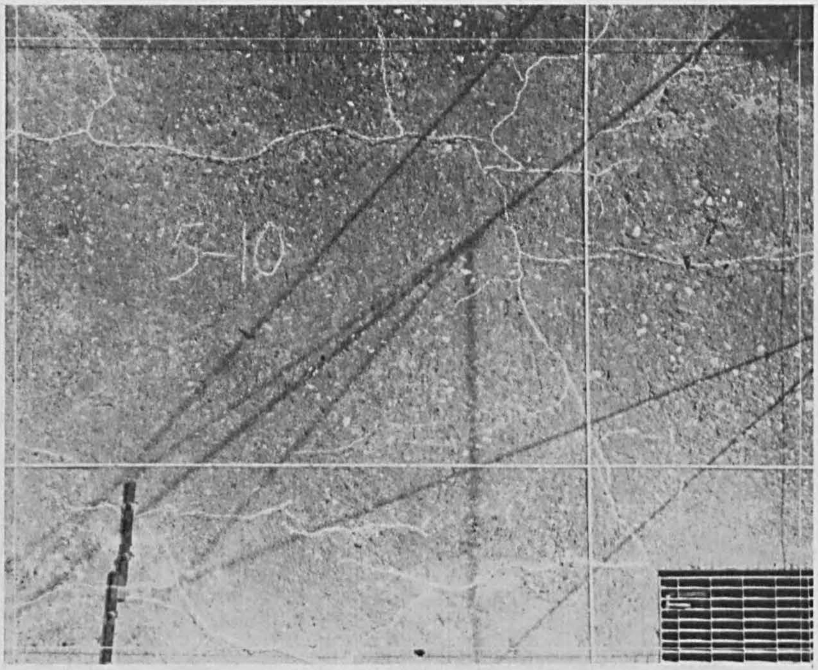


a. 1948



b. 1976

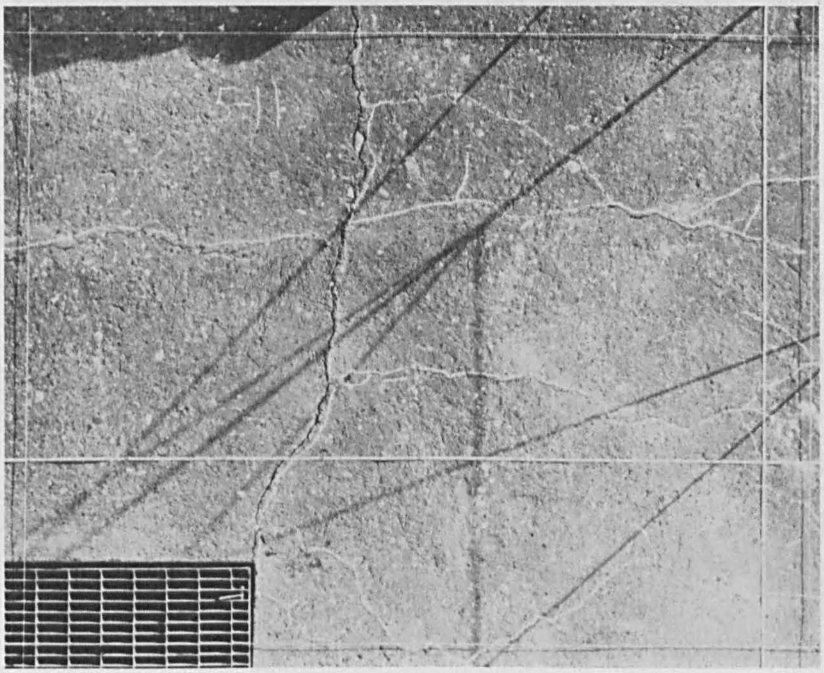
Fig. 27 Top surface, Monolith No. 5.



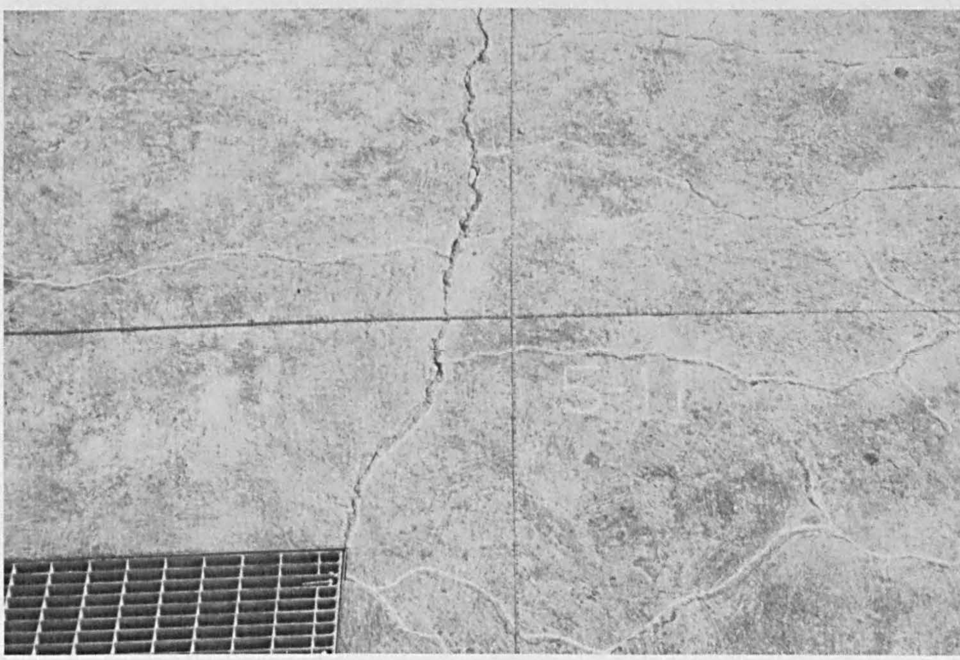
a. 1948

b. 1976

Fig. 28 Top surface, Monolith No. 5.

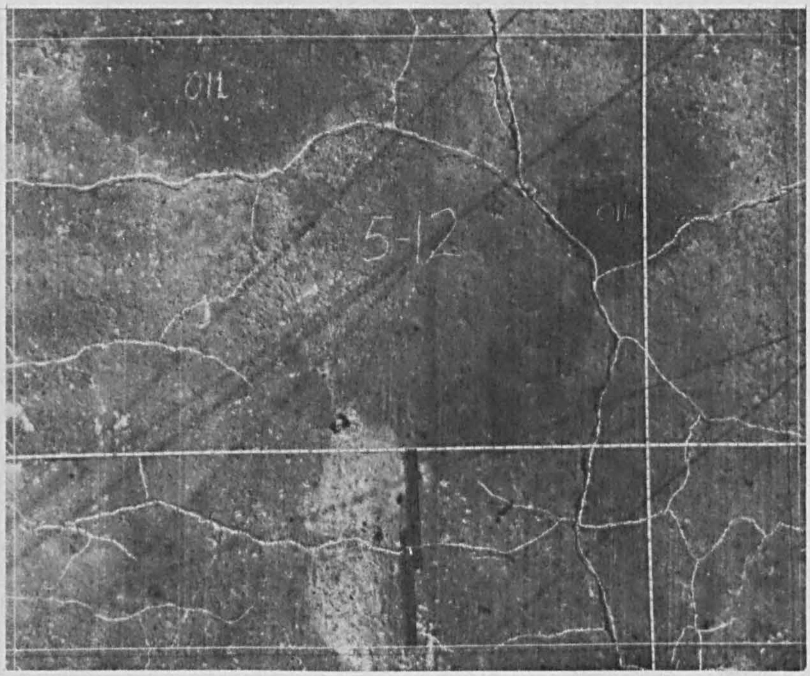


a. 1948

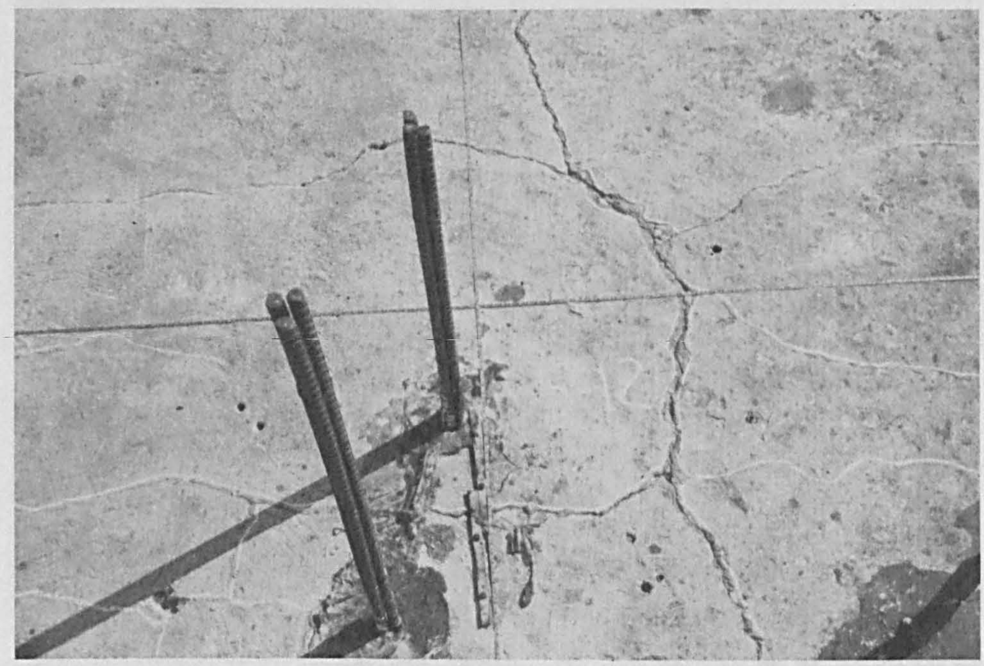


b. 1976

Fig. 29 Top surface, Monolith No. 5.

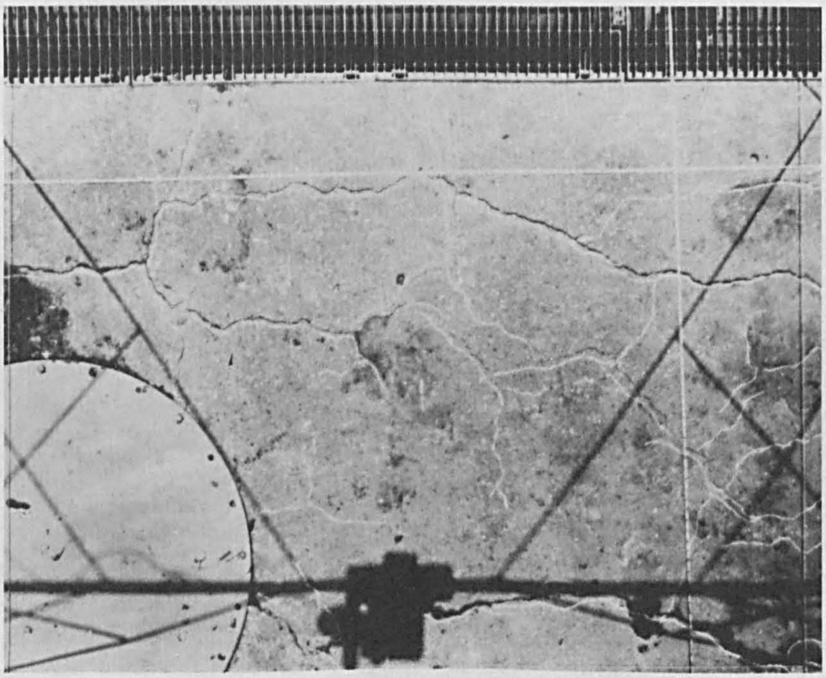


a. 1948



b. 1976

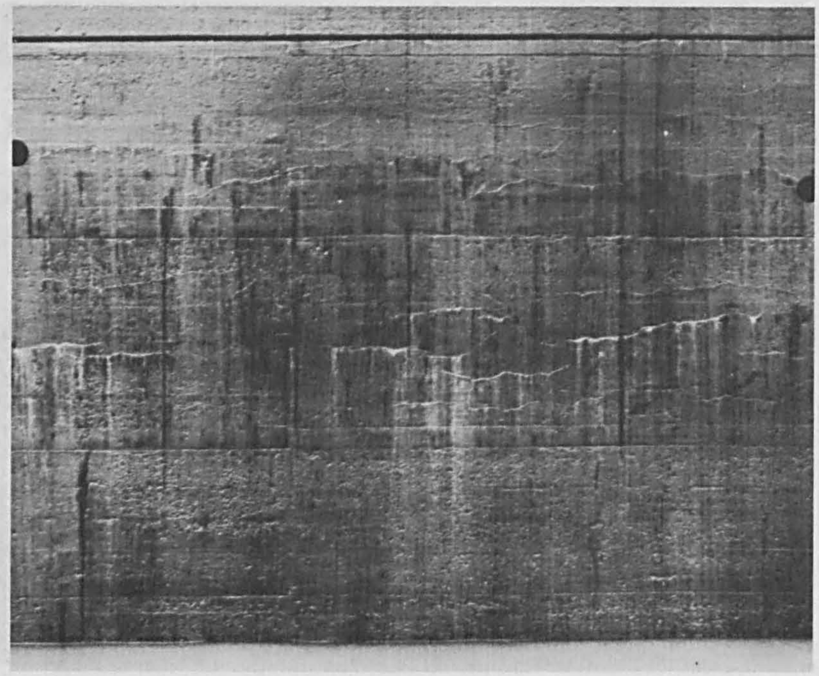
Fig. 30 Top surface, Monolith No. 5.



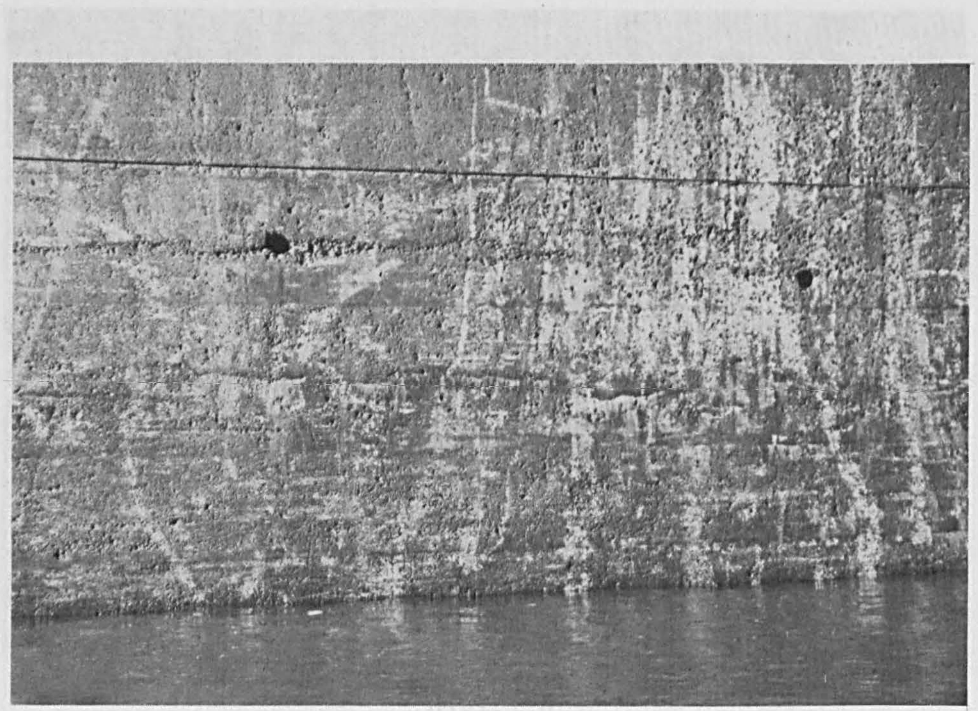
a. 1948

b. 1976

Fig. 31 Top surface, Monolith No. 5.



a. 1948



b. 1976

Fig. 32 Monolith No. 60, lock chamber face.

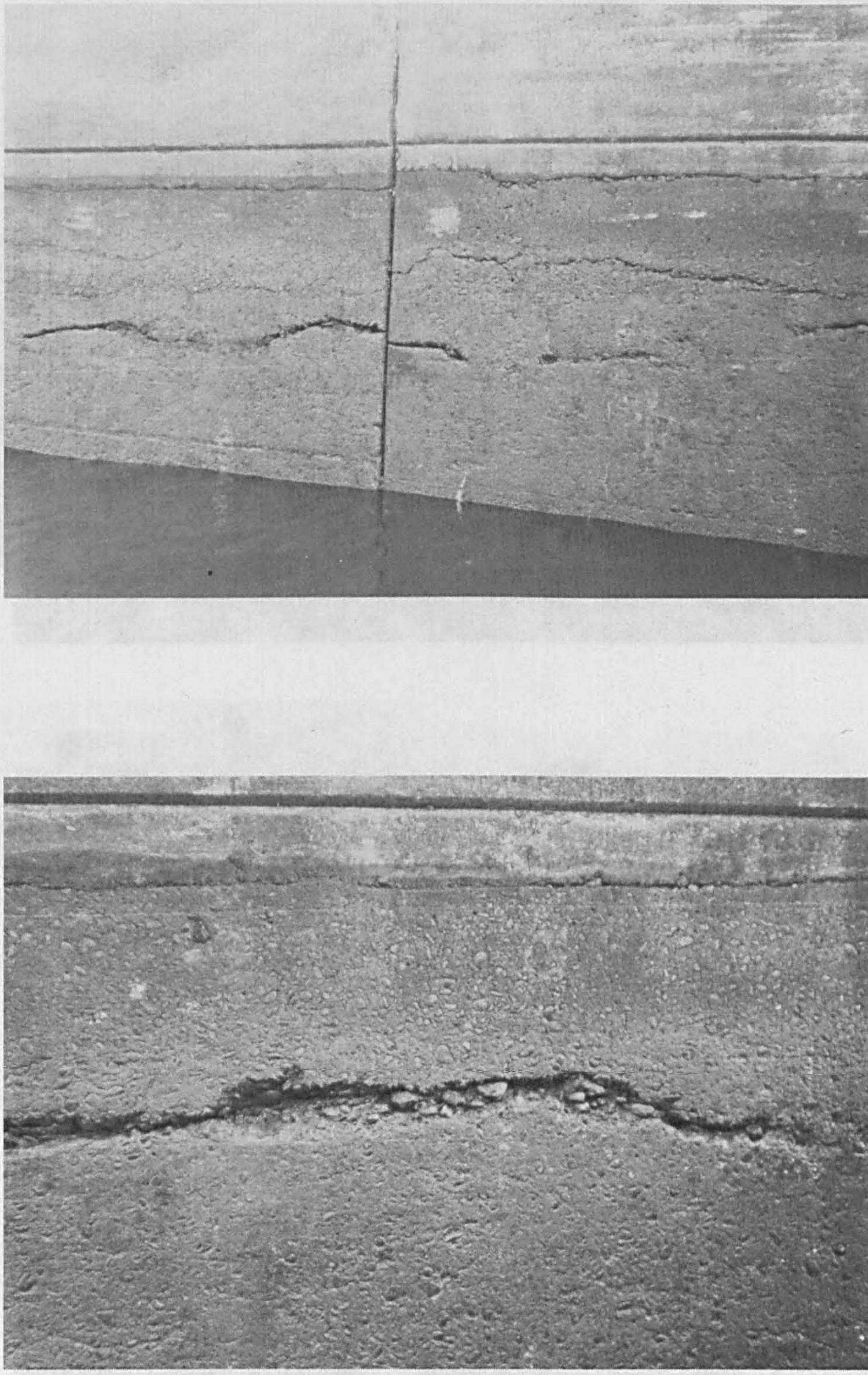
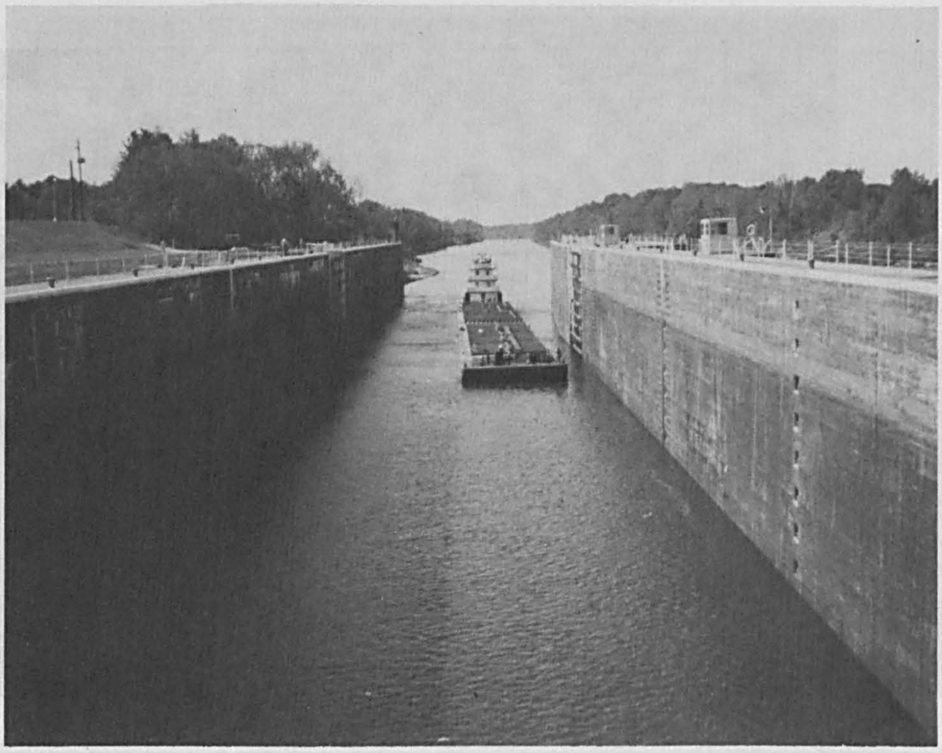


Fig.35 Land wall, lock chamber face.



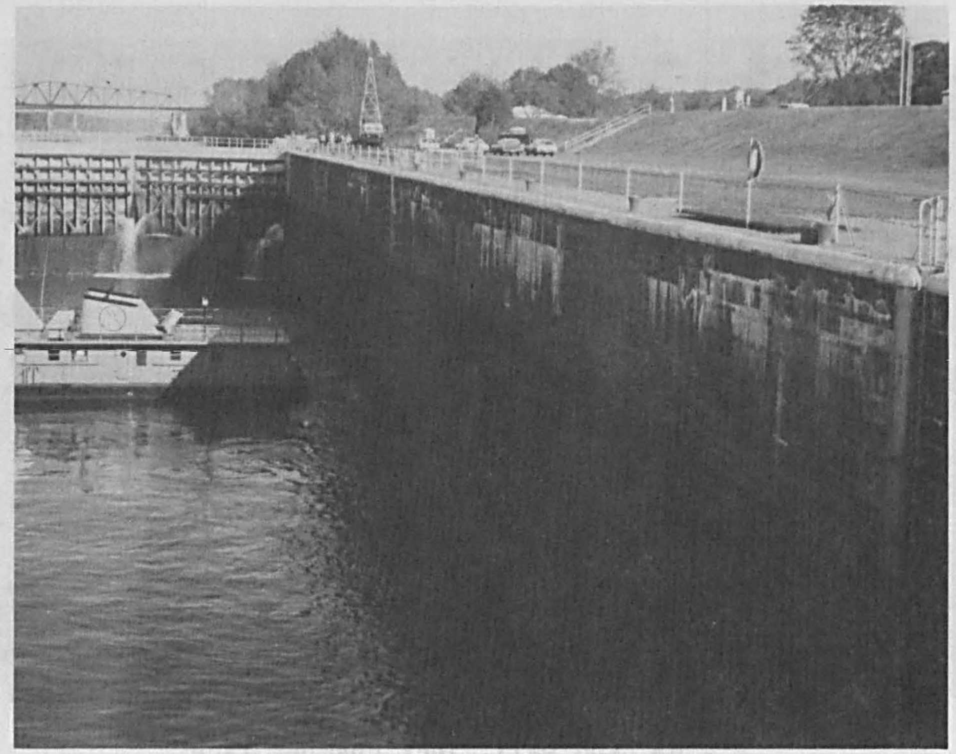
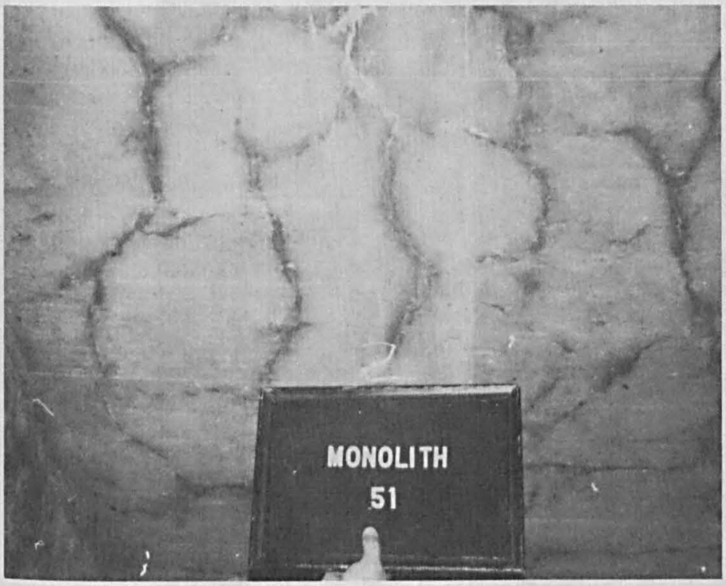


Fig. 34 General condition of lock chamber faces.



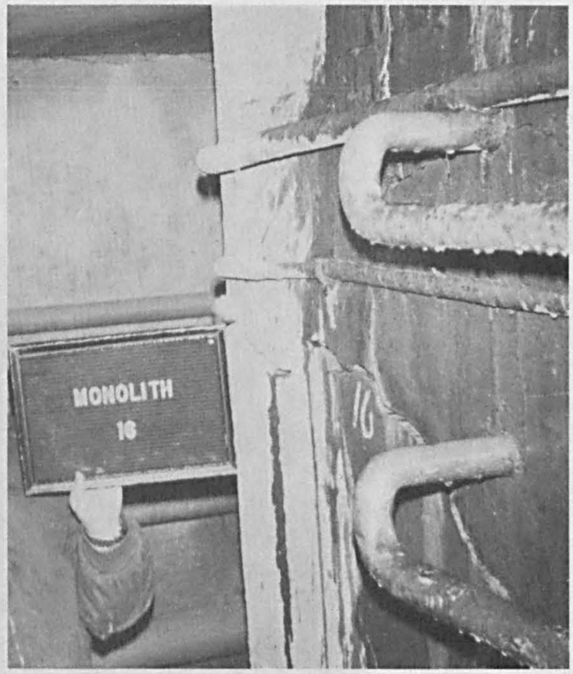
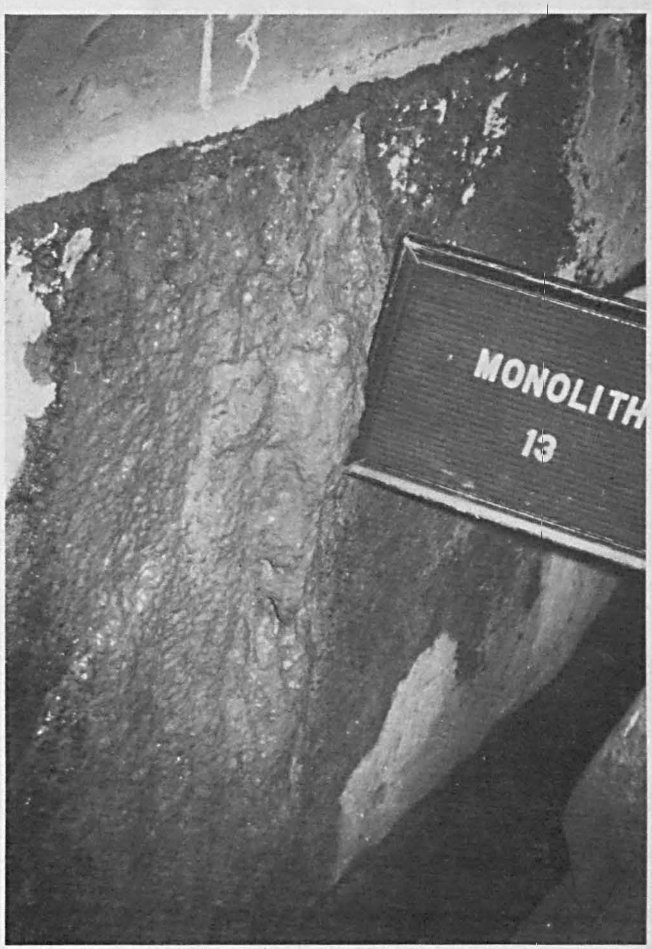


Fig. 35 Examples of cracking in inspection galleries.



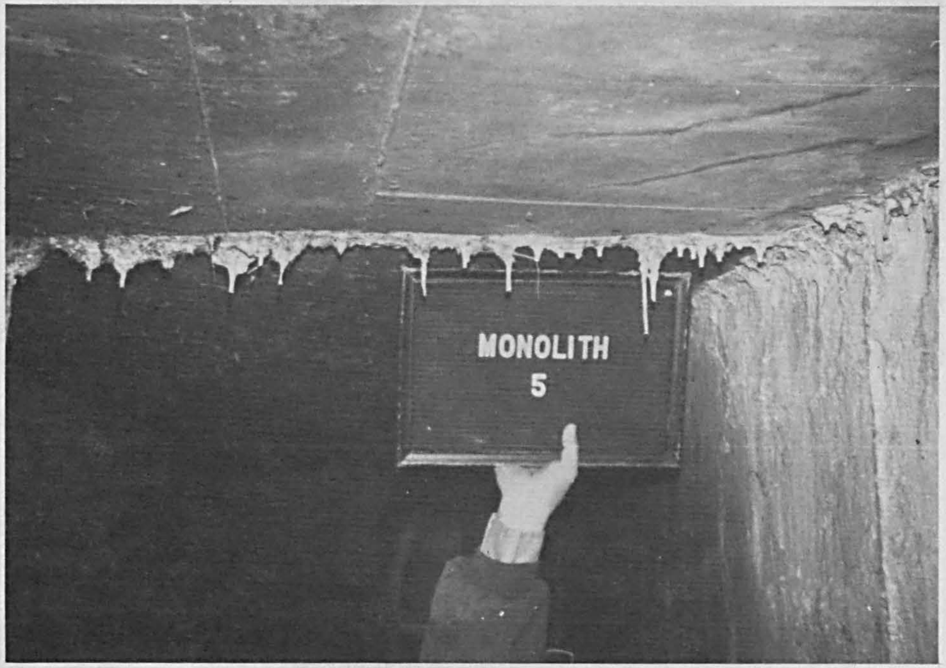
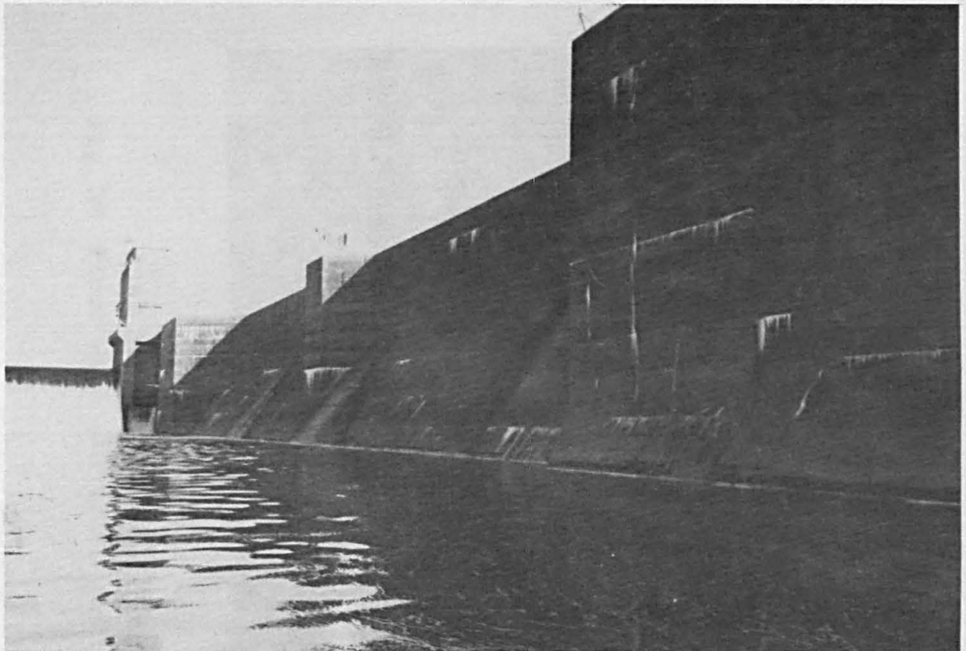
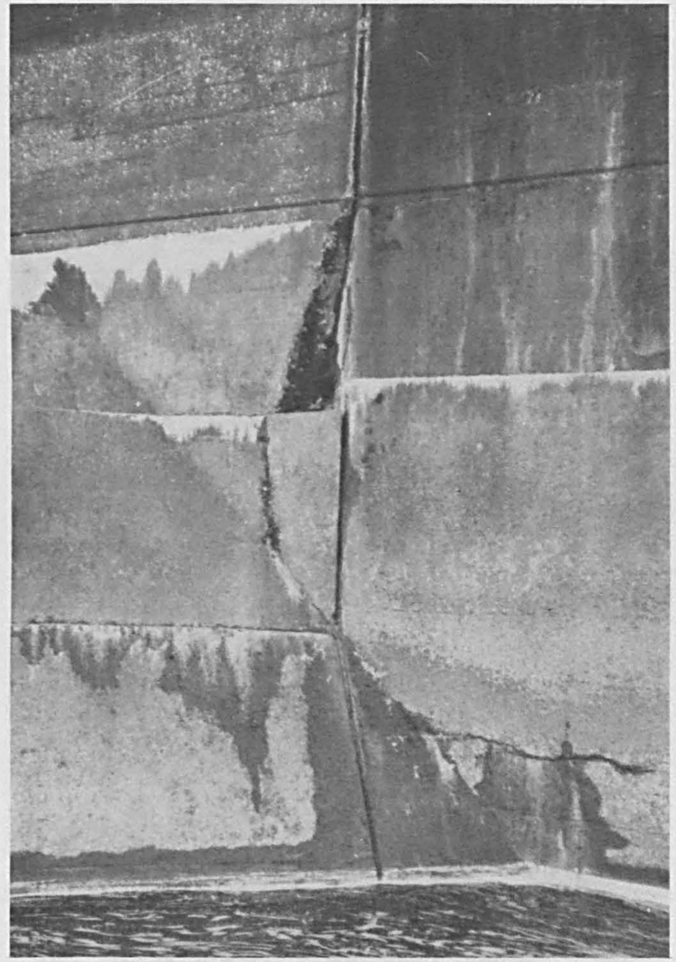


Fig.36 Typical deposits of gel resulting from chemical reaction.

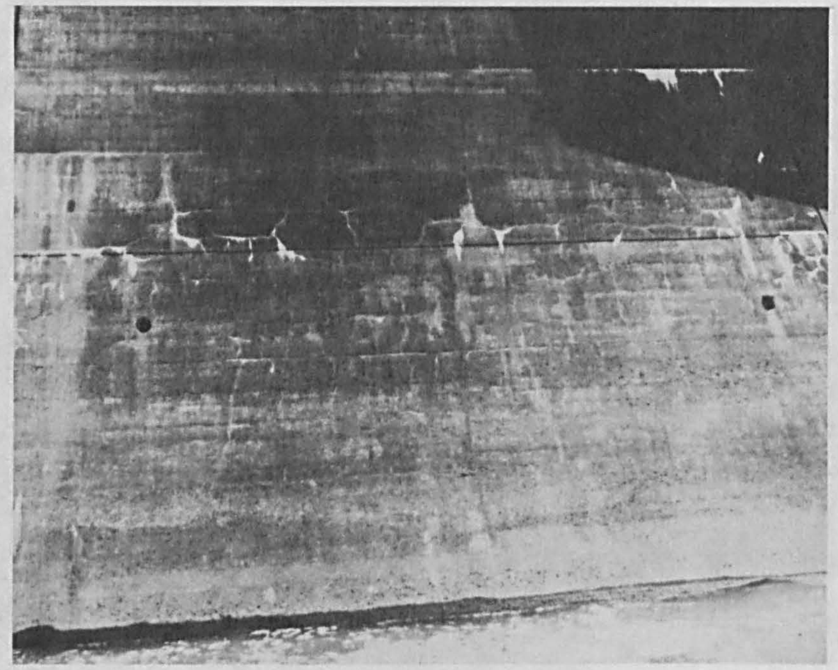


a. Overall

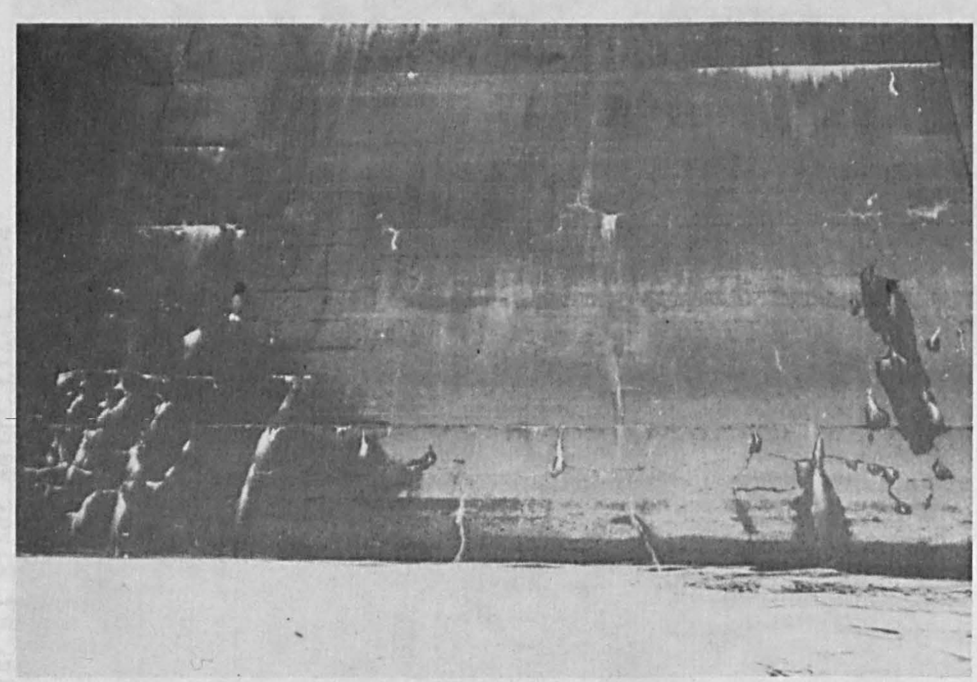


b. Joint between Monolith Nos. 65 and 66

Fig. 37 River wall, river face.

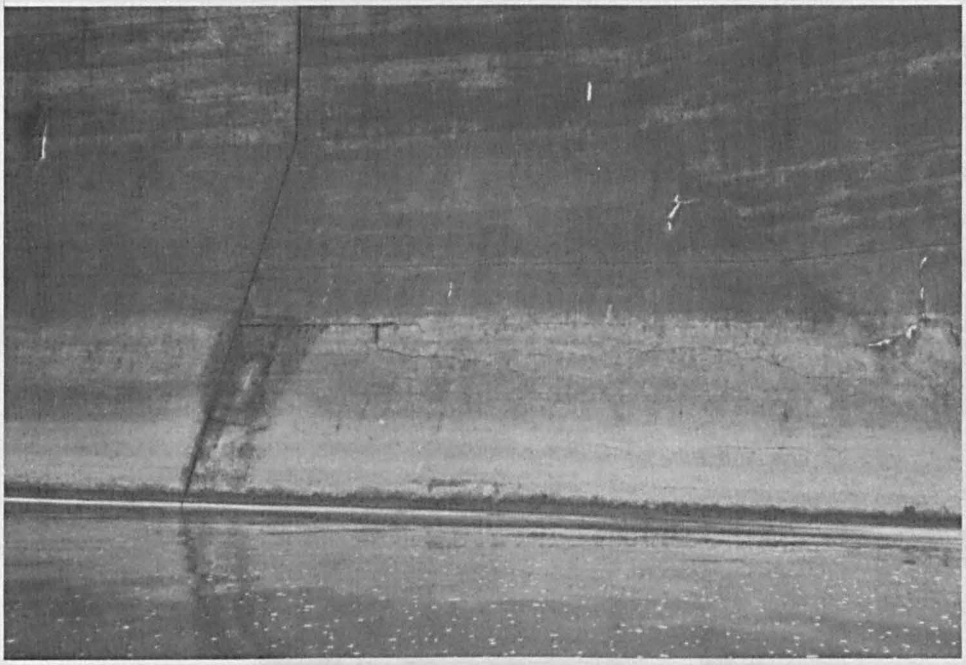


a. 1948

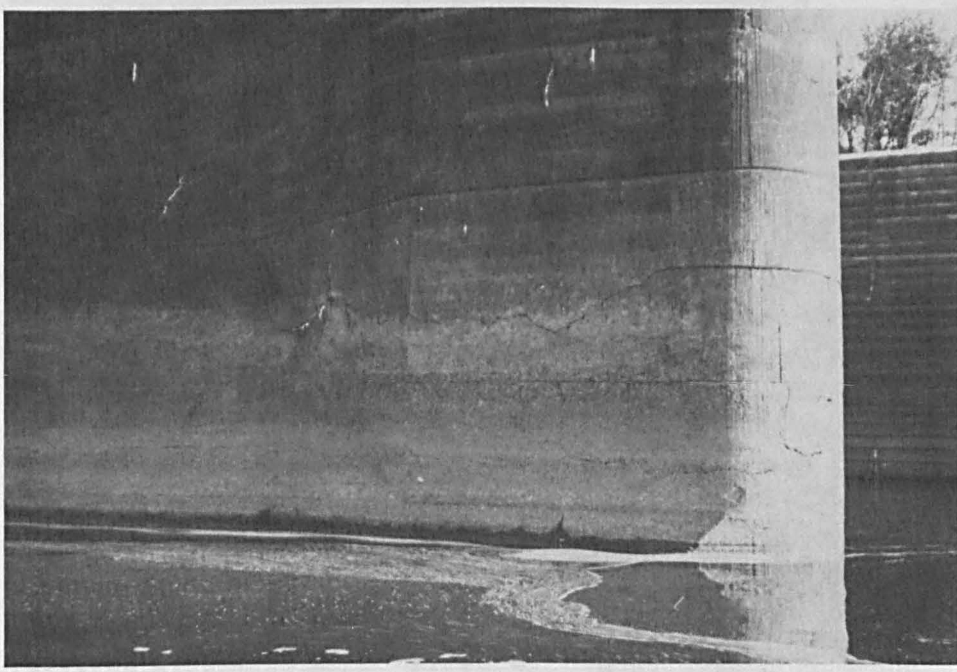


b. 1976

Fig. 38 Monolith No. 60, river face.

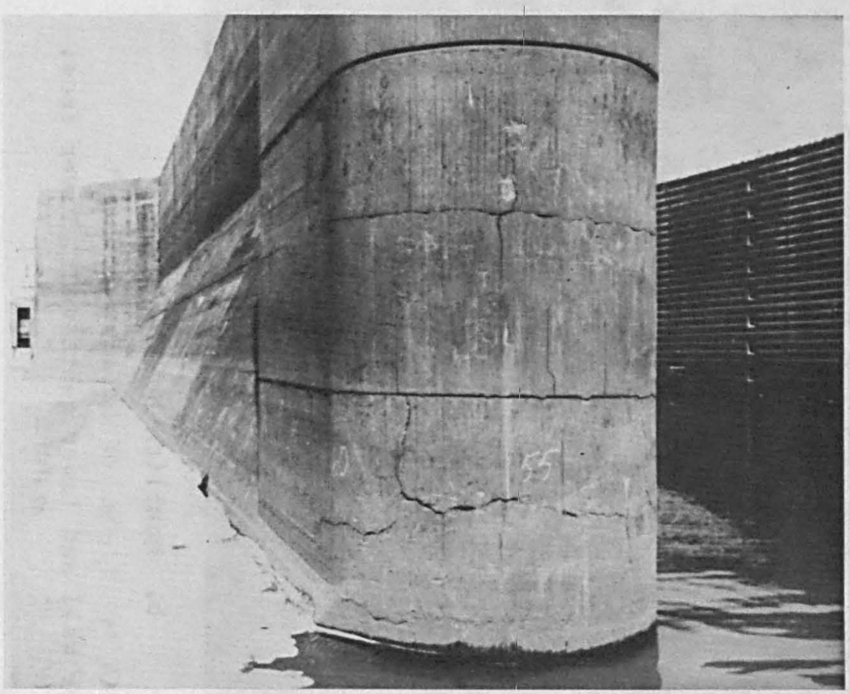


a. Monolith No. 65

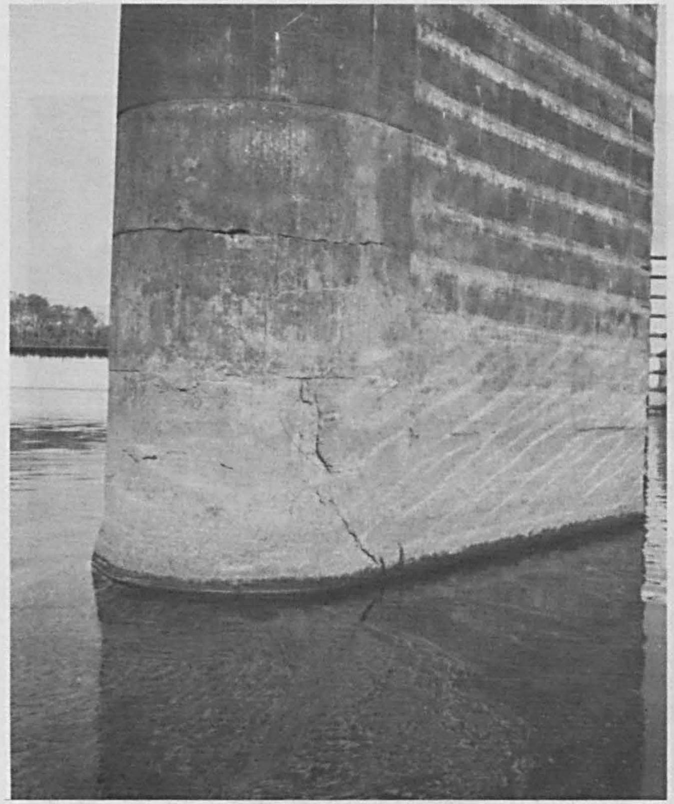


b. Monolith No. 74

Fig. 39 River wall and lower guard wall, river face.

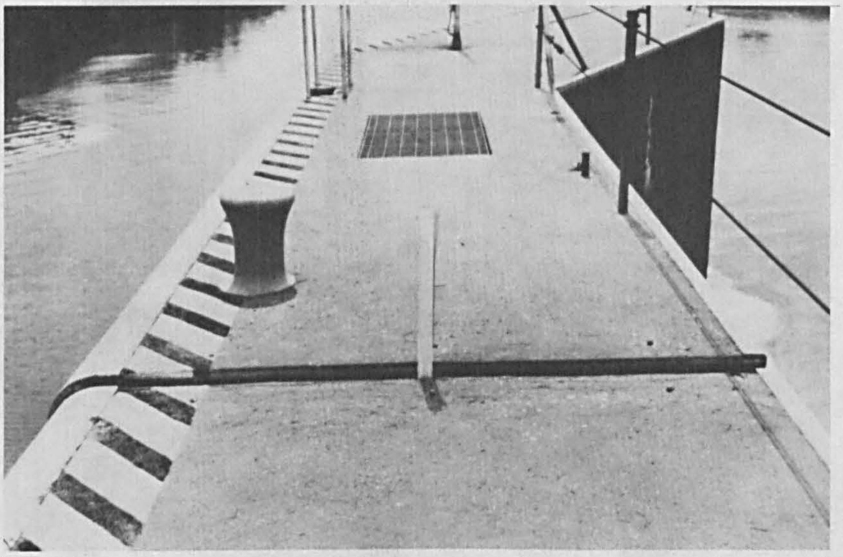


a. 1948

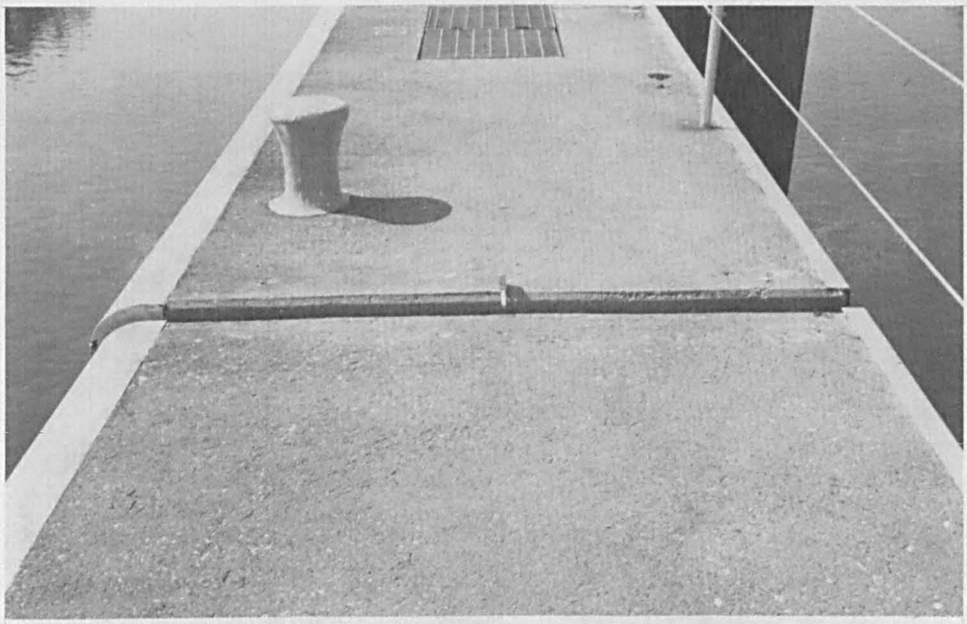


b. 1976

Fig. 40 Nosing, lower guard wall, Monolith No. 74.

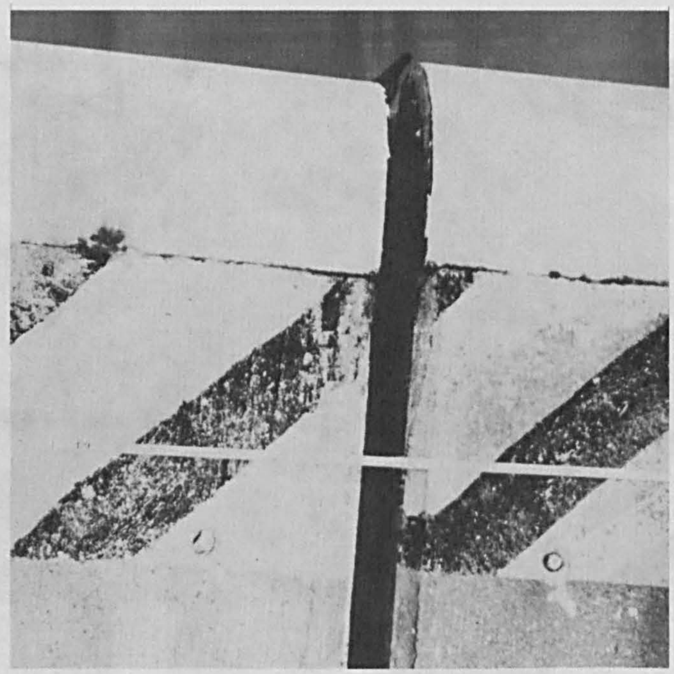


a. 1948

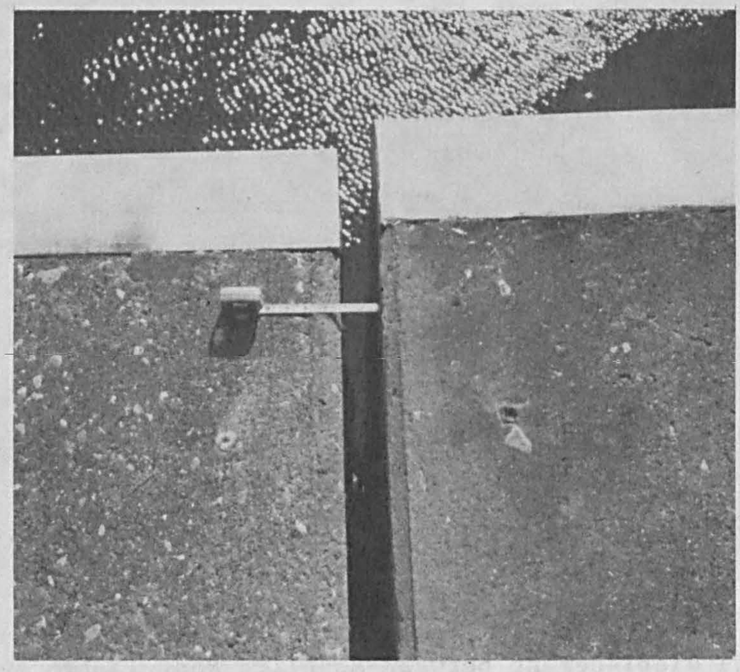


b. 1976

Fig. 41 Vertical displacement between Monolith Nos. 73 (foreground) and 74.



a. 1948



b. 1976

Fig. 42 Longitudinal displacement between Monolith Nos. 73 (left) and 74.

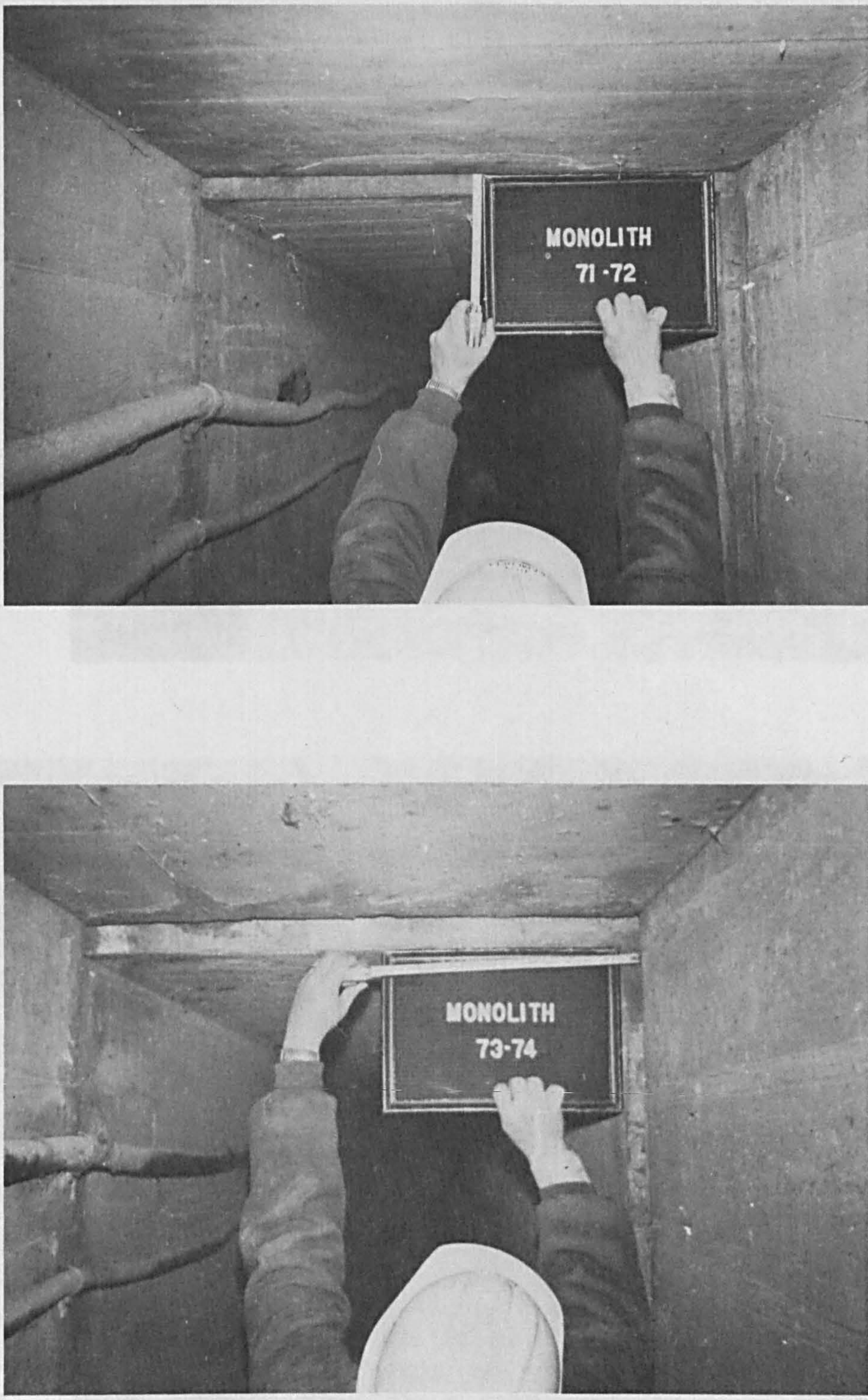
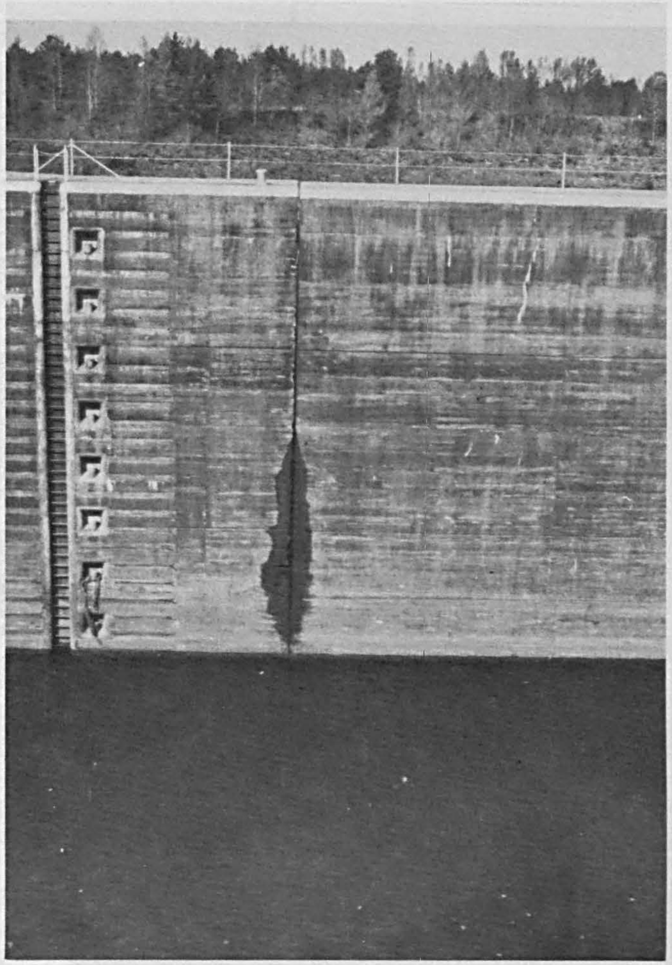
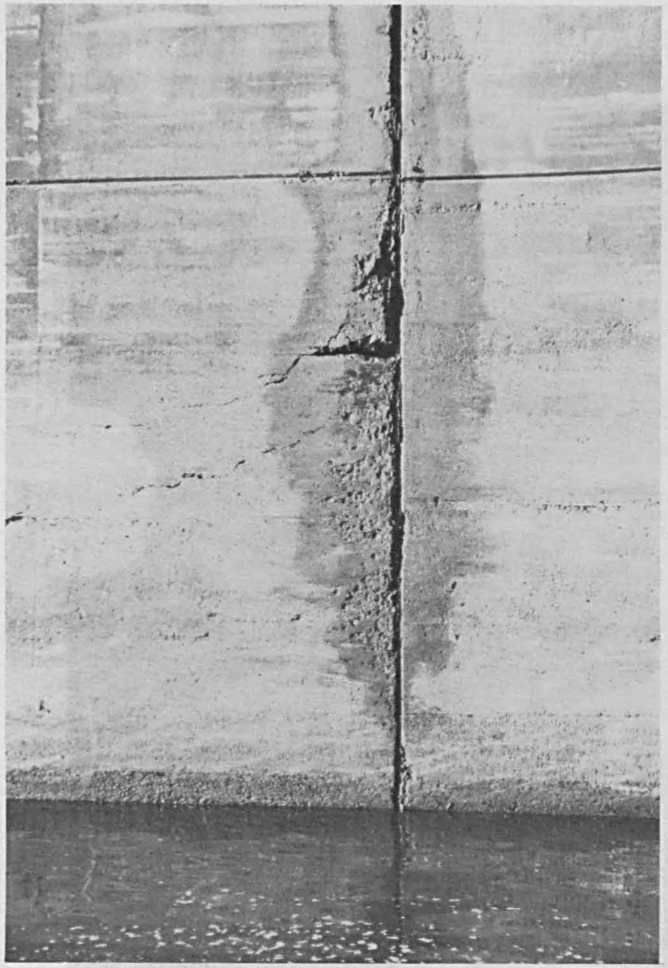


Fig. 43 Gallery displacements, lower guard wall.

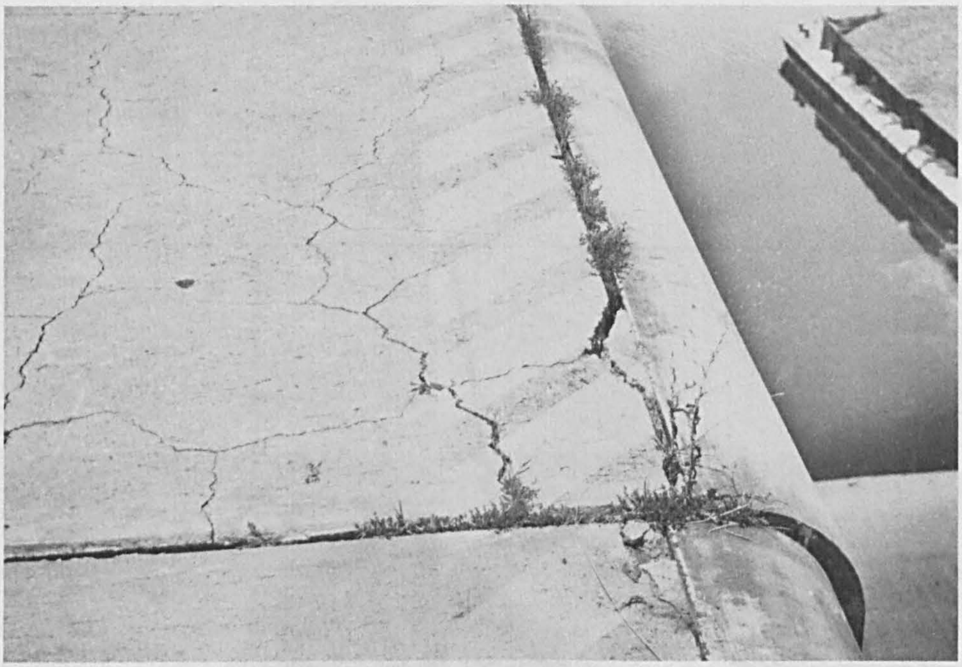


a. Overall

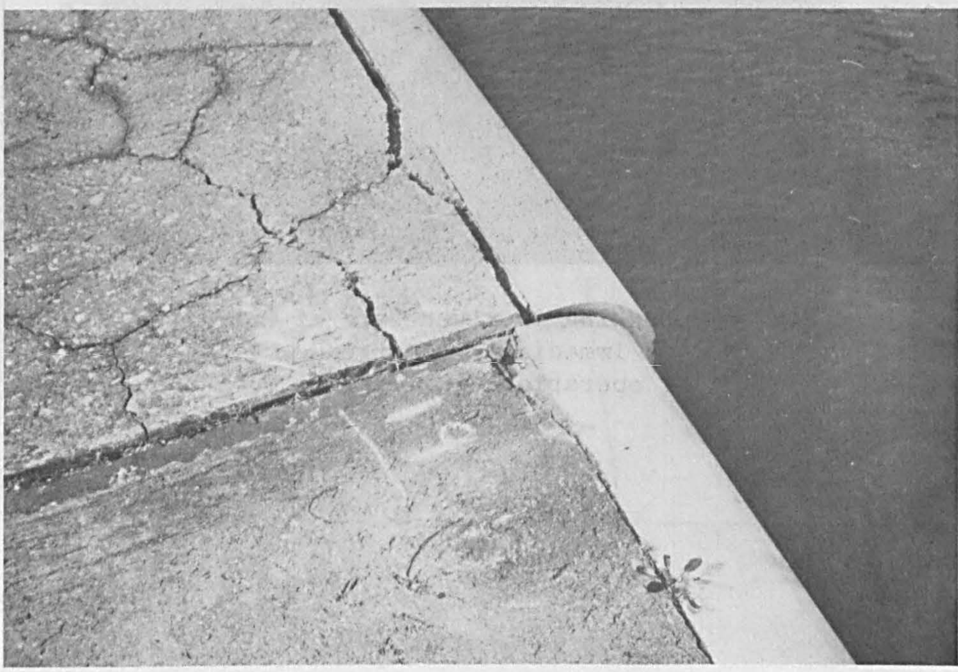


b. Closeup

Fig. 44 Deterioration of joint between Monolith Nos. 73 and 74.



a. 1954



b. 1976

Fig 45 Displacement Between Monolith Nos. 19 & 20.

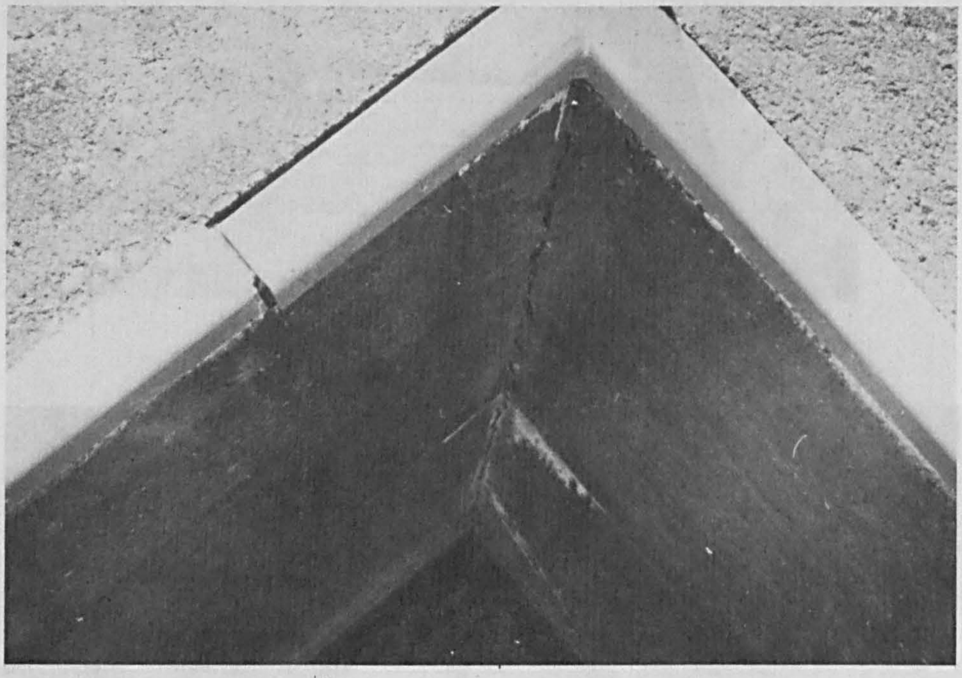


Fig. 46 Cracking in river face of Monolith No. 54 immediately upstream of operations building.

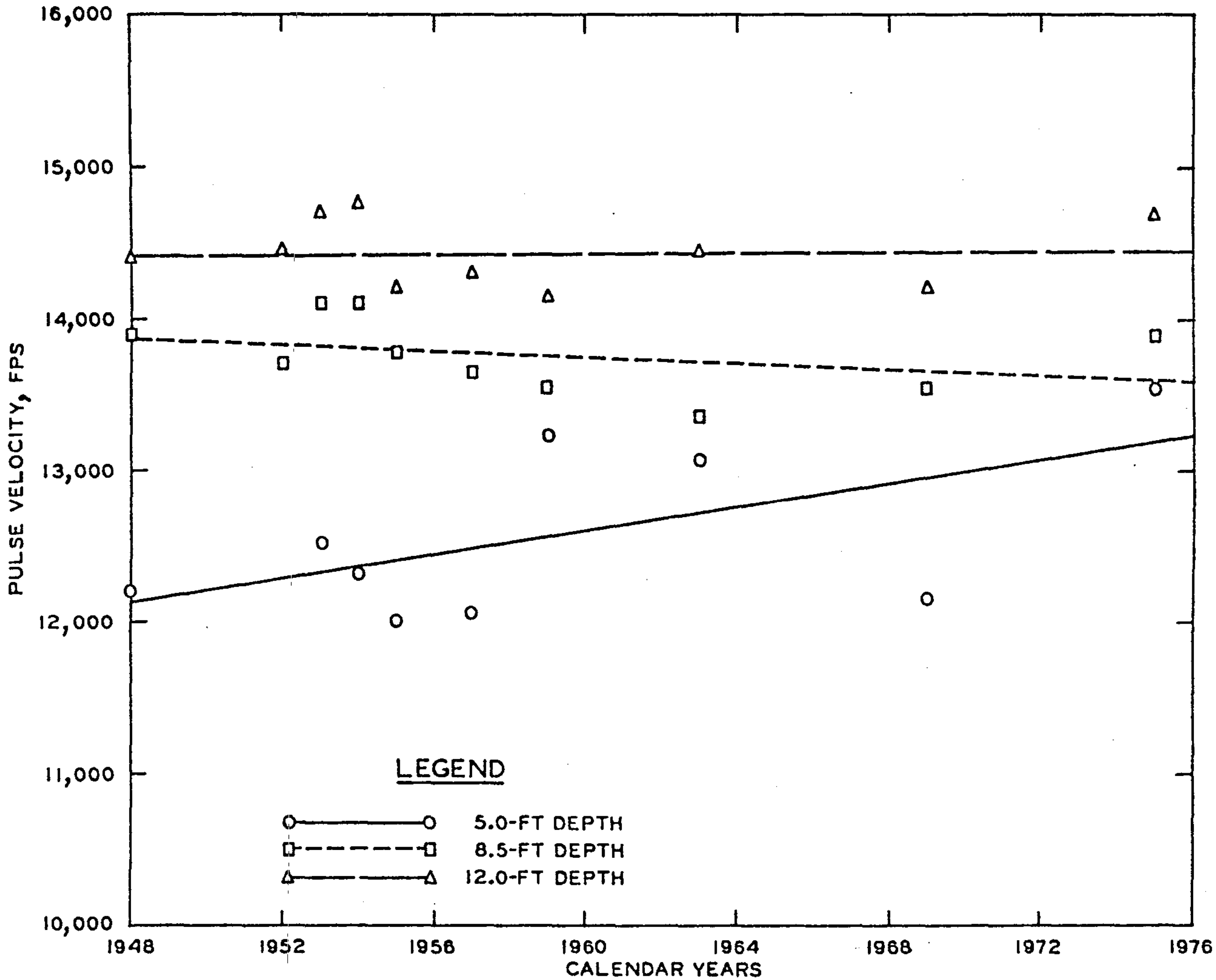


Figure 47. Concrete pulse velocity, monolith No. 5.

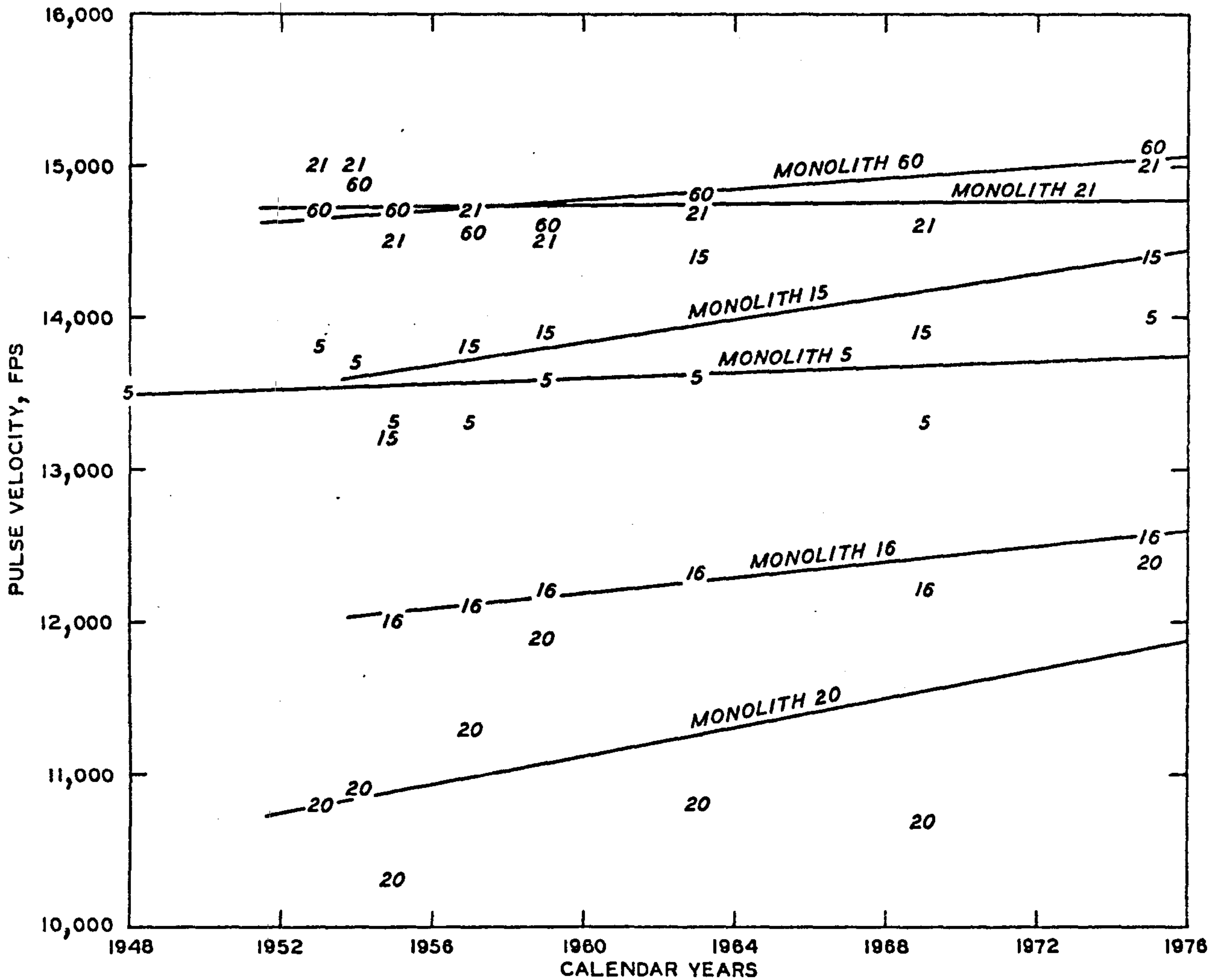


Figure 48. Pulse velocity tests.
1948-1975

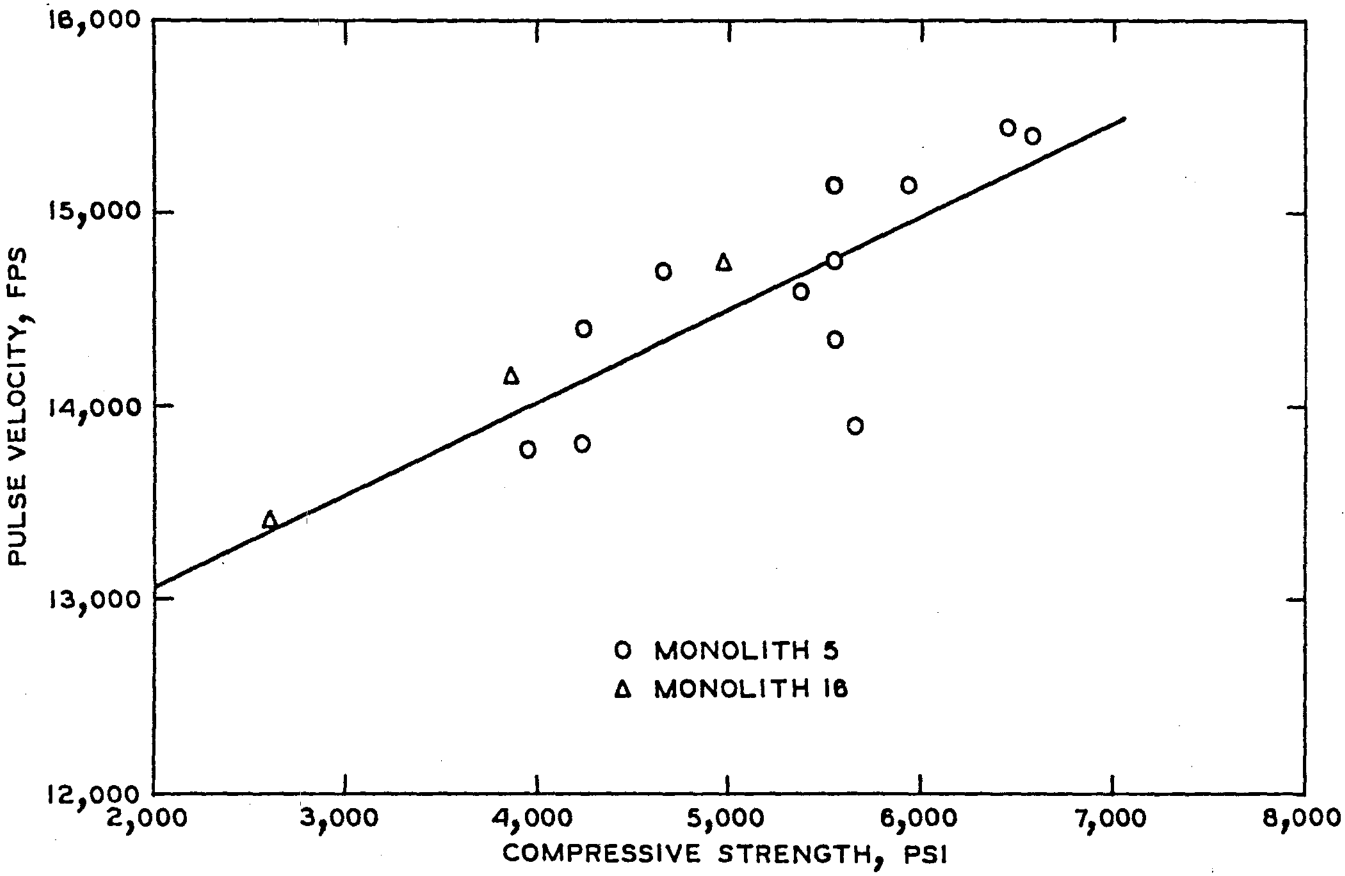


Figure 49. Relationship between pulse velocity and compressive strength - Series 1 tests.

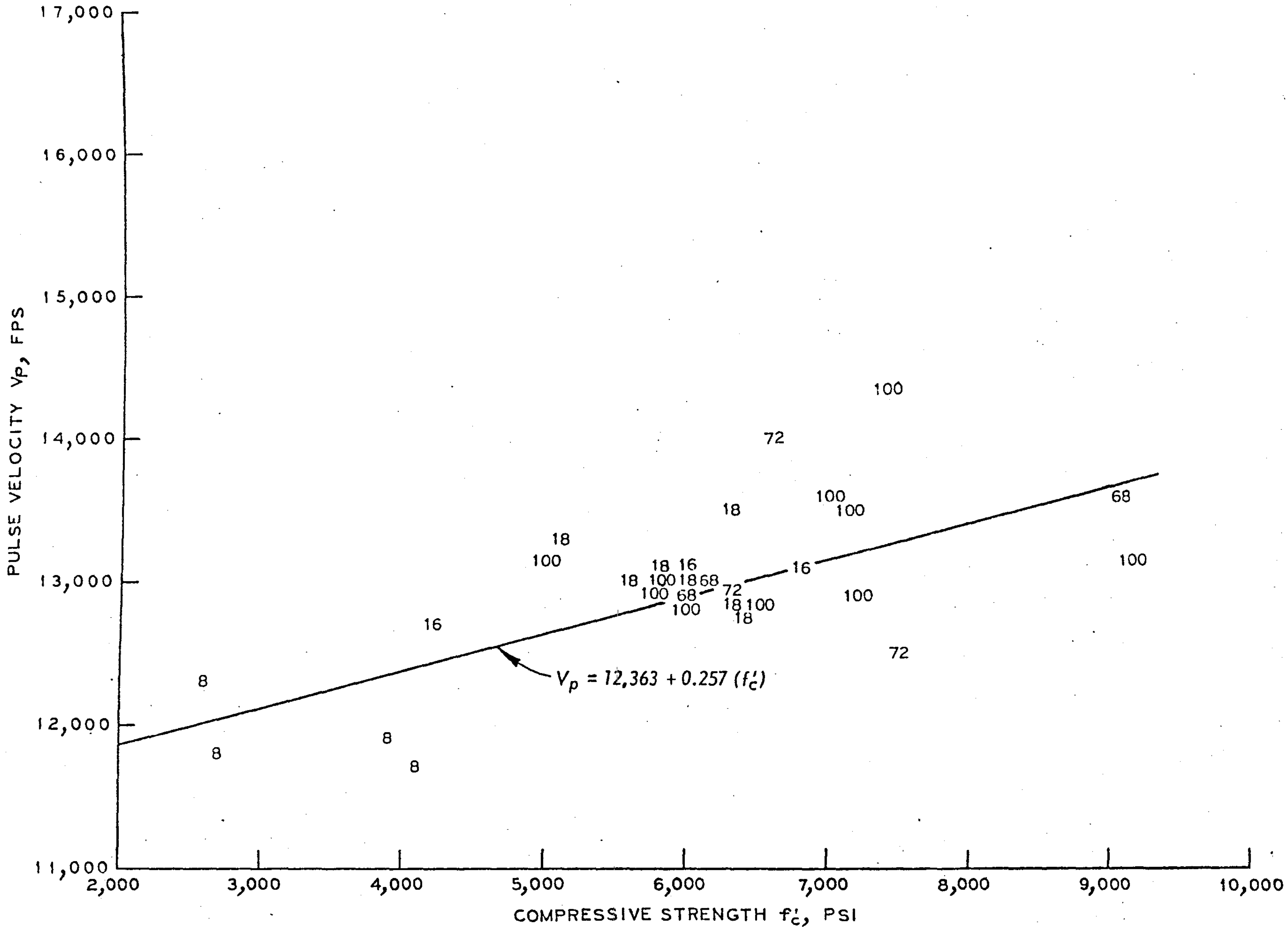


Figure 50. Relationship between pulse velocity and compressive strength for various monoliths - Series 2 tests.

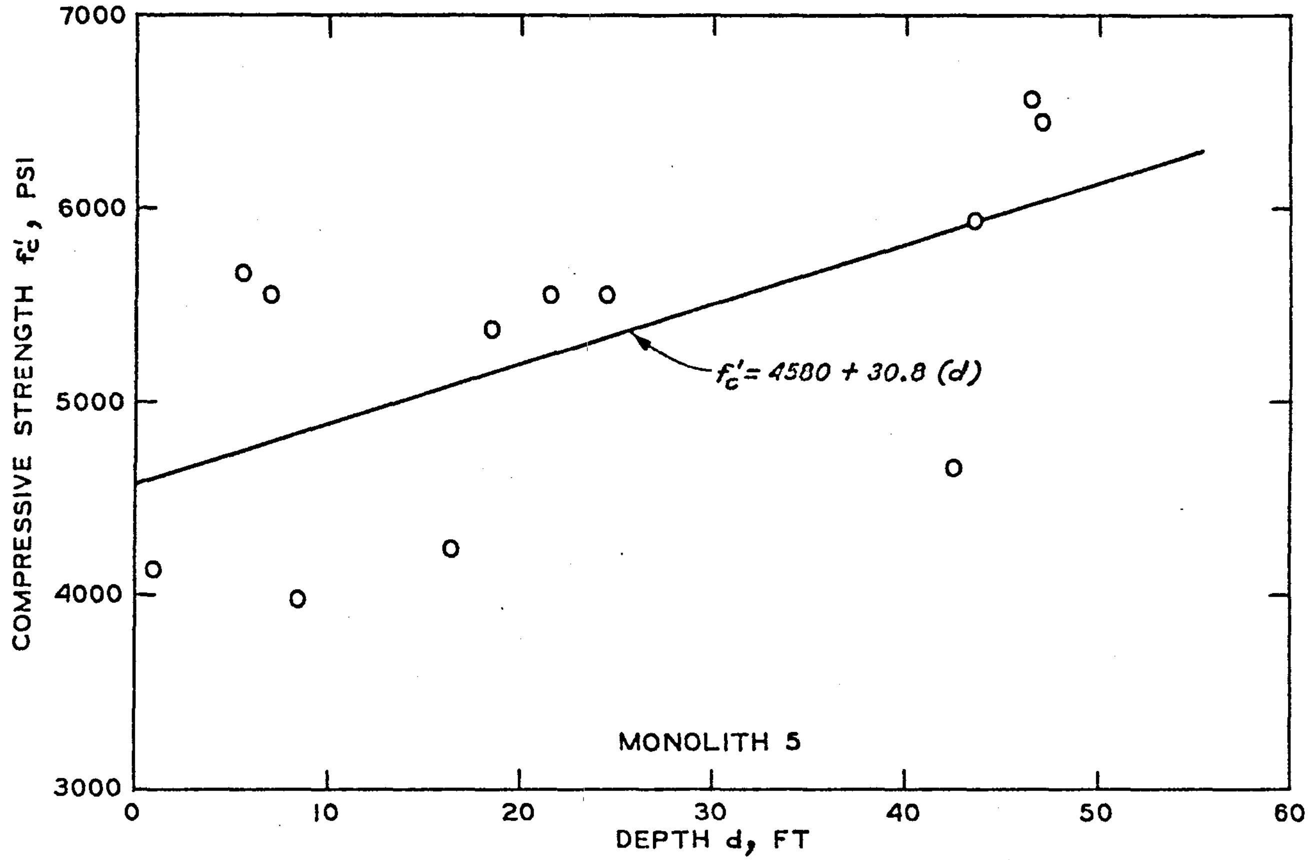


Figure 51. Variation in compressive strength with depth.

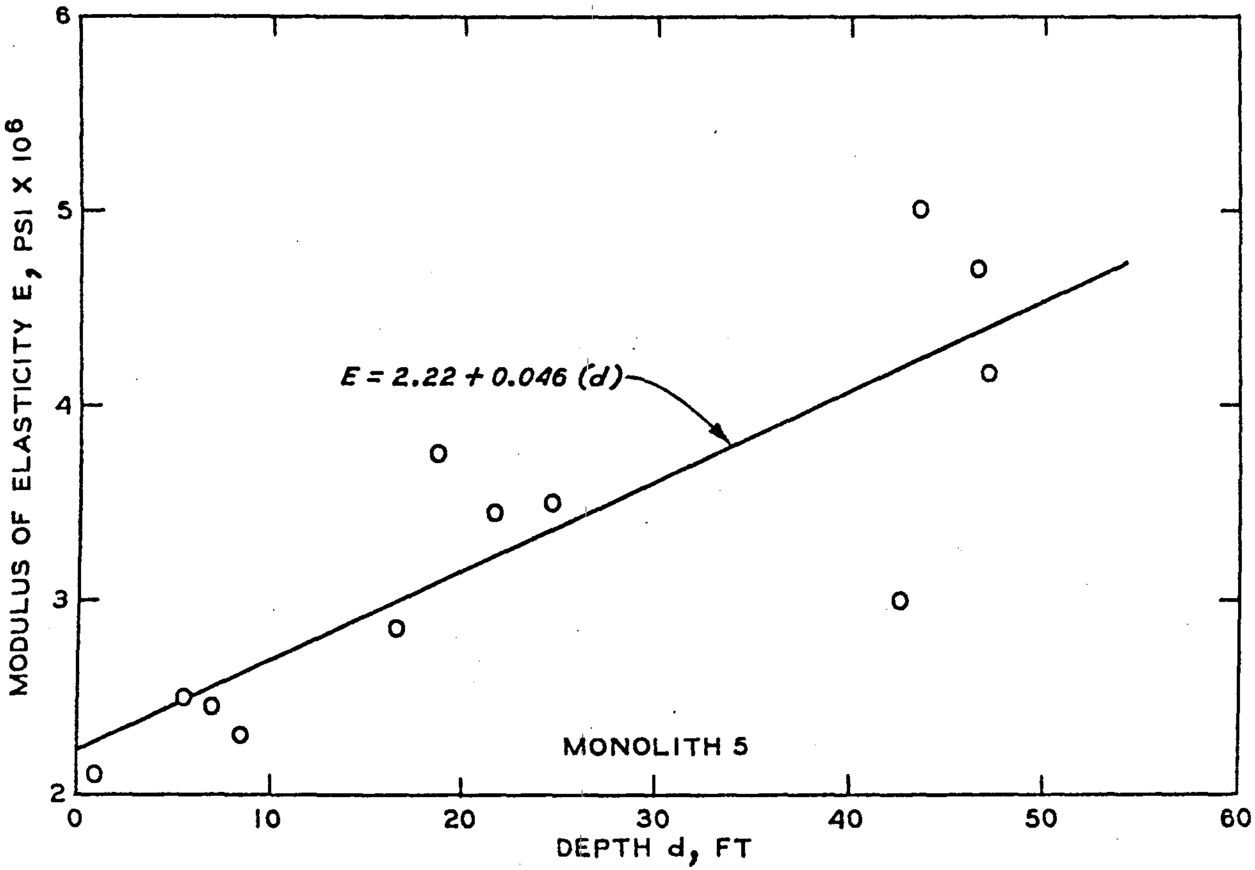


Figure 52. Relationship between modulus of elasticity and depth.

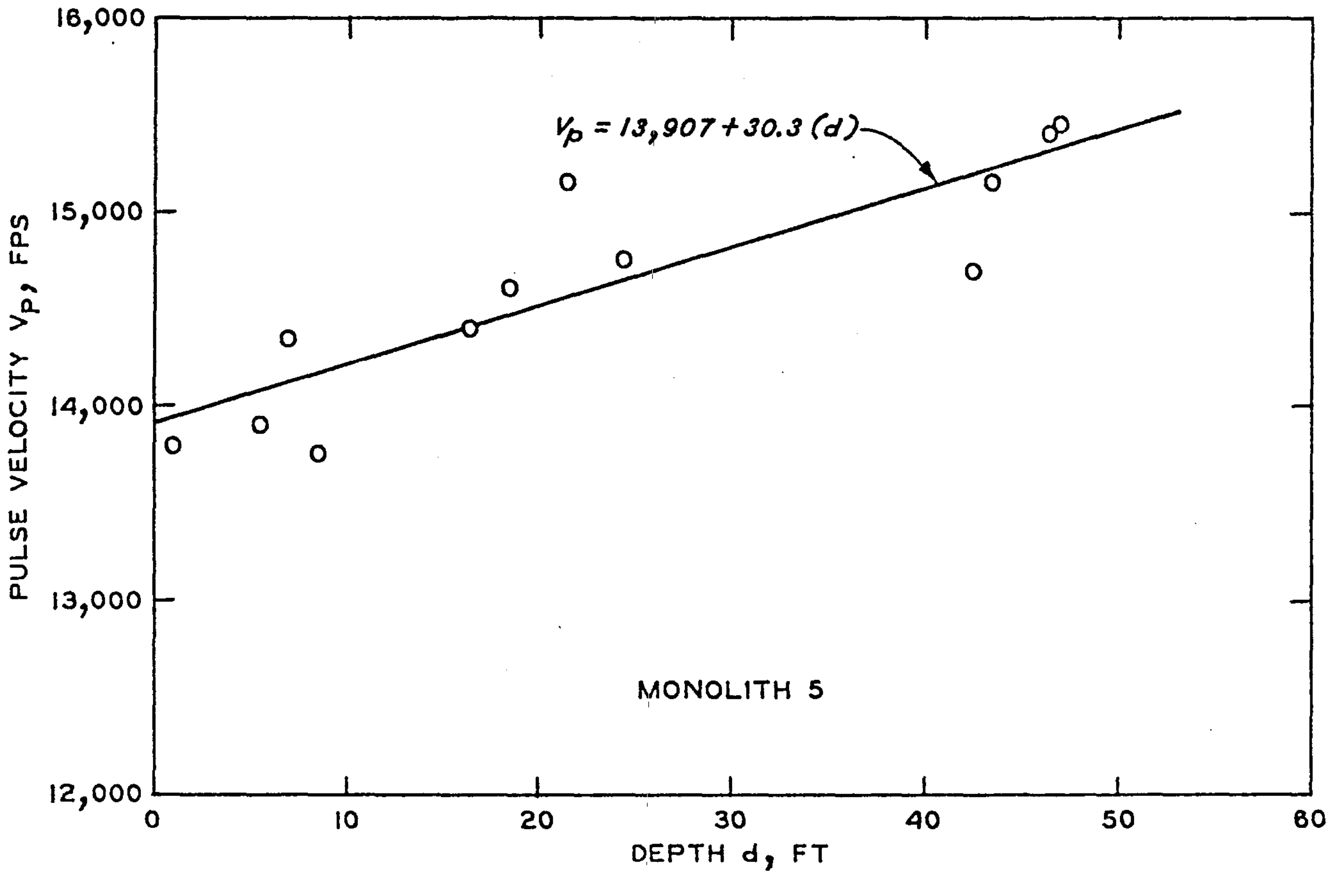


Figure 53. Relationship between pulse velocity and depth.

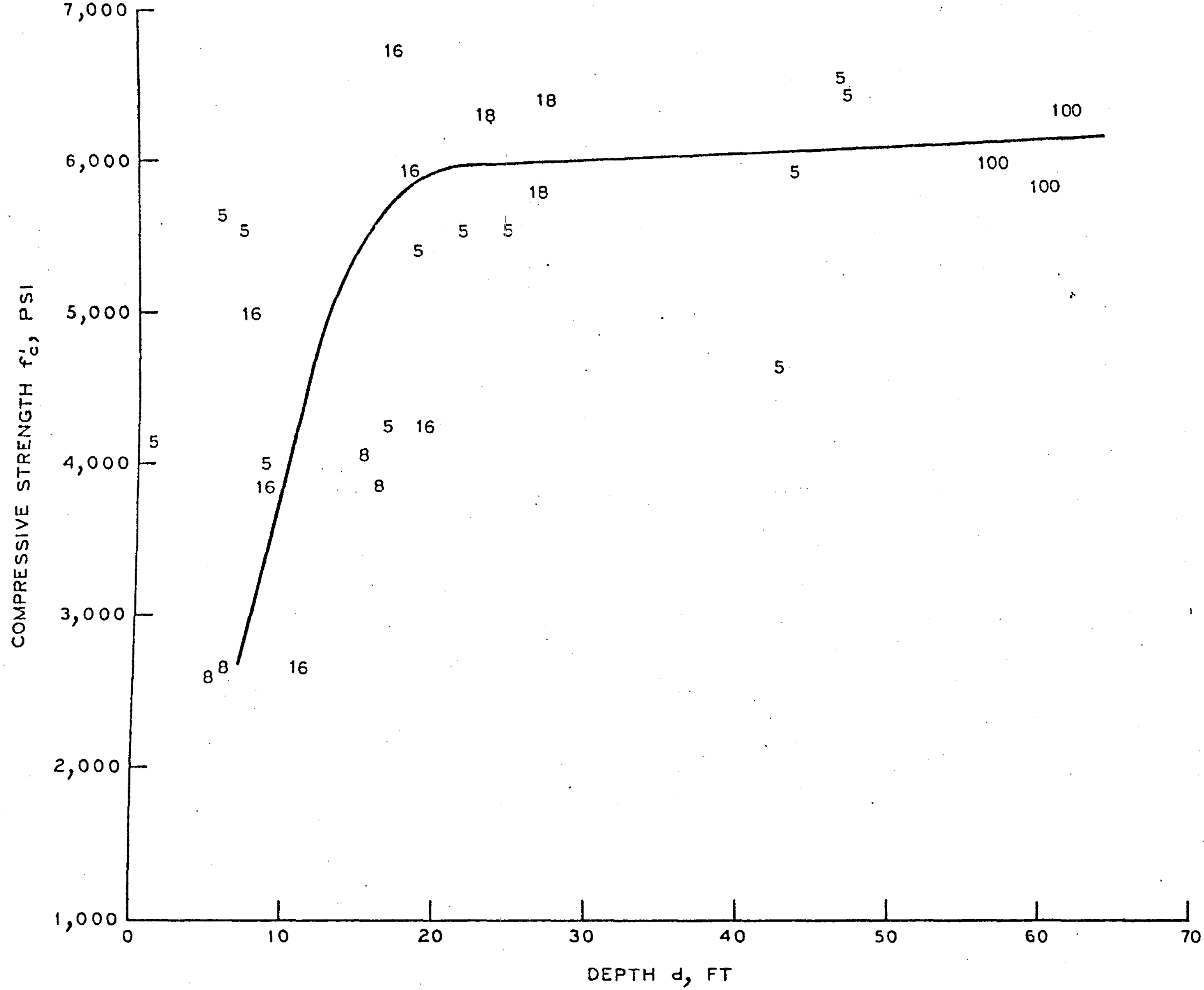


Figure 54. Variation in compressive strength with depth for various monoliths containing the higher alkali cement.

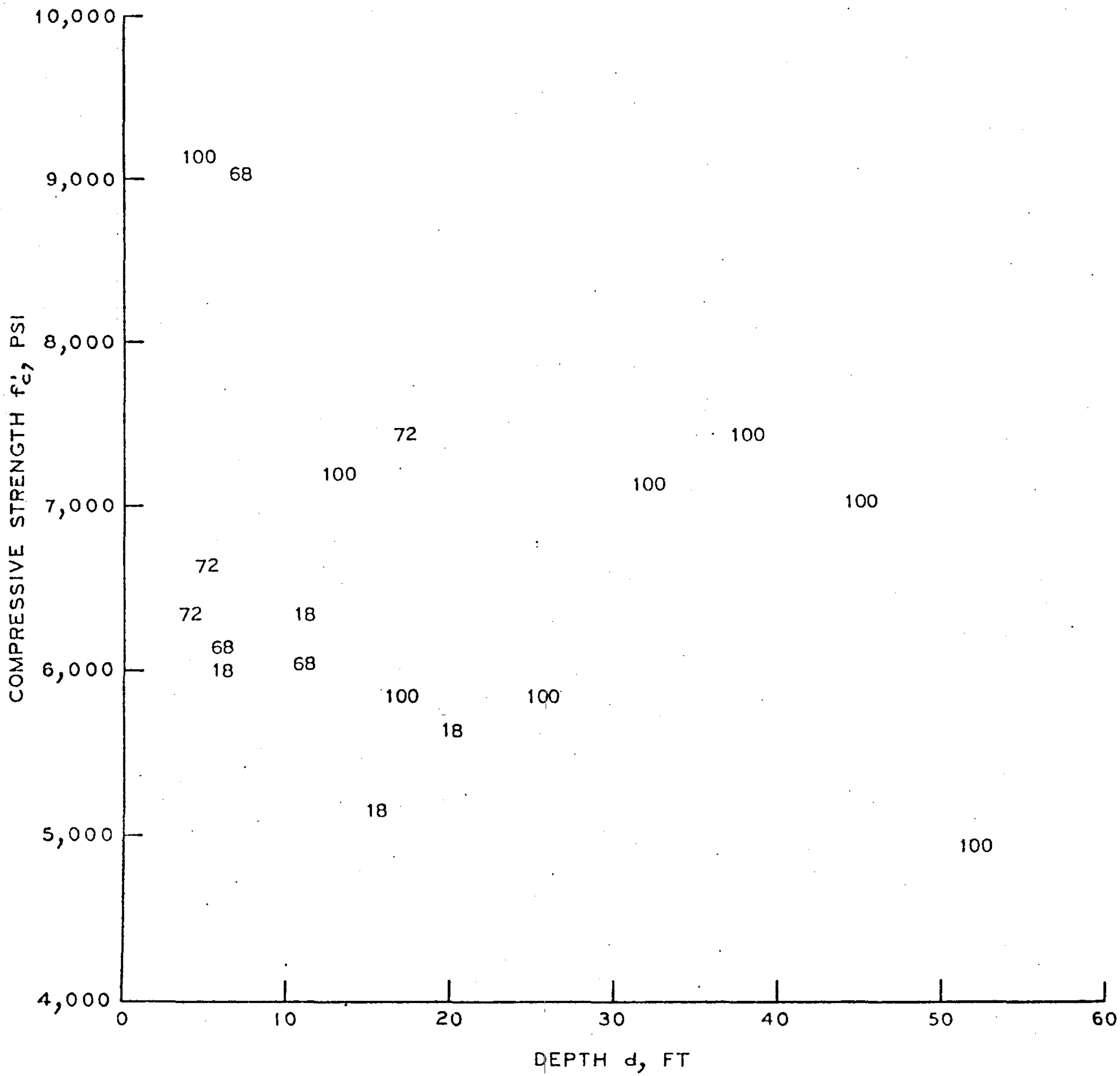


Figure 55. Variation in compressive strength with depth for various monoliths containing the lower alkali cement.

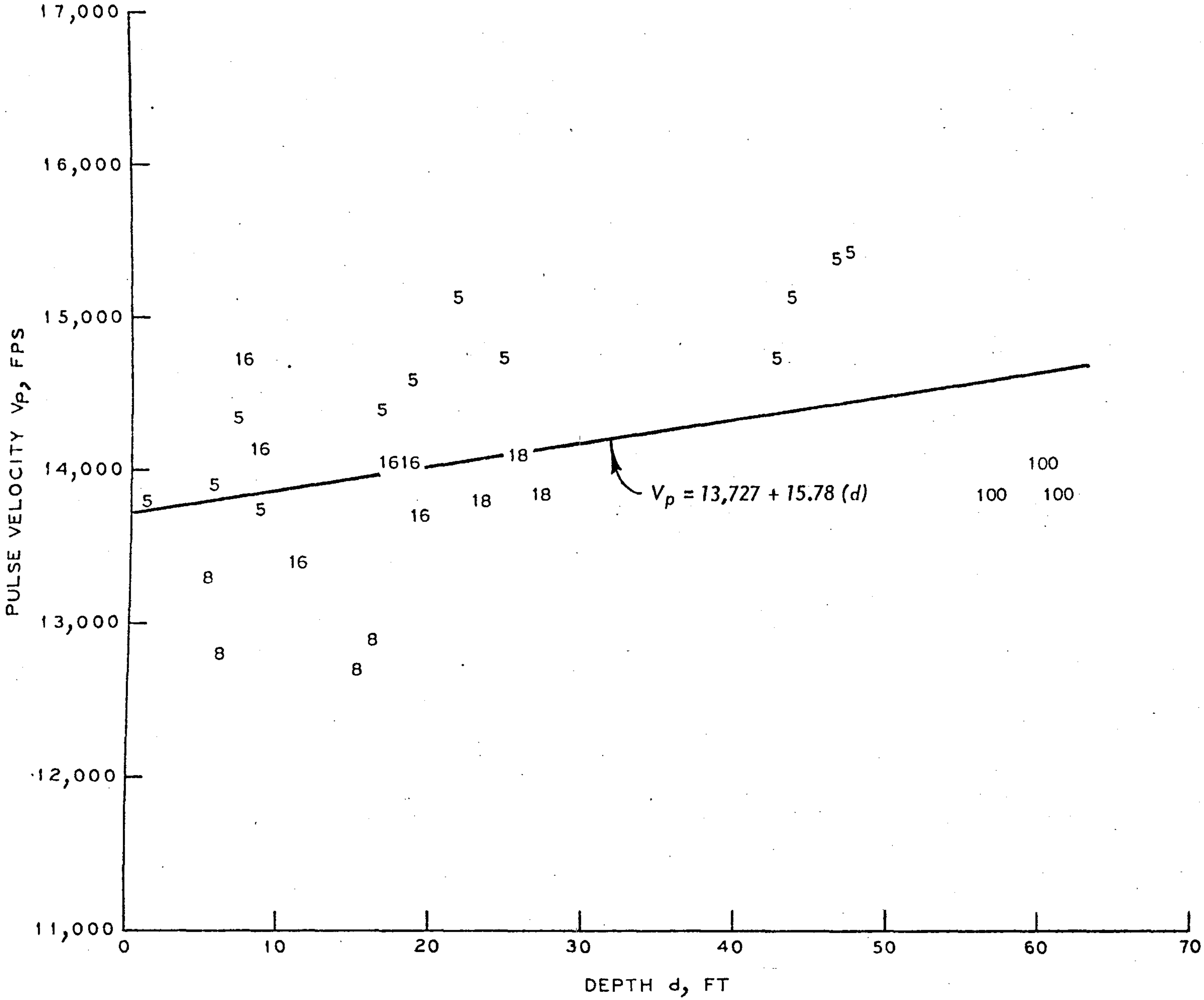


Figure 56. Variation in pulse velocity with depth for various monoliths containing the higher alkali cement.

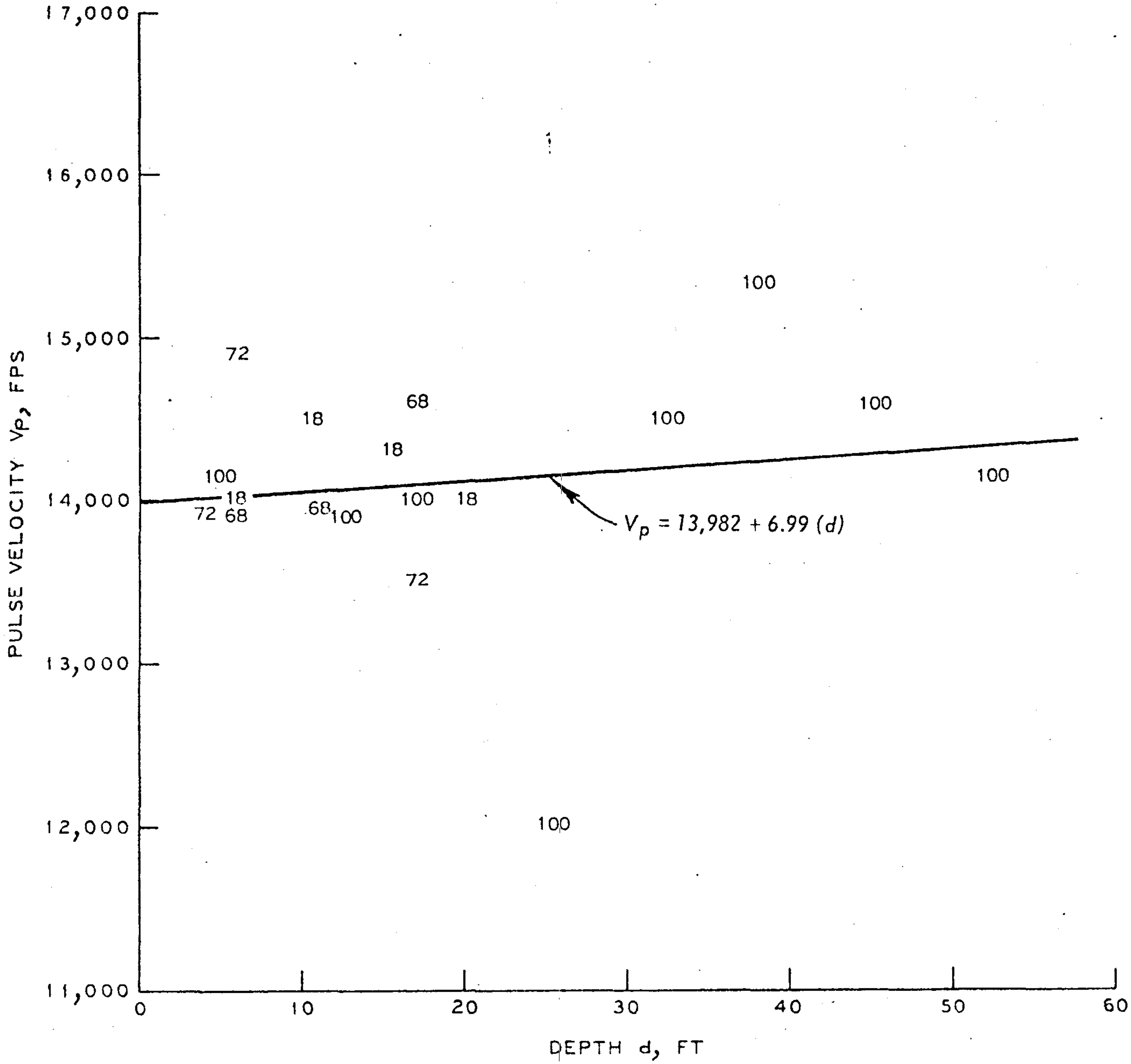


Figure 57. Variation in pulse velocity with depth for various monoliths containing the lower alkali cement.

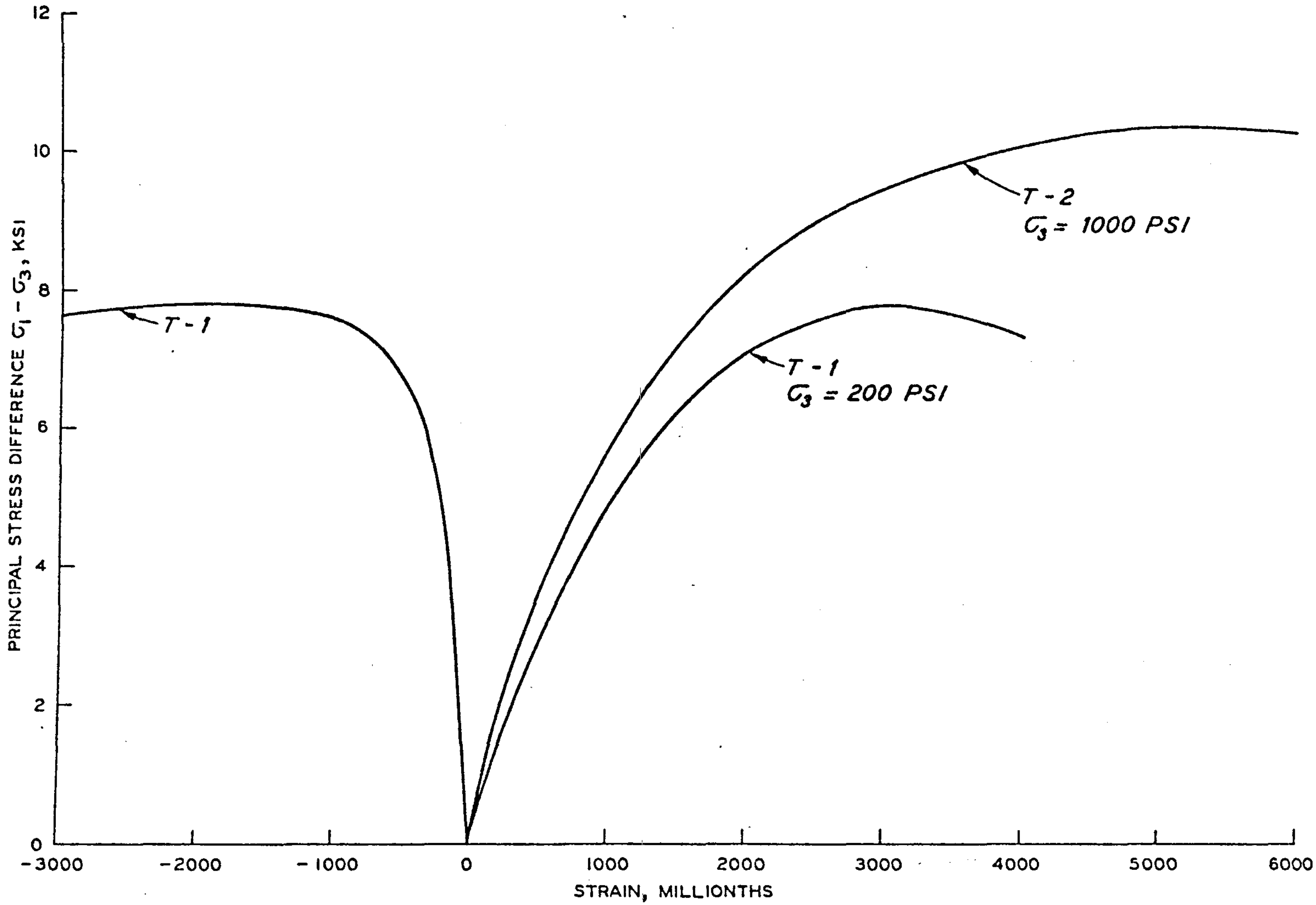


Figure 58. Triaxial test results, concrete cores, monolith No. 5.

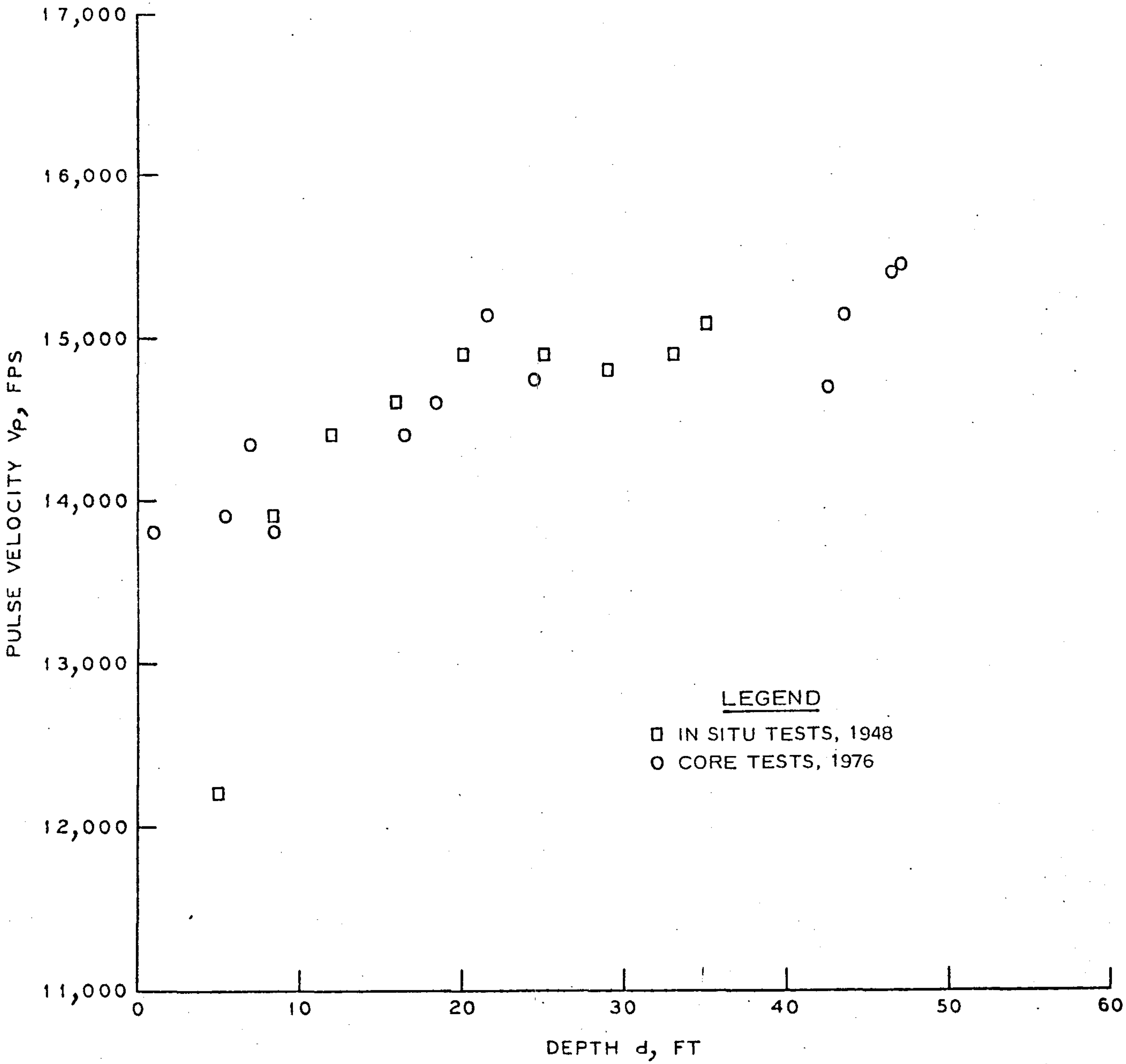


Figure 59. Comparison of tests on cores and in-situ tests, Monolith No. 5.

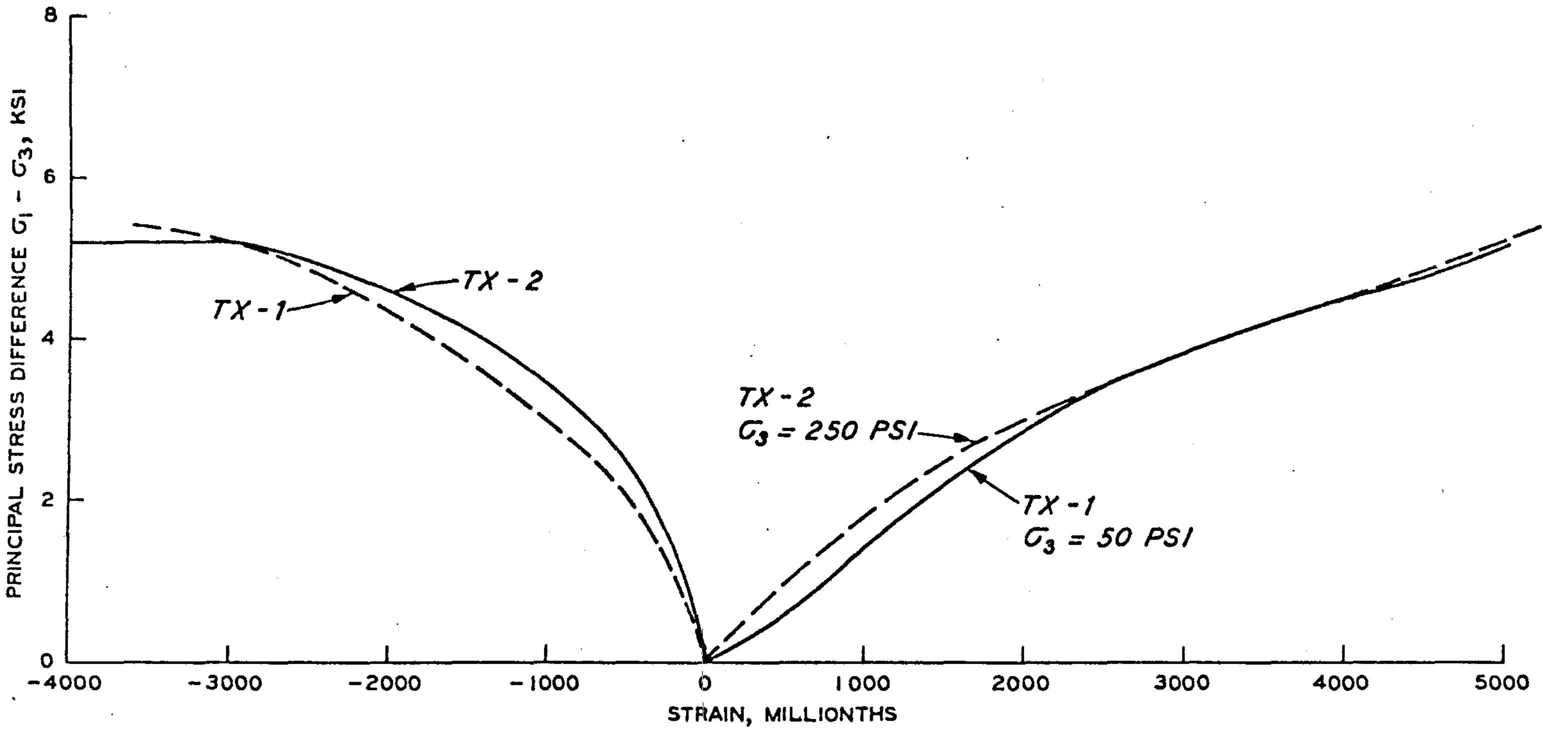


Figure 60. Triaxial test results, foundation cores, monolith No. 5.

PART III: STRESS ANALYSIS

Reduced concrete strengths resulting from alkali-silica reactions initiated concern regarding magnitude and location of stress concentrations within certain lock wall monoliths. In the evaluation of the lock, gate monoliths were singled out because of their importance in the operation of the lock and because of the magnitude of their applied loads. The upstream landwall gate monolith was of particular concern because it had the most extensive surface cracking of the four gate monoliths and was built entirely with the higher alkali cement. Consequently, the upstream landwall gate monolith was determined to be the most critical monolith in the lock. As such, it was selected as the subject of a stress analysis investigation to determine the magnitude and location of stress concentrations and to evaluate these results with respect to current design criteria.

Solution Methods

A 2-D plain strain and a 3-D solid element stress analysis were considered as potential solution methods. The 2-D analysis offered:

a. Reasonable computer cost.

b. Graphical presentation of output.

c. Orthotropic material properties as input.

This type of analysis, however, is restricted to the x-y plane and does not allow the distribution of stresses in the Z direction due to point loads in the x-y plane. Consequently, a 2-D analysis of the gate monolith containing point loads would, at best, yield questionable results.

The 3-D solid element stress analysis would model a structure that had point loads and other changes along the Z axis, such as variation in loadings and geometry. The disadvantages of the 3-D analysis were:

a. Higher computer cost due to increase in problem size.

b. No graphical presentation of output.

c. Isotropic material properties as input.

When the two potential solution methods were compared, it was determined that the monolith could best be modeled by a 3-D analysis because of the following conditions in loadings and geometry of the structure:

a. Point loading:

1. Pintle (gate weight and free-hanging gate forces)

2. Top of recess (gate thrust)

3. Gate anchors (free-hanging gate forces).

b. Variable soil loadings.

c. Variable geometry:

1. Gate and bulkhead recesses

2. Varying pipe gallery location.

Finite Element Grid

Monolith geometry and loading are shown in figures 61 through 65. A finite element 3-D grid was constructed to model the structure and 30 ft of foundation below the structure. The grid contained 2746 nodes and 2061 elements and is shown in figures 66 through 76. A finer grid was used around cutouts to improve the accuracy of the output for these potentially critical areas of stress. A width- or height-to-length ratio of 1.0 to 7.86 was used for minimum element size.

Input Data

The structure was divided into three zones to reflect the decrease in concrete strength with the increase in elevation. These zones and their material properties were as follows:

Zone	Elevation	Modulus Elasticity psi x 10^6	Poisson's Ratio
A	129.6-140.0 ft	2.34	0.144
B	111.0-129.6 ft	3.38	0.158
C	77.8-111.0 ft	4.20	0.204

The foundation was classified into four material zones as previously discussed in the description of foundation core tests. Material properties for these zones are shown in Table 5 .

Three load cases were used in the analysis, as follows:

Load Case	Operation	Description
1	Normal	Upper pool water level upstream and downstream of gate
2.	Normal	Upper pool water level upstream of gate and lower pool downstream of gate
3	Maintenance	Lock chamber dewatered

Gate loads and uplift pressures were different for each case (figures 62 through 65). The hydrostatic pressures in the intake culvert were at a constant upper pool head for all cases.

Based on the results from a previous subsurface investigation by the Mobile District[5], the following properties were used to calculate backfill pressures:

Moist Unit Weight = 128.8 lbs/cu ft

Submerged Unit Weight = 67.7 lbs/cu ft

ϕ Angle = 30.0°

Average Water Table Elevation= 125.0 ft

The at-rest earth coefficient, K_o, for the backfill was calculated using the following equation from reference 6:

$$K_o = 1 - \sin \phi = 0.5$$

As dictated by the Structural Analysis Program 4 input,pressure loadings on negat elements faces were applied as element pressures and on positive faces as nodal loads. As a result there were some 1600 nodal loads used as input. Gate loads were applied as point loads, as follows:

Load Case	Description
1	Free-hanging weight minus buoyancy: weight applied at pintle and moment forces applied at pintle and gate anchors
2	Thrust loads: weight applied at pintle and the portion of the thrust load not taken by the sill applied at the top of the monolith
3	Free-hanging weight: weight applied at pintle and moment forces applied at pintle and gate anchors

Uplift was applied simultaneously to the structure and the foundation through interface foundation elements. For the first computer run, these elements were used to make the interface between the structure and the foundation continuous. This allowed unrealistic tension to develop between the structure and the foundation. To correct this a second run was to be made deleting interface elements that transferred such tension. Also, the pressures due to headloss in these element areas were to be replaced, with full hydrostatic pressures and uplift recalculated.

A data check run showed this problem to be the largest stress analysis problem ever attempted on the WES G-635 computer. The global stiffness matrix consisted of 7412 equations with a bandwidth of 812. During the first solution attempt, storage was exceeded while writing to the scratch disc pack (four million-word capacity) that solved the global stiffness matrix. It was later determined that the matrix solution was approximately one-third complete and would require a multireel tape file having a twelve million word-storage capacity to complete. The total solution time was estimated to be in excess of ten hours.

At this point in time, the WES G-635 system did not have a 3-D bandwidth-minimizer program available. The minimization was attempted manually, with the results being less than optimum. Therefore, the efficiency of the SAP 4 program was reduced. A new version of SAP (SAP 5) was added later to the system, that does have a 3-D bandwidth minimizer. As part of the check on the new code it was possible to use the 3-D bandwidth-minimizer capability on the Oliver stress analysis problem with results as follows:

	Before	After
Bandwidth	812	717
Global matrix size, words	12,989,710	10,680,480

The solution time with a minimized grid was estimated to be 11 hours on the WES G-635 computer. Since the structure and foundation were linked together at common node points, a second solution run would be required to eliminate tension between the two. The computer cost for the two runs required was estimated at $3300 (22 hr x $150/hr). Since there was some concern about the capability of generating a file of sufficient size to accommodate the golbal matrix, a sample run was made in which a file was successfully created containing 10,752,000 words storage.

It was decided in discussions with District personnel that time and funding constraints would not allow completion of the stress analysis at this time. Therefore, work on the analysis was terminated, and all data was stored on cards and filed at this office for possible future use with the SAP 5 program.

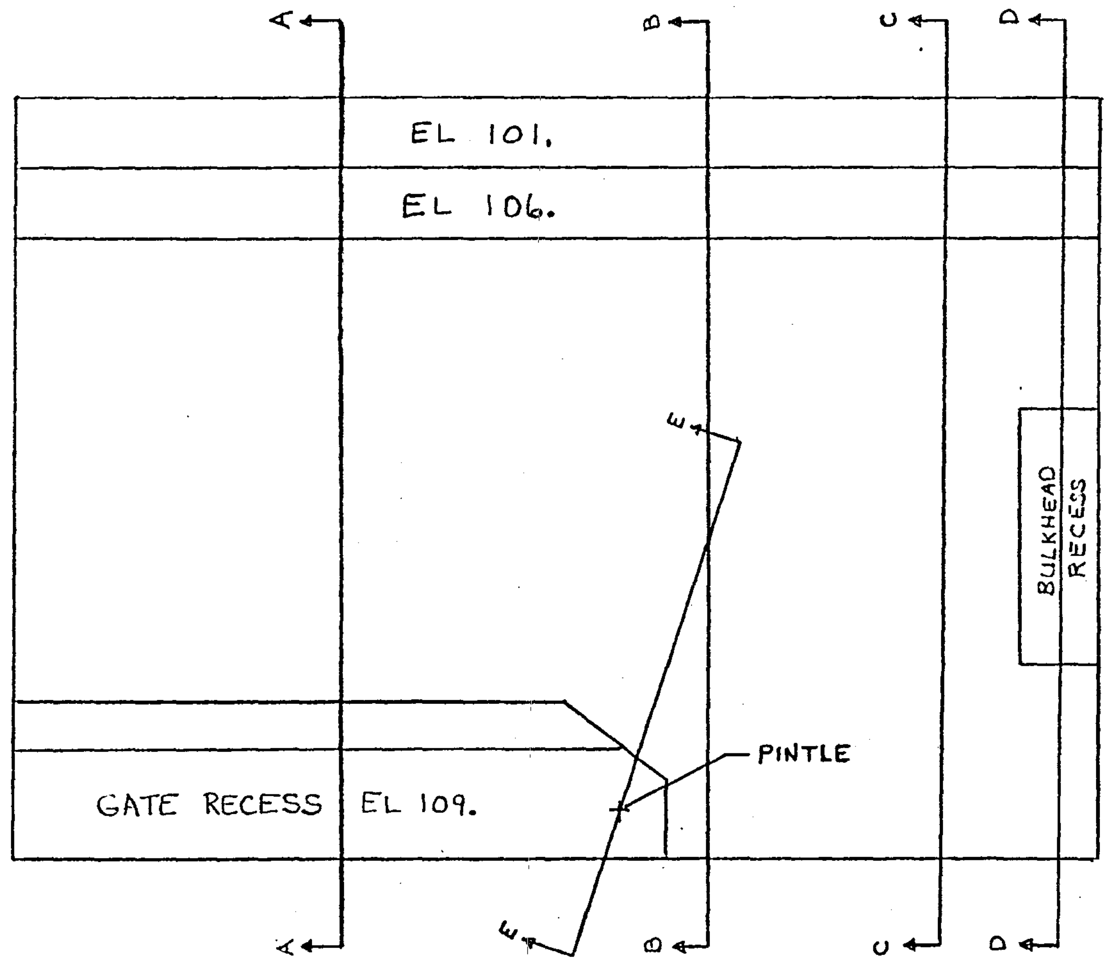


Figure 61. Plan View, Monolith No. 5.

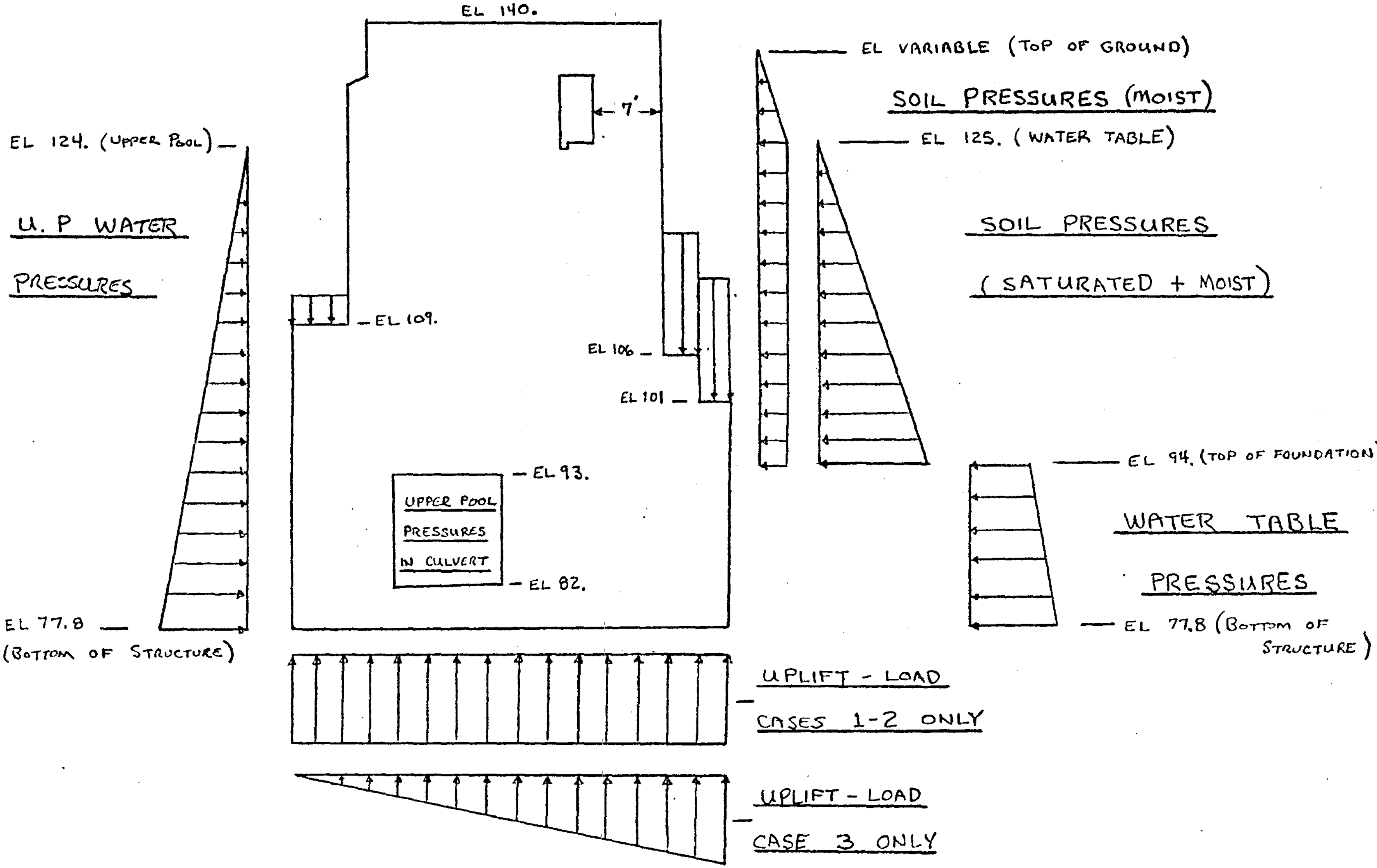


Figure 62. Load Diagrams, Load Cases 1-3, Section A-A, Monolith No. 5.

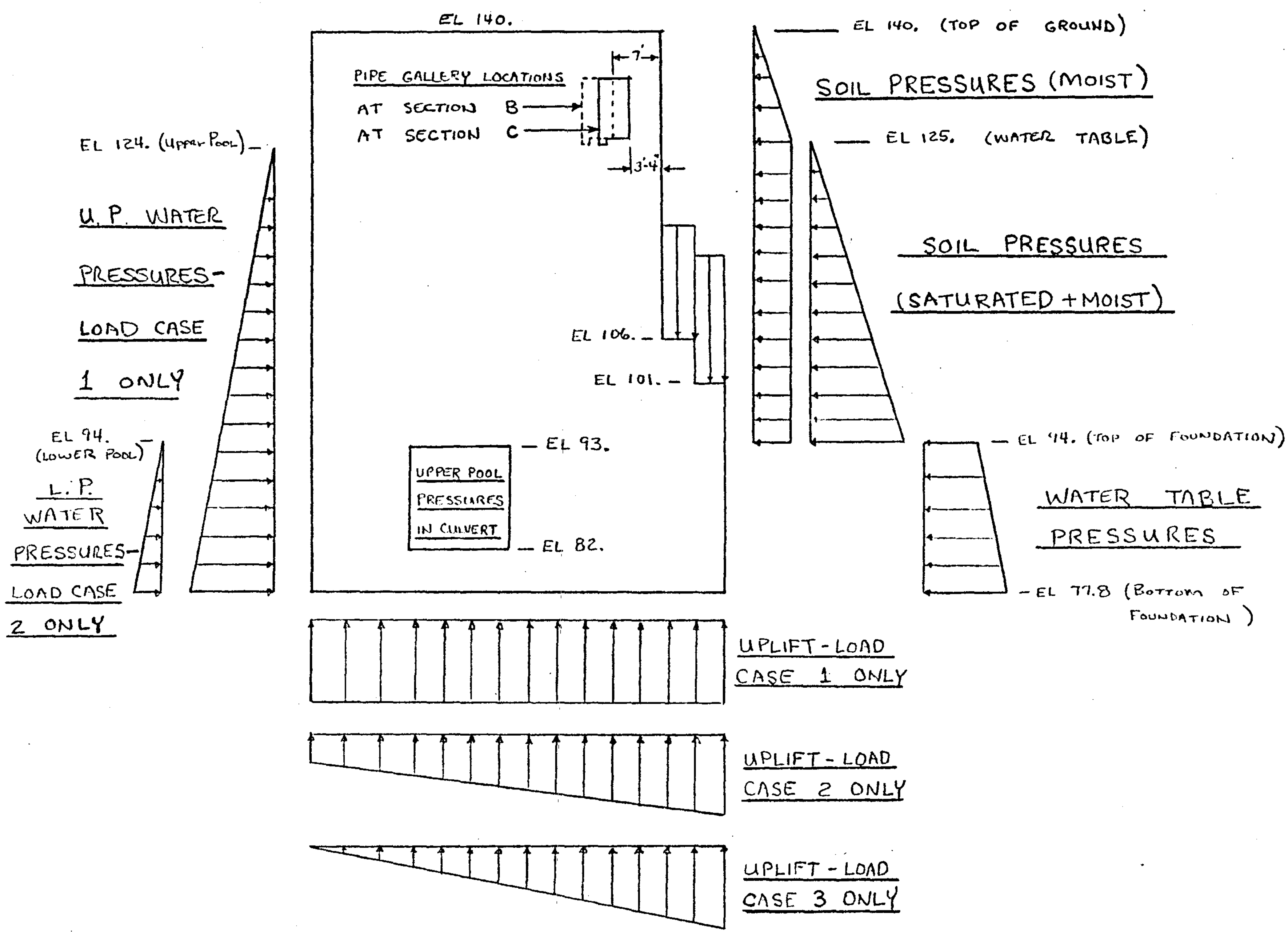


Figure 63. Load Diagrams, Load Cases 1-3, Sections B-B and C-C, Monolith No. 5.

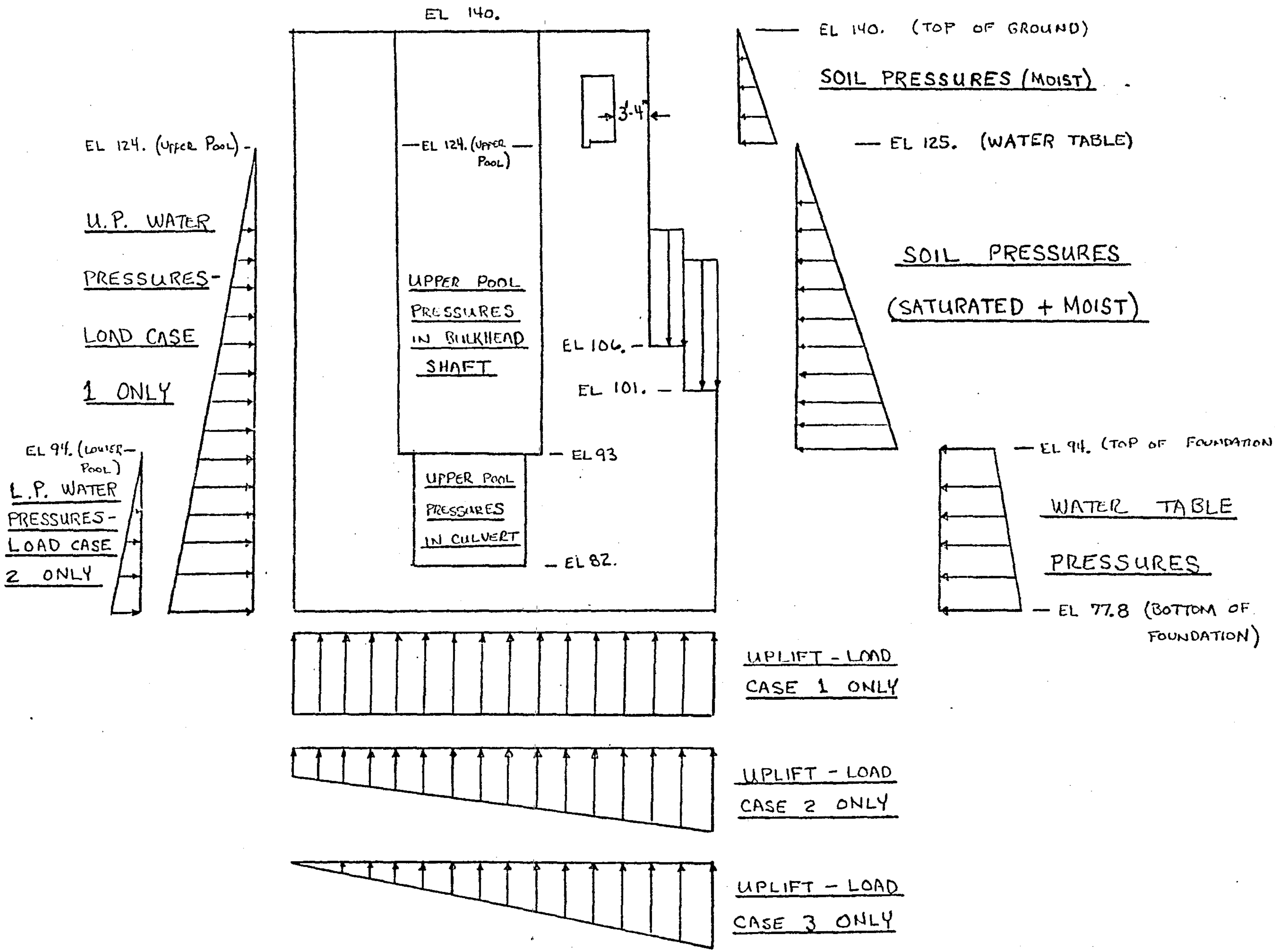


Figure 64. Load Diagrams, Load Cases 1-3, Section D-D, Monolith No. 5.

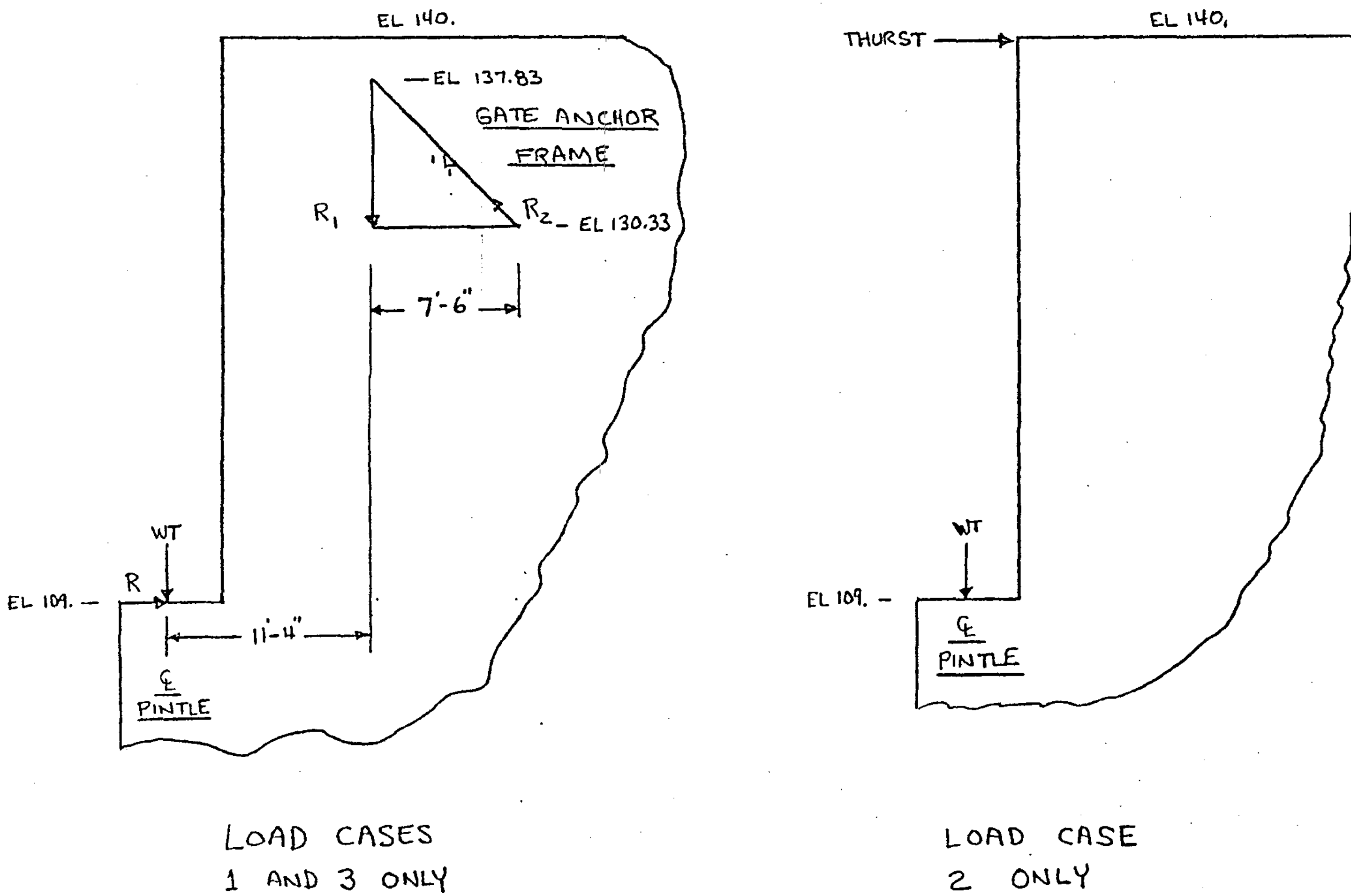


Figure 65. Gate Loads, Load Cases 1-3, Section E-E, Monolith No. 5.

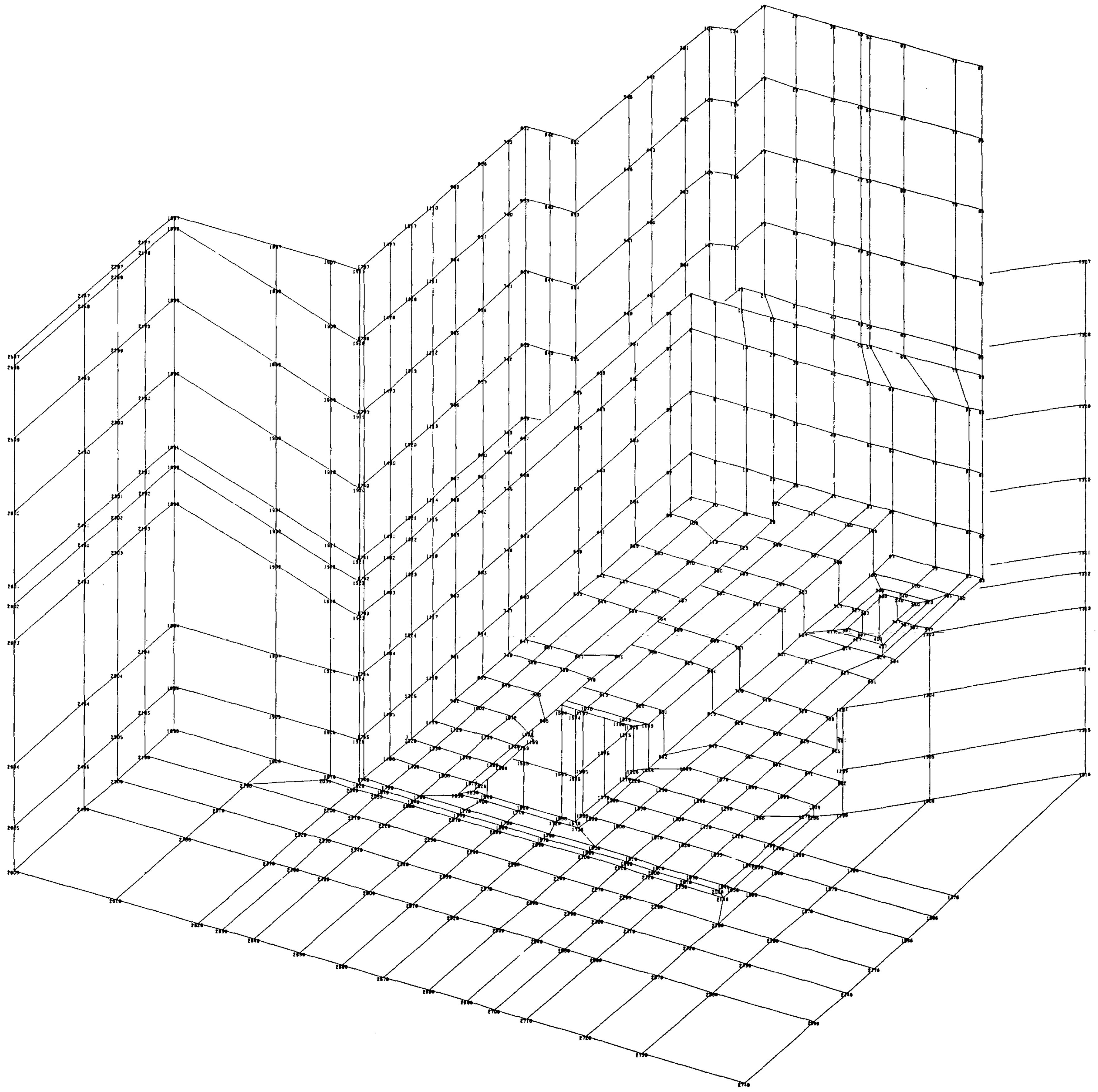


Figure 66. Finite element grid, monolith 5

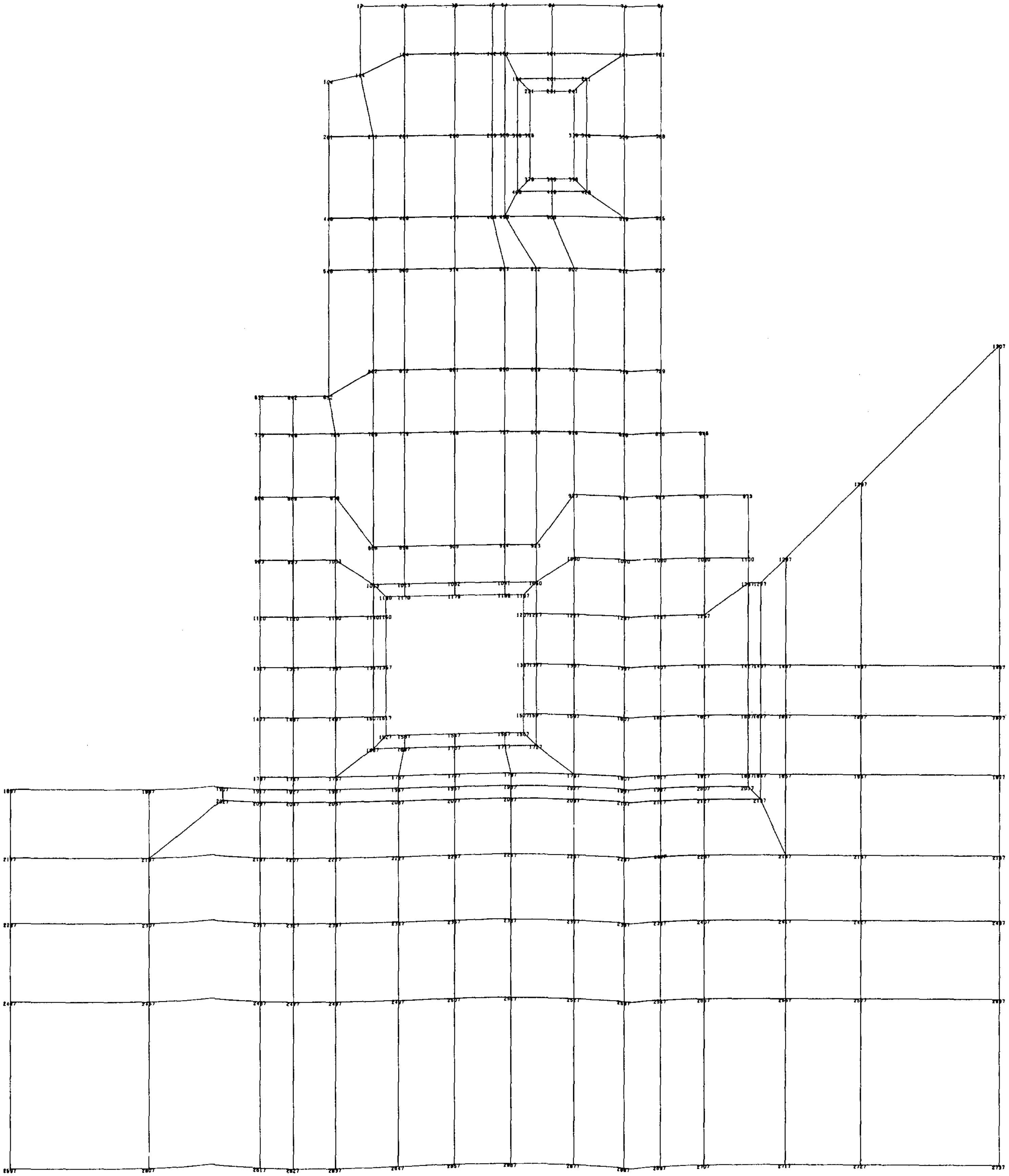


Figure 67. Finite element grid, section view of gate recess, monolith 5

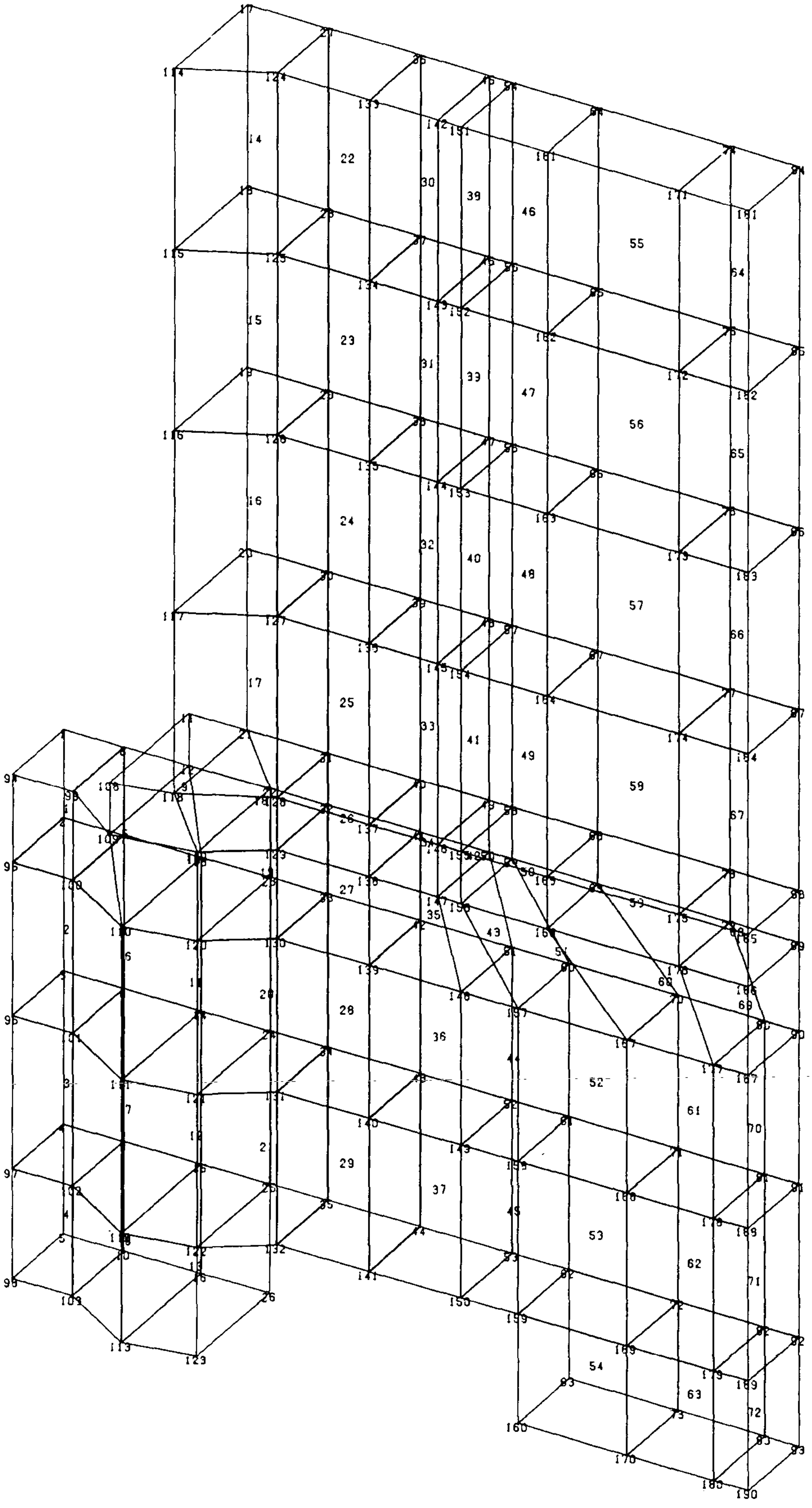


Figure 68. Finite element grid, top layer of elements with hidden lines shown, monolith 5

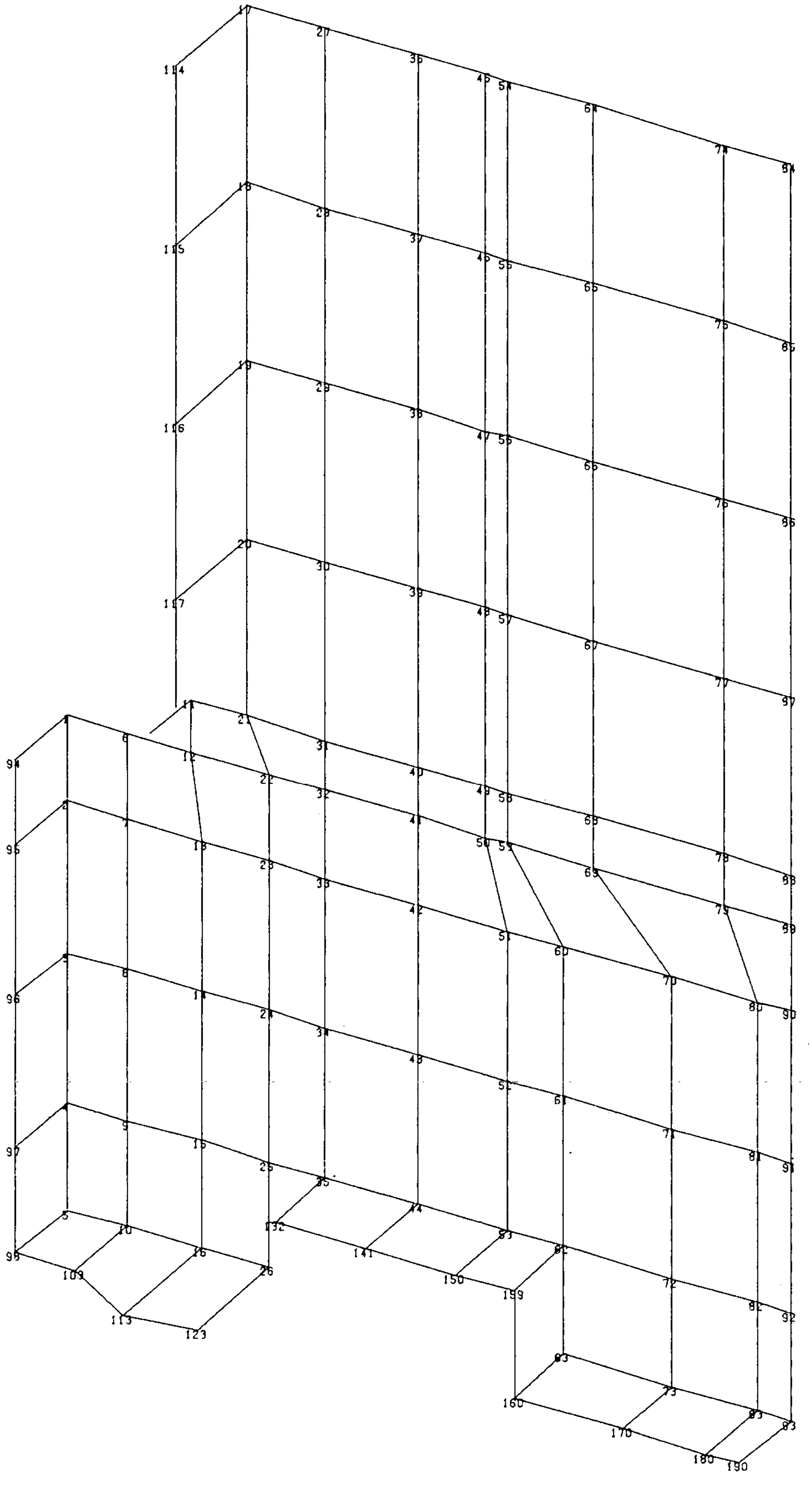


Figure 69. Finite element grid, top layer of elements without hidden lines shown, monolith 5

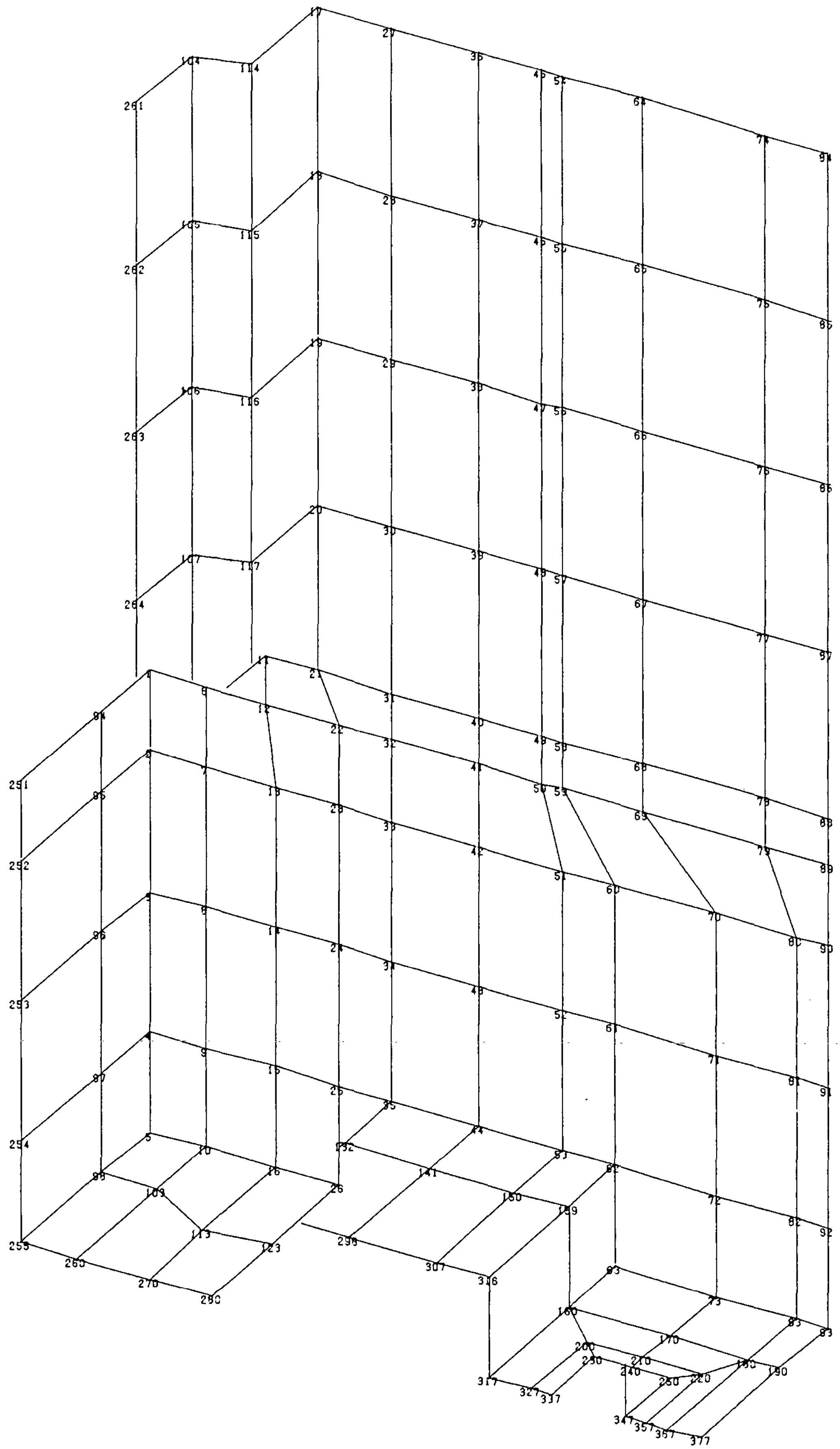


Figure 70. Finite element grid, zone A, concrete, monolith 5

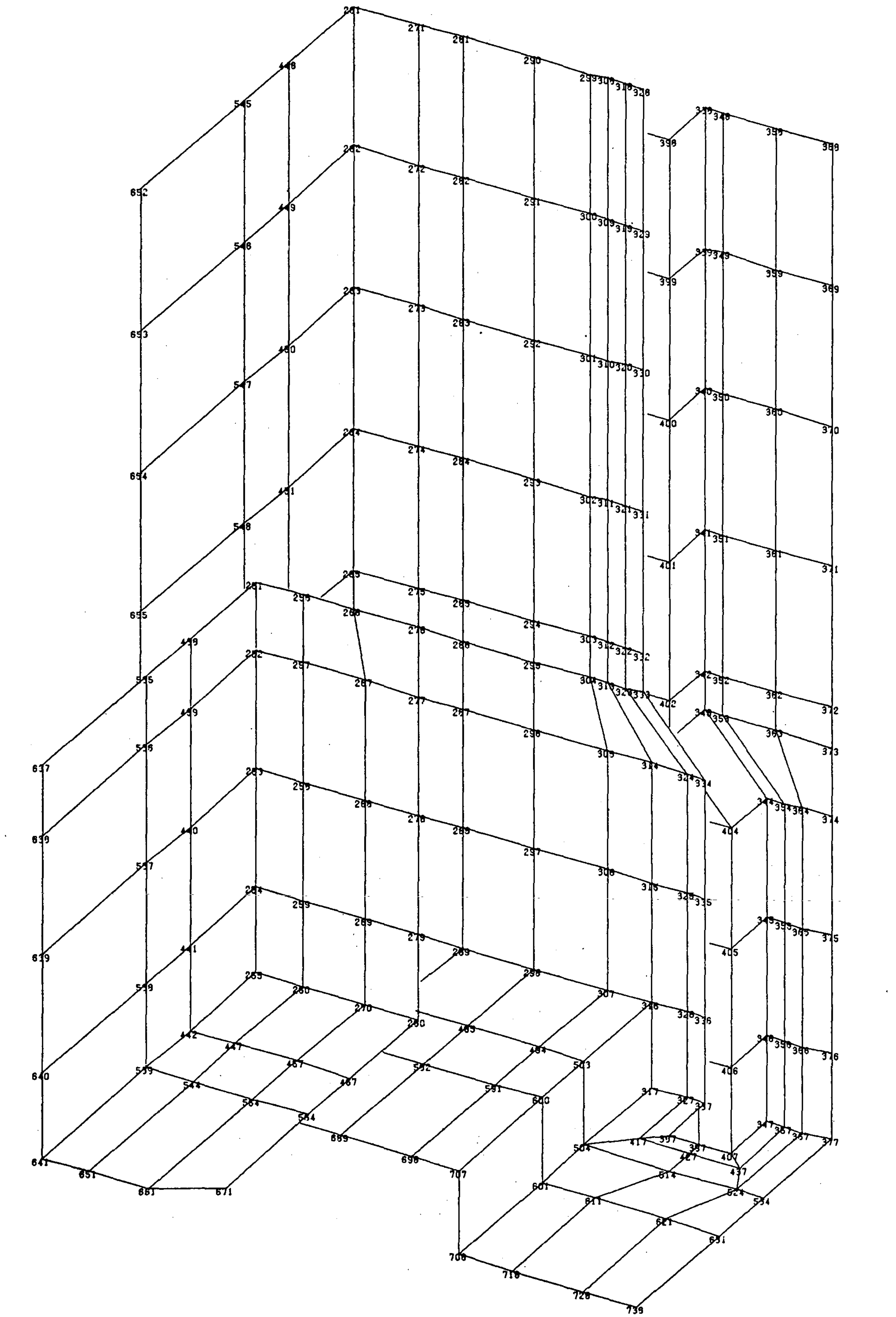


Figure 71. Finite element grid, zone B, concrete, monolith 5

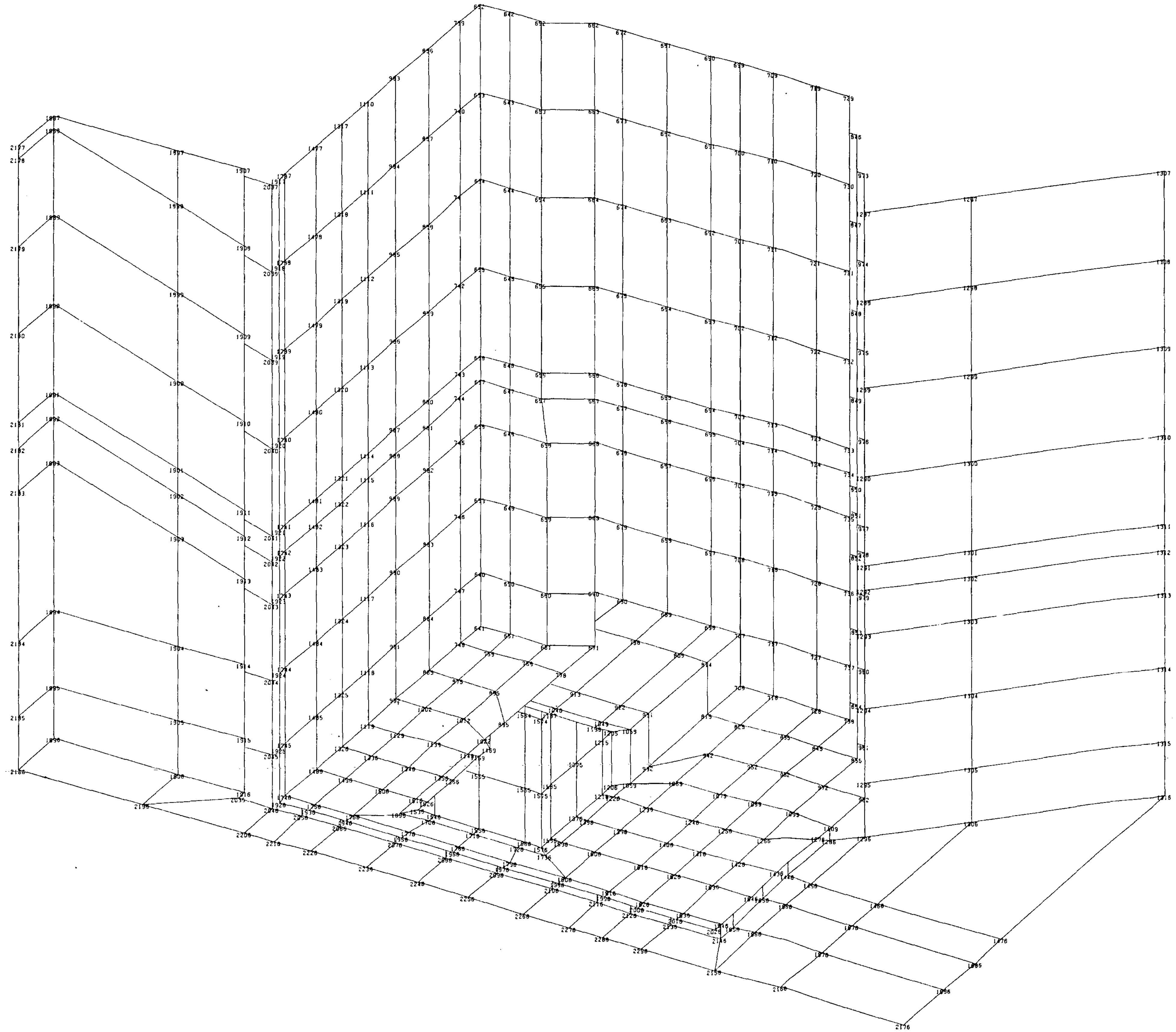


Figure 72. Finite element grid, zone C (concrete) and zone D (foundation), monolith 5

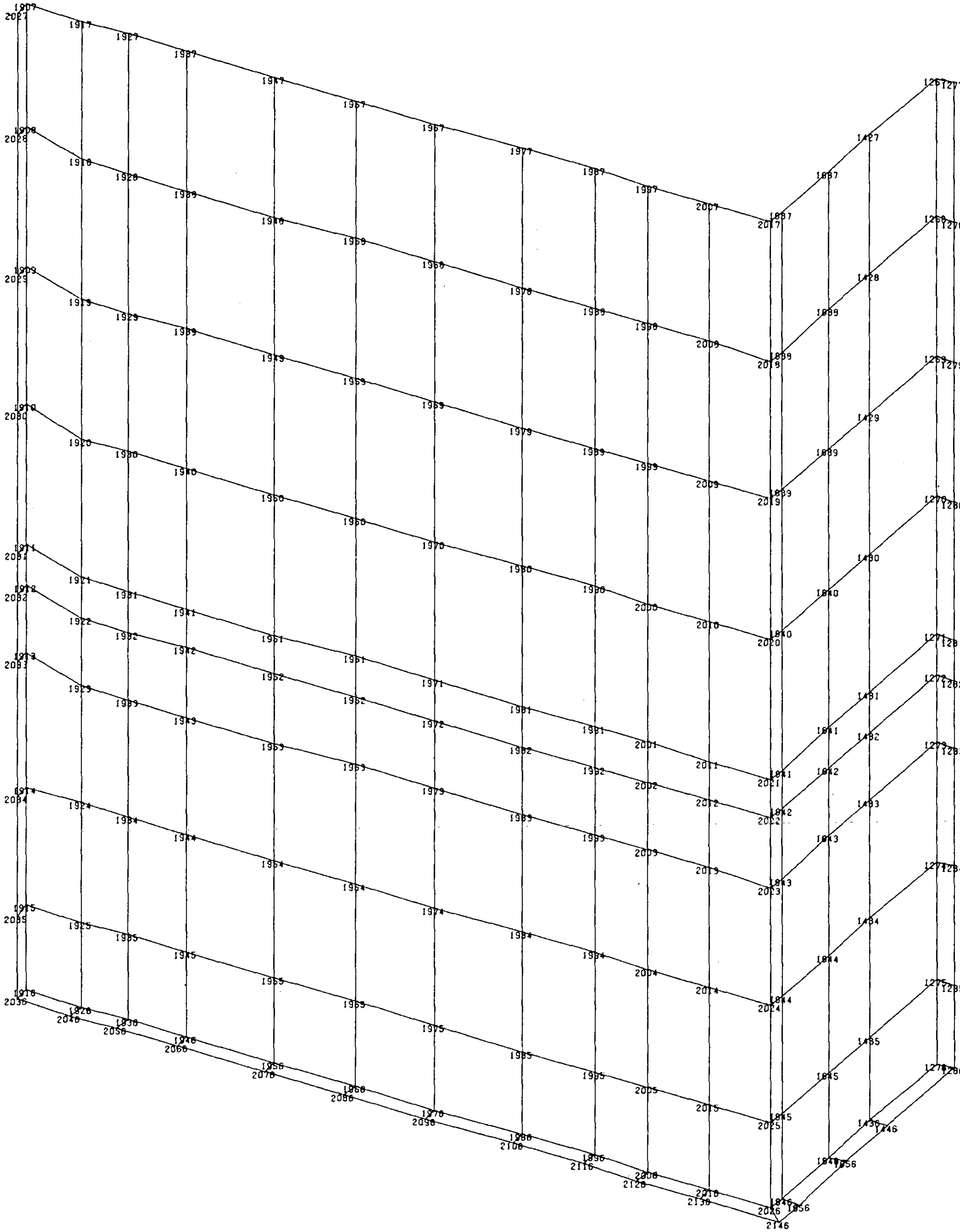


Figure 73. Finite element grid, interface elements, zone D, foundation, monolith 5

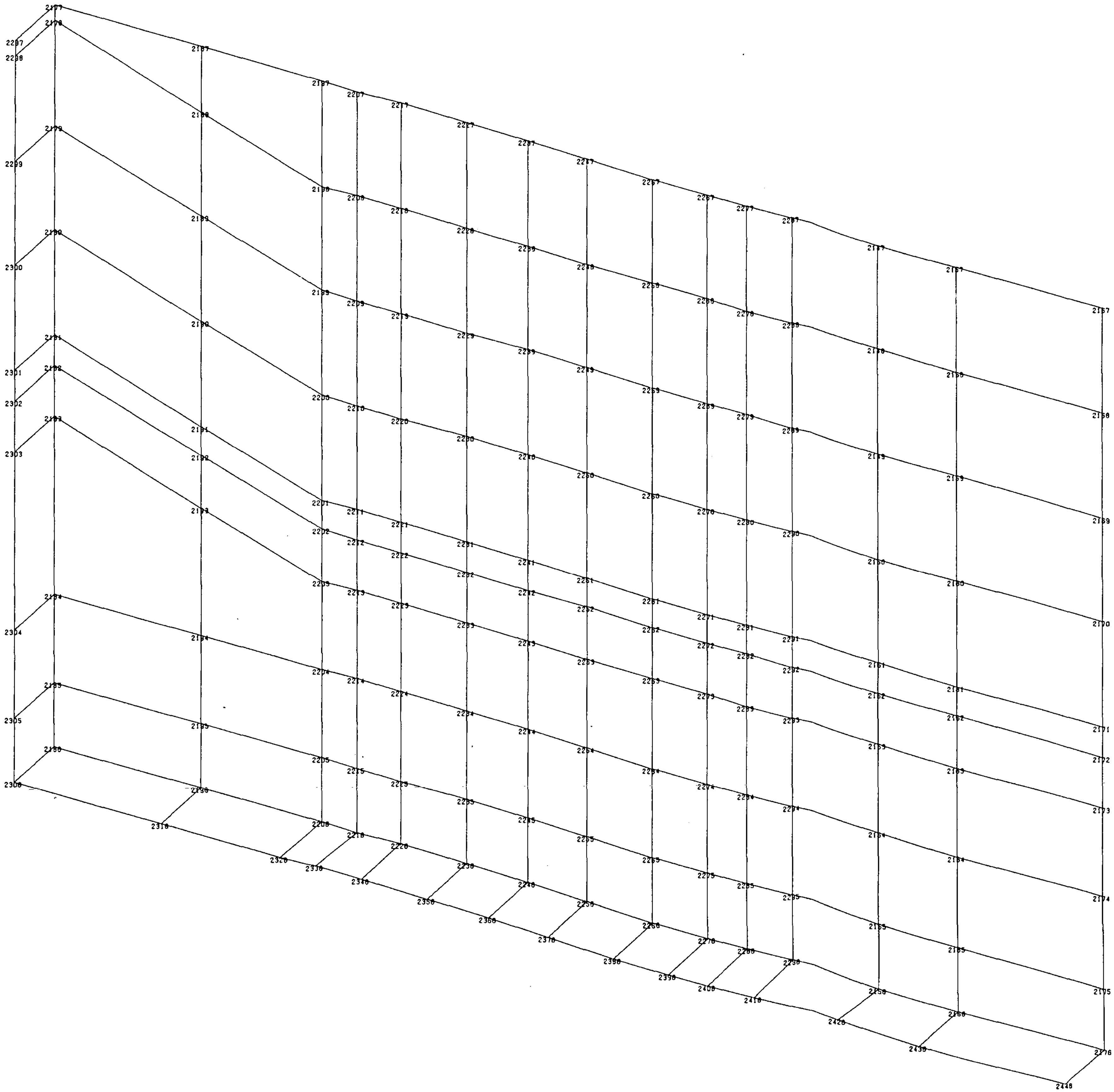


Figure 74. Finite element grid, interface elements, zone E, foundation, monolith 5

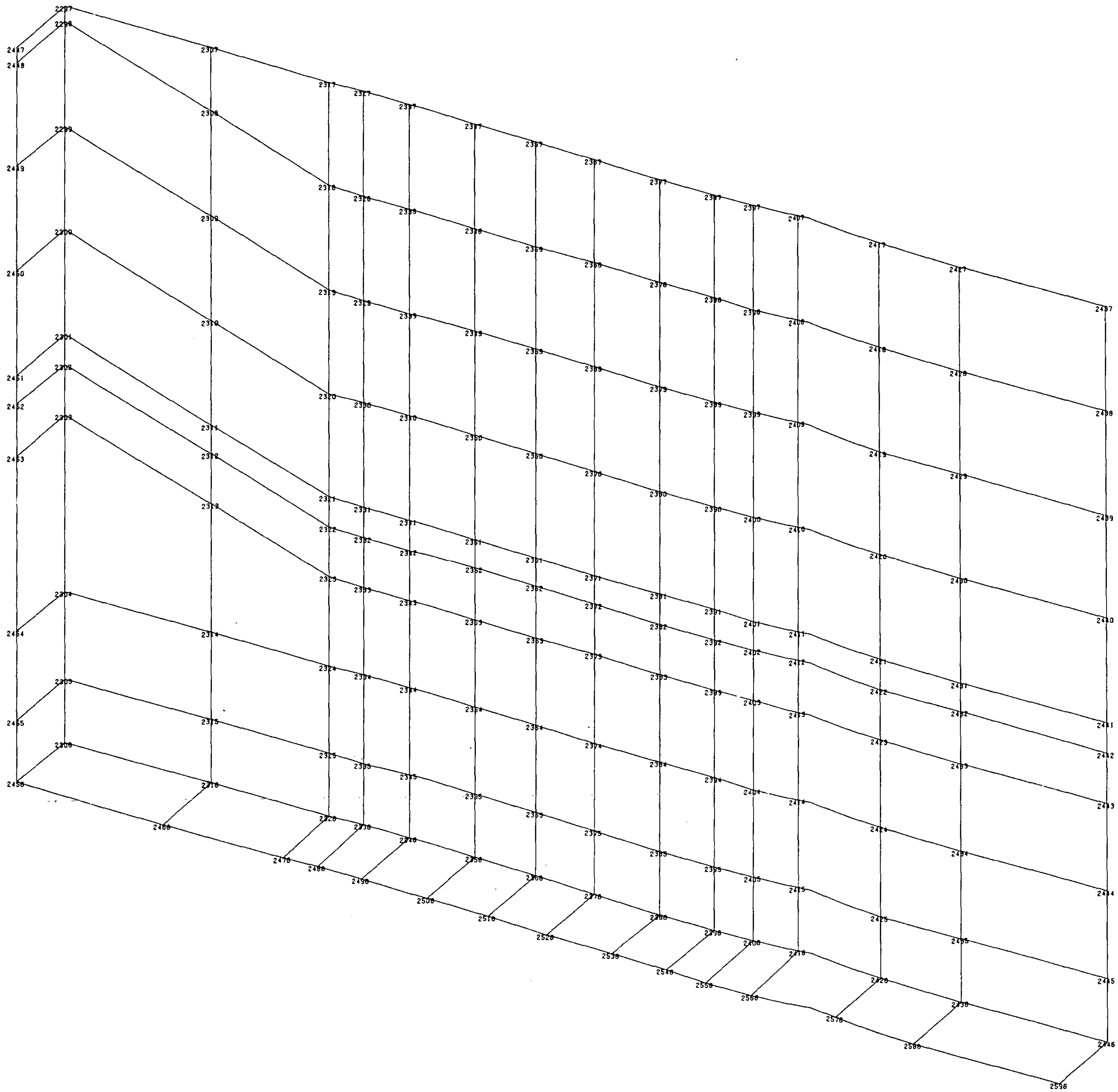


Figure 75. Finite element grid, zone F, foundation, monolith 5

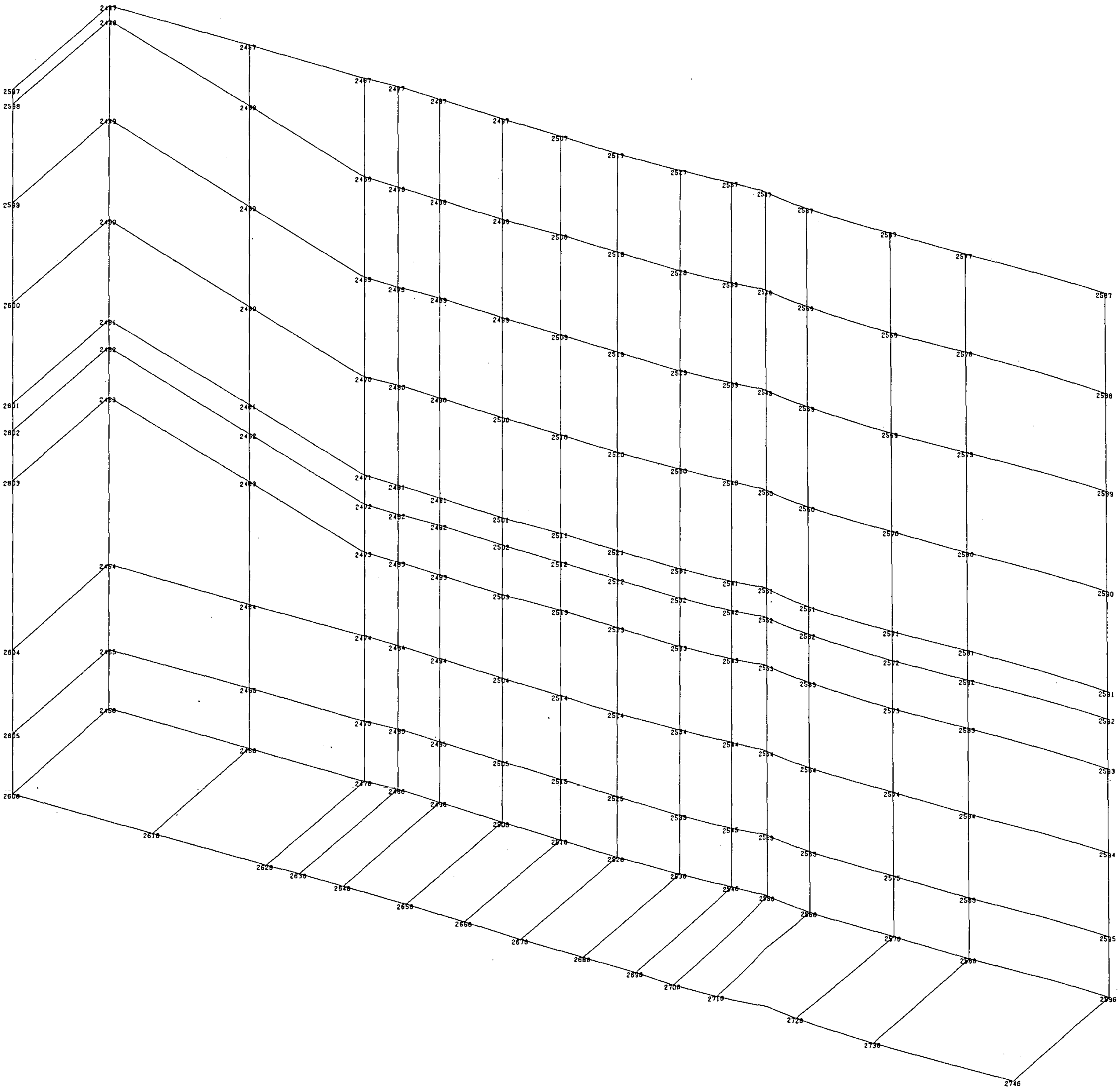


Figure 76. Finite element grid, zone G, foundation, monolith 5

PART IV: DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

Although the exact time is not known, sometime after completion of the structure in 1939 small cracks appeared in the top surfaces and faces of the lock walls. This cracking increased progressively and by 1947 had reached such serious proportions that a special investigation was made to determine its cause. Results of this investigation indicated the primary cause of cracking and disintegration was alkali-silica reaction. The internal expansion and external cracking resulting from this reaction appear to have continued at a greatly diminished rate during the ensuing years.

The petrographic examination of concrete core from Monolith No. 5 shows evidence of alkali-silica reaction the full depth of this monolith. Similar results were obtained on concrete from Monolith No. 16, which was drilled to a depth of approximately 25 ft. In both cases the evidence of alkali-silica reaction decreases with depth, and the major effects of the reaction appear to be concentrated in the upper few feet of each core.

Length-change data for concrete from Monolith No. 5 stored at 100 percent RH and 100 F show an increase with time and depth. Similar results were obtained for concrete from Monolith No. 16, to a minor degree. However, all of the data indicate enough expansion to show that the potential for expansion due to alkali-silica reaction is still present in the concrete under these conditions of high moisture and temperature. Similar data for cores stored at high moisture conditions and temperatures of approximately 70 F would provide an interesting comparison, since they would more nearly simulate possible field conditions.

A first impression of the current concrete cracking is not unlike that of Prof. R. W. Carlson, who observed in 1948,[7] "... lock wall is so badly cracked that the natural impression would be that it is about to collapse." However, as Prof. Carlson later stated, "... pulse velocity tests tell a different story, and probably the true one." These results indicated practically sound concrete for most of the wall, with serious internal disintegration only near the top, and Prof. Carlson concluded that most of the cracking was confined to the surface and that the interior was sound. Subsequent soniscope tests continued to indicate generally good-quality concrete. In situ pulse velocity data obtained during the period 1948-1976 indicated that, of the monoliths tested, only the concrete in Monolith Nos. 16 and 20 would be classified as questionable. However, it should be noted that the same data indicate the concrete in these monoliths is not experiencing progressive deterioration; in fact, the trend is for increased pulse velocities since tests were initiated.

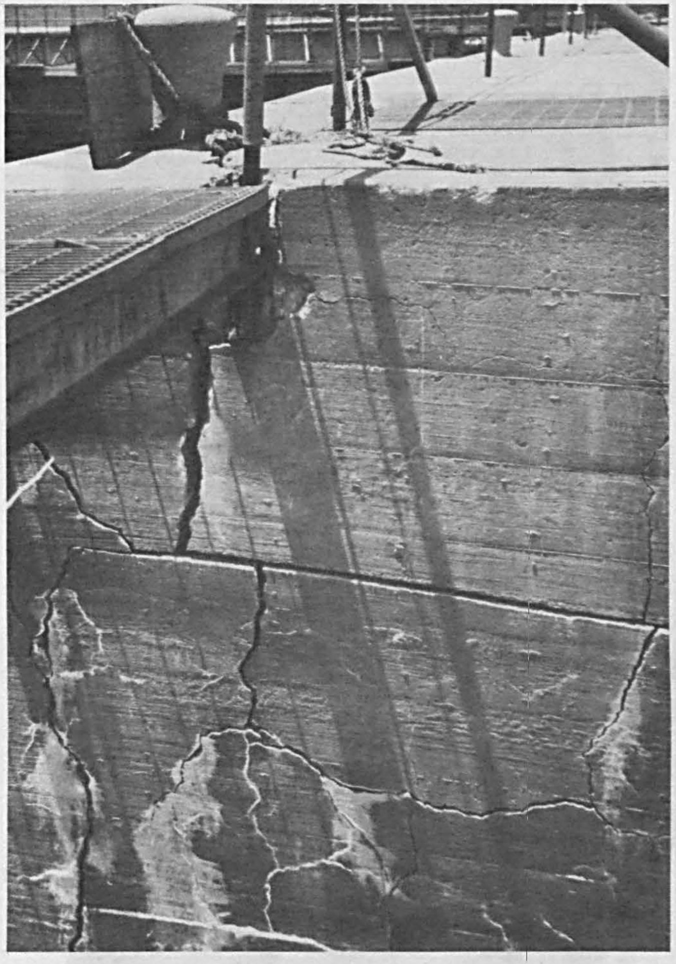
The intensity of surface cracking in the lock wall monoliths generally decreases with distance from the surface, and, for the most part, is limited to the upper 20 ft of the monoliths. A comparison between current surface cracking and relative displacements of adjacent monoliths and that present in 1948 indicates that crack patterns and widths and monolith displacements have, in general, not undergone any drastic changes since the initial investigation.

Results of tests to determine material properties correlate generally with previous tests and other phases of the current condition survey. Of the 46 concrete specimens tested, only three compressive strengths less than 3800 psi were obtained. The ultrasonic pulse velocity of the concrete cores generally increased with increased compressive strength, and all results were in the range (12,000-15,000 fps) of generally good concrete, or better. The compressive strength and pulse velocity of the higher-alkali concrete generally increased with depth, the compressive strength increasing with depth to approximately 20 ft, then stabilizing.

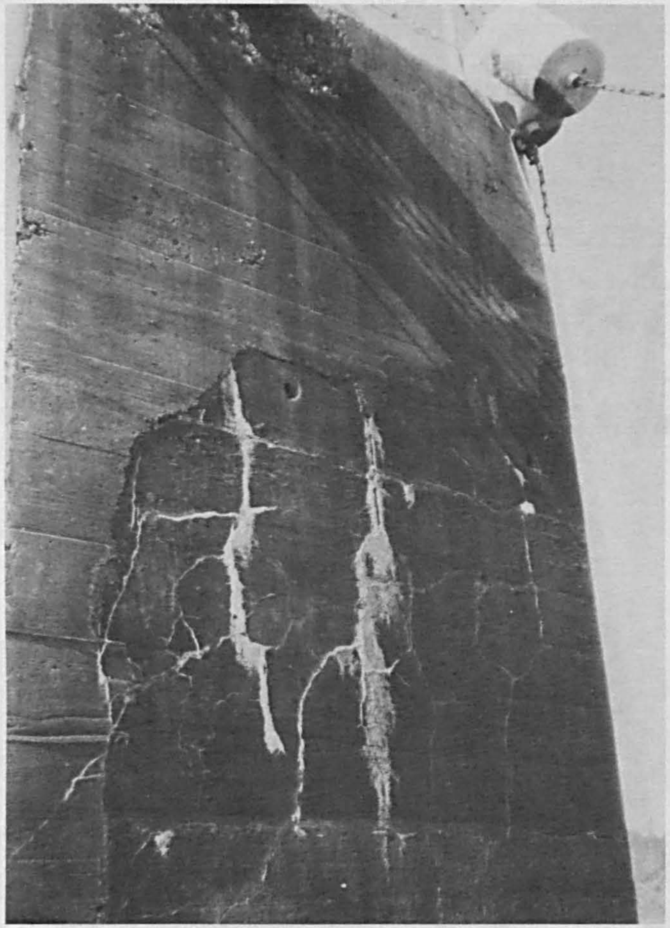
While it is regretted that time and funding constraints prevented pursuing the stress analysis to a final conclusion, results of the other phases of the investigation tend to minimize this concern. In particular, the results of material property tests indicating the current concrete quality to be generally good and substantially unchanged from the initial investigation in 1948 tends to alleviate the concern regarding the effect of reduced concrete strengths on the magnitude and location of stress concentrations within the monolith.

Based on the results of this investigation and comparison with previous work on this structure, it appears that the concrete, despite extensive cracking in some monoliths, is of generally good quality. In those areas of obvious distress, it appears that the condition of the concrete has stabilized over the years, even though the concrete still has the potential for expansion due to alkali-silica reaction. It is believed that any increases in cracking in recent years is more likely attributed to physical deterioration, particularly freezing and thawing, than to the direct effects of continued alkali-silica reaction.

Extensive repairs and/or rehabilitation of the existing 460-ft lock chamber do not appear necessary at the present. Specific points identified through continuing systematic inspections as requiring maintenance would be exceptions. In such cases a procedure similar to that previously employed to repair a portion of Monolith No. 51 is recommended (Fig. 77). This procedure involves removing approximately 1-3 ft of surface concrete, depending on the extent of deterioration, and replacing it with a new reinforced concrete cap block. After 20 years the repair to Monolith No. 51 exhibits only two small cracks, despite significant cracking in the unrepaired portion of the monolith (Fig. 78). The difficulty involved in arresting a propagating crack should be recognized; and the fact that the repair has a minimum of cracking may be further evidence of chemical reaction stabilization as much as 20 years ago.



a. Before



b. After (20 yrs. later)

Fig. 77. Repair of Monolith No. 51.

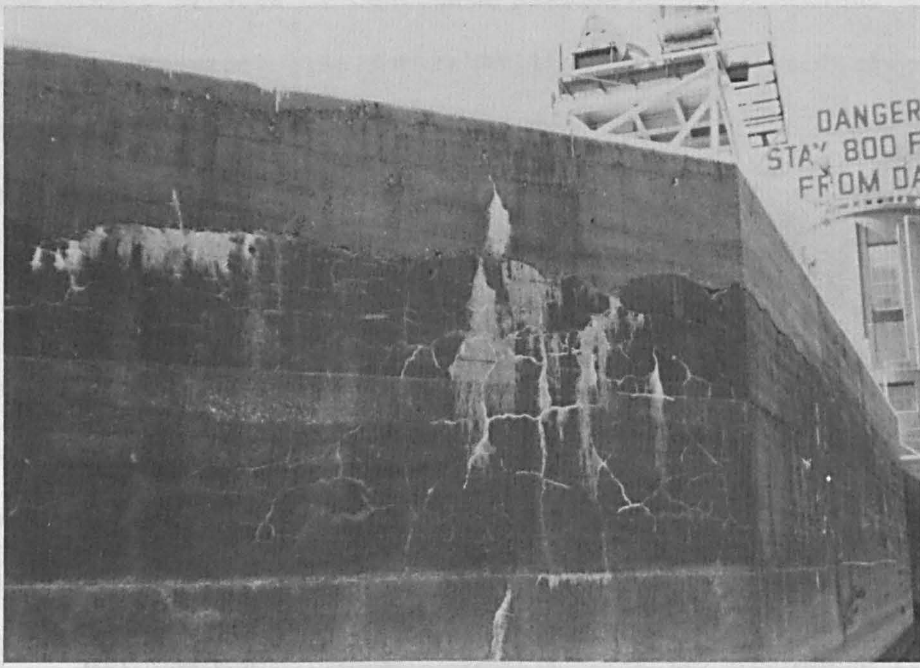


Fig. 78. Portions of Monolith No. 51 repair
after approximately 20 years.

REFERENCES

1. US Army Engineer Waterways Experiment Station, CE, "Handbook for Concrete and Cement," Aug 1949 (with quarterly supplements), Vicksburg, MS.

2. Roshore, E. C., "Field Soniscope Tests of Concrete; 1953 Tests," Technical Memorandum No. 6-383, Report 1, Apr 1954, US Army Engineer Waterways Experiment Station, CE.

3. Roshore, E. C., "Field Soniscope Tests of Concrete; 1953 - 1957 Tests," Technical Memorandum No. 6-303, Report 2, Mar 1958, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

4. Thornton, H. T., Jr., "Field Soniscope Tests of Concrete; Ten-Year Summary of Results," Technical Memorandum No. 6-383, Report 3, Mar 1967, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS

5. Lamar, J. H., "Oliver Lock and Dam - Subsurface Investigation," Mar 1971, US Army Engineer District, Mobile, CE, Mobile, AL.

6. ___________, "Retaining Walls," Engineer Manual No. 1110-2-2502, November 1975, Office, Chief of Engineers, Washington, DC.

7. Carlson, R. W., "Wave Velocity Apparatus and Its Place in Non-Destructive Testing of Concrete in Place," Memorandum for Office, Chief of Engineers, November 1948.

TABLE 1

SONISCOPE TESTS

ROOF OF INSPECTION TUNNELS TO TOP OF LOCK

Monolith No.	Station No.	Path Length, ft	Pulse Velocity, fps 1953	1954	1955	1957	1959	1963	1969	1975
15	(27)*	7.85	--	--	13,285	13,820	13,920	14,375	13,870	14,405
16	(35)	↓	--	--	12,020	12,095	12,135	12,210	12,150	12,745
↓	(42)	↓	--	--	12,020	12,115	12,190	12,500	12,460	12,850
↓	(52)	↓	--	--	12,115	12,220	12,150	12,210	12,060	12,440
20	30-10V	4.42	**	**	9,910	9,405	11,135	6,770+	8,420	10,550
↓	30-3V	↓	10,160	10,400	9,865	11,220	11,570	10,575	11,510	12,885
↓	30-1.5V	↓	10,885	10,890	10,475	11,725	12,555	12,450	10,730	12,850
↓	30V	↓	10,860	10,835	10,400	11,600	11,880	11,725	11,600	12,775
↓	30+1.5V	↓	11,335	11,515	10,995	12,520	12,555	12,520	11,420	12,850
21	5	↓	14,880	14,850	14,305	14,635	14,445	14,585	14,540	14,985
↓	10	↓	15,085	15,155	14,635	14,735	14,585	14,830	14,585	15,035
60	31H	7.97	15,125	15,035	14,895	14,870	14,730	14,850	***	15,300
↓	32H	↓	14,570	14,545	14,360	14,260	14,310	14,595	***	14,870
	33H		14,335	15,010	14,870	14,760	14,730	14,815	***	15,210

* Parentheses around station numbers indicate that these stations were established in 1955.
** No readings obtained at this station in 1953 and 1954.
\+ Cracks up to 1 in wide were observed in Monolith 20 in 1963.
*** No readable signal obtained

TABLE 2

OLIVER LOCK AND DAM

CONCRETE CORE TESTS

Monolith No.	Spec. No.	Approx. Depth, ft	Pulse Velocity, fps	Unconfined Compressive Strength, psi	Modulus of Elasticity,* psi x 10^6	Poisson's Ratio
5	1-A	1.0	13,812	4120	2.10	.122
	2-A	5.5	13,914	5670	2.50	.159
	3-A	7.0	14,347	5530	2.45	.173
	4-A	8.5	13,770	3970	2.31	.122
			13,960	4820	2.34	.144
5	1-B	16.5	14,391	4230	2.86	.128
	2-B	18.5	14,622	5380	3.74	.153
	3-B	21.5	15,136	5540	3.46	.181
	4-B	24.5	14,755	5550	3.47	.169
			14,726	5180	3.38	.158
5	1-C	42.5	14,711	4670	2.99	.167
	2-C	43.5	15,136	5930	4.98	.218
	3-C	46.5	15,418	6580	4.70	.250
	4-C	47.0	15,441	6450	4.13	.179
			15,176	5910	4.20	.204
16	1-H	7.5	14,733	4980	2.77	.278
	2-H	8.5	14,170	3860	2.03	.163
	3-H	11.0	13,408	2610	1.37	.342
			14,104	3820	2.06	.261

* Secant modulus of elasticity determined at 50% of ultimate stress.

Table 3
Concrete Core Tests

Monolith No.	Spec. No.	Approx. Depth, ft	Pulse Velocity, fps	Unconfined Compressive Strength, psi
8	8-A	5.2	13,316	2580
	8-B	6.3	12,820	2670
	8-C	15.2	12,657	4080
	8-D	16.2	12,900	3880
16	16-J	16.9	14,062	6760
	16-K	17.9	14,062	5950
	16-L	19.0	13,690	4240
18	18-A	6.2	13,975	5990
	18-B	10.8	14,492	6320
	18-C	15.5	14,273	5140
	18-D	19.7	13,975	5650
	18-E	23.0	13,833	6310
	18-F	25.5	14,062	5780
	18-G	27.1	13,854	6380
68	68-A	6.1	13,945	6110
	68-B	11.0	13,916	6030
	68-C	16.9	14,583	9050
72	72-A	4.0	13,860	6340
	72-B	5.0	14,945	6620
	72-C	17.2	13,500	7460
100 (Spillway)	S-A	5.4	14,149	9150
	S-B	12.6	13,860	7210
	S-C	16.9	13,981	5860
	S-D	25.5	12,000	5850
	S-E	32.4	14,527	7150
	S-F	38.2	15,345	7410
	S-G	44.9	14,583	7030
	S-H	51.6	14,147	4960
	S-I	56.6	13,833	5990
	S-J	59.6	14,062	5840
	S-K	61.3	13,854	6380

Table 4

Triaxial Tests

Concrete and Foundation Cores

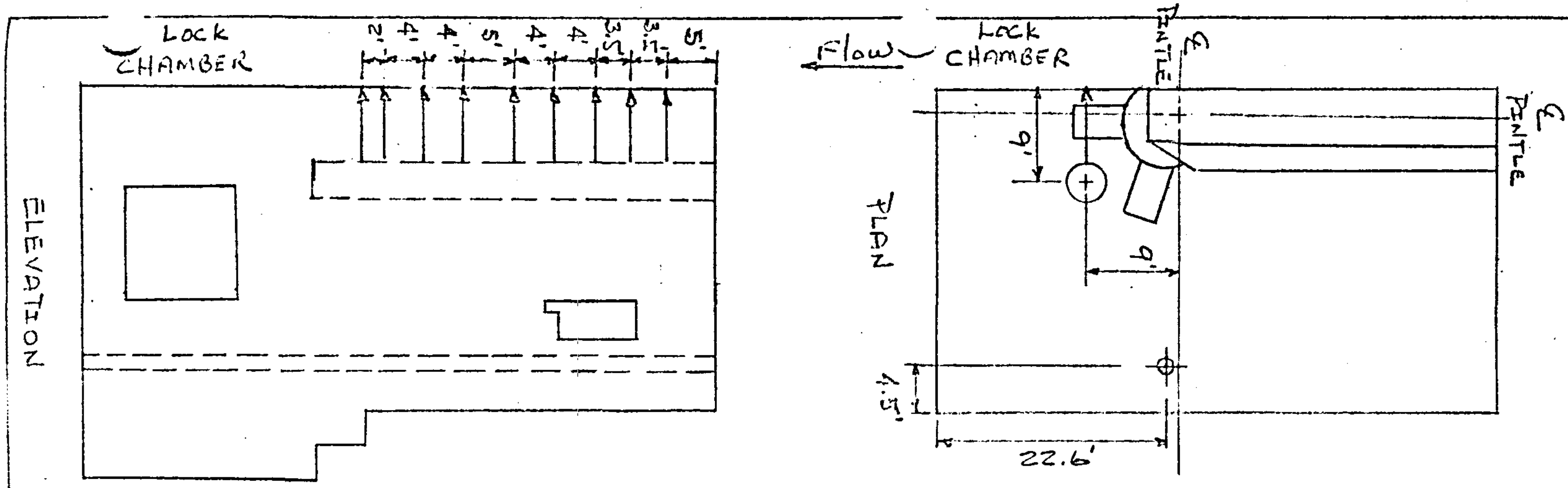
Mono. No.	Specimen No.	Approx. Depth, ft	Min Prin Stress, σ_3 psi	Max Prin Stress, σ_1 psi	Modulus of Elasticity, psi X 10^6	Poisson's Ratio
			Concrete			
5	T-1	40.5	200	8080	5.39	.227
	T-2	48.5	1000	11440	5.84	——
			Foundation			
5	Tx-1	87.0	50	5220	1.45	.306
	Tx-2	90.0	250	5680	1.58	.500

Table 5

Oliver L&D Foundation Core Tests

Mono. No.	Spec. No.	Approx. Depth, ft	Unconfined Compressive Strength, psi	Modulus of Elasticity,* psi X10^6	Poisson's Ratio
5	2-D	63.1-68.5	5120	1.49	0.064
	3-D		4430	1.09	0.254
			4780	1.29	0.159
5	1-E	68.5-73.7	4400	1.68	0.233
	2-E		11920	2.68	0.128
			8160	2.18	0.180
5	1-F	73.7-79.9	4790	1.15	0.293
	2-F		5230	1.16	0.289
			5010	1.16	0.291
5	1-G	79.9-91.4	4160	0.86	0.296
	2-G		3760	0.66	0.311
			3960	0.76	0.304

*Secant modulus of elasticity determined at 50% of ultimate stress.

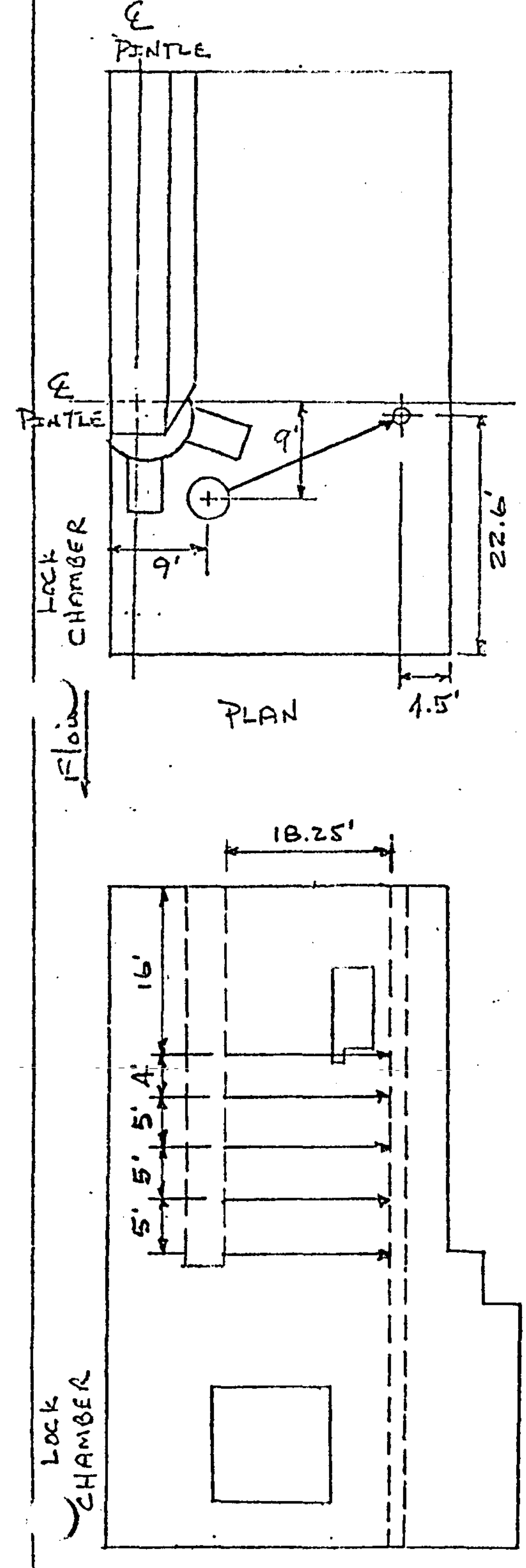


Pulse Velocity, fps

Depth, ft	1948	1952	1953	1954	1955	1957	1959	1963	1969	1975
5	12,200	-	12,510	12,310	12,020	12,055	13,230	13,070	12,155	13,535
8.5	13,390	13,680	14,110	14,085	13,780	13,655	13,560	13,345	13,535	13,880
12	14,390	14,430	14,710	14,770	14,190	14,270	14,165	14,430	14,215	14,710
16	14,610	14,215		13,485						
20	14,890	14,215		14,740						
25	14,910	14,295		14,795						
29	14,810									
33	14,930									
35	15,110									

SONISCOPE
TEST RESULTS
MONOLITH NO. 5

CHECKED BY:	DATE:	SHEET NO.

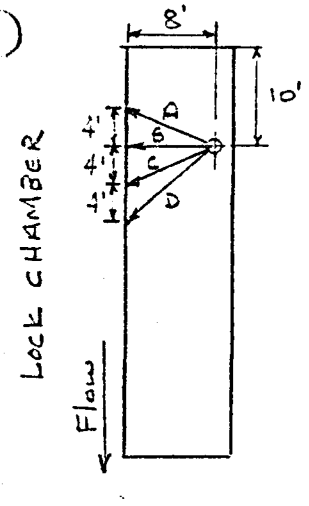


ELEVATION

Depth, ft	Pulse Velocity, fps
16	14,485
20	14,485
25	14,425
30	14,315
35	13,775

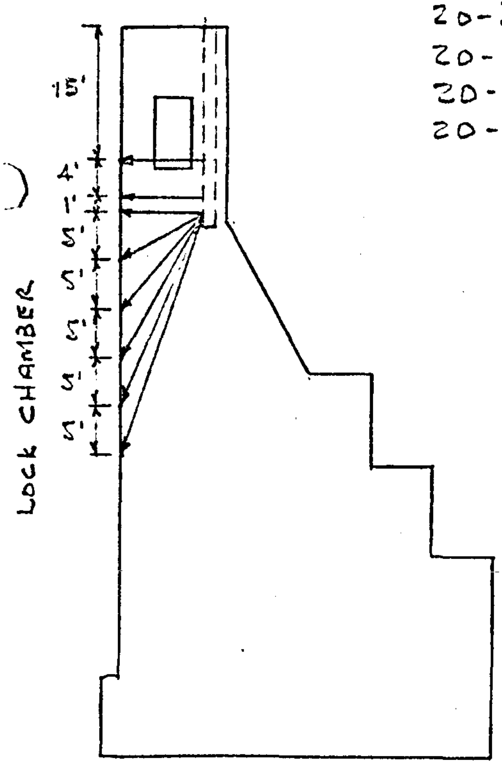
1976
SONISCOPE
TEST RESULTS
MONOLITH NO. 5

PLATE 2



PLAN

Depth, ft.	Test Path Designation	Test Path Length, ft.	Pulse Velocity, fps
15	A	8.75	13,360
	B	7.83	13,625
	C	8.75	14,000
	D	11.25	14,330
19	A	8.75	14,465
	B	7.83	14,920
	C	8.75	14,585
	D	11.25	15,000
20	B	7.83	14,920
20-25	–	9.29	14,870
20-30	–	12.70	14,600
20-35	–	16.92	14,590
20-40	–	21.48	8,840
20-45	–	26.20	14,435

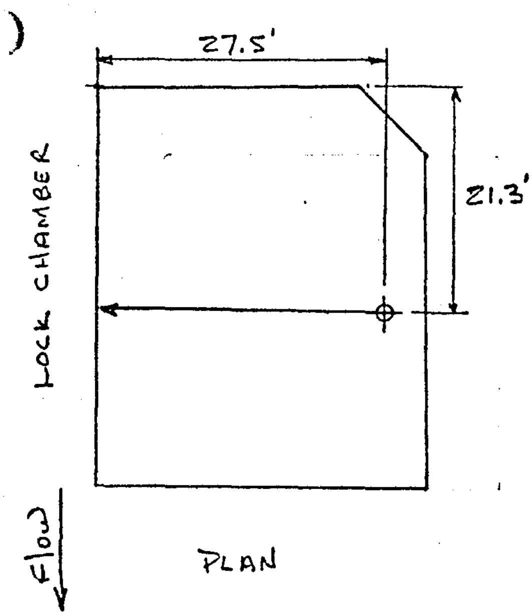


ELEVATION

1976
SONISCOPE
TEST RESULTS
MONOLITH No. 8

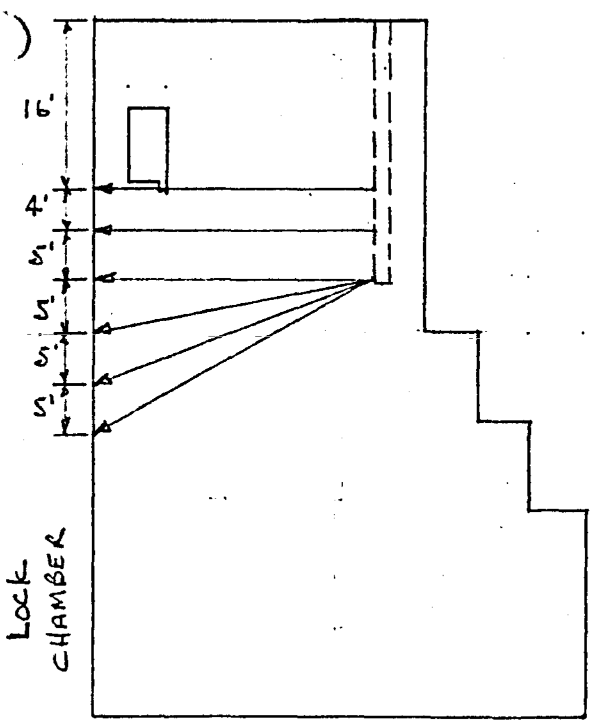
WES F... NO. 1255
REV ... 1968

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.



PLAN

Depth, ft.	Path Length, ft.	Pulse Velocity, fps
16	27.50	14,415
20	27.50	14,800
25	27.50	14,380
25-30	28.20	14,170
25-35	29.50	14,215
25-40	No Signal	



ELEVATION

1976

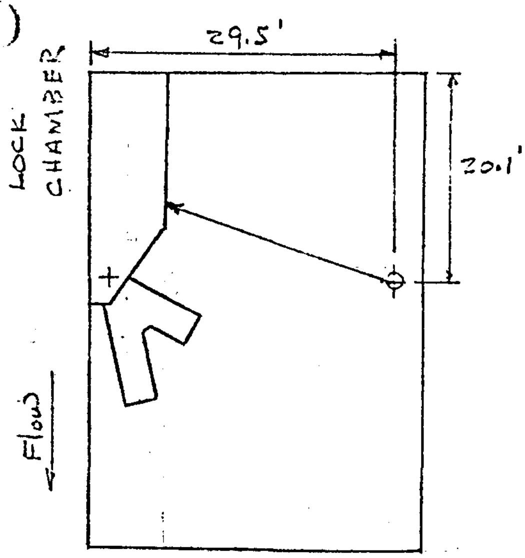
SONISCOPE

TEST RESULTS

MONOLITH NO. 16

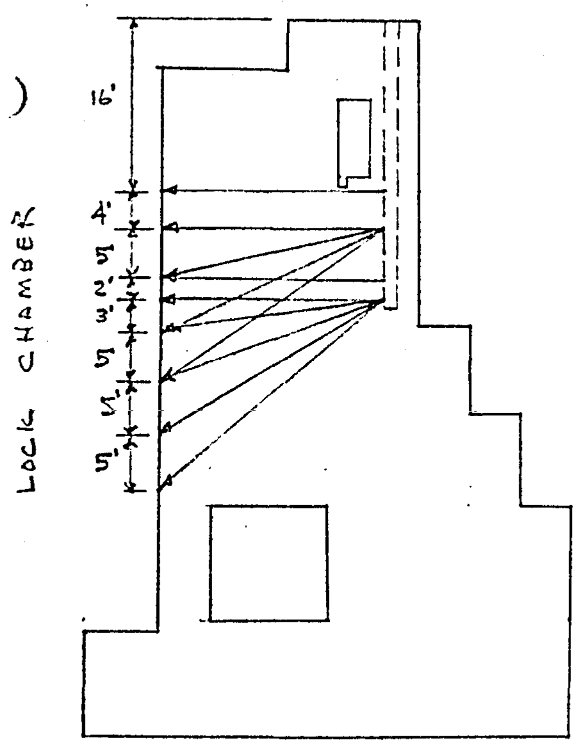
PLATE 4

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.



PLAN

Depth, ft.	Path Length, ft.	Pulse Velocity, fps
16	22.65	14,950
20	22.65	15,150
20-25	23.20	15,210
20-30	24.76	14,825
20-35	27.17	14,805
25	22.65	15,100
27	22.65	14,805
27-30	22.85	14,460
27-35	24.02	14,470
27-40	26.12	14,235
27-45	Noisy - No Signal	



ELEVATION

1976

SONISCOPE

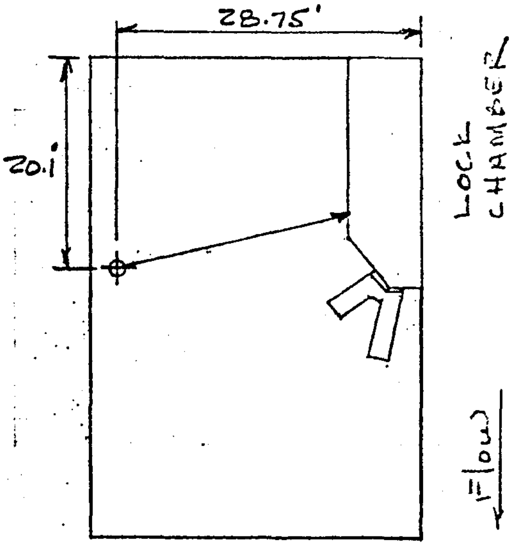
TEST RESULTS

MONOLITH NO. 13

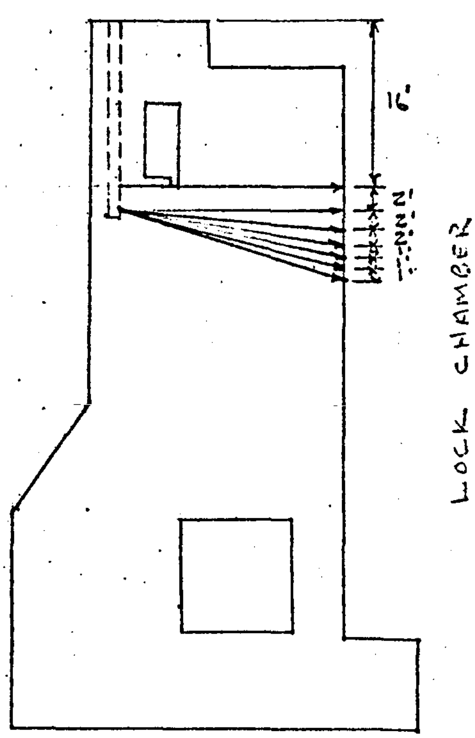
PLATE 5

WES FORM NO.

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.



PLAN



ELEVATION

Depth, ft.	Path Length, ft.	Pulse Velocity, fps
16	22.00	14,915
18	22.00	14,865
18-20	22.09	14,775
18-22	22.36	14,810
18-23	22.56	14,840
18-24	22.86	14,900
18-25	Faded out	

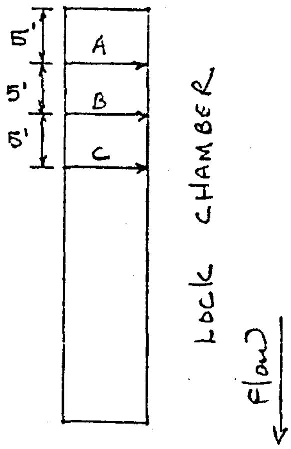
1976

SONISCOPE

TEST RESULTS

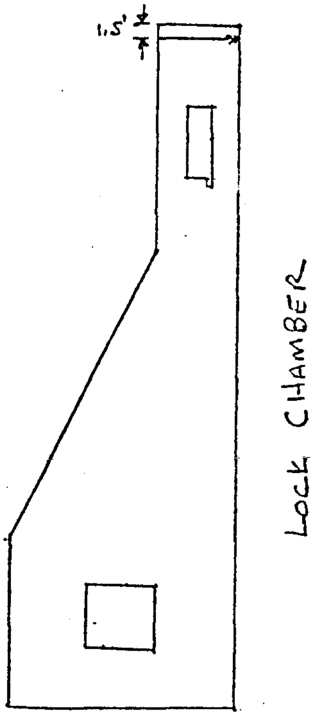
MONOLITH NO. 68

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET



PLAN

Depth, ft.	Test Path Designation	Test Path Length, ft	Pulse Velocity, fps
1.5	A	8.0	14,390
1.5	B	8.0	14,840
1.5	C	8.0	14,545



ELEVATION

1976

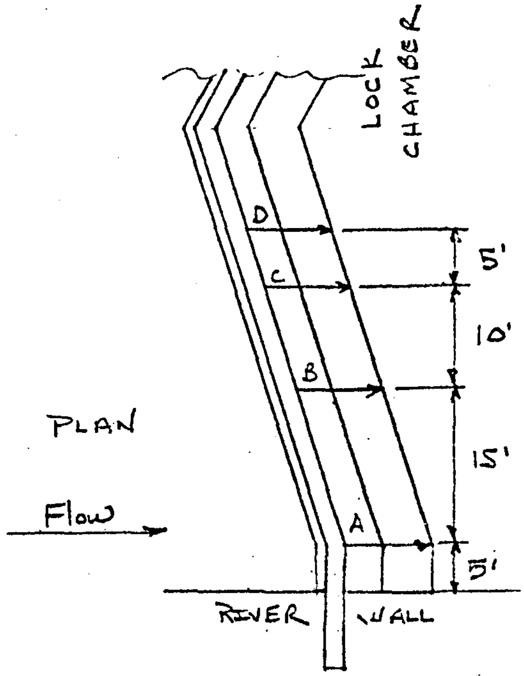
SONISCOPE

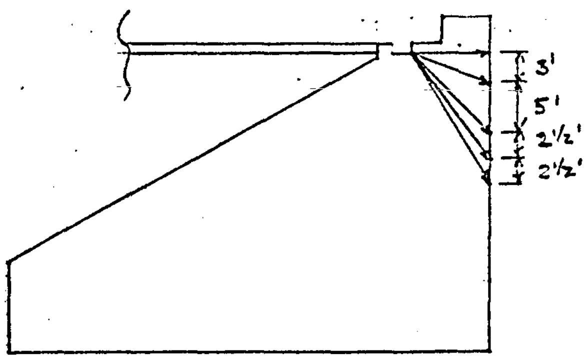
TEST RESULTS

MONOLITH NO. 72

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

Station	Depth, ft.	Path Length, ft.	Pulse Velocity, fps
A	0	8.00	14,545
	0-3	8.54	14,000
	0-8	11.31	14,800
	0-10.5	13.20	14,425
	0-12	15.26	14,815
B	0	8.00	14,680
	0-3	8.54	14,600
	0-8	11.31	15,390
	0-10.5	13.20	14,665
	0-12	15.26	14,890
C	0	8.00	15,240
	0-3	8.54	15,115
	0-8	11.31	15,600
	0-10.5	13.20	15,715
	0-12	15.26	15,570
D	0	8.00	15,240
	0-3	8.54	14,980
	0-8	11.31	15,495
	0-10.5	13.20	15,530
	0-12	15.26	15,895





ELEVATION

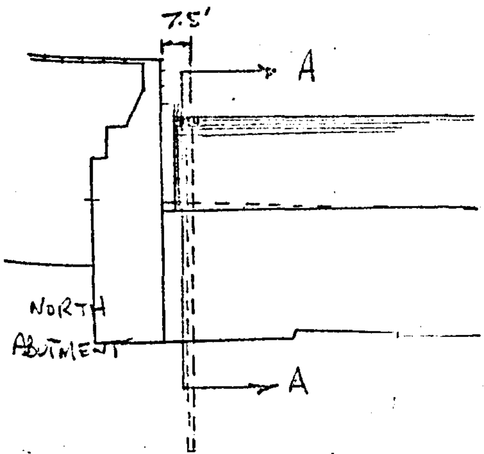
1976

SONISCOPE

TEST RESULTS

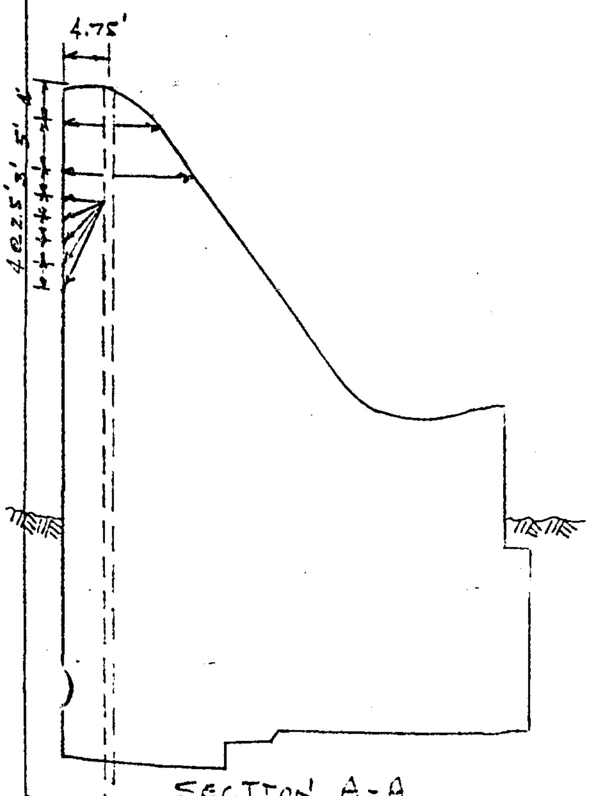
UPPER MITER SILL

SUBJECT:	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.



ELEVATION

Depth, ft.	Path Length, ft.	Pulse Velocity, fps
	Upstream	
4	4.42	13,590
9	4.42	13,590
12	4.42	12,990
12-14	4.35	14,055
12-16.5	6.31	15,195
12-19	8.28	15,615
12-21.5	10.48	15,635
	Downstream	
4	12.0	14,815
9	17.0	15,110



SECTION A-A

1976

SONISCOPE

TEST RESULTS

FIXED-CREST SPILLWAY

PLATE 9

APPENDIX A

HISTORICAL DATA

Historical Data Pertaining to Concrete Placed in the Tuscaloosa Lock and Dam

General

1. The Tuscaloosa Lock and Dam is located on the Warrior River at Tuscaloosa, Alabama, and was constructed by the Hardaway Contracting Company of Columbus, Georgia, under Government Contract No. W-569-eng. 1503, dated 2 December 1936. The placing of concrete was commenced 11 October 1937 and completed 15 September 1939.

Cement

2. All cement was furnished by the contractor, approximately 90,000 barrels being secured from the Alpha Portland Cement Company, and the balance, approximately 120,000 barrels, from the Penn-Dixie Cement Company. All tests of the cement were made by the Pittsburgh Testing Laboratory, Birmingham, Alabama under a contract with the construction contractor.

3. The Government contract for the construction of this lock and dam required that the cement "conform to Federal Specifications SS-C-191, for 'Cement; Portland', Oct. 14, 1930, with the following exceptions:

"Compound Composition: It shall contain not more than 55 percent Tri-Calcium-Silicate compound ($3CaO \cdot SiO_2$) nor more than 8 percent Tri-Calcium-Aluminate compound ($3CaO \cdot Al_2O_3$), computed from finished cement analysis in accordance with the method outlined by R. H. Bogue in Paper No. 21, Portland Cement Association Fellowship, on Calculation of the Compounds in Portland Cement."

4. Tests were made in the bins at the mill and a complete test was made of each 2,000 barrels of cement. Reports of all tests made were furnished the Government and a certificate was also furnished for all cars of cement delivered for use in the lock and dam, giving the number of the bin from which taken and the laboratory test numbers and dates. All cement used complied fully with the specifications. No High Early Strength Cement was used.

5. The cement was delivered to the site of the work in bulk. On arrival at the site the cement was discharged from the cars into a hopper from which it was conveyed by screw conveyor to a belt conveyor which transported it to a steel silo of 1,000 barrel capacity located at the mixers, from which it passed over batching scales to the mixers.

6. The first car of Alpha cement was received at the site 7 October 1937 and the last car 7 March 1938. The first car of Penn-Dixie Cement was received 7 March 1938 and with the exception of the cement remaining in the silo and two cars of Alpha cement containing a total of 673.7 barrels received on 7 March 1938, Penn-Dixie cement was used in all concrete placed after this date.

7. Alpha cement was used in the following monoliths:

Entire Monoliths Nos. 1, 2, 4, 5, 8, 9, 10, 12, 13, 14, 16, 19, 20, 21, 22-24, 25, 32 to 41 incl. and 51.

~~Top portion of Monoliths Nos. 2, 4 and 14.~~

Lower portion of Monoliths Nos. 3, 6, 7, 11, 15, 17, 18, 23, 26, 52 to 74 incl., 100 to 115 incl., 153, 154 and 156.

Penn-Dixie cement was used in all other monoliths or portions of monoliths.

Fine and Coarse Aggregate

8. The fine and coarse aggregates used consisted of natural sand and gravel and was secured from the Montgomery Gravel Company, Montgomery, Alabama. No admixture was used.

9. Practically all of this aggregate was taken from a pit located approximately 9 miles west of Montgomery. For a short period it was necessary to secure some of the larger sizes of the coarse aggregate from another pit in the same vicinity, with a few cars of this larger aggregate being secured from Selma, Alabama.

10. The primary pit consisted of an artificial lake near the east bank of the Alabama River. All of the sand was secured from this pit. The material was taken from the pit by hydraulic dredge without cutterhead and was discharged onto vibrating screens located on the bank. The screening plant could be adjusted to produce a very wide range of sizes and the materials, both sand and gravel, were thoroughly washed by high pressure jets while passing through the screen. Coarse aggregate was loaded into the railroad cars in two ranges in sizes, gravel ranging in size from No. 4 sieve to 1/2 maximum size (3/4 inch) being loaded into separate cars from gravel over 3/4 inch to maximum size (1-1/2 inch). On arrival at the site of the work, the coarse aggregate was dumped from the bottom dump cars into two batching bins, one bin for each range in size. From these bins it passed over batching scales to a belt conveyor which transported it to the batch hoppers at the mixers. There was no separation in the sizes of sand. On arrival at the site the sand was dumped into two batching bins and passed from these bins to the batching hoppers at the mixers in the same manner and on the same belt conveyor as the gravel.

11. To determine whether the pit was satisfactory as a source for the aggregate the Government engaged the Pittsburgh Testing Laboratory, Birmingham, Alabama, to examine the pit and to make tests of the materials, however all tests of the aggregate delivered for use in the work were made by the Southern Testing Laboratories, Birmingham, Alabama, under Government Contract No. W-569-eng. 1675, dated 9 September 1937. Sieve analyses of each car of aggregate were made by Government employees.

12. The first shipments of aggregate were received at the site during the month of September 1937 and for a short period the sieve analyses for acceptance purposes were made at the site of the work, after which the aggregates were analysed and accepted at the pit with the material being spot checked at the site.

13. Considerable difficulty was encountered in producing sand with the required quantity of fines, and some differences were found in the analyses at the pit and the spot check analyses at the site. This was thoroughly investigated and resulted in the analyses for acceptance purposes being again made at the site. After adjustments were made in the screens to produce the sand gradations required by the specifications, at both the pit and the site of the work, the sieve analysis for acceptance purposes was again made at the pit and spot checked at the site.

14. All chemical and physical test reports made by the Southern Testing Laboratories on aggregate indicated full compliance with the specifications with the very rare exception of an occasional report showing Plate II or Plate I-II for organic matter in the sand.

Mixing Water

15. Water used in all concrete mixes was from the waterworks system of the city of Tuscaloosa. The water for each batch of concrete was weighed on scales, for that purpose, located at the mixers. The quantity of water added at the mixer was the quantity required for the mix less the quantity of water in the aggregate.

Concrete

16. After numerous mixes were designed the mix adopted for Class B concrete consisted of 846 lbs. cement, 2460 lbs. sand, 1620 lbs. gravel No. 4 to 3/4 inch, and 2862 lbs. gravel over 3/4 inch to 1-1/2 inch, for a 2 cubic yard batch. This mix was used from the beginning of the concrete work to 23 November 1937. At this time the batch was changed from 2 cubic yards to 2.2 cubic yards, and consisted of 931 lbs. cement, 2600 lbs. sand, 1990 lbs. gravel No. 4 to 3/4 inch, 3100 lbs. gravel over 3/4 inch to 1-1/2 inch, which was continued until 5 January 1938. At this time a series of experiments were started in an effort to eliminate air pits which appeared on the faces of the concrete walls. These experiments consisted principally of increasing the cement content and varying the ratio of the two sizes of coarse aggregate. This experimental work continued until about 19 February 1938, at which time a mix consisting of 951 lbs. cement, 2600 lbs. sand, 1740 lbs. gravel No. 4 to 3/4 inch, and 3244 lbs. gravel over 3/4 inch to 1-1/2 inch was adopted and used with slight variations in the ratio of the two sizes of coarse aggregate until 18 May 1939. After this date all of the Class B concrete remaining to be placed was in the dam and the mix used consisted of 845 lbs. cement, 2400 lbs. sand, 1780 lbs. gravel No. 4 to 3/4 inch, 2682 lbs. gravel over 3/4 inch to 1-1/2 inch for a 2 cubic yard batch, with slight variations in the coarse aggregate for control purposes.

17. Very little Class A concrete was required. All compression tests using 6 inch by 12 inch cylinders indicated compression strengths at 28 days higher than the 3,000 pounds per square inch for Class B and 3,400 pounds for Class A concrete required by the contract.

18. The concrete was mixed in two 2 cubic yard mixers of the tilting type and was transported to the monoliths by cableway, cableway and whirler derrick, and by belt conveyor and tram track to whirler derrick which placed it in the form. The buckets were the ~~Blaw-Knox~~ Dravo cylindrical, bottom dump, with capacity of 3 cubic yards.

19. After the concrete was dumped from the buckets it was vibrated with approved vibrators to reduce the pile to the 24 inch maximum thickness of layer required by the contract and to secure thorough compaction. The vibrating was supplemented by hand spading adjacent to the forms.

20. When concrete was placed with the ambient temperature less than 35 degrees F. the contractor was required to heat the materials to produce a temperature of the concrete, when placed, of not less than 50 degrees F.

21. The depths of the concrete lifts varied from four feet to approximately eight feet with the depth between five and seven feet predominating. There was no limit for the time interval between the pouring of lifts and in a number of cases lifts were poured on successive days.

22. All concrete was kept wet during the 14 day curing period required by the contract by covering with water and sprinkling by means of perforated pipe.

Miscellaneous

23. There were no indications at any time that the quality of the concrete as to strength and durability was not adequate. The only question arising relative to the adequacy of the mix was in connection with the faces of the wall which contained numerous small air pits and an occasional small area of honeycomb. In an effort to eliminate the air pits various changes were made in the concrete mix consisting principally in increasing the cement content and changes in the ratio of large coarse aggregate to small coarse aggregate.

24. Several years (the exact time not known) after the completion of the structure small cracks appeared in the top surfaces and faces of the lock wall, and this cracking has increased progressively until the present time. The cracks did not reach serious proportions until this year (1947) when it was decided that a special investigation should be made to determine its cause and action necessary to prevent further deterioration.

Available Data

25. The following records relative to the concrete placed in the Tuscaloosa Lock and Dam are available in the District Office, Mobile, Alabama:

Test reports on cement by the Pittsburgh Testing Laboratory, Birmingham, Alabama.

Test reports on fine and coarse aggregates by the Southern Testing Laboratories, Birmingham, Alabama.

Sieve analyses of fine and coarse aggregate, by Government personnel and the Pittsburgh Testing Laboratory.

Tabulated record of cars of cement received giving date received, number of barrels, bin from which shipped, and analyses for each brand of cement.

Tabulated record of each day's pour of concrete giving date, monolith number, mix, slump, and quantity placed.

Concrete progress chart showing the number of each monolith and the date on which each concrete lift was poured.

26. There are attached hereto prints of typical sheets of the tabulated records and chart listed in the above paragraph.

Prepared 17 November 1947
by F. F. Gatlin, Principal Engineer

COMPARISON OF CHEMICAL ANALYSIS OF ALPHA AND PENN-DIXIE CEMENT USED IN THE TUSCALOOSA LOCK AND DAM

	Alpha					Penn-Dixie				
Bin No.	3	6	8	14	4	15	12	13	7	10
Silica	19.39	20.93	20.62	20.83	21.20	20.85	20.85	20.88	20.88	20.97
Iron Oxide	5.94	4.78	5.01	5.09	4.57	5.16	5.07	5.06	5.27	5.14
Alumina	6.24	5.73	5.93	5.65	5.73	5.80	5.80	5.87	5.90	5.74
Lime	60.70	59.72	61.77	61.57	61.93	62.16	61.85	62.41	62.36	61.63
Magnesia	2.90	2.65	2.86	2.74	2.86	3.82	4.16	3.53	3.50	3.89
Sulphuric-Anhydride	1.63	1.93	1.80	1.85	1.73	1.87	1.94	1.78	1.67	1.82
Tri-Calcium Aluminate	6.45	7.06	7.20	6.33	7.42	6.60	6.75	6.96	6.70	6.50
Tri-Calcium Silicate	44.65	41.20	42.44	41.77	41.07	42.90	41.52	43.50	43.18	40.44

Average

	Alpha		Penn-Dixie	
Silica	20.49	-	20.89	✓
Iron Oxide	5.08	-	5.14	✓
Alumina	5.86	✓	5.82	-
Lime	61.14	-	62.08	✓
Magnesia	2.80	-	3.78	✓
Sulphuric Anhydride	1.79	-	1.82	✓
Tri-Calcium Aluminate	6.89	✓	6.70	-
Tri-Calcium Silicate	42.23	-	42.30	✓

APPENDIX B

BOARD OF CONSULTANTS REPORT
NOVEMBER 1947

15 December 1947

COPY

SUBJECT: Investigation of Disintegration of the Concrete in Tuscaloosa Lock and Dam

FROM: Board of Consultants

MEMORANDUM TO: District Engineer
Corps of Engineers
Mobile District

1. In accordance with previous arrangements the undersigned assembled at Tuscaloosa on the morning of 24 November and inspected the operating gallery and all parts of the lock walls readily visible by drawing down the water in the lock. It was not possible to make an inspection of the dam, since water was flowing over the crest but the board was advised that when the crest of the dam was unwatered a short time ago there was no visible cracking on the crest or the downstream slope of the ogee.

2. After a close inspection of the various monoliths in the lock walls and a review of the data presented for use at this conference, it was evident that the cracking in monoliths built with Alpha Portland cement from Birmingham is in a more advanced stage than in the monoliths built with Penn-Dixie Portland cement. This statement holds regardless of the position of the cement in the monoliths in so far as surface indications are concerned. Until cores have been drilled from some of the monoliths affected by the cracking it will not be possible to determine whether there is any difference between the two ~~comments~~ cements relative to depth or extent of cracking in the interior.

3. There are no indications on the structure or in the data furnished that the workmanship or inspection was in any way responsible for the present condition of the lock. All concrete surfaces, whether horizontal or vertical, give evidence of well placed low slump concrete.

4. As a basis for determining the extent of repairs to the lock walls that should be undertaken at this time or in the near future, the following points were agreed upon after a thorough discussion of the various features involved in possible repair work:

a. Map all cracks in Blocks 4, 5, 6, 20, 51, 54, 55 and 60 by either pantographic methods or by photographic methods, marking all cracks with chalk in surfaces to be photographed so that the cracks will be readily visible.

b. Locate the center of all four lock-gate pintels by intersection from readily accessible permanent monuments so as to be able to measure all future movements due to temperature changes in the concrete of the monoliths or to further expansive cracking that may take place in the lock-gate blocks. All surveying done in connection with pintel intersection location should be accomplished between daylight and sun-up so as to avoid errors due to temperature variation after sun-up. Temperature records and weather condition should also accompany this work.

c. Make a crack width measurement survey of the top of the lock walls by photographic methods using a tripod to support the camera at a fixed distance above the top of the lock walls. A steel scale should be placed across the crack on the center line of the camera. The location of the center line of the camera for all such pictures should be suitably marked so that at any future time duplicate photographs can be taken and the variation in the crack width be determined by comparison. This survey should be accompanied by suitable measurements of the overall width of lock walls so that in the future a repetition of these measurements under similar temperature conditions will give a reasonable estimate of overall expansive growth of the concrete during the intervening period.

d. Drill two 6" diamond drill holes in both vertical faces of Block No. 60 in the cracked area constructed with Alpha cement. The depth of each hole should be sufficient to obtain at least three uncracked cores, each 18" in length.

e. Drill a 6" diamond drill hole at location selected for the initial 36" calyx hole in Gate Block 5 and follow the drilling of this 6" core with the drilling of the 36" core to El. 99 ±. The pieces of the 36" core should be assembled in sequence adjacent to the lock wall. A complete photographic record of the entire surface of the 36" calyx hole should be obtained and suitable prints made and transmitted to the members of the board as soon as practicable and prior to the next board meeting if possible. Cracks in the hole should be outlined with chalk and orientation of gate block indicated so that cracking can be evaluated relative to possible effect on stability of gate block. If the cracking in Block 5 is as deep seated as appears from the surface, it is proposed to recommend that this block be reinforced, the design of any necessary reinforcing to be determined after a careful engineering study of the plans by which this was constructed and the information provided by an inspection of the 36" core hole, and further discussion of this particular block at the next Board meeting.

f. Careful consideration was given to the possibility of waterproofing the top of the lock walls so as to prevent entrance of rain water into the cracked areas. Since a satisfactory method of waterproofing did not appear practical to all the members of the board, it was decided to postpone any recommendations relative to waterproofing until the January meeting of the Board.

g. Store all 6" cores in boxes in damp sawdust and retain at the site of the work until after the January meeting.

h. It is recommended that the Concrete Research Division of the Waterways Experiment Station be authorized to make such studies of cement, sand, gravel and concrete specimens from this lock as is necessary to determine the cause of the cracking and disintegration of the concrete in the lock walls. It is also recommended that a few 6" cores be drilled in the ogee crest of the dam to determine whether there is any evidence of the same type of cracking in the crest that is taking place in the lock walls. Due to more or less constant temperature, cracking in the crest may be in evidence under the microscope but not sufficiently advanced to be recognized by the naked eye.

i. In Block 51 cut the eye beams free from contact with the loose block of concrete at the upstream corner and tie this loose block back into solid concrete with reinforcement bars suitably embedded in the block and into the solid concrete.

j. It is recommended that an extensive crack survey of all blocks in the lock, except as noted herein, should be postponed for further consideration during the second meeting of the Board in January. At this time it is proposed to give careful consideration to a complete survey of all blocks and all cracks. By that time, or shortly thereafter, minimum temperature and maximum opening of the cracks for the winter season will be in effect.

5. The conference adjourned at 3:00 P.M. on 25 November with the understanding that the second meeting would be called by Mr. Catlin as soon as the 36" calyx hole in Block 5 was approximately completed. The tentative date for this meeting was suggested for the week of 12 January 1948.

6. The Board of Consultants was assisted in the inspection and in the discussions by Mr. Ivan L. Tyler, Portland Cement Association, Chicago, Illinois
Mr. C. C. Olsen, Portland Cement Association, Chicago, Illinois
Mr. Clifford, Reed, Jr., South Atlantic Division
Mr. Bryant Mather, Concrete Research Division, Waterways Experiment Station

(SIGNED) H. K. COOK

F. F. CATLIN

J. C. SPRAGUE

B. W. STEELE, Chairman

APPENDIX C

BOARD OF CONSULTANTS REPORT
JANUARY 1948

ADDRESS REPLY TO:
CHIEF OF ENGINEERS
WAR DEPARTMENT
WASHINGTON 25, D. C.

REFER TO FILE No. ENGEE

WAR DEPARTMENT
OFFICE OF THE CHIEF OF ENGINEERS
WASHINGTON 25, D. C.

18 March 1948

SUBJECT: Investigation of Disintegration of the Concrete Walls in Tuscaloosa Lock and Dam

FROM: Board of Consultants

TO: The District Engineer
Corps of Engineers
MOBILE, ALABAMA

1. The undersigned in accordance with the agreement made during the meeting at Tuscaloosa in November, assembled for a second meeting on the morning of 13 January and spent the following two days in inspecting the results of core drilling operations and in discussing further investigations and the necessity for repair work.

2. After a careful inspection of the 36" calyx core and the calyx hole, there does not appear to be any necessity for immediate repairs to monolith #5, which is the upstream lock gate monolith on the land side of the lock and is the gate monolith that is the most effected by surface cracking.

3. Although the cracking in monolith #5 is not as serious as it appeared from surface indications before the 36" hole was drilled, it is believed that the internal growth in the concrete will continue and that the cracking will be progressive. Hence a thorough system of checks on gate pintle center line movements in any direction should be developed and periodic measurements made by the Tuscaloosa Operations Forces.

4. It is believed that with the completion of the 6" hole in monolith 3, the cores now available are sufficient for the investigations to be conducted at Clinton and at Mariamont Laboratories.

5. In view of the probable continued growth of concrete and development of cracks it is believed that the District should evaluate briefly the cracking on all monoliths as of today for use in comparison of crack development in later years.

6. In view of the non-appearance of any type of cracking in the 36" hole or of any indication that structural cracks might develop because of the 36" hole, it is believed that it would be desirable to leave this calyx hole open for future observation and that it should be suitably covered with a door that can be readily opened for inspection at anytime.

7. Further consideration was given to the possibility of water-proofing the top of the lock walls. After a thorough discussion it was concluded

that any such attempt would serve no useful purpose and would prevent the opportunity of future observation of crack development.

8. In addition to the monoliths mentioned in the Board's report dated December 15, it has been decided to add Monoliths 1, 12 and the lower portion of 74 to the monoliths selected for detailed photographs. The structural cracks, as well as the pattern cracking in monolith #1 are of particular interest and may be worth watching and measuring if concrete growth during the coming warm weather is as accelerated in the summer season as it appeared to be during the summer of 1947.

9. After a thorough discussion of a system of recording monolith conditions of cracking, it was decided to leave this detail to Mr. Gatlin to work out at the site.

10. Due to the importance of the work load at Clinton, the studies to be made by the CRD will not take priority over any pending investigation on which contract work depends and hence a report by Clinton on the laboratory studies of the Tuscaloosa cores will probably not be issued for several months.

11. It is the understanding of the Board that the CRD will make a report to the Board after its studies are completed and that the Board will submit a report to the District Engineer relative to the causes of the cracking sometime later this year, preferably after the effect of hot weather and moisture during the coming summer season are in evidence.

12. The following were present part of the time during the inspection and discussion:

Mr. Ivan L. Tyler, Portland Cement Association
Chicago, Illinois

Mr. C. C. Olsen, Portland Cement Association
Chicago, Illinois

/s/ Herbert K. Cook
H. K. COOK

F. F. GATLIN

J. C. SPRAGUE

B. W. STEELE, Chairman

APPENDIX D

LABORATORY INVESTIGATION OF CONCRETE DISINTEGRATION AUGUST 1949

CORPS OF ENGINEERS, U. S. ARMY

****

DISINTEGRATION OF CONCRETE FROM

TUSCALOOSA LOCK AND DAM

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

****

25 August 1949

Disintegration of Concrete from

Tuscaloosa Lock and Dam

1. Correspondence from the Division Engineer, South Atlantic Division, dated 27 October 1947, subject: "Tuscaloosa Lock and Dam" stated that the concrete in the lockwalls of the Tuscaloosa Lock and Dam had been inspected and that portions of it had been found to be in an advanced stage of disintegration. This letter also announced the decision to appoint a Board to study and report on the condition of the concrete and to recommend remedial measures.

2. Correspondence from the District Engineer, Mobile District, dated 17 November 1947, forwarded copies of "Historical Data", "Cement Data", "Combined Concrete Record", and "Concrete Progress Chart." These data record that the Tuscaloosa Lock and Dam was constructed between 11 October 1937 and 15 September 1939; that the cement used consisted of 90,000 bbl. from the Alpha Portland Cement Co. and 120,000 bbl. from the Penn-Dixie Cement Co.; that the cement was required to conform to Federal Specifications SS-C-191 except that it should contain not more than 55 per cent tricalcium silicate and not more than 8 per cent tricalcium aluminate; that the actual calculated tricalcium aluminate content ranged from 7.42 to 6.33 per cent and averaged 6.39 for the Alpha Cement, ranged from 6.96 to 6.50 and averaged 6.70 for the Penn-Dixie Cement; that the mixing water came from the Tuscaloosa city water supply; and the aggregates consisted principally of natural sand and gravel from the Montgomery Gravel Co. pit located approximately 9 miles west of Montgomery, Ala. on the east bank of the Alabama River. It is understood that the Penn-Dixie Cement was manufactured at

Richard City, Tennessee, and the Alpha Cement at Birmingham, Alabama.

3. The meeting of the Board held at Tuscaloosa, Ala. on 24-25 November 1947 developed the following points as recorded in its report dated 15 December 1947:

a. Cracking is more advanced in monoliths built with Alpha cement than in those built with Penn-Dixie.

b. Cores should be drilled from the structure.

c. A crack survey should be made.

4. The meeting of the Board on 13-15 January 1948 developed the following points as recorded in its report dated 18 March 1948:

a. The cracking in Monolith #5 is not as serious as it appeared from surface indications but it is believed that the internal growth of the concrete will continue and that the cracking will be progressive.

b. The studies to be conducted by the Concrete Research Division of the Waterways Experiment Station should not take priority over pending investigations for other projects.

5. Correspondence from the District Engineer, Mobile District, dated 12 February 1948, subject: "Investigation of Disintegration of the Concrete in Tuscaloosa Lock and Dam" requested that this office conduct tests to determine the cause of the cracking and disintegration of the concrete in the lock walls.

6. In accordance with arrangements made between the Office, Chief of Engineers and the Portland Cement Association tests were conducted on 16-19 June 1948 using the wave velocity apparatus owned by the Association to determine the velocity characteristics of the concrete in monolith 5. In accordance with a request from the Portland Cement Association this office

determined values for shear modulus of selected cores to provide data to be used in calculations based on wave velocity. The results of these tests are given in a later section of this report.

7. In accordance with the request contained in the correspondence referred to in par. 5 above, this office undertook a program of investigation involving petrographic and other studies of the concrete specimens taken from the lock wall and shipped to this laboratory in accordance with correspondence from the Resident Engineer, Tuscaloosa, Ala., dated 4 February 1948. The results of the petrographic study are set forth in summary and are detailed on the attached LMW form 557 and inclosures thereto. The results of the other studies are given below.

8. Strength and Elastic Properties. Samples of 4 3/4-in. diameter cores extracted from various monoliths of Tuscaloosa Lock and Dam were received 4 February 1948. Monoliths represented are 3, 5, 20, and 60. All cores except those from Monolith 60 were drilled vertically; those from Monolith 60 were drilled horizontally. Core lengths varied from about 6 to 40 in. All cores contained the same aggregates, and all except those from Monolith 3 from depths less than 10 ft contained the same cement. All sections of 4 3/4-in. core of lengths greater than 10 in. were tested for dynamic modulus of elasticity, in an as-received condition. This was done by averaging the lengths determined at several points of the end surfaces for each core, determining the resonant fundamental flexural frequencies, and computing the moduli. Following this, six representative cores were taken, and the ends sawed plane on a diamond cut-off wheel. Fundamental flexural and torsional frequencies were determined on these cores. Moduli of elasticity and rigidity, and Poisson's ratio were calculated from these readings.

The moduli of elasticity determined on the cores in the as-received condition are given in Table 1. This table gives also the average lengths, weights, and flexural frequencies of these cores.

Table 2 shows the moduli of elasticity and rigidity, lengths, weights, and fundamental flexural and torsional frequencies, for the six selected cores. The table also gives Poisson's ratio for these cores.

The results given in Tables 1 and 2 are the best that have been obtained. The flexural frequencies and moduli of elasticity are believed to be fairly accurate in most cases but due to the averaging of lengths on all cores, and the existence of cracking (visually determined) in several, the moduli of elasticity should be considered merely indicative of the state of the concrete, and not exact. In addition internal cracking is quite probable in several of the cores tested, which would lead to less precise readings of flexural frequencies.

Compressive strength and modulus of elasticity (static) were determined on nine sections of core. These data are given in Table 3.

9. Wave Velocity: The wave velocity tests referred to in paragraph 6 above provided additional data on the strength and elastic properties of the concrete. These tests were conducted with an apparatus consisting essentially of a pulse repetition oscillator, a transmitter, a timing wave circuit connected to an oscilloscope, and suitable amplifiers. Data were developed from these tests from which values for Young's modulus were calculated. The most interesting results were obtained from tests on monolith 5 at various depths from 5 ft to 35 ft in the 36-in. diameter calyx core hole. A progressive increase in velocity from 12,200 ft/sec at the 5-ft depth to 15,110 ft/sec at the 35-ft depth was found. The

velocities correspond to values of Young's modulus of from approximately 3.7 to approximately 5.8 x 10^6. The individual values are tabulated below:

Data on Monolith 5, Tuscaloosa
Lock and Dam

Depth feet	Velocity	Young's Modulus x 10^{-6} psi (a)	Young's Modulus x 10^{-6} psi (b)
5.0	12,200	3.9	3.5
8.5	13,890	5.3	4.7
12.0	14,390	5.8	5.2
16.0	14,610	6.0	5.3
20.0	14,890	6.1	5.4
25.0	14,910	5.8	5.2
29.0	14,810	6.1	5.4
33.0	14,930	6.2	5.5
35.0	15,110	6.2	5.5

(a) Computed as a massive structure
(b) Computed as a slab

All the data developed from these tests are given in Table 4.

10. These data are discussed by Prof. R. W. Carlson in a memorandum entitled "Wave Velocity Apparatus and its Place in Non-Destructive Testing of Concrete in Place", dated 17 November 1948 prepared for the Office, Chief of Engineers, as follows:

"The Tuscaloosa Lock Wall is so badly cracked that the natural impression would be that it is about to collapse. But the wave velocity tests tell a different story and probably the true one. The wave velocity was found to vary from 15,110 ft per second at a depth of 35 feet below the top, to 12,200 at a depth of 5 feet. The change was systematic and gradual, except near the top where the change was more rapid. These results indicate practically sound concrete for most of the wall, with serious internal disintegration only near the top. The conclusion is that most of the cracking is confined to the surface and that the interior is sound. The systematic variations in wave velocity indicate that much more could have been learned had earlier measurements been made for reference. Small differences in wave velocity can be measured reliably."

11. Volume Change: Three 6-in. diameter cores were drilled in the Concrete Research Division laboratory from the top of the 36-in. calyx section from hole 5-2 (Monolith 5). These cores were fitted with inserts to permit measurements of length change, soaked in water, and stored at 100 F, over or immersed in water. Measurements to an exposure period of 1 year indicate a progressive expansion for a relatively short period of time followed by equilibrium as shown below:

Specimen No.	Exposure	Maximum Expansion, at one year per cent	Length of Exposure to reach maximum, days
1	Over water	+ 0.08	11 days
2	In water	+ 0.06	80 days
3	Over water	+ 0.06	80 days

12. Aggregate: Pertinent data on the aggregate present in the concrete are given in the attached petrographic report. The information contained therein may be compared with that given in the petrographic report forwarded by this office to the South Atlantic Division Laboratory with correspondence dated 1 June 1948, subject: "Reports of Tests on Aggregates, Tennessee - Tombigbee Project", covering a sample of gravel from the Roquemore Gravel Co., Montgomery, Ala. This sample was examined in the sizes from 1 in. to No. 4 and had more than 50 per cent quartz in all sizes smaller than 1 in. and 46 per cent quartz in the fraction retained on 1 in. Chert was present as follows: 39 per cent on 1 in., 31 per cent on 3/4 in., 35 per cent on 1/2 in., 22 per cent on 3/8 in., and 36 per cent on No. 4. The remainder

of the sample consists of quartzite, sandstone, and granite. Chalcedony was found in one or two of the chert particles; most of the particles which were powdered and examined were found to contain no material with an index of refraction lower than that of quartz. Physical tests on sand and gravel samples from the Roquemore deposit are summarized below:

Bulk specific gravity, saturated
surface dry: sand = 2.65, gravel = 2.62

Absorption, per cent: sand = 0.4, gravel = 0.7

Percentage of particles lighter than 2.40
after 5 hr boiling: gravel = 2.1

Loss after 5 cycles of magnesium sulfate
soundness test, per cent: sand = 4.7, gravel = 4.0

Linear coefficient of thermal expansion of
sand mortar x 10^6 per deg F = 7.4

Durability factor of concrete after 300 cycles
of accelerated freezing and thawing = 46

13. Chemical Data: Results of the quick chemical test for reactivity of aggregates sampled from the concrete are given in the attached petrographic report. Information supplied from the Geological Survey of Alabama by Stewart J. Lloyd, Assistant State Geologist, states that an analysis of a sample of water from the Warrior River above Tuscaloosa, Alabama, showed a sulfate content of 10 parts per million of the SO_4 radical. This is not regarded as an excessive amount since it is reported that the city water supply of Birmingham, Ala. on analysis in 1932 showed 61 ppm of SO_4.

Two samples of the gel reaction product from the Tuscaloosa lock wall concrete were subjected to chemical analysis. Sample No. 1 consisted of material scraped from the wall of the 36-in. core hole in Monolith 5 on 16-19 June 1948. Sample No. 2 was collected from pockets in 5 3/4-in.

cores from holes 20-1 and 60-1 after the concrete had dried in laboratory air. The results of these analyses are as follows:

	Per Cent	
	Sample No. 1	Sample No. 2
Moisture loss at 105 C	34.6	9.81
Composition calculated on dry weight:		
SiO_2	49.82	61.73
CaO	21.11	12.28
Al_2O_3	1.15	2.17
Fe_2O_3	0.81	0.47
SO_3	0.16	0.00
Insoluble	5.58	8.1
Ignition loss	-	14.84
Calculated calcium sulfoaluminate	0.47	0.0

14. Conclusions: It is concluded that the disintegration and cracking has been caused, at least primarily, by deleterious chemical reaction between the alkalies in the cement and unstable silica in the aggregate. A study of aggregate particles from the concrete has revealed that approximately 70 per cent of the chert pebbles consist, at least in part, of the material known as "chalcedony", which contains opal. Chalcedony is known to be one of the materials which is capable of participating in a deleterious chemical reaction in concrete. The study of the aggregate has not revealed the presence of any other constituent which is regarded as capable of participating in such a reaction.

The study of the concrete specimens has confirmed the indications developed from the examination of the structure and the physical tests of the concrete both on specimens and in the structure, that the cracking is largely confined to near surface zones and is more pronounced in those portions of the structure in which it is reported that Alpha cement was used. Although specific data on the alkali contents of the cements used in this project are not available, it is regarded as probable that the Alpha cement contained a larger percentage of alkalies than did the Penn-Dixie. The microscope examination of the concrete specimens reveals, however, that gel, which is the characteristic product of the chemical reaction, is present in all of the concrete specimens examined without regard to the brand of cement used or the depth in the structure.

The concrete is characterized not only by the presence of the gel referred to above, but also by the presence of deposits of crystals of calcium sulfoaluminate. Calcium sulfoaluminate is the normal product of the reaction of the gypsum which is interground with the cement for the purpose of controlling time of set and the calcium aluminate in the cement. In normal, non-deteriorated concrete the calcium sulfoaluminate is widely distributed in the cement paste and does not appear as deposits of crystals. Concrete which has suffered deterioration from any cause whatsoever frequently exhibits crystalline deposits of this material. Unless it can be shown that additional sulfate has been provided from an external source for further reaction with the aluminate portion of the cement, such deposits do not indicate that the concrete containing them has undergone any deterioration due to sulfate attack. In the case of the Tuscaloosa concrete, since the available information does not indicate an additional source of sulfate, and since the evidence of deterioration due to alkali-chalcedony reaction is thoroughly established, it is not regarded as likely that there was a significant sulfate-attack factor involved in this occurrence.

TABLE 1

DYNAMIC MODULUS OF ELASTICITY OF CONCRETE CORES

TUSCALOOSA LOCK AND DAM

Specimen	Depth	Modulus of Elasticity $E \times 10^{-6}$ psi
3-1	0.0	6.40
3-1	3.0 - 4.2	6.09
3-1	8.2	6.31
3-1	11.8	6.08
Average		6.15
5-1	0.0 - 1.5	2.40
5-1	6.0 - 7.5	1.51
5-1	7.5 - 9.0	4.33
5-1	9.0 - 10.0	4.15
5-1	12.5 - 13.5	5.90
5-1	18.9 - 23.8 Sect 1	5.71
5-1	18.9 - 23.8 Sect 2	5.26
5-1	23.8 - 28.8 Sect 1	6.50
5-1	23.8 - 28.8 Sect 2	5.65
5-1	28.8 - 33.5 Sect 1	5.31
5-1	28.8 - 33.5 Sect 2	5.56
5-1	33.5 - 39.6 Sect 1	6.03
5-1	33.5 - 39.6 Sect 2	5.28
5-1	33.5 - 39.6 Sect 3	6.26
Average		4.99
20-1	3.7 - 6.9	3.34
Average		3.34
60-1	0.0 - 4.8 Sect 1	2.59
60-1	0.0 - 4.8 Sect 2	3.26
60-1	4.8 - 10.2 Sect 1	3.85
60-1	4.8 - 10.2 Sect 3	3.74
Average		3.36
60-2	0.0 - 4.8 Sect 1	2.00
60-2	0.0 - 4.8 Sect 2	3.30
60-2	4.8 - 10.2 Sect 1	3.26
60-2	4.8 - 10.2 Sect 2 (Part 1)	4.34
60-2	4.8 - 10.2 Sect 2 (Part 2)	4.45
Average		3.47

TABLE 1 (Concluded)

Specimen	Depth	Modulus of Elasticity $E \times 10^{-6}$ psi
60-3	0.0 - 6.9 Sect 1	2.60
60-3	0.0 - 6.9 Sect 2 (Part 1)	2.75
60-3	0.0 - 6.9 Sect 3	4.11
Average		3.15
60-4	0.0 - 4.8 Sect 1	1.90
60-4	0.0 - 4.8 Sect 2	3.45
60-4	4.8 - 8.2 Sect 2 (Part 1)	0.99
60-4	4.8 - 8.2 Sect 2 (Part 2)	1.03
Average		1.86

TABLE 2

DETERMINATION OF POISSON'S RATIO OF CORES FROM TUSCALOOSA LOCK & DAM

Monolith No.	Section and Core No.	Depth (ft.)	Length (in.)	Weight (lb.)	Fundamental Frequencies cps		Moduli x 10^{-6} psi		Poisson's Ratio
					Flexural	Torsional	E	G	
3(1)	3-1	14.8	14	21.6	2790	3050	5.8	1.6	0.775
5	5-1 (Upper)	13.5-18.9	$13\frac{1}{4}$	20.1	2510	3100	3.8	1.5	0.275
5(2)	5-1 (Middle)	13.5-18.9	$10\frac{7}{8}$	16.8	3450	4150	3.9	1.8	0.600
20	20-1 (Lower)	6.9- 9.4	$10\frac{1}{2}$	16.5	3450	3750	3.6	1.4	0.249
60	60-1 (Inner)	4.8-10.2	$13\frac{1}{4}$	20.2	2320	2900	3.3	1.3	0.244
60	60-4 (Outer)	4.8- 8.2	14	21.2	2310	3030	3.9	1.6	0.233

(1) Core #3 - Poisson's ratio is high. Torsional frequency is high, and True Torsional maximum may be masked by flexural maximum.

(2) Core #5-1(Middle) - Torsional frequency is high. This section believed to have internal cracking.

TABLE 3

Compressive Strength and Static Modulus

TUSCALOOSA LOCK AND DAM

Core Tests (4.75" Diameter Cores)

Core No.	Hole No.	Depth Ft.	Compr. Str. psi	Mod. Elast. psi
Con-1 (1)	3-1	11.8-13.7	6850	4,640,000
Con-1 (2)	3-1	4.2- 6.5	7040	6,782,000
Con-2	5-1	19.0-23.0	5140	4,520,000
Con-3	20-1	3.7- 6.9	4265	2,024,000
Con-4	60-1	4.8- 7.0	3520	2,180,000
Con-5 (1)	60-2	0- 4.8	2575	1,242,000
Con-5 (2)	60-2	4.8-10.2	3785	2,710,000
Con-6	60-3	0- 6.9	2890	1,360,000
Con-7	60-4	0- 4.8	3005	1,940,000

Note: All cores showed white deposits. Very few deposits in 60-2 (4.8-10.2) and in 3-1 (11.8-13.7)

6-in. Diameter Core Drilled from 36-in. Core, Lower Section

Con-8 (1)	5200	2,685,000

TABLE 4

WAVE VELOCITIES IN TUSCALOOSA LOCK

(Data Provided by Portland Cement Association)

Date, 1948	Monolith & Test Number	Location of Test	Wave Velocity
6/17	5-1	35' below top of 36" core hole to lock wall	15,110
	5-2	33' " " " " " " " " "	14,930
	5-3	29' " " " " " " " " "	14,810
	5-4	25' " " " " " " " " "	14,910
	5-5	20' " " " " " " " " "	14,890
	5-6	16' " " " " " " " " "	14,610
	5-7	12' " " " " " " " " "	14,390
	5-8	8.5' " " " " " " " " "	13,890
	5-9	5' " " " " " " " " "	12,200
6/16	6-1V*	46.6' from D.S. Joint, 5' S. of lock wall	14,100
	6-2V	18' from D. S. Joint	14,100
	6-3V	3' from D. S. Joint	13,100
8/19	6-1+3H**	43.6' from D. S. Joint	14,620
	6-1+6H	40.6' " " "	14,620
	6-2H	17.7' " " "	14,300
	6-3H	3' " " "	14,300
	6-AS	These tests made on surface of monolith along	12,000
		N-S line 1.5' from D.S. Joint. Transmitter	7,240
	6-BS	was held against S. face of monolith, 3" below	12,250
		surface. Receiver held at points along line	7,110
	6-CS	3', 6', 9', 12' and 15' from S. edge. (Second	12,670
		velocity is that of the Rayleigh Wave.)	7,430
	6-DS		12,830
			7,370
	6-ES		12,340
			7,250
6/16	7-4V	3' from U.S. Joint	13,820
	7-5V	22.65' from D.S. Joint	14,360
	7-6V	8' from D. S. Joint	14,100
6/18	7-4H	3' from U.S. Joint	13,960
	7-5H	22.65' from D.S. Joint	14,620
	7-6H	8' from D.S. Joint	14,620
6/16	8-8+4V***	13.05' from D.S. Joint	11,650
6/18	8-7H	8.4' from U.S. Joint	14,200
	8-8H	17.05' from D.S. Joint	13,960
	8-9H	3' from D.S. Joint	13,300

Date, 1948	Monolith & Test Number	Location of Test	Wave Velocity
6/18	9-10H	3' from U.S. Joint	13,960
	9-11H	Center of Monolith 9	13,960
	9-12H	3' from D.S. Joint	13,330
6/18	10-13H	3' from U.S. Joint	13,330
	10-14H	Center of Monolith 10	12,780
	10-15H	3' from D.S. Joint	13,330
6/18	11-16H	3' from U.S. Joint	14,200
	11-17H	Center of Monolith 11	14,200
	11-18H	3' from D.S. Joint	13,960
	11-17AH***	6" below 11-17H	14,200
6/19	11-16V	3' from U.S. Joint	14,180
	11-17V	Center of Monolith 11	14,060
	11-18V	3' from D.S. Joint	No signal
6/19	12-19H	3' from U.S. Joint	13,330
	12-20H	Center of Monolith 12	13,050
	12-21H	3' from D.S. Joint	13,640
6/19	13-22H	3' from U.S. Joint	15,000
	13-23H	Center of Monolith 13	13,960
	13-24H	3' from D.S. Joint	13,960
6/19	14-25H	3' from U.S. Joint	13,960
	14-26H	Center of Monolith 14	13,640
	14-27H	3' from D.S. Joint	13,960
6/19	20-28H	28.5' from D.S. Joint, 9' below top of Monolith	13,890
	20-29H	18.5' " " " " " " " "	13,890
	20-30H	3' " " " " " " " "	14,700
		(all H shots in 20 made to S. face of monolith)	
	~~20-30V~~	3' from D.S. Joint	12,880
	20-30-3V	6' " " "	12,160
	20-30-1.5V	4.5' from D.S. Joint	12,330
	20-30+1.5V	1.5' " " "	12,880
	20-30-10V	13' " " "	11,850
6/19	60-34	3' from D.S. Joint, center 8th lift from top	13,030
	60-35	3' from D.S. Joint, " 7th " " "	14,000
	60-36	3' from U.S. Joint, " 8th " " "	12,930
	60-37	3' from U.S. Joint, " 7th " " "	13,870
	60-31H	3' from U.S. Joint, 3.25' below tunnel roof	13,050
	60-32H	17.75' from U.S. Joint, 3.25' below tunnel roof	13,050
	60-33H	3' from D.S. Joint, 3.25' below tunnel roof	13,050
	60-31V	3' " U.S. "	12,860
	60-32V	17.75' from U.S. Joint	12,860
	60-33V	3' from D.S. Joint	12,960

* - Letter "V" denotes shot made in vertical direction from tunnel roof. Unless otherwise noted, all "V" shots made over ℄ of tunnel.

** - Letter "H" denotes shot made in horizontal direction from tunnel wall to lock wall. Unless otherwise noted, all "H" shots made 4.3' above floor level of tunnel.

*** - This was the only location in Monolith 8 at which a vertical shot was successfully attempted.

**** - Routine shots in Monolith 11 were in 3rd lift from top. Shot 11-17AH was in 4th lift from top.

PETROGRAPHIC REPORT

CORPS OF ENGINEER } S. ARMY MISSISSIPPI RIVER COMMISSION WATERWAYS EXPERIMENT STATION	PETROGRAPHIC REPORT	CONCRETE RESEARCH DIVISION P. O. BOX 217 CLINTON, MISSISSIPPI	
SYMBOL: MOB-4 6078	PROJECT: Tuscaloosa Lock and Dam	DATE REPORT SUBMITTED: 23 Aug 1949	INITIALS: KM
SERIAL NO: MOB-4 CON-1 through CON-9	SOURCE: Tuscaloosa Lock		

1. Samples. The samples consist of cores extracted from the subject structure described as follows:

CRD Ser. No. MOB-4	Diameter, in.	Hole No.	No. of Boxes	Approximate Length, ft.
CON-1	4 3/4	3-1	2	10.5
2	4 3/4	5-1	6	38.7
3	4 3/4	20-1	2	8.0
4	4 3/4	60-1	2	8.8
5	4 3/4	60-2	2	9.1
6	4 3/4	60-3	1	6.6
7	4 3/4	60-4	2	8.2
8	36	5-2	-	2
9	36	5-2	-	5

The first number in the identification of each hole is the monolith number. All of the samples were received at this office by truck from Tuscaloosa on 4 February 1948. All of the samples received were taken from vertical holes except those from monolith 60 which were taken horizontally. All of the cores received contain the same aggregates, and all contain Alpha cement except those from hole 3-1 from depths less than 10.0 ft.

2. Summary. Petrographic examination has been made of a total of seven 4 3/4-in. and two 36-in. diameter cores from monoliths 3, 5, 20, and 60 of Tuscaloosa Lock, Tuscaloosa, Ala. All of the cores contain visible cracks of various widths in the upper or surface sections; some of them contain wide weathered cracks. All of the cores, regardless of brand of cement, contain deposits of gel in voids, cracks, and aggregate particles in all sections, regardless of depth from the upper or outer surface of the structure. All of the cores also contain calcium sulfoaluminate in all sections. However, the widest fresh cracks are coated with gel and the adjoining paste is soaked with gel. In all of the concrete, there is more gel than there is sulfoaluminate; more voids are filled with gel than with sulfoaluminate. Sulfoaluminate is most developed in the most highly leached and cracked concrete. It is therefore believed that the gel produced by alkali-aggregate reaction is the major cause of the cracking of the upper and outer portions of the cores.

Chalcedonic chert is the only constituent found in the coarse or fine aggregate which is known to be capable of deleterious reaction with the minor alkalies of portland cement. Such chert is a much more important constituent of the coarse aggregate than of the fine aggregate. Based on counts of over 5000 particles on sawed surfaces of the 4 3/4-in. cores 42 per cent of the total coarse aggregate is chert and 58 per cent consists of other types, principally highly metamorphic quartzite and vein quartz. The fine aggregate is principally quartz, with feldspar and some chert. Determinations of 68 chert pebbles close to gel pockets showed that 71 per cent of them consisted of or contained chalcedony. Since the pebbles were selected because they were close to gel, it is believed that they give too high a figure for the chalcedony content of the coarse aggregate. Assuming that 50 to 70 per cent of the total chert is chalcedonic, the percentage of chalcedony in the coarse aggregate would amount to 21 to 30. The average of 40 determinations of index of refraction of the chalcedony particles is 1.5365. Using this value and assuming chalcedonies of varying opal content, the opal content of the coarse aggregate is estimated as 1 to 5 per cent. Further confirmation of the hypothesis that the chalcedonic chert is the reactive constituent is found in the preferential association of gel with chert rather than with any other of the types of aggregate.

While cracks and gel are most developed in the upper and outer sections of the core, all sections of all cores contain gel, and gel grew on the core surfaces after they were drilled and stored in a damp condition. The late gel grew more abundantly on the middle and lower sections of the cores. This fact suggests that while reaction in the structure had only gone far enough to crack the concrete near the surface, all the necessary ingredients for reaction are present in the deeper concrete and that the production of gel in this concrete is accelerated when a source of moisture is provided. As an example of the speed with which reaction can develop, under favorable conditions, a thin section blank from core 5-1, section 7.5 - 9.0 ft, was ground smooth and exposed to laboratory air (warm, humid) for about 36 hr. When the blank was examined, it was found that three gel pockets and one chalcedony particle on the ground surface had taken up water from the air and swelled up perceptibly above the general level of the ground surface. The concrete containing Alpha cement shows more signs of reaction than that containing Penn-Dixie cement, but the concrete containing Penn-Dixie does contain cracks in the upper section and gel throughout the core.

10 Incls
1. Detailed Petrography
2. - 4. Tables 1 - 3
5. Fig. 1
6. - 10. Photographs 1 - 5

Detailed Petrography

1. Test Procedure. The two 36-in. cores were inspected and the seven 4 3/4-in. cores were examined megascopically and logged for the presence of gel and cracks, and for general condition and appearance. Selected sections of each core were examined on the drilled surface using a stereoscopic microscope. Selected sections of each core were sawed transversely or longitudinally and examined under the stereoscopic microscope. A number of sections were broken and the broken surfaces examined using the stereoscopic microscope. Megascopic counts were made of coarse aggregate and gel on sawed surfaces. Sixty-eight particles of chert that were associated with gel were crushed and examined in immersion media to discover whether they were composed of quartz or chalcedony. The index of refraction of 40 particles of chalcedony was determined. Many samples of gel were examined in immersion media. Fourteen thin sections were prepared and examined, and several photographs made.

2. Concrete Cores. The cores examined included all those received except 6 sections tested for torsional frequency and subsequently shipped to the Portland Cement Association and 9 sections tested for compressive strength and static modulus of elasticity. All of the 4 3/4-in. core had been packed in damp cedar sawdust. At the time that the petrographic examination began, the sawdust and the concrete were dry. The wet sawdust had stained many of the core sections brown. Results of the examination of the cores are summarized in table 1. The composition of the aggregate, the condition of the concrete, the relation of gel and chert, and the types of exudates and deposits found in the concrete are discussed in paragraphs 3 through 6.

3. Composition of Aggregate.

a. Coarse Aggregate. Identification of the coarse aggregate particles intersected by sawed surfaces (table 2) indicates that about 40 per cent of the coarse aggregate is chert, and 60 per cent consists of other types of material, principally vein quartz, quartzite and sandstone, with very small quantities of granitic gneiss, and ochre or limonite. Sixty-eight chert particles which were associated with gel pockets were examined in immersion media; 48 of these particles (71 per cent of those examined) contained or consisted of chalcedony. The index of refraction of 40 of the chalcedony particles was determined (Fig. 1); the average of the 40 indices is 1.5365 with an observed range from 1.5240 to 1.5420. Since the chert particles were deliberately selected from those which had adjoining gel pockets, it is believed that the calculated percentage of chalcedonic chert, 71, is a maximum. If it is assumed that from 50 to 70 per cent of the chert is chalcedonic, the chalcedony content of the coarse aggregate is calculated as 19 to 30 per cent. Using the determined average value of the index of refraction of the chalcedony, and curves given by

Donnay,(1) the opal content of the chalcedony can be estimated as between 5.5 and 17.5 per cent, and the opal content of the total coarse aggregate as between 1 and 5 per cent.

The silica solubility and reduction in alkalinity was determined on a composite sample taken from particles determined as chalcedony. Duplicate tests gave $S_c = 477$ and $R_c = 46$, $S_c/R_c = 10.4$.

b. Fine Aggregate. The fine aggregate is natural sand composed principally of quartz, with some feldspar and a small amount of other minerals, and chert which is particularly conspicuous in larger sizes. According to information provided by the Mobile District, difficulty was encountered in producing sand with the required amount of fines. The amount of fine sand in the mortar was relatively small, and consequently there are larger areas of cement paste in the mortar than there usually are in the mortar of concrete made with sand graded in accordance with the current Guide Specifications.

4. Condition of the Cores. All of the drilled surfaces showed white films or mounds of gel, on areas of mortar and on chert coarse aggregate, but not on coarse aggregate of other types. Frequently the gel entirely inclosed sawdust particles; mounds of gel rose up as much as 1/8 in. above the core surface (Photograph 1a). These two facts make it plain that the deposition took place after the cores were drilled. All of the drilled surfaces cut air pockets and underside voids containing white or clear exudates with outer surfaces continuous with those of the adjacent paste and mortar, and accordingly older than the drilling of the core. A number of the drilled surfaces show cracks visible to the naked eye. A few show such cracks filled or partly filled with exudate. (Photograph 1b). The smoother surfaces produced by sawing with a diamond-edged blade reveal more cracks than appear on the outside of the core. When the concrete was broken open and the surfaces of the largest cracks examined, the surfaces were found to be covered with dessicated gel, which filled the crack and saturated the adjoining mortar. Some of the drilled surfaces intersect empty pebble sockets where poorly-bonded aggregate was lost during drilling. Some of the surfaces intersect coarse aggregate particles containing cracks visible to the naked eye. Most of the cracked particles are chert; a few are quartzite or sandstone. Cores from holes 3-1, 20-1, 60-1, 60-2, 60-3 appear fairly dense and free of large voids on the drilled and sawed surfaces. In those cores the concrete near the outside of the structure appears generally more dense than the concrete in the interior. The core from hole 60-4 is noticeably less dense than those mentioned above. Core from hole 5-1 is fairly dense to a depth of 7.5 ft, contains numerous large irregular voids from 7.5 to 13 ft, is conspicuously honeycombed around 13 to 14 ft (Photograph 2); from 14 to 39.6 ft it contains many spherical to irregular voids (Photograph 3). In all of the concrete, megascopic voids are more common

(1)Donnay, J. D. H., La birefringence de forme dans la calcedoine, Annales de la Societe Geologique de Belge, pp 289-302, 1936.

near coarse aggregate particles than away from them (Photographs 1b through 3). Many of the pebbles have relatively narrow but extensive underside voids.

5. Relation of Chert and Gel Pockets. The gel which grew on the cores after they were stored in sawdust was located on chert particles or on mortar, not on other varieties of coarse aggregate. The gel intersected by the sawed surfaces adjoined coarse aggregate of all the types present, or occupied cracks or voids in the mortar. Although particles of all the types of coarse aggregate could be found with adjacent gel, gel within coarse aggregate was confined almost entirely to chert pebbles. The exceptions were found in highly fractured quartzite pebbles. The association of gel with chert appeared to be more common than the association of gel with any of the other types of coarse aggregate (Photographs 4,5). To test this indication, megascopic counts were made of sawed surfaces to determine the number of chert particles with and without associated gel, the number of other coarse aggregate particles with and without associated gel, and the number of gel pockets or linings with no visible association with a coarse aggregate particle (Table 3). On 57 sawed surfaces counted, 57 per cent of the total coarse aggregate belonged to other varieties than chert, of that percentage 53 per cent did not have associated gel and 4 per cent had associated gel; 41 per cent was chert, of which 36 per cent did not have and 5 had associated gel. The total number of megascopic gel pockets was 580, of which 110 were apparently isolated, 281 adjoined chert and 189 adjoined other types of coarse aggregate. These figures leave out of account the possible connections in depth of the apparently isolated pockets. It is possible to calculate whether the observed distribution of gel pockets with respect to chalcedony and other types of aggregate is a random one, or whether the distribution indicates an association between gel and chalcedony greater than that likely to arise by chance.(1) Those calculations were made for the totals of all the surfaces counted. The probability that the association found would occur by chance is less than 1 in 1000.

Table 3 also indicates that the percentage of gel pockets in core 3-1, containing Penn-Dixie cement, is lower than it is in any of the cores containing Alpha cement.

6. Exudates and Deposits in the Concrete. Examination of drilled, sawed, and broken surfaces of the concrete shows that several types of deposits are present in voids and cracks and on aggregate particles.

a. Gel. The exudates on the core surfaces which are later than the drilling of the cores are white to translucent bluish, laminated, often show shrinkage cracks, and are brittle when they are dry. The samples examined in immersion media were isotropic gels of lower indices and with fewer

(1) R. A. Fisher, Statistical Methods for Research Workers, G. E. Stechert, 1946, pp 85-99.

crystalline inclusionsthan the gels which grew in the structure. Two varieties of gel differing in appearance are found in pockets and lining cracks. One is clear to translucent, rubbery to brittle, and usually forms the outer shell of the lining if two varieties are present in one void. The other is white, opaque dull, rubbery to powdery to brittle, and usually forms the inner core if both varieties are present in one void. Two examples were found where the translucent gel occupied the interior and the opaque gel the periphery of the filling. Under the petrographic microscope, three structural varieties were found. One variety was anisotropic, with aggregate polarization, low birefringence, and wavy extinction of the type developed in strained glass. The second variety under crossed nichols had a very fine-grained "pepper-and-salt" appearance resembling chert or fine-grained calcium hydroxide; it was interpreted as inclusions of minute crystals of calcium hydroxide in gel. Some of the gel with the pepper-and-salt inclusions also contains irregular or rhombic inclusions of calcium carbonate. Calcium carbonate is the only inclusion in some of the gel samples. The third variety was clear, isotropic, and usually contained fewer inclusions than the other types. Examples of all three types were found in the same pocket in some cases. The indices of refraction of all three varieties ranged; the range observed in the salt-and-pepper type was 1.478 to 1.511; in the type shewing aggregate polarization from 1.480 to 1.502. Isotropic gel later than the drilling of the cores had indices from 1.465 to 1.487. All of the variations found suggest that gels of differing composition may form within a relatively small volume of concrete with changing conditions and at different times. There are many voids which were entirely filled with gel, or which evidently were once entirely filled but the filling shrank. However, most of the void space in the concrete is still empty, even in areas close to gel-filled cracks. Many voids contain no gel, or a thin partial lining of clear brittle gel. Gel was found in all sections of all the cores, regardless of type of cement or distance from the outer surface of the structure. The most abundant gel, and the greatest amount of gel-permeated and gel-whitened paste is found near cracks in the upper or outer sections. In the longest core, 5-1, gel and cracking decrease from top to bottom in the core, but gel is present in the bottom section.

b. Calcium Carbonate. Carbonation of the cement paste adjoining cracks was found in thin sections from depths up to 20 feet from the exterior. A small amount of carbonation was found in one thin section from core 5-1, depth about 39 feet. It is believed that this carbonation took place after the core was drilled. The amount and extent of carbonation did not appear to be unusual, and is not regarded as significant except that it indicates that concrete as deep as 20 feet was accessible to air.

c. Calcium Sulfoaluminate. Calcium sulfoaluminate was found in voids in every section of every core, regardless of distance from the surface of the structure. It was most abundant in each core near the outer or upper surface of the structure. In the top section of 3-1, which was represented by a series of chunks and fragments, sulfoaluminate was very abundant in voids and in pebble sockets. In the cracked outer section of 60-3, about 8 in. from the surfaces, mats, rosettes, and spherulites of sulfoaluminate filled voids, lined pebble sockets, and coated sand grains. With increasing depths, the sulfoaluminate was found in tufts, rosettes, and linings in the voids, but not as linings of pebble sockets.

d. Calcium Hydroxide. Core 5-2 from depths 38.4 to 40.2 ft contained well-developed tufts and rosettes of sulfoaluminate in voids. In some of the voids, the sulfoaluminate was associated with clear colorless plates of calcium hydroxide. The calcium hydroxide is less abundant than the sulfoaluminate, but is not uncommon. This is the first example of calcium hydroxide in crystals in voids found in field concrete examined by this office. Since accessible calcium hydroxide is fairly easily converted to calcium carbonate by moderately dry air containing carbon dioxide(1) the significance of the calcium hydroxide crystals in voids may be interpreted as follows: At some period in its history, the concrete contained enough circulating solutions to leach some of hydroxide from the paste and redeposit it as crystals in voids. The crystals persisted because the concrete never dried out enough to permit the crystals to be exposed to relatively dry air containing CO_2.

(1) F. M. Lea and C. H. Desch, The Chemistry of Cement and Concrete, London, 1935, p. 328.

Table 1

Condition of 4 3/4-in. cores from Tuscaloosa Lock (MOB-4 CON-1 through CON-7)

Monolith and Hole Number	Cement	Visible Cracks	Gel		Sulfoaluminate
			Exuded on Drilled Surfaces	In Pockets and Cracks	
3-1	Penn-Dixie Alpha	Yes No	Yes "	Yes "	Yes "
5-1	Alpha	(a)	Yes	Yes	Yes
20-1	Alpha	Yes	Yes	Yes	Yes
60-1, 2,3,4,	Alpha	Yes	Yes	Yes	Yes

(a) Cracks visible in sections from 0.0 to 7.5 ft in depth only.

Table 2

Composition of Coarse Aggregate in Tuscaloosa Lock, as Determined by Counts of Particles Intersected on Sawed Surfaces

Monolith and Hole No.	Sawed Surfaces Counted	Number of Coarse Aggregate Particles, %[a]	
		Chert[b]	Other[d]
3-1	4	41	59
5-1	33	42	58
20-1	9	42	58
60-1	3	37	63
60-2	3	38	62
60-3	2	32	68
60-4	3	42	58
Average of all cores[d]	-	42	58

(a) Calculated as a percentage of the total number of coarse aggregate particles intersected on the sawed surface.
(b) Chert determined by megascopic examination, without any distinction made between chert consisting of quartz and chert consisting of chalcedony.
(c) The coarse aggregate constituents other than chert are quartz, quartzite, sandstone, with a few particles of granitic gneiss.
(d) Arithmetic averages, based on a total of 5518 coarse aggregate particles.

Table 3

Distribution of Gel Pockets on Sawed Surfaces of
Concrete from Tuscaloosa Lock

Monolith and Hole Number	Numbers Intersected on Sawed Surfaces, as Per Cent: Coarse Aggregate Not Associated with Gel		Coarse Aggregate Associated with Gel		Isolated Gel Pockets
	Chert	Other	Chert	Other	
3-1	35	56	5	2	1
5-1	36	56	6	2	1
20-1	38	46	5	5	3
60-1	31	57	4	3	2
60-2	32	55	6	5	3
60-3	13	46	9	13	13
60-4	34	48	6	7	4
Average, all 7 cores	36	53	5	4	2
Numerical totals	2032	3016	281	189	110

D28

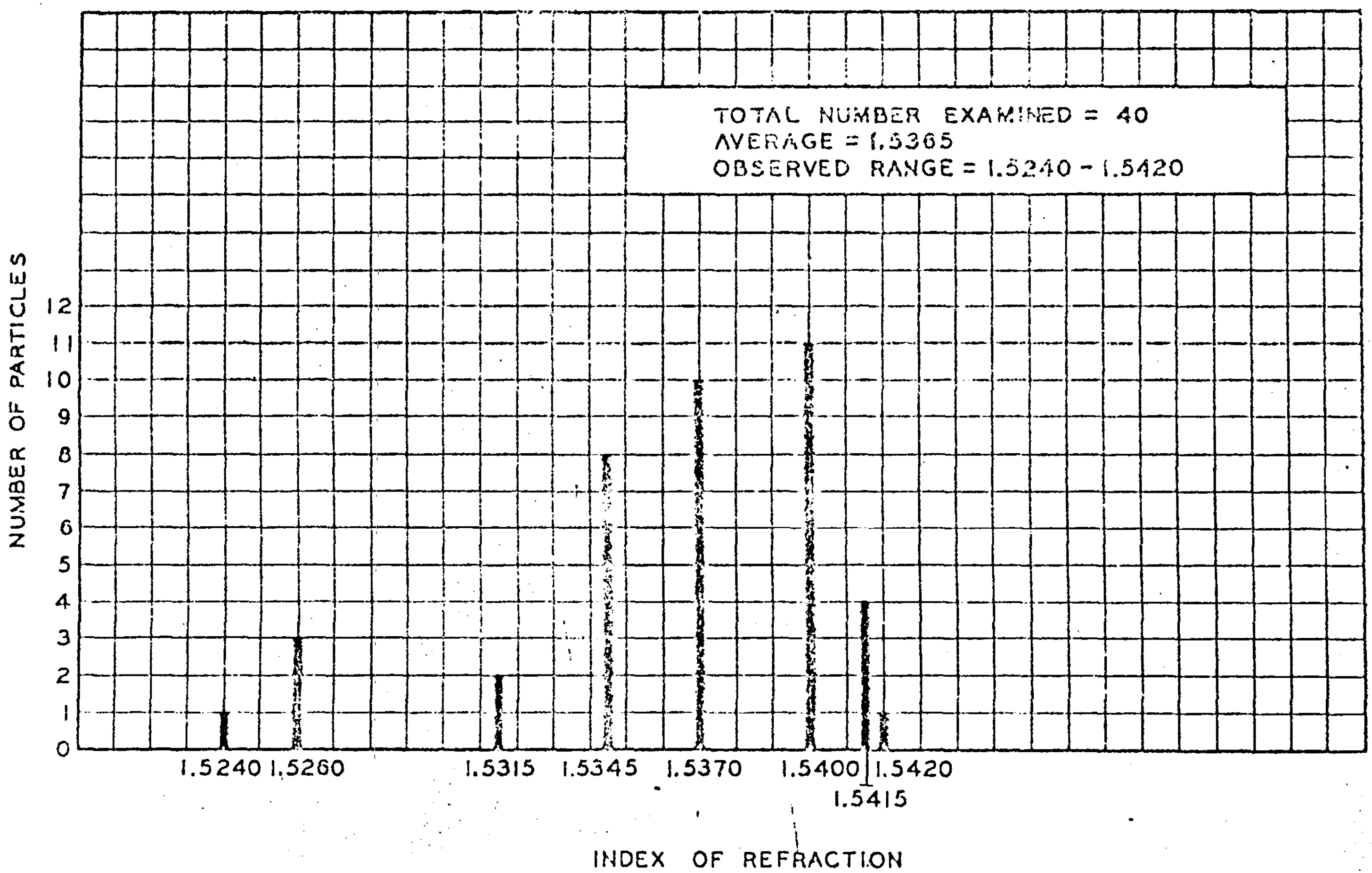
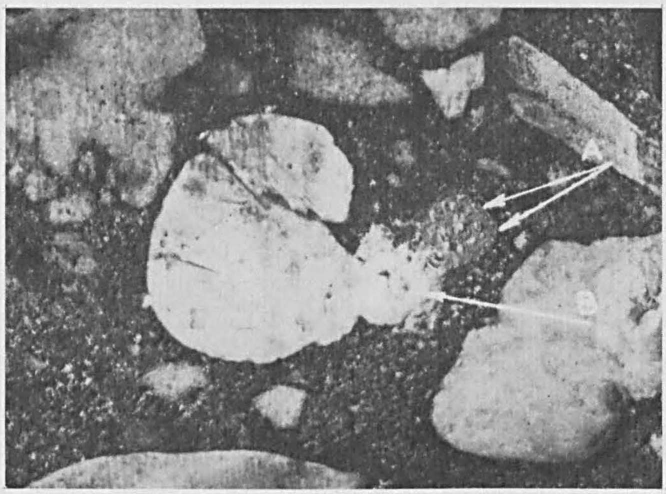


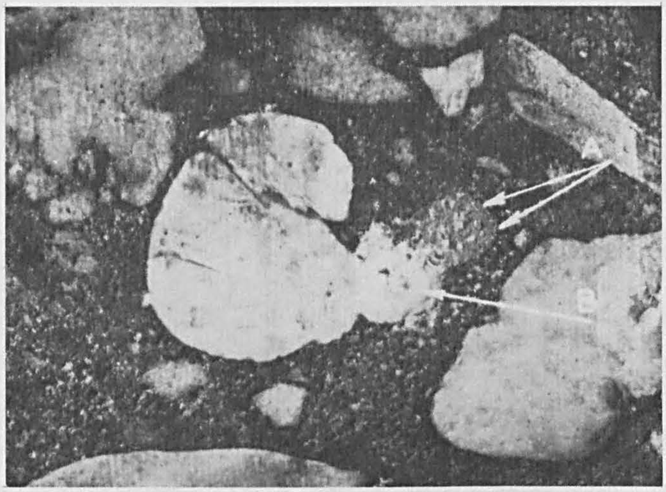
FIG. 1 - DISTRIBUTION OF CHALCEDONIC CHERT COARSE AGGREGATE PARTICLES SELECTED FROM CONCRETE FROM TUSCALOOSA LOCK, WITH RESPECT TO INDEX OF REFRACTION



A. FLECKS OF GEL WITH INCLUDED SAWDUST (A) AND MOUND OF GEL (B) WHICH GREW ON CORE SURFACE AFTER DRILLING. ABOUT NATURAL SIZE. CORE 5-1, DEPTH 7.5-9.0

B. OPEN CRACK. (A-A). PARTIALLY FILLED WITH GEL (ARROWS). THE CRACK REACHES ITS GREATEST WIDTH IN THE CHERT PEBBLE NEAR THE CENTER OF THE PHOTOGRAPH. ABOUT NATURAL SIZE. CORE 60-1, 7 IN. FROM FORMED SURFACE.

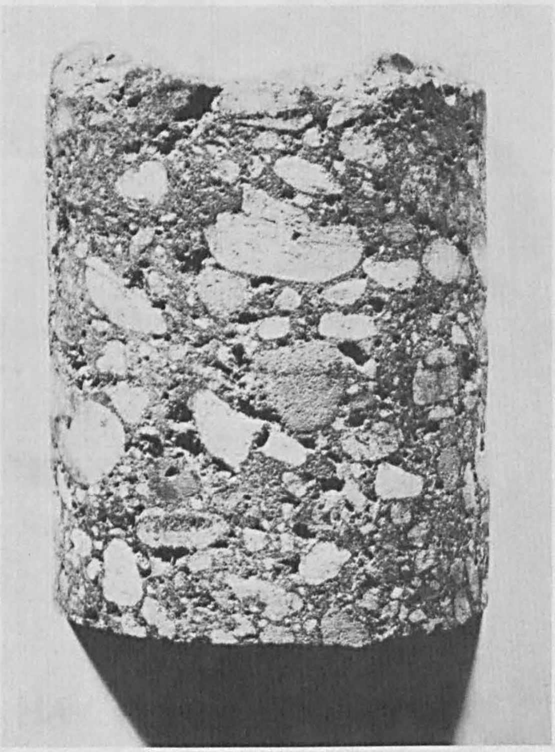
DRILLED SURFACES OF CORES FROM TUSCALOOSA LOCK



A. FLECKS OF GEL WITH INCLUDED SAWDUST (A) AND MOUND OF GEL (B) WHICH GREW ON CORE SURFACE AFTER DRILLING. ABOUT NATURAL SIZE. CORE 5-1, DEPTH 7.5-9.0

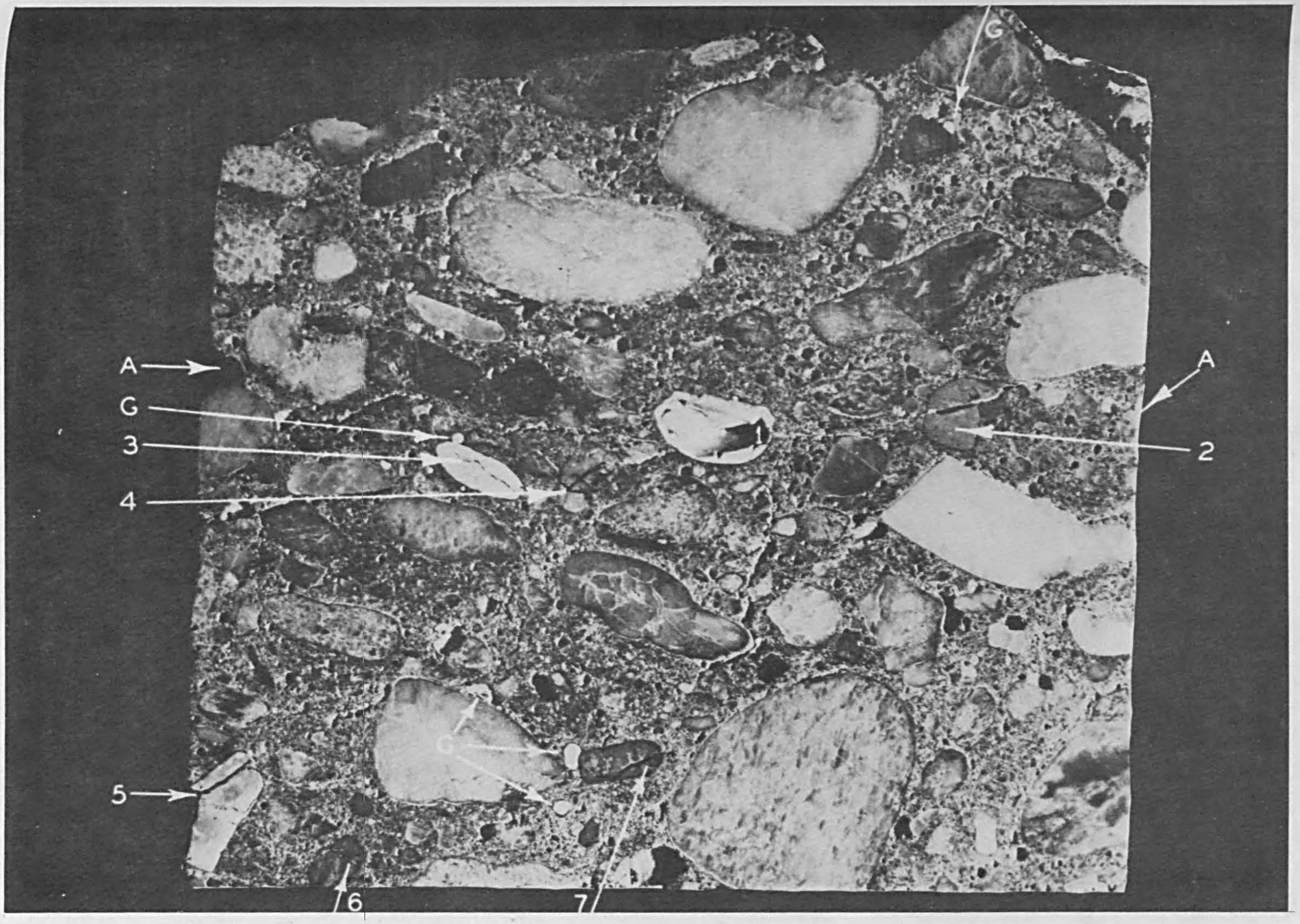
B. OPEN CRACK. (A-A). PARTIALLY FILLED WITH GEL (ARROWS). THE CRACK REACHES ITS GREATEST WIDTH IN THE CHERT PEBBLE NEAR THE CENTER OF THE PHOTOGRAPH. ABOUT NATURAL SIZE. CORE 60-1, 7 IN. FROM FORMED SURFACE.

DRILLED SURFACES OF CORES FROM TUSCALOOSA LOCK



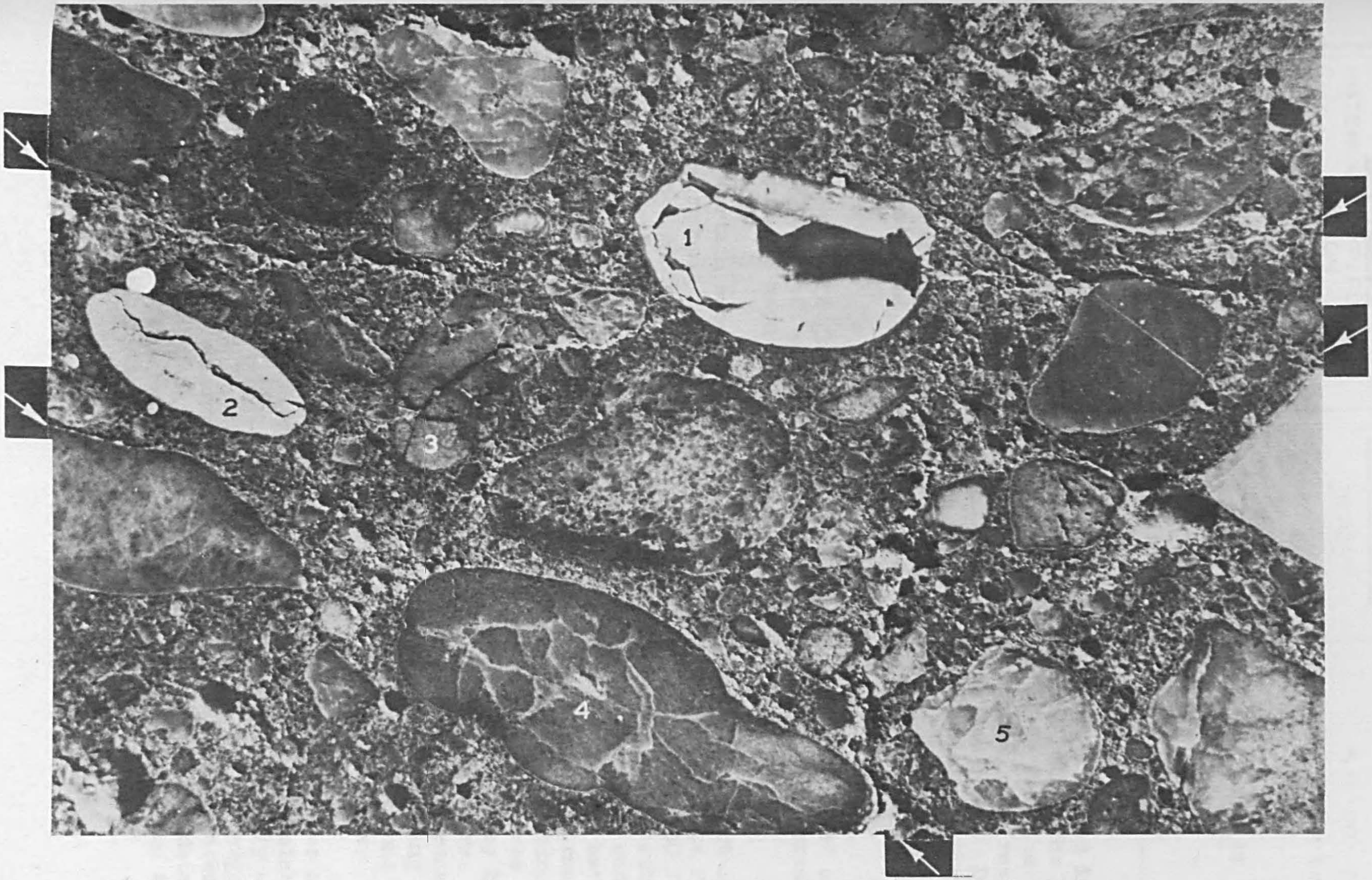
CONCRETE WITH NUMEROUS LARGE VOIDS, DEPTH ABOUT 33 FT., CORE 5-1. THE SURFACE IS TYPICAL OF THE CORE FROM 14.5 FT. TO 39.6 FT. DIRECTION OF PLACEMENT TOWARDS BOTTOM OF PHOTOGRAPH. THE VOIDS ARE MORE FREQUENT ADJACENT TO COARSE AGGREGATE PARTICLES.

DRILLED SURFACE OF CORE 5-1 FROM TUSCALOOSA LOCK



SAWED SURFACE OF CORE CUT LONGITUDINALLY. THE DIRECTION OF PLACEMENT IS TOWARD THE TOP OF THE PHOTOGRAPH. A-A INDICATES THE ENDS OF A CRACK SYSTEM WHICH CAN BE TRACED ACROSS THE CORE, PASSING THROUGH TWO CHERT PEBBLES (I AND 2). THE WHITE CHERT PEBBLE, I, HAS AN OUTER ZONE VERY FIRMLY BONDED TO THE MATRIX AND THE CENTER OF THE PEBBLE SEPARATED BY CRACKS FROM THE OUTER ZONE. THE CHERT PEBBLES INDICATED BY ARROWS (2 THROUGH 7) SHOW THE WIDE CRACKS COMMON IN CHERT PARTICLES IN THIS CONCRETE. IN PEBBLE 2 THE GEL FILLING OF PARTS OF THE CRACK SYSTEM CAN BE SEEN AT THE LEFT AND RIGHT. G INDICATES GEL POCKETS.

CORE FROM TUSCALOOSA LOCK, HOLE 5-I, DEPTHS I.5 - 2.0 FT.



PART OF AREA SHOWN IN PHOTOGRAPH 1, X 3.4. ARROWS AT MARGIN MARK ENDS OF CRACK. 1,2,3 ARE CHERT PEBBLES WITH WELL-DEVELOPED WIDE CRACKS; THOSE IN 3 ARE LOCALLY GEL-FILLED. 4,5, ARE CRACKED PEBBLES; THE CRACKS ARE NARROWER THAN THOSE IN 1,2,3.

CORE FROM TUSCALOOSA LOCK, HOLE 5-1, DEPTHS 1.5 - 2.0 FT.

CORPS OF ENGINEERS, U. S. ARMY XXXXXXXXXXXXXXXXXX XXXXXXXXXXX WATERWAYS EXPERIMENT STATION	PETROGRAPHIC REPORT	CONCRETE RESEARCH DIVISION P. O. BOX 217 CLINTON, MISSISSIPPI

6078

SYMBOL: MOB-4 6078	PROJECT: Tuscaloosa	DATE REPORT SUBMITTED: 19 Oct 49	INITIALS: KM
SERIAL NO: MOB-4 CON-2	SOURCE: Tuscaloosa Lock and Dam		

/de

1. Reference is made to the petrographic report included in "Disintegration of Concrete from Tuscaloosa Lock and Dam". The last paragraph of the summary mentions a thin section blank from the 7.5 - 9.0 ft section of core 5-1. This blank was impregnated with resin, ground smooth, and exposed to warm humid air for about 36 hr. During this exposure, three gel pockets and one chalcedony particle on the ground surface took up water from the air and swelled. Photograph 1 is a photograph of the specimen at a magnification of 4x, taken about 23 August 1949.

2. About 24 August 1949 the blank was stored in a closed container over water, and left until 17 October 1949. It was then re-examined and photograph 2 was made at the same magnification.

3. Certain comments on the rapid and obvious development of the reaction-products in this specimen suggest themselves. In the first place, the specimen has been heated well above 100 F (probably to 200 F at least) in the process of impregnating it with resin. During that heating, a more abrupt thermal change took place in it than it had ever undergone before. The cement paste lost some water which it would not have lost in laboratory storage at ambient temperatures. Probably the thermal shock and partial dehydration suddenly opened up the general structure and particularly the mortar-coarse aggregate boundaries to a degree unusual in concrete undergoing mild weathering. Thus the specimen may have been rendered unusually susceptible to a later addition of moisture. In the second place, while the resin apparently did not thoroughly penetrate the interior, it formed a coating around all the surfaces including that from which it was later removed by grinding. All the manifestations of activity are probably therefore concentrated on the one ground surface.

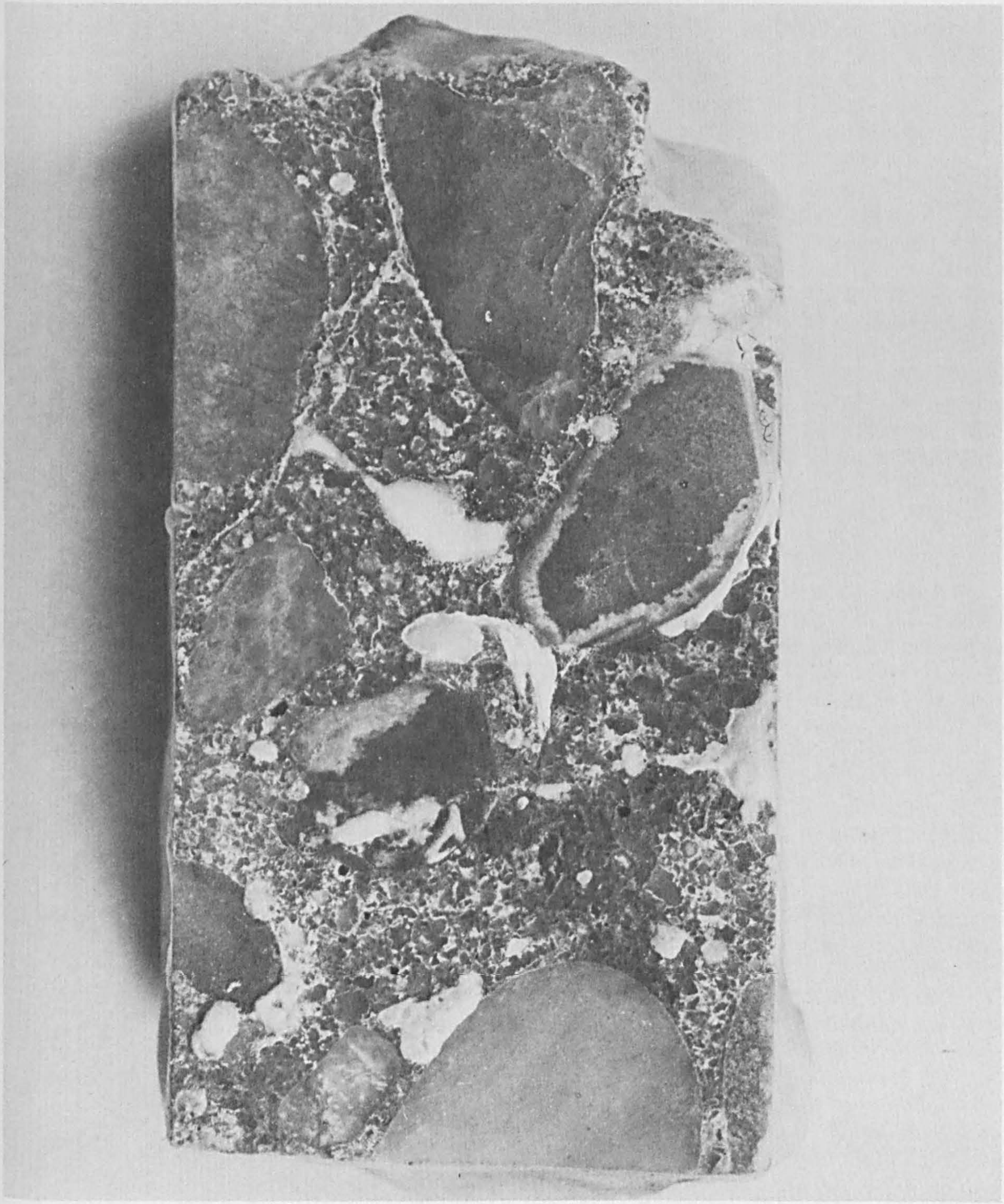
4. The comments in 3 above are made because other pieces of Tuscaloosa concrete have been stored in the same closed container since July 1948 without developing any visible changes except fairly minor increases in gel deposits. Therefore it is not believed or suggested that deterioration of the concrete in Tuscaloosa lock may be expected to proceed at the rapid rate suggested by the two photographs and dates. It is believed that the photographs do tend to confirm the suspicion that the reactive potential in the concrete has not been exhausted.

2 Incls
1, 2. Photographs 1, 2

LMW FORM NO. 557

THIN SECTION BLANK IMPREGNATED WITH RESIN, GROUND, EXPOSED TO WARM HUMID AIR FOR ABOUT 36 HRS. MAGNIFICATION 4X, PHOTOGRAPHED ABOUT 23 AUGUST 1949. TWO LARGE GEL POCKETS (UPPER RIGHT AND UPPER CENTER) HAD TAKEN UP MOISTURE AND SWELLED. THE CHALCEDONIC PEBBLE (LOWER LEFT CENTER) HAD TAKEN UP MOISTURE, AND THERE WAS A WET SPOT ON A SAND GRAIN AT THE LOWER LEFT.

THIN SECTION BLANK, CORE 5-1, 7.5-9.0 FT SECTION, TUSCALOOSA LOCK.



SAME SPECIMEN AFTER STORAGE OVER WATER AT ROOM TEMPERATURE BETWEEN 24 AUGUST 1949 AND 18 OCTOBER 1949. THE GEL POCKETS SHOWN IN THE PREVIOUS PHOTOGRAPH HAVE ENLARGED; MANY MORE HAVE APPEARED; CRACKS WHICH WERE NOT OBVIOUS BEFORE ARE NOW TRACED IN REACTION PRODUCT.

THIN SECTION BLANK, CORE 5-1, 7.5-9.0 FT SECTION, TUSCALOOSA LOCK.

APPENDIX E

BOARD OF CONSULTANTS REPORT
OCTOBER 1949

SUBJECT: Investigation of Disintegration of the Concrete in Tuscaloosa Lock and Dam

TO: The District Engineer
Mobile District
Corps of Engineers
MOBILE, ALABAMA

FROM: The Board of Consultants

The undersigned convened on the morning of October 24th in accordance with previous arrangements and reviewed the field data assembled by the Mobile District since the last Board Meeting, inspected the condition of cracking in the lock walls, and discussed the alkali-aggregate problem involved in this and similar structures.

FIELD DATA SECURED SINCE THE LAST MEETING OF THE BOARD

There is attached hereto a typed tabulation of elevations of selected points on the lock walls taken at different dates between January 1948 and September 1949, a drawing showing an outline of the upper end of the lock walls in plan with a tabulation showing horizontal distances measured between marked points on the tops of various monoliths taken between January 1948 and September 1949. Comparison of maximum, minimum, and average differences in measurements, both between the Spring of 1948 and the Spring of 1949 and the Fall of 1948 and the Fall of 1949, and the overall differences between January 1948 and September 1949 indicates that growth has continued at varying rates with the maximum horizontal distance between certain points of .028 feet. Many points show only a difference of 0.001 ft. It is therefore concluded that internal expansion and external cracking are continuing throughout the various monoliths of the lock walls, although at a decreasing rate as compared to the Calander Year 1947 and probably 1946.

Careful examination of Monolith No. 5, including an inspection of the 36" hole adjacent to the pintle bearing, did not indicate any great change in the condition of this monolith since the last meeting of the Board, although it is stated that the pintle has moved a distance of .015 feet. There is some evi- of displacement in connection with the operating mechanism for the gate in Monolith No. 5, but at this time the displacement is not sufficient to cause any concern other than to emphasize the desirability of keeping track of such indications for possible future correction if the displacement should continue to increase. The cracking in Monolith No. 51 has increased appreciably since the last meeting and it has been necessary to discontinue use of the mooring bit located at the upstream end of this monolith. Cracking in the top of Monolith No. 20 has apparently increased somewhat since the last inspection. Deflection toward the lock of the top of Monolith No. 74, and also in a downstre direction from Monolith No. 73, appeared to have increased since the last inspection but a rough check of previous data indicated that the increase, if any, was of minor amount and probably in line with increased cracking in general only.

An interesting observation was made on Monoliths Nos. 70, 71, 72, 73 and 74. Monoliths 71 and 73 above the water surface are made of Penn-Dixie cement and cracking in these monoliths is relatively negligible, especially on the 45° bevel on the river side of the lock wall. In contrast to this apparent lack of deflection, pattern cracking on the beveled portion of the river side of the lock wall in Monoliths 70, 72 and 74 is very noticeable and the top of these monoliths have deflected toward the lock an appreciable amount, varying from the order of 1/2" to the order of 1½" in Monolith 74. It seems evident that the internal growth---apparently caused by the higher alkali content---of the Alpha cement in the beveled portion of Monoliths 70, 72 and 74 has been responsible for this tipping action.

LABORATORY DATA PREPARED BY W.E.S.

There is attached a copy of the report entitled "Disintegration of Concrete from Tuscaloosa Lock and Dam" dated 25 August 1949. The purpose of the studies and tests conducted by the Concrete Research Division was to definitely determine, if possible, the cause of the extensive cracking in Tuscaloosa Lock. The principal conclusion of the Concrete Research Division is that "Chalcedonic chert is the only constituent found in the coarse or fine aggregate which is known to be capable of deleterious reaction with the minor alkalis of Portland cement. Such chert is a more important constituent of the coarse aggregate than of the fine aggregate." In the last paragraph of the summary it is noted that "While reaction in the structure had only gone far enough to crack the concrete near the surface, all the necessary ingredients for reaction are present in the deeper concrete and that the production of gel in this concrete is accelerated when a source of moisture is provided." In the closing sentence in this summary, the following is noted: "The concrete containing Alpha cement shows more signs of reaction than that contained by Penn-Dixie cement, but the concrete containing Penn-Dixie does contain cracks in the upper section and gel throughout the core." It is believed that the Clinton laboratory report is sufficiently exhaustive to serve the purpose of this investigation and that any further investigational work in connection with the cores available should be done as a part of the general program of investigations of alkali-aggregate reaction being conducted by the Concrete Research Division under CWI Test No. 603.

FUTURE PROGRAM OF INVESTIGATIONS

It was agreed by all members of the Board that any further investigation of alkali-aggregate reaction in connection with Tuscaloosa Lock was not necessary insofar as the maintenance and operation of the lock for future use is concerned, but all concurred in the desirability of semi-annual measurements to be taken in September and April of each year for the purpose of checking the location of all four pintle bearings as well as other prominent points on top of the lock wall that are being watched. It was also concluded that it would be desirable to check the horizontal distance across the lock between pintle bearings as a continuing check on any possible movement of the pintles in a lock-ward direction. It would appear that any movement that would be detrimental to the maintenance and operation of the lock in the future would have to be toward the lock. It is also suggested that a close check on the operating mechanism in Monolith No. 5 be maintained. From observation during the inspection on the 24th it would appear that indications of the stem guide relative to deflection, either in a horizontal or a vertical direction, and the distance between the guide and the roller, as measured by feeler gauge at definite points, might be as satisfactory an indication of growth and movement as any place such measurements could be made on the operating machinery.

Mr. Tyler suggested that they would be interested in making wave velocity measurements at the points previously measured in June 1948 whenever they were in this territory for the purpose of making wave velocity measurements on other structures. It was the unanimous opinion of the board that P.C.A. should be encouraged to continue wave velocity measurements previously made at least until such time that the Clinton laboratory has available wave velocity apparatus of its own for keeping track of growth and movement in certain structures such as Tuscaloosa Lock.

MAINTENANCE AND REPAIRS

It was the unanimous opinion of all present at this meeting that due to continued internal growth and external shrinkage, even though at a slower rate, it is highly impracticable to make any extensive repairs at this time, or probably at any time in the course of the next year or two. Unless Tuscaloosa Lock concrete performs differently than other structures similarly affected, growth will continue for an undetermined period and until it ceases it would not appear desirable to make extensive repairs except to specific points that require some maintenance, as in the case of Monolith No. 51 where the concrete around the mooring bit has already deteriorated to such an extent as to make it dangerous to use. It therefore appears that something must be done to recondition the upper part of Monolith No. 51 and put the mooring bit back into useable condition. For this purpose and for the purpose of obtaining some data on possible future repairs for the top of the lock wall monoliths that are extensively cracked, the Board recommends that the concrete in the top of Monolith No. 51 be removed and a reinforced concrete cap block cast on top of this monolith. The extent of the concrete to be removed can be determined only after its excavation is started. It is believed that some reinforcing will be necessary to prevent immediate cracking due to growth of the concrete under the cap and it is suggested that 1-inch or 1-1/8-inch square bars at approximately 15-inch centers both ways be imbedded about 3 inches below the surface of the new concrete cap block. The small cost of the reinforcing should provide an excellent illustration of what can be accomplished in this respect and whether it will be desirable in future repairs to use reinforcing steel.

No further meetings of the Board are under consideration at this time. It is assumed that when conditions develop that would necessitate further consideration by the Board, the Mobile District will call such a meeting.

An invitation was extended to the Portland Cement Association to be present at this meeting and Mr. Ivan L. Tyler, Manager of Field Research, was present throughout the inspection and later discussions.

H. K. COOK

C. B. WESTON

J. C. SPRAGUE

E. W. STEELE

Attached:
1. Tabulation of elevations of selected points on lock walls.
2. Drawing showing outline of upper end of lock walls in plan with a tabulation showing horizontal distances between marked points on tops of various monoliths.
3. Report "Disintegration of Concrete from Tuscaloosa Lock and Dam" dtd 25 August 1949.

APPENDIX F

PETROGRAPHIC REPORT
DECEMBER 1976

Corps of Engineers, USAE Waterways Experiment Station	Petrographic Report	Concrete Laboratory P. O. Box 631 Vicksburg, Mississippi
Project Condition Survey and Stress Analysis, Oliver Lock and Dam		Date 20 December 1976 ADB

Background

1. The structure was built between 1937 and 1939 as Tuscaloosa Lock and Dam. It was later named Oliver Lock and Dam. A report by the U. S. Army Engineer Waterways Experiment Station (WES)[1] dated August 1949 identified the deleterious chemical reaction that had occurred in about 10 years as the alkali-silica reaction.

2. Approximately 25 years later two more cores have been examined. The questions that were to be answered were:

a. Does the entire length of each concrete core show evidence of alkali-silica reaction?

b. If present, has the reaction exhausted its expansive potential?

Samples

3. Two cores of nominal 4-in. diameter were received in November 1976 from the U. S. Army Engineer District, Mobile, for examination and testing. They are identified below:

Concrete Laboratory Serial No.	Field Data
MOB-4 CON-15 (concrete) and MOB-4 DC-1 (rock)	One core consisting of approximately 63 ft of concrete and 28 ft of foundation rock from monolith 5 (land wall). It was located at Sta 0+01.5B and 27.5 ft from the chamber wall.
MOB-4 CON-16	One concrete core approximately 25 ft long from monolith 16 (land wall). It was located at Sta 4+31B and 27.5 ft from the chamber wall.

Test procedure

4. Each core was logged. Some of the pieces of concrete in each core and all of the pieces of foundation rock from monolith 5 had been sealed in a wrapping of cheesecloth and wax to preserve them at field moisture conditions. Since the concrete appeared to be generally uniform in appearance, these pieces were not unwrapped during the logging. The sealed pieces of foundation rock were opened, inspected quickly, and immediately resealed.

WES FORM No. 1115
Rev Feb 1970

5. Petrographic samples were taken from the following portions of the cores.

Sample	Monolith	Depth Interval, ft
Concrete	5	3.5-4.2
Alkali-silica gel		20.7
Concrete		30.9-32.7
Concrete		53.3-54.3
Concrete		59.9-60.6
Foundation rock (shale, sandstone)		70.3-71.0
Foundation rock (shale)		85.5-85.9
Concrete	16	0.0-0.9
Concrete		1.2-1.4
Concrete		3.3-4.0
Concrete		5.6-6.9
Concrete		21.5-21.9
Concrete		24.5-25.3

Drilling was discontinued in monolith 16 when the hole was 25.3 ft deep. The color of the rock samples was determined.[2]

6. Samples of concrete from the above pieces were selected for length-change measurements from the top, middle, and bottom concrete portions of the monolith 5 core and from the top and bottom portions of the monolith 16 core. Each piece was sawed to a length ranging from about 6 to 11 in. and fitted with metal inserts. The five pieces of core were measured, stored in water overnight, and remeasured; this latter value was taken as the reference length. The specimens were then stored over water at 100 percent relative humidity (RH) and 100°F in general accordance with CRD-C 123-72[3] and measured weekly. The intent of storage at the high moisture and temperature conditions is to determine if any expansive potential remains in the concrete. It is not intended to simulate field conditions.

7. The remainder of the petrographic samples were examined with a stereomicroscope. This included examination of broken surfaces and of some surfaces that had been sawed and then ground to enhance detail.

8. Thin sections of the foundation rock from the 70.3 to 71.0 ft interval were prepared and examined with a polarizing microscope.

9. Selected portions of the foundation rock were examined by X-ray diffraction to determine their mineralogical composition. Saturation with glycerol and heat treatment were used along with X-ray diffraction to assist in characterization of the 14Å clay mineral in the shale.

10. The sample of alkali-silica gel from the 20.7-ft depth of the monolith 5 core was ground and X-rayed as a tightly-packed powder; powder immersion mounts of it were examined with a polarizing microscope.

11. All of the X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

Results

12. The logs of the two cores are attached (Figures 1, 2). Inspection of the core during the logging and of other data showed:

a. There is evidence of alkali-silica reaction from top to bottom of the concrete in each core. This is seen as white alkali-silica gel in voids and on broken surfaces, as rimmed chert particles, and as cracks that traverse mortar or aggregates or both.

b. The evidence of alkali-silica reaction decreases with depth. The major effects of the reaction appear to be concentrated in the upper few feet of each core. This observation is not unusual and probably is connected with the increase in restraint with depth.

c. Breaks in the concrete that appeared to have been present before the cores were drilled are listed below:

Monolith 5 - 2.2, 2.8, 3.3, 3.5, 4.5, 9.3, 9.7, 10.4, 15.4, 19.3, 39.7(?), and a vertical crack between 53.3 and 54.3 ft.

Monolith 16 - 3.0, an almost vertical crack between 3.0 and 3.6, fragmented between 3.6 and 4.0, and 5.4 ft.

d. Aside from the difference in amount of alkali-silica reaction with depth the concrete appeared homogeneous. The coarse aggregate was chert, quartz, and quartzite of about 1-1/2-in. maximum size; the fine aggregate was natural sand. There were scattered small areas of honeycomb due to poor consolidation. According to information on the Black Warrior, Warrior, and Tombigbee Rivers Concrete Progress Chart,[1] the cement in all of both cores should be the higher alkali Alpha brand.[1]

e. The foundation rock in the lower 28 ft of the core from monolith 5 is dark gray (N3)[2] shale and light gray (N7)[2] fine-grained sandstone. Some areas of the core are all shale, others are all sandstone, and some are alternating thin layers of the two. The sandstone is identified as siltstone on Figure 1, but it is more properly called sandstone. Scattered small patches and layers of tan clay (dark yellowish brown, 10YR 4/2)[2] are indicated on Figure 1. This material turned out to be clayey concentrations of siderite ($FeCO_3$) when examined by X-ray diffraction.

f. The foundation material also included 0.4 ft of coal between 79.9 and 80.3 ft. There were traces of coal at scattered intervals below this area.

g. All of the breaks in the foundation rock appeared to be fresh breaks that were associated with the drilling process.

h. Air drying of the shale sample from the 85 ft depth did not produce appreciable cracking.

13. Length-change data for five samples of concrete are shown in Table 1. The values for the three pieces from monolith 5 show an increase with time and with depth. The values for the two pieces from monolith 16 show these trends to a minor degree. However, all of the data indicate enough expansion to show that the potential for expansion due to the alkali-silica reaction is still present in the concrete represented under these conditions of high moisture and temperature. Similar data for cores stored at high moisture conditions and temperatures around 70°F would provide an interesting contrast since they would more nearly simulate possible field conditions.

14. Examination of the petrographic samples of concrete verified the preliminary core inspection and agreed with previous data[1] about the presence of alkali-silica reaction. The sample of alkali-silica gel showed varieties similar to those described before.[1] The refractive index of the anisotropic types was above 1.486 while that for the isotropic type was below this value. All varieties had refractive indices below 1.544. The X-ray pattern of this gel showed spacings similar to those listed for other gel[4] but no specific identification was made. The spacings are shown below:

Spacing, Å	Relative Intensity
10.5	Medium
8.8	Weak
6.6	Very weak
~~5.6~~	~~Very weak~~
3.59	Weak
3.07	Strong
3.03	Strong (probably calcite)
2.81	Weak
2.14	Weak
1.98	Weak
1.84	Weak
1.67	Weak
1.64	Very weak
1.54	Very weak

The X-ray pattern indicated that the gel was a mixture of crystalline phase(s) and amorphous material.

15. Petrographic examination of the samples of foundation rock showed that the shale and sandstone were composed of micaceous minerals (chlorite, muscovite, biotite), clays (kaolinite, clay-mica), and nonclays (quartz, feldspars). The shale contained detectable siderite and the sandstone showed detectable calcite. The shale contained more micaceous and clayey material while the sandstone contained more quartz and feldspar. The siderite mentioned earlier also contained small amounts of the same constituents as the shale and the sandstone.

16. Examination of the thin sections showed that the grain size of the rock was usually about 120 by 120 μm with some particles up to 250 by 250 μm. These sizes meant that the rock should be classified as a fine-grained sandstone rather than siltstone. The grain size was fairly uniform. Most of the quartz grains were anhedral in shape. There were both quartz grain to grain contacts and some instances of mica or clay between the sand grains. Therefore, the rock is fine-grained, micaceous sandstone cemented mainly by silica.

Conclusions

17. The full lengths of both concrete cores show evidence of alkali-silica reaction. The reaction is more pronounced in the upper few feet of the cores.

18. Length-changes of concrete specimens from both cores stored at 100 percent RH and 100°F show that the concrete still has expansive potential.

19. The foundation rock is shale, fine-grained sandstone, and closely spaced alternating layers of these materials.

20. There is a thin layer (0.4 ft) of coal at a depth of about 80 ft in the core from monolith 5.

Recommendations

21. It is recommended that the present length-change measurements be continued to determine how much expansion will occur under these conditions. It is further suggested that companion specimens be prepared for storage at 100 percent RH and about 75°F with periodic measurement to determine how much of the expansion is due to elevated temperature.

REFERENCES

1. "Disintegration of Concrete from Tuscaloosa Lock and Dam," U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss., Aug 1949.

2. The Rock Color Chart Committee, National Research Council, Rock-Color Chart, Washington, D. C., 1963.

3. U. S. Army Engineer Waterways Experiment Station, CE, Handbook for Concrete and Cement, with quarterly supplements, Vicksburg, Miss., Aug 1949.

4. Buck, A. D., and Mather, Katharine, "Concrete Cores from Dry Dock No. 2, Charleston Naval Shipyard, S. C.," U. S. Army Engineer Waterways Experiment Station, Miscellaneous Paper C-69-6, Vicksburg, Miss., Jun 1969.

Table 1

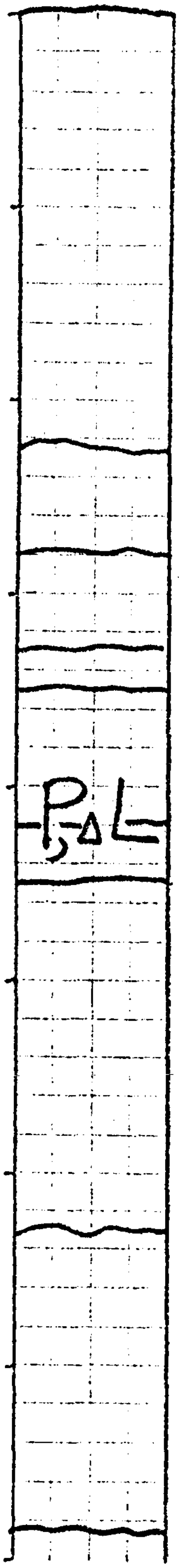
Length-Change of Concrete Cores from Monoliths 5 and 16, Oliver Lock and Dam

Monolith	Specimen	Approximate Depth, ft	Length-Change at Ages Shown Below, % (a)(b)						
			7-day	14-day	21-day	28-day	56-day	84-day	271-day
5	3	4	0.058	0.115	0.130	0.188	0.231	0.246	(c)
	4	31	0.066	0.085	0.123	0.132	0.132	0.142	0.160
	5	60	0.140	0.218	0.249	0.249	0.280	0.312	(c)
16	1	1	0.062	0.072	0.062	0.103	0.145	0.176	0.280
	2	25	0.034	0.079	0.079	0.090	0.113	0.102	0.135

(a) All values are positive.
(b) Values in Reference 1 for three specimens from Monolith 5 showed that it took from 11 to 80 days to expand from 0.06 to 0.08 percent. The specimens did not show additional expansions when measurements were stopped at 1 year.
(c) Not determined.

Log of 4-in.-Diameter Core No. MOB-4 CON-15 (Concrete) and MOB-4 DC-1 (Rock) from Oliver Lock and Dam Monolith 5* (Land Wall)

El, ft 140 | Depth, ft



Depth, ft	
0	Finished surface
	Old break
	Old break
	Old break
	Old break
4	New break
	Old break
6	New break End box 1 Box indicates depth is 6.5 ft
8	New break

Location: Sta 0 + 01.5B 27.5 ft from face of chamber wall

Coarse aggregate: Natural gravel (chert, quartzite, and quartz). Maximum size about 1-1/2 in.

Fine aggregate: Natural sand

Alkali-silica reaction gel in voids and on broken surfaces. Rims on some aggregate particles. Some chert is badly cracked. Evidence of reaction decreases with depth.

No lift contacts are evident.

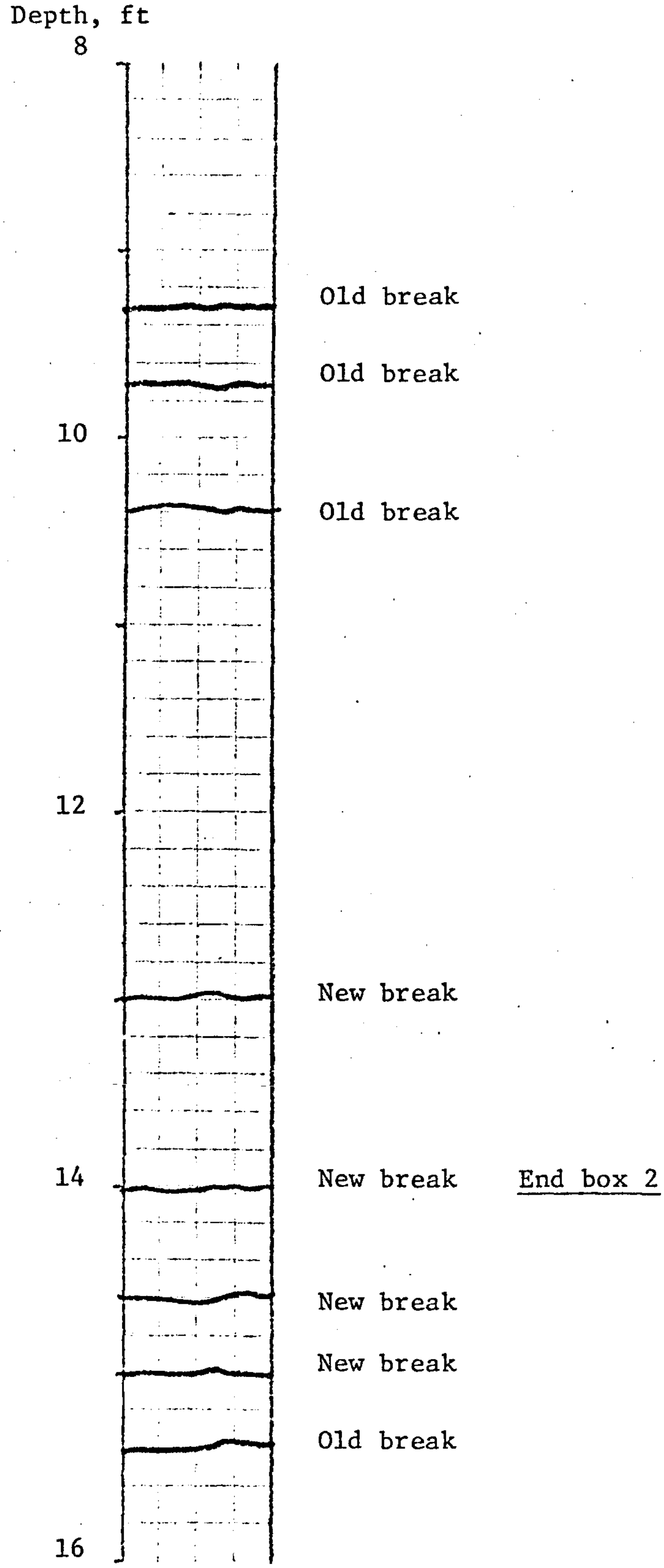
P - Petrographic sample
ΔL - Length-change sample

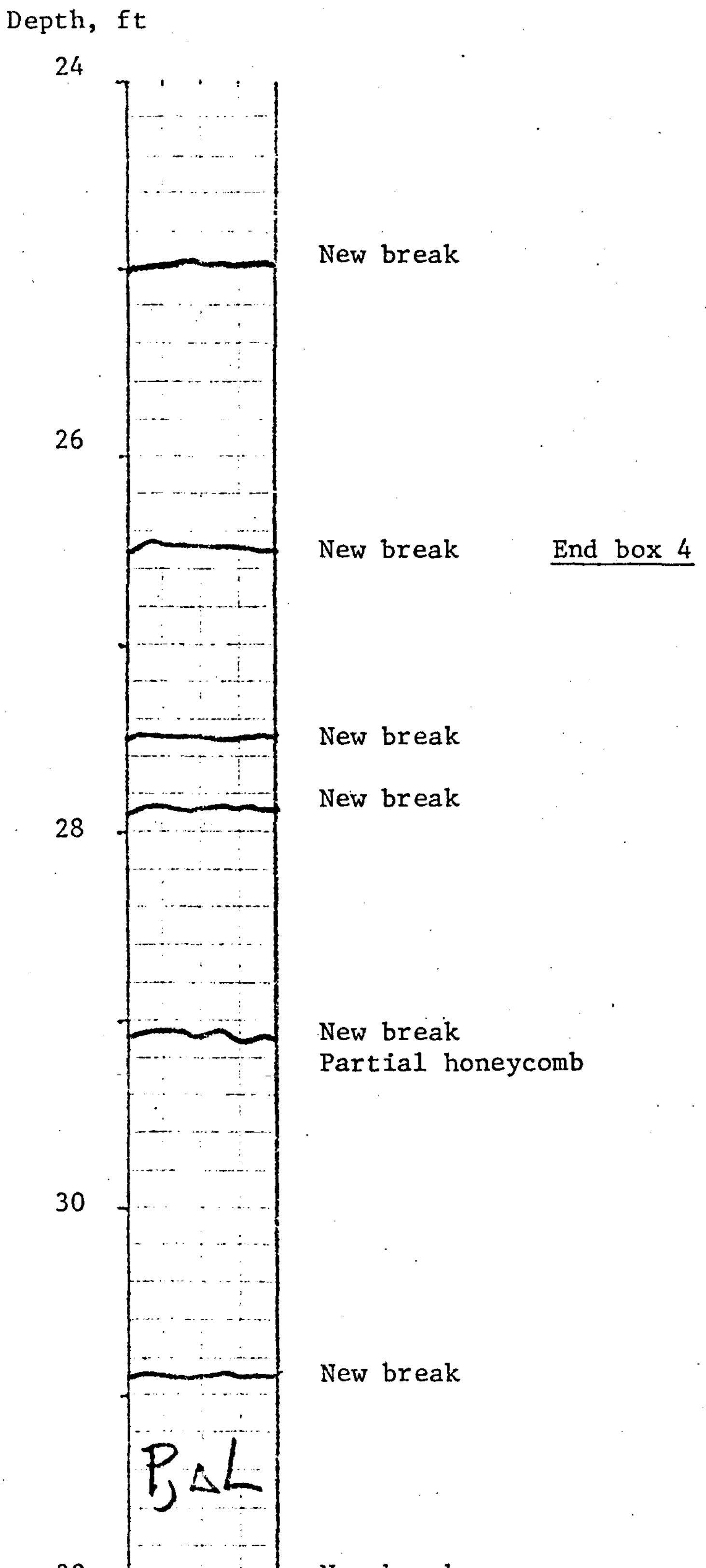
Gel is evident on the new breaks

Vertical scale:
0.2 in. = 1 ft
No horizontal scale.

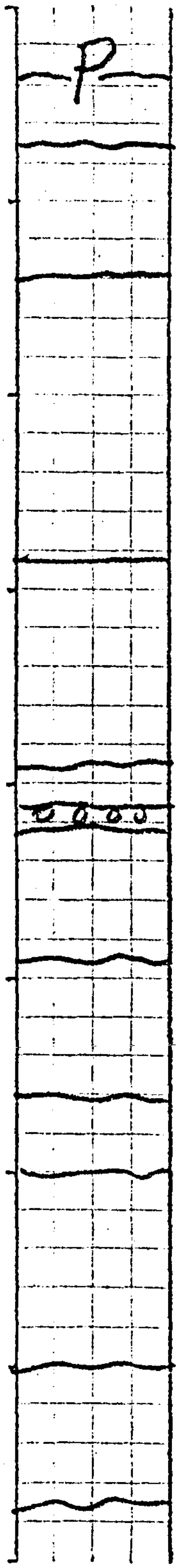
*Note: The first 63.1 ft is concrete (MOB-4 CON-15).
The rest is foundation rock (MOB-4 DC-1).

Figure 1.


Depth, ft
8
Old break
Old break
10
Old break
12
New break
14
New break
End box 2
New break
New break
Old break
16


Depth, ft
24
New break
26
New break
End box 4
New break
New break
28
New break
Partial honeycomb
30
New break
32
New break

Depth, ft



32

New break

New break — End box 5

New break

34

New break

New break — About 12 in. of honeycomb

36

Loose rock

Loose rock

New break

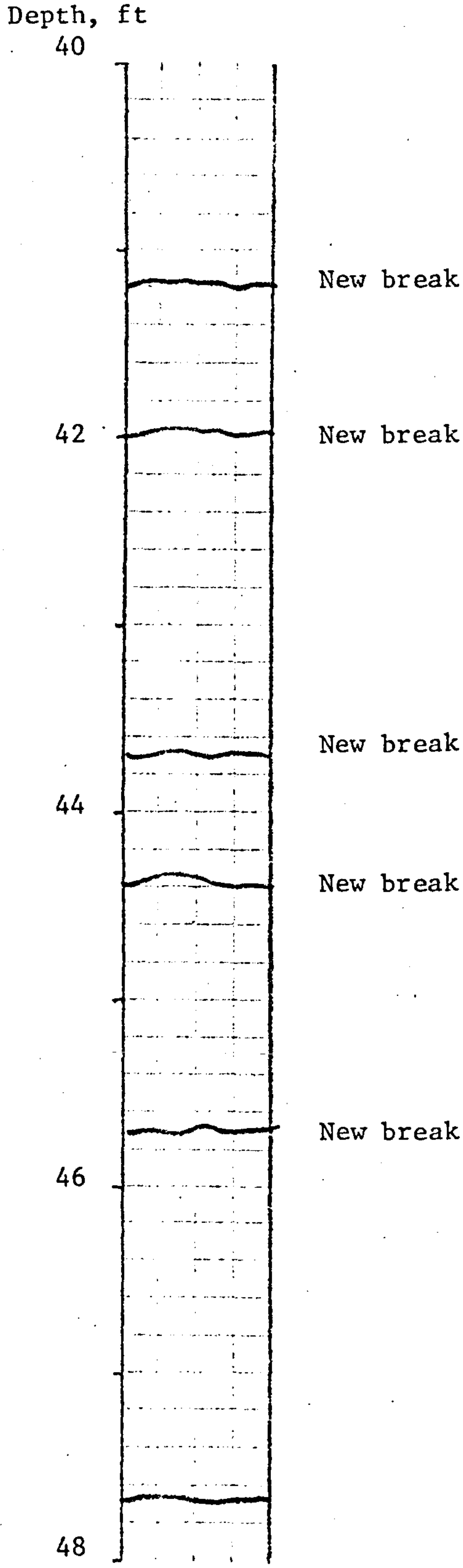
38 New break

New break — End box 6

Old? break

40

F12



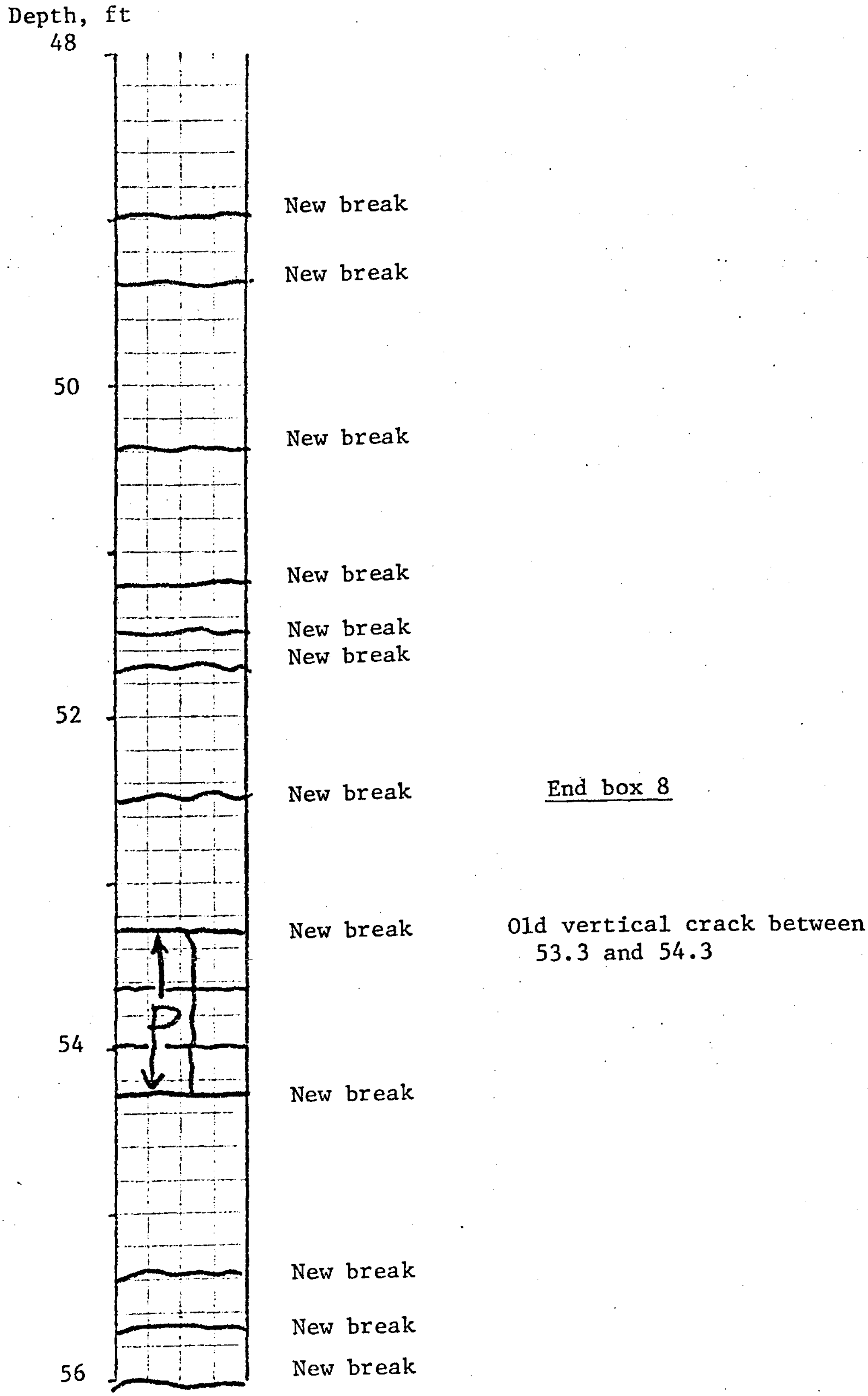
Piece from 39.7 to 41.2 ft was sealed in wax. Not opened.

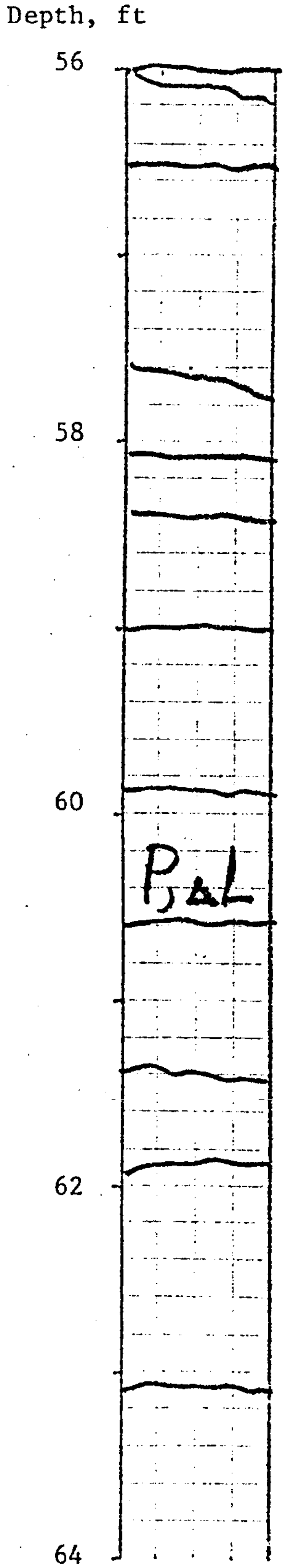
Piece from 42 to 43.7 ft was sealed in wax. Not opened.

Poor consolidation for an inch or two.

End box 7

The two pieces from 45.7 to 49 ft were sealed in wax. Not opened.


Depth, ft
48
New break
New break
50
New break
New break
New break
New break
52
New break
End box 8
New break
Old vertical crack between 53.3 and 54.3
P
54
New break
New break
New break
56
New break



New break

New break

New break

New break

New break   End box 9   Piece from 59 to 59.9 is sealed. Not opened.

New break

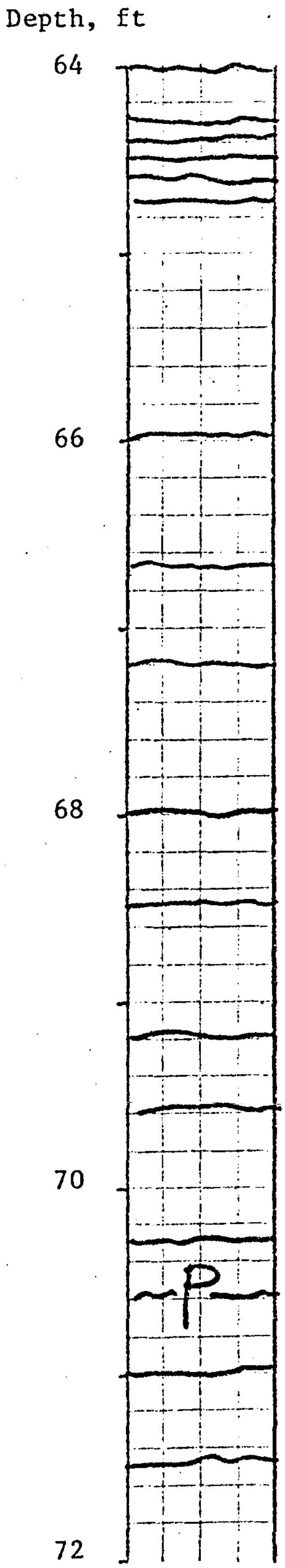
New break

Sealed in wax. Not opened.

New break

New break

Tight contact of concrete with blackish shale foundation rock. Rock is flat lying inter-fingered black shale and gray siltstone.



Same rock. Top 0.3 ft intact. Remainder split into thin layers.

End box 10

Intact flat lying rock from 64.7 to 66 ft. The rock is thin bedded interfingered black shale and gray siltstone.

Intact rock generally as above 66 to 66.7. Zones of shale and siltstone starting to show.

Thin layered shale and siltstone.

As above.

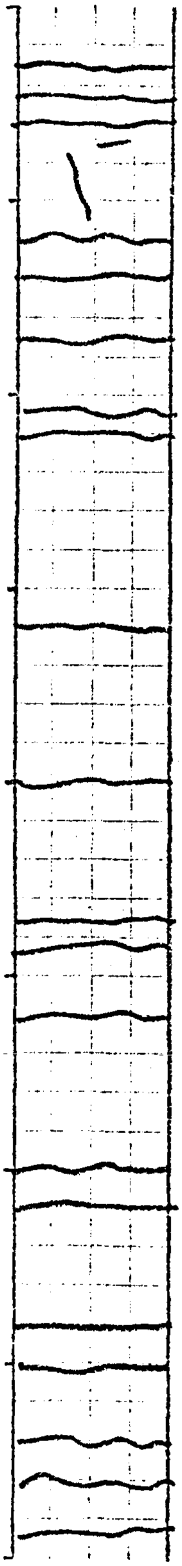
As above

68.5 to 66.2. Top half largely siltstone with thin seam (about 1/4 in.) tan clay at 68.7. Bottom half is thin bedded as above.
66.2 to 66.6. Upper two-thirds siltstone. Remainder thin bedded as above.
66.6 to 70.3. Top half thin bedded shale and siltstone. Remainder is siltstone.
70.3 to 70.6. Zoned shale and siltstone.
70.6 to 71. Zoned shale and siltstone.
71 to 71.5. Siltstone with about 1/4-in. seam of tan clay near bottom of piece.
71.5 to 72.3. Essentially all siltstone with scattered thin stringers of tan clay.

End box 11

Depth, ft

72



74

76

78

80

72.3 to 72.6. Largely siltstone with thin stringers of black shale.

72.6 to 73.2. Siltstone with thin stringer of coal. Healed vertical crack.

73.2 to 73.7. Siltstone with some thin stringers of clay and shale.

73.7 to 74.2. Black shale with some tan clay.

74.2 to 75.2. Intact flat bedded black shale with small areas of tan clay.

75.2 to 76. Same as above piece.

76 to 76.7. As above piece.

76.7 to 77.2. Top 0.2 ft interfingered thin bedded shale and siltstone as before. Bottom portion is shale.

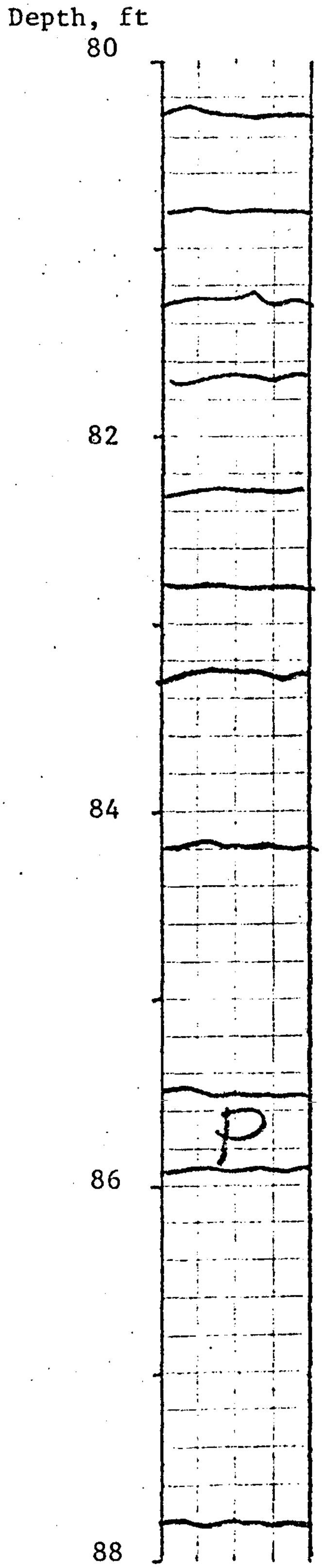
77.2 to 78. Black shale

<u>End box 12</u>

78 to 78.8. Same as above piece.

78.8 to 79.4. Essentially as above; becoming thicker bedded. Crack at 79 ft.

Same as above.



79.9 to 80.3 is broken fragments of coal.

Black shale again.

As above. Thin bedded.

81.3 to 82.3. As above.

82.3 to 83.3. As above. In addition, there are lines of coal at about 1/3 and 2/3 points.

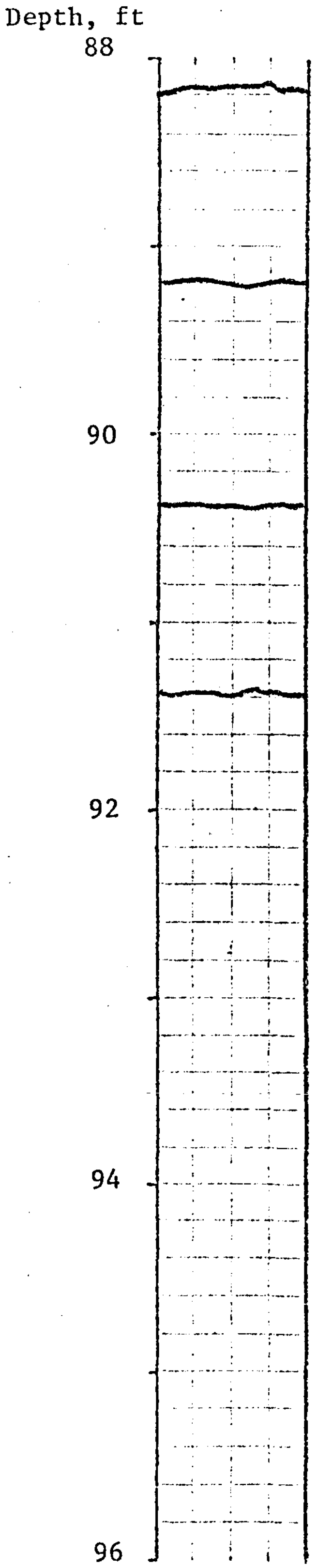
Thin bedded shale as above. Thin stringers of coal at a couple of spots.

End box 13

As above piece.

Thin bedded black shale.

As above piece with one thin stringer of coal near top of piece.



87.8 to 88.2. As above piece.

As above piece. One thin stringer of coal 0.3 from top of piece.

Thin bedded black shale.

As above. Thin seam of siltstone near top of piece.

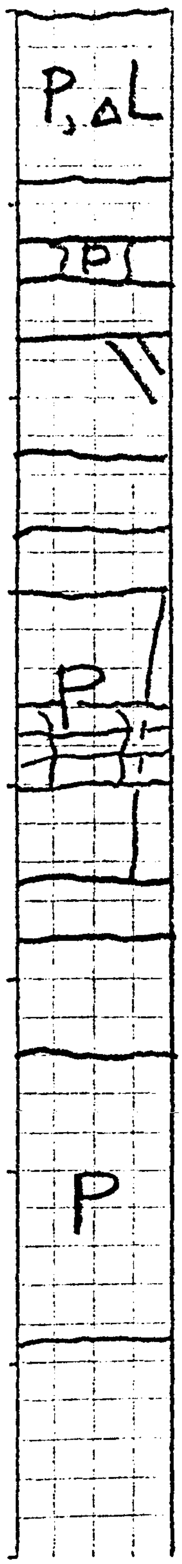
End box 14
<u>End of hole</u>

NOTE: The breaks in the foundation rock appeared to be new.

Log of 4-in.-Diameter Concrete Core No. MOB-4 CON-16
from Oliver Lock and Dam Monolith 16 (Land Wall)

El, ft
140

Depth, ft



Depth, ft	Description
0	Finished surface
	New break
	New break
	New break
	New break
2	Portion of hollow pipe, about 2-in. diameter
	New break
	New break
	Old break
	Old near vertical crack
4	Fragmented
	End of vertical crack
	New break
	New break
	Old break
6	Honeycomb area
	New break
8	

End box 1

Pieces in the interval from 6.9 to 20.7 were sealed in wax. They were not opened.

Location: Sta 4 + 31B
27.5 ft from face of chamber wall

Same aggregate as in monolith 5.

The concrete shows white gel due to alkali-silica reaction.

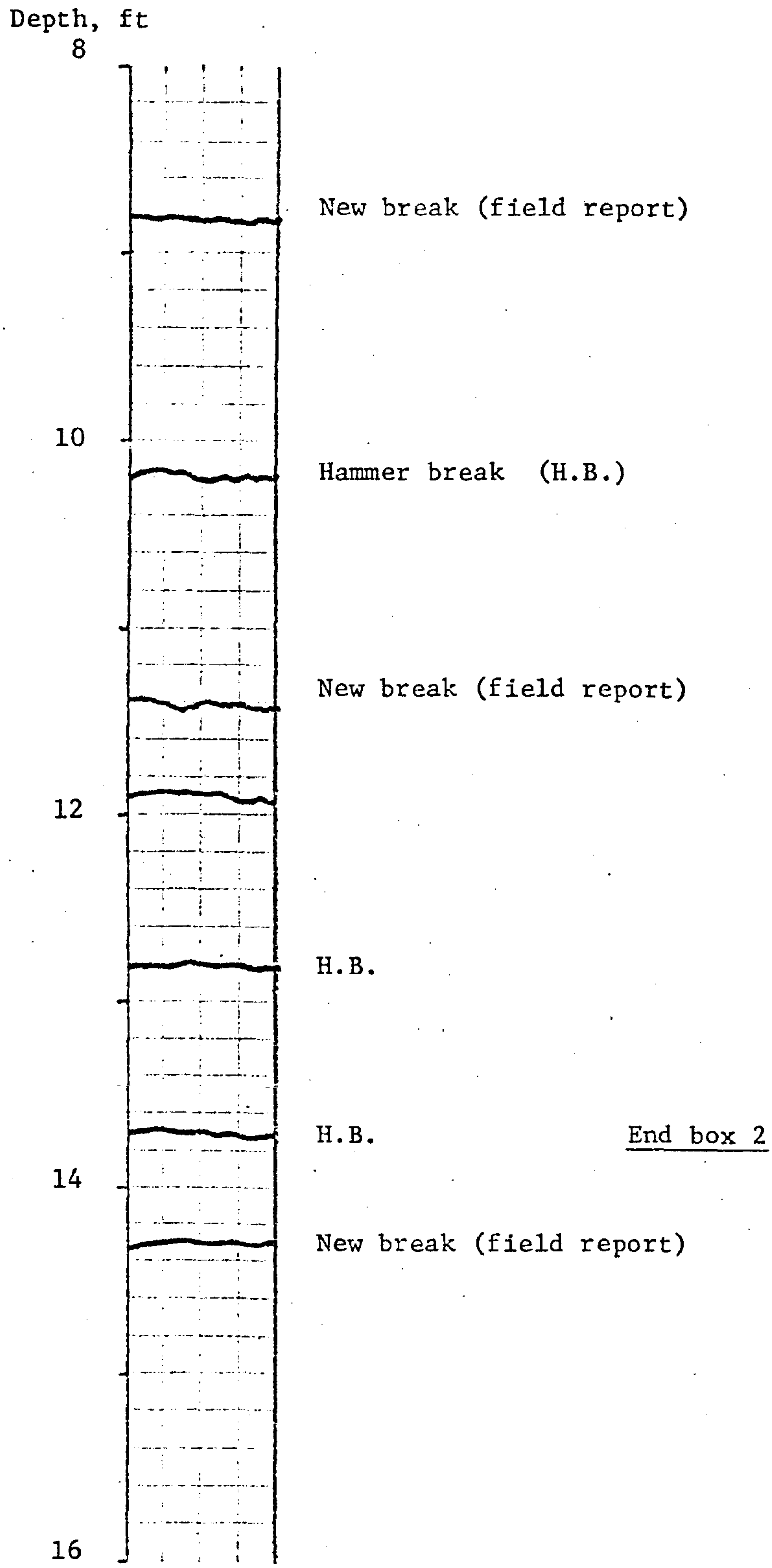
P – Petrographic sample
ΔL – Length-change sample

Lift contacts were not evident.

Vertical scale:
0.2 in. = 1 ft

No horizontal scale.

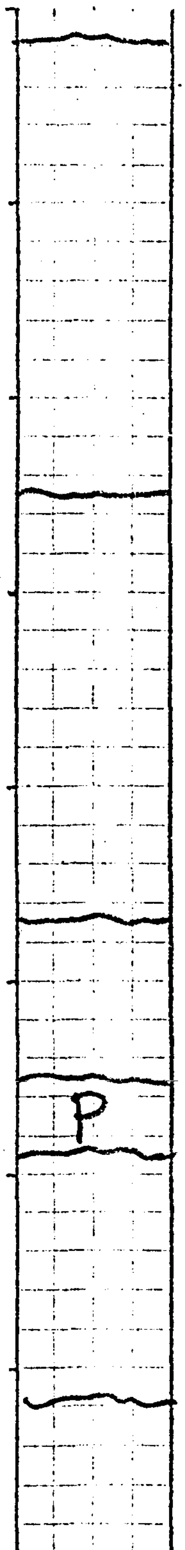
Figure 2.


Depth, ft
8
New break (field report)
10
Hammer break (H.B.)
New break (field report)
12
H.B.
H.B.
End box 2
14
New break (field report)
16

Depth, ft

16

18



New break (field report) <u>End box 3</u>

20

End of waxed pieces

New break

New break

New break

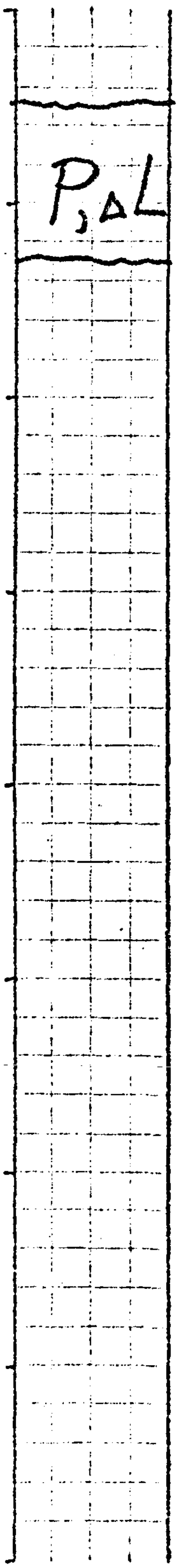
22

New break

24

Depth, ft

24



New break

New break   <u>End box 4</u>   This is all of the core that was received from this hole.

Log of Core MOB-4 DC-2 from the Spillway Foundation, Oliver Lock and Dam

El, ft 68 Depth, ft

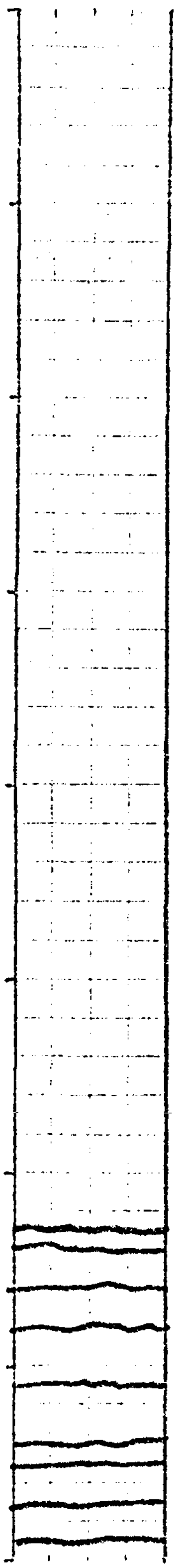
56

58

60

62

64



Top 62.3 ft of core is concrete. Not examined.

Location of hole. 4.75 ft down-stream from dam face, 7.5 ft south of North Abutment.

Elevation at top of hole 124 ft.

Summary of foundation rock.
10.1 ft of grayish shale that appears to become blacker and more compact with increasing depth.
1.9 ft of alternating thin layers of shale and fine-grained sand-stone.
2.8 ft of fine-grained sandstone.
1.1 ft of blackish shale.
0.9 ft of interlayered shale and sandstone as above.
16.8 ft total

Thin seams (<0.1-ft thick) of coal at 63.1 and 63.7 ft.

Foundation rock starts at 62.3 ft.

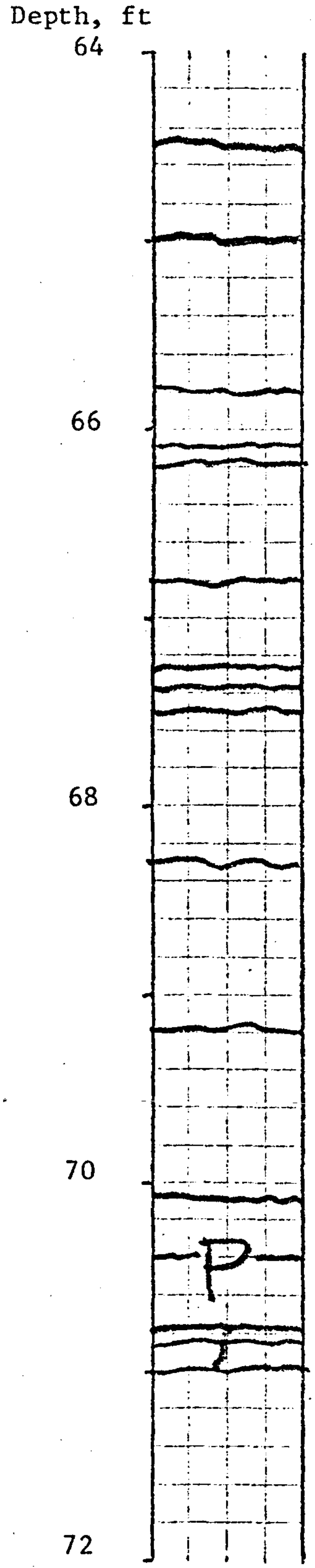
Vertical scale: 1 in. = 1 ft
No horizontal scale.
P - Petrographic sample taken.

Rock is shale from 62.3 to 72.4 ft.

0.05-ft coal at 63.1 ft.

Seam of coal at 63.7 ft.

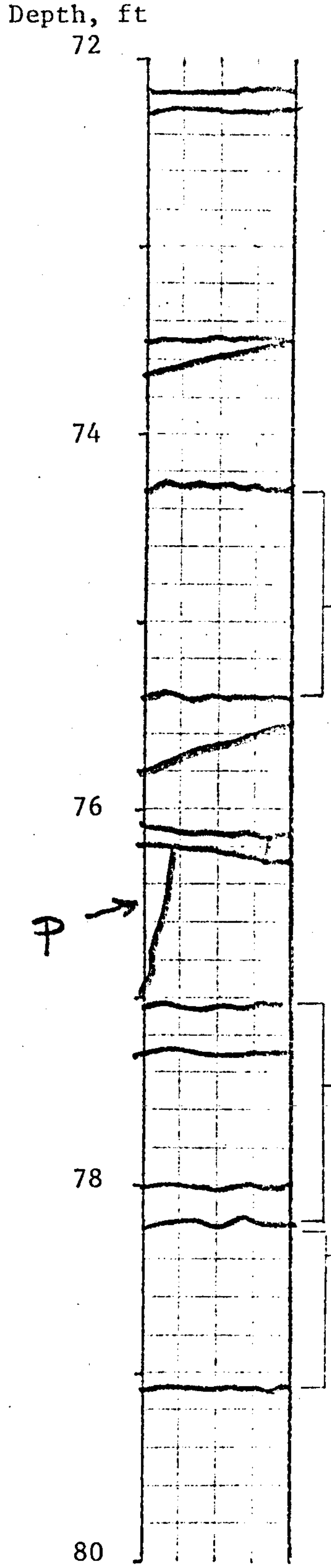
Figure 3



All shale on this sheet

Bottom of Box 10

Vertical break, 70.8 - 71.0, probably old.



End Box 11

The interval from 72.4 to 74.3 ft shows traces of sandstone interfingering with the shale.

Old(?) break dips about 20°.

May be old break

Becoming mostly sandstone in this interval

May be old break on shale seam. Dips 30°.

Old steep break

Sandstone to here with scattered shale

Shale

Interbedded shale and sandstone

End Box 12. End of core. Note on box stated that core from 79.1 to 80.7 was discarded.

APPENDIX G

FIELD DRILLING LOGS

Hole No. Monolith 5

DRILLING LOG	DIVISION: SOUTH ATLANTIC	INSTALLATION: MOBILE DISTRICT	SHEET 1 OF 2 SHEETS

1. PROJECT	OLIVER LOCK
2. LOCATION (Coordinates or Station)	27.5' From Land Wall - Sta. 0+01.5 B, Chamber Face
3. DRILLING AGENCY	MOBILE DISTRICT
4. HOLE NO. (As shown on drawing title and file number)	Monolith 5
5. NAME OF DRILLER	BILL McILWAIN
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF ~~OVERBURDEN~~ Concrete	63.1'
8. DEPTH DRILLED INTO ROCK	29.2'
9. TOTAL DEPTH OF HOLE	92.3'
10. SIZE AND TYPE OF BIT	4" x 5 1/2" Diamond
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MSL
12. MANUFACTURER'S DESIGNATION OF DRILL	FAILING 314
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED: — UNDISTURBED: —
14. TOTAL NUMBER CORE BOXES	14
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 11-2-76 COMPLETED 11-5-76
17. ELEVATION TOP OF HOLE	140.0
18. TOTAL CORE RECOVERY FOR BORING	100 %
19. SIGNATURE OF INSPECTOR	C.W. KLING

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
				100		
	5			100		EL. 137.7-135.3 CRACKED AT 1/2 FT. INTERVALS
				100		EL. 133.5 HAMMER BREAK
	10					EL. 131.9 VIBRATION BREAK
				100		EL. 126.9, 125.9 & 123.2 HAMMER BREAK
	15					
				100		EL. 123.2 - 121.6 CRACKED AT .4' INTERVALS
	20					EL. 121.5, 2" PIPE
						EL. 119.2 CORE BROKEN AT CHALCEDONY POCKET
	25		CONCRETE	100		
						EL. 116.3 VIBRATION BREAK
	30					EL. 111.3 CHALCEDONY BREAK
				100		EL. 106.9 CHALCEDONY VOIDS
	35					EL. 104.1-103.5 HONEYCOMBED CONCRETE, SOME MORTAR WASHED OUT BY DRILL WATER W/GRAVEL RECOVERED
	40			100		
						EL. 102.0 & 101.0 HAMMER BREAK
	45			100		
	50					

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 140.0	Hole No. Monolith 5
PROJECT OLIVER LOCK	INSTALLATION MOBILE DISTRICT	SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	50					EL. 86.7-85.7 VERTICAL CRACK W/BROKEN CONC.
	55		CONCRETE	100		
	60			100		
-76.9	63.1			100		CONCRETE-FOUNDATION CONTACT
	65					
	70			100		EL. 67.3 DISCARDED 0.4' OF CORE BROKEN UP IN BIT
	75			100		EL. 76.9-47.7 NO APPARENT OPEN CRACKS IN FOUNDATION. ALL BREAKS LOOK FRESH, PROBABLY DUE TO DRILL VIBRATION.
	79.8		COAL			
	80.2					
	85			100		DISCARDED 1' OF CORE AT BOTTOM OF HOLE
	90			100		
-47.7	92.3		BOTTOM OF HOLE			

ENG FORM 1836-A JUN 67 | PROJECT OLIVER LOCK | HOLE NO. Mon. 5

Hole No. Monolith 8

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION	SHEET 1 OF 1 SHEETS
1. PROJECT OLIVER LOCK		10. SIZE AND TYPE OF BIT 4" x 5½" Diamond	
2. LOCATION (Coordinates or Station) Land Wall Sta. 1+27B, 8' from Chamber face		11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL	
3. DRILLING AGENCY MOBILE DISTRICT		12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 314	
4. HOLE NO. (As shown on drawing title and file number) Monolith 8		13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED / UNDISTURBED
5. NAME OF DRILLER BILL McILWAIN		14. TOTAL NUMBER CORE BOXES 3	
		15. ELEVATION GROUND WATER	
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.		16. DATE HOLE STARTED 11-11-76	COMPLETED 11-11-76
7. THICKNESS OF ~~OVERBURDEN~~ Concrete 20.7'		17. ELEVATION TOP OF HOLE 140.0	
8. DEPTH DRILLED INTO ROCK 0.0		18. TOTAL CORE RECOVERY FOR BORING 90 %	
9. TOTAL DEPTH OF HOLE 20.7'		19. SIGNATURE OF INSPECTOR C.W. KLING	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
				100		EL. 139.2 OPEN CRACK
				26		EL. 138.0 - 135.8 ZONE OF CORE LOSS, LOOSE AND BROKEN
	5			93		EL. 134.2 VIBRATION BREAK
	10		CONCRETE	100		EL. 132.3 CORE BROKEN AT WHAT APPEARS TO BE A LIFT LINE
	15			100		EL. 130.8 & 129.7 HAMMER BREAK; EL. 125.3 & 121.5 VIBRATION BREAK
	20			100		EL. 122.9 HAMMER BREAK
119.3	20.7		BOTTOM OF HOLE			

ENG FORM 1836 (1 APR 63) PREVIOUS EDITIONS MAY BE USED — PROJECT OLIVER LOCK — HOLE NO. Mon. 8

Hole No. Monolith 16

DRILLING LOG	DIVISION: SOUTH ATLANTIC	INSTALLATION: MOBILE DISTRICT	SHEET 1 OF 1 SHEETS

1. PROJECT: OLIVER LOCK	10. SIZE AND TYPE OF BIT: 4" x 5 1/2" Diamond
2. LOCATION (Coordinates or Station): 27.5' from Land Wall Sta. 4+31B, Chamber face	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 314
4. HOLE NO. (As shown on drawing title and file number): Monolith 16	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: BILL McILWAIN	14. TOTAL NUMBER CORE BOXES: 4
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ___ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 11-8-76; COMPLETED 11-9-76
7. THICKNESS OF ~~OVERBURDEN~~ Concrete 25.3'	17. ELEVATION TOP OF HOLE: 140.0
8. DEPTH DRILLED INTO ROCK: 0.0	18. TOTAL CORE RECOVERY FOR BORING: 100 %
9. TOTAL DEPTH OF HOLE: 25.3'	19. SIGNATURE OF INSPECTOR: C.W. KLING

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
				100		EL. 139.1, 138.4, 137.6, 137.1 & 136.8 OPEN CRACKS
	5			100		EL. 138.8 - 138.5 BROKEN
						EL. 134.5 VIBRATION BREAK
	10			100		EL. 133.1 BAD CONC. CHALCEDONY PKT. EL. 131.2 & 128.8 VIBRATION BREAKS
			CONCRETE	100		EL. 129.8 HAMMER BREAK
	15					EL. 127.4 & 126.3 HAMMER BREAKS
				100		EL. 125.7 & 121.5 VIBRATION BREAKS
	20			100		EL. 138.4 LOST PRESSURE THROUGH VERT. CRACK, HIT 2" STEEL PIPE
114.7	25.3		BOTTOM OF HOLE			EL. 118.5, 118.0 & 116.8 HAMMER BREAKS
						EL. 115.5 VIBRATION BREAK

ENG FORM 1836, 1 APR 63 — PREVIOUS EDITIONS MAY BE USED

PROJECT: OLIVER LOCK | HOLE NO. Mon. 16

Hole No. Monolith 18

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MOBILE DISTRICT	SHEET 1 OF 1 SHEETS

1. PROJECT: OLIVER LOCK	10. SIZE AND TYPE OF BIT: 4" x 5 1/2" Diamond
2. LOCATION (Coordinates or Station): Sta. 5+20.58, 29.5' from Chamber face	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 314
4. HOLE NO. (As shown on drawing title and file number): Monolith 18	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: BILL McILWAIN	14. TOTAL NUMBER CORE BOXES: 4
6. DIRECTION OF HOLE: X VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF ~~OVERBURDEN~~ Concrete 27.7'	16. DATE HOLE: STARTED 11-9-76; COMPLETED 11-10-76
8. DEPTH DRILLED INTO ROCK: 0.0	17. ELEVATION TOP OF HOLE: 140.0
9. TOTAL DEPTH OF HOLE: 27.7'	18. TOTAL CORE RECOVERY FOR BORING: 100 %
	19. SIGNATURE OF INSPECTOR: C.W. KLING

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			CONCRETE	100		EL. 131.8 3/4" STEEL ROD ON SIDE OF CORE
				100		
	5			100		EL. 128.6, 127.1 & 125.9 VIBRATION BREAKS
				100		EL. 126.4 HAMMER BREAK
	10			100		EL. 123.3 CORE BROKEN IN BIT AT 2" PIPE
	15					EL. 122.4 & 118.6 VIBRATION BREAKS
				100		EL. 117.0 LOOKS LIKE LIFT JOINT
	20					EL. 115.6 VIBRATION BREAK
				100		EL. 113.8 HAMMER BREAK
	25					EL. 113.8 - 112.3 CORE DISCARDED
112.3	27.7		BOTTOM OF HOLE			

ENG FORM 1836, 1 APR 63 — PREVIOUS EDITIONS MAY BE USED — PROJECT: OLIVER LOCK — HOLE NO.: Mon. 18

Hole No. Monolith 68

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MOBILE DISTRICT	SHEET 1 OF 1 SHEETS

1. PROJECT: OLIVER LOCK	10. SIZE AND TYPE OF BIT: 4" x 5½" Diamond
2. LOCATION (Coordinates or Station): 28.75' from River Wall, Sta. 5+20.58, Chamber face	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 314
4. HOLE NO. (As shown on drawing title and file number): Monolith 68	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: BILL McILWAIN	14. TOTAL NUMBER CORE BOXES: 3
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 11-15-76; COMPLETED 11-18-76
7. THICKNESS OF ~~OVERBURDEN~~ Concrete: 18.7'	17. ELEVATION TOP OF HOLE: 140.0
8. DEPTH DRILLED INTO ROCK: 0.0	18. TOTAL CORE RECOVERY FOR BORING: 100 %
9. TOTAL DEPTH OF HOLE: 18.7'	19. SIGNATURE OF INSPECTOR: LEO CAIN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
				100		EL. 133.4 HAMMER BREAK
				100		EL. 131.8, 125.5 & 123.8 VIBRATION BREAKS
	5			100		EL. 130.8-130.3 WORE OUT BIT
			CONCRETE	100		
	10			100		USED UP 4 BITS IN FIRST 12.5'
				100		
	15			100		
				100		
121.3	18.7		BOTTOM OF HOLE			

ENG FORM 1 APR 63 1836 PREVIOUS EDITIONS MAY BE USED | PROJECT OLIVER LOCK | HOLE NO. Mon. 68

Hole No. Monolith 72

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MOBILE DISTRICT	SHEET 1 OF 1 SHEETS

1. PROJECT: OLIVER LOCK	10. SIZE AND TYPE OF BIT: 4" x 5½" Diamond
2. LOCATION (Coordinates or Station): Sta. 6+80±, 4.5' from Chamber face, River wall	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 314
4. HOLE NO. (As shown on drawing title and file number): Monolith 72	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: BILL McILWAIN	14. TOTAL NUMBER CORE BOXES: 2
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 11-19-76; COMPLETED 11-24-76
7. THICKNESS OF ~~OVERBURDEN~~ CONCRETE: 18.3'	17. ELEVATION TOP OF HOLE: 140.0
8. DEPTH DRILLED INTO ROCK: 0.0'	18. TOTAL CORE RECOVERY FOR BORING: 100 %
9. TOTAL DEPTH OF HOLE: 18.3'	19. SIGNATURE OF INSPECTOR: C.W. KLING

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			CONCRETE	100, 100, 100, 100		EL. 136.3, ½"φ STEEL BAR EL. 134.7 VIBRATION BREAK EL. 133.7 FIGURE "8" SHAPED JUNK STEEL EL. 133.3 HORIZ. CRACK, LOST SOME DRILL WATER WHICH CAME OUT ON SIDE OF WALL
130.4	9.6		GALLERY	100		
123.4	16.6		CONCRETE	100		
121.7	18.3		BOTTOM OF HOLE			EL. 132.2-130.4, 123.4-121.7 VERY HARD CONC. DRILL BITS WORE OUT RAPIDLY

ENG FORM 1836 (1 APR 63) PREVIOUS EDITIONS MAY BE USED

PROJECT: OLIVER LOCK — HOLE NO. Mon. 72

Hole No. SPILLWAY

DRILLING LOG	DIVISION: SOUTH ATLANTIC	INSTALLATION: MOBILE DISTRICT	SHEET 1 OF 2 SHEETS

1. PROJECT: OLIVER SPILLWAY
2. LOCATION (Coordinates or Station): 4.75' D.S. from 7.5' South of North Abutment, Dam face
3. DRILLING AGENCY: MOBILE DISTRICT
4. HOLE NO. (As shown on drawing title and file number): SPILLWAY
5. NAME OF DRILLER: BILL McILWAIN
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF ~~OVERBURDEN~~ Concrete: 62.3'
8. DEPTH DRILLED INTO ROCK: 18.4'
9. TOTAL DEPTH OF HOLE: 80.7'
10. SIZE AND TYPE OF BIT: 4" x 5 1/2" Diamond
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 314
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED ____ UNDISTURBED ____
14. TOTAL NUMBER CORE BOXES: 12
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 12-1-76 COMPLETED 12-5-76
17. ELEVATION TOP OF HOLE: 124.0
18. TOTAL CORE RECOVERY FOR BORING: 100 %
19. SIGNATURE OF INSPECTOR: C.W. KLING

ELEVATION (a)	DEPTH (b)	LEGEND (c)	CLASSIFICATION OF MATERIALS (Description) (d)	% CORE RECOVERY (e)	BOX OR SAMPLE NO. (f)	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) (g)
	0–5		CONCRETE	100		EL. 123.1 & 121.8 VIBRATION CRACKS; EL. 120.8 & 120.3 HAMMER BREAK
	5			100		EL. 117.8, 117.0 & 115.6 HAMMER BREAK HONEYCOMBED CONC.
	10			100		EL. 114.5 - 113.8 0.1' OF MORTAR WASHED OUT AT EL. 113.8. RECOVERED LOOSE GRAVEL AROUND EDGES. CENTER CONTINUOUS.
	15			100		
	20			100		EL. 107.6 & 106.8 HAMMER BREAKS; EL. 110.4-107.6 WAXED
	25			100		EL. 105.5, 105.0 & 104.6 HAMMER BREAKS; EL. 104.6 - 102.6 WAXED
	30			100		EL. 101.9, 101.2 & 98.0 HAMMER BREAKS; EL. 101.2 - 98.0 WAXED
	35			100		EL. 96.2 VIBRATION BREAK; EL. 94.2 & 94.5 HAMMER BREAKS
	40			100		EL. 91.9, 91.. & 90.5 HAMMER BREAKS; EL. 90.4-86.1 VERT. JOINT
	45			100		EL. 85.3 VIBRATION BREAK; EL. 82.8 & 82.3 HAMMER BREAK
	50			100		EL. 82.3 - 78.6 WAXED

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 124.0	Hole No. SPILLWAY
PROJECT OLIVER SPILLWAY	INSTALLATION MOBILE DISTRICT	SHEET 2 of 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	50			100		EL. 78.6-78.1 WAXED
						EL. 75.1 HAMMER BREAK
	55		CONCRETE	100		EL. 71.9 VIBRATION BREAK
						EL. 73.0-72.0 POOR CONCRETE
	60			100		EL. 67.0 VIBRATION BREAK
61.7	62.3					CONCRETE-FOUNDATION CONTACT
	65			100		EL. 68.3 HAMMER BREAK
				100		EL. 65.8, 65.0 & 63.5 HAMMER BREAKS
	70					EL. 61.7 - 59.5 JOINTED AT .25' INTERVALS
				100		EL. 56.8, 56.7, 55.5, 54.9 & 53.4 BREAKS
	75					EL. 54.0, 0.1' CRUMBLY MUDDY JOINT
				100		EL. 53.1 MUD FILLED JOINT
43.3	80.7					
			BOTTOM OF HOLE			EL. 53.2-53.0 VERTICAL CRACK
						EL. 51.7, 51.6, 50.6, 50.4, 49.8 & 48.8 BEDDING PLANE BREAKS
						EL. 47.8-47.1 VERT. CRACK
						EL. 47.8, 46.6, 45.9, 44.9, 43.7, 43.5 & 43.4 BEDDING PLANE BREAKS
						EL. 44.9 - 45.3 CORE DISCARDED

ENG FORM 1836-A JUN 67 — PROJECT OLIVER SPILLWAY — HOLE NO.

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

McDonald, James E
An investigation of concrete condition, William Bacon Oliver Lock and Spillway, by James E. McDonald ₍and₎ Roy L. Campbell. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1977.
1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper C-77-5)
Prepared for U. S. Army Engineer District, Mobile, Mobile, Alabama.
Includes bibliography.

1. Alkali-aggregate reaction. 2. Concrete cracking. 3. Concrete deterioration. 4. Condition survey. 5. Navigation locks. 6. Overflow spillway. 7. Repair and rehabilitation. I. Campbell, Roy L., joint author. II. U. S. Army Engineer District, Mobile. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper C-77-5)
TA7.W34m no.C-77-5