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Case Histories of Earth Pressure-Induced Cracking of Locks

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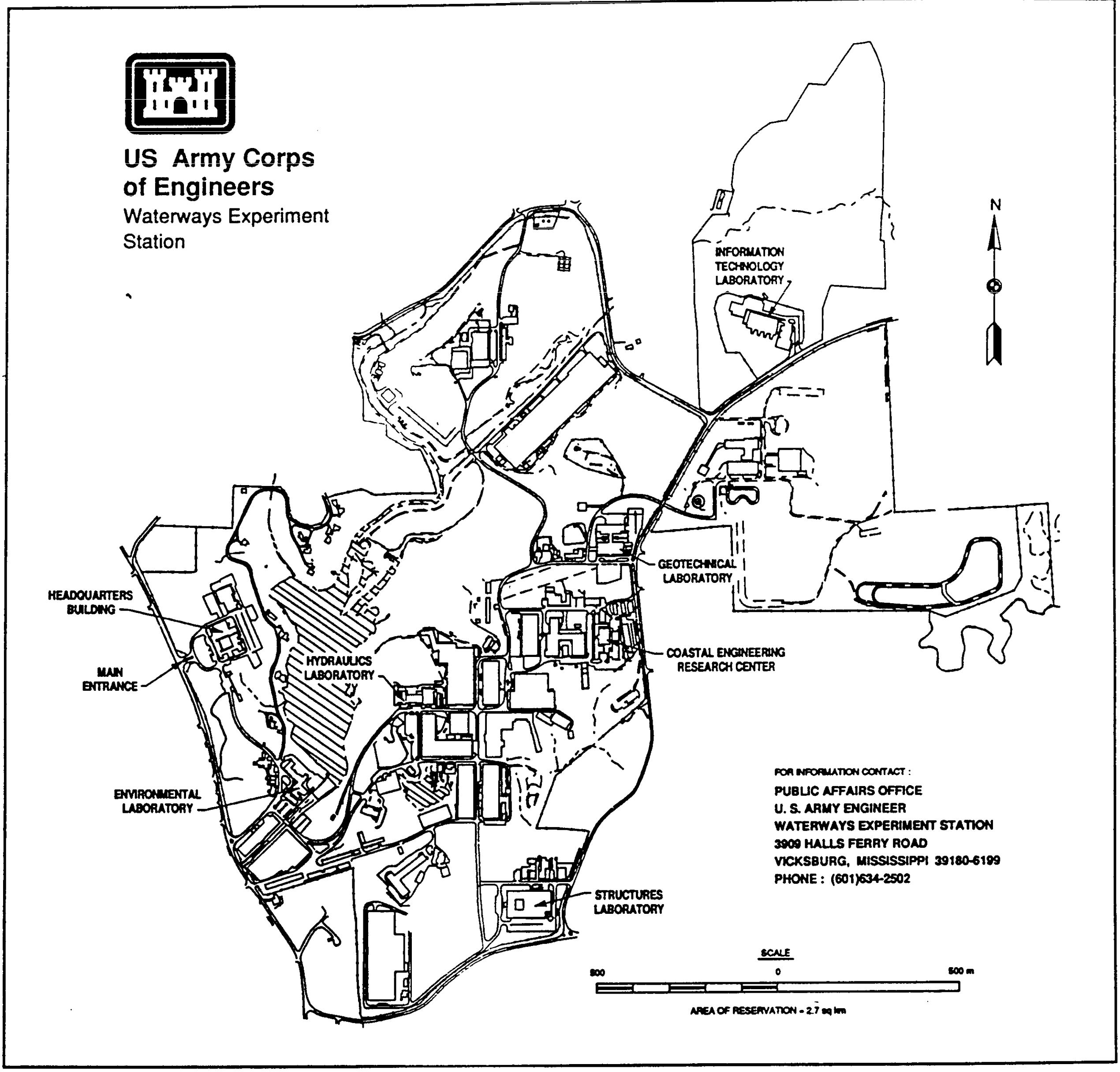

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Table Of Contents

List of Tables

List of Figures

Preface

The work described herein was sponsored by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Civil Works Research and Development program on Structural Engineering (CWR&D). The work was performed under Civil Works Work Unit 31713, "Soil-Structure Interaction Studies of Walls," for which Dr. Robert Hall, Structures Laboratory (SL), US Army Engineer Waterways Experiment Station (WES), is Problem Area Leader and Dr. Reed L. Mosher, SL, is the Principal Investigator. The HQUSACE Technical Monitor is Mr. Lucian Guthrie.

The work was performed at the Information Technology Laboratory (ITL), WES, by Dr. Robert M. Ebeling and Mr. Robert C. Patev. Dr. Reed Mosher, SL, was the author of the scope of work for this work unit and Mr. Mike Pace was the work unit monitor at ITL. The report was written and prepared by Dr. Ebeling and Mr. Patev under the direct supervision of Mr. H. Wayne Jones, Chief, CAED, ITL, and Dr. N. Radhakrishnan, Director, ITL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI (Metric) Units of Measure

Non-SI units of measurement can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
pounds per cubic foot	157.0	Newtons per cubic meter
pounds per square inch	6.8948	kilopascal
inch	25.4	millimeter
ton	2.224	kilonewton
pounds per inch	175.13	Newton per meter

1 Introduction

The US Army Corps of Engineers is responsible for designing and maintaining a large number of navigation and flood-control structures. Massive unreinforced concrete gravity walls serve many uses at many of these hydraulic structures. These concrete gravity structures are used as lock walls, are typically founded on rock, and are subjected to large differential water and earth loadings. These structures must maintain their internal structural integrity and be stable with respect to sliding and overturning. However, some rock-founded, unreinforced, concrete gravity lock walls have experienced cracking as a result of earth loadings in excess of those anticipated during structural design. This report summarizes the existing information on four locks which have experienced cracking within the unreinforced lock walls. A fifth lock which was remediated to avoid cracking is also discussed. All five lock walls retain backfill. Backfill loads were found to be the primary type of loading on the walls.

The four case histories of earth pressure-induced cracking of unreinforced mass concrete lock walls to be discussed are Snell and Eisenhower Locks on the St. Lawrence Seaway, Millers Ferry Lock on the Alabama River in Alabama, Holt Lock on the Black Warrior River in Alabama, and Demopolis Lock on the Tombigbee River in Alabama. Demopolis Lock is not known to have cracked to date but was remediated because of similarities between it and Miller's Ferry and Holt Locks. All five locks retain soil, which accounts for a significant portion of the total horizontal loading along the backs of the lock walls. All pertinent information is described for each case history, including design loadings. Earth pressure loadings used in the design of the gravity retaining lock walls are included for all locks.

Each of the five case histories contains uncertainty regarding details associated with one or more of the following issues: the history of construction and loading of the lock walls, the variability in the material properties within the as-built structure and backfill, and a lack of data for the characterization of all engineering material properties. However, some case histories are more complete than others. For example, the case histories of Snell and Eisenhower Locks are more complete due to the availability of pressure meter test (PMT) data and hydrofracture (HF) test data in the backfills. The PMT and HF data allow for the characterization of magnitude of the horizontal earth pressure forces (i.e., the demand) that the backfill applies to the lock walls. The PMT and HF data also allow for a comparison of the earth pressures that the backfill exerts on the lock walls with the pressures used in the design of Snell and Eisenhower Locks.

Factors contributing to cracking of the monoliths, other than earth pressures, are identified for each case history. The rehabilitation or remediation applied to each of the five locks is also described.

Chapter 2 describes the case history of cracking in the walls of Snell and Eisenhower Locks. These two locks are close to each other on the St. Lawrence Seaway. Both locks were constructed between 1955 and 1958. The locks are of similar geometry and have nearly identical design concrete mixtures which varies throughout the structures depending on location. The aggregates used in the concrete mixtures came from the same borrow pits. Each of the walls at both of the locks was backfilled with glacial till. During a 1967 inspection of the locks, a crack was observed to extend from the landward-ceiling corner of the culvert through to the exterior face of the back faces of the four (North and South) walls comprising each of the locks. These two locks were rehabilitated from 1967 through 1969 and consisted of the installation of post-tensioned anchors.

Chapters 3 and 4 describes the case history of cracking in the walls of Miller's Ferry Lock and Holt Lock, respectively. Like Snell and Eisenhower, the design of these two locks mirrored each other in many respects. Both the locks were constructed and operational during the mid- to late-1960's. The design concrete mix for both locks was specified to the same three grades of compressive strengths. Most notably, the materials used for the backfill and compacted behind the chamber wall had similar material properties. While the cracks in the structures were discovered at different times, Holt Lock in 1981 and Miller's Ferry Lock in 1990, both were discovered after an increase in the saturation level of the backfill. This rise in water levels was most likely due to a flood event. The rehabilitation and remediation of these structures were primarily accomplished through the use of post-tensioned anchors and removal of the backfill.

Chapter 5 describes the remediation of Demopolis Lock to prevent the potential of cracking of the chamber walls. Demopolis Lock was built in the late 1940's and was fully operational by the mid-1950's. Like Holt and Miller's Ferry, Demopolis also had a silty compacted backfill material that would retain a water table higher than was assigned in the original design. Piezometer readings taken within the backfill behind the chamber wall indicated that the phreatic surface existed at or near the upper pool elevation. Again, after a flood in March 1989, problems associated with a high water table prompted stability analyses by the Mobile District. These analyses showed a need to improve the overall stability of the structure so cracking would not occur in the future. The remediation of Demopolis Lock in 1990 and 1991 involved the removal of 25 ft of backfill from behind the chamber monoliths.

2 Snell and Eisenhower Locks

Introduction

The Snell and Eisenhower Locks were constructed between 1955 and 1958 as part of an international cooperative effort to build the Saint Lawrence Seaway. The project was placed in service in the spring of 1959. The U. S. portion of the project was authorized by the Wiley-Dondero Act of 13 May 1954. This act also created the Saint Lawrence Seaway Development Corporation (SLSDC) to construct, operate, and maintain the locks. SLSDC contracted with the Corps of Engineers to design and construct these two locks.

The Eisenhower and Snell Locks are located in the Wiley-Dondero Canal portion of the Saint Lawrence River just north of Massena, NY. The locks are about 4 miles apart and together they allow vessels to transit around the Saint Lawrence Power Project.

The Eisenhower and Snell Locks have lifts of 38 to 42 and 45 to 49 ft, respectively. The chamber dimensions are 80 ft in width and 860 ft in length from upstream miter gate to downstream miter gate, and the locks have 30 ft of water depth over the sills.

The Lock Walls and Concrete Design Mixtures

Figures 1 and 2 shows typical cross sections through the chambers at Snell and Eisenhower locks, respectively. These four rock-founded gravity retaining structures comprising the two locks were designed in 1942 by the US Army Corps of Engineers using then state-of-the-art practices (USACE 1942). The lock walls were designed as mass concrete structures. Buck, Mather, and Thorton, 1967, and Mather, 1967, provide details regarding the concrete mixtures and construction specifications for the lock walls. The following information is taken from the referenced reports.

Each monolith comprises about 40 percent interior grade concrete and 60 percent exterior grade concrete. Table 1 gives typical concrete mixtures. The concrete mixtures for exterior grade concrete differ

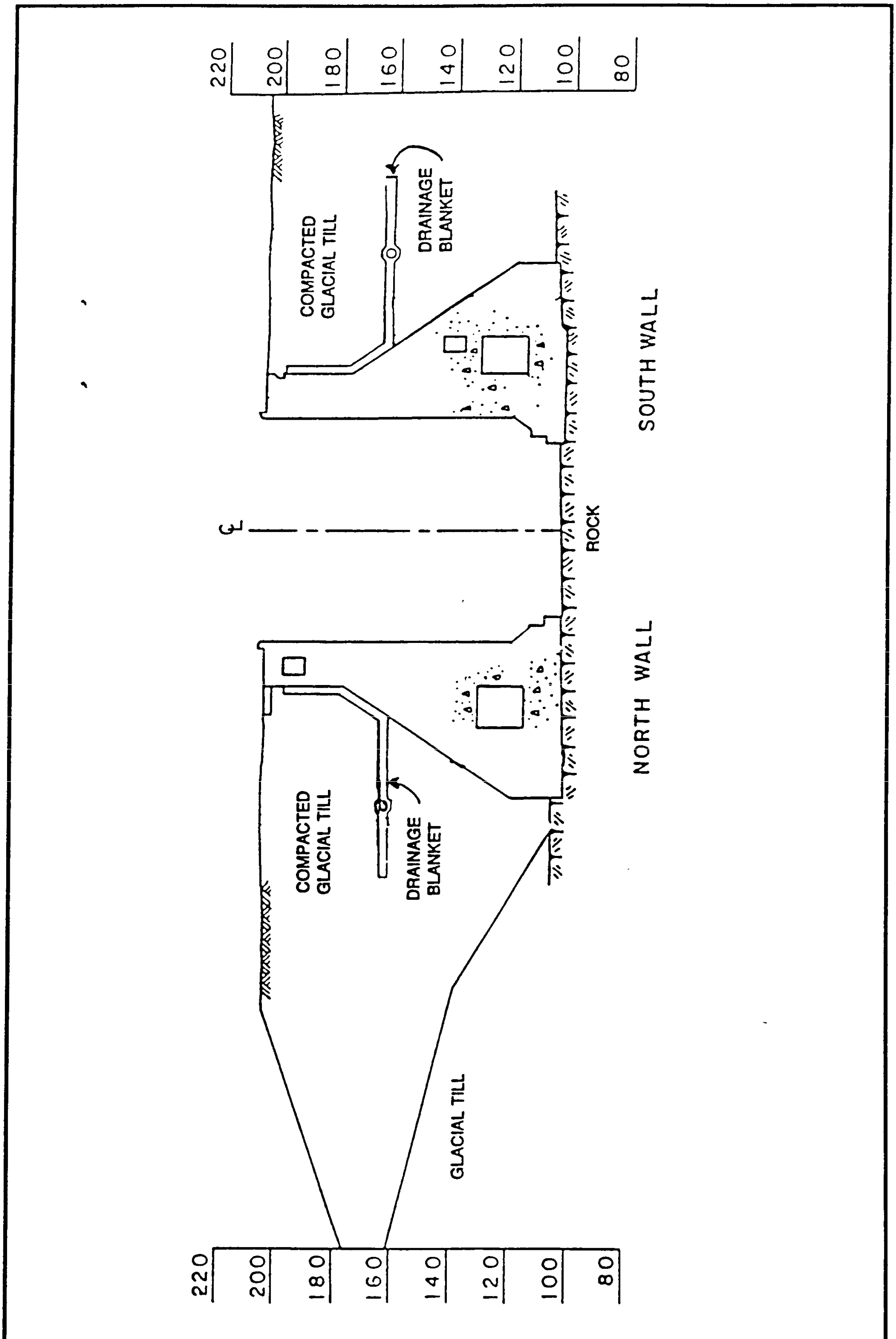


Figure 1. Typical cross section through the chamber at Snell Lock

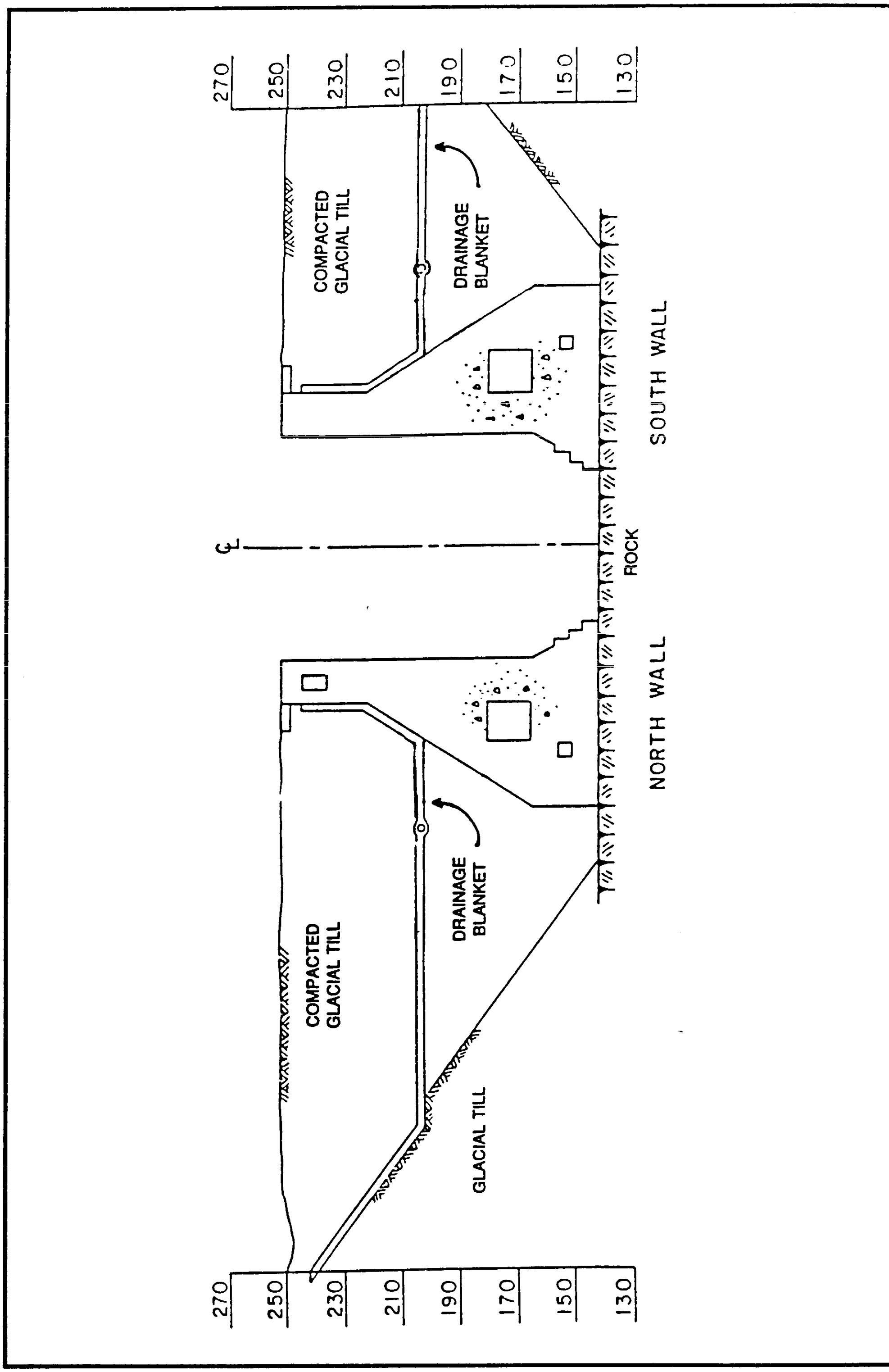


Figure 2. Typical cross section through the chamber at Eisenhower Lock

according to the maximum size aggregate being used, i.e., 6 or 3 in. More than 80 percent of the concrete contained 6 in. aggregate. Type II Portland cement (moderate heat of hydration) was obtained from a number of sources. The specifications permitted the use of natural cement as a replacement for 25 percent by weight of the Portland cement. The contractor for the Eisenhower Lock elected to use natural cement as a replacement for Portland cement, while the contractor for Snell Lock did not.

The use of interior concrete resulted in a reduction in heat evolution and, ultimately, a cost savings. Layers of concrete were required to be 20 in. thick, and lifts were restricted to a height of 5 ft in monoliths more than 16 ft wide. It was required that 120 hours elapse between lifts. Concrete was required to be moist-cured for 14 days, except in isolated cases in which membrane-forming curing compounds were permitted. During cold weather, the concrete was to be maintained at a temperature above 40 F for at least five days and above freezing for the remaining 9 days of the 14-day curing period. Concrete was required to be at a temperature of at least 40 F and not more than 60 F when placed.

All concrete was air-entrained (Table 1). The coarse aggregate was crushed stone, and the fine aggregate was either crushed or natural sand or a combination of both. The crushed stone was dolomite from Beekmantown formation produced near Helena, NY, about 12 miles from the job site. A natural sand was blended with manufactured sand during much of the work to facilitate compliance with grading requirements. The proportion of natural sand varied from 0 to 25 percent, and was greater near the completion of the work.

Table 1 Typical Concrete Mixtures For Snell and Eisenhower Locks (from Buck, Mather, and Thorton, 1967)

Use	Exterior	Exterior	Interior
Max. size aggregate, in.	6	3	6
Water-cement ratio, wt	0.49	0.49	0.64
Cement factor, bags/cu yd	3.80 to 3.88	4.20	2.75
Ratio of fine to total aggregate, % by vol.	23	28	23
Air, %*	6.1	5.7	6.2
Slump, in.	1-1/2 to 2-1/2	1-1/2 to 2-1/2	1-1/2 to 2-1/2

* In portion of concrete mixture smaller than the 1-1/2-in. sieve.

Dates of Construction of Lock Walls and Backfilling

Eisenhower and Snell Locks were constructed during 1956 and 1957. The first construction season (1956) ended with the onset of winter and saw concrete placement for the four lock walls to within the region defined by the floor of the culverts. The last lift was placed on 24 October at Eisenhower Lock and on 9 November at Snell Lock. No construction took place during the winter season due to temperature restrictions imposed by the Corps of Engineers on the curing of the concrete. Construction of the lock walls started again in the spring of 1957, and the last lift was placed on 8 June at Eisenhower Lock and 23 July at Snell Lock. The remaining 90 to 95 percent of backfilling (in elevation) of all four lock walls commenced about this time.

Concrete Deterioration at Eisenhower Lock

The concrete deterioration problem at Eisenhower Lock has been linked to the natural cement used in the concrete mix. The mix at Eisenhower contained 25 percent by weight natural cement and 75 percent by weight Portland cement. Review of the available data and reports on the concrete deterioration indicate that the mechanism of the concrete deterioration was freezing of water in the pores of the concrete. The mechanism of the concrete deterioration is clear. However, the reason that the concrete at Eisenhower Lock is less resistant to deterioration than the concrete at Snell Lock is less clear.

The concrete mixture at both Eisenhower and Snell Locks varies throughout the structures depending on the locations. The concrete mix design was the same at both locks except for the 25 percent by weight of natural cement. A detailed investigation of concrete at the two locks was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) (Buck, Mather, Thorton 1967).

Both the Corps (Buck, Mather, Thorton, 1967) and Harza Engineering Company (Harza, 1981) cited the slow development of the strength of the concrete at Eisenhower lock as the most plausible reason for the lower resistance to frost damage. The available evidence from the construction records and laboratory experiments shows that the Eisenhower concrete developed strength more slowly than did the Snell concrete. Based on the construction data, it took about 12 and 37 days, respectively, for the Snell and Eisenhower exterior grade concrete made in 1956 to reach a strength of 3,000 psi (Buck, Mather, Thorton, 1967).

It was required that the concrete be kept at a temperature above 40 F for 5 days and above 32 F for 14 days. Buck, Mather, and Thorton reported that climatological data at Eisenhower and Snell Locks show that the cores for the concrete placed 24-27 September and 2-26 October 1956 would have been subject to freezing at an age between 14 and 18 days. The exterior concrete at Eisenhower Lock placed during 1956 had an average 28-day compressive strength of 2812 psi as compared to a 28-day compressive strength of 3954 psi at Snell Lock. The results of tests of cylinders made during construction showed significant differences that persisted to the greatest age at which such tests were made. For example, the 6-month averages were 3810 psi for Eisenhower and 5080 psi for Snell. Yet by 1966, samples of nondeteriorated concrete from comparable locations within Eisenhower and Snell Locks had compressive strengths approaching one another, 5160 psi (range 4190 to 5860 psi) and 5550 psi (range 4760 to 6450 psi), respectively (Buck, Mather, Thorton, 1967).

This was regarded by Buck, Mather, and Thorton as the most probable reason for the lower durability of the concrete at Eisenhower Lock. If the concrete had matured enough, it should have been just as frost resistant as the Snell concrete has proven to be in service. The freezing of the low-frost-resistant concrete had the effect of introducing additional void space, such as microcracks, that would not otherwise be present. This additional void space, beyond that which the entrained air-void system had been provided to protect against, would provide the location in which additional water that could freeze and produce progressive deterioration of the concrete.

A second study of Eisenhower and Snell Locks was conducted in 1991 by Mosher, Bevins, and Neeley. Six-in.-diameter concrete cores were recovered over the entire height of six lock monoliths (four at Eisenhower Lock and two at Snell Lock). Figure 3 shows the compressive strengths measured on 19 samples taken from Eisenhower Lock and on 10 samples taken from Snell Lock. The compressive strengths averaged 5230 psi (range 4070 to 6050 psi) and 6620 psi (range 3730 to 8590 psi), respectively. The average compressive strength for the 1991 tests of cylinders taken from Snell Lock was more than 1000 psi greater than the average compressive strength measured in 1966. However, the average compressive strengths are nearly the same in the 1966 and 1991 studies for Eisenhower Lock.

The lower average value for compressive strength at Eisenhower from the Mosher, Bevins and Neeley (1991) study is biased because of the larger

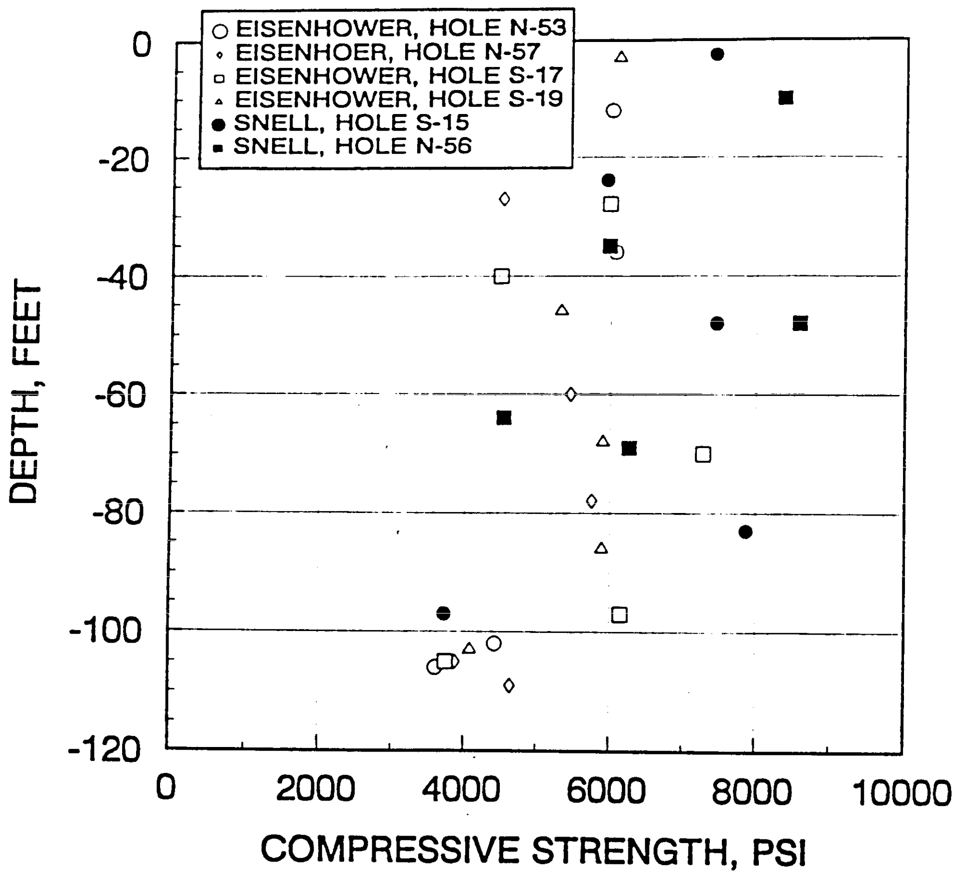


Figure 3. Variation of concrete compressive strength with depth (from Mosher, Bevins, andNeeley1991)

number of test specimens from the lower portion of the wall when compared with the number of specimens from Snell, six from Eisenhower and only one from Snell. Using the average compressive strength from specimens taken from the upper portion of Eisenhower borings and comparing that to the Snell average, the difference is only 9 percent, which is approximately the same difference reported by Buck, Mather, and Thorton (1967).

Extensive concrete repairs have been made to the chamber faces, filling and emptying culverts, gate recesses, pintle bases, and sills at Eisenhower Lock. SLSDC has had an aggressive program to repair and replace deteriorated concrete. Mosher, Bevins, and Neeley (1991) concluded that the concrete deterioration at Eisenhower Lock will be a continuing problem.

Culvert Cracks at Snell and Eisenhower Locks

In January 1967 during inspections of the Eisenhower Lock filling and emptying culverts by Saint Lawrence Seaway Development Corporation and Corps of Engineers personnel immediately after winter dewatering, a continuous crack was observed along the landward-ceiling corner of the culvert in the north wall. Further investigation revealed that this crack was continuous from the culvert through to the exterior backfilled face of the lock wall. At the time, the crack leaked water in various amounts along its entire length, and fresh spalls of concrete were found lying on the culvert floor beneath it. Subsequent detailed inspections and other pertinent investigations revealed that the crack extended along the culvert between the upper and lower valve monoliths.

After initial discovery of the crack in the north culvert, a close inspection was made in the south culvert. The same kind of crack that existed in the north culvert was present in the south culvert at its landward-ceiling corner, as shown in Figure 4. Its longitudinal extent was the same as that of the north culvert crack. Examination of the Snell Lock culverts revealed similar cracks in that lock.

With these cracks extending through to the backfill, the overall stability of the lock walls became a matter of serious concern. Under certain conditions, all wall loads must be absorbed by the 15-ft-thick section between the culverts and the faces of the lock chambers. This was thought to be especially serious with respect to Eisenhower Lock where portions of this section were deteriorated and, thus, less capable of supporting the imposed loads. The core boring program underway concerning the

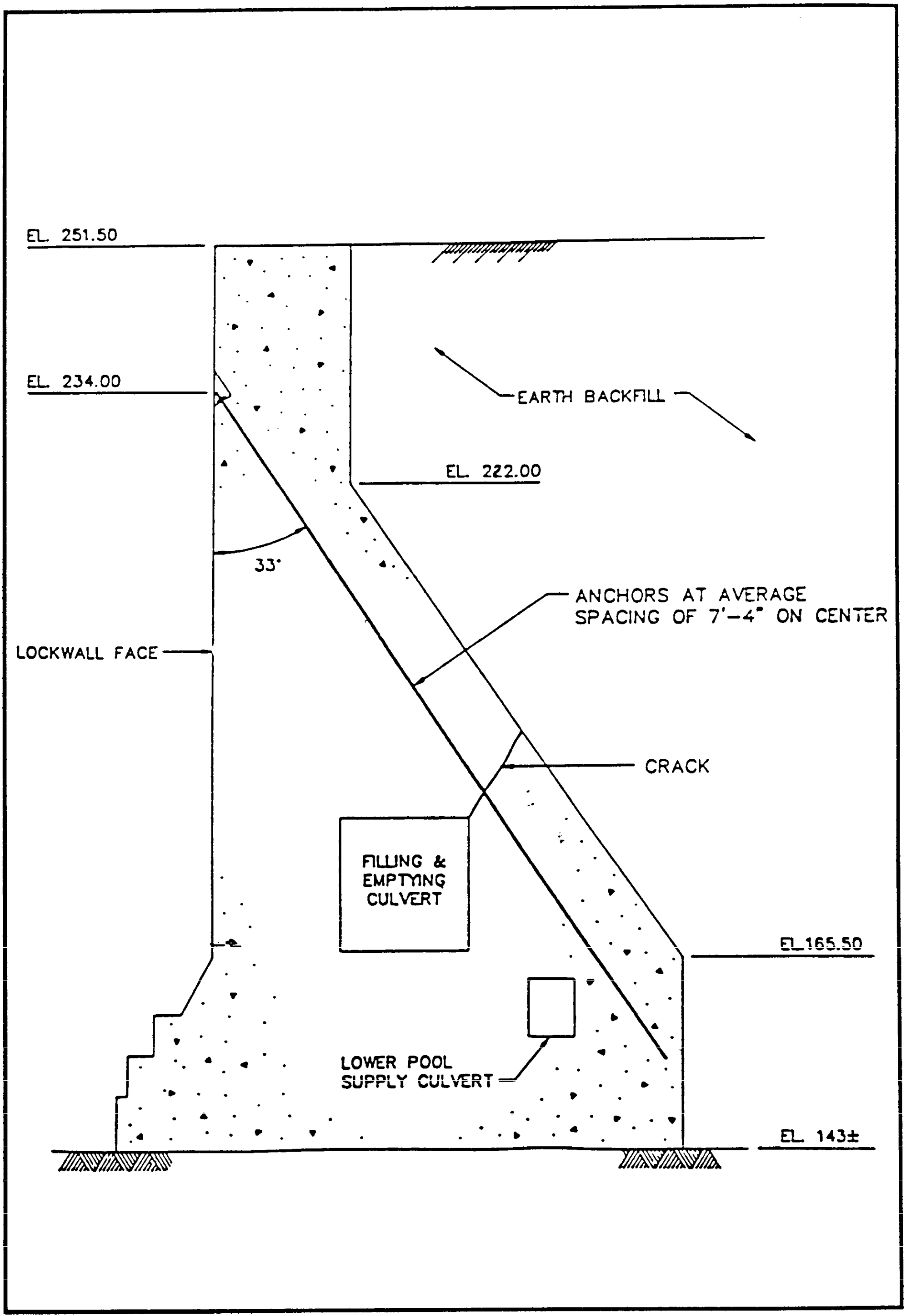


Figure 4. Typical section through south wall chamber monoliths at Eisenhower Lock

problem of deterioration was enlarged to include exploration of these cracks. To obtain additional data on the extent of the cracks and condition of the surrounding concrete, joint meters were installed across the cracks to measure changes in the sizes of the cracks during lock operations. Bar joints were installed across the lock chambers to measure relative movements of the lock walls, and an inclinometer was used to measure tilting of the lock walls during operation. Alignment control was set up to measure any lateral displacement of the wall, and piezometers were installed in the backfill areas to provide information on saturation levels and drainage patterns. Correlated flow measurements were taken of flows in the backfill drains.

Based on this information and the information gained by the 1966-67 concrete survey, a determination was made by a Saint Lawrence Seaway Development Corporation convened Board of Consultants that a complete rehabilitation program was necessary to guarantee continued structural integrity and stability and to ensure ability to operate the locks. In a letter dated June 26, 1967, from the Administrator, the Corps of Engineers was requested to perform the necessary design and contracting services concerning the proposed rehabilitation program for the Eisenhower and Snell Locks to restore the locks to a condition of full stability.

Priority was given to Eisenhower Lock. The rehabilitation work for the crack consisted of placing post-tensioned anchors across the culvert crack (both walls). This was accomplished during the winter shutdown of 1967-68 by contract with Peter Kiewit & Sons. Similar post-tensioned anchors were placed across the culvert cracks at Snell Lock during the winter of 1968-69 under contract with Morrison-Knudsen.

Rehabilitation of Snell and Eisenhower Locks Using Post-Tensioned Anchors

In the winters of 1967-68 and 1968-69, post-tensioned anchors were installed in Eisenhower and Snell Locks, respectively. The north and south walls of the Eisenhower and Snell Locks have 14 monoliths with narrow tops and sloping backs. Figures 1 and 2 show typical sections. These walls in the chamber portion of the locks are 606 ft long. Eighty-two and eighty-three anchors were installed in the north and south walls, respectively, of each lock as shown in Figure 4. Six 636-kip anchors were installed in each monolith. The average spacing of the anchors was 7.33 ft.

Review of data and stability analyses show that the saturation level in the backfill of Eisenhower Lock was at el 221 ft[1] at the time of anchor installation. This elevation is 16 ft higher than was designed for originally. Recent field inspection of drainage pipe and a dye tracing study by Gannet Fleming Geotechnical Engineering, Inc. (1986) of the drainage blanket show that the drainage pipes are operational and continuous. From historical data and recent observations, it was determined that the static groundwater level is at the drain invert in the drainage blanket for the soil below the blanket. These data also show that the soil is saturated up to 18 ft above the drain in the same locations. These high piezometer levels observed in the upper portion are the result of a perched water table fed by the water level in the natural soil. While the drainage blanket and pipe are functioning, they are not connected to the soil above the drainage blanket.

In February 1989, the Saint Lawrence Seaway Development Corporation and the Corps of Engineers conducted an anchor investigation program at Eisenhower Lock. The objective of the investigation was to determine whether the post-tensioned anchors in the chamber monoliths at Eisenhower Lock have sustained any significant corrosion due to water leakage through the existing culvert cracks. Of the 165 anchors in Eisenhower Lock, two anchors were examined, one in monolith N-51 and one in monolith S-17, at locations near the greatest amount of leakage through the existing culvert cracks. Significant corrosion was considered to have the greatest potential at these locations. The investigation consisted of excavating the concrete from inside the culvert to expose a short section of each anchor for visual inspection and dimensional measurements.

Results of the anchor investigation showed that the grout was intact and completely surrounded by the anchor strands in the exposed areas. The anchor strands were observed to be as shiny as new and there was no evidence of any surface corrosion or pitting. The results of this investigation showed that the anchors were in excellent condition. It was further concluded that post-tensioned anchors in Eisenhower Lock should remain structurally sound and should adequately serve the anticipated life expectancy of the lock. It was concluded that in any future structural evaluation of the lock, the existing anchors should be assumed to be 100 percent effective.

[1] All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Splitting Tensile Test Measurements

Thirty-one of the 6-in.-diameter concrete cores recovered in 1991 by Mosher, Bevins, and Neeley were used to measure the splitting tensile strength of the concrete that comprises the Eisenhower and Snell Locks. Figure 5 shows the distribution with elevation of the tensile splitting strengths measured on 21 samples taken from Eisenhower Lock and on 10 samples taken from Snell Lock. The tensile splitting strengths averaged 581 psi (range 390 to 790 psi) and 650 psi (range 495 to 930 psi), respectively.

Earth Pressures

The four rock-founded, massive gravity retaining structures comprising the two locks were designed in 1942 by the US Army Corps of Engineers using then state-of-the-art practices (USACE 1942). The horizontal earth pressures used in the designs assumed an equivalent fluid pressure of 33 psf and 93 psf per foot of depth for the moist and submerged glacial till, respectively (USAE 1942, or Diviney 1990).

The sizes of the excavations during construction of the locks were significant given the sizes of the monoliths (Figures 1 and 2). The excavated glacial till, consisting of fine to coarse gravel and fine to coarse sand with some silt, was stockpiled at the respective sites. The glacial till at Snell Lock is more fine grained than the till at Eisenhower Lock (Figure 2 gradation curves in Diviney 1990). Backfilling commenced immediately after construction of the monoliths was completed. Large off-road dump trucks and heavy, self-propelled and dozer-drawn compactors were used to place and compact the backfill (Diviney 1990).

The in-place density of the backfill soil has been a point of controversy for some time because of the high values measured during in-situ tests (Mosher, Bevins, and Neeley 1991). Assumed moist densities from previous studies have ranged from 125 pcf in the 1955 Corps Design Memorandum to 140 pcf used in the Harza Engineers' study (1981). Measured backfill density values from in-place tests range from a low of 135.5 pcf to a high of 150.6 pcf (Empire Soils Investigation, Inc. 1985). Mosher, Bevins and Neeley (1991) evaluated all available information on density measurements at both locks and assigned total unit weights of 140 and 148 pcf to the backfill soils of Eisenhower and Snell Locks, respectively, for their SSI analyses of the two locks.

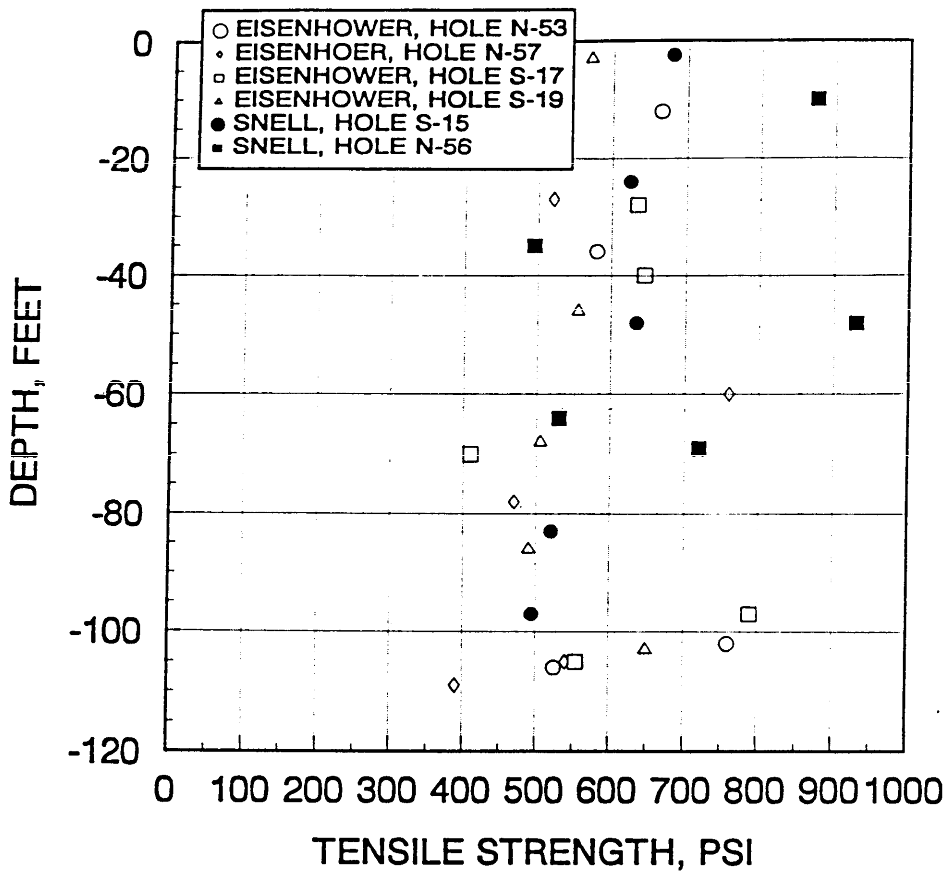


Figure 5. Variation of concrete tensile strengths with depth (from Mosher, Bevins, andNeeley1991)

Using these values for the unit weights of soil backfill, the equivalent fluid pressures used in the design of the two locks correspond to horizontal earth pressure coefficient, K_h values between 0.21 and 0.24 for moist and submerged backfill soils.

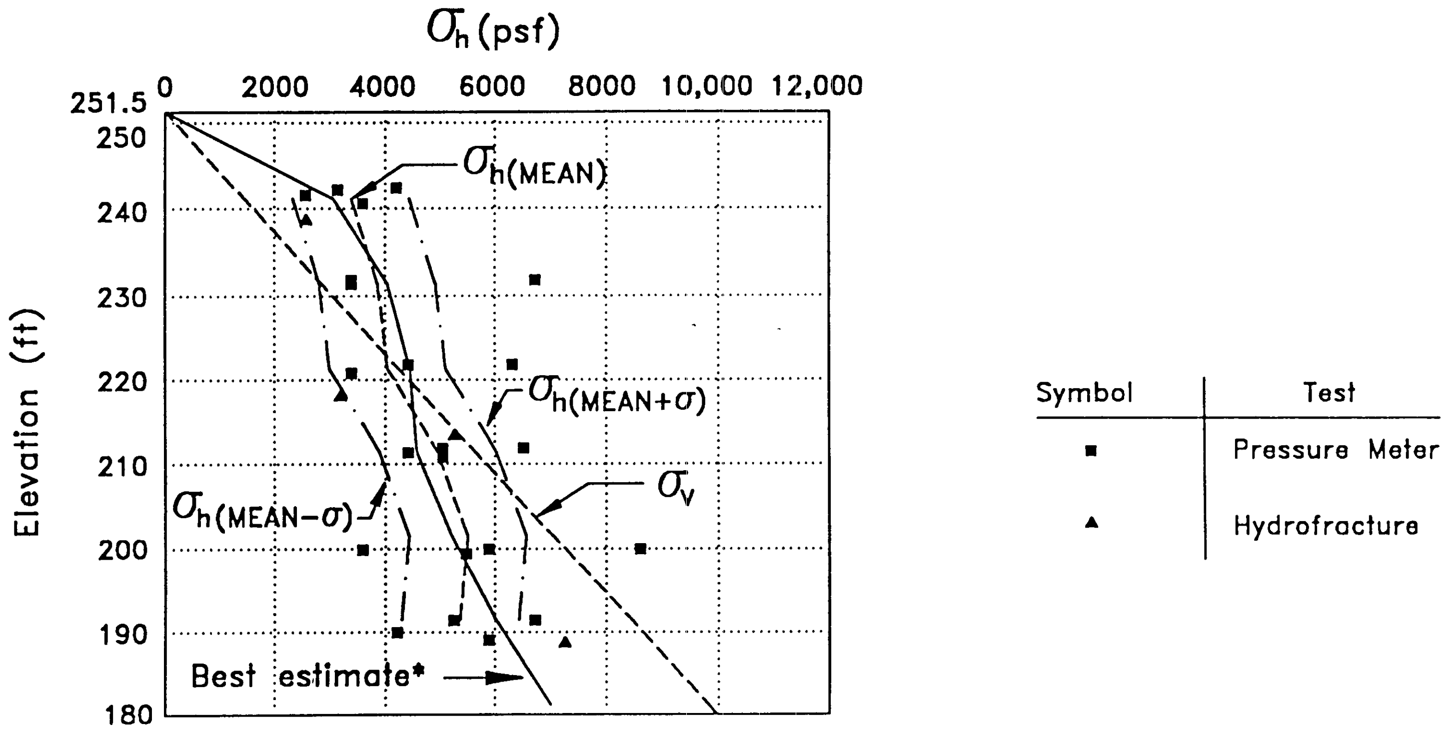
In-situ horizontal earth pressure investigations

In 1986, an in-situ testing program was conducted using pressure meter testing (PMT) and hydrofracture testing (HF) in the backfills of the two locks to determine the state of horizontal (total) stress. Piezometers were also installed during the field investigations to determine the pore water pressures within the backfills. Forty-three successful PMT tests were conducted in eight bore holes (four at each lock) made to 60-ft depths through the backfill over the heels of the monoliths. Twenty HF tests were conducted in seven of the PMT test boreholes. Details regarding the tests, measurements, and their interpretation are described in Schmertmann (1986), Goldberg-Zoino (1986), Gannett Fleming (1986), and Diviney (1990).

Figures 6 and 7 show the variation in horizontal (total) stress, σ_h, with elevation from PMT and HF testing results for Eisenhower and Snell Locks, respectively. The three dashed lines in these figures are the mean, mean plus standard deviation, and mean minus standard deviation computed from the PMT data. The short dashed line shows the variation in total overburden pressure, σ_v, with elevation and is included for reference. The solid line designated as best estimate was computed from Snell and Eisenhower Locks combined data, as reported in Diviney (1990), after Schmertmann (1986).

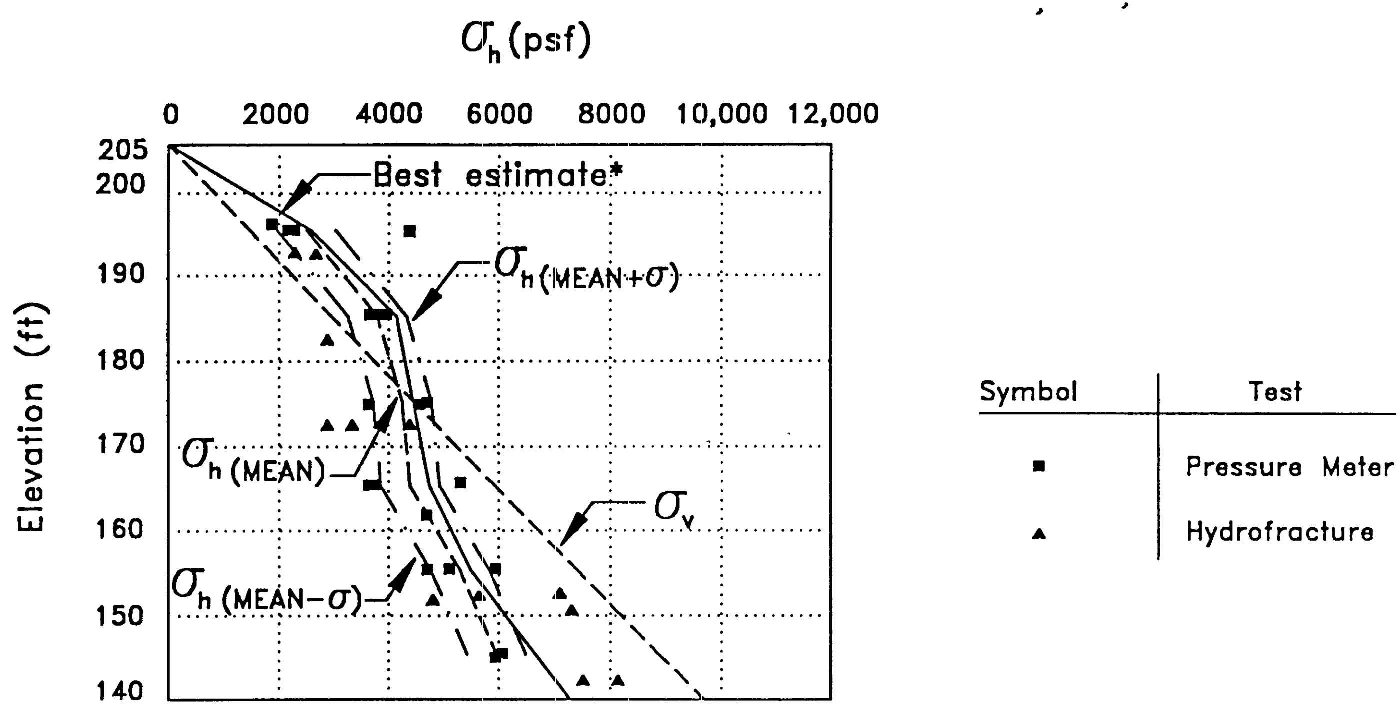
The statistical evaluations of the PMT test data shown in Figures 6 and 7 were made using weighting factors based on Schmertmann's interpretation of the data. Schmertmann's interpretation of the test data included a qualitative evaluation of each test (Schmertmann 1986 or Diviney 1990). Schmertmann rated each data point as either very good, good, fair, or poor. A subjective weight equal to 1.0, 0.5, 0.25 or 0, respectively, was assigned to each data point in this study according to Schmertmann's rating. These subjective weights are designated as p_i in subsequent equations.

The average horizontal (total) earth pressure was calculated at each elevation of testing using the PMT data, designated as lower case x_i, using



* Best estimate computed from Snell and Eisenhower Locks combined data as reported in Diviney (1990), after Schmertmann (1986).

Figure 6. Horizontal stress from pressure meter and hydrofracture testing at Eisenhower Lock



* Best estimate computed from Snell and Eisenhower Locks combined data as reported in Diviney (1990), after Schmertmann (1985).

Figure 7. Horizontal stress from pressure meter and hydrofracture testing at Snell Lock

$$E(X)_{el} = \sum_{i}^{n} wf_i * x_i \tag{1}$$

where the weighting factor wf_i for each data point is given by

$$wf_i = \frac{p_i}{\sum_{i}^{n} p_i} \tag{2}$$

$E(X)_{el}$ designates the average or mean value of horizontal (total) earth pressure for the n values of PMT data at a specified elevation. The value of n was 4 or less at each elevation. Equation 2 guarantees that the sum of the weighting factors, wf_i, applied to each corresponding value of PMT test data x_i in equation 1, equals 1.0 at each elevation of testing.

The standard deviation of the weighted PMT data was computed from the variance of the data about the variation in $E(X)_{el}$ with elevation. Recall that the average, or more precisely, the expected value of the PMT data was computed at each elevation for which the in-situ tests were conducted using equation 1. The variance about $E(X)_{el}$ in Figures 6 and 7 is given by

$$\begin{aligned} Var(X) &= E(X - E(X)_{el})^2 \\ &= \sum_{i}^{N} WF_i * (x_i - E(X)_{el})^2 \end{aligned} \tag{3}$$

where the weighting factor WF_i for each data point is given by

$$WF_i = \frac{p_i}{\sum_{i}^{N} p_i} \tag{4}$$

Note that the variance in PMT data about $E(X)_{el}$ is over the entire 60-ft depth of testing. Thus, the numbers of PMT data points N equal 24 and 19 for Eisenhower and Snell Locks, respectively, in equations 3 and 4. Equation 4 guarantees that the sum of the weighting factors, WF_i, applied to each corresponding value of PMT test data x_i in equation 3 equals 1.0. The standard deviation $\sigma(X)$ of the PMT test data is given by

$$\sigma(X) = \sqrt{Var(X)} \tag{5}$$

The standard deviations of the PMT data for Eisenhower and Snell Locks are 1063 psf and 530 psf, respectively.

Schmertmann concluded that the tests at Snell Lock were of better quality. The computed value of standard deviation for the PMT test data for Snell Lock being approximately one-half the value for Eisenhower Lock supports Schmertmann's conclusion if the "scatter" of the data is used as a measure of quality.

Figures 6 and 7 show the statistical evaluations made in this study (with subjective weights assigned to each data point based on Schmertmann's qualitative evaluation of the test data) are in agreement with the best estimate reported in Diviney (1990) after Schmertmann (1986).

Twenty-eight vibrating wire piezometers were installed in select boreholes at both locks. Data measured with this instrumentation were used to develop the distributions of pore water pressures with elevation in the backfills of Eisenhower and Snell Locks, as shown in Figures 8 and 9, respectively. The piezometers indicated a perched water table approximately 30 ft below the surfaces of the backfill at both locks. The water pressures are hydrostatic to the top of drains in the backfills at both locks. These drains, shown in Figures 1 and 2, are at midheight (approximately) within the backfills. Hydrostatic water pressures were measured below the drains in the backfills and are shown in Figures 1 and 2.

The presence of the perched water table above the drains in the backfills was an unanticipated source of load on the two locks. However, the SSI studies of the two locks by Mosher, Bevins, and Neeley (1991) demonstrated that this factor alone could not have been responsible for the cracks in the sections of the four lock walls.

Figures 8 and 9 shows the mean, mean plus standard deviation, and mean minus standard deviation of the horizontal total earth pressure σ_h computed from the PMT data for the two locks. The corresponding horizontal effective earth pressure σ'_h distributions are also shown in these figures.

Figures 10 and 11 shows the mean, mean plus standard deviation, and mean minus standard deviation of the horizontal effective earth pressure σ'_h for the two locks. The corresponding horizontal earth pressure coefficient K_h distributions are also shown in these figures.

Figures 12 and 13 show the statistical evaluations made in this study in terms of K_h for the two locks to be in agreement with the best estimate reported in Diviney (1990) after Schmertmann (1986). Recall that their

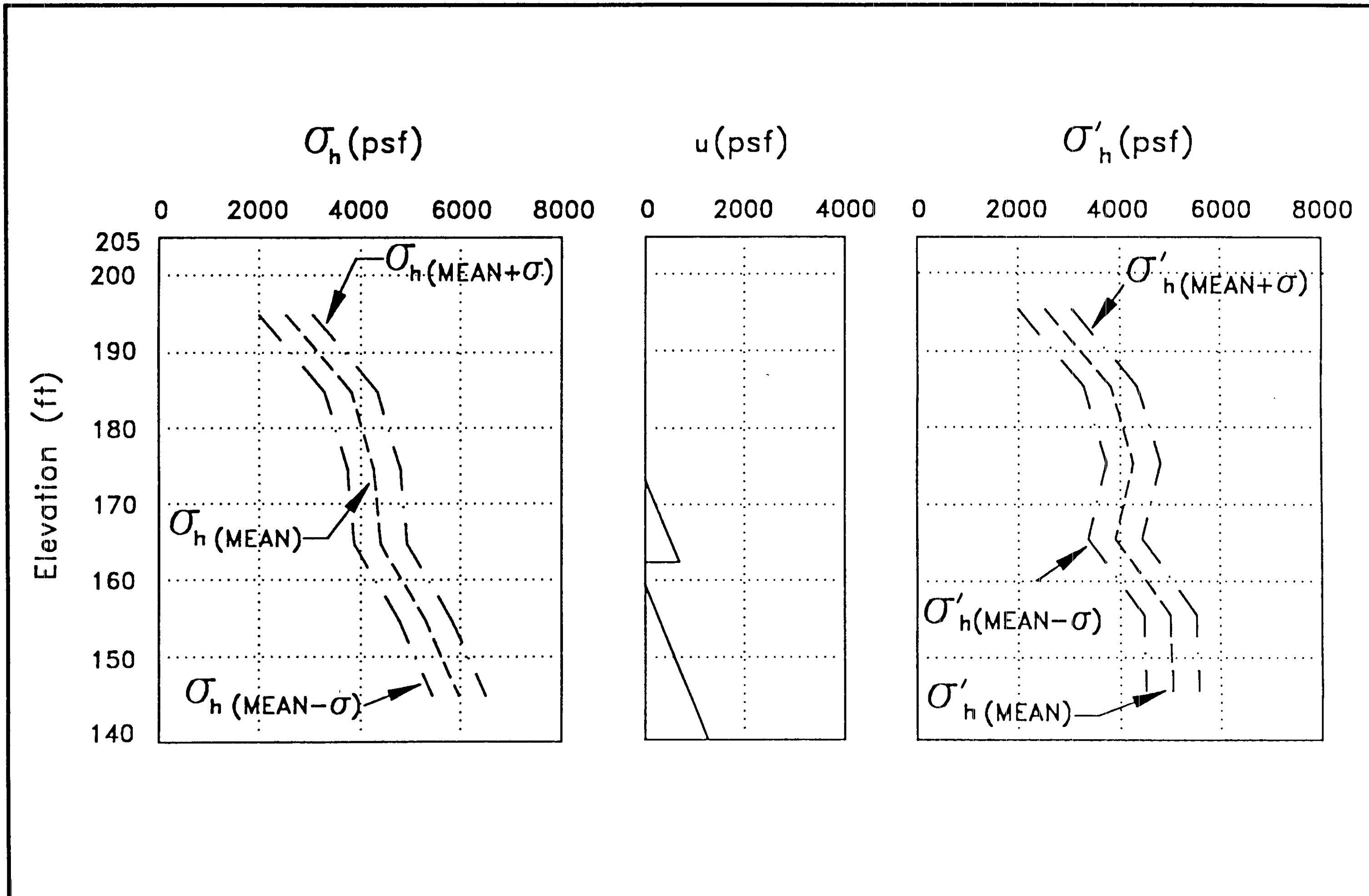


Figure 8. Distribution of σ_h, u, and σ'_h with elevation from statistical evaluation of pressure meter test data at Snell Lock

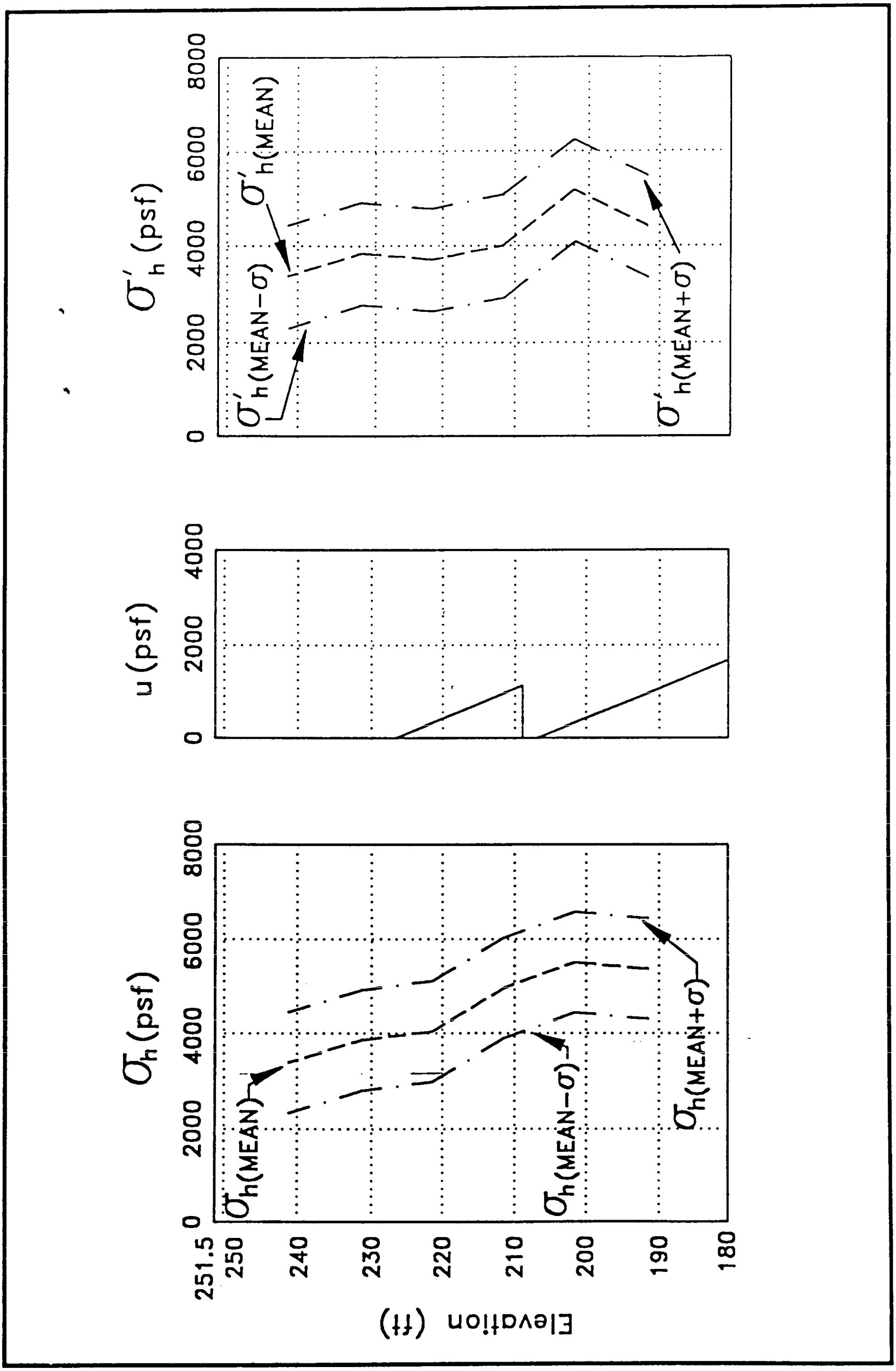


Figure 9. Distribution of σ_h, u, and σ'_h with elevation from statistical evaluation of pressure meter test data at Eisenhower Lock

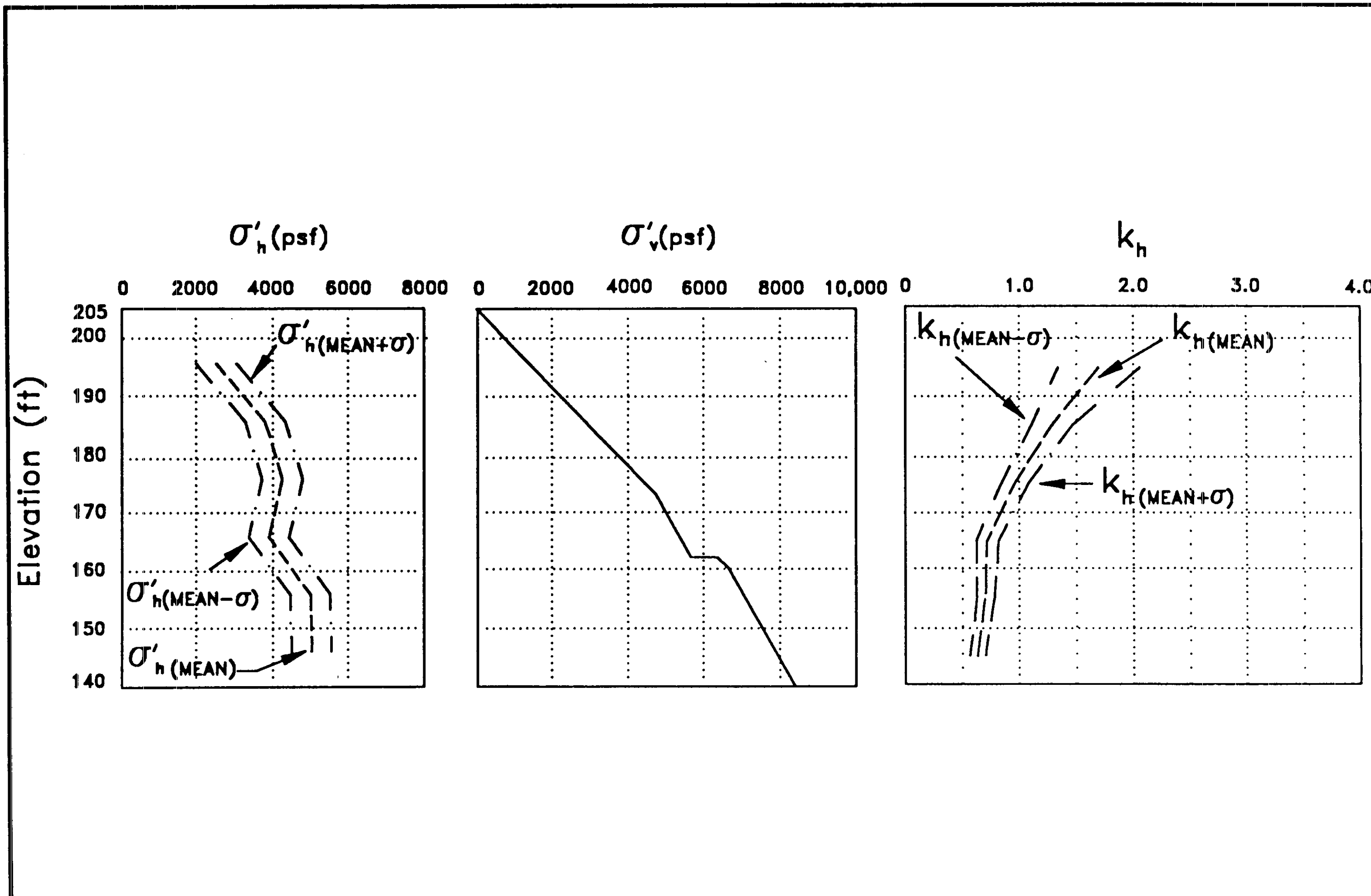


Figure 10. Distribution of σ'_h , σ'_v , and K_h with elevation from statistical evaluation of pressure meter test data at Snell Lock

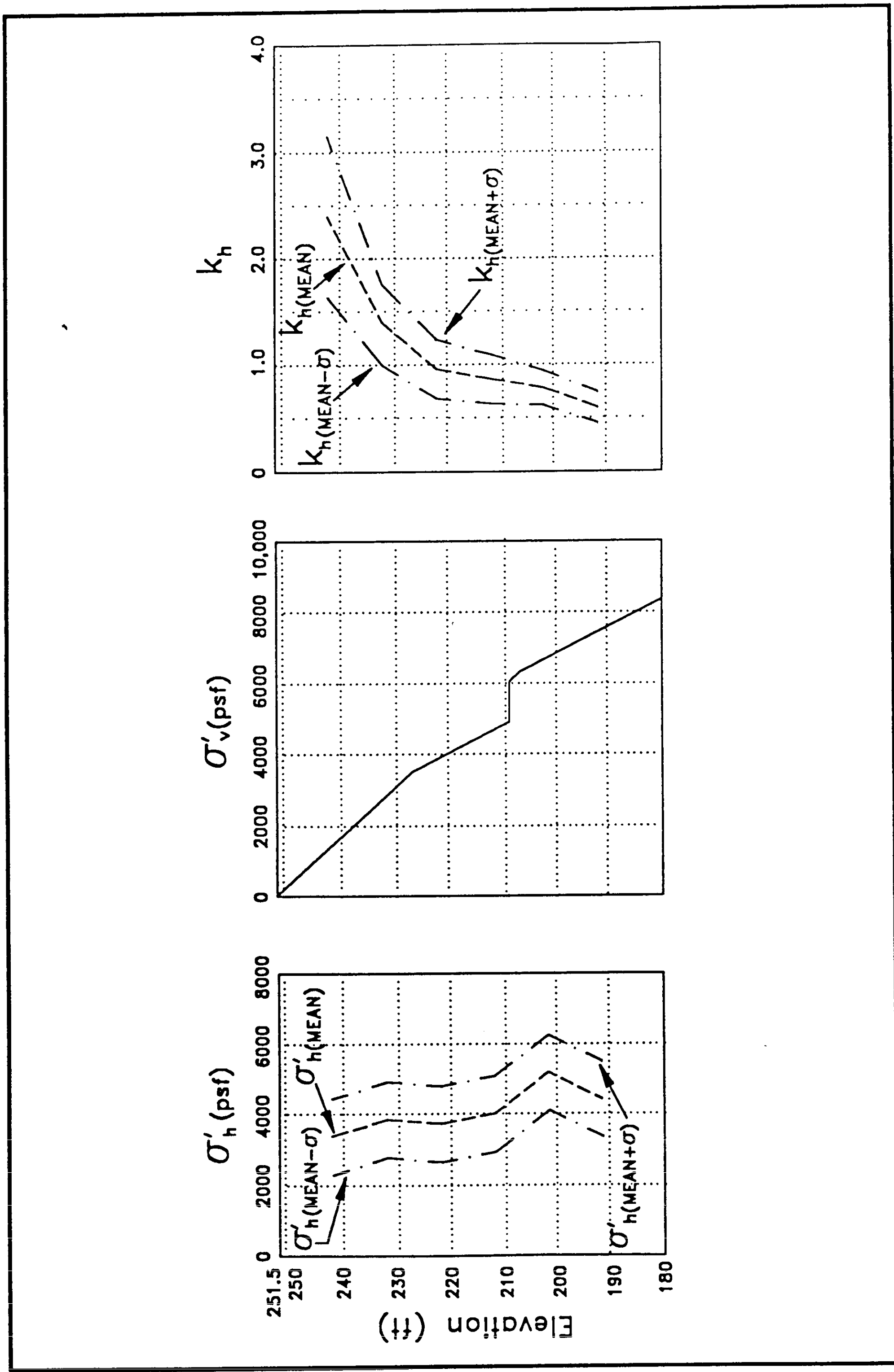
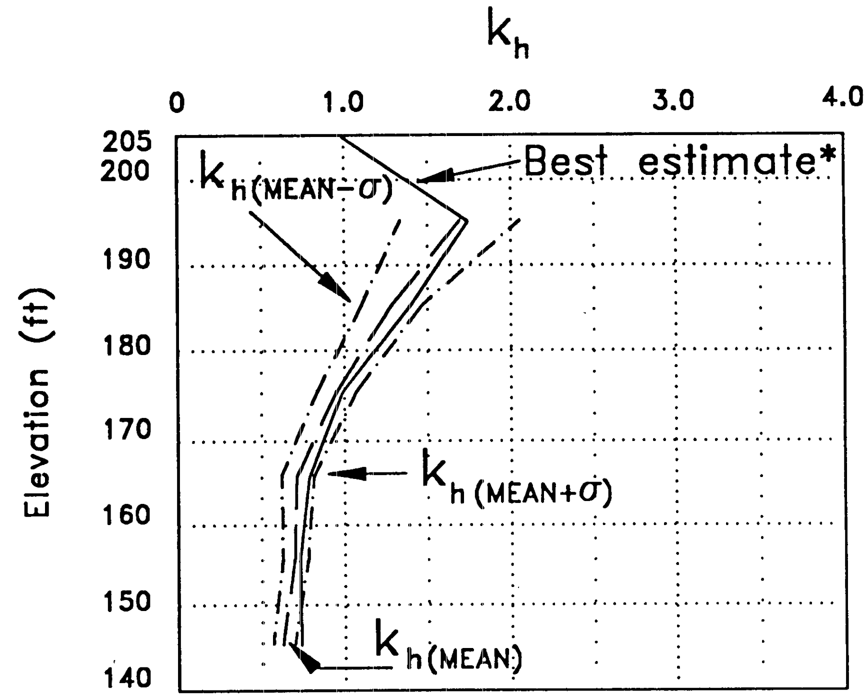
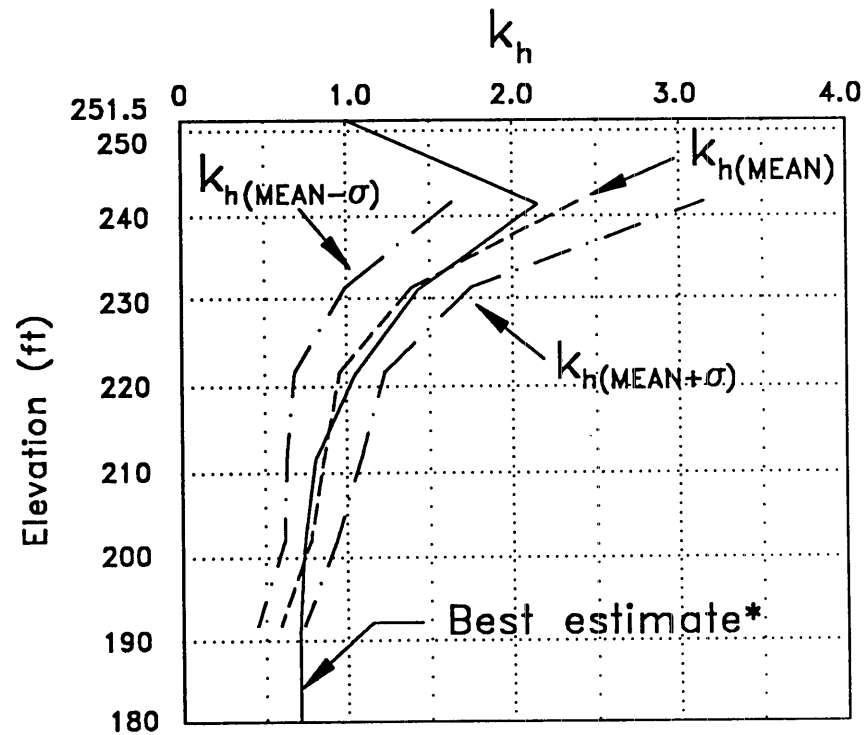


Figure 11. Distribution of σ'_h, σ'_v, and K_h with elevation from statistical evaluation of pressure meter test data at Eisenhower Lock



* Best estimate computed from Snell and Eisenhower Locks combined data as reported in Diviney (1990), after Schmertmann (1986).

Figure 12. Distribution of K_h with elevation from statistical evaluation of pressure meter test data at Snell Lock



* Best estimate computed from Snell and Eisenhower Locks combined data as reported in Diviney (1990), after Schmertmann (1986).

Figure 13. Distribution of K_h with elevation from statistical evaluation of pressure meter test data at Eisenhower Lock

best estimate was computed using combined data from Snell and Eisenhower Locks.

In addition, Filz and Duncan (Figure 7.13, 1992, or Figure 6, 1996), using the Duncan and Seed (1986) compaction-induced earth pressure theory, applied their analytical model for simulating compaction-induced earth pressures to a model backfill for Snell Lock. Their results showed agreement with the PMT and HF test results when the heaviest compactor was used in the model.

Conclusions

The primary means of loading that caused the cracking of the four culvert walls are the lateral earth loads. These structures were designed in the early 1940's as massive concrete structures using equivalent fluid pressures to account for the load imposed by the backfill. One index used by engineers to characterize the magnitude of earth pressures is the horizontal earth pressure coefficient K_h. The values for K_h corresponding to equivalent fluid pressure used in the design of the lock walls range from 0.21 to 0.24. The results of in-situ testing (PMT and HF tests) show K_h to range from 0.7 to above 2.0, depending on elevation within the backfill (Figures 12 and 13). The overcompaction of the backfill resulted in earth pressures greater than those anticipated during the design of the lock walls by factors ranging from 3 to 10, depending on the elevation in the backfill.

Rehabilitation was accomplished using 165 post-tensioned anchors installed in all four walls of Snell and Eisenhower Locks. Six 636-kip anchors were installed in each monolith. The average spacing of the anchors was 7.33 ft.

3 Miller's Ferry Lock and Dam

Introduction

Miller's Ferry Lock and Dam is located about 142.2 miles above the mouth of the Alabama River near Camden, Alabama. Construction of the lock and dam began in April 1963, and the lock was in operation in June 1968. The lock and dam was authorized for construction by Congress in Section 2 of the River and Harbor Act of 1945, Public Law 14. In addition to the lock and dam, the project included the design and construction of a hydropower powerhouse with three 25,330-kW generator units, each with a turbine intake.

The lock and dam is a massive concrete gravity structure that is founded on a chalky limestone deposit (Prairie Bluff Formation) about 11 to 12 ft thick (USACE 1963a). The lock consists of a 600- by 84- ft chamber with a maximum lift of 45 ft. The dam has a gated spillway with 17 tainter gates and an overall length of 1,012 ft. The design upper pool for the structure is el 80.0. The design lower pool is at el 35.0. Figure 14 shows the location, layout, and typical cross sections for Miller's Ferry Lock and Dam. Figure 15 shows the cross section for a chamber monolith on the land wall.

Design Concrete Mix

The massive concrete used for Miller's Ferry Lock and Dam was specified according to three grades of concrete based on minimum 28 day compressive strength. The nominal cement used in the mix was 3.75 bags per cubic yard (USACE 1962c). Specific provisions were made in the contract to allow the contractor to use fly ash as an option for a cement replacement material. However, if fly ash was used in the mix, the minimum compressive strength of the concrete was taken at 90 days for field control purposes.

The three grades of mass concrete mix were specified at Miller's Ferry Lock were as follows (USACE 1962c, 1963b):

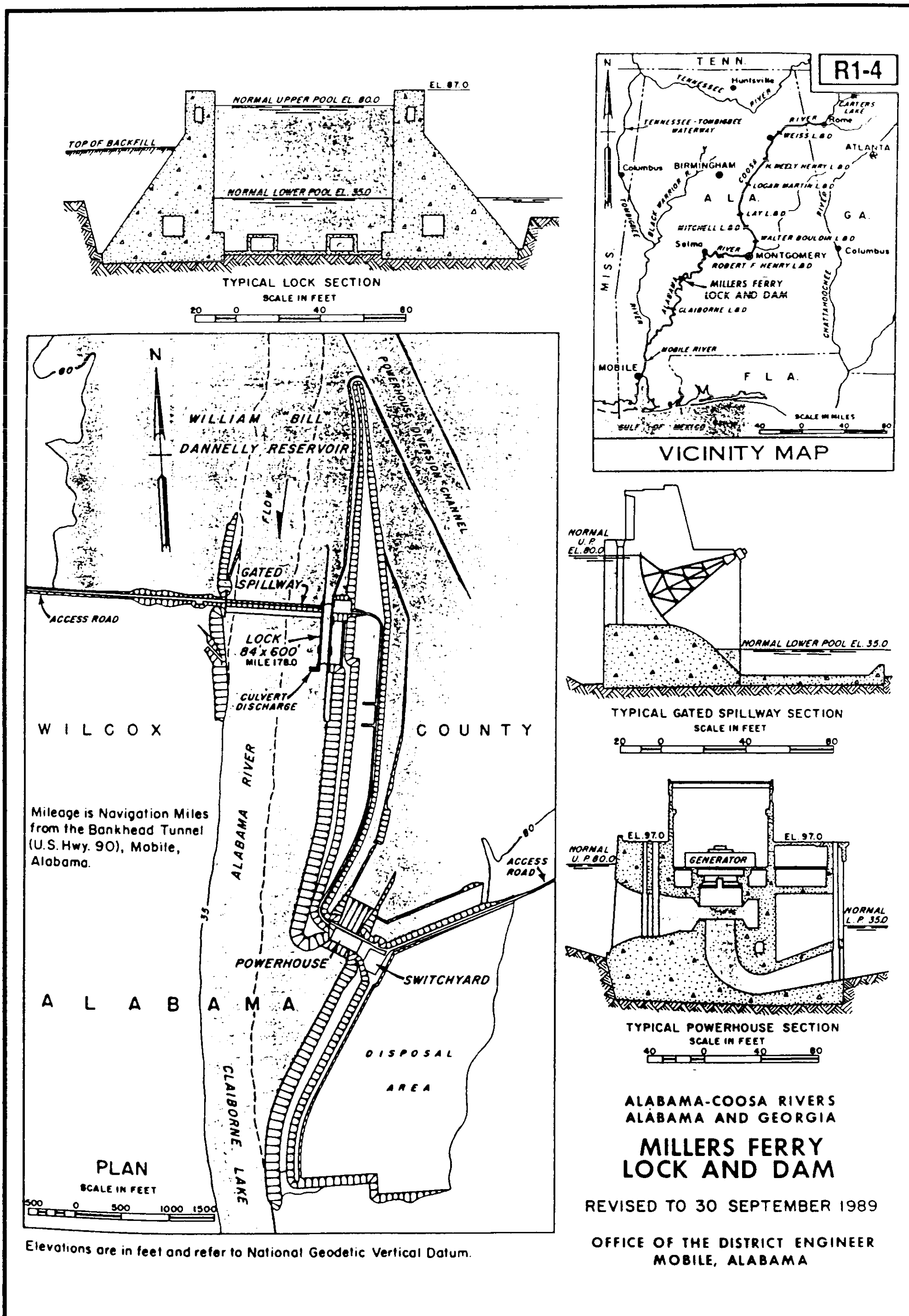


Figure 14. Location, layout, and typical cross sections for Miller's Ferry Lock and Dam (after USACE 1993)

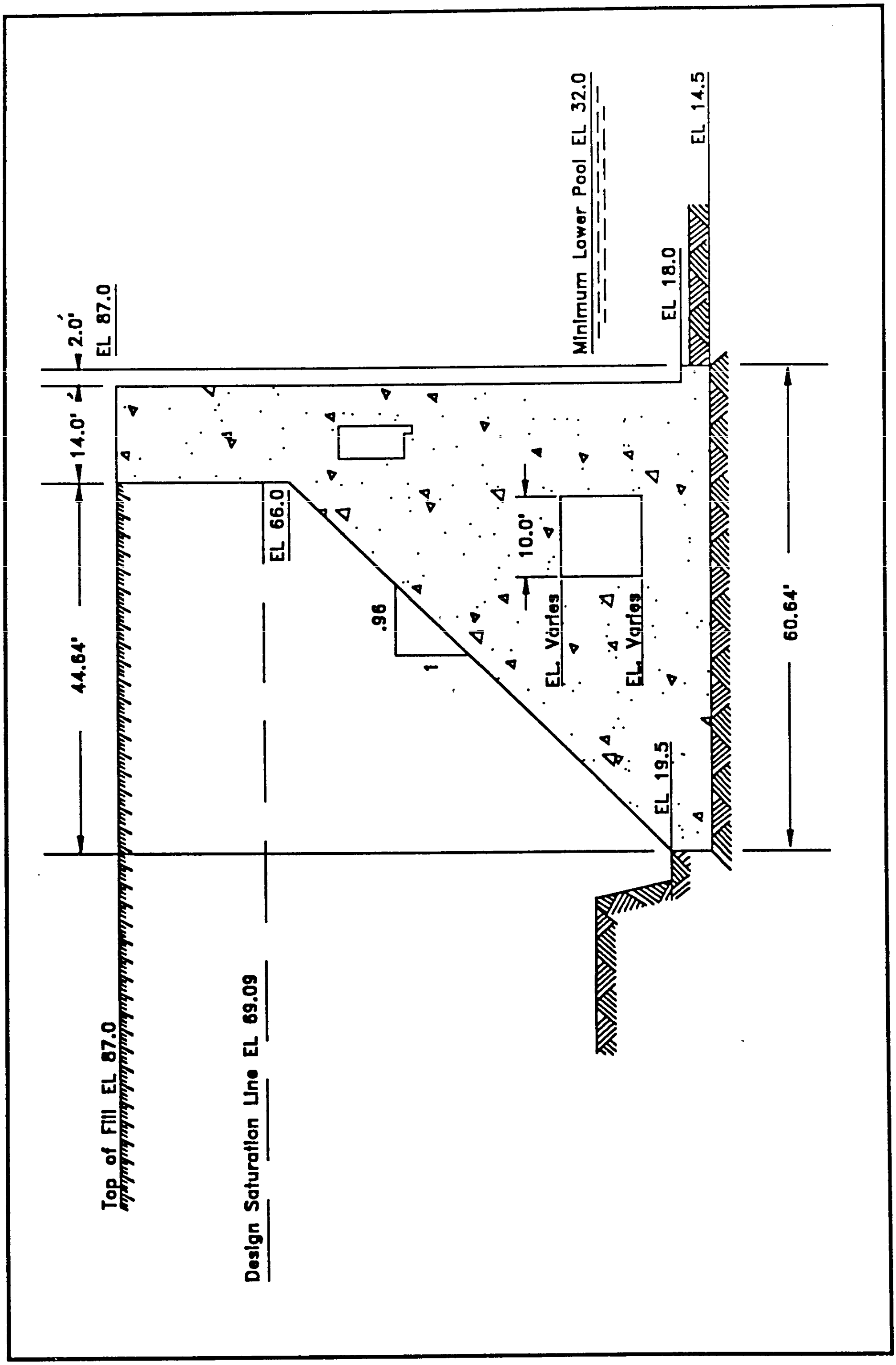


Figure 15. Typical cross section of chamber monolith on land wall at Miller's Ferry Lock

a <u>2,000 psi</u> - Used for the mass concrete of large gravity sections of the lock. The maximum aggregate size was specified as 6 in.

b. <u>3,000 psi</u> - Used for the exterior shell of large gravity sections as a shell for deterioration purposes. The exterior shell had a minimum depth of 5 ft from exterior surfaces. The maximum aggregate size would be 6 in. within one lift of the top of lock wall, where the maximum aggregate size could be 3 in.

c. <u>5,000 psi</u> - Used for the lining of the water passages for the emptying and filling system. This concrete was provided for a short distance upstream and downstream of the culvert valves where velocities were considered high. The concrete was placed during regular lift placement as an interior lining 5 ft from the surfaces with a maximum aggregate size of 1-1/2 in. This mix was also used for reinforced concrete areas in the floor-culvert systems, culvert discharge structure, and spillway piers and at the post-tensioned gate anchorages.

The sources of the course and fine aggregates were seven local quarries within a 125-mile radius of the construction site. The course aggregates were primarily limestone, dolomite, and granitic gneiss. Their sizes ranged from 4 to a maximum of 6 in. Because these course aggregates tested as innocuous, they did not require the use of a low-alkali cement (USACE 1962a).

The fine aggregates were taken from riverbed deposits upstream in the Alabama River and were less than 2 in. in size. All the sources of fine aggregates required the use of a low-alkali cement since tests indicated the presence of deleterious material within the fine aggregate (USACE 1962a). The water used in the mass concrete mix was taken directly from the river because local sources of artesian water were too high in temperature.

Design Earth Pressures

Two different materials were used to backfill behind the landside lock wall at Miller's Ferry Lock. The first material was a silty sand that came from an area near the excavation for the lock. This material was stockpiled off site and allowed to dry. Since the amount of material removed at the site was insufficient to fill completely behind the lock wall, the silty sand material was placed only within the "theoretical active pressure wedge" behind the wall (USACE 1963b). The lock design memorandum (USACE 1963b) reports the silty sand with an angle of internal friction between 14 and 36 degrees and a cohesion of 0.00 tsf. Details regarding the types of tests conducted were not available.

The second backfill material comprised lean and sandy clays. The lock design memorandum (USACE 1963b) states that the clays had an average internal friction angle of 17 degrees and a cohesion around 0.43 tsf. Details regarding the types of tests conducted were not available. No data are available for the plastic or liquid limits of these clays from the laboratory testing for use in examining creep effects in this backfill. Also, the backfill behind the lock wall was compacted by rollers after the completion of the chamber monoliths. The total amount of backfill compacted behind the landside lock wall was about 100,700 yd^3 of material.

Table 2 shows the design parameters for the silty sand. Table 3 shows the design equivalent fluid pressure based on the active pressure computed using Rankine's formula and the at-rest horizontal pressures based on a horizontal coefficient K_h of 0.5. The stability of all backfilled monoliths was checked using at-rest earth pressures (USACE 1963b).

Table 2 Design parameters for silty sand backfill at Miller's Ferry Lock

Specific Gravity	2.66
Angle of internal friction	30 deg
Cohesion	0.00 tsf
Percentage of voids	36
Dry weight	100 pcf
Moist weight	115 pcf
Saturated weight	125 pcf
Submerged (buoyant) weight	62.5 pcf

Table 3 Design equivalent fluid pressures based on active and at-rest pressures at Miller's Ferry Lock

Active Pressures	$lb/ft^2/ft$ of height
Dry	33
Moist	38
Saturated	83
At-Rest Pressures	
Dry	50
Moist	58
Saturated	94

Culvert Cracking at Miller's Ferry Lock

Miller's Ferry Lock had been in service for almost 25 years when a problem was detected in April 1990. During an inspection of the lock, Mobile District personnel discovered that water had been escaping behind the upper valve monolith 6L. This indicated that there might be a crack in one of the filling culvert monoliths. This was confirmed by further observation of upstream monoliths 5L and 4L, which were misaligned relative to the remainder of the lock chamber by almost 1/2 in. (USACE 1993).

The investigation to examine the location and extent of the crack was conducted in two phases. The first phase involved the drilling of exploratory holes through the esplanade into monoliths 5L and 4L. The exploratory drilling was used to map crack width and direction in each of the monolith sections. The second phase used an underwater video camera to confirm the crack location from inside the culvert. The video showed that there was a continuous crack through the top corner of the culvert to the back of the monolith. However, it appeared that the flows in the culvert were not very high and had not reached extreme levels. From both phases of the investigation, the Mobile District concluded that the crack in the culvert was at an approximate 45 degree angle to the outside faces of the monoliths (USACE 1993). Figure 16 shows the location of the crack within monoliths 5L and 4L.

As part of the exploratory drilling process, soil samples from the backfill were taken and tested in the laboratory. From this investigation, it was determined that the backfill contained a large zone of very fine impervious material (USACE 1993). This material was able to retain the saturation level behind the lock wall at a higher elevation than that used in the original design.

Piezometer readings in the backfill indicated that the saturation line (or phreatic surface) could range from el 50 to el 65, depending upon the season. However, in March 1990 a flood occurred on the Tombigbee River and the flood stage reached el 83. At this time, the saturation level in the backfill moved up to approximately el 82 due to the low permeability of the fine materials in the backfill. It appears that this flood event triggered or exposed the cracking problems at Miller's Ferry, which were discovered during the inspection in April 1990.

Stability and structural integrity analyses were performed on the lock structure by the Mobile District. These analyses were used to determine

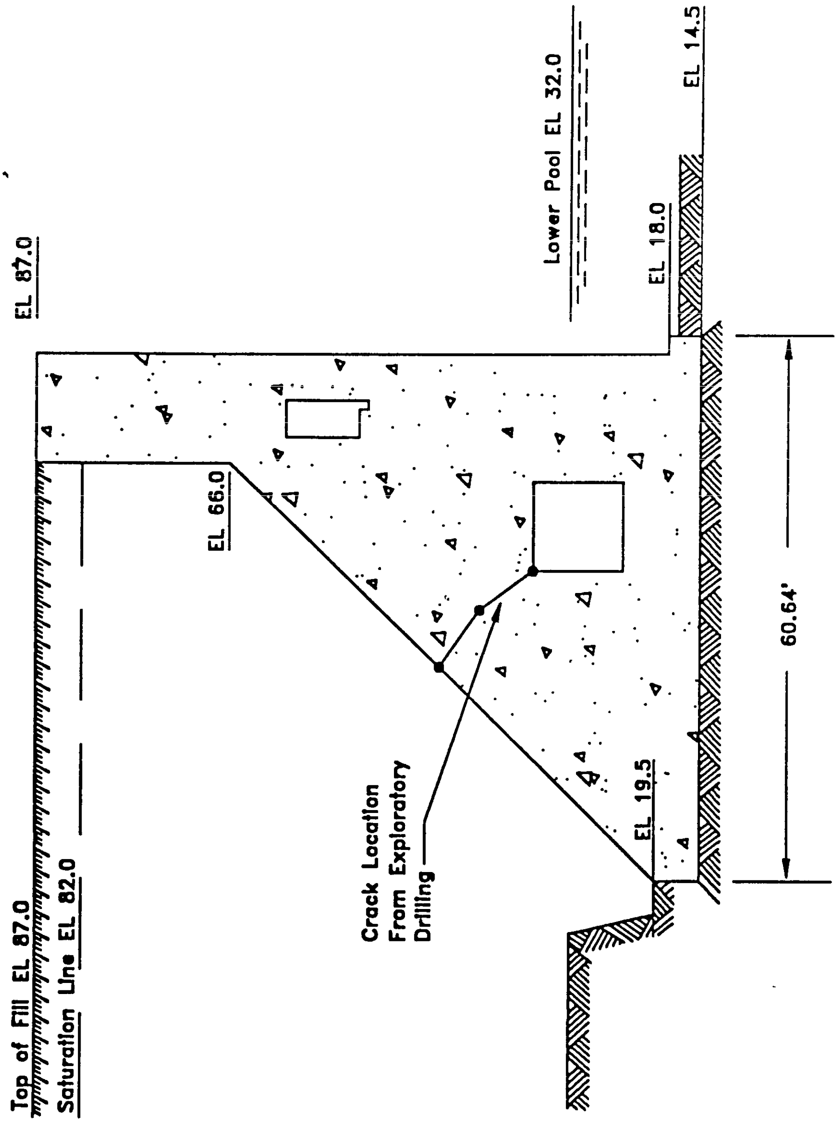


Figure 16. Location of crack in monoliths 4L and 5L at Miller's Ferry Lock

the forces which caused the cracking to develop as well as to develop a plan for the repair and stabilization of the structure. The analyses used the test results from the soil testing investigation. A saturated unit weight of 135 pcf was used for the soil. This was almost 10 pcf over the value used in the design of the lock. The 1990 stability analyses used a horizontal earth pressure coefficient K_h set equal to the at-rest pressure coefficient K_o of 0.7. This value of K_o is 40 percent greater than that used in the original design.

Remediation/Rehabilitation of Lock Walls

The rehabilitation of the cracked monoliths sections was accomplished by the installation of post-tensioned anchors. Nine tension anchors, each containing twelve 0.6-in.-diameter strands, were used to rehabilitate the structure. The design load for the anchors was 422 kips. Figure 17 shows the typical anchor installation. After placement of the anchors and lockoff in March 1991, there was serious concern that the anchors would hold only approximately 60 to 70 percent of their design prestress (USACE 1993). However, in a liftoff test in October 1991, all the nine anchors held about 100 percent or more of their design prestress.

After the anchors had been installed, the cracks in the culverts were sealed with a grout mixture. This grouting mixture in the crack helped to maintain a continuity of the contacts between the two separated surfaces. This was accomplished by divers using an insert drilled into the upper left corner of the culvert that pumped the grout into the crack. The insert holes were then later filled and sealed with an epoxy sealer. This grouting process was successful, but it did take more than one attempt to stop the flow. Even today the grout does cut off most of the culvert flow into the backfill even though some water may still be present at times behind the monoliths (USACE 1993).

The site was also remediated by the removal of material behind the affected monoliths to lower elevations. The backfill behind monolith 4L was lowered to el 68 and to el 64 behind monoliths 5L and 6L. The option of removing material and replacing it with a much freer draining material was discussed. However, it was ruled out due to the increased expense. The total cost of this remediation/rehabilitation was around $473,000.00, in 1991 dollars.

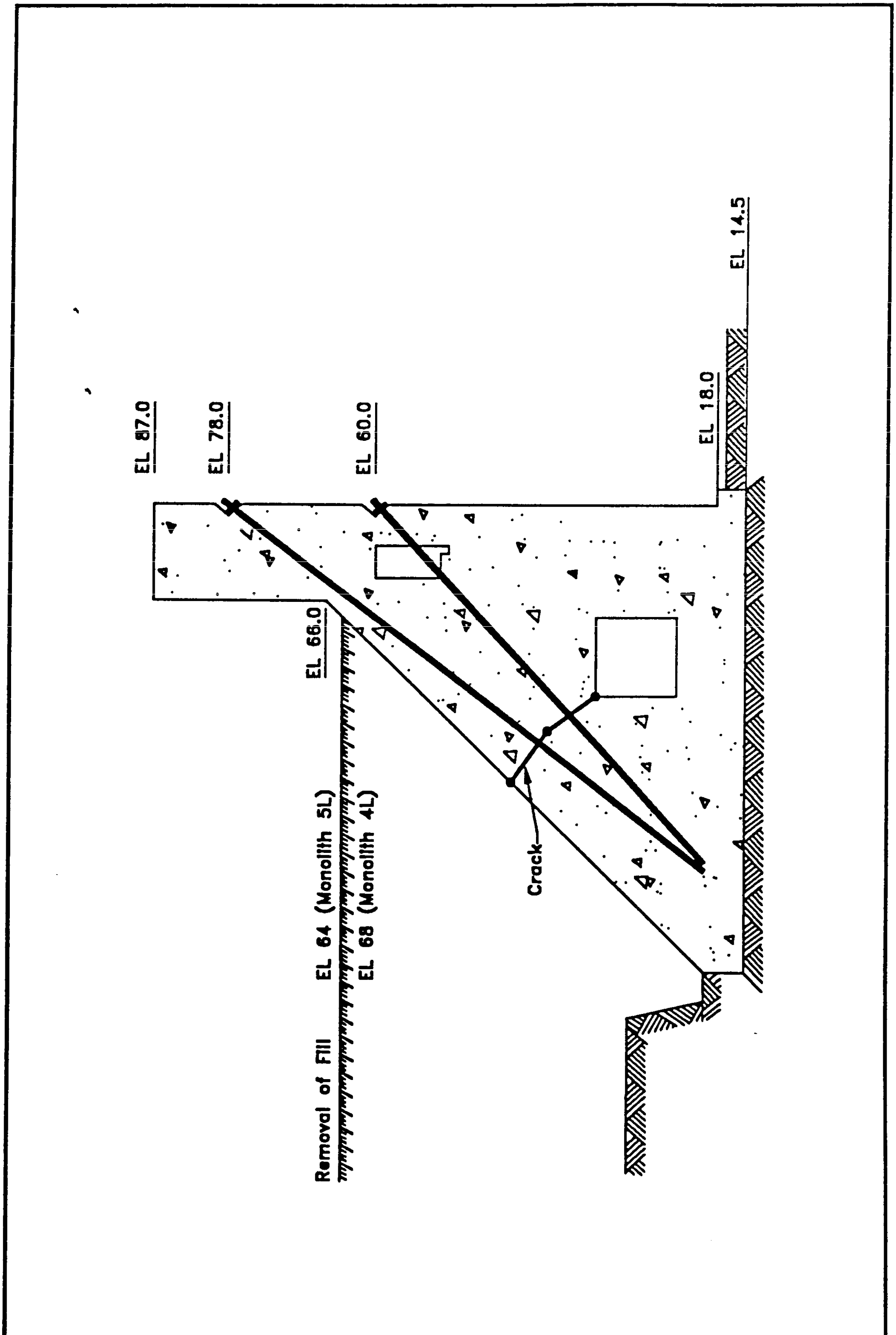


Figure 17. Typical installation of anchors and remediation of soil at Miller's Ferry Lock

Conclusions

The cracking in the chamber monoliths at Miller's Ferry Lock was primarily due to earth-induced pressures caused by the compaction of the lean and sandy clays in the backfill within the "theoretical active wedge". A water table was retained within the compacted clay backfill at an elevation which was much higher than that anticipated during the design of the lock walls. In addition, the compressive strength of concrete mix was not very high when compared with those of Snell and Eisenhower Locks (somewhere around 3000 psi). Therefore, the tensile capacity of the concrete is expected to have been very low since tensile strength of concrete is strongly correlated to compressive strength.

Stability analyses of Miller's Ferry indicate that an at-rest earth pressure coefficient of approximately 0.7 would not meet current Corps design criteria for stability and result in the cracking of the monoliths at Miller's Ferry. The rehabilitation/remediation of Miller's Ferry Lock was accomplished by installing post-tensioned anchors and by removing 19 ft of soil from behind the chamber monoliths, respectively.

4 Holt Lock and Dam

Introduction

Holt Lock and Dam is located about 155 miles above the mouth of the Black Warrior River near Tuscaloosa, Alabama. Holt Lock was built as a replacement for Locks 13, 14, 15, and 16 on the Black Warrior River. Construction of the lock and dam was initiated in 1962, and the lock was opened to navigation in 1966. The lock and dam was authorized for construction by Congress in Section 6 of the River and Harbor Act of 1909, and by Section 12, approved 25 July 1912. In addition to the lock and dam, a spillway and powerhouse were completed in 1969. The powerhouse is owned and operated by the Alabama Power Company and was originally constructed as an integral part of the dam.

The lock and dam are massive concrete gravity structures that are founded on thick shale and sandy shale beds of the Pottsville formation (USACE 1961). Thin beds of coal seams are also present at the site. The lock consists of a 600- by 110- ft chamber with a maximum lift of 63.6 ft. The dam has a gated spillway with 14 tainter gates and an overall length of 680 ft. The design upper pool for the structure is el 186.5, and the design lower pool is el 122.9. Figure 18 shows the location, layout, and cross sections for Holt Lock and Dam. Figure 19 shows the cross section for a chamber monolith on the land wall.

Design Concrete Mix

The massive concrete used for Holt Lock and Dam was specified by three grades of concrete based on minimum 28 day compressive strength. The mix was proportioned in a ratio of cement to coarse aggregate to fine aggregate in 0.750 bbls: 1.510 tons: 0.370 tons (USACE 1962b). The nominal cement used in the mix was 3.75 bags/yd^3 (USACE 1962a). In addition, direct provisions were made in the contract which allowed the contractor an option to use fly ash as a cement replacement material. However, if fly ash was used in the mix, then the minimum compressive strength was taken at 90 days for field control purposes.

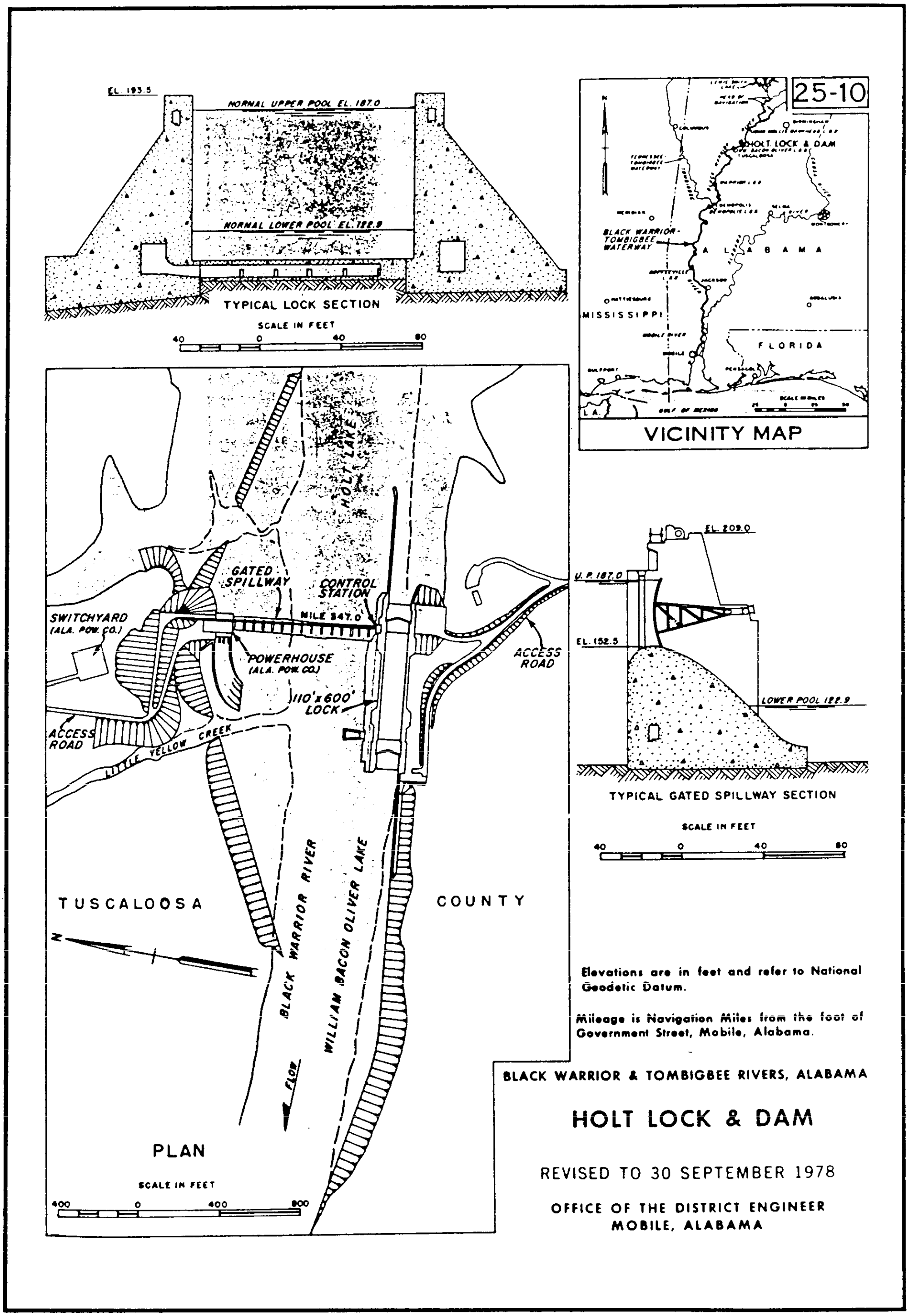


Figure 18. Location, layout, and typical cross sections for Holt Lock and Dam (after USACE 1985)

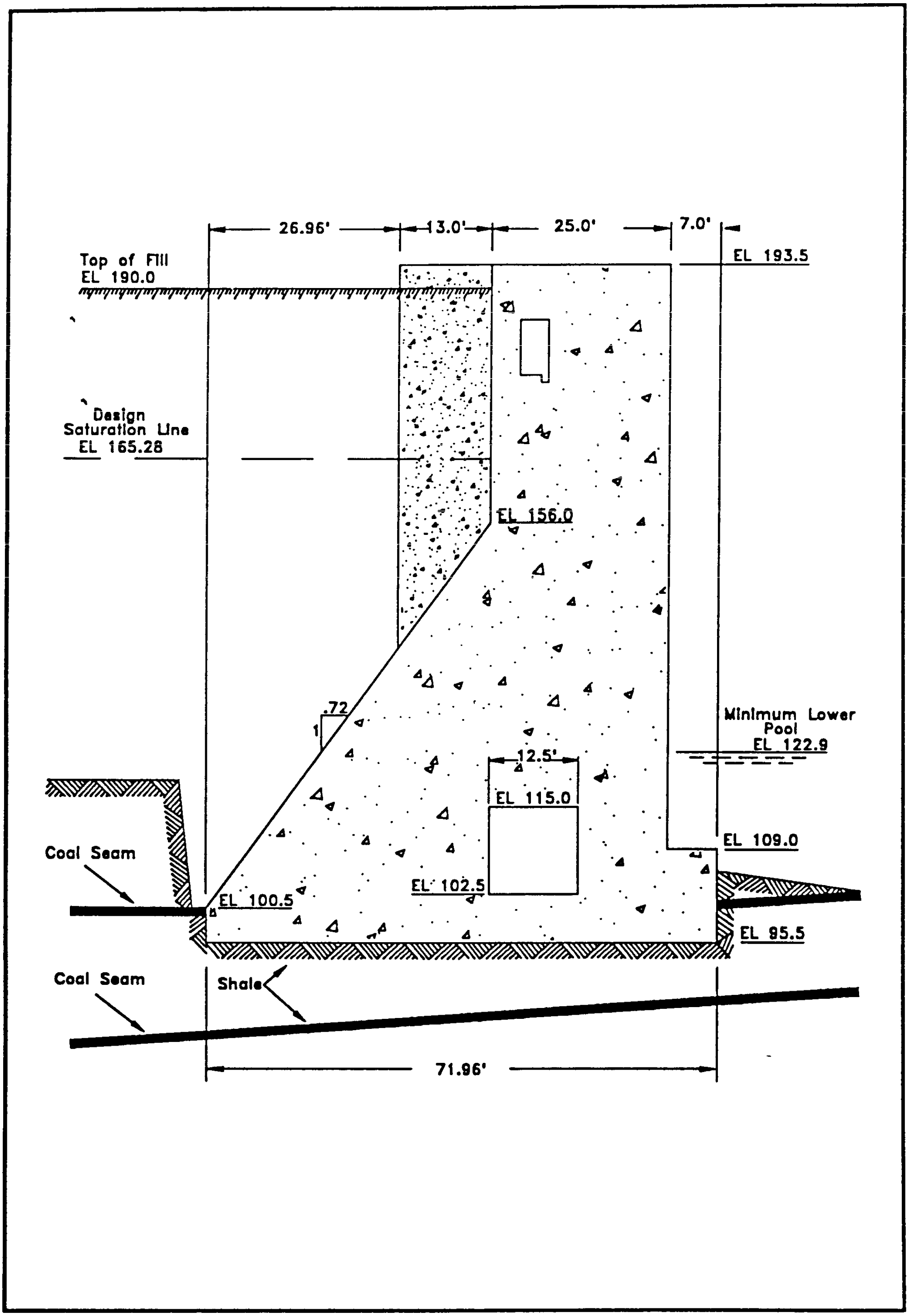


Figure 19. Typical cross section of chamber monolith on land wall at Holt Lock

The three grades of concrete mix used at Holt Lock were as follows (USACE 1962b):

a. 2,000 psi - Used for the mass concrete of large gravity sections of the lock. The maximum aggregate size would be 6 in.

b. 3,000 psi - Used for the exterior shell of large gravity sections as a shell for deterioration purposes. The exterior shell has a minimum depth of 5 ft from exterior surfaces. The maximum aggregate size would be 6 in. within one lift of the top of lock wall, where the maximum would be 3 in.

c. 5,000 psi - Used for the lining of the water passages for the emptying and filling system. This concrete was provided for a short distance upstream and downstream of the culvert valves where velocities were considered high. The concrete was placed during regular lift placement as an interior lining 5 ft from the surfaces with a maximum aggregate size of 1-1/2 in. This mix was also used for reinforced concrete areas in the floor-culvert systems, the culvert discharge structure, and spillway piers and at the post-tensioned gate anchorages.

The course and fine aggregates were taken from six local quarries within a 150 mile radius of the construction site. The course aggregates were primarily limestone, sandstone, and granitic gneiss. Their sizes ranged from 4 to a maximum of 6 in. Because these course aggregates tested as innocuous, they did not require the use of a low-alkali cement (USACE 1962a).

The fine aggregates were taken from riverbed deposits of natural sand and gravel on the Alabama and Tombigbee Rivers. The size of the fine aggregates was less than 2 in. All the fine aggregates required the use of a low-alkali cement since they contained deleterious materials (USACE 1962a).

The water used in the mix was taken directly from the river just above the construction site. Tests performed on the river water showed that it had a pH of 7.0, with a chloride content of 6 PPM and a sulfate content of 53 PPM. Temperature control analyses indicated that the 60 F temperature gradient would be exceed during the months of June, July, August, and September. This necessitated the use of temperature control procedures for lifts poured during these months (USACE 1962a). Additionally, the annual mean minimum temperatures did not reach the freezing point of 32 F. This permitted the pouring of concrete throughout the year.

Design Earth Pressures

The material used for backfill behind the lock wall was a silty sand material that came from a location just upstream of the lock excavation site. The material was removed and stockpiled offsite where it was allowed to completely dry. This material removed near the site was insufficient to completely fill behind the lock wall. This silty sand material was placed only in the "theoretical active pressure wedge" behind the wall (USACE 1962b). The remainder of the backfill comprised lean and sandy clays. All the backfill behind the lock wall was compacted by rollers after the construction of the chamber monoliths was complete. The total amount of backfill compacted behind the left lock wall was around 186,800 yd^3 of material.

The lock design memorandum reports (USACE 1962b) that the lean and sandy clays had an internal friction angle of 21 degrees and a cohesion of 0.27 tsf. The silty sand had an angle of internal friction of 25 degrees and a cohesion of 0.11 tsf. Details regarding the types of tests conducted were not available. For examining creep effects, the lean and sandy clays had a liquid limit between 30 to 45 and a plastic limit between 20 to 30.

Table 4 shows the design parameters for the silty sand backfill. Table 5 shows the design equivalent fluid pressures based on the active pressure computed using Rankine's formula and the at-rest horizontal pressures based on a horizontal coefficient of 0.5. The stability of all backfilled monoliths was checked using at-rest earth pressures (USACE 1962b).

Table 4 Design parameters for backfill at Holt Lock

Specific Gravity	2.66
Angle of internal friction	30 deg
Cohesion	0.00 tsf
Percentage of voids	36
Dry weight	106 pcf
Moist weight	117 pcf
Saturated weight	129 pcf
Submerged (buoyant) weight	66.0 pcf

Table 5 Design equivalent fluid pressures based on the active and at-rest earth pressures at Holt Lock	
Active Pressures	lb/ft^2/ft of height
Dry	35
Moist	39
Saturated	85
At-Rest Pressures	
Dry	53
Moist	58
Saturated	95

Culvert Cracking at Holt Lock

Significant movement of Holt Lock monolith 7L was noticed in mid-1980 (USACE 1981). Alignment surveys indicated movements of the monolith in the range of 1 to 2 in. An investigation was undertaken by the Mobile District to investigate the probable causes. Relief wells were drilled and piezometers installed to investigate the saturation level in the backfill behind the lock wall. The piezometer heads indicated that the saturation level (or phreatic surface) was around el 178 (USACE 1981). This was almost 15 ft higher than that used in the original design.

Recommendations were first made to install shear keys at the contact monolith joints as well as to install relief wells to relieve the backfill pressure. In addition, waterstops were placed at the monolith joints to stop the flow of water. During the drilling installation of the waterstops, a crack was discovered in monolith 6L. Subsequent drilling revealed additional cracks in monolith 7L.

The crack in monolith 6L extended horizontally across the section at el 148.5. The crack was completely open, and staining was present at the downstream end. The upstream end was closed and had not yet stained. The crack in monolith 7L was located between the top of culvert and the back face of the monolith near el 120. Continued drilling across the monolith indicated that the crack was at a 45 degree angle to the backface (USACE 1991). The crack opening was about 3/8 in. and allowed a flow that ranged from 100-500 gpm. Figure 20 shows the crack locations for monolith 7L.

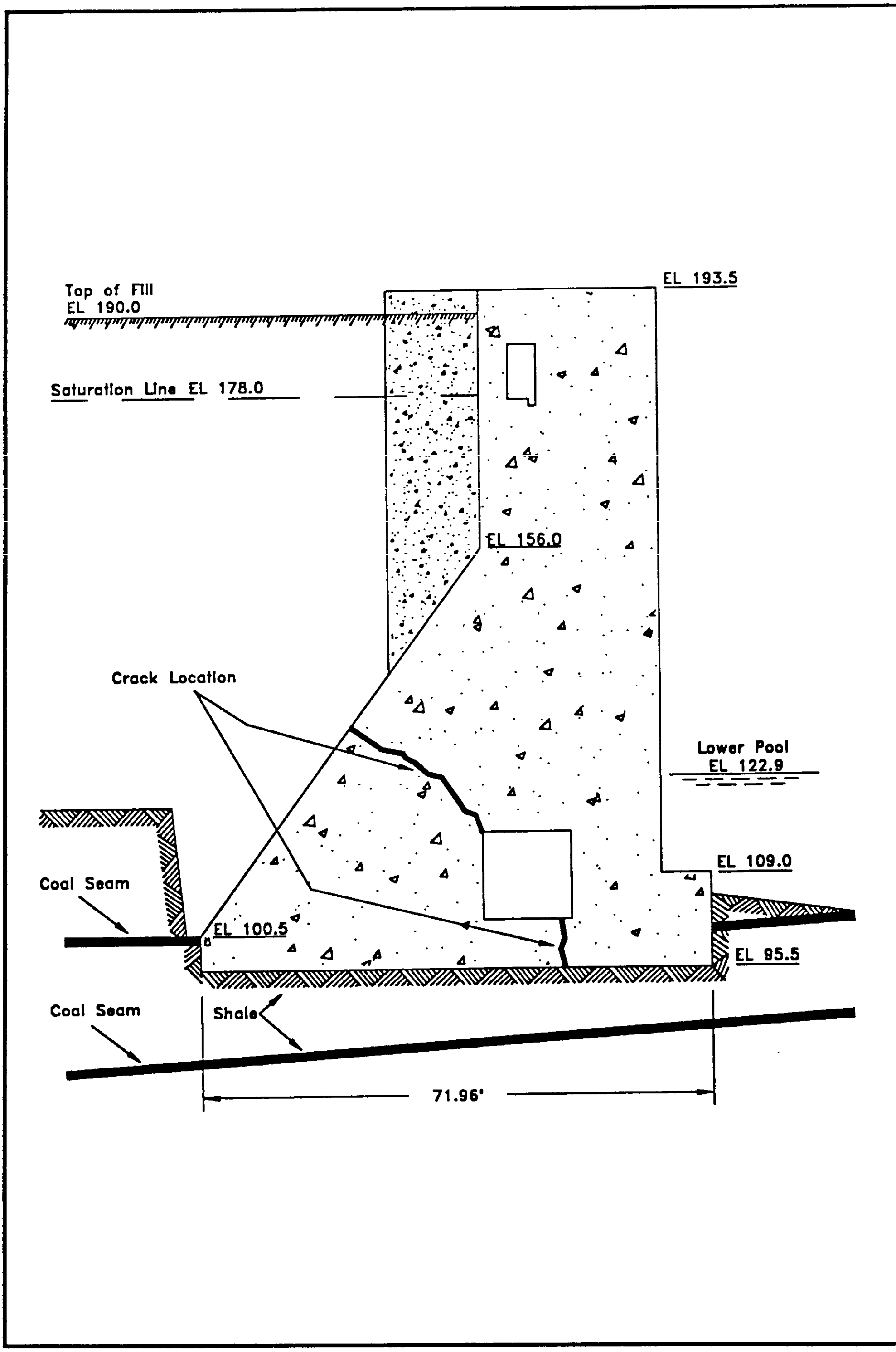


Figure 20. Locations of cracks in monolith 7L at Holt Lock

Recent inspection of the structure (USACE 1985) has revealed 4- to 5-in.-diameter boils at the surface of the backfill about 10 ft from the lock wall behind monolith 7L and 8L joints. The relief drains flow is less than 5 gpm and usually clear. Lock uplift cells 7,8, and 9 are present in monolith 6L, but appear to be clogged because they tend to give readings higher than the upper pool elevations. A stability analysis was conducted by the Mobile District on the chamber structures. The results indicated that the lateral earth pressure coefficients were in the range of 0.8 to 1.0 would result in the cracking of the monoliths (USACE 1981).

Remediation/Rehabilitation of Lock Walls

The rehabilitation of the cracked monoliths at Holt Lock was accomplished by installing post-tensioned anchors in August 1981. Six post-tensioned rock anchors were installed in monolith 6L. The anchors were prestressed with a 600-kip anchor force, and the bond length was 30 ft. These anchors were 0.6-in.-diameter, seven-wire, with 17 strands per anchor. The anchors were placed in the hole, grouted, and set for 10 days before stressing.

Ten anchors were installed in monolith 7L. They were designed for an anchor force of 667 kips. The anchors were 0.6-in.-diameter, seven-wire like monolith 6L, but had 19 strands instead of 17 strands per anchor. The bond length in monolith 7L was 20 ft. Additional remediation at Holt Lock included removing 25 ft of backfill from behind the lock wall and the grouting of the cracks in monoliths 6L and 7L. Figure 21 shows the typical anchor installation in monolith 7L.

Conclusions

Like Miller's Ferry Lock, the cracking in the chamber monoliths at Holt Lock was due to earth-induced pressures caused by the overcompaction of a dry lean clay backfill. A water table was retained within the compacted backfill at a higher elevation than anticipated during design. The concrete mix is likely to have had a compressive strength near 3000 psi, which yielded a very low tensile strength.

Stability analyses indicate that an at-rest earth pressure coefficient around 0.8 to 1.0 would not meet current Corps design criteria for stability and result in cracking of the monoliths at Holt Lock. The rehabilitation and remediation of Holt Lock was accomplished through the use of post-tensioned anchors and the removal of about 25 ft of soil from behind the filling chamber monoliths.

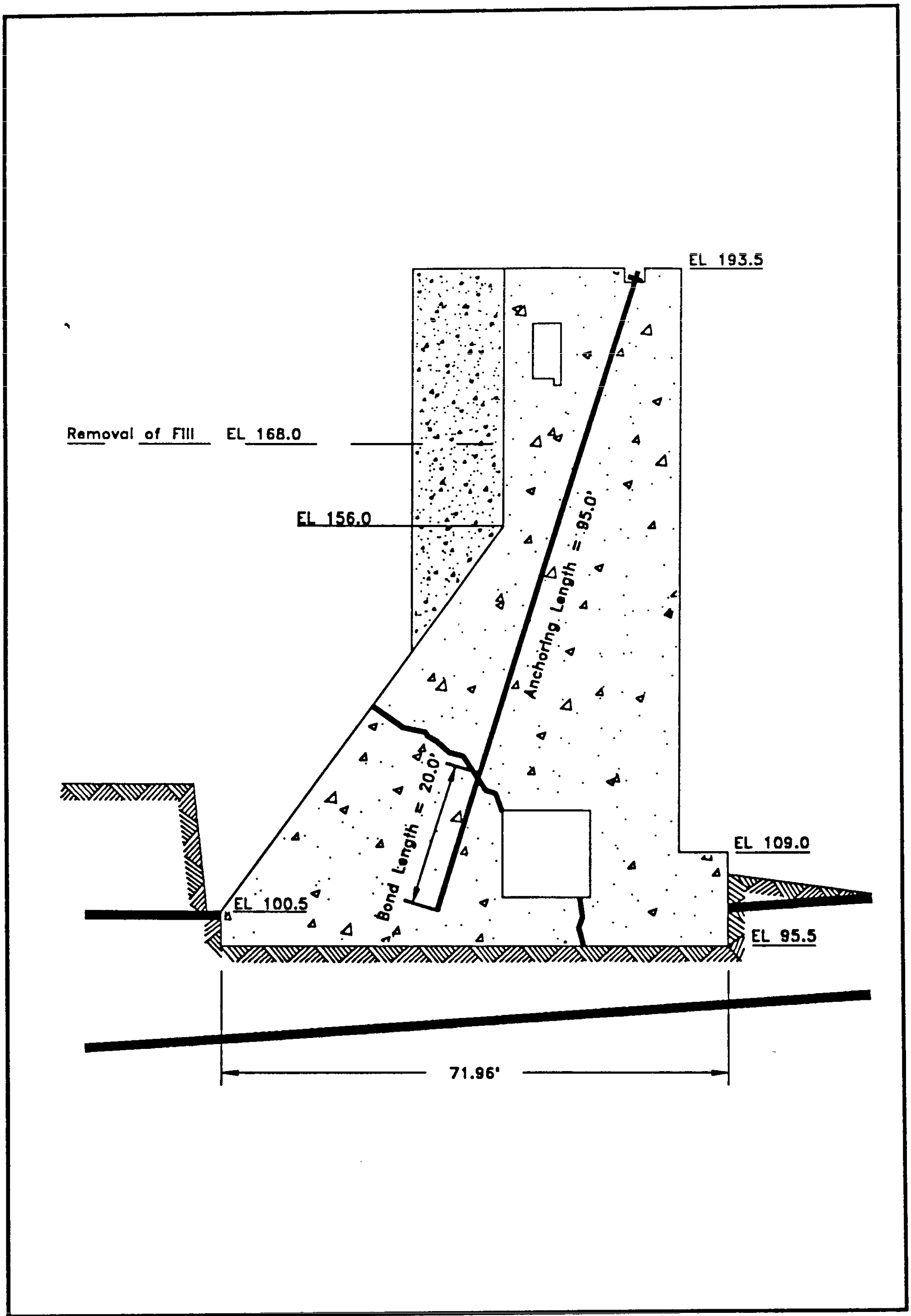


Figure 21. Remediation and typical installation of anchors at Holt Lock monolith 7L

5 Demopolis Lock

Introduction

Demopolis Lock and Dam is located on the Tombigbee River about 3.6 miles from the confluence of the Tombigbee and Black Warrior Rivers. The lock was authorized by the River and Harbor Act of 1945, Public Law 14 of the 79th Congress. The project was built to replace four locks and dams upstream of the proposed structure and permit the improvements to the Tombigbee River for access to commercial navigation traffic. The construction of the lock and dam was initiated in 1949, and the project was completed and in service in the latter part of 1955.

The lock and dam are massive concrete gravity structures that are founded on a chalk deposit over 500 ft thick (USACE 1948). The lock consists of a 600- by 110-ft chamber with a maximum lift of 40 ft. The dam is a concrete gravity structure with a fixed crest spillway with a total length of 1,485 ft. The design upper pool for the structure is at el 73.0 and the design lower pool is at el 33.0. Figure 22 shows the location, layout, and typical cross sections of the lock. Figure 23 shows the cross section for a chamber monolith on the land side.

Design Concrete Mix

The massive concrete used for Demopolis was of two types (USACE 1948): Type D concrete for "mild climate", which was used in the lock and dam structure from the foundation to about the elevation of the lower pool (el 33); and Type A concrete for "mild climate", which was used above that elevation to the top of the lock wall. These definitions for "Type" are in accordance with the Engineering Manual for Civil Works, Chapter XII, Recommended Practice for Concrete and Reinforced Concrete (USACE 1932). Depending upon the "Type" of concrete, the 28 day minimum compressive strengths ranged from 2,000 to 3,000 psi.

The large aggregates of the mix were taken from quarries between Tuscaloosa and Old Lock No. 17. A blue sandstone was considered the best source of manufactured coarse aggregate and ranged in size from 3

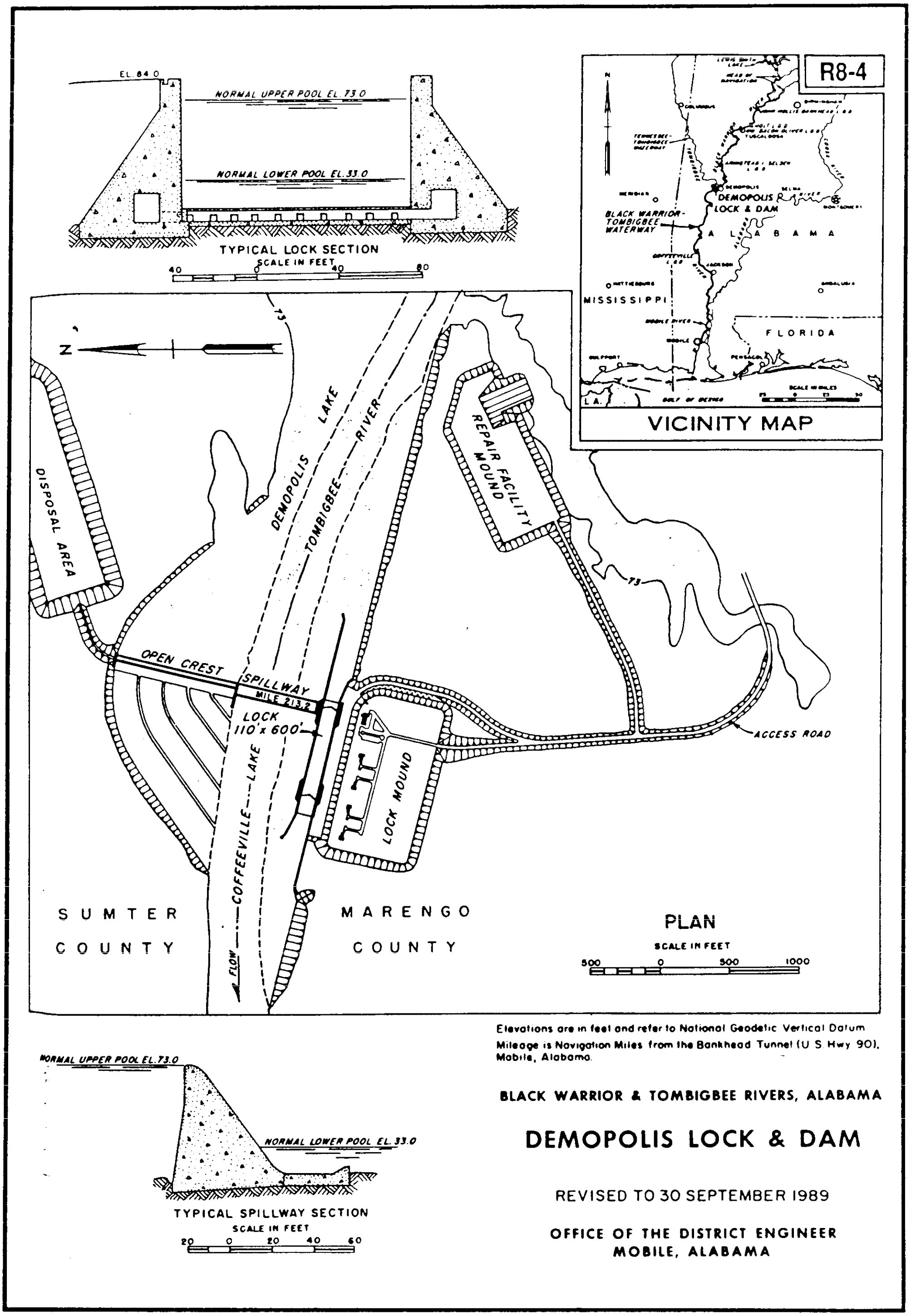


Figure 22. Location, layout, and typical cross sections for Demopolis Lock and Dam (after USACE 1992)

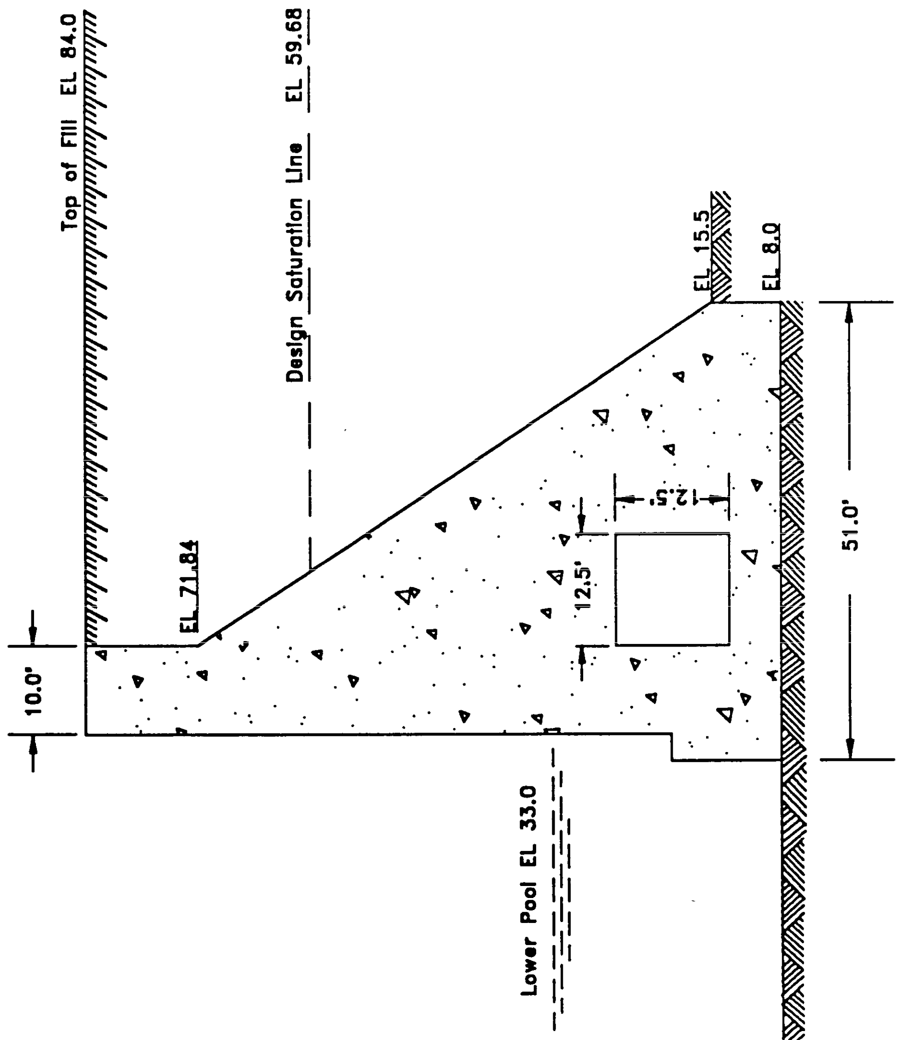


Figure 23. Typical cross section of chamber monolith on land wall at Demopolis Lock

to 4 in. The fine aggregates came from natural sources near the lock site. These fine aggregates had a maximum size of about 1-1/2 in. The gravel directly on site, which was only 1 in., was considered suitable only for exposed concrete with a high cement content.

Slag was not used because of its low specific gravity and because it is not a good material for use in mass concrete. The cement type or content for the lock was not specified since the design mix had not been specified at the time the Design Memorandum was written. This was because of concerns with the alkali-reactivity of the large and fine aggregates and the need for the use of a low-alkali cement. The assumed unit weight for the mass concrete was 150 pcf.

Design Earth Pressures

The lock wall was backfilled with materials from excavations in the river immediately upstream of the dam. These materials were dredged and allowed to partially dry before placement and compaction behind the lock walls. In addition, there was some concern that the entire structure would be submerged at times of high water, i.e., during flood conditions. As the waters would recede, a temporary water table would exist with the top of the lock wall in the backfill (USACE 1948). This raised water table in conjunction with the drawdown of the pool some 40 ft could cause stability problems. This condition is reflected in the design calculations for the structure because this has the lowest factor of safety against overturning of 1.57 (USACE 1948).

The backfill material at Demopolis Lock was assumed to be a silty sand material. The unit weights of the backfill were 110, 120, and 130 pcf for dry, moist, and saturated unit weights, respectively. The lock design memorandum (USACE 1948) reports that the angle of internal friction was 33.5 degrees and the material had 32 percent voids. Details regarding the types of tests conducted were not available. Table 6 shows the design equivalent fluid pressures based on active earth pressures. The amount of compacted fill at Demopolis for the embankment and esplanade was 1,176,000 ft^3.

Table 6 Design equivalent fluid pressures based on the active pressures at Demopolis Lock

Active Pressures	lb/ft^2/ ft of height
Dry	32
Moist	40
Saturated	82

Remediation of Structure

Demopolis Lock had been in service for approximately 35 years before any serious problems were encountered. Piezometer readings from 1980 to 1990 in the backfill showed that the saturation line (or phreatic surface) in the backfill was much higher than that used in the original design. In fact, the elevation of the saturation line behind monoliths 7L and 8L was generally at or near the upper pool elevation (USACE 1992).

Remediation of Demopolis Lock was complicated by a flood on the Tombigbee in March of 1989. The flood stage was around el 77. Based on past experiences with other locks in the District, i.e., Miller's Ferry and Holt Locks and knowledge that a high water table was already present at the site, the potential for a severe problem existed. This prompted the Mobile District to examine the stability of the lock in 1989 (USACE 1992). In the stability analysis, the value assigned to the at-rest earth pressure coefficient K_o for the backfill was 0.7. This was estimated from previous investigations of both Miller's Ferry and Holt Lock.

Based on the results of the stability evaluation, in 1990 and 1991 the Mobile District removed about 20 ft of backfill from behind the left lock wall, adjacent to monoliths 7L and 8L, down to el 63.5 (about esplanade level). Figure 24 shows the remediation of backfill for monoliths 7L and 8L at Demopolis Lock.

Conclusions

The remediation of Demopolis Lock was crucial in preventing any future cracking of the chamber monoliths. The material compacted behind the lock walls was a silty sand material. This backfill permitted the water table behind the lock walls to be retained at or near upper pool elevation. This was a concern given previous cracking problems at similar locks in the region. Stability analyses were performed by the Mobile District (1992) using an at-rest earth pressure coefficient K_o of 0.7. In order to maintain the current Corps of Engineers criteria for stability, the remediation of almost 20 ft of backfill from behind the chamber monoliths was required.

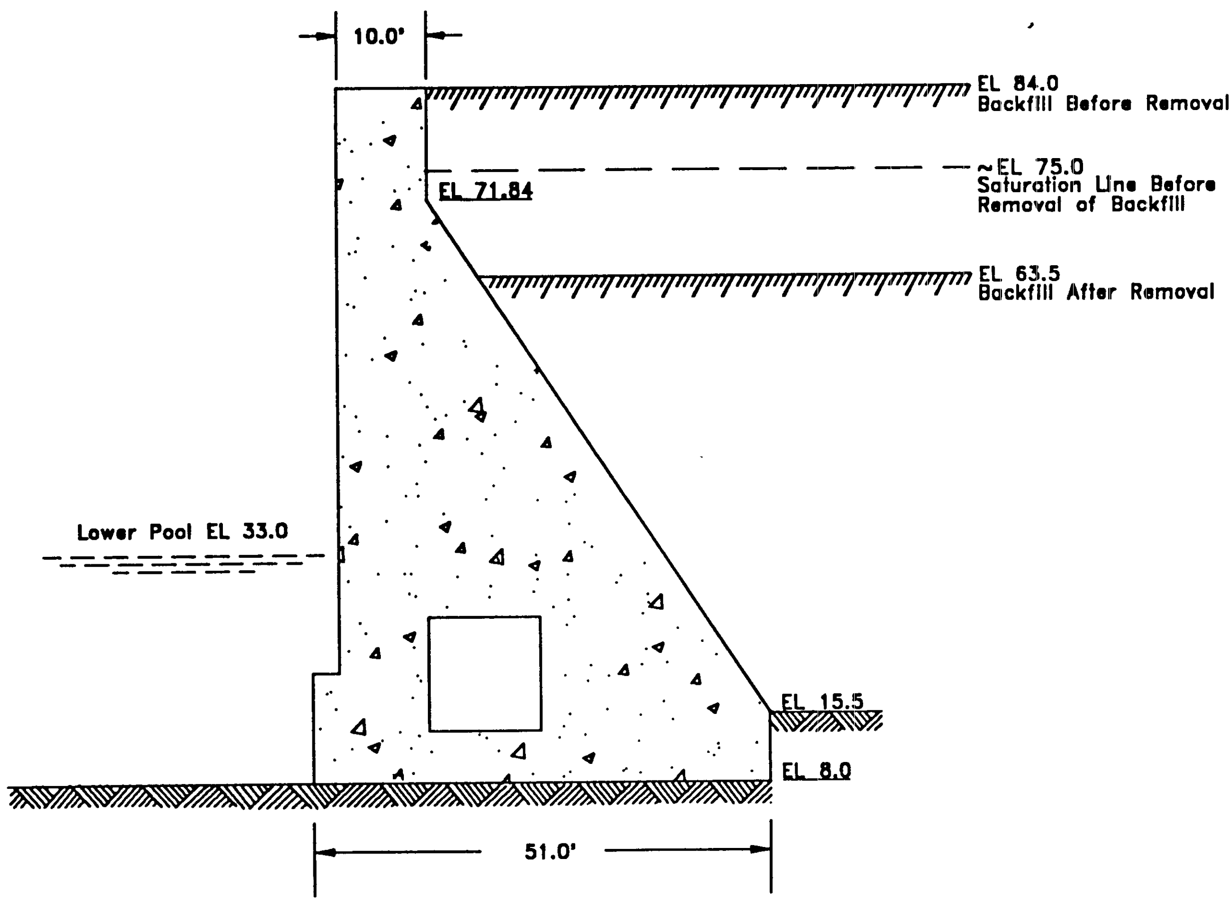


Figure 24. Remediation of backfill at monoliths 7L and 8L at Demopolis Lock

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13.ABSTRACT *(Maximum 200 words)*

This technical report presents four case histories of earth pressure-induced cracking of unreinforced mass concrete lock walls. The lock walls to be discussed are Snell and Eisenhower Locks on the St. Lawrence Seaway, Miller's Ferry Lock on the Alabama River in Alabama, Holt Lock on the Black Warrior River in Alabama, and Demopolis Lock on the Tombigbee River in Alabama. Demopolis Lock is not known to have cracked to date but was remediated because of similarities between it and Miller's Ferry and Holt Locks. All five locks retain soil, which accounts for a significant portion of the total horizontal loading along the backs of the lock walls. All pertinent information is described for each case history, including design loadings. Earth pressure loadings used in the design of the gravity retaining lock walls are included for all locks.

Factors contributing to cracking of the monoliths, other than earth pressures, are identified for each case history. The rehabilitation or remediation applied to each of the five locks is also discussed.

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