STATE OF CALIFORNIA

THE RESOURCES AGENCY

Department of Water Resources

INVESTIGATION OF FAILURE

BALDWIN HILLS RESERVOIR

APRIL 1964

HUGO FISHER	EDMUND G. BROWN	WILLIAM E. WARNE
Administrator	*Governor*	*Director*
The Resources Agency	State of California	Department of Water Resources

PREFACE

This report presents the findings of the State Engineering Board of Inquiry in its investigation of failure of the Baldwin Hills Reservoir in Los Angeles, California, on December 14, 1963. The disaster has emphasized forcefully the extensive damage and threat to life that even a relatively small amount of water can bring when suddenly released onto a highly developed area. It is important, therefore, that the facts disclosed in this comprehensive inquiry be made known to give a measure of guidance to those who have responsibility for design, construction, and operation of such reservoirs. Recognizing that water and its storage are essentials of life in our communities, it is imperative that the facilities which provide these benefits be made as safe as current engineering knowledge permits, reinforced by the lessons learned from the unfortunate experience at Baldwin Hills.

The groundwork for a full-scale state investigation was laid immediately following the disaster as a result of initial inspections of the broken reservoir and conferences among Director William E. Warne; Chief Engineer Alfred R. Golzé; Walter A. Brown, head of the Supervision of Dam Safety Office; and other members of the staff of the California Department of Water Resources. On December 19, 1963, Mr. Warne issued an order directing that an Engineering Board of Inquiry be organized and designated Mr. Robert B. Jansen as Chairman of the Board. The investigation was assigned the highest priority, with the objective of snbmittal of a definitive report to Resources Agency Administrator Hugo Fisher and Governor Brown in accordance with their requests.

The Engineering Board of Inquiry is indebted to the following members of the Department of Water Resources organization for their assistance in conducting the investigation: James E. Ley and Arthur B. Arnold and the data collection task force which they supervised; William J. Ellis, Ernest M. Weber, William M. Gibson, and William F. Harley for special studies; and Roy C. Wong, William H. Montgomery, and Helen J. Ribbeck for staff assistance. The Board is also grateful to the personnel of the Laboratories Branch for materials testing and to the many others who contributed to preparation of the report.

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STATE OF CALIFORNIA—RESOURCES AGENCY

EDMUND G. BROWN, *Governor*

DEPARTMENT OF WATER RESOURCES

P. O. BOX 388
SACRAMENTO

May 8, 1964

Honorable Edmund G. Brown
Governor of California
State Capitol
Sacramento, California

Dear Governor Brown:

Immediately after the failure of Baldwin Hills Reservoir in Los Angeles, I ordered an investigation to determine the cause and manner of failure. This investigation has now been completed, and I am pleased to submit herewith a full report on the matter.

The investigation included, in addition to a study of the facts related to the failure, a review of the design, construction, and operation of the reservoir. Consequently, it is hoped that this report will be useful not only as a study of the failure and its cause, but as a technical reference for those who might design, build, and operate similar facilities in the future.

Sincerely yours,

William E. Warne

William E. Warne
Director

LETTER REPORT FROM CONSULTING BOARD

April 14, 1964

Mr. R. B. Jansen, Chairman
Engineering Board of Inquiry
Baldwin Hills Reservoir Failure
P. O. Box 388
Sacramento 2, California

Baldwin Hills Reservoir Failure
Consulting Board Report

Dear Mr. Jansen:

This Consulting Board, J. Barry Cooke, Thomas M. Leps, and Roger Rhoades, was appointed by telephone on the day of the failure of the Baldwin Hills Dam and Reservoir, December 14, 1963. On the morning of December 15, the Board met at the site with representatives of the State to inspect the conditions and assist in outlining field investigations that would help in the determination of the manner and causes of the failure. Since December 15, the Consulting Board has met with the State Engineering Board of Inquiry at site inspections and at conferences directed toward furthering the acquisition of data, analysis of data, and the writing of the report.

The State of California report, "Investigation of Failure, Baldwin Hills Reservoir," dated April 1964, presents the work of the State Engineering Board of Inquiry. The objectives of the Board of Inquiry were to determine the way in which failure occurred and the physical causes of the failure. The Consulting Board is in agreement with the findings which the Board of Inquiry presents in the above report.

The reservoir was a basin having four sides, carved and constructed on the top of a hill. An impervious compacted clay blanket covered all excavated slopes and constructed embankments. The blanket was 10 feet thick on the reservoir floor, tapering up the slopes to a lesser thickness. Under the blanket was a four inch thick porous concrete drainage layer which was placed over an asphalt seal coating. The main dam constituted the North Side. Failure took place by piping and erosion in and along a steeply dipping fault that passed under the reservoir and through the East Abutment of the main dam.

This in summary was the manner of failure: A gradual deterioration of the foundation took place during the life of structure and culminated with sudden failure on December 14, 1963. The porous concrete drain was damaged by early small movements at the fault, and leakage water found its way into the fault. These earth movements were mainly caused by land subsidence, locally concentrated along the fault which was a weak plane. During the life of the reservoir, erosion took place in the fault under the undamaged blanket and partially damaged drain. The narrow width of the fault permitted the porous concrete drain to span openings that were developing under the drain. These occurrences were gradual and progressive. The perviousness of the fault permitted the water to disappear into the hill without emerging on the downstream abutment. Movement occurred at the fault on December 14, rupturing the impervious blanket and admitting full reservoir pressure to the fault and to the drainage system for the first time. The full reservoir pressure in the fault forced an outlet to the surface at a point low down on the east abutment of the main dam. Flow developed in the pervious and erodible fault zone and foundation rock. The flow and erosion increased rapidly, a cavernous opening piped through the abutment, the overlying foundation and embankment collapsed into this opening and the reservoir drained quickly and completely.

The physical cause of the failure was earth movement. The earth movement, due to subsidence, manifested itself by opening and by offsetting at a fault, a plane of weakness. Erodible material in and adjacent to the fault provided conditions that permitted rapid and complete failure.

The earth movements and the weak and erodible nature of the foundation were considered when the dam was designed. The geological investigations were thorough. The dam was built carefully. Instrumentation, measurements and operating inspections were thorough. Drainage had been provided in the abutment that failed. However, in spite of careful design, construction and constant surveillance, the reservoir failed, and lessons are to be learned from the failure.

It was agreed by the Consulting Board and the Board of Inquiry that the report, "Investigation of Failure, Baldwin Hills Reservoir," should be comprehensive, not only to serve its direct purpose, but to provide all attendant data that might improve the safety of future dams generally.

Respectfully submitted,

Barry Cooke

J. Barry Cooke, Chairman

TM Leps

Thomas M. Leps

Roger Rhoades

Roger Rhoades

ACKNOWLEDGMENTS

This report would not be complete without special recognition of the outstanding cooperation received from the Department of Water and Power, City of Los Angeles, during the course of the investigation. Its staff was very helpful in providing files, records, and other background material in addition to personal accounts of events on the day of failure. Men and equipment were furnished on short notice to undertake the extensive exploration program necessary at the reservoir site. The sum total of these efforts represents a major contribution to this investigation, one which not only facilitated the work of the Engineering Board of Inquiry, but added to the completeness of this report. This cooperation is gratefully acknowledged.

Special mention is also made of the following organizations, all of which contributed in some way to this investigation:

U. S. Coast and Geodetic Survey
U. S. Geological Survey
U. S. Corps of Engineers
California Division of Oil and Gas, Department of Natural Resources
California Division of Highways, Department of Public Works
Los Angeles County Engineer
Los Angeles Times
Los Angeles Herald-Examiner
Standard Oil Company of California
Television Station KTLA
Television Station KNXT

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

EDMUND G. BROWN, *Governor*

HUGO FISHER, *Administrator*, The Resources Agency

WILLIAM E. WARNE, *Director*, Department of Water Resources

ALFRED R. GOLZÉ, *Chief Engineer*

This investigation was conducted by the

ENGINEERING BOARD OF INQUIRY

Robert B. Jansen, Chairman	Deputy Division Engineer Division of Design and Construction
Gordon W. Dukleth	State Engineering Representative Federal-State San Luis Project
Bernard B. Gordon	Department Soils Engineer
Laurence B. James	Chief Engineering Geologist
Clyde E. Shields	Project Construction Engineer North San Joaquin Division, State Water Project

CONSULTING BOARD

J. Barry Cooke, Chairman	Consulting Engineer
Thomas M. Leps	Consulting Engineer
Roger Rhoades	Consulting Engineering Geologist

SPECIAL CONSULTANT

Dr. Pierre St.-Amand	Consulting Seismologist

CHAPTER 1

INTRODUCTION

On Saturday, December 14, 1963, at about 11:15 a.m., the caretaker at the Baldwin Hills Reservoir in Los Angeles, California, detected an unusual sound of running water in the spillway discharge pipe at that facility. This was the first of a series of observations made by various persons witnessing a catastrophe in the making. By 3:38 p.m. the reservoir was ruptured and the churning water escaping through the breach had brought devastation to the communities in its path.

Baldwin Hills Reservoir is located on the north slope of the Baldwin Hills, approximately eight miles southwest of the Los Angeles City Hall. (Plate 1) These are the highest hills in the southwest part of Los Angeles, making this a logical location for a reservoir with the necessary elevation to serve the south and southwest sections of the city, extending to the industrial areas adjacent to Los Angeles International Airport. At the time of site selection the Baldwin Hills were relatively undeveloped except for an oil field lying generally south and west of the proposed reservoir site.

Most of the Los Angeles water supply is imported from distant points through long aqueduct lines. This necessitates maintaining a large close-in storage capacity to assure sufficient water for both peaking and emergency purposes. Because of the great area to be served, long lines are required for water distribution inside the city. For this reason strategically located storage, such as at Baldwin Hills Reservoir, is important to the efficient and economical operation of the water system.

From the time it was placed into service in 1951, the Baldwin Hills Reservoir had been regarded as a model of engineering excellence and a source of pride to its builder and owner, the Los Angeles Department of Water and Power. Painstaking care had been taken in its design and construction. It had been kept under close surveillance during its 12 years of operation. Its function of providing water service to the rapidly expanding southwest part of the City of Los Angeles had been well performed.

As darkness settled over the scene on the north edge of the Baldwin Hills on that Saturday, there were imperative questions which had to be answered: How had the reservoir failed? What had caused the failure?

The investigation which is the subject of this report was conducted to find the answers to these questions.

PURPOSE, SCOPE, AND AUTHORITY OF INVESTIGATION

On December 19, 1963, William E. Warne, Director, Department of Water Resources, issued an order creating an Engineering Board of Inquiry for the purpose of accomplishing a full investigation of the failure of the Baldwin Hills Reservoir. This directive designated Robert B. Jansen, Deputy Division Engineer of the department's Division of Design and Construction, as Chairman of this Board and delegated to him the Director's authority under Section 124 of the State Water Code to organize the Engineering Board of Inquiry and to conduct the investigation.

The Director's order included authorization for the Chairman of the Engineering Board of Inquiry to:

1. Select the members of his Board from employees of the Department of Water Resources, subject to the approval of the Chief Engineer.
2. Engage a consulting board to assist the Engineering Board of Inquiry.
3. Retain such additional experts and consultants as might be required during the course of the investigation.
4. Seek the assistance of other state agencies as required.
5. Personally, or through his designated representative, audit hearings of other agencies regarding the subject of this investigation and secure copies of transcripts, exhibits, books, and records as required.

The Chairman's recommendations for membership of the Board were submitted to, and approved by, Chief Engineer Alfred R. Golzé on December 20, 1963. The members were: Robert B. Jansen; Gordon W. Dukleth, State Engineering Representative, Federal-State San Luis Project; Bernard B. Gordon, Department Soils Engineer; Laurence B. James, Chief Engineering Geologist; and Clyde E. Shields, Project Construction Engineer, North San Joaquin Division, State Water Project.

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The objectives of this investigation were:

1. To determine the way in which the reservoir failed.

2. To determine the physical causes of the failure.

The investigation was to include:

1. Review of the design of the dam and reservoir.

2. Review of the construction of the dam and reservoir.

3. Review of the operation and maintenance of the dam and reservoir.

4. Study of the natural phenomena which might be related to the failure, including, but not limited to, subsidence and seismic movement.

PROCEDURES

The investigation by the Engineering Board of Inquiry was started immediately upon issuance of the Director's order. Steps were taken to acquaint the Board with data collected by state personnel in the hours and days following the reservoir failure. The organization of a task force of employees in the department's Southern District Office in Los Angeles had been started on December 18, 1963, for purposes of on-site data collection. This task force was assigned to work under the general direction of the Engineering Board of Inquiry.

In conducting the investigation, design files, exploration reports, and inspection reports compiled during construction and operation and maintenance of the reservoir were reviewed. Field work on the dam, reservoir, and adjacent area was undertaken as required.

A Board of Consultants was retained to assist the Engineering Board of Inquiry. This advisory group consisted of: Chairman J. Barry Cooke, Consulting Engineer; Thomas M. Leps, Consulting Engineer; and Roger Rhoades, Consulting Geologist. The Board of Consultants provided valuable advice in the determination of the scope of investigation and of methods of data collection and analysis.

Dr. Pierre St.-Amand, Consulting Seismologist, served the Engineering Board of Inquiry in the specialized fields of seismicity, tectonics, and instrumentation.

The work of the task force included compiling detailed logs of core holes and exploratory excavations. All visible cracks in the reservoir and in the areas adjacent to the reservoir were mapped and photographed. Liaison was established with the Department of Water and Power, City of Los Angeles. Pertinent geologic and engineering reports on the site were obtained from that agency, as well as engineering plans and specifications and a final construction report.

Data which were obtained from the Department of Water and Power included instrumentation readings and plots of settlement, horizontal and lateral movement, and recorded flows in the reservoir underdrains and the drainage system under the embankment of the main dam.

The Department of Water and Power cooperated fully in conducting exploration by drilling and excavation at the reservoir and by establishing survey control points to enable detection of any additional movement following the failure. The measurements on these points and all other data were made available to the Engineering Board of Inquiry for analysis.

In support of the investigation by the Board, the Department of Water Resources Laboratories Branch assigned a crew to obtain samples of materials at the Baldwin Hills site. Testing of materials was performed by the soils, concrete, and chemical laboratories.

Services of the Corrosion Engineer of the Department of Water Resources were obtained for conducting a special investigation of corrosion in the circulator conduit system of the reservoir.

As part of the investigation, data were collected on the seismic history of the Baldwin Hills area and on the locations of faults and earthquake epicenters.

A study was made of the history of extraction of potable water from the underground basins in the Baldwin Hills vicinity.

Information was obtained from the State Division of Oil and Gas and from the Standard Oil Company of California on the geology and oil production activity in the Inglewood Oil Field.

Inquiries were made of all agencies that were regarded as possibly having survey records related to the Baldwin Hills area. Data on horizontal and vertical control were provided by the U. S. Geological Survey, the U. S. Coast and Geodetic Survey, and local agencies.

Staff photographers of the Department of Water Resources and Department of Water and Power made a photographic record of all pertinent on-site geology, exploratory activity, and evidence of the failure. Photographic coverage of the dam failure was obtained from local television stations, other news media, individuals, and the Department of Water and Power.

Eye witnesses to the failure, and others having knowledge of the reservoir and its environs, were interviewed by members of the task force or by the Engineering Board of Inquiry.

Observations made by individuals, field and office data used in the investigation, maps, charts, photographs and motion picture films, newspaper accounts, copies of relevant public records, and all other supporting information were fully documented, indexed, and incorporated into the official record of this investigation.

All public hearings held in connection with the Baldwin Hills disaster were attended by representatives of the Engineering Board of Inquiry, and all pertinent information disclosed therein has been investigated.

SUMMARY OF FINDINGS

The investigation by the Engineering Board of Inquiry of the failure of the Baldwin Hills Reservoir has resulted in the following findings:

How Did the Reservoir Fail?

Earth movement occurred at the reservoir on December 14, 1963, following an apparent long-term development of stress and displacement in the foundation. The movement was apparently not seismic but took place at faults which were planes of foundation weakness.

Foundation displacement resulted in rupture of the reservoir lining and consequent entry of water under pressure into a pervious and erodible fault. Erosion in the fault and adjacent foundation proceeded rapidly, causing uncontrolled leakage through the east abutment of the main dam.

What Caused the Failure?

Evidence indicates that the earth movement which triggered the reservoir failure was caused primarily by land subsidence which has been experienced in the vicinity for many years. This relatively fast settlement over a limited area is superimposed on more general, slow, and widespread tectonic deformations along the Newport-Inglewood fault system.

Distortion of the land surface has resulted in an increase in the reservoir dimension generally in a northeast-southwest direction, with several inches of elongation during the life of the reservoir. The tension accompanying this distortion apparently caused opening at pre-existing foundation faults. Subsidence is manifested in displacement of foundation blocks at these faults. Such movements led to rupture of the reservoir lining, which was designed to preclude the passage of water into the foundation, known to be highly susceptible to deterioration by the action of water. There is evidence that continuous leaking of water had created cavities in the foundation at the faults. In some places it appears that the reservoir lining was able to bridge these cavities as they developed. With the earth movement on December 14 the lining collapsed into these cavities and accelerated the failure process.

A fractured fault zone cutting through the east abutment of the main dam, together with the erodible foundation strata, provided minimal resistance to flow.

CHAPTER II

FAILURE OF RESERVOIR

The day of December 14, 1963, began quietly. At 7:45 a.m. the reservoir keeper (caretaker) of the Baldwin Hills Reservoir, Mr. Revere Wells, drove up the roadway from La Brea Avenue to the reservoir site. After making his routine readings of the inflow and outflow meters at the tunnel portal, as was his daily practice, he drove up the roadway to the caretaker's station on the east side of the reservoir. He crossed the footbridge to the gate tower and observed the water level in the reservoir. All the readings taken and observations made at the tower were regarded by Mr. Wells as normal at that time. A few minutes after 8:00 a.m. he made a telephone report of his observations to the Western District switchboard of the Department of Water and Power. Then for nearly an hour he stationed himself by the telephone at the caretaker's shelter awaiting operating instructions for the day.

Beginning at about 9:00 a.m. the caretaker made his routine tour of the reservoir, moving in a clockwise direction along the upper roadway outside the reservoir fence. This required between 20 and 30 minutes. He observed nothing unusual during that patrol.

At about 9:30 a.m. he returned to his station on the east side of the reservoir and for the next hour and a quarter maintained a routine vigil from that point.

At about 10:45 a.m. Wells prepared to make a regular inspection of the spillway catch basins on the north face of the main dam, as shown on Plates 2 and 3. He walked across the dam and unlocked the barricade at the northwest corner of the reservoir. He walked back to the northeast corner barricade and unlocked it. He then went inside the fence and checked the water in the reservoir for any material that might be lodged against the spillway intake. He was standing over the spillway intake when he heard a light but unusual sound of rushing water. The time of this observation was estimated to be approximately 11:15 a.m.

Wells walked across the inner roadway to a small catch basin over the spillway pipe where it was possible to hear better. The acoustics inside this basin enabled the detection of the slightest noises. The usual sound which would have been heard at this point was that of a low flow of drainage water, normally between 10 and 13 gallons per minute (gpm), dropping from the 24-inch blowoff pipe into the spillway pipe at the junction of the two. At this location, the sound of flowing water was unusually loud. Listening for a few minutes, he had the impression that the flow from the drainage system was increasing.

Since it was not possible to see the drainage flow at this point, he drove to the catch basin near Manhole A, where it could be seen. He looked into the basin and saw at once that there was about four or five times the usual amount of water flowing in the 42-inch spillway pipe, and the water was muddy. Realizing that this water was coming from the drainage inspection chamber beneath the reservoir, he drove back up onto the reservoir roadway and down to the inlet tunnel portal, east of the reservoir. He walked through the tunnel, around the tower, to the drainage inspection chamber, where he observed that the upper floor of the chamber was still dry. Standing at the steps in the chamber, near the gate tower, he could see that the lower floor of the chamber was covered with water. He observed that the three reservoir underdrain pipes nearest him were "blowing like fire hoses", discharging muddy water at a high rate. Under normal conditions, these drains would be carrying the seepage which penetrated the compacted earth lining on the east side of the reservoir. The water "appeared to be whitish," and the streams appeared to be impinging near the base of the north wall of the drainage inspection chamber. To enter the drainage inspection chamber it is necessary to descend three steps to the first platform and then four more steps to reach the floor of the chamber. Wells reported that soon after 11:15 a.m. water had risen up over the lowest four steps. He was concerned about a crack in the drainage inspection chamber which had been under observation for several years. He expected that, if the crack had opened, there would be an inflowing curtain of water at that point; but he did not observe any such curtain flow. It was his impression that the crack in the drainage inspection chamber had not yet opened.

Having made these observations, Mr. Wells, at about 11:35 a.m., went immediately to his caretaker's station on the rim of the reservoir and placed a telephone call to Mr. Patrick Doherty, Operations Field Superintendent of the Western District.

Wells then went back into the drainage inspection chamber. Removing his shoes and proceeding barefooted, he attempted to get farther back into the chamber. At that time the water had risen above the lowest of the three highest steps. Wells could feel silt with his bare feet on those steps. He reported that the water was "just blowing, making a terrific racket, and blowing right out." Wells reported that the drain which was discharging the greatest was the drain nearest him, which was the southeast toe drain.

At 12:05 p.m. Doherty and Oscar Graf, a valve foreman who had been summoned by Doherty, arrived separately at the tunnel portal. They observed that no water was flowing from the portal at that time.

After telephoning Mr. Gerard Wyss, Assistant Head, Water Operating Division, Doherty joined Graf and Wells to proceed to the drainage inspection chamber. Doherty reported that the water was just starting to flow past the tower, having covered both series of steps in the chamber. He looked specifically to see if there was an opening of the cracks in the chamber, but he could not see any. He could see water issuing at high velocity from the first three drains. He reported that the first drain, the southeast toe drain, was spouting over the top of the weir box, across the chamber, and impinging on the far wall. When Doherty looked for flow through the cracks in the chamber, some leakage was apparent but not a large inflow. The lights in the tunnel and the chamber were still operable at that time. Layout of the drain system is shown on Plate 4.

Gerard Wyss arrived at the inlet tunnel at about 12:15 p.m. He entered the tunnel and was about halfway between the portal and the drainage inspection chamber when he met Doherty, Wells, and Graf coming out. He did not proceed farther. At that time water from the drainage inspection chamber had reached the tunnel portal and was flowing to La Brea Avenue.

At 12:20 p.m., a decision was made to begin emptying the reservoir, and procedures for reservoir drainage were activated. By this time a heavy flow of water was beginning to run down La Brea Avenue. Steps were taken to notify the police department of the situation. Doherty closed the valve on the inlet line at the tunnel portal whereby all inflow to the Baldwin Hills Reservoir was stopped. Flow through the outlet line continued.

It appears to be the consensus of the Water and Power personnel involved that the final stage of the structural failure of the drainage inspection chamber did not occur until shortly after 12:00 noon. It has been reported that, at about 12:20 p.m., water discharging at the chamber started rising rapidly. It appeared that the chamber cracks opened at about that time. Those on the ground reported no evidence of earth tremors at the time of the assumed failure.

At about 12:45 p.m. Mr. Richard E. Hemborg, Engineer in Charge, Water Operating Division, and Mr. William Tate, Supervisor of the Foundations and Structures Maintenance Section, responding to a telephone summons by Wyss, separately arrived at the tunnel portal.

From his home immediately north of Baldwin Hills, Tate had driven south on La Brea Avenue and encountered "a river of muddy water." He was not able to continue on La Brea and therefore retraced his route, approaching by way of Cloverdale Avenue to the top of the main dam and down to the inlet tunnel portal. He estimated that there was 10 or 15 cubic feet per second (cfs) of muddy water flowing from the tunnel when he arrived. The rate of flow from the tunnel was reportedly increasing. He found operations personnel standing in water to adjust the operating valves.

At about 1:00 p.m. Tate drove to the reservoir rim with Wells with the intention of inspecting the catch basins on the north side of the main dam. Wells had informed Tate about hearing water running through the 24-inch blowoff pipe. Starting to drive across the main dam, the two men looked along the downstream face. Tate reported seeing muddy water emerging from the east abutment in an area estimated to be about 5 feet in diameter, at a point about 10 feet above the Elevation 390 berm. Water was bubbling out and running toward the catch basin.

The two men immediately returned to the tunnel portal to report their discovery to Wyss and Hemborg. Mr. Tate reports that he did not notice any enlargement of an old crack in the crest of the dam. He returned to his home in the threatened area and advised his wife to alert the neighborhood.

Tate reports that he and Wells were the first to observe the leakage emerging downstream of the dam. He believes that the flow through the structure had just begun because the wet spot was not more than 4 or 5 feet in diameter. He estimates that the water running into the surface drain had not flowed for a distance greater than about 4 or 5 feet. The surface drain was a concrete gutter constructed to carry rainwater to the catch basin. Tate is of the opinion that the leakage was emerging from the abutment and not from the embankment. He estimates that the first time he saw this leakage must have been within 15 minutes of the time that it started. Upon returning from his home, in about a half hour, the flow had increased so that it was running down the surface drain into the catch basin near Manhole A. (Photos 1 through 4)

Dr. A. I. Coleman, a resident of Cloverdale Avenue just below the dam, viewed the failure with the aid of binoculars. He reported that at about 1:00 p.m. he noticed water emerging from two places on the downstream slope of the dam, "30 feet and 50 feet from the crest."

Between 1:00 and 2:00 p.m. adjustments were made in the operation of the water system to accelerate the draining of the reservoir. The reservoir inlet valve was reopened to reverse the flow and permit discharge through a Howell-Bunger valve near Rodeo Road and La Brea Avenue into a storm drain leading to Ballona Creek. Adjustments were also made in the water distribution system outlet line operation to obtain increased flow to the areas of water consumption. A relief valve was opened at Venice Boulevard and Overland Avenue to allow discharge of about 20 cfs into a local storm drain. It was estimated that maximum controlled draft from the reservoir during these emergency draining efforts reached approximately 450 cfs.

Mr. Wyss reported that about ⅛-inch movement was noted in the inlet line of the reservoir after failure. He believes that it could be attributed to the vibrations during drainage attempts. He reports that they were passing so much water through the inlet line that the line actually lifted from its piers. The line vibrated so violently that it was necessary to partially close the Howell-Bunger valve because it was feared that pipe joints would pull apart at the flexible couplings.

The outlet line, at maximum discharge, apparently exceeded the capacity of the tower outlet gates. It vibrated severely because of drawing air into the line.

At about 1:15 p.m. Wyss and Wells went to the downstream face of the dam and drove onto the 390 berm to appraise the seriousness of the abutment leakage. Wells reported that the leakage area was about 5 or 6 feet above the berm and the water had run down to the berm and then about 10 or 12 feet over to the surface drain that continues to the catch basin on the berm at Elevation 340. They did not observe any seepage from elevations higher than this, nor did they see other areas of seepage on the downstream face of the dam. The leakage area just above the 390 berm was the only trouble spot that they saw. It was "like a 12-foot circle." There was silt being discharged with the water. Wyss estimated that the flow then was perhaps 10 gpm. The fact that it was carrying silt was a warning that failure by breaching was possible.

Returning to the tunnel portal, Wyss reported the severity of the leakage to Richard Hemborg. It was at this time that the decision was made that the area below the main dam should be evacuated. At 1:30 p.m. Hemborg telephoned the Los Angeles Police Department to request that the area in danger be evacuated.

At about the same time Tate drove back to and across the main dam. He reports the crack across the crest at Station 8+93.5 had begun to open and leakage water from the downstream face was flowing to the catch basin near Manhole A. (Plate 5) He watched the crack until 1:45 p.m. and reports that it widened from "¼ inch to 3 inches" in a period of about 15 minutes. This crack was at the beginning of the northeast curve where the embankment contacted the abutment. The last time Tate saw the crack was about 1:45 p.m., when it appeared to be opening at such a rate that he was fearful of driving across it. He noted that the crack had extended across the pavement and was apparent in the downstream slope, immediately above the point where the leakage was appearing at the 390 berm. At that time, the flow was still confined within the gutter.

The Chief Engineer of Water Works, Max K. Socha, arrived at the reservoir between 1:35 p.m. and 1:45 p.m. and went with Tate and Doherty to the dam crest to observe the leakage at the east abutment. They saw that the rate of discharge had increased. Mr. Socha examined the crack in the roadway pavement on the dam crest almost directly above the apparent line of seepage through the abutment.

A few minutes later Socha received a telephone call from Los Angeles Chief of Police William H. Parker, who was calling to confirm the nature of the emergency. He estimated for Chief Parker that there might be no more than two hours in which to evacuate the area. He also attempted to define the approximate extent of the area which might be subjected to flooding. It was decided to evacuate the area south of Jefferson Boulevard, between La Cienega Boulevard and La Brea Avenue, extending southward to Baldwin Hills.

Two police officers in a patrol car drove up to Tate's car at about this time. They had discovered the water flowing down La Brea Avenue and had come to investigate. Tate took them to see Wyss and heard him tell the officers that evacuation had been requested. He estimates that the time then was between 1:30 and 2:00 p.m. Mr. Tate thereupon departed from the site to assist in the evacuation of the threatened area.

Mr. Lee E. Clark, construction foreman, was summoned from his home and arrived at the reservoir at about 1:30 p.m. Upon arrival he noticed water flowing over the 390 berm on the downstream face of the main dam and down the gutter toward the upper catch basin. He observed the crack in the earth embankment north of the upper roadway pavement of the dam and noted that it was opening rapidly. He also saw whirlpools or vortices on the water surface between the gate tower and the dam and adjacent to the dam near the northeast corner of the reservoir.

At about 2:00 p.m. Richard Hemborg inspected the discharge at the east abutment downstream of the dam. He described it as "a sheet of water shooting through that little crack—a crack maybe 15 or 20 feet high—right above the 390 berm." Streets below the dam were dry and clean then. The leakage water was being contained below the dam, as it would be for another 90 minutes. The increasing flow through the east abutment was now discharging into a detention basin formed by the road embankment of Cloverdale Avenue. Not until about 3:30 p.m., eight minutes before the total breach, would the detention basin overflow and begin flooding residential streets.

At 2:20 p.m. a sigalert broadcast was made on all radio and television stations. Some sixty patrol units responded and began warning residents thought to be in danger at that time.

At about this time receding water in the reservoir revealed the top of a 3-foot wide break on the inner slope directly opposite the widening break on the downstream face. (Photos 5 through 12) The full extent of the break was hidden under water, but engineers agreed that "it looked as if you could drop in a plug of sandbags and slow the outflow down." A call for sand and bags went to the Western District yard of the Department of Water and Power on Venice Boulevard.

At 2:30 p.m. measures were taken to remove any debris which might restrict discharge of water into the storm drain system above the Cloverdale Avenue residential area.

At this time it was observed that the crack across the roadway on the crest of the main dam was continuing to widen. (Photo 13) Leakage through the abutment was steadily increasing, as was the discharge from the drainage inspection chamber through the tunnel portal. All of the leaking water was carrying appreciable sediment.

Colonel E. G. Peacock, District Engineer, U. S. Army Engineer District, Los Angeles, arrived at the reservoir at approximately 2:30 p.m. He noticed that the water surface was at the hole in the upstream face of the main dam. He observed that, at approximately one-third of the distance from the embankment to the gate tower, there was an area of the water surface which appeared to him to be similar to an oil slick. He observed a 1-inch crack in the dam crest at the east abutment. In walking downstream along the east abutment he observed leakage which he estimated to be "equivalent to the flow in a 14-inch pipe." These flows were below the upper paved berm at Elevation 430. However, there had been flows at a higher elevation, as evidenced by flattened vegetation. The eroding action was in a vertical slot progressing upstream. Colonel Peacock emphasized that there was not a large surge of water at any time as the reservoir failed.

Mr. Clark arrived at 2:45 p.m. with the first truckload of sand and bags. He was lowered on a safety rope to the hole on the upstream dam face. He first used a sledge to break away the undermined asphaltic paving. An effort was then made to stem the flow by dropping sandbags into the hole. The bags disappeared without evidence of impeding the flow, and the futility of continuing with this remedy was soon realized. Mr. G. R. Holman, Central District Office engineer, assisting in the sandbagging attempt, recalls: "When the water level dropped a little more, we could see a hole as big as a living room inside the crack. The outlet was invisible . . . ."

Dr. Coleman reports that at 2:30 p.m. he noticed water "cascading" out of the break. Between 3:00 p.m. and 3:15 p.m. he noticed that the "two holes" joined and the detention basin was filling.

At 3:00 p.m. it was ordered that a drain valve at Centinela and Florence Avenues be opened to provide an additional discharge from Baldwin Hills Reservoir of about 30 cfs.

It was 3:15 p.m. The breach on the upstream face was about 10 feet wide. Water was pouring in violent agitation as it discharged through the rapidly enlarging breach. A few minutes more and the detention basin could no longer hold it. Water spilled over the basin edge and into the residential streets below.

Meanwhile, assigned to streets in teams of two, policemen raced from house to house, warning residents to evacuate while there was time.

At about 3:00 p.m. the Los Angeles and Dorsey High Schools opened their gymnasiums for use as evacuation centers. Red Cross crews were immediately mobilized to give assistance. Evacuation center locations were broadcast by sigalert. By this time about 200 state and local police officers were on duty to seal off the danger zone.

The reservoir water level was falling steadily under combined effect of uncontrolled flows through the dam and the inlet tunnel plus a blowoff into the Ballona Creek storm drain through the reversed inlet line and vastly increased service demands loaded on the outlet line. By 3:30 p.m. a heavy discharge was issuing from the widening breach at the east abutment.

In the final stages of failure muddy water was gushing from a slot extending the full height of the abutment crack. Then, suddenly, there was an eruption of muddy water and fragments of earth near the bottom, forming a large hole into which the upper areas of abutment and embankment collapsed. By 3:38 p.m. the failure was regarded as essentially complete. (Photos 14 through 17)

Water surging from the breach swept through the residential subdivision immediately below, washing houses from their foundations and in many cases obliterating the foundations. At the base of the hills the flow fanned outward swiftly, engulfing the heavily populated community below.

The area reportedly flooded by the waters has been described as bounded approximately by Ballona Creek on the west, La Brea Avenue on the east, Jefferson Boulevard on the north, and the Baldwin Hills on the south. The most seriously damaged section was limited on the east by La Brea Avenue, on the west by Hauser Boulevard, on the north by Jefferson Boulevard, and on the south by the Baldwin Hills.

By 6:50 p.m. flood water had receded to a point where it was possible to move in heavy equipment and begin the tremendous task of cleanup and restoration.

The toll was heavy: Scores of homes destroyed, total damage estimated at more than $15,000,000, and five people dead. But early warning and prompt action by Department of Water and Power personnel had averted a much greater tragedy. Their quick recognition of the danger and their forthright action undoubtedly saved many lives.

CHAPTER III

GEOLOGY

Numerous geological investigations have been completed of the Los Angeles Coastal Plain—many to considerable detail. The discussion of regional geology herein presented is generalized and limited to the most basic geologic features of that part of the Coastal Plain that lies within Los Angeles County. These features are shown on Plate 6.

An approximate geologic time scale is presented in Table III-1 for those who may be unfamiliar with the names of the geologic epochs and periods.

DESCRIPTION OF AREA

The Baldwin Hills are the northernmost and highest (maximum elevation approximately 510 feet) of a chain of ridges which trend northwest-southeast and extend about 42 miles between Beverly Hills in Los Angeles County and Newport Beach in Orange County. This alignment of ridges, known as the Newport-Inglewood uplift, accommodates 14 major producing oil fields. Inglewood Oil Field, which adjoins the reservoir on the south and west, is one of the northernmost of these fields.

Ballona Gap skirts the northern and western slopes of Baldwin Hills and separates them from the plains and piedmont slopes that adjoin the Santa Monica Mountains to the north. The gap is a narrow alluvial plain which interconnects Downey Plain on the east with Santa Monica Bay on the west. Culver City, the home of motion picture studios, is situated in the center of Ballona Gap immediately to the west of Baldwin Hills.

The east slopes of Baldwin Hills dip beneath the Downey Plain, an extensive alluvial surface which extends eastward through Los Angeles County and into Orange County. The southwest slopes join the Torrance-Santa Monica Plain which meets the shore of Santa Monica Bay at Playa Del Rey five miles to the west. Los Angeles International Airport is located six miles to the southwest of the reservoir.

In the early days Baldwin Hills provided pasture for the sheep of the 4,000-acre Las Cienegas Ranch. The owner, E. J. (Lucky) Baldwin, for whom the hills were named, was a spectacular promoter regarded by many as America's first realtor. Reportedly, Las Cienegas Ranch, including Baldwin Hills, was forfeited to "Lucky" Baldwin in the 1870's as partial security on a note for $210,000.[1] With discovery of Inglewood Oil Field in 1924, its value was enhanced greatly, and the quiet pastoral setting of Spanish land grant days was transformed to a scene of intense industrial activity.

[1] Glasscock, C. B. "Lucky Baldwin." Bobbs-Merrill Company. 1933.

TABLE III-1

GEOLOGIC TIME SCALE

Period	Epoch	Approximate length of period in millions of years	Approximate millions of years since period commenced
Cenozoic Era			
Quaternary	Recent	(10,000 years)	(10,000 years)
	Pleistocene	1	1
Tertiary	Pliocene	10	11
	Miocene	14	25
	Oligocene	15	40
	Eocene	20	60
	Paleocene	10	70
Mesozoic Era			
Cretaceous		65	135
Jurassic		45	180
Triassic		45	225
Paleozoic Era			
Permian		45	270
Carboniferous		80	350
Devonian		50	400
Silurian		40	440
Ordovician		60	500
Cambrian		100	600
Precambrian			4½ (Billion Years)

REGIONAL GEOLOGY

The Coastal Plain in Los Angeles County, in which Baldwin Hills are included, covers an area of roughly 600 square miles. Its most prominent land forms include bordering highlands, river channels that drain the area, a broad alluvial expanse known as the Downey Plain, a similar but less extensive area called the Torrance-Santa Monica Plain, and an alignment of low hills designated as the Newport-Inglewood uplift.

With the exception of the bordering highlands, the formations exposed are almost entirely sedimentary. Those of Pleistocene and Tertiary geologic age have heen faulted and folded to varying degrees, in some instances creating structural domes in which oil and natural gas have accumulated. In all, twenty-four producing oil fields lie within or immediately adjacent to the Los Angeles Coastal Plain, making it one of the major oil producing areas of the United States.

The highland borders of the Coastal Plain are in part crystalline rocks and in part consolidated sediments. They are the result of tectonic uplifts that occurred largely in late Pleistocene time, and they stand in moderate to bold relief, attaining elevations of 2,126 feet in Santa Monica Mountains on the north, 1,387 feet in Puente Hills on the northeast, and 1,480 feet in Palos Verdes Hills at the southwest corner of the region. The Pacific Ocean lies to the south and west.

Three intermittent rivers cross the Coastal Plain in Los Angeles County, originating in watersheds to the north and flowing in a general southerly direction to the ocean. Named from east to west, these are the San Gabriel, Rio Hondo, and Los Angeles Rivers. Only during infrequent heavy rains do these streams carry significant flows.

Ballona flood channel, which conveyed to the ocean the flood that resulted from the reservoir failure, lies north of Baldwin Hills in the Ballona Gap. This channel follows an old, abandoned course of the Los Angeles River, which, prior to 1825, flowed westward to Santa Monica Bay.[2]

Downey Plain occupies the eastern part of the Coastal Plain in Los Angeles County and is the dominant lowland area. Beneath its surface are sedimentary deposits probably exceeding 40,000 feet in thickness at the deepest point. The shallow sediments are largely the products of slope wash from the bordering highlands and the compounding of the alluvial fans of the three contributory rivers. The bulk of the deposits, however, consists of marine sediments of Tertiary age. These have been deposited in a broad trough designated the Paramount syncline, which trends generally northwest-southeast. Southwest of the axis of this trough the strata warp upward to form the northeast limb of the Newport-Inglewood uplift.

A second expanse of low relief, the Torrance-Santa Monica Plain, lies west of Downey Plain and southwest of Baldwin Hills. It is underlain by sediments largely of marine origin and Tertiary age, which rest upon a basement complex of igneous and metamorphic rock. The sediments attain their greatest thickness, which is probably as much as 15,000 feet, along the axis of a trough called the Gardena syncline. East of Gardena syncline the formations rise to form the southwest limb of the Newport-Inglewood uplift.

Inglewood Fault

Inglewood fault is the most northerly of four principal faults of the Newport-Inglewood uplift. It extends southeastward from Beverly Hills, passing through Baldwin Hills and the City of Inglewood, terminating about one-half mile southeast of Hollywood Park Race Track. Its length is about nine miles; however, its continuity is interrupted in Baldwin Hills by cross faults which offset the main fault and divide it into seven distinct segments. Inglewood fault passes about 500 feet west of Baldwin Hills Reservoir (Photo 18), and is marked here as an escarpment on which over 270 feet of vertical displacement has been measured, the west side being downdropped. It has been estimated that about 1,500 feet of lateral movement has occurred on Inglewood fault in Baldwin Hills.[3] This movement has been predominantly right lateral, the east side of the fault moving to the southeast with respect to the west side.

North of Baldwin Hills, Inglewood fault passes beneath the shallow deposits of Recent alluvium that underlie Ballona Gap. These deposits comprise a mantle of low permeability up to 40 feet thick which is underlain by a tongue of gravel that extends from Downey Plain to the ocean. The lower alluvial member is called the "50-foot gravel," which has been tapped in the past by water wells. Since the "50-foot gravel" was at one time an important source of water supply, it has been subjected to geologic scrutiny. These ground water studies have shown that late Pleistocene aquifers in Ballona Gap have been transected by the Inglewood, Charnock, and Overland faults but that there is no evidence that the aquifers of Recent age have been displaced.[4]

In Baldwin Hills, where the scarp of Inglewood fault stands over 200 feet in height, detailed investigation by the U. S. Geological Survey has revealed no evidence of recent lateral displacement such as offset stream channels or fan deposits.[5]

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[2] Layne, G. L. "Annals of Los Angeles from the Arrival of the First White Man to the Civil War, 1769 to 1861." Special Publication No. 9. California Historical Society. 1935.

[3] Driver, H. L. "Inglewood Oil Field." Bulletin 118. California State Department of Natural Resources. 1943.

[4] Poland, J. F. and others. "Geology, Hydrology, and Chemical Character of Ground Waters in the Torrance-Santa Monica Area, California." U. S. Geological Survey, Water Supply Paper 1461. 1959.

[5] U. S. Geological Survey. "Preliminary Report on Recent Surface Movements Through July 1962 in the Baldwin Hills, Los Angeles County, California." Open File Report. January 1964.

Beneath Ballona Gap the late Pleistocene aquifers have been downdropped along the east side of Inglewood fault. This movement is in contrast to the displacement in Baldwin Hills where it was the west side of the fault that dropped. Thus a scissors action appears to have occurred on Inglewood fault around a pivoted point located somewhere in Ballona Gap.[4]

North of Ballona Gap, Inglewood fault has been tentatively extended to Beverly Hills on the basis of an escarpment which aligns with the fault in Baldwin Hills. The trace of this fault is lost in the alluvial deposits in Beverly Hills, and its extension is not discernible in the crystalline rocks of the Santa Monica Mountains farther north.

In Baldwin Hills a second fault parallels Inglewood fault about 2,000 feet to the west. The block between these faults has dropped, resulting in a graben structure which contains the main Inglewood Oil Field. This graben and its boundary faults are cut by several lesser faults which trend generally northeast-southwest. These lesser faults offset the Inglewood fault and its parallel companion and obviously, therefore, have moved more recently than the Inglewood fault.

GEOLOGY OF THE DAM AND RESERVOIR SITE

Principal Features

Baldwin Hills Dam and Reservoir were constructed entirely on sedimentary deposits of marine and littoral origin and of Pliocene and Pleistocene age. From oldest to youngest, these include the Pico formation of upper Pliocene age, the Inglewood formation of lower Pleistocene age, and the Palos Verdes formation of upper Pleistocene age. A geologic map is shown on Plate 7.

The Pico formation underlies the Inglewood formation, which in turn underlies the Palos Verdes formation, the latter contact being an unconformity. The embankment of the main dam rests generally on and against Pico formation at elevations below about 340 feet, against Inglewood formation below 470 feet in the east abutment, and against Palos Verdes formation near the crest.[6]

The strata at the site are, for the most part, gently tilted, with a predominant northwesterly strike and southwesterly dip, although there are several local variations due to warping. The average dip is reported as about five to seven degrees. Two minor synclines and an anticline were mapped near the axis of the dam during construction, but these structures now lie buried beneath the embankment.

Seven minor faults were mapped in the reservoir vicinity during construction. The traces of two of these, which are designated as Faults I and V, are now clearly discernible in the reservoir floor. Fault II is closely associated with Fault I and has been considered to be a part of the Fault I zone in this report. The trace of Fault I is spectacularly apparent in the frontispiece of this report where it appears as a crack in the reservoir extending through the breach in the dam.

Past Investigations

The earliest report of record on the geology of Baldwin Hills Reservoir was prepared by Mr. Milton D. Burris, former employee of the Department of Water and Power.[7] Mr. Burris referred to geological investigations including auger holes, test pits, and 16-inch drill holes that had been completed at the site during the four years preceding his report. He discussed the local stratigraphy, structure, and regional seismicity and presented a preliminary geologic map, geologic sections, and photos of the site. His conclusions were:

> "The following are brief statements of the reasons why the reservoir site is recommended:
>
> 1. The homogeneous nature of material upon which the lower portion of the reservoir is to be built.
>
> 2. The lack of oil production in this northern portion of the east block which might eventually cause subsidence in the area.
>
> 3. The logs of these abandoned wells in this area show increasing hardness in the formations at greater depths. This is especially true of the 8,775-foot well completed by the Royalty Service Corporation in 1941.
>
> 4. The excellent exposures through and beyond the proposed site in both east-west and north-south directions. A thorough examination has failed to show any faulting or differential movement of recent times.
>
> 5. No evidence of beds of conglomerate, gravels or unconsolidated sands in the proposed site."

A second investigation and report on the geology of the dam and reservoir site was completed in 1945 by Mr. Frank J. Baudino, geologist for the Department of Water and Power. The Engineering Board of Inquiry has been unable to locate copies of Mr. Baudino's geologic report. However, during an interview by the Board, Mr. Baudino recalled that in the conclusion to his report he had recommended against construction of a reservoir at this site because the foundation materials were soft and erodible.

On February 25, 1947, Mr. Chester Marliave, Consulting Geologist and member of the consulting board retained by the Los Angeles Department of Water and Power to review plans and feasibility of the proposed project, submitted a report in which he en-

[6] Wilson, R. R. "Geology of the Baldwin Hills Reservoir Site and Immediate Vicinity." Unpublished report, Los Angeles Department of Water and Power. 1949.

[7] Burris, M. D. "Geology of the Proposed Baldwin Hills Reservoir Site and Immediate Vicinity." Unpublished report, Los Angeles Department of Water and Power. February 1944.

dorsed findings set forth in Mr. Burris' report of February, 1944.[8] He reviewed the geologic history of the region and set forth conclusions and recommendations. In his observations he noted:

". . . The main Inglewood Fault was located in the field and found to be about 400 feet distant from the westerly edge of the proposed reservoir but no traces of it were observed to pass through the reservoir site.

"There may be some minor faulting across the hills transverse to the Inglewood Fault zone but none having any significance was observed passing through the reservoir location.

". . . Because of the proximity of the Inglewood fault it is to be expected that there would be some distortion in the structure of the surface sediments at the reservoir site.

"Precise leveling records show that the crest of the Baldwin Hills near the proposed reservoir site has been sinking at the rate of .01 to .03 feet per year in recent years. . . ."

Mr. Marliave concluded:

"The sediments on which the proposed reservoir will rest are rather pervious. They are poorly consolidated and some of the sand and gravel strata will permit percolation of water through them . . . Portions of the Baldwin Hills and of the adjacent areas within the Downey basin are known to be rising or subsiding with reference to sea level, and the block of ground on which the reservoir site is situated may rise or sink or even be tilted a small amount over a period of years. The tectonic forces causing such movements generally act very slowly and should create no hazard to the flexible type of construction provided for the proposed reservoir . . . The proposed design appears to be amply suitable to care for any obvious weaknesses in the foundations, but it would be well to follow the progress of the excavations closely for evidence of any geological weaknesses not yet disclosed."

The investigations of both Mr. Burris and Mr. Marliave were undertaken prior to stripping of the site. Consequently, it is to be expected that some of the geologic details disclosed in subsequent investigations would not be discernible at the time of their inspections.

The fourth and most detailed geologic report on the Baldwin Hills site was submitted in February, 1949, by Mr. R. R. Wilson, geologist for the Department of Water and Power.[6] This report contains detailed geologic mapping of the reservoir area compiled during construction, indicating the bedrock features after stripping. These data are supplemented by bedrock contour maps at a scale of 1 inch equals 40 feet.

[8] Marliave, C. "Memorandum on Geological Conditions Found at the Baldwin Hills Reservoir Site." From files of Los Angeles Department of Water and Power. February 25, 1947.

Physical Qualities of the Geologic Formations

The three formations upon which the dam and reservoir are constructed consist of sediments which are in part loose and in part moderately consolidated.

The uppermost formation, the Palos Verdes, is mostly poorly consolidated silts, sands, and gravels. It is highly erodible as evidenced by low density, lack of cementing media, and by sharp gullies where surface runoff has concentrated in streams. (Photos 19 and 20)

The Inglewood formation, which underlies the Palos Verdes, was well exposed in the breach created during failure. Here it consists mostly of nearly flat lying strata ranging in thickness from ½ inch to 2 feet. Some of these strata were found to consist of moderately hard sedimentary rock, but others are composed of loose, powdery sands and silts. The loose materials are incapable of resisting even mild erosive action.

The deepest formation in the foundation, the Pico, is generally denser and more massive than the younger overlying formations and more resistant to erosion.

With regard to the qualities of the formations, which were exposed during construction and subsequently covered, pertinent descriptions are excerpted from the report of the Department of Water and Power.[6]

"The formation (Palos Verdes) is loosely consolidated with the exception of a few beds of soft sandstone 3 to 6 inches thick in the upper part of the formation, exposed in the rim of the reservoir . . . The bottom and roughly the lower half of the sides of the reservoir, and the foundation of the dam down to Elevation 335± consist of the Inglewood formation. . . .

"The sediments consist of light to yellowish gray, and yellowish to reddish brown fine sands, silts, and clays interbedded. The silts often contain fine sand and clay and vary from fine sandy to clayey silts. Some of the thin silt beds are cemented by iron oxide. In the lower part of the Inglewood, exposed by excavation, there are occasional beds of limestone up to one inch thick and occasional beds of gray, fine sand containing limestone nodules up to 2 or 3 inches thick and about 6 inches long. The sand and silt beds vary in thickness from less than an inch to about 4 feet, while the clay beds are usually less than 2 inches thick. . . .

"The Pico sediments are remarkably homogeneous with the exception of an occasional small lens of sand and gravel. They consist chiefly of dusky blue and gray to dark gray silt with varying proportions of very fine sand and clay, and may be called fine sandy clayey silt. There are occasional beds of fine sandy silt up to 4 feet thick and of silty clay up to 1½ feet thick. Now and then the silt beds are separated by a thin (often paper-thin) bed of light gray very fine sand . . . There are occasional small limestone nodules and spherical or tubular limonitic concretions. Some carbonaceous material is present in the form of small fragments, and streaks of black to dark bluish gray finely divided material.

"The Pico sediments are consolidated. In fact some of the beds are so well consolidated that they may be partially cemented. These beds would be more appropriately described as soft siltstone rather than just silt. . . ."

Faults

Most of the known faults in the reservoir were disclosed at the time of construction and were subsequently covered by embankment. (Photo 21) Pertinent observations noted when the faults were visible are quoted below[6]: (Survey stationing is referenced to the 12-inch tile drain as shown on Plate 4.)

"A few faults . . . which did not show at the ground surface, have been uncovered by excavation in the dam and reservoir sites. Most of them are normal faults trending approximately north-south and dipping westerly from 55° to 89° with an average of about 70°. They contain gouge which is usually a moist to very moist reddish brown clay, the color being due to iron oxide. The gouge is usually ⅛″ or less thick. A maximum thickness of 4″ was found along a short section of Fault I. The gouge contains fresh slickensides which indicate, in the opinion of the writer, that these faults are probably still active.

"The most important of the faults are Fault I and Fault II. They may branch off of the Inglewood fault at depth. Fault I has been traced from the southeastern corner of the reservoir to the north side of the reservoir and along the east abutment of the dam to about Station 5+50. There is a fault-zone along the trend produced of Fault I from Station 5+50 to 7+50.

"At elevation 446′, Fault I was about 9′ east of the center of the original gate tower site. It is a normal fault dipping westerly at an average angle of 75° to 80°. The trend varies from N. 8° W. to N. 9° E. The faulted zone is 2 to 3 feet wide. It was possible to measure the vertical displacement at three points on the fault as follows:

1. Near original gate tower site—13 feet.
2. About 200 feet northerly from original gate tower site—16½ feet.
3. North side of reservoir—26 feet.

It has not been found possible to measure the amount of horizontal displacement. At one place on the fault plane, located about 55′ northerly from the original gate tower site, horizontal striae were found showing that the last movement at this location was entirely horizontal with no vertical displacement. . . .

"Gouge, slickensides, and cementation of the silts and fine sands are prominent features along Fault I. The gouge is a reddish-brown, moist clay, usually about ⅛″ thick; however, beginning at a point about 100 feet northerly from the original gate tower site there is a zone over 100 feet long where the gouge is 2 to 4 inches thick. . . . The slickensides appear to be very fresh, indicating that the fault is probably still active.

* * * * *

"When it was discovered that Fault I came near the center of the gate-tower site, an east-west cut was made with carryalls across and normal to the strike of Fault I at the gate tower site. Within a few days the cut was about 20′ deep with steep banks. The southern bank (on which the center of the gate tower site was located) was about 150′ and the northern bank about 100′ long. The western end of the northern bank was located at Fault I. From this point, northerly for a distance of over 200′, excavation by carryalls along Fault I created a bank several feet high. Fine exposures of the formations were made by these excavations so it was possible to make detailed transverse cross-sectional studies. . . .

"The subgrade elevation of the gate tower foundation is 389′. Judging from the dips taken on Fault I, it was estimated that the fault plane would come within a foot or two of the center of the gate tower site at elevation 389. Since Fault I is possibly still active and since the reservoir is located in a seismic area, a decision was made to move the location of the center of the gate tower 48′ easterly along the inlet and outlet lines. This new location should place the entire foundation of the gate tower east of Fault I and also Fault II at subgrade elevation 389′.

"Fault II has been traced from the south bank of the east-west cut near the original gate tower site northerly for a distance of about 250′. . . . The average strike and dip is N. 14° E., 80° northwest. In the south bank of the cut, the elevations of the Palos Verdes-Inglewood contact were taken on both sides of the fault. The vertical displacement amounted to 17′ . . . Thin layers of gouge, some ferruginous cementation of the formation, and slickensides occur on Fault II but are not as prominent as similar features on Fault I.

"There is a zone of jointing and minor faulting west of Fault I, from one to two hundred feet wide. Some of the joints contain thin layers of reddish-brown to dark gray, clayey gouge. Most of them can be traced for only a few feet, and the amount of movement on the few faults where it could be measured was usually less than 2 or 3 inches.

"A few minor faults were observed in the east abutment between Stations 6+00 and 7+50. They were more numerous on the uppermost (392′± elev.) berm, being the nearest to Fault I. On January 17, 1948, immediately after the completion of the 392′ berm, a few minor faults, containing very little gouge, were found on this berm between Stations 6+50 and 7+00. Some of them could be traced for 30′ or more. They were striking N. 38° to 43° E. and dipping 40° to 46° S.E. (into the abutment). The horizontal displacement on one of these faults was 6 feet. As the faults were exposed on a horizontal plane only, it was not possible to determine whether they are normal or reverse."

Joints

Several joints at the site which were exposed during construction are now obscured by embankment. The following therefore is quoted from descriptions made prior to completion of the reservoir.[6]

"Joints are common in the damsite. . . . They occur in both Pico and Inglewood formations, but are more common in the older and better consolidated Pico silts. Most of the joints appear to be tension joints but a few shear joints were also found. The joints, with the exception of some observed in the east abutment, do not seem to fit any particular pattern, as for example that of the strain-ellipsoid, using the trends of the Inglewood fault and Fault I for orientation of the strain-ellipsoid.

"The tension joints vary in trend from N. 15° W. to N. 60° E. with an average trend of about N. 15° E. The dips range from about 30 degrees to vertical with most of them being steeper than 55 degrees. No particular direction is favored by the dips although on the east abutment there are more westerly dipping joint-planes, while on the west abutment there are more easterly dipping joint planes. This is due to the fact that the joint-planes dipping away from the abutment are more easily exposed, so they appear to be more numerous than those dipping into the abutment. Most of the joints are rather tight. About 20% are slightly open and a few have opposite planes from ⅛ inch to ¼ inch apart.

"The open joints are usually coated with iron and/or manganese oxide, and frequently contain gypsum. The gypsum occurs in two forms, either as a white, fine granular, powdery coating . . . or as a seam of ordinary crystalline gypsum averaging about ⅛ inch and rarely exceeding ½ inch in thickness. Some of the seams, usually the thicker ones, have been partially leached by percolating waters.

"Between Manholes 'A' and 'B' the trend of the ravine in which the dam is located is N. 17° E. and it appears to have been controlled by jointing, as the average trend of the joints is about N. 15° E.

"Two areas were observed in which there are a few compression or shear joints, one located in the west abutment between Stations 9+50 and 10+10, the other in the east abutment between Stations 5+50 and 7+50. The joint planes show signs of movement such as grooves, striations, and weak slickensides, so technically they are faults, but the amount of movement, when it could be measured, was seldom more than 2″ or 3″, therefore they are classified as joints rather than faults.

"The shear joints in the west abutment strike N. 40° to 50° E., and dip 40° to 55° southeasterly. The shear joint at Station 9+55 contains grooves which dip 17° in the direction N. 65° E. At Station 10+05 another grooved shear joint was noted, with striae dipping 10° in the direction N. 70° E. . . .

"The shear joints located in the east abutment between Stations 5+50 and 7+50 are in a zone west of Fault I produced northward. They vary in strike from N. 11° W. to N. 72° E., with an average strike of about N. 35° E. Most of the dips are steeper than 45°. The direction of the dip is variable being into as well as away from the east abutment.

* * * * *

"A 4′ x 8′ test pit, 17′ deep, was dug in the east abutment on the 392′ berm opposite Sta. 6+65. Previously, 4 vertical holes, V-1 to V-4, were drilled at this location for the purpose of intersecting a horizontal hole, H-2. All four of these holes lost their return water at about 12.4′ depth. Upon completion of drilling, the test pit was dug in order to determine the cause of the drill-water loss. Several joints and a few faults were exposed in the walls of the test-pit. Below the 10′ depth some of the joints were slightly open (about ⅛″) and along a few of these were zones up to 4″ wide of broken, very permeable silt which must have absorbed the drill-water rapidly. All the faults were normal, and none had a max. total displacement of more than 1.5′."

INGLEWOOD OIL FIELD

Inglewood Oil Field occupies the westerly part of Baldwin Hills, covering 1,180 acres and including 611 producing wells. The field adjoins the reservoir on the south and west, the nearest production being from three wells located within 700 feet of the south rim. The approximate limits of production are shown on Plate 8.

Inglewood fault divides the field into two distinct components known as the east and west pools. There is no interconnection between these pools, and they are in effect two separate oil fields. Baldwin Hills Reservoir adjoins the east pool.

Oil and gas are produced from nine separate zones which range from about 950 to over 10,000 feet in depth. The most productive is the Vickers zone with 294 producing wells, of which 86 lie east of Inglewood fault in the east pool. With exception of the Investment zone, which is a minor producer, the Vickers is the shallowest producing zone. The oil wells nearest the reservoir enter the Vickers zone at an elevation between 900 and 1,000 feet below sea level.

The Vickers zone is present beneath Baldwin Hills Reservoir, but it reportedly contains no producible oil in this area. The northern boundary of the oil pool was located roughly 200 feet south of the reservoir at the time of discovery of the oil field, and this boundary has shifted to the south farther from the reservoir since that time.[9]

Historical Development

Inglewood Oil Field was discovered in September, 1924, by Standard Oil Company of California at Los Angeles Investment Well No. 1-1 and was rapidly developed by Standard and other producers. When the field was only 20 months old it had produced approximately 25,500,000 barrels of oil, equivalent to a yield of 29,145 barrels per acre from 875 acres and 156 producing wells.

Development continued in a vertical direction with the lowermost producing horizon discovered by the R. R. Bush Oil Company at Sentous No. 1. This well was completed in the Middle Miocene in September, 1940. The deepest well is Standard Oil Company of California Baldwin-Cienega No. 105. (Plate 8) Total depth of this well was 12,276 feet, and it bottomed in a schist—possibly Jurassic in age.

Wells were drilled using very close spacing (commonly two-acre spacing, while the industry average in California is about 10 acres). This should mean uniform drainage throughout the oil producing horizons and fairly uniform fluid pressures.

Many of the early wells had initial production rates in excess of 1,000 barrels of oil per day.[10] It was not uncommon for a well, drilled on two-acre spacing, to produce 100 to 200 barrels of oil per day despite large cumulative withdrawals from nearby older wells averaging 25 barrels per day. Exploitation wells are still being drilled, and the State Division of Oil and Gas reports that additional wells are extending the producing limits of the field beyond those presently established.

[9] Horn, A. J., Petroleum Engineer, Standard Oil Company of California. Interview with Engineering Board of Inquiry. February 5, 1964.

[10] Oefelein, F. H. and Walker, J. W. "Field Case History—Vickers East Zone Water Flood, Inglewood Field, California." Preprint of the Society of Petroleum Engineers AIME Paper SPE 699. October 24, 1963.

Cumulative production totals as of September, 1963, were:[11]

Oil	239,145,000 barrels, or 31,000 acre-feet
Water (gross)	364,017,000 barrels, or 47,000 acre-feet
Injection water	83,099,000 barrels, or 11,000 acre-feet
Water (net)	280,918,000 barrels, or 36,000 acre-feet
Gas (approx.)	164 billion cubic feet

Therefore, a net of 67,000 acre-feet of oil and water has been taken from below approximately 1,000 surface acres. Production by years is shown on Plate 9.

Structure

The Inglewood Field is an elongated faulted anticline. It is one of fourteen oil fields located along the Newport-Inglewood uplift. Other important fields on this structure, listed in north to south order, are Rosecrans, Dominguez, Long Beach, Seal Beach, and Huntington Beach.

The interior of the Inglewood Oil Field is intricately faulted generally parallel to the main Inglewood northwest-southeast trend. This structure is shown in cross section on Plate 10. Certain faults which have been mapped, oriented more north-south, tend to have vertical displacements of 40 to 80 feet. One such fault of particular interest has been mapped by Standard Oil of California trending north into the Baldwin Hills Reservoir area. This was designated by the Company as the "Reservoir Fault," and its position was fixed by electric log correlations, bore hole pressure information, and barriers discovered during the injection program.[9] It should be noted that the subsurface locations of the reservoir fault and of other faults shown are to some extent a matter of interpretation and that other competent geologists might produce different and equally plausible versions from the same basic data. The inclination of the fault plane as shown on the geologic section is about 10 degrees less than can be measured on the surface exposure of Fault I. Nevertheless, it seems likely that the "Reservoir Fault" is the subsurface projection of Fault I.

Stratigraphy

A stratigraphic column for Inglewood Field is contained in Table III-2. Comment here will be limited to the highly productive Vickers oil zone, which consists of interbedded soft shale and unconsolidated sand. These sands are described as fine to coarse grained and generally silty and friable.[10] Cores show alternating shale and sand intervals commonly 1 to 5 feet thick with few continuous shale intervals greater than 20 feet thick. Average porosity for the zone is reported as 35 percent and the maximum net sand as 43.4 percent.

An examination of electric logs indicates a lithology essentially similar to the above for the remainder of the section down to the basement schist.

[11] California State Division of Oil and Gas. Material from files. 1964.

Oil Reservoir Characteristics

There are three common types of recovery mechanisms in oil fields: Water, gas cap, and solution gas. Of these, solution gas is considered to be the least efficient because production results in a rapid bottom hole pressure drop. With natural water drive, on the other hand, there may be no significant bottom hole pressure decline through the life of the field. Gas cap drive has never been significantly active in Inglewood Oil Field.

The recovery mechanism in Vickers east pool prior to water flooding was solution gas drive with a limited water drive.[10] This means that the oil, for the most part, is propelled out of the reservoir sand by dissolved gas which tends to come out of solution when pressure is released by a bore hole.

The original reservoir fluid pressure in Vickers east pool (1,330 feet subsea) was estimated by Standard Oil of California at 575 pounds per square inch absolute (psia). This is the point shown for December, 1924, on the pressure graph depicted on Plate 9. The same level in 1954 prior to the water flood measured about 50 psia. Vickers east pressures were raised by water injection to about 225 psia by the end of 1963. However, a small section of the Vickers east pool reportedly had a natural water drive and, accordingly, no significant drop in bottom hole pressures.[9] This natural water drive block is bounded by the "Reservoir Fault," the next fault to the west, and the main Inglewood trend southwest of the Baldwin Hills Reservoir. Pressures from this water drive section were excluded from the Standard Oil Company's computations for the pressure-elevation graph, Plate 9.

Reportedly, sand production has never been a problem at Inglewood Oil Field.[9]

TABLE III-2

**STRATIGRAPHIC COLUMN
INGLEWOOD OIL FIELD**

Series	Formation		Zone	Average thickness	Remarks
Recent	Alluvium		———	25'–100'	No production
Pleistocene	San Pedro		———	150'–200'	No production
Pliocene	Pico	Upper	———	100'–300'	No production
		Middle	Investment	200'–800'	200' East block 800' West block
		Lower	Vickers	1700'	1200' East block
	Repetto	Upper	Rindge	1000'	———
		Middle	U. Rubel	300'	———
			L. Rubel	700'	———
		Lower	U. Moynier	400'	———
			L. Moynier	700'	———
Miocene	Delmontian		Bradna	650'–1800'±	400' nonproductive between P/M and Bradna
	Mohnian	Middle	City of Inglewood	0'–250'	———
		Lower	Nodular Shale	150'	Approximately top 400' productive
	Luisian		Sentous	1700'	
	Relizian		Topanga	1500'±	No production
Jurassic?	Schist		———	?	No production

From—Oefelein, F. H. and Walker, J. W. "Field Case History—Vickers East Zone Water Flood, Inglewood Field, California." Preprint of the Society of Petroleum Engineers AIME Paper SPE 699. October 24, 1963.

CHAPTER IV

PROJECT PLANNING AND DESIGN

PLANNING OF PROJECT

Water System Requirements

A study[1] by the Department of Water and Power in the early 1940's revealed that the Los Angeles water system was becoming inadequate to supply peak demands. It was concluded that construction of Baldwin Hills Reservoir, along with other system improvements, would double the capacity of the distribution system and would provide satisfactory service to an additional 750,000 persons in the growing southern and western parts of Los Angeles.

It was estimated that the Baldwin Hills Reservoir would make possible the delivery of 67 cfs additional mean annual flow to the service area. The inlet capacity required at Baldwin Hills for the future maximum 24 hours would be 85 cfs, and the outlet capacity 144 cfs.

Based upon the studies of the water system requirements, it was concluded that the Baldwin Hills Reservoir project would be the most desirable plan of development. It would require the least modification of existing facilities to provide the desired service.

Plan for Development

The Engineering Board of Inquiry was unable to secure definitive information on studies made of alternative storage sites. It appears that there were no desirable alternatives. The topography of Baldwin Hills, offering the logical location having sufficient elevation near the area to be served by the reservoir, determined the location of the site. The reservoir would be built in the first ravine west of La Brea Avenue at the north flank of Baldwin Hills. The plan provided for Baldwin Hills Reservoir to be supplied from Lower Franklin Reservoir, which is located to the north across Ballona Gap. (Plate 1)

The minimum storage requirements set for Baldwin Hills Reservoir were to provide approximately 200 acre-feet for peaking demands. However, it was determined that appreciably more storage could be developed economically, providing close-in storage in the system as a protection against outages. It was therefore decided that the reservoir with the greater storage should be constructed. The reservoir capacity would be 897 acre-feet with a water surface area of 19.57 acres at the spillway lip, Elevation 477.50.

The reservoir bowl would be excavated into the head of the ravine. To provide a uniform reservoir floor, part of the subgrade would have to be constructed of compacted earth material. There would be a number of small embankments on the perimeter of the reservoir, and a major dam on the north side which would be approximately 155 feet high at the axis. The embankments would be built from the materials excavated for the reservoir bowl. Select materials would be reserved by stockpiling for the impervious compacted earth lining as the essential feature in controlling seepage from the reservoir.

The site of the proposed reservoir was very close to the Inglewood fault, regarded as one of the most active faults in California. For this reason, it was decided that the dam and appurtenances should be designed to withstand a seismic acceleration of 0.2 gravity in a horizontal direction.

The general plan is shown on Plate 2.

Exploration and Testing

The low density and the susceptibility of the foundation materials to water erosion were recognized early by the engineers and geologists who were exploring the site. The site was first explored in the early 1940's. Much of the more detailed foundation exploration for the main dam was made after the construction of the project was under way. The earliest soils exploration, other than for general surface reconnaissance, was in March, 1941. At the extreme south end of the reservoir five 4-inch auger borings were put down. (Plate 11) The borings were made with a regular post-hole auger with necessary pipe extensions to allow penetration to a maximum depth of 24 feet. The purpose was to classify the soils, to obtain logs of the borings, and to take selected penetration needle readings on remolded specimens at each 2 feet of depth.

In March, April, and May of 1941, 14 test shafts were dug with the primary purpose of measuring the in-place density of the foundation soils. These shafts were located at the south end of the reservoir site and were carried to a maximum of 13 feet in depth. Laboratory testing was conducted to determine the compaction characteristics, percolation-settlement rates, and penetration needle readings. In April, 1941, in order to penetrate considerably more depth, five borings were made, using a truck-mounted auger. With this machine it was possible to drill to a maximum depth of 60 feet. The resultant data were assembled in the form of logs for each boring, with sam-

[1] Los Angeles Department of Water and Power "Preliminary Report on the Function and Operation of the Proposed Baldwin Hills Reservoir", unpublished report. March, 1943.

ples obtained for each 5 feet of depth. Laboratory testing determined compaction characteristics for each sample and for various composites which were prepared in the laboratory, as well as the accompanying percolation-settlement rates, field moisture content, specific gravity, and gradation.

This preliminary work determined the characteristics of the surface or near-surface materials, not only as a foundation for the reservoir but also for the support of the main dam and various other fills. Equally important was determination of the characteristics of the material which would be excavated in forming the reservoir bowl and which would be used in the main dam, in the peripheral fills, and in the reservoir lining. Representative test data are presented in Table IV-1.

In 1943 additional borings were made to obtain more complete information on foundation soils and fill materials as well as to extend the exploration farther to the north into the area which is now occupied by the main dam. This additional exploration consisted generally of extending the 4-inch auger boring system into a grid approximately 200 feet on centers. These borings were carried to a maximum depth of 13 feet, with composite samples being obtained at 5-foot intervals. Extensive laboratory compaction, percolation-settlement, and shear tests were made.

Accompanying this sampling and boring investigation, the Department of Water and Power continued with surface reconnaissance of the area. In the early files it was reported that the entire area was deeply incised. It was further noted that the ravines followed planes of structural weakness created by diastrophism. The entire area was covered with a mantle of well weathered top soil. Low-density sand lenses were discovered in a number of the borings, indicating that provisions for drainage should be considered in design of the reservoir bowl to preclude the saturation of the foundation by percolating water.

Testing of the soils indicated that, with rare exception, all of the material to be excavated could be used for an earthfill dam with conservative side slopes extending to a height of 200 feet. It was expected also that, by proper mixing and blending, any of the materials to be encountered in the excavation could be used for fill purposes. In the planning studies, however, it was contemplated that blending would not be necessary, as there appeared to be more than enough material available for use in a stable dam section and an impervious reservoir lining. It was recognized that a final answer to this problem would not be found until the area of the reservoir was stripped during construction.

From these preliminary investigations, it appeared feasible to design and construct the necessary compacted earth embankments and a compacted earth reservoir lining with the materials required to be excavated for the reservoir bowl. Of the greatest concern was the necessity for protecting the foundation from saturation and from erosion due to percolating water.

Surveying

The records show that the first surveying at the reservoir site by the Department of Water and Power was done in 1939. Horizontal and vertical control points were established, and topographic maps of the area were made.

The problem of earth movement was suspected at that time. The surveyors were reported to have had difficulty maintaining stable horizontal and vertical control stations. The surveys were referenced to the triangulation network established by the Survey Division of the Los Angeles County Engineer's office. This was a first order triangulation survey network accepted by the U. S. Coast and Geodetic Survey.

DESIGN OF FACILITIES

No formal design report presenting assumptions, criteria, and reasoning involved in the design has been located. The following comments have been assembled through an exhaustive review of the design files.

Foundation

Preliminary investigation had disclosed that the foundation consisted of sediments which were loosely

TABLE IV-1

REPRESENTATIVE PRECONSTRUCTION LABORATORY DATA

Type of test	Unit	Number of tests	Average	Maximum	Minimum
Field Moisture [a]	percent	51	17.1	36.2	6.3
Field Density [a]	pcf	51	105.3	120.7	81.3
Percolation-Settlement [b]					
Percolation Rate	ft/yr	47	1.99	46.2	0.0006
Consolidated Dry Weight	pcf	47	118.0	128.8	86.5
Shear [c]					
Cohesion	psi	17	7.8	15.5	5.6
Friction Angle	degrees	17	26° 45′	48° 15′	9° 30′
Coefficient of Friction		17	0.503	1.112	0.167

[a] Data obtained from 14 shafts (4 to 13 feet deep).
[b] Data obtained from compacted samples from entire exploration.
[c] Data obtained from selected compacted samples.

compacted and susceptible to densification and erosion by running water. The bottom of the reservoir would be in the Inglewood formation, which consisted of thinly bedded and horizontally stratified lenses of loose and alternately harder sands and silty sands.

The existence of faults in the foundation area was known. The amount of future activity was anticipated to be minor. Special precautions were taken in the design of some of the appurtenant structures because of the faulting.

Most of the foundation for the main dam would be on the Pico formation, which was known to be considerably more massive and more competent than the overlying Inglewood although still leaving much to be desired. There was evidence of old slides at the east abutment downstream of the main dam. This area proved to be unstable during removal of the toe of the slope during construction.

Exploration and Testing

Exploration extended from early in 1941 into the construction period, reflecting delays due to the war years as well as the care which went into the field investigation and the design. Tabulations were completed of all the soil testing which had been accomplished in the 1941 and 1943 investigations. Representative values were obtained from these investigations for shear strength and percolation-settlement rates. Compaction results are given in Table IV-1. The location of the preconstruction exploration is shown on Plate 11.

It has been reported that water was introduced into the bottom of some of the auger holes and shafts for a qualitative type of in-situ percolation and subsidence testing. The water disappeared very rapidly, causing compaction and resulting subsidence. The tests rather dramatically demonstrated the unstable nature of the loose, fine, silty, and sandy foundation materials when saturated. It was stated in the Water and Power files:

> ". . . it should be remembered that very little percolating water now reaches the low density sand lenses underlying a portion of the reservoir area, and the leakage rate should be held to a low figure."

While not a normal part of the design investigation, a considerable amount of core drilling was accomplished from May, 1947, through the end of that year. Additional drilling was done during 1948. This work included core borings which were drilled into the foundation of the main dam. These holes were utilized as drains for intercepting seepage. Cores from many of the holes were selected and tested in the laboratory. Locations of these bore holes are shown on Plate 11 and pertinent data are presented in Tables IV-2 and IV-3. All borings were logged, backfilled with coarse sand or pea gravel, and connected into the foundation underdrain system. Some of these borings also functioned for instrumentation.

TABLE IV-2

SUMMARY OF PRECONSTRUCTION EXPLORATION

Type	Exploration size inches	Number	Depth: Maximum feet	Depth: Minimum feet	Depth: Total feet	Purpose	Dates accomplished
Hand Auger	4	40	45	10	452	Shallow foundation; sources of materials for embankment and/or lining; compaction characteristics.	March-May 1941 May 1943
Power Auger	16	5	53	40	213	Foundation; sources of material for embankment and/or lining; samples for compaction studies.	April 1941
Shafts	[a]	14	13	4	111	Shallow foundation and sources of borrow materials determination of in-place density, moisture content, and needle readings; samples for compaction studies.	March-May 1941
Rotary [b]	2½ (1⅞ core)	2	130	90	220	Deep foundation for drainage wells. Some cores taken for laboratory study.	May-July 1947
	4 (3 core)	1	----------	----------	35	Hole drilled to intersect horizontal Hole H-2; backfilled with coarse sand.	May 1948
Cable Tool [c]	6 (2¼ core)	20	153	101	2,800	Drainage wells in dam foundation	May-Dec. 1947
Horizontal [d]	4 (3 core)	3	115	90	305	Logs and 3-inch cores were taken. Holes were backfilled with sand and were used as foundation drains.	April-June 1948
Horizontal Rotary (auger) [d]	5	10	257	10	2,235	No cores taken; holes were backfilled with pea gravel and utilized as drains discharging into Manholes A and B.	Jan.-Nov. 1949

[a] Minimum size to permit hand excavation.
[b] Designation: C Series
[c] Designation: D Series
[d] Designation: H Series

TABLE IV-3

SELECTED TABULATED DATA FROM CORE BORINGS

Boring					Shear strength									
					Cohesion psf			Angle of friction degrees				Tensile strength psi		
Number	Size, inches	Core size, inches	Depth, feet	No. of tests	Average	Maximum	Minimum	Average	Maximum	Minimum	No. of tests	Average	Maximum	Minimum
C-1	—	1⅞	130	42	1,088	2,200	360	46	82	0	—	—	—	—
C-2	—	1⅞	90	25	950	1,630	308	41	63	0	1	3.72	—	—
Manhole A	—	—	Subgrade	4	1,540	2,840	1,020	39	64	0	—	—	—	—
D-1	6	2¼	152.5	1	880	—	—	38	—	—	—	—	—	—
West Abutment	—	—	Subgrade	2	1,380	1,440	1,320	62	67	57	—	—	—	—
East Abutment	—	—	Subgrade	1	1,740	—	—	36	—	—	—	—	—	—
D-3	6	2¼	101	2	1,405	1,560	1,250	19	31	8	1	14.7	—	—
D-4	6	2¼	101	2	597	720	475	30	61	0	1	3.13	—	—
D-5	6	2¼	151	3	760	1,370	450	47	67	15	1	1.85	—	—
D-6	6	2¼	101.5	2	1,295	1,470	1,120	34	45	24	2	3.1	4.8	1.4
D-7	6	2¼	151	3	1,003	1,200	820	56	63	51	1	5.3	—	—
D-8	6	2¼	101	2	1,238	1,267	1,210	45	65	26	2	5.7	5.7	5.6
D-9	6	2¼	152	3	1,080	1,300	720	63	72	57	—	—	—	—
D-10	6	2¼	151	3	1,107	1,510	850	44	57	32	—	—	—	—
D-11	6	2¼	151	3	1,700	1,860	1,410	53	62	44	—	—	—	—
D-12	6	2¼	151	3	1,743	2,710	1,180	13	39	0	—	—	—	—
D-13	6	2¼	151	3	470	790	144	53	71	19	—	—	—	—
D-14	6	2¼	153	3	1,310	1,580	590	28	65	28	—	—	—	—
D-15	6	2¼	152	3	1,600	1,760	1,300	0	7	0	—	—	—	—
D-16	6	2¼	151	3	1,500	1,930	1,220	18	54	0	—	—	—	—
D-17	6	2¼	152	3	820	1,200	500	55	69	37	—	—	—	—
D-18	6	2¼	152	3	1,150	1,960	620	44	68	2	—	—	—	—
D-19	6	2¼	152	3	1,550	1,900	1,220	38	62	10	—	—	—	—
D-20	6	2¼	152	2	1,340	1,500	720	40	78	41	—	—	—	—

Embankment

Besides the main dam embankment at the north end of the reservoir there were five minor embankments required to close low areas in the reservoir perimeter. The embankments were designed to be built of the material removed during excavation of the reservoir bowl. As a result of the extended exploration and testing program, it was determined that the material would require the addition of moisture and adequate blending for use in the dam. It would be placed at a density consistent with the objective of obtaining sufficient strength, impermeability, and minimum compressibility. The embankment design required an impervious reservoir lining and an underdrainage system which would ensure that the phreatic line would not rise into the embankments to impair their safety. Stability studies were performed in the conventional way by use of the sliding circle method of analysis.

Reservoir Lining

The reservoir lining design selected was a sandwich type of construction to inhibit the passage of seepage water and to collect and drain away all leakage which got through the relatively impervious compacted earth lining. (Plate 12)

The uppermost member was to be a 3-inch thickness of porous asphaltic paving designed to prevent wave wash on the underlying earth lining, to facilitate cleaning of the reservoir, and to prevent aquatic weed growth. Immediately below the paving was to be the main impervious member, a compacted earth lining, which was designed to be 10 feet thick in the bottom of the reservoir, graduating to 5 feet in thickness normal to the slope at the top. Below the compacted earth lining was a 4-inch cemented pea-gravel drain designed to act as a collector for all seepage finding its way through the compacted earth lining. A system of 4-inch clay tile pipes was to convey this water into the drainage inspection chamber. The pea-gravel drain was to be capped with a ¼-inch porous sand gunite layer to prevent infiltration of compacted earth.

The pea-gravel layer was cemented for stabilization. Consideration was given to the use of asphalt as an alternative binder for the pea gravel; but this was rejected, after tests were performed, because of the possibility that cold flow might result, leading to failure of the earth blanket on the slopes of the reservoir. By using portland cement, the designers expected that there would be no plane of weakness on the inside slope of the reservoir.

Immediately below the pea gravel was to be an asphaltic membrane about ¼ inch thick, applied directly on the foundation. In certain areas this membrane was to be reinforced with cloth fabric. The function of the membrane was to prevent all water from reaching the foundation. The security of the reservoir would be dependent upon the watertightness of the asphaltic membrane.

Drainage System

Two separate systems were designed to provide drainage: The foundation drainage system was to drain foundations under the earth embankments; the reservoir underdrainage system was to collect all seepage passing through the earth lining and to drain it from under the reservoir floor through a central observation and measurement station. The general arrangement of the drainage system is shown on Plate 4.

Each of the five smaller embankments was designed to have a separate drain system. These were to be 4-inch vitrified clay tile drains with open joints, bedded in concrete, with the upper half covered with cemented pea gravel to permit water entry.

In the bottom of the ravine containing the main embankment a 12-inch vitrified clay tile drain, also with open joints, was to be placed and bedded and covered in the same manner as the 4-inch drains. It was to pass through Manhole A where seepage could be observed. A number of foundation drainage wells were located along the side of the 12-inch tile drain. Horizontal holes were drilled into the abutments and filled with sand, along with other vertical holes into the foundation, all connected to 4-inch drain tile systems discharging into the 12-inch tile bottom drain. The 12-inch tile drain was to continue through Manhole B and finally to discharge into the city storm drain at Cloverdale Avenue, as shown on Plate 13.

Radiating from each of the manholes were other horizontal drill holes, backfilled with sand, which would pick up seepage from deep in the foundation and carry it to the manholes, where the discharges could be measured.

The reservoir underdrains were to be 4-inch vitrified clay tile pipes embedded in concrete and covered with cemented pea gravel to conduct reservoir bottom leakage to the drainage inspection chamber for measurement. Generally, the tile pipes were to be laid with open joints to admit reservoir seepage. There were to be a total of eight reservoir underdrain lines and a tower base drain to intercept all the water that seeped through the impervious earth lining.

The collecting underdrain lines were designated as the southeast toe, northeast toe, fault, south toe, south bottom, west toe, north bottom, and north toe drains. The latter fi~ were to enter the drainage inspection chamber as a cluster of five embedded in a single concrete encasement. The covering of the tile drains was to be an integral part of the pea-gravel drain completely underlying the reservoir. The concrete cradle, or bedding, was to be placed in a trench in the reservoir bottom on top of the continuous asphaltic membrane. (Plate 12)

The west toe drain was to be in the form of a "T" in plan, the east-west stem of the "T" being encased in concrete throughout its length. The stem encasement was to interrupt the pea-gravel drain, thereby dividing the reservoir bottom into approximately equal halves and separating the north and south bottom drains for seepage source identification.

Each of the reservoir bottom drains was to have its terminus at weir boxes in the drainage inspection chamber. At the chamber, seepage water was to be discharged through check valves into the 24-inch blowoff line used for draining the reservoir.

Water Control Facilities

Baldwin Hills Reservoir was designed to be served by mortar-lined steel inlet and outlet lines located in tunnels through the east side. The inlet line was to be 57 inches and the outlet line 66 inches, inside diameter. The lines were to be carried on ring girder supports through the tunnels, with a walkway passage beside each pipe, the inlet passage extending to the drainage inspection chamber. The inlet line was to be continuous through the gate tower, feeding two circulator lines leading to the west side of the reservoir after bifurcation at the base of the east slope. The inlet line would have a 36-inch roto valve just outside the tunnel portal, and a 32-inch Venturi tube immediately to the east for flow measurement.

It was considered prudent that inlet and outlet lines be supported on ring girders within the tunnels instead of being encased in concrete, so as to ensure against their being damaged by earth movement.

Discharge through the outlet line was to be controlled at the gate tower, with a 36-inch Venturi tube at the chlorinating station, about 70 feet outside the tunnel portal. Both lines would lead to La Brea Avenue, with the outlet line having a 42-inch manually controlled roto valve near La Brea.

The circulator pipes were to be supported on concrete piers across the reservoir. They were to be 39-inch reinforced concrete pipe in 8-foot sections with collared joints.

The gate tower was to be located at the east side of the reservoir, with gates at various levels for withdrawal of reservoir water through the tower into the 66-inch outlet line. There would be five sluice gates, each 2 feet 6 inches by 4 feet 0 inches, for that purpose. There would be a 24-inch gate valve at Elevation 411.5, and a square-frame sluice gate to the 24-inch blowoff line at the base of the tower. Each gate would have a trash screen. A water stage recorder would be located at the top of the tower with a stilling well attached to the inside of the tower. A footbridge would connect the tower to the east roadway.

The 24-inch blowoff line for reservoir draining purposes was to pass through the drainage inspection chamber to safeguard the pipe from possible damage where it crosses known faults in the foundation. Mechanical couplings on the blowoff line inside the chamber would provide flexibility. The blowoff line would continue through the end of the chamber, along the reservoir foundation, and across the east abutment of the main dam, ultimately discharging into the 42-inch spillway conduit at a junction box just below the 390 berm. It would be welded steel pipe encased in a concrete jacket with cutoff collars to prevent seepage along the line. It would have ½-inch mortar lining with coal tar undercoating.

The inlet and outlet tunnels would be 9 feet 8 inches in diameter with reinforced concrete linings. Their centerlines would be about 40 feet apart, with each tunnel being about 380 feet long.

Spillway

The spillway weir, at Elevation 477.5, would be of the side channel type with a crest length of 100 feet. The spillway discharge conduit would be a pipe laid approximately on the foundation line at the east abutment of the main dam, beginning with about 170 feet of 48-inch reinforced concrete pipe. The remainder would be 42-inch pipe. It would be equipped with air vents at critical locations. Four drop inlets for accepting surface drainage would be provided. The first drop inlet would be on the Elevation 479 roadway at the east end of the main dam; the second on the 340 berm; the third on the 250 berm; and the fourth downstream of the dam with a lip at Elevation 219. The spillway was designed to pass 181 cfs under a 0.62-foot head on the weir.

Instrumentation

The drainage inspection chamber was the focal point for the measurement of all seepage flows discharging from the reservoir underdrainage system. It is a rectangular structure with inside dimensions 6 feet 6 inches high by 5 feet 6 inches wide. It is about 80 feet long, beginning over the gate tower footing and extending westerly under the reservoir. It has a free joint where it leaves the top of the tower footing. From there it is a continuous concrete structure. The westerly end wall is at Station 0+98. The structure crosses Fault I at approximately Station 0+85. It has 16-inch side walls and 19-inch top and bottom slabs.

The drains enter the chamber at the locations diagrammatically shown on Plate 4. Weir boxes inside the chamber are arranged to provide for measuring individual flow from each drain line entering the chamber as well as total flow from the reservoir underdrainage system.

Foundation drainage into the 12-inch tile drain in the bottom of the ravine is measured at the base of Manhole A. The flow is then led to Manhole B by continuation of the 12-inch tile drain, where accretions to the flow between manholes are measured. There are six manometers in Manhole A and three in Manhole B for use in measuring foundation settlement at several locations along the 12-inch tile drain. Additionally, there is one copper tube entering each of these manholes for measuring foundation water levels in Wells C-1 and C-2. (Plate 13)

A deep bench mark, BHBM 126, is located at the base of Manhole A. It was established on top of a pipe stem, extending into a drill hole and embedded in concrete 152.2 feet below the manhole floor.

A test culvert for experimentation was included in the design, extending 120 feet to the northeast from Manhole A at Elevation 310. Its purpose was to determine structural strain produced by the overlying embankment. Also, pressure cells were located in the outside walls of the structure to measure earth pressures as the fill was constructed around and above the culvert.

CHAPTER V

CONSTRUCTION

The first construction work for Baldwin Hills Dam and Reservoir was started by Department of Water and Power construction forces on January 13, 1947. The work was completed by contract and water was turned into the reservoir at a dedication ceremony on April 18, 1951.

The plans and specifications for Baldwin Hills Dam and Reservoir were prepared by the Water System Division of the Department of Water and Power, City of Los Angeles, and approved by the Chief Engineer of Water Works and Deputy General Manager of the Water System. Plates 2, 4, 12, and 13 show the plan and details of this facility.

Application No. 6-40, for the approval of plans and specifications for construction of Baldwin Hills Reservoir, was filed with the State Supervision of Dam Safety Office on July 17, 1944. The application was approved on June 17, 1946. Periodic inspections were made during construction. An Order Authorizing Use of the Dam was issued on September 26, 1950. After submittal of final cost data and as-built drawings, the State issued a Certificate of Approval on May 17, 1956.

FORCE ACCOUNT WORK

Work done by force account (by Department of Water and Power construction forces) was as follows: Excavation of foundations and stockpiling of excavated material, including the rough excavation of the reservoir bowl; construction of the main dam and foundation drainage system, including the 12-inch drain and Manholes A and B; construction of the 42-inch spillway pipe and 24-inch blowoff line through the dam and outfall to discharge into the city storm drain; and construction of miscellaneous appurtenances.

CONTRACT WORK

Principal contract work was done under Specification No. 6912 by M. L. Kemper Construction Company. This contract included completion of the main dam and the peripheral Fills 1, 2, 3, 4, and 5 (east access road); reservoir foundation rough excavation and trimming; construction of reservoir underdrainage system (asphaltic membrane, cemented pea gravel, and tile drains); compacted earth lining over entire reservoir floor and slopes; spillway; gate tower; drainage inspection chamber; circulator lines; inlet and outlet tunnels; paved roadways and paved reservoir surface on floor and side slopes. Work started July 19, 1949, and was completed December 28, 1950.

Contract No. 7276 with Humiston and Rosendahl Company for construction of the steel inlet and outlet pipelines was started January 4, 1951, and completed May 7, 1951.

CONSTRUCTION SUPERVISION

All work was thoroughly inspected daily by the forces of the Field Engineering Division, Department of Water and Power. Written daily inspection reports were made. The engineering forces under the Field Engineer consisted of as many as 15 inspectors, two survey parties, and two laboratory technicians at the job site. A field laboratory was established at the site to provide close control of daily earthwork operations. Final test results were completed at the permanent laboratory of Water and Power.

EARTHWORK AND DRAINAGE

Earthwork Control

Earthwork was accomplished following the fundamental principles of construction control formulated by Mr. R. R. Proctor in his capacity as Field Engineer in the Department of Water and Power. These principles, first described by Proctor in 1933,[1] are the basis for modern earthwork control methods utilized in this country and throughout the world today. However, there are some differences in the way the Department of Water and Power applied these principles and the way today's accepted standards are written. Fundamental principles of the close relationship between moisture content of the soil at placement and the compacted density in achieving a desired strength and impermeability in compacted soils are implicit in all systems, as are the effects of varying compactive effort on final density. The system utilized by Proctor involved the concept of the "Indicated Saturated Penetration Resistance," defined as ". . . . The saturated penetration resistance of a soil as determined by the compacted penetration resistance of the soil at the same moisture content that is required to saturate it. . . ."[2]

In applying this method, the indicated saturated penetration resistance (I.S.P.R.) is determined from the laboratory compaction test, the results of which

[1] Engineering News Record. "Fundamental Principles of Soil Compaction." August 31, September 7, September 21, and September 28, 1933; a series of four articles.

[2] Proctor, R. R. "Laboratory Soil Compaction Methods, Penetration Resistance Measurements, and the Indicated Saturated Penetration Resistance." Volume V, pp. 242-247 Incl. Proceedings Second International Conference on Soil Mechanics and Foundation Engineering. Rotterdam. June 21-30, 1948.

are expressed as curves of dry weight, plasticity needle penetration resistance, and zero air voids all plotted against moisture content.

As stated by Proctor:[1]

"... The ultimate capacity for water absorption in this soil, when compacted at various moisture contents and by different methods can be determined ... by noting the moisture content at the intersection of the zero air-voids curve and the dry weight of the compacted soil."

In applying the Proctor system:

"... Preliminary investigations are made to determine the minimum required soil density, which is expressed in terms of the indicated saturated penetration resistance for use in field control. Soil density tests of the work are tabulated in terms of the indicated saturated penetration resistance. Percolation, consolidation, and shear tests are performed periodically on soils removed from the work by placing them in the test apparatus at the mean indicated saturated penetration resistance of the work. Field density tests of compacted or natural soils determine their indicated saturated penetration resistance, from which the anticipated settlement under loading can be determined. ..."[2]

In contrast with the Proctor approach, most agencies today control required compaction by specifying a percentage relative compaction, which is a percentage expressing the ratio of the required field density to the maximum achievable density as obtained in the laboratory test.[3]

The Department of Water and Power provided a field laboratory equipped to control daily earthwork operations. The field laboratory was responsible for making daily routine compaction tests of materials from excavation, from stockpiles, or from borrow. Moisture content was checked by penetration needle readings of materials before they were placed on the fill and of materials on the fill just before they were rolled. Completeness of compaction after rolling was checked both by the needle and by in-place density tests of the compacted fill.

Close support was given to the field organization by the central laboratory of Water and Power. Tests were run in the central laboratory on remolded samples obtained by the field laboratory. Selected samples were subjected to tests for direct shear, percolation, and settlement, in addition to the routine tests conducted on all fill control samples. Test procedures which are different from those utilized by other agencies are described elsewhere.[1, 4]

[3] United States Department of the Interior, Bureau of Reclamation. "Earth Manual." 1st Edition. July 1963.

In its review of the project the consulting board retained by Water and Power, in a report dated February 25, 1947, stated:

"Because of the strategic location of the site and the known seismic activity of the region, the Board recommends that there be complete field control of the embankment material and that the quality and supervision of all the work be of the same high standard that has been maintained by the Department on similar structures in the past.

"With the above considerations, your Board approves the feasibility of the proposed Baldwin Hills Project, the suitability of the site, and the plans of design."

Dam and Embankments

Stripping operations (removal and disposal of unsuitable grass, sod, roots, brush, and other objectionable material) commenced January 13, 1947, at the lower end of the ravine about 175 feet south of Sanchez Avenue and continued south to encompass the entire reservoir area. Waste material was placed in disposal areas, the largest of which was on the lower part of the gully below Fill 4, southwesterly of the reservoir. This work was started with Department forces.

Similarly the construction of the dam and reservoir started at the lower end and proceeded up the ravine. Pipelines, catch basins, and manholes were constructed as work progressed. Excavated material was placed in stockpiles for later use, directly in compacted pipe backfill, or directly into embankment.

The excavation proceeded into the reservoir bowl while waiting for construction of Manholes A and B and completion of the 12-inch drain pipeline. The excavated material was carefully selected. Sufficient (approximately 350,000 cubic yards) material suitable for the compacted earth lining was placed in special stockpiles. Excavated material not suitable for earth lining but suitable for compacted embankment for the dam and peripheral fills was placed either in stockpiles or directly into embankments.

All loose material was removed from the foundation of the dam to a depth necessary to obtain relatively tight material. In excavating the abutment foundations, the work was carried out in a series of level benches with rather steep slopes ($\frac{1}{2}$ to 1) between benches. These excavated benches ran about at right angles to the axis of the main dam. No cutoff trench parallel to the axis of the dam was excavated either in the bottom of the foundation or in the abutments; nor was there any deep foundation treatment. Satisfactory foundation material would ideally consist of

[4] Proctor, R. R. "Construction and Operation Details for a Simple Machine to Test Soils in Double Shear." Volume VII, pp. 61-64. Proceedings Second International Conference on Soil Mechanics and Foundation Engineering. Rotterdam. June 21-30, 1948.

firm ground free of all loose material and as dense as, or better than, could be obtained by removal and replacement with compacted embankment using the materials available. Quoting Proctor:

"... The depth to which foundation soils should be stripped is that to which the dry weight in place is the same as required if the soil were to be placed in the fill. If the foundation soils are too deep for this to be accomplished, the cross-section of the dam should be carefully designed to meet the conditions of stability that are thus imposed. The compacted fill should then be placed at the same saturated plasticity as the foundation in order to result in a fill of sufficient plasticity to conform to such settlement as will take place in the foundation. The careful use of the principles that have been set forth should permit the design and construction of compacted earth dams of known safety at locations that have been considered unsuitable for this purpose." [1]

During excavation of the east abutment, a substantial slide occurred opposite Station 8+25 (12-inch drain stationing). This slide was first noted on January 5, 1948, as a slight movement on a clay bed exposed near the toe of excavation. Most of the movement that developed was between Station 7+00 and 9+10. This slide movement developed progressively and was measured at observation points at several locations. The slide continued for approximately two weeks with an average rate of movement of 0.03 foot per day. Maximum amount of movement recorded at Station 8+25 was 0.43 foot; the average total movement was 0.39 foot. (Photos 22 and 23) Wilson noted that this movement occurred for the most part in a 3- to 4-inch thick bed of moist brown clay. He concluded that the causes were: [5]

1. A lubricated slip bed which was moist enough for motion but was not saturated,
2. The presence of a synclinal fold dipping into the excavation, and
3. Excavation at the toe of slope which removed the support for the hillside mass.

Remedial measures included partial removal of the slide and installation of permanent drainage consisting of hand auger holes drilled horizontally approximately 50 feet into the abutment, packed with sand and connected into the permanent tile drain system which was installed under the dam. The open crack at the top of the slide was filled by excavating a trench about 300 feet long, 1 to 2 feet deep, and 1 foot wide along the crack and backfilling with relatively impervious compacted backfill.

Excavation at the east abutment and at the gate tower foundation exposed Fault I and auxiliary faults extending through the original tower site and through the east abutment nearly in a north-south direction. After having inspected the site the consulting board reported on January 20, 1948: [6]

"... It is very unlikely that any appreciable movement will again occur along these auxiliary faults... (The hoard recommended moving the tower so that) the tower will be founded on a mass of undisturbed material that is not traversed or undercut by this particular fault."

As a result of these exposures the tower location was moved easterly about 48 feet. No indication was found in the records that any special exploration of the fault zone below subgrade was performed either at the east abutment or in the reservoir.

All wet spots in the dam foundation and in the east and west abutments caused by ground water were drained by the foundation drainage system installed as shown on Plate 4. This system consists of numerous vertical and horizontal drill holes packed with pea gravel and local areas covered with cemented pea gravel and drained with 4-inch clay tile pipe. These drains discharge into Manholes A or B. Some of the detail used is illustrated in Photos 24 and 25.

The placing of compacted earth material in the dam embankment started at the downstream toe and proceeded in horizontal lifts. The material was either hauled directly from excavation or from temporary stockpiles set aside for this purpose during excavation. Moisture was added in most cases at the excavation site. The material was loaded and hauled by scrapers, mostly of 15 cubic yards capacity.

The material was spread on the fill in layers of 9-inch thickness. Water was then added, if necessary. The entire layer and the top few inches of the previous layer were then mixed with a Seaman or a Wood's pulvimixer, pulled by a tractor (Photo 26). Generally one trip with the pulvimixer was all that was necessary to obtain the required uniform moisture content. However, if the material was still too dry, the process was repeated. Close control was kept of this operation.

After adequate moisture conditioning, compaction of the embankment was accomplished with sheepsfoot rollers drawn by tractors. Sixteen roller passes were made over most of the layers in order to obtain the proper compaction.

Three rollers were used for most of this work. One roller was built to the U. S. Bureau of Reclamation specifications. It consisted of two drums, each measuring 5 feet long and 5 feet in diameter, outside measurements. Each drum had six rows of teeth, with 20 teeth in a row. The area of the tooth pad was 7 square inches, and the length of the tooth was 9 inches. The

[5] Wilson, R. R. "Geology of the Baldwin Hills Reservoir Site." Unpublished report of Los Angeles Department Of Water and Power. February 1949.

[6] Consulting Board: H. A. Van Norman, Consulting Engineer; Charles T. Leeds, Consulting Engineer; and Chester Marliave, Consulting Geologist. Letter report to Mr. Samuel B. Morris, General Manager and Chief Engineer, Los Angeles Department of Water and Power. January 20, 1948.

drums were loaded with sand and water so that the total weight of the roller was 42,700 pounds. Two other rollers, built to Water and Power specifications, were used. Each roller had two drums, each 5 feet long and 4 feet in diameter, outside measurement. Each drum had six rows of teeth with 13 in a row. The total weight of the roller loaded with sand and water was 41,600 pounds.

Approximately 1,281,000 cubic yards of compacted embankment material was placed in the main dam. An effort was made to place the materials in the dam embankment so that the most impervious materials, containing more silts and clays, would be near the reservoir side of the dam, while the sand and more pervious material would be near the downstream side. Control data as recorded in the final construction report are summarized in Table V-1.

Special attention was given to the selection and placing of the more silty and clayey materials directly on the foundation at abutments so as to obtain maximum bond. All loose dry material was removed from the abutments prior to placing properly conditioned fill material. Compaction adjacent to the vertical faces at the abutment was by hand pneumatic methods. The sheepsfoot rollers were operated close to the nearly vertical faces, and the number of roller passes was nearly doubled at the abutments to ensure that adequate compaction was obtained. (Photos 27, 28, and 29)

After the dam embankment reached the elevation of subgrade of the reservoir, sufficient overfilling was placed on the interior face of the dam so that at least 1 foot of compacted fill existed outside of the final trim line, as measured normal to the slope. This overfilled material was trimmed off by dozers pushing the excess to the top of the fill where it was reconditioned and mixed with the incoming new material.

As noted on Plate 2, there are five peripheral fills in small ravines on the east, south, and west sides of the reservoir. These fills, numbered counterclockwise starting at the northwest corner of the reservoir, were built up in essentially the same manner as the main dam. A clay tile pipe foundation drain was installed under each fill in the bottom of the arroyo. Total volume in these fills was approximately 275,000 cubic yards.

Reservoir Bowl

Rough excavation of the reservoir bowl left about 1 foot of original ground above the finished subgrade, and the overfilling during construction of compacted embankments left about the same amount to be trimmed to finish subgrade. This was true of both the sides and the bottom of the reservoir.

Final trimming of the reservoir interior subgrade was accomplished with a motor patrol grader and by hand. Starting at the top of the reservoir, the grader, bridled to a tractor equipped with a winch, was operated along the slope. The tractor traveled along on the roadway above at the same speed as the grader and by means of the cable and winch kept the grader from overturning. (Photo 30)

Material was peeled off in thin layers down the slope to where it was necessary to push it by bulldozer farther down the slope to where it could be loaded by scraper and transported to other fill areas. Any unevenness left after final trim by the motor patrol was smoothed off by hand, using a mattock, and finally by hand brooming to clean the slopes of all loose material. After cleanup of an area, and before the surface had time to dry out and crack, the asphaltic membrane was applied. This method of trimming and applying asphalt was carried on continuously around the reservoir from the top to the floor. As the slope became less steep near the bottom, it was not necessary to bridle the motor patrol to the winch above.

The reservoir bottom was trimmed to subgrade in much the same manner. Scrapers carried off most of the material, and final trimming was by motor patrol grader. Panel areas about 100 feet in width were trimmed and the asphaltic membrane applied.

The reservoir underdrainage tile lines, as shown on Plate 4, were placed in ditches that were excavated

TABLE V-1

CONTROL DATA FOR MAIN DAM AND FILLS

Item	Number of tests	Moisture content, percent	Dry density, pcf	Penetration needle		Equivalent compactive effort, ft.lbs./cu.ft.
				At placement, psi	At Saturation (I.S.P.R.), [a] psi	
Main Dam						
Embankment	559	15.2	111.6	1,242	246	15,070
Foundation	81	19.1	99.0	1,559	136	
Hand Tamp	7	19.3	102.6	800	174	
Fills 1 to 5						
Embankment	118	13.9	111.5	1,192	174	16,774
Foundation	16	16.3	98.6	2,366	88	9,500

[a] Minimum requirement 200 psi.

below the final reservoir subgrade. These ditches were rough graded by motor patrol grader and then finished by hand. After final trimming the ditch surfaces were covered with the asphaltic membrane.

The subgrade of the reservoir bowl after trimming consisted partly of excavated natural ground and partly of compacted fill. Due to the variation of materials in the reservoir foundation, with some areas entirely of sands, and others of loose or well consolidated silts, it was necessary to use different materials to stabilize the subgrade. Considerable experimentation was done to determine the best methods and procedures. It was decided to use various grades of slow curing asphalts as penetrating and primer coats. The number of coats applied would depend on the porosity of the soil, with the more porous soils being treated with lighter asphalt to obtain greater penetration. Then the heavier asphalts were applied to build up and stabilize the area. On areas that were well consolidated, only one penetration coat was used.

After the subgrade was well stabilized, a rather heavy first coat of 40-50 grade penetration asphalt was applied. As soon as this coat had cooled, a second coat was applied. This built up the membrane thickness to about ¼ inch.

Experimental test panels placed on the reservoir slope indicated the need for reinforcement of the asphaltic membrane to avoid heel damage from men walking on the slope. A cotton cloth reinforcing with a special open weave was tried and proved successful. This fabric was placed between the first and second coats of asphalt. The second coat thoroughly penetrated the fabric to form a reinforced membrane also about ¼ inch thick. This reinforcing was used mostly on the slope where the foundation was especially sandy and loose and on the bottom and sides of all ditches for the drainage tile. The cotton fabric was woven in rolls 5 feet in width with 40 strands per inch longitudinally (warp) and 25 strands per inch transversely (fill). (Photos 31 and 32) All of the asphalt materials were applied at 300° to 425° F. by hand-held pressure nozzles, connected by hose to hot tank delivery trucks. The cloth was applied by brooming and nailing with eight-penny nails onto the first coat of 40-50 asphalt. Total quantities of material used for the membrane were approximately 1,100 tons of asphalt and 300,000 square feet of fabric.

Examination of the daily inspectors' reports indicates that, although the placing of this very important asphaltic membrane was a difficult operation, it was conscientiously carried out and rigidly inspected.

On the floor of the reservoir, after the final application of the membrane, a thin layer of loose pea gravel was applied. As the weather was rather cold (November-December), this pea gravel was rolled into the asphalt with a light flat-wheeled roller. The purpose of the rolled pea gravel was to form a firm bond between the membrane and the 4-inch cemented pea-gravel layer. (Photo 33)

The pea gravel for the reservoir underdrain was mixed with cement, water, and calcium chloride using a small portable batch plant and concrete mixer. Specified cement content was 1 sack of cement per cubic yard. After mixing, the cemented pea gravel was hauled in small dump trucks to the reservoir floor, where it was dumped into a widened laydown concrete bucket. A crane picked up this bucket, and the material was spread from the bucket in a wide semicircle. It was then further spread and leveled by hand, using shovels and rakes, to obtain a thickness of 4 inches. This method was used to cover the entire reservoir floor. (Photo 34)

The cemented pea gravel was placed on the slopes using a contractor-built spreader which was mounted on the frame of an old trenching machine, using its tracks and power assembly. This spreader had a receiving hopper installed on the front end so the light trucks delivering pea gravel from the mixing plant could dump directly into the hopper. The spreader was designed to distribute the cemented pea gravel on the slope through a strike-off hopper. Moving at 8 feet per minute, the spreader placed a 4-inch layer in an 8-foot-wide course. Traveling on the compacted earth lining as it was built around the sides, the machine laid a course of cemented pea gravel on the slope. This method was used to cover the entire reservoir slope. (Photo 35)

After the cemented pea gravel had set up, the surface was sealed with a ¼-inch layer of sand gunite. The gunite was to serve as a filter by allowing the passage of water but preventing infiltration of soil particles from the compacted earth lining.

As shown on Plate 4, a system of 4-inch tile drains was laid in shallow trenches in the reservoir floor. The drain lines that were designed to collect water were embedded in concrete up to the spring line, with open joints above the concrete. In cases where the drain was not designed to pick up seepage, the tile was totally encased in concrete. After the concrete set up, cemented pea gravel was laid around and over the pipe to be flush with the 4-inch layer of cemented pea gravel in the reservoir floor. (Photos 36 and 37) Approximate quantities of materials used in the drainage system were 8,600 linear feet of 4-inch tile, 370 cubic yards of concrete, 18,600 cubic yards of cemented pea gravel, and 75 cubic yards of gunite.

After the cemented pea-gravel drain was completed in the bottom of the reservoir, hauling and placing of the compacted earth lining from the southeast stockpile was started. This stockpile was premoistened and mixed with a power shovel raking the vertical face to obtain mixing. The mixed material was placed behind the shovel as it worked into the stockpile.

The mixed material was loaded and hauled by scrapers of 15 cubic yards capacity pulled by rubbertired prime movers. The first material placed on the cemented pea gravel was spread in a single 18-inch layer so as to avoid damage to the pea gravel. This first lift was smoothed with a motor grader and then rolled with sheepsfoot rollers using about double the number of trips required for the normal 9-inch layer. (Photos 38, 39, and 40)

After the first 18-inch layer was compacted, the following layers were placed in loose 9-inch layers which compacted to about 6 inches. The methods used were, in general, the same as those used in compacting earth material in the dam embankment. After spreading the incoming material, water was added where necessary, and then mixing was done with a pulvimixer. After the material was uniformly mixed, it was rolled with a sheepsfoot roller. Generally 20 trips of the roller were required to ensure adequate compaction.

The backfill material around concrete structures such as gate tower, drainage inspection chamber, and spillway was carefully compacted. Small areas were compacted using a hand-operated pavement breaker weighing 90 pounds and fitted with a round metal plate 6 inches in diameter. In larger areas a pavement breaker fitted with a tamping shoe about 16 inches square was used.

After the compacted earth lining in the bottom of the reservoir was constructed to within 1 foot of finished grade, the inside slopes were started. The material was placed in the same manner in 9-inch loose layers, smoothed off, watered, mixed with a pulvimixer, and rolled. The slopes were overfilled at least 1 foot to ensure compaction of the material out to the finished slope. The material from the overfilling was later trimmed off downward in the same manner as was used for final trimming of the subgrade for the asphaltic membrane. This material was then reconditioned and used to complete the reservoir bottom to finish grade. (Photo 41)

A total of 177,000 cubic yards of impervious earth material was compacted in the reservoir floor lining and 149,000 cubic yards was compacted in the side slope lining. The material was a silty clay. Average densities obtained for the different areas are contained in Table V-2.

Minimum requirement for the compacted earth lining was an indicated saturated penetration resistance of 300 psi. Actually the minimum requirement was not always met, but it does not appear that this was critical. The fundamental requirement for the earth lining was impermeability.

The thorough detail involved in controlling the earthwork is evidenced by the numerous compaction cylinder tests made either in the field or central laboratory.

Following completion of the compacted earth lining to grade, the surface was trimmed and a 3-inch aspahaltic pavement was laid over the entire inside surface of the reservoir. (Photo 42)

TABLE V-2

CONTROL DATA FOR COMPACTED EARTH IN RESERVOIR

Item	Number of tests	Moisture content, percent	Dry density, pcf	Penetration needle		Equivalent compactive effort, ft.lbs./cu.ft.
				At placement, psi	At Saturation (I.S.P.R.),[a] psi	
Bottom lining	66	13.1	113.5	1,344	180	16,636
Slope lining	94	14.1	114.5	1,384	354	21,238
Reservoir subgrade	11	17.2	98.2	2,093	117	10,790
Gate tower subgrade	9	13.8	113.9	1,081	244	18,000
Gate tower hand tamp	5	15.0	114.2	950	271	20,000

[a] Minimum requirement 300 psi.

APPURTENANT FACILITIES

Gate Tower

A circular reinforced concrete gate tower was constructed at the east side of the reservoir. The tower has a total height of 106 feet 2 inches, and it is supported by a circular reinforced concrete base 47 feet in diameter and 7 feet 2 inches thick. The barrel of the tower has an inside diameter of 14 feet with sidewalls varying in thickness from 4 feet 4 inches at the bottom to 20 inches at the top.

The excavation was carried down on a 1:1 slope to an elevation 2 inches below foundation grade. After careful investigation of the foundation bearing values to ensure adequate stability, a 2-inch pad of sand-cement mortar was placed on the foundation surface so as to have a clean and firm hase on which to erect reinforcing steel and to support the forms. A total of 466 cubic yards of concrete was placed in the base. Average compressive strength in 28 days, as determined from test cylinders, was 3,460 psi.

After the base of the tower was complete, the various structures resting on the base were formed and concrete was placed. These included the inlet line, the outlet line, the walkway for access from the inlet tunnel to the drainage inspection chamber, a section of the drainage inspection chamber, the 24-inch blowoff line, the tower base drain, and the first 16-foot pour of the tower itself. Because of the changed location of the gate tower, requiring excavation into the east slope, it was necessary to design and construct concrete approach channels leading from the surface of the finished slope into the tower gates at Elevations 411.5 and 418.0. These channels were constructed on compacted backfill and butted against the tower without watertight seals.

Drainage Inspection Chamber

The excavation for the drainage inspection chamber was made at the same time as the excavation for the base of the gate tower, and construction proceeded concurrently with that of the tower. The chamber is located as shown on Plate 4.

A 2-inch pad of sand-cement mortar was placed on the foundation grade of the chamber to support the reinforcing steel and the forms. Outside forms were built, followed by erection of the reinforcing steel for the floor and sidewalls. The concrete floor was then placed and the 24-inch blowoff pipe was installed. The inside wall and roof slab forms were erected next, and the concrete was placed in the sidewalls. After a 3-hour interval to allow for settlement, the roof slab was placed. The weir boxes, check valves, and drain lines were installed later. (Photo 43)

The necessary drain lines, cemented pea gravel, and asphaltic membrane were carefully installed as the backfill was brought up around these structures.

Inlet and Outlet Tunnels

The material encountered in driving the tunnels was mostly of a consolidated, laminated, silty nature, with thin layers of sand and clay. Excavation was by hand, using an air spade. The tunnel cars were loaded with a mucking machine and some hand shoveling. Metal liner plates were installed as the heading progressed. The tunnels were lined with concrete at the rate of about one 30-foot placement per day, using a collapsible form which rode on the tunnel cars. After the tunnels were lined, the concrete invert was placed. About five months after placing the invert, the tunnel was grouted through grout pipes using a mix of 1 sack of bentonite, 11 sacks of cement, and 110 gallons of water.

Blowoff and Spillway

The 24-inch blowoff line was laid in the foundation under the reservoir and crossed the east abutment of the main dam. The pipe was placed in a trench 3 feet 6 inches wide and of varying depths. A mat of ½-inch reinforcing bars was placed above and below the pipe, and circumferential ½-inch reinforcing bars were used. The pipe was placed on 8-inch concrete blocks, and the entire trench surrounding the pipe was filled with concrete to about 10 inches above the top of the pipe. Concrete cutoff collars were provided at 30-foot intervals along the pipe. Connection to inspection chamber and junction with spillway pipe are shown on Photos 44 and 45.

The compacted earthfill was built to about 1 foot above the lower reservoir roadway. The overfill was then removed to approximately final grade of the roadway, Elevation 479. A small trenching machine was used to cut a vertical bank that was to be the outside form for the rear wall of the concrete spillway structure. This vertical back face was protected from raveling with the application of a thin layer, ¼ to ½ inch, of gunite. The concrete on the bottom and backside of this structure was placed directly against the ground. The front of the structure was formed and, after placement of concrete, was backfilled with selected and compacted earthfill. The transition section from the spillway to the 48-inch pipeline was formed and concrete placed at the same time as the spillway.

The spillway pipe was laid in compacted fill of the main dam, close to the east abutment, and continued down the arroyo to connect with a city storm drain. The pipe was cradled in concrete with concrete cutoff walls approximately 25 feet apart. The compacted backfill about the pipe was carefully selected and thoroughly compacted to form an integral part of the main dam embankment.

Chlorinating Station

The chlorinating station was constructed just easterly of the inlet and outlet tunnel portals at the east access road. This work was done under contract by the M. F. Kemper Construction Company. Work was started March 26, 1951, and was completed August 2, 1951.

Parapet

A concrete parapet wall was placed between the Elevation 479 roadway and the top of the inside slopes of the reservoir. After completion of the fill for the Elevation 479 roadway and the trimming of the reservoir lining to subgrade of the paving, a small trenching machine was used to make the excavation for the wall footing. The parapet wall was constructed in alternate sections 21 feet long.

Roads

There were two roadways built around the perimeter of the reservoir; the inner road at Elevation 479 and the outside public road at Elevation 482.

Another road was constructed from the Elevation 482 roadway at the northwest corner of the reservoir down the north face of the main dam to and including the Elevation 250 berm.

An access roadway was also built between La Brea Avenue and the east side of the reservoir. This road follows the same gully as the inlet and outlet lines and joins the Elevation 482 roadway about 80 feet north of the southeast curve.

Fences

A galvanized chain link fence was constructed on the reservoir side of the Elevation 482 roadway to prevent public access to the reservoir. This fence was 6 feet high with 14-inch extension arms carrying 3 strands of barbed wire.

Cast Iron Water Line

In 1958 a 12-inch water line was constructed from the water tank just south of the reservoir, around the south end and west side of the reservoir, to supply the residential area northwest of the reservoir. This line was constructed with cast iron pipe using a bell-and-spigot rubber-seal joint.

A panoramic view of the nearly completed reservoir is shown in Photo 46.

CHAPTER VI

OPERATION, MAINTENANCE, AND SURVEILLANCE

The Baldwin Hills Reservoir was in service continuously from July, 1951, until failure on December 14, 1963, with the excepton of a short period in 1957, when it was drained for cleaning and repairs. During more than 12 years of operation, apparently no difficulties were experienced that interfered with the satisfactory performance of the reservoir.

The reservoir was kept under strict surveillance by means of a complex system of safeguards. Daily inspection was made by the caretaker of the seepage from drain networks underlying the entire reservoir and its embankments. Monthly surveys were conducted to detect movements at the reservoir and in the surrounding area. A squad of maintenance specialists inspected the reservoir once each month, on the alert for all factors related to the safety of the facility. Instrumentation at the site, such as strain gages, seismoscopes, and tiltmeters, was carefully planned and closely watched.

The State Supervision of Dam Safety Office made annual maintenance inspections of the dam during the period of operation. The last inspection was made on April 3, 1963.

SYSTEM OPERATION

Much of the water supply for the City of Los Angeles is conveyed from distant sources, the principal importations being from the Owens Valley via the city's own Los Angeles Aqueduct and from the Colorado River via the Colorado River Aqueduct of the Metropolitan Water District of Southern California. These waters, along with supplies developed locally, are distributed through an extensive system of pipelines and storage facilities to service areas ranging greatly in elevation.

The operation of the water system of the City of Los Angeles is divided by the Department of Water and Power into five districts to provide decentralized operation. Each of these districts covers a large geographical area. The general functions of a district and the operating characteristics of the system are made known generally by oral instructions to the district operations personnel.

The water system is operated so that service is provided from close-in storage to satisfy the peaking summer demand. Service areas are supplied either by the close-in reservoirs or by regulators from the large reservoirs in the service zone.

RESERVOIR OPERATION AND MAINTENANCE

The Baldwin Hills Reservoir was designed to "ride" on the system, responding automatically to fluctuations of demand in the service areas. Generally water would enter the reservoir at a uniform rate. Metropolitan Water District water is brought into the area in the summer on a "block loading" basis, that is, at a relatively continuous rate. The major demands in the southwest part of the City of Los Angeles would be met by outflow from the reservoir.

In the winter the demand in the area is considerably reduced, and the drawdown of the reservoir was, therefore, less than in summer. Normally the reservoir level would fluctuate through only the upper few feet of storage. The balance was reserve storage available for use in the event of any failure of the trunk lines or any other unusual operating condition. The Baldwin Hills Reservoir was regarded as an extremely valuable reservoir for that purpose, since it provided close-in storage for the entire service area as far south as the Los Angeles International Airport.

Normal Operation

Operation of the reservoir involved controlling the inflow to meet a 24-hour cycle in withdrawing and recharging the reservoir. It was large enough to meet the peak 24-hour demand on the system without any unusual operating condition. The daily variation in reservoir stage would be as much as five feet. Operating personnel were familiar with the control, which had to be set daily to meet the demands. Apparently there was nothing unusual in the hydraulic operation of this reservoir. This was regarded as a reliable reservoir, relatively simple to operate.

Emergency Operation

Specific detailed written instructions were not issued for action in the event of emergency at the Baldwin Hills Reservoir. Each reservoir caretaker of Water and Power is orally instructed to report any unusual occurrence at the facility for which he is responsible. In addition, all men in the Western District were expected to report to their headquarters upon learning of any emergency.

There were three methods by which water could be drained from the reservoir. Operations personnel were intimately familiar with all of them. Apparently it was not anticipated that an emergency could develop which would require the use of all three of these means at once, as was necessary on December 14, 1963.

The principal method of draining the reservoir was by reversing the flow through the 57-inch inlet pipe. This line was equipped with a large-capacity Howell-Bunger valve near the intersection of Rodeo Road and La Brea Avenue. By this means it was possible to "blowoff" the main supply line from the reservoir through the valve, wasting the water into the Ballona Creek channel by way of the local storm drain system.

A second means of draining the reservoir was to adjust the total system so that the highest demand was placed upon Baldwin Hills Reservoir to meet the needs of the service area. In this way, maximum discharge directly into the system was achieved.

Thirdly, emergency release could be made via the 24-inch blowoff line under the reservoir. This line joins the spillway pipe near the axis of the main dam, and flows are thus delivered into the storm drain north of the reservoir.

It has been reported that on December 14, 1963, the controlled maximum discharge from the reservoir under emergency conditions was about 450 cfs.

Even though all three methods of release were employed to the greatest possible extent, the storage volume was only fractionally reduced before failure occurred.

Reservoir Caretaker

A reservoir caretaker was stationed regularly at the Baldwin Hills Reservoir between the hours of 7:45 a.m. and 4:15 p.m. seven days a week. Between the hours of 4:15 p.m. and 7:45 a.m., the reservoir was not under surveillance. Each caretaker routinely had 10 successive days of duty, followed by four days relief. (Revere Wells had begun his duty tour on Wednesday, December 11, 1963, three days before the disaster.)

During the first few months of operation of the Baldwin Hills Reservoir two caretakers reportedly were kept on duty, one on the day shift and one on the swing shift. After the reservoir had been in operation for a period without difficulty, the swing shift was eliminated.

The duties of the caretaker included reading the inflow and outflow meters and the reservoir stage, telephoning these data to the Western District office, and inspecting the drainage inspection chamber under the reservoir. He was also charged with the responsibility of removal of foreign matter from the reservoir, maintenance of landscaped areas, operation of valves, maintaining drainage ditches, patrolling of the reservoir area to detect unsanitary conditions, operation of the tower gates, and general detailed surveillance. He kept a daily log of his activities, making particular note of anything extraordinary. A copy of this log was submitted weekly to the Western District headquarters.

There were two daily tasks that the caretaker performed with certainty: He inspected the drainage inspection chamber, and he made inside and outside patrols along the reservoir fence.

Although the detailed analysis of seepage records is made by engineering personnel, the caretakers at Water and Power facilities become familiar with the amount of seepage which is normal. In the event of unusual flows from the drainage system, they are expected immediately to call the District office and to notify the Foundations and Structures Maintenance Section of the Water Engineering Design Division.

The caretaker at Baldwin Hills was aware that normal discharge from the reservoir underdrain system would be clear water without silt. The presence of turbidity was to be reported immediately. Any change in the character of the drainage would be regarded as extremely significant.

It was not the specific function of the caretaker to measure differential movement in structures at the known fault zones. The Foundations and Structures Maintenance Section has the responsibility to maintain continuous observation of problem areas such as this at all of the reservoirs in the water system. The caretaker at Baldwin Hills, however, was familiar with the surveillance program at his reservoir and did, as a matter of interest, frequently observe known points of special concern. For example, strain-gage points were installed on both sides of several cracks in the drainage inspection chamber and around the perimeter of the reservoir. Periodic checks were made on these points. The reservoir keeper was familiar with this instrumentation and provided assistance by keeping a general watch on these areas during the course of his normal routine.

Similarly, in the event of an earthquake, he would make a general inspection, looking for anything unusual, particularly at the known points of possible movement. He understood that if an earthquake occurred, he was to make a special inspection of the drainage inspection chamber. He knew that there was a fault zone crossing the chamber. If he noticed any change in the condition of the drains or of the cracks in the structure, he was to notify his superiors.

SURVEILLANCE

Water and Power conducted a systematic and comprehensive inspection program to measure the performance of the Baldwin Hills Dam and Reservoir. The surveillance record is generally from 1948 until the time of failure.

Various measuring devices to detect structural strain or earth movement were installed at strategic locations and were carefully monitored. The reservoir and its environs were checked regularly to detect all movements.

Engineering Surveillance

The Foundations and Structures Maintenance Section routinely devoted one full day of each month to the inspection of Baldwin Hills Reservoir. These inspections followed a well established procedure. If need for maintenance was noted, it was brought to the attention of the reservoir keeper for action. This might involve such work as unplugging an obstructed drain or cleaning out the spillway. The regular inspection required a patrol of the surface drains along the abutments on both sides of the downstream face of the main dam. It also included patrolling the slopes, spillway, roadways, curbs, and the surrounding areas. Structural cracks were measured in the tunnels, drainage inspection chamber, curbs, and the spillway. Also, measurement was made of drain discharge.

During each inspection of the reservoir the engineering maintenance personnel made it a practice to inspect the immediate vicinity of the spillway because they knew that this was a potential danger zone. They were aware of the fault zone which existed in this area. Therefore they gave special attention to the area between the crest of the main dam and the catch basin near Manhole A. They made it a point to examine the hills in that immediate area looking for movement or cracks. Nothing significant was ever reported as a result of these patrols.

If there was an earth tremor of even slight magnitude in the vicinity, special inspections of the reservoir were made by the engineers.

The last regular inspection by the Foundations and Structures Maintenance Section prior to the failure was on November 26, 1963. Evidently none of the observations at that time were regarded with concern.

The drainage was inspected and measured by hydrographers of the Water Operating Division. It was also their practice to take samples of the seepage water for turbidity analysis. The activities of the hydrographers, during a typical weekly inspection at Baldwin Hills, included entering Manholes A and B and measuring the drainage from the horizontal drain holes. Also included in the inspection was an examination of the catch basins and the storm drains to make sure that they were unobstructed.

During each inspection the engineers and hydrographers carried field books containing records of all measurements taken at the reservoir for the previous two or three years. This enabled field comparison of new readings with the past measurements. The field men were held responsible for notifying their supervisors if any unusual change was noted.

In addition to the surveillance activities described above, the Foundations and Structures Maintenance Section also had responsibility for collecting and analyzing the seepage measurements made by the hydrographers. The movement and settlement data brought in each month were turned over to one man in the office whose exclusive function was plotting data. If this man noticed any change in the movement or settlement, flow of drains, strain-gage readings, or well-water elevations, he was obligated to bring such change to the attention of his supervisor.

Surveillance was also maintained over local drainage waters, particularly in the residential areas adjoining the reservoir. The investigation by the Engineering Board of Inquiry disclosed no reports of damage from such flows during the life of the reservoir.

Reservoir Underdrains

Records of flow from the drainage system under the reservoir lining were maintained continuously from the beginning of operation of the reservoir. Flows of the eight sectional drains that discharge into the drainage inspection chamber, as well as drainage from the tower base, could be measured by weirs in the chamber before discharge into the 24-inch blowoff line. (Photo 47)

In the early years of operation the drainage system under the reservoir required much maintenance. The records indicate that appreciable quantities of asphalt flowed through the system from the westerly underside of the reservoir. (Photo 48) Calcium carbonate deposits formed in the drains, requiring frequent attention by maintenance forces to prevent plugging.

The obstructions in some of the drain lines caused by clogging, and possibly to some extent by displacement, led to a reduction in the total seepage flow entering the drainage inspection chamber. Seepage collected at that point varied over the years of operation from a high of about 23 gallons per minute (gpm) to a low of approximately 7 gpm. During the later years of service, the flow from the reservoir drains tended to become more uniform. Beginning in the spring of 1963, however, there was a slight, but detectable and consistent, uptrend in the measured reservoir seepage.

Embankment Drains

Extensive provisions were made for drainage of the embankment foundations. The drains were observed and discharges measured monthly.

Throughout the project operation period no seepage was observed in the drains for Fills 1 to 5. It must therefore be assumed that these peripheral fills were never saturated.

Initially, there were three manholes along the 12-inch drain line which permitted measurement of seepage from the foundation. These were Manholes A and B and the junction manhole at Spillway Station 15+00. The manhole at Station 15+00 became inaccessible during 1954 because of real estate developments.

The 12-inch tile drain, with its connected drain holes, produced a nearly uniform discharge over the life of the project. From the time the reservoir went into service in 1951 until December of 1963, the flow varied almost lineally from about 15 to 10 gallons per hour (gph) as measured in Manhole A. The 10 gph rate persisted substantially constant over the last four years.

Records of Water and Power show that each of the horizontal drain holes radiating from Manhole A discharged essentially a constant 0.2 gph with the exception of H-7, which discharged approximately 0.4 gph with some variations virtually to the time of failure.

Three horizontal drains discharging into Manhole B, H-9, H-11, and H-12, were always dry. The fourth horizontal drain hole, H-10, discharged water at a variable rate from 0.09 to 0.14 gph throughout the period of observation with the exception of 1963. This flow increased slightly with time.

During the last year of reservoir operation, all of the discharging horizontal drains under the main dam experienced rapid variation whereby the drainage reduced to zero and in some cases increased to its former amount followed by continued fluctuations.

Flow from the 12-inch tile drain into Manhole B showed the same general characteristics as the flow into Manhole A, with a virtually constant differential of about 1.5 gph due to flow accretions between Manholes A and B.

The reservoir keeper has reported that he never observed any evidence of seeping water on the downstream face of the main dam, such as excessive growth of grass.

Observation Wells

Periodic inspections were made of observation wells at the perimeter of the reservoir to check for indications of water in the foundation or embankments. Reportedly there was never any water found in these wells.

Movement

A careful vigil was kept by Water and Power over the reservoir and the immediate vicinity in order to detect any movement which might endanger the facility. In addition to the regular patrols by the caretaker, monthly surveys and inspections were made by engineering personnel.

Measuring devices were installed at the reservoir and its appurtenances for the purpose of showing any strain developing in the structures or any earth movements. These included such instrumentation as strain gages, seismoscopes, tiltmeters, and foundation settlement measuring devices. Special attention was paid to any possible rupture of utilities in the vicinity. There was no reported leaking or cracking in the local sewer and water lines. Instrumentation is shown on Plate 5.

Surveys. The Water and Power surveyors made monthly checks for movement on the following:

1. A line 20 feet north of and parallel to the main dam axis.
2. A line along the 390 berm.
3. A line along the 340 berm.
4. The parapet wall.
5. The gate tower.
6. Lines along the inlet and outlet tunnels and in the drainage inspection chamber.

In addition, settlement checks were run on points located as follows:

1. A series of points extending from the northeast corner of the reservoir to north of the downstream toe of the main dam.
2. A series of points extending from the northeast corner of the reservoir, past the chlorinating station, to La Brea Avenue.
3. Points in Manhole A and the test culvert.
4. Points in Manhole B.
5. Piers of the elevated water tank south of the reservoir.
6. Numerous bench marks in the vicinity.

Strain Gages. Movements were monitored at strategic points by means of strain gages. Readings of these instruments were made several times each year at varying intervals.

Points of measurement included the cracks in the drainage inspection chamber, the free joints and cracks in the inlet and outlet tunnels, and the spillway structure and conduit.

Seismoscopes. In October, 1961, two strong-motion earthquake recorders were installed on the reservoir rim at the north and east sides. The instrument class used was the U.S. Coast and Geodetic Survey Seismoscope, also known as the Wilmot survey type earthquake recorder. It is the model developed by the California Institute of Technology and the Wilmot Engraving Company from the U.S.C. & G.S. prototype. The amplitude of a strong temblor is recorded by a device which scribes a mark on a smoked glass plate. These instruments never recorded movement at the site.

Tiltmeters. There were tiltmeters on the east side and the west side of the reservoir and on the 340 berm of the main dam. These tiltmeters were 2-inch pipes extending about 15 feet below the ground level inside a 6-inch casing so that the pipe was free to move. The tiltmeter was oriented in a north-south direction and readings made by means of a level bubble. Tiltmeters were read and results plotted on a monthly basis. No significant deformation was noted.

Foundation Settlement Measuring Devices. Monthly measurements were made of elevations at nine locations on the dam foundation adjacent to the 12-inch drain line. These observations were made by means of manometers in Manholes A and B. This instrumentation was designed to detect any settlement of the foundation along the 12-inch drain.

Operational Water

Water and Power maintained continuous records of the water levels in the reservoir from its initial filling in April, 1951, until the time of failure. These records were obtained by means of a water stage recorder located in the gate tower. Also, daily inflow and outflow records were maintained for the same period.

History of Operation, Maintenance, and Surveillance [1]

Events of significance in the life of the Baldwin Hills Reservoir include: (Plate 14)

On April 18, 1951, the reservoir dedication was held.

On May 4 and 5, 1951, dye tests were run on the drain system to trace the flow of seepage.

On May 5, 1951, the reservoir was drained because of an excessive amount of leakage. The asphaltic lining was observed to have buckled along the toe of the inside slope at the east side of the reservoir. Also, a ¾-inch settlement had occurred between the Elevation 418 channel inlet structure and the gate tower. (Photos 49 and 50)

In the period from May 14, 1951, to June 18, 1951, repairs were made. The roofing-paper gaskets in the joints between the channel inlet structures and the gate tower were removed and replaced with rubber gaskets. Holes were drilled through the compacted earth lining to the pea-gravel drain at the gate tower. Grout was poured into these holes in an effort to seal the drain at the tower to eliminate the leakage.

During June, 1951, the reservoir was refilled.

On June 19, 1951, due to variations in drainage measurements, a schedule of five readings per day was initiated for measuring the reservoir underdrainage. This was continued for about two years. Thereafter, measurements were made on a weekly basis.

Some of the reservoir underdrains were initially delivering substantial quantities of asphalt at a rate reported to be "approximately 5 gallons in 48 hours." This soon reduced to "5 gallons per month" and continued to decrease until by September 1, 1953, the total discharge of asphalt was "one gallon or less per month."

On June 19, 1951, as refilling of the reservoir was begun, the average total flow of seepage water from the underdrains was 0.03 gpm. By the end of June, 1951, the average total flow had increased to 2.98 gpm. On the last day of July, 1951, the flow had increased to about 21 gpm. By then traces of calcium carbonate had appeared in the discharge pipe of the southeast toe drain.

On July 2, 1951, the reservoir was placed into service.

During August, 1951, the southeast toe drain was discharging approximately 10 gpm out of a total flow of about 23 gpm. This drain had apparently developed a restriction causing a cross flow from the southeast drain into the fault drain through the pea gravel. The flow in the fault drain then increased to about 12 gpm, with a simultaneous reduction in the southeast toe drain discharge.

During September, 1951, calcium carbonate continued to deposit in the reservoir drains and weirs. The 6-inch check valves between the weirs and blowoff line required cleaning several times. During this month the fault drain discharged about 11 gpm of a total flow of approximately 21 gpm.

By October, 1951, the increased flow in the fault drain had developed a restriction of calcium carbonate, and drainage water was apparently caused to flow across into the south bottom drain.

A series of tests conducted by Water and Power verified that, when calcium carbonate deposits caused a restriction in the southeast toe drain, water would flow into the fault drain and, to a lesser extent, into the tower base drain. When the fault drain system became restricted, the drainage would then flow into the south bottom drain.

On October 11, 1951, the use of a hydrochloric acid solution was begun intermittently to unclog the drains. In the early stages of the operations, backflushing of the southeast toe drain with hydrochloric acid opened a passage through the pea-gravel drain between the southeast toe and the tower base drain. To remedy this, a "3.5-percent bentonite and sawdust" mixture was injected into the southeast toe drain and allowed to overflow into the tower base drain. As soon as the sawdust and bentonite appeared in the tower base drain discharge, the injection of the mixture was stopped. The discharge from the tower base drain then decreased to a flow considered normal.

Various procedures were used to keep the drainage system operable in the period from 1951 to 1953. Yet clogging of the drainage system remained a problem.

By the end of 1953, total measured reservoir seepage decreased to about 10 gpm from a flow of approximately 23 gpm in 1951.

On October 29, 1951, a crack was discovered in the drainage inspection chamber at Station 0+70.3. (Photo 51) It measured $\frac{3}{32}$ inch on the ceiling and a hairline on the floor. Strain-gage points were set across this crack and regular measurements were made

[1] Los Angeles Department of Water and Power. From detailed reports in files.

of the opening from the time of its inception. The crack was open enough during the years before failure that it was possible to see the steel reinforcing rods inside the wall. Leakage through the opening was reportedly negligible prior to the disaster.

During 1952 acid treatment of the drains under the reservoir was continued.

On December 21, 1952, there was a reported earthquake.

On December 22, 1952, cracks in the embankment of the main dam were repaired. The principal crack was located at the top of the north slope, about 1 foot north of and parallel to the north curb of the outside road, beginning 30 to 50 feet west of the spillway and having a length of about 90 feet. The crack was estimated to be ¼ to ½ inch wide and 18 inches deep. Repair was reportedly made by hand-tamping sand into the opening. At that time observation was also made of small surface slides on the slope below the crack.

Early in 1953 it was observed that a joint in the parapet wall at Station 9+14.4 at the east abutment of the main dam had opened about ¼ inch. (Photo 52)

On February 13, 1954, following a heavy rainfall, a minor slide developed on the downstream slope of the main dam just below the roadway.

In the period 1953 to 1956 intermittent treatment of drains under the reservoir continued.

In May, 1955, the opening of the parapet wall joint at Station 9+14.4 had increased to about ½ inch.

In November, 1955, a plumber's "snake" was passed through the 90-degree elbows in all the drain pipes near the drainage inspection chamber except in the northeast toe drain. Such mechanical cleaning, followed by pressure water flushing, apparently kept the drainage system fnnctioning.

In 1956 calcium carbonate deposits were continuing to accumulate in the northeast toe drain.

On August 7, 1956, a 6.5-inch hole was cut through the concrete wall of the drainage inspection chamber to eliminate two 90-degree elbows in the southeast toe drain. A 3-inch hole was cut into the top elbow in the drain, permitting insertion of a permanent 3-inch pipe to provide a straight access into the drain.

The five drains which enter the drainage inspection chamber near the westerly end (the north toe, the south toe, the north bottom, the south bottom, and the west toe drains) discharged a rather consistent total flow, although there was evidence of some cross flow among the drains. On August 8, 1956, plumbers inserted a snake into each of these five drains beyond the 90-degree elbows. This was followed by water flushings, and at that time there was no apparent cross flow, suggesting that a suspected separation in the pipes was closed.

On August 30, 1956, plumbers inserted a snake and a 2-inch spiral reamer into the southeast toe drain. They detected what they regarded as an obstruction 63 feet from the drainage inspection chamber, but they were able to work the entire snake into the drain, a distance of 190 feet. The equipment was then removed and a 3-inch spiral reamer inserted, but they were unable to pass the 63-foot point.

By October, 1956, the total measured seepage from the reservoir underdrains had decreased to approximately 7 gpm from about 23 gpm in 1951. This decrease was attributed to deposits of calcium carbonate and asphalt in the drainage system. The asphalt, however, probably did not cause appreciable stoppage, since it remained fluid.

On October 2, 1956, 207 feet of ¾-inch polyethylene plastic pipe was inserted into the southeast toe drain. The next day 24 gallons of hydrochloric solution was pumped through the plastic pipe into the sontheast toe drain. The acid was left in the drain for four hours, and then the drain was flushed with water. The return water carried a heavy concentration of calcium carbonate particles and some asphalt.

In February, 1957, the joint in the parapet wall at Station 9+14.4 was open ⅝ inch.

In the period March 13 to 16, 1957, the reservoir was emptied for cleaning and repairs. It is reported that corroded bolts on some of the slide gates in the gate tower were replaced at that time. While the reservoir was empty, the lining was completely cleaned and inspected. It was discovered that there had been some cracking in the thin cement coating which had been placed on the asphaltic paving. This coating averaged about ¼ inch in thickness. The paving was observed to have lost its cohesion in places to an average depth of about ½ inch.

The cracks found in the cement coating were judged by the inspectors to be caused by the creeping of the asphaltic paving on the slopes of the reservoir. Although no significant separation was found in the asphaltic paving, observation was made in some locations of an overthrust ranging up to 2 inches.

The bottom of the reservoir was found to be in good condition except for some general crazing and a few small soft spots. The records of Water and Power indicate that these spots were repaired.

During the shutdown precise levels were run on the circulator lines in the reservoir. It was revealed that settlement had occurred up to about 0.2 foot at the inlet structures of these two lines. There was also an unequal settlement of about 0.05 foot in the circulator transition at a distance of about 20 feet from the gate tower. There was some very minor leakage from the circulator pipe joints near this transition.

On February 26, 1958, all underdrains were backflushed. Observation was made then of a new crack in the bottom of the drainage inspection chamber at Station 0+89.

On June 16, 1958, the northeast toe drain was flushed.

On June 30, 1958, the parapet joint on the main dam at Station 9+14.4 was open ¾ inch.

On July 9, 1958, 1-inch copper pipe and valves from the inlet line were installed at the three weir boxes in the drainage inspection chamber for flushing purposes.

On January 22, 1960, strain-gage points were set across new cracks in the inspection chamber at Station 0+43 and at Station 0+54. The crack at 0+43 was described as being open $1/64$ of an inch at the top on the north wall and a hairline crack at the south wall and the bottom. Up to this time all of the cracks were reported to be hairline cracks except the one at Station 0+70.3.

Also on January 22, 1960, a new crack of ⅛ inch width was found across the 479 roadway on the main dam opposite Station 8+93.5. Although it was not a large crack, it was kept under surveillance from its inception. Reports indicate that it did not appear to be increasing.

On October 18, 1961, two strong-motion earthquake recorders were installed at the Baldwin Hills Reservoir, one on the crest of the main dam and the other on the east side of the reservoir.

The reservoir keeper reports that there was "duck grass" growing around the sides of the reservoir in 1962. The reservoir was lowered about 20 feet so that the grass could be removed.

In November, 1962, four observation wells were drilled to detect seepage. These were 3-inch holes cased with pipe 2 inches in diameter, about 100 feet in depth below the reservoir crest. They were located 8 feet out from the parapet wall and were spaced around the perimeter, as shown on Plate 5.

On March 19, 1963, all drains into the drainage inspection chamber were flushed.

In the weeks immediately preceding failure an apparent uplift developed in the inlet tunnel, the gate tower, and the portion of the drainage inspection chamber east of Fault I.

The Baldwin Hills Reservoir was a closely observed feature of the water system of the Los Angeles Department of Water and Power. This vigilance was imperative because of the complex geologic setting and highly developed urban surroundings. It would be rare indeed to find a feature of such limited storage in any other system where such extensive surveillance data were obtained.

CHAPTER VII

EARTH MOVEMENT

The area in which Baldwin Hills are located experienced severe tectonic deformation during late Pleistocene time. There is little evidence, however, to indicate that distortion has persisted at a significant rate throughout the last 10,000-year period. Recent alluvial outwash shows no appreciable disruption where deposited across faults, and younger natural drainage channels are not noticeably offset. Within the past several decades earth movements have accelerated, as manifested by the shifting and settling of survey stations and the development of conspicuous earth cracks, particularly in an area southeast of the reservoir.

MANIFESTATIONS OF MOVEMENT

Observations of Subsidence

In 1917, Water and Power established bench marks at the site of Centinela Reservoir, a proposed storage facility about 3,000 feet southwesterly of Baldwin Hills Dam. These surveys provided the earliest records of land surface elevations within what is now the subsidence bowl in Baldwin Hills. In 1943 levels again were run to some of these bench marks, and the changes of elevation since 1917 were determined.[1] With these data and the results of the subsequent levelings conducted by Water and Power, it has been possible to estimate the general regimen of subsidence. The trends are shown graphically on Plate 15, on which a maximum estimated subsidence is shown to have occurred approximately at the location of Bench Mark PBM 122 about one-half mile westerly of Baldwin Hills Reservoir. This subsidence is estimated to have been about 9 feet between 1917 and 1962. Plate 16 is a map showing current annual rates of subsidence and the limits of the subsidence bowl.

Surveys of the Baldwin Hills Reservoir site were conducted by Water and Power in 1939, when bench marks were set and a proposed axis established for the main dam. The 1939 surveys also included level lines across Baldwin Hills that connected bench marks near the reservoir with outlying bench marks which had been established earlier by the U. S. Coast and Geodetic Survey.

The area of survey was progressively extended at successive four-year intervals, and in 1962 was bounded on the south by Slauson Avenue, on the east by La Brea and Vernon Avenues, on the north by Santa Barbara Avenue and Rodeo Road, and on the west by Jefferson Boulevard and Overland Avenue. By then many new lines of levels interlaced the hills.

At the completion of each leveling, an annual rate of change of elevation was computed for each bench mark. Maps were prepared from these data showing the locations of bench marks and lines of equal rate of subsidence (isobase lines). This information has been presented in a series of unpublished reports by staff members of Water and Power.[1, 2]

Of particular importance in establishing the long-time history of subsidence in the area are the data in Report 331.[1] These data indicate that a temporary bench mark, which for convenience is called "BM Saddle", subsided 4.2 feet between 1917 and 1943. "Saddle" is located about 2,700 feet southeasterly from the center of the subsidence bowl defined by the contours on Plate 16. Isobase lines that were drawn by Water and Power at four-year intervals during the period 1950 through 1962 indicate that the center of the bowl has remained in almost the same position horizontally during that twelve-year period. Accordingly, there is no basis for assuming it migrated substantially in earlier years.

Using the foregoing facts, together with indications from the isobase maps that the center of the bowl subsided more than "BM Saddle" by a factor of about,

Report 331-C, 0.200 to 0.160 = 1.25
Report 331-D, 0.210 to 0.157 = 1.34
Report 331-E, 0.145 to 0.112 = 1.30

Average = 1.3

it is computed that between 1917 and 1943 a point at the present center of the bowl of subsidence experienced a vertical subsidence of about 1.3 by 4.2 = 5.5 feet.

PBM 122 is located reasonably close to the center of the bowl, and it has been adopted as a reference bench mark. The records of PBM 122 apparently begin in March, 1950. The subsidence of this point has been estimated by Leps for the years 1917 to 1950.[3] The subsidence between 1943 and 1950 was approximated by examining the measured performance between 1950 and 1958 and extrapolating backwards. For that period, PBM 122 subsided an average of from 0.20 to 0.21 foot per year. Assuming that the

[1] Hayes, S. A. "Report on Earth Movement Measurements in the Baldwin Hills and Inglewood Area." Reports Nos. 331 (1943), 331-A (1947), 331-B (1951), 331-C (1955), and 331-D (1959).

[2] Walley, F. J. "Report of Elevation Changes of Bench Marks and Land Subsidence in the Baldwin Hills Reservoir Area." No. 331-E. Unpublished report of Los Angeles Department of Water and Power, September 1963.

[3] Leps, T. M., Consultant to the Engineering Board of Inquiry. Letters to R. B. Jansen dated March 12, 1964, and April 12, 1964.

annual rate for this 8-year period is a reasonable approximation for the preceding 7 years, PBM 122 subsided about 1.4 feet between 1943 and 1950. This amount appears consistent with actual measured subsidences during that period at the following bench marks in the subsidence area, in relation to their distances from the center of the subsidence bowl:

PBM 31 (Baldwin Aux.)	1943-1950	0.45 foot
PBM 12 (La Brea and Stocker)	1943-1950	0.22
PBM 68 (BM "D-D")	1946-1950	0.40

In summary at this point, then, the center of the bowl of subsidence subsided about as follows:

1917 to 1943, 5.5 feet, as estimated from the measured subsidence of nearby point "Saddle".

1943 to 1950, 1.4 feet, as estimated from its measured subsidence during the years 1950-1958.

From March, 1950, to August, 1954, PBM 122 subsided a measured 0.20 foot per year for an incremental amount of 0.90 foot. Thus, by August, 1954, this point in the bowl of subsidence had subsided about 7.8 feet since 1917.

From August, 1954, to August, 1958, PBM 122 subsided a measured 0.21 foot per year for an incremental amount of 0.84 foot. Thus by August, 1958, this point in the bowl had subsided about 8.6 feet since 1917.

From August, 1958, to about August, 1962, the position occupied by PBM 122 (which apparently had been destroyed during the period) subsided a measured 0.14 foot per year for an incremental amount of 0.56 foot. Thus, by August, 1962, this point in the bowl had subsided about 9.2 feet. Bringing this estimate up to present date, it is probable that the location of PBM 122 has subsided about 9.7 feet since 1917. In view of the gaps in the data and the probably low order of precision of the 1917 to 1943 subsidence measurement, the estimate of 9.7 feet total subsidence since 1917 is probably accurate to within plus or minus 1 foot.

A plot of the data developed above for the center of the bowl against calendar time, and in relation to measured subsidences of other bench marks in the subsidence bowl, indicates that the subsidence of the center of the bowl may have started any time before 1926. It is not possible from the data so far available to fix the date of initial subsidence any more closely. It must also be recognized that no data covering conditions prior to the year 1917 have been discovered; and it must be conceded, in the absence of reasonable proof, that the area may have experienced measurable uplift or subsidence prior to 1917.[3]

The isobase lines drawn by Water and Power show also an area of uplift east of La Brea Avenue in which the maximum rate of rise between 1958 and 1962 was about 0.01 foot per year.[2] Since a virtual uplifting of so large a region suggests tectonic participation, the survey procedure was analyzed to determine whether the uplift was actual or possibly relative to some unstable reference. It was learned that the contour of zero elevation change was controlled by Bench Mark PBM 40-C, located about 1,000 feet northeast of the northeast corner of the reservoir, and that this bench mark had been regarded as fixed throughout the period of surveys. Its stability had been checked by levels run from Bench Mark PBM 1 at the intersection of Centinela Avenue and Market Street, two and one-half miles south of the reservoir. Although these surveys suggested PBM 40-C was stable, there was question with regard to the fixity of the control Bench Mark PBM 1. Consequently, the Engineering Board of Inquiry arranged with the U. S. Coast and Geodetic Survey for a first order leveling from the tide gage at San Pedro to several bench marks in Baldwin Hills.

At the time of this writing the U. S. Coast and Geodetic Survey has completed leveling to PBM 1 and PBM 40-C and to several other select bench marks on or near Baldwin Hills. The levels commenced and closed at Los Angeles City Hall, which is regarded as a relatively stable station. The field work soon will be extended to the continental datum at the San Pedro tide gage. The tentative results of these surveys of March and April, 1964, are presented in Table VII-1, in which there are shown also elevations determined in past years by U. S. Coast and Geodetic Survey, Los Angeles County Engineer, and Los Angeles Department of Water and Power. The results as tabulated indicate in general a subsidence of all stations, including the references PBM 1 and PBM 40-C, which were assumed stable by the Department of Water and Power. These results should be considered tentative pending a tie-in at the tide gage. However, it appears reasonably certain at this time that no areas of absolute uplift exist in the vicinity of the reservoir.

The Survey Division of the Department of the County Engineer, Los Angeles County, has checked horizontal movements by triangulation in the Baldwin Hills area, beginning in 1934.[4] The results of the measurements for the area adjacent to the Baldwin Hills are presented on Plate 16, where the horizontal vectors indicate the direction and magnitude of horizontal movement of bench marks. This movement is best exemplified by examining the station designated as "Baldwin Aux." which is located beneath an elevated steel water tank south of the reservoir. In the period from 1934 to 1961 this station moved 2.21 feet horizontally in the direction of South 64° 29′ West. From 1961 to 1963, an additional 0.28 foot of horizontal movement was noted in this same direction. The horizontal motion is generally directed towards the trough of subsidence which is located southwesterly of the reservoir.

[4]Alexander, I. H. "Horizontal Earth Movement in the Baldwin Hills, Los Angeles Area." Journal of Geophysical Research. June, 1962.

TABLE VII-1

ELEVATIONS AT SELECTED BENCH MARKS

Bench mark	1933	1934	1943	1946	1950	1954	1955	1956	1958	1960	1961	1962	1964
PBM 1 Control bench mark	139.954	--------	140.004	--------	--------	--------	139.679	139.620	--------	139.598	--------	--------	**139.551**
PBM 68	--------	--------	317.449	317.110	316.686	316.099	--------	--------	315.590	--------	--------	315.189	--------
BHBM 126 Toe of dam	--------	--------	--------	--------	261.182	261.067	--------	--------	--------	--------	--------	--------	**258.963**
BHBM 129 Top intake tower	--------	--------	--------	--------	481.821	481.595	--------	--------	481.458	--------	--------	--------	**481.084**
PBM 40-C Control bench mark	--------	--------	--------	--------	--------	456.787	--------	--------	456.746	--------	--------	456.743	**456.317**
PBM 31 Baldwin auxiliary	--------	**512.151**	(511.679)	**511.354**	(511.239)	(510.940)	--------	--------	(510.652)	--------	510.113	--------	**510.050**
PBM 26	--------	--------	(504.327)	**503.988**	(503.853)	(503.456)	--------	--------	(503.103)	--------	502.533	--------	**502.416**
PBM 30 Baldwin	--------	**513.093**	(512.612)	**512.282**	(512.161)	(511.867)	--------	--------	(511.574)	--------	511.031	--------	**510.971**

Note: Elevations in parentheses from Los Angeles Department of Water and Power; those boldface by U. S. Coast and Geodetic Survey; all others by Los Angeles County Engineer. Elevations shown for 1964 are tentative.

Observations of Cracking

The Engineering Board of Inquiry instigated a crack survey of the Baldwin Hills vicinity. The cracks located in the reservoir after failure and those that were traced out in the vicinity around the La Brea-Stocker area are shown on Plates 17a and 17b.

Earthcrack 1 developed near the intersection of Stocker Street and La Brea Avenue in May, 1957. It appeared in the asphaltic surfacing of the playground of the nearby Windsor Hills School in October, 1957. The Los Angeles School Districts maintained surveillance of the schoolyard crack from the time of its detection until July, 1962, at which time it was obscured by resurfacing. Observations reveal similarities in the line of rupture through the school yard and the cracking along Fault I in the floor of Baldwin Hills Reservoir.[5] In both instances irregularity in gap width was apparent along the lengths of the cracks; vertical offset had occurred but there was no horizontal displacement along the crack. There was a striking likeness in the general appearance of the cracking there and that at Baldwin Hills Reservoir. Such similarities were noted in other areas, as illustrated in Photos 53 through 58.

Similarities between the cracks in the reservoir and the cracks that have developed elsewhere in Baldwin Hills are as follows:

1. A general parallelism to preexisting faults.
2. A common orientation generally north-south in direction.
3. Steep dip.
4. Gaps which vary in width along the length of each crack.
5. No horizontal displacement along the cracks.
6. Occurrence in areas where rate of subsidence is changing markedly within short distances.

In the fall of 1963, the Los Angeles City Engineering Survey Section set up a series of three control lines across the earth cracks in this area. These cracks range from 2,500 to 3,800 feet southeasterly from the southeast corner of the reservoir. Water and Power reports that its surveyors have cooperated with the City Engineer's forces in making the periodic checks, but no significant change has been detected.

In 1960 a subcommittee of the Geological Hazards Committee of the City of Los Angeles reported cracking in the La Brea-Stocker vicinity. Also, in April of 1963, a Stocker-La Brea Fault Zone Study Committee composed of representatives of public and private interests was formed to study this area of general cracking. As part of its investigation, the Engineering Board of Inquiry had discussions with individual members of these two groups.

On February 19, 1963, following a rain, a small slide occurred along the west embankment of La Brea Avenue about 700 feet south of Stocker Street near Earthcrack 8. Salty water issued from a saturated area associated with the slide. This suggested that brine injected at depth in connection with repressurization of Inglewood Oil Field might be finding its way to the surface. However, laboratory analyses have shown that samples of this seepage differed in chemical character from the brines used in connection with oil recovery.

In January, 1963, Mr. Perry Ehlig, Consulting Engineering Geologist, inspected the vicinity of Baldwin Hills Dam in connection with an investigation he was conducting for a client. Ehlig included in his scrutiny the east end of the main dam and its east abutment. He searched particularly for indications of subsidence cracking and seepage. He has reported that

[5] Omstead, H., Architectural Planning Branch, Los Angeles City School Districts. Letter report with enclosed photographs. January 9, 1964.

his observations were conducted meticulously and that no such evidence was found.[6]

POSSIBLE CAUSES OF EARTH MOVEMENT

In investigating the cracking and elevation changes within, and in the vicinity of, Baldwin Hills Reservoir, consideration was given to earthquakes, groundwater extraction, oil and gas production, and tectonics as possible causes.

Earthquakes

Baldwin Hills is an integral feature of the Newport-Inglewood uplift, an alignment of faults, structural domes, and saddles that has been the source of comparatively frequent earthquakes. Most notable of the shocks occurring on Newport-Inglewood uplift in historic times are the temblors of June 21, 1920, near Inglewood, and March 10, 1933, near Long Beach. The 1920 shock was probably centered three to four miles southerly of the site of Baldwin Hills Reservoir, and its intensity was sufficient to wreck a two-story school building, topple walls, and break windows.[7, 8, 9] The 1933 earthquake is believed to have been centered offshore, southeast of Newport, California.[10] It is classified a major disaster in which 115 lives were lost and an estimated 40 million dollars in damages incurred.[9] Numerous lesser instrumentally-recorded shocks have been attributed to the Newport-Inglewood uplift. The locations of these are shown on Plate 18, in which one of the conspicuous groups of epicenters is depicted near Baldwin Hills.

On October 21, 1941, fifteen oil wells in Dominguez Field on Newport-Inglewood uplift were crushed by an earthquake.[11]

On February 18, 1963, Standard Oil Company Wells Stocker No. 5 and No. 17 were damaged within the Vickers zone by a reported earthquake; and on March 10, 1963, the Baldwin Cienega Well No. 27 was damaged at a depth of 1,520 feet by another temblor.[12] (Plate 8)

On January 2, 1964, the services of Dr. Pierre St.-Amand, Consultant in Seismology and Engineering Geology, were obtained by the Engineering Board of Inquiry for the specific purpose of investigating the seismic aspects of the failure of Baldwin Hills Dam and Reservoir.

The investigation of seismic phenomena by Dr. St.-Amand included an analysis of seismographic records obtained by the Seismological Observatory at California Institute of Technology during the period 1934 to December 15, 1963; interrogation of residents near Baldwin Hills as to whether they had felt vibrations in the months preceding failure; a field canvass for surficial indications of a minor earthquake; and the installation of a seismograph at the reservoir site to detect and record any unusual events subsequent to failure.

Upon completion of his studies, Dr. St.-Amand concluded, as follows: [13]

> "At present it is quite clear that there were no earthquakes strong enough to have been responsible for any important inertial effects in the months preceding dam failure or except for the Kern County shocks in 1952, in the life of the dam. An earthquake could not have caused the failure by inertial effects on the dam and associated structures.
>
> "It is possible that movements during earthquakes could in the past have ruptured tile drains etc. and indirectly contributed to the failure but even so, failure could have been consequent to slow creep on the faults."

The seismographs at California Institute of Technology are believed to be sufficiently sensitive to have detected a tremor with an energy level about 1/100 of that which could have been perceived by any person in the vicinity of the reservoir site. Consequently, the tremors sensed by some residents on the day of the failure have been attributed to some other kind of vibrations such as those caused by trucks, sonic booms, or heavy equipment. None of the local citizens interviewed by Dr. St.-Amand reported feeling shocks, although there were reports of minor quakes from other sources.

An earthquake may or may not attend tectonic displacement along a fault. Nonvibratory offsets on faults have occurred on the San Andreas fault at Almaden Winery near Hollister, California,[14] on a thrust fault in Buena Vista Hills in Kern County, California,[15] and along a fault zone two miles in length located eight miles north of Bakersfield, California.[16] In each of these instances displacement was observed to have occurred, but no vibrations were felt, nor were they recorded by the nearest seismographs.

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[6] Ehlig, P. Oral communication to Mr. L. B. James. April 1, 1964.
[7] Taber, S. "The Inglewood Earthquake in Southern California, June 21, 1920." Volume 10. Bulletin, Seismological Society of America. 1920.
[8] Kew, W. S. W. "Geological Evidence Bearing on the Inglewood Earthquake of June 21, 1920." Volume 13. Bulletin, Seismological Society of America. 1923.
[9] U. S. Coast and Geodetic Survey. "Earthquake History of United States." Part 2, "Stronger Earthquakes of California and Western Nevada." Publication No. 41-1 revised 1960 edition.
[10] Benioff, H. "Determination of Extent of Faulting with Application to the Long Beach Earthquake." Volume 28, No. 2. Bulletin, Seismological Society of America. April, 1938.
[11] Bravinder, K. M. "Los Angeles Basin Earthquake of October 21, 1941, and Its Effect on Certain Producing Wells in Dominguez Field, Los Angeles County, California." Volume 26, No. 3. Bulletin, American Association of Petroleum Geologists. 1942.
[12] Horn, A. J., Standard Oil Company of California. Interview with Engineering Board of Inquiry. February 5, 1964.
[13] St.-Amand, P., Special Consultant to the Engineering Board of Inquiry. Letter to R. B. Jansen, subject: "Seismic Aspects of the Baldwin Hills Dam Failure." January 13, 1964.
[14] Tocher, D. "Creep on San Andreas Fault." Volume 50, No. 3. Bulletin, Seismological Society of America. July, 1960.
[15] Koch, T. W. "Analysis and Effects of Current Movement on an Active Fault in Buena Vista Oil Field, Kern County, California." Volume 17. American Association of Petroleum Geologists. 1933.
[16] Hill, M. L. "Tectonics of Faulting in Southern California." Bulletin 170, Chapter IV. California Division of Mines. 1954.

Such movement on faults is referred to as "slow creep," and it is to this phenomenon that Dr. St.-Amand refers in the second part of his conclusion. The absence of an earthquake during the displacement of Faults I and V, therefore, does not preclude the possibility of tectonic motivation.

Groundwater Extraction

Subsidence due to intensive and prolonged pumping of water wells is occurring in several parts of California. In the San Joaquin Valley as much as 23 feet of settlement has been attributed to this cause, with annual rates of as much as 1.0 foot per year.[17] In view of this subsidence elsewhere in the state, the Engineering Board of Inquiry initiated a hydrogeologic investigation to determine whether groundwater extraction may have contributed significantly to the elevation changes occurring near Baldwin Hills Reservoir. An area bounded on the west by La Cienega Boulevard, on the northeast by the toe of Baldwin Hills, and on the south by Stocker Street, was studied. The findings are given in the following enumerated paragraphs:

1. The Baldwin Hills are an anticlinal structure underlain chiefly by nonwater-bearing Pliocene formation which is, in turn, overlain by a thin section of Pleistocene formations.

2. Historically, the Pleistocene formations in the area designated for study have never been saturated, nor has there been any extraction of ground water from them.

3. All extractions of ground water in the general vicinity of the Baldwin Hills have been made from aquifers which occur as veneer overlying the foundation material or aquifers which do not exist in the study area.

4. The extraction of ground water from wells in the peripheral margins of the hills would have no influence on subsidence in the area of the reservoir, since all groundwater extraction has taken place from sediments overlying formations which constitute the foundation of the reservoir.

Oil and Gas Production

Land subsidence may attend or follow the extraction of oil and gas. One of the earliest examples occurred at Goose Creek Oil Field, about 25 miles southeast of Houston, Texas. Here about three feet of settlement occurred over a period of eight years.[18, 19] At later dates, repeated levelings by numerous agencies indicated that accentuated subsidence also had occurred on the Los Angeles Coastal Plain at Wilmington, Huntington Beach, Santa Fe Springs, Torrance, Inglewood, Salt Lake (Beverly Hills), and Playa del Rey Oil Fields.[20, 21, 22]

Oil field subsidence is attributed to decrease in pressure within the subsurface reservoir as a result of exploitation of oil, gas, and brine. The natural pressures within the oil structures oppose subsidence. As these pressures are reduced, consolidation occurs, except to the extent that increased load may be transferred to strata overlying a producing zone and be borne by arch effect. Where such arch effect does not develop, a finite decrease in reservoir pressure is tantamount to increasing surface loading a like amount.

Probably the most publicized example of oil field subsidence occurred at Wilmington, California. Here a settlement of about 28 feet and attendant horizontal displacements of as much as 6 feet have been measured. Some of the factors which link the bulk of subsidence at Wilmington Field with oil production follow:

1. Contours of settlement (isobases) were roughly elliptical in outline, with the major axis of the ellipse approximately congruent with the axis of the elongate domal structure of the oil field.

2. Bench marks whose elevations were fairly stable prior to production from the field subsided markedly after 1937, the year of discovery.

3. Subsidence progressed continuously with production of the field and covered an area which extends a little beyond the limits of production.

4. Subsidence has been curtailed by repressurization of the oil producing zones.

While there are some obvious differences between Wilmington and Inglewood Oil Fields, most notably in topography and geologic structure, there are also some similarities.[3] These are noted as follows:

1. Both vicinities have experienced substantial vertical subsidence, the maximum at Terminal Island being about 28 feet and that at Baldwin Hills being in the order of 9 feet.

2. Both vicinities have experienced subsidence over large horizontal areas, the shape of which is roughly elliptical. At Terminal Island the major and minor axes of the ellipse are roughly 35,000 and 23,000 feet, while at Baldwin Hills the axes are about 14,000 and 10,000 feet.

3. Both vicinities have experienced maximum subsidence in a small area near the centers of the ellipses, and the amount of subsidence progressively diminishes towards the edges of the ellipses, thus forming actual though imperceptible "bowls" of subsidence. (Plate 19)

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17 Poland, J. F., Research Geologist, U. S. Geological Survey, Oral communication to Mr. L. B. James. March 25, 1964.

18 Minor, H. E. "Goose Creek Oil Field, Harris County, Texas." Volume 9. Bulletin, American Association of Petroleum Geologists. 1925.

19 Pratt, W. E. and Johnson, D. W. "Local Subsidence of the Goose Creek Field." Volume 34. Journal of Geology. 1926.

20 Grant, U. S. and Sheppard, W. E. "Some Recent Changes in Elevation in the Los Angeles Region of Southern California and Their Probable Significance." Volume 29. No. 2. Bulletin, Seismological Society of America. 1939.

21 Grant, U. S. "Subsidence and Elevation in the Los Angeles Region." 75th Anniversary Volume. Science in the University Press. 1944.

22 Grant, U. S. "Subsidence in the Wilmington Oil Field." Bulletin 170, Chapter X. California Division of Mines. 1954.

4. In both vicinities the bowls are concave upwards near their centers, with their surface curvatures changing to convex upwards in some areas near the margins of the bowls.

5. In both vicinities, where the bowl surface is concave upwards, the ground surface is subjected to horizontal compression; where the bowl is convex upward, the ground surface is subjected to horizontal tension.

6. In both vicinities accurate triangulation has shown that survey monuments within the bowls of subsidence have moved horizontally, generally toward the centers of the bowls.

7. In both vicinities the shapes and locations of the subsidence bowls place them almost symmetrically above the subsurface outlines of important oil and gas producing fields.

The Inglewood Field has certain characteristics in common with other oil fields known to be subsiding:

1. The fluid production is from unconsolidated sands interbedded with soft shales.

2. The fluid production per acre is high since Inglewood is actually several oil fields, one on top of another.

3. The natural drive mechanism for most of the field has been dissolved gas. Thus, production has been accompanied by large fluid pressure declines in spite of secondary recovery measures. An increased vertical force is imposed on the rock structure because these oil pool pressures have been reduced.

The graphs of Plate 15 indicate that subsidence has progressed continuously since the mid-1920's, but they give no indication of the elevation changes, if any, prior to that time. The Engineering Board of Inquiry was unable to locate any survey records or other evidence showing that elevation changes either had or had not occurred earlier. In its canvass for this information the Board queried the Los Angeles Department of Water and Power, the City of Los Angeles Geodetic Section, the Standard Oil Company of California, the Los Angeles County Engineer, the U. S. Geological Survey, and the U. S. Coast and Geodetic Survey. Several of these agencies are continuing to search for old survey data and, should their efforts prove successful, much may be learned concerning the relative contribution of the factors causing subsidence.

Damage to oil wells resulting from subsurface movement has been more prevalent in and above the highly productive Vickers zone of Inglewood Oil Field than at greater depths, and this apparent concentration of damage in the shallow zones has been attributed to the fact that many more wells penetrate the Vickers zone than extend to deeper levels.[12] A second possible explanation would be that the earth movements that have occurred were largely concentrated within and/or above this productive oil zone.

Tectonic Activity

Tectonic processes are the natural phenomena responsible for creating crustal deformations. The question arises as to what extent they may be responsible for the contemporary subsidence and cracking in Baldwin Hills and for the movement on Fault I that led to failure of the reservoir.

Folding and faulting of the Cenozoic rocks of Los Angeles Coastal Plain may have commenced as early as middle Miocene time. There is abundant evidence that tectonic distortion has persisted along the Newport-Inglewood uplift, including the Baldwin Hills, during the last 500,000 years. In fact, most of the present physiographic features are believed to have been created during this interval.

A study of the behavior of more than 9,000 survey stations in Los Angeles Coastal Plain leveled by the Los Angeles City Engineer during the last 25 years indicates that in general the sedimentary basins are subsiding, whereas some of the stations in the foothills are rising.[23]

Significant deformation of parts of Newport-Inglewood uplift has occurred within the last 10,000 years. This is evidenced by anticlinal folds which have risen sufficiently rapidly to prevent breaching by existing streams,[24] and by the remarkably smooth surface of Dominguez Hill which shows little erosion though it consists of only slightly consolidated sediments.[25] According to Dr. U. S. Grant,[21] repeated surveys near Inglewood, three miles south of Baldwin Hills Reservoir, have disclosed contemporary uplifting amounting to as much as 0.03 foot per year. This appears to be related to folding accompanying slow movement along the Newport-Inglewood uplift. However, in Baldwin Hills no reliable geologic factors are found from which to estimate the amount of tectonic deformation that may have occurred during the past 10,000 years. It has been suggested that thick peat beds existing northeast of Baldwin Hills may have formed behind barriers across Ballona Creek which were created by recent uplifts occurring along Inglewood fault.[20] That such uplift occurred is only speculative, however, and no estimates of the possible displacement were made. On the other hand, the lack of deformation in the "50-foot gravel" aquifer, where this deposit is transected in Ballona Gap by Inglewood fault, suggests that fault movement has been limited or nonexistent near Baldwin Hills during the Recent period.

[23] Stone, R. "Geologic and Engineering Significance of Changes in Elevation by Precise Leveling, Los Angeles Area, California." Abstract of paper presented to the Geological Society of America at San Diego, California. March, 1961.
[24] Jahns, R. H. "Investigations and Problems of Southern California Geology." Bulletin 170, Chapter 2. California Division of Mines. 1954.
[25] Vickery, F. F. "The Interpretation of the Physiography of the Los Angeles Coastal Belt." Volume 11. Bulletin, American Association of Petroleum Geologists. 1927.

The maximum current rate of subsidence in Baldwin Hills is believed to be about 0.2 foot per year as measured by the Los Angeles County Engineer. (Plate 16) Using this rate, it is possible to estimate the time required to form the graben structure in the dome of the Inglewood Oil Field. It has been noted that about 270 feet of displacement has been measured on Inglewood fault along the east side of the graben. If present subsidence is related to movement of the graben and the current subsidence rates are indicative of the rate at which the graben has dropped, it has taken only 1,350 years for this structure to form. From a geological point of view, this is a short period of time for so large a deformation. Moreover, there is no evidence of significant recent lateral or vertical movement on Inglewood fault in this area during this period. This suggests, therefore, that the subsidence now occurring is too rapid to be accounted for solely as a continuation of the tectonic movement involved in creation and growth of the graben.

With regard to the genesis of oil field structures along Newport-Inglewood uplift, a generally accepted theory attributes them to lateral movements along a deep-seated fault in the basement rocks which parallels the trend of the uplift. The thick series of sediments overlying this fault responds to these displacements by wrinkling and puckering. The structures so created align en echelon with their axes askew to the general trend of the uplift. These folds never quite parallel the shearing because of the component of friction that develops along the fault plane.[26] This theory has been demonstrated by a simple device consisting of two cards placed side by side with a thin tissue pasted across the juncture. When the cards are shifted horizontally, miniature folds appear in the tissue. These are oriented en echelon as are the domal folds along Newport-Inglewood uplift. If the theory is correct, the creation of Inglewood Oil Field dome, the graben structure, and related faults were all associated with right lateral movement on Inglewood fault, i.e., movement wherein the east side of the fault moved relatively southward with respect to the west side. It appears significant that the vectors of horizontal movements of survey stations on Baldwin Hills, as depicted on Plate 16, point roughly toward the center of subsidence in Inglewood Oil Field, and would therefore appear to be generally opposite in direction to the tectonic movements required to stretch the crest of the oil field dome and thus create the down-dropped block west of the reservoir.

Any attempt to determine with reasonable accuracy the degree to which tectonic or oil field activities are contributing to contemporary subsidence would require further investigation. Primarily a better understanding is needed of the earth movements occurring at depth including identification and delimitation of the zones that are undergoing compaction and the rates involved. The studies required to fulfill such an objective would be lengthy and beyond the scope of the investigation by the Engineering Board of Inquiry.

[26] Ferguson, R. N. and Willis, C. G. "Dynamics of Oil Field Structures in Southern California." Volume 8. Bulletin, American Association of Petroleum Geologists. 1924.

CHAPTER VIII

INVESTIGATION AFTER FAILURE

SURFACE INVESTIGATION

Observations following the failure showed that, although the reservoir floor was covered with about 2 inches of fine silt and clay, a continuous crack was visible across the reservoir floor approximately parallel to and near the toe of the east reservoir slope. Vertical displacement averaged about 2 inches. It was as much as 7 inches in some locations, with the west side of the crack down with relation to the east side. (Photos 59 and 60)

The crack extended up the south slope of the reservoir east of the access stairway but with very little vertical offset. Small cracks also extended about 5 feet south across the road surface outside the fence opposite the curb joints at Dam Stations 21+75 and 21+90. At Station 21+75 the construction joint in the curb along the fence line was open and displaced vertically about ¼ inch, with the west side down with relation to the east side. A filling of fine soil stood freely in the crack. The presence and condition of the filling attested to the very recent opening of the joint.

The broken surface of the asphaltic paving was fresh and unstained. The aggregate in the paving glistened, and the asphalt was very black and lustrous. In at least one location the surface had apparently broken in two or more stages wherein there were minute organisms, fresh water ostracods in the presence of sediments, affixed to part of the 3-inch asphaltic paving thickness. The upper part of the broken surface exhibited lustre. After three days of exposure the offset surfaces of the asphaltic paving had become dull and no longer had the shiny appearance that was noted during the inspection on the day following the failure.

The two transverse joints in the circulator line connection conduit farthest from the tower had opened and were offset. The condition of the exposed expansion joint filler and the joint surface suggested that this opening had occurred very recently. The joint openings were observed to have increased somewhat during the first three days following the failure. (Photo 61)

Cracking and parting were observed at the Elevation 418 gate approach channel. This was on the line of the crack in the reservoir floor. The offset also apparently increased in the ensuing three days.

Sink holes of varying dimensions could be seen at several locations along the continuous crack. Large sink holes were located about midway between the gate tower and the breach in the main dam, over the drainage inspection chamber, and at the south end of the reservoir. Others of smaller size were also observed. Flow through the large sink holes undoubtedly caused vortices reported by eyewitnesses to the failure. (Photos 62 through 66)

An exposed end of the 4-inch clay tile drain along the toe of the north slope could be seen in the breach. Also, the pea-gravel drain was exposed in both sides of the breach. Calcium deposits were apparent in the bottom of the pea gravel but were not extensive enough to block drainage.

The dam embankment exposed at the top of the breach appeared to be moist but not saturated. In some places where the water had washed over the downstream berm there could be seen the imprints of the sheepsfoot roller used to compact the embankment. In breaching the east abutment the escaping waters left a steep incision in the Inglewood formation, but in the Pico formation cutting was retarded. The scarp of a waterfall about 30 feet high remained after the reservoir had emptied, a testimonial to the relatively greater resistance offered by the Pico sediments to incision.

Several shear zones were exposed on the west side, and fresh slickensided surfaces were noted. There was a pronounced but small parting along one joint surface which suggested very recent movement. Throughout the breach the Inglewood formation was well exposed, and its horizontal stratification was plainly discernible. The flowing water had scoured the thinly bedded weaker laminae, leaving the more indurated strata standing out in bold relief. (Plate 20) It was not possible to trace any of these horizontal strata throughout the breach, but continuity may well exist by some devious paths. The weaker uncemented materials in many strata appeared similar to beach sands. The various laminations ranged in thickness from a few inches to about 2 feet. (Photos 67 through 71)

The spillway conduit between about Stations 2+40 and 5+60 was lost during erosion of the breach. Water from the reservoir underdrains, discharging through the blowoff line into the spillway conduit, could be heard running in the spillway pipe at an air vent at Spillway Station 5+79 on the day after the failure.

Inspection was made of pre-existing cracks in the inlet tunnel between the east portal and the gate tower. There was no apparent change in the conditions of any of these cracks.

Immediate inspection of the drainage inspection chamber was not possible. Deep mud deposits prevented access beyond Station 0+28. It was entered on December 18, when the first detailed inspection was made. A transverse break was observed at Station 0+70.3. (Plate 21) The maximum opening at the break was about 2 inches at the top. There was only a hairline crack at the bottom. There was a left lateral offset of ½ inch and a vertical offset of about ¾ inch, with the west side of the break up in relation to the east side. (Photo 72) Water was observed to be dripping from the break at the top of the chamber. There was a small continuous flow from the north wall about 4 feet from the top, just above a mud deposit. Several ⅜-inch-diameter horizontal reinforcing bars had broken, and fresh rust was observed at the exposed ends of the broken steel. Another transverse break was found in the drainage inspection chamber at Station 0+89 with a maximum bottom opening of about 1 inch. There was a hairline crack at the top. A small continuous flow was observed from this break. The break did not show lateral offset, but there was a vertical displacement of about ⅛ inch, with the east side higher. (Photo 73)

Water and Power measurements of drain flows on December 18 were as follows:

Southeast toe	0.37 gpm
Fault	3.5 gpm
Northeast toe	0.5 gpm
North toe	0.11 gpm
North bottom	0.27 gpm
West toe	1.37 gpm
South bottom	Drip
South toe	None

Flow from the fault drain was discolored.

On December 18, 1963, a minor crack was found near the center of the reservoir floor, trending in a north-south direction. (Plate 22a) It was revealed by the drying of sediments covering the reservoir floor adjacent to the crack. Later subsurface investigation showed that this crack was practically coincident with Fault V. Cleaning of the area disclosed very little displacement; however, it appeared that water had been flowing through the crack into the compacted earth lining.

One joint in the south circulator line at the Fault V crack crossing had opened about ½ inch at the top but remained closed at the bottom. At one of the supports the pipe and grout pad had slid about 1 inch to the west. At other supports fresh separations between the pipe and the grout pad and between the grout pad and the support were noted. This movement had caused the spalling of concrete at some places. The spalls had dropped onto the silt that covered the reservoir floor. From their lack of silt covering it appeared that they had fallen after the reservoir was empty.

An examination was made of the steel liner plate separations inside the circulator line connector conduit. This was to determine if corrosion products at joint connections would indicate successive fault displacements and give a qualitative estimate of how long a time was involved in the failure process.

An inspection on January 4, 1964, revealed cracks in the embankment immediately downstream from the 340 berm on the main dam. It appeared that the openings had been increased in size by water erosion. The most pronounced crack was observed at the east side of the embankment at about Elevation 325. There was some vertical displacement across the crack. (Photo 74)

The steel door into Manhole A had been stove in by the force of the flood waters, which had caused the manhole to become filled with water and debris. The manhole and adjacent test conduit were inspected on January 4, 1964, after having been freed of debris. The bottom of the test conduit still had a deposit of silt approximately 1 foot deep. No cracking or displacement of structural significance was observed. At that time none of the six horizontal drains entering at the bottom of the manhole was flowing more than a rapid drip. The gravel-packed faces of all of these drains were coated with silt, except Drain H-13, which had been washed clear. Drain H-13 extends into the east abutment. (Plate 4) It appears possible that the silt was deposited by the muddy flood water entering the manhole and was washed away by drain discharge after the failure.

The 12-inch drain entering the manhole was discharging slightly. The two 4-inch clay tile drains on each side were wet. A chemical analysis was run on a sample of deposits found in the bottom of the 12-inch drain at the manhole. The deposits, consisting of about 90 percent calcium and 10 percent magnesium salts, were found to be soluble in hydrochloric acid.

Immediately following the failure, straddler points were established across the break in the asphaltic paving at a number of selected locations. (Photo 75) These were steel pegs placed as reference points. Daily measurements were then taken to detect continuing movement at the faults. The results were inconsequential.

On January 4, 1964, Water and Power personnel made an interior inspection of the 42-inch spillway pipe from approximately Station 4+75, which was exposed in the breach, to Station 24+64. They found no separation or signs of distress in this pipe except for the section of pipe under the subdivision road fill located downstream from the dam. In this section there were some signs of settlement and minor cracking but no open cracks of significance.

It was reported by Water and Power that the most recent hydrographic observations at the reservoir prior to the failure had revealed no discoloration of the water in the drains nor any siltation in the manholes or in the drain lines. Seepage measurements had been obtained volumetrically by catching the flow for a given interval in a calibrated container. It can be reasonably assumed, therefore, that any turbidity of the seepage water would have been evident to the hydrographers.

On February 12, 1964, Water and Power personnel conducted underdrain flow tests. The southeast toe drain was tested from Excavation No. 8 to the drainage inspection chamber and the north and south bottom drains from Excavation No. 11 to the chamber. The following conclusions were reached from these tests:

1. The southeast toe drain was open, and there was no appreciable loss of water.

2. The north bottom drain was open, and there was no appreciable loss of water.

3. The south bottom drain was plugged between Excavation No. 7A and the drainage inspection chamber.

EARTHQUAKE REPORTS

The City of Los Angeles Police Department reported that no calls were received on December 13 or December 14, 1963, reporting earthquakes.

Following the failure of the reservoir, investigators for Water and Power interviewed people at 136 locations in the vicinity of the Baldwin Hills Reservoir. Of those interviewed, 52 percent stated that they heard or felt no unusual noises or movements prior to the failure, 27 percent reported that most of the tremors they felt were due to sonic booms or jet airplanes, 12 percent stated that they had felt earthquakes, and 9 percent reported tremors due to unknown causes. Of the 12 percent claiming to have felt earthquakes, only a few were sure of the exact time of occurrence. Three people reported a rumble or earthquake between 8:30 a.m. and 9:30 a.m. on Thursday, December 12. Two persons reported an earthquake between 8:30 a.m. and 9:30 a.m. on Saturday, December 14. One reported feeling a sharp earth shock at 5:00 a.m. on either December 13 or December 14. All but one of the people reporting earthquakes were located in a small area southwest of the intersection of Slauson and La Brea Avenues.

SUBSURFACE INVESTIGATION

As part of this investigation, a total of 15 excavations were made in and adjacent to the reservoir, and seven exploratory holes were drilled. These were mapped and logged by geologists of the Department of Water Resources task force, and soil sampling was conducted by a team from the department laboratories in Bryte, California. On Plates 22a through 22m are shown the post-failure exploration data. All excavations and borings from which samples and data were obtained for use by the Engineering Board of Inquiry were made by forces of the Department of Water and Power. (Photo 76)

Excavations Nos. 1, 2, 4, 5, 6, 7, 8, 12, and 12A were located across the main line of rupture along Fault I within the reservoir to investigate the nature of the failure movement.

Excavations Nos. 3, 11, and 13 were across the reservoir bottom crack along Fault V. Excavation No. 13 was companion to No. 3, immediately on the north of the south circulator line, in an attempt to discover additional evidence of movement along Fault V. Excavation No. 11 was made along the group of tile drains which traverse the reservoir from west to east, draining the reservoir bottom into the drainage inspection chamber.

Excavation No. 9 was a drift under the breach, along Fault I, with the bottom at Elevation 367. It was 256 feet long, ending downstream of the dam axis. The drift was entered through a 50-foot shaft at the head of the breach.

Excavations Nos. 10 and 10A were made in an unsuccessful attempt to intercept the presumed location of Fault V where it was estimated to cross the 24-inch blowoff line.

Excavation No. 14 was located in the area south of the reservoir along the extension of Fault I.

Excavation No. 15 was in the extreme north end of the breach.

Excavation No. 7 had as its fundamental purpose the uncovering of the drainage inspection chamber and its foundation. Initially an area to the south of the chamber was excavated so that the chamber wall was exposed. However, rain on January 18, 1964, resulted in collapse of most of the excavations before any examination could be made of Excavation No. 7. In the final exploration, Excavation No. 7B was made to replace Excavation No. 7. This was excavated on the north side of the drainage inspection chamber and extended between the two main breaks in the chamber. Photos 77 through 80 show the breaks in the north face of the chamber.

Excavation No. 7A was located west of the drainage inspection chamber to expose the group of five drain pipes which enter the chamber near its west end. The purpose was to investigate a presumed break in the drainage pipes but no such break was uncovered.

At selected locations, relatively undisturbed soil samples were taken for moisture and in-place density tests and for special testing for permeability, shear, and consolidation. In addition, penetration needle readings were taken in the foundation on each side of the faults exposed in Excavations Nos. 1 and 2. A number of push-tube undisturbed samples, disturbed jar samples, and bag samples were taken and sent to the laboratory for analysis.

Three sampling points were located in the breached dam. These are designated at EFB-S1, EFB-S2, and WFB-S1. Undisturbed brass liner samples, jar samples, and bag samples were collected from these locations in the breach.

Core samples were taken from the compacted earth lining at five specific locations in the reservoir. One boring was located beside each of Excavations Nos. 1, 2, and 3. A fourth boring, designated BH-R4, was located in an apparent soft spot on the west side of the reservoir floor between the circulator lines; and a fifth boring, BH-R5, was located at an undisturbed portion of the floor near the north circulator. These borings were also logged by task force geologists, and 61 undisturbed core samples in brass liners were collected for laboratory analysis.

A 40-inch bucket auger hole, designated BAH-4, was drilled to intercept the southern end of the 12-inch tile drain. The purpose of this hole was to check alignment and possible settlement of the 12-inch line. Brass liner core samples were collected from this hole as well as jar samples. The hole penetrated the compacted earth lining, the pea-gravel drain, and finally the compacted earthfill which had been placed in the bottom of the reservoir to fill the ravine which existed previous to construction.

TESTING

Soils

Laboratory analyses for identification purposes primarily were made of selected soil samples obtained from the test pits, trenches, bore holes, shafts, and drift described previously. Materials in the natural foundation of the reservoir underlying the pea-gravel drain were examined and tested for in-place density and natural moisture content. Where loose sands or silty sands were encountered, laboratory compaction tests were also conducted so that an estimate could be made of the relative compaction of the material. Selected undisturbed samples were tested to determine the coefficient of permeability at in-situ density. In general, every sample tested was analyzed for grain size distribution and specific gravity; for plastic soils the Atterburg limits were also determined.

The foundation materials were variable in grain-size distribution but were essentially fine-grained silts and sands (with minor amounts of clay) with about 25 to 95 percent by weight finer than 0.074 millimeters (No. 200 U. S. Standard Sieve Size) and from about 5 to 35 percent finer than 0.005 millimeters. The uniformity coefficient, which is the ratio of the grain diameter at which 60 percent is finer to the diameter at which 10 percent is finer, varied from 10 to 100. The soil classifications and sampling points are shown on the logs of excavations.

Samples of the representative loose soils from the reservoir foundation and from the breach were tested to determine if the soils structure of these materials was subject to collapse when loaded or when saturated. It appears from the results of these tests that all samples which were obtained had at some time been subjected to wetting, possibly during the course of the failure; and, therefore, very little additional structural collapse or densification was noted in these laboratory tests.

Penetration needle readings were taken on in-place foundation materials in the immediate vicinity of a fault and to either side of that fault to determine the possibility of moisture migration. This was done by taking the readings on a grid system which used the fault as the approximate middle of the grid. These readings were taken on the north face of Excavations Nos. 1, 2, 3, and 6 and on the south face of Excavations Nos. 1 and 2. Similar readings were taken in the compacted earth lining on the north face of Excavation No. 8 and the south face of Excavation No. 11. The results were inconclusive.

On samples selected from the borings through the earth lining material, moisture and density profiles were determined. Based on these profiles, specimens for triaxial shear tests were tested in two conditions to determine if there had been any hardening or stiffening of the compacted earth lining due to aging:

1. Samples obtained from the drilling which, in effect, simulated the placement condition.
2. Companion samples which were remolded at the same moisture content to the same density.

The resulting stress-strain curves were compared, and it was noted that there was no appreciable difference in the shape of these curves. It would appear from these results, therefore, that the compacted earth lining material retained its plasticity during the life of the project. It was able to deform a substantial amount without any fracturing. In several areas considerable rupture occurred in the pea-gravel drain, and yet the overlying earth lining material appeared to have spanned the weakened foundation without appreciable structural damage. Selected laboratory testing data are summarized in Table VIII-1.

In Excavation No. 7B, a white sandy deposit, elliptical in shape and from 12 to 15 inches across, was noted immediately adjacent to Fault I and at about the level of the middle of the inspection chamber. Chemical analyses were made of this soil to determine if it represented a possible spot where percolating water leached out the calcium from the cemented pea-gravel drain and deposited it in the fault zone. Results of this chemical testing indicated that the white deposit had the same characteristics as the white deposit found in the cemented pea-gravel drain, i.e., containing a measurable percentage of sulphates of calcium and sodium. However, total soluble constituents amounted to only 0.25 percent.

An attempt was made to perform a crude permeability test in the bottom of Excavation No. 7B directly over Fault I. Water percolated into the foundation so rapidly that it was impossible to maintain a constant pressure head.

TABLE VIII-1

POST-FAILURE SOIL TESTING DATA

	Number of samples	Average	Maximum	Minimum
Foundation Samples				
In-Situ Density, pcf	57	92.4	100.6	78.6
Laboratory Maximum Density [a], pcf	8	101.9	107.9	98.3
In-Situ Permeability, feet per day	29	1.4	4.6	Nil
Consolidation				
Load up to 2.8 tsf, percent	2	1.26	1.30	1.22
After Saturation, percent	2	1.46	1.53	1.39
Compacted Earth Lining Samples				
In-Situ Density, pcf	59	110.2	116.4	96.8
In-Situ Moisture Content, percent	61	18.7	33.9	15.2
Laboratory Maximum Density [a], pcf	1	119.6	----	----
Optimum Water Content, percent	1	13.2	----	----

[a] Based on Department of Water Resources' standard 20,000 foot-pounds per cubic foot compactive effort.

Pea-gravel Drain

Samples of the pea-gravel drain were selected from areas where the drain had enough cementation to develop flexural strength. Representative pieces were taken to the department Concrete Laboratory where test specimens were subjected to the standard test for flexural strength for concrete beams (ASTM Designation: C 293-59). The results of the tests are summarized in Table VIII-2.

TABLE VIII-2

PEA-GRAVEL DRAIN TESTING DATA

Specimen	*Modulus of Rupture, psi*
A	50
B1	91
B2	89
B3	108
C	49

Mathematical analysis indicates the cemented pea-gravel drain represented by the test specimens to have sufficient beam strength to bridge a void of from 6 to 10 inches before rupturing, under a load corresponding to 10 feet of saturated earth lining and 65 feet of reservoir water. (Photos 81 and 82)

In addition, chemical analyses were made of a white deposit which was found in the pea-gravel drain in many places. This deposit is apparently the same calcium deposit encountered during the operation of the reservoir which necessitated constant cleaning in order to keep the drainage system open. The test results indicate that the foreign material is basically calcium, being in the forms of calcium carbonate and calcium sulphate. One sample had a calcium sulphate content of 0.83 percent, a concentration which is believed to be high enough to cause deterioration of concrete.

Asphaltic Membrane

Samples of asphaltic membrane material obtained from both the breached area and Excavation No. 11 were submitted to the State Division of Highways asphalt laboratory in Sacramento for analysis. One specimen included fabric which had been used where membrane was placed against soft or loose foundation soil.

Results of analyses are contained in the following summary:

Test on Membrane Asphalt

Penetration: 45
Softening Point: 123°F.
Ductility: Over 100 centimeters

Tests on Fabric		*ASTM Designation: D 173 Specified Limits*
Thread count, warp	30	26–32
Thread count, fill	21	24–32
Tensile Strength, Warp	72 pounds	Greater than 50 pounds
Tensile Strength, Fill	55 pounds	Greater than 50 pounds

SURVEYING

After the failure, Water and Power conducted surveys to determine the extent and direction of horizontal and vertical movement of all observation points previously established and observed at the site. This included: the inlet and outlet tunnels; gate tower; drainage inspection chamber; circulator lines; reservoir parapet wall; a line 20 feet north of the main dam axis; the 390 berm; the 340 berm; the piers at the base of the water tank south of the reservoir; and Manholes A and B.

The survey crews set straddler points along the trace of Fault I on the reservoir floor and made frequent measurements after the failure. They located all exploratory holes, trenches, and shafts; established vertical control to enable location of soil sampling points in the excavations and in the breach; and prepared a topographic map of the breach. They also determined elevations along a series of east-west lines on the reservoir floor for comparison with "as-built" conditions. Two selected cross sections are shown on Plate **23**.

Precise levels were run to check the elevation of control bench marks. These levels were run from PBM 40-C. (Plate 16)

EARTH MOVEMENT

Seismology

Seismograph records of the California Institute of Technology were obtained and reviewed by the Special Consultant to the Engineering Board of Inquiry, Dr. Pierre St.-Amand. The seismoscopes installed at the reservoir site were examined for indications of movement. A seismograph has been installed in the chlorinating station to detect post-failure tremors.

Groundwater Extraction

Department specialists made a study of potable groundwater extraction in the area as it might affect the Baldwin Hills Reservoir, and a report was made to the Engineering Board of Inquiry.

Oil Field Activity

The Board had discussions with representatives of the State Division of Oil and Gas, the Standard Oil Company of California, and others regarding activity in the Inglewood Oil Field. Data were obtained on petroleum production and repressurization.

INTERVIEWS

The Engineering Board of Inquiry also conducted interviews with persons having knowledge of Baldwin Hills Reservoir from its inception and with others having knowledge of the near vicinity. Individual Board members also interviewed professional persons informally. Transcripts were made of most of these discussions. Following is a list of those interviewed.

Personnel of the Department of Water and Power:

- Mr. Robert L. Brady, Resident Engineer, Water Engineering Design Division
- Mr. D. E. Bundy, Assistant Head, Water Engineering Design Division
- Mr. J. G. Cowan, Assistant Chief Engineer of Water Works
- Mr. O. N. Denman, Material Testing Engineer
- Mr. P. D. Doherty, Field Superintendent, Western District
- Mr. S. S. Green, Senior Engineer, Inspection and Research
- Mr. H. B. Hemborg, Principal Engineer, Project Engineering and Inspection
- Mr. R. E. Hemborg, Engineer in Charge, Water Operating Division
- Mr. E. D. Hoag, District Superintendent, Western District
- Mr. N. M. Imbertson, Head, Water Operating Division, retired
- Mr. C. J. Itter, Engineer of Design, Design Division
- Mr. W. J. Simon, Senior Engineer, Dams and Foundations
- Mr. L. E. Tabor, Specification Engineer
- Mr. W. Tate, Supervisor, Foundations and Structures Maintenance Section
- Mr. V. Vancott, Civil Engineering Associate, Water Design Division, retired
- Mr. F. E. Walley, Survey Supervisor
- Mr. R. Wells, Baldwin Hills Reservoir Keeper
- Mr. R. R. Wilson, Geologist
- Mr. B. A. Wright, Jr., Inspection Engineer
- Mr. G. Wyss, Assistant Head, Water Operating Division

- Mr. J. A. Lambie, Los Angeles County Engineer, and three of his staff
- Mr. A. J. Horn, Petroleum Engineer, Standard Oil Company of California
- Mr. F. J. Baudino, Los Angeles structural engineer, former geologist with Department of Water and Power
- Dr. Thomas Clements, Professor of Geology, University of Southern California
- Dr. J. C. Crowell, Head of Geology Department, University of California at Los Angeles
- Dr. Robert Stone, President, Stone Geological Services, Inc.
- Mr. David Campbell, former geologist with Department of Water and Power
- Dr. Carrol M. Beeson, Professor of Petroleum Engineering, University of Southern California
- Mr. H. Omstead, Architectural Planning Branch, Los Angeles City School Districts
- Mr. Perry Ehlig, Consulting Engineering Geologist
- Mr. J. V. Spielman, State Supervision of Dam Safety Office, retired
- Mr. R. E. Stephenson, State Supervision of Dam Safety Office

RECORD EXAMINATION

The Engineering Board of Inquiry requested and received selected records from the archives of the Department of Water and Power for examination and analysis. Excellent cooperation was given by the Department of Water and Power in the prompt furnishing of the requested data.

Rainfall

Records of precipitation for the period from July 1 to December 14, 1963 at the Baldwin Hills Reservoir station, maintained by Water and Power, were obtained. Total measured rainfall in this interval was 3.25 inches, of which 1.22 inch fell on November 20. There was no precipitation in the period from November 21 to December 14, inclusive.

Seepage

Water and Power seepage records for the reservoir underdrains and foundation drains were carefully examined. (Plates 24a through 24f)

Movement

Data pertaining to strain gage measurements on cracks in structures, instrument surveys on bench marks and horizontal control stations about the reservoir, aud data from foundation settlement measuring devices were carefully examined. Survey data on the subsidence area and reports pertaining thereto were also analyzed. (Plates 25a through 25e)

Construction

The Department of Water and Power final construction report, soils testing data, fill placing reports, coucrete cylinder records, daily inspection reports, "as-built" drawings, and miscellaneous letters, data, and reports were analyzed by the Engineering Board of Inquiry. Files of the Supervision of Dam Safety Office, State of California, were reviewed.

Operation and Maintenance

All of the Department of Water and Power operation and maintenance records embodied in special reports on drain treatments and associated studies, miscellaneous letters and memoranda, caretaker's daily log, water inventory data, and accounts of maintenance were examined.

CHAPTER IX

ANALYSIS OF FAILURE

To approach a comprehensive understanding of the mechanism of failure, it is necessary to examine conditions and events which have developed in the Baldwin Hills over a period of several decades. This includes manifestations of natural phenomena in the area as well as circumstances at the reservoir itself.

ANTECEDENT CONDITIONS

Facts available in the period prior to failure and warranting analysis are drawn from the characteristics of the dam and reservoir foundations, regional geology and earth movement, earthquakes, displacements at the reservoir, seepage, oil field activity, and groundwater extraction.

Examination of reservoir surveillance records relative to movements and leakage gives subtle indication that there were disquieting events occurring. New cracks had been discovered in the drainage inspection chamber; and there were changes in the seepage regimen.

Geology

Baldwin Hills Dam and the bowl of the reservoir were constructed on three sedimentary formations, largely of marine and littoral origin. The Palos Verdes formation, consisting of a loose pebbly sand, is exposed around the reservoir rim and forms the upper part of the abutments of the main dam. It is underlain by the Inglewood formation, composed of stratified sands, powdery silts, and clays of varying degrees of induration. The lowest formation in the abutments of the main dam is the Pico. This is largely a massive clayey to fine sandy silt, consolidated to the verge of being a siltstone, and including a few streaks of fine sand. These formations were found to contain loosely coherent members which exhibit little apparent strength and little capability to resist erosion. They are also cut by numerous joints.

Baldwin Hills are part of a seismically active zone known as the Newport-Inglewood uplift. This structure was the source of the Inglewood earthquake of 1920, the Long Beach earthquake of 1933, and numerous lesser shocks. However, investigation of recent seismic activities, including analysis of seismographic data, has shown that no recorded earthquake of sufficient intensity to cause inertial damage to the dam occurred within the life of the project.

The structure of the hills is a faulted, anticlinal dome. The most prominent fault, the Inglewood, displays evidence of late-Pleistocene displacement but no clear indications of appreciable offset within the last 10,000 years. (Plate 7)

Several lesser faults are associated with the Inglewood fault; and two of these, Faults I and V, transect the reservoir. These faults dip steeply; and, since they are planes of ancient rupture, they constitute weak zones in the foundation along which movement might be expected.

Inglewood Oil Field underlies Baldwin Hills, and its boundary extends to within a few hundred feet of the reservoir. The geologic structure of this field, the character of the oil-bearing deposits, the comparatively shallow depth of these deposits, and the solution gas drive that prevailed in the early stages of production provide a geological environment favorable for subsidence.

Earth Movement

Surveys of the Baldwin Hills indicate that earth movements have been taking place in both vertical and horizontal directions. The earliest reliable leveling was conducted in 1917; and, through comparison of the results of this and several subsequent surveys, it is estimated that a maximum of about 9 feet of subsidence occurred during the 1917-1963 interval. (Plate 15) The area affected resembles an elliptical bowl with its center situated about one-half mile westerly of the reservoir and its periphery extending to the east beyond La Brea Avenue. (Plate 16) Subsidence at the reservoir has aggregated about 3 feet during this period. Its southwest corner has dropped farther than the northeast corner, indicating a northeast-southwest tilt. Between 1947 and 1962 the elevation difference between these corners amounted to about one-half foot.

Triangulation surveys conducted in 1934, 1961, and 1963 have disclosed stations in the Baldwin Hills to be moving laterally in the general direction of the subsidence trough. Moreover, measurements of the dimensions of the reservoir indicate a progressive elongation of the northeast-southwest diagonal between 1950 and 1963 of about 0.4 foot.

Creation of the graben in the Inglewood Field required a stretching of the earth's crust, resulting in planes of rupture between which the block could drop. Thus, the directions of the primordial tectonic movements were outward, away from the axis of the graben and opposite in direction from the forces believed responsible for its formation. Therefore, it seems unlikely that the contemporary horizontal movements are a continuation of the tectonic deformations associated with the development of the graben.

By projecting the present subsidence rate of about 0.2 foot per year back in time, it is apparent that the 270 feet displacement measured in the east wall of the Baldwin Hills graben would have occurred in about 1,350 years. This rapid movement is not compatible with geologic evidence indicating insignificant movement along these faults during this interval.

There are similarities between Inglewood Oil Field and other oil fields which are known to be subsiding as a result of oil production. The geology, oil field pressure decline, high production rate, and relatively shallow production zone at Inglewood Field fit the subsidence syndrome as established at these other fields.

Earth Cracking

The cracks in the Baldwin Hills Reservoir are similar to earth cracks that have developed elsewhere in Baldwin Hills in recent years. The cracks are generally alike in lack of longitudinal displacement along the crack, extent of vertical offset, opening, and orientation. The cracks were formed near the periphery of the subsidence bowl in areas where tensile forces would be expected to develop as a result of down-warping of the surface sediments. In general, both the reservoir cracks and those in the vicinity are strikingly parallel to old faults and joints.

In view of the similarities noted, it appears probable that the cracking along Faults I and V in Baldwin Hills Reservoir and the earth cracking observed southeast of the reservoir have been caused by the same phenomena. Both appear to be related to the subsidence taking place in Baldwin Hills.

Displacement at Reservoir

Throughout the period of operation, displacement and cracking occurred in the reservoir and its appurtenances. It appears likely that some of these evidences of movement can be related directly to the chain of circumstances leading to failure. These would include:

1. Cracking of the drainage inspection chamber beginning in 1951.
2. Opening of the north parapet wall joint at Station 9+14.4.
3. Development of a crack across the lower roadway on the main dam at Station 8+93.5 in 1960 and accompanying opening of adjacent parapet wall joint.
4. Interruption of clay tile drains.
5. Cracking of the curb at the southeast rim of the reservoir.
6. Joint displacement in the circulator transition.
7. Settlement of the north and south circulator lines.
8. Distortion of the reservoir as manifested by elongation of certain of its dimensions.
9. Settlement near Station 25+50 on the south parapet wall.
10. Continuing settlement of the north parapet wall near Station 6+50.
11. Relative uplift of structures east of Fault I in the weeks immediately preceding failure.
12. Continuing downward movement of foundation settlement measuring devices beneath the main dam.

Other effects of movement which are not directly related to the failure, or which have questionable significance in analysis of causative factors, include:

1. Buckling of the asphaltic paving at the easterly inner toe of slope in 1951.
2. Settlement of the Elevation 418 channel inlet structure at the gate tower in 1951.
3. Cracks and surface slides on the crest and north face of the main dam in 1952.

Seepage

Throughout the life of the reservoir there had been problems of drain calcification which required treatment by flushing with hydrochloric acid and cleaning by mechanical methods. For a period of time, flow from the west toe drain contained asphalt, which gradually diminished and then virtually ceased.

The quantity of seepage through the reservoir bottom decreased from about 23 gpm at the beginning of service to about 9 gpm in early 1963. (Plate 24a) Although the general trend was a uniform decrease, there were substantial variations in the flow throughout the period. Beginning about April, 1963, the flow began to increase perceptibly from 9 to about 13 gpm at the time of failure. (Plate 24d) This increasing rate of drain discharge in the last year followed reported earth tremors of March 10, 1963. There was an earlier shake reported in the area on February 18, 1963.

Flows of horizontal drains discharging into Manholes A and B varied radically during 1963. (Plates 24e and 24f) It is possible that this drain flow variation was related to the tremors mentioned above.

Examination of some of the fragments of the asphaltic membrane underlying the cemented pea-gravel drain disclosed that there were small holes in it which could allow free passage of drainage water. The gross area of such holes is not known, but it is improbable that an area as large as this reservoir could have been covered with a thin asphaltic membrane without having unavoidable openings in it.

Oil Field Activity

Production at Inglewood Oil Field commenced in 1924, and between that time and the day of failure about 67,000 net acre-feet of oil and water were extracted from 611 wells contained within approximately 1,180 surface acres. An attendant drop in fluid pressure of about 520 psi occurred in part of the field. (Plate 9)

The field is known to contain nine producing zones ranging in depth from about 950 to over 10,000 feet. The most productive of these, the Vickers zone, is the shallowest major producing zone. Limits of production from that zone extend to within about 300 feet south of Baldwin Hills Reservoir, at which point the top of the zone is about 1,500 feet lower in elevation than the reservoir.

Three oil field repressurization projects have been started: Vickers East in 1954; Vickers West in 1962; and Rubel in 1960. The injection of brines into the oil zones as a part of this program has raised fluid pressures in the Vickers East pool about 170 psi. The natural drive mechanism for most of the field has been dissolved gas, a type of drive which leads to substantial fluid pressure lowering in spite of secondary recovery measures.

Groundwater Extraction

The formations underlying Baldwin Hills are devoid of significant quantities of potable ground water, and hence pumpage from water wells has never posed a threat of land subsidence in this area.

CONDITIONS ON THE DAY OF FAILURE

Significant deductions can be made regarding causative factors hy analysis of observations made on the day of the disaster. The Baldwin Hills Reservoir failure was unique in that professional engineers and others were present to witness and report on the events. Valuable data have been derived from this coverage.

General

There was no rainfall on December 14 or in the period immediately preceding which could have caused enough wetting of the earth to initiate land movements.

There were no earthquakes recorded at the California Institute of Technology on December 14 in the vicinity of Baldwin Hills. Some nearby residents reported earthquakes to interviewers subsequent to the failure, but no one reported them to the Los Angeles Police Department at the time of incidence. The authenticity of such reports is questioned.

Main Dam

Displacement of the main dam foundation during the failure evidently was confined narrowly along the trend of Fault I. It appears that the leakage which was discovered at about Elevation 400 in the east abutment of the dam was made possible by foundation rupture at the fault. The muddy water observed issuing from the abutment very probably had found an escape passage through the highly erodible and broken foundation materials along the fault zone.

Almost simultaneously with the development of noticeable abutment leakage, a crack opened across the crest of the dam. This might be attributed at its inception to foundation movement, but its final rapid enlargement was a secondary effect caused by the erosion of underlying materials.

Drainage System

On the day before the catastrophe, when the caretaker made his inspection, the reservoir and foundation drainage systems were reported to be functioning normally. Seepage water was reported to be clear and no change in rate of flow was apparent.

Conditions were radically different on the morning of December 14. Shortly after 11:00 a.m. the total drainage flow was observed to be several times normal, and the leaking water was muddy. It would be difficult to estimate exactly when the sudden change took place, during the night of December 13 or the morning of December 14. Actually, a precise pinpointing of this time is not essential to analysis of the failure. It suffices to know that the reservoir became seriously damaged in a period of a few hours before discovery of trouble.

The fact that the drains from the southeast toe, the northeast toe, and the fault were "blowing like fire hoses" is clear evidence of rupture at the east edge of the reservoir bottom. Undoubtedly most of the discharge detected by the caretaker in the spillway conduit under the main dam came from this source.

When the drainage inspection chamber was first examined on December 14, no new or increased rupturing of the structure was seen. Although the easterly three drains were ejecting abnormal amounts of muddy water, it did not appear at that time (soon after 11:15 a.m.) that there was any unusually large flow issuing from cracks in the chamber. Less than an hour later leakage had developed to such an extent that the inlet tunnel was discharging water onto the street system. The apparent lag in initiation of discharge from the structural cracks may be ascribable either to delay in final opening of the fractures or to the time required for leaking water to erode through the compacted earth backfill around the structure. Observers of the Department of Water and Power were of the general impression that the chamber cracks opened abruptly at about noon.

It would appear that the system of underdrains did afford a few hours of advance warning of the impending disaster by signaling that reservoir pressures had been introduced into the southeast toe, northeast toe, and fault drains.

CONDITIONS AFTER FAILURE

A thorough examination of the reservoir and its environs was made in the 90-day period subsequent to failure. The investigation focused on evidence of foundation and lining displacement and erosion, drainage system rupture, movement of appurtenant facilities, and recent general cracking in the area. A thorough understanding of these factors is essential to an adequate and useful analysis of the failure.

Foundation Characteristics

Examination of test excavations along Faults I and V, with attendant testing of representative samples, confirmed the poor quality of the foundation. A 0.15-inch rainfall on January 18, 1964, led to collapse of the various open trenches and shafts, attesting further to the erodibility of these materials.

In all of the excavations across Faults I and V it was found that the west side was down with respect to the east side. This was evident in the rupture of the asphaltic reservoir paving but was more pronounced in the rupturing of the cemented pea-gravel drain, where offsetting in excess of 7 inches was observed in some of the excavations. These displacements were generally extensions of rupture in the foundation.

No horizontal displacement occurred along the faults during their most recent movements, as indicated by absence of lateral offsetting in the clay tile underdrains.

Along Fault I cavities were found directly under the cemented pea-gravel drain in many of the excavations. There was evidence that many of these cavities had at one time been larger but had subsequently become filled with transported material, the apparent preponderant source of which was the compacted earth lining. It appears that the cemented pea-gravel had bridged these cavities until they became of sufficient width that the structural strength of the drain was exceeded. Then, as was revealed in Excavation No. 1, the pea gravel collapsed, resulting in filling the cavity with broken portions of the drain and with soil from the compacted earth lining. (Plate 22d)

In the north face of Excavation No. 3 a cavity large enough to admit a man was found about 8 feet below the pea-gravel drain. (Photos 83 and 84 and Plate 22f) It extended in a northerly direction along the west side of Fault V. Although the pea-gravel drain at the fault was displaced, it had not collapsed. There was slight calcification on the surfaces of the cavity, suggesting that the cavern had existed for some period of time.

There was also a cavity in the foundation at Excavation No. 13. To establish whether there was continuity between the cavities, water was introduced and found its way between the two excavations.

Excavation No. 11 exposed the tile drains crossing Fault V. The drains were broken near the fault line. (Plate 22j) It appears that reservoir water had seeped through the lining into the foundation at Fault V.

Excavation No. 4 also disclosed cavities existing in the natural formation as much as 47 feet below the pea-gravel drain adjacent to Fault I. It appears reasonably certain that these cavities had existed for some length of time, predating the day of failure.

Excavation No. 8 was made specifically to expose the assumed crossing of Fault I by the southeast toe drain. The compacted earth lining was carefully removed from the pea-gravel drain. It was found that the drain and membrane were ruptured at the fault line, with the west side down about ¼ inch. The fractured pea-gravel drain was carefully removed. An open cavity was found immediately beneath it, extending between the southeast toe drain and an adjacent sink hole in the reservoir floor. The cavity was approximately 1 foot wide and 6 inches high. The 4-inch clay tile in the southeast toe drain was broken at the fault line. (Photo 85) It was evident that this break had occurred at some time in the past because the broken surface had an incrustation of calcium carbonate. Photos 86 and 87 show a fragment of the 4-inch tile with the calcium deposit.

At various locations along the drift (Excavation No. 9) cavities were found at the fault line. (Photos 88 and 89 and Plate 22i) The comparatively smooth and straight fault planes showed no signs of erosion, suggesting that separation between fault blocks was the primary action in creating the gaps. Fine sand was commonly found in some of the open fault joints. In two locations along the drift, near the base of the breach, pieces of asphaltic membrane and pea gravel were found.

Reservoir Lining and Drainage System

The asphaltic paving was fractured completely across the reservoir along the trace of Fault I. The crack extended into the south crest roadway and disappeared a few feet south of the fence and curb line. There were major caved areas or sinkholes adjacent to the breach, over the drainage inspection chamber, and at the base of the south reservoir slope along the crack. There were a number of minor sinkholes on the same line. The west side of the crack was offset downward as much as 4 inches with respect to the east. Evidence of increasing displacement was the presence of ostracods clinging to part of the broken paving surface. The remainder exhibited the lustre of a fresh break, which undoubtedly occurred on the day of reservoir failure. This lends credence to the conclusion that the paving broke in at least two stages.

There was a minor crack in the reservoir floor at the south circulator line along the trace of Fault V. There were no sinkholes along this crack, but there had been flow through it.

In many of the test excavations it was difficult to detect visually shearing or tension disturbance through the compacted earth lining over or adjacent to the fault lines. In others the only evidence of lining failure was a zone of saturation which could be easily penetrated by a probe. However, there was a well-defined crack through the lining in Excavation No. 8.

Displacements at Faults I and V were characterized by the west side being down with respect to the east side. The reservoir bottom was constructed to slope to the east to provide a positive drainage gradient toward the drainage inspection chamber. It appears likely that minor offsets in the pea-gravel drain pre-existed at the fault lines and that seepage water was intercepted at the offsets. The amount of water so intercepted could have been small. However, it would have been concentrated by being forced to flow along the displacement until reaching places where it could escape into the foundation.

Examination of the 12-inch clay tile drain between Bore Hole BAH-4 and Manhole A showed the line to be clear, without evident disruption.

A study of the logs of the several borings and test excavations revealed that the thickness of the compacted earth lining varied from 8.5 to 10.2 feet at various locations in the reservoir bottom. Analysis of these variations does not indicate any correlation with areas of failure.

Reservoir Vicinity

Following failure, Fault I was traced across the floor of the reservoir to an exposure in Excavation No. 14, 250 feet south of the rim of the reservoir. North of the main dam the trace of the fault becomes obscure. A crack in the detention basin on the south side of Cloverdale Avenue was observed, but it could not be determined if this crack was associated with the fault.

FOUNDATION SETTLEMENT ANALYSIS

During construction of the main dam, foundation settlement measuring devices had been placed at nine selected stations along the 12-inch drain. (Plate 13)

The readings of these settlement devices were recorded periodically over the life of the project. Results are plotted on Plate 26 and selected data are tabulated in Table IX-1. Elevation datum is Bench Mark PBM 40-C, which has been determined to be subsiding.

It appears that this foundation settlement took place in about four years, with the longest settlement period approximately 55 months. However, the precise differentiation between foundation settlement under embankment load and that due to other causes is rather difficult to make at this site. The amount of the incremental settlement and its relationship to the total are evidence that there has been a substantial vertical movement other than related to the compression of loose foundation materials. As an example of this, for Settlement Device No. 2, where approximately 0.65 foot settlement occurred in the first three years, an additional 0.78 foot occurred in the succeeding period. The shape of the curves for Settlement Devices Nos. 2, 3, and 6 is revealing in that, once the effect of foundation consolidation begins to disappear, a definite continuing downward motion is apparent. The curves for Settlement Devices Nos. 1, 4, and 5, which are not included, have a similar shape. For Settlement Devices Nos. 7, 8, and 9 the trend is still downward, yet it is at a noticeably different rate. The curves also have a different shape, indicating that these settlement devices are reacting to a different combination of loads.

Analysis of results of Water and Power surface settlement observations at the reservoir indicates that a trough of major settlement had been developing along an alignment close to the trace of Fault V. (Plates 25a through 25c)

Settlement-versus-time data for selected reference points were replotted into semilogarithmic form where it is often possible to separate the settlement due to foundation consolidation from settlement due to other causes. (Plate 27) This type of plotting accentuates long-term effects as opposed to short-term effects. Set-

TABLE IX-1

FOUNDATION SETTLEMENT DATA

Settlement measuring device number	Stationing along 12-inch drain	Approximate height of fill, feet	Total settlement, feet	Approximate foundation settlement under fill load, feet	Incremental settlement, feet
1	3+67	95	1.05	0.20	0.85
2	4+58	157	1.43	0.65	0.78
3	5+49	154	1.45	0.80	0.65
4	6+39	127	1.23	0.65	0.58
5	7+29	106	1.08	0.65	0.43
6	8+21	105	1.08	0.55	0.53
7	10+45	91	0.65	0.40	0.25
8	11+37	98	0.45	0.20	0.25
9	12+28	70	0.35	0.15	0.20

tlement during the life of the reservoir was as follows: (Plate 5)

Reference Point	Settlement in Feet
South Parapet Wall, Station 25+50	0.80
South Circulator Pipe, Station 3+00	0.90
North Circulator Pipe, Station 3+50	1.18
North Parapet Wall, Station 6+50	0.94
20 Feet North of Dam Axis, Station 6+50	1.07
390 Berm, Station 9+50	0.82
340 Berm, Station 3+50	0.50

These points have undergone major settlement over the life of the reservoir; some of them are on the surface of the natural ground and some on fill, and yet they have all behaved in essentially the same way. It appears therefore that the compression of the fill is not the only significant factor in the overall surface settlement which has been measured.

The record for PBM 31, which is a bench mark located approximately 700 feet southerly of the reservoir at the location of the elevated steel water tank and which is located along an extension of Fault I, has been included for comparison only.

Instrument surveys of the gate tower, inlet and outlet tunnels, and drainage inspection chamber were made during the week after failure. With respect to measurements made on November 20, 1963, there was relative uplift through the inlet tunnel of 0.01 foot at the east portal, 0.11 foot at the tower, and 0.17 foot in the drainage inspection chamber easterly of Fault I. There was no measured change in the chamber elevation west of Fault I (Plate 25e). The top of the gate tower had moved to the east 0.12 foot.

Surveys made on January 1, 1964, showed an additional uplift of about 0.015 foot in the inlet tunnel since the survey of December 17, 1963. During the same interval the westerly end of the chamber underwent no further elevation change.

With the exception of the west end of the chamber, there had been an uplift between October 20, 1963, and November 20, 1963, of about 0.01 foot in the tower, the tunnels, and in the rest of the chamber.

The movement of the gate tower base (Plate 25d) had been consistently downward over the life of the structure, reaching a maximum of 0.55 foot before suddenly uplifting 0.11 foot at time of failure. A feature of the record is the jerky nature of the settlement plot indicating short-duration uplift superimposed on a steady downward trending curve. Such a condition could be accounted for by rapid release of strain energy which had been developed as a direct result of downdrag of the foundation mass west of Fault I on the foundation block to the east. As the settlement progressed, the strain energy could be built up to a point at which the frictional resistance across the fault surfaces was exceeded. A rapid slip could result. This could have continued until the day of failure, when the sudden release of the reservoir load was combined with the release of the strain energy. A measure of the resilient rebound of the foundation is indicated on the settlement curve during 1957, when about 0.03 foot rebound was measured during draining of the reservoir.

A low-density sedimentary foundation can be expected to compress appreciably when subjected to the load of a major embankment such as the main dam. Similarly, such a high embankment will compress internally under its own weight. Both of these kinds of movement will, however, decrease and cease with time. Both types of settlement have been measured by Water and Power at Baldwin Hills. Analysis of these data has indicated that:

1. Both the foundation and embankment settlements were well within the tolerances acceptable for good construction practice.

2. Most of the vertical movement which could be attributed to these causes virtually ceased in the early life of the reservoir.

3. Substantial continuing settlement for both surface and foundation reference points has been measured, and this settlement has followed the same pattern as that of bench marks located elsewhere in the subsidence bowl.

4. A trough of maximum settlement has been defined which crosses the reservoir in a north-south direction and which is parallel to and just westerly of the trace of Fault V. This zone curved slightly to the west between Manholes A and B. The settlement trough suggests that foundation deterioration was in progress along Fault V as well as along Fault I. (Plate 23)

5. Increasing vertical movement at Fault I in the few weeks immediately preceding failure was evidenced by reversal of the settlement trend of the gate tower and the structures immediately adjacent to it. This uplift was similar to that which had been occurring spasmodically during the life of the reservoir but was greater in magnitude than any encountered before.

ANALYSIS OF MOVEMENT IN STRUCTURES

Examination of the record of construction joint separation of the reservoir parapet wall showed progressive displacement at Station 9+14.4 beginning shortly after the reservoir was put into operation. The separation widened to about ¾ inch prior to the failure. An adjacent joint at Station 8+93.5 was opening in a similar manner, and it was at this location that the widening was greatly accelerated as the failure became complete on December 14. The south parapet wall and curb joint openings first observed after the failure were approximately on the trace of Fault I. These openings indicate progressive movement at the fault culminating in a rapidly accelerating displacement at about the time of failure.

Cracking of the drainage inspection chamber at Station 0+70.3 started in 1951. The crack at Station 0+89 was first observed in 1958, followed by the cracks at Stations 0+43 and 0+54 in 1960. During the month of November, 1963, each of these cracks opened appreciably. Table IX-2 presents crack data for the year preceding failure. It was not necessary to have large forces to open these cracks since the chamber had only minimal temperature reinforcement in the longitudinal direction.

The rotation of the middle broken section of the chamber was probably due to nonuniform weakening of the structure foundation, combined with the relative uplift east of Fault I.

Strain gage readings at Spillway Stations 3+10 and 3+33 show continuously increasing displacement. The spillway in this reach was lost in the breaching of the dam. The cumulative strains were in the order of ¼ inch maximum.

An examination of the steel shell in the connector conduit between the tower and the circulator lines was made by the Corrosion Specialist, Department of Water Resources, for evidence of progressive settlement in the connector. The two construction joints in the connector concrete encasement had undergone vertical and longitudinal displacement. The report of this investigation states:[1]

"Examination of interior surfaces of the tower connection revealed a crack in the cement mortar lining, around the periphery of the pipe, at the flexible joint nearest the outlet tower. This crack . . . varies in width from 1 inch at the bottom of the pipe to 2 inches at the top of the pipe. A ¼-inch wide crack was also found in the cement mortar lining around the periphery of the pipe at the flexible joint farthest from the outlet tower. At the time of original installation, the gap which existed in the cement mortar lining at the joints, was filled with hand placed cement mortar . . . Sufficient evidence was present to determine that hand-placed mortar completely filled the gap in the lining when originally installed. Therefore, the presence of cracks in the cement mortar lining at the flexible joints is positive evidence that movement has occurred since installation of the tower outlet. The 2-inch crack at the flexible joint nearest the outlet tower indicates that considerable movement has occurred at this point.

"A build up of corrosion products was found on the interior surface of the steel bell ring exposed at the base of the 2-inch crack in the cement mortar lining . . . The circumferential band of steel exposed by movement of the joint was 2 inches wide and corrosion products covered 1½ inches of this width. A ½-inch wide strip, adjacent to the spigot end of the joint was completely free of corrosion products . . . The absence of corrosion products on this area indicates recent movement of the joint. If this area had been exposed to reservoir water for a long period of time, we would expect a build up of corrosion products similar to that found on the remaining area of steel. However, since the ½-inch steel strip was completely free of even minor corrosion, it is concluded that this area was exposed by recent movement of the joint which occurred immediately prior to, during, or after dam failure.

"The remaining 1½-inch width of exposed steel shows a build up of corrosion products to a height of ¾-inch from the steel surface. This would require exposure of the steel surface to the corrosive influence of the water for a long period of time. . . . We cannot determine the actual exposure time involved in this formation, however, based upon past

[1] Ellis, W. J., Memorandum report to Robert B. Jansen. "Baldwin Hills Dam and Reservoir—Tower Connection Corrosion Inspection." April 10, 1964.

TABLE IX-2

DRAINAGE INSPECTION CHAMBER
STRUCTURAL CRACKING DATA

Date of measurement	Total crack opening				
	Station 0+43, North top, inches	Station 0+54, South bottom, inches	Station 0+70.3		Station 0+89, Bottom average, inches
			Top average, inches	Bottom average, inches	
December 19, 1962	0.028	0.035	0.441	0.140	0.225
January 25, 1963	0.032	0.039	0.447	0.144	0.227
February 20, 1963	0.031	0.037	0.457	0.145	0.235
March 25, 1963	0.033	0.041	0.464	0.149	0.241
April 26, 1963	0.039	0.051	0.468	0.154	0.231
May 24, 1963	0.035	0.053	0.470	0.154	0.238
June 25, 1963	0.037	0.049	0.465	0.146	0.238
July 26, 1963	0.032	0.046	0.453	0.146	0.242
August 30, 1963	0.031	0.054	0.457	0.146	0.234
September 27, 1963	0.031	0.046	0.455	0.157	0.236
October 30, 1963	0.022	0.045	0.455	0.146	0.234
November 26, 1963	0.070	0.088	0.488	0.202	0.275

Source of Data: L.A. Dept. of Water and Power.

experience, I would feel that a period of 5 years or more would be a conservative estimate. . . .

"Heavy calcium deposits were found at one location within the joint crack of the lining near the top of the pipe. This deposit was approximately 1 inch in thickness and extended from the steel surface to the interior surface of the cement mortar lining. The amount of calcium deposited in this area indicates a considerable period of time involved. Actual time required for formation of the calcium would be difficult if not impossible to determine, however, it serves to substantiate the conclusion that the crack in the lining has existed for a number of years." (Photos 90 and 91)

The measurements of displacement of the appurtenant structures, including the corrosion analysis of the circulator connector pipe, indicate that the movements leading to failure had been occurring over a substantial period of time with a final movement on or about the day of failure.

CHAPTER X

CONCLUSIONS

The events and conditions which have been analyzed now can be categorized as to their relative significance as contributing influences in the process of destruction.

FACTORS NOT RELATED TO FAILURE

Evidence examined by the Engineering Board of Inquiry indicates that the following factors did not have any significant causal relationship to the failure of the reservoir.

Extraction of Potable Ground Water

Records show that no potable groundwater withdrawal of consequence has been made from the formations underlying the reservoir. Evidently, none of the land subsidence affecting the facility stemmed from this source.

Earthquake on Day of Failure

Although several people located south and southwest of the reservoir have reported feeling tremors of uncertain source during the week of failure, there is not any recorded instrumental evidence of an important earthquake in that period. It is possible that mild tremors from time to time during the life of the facility may have contributed in a minor way to its deterioration. But it seems reasonable to conclude from the available facts that the failure was not primarily of seismic origin.

Massive Landslide

This investigation did not uncover any evidence which would link the failure to landsliding of a substantial mass of the Baldwin Hills due to slope instability. Local landsliding has been observed in the area for many years, and evidences of current movement of limited extent can be seen in various locations around the reservoir. Neither kind of sliding is regarded as of importance in this analysis.

EVOLUTION OF FAILURE

General earth movement in the area of investigation can be characterized as a widespread, long-term tectonic activity upon which has been superimposed a relatively rapid subsidence confined to a limited area close to the reservoir.

Evidence indicates that the primary actuating movement which led to the failure of the Baldwin Hills Reservoir stemmed from land subsidence in the vicinity, with the center of depression located southwesterly of the reservoir. This movement set up tensile stresses in the earth's surface at the reservoir, which lies practically on the rim of the subsidence bowl.

The foundation of the reservoir is broken by several faults which are probably associated with the nearby Newport-Inglewood fault system. These faults, trending generally in a north-south direction, represent planes of weakness in the foundation at which tensile stresses such as those developed by subsidence might be expected to find relief.

Stretching of the ground surface in the vicinity apparently resulted in opening of the foundation at these faults and dropping of the foundation blocks simulating a staircase descending to the west. There is little doubt that the subsidence began long before construction of the reservoir and continued throughout its life.

Opening of the foundation faults and the accompanying vertical displacement at these lines of weakness evidently led to rupture of the asphaltic membrane, the pea-gravel drain and clay tile pipes underlying the reservoir. Vertical offsets in the drain system would form, in effect, miniature barriers to the seepage flow from the west side of the reservoir since the floor of the reservoir slopes downward to the east. The displacement, with the east side of offset higher than the west, would provide a ready means of interception of leakage and for diversion of the flow into the foundation along the lines of maximum foundation weakness.

Although there was probably some leakage occurring into the reservoir foundations generally, the most significant and most damaging leakage was concentrated at the foundation faults, where the greatest hazard existed. It is regarded as quite possible that long-term leakage into the faulted foundation during the years of reservoir operation resulted in creation of cavities not only immediately beneath the reservoir lining but at depth in the foundation at the faults. It appears that such cavities were formed by leaking water moving downward into the faults, causing particle readjustment and erosion of foundation materials into the fault openings created by earth tension.

The lining of the reservoir functioned in such a way that the asphaltic surfacing and the compacted earth lining effectively concealed the disruptive process taking place beneath in the pea-gravel drain, the clay tile pipes, the asphaltic membrane, and in the foundation. The pea-gravel drain and the clay tile pipes and their encasement were relatively rigid elements which would tend to span the cavities which were probably developing in the immediately underlying foundation.

At the time of failure, it appears that comparatively rapid opening and vertical displacement occurred at the faults in the foundation of the reservoir, such movement being of sufficient magnitude to open cracks through the entire thickness of the reservoir lining and to cause its collapse into foundation cavities immediately beneath the lining. This introduced essentially full reservoir pressures into the underdrain system and into the foundation. The general foundation movement involved only a small fraction of a foot of vertical displacement at the faults, as evidenced by measurements north and south of the reservoir following failure and also by relative uplift of the gate tower and the easterly part of the drainage inspection chamber. The remainder of the observed vertical offset in the reservoir lining, and also the final rupturing of the chamber, can probably be attributed primarily to general weakening of the foundation at the faults due to water action.

Once leaking water was admitted to the reservoir foundation in great quantities, failure of the facility was inevitable. The general weakness and fracturing of the foundation at the faults enabled rapid erosion along short lines of percolation into the drainage inspection chamber and through the east abutment of the main dam. Erodibility of the dam foundation with its laminations of loose, fine sands and silt contributed to the acceleration of the failure process.

THE FAILURE IN RETROSPECT

It appears that the stage for destruction of the Baldwin Hills Reservoir was being set even before conception of the facility. Although the significance of natural and man-made phenomena as potentially dangerous factors would have been difficult to recognize at Baldwin Hills 20 or 30 years ago, the forces which ultimately led to trouble were in motion then.

The experience of December 14, 1963, has demonstrated forcefully again that a structure is only as strong as its weakest essential component. Geologic elements at the Baldwin Hills site would provide a marginal foundation for any open facility for confinement of water.

Aware of the inherent deficiencies of the formations at the site, the designers and builders of the reservoir attempted to assemble an integral structure which would meet all the requirements for stability. Their work was a success as long as the reservoir and its foundation remained together and functioned as a unit. But powerful and inexorable forces were acting to disrupt this harmony.

Sitting on the flank of the sensitive Newport-Inglewood fault system with its associated tectonic restlessness, at the rim of a rapidly depressing subsidence basin, on a foundation adversely influenced by water, this reservoir was called upon to do more than it was able to do.

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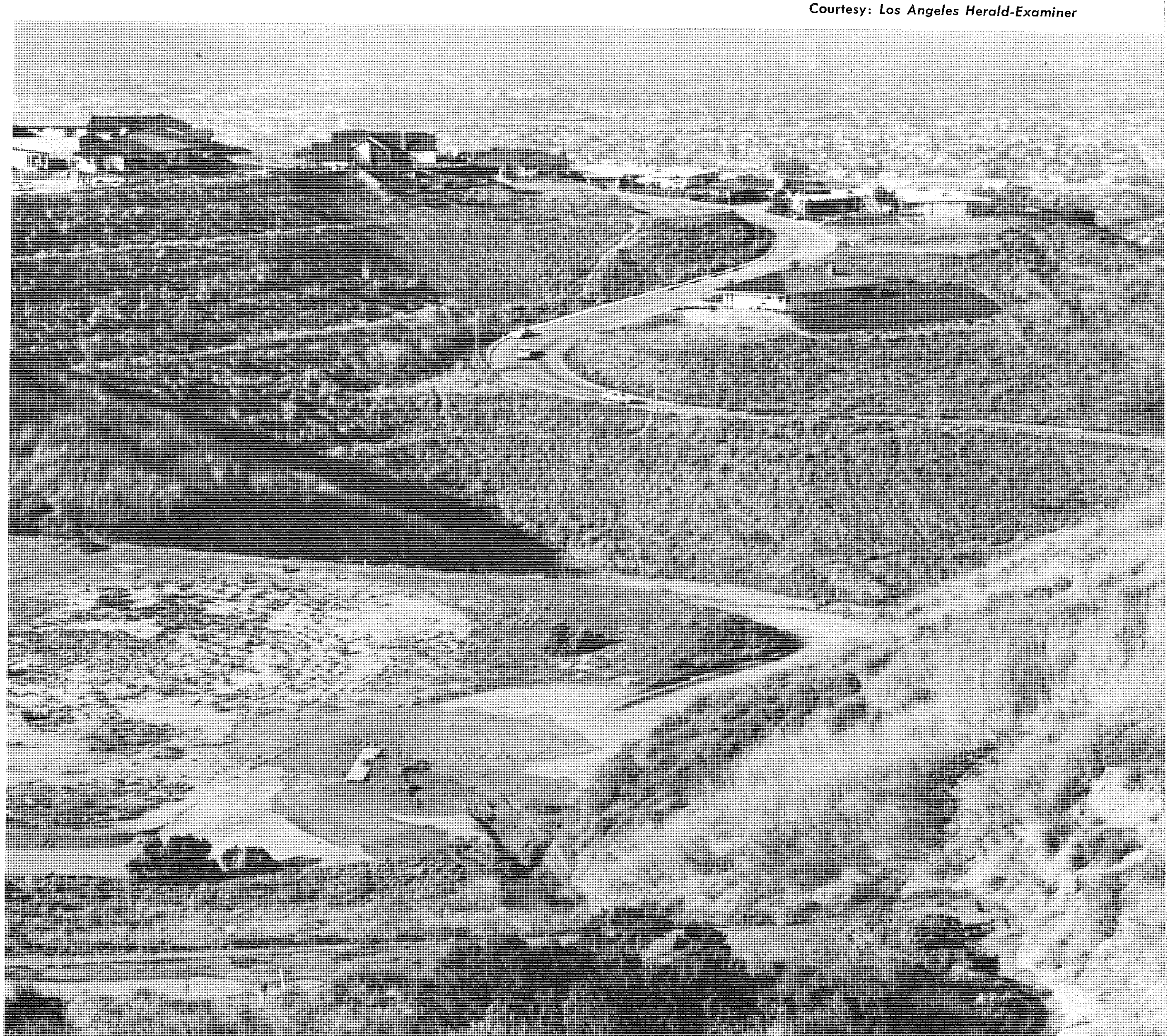
PHOTOGRAPHS

Photos 1 through 4 show the progressive failure of the dam on its downstream side.

PHOTO 1. Discharge at east abutment of the main dam at about 2:15 p.m. Note flow entering catch basin.

Courtesy: *Los Angeles Herald-Examiner*



Courtesy: *Los Angeles Times*

PHOTO 2. Flow spreading on Elevation 340 berm. Estimated time 2:30 p.m. Note submergence of catch basin and cavities developing at gutter.

PHOTO 3. Accelerating discharge at about 3:30 p.m. Observe ponding at street embankment.

Courtesy: *Los Angeles Times*

PHOTO 4. Flow through breach during final stage of failure, at about 4:00 p.m. Spray caused by material falling into flow.

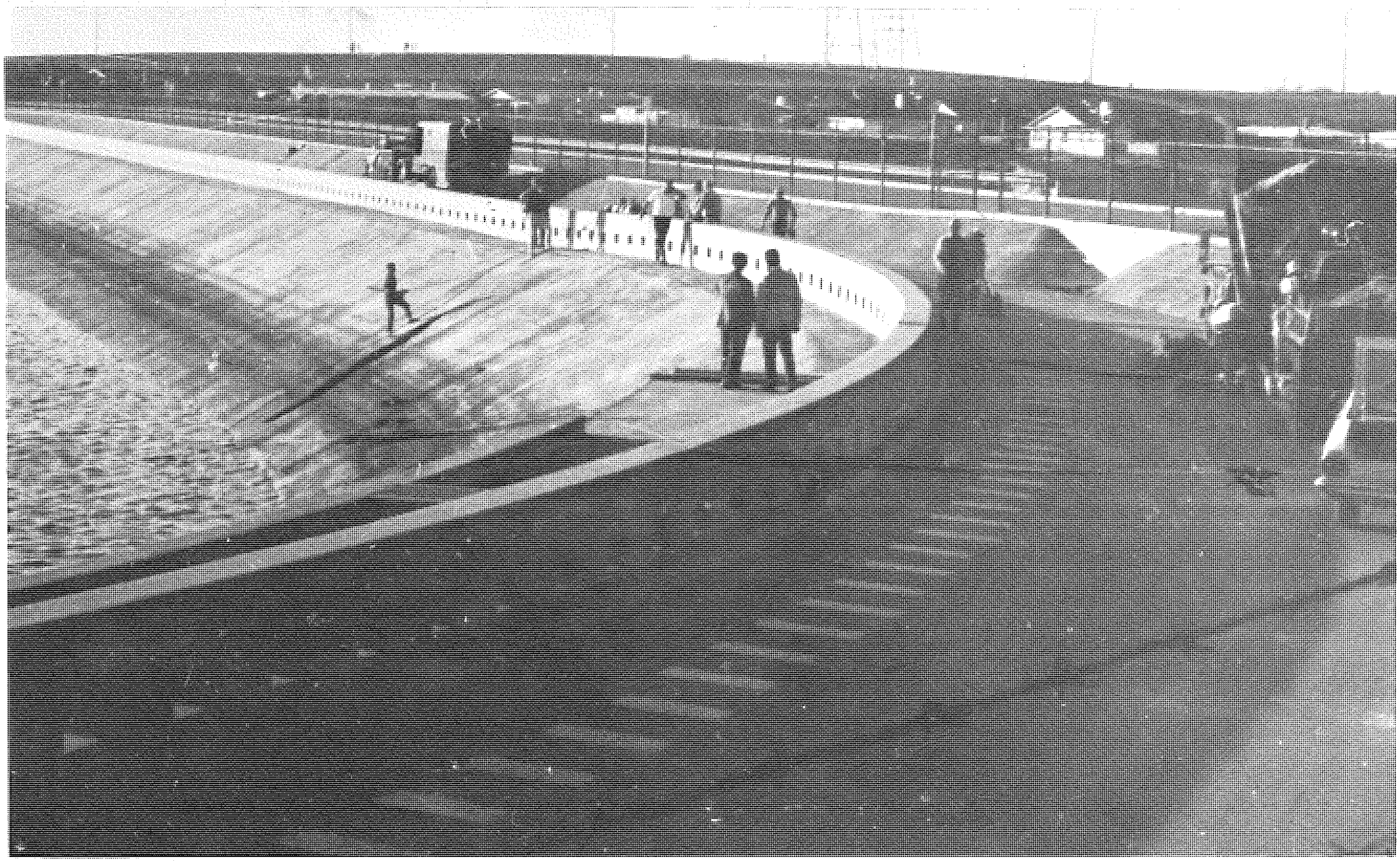
Photos 5 through 12 show the progressive failure of the dam on its upstream face.

Courtesy: *Los Angeles Herald-Examiner*

PHOTO 5. Discovery of hole on face of main dam at approximately 2:20 p m.

PHOTO 6. Attempt at sandbagging hole in main dam, at about 2:50 p.m.

Courtesy: *Los Angeles Times*

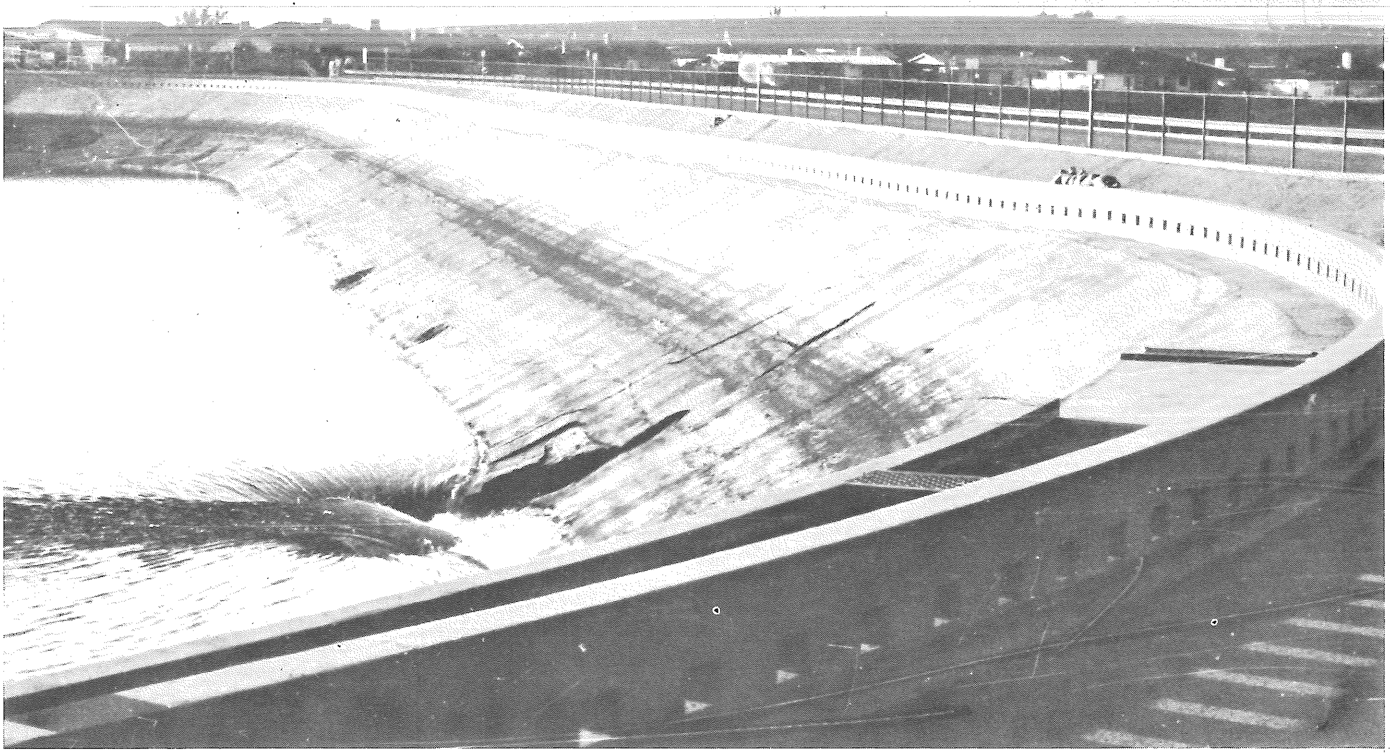


Courtesy: *Los Angeles Times*

PHOTO 7. Efforts toward plugging hole proved futile. Note lowering of water level in race to empty the reservoir. Estimated time 3:00 p.m.

PHOTO 8. Appearance of a vortex on the water surface indicating the start of a major breakthrough, at about 3:20 p.m.

Courtesy: *Los Angeles Times*

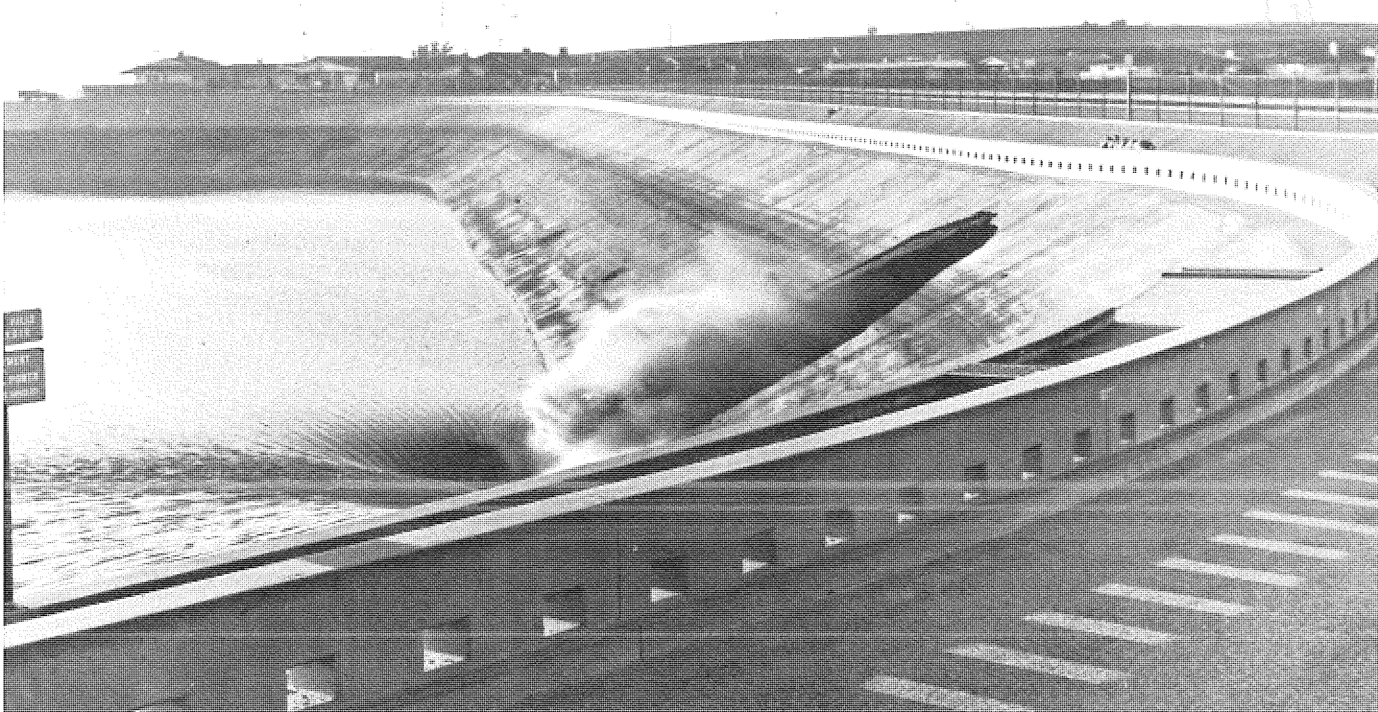


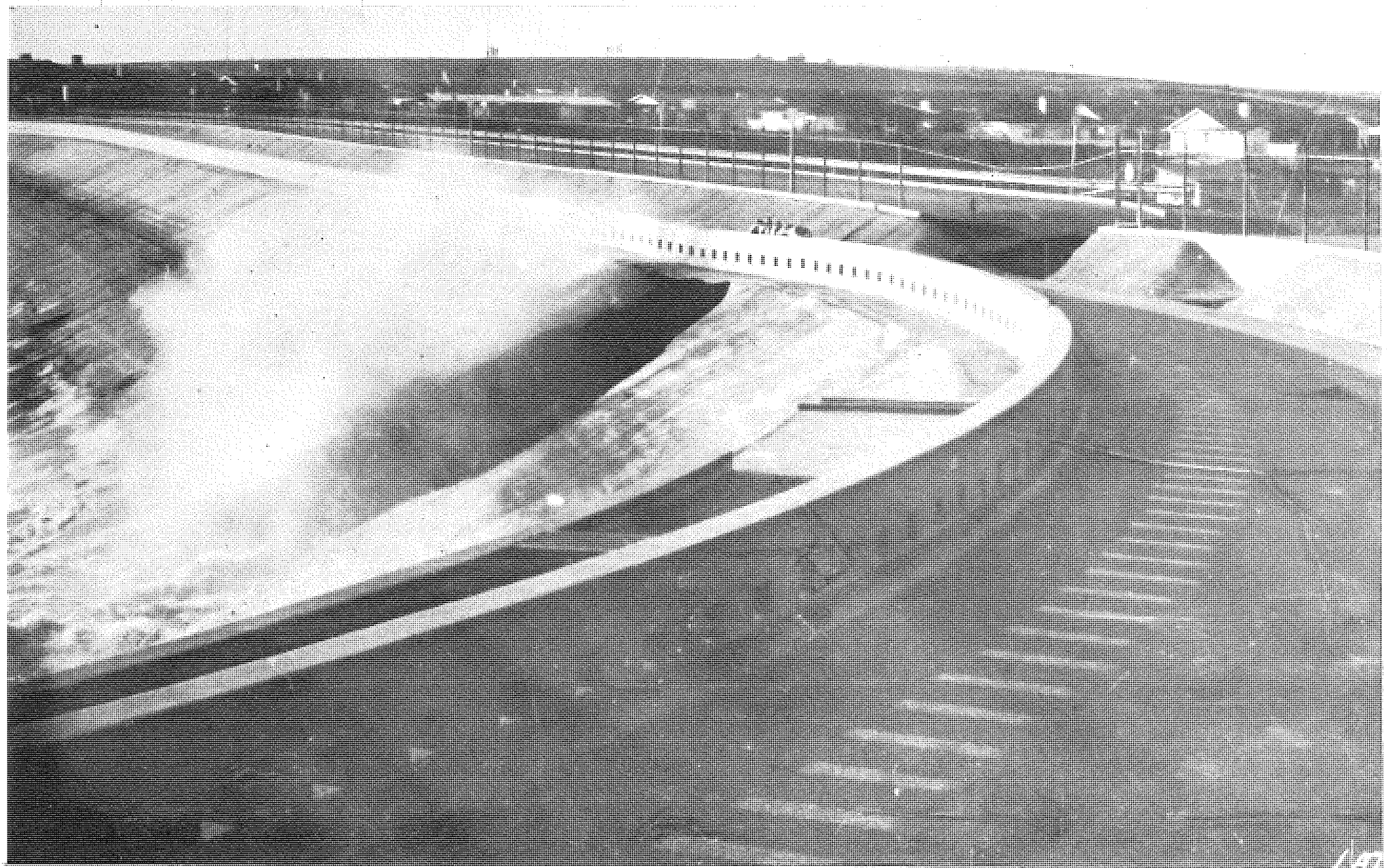
Courtesy: *Los Angeles Times*

PHOTO 9. Widening of the break at approximately 3:25 p.m.

PHOTO 10. The break, at about 3:30 p.m.

Courtesy: *Los Angeles Times*



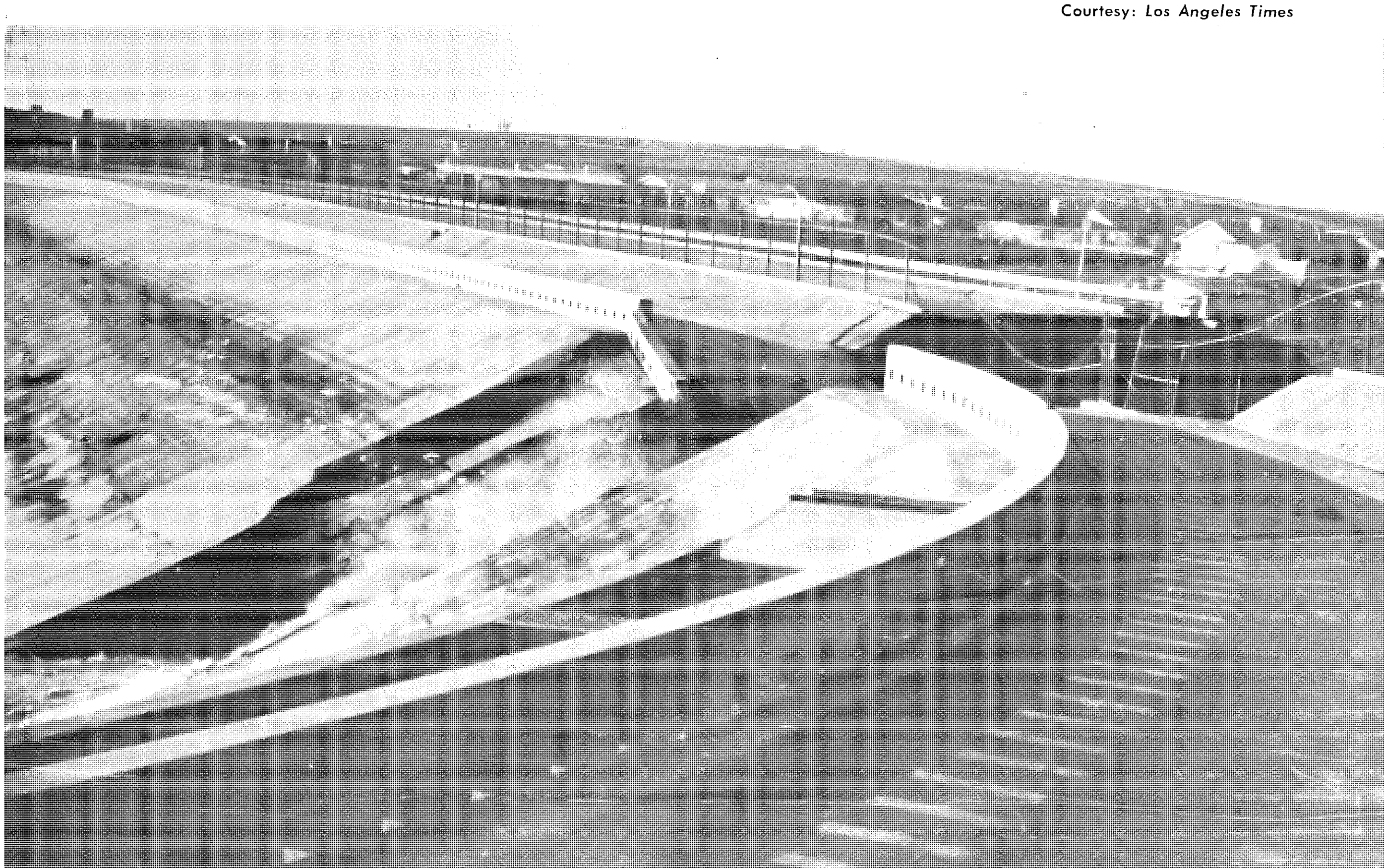


Courtesy: *Los Angeles Times*

PHOTO 11. Dam crest roadway and parapet wall starting to collapse at about 3:35 p.m.

PHOTO 12. Total collapse of the dam crest roadway at 3:38 p.m.

Courtesy: *Los Angeles Times*

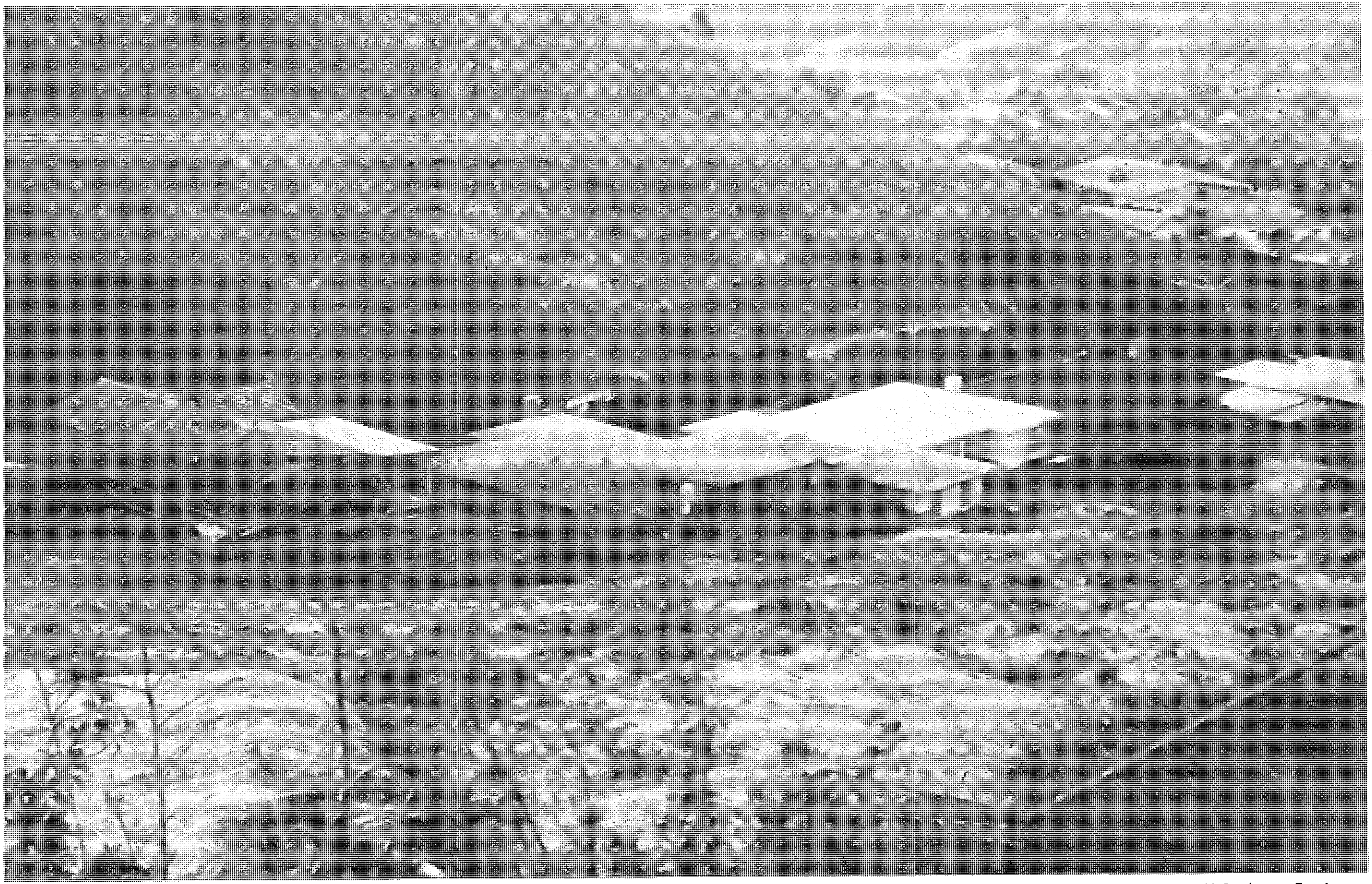


Courtesy: *Los Angeles Herald-Examiner*

PHOTO 13. Crack at crest of main dam near east abutment.

PHOTO 14. Discharge following dam crest collapse, at approximately 3:40 p.m.

Courtesy: *Los Angeles Times*



Courtesy: *U.S. Army Engineers*

PHOTO 15. Cloverdale Avenue after flood crest. Homes on right side of street washed away.

PHOTO 16. Breach with failure complete and reservoir almost empty.

Courtesy: *Los Angeles Times*

Courtesy: *Los Angeles Times*

PHOTO 17. Aerial view of the reservoir at about 4:00 p.m.

PHOTO 18. View looking north toward reservoir. Note physiographic expression of Inglewood fault indicated by arrows.

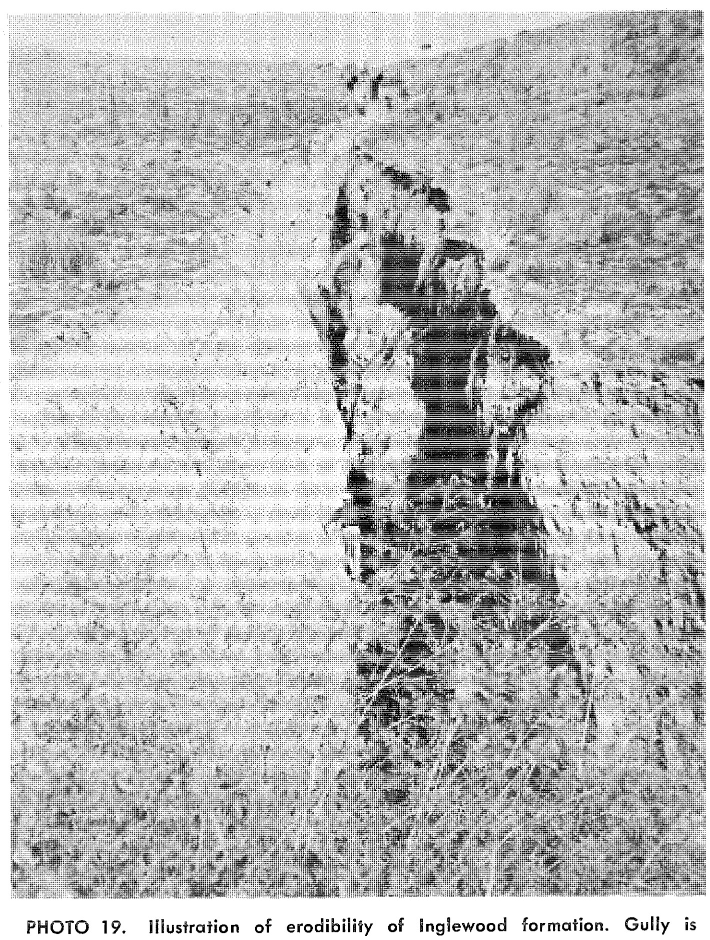


PHOTO 19. Illustration of erodibility of Inglewood formation. Gully is about 40 feet deep.

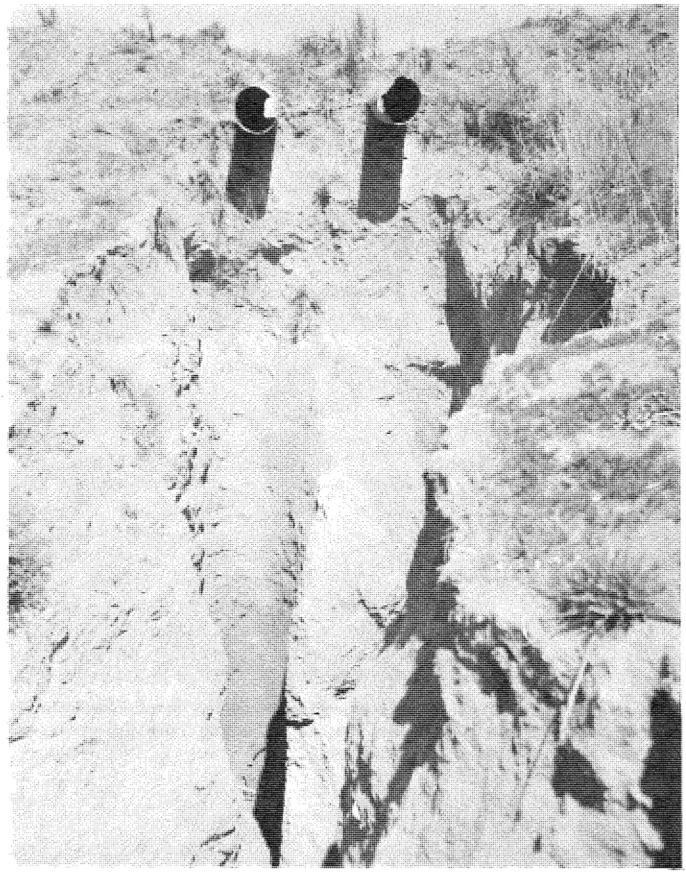


PHOTO 20. Head of the gully shown in Photo 19. Erosion is due to runoff from road above culverts.

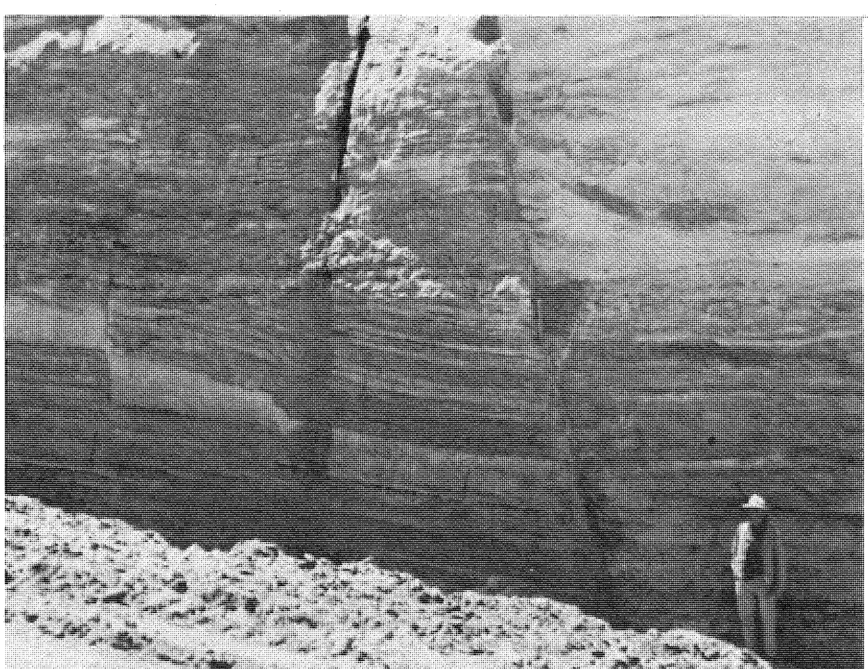


PHOTO 21. South face of an excavation into zone of Fault I near gate tower site, April 29, 1948. Total offset across the four faults shown estimated in excess of 30 feet.

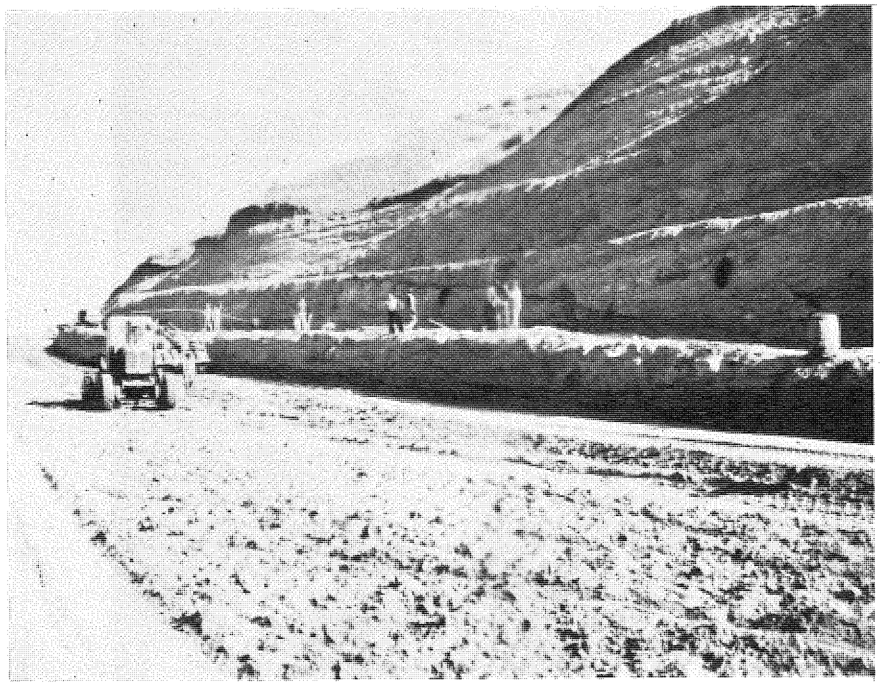


PHOTO 22. Slide area in east abutment looking northeasterly. Men shown are drilling horizontal drain holes into the abutment below the slide. Sloping dark line above men is clay slip-bed.

PHOTO 23. Close-up of slip-bed showing the magnitude of movement, which measured a maximum of 0.43 foot.

PHOTO 24. Foundation drainage details—Cradles for 4-inch and 12-inch tile and copper tubing for settlement measuring devices.

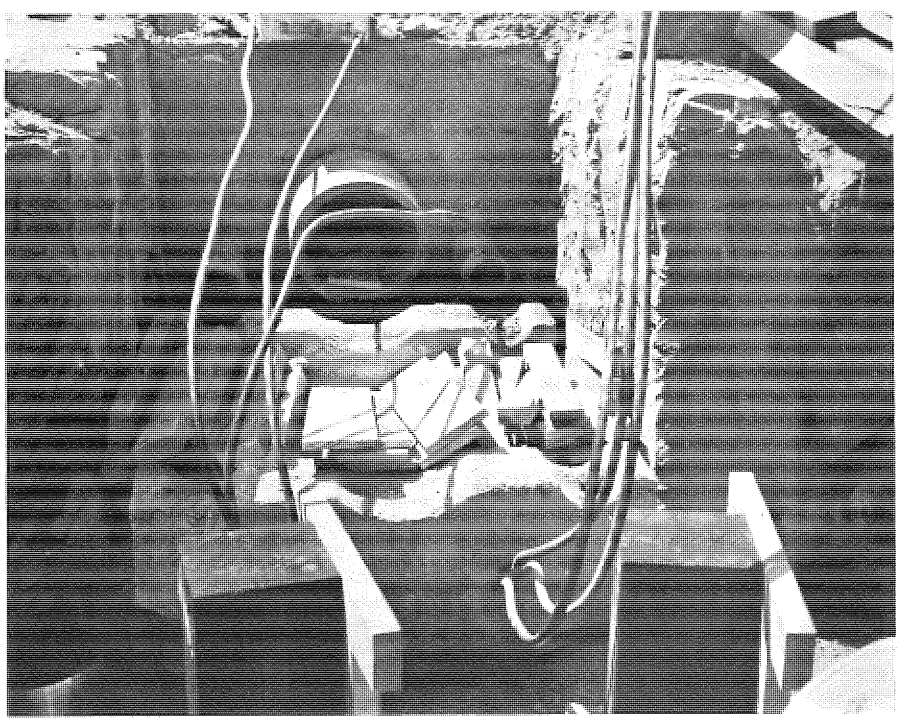


PHOTO 25. Looking upstream from Manhole A, showing 12-inch and 4-inch tile drains and copper tubing.

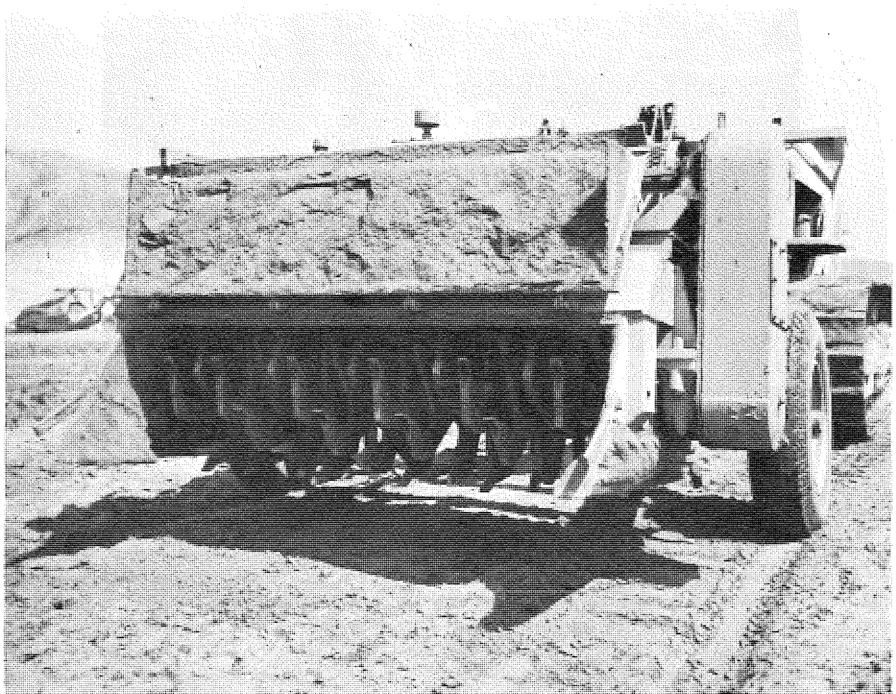


PHOTO 26. Wood's Pulvimixer.

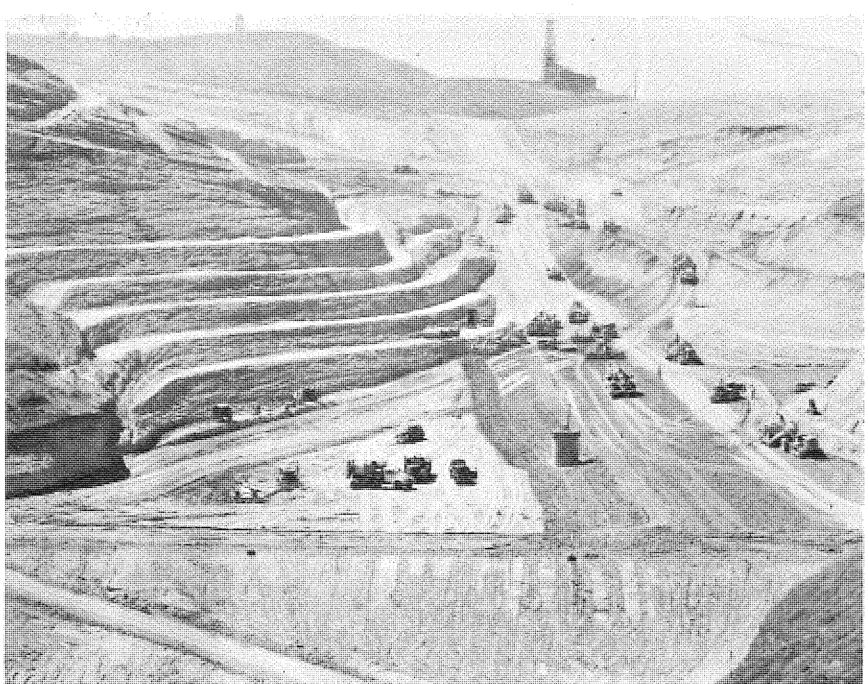


PHOTO 27. Looking upstream, showing benching on east abutment of main dam, Manhole A, 42-inch spillway pipe and catch basin on Elevation 340 berm.

PHOTO 28. Looking upstream at main dam. Note benching of east abutment above Elevation 340 berm, also catch basin and Manhole B at Elevation 250 berm.

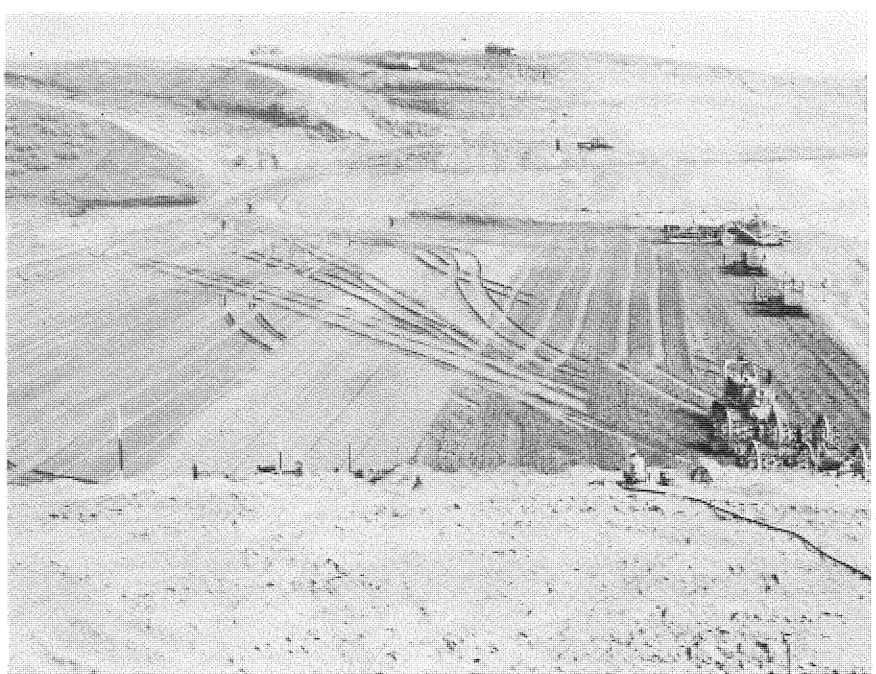


PHOTO 29. Looking east at rolled fill in main dam.

PHOTO 30. Winch tractor and motor grader trimming inside slope.

PHOTO 31. Fabric-reinforced asphaltic membrane on west slope.

PHOTO 32. West toe drain ditch after placing fabric reinforcing.

PHOTO 33. Rolling loose pea gravel into asphaltic membrane.

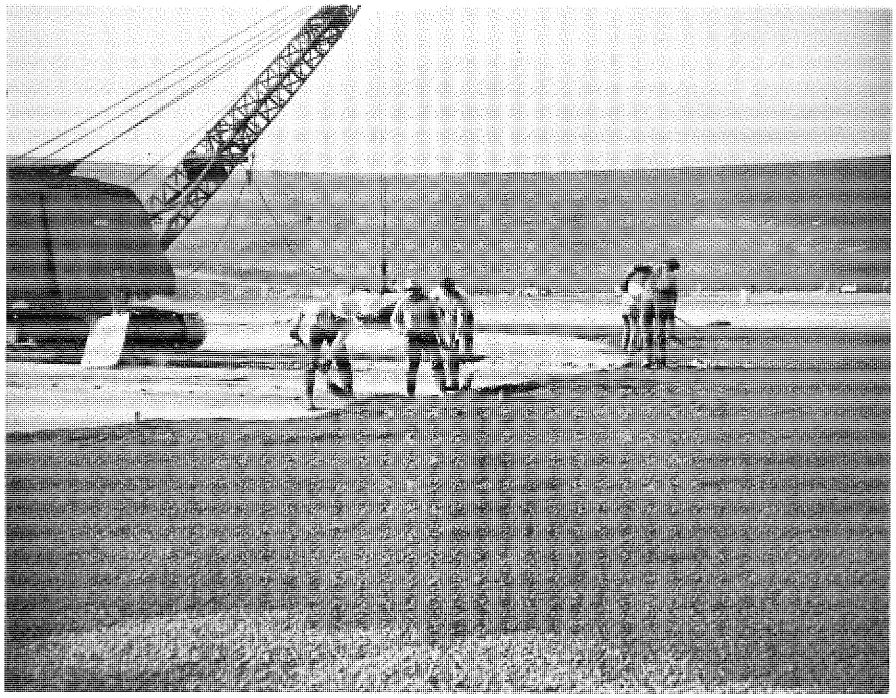


PHOTO 34. Spreading cemented pea gravel. Note grade stakes apparently driven through asphaltic membrane.

PHOTO 35. Kemper pea-gravel slope spreader.

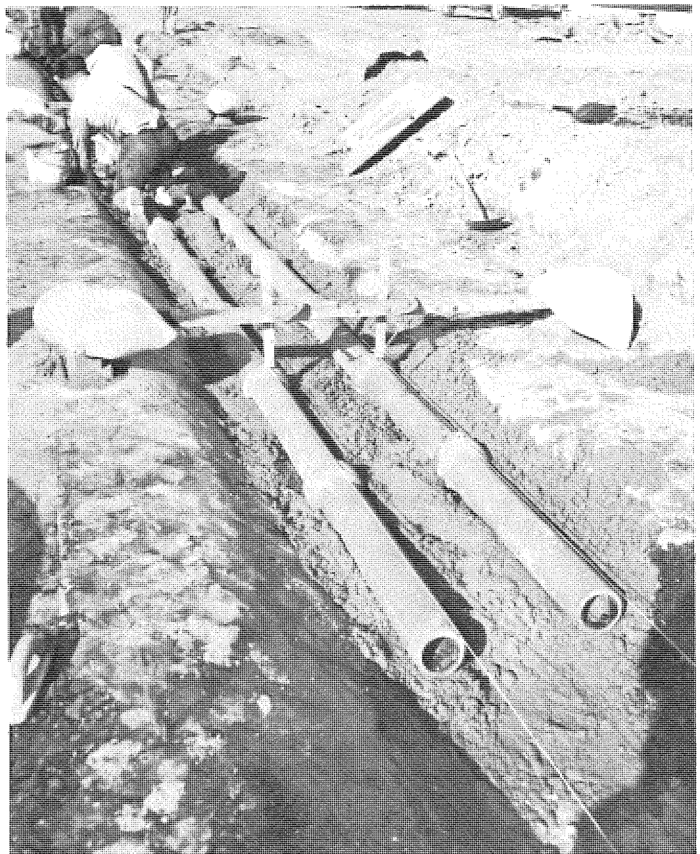


PHOTO 36. Installation of 4-inch tile drains in reservoir bottom.

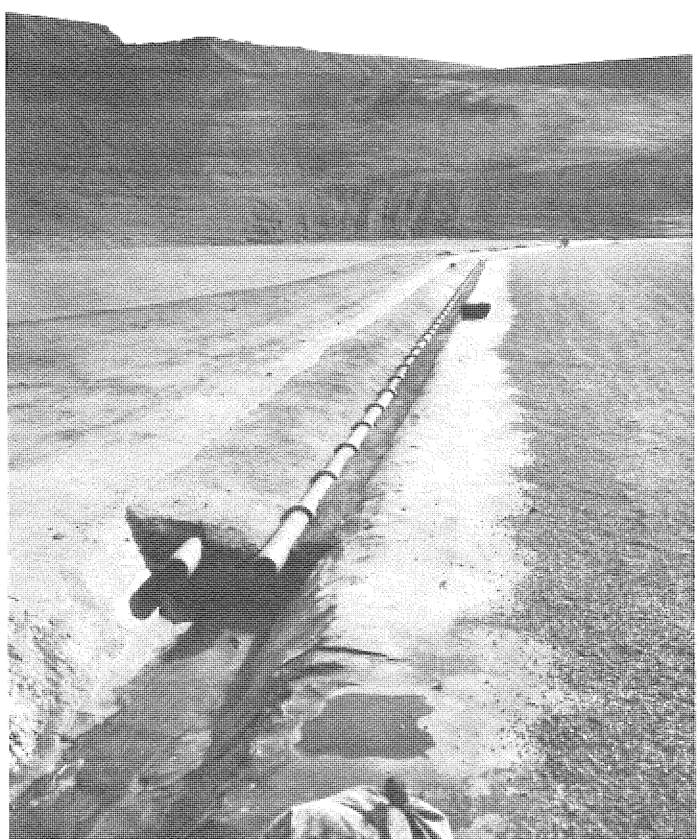


PHOTO 37. Installation of 4-inch tile drains. Note one line completely encased in concrete and other with open joints on top.

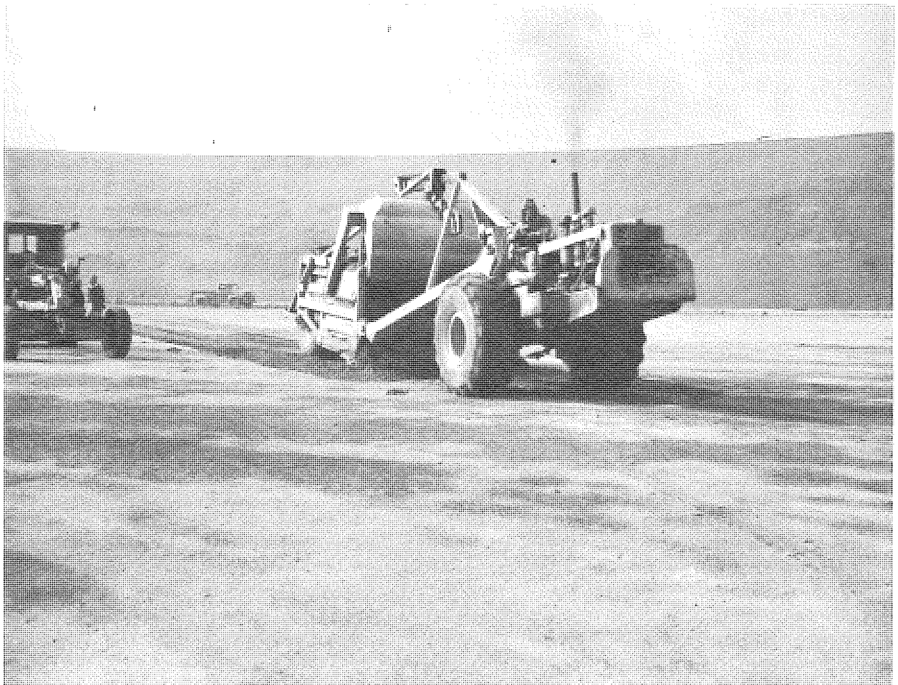


PHOTO 38. Scraper spreading first layer of earth on cemented pea gravel. Note equipment operating directly on surface of pea gravel.

PHOTO 39. Motor grader spreading first layer of earth on cemented pea gravel.

PHOTO 40. Crack in 4-inch cemented pea gravel discovered during construction.

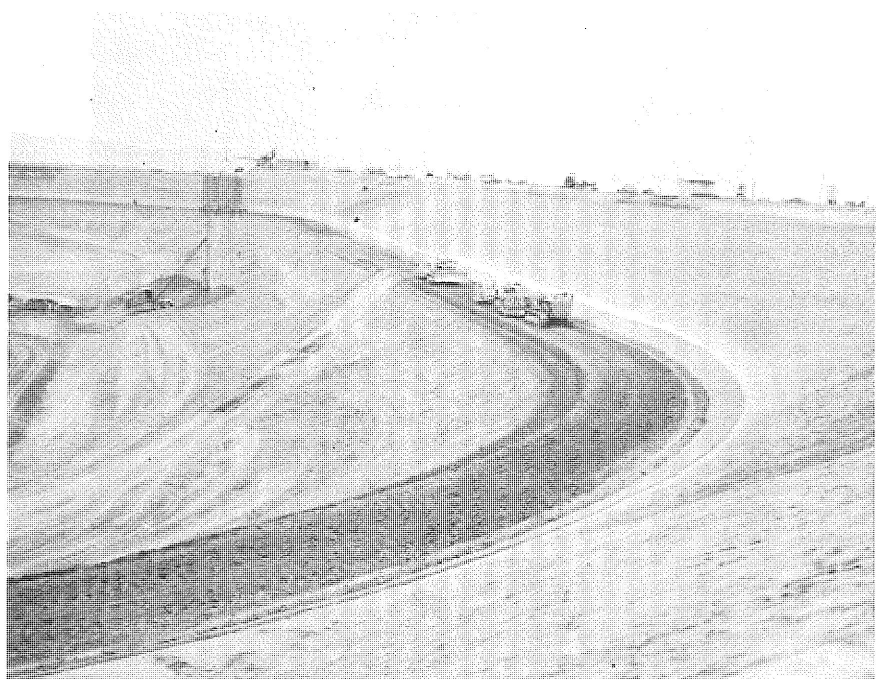


PHOTO 41. Placing compacted earth lining on inside slopes.

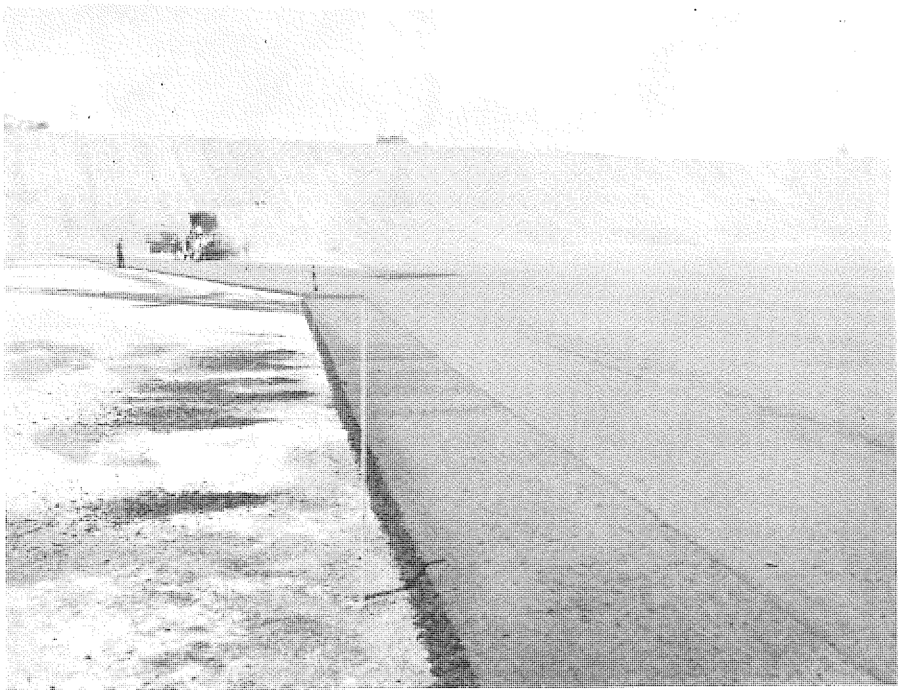


PHOTO 42. Paving reservoir bottom.

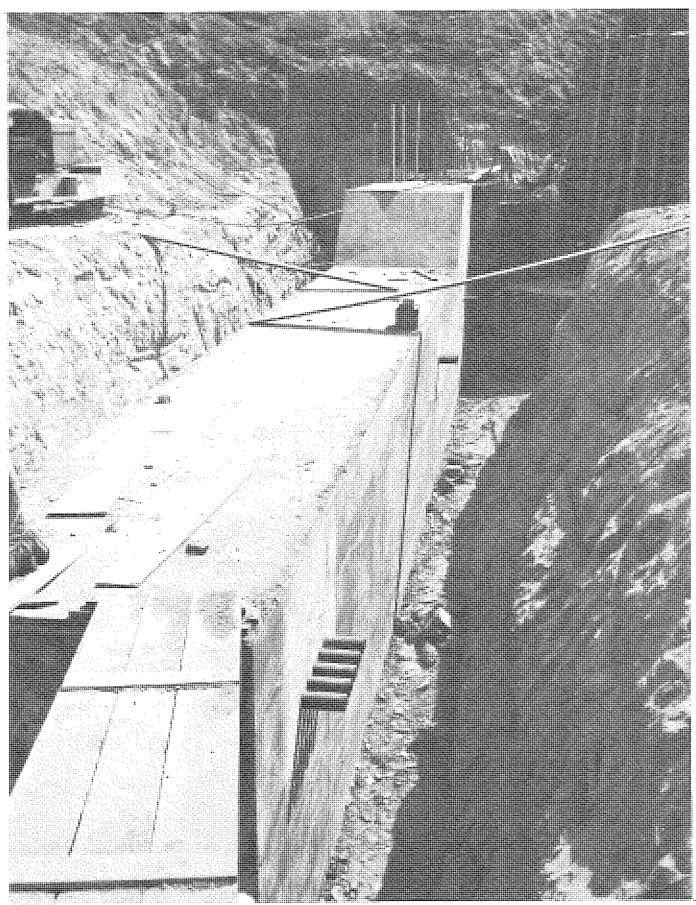


PHOTO 43. Drainage inspection chamber. Note pipes for drain lines.

PHOTO 44. Junction of 24-inch blowoff and inspection chamber.

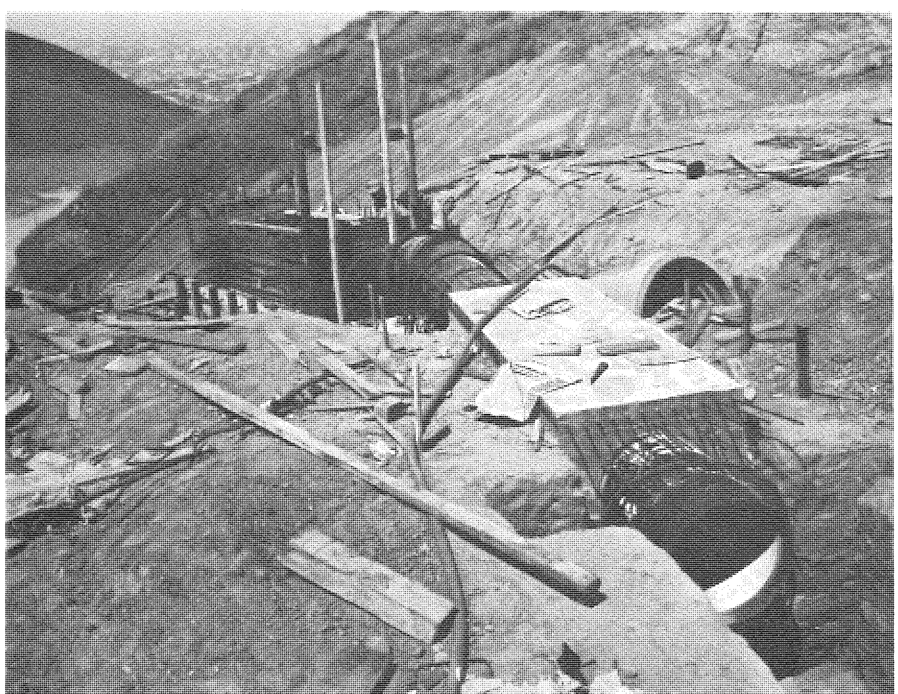


PHOTO 45. Junction of 24-inch blowoff and 42-inch spillway lines.

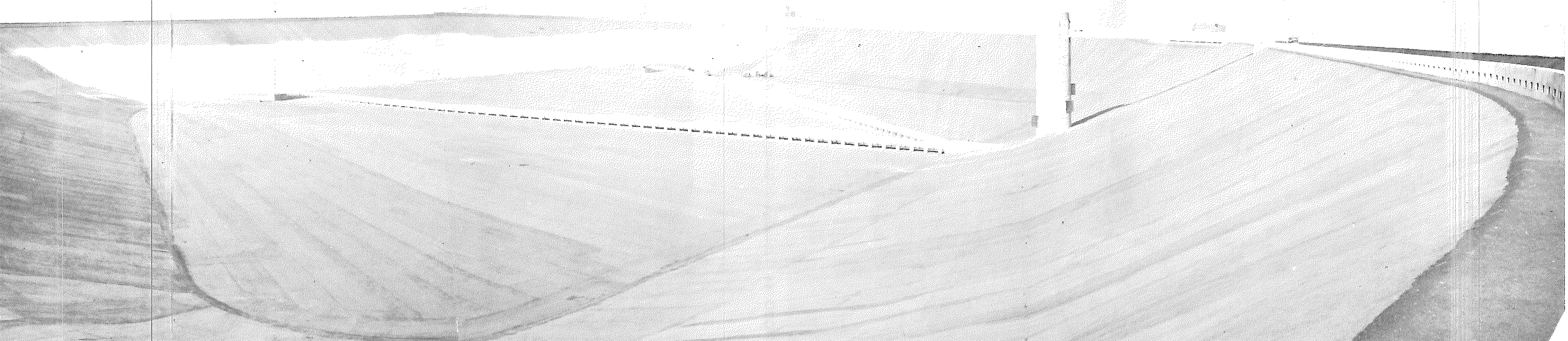


PHOTO 46. Panorama from southeast corner. West slope and northwest corner partly sprayed with cement. Circulator lines in center. Gate tower and footbridge right, with spillway opening visible under the bridge.

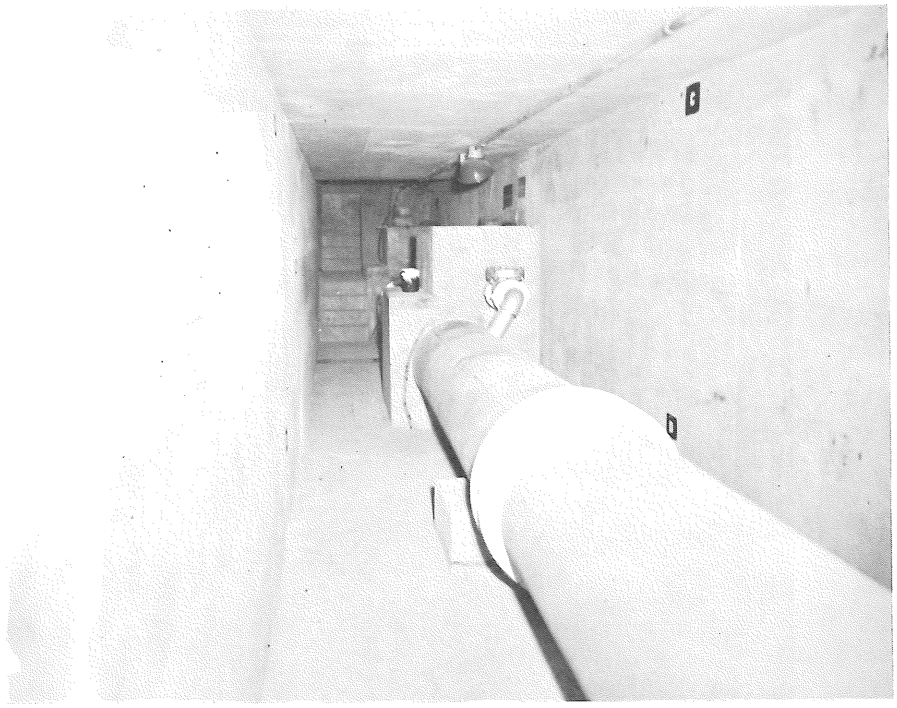


PHOTO 47. Drainage inspection chamber, January 10, 1952, showing 24-inch blowoff pipe in foreground and weir box for southeast toe, northeast toe, and fault drains in rear. Labels C and D on south wall locate crack at Station 0+70.3.

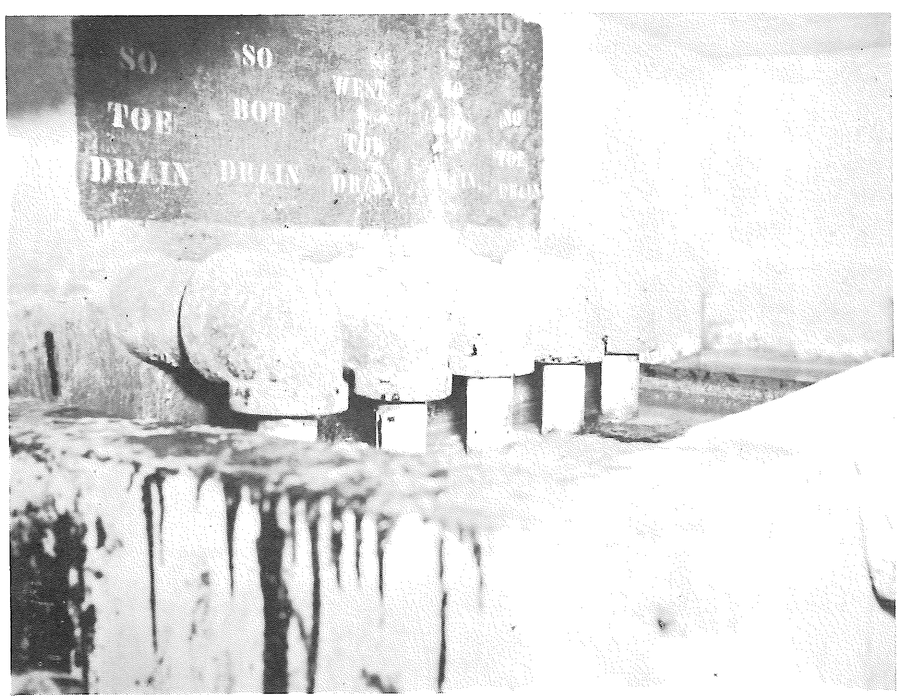


PHOTO 48. Drains at west end of drainage inspection chamber, January 10, 1952. Note evidence of asphalt discharge.

PHOTO 49. Buckling of asphaltic paving, May 11, 1951.

PHOTO 50. Settlement of gate approach channel, May 11, 1951

PHOTO 51. Crack in drainage inspection chamber at Station 0+70.3, January 10, 1952.

PHOTO 52. Opening in north parapet wall at Station 9+14.4, May 16, 1955.

PHOTO 53. Earthcrack 2. Looking north across restaurant parking lot near southwest corner of Stocker Street and La Brea Avenue. Maximum vertical displacement 2 inches. Opening 1 inch.

PHOTO 54. Earthcrack 2 near northwest corner of Stocker Street and La Brea Avenue, showing pothole which is typical of this type of earthcrack.

PHOTO 55. Earthcrack 4 near La Brea Avenue. A typical open crack in undeveloped field.

PHOTO 56. Earthcrack 6 near Don Miguel and Don Lorenzo Drives. Crack offsets sidewalk about 1 inch.

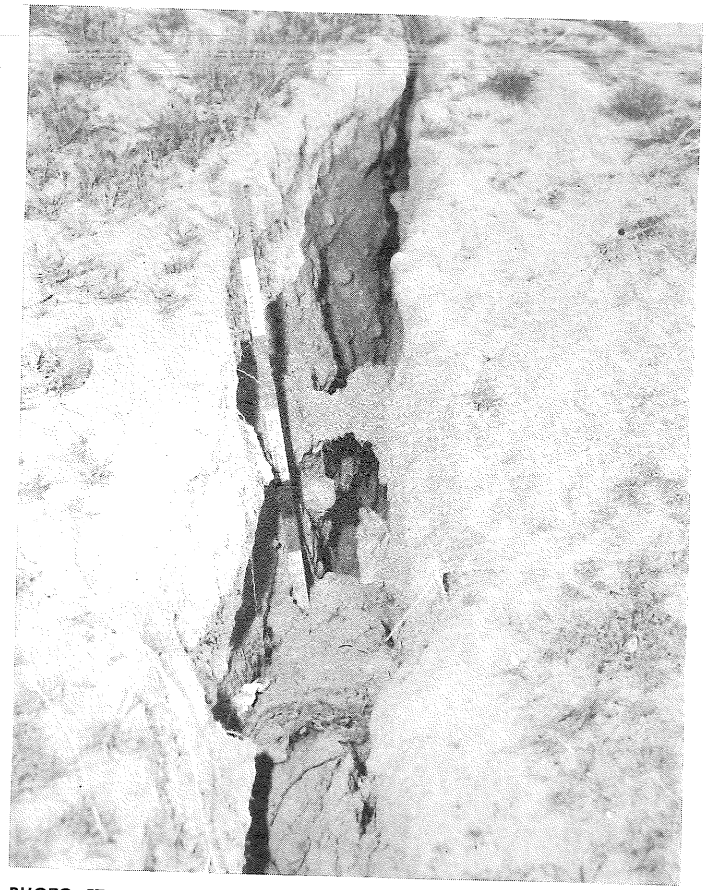


PHOTO 57. Earthcrack 9. Looking north showing open crack near base of slope northwest of detention basin and south of Cloverdale Avenue.

PHOTO 58. Earthcrack 10. Looking north at open eroded crack in south bank of Cloverdale Avenue.

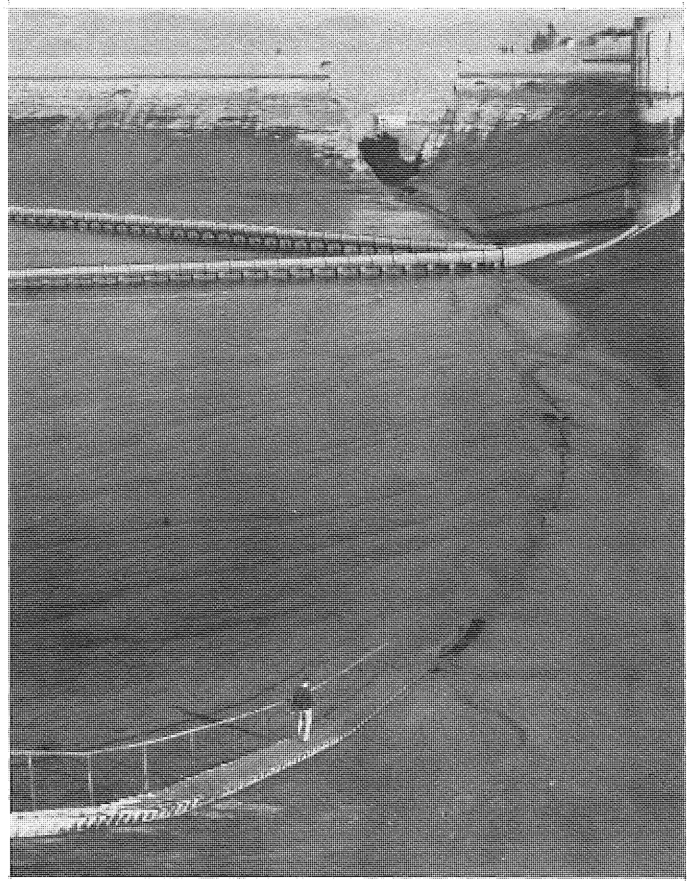


PHOTO 59. Ruptured reservoir lining and breach in main dam, December 15, 1963.

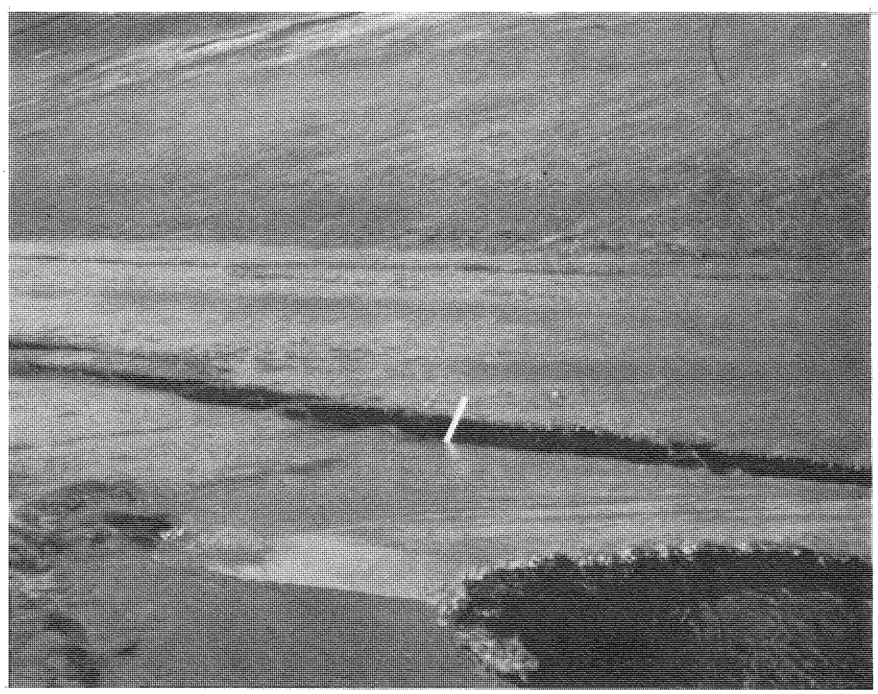


PHOTO 60. Typical displacement and collapse at Fault I, December 15, 1963.

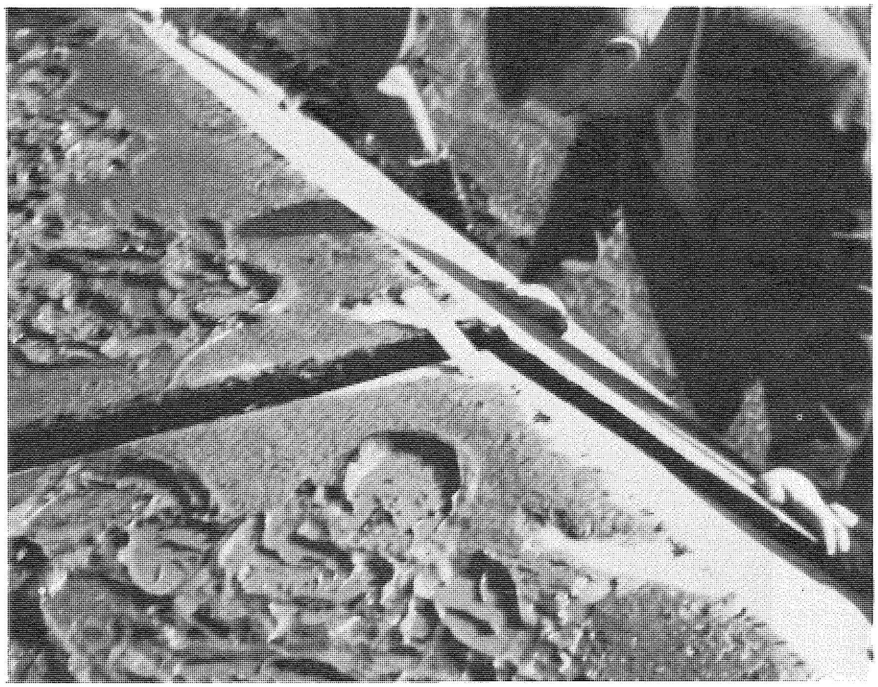


PHOTO 61. Opening of second joint from gate tower in circulator connection conduit, December 15, 1963.

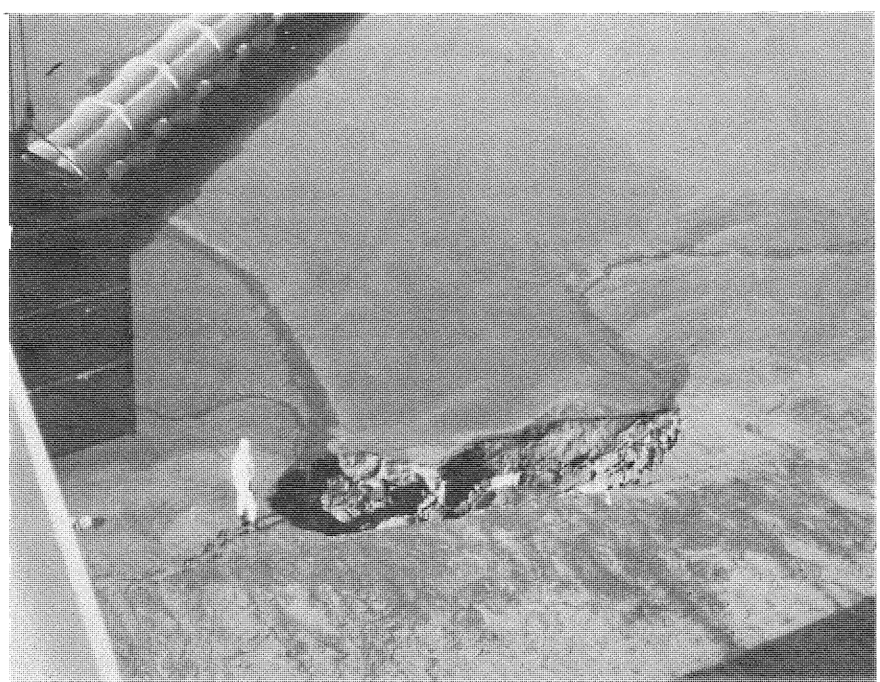


PHOTO 62. Asphaltic paving and compacted earth lining adjacent to subsurface drainage inspection chamber, December 15, 1963.

PHOTO 63. Ruptured lining near drainage inspection chamber, December 16, 1963.

PHOTO 64. Sink holes and collapse features west of surface expression of Fault I along east side of reservoir.

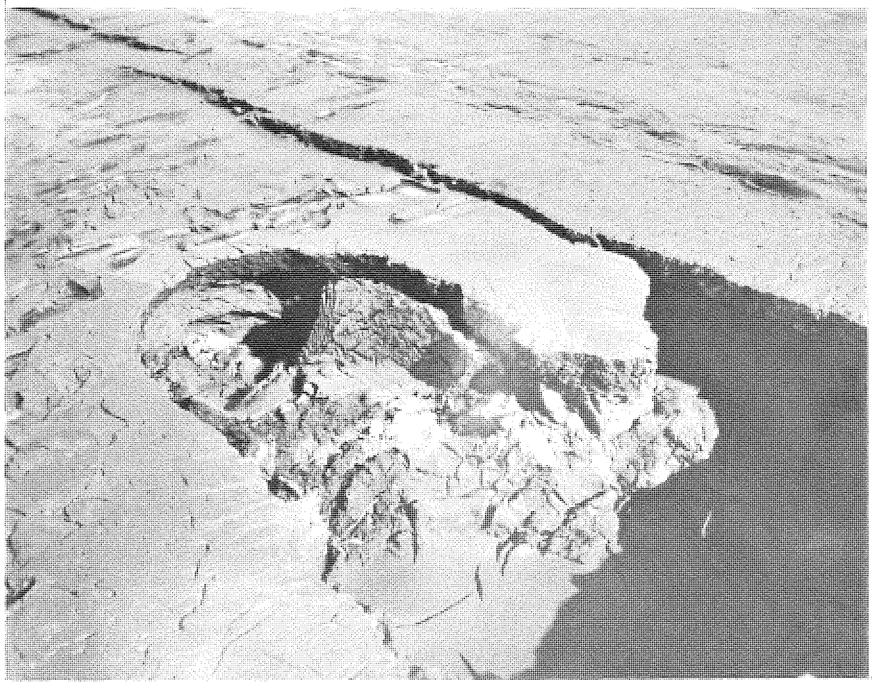


PHOTO 65. Collapse feature west of surface expression of Fault I north of Excavation No. 2.

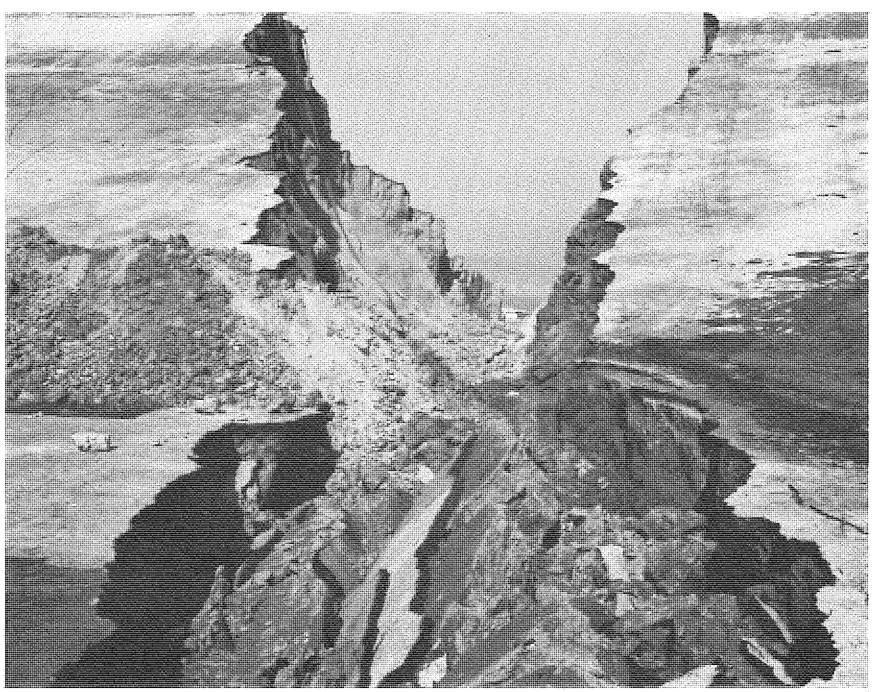


PHOTO 66. Breach seen from inside reservoir, December 22, 1963.

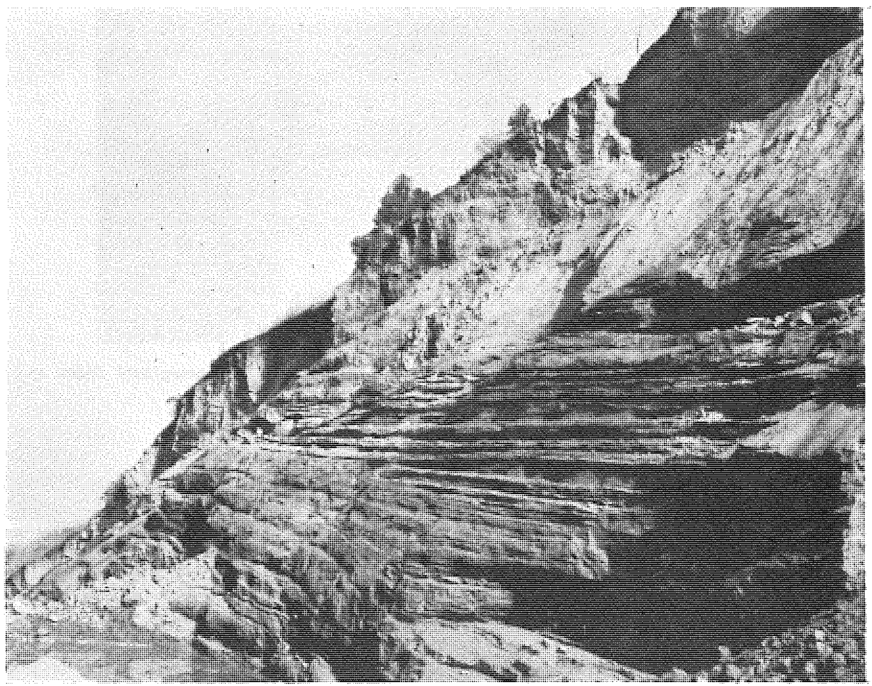


PHOTO 67. Effects of water erosion on foundation strata on east face of breach, December 22, 1963.

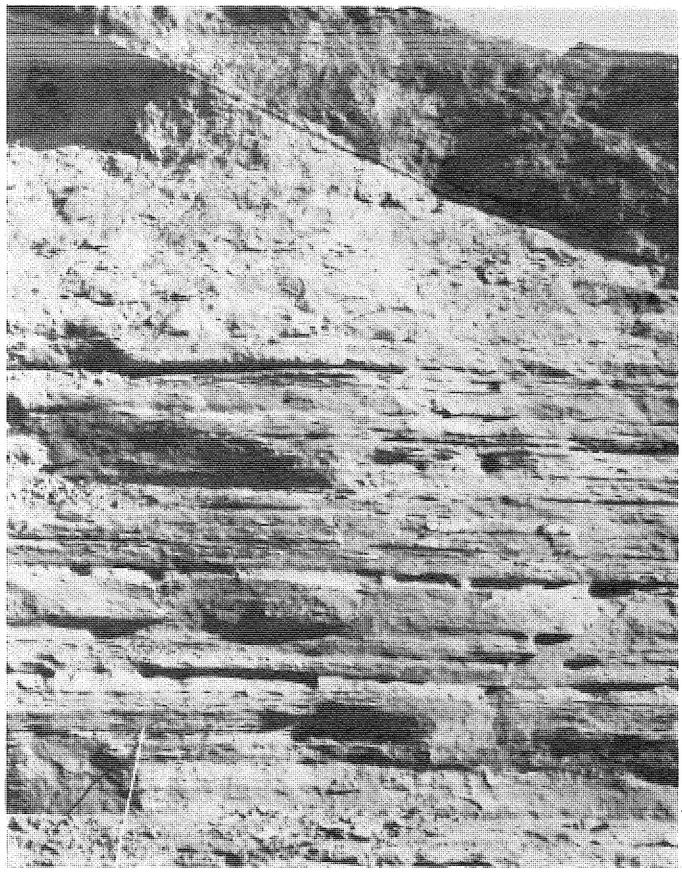


PHOTO 68. Stratification in foundation at east side of breach. Note pea-gravel drain and compacted earth lining above. December 28, 1963.

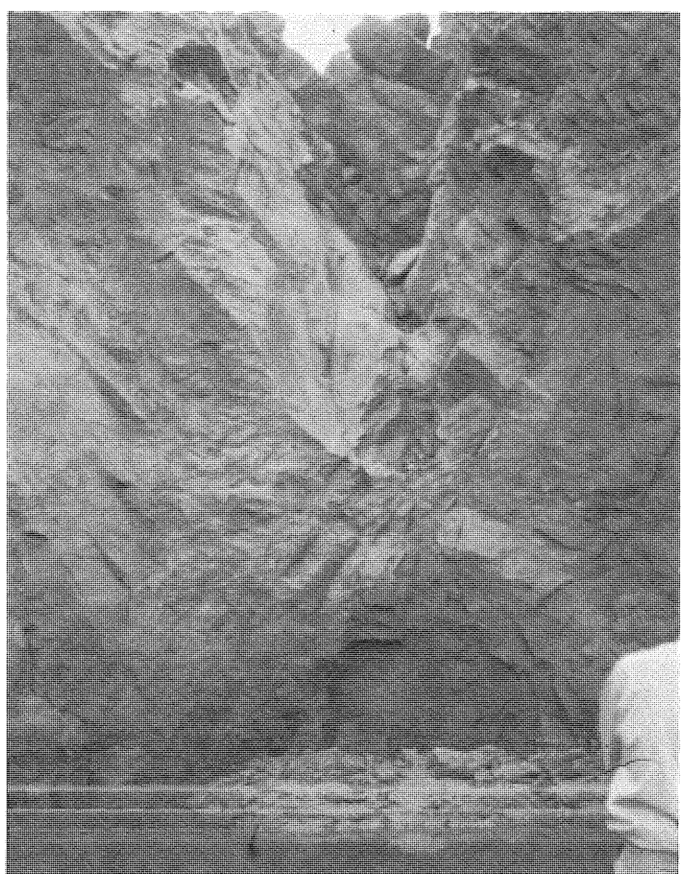


PHOTO 69. Looking upstream at faulted Pico formation in lower end of breach, December 16, 1963.

PHOTO 70. Landslide plane exposed in east side of breach, believed to be part of slide encountered during construction of dam.

PHOTO 71. East side of breach showing spillway pipe and reservoir lining, December 22, 1963.

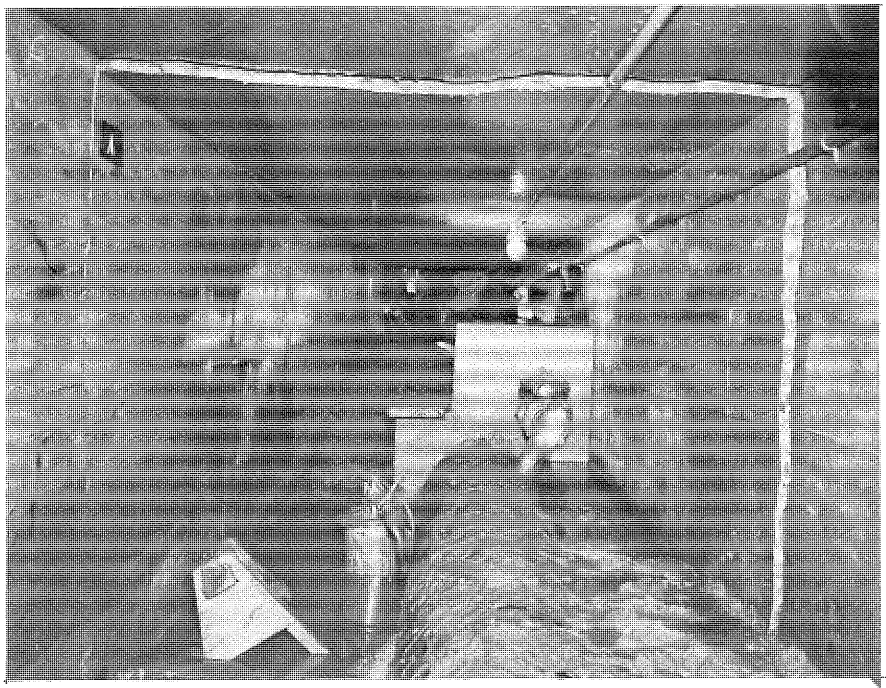


PHOTO 72. Drainage inspection chamber, showing crack at Station 0+70.3, December 20, 1963.

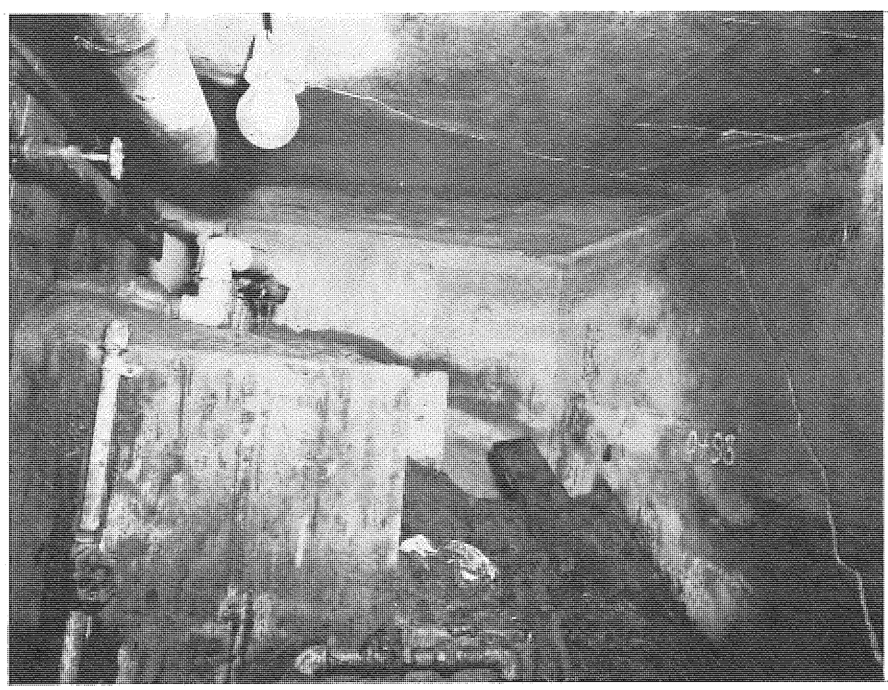


PHOTO 73. West end of drainage inspection chamber, showing crack at Station 0+89, drains, and weir box, December 20, 1963

PHOTO 74. Scour and cracking in main dam downstream from 340 berm, January 4, 1964.

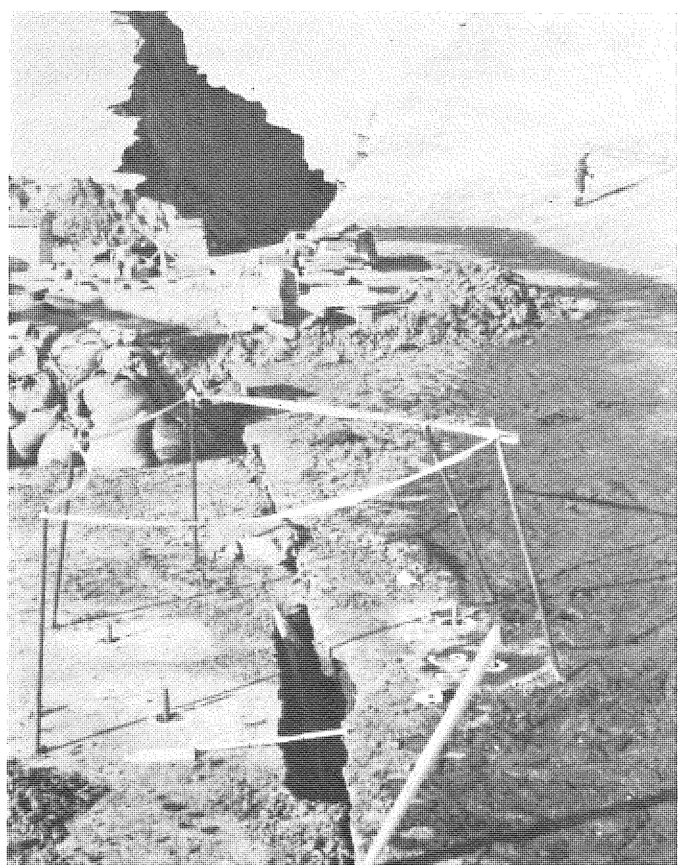


PHOTO 75. Lining separation and straddler points set after failure, December 28, 1963.

PHOTO 76. Exploratory activity at reservoir, January 6, 1964. Excavation No. 8 in foreground.

PHOTO 77. Break in north wall of drainage inspection chamber at Station 0+70.3, as exposed in Excavation No. 7B.

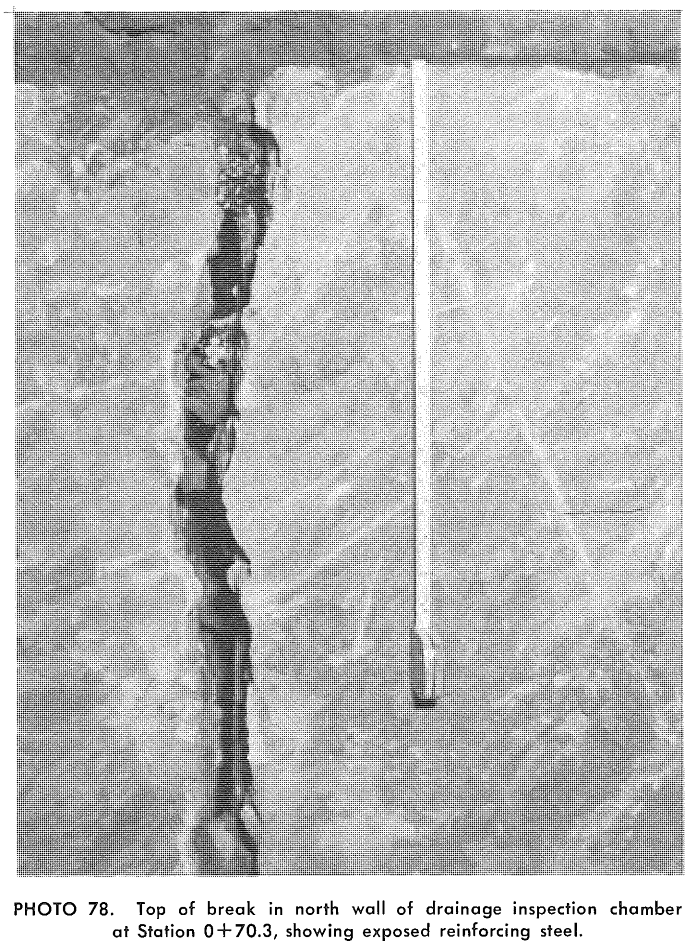


PHOTO 78. Top of break in north wall of drainage inspection chamber at Station 0+70.3, showing exposed reinforcing steel.

PHOTO 79. Break in north wall of drainage inspection chamber at Station 0+89 as exposed in Excavation No. 7B. Break closes to hairline crack at top.

PHOTO 80. Bottom of break in north wall of drainage inspection chamber of Station 0+89.

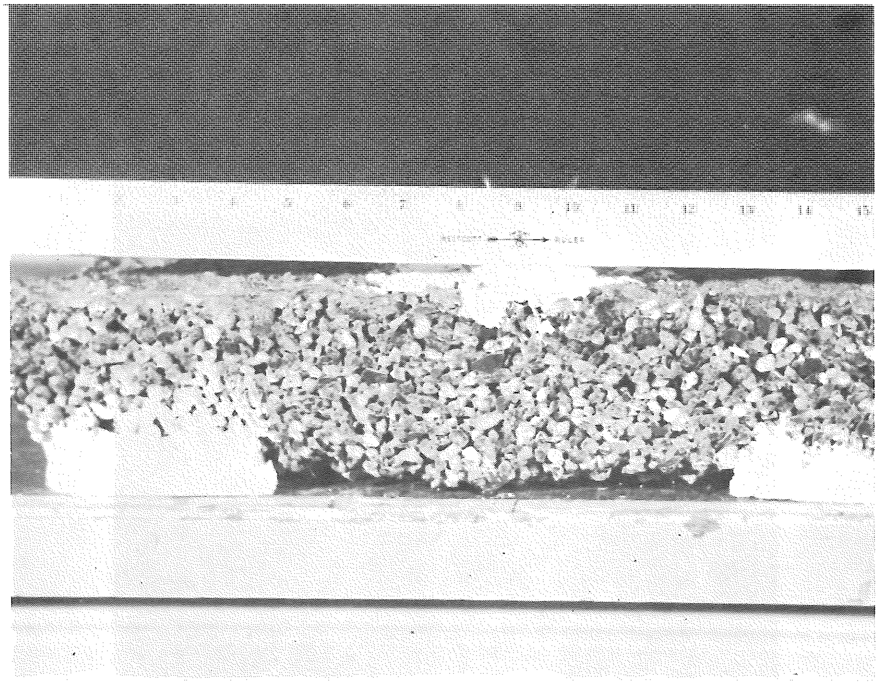


PHOTO 81. Pea-gravel drain Flexure Test Specimen B1 after cutting with diamond saw (length to depth ratio approximately 3).

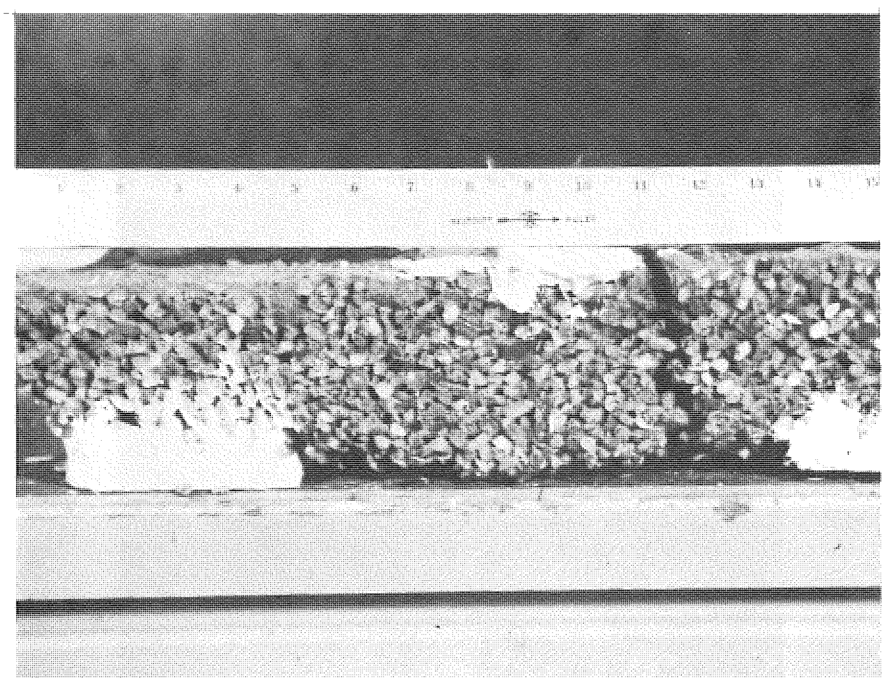


PHOTO 82. Pea-gravel drain Flexure Test Specimen B1 after testing. Failure occurred with a concentrated load of 270 pounds, indicating a modulus of rupture of 91 psi.

PHOTO 83. Cavity as first exposed near bottom of Excavation No. 3. Hole is about 5 inches in diameter. Fault V forms right edge of cavity.

PHOTO 84. Cavity near bottom of Excavation No. 3 after digging farther into wall of trench. Cavity is sufficiently large for man to enter.

PHOTOS 85. Broken 4-inch clay tile drain in Excavation No. 8. Faces of break were coated with a calcium carbonate deposit.

PHOTO 86. Broken clay tile pipe taken from Excavation No. 8, showing calcium carbonate deposit on face of old fracture.

PHOTO 87 Another view of broken clay tile pipe taken from Excavation No. 8.

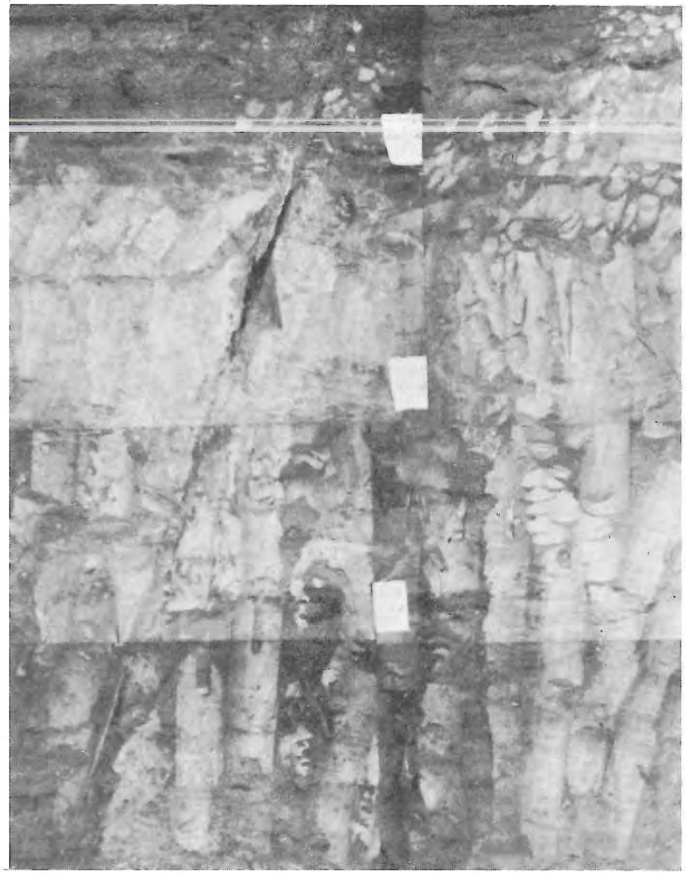


PHOTO 88. Mosaic showing fault plane at Station 0+16 in Excavation No. 9, a point directly beneath south toe of main dam. Fault is partially filled with loose silt. A cavity appears in upper half of photo.

PHOTO 89. Mosaic showing cavity along Fault I at Station 0+62 in Excavation No. 9. Cavity extends beyond face to distance of 4.5 feet.

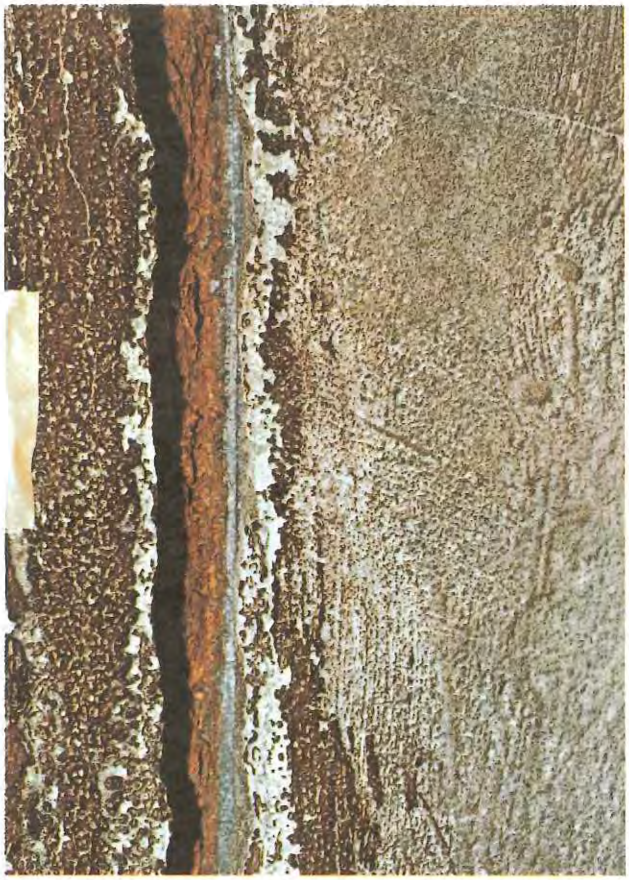


PHOTO 90. Interior of circulator connector conduit, showing corrosion of steel bell ring at a joint where the cement mortar lining separated. Blue line is uncorroded steel recently exposed.

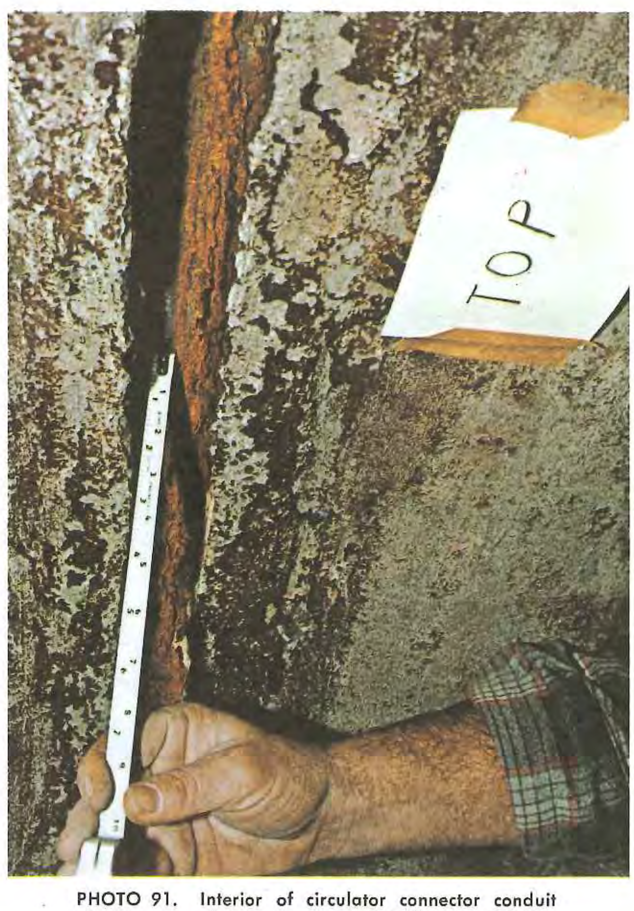


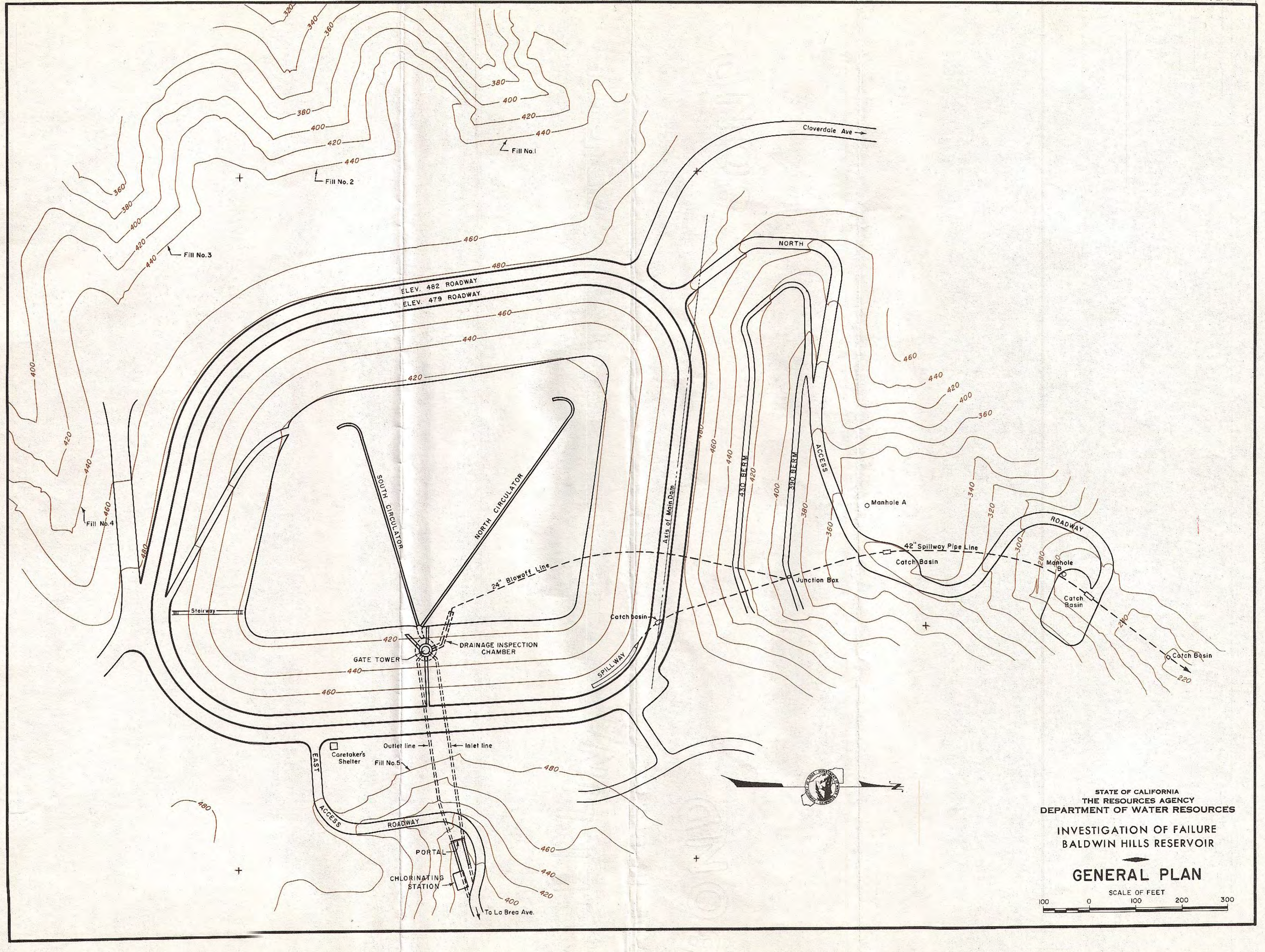
PHOTO 91. Interior of circulator connector conduit showing corrosion layers on steel bell ring at lining separation.

PLATES

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

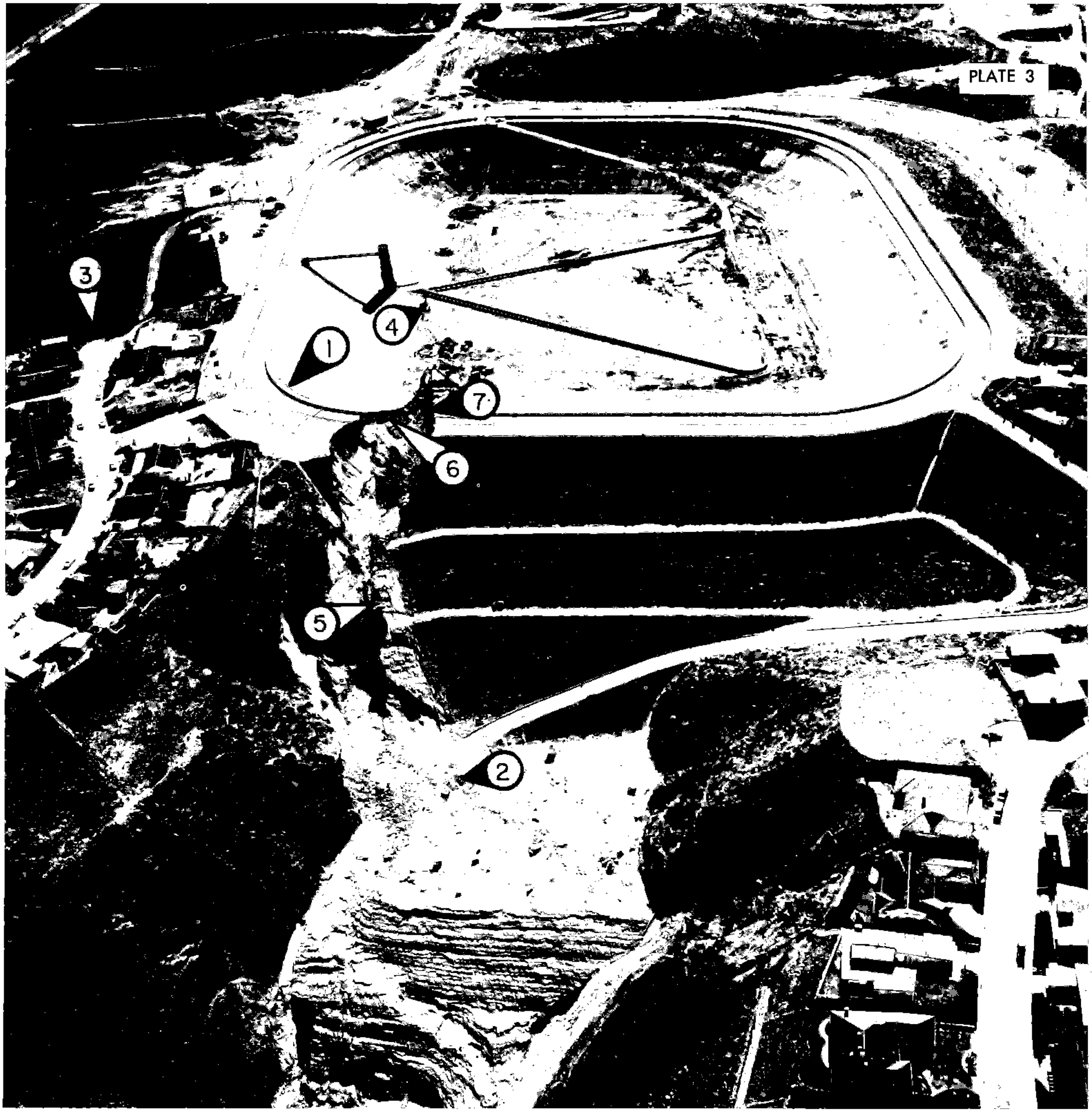
AREA MAP



STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

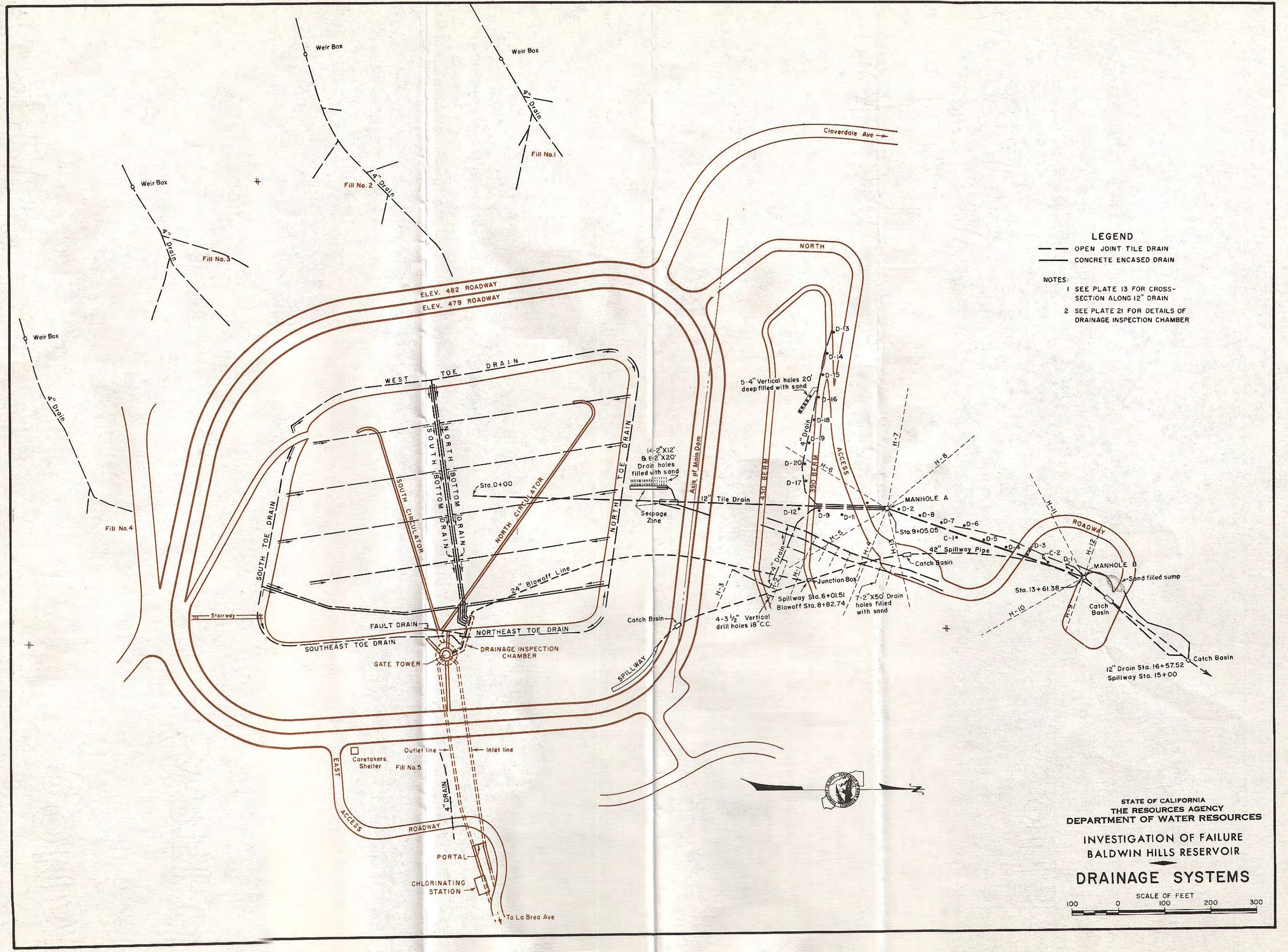
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

GENERAL PLAN



EVENTS ON DAY OF FAILURE

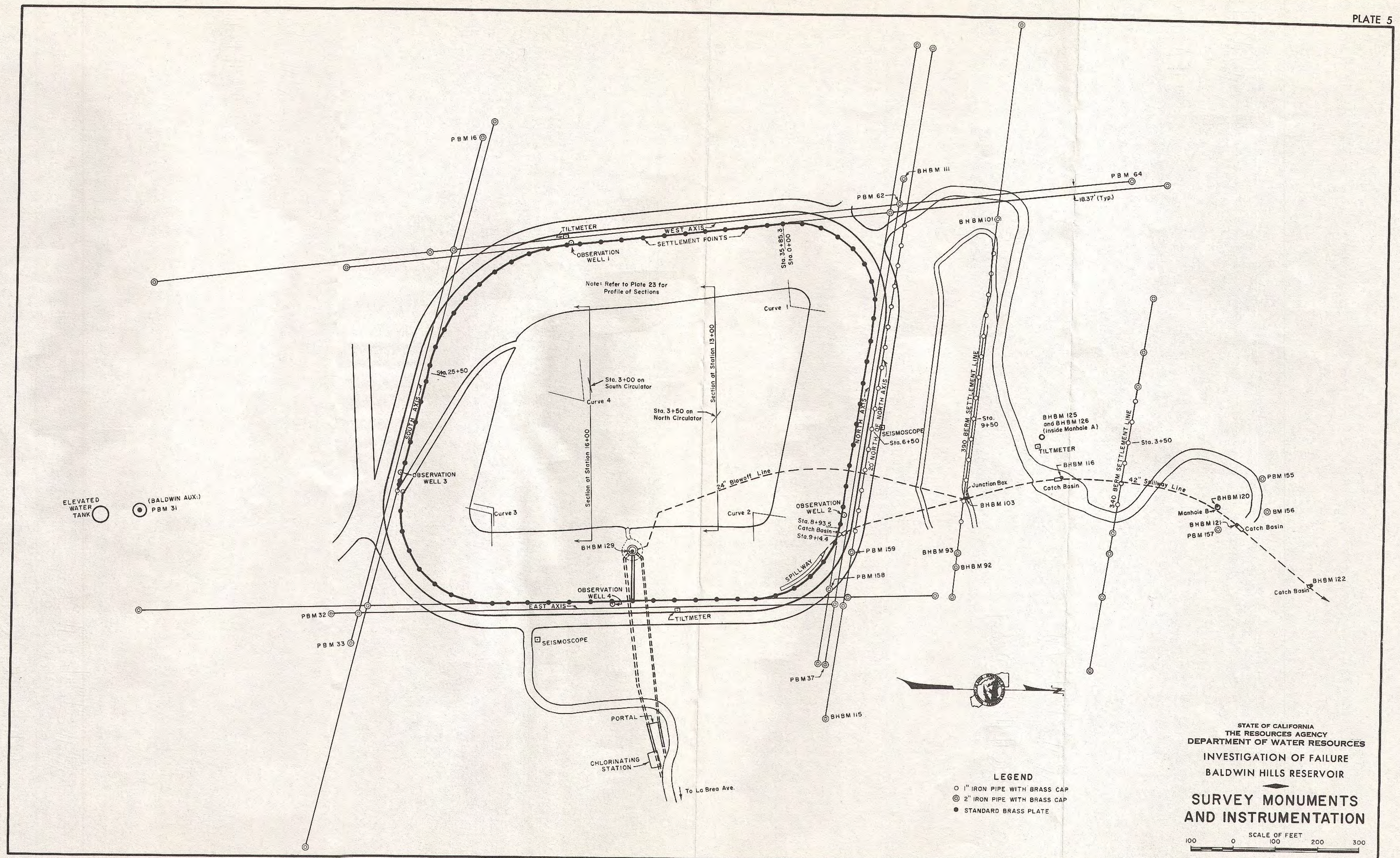
1. Caretaker noticed an unusual sound of water at spillway intake at about 11:15 a.m.
2. At the catch basin, caretaker observed an unusually high flow of drainage water running through spillway pipe.
3. Caretaker entered inlet tunnel portal to reach drainage inspection chomber.
4. Inside the chamber, beneath the reservoir, drains were found to be discharging at an unusually high rate.
5. Leakage from reservoir was first observed on downstream side of dam at a point about 10 feet above 390 berm.
6. First surface evidence of possible breach occurred at Station 8+93.5, where a crack across the main dam opened and rapidly widened.
7. As flow through dam continued, an opening became evident on inside face of dam. The breach was complete by 3:38 p.m.



STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

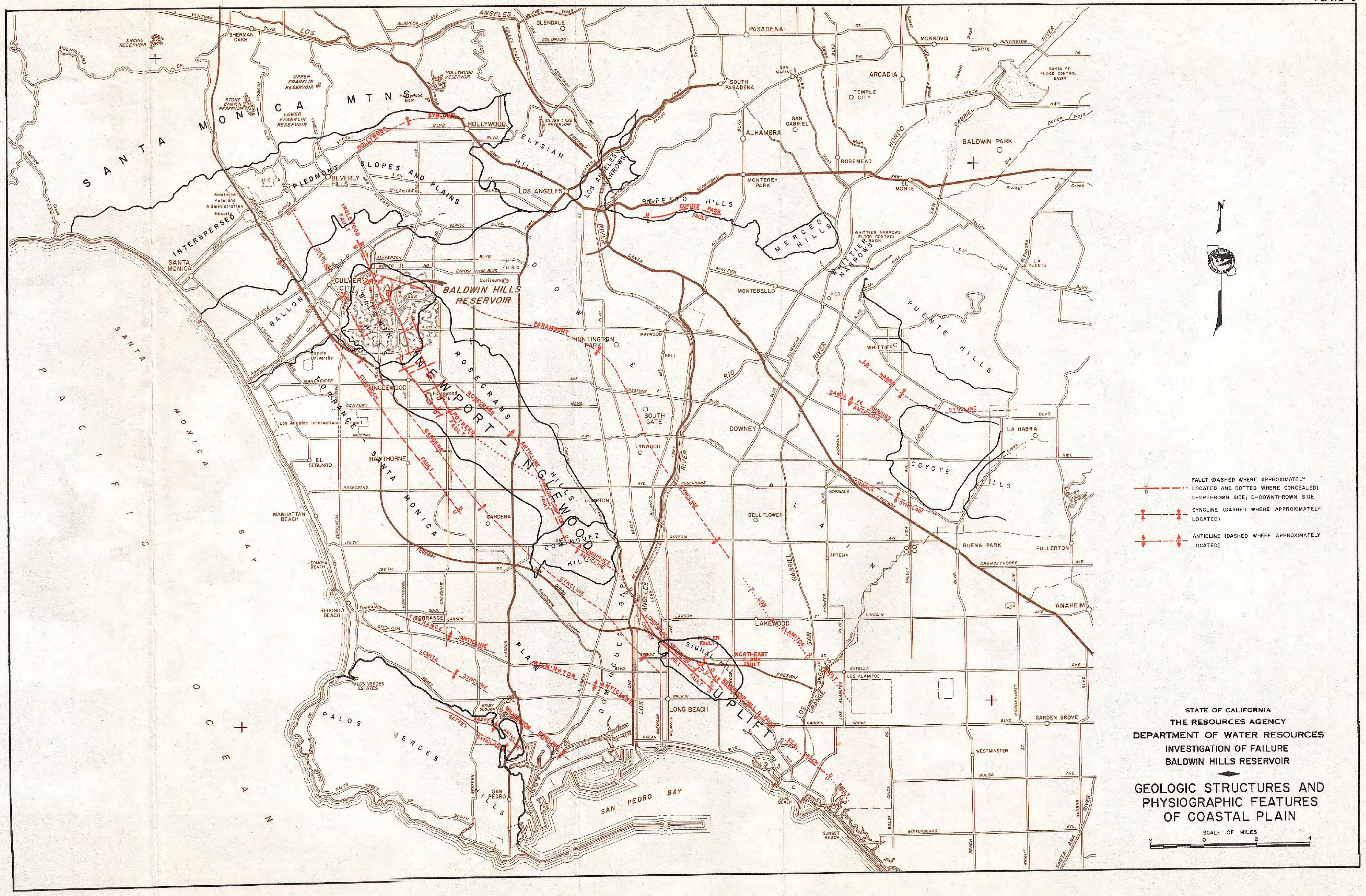
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

DRAINAGE SYSTEMS



STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

SURVEY MONUMENTS AND INSTRUMENTATION



STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

GEOLOGIC STRUCTURES AND PHYSIOGRAPHIC FEATURES OF COASTAL PLAIN

SCALE OF MILES

LEGEND

AGE

Age		
RECENT	Qal	ALLUVIUM
PLEISTOCENE	Qpv	PALOS VERDES FORMATION
PLEISTOCENE	Qi	INGLEWOOD FORMATION
PLIO-PLEISTOCENE	TQ	PICO FORMATION OR PICO-INGLEWOOD FORMATIONS UNDIFFERENTIATED

ls — AREA OF SLUMPING OR MINOR LANDSLIDING

CONTACT, DASHED WHERE INFERRED OR APPROXIMATELY LOCATED

U / D — FAULT, DASHED WHERE INFERRED OR APPROXIMATELY LOCATED, U, UPTHROWN SIDE; D, DOWNTHROWN SIDE.

12 — STRIKE AND DIP OF BEDS

⊕ HORIZONTAL BEDS

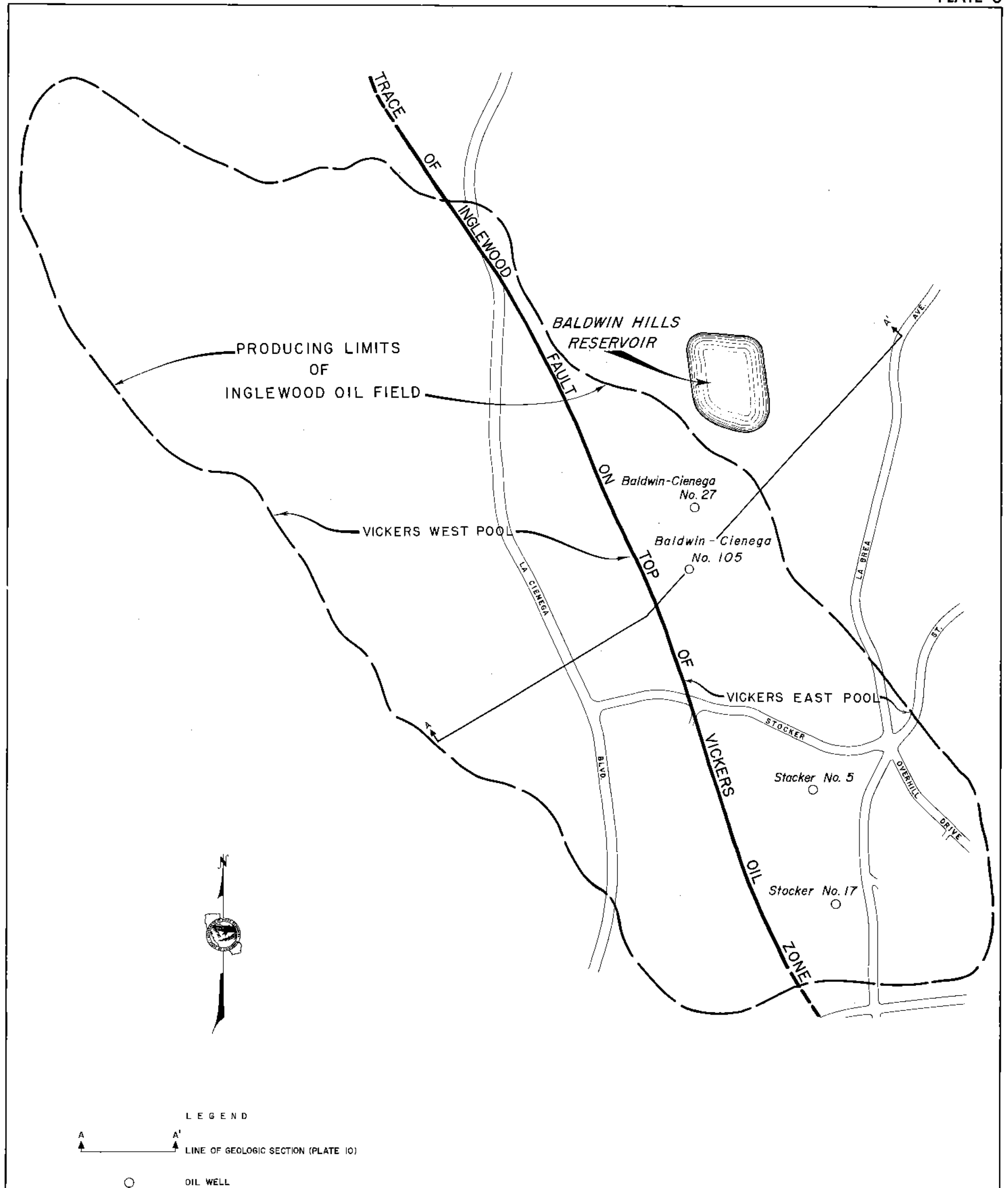
GEOLOGY LARGELY FROM R. R. WILSON (1949), T. L. BEAR AND P. H. GARDETT (1955), AND R. O. CASTLE (1959), BUT WITH MINOR ADDITIONS AND REVISIONS.

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

GEOLOGIC MAP

SCALE OF FEET
0 500 1000

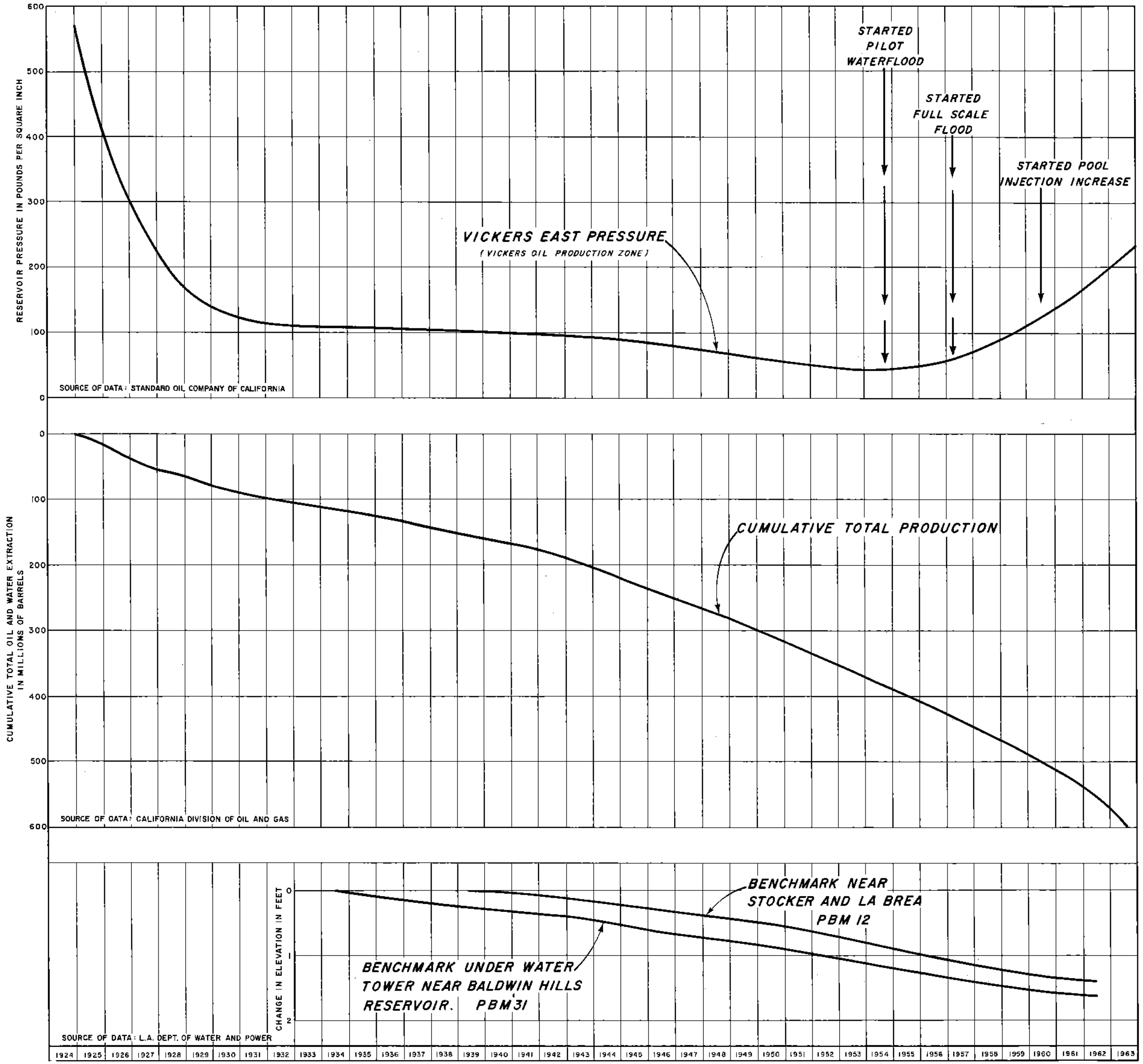


STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

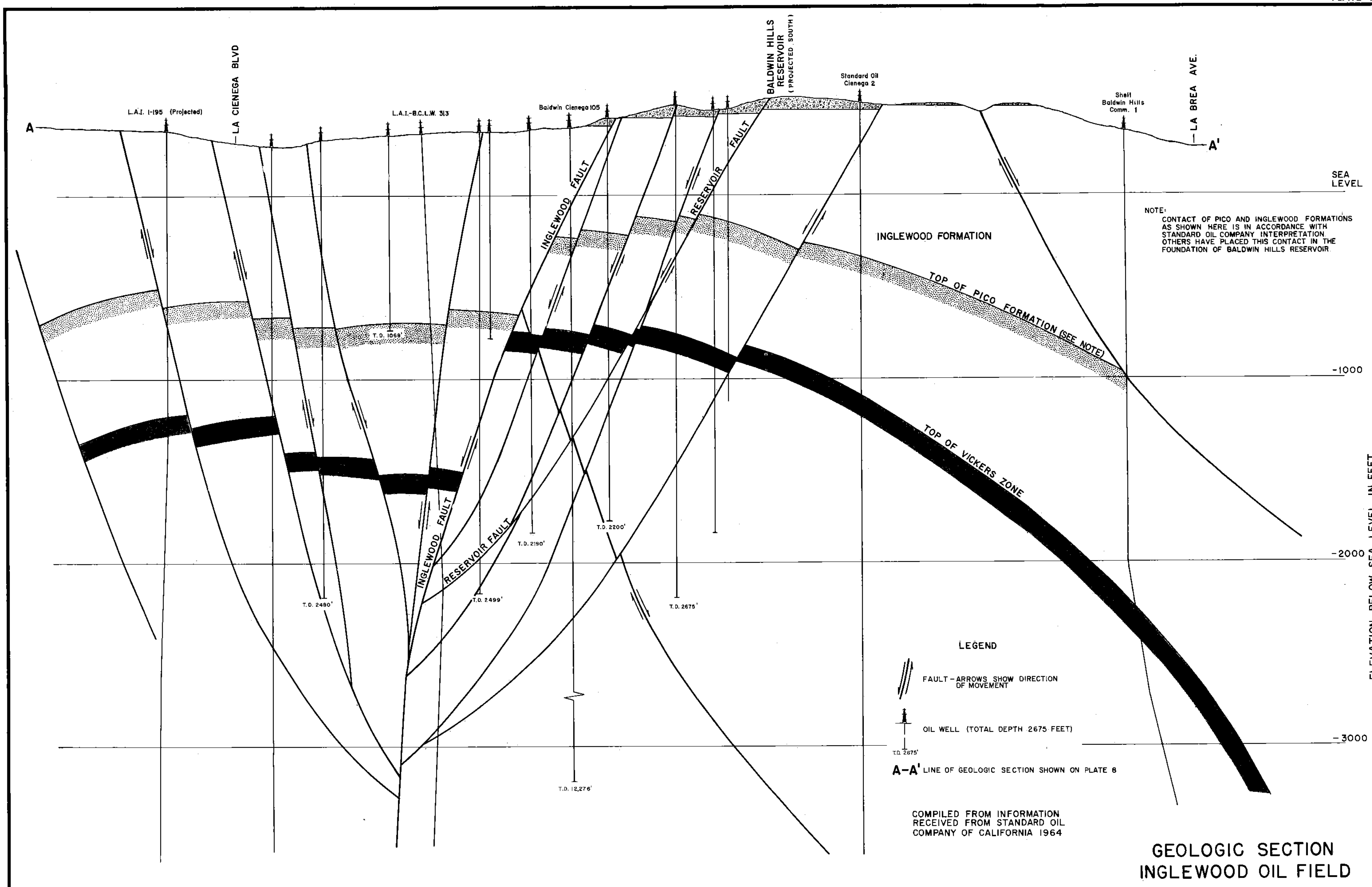
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

BOUNDARY OF INGLEWOOD OIL FIELD

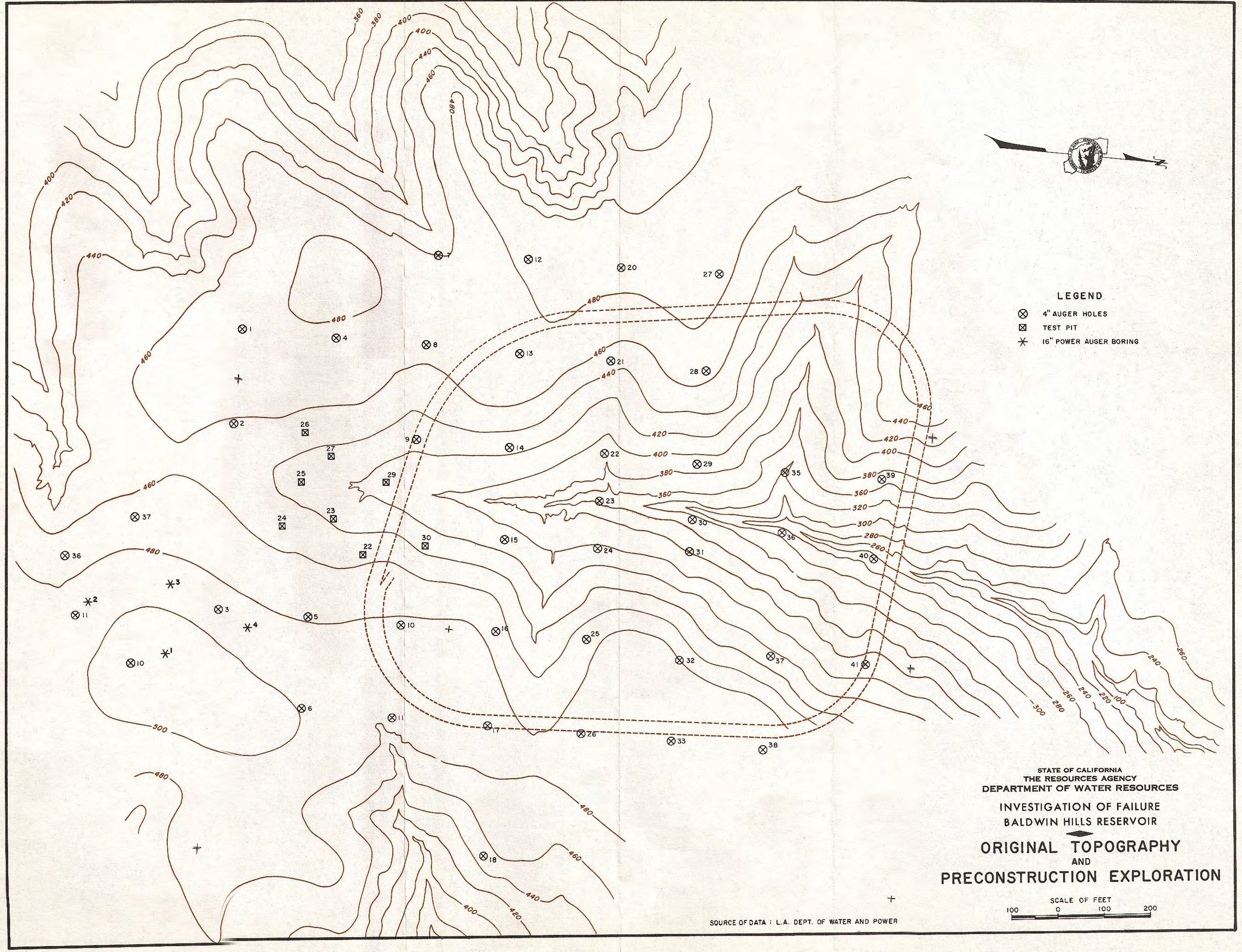
SCALE IN FEET



FLUID PRESSURE, PRODUCTION, BENCH MARK ELEVATION CHANGES — INGLEWOOD OIL FIELD



GEOLOGIC SECTION
INGLEWOOD OIL FIELD



STATE OF CALIFORNIA
THE RESOURCES AGENCY
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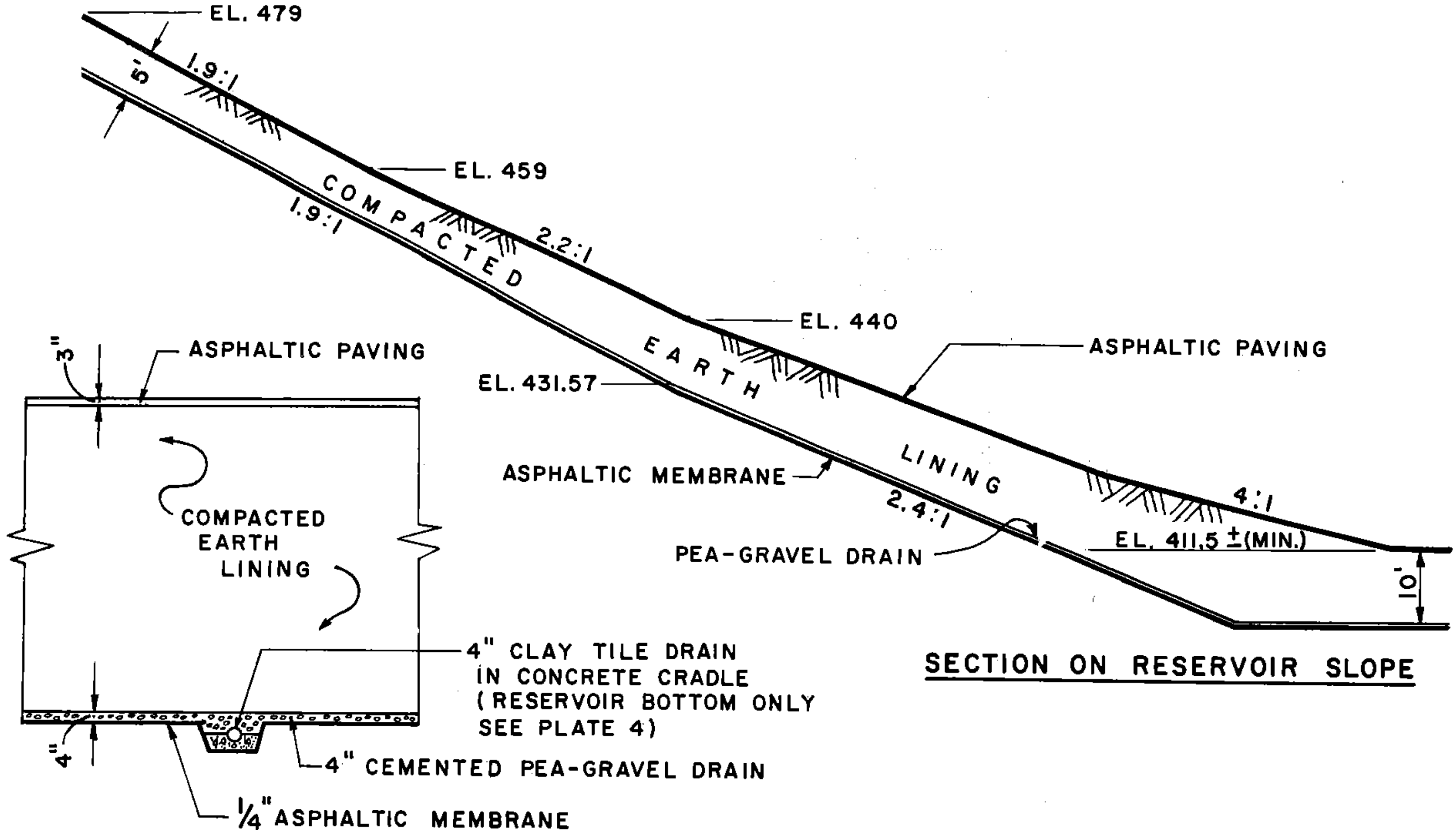
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

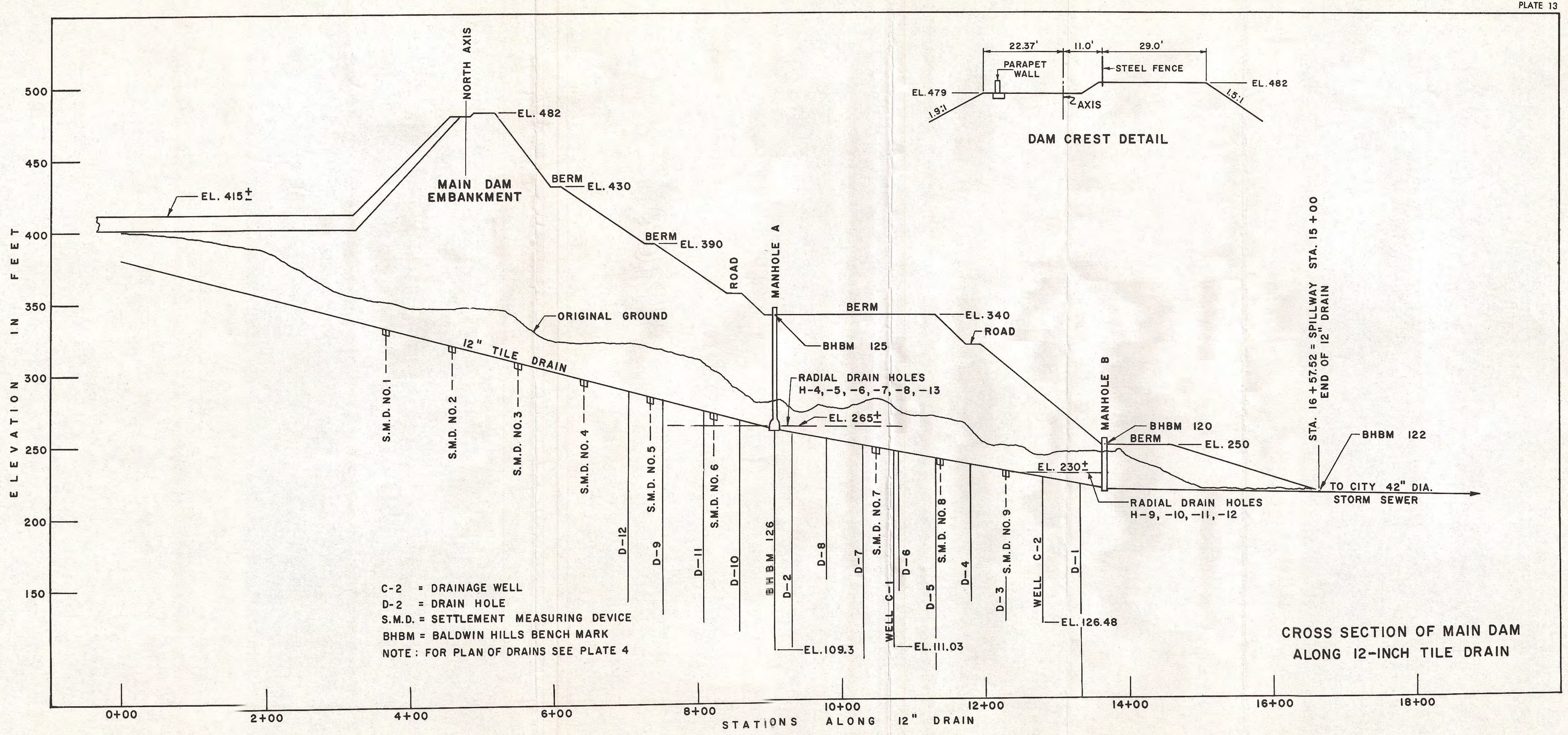
ORIGINAL TOPOGRAPHY AND PRECONSTRUCTION EXPLORATION



SECTION ON RESERVOIR SLOPE

TYPICAL SECTION THROUGH LINING

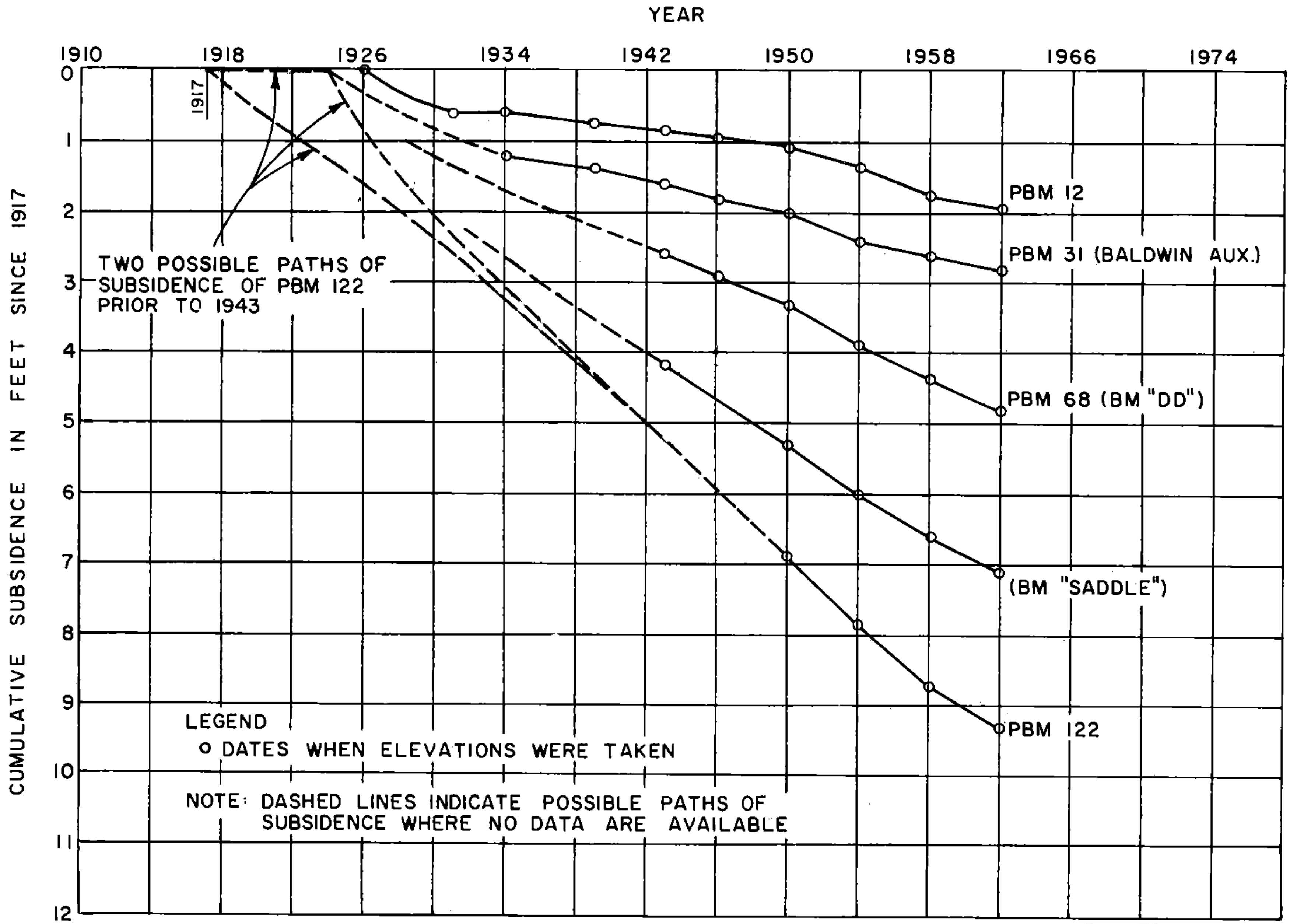
RESERVOIR LINING



CROSS SECTION OF MAIN DAM ALONG 12-INCH TILE DRAIN

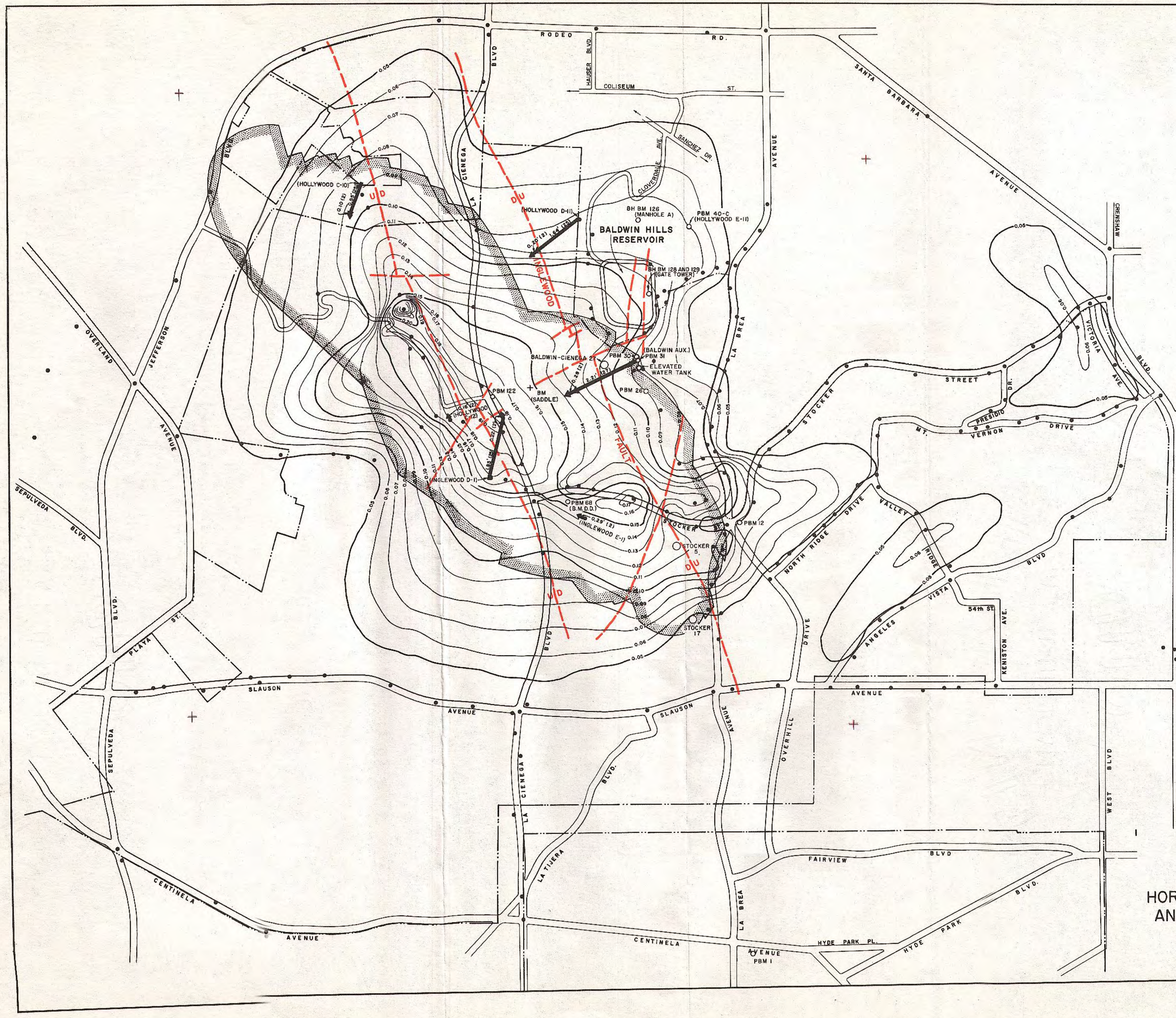
CHRONOLOGY ON OPERATION OF BALDWIN HILLS RESERVOIR

1951	April 11 First water in reservoir April 18 Dedication May 5 Drained for repairs June 19 Started refilling reservoir June 19 Initiated 5 daily drain measurements July 2 Reservoir placed into service Oct. 10 Started acid flushing of pea-gravel drain Oct. 29 Crack in chamber at Station 0+70.3
1952	July 10 Earthquake at 1:45 a.m., also July 21, 25, and 29 Dec. 21 Earthquake reported—longitudinal dam cracks filled
1953	Jan. 27 Tiltmeter installed near Manhole A Mar. 27 Parapet joint (Station 9+14.4) open ¼ inch Uncertain date—Started weekly drain measurements
1954	Jan. 12 Earthquake reported Feb. 13 Minor slide on main dam
1955	May 16 Parapet joint (Station 9+14.4) open ½ inch
1956	May 17 Certificate of approval issued by State Supervision of Dam Safety Office
1957	Mar. 13 Reservoir drained for cleaning and repairs Mar. 21-25 Reservoir refilled
1958	Feb. 26 Backflushed bottom drains. Crack in chamber at Station 0+89 April 28 Worked on bottom drains with hose, snake, etc. June 30 Parapet joint (Station 9+14.4) open ¾ inch
1959	June Northeast-southwest diagonal of reservoir reported to be elongated about 0.5 foot
1960	Jan. 22 Chamber cracks at Stations 0+43 and 0+54 Jan. 22 Crack across 479 roadway on main dam at Station 8+93.5
1961	April 5 Terminal Island earthquake April 13 Long Beach earthquake July 28 Earthquake reported Oct. 18 Installed seismoscopes Oct. 20 Orange County earthquake Oct. 23 Earthquake reported
1962	Sept. 1 West Los Angeles earthquake reported Oct. 29 Earthquake reported Nov. Drilled 4 observation wells
1963	Feb. 18 Earthquake reported Mar. 10 Earthquake reported Mar. 19 Flushed all drains April 3 Last annual inspection by State Supervision of Dam Safety Office May 7 Earthquake reported Nov. Increased crack openings in drainage inspection chamber Dec. 14 Failure



From T.M. Leps, 1964

SUBSIDENCE OF BENCH MARKS
BALDWIN HILLS AREA



STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

HORIZONTAL MOVEMENTS AND SUBSIDENCE RATES IN BALDWIN HILLS

EARTH CRACK DATA

Crack Number	First Noted	Location	Movement	Direction	Damage	Remarks
1	May 1957 Los Angeles Investment Company	Overhill Dr., Windsor Hills School	Down on West 3-6″	N 11°–14° E	Street, school ground, garage floor, extensive cracking in several residences	Open irregular cracks and pot holes in fields.
2	March 1958 Los Angeles Department of Water and Power	Stocker Ave.	Down on East 1¾″	N 10° E	Street, curb, parking lot	Pot holes and cracks in field.
3	March 1958 Los Angeles Department of Water and Power	Stocker Ave., West of La Brea Ave.	Down on East 1¼″	N 10° E	Street, curb	Open crack in curb 1½″; opposite fault zone in road cut.
4	January 1958 Los Angeles Department of Water and Power	Stocker Ave. to Don Lorenzo Dr.	Down on East 1½″	N 19°–25° E	Street, curb, apartment building	Intermittent trace, open cracks and pot holes in field.
5	March 1963 Los Angeles County Engineer's Office	Northwest of Windsor Hills School yard	Mainly down on East ½″	N 8°–9° E	Street, curb	Trace is offset slightly; open cracks on curb.
6	February 1964 Department of Water Resources	Don Lorenzo Dr. and Don Miguel Dr.	Down on West 1″	N 25°–32° E	Street, curb, new apartment building	Swale type offset 15 feet wide.
7	February 1964 Department of Water Resources	Don Miguel Dr.	Down on West ½″	N 2° W	Street and curbs	
8	February 20, 1963 Los Angeles Department of Water and Power	La Brea Ave.	Down on West 6″	Parallels street	Street, landslide, broken residential water pipe	Crack reported as being open 2″. Possible damage to Oil Well Nos. 5, 17 and 27. Injection Well No. 4 was started in April and shut down in March 1963.
9	February 1964 Department of Water Resources	Catch basin	Possibly down on East			
10	February 1964 Department of Water Resources	Cloverdale Ave.	No vertical movement observed	N 24°–32° E	Street, curb and possible house damage	
Embankment crack	December 1963	Downstream toe of dam	West side down ½″	Apparent alignment with trace of Fault I		
Fault I earth crack	December 14, 1963	Reservoir and South beyond water tank (Possibly)	West side down 6″	N 8°–10° W, Dip 73°–86° W	Inspection chamber pea-gravel drain, tile drain	Crack in asphalt membrane, impervious embankment material, curb and road.
Fault V earth crack	December 1963	Reservoir only	West side down 4″	N 7° W–N 5° E, Dip 65°–72° W	Circulator lines, pea-gravel drain, tile drain	Crack in asphalt membrane, and impervious embankment material.

LEGEND

EARTH CRACK, DASHED WHERE INFERRED OR APPROXIMATELY LOCATED; U, UPTHROWN SIDE; D, DOWNTHROWN SIDE.

INJECTION WELL

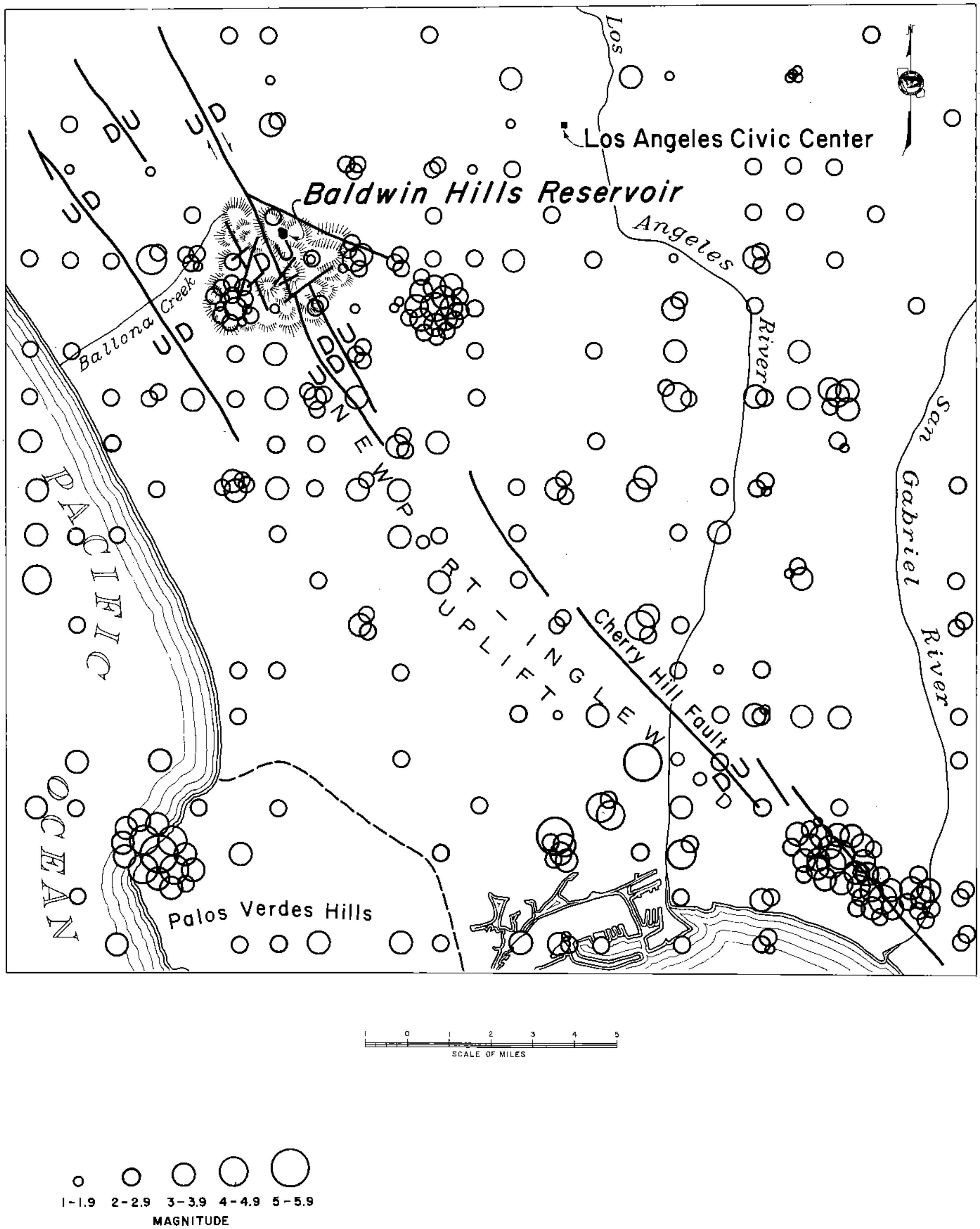
NOTE: INJECTION WELLS APPROXIMATELY LOCATED FROM MAP OF INGLEWOOD OIL FIELD, APRIL 27, 1963, DIVISION OF OIL AND GAS.

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

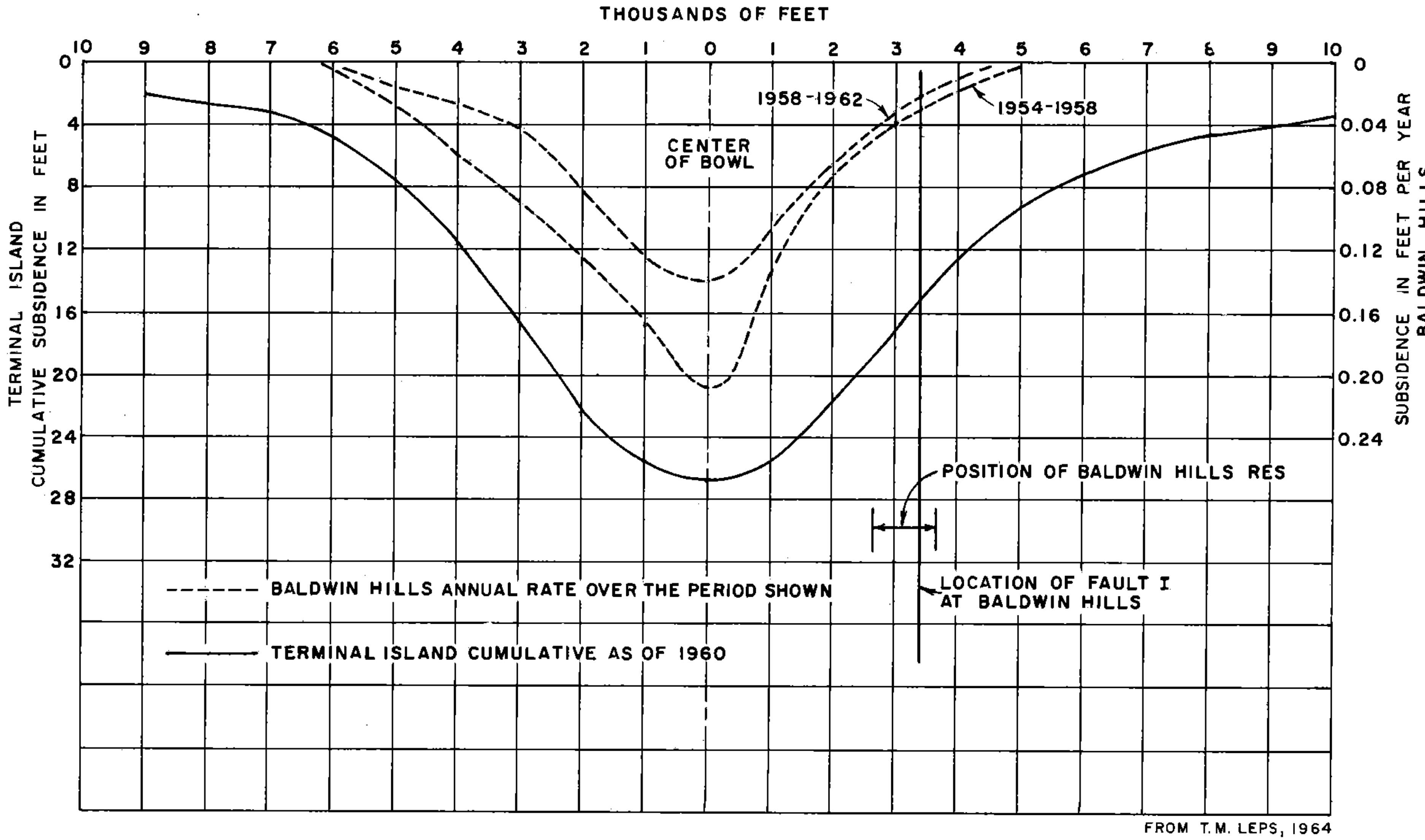
LOCATION OF EARTHCRACKS

SCALE OF FEET
0 500 1000

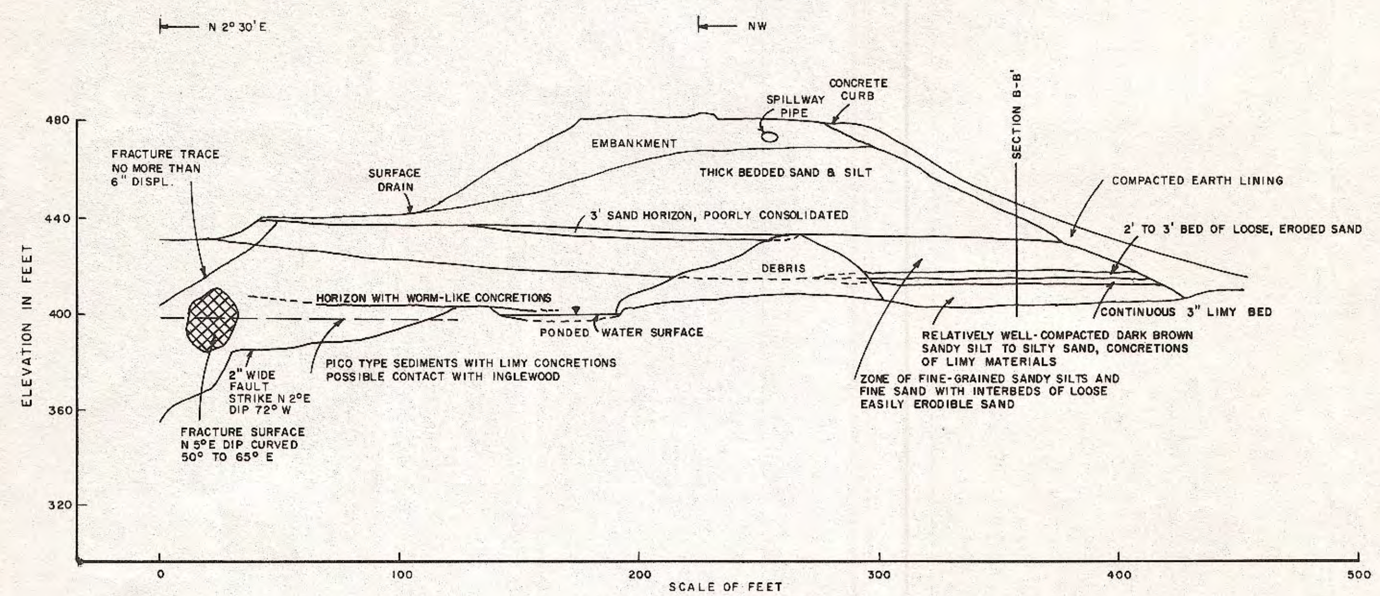


EPICENTER LOCATIONS
IN THE VICINITY OF
NEWPORT - INGLEWOOD UPLIFT
LOS ANGELES COASTAL PLAIN
CALIFORNIA
JANUARY 1, 1934 TO MAY 31, 1963

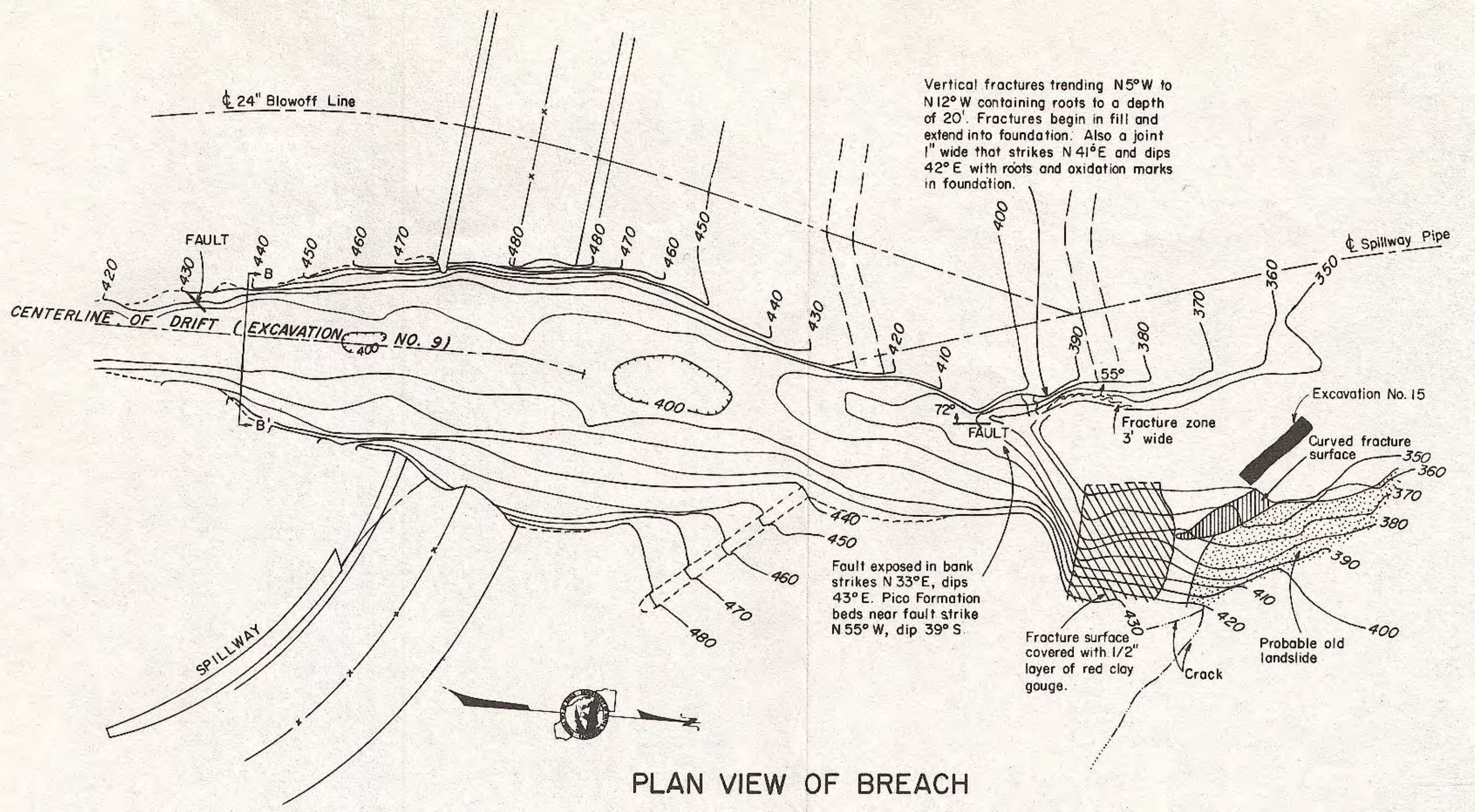
AFTER C.F. RICHTER, C.R. ALLEN, J.M. NORDQUIST AND P. ST.-AMAND,
SEISMOLOGICAL LABORATORY, CALIFORNIA INSTITUTE OF TECHNOLOGY.



SUBSIDENCE COMPARISON
BALDWIN HILLS AND TERMINAL ISLAND



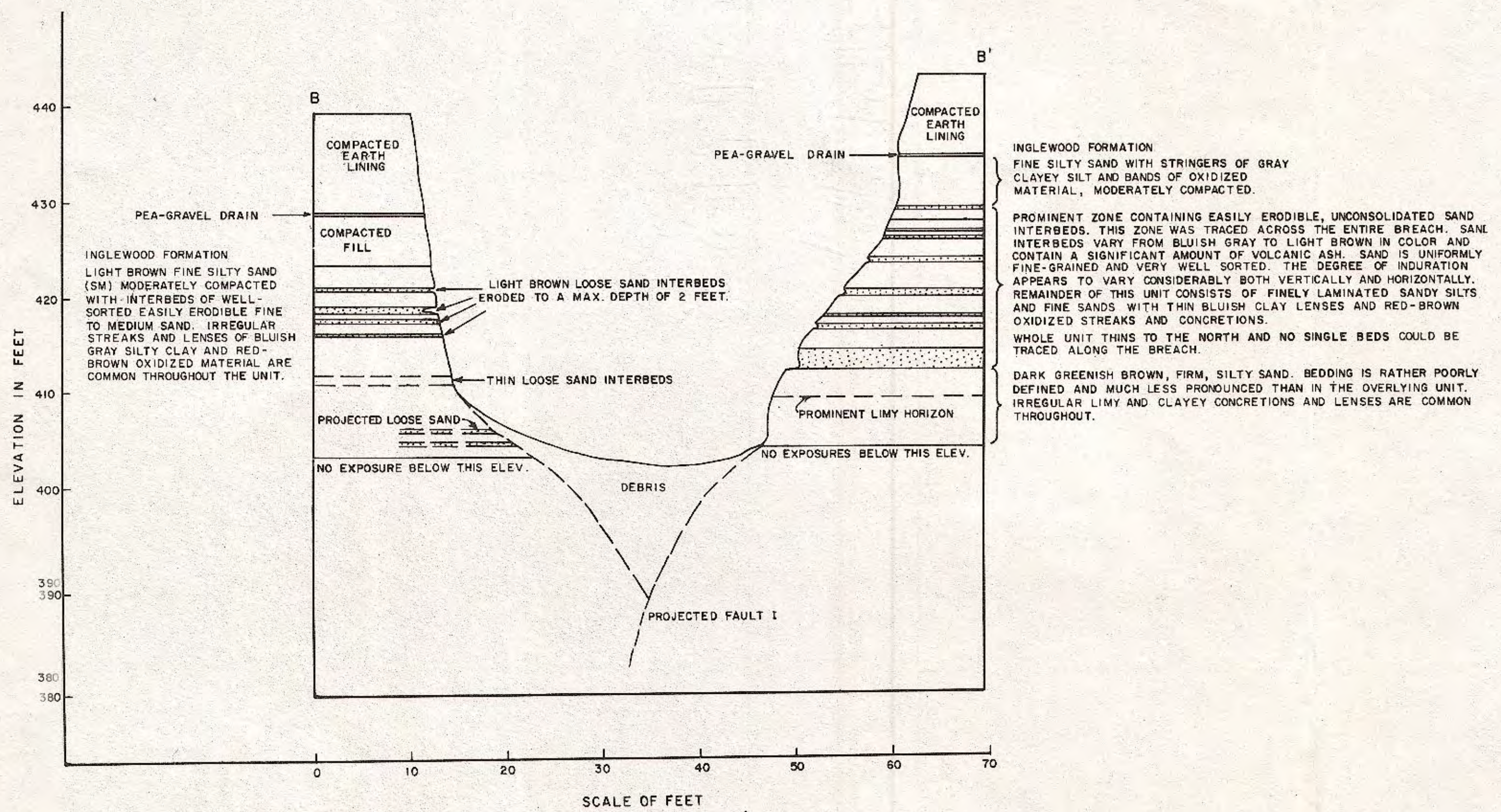
EAST FACE



PLAN VIEW OF BREACH

0 50 100

SCALE OF FEET

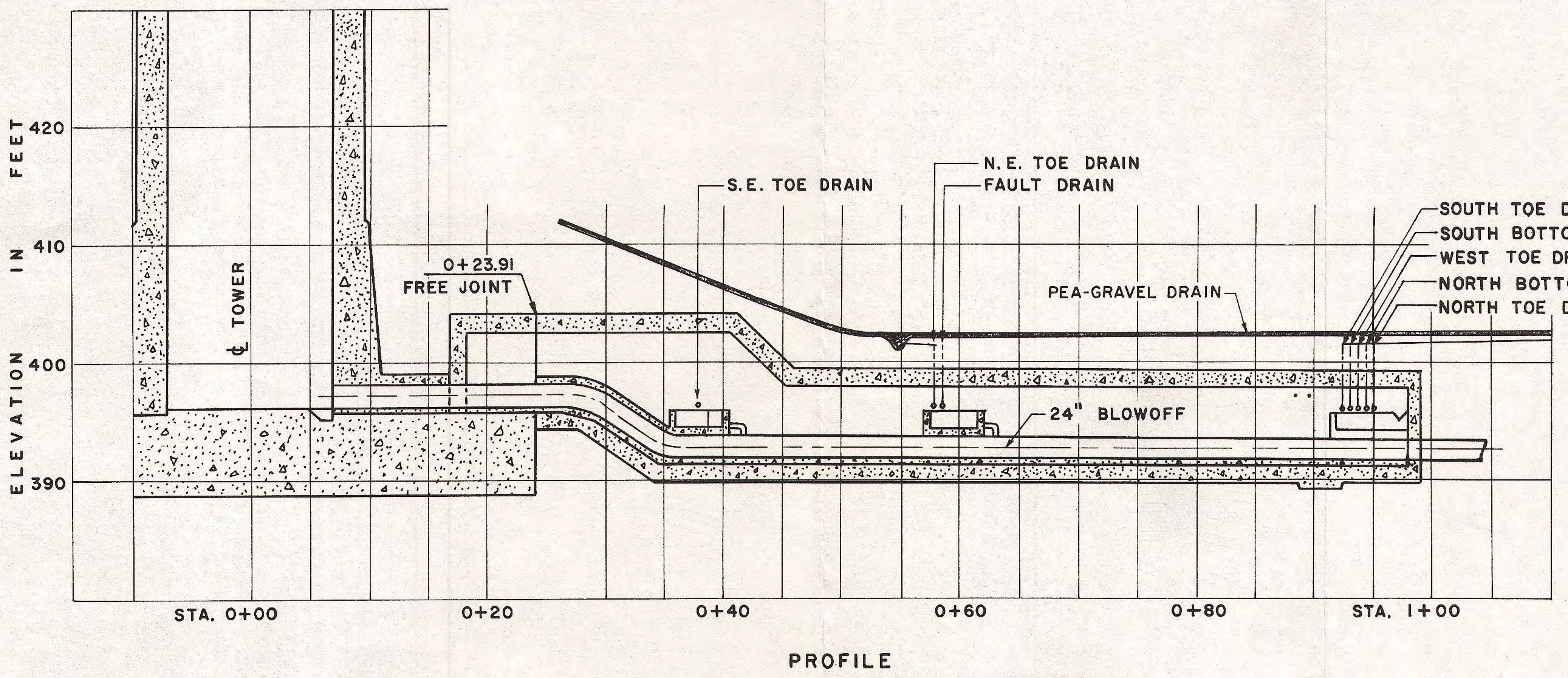


SECTION B-B'

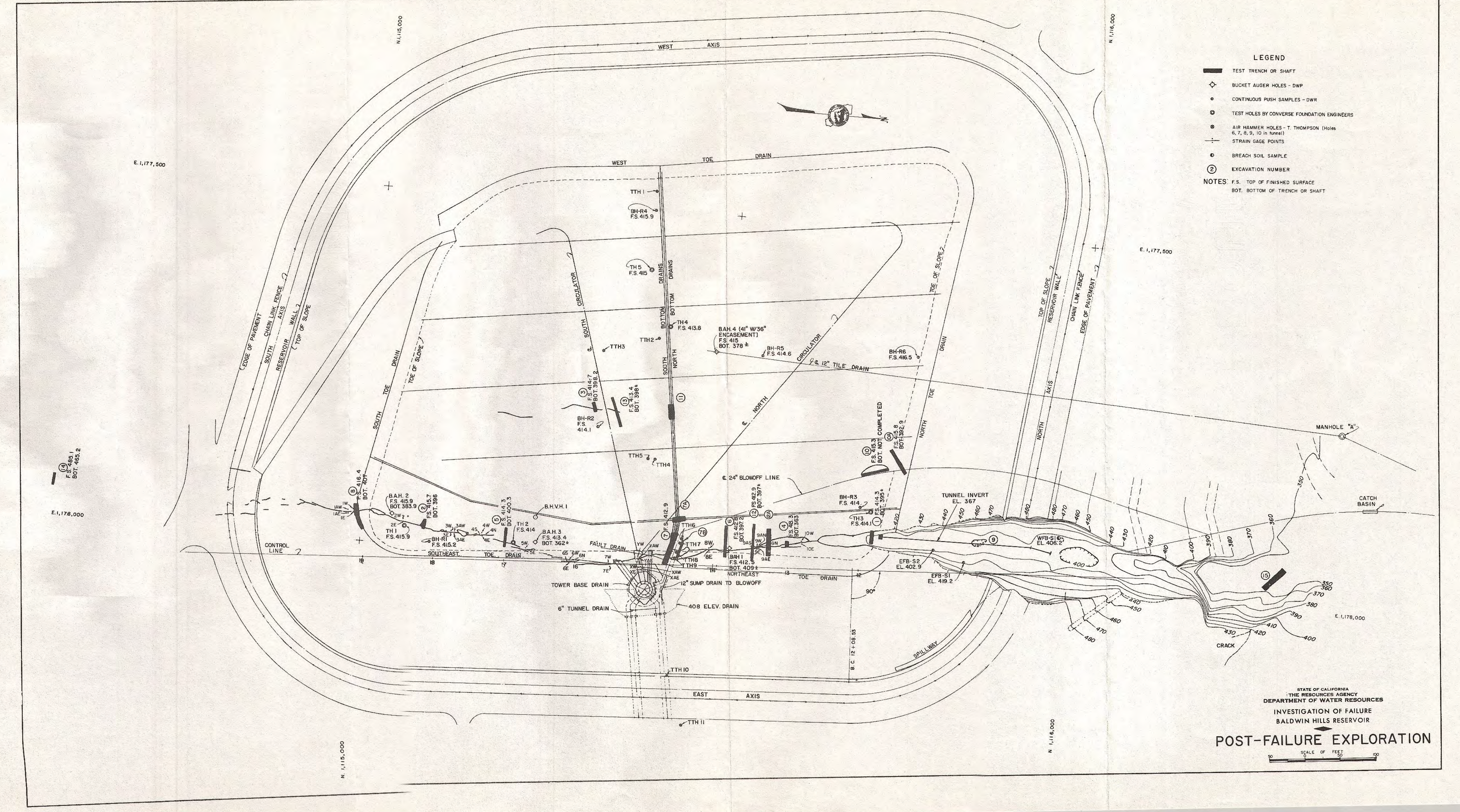
STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

GEOLOGIC PLAN AND CROSS SECTIONS OF BREACH



DRAINAGE INSPECTION CHAMBER

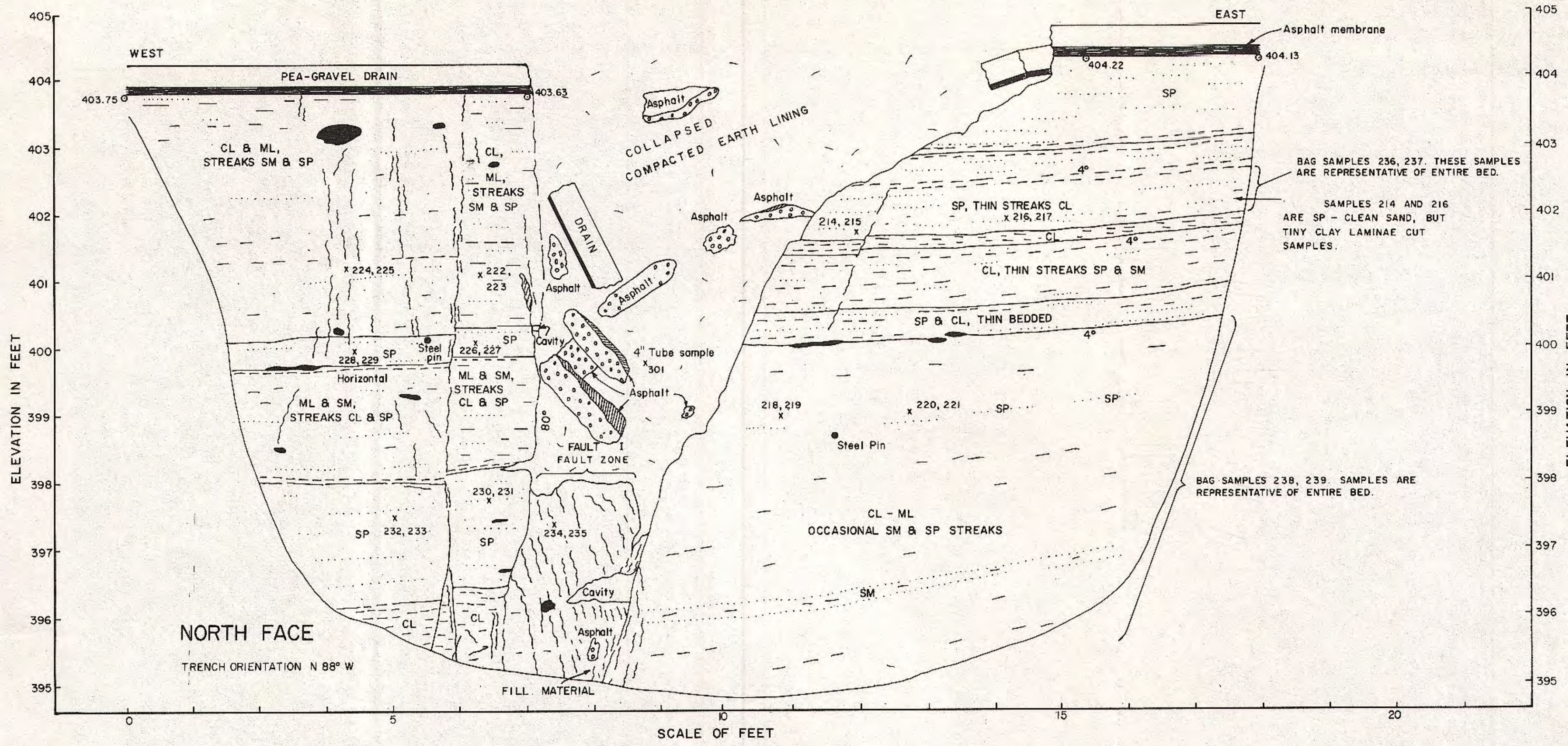

LEGEND
TEST TRENCH OR SHAFT
BUCKET AUGER HOLES - DWP
CONTINUOUS PUSH SAMPLES - DWR
TEST HOLES BY CONVERSE FOUNDATION ENGINEERS
AIR HAMMER HOLES - T. THOMPSON (Holes 6, 7, 8, 9, 10 in tunnel)
STRAIN GAGE POINTS
BREACH SOIL SAMPLE
EXCAVATION NUMBER
NOTES: F.S. TOP OF FINISHED SURFACE
BOT. BOTTOM OF TRENCH OR SHAFT
WEST AXIS
EAST AXIS
WEST TOE DRAIN
SOUTHEAST TOE DRAIN
NORTHEAST TOE DRAIN
SOUTH TOE DRAIN
NORTH TOE DRAIN
SOUTH BOTTOM DRAINS
NORTH BOTTOM DRAINS
SOUTH CIRCULATOR
NORTH CIRCULATOR
12" TILE DRAIN
24" BLOWOFF LINE
FAULT DRAIN
TOWER BASE DRAIN
12" SUMP DRAIN TO BLOWOFF
6" TUNNEL DRAIN
408 ELEV. DRAIN
CONTROL LINE
SPILLWAY
TUNNEL INVERT EL. 367
MANHOLE "A"
CATCH BASIN
CRACK
E. 1,177,500
E. 1,178,000
N. 1,115,000
N. 1,116,000
STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR
POST-FAILURE EXPLORATION
SCALE OF FEET
50 0 50 100

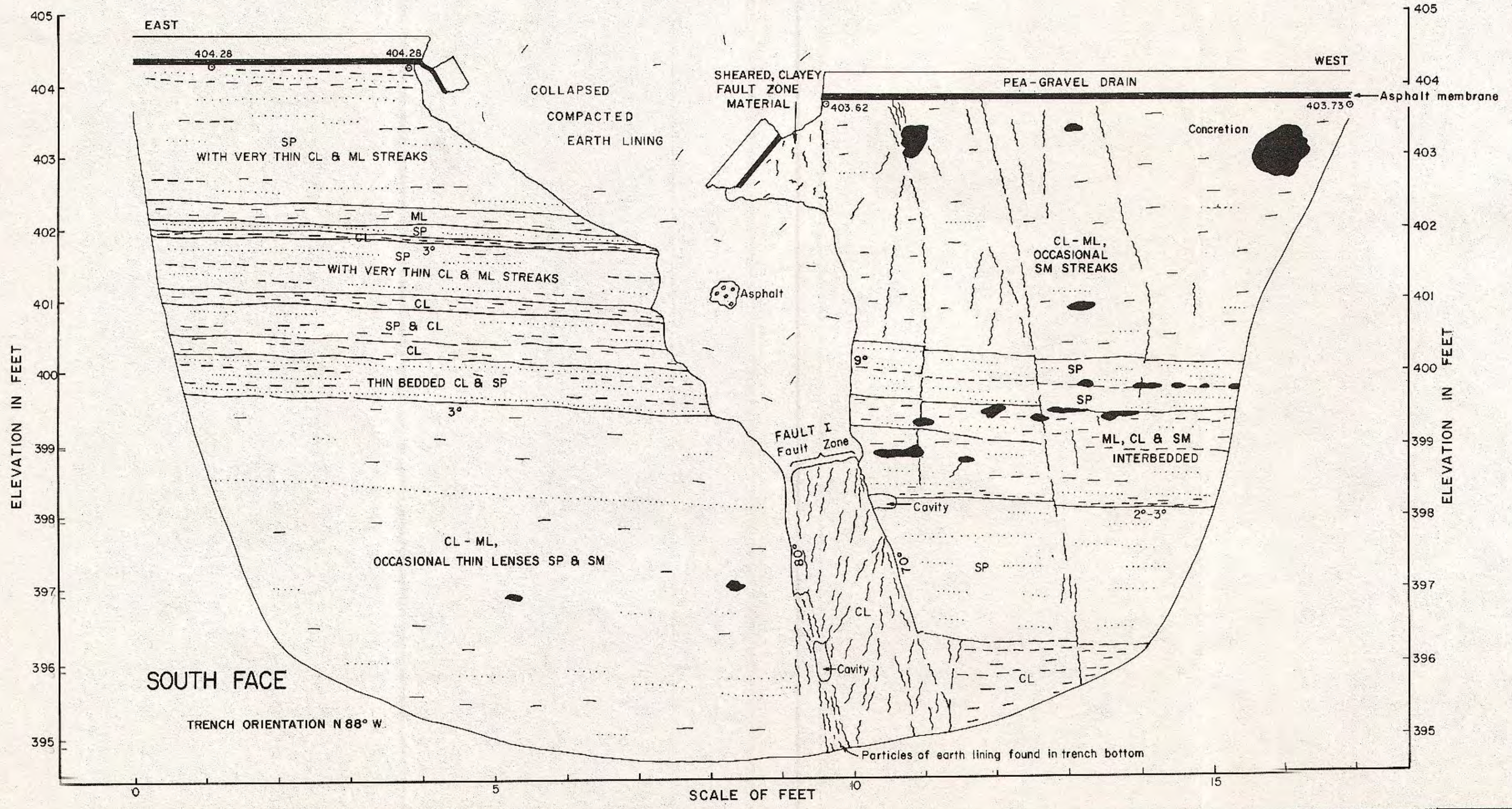
EXPLORATION DATA

Excavation or bore-hole number	Type and/or method	Purpose	Date completed	Depth	Sampling data	Remarks—significant results
1	Trench, Gradall	Inspect Fault I near breach	12-16-63	20′	12 push tube samples, 13 jar samples, 2 bag samples	Intersected Fault I. Found separation on drain approximately 0.4 feet.
2	Trench, Gradall	Inspect Faults I and II near south end of reservoir	12-17-63	17′	19 push tube samples, 23 jar samples	Intersected faults. Found separation on drain approximately 0.4 feet.
3	Trench, Gradall	Investigate Fault V near reservoir center	12-18-63	19′	11 push tube samples, 12 jar samples, 2 bag samples	Found Fault V, separation along drain approximately 0.3 feet. Large cavity found near trench bottom.
4	Shaft, clamshell and soft rock mining	Investigate nature of Fault I at depth	1-11-64	50′	DWP conducted extensive sampling—1 jar sample by DWR	Nature of Fault I was observed along entire depth.
5	Trench, Gradall	Further determine conditions on Faults I and II	1- 2-64	15′	None	Intersected faults. Approximately 0.5 feet aggregate separation observed on both faults.
6	Trench, Gradall	Further determine condition of Fault I	1- 3-64	17′	18 jar samples, 23 push tube samples, 4 bag samples	Intersected fault. Approximately 0.6 feet separation on pea-gravel drain.
7	Trench, Gradall and clamshell	Inspect chamber exterior in respect to faulting	1-18-64	26′	None	Trace of fault does not correspond to crack in chamber.
8	Trench, Gradall	Inspect intersection of Fault I and southeast toe drain	1- 6-64	13′	10 push tube samples, 1 bag of lining material only	Cracked face of tile coated with secondary material.
9	Shaft and tunnel, clamshell and soft rock mining	Investigate Fault I beneath breach	2-17-64	47′ shaft, 256′ tunnel	None	Asphaltic material from membrane found at Station 0+27 and 0+34.
10	Trench, Gradall	Inspect intersection of Fault V and 24-inch blowoff	1- 8-64	24′	None	Could not find fault.
10A	Trench, Gradall	Inspect intersection of Fault V and 24-inch blowoff	1-13-64	25′	Some by the Department of Water and Power	24-inch pipe intersected—no cracks or evidence of fault found.
11	Trench, Gradall	Inspect intersection of Fault V and three center tile drains	1-15-64	16′	4 push tube samples south face of lining material only	Cracked faces of tiles coated with secondary material.
12	Trench, Gradall	Further determine conditions on Fault I	1-16-64	18′	None	Very loose sands observed along fault zone.
13	Trench, Gradall	Find extension of cavity found on Fault V in Excavation No. 3	1-20-64	16′	None	Cavity found beneath pea-gravel drain which extends toward Excavation No. 3.
14	Trench, Gradall	Extend knowledge of Fault I south of reservoir	1-23-64	23′	None	Trace of Fault I established.

EXPLORATION DATA

Excavation or bore-hole number	Type and/or method	Purpose	Date completed	Depth	Sampling data	Remarks—significant results
15	Trench, Gradall	Find extension of faults north of breach	1-25-64	25′	None	Faults not definitely located; fill is too deep.
BH-R1	Rotary CP 8 drill	Sample blanket material near Faults I and II	1- 3-64	10.0′	9-2½″ push tube samples	
BH-R2	Rotary CP 8 drill	Sample blanket material near Fault V	1- 3-64	10.4′	13-2½″ push tube samples	
BH-R3	Rotary CP 8 drill	Sample lining material near breach	1- 3-64	9.8′	12-2½″ push tube samples	
BH-R4	Rotary CP 8 drill	Sample lining in 'soft' area, west center of reservoir	1- 4-64	9.5′	13-2½″ push tube samples	
BH-R5	Rotary CP 8 drill	Sample lining in area of no known disturbance	1- 4-64	10.2′	14-2½″ push tube samples	
BH-R6	Bucket auger drill	Sample compacted fill beneath	1- 9-64	21.9′	13-2½″ push tube samples	
BH-R7	40-inch bucket auger drill	Inspect condition of 12-inch tile drain	1-16-64	34.0′	10 jar samples	12-inch drain found to be open to Manhole "A"—Department of Water and Power calls this hole BAH No. 4.
BAH 1 thru BAH 3	Bucket auger drill	Sample lining and foundation materials	12-27-63			Not recorded by department.
Converse Engineers TH 1 thru TH 5	Bucket auger drill	Sample lining and foundation materials	1- 8-64		3″ ring drive samples	Not recorded by department.
EFB-S1	Hand sampling	Sample pertinent materials in breach	1- 2-64	Near surface	2 jar samples, 2-2½″ push tubes, 1 bag for compaction	
EFB-S2	Hand sampling	Sample pertinent materials in breach	1- 2-64	Near surface	2 jars, 2-2½″ push tubes, 1 bag for compaction	
WFB-S1	Hand sampling	Sample materials in breach	1- 2-64	Near surface	2 jar samples, 2½″ push tubes, 1 bag for compaction	
BHVH-1	Rotary-vacuum type rig	Obtain data on syndets, chlorides and moisture	2-12-64	139′	Continuous bag samples	
7A	Trench, Gradall	Find and determine condition of tile drains near chamber	2-11-64	10′	None	Drains were found undisturbed.
12A	Trench, Gradall	Further determine nature of Fault I	2-10-64	18′	None	Very loose soft soils were found in trench.
7B	Trench, Gradall	Inspect north exterior chamber wall and footing at Fault I	3-14-64	26′	None	Foundation percolation test made 3-19-64. No evidence of scour under chamber.





LEGEND

- BEDDED SAND
- SAND WITH NO APPARENT BEDDING

(GRAIN SIZE LARGER THAN NO. 200 SIEVE, SMALLER THAN NO. 4 SIEVE.)

- BEDDED SILT OR CLAY
- SILT OR CLAY WITH NO APPARENT BEDDING

(GRAIN SIZE SMALLER THAN NO. 200 SIEVE.)

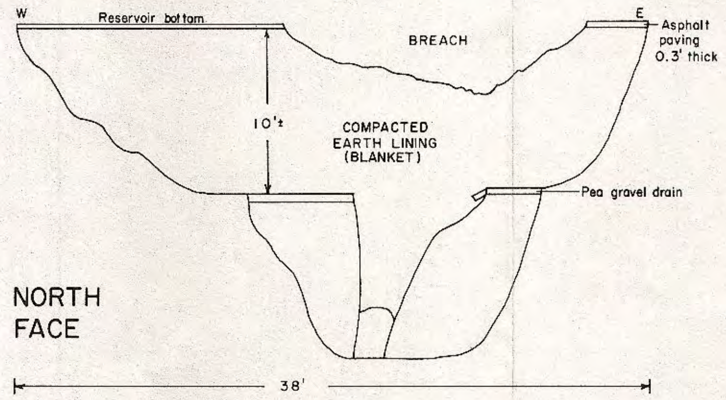
- CONTACT BETWEEN BEDS OR FAULTED MATERIALS, DASHED WHERE INDEFINITE.
- MINOR SHEAR
- 264, 265 — SAMPLING POINT. LAB NUMBERS INDICATED; FIRST NUMBER IS A 2-1/2" JACKED TUBE SAMPLE, SECOND NUMBER IS A JAR SAMPLE. VARIATIONS OF ABOVE ARE SHOWN ON TRENCH LOG. SAMPLES WERE TAKEN HORIZONTALLY, THAT IS PARALLEL TO BEDDING. FEW VERTICAL SAMPLES TAKEN ARE INDICATED BY
- CONCRETION, GENERALLY COMPOSED OF $CaCO_3$ and/or Fe_2O_3

UNIFIED SOIL CLASSIFICATION SYMBOLS USED ON TRENCH LOGS

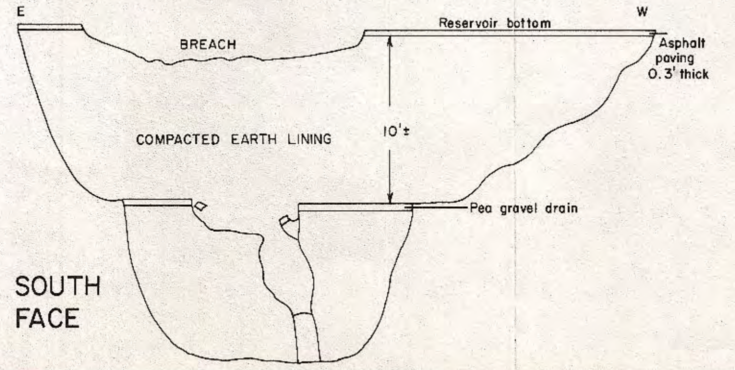
SP	POORLY GRADED SAND, PREDOMINANTLY ONE SIZE.
SM	SILTY SAND, POORLY GRADED SAND-SILT MIXTURES, NON-PLASTIC FINES
SC	CLAYEY SAND, POORLY GRADED SAND-CLAY MIXTURES, PLASTIC FINES
ML	INORGANIC SILT AND VERY FINE SAND, SILTY OR CLAYEY FINE SAND WITH SLIGHT PLASTICITY.
CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, SANDY CLAY, SILTY CLAY, LEAN CLAY

GENERAL NOTES

1. ALL MATERIAL DESIGNATED SP ON THESE TRENCH LOGS IS FINE TO MEDIUM SAND, GRAY TO BUFF, CLEAN, POORLY GRADED, LOOSE, MOIST, MICACEOUS, ARKOSIC, FREQUENTLY ASHY, GENERALLY LOW DENSITY, APPARENTLY HIGHLY PERMEABLE, EASILY ERODIBLE.
2. CL AND ML IS GENERALLY MORE FIRM AND COMPACT THAN SP. CL AND ML IS MOIST, THIN BEDDED TO MASSIVE, AND FREQUENTLY IRON-OXIDE-STAINED.
3. FAULTED MATERIAL IS USUALLY A MIXTURE OF LOOSE SAND, SILT, AND CLAY WITH NO APPARENT STRUCTURE.
4. MOST BEDS OF THE INGLEWOOD FORMATION CAN BE TRACED 20 OR 30 FEET, BUT PROBABLY CANNOT BE TRACED ACROSS THE RESERVOIR THE INGLEWOOD FORMATION CONSISTS OF THIN TO THICK BEDDED, INTERBEDDED CLAY AND SILT AND SAND, WHICH ARE POORLY CONSOLIDATED.
5. TRENCHES WERE EXCAVATED WITH A GRADALL.
6. SOILS CLASSIFICATION BASED ON FIELD DETERMINATIONS.



NORTH FACE



SOUTH FACE

NOTES:

1. NEARLY ALL SHEAR FACES THIS TRENCH WERE COATED WITH SOFT, WHITE, CALCAREOUS MATERIAL.
2. AVERAGE DIP OF FAULT THIS TRENCH 75° WESTERLY, STRIKE N 8° E. AVERAGE DIP OF BEDS 3° - 4° WESTERLY, STRIKE N 75° W. SEDIMENTS THIS TRENCH ARE INGLEWOOD FORMATION.
3. A GREAT DEAL OF FOREIGN MATERIAL IS PRESENT ALONG FAULT ZONE. THIS TRENCH IS LOCATED IN FAILURE PATH JUST ON NORTHERLY EDGE OF RESERVOIR BOTTOM.

STATE OF CALIFORNIA
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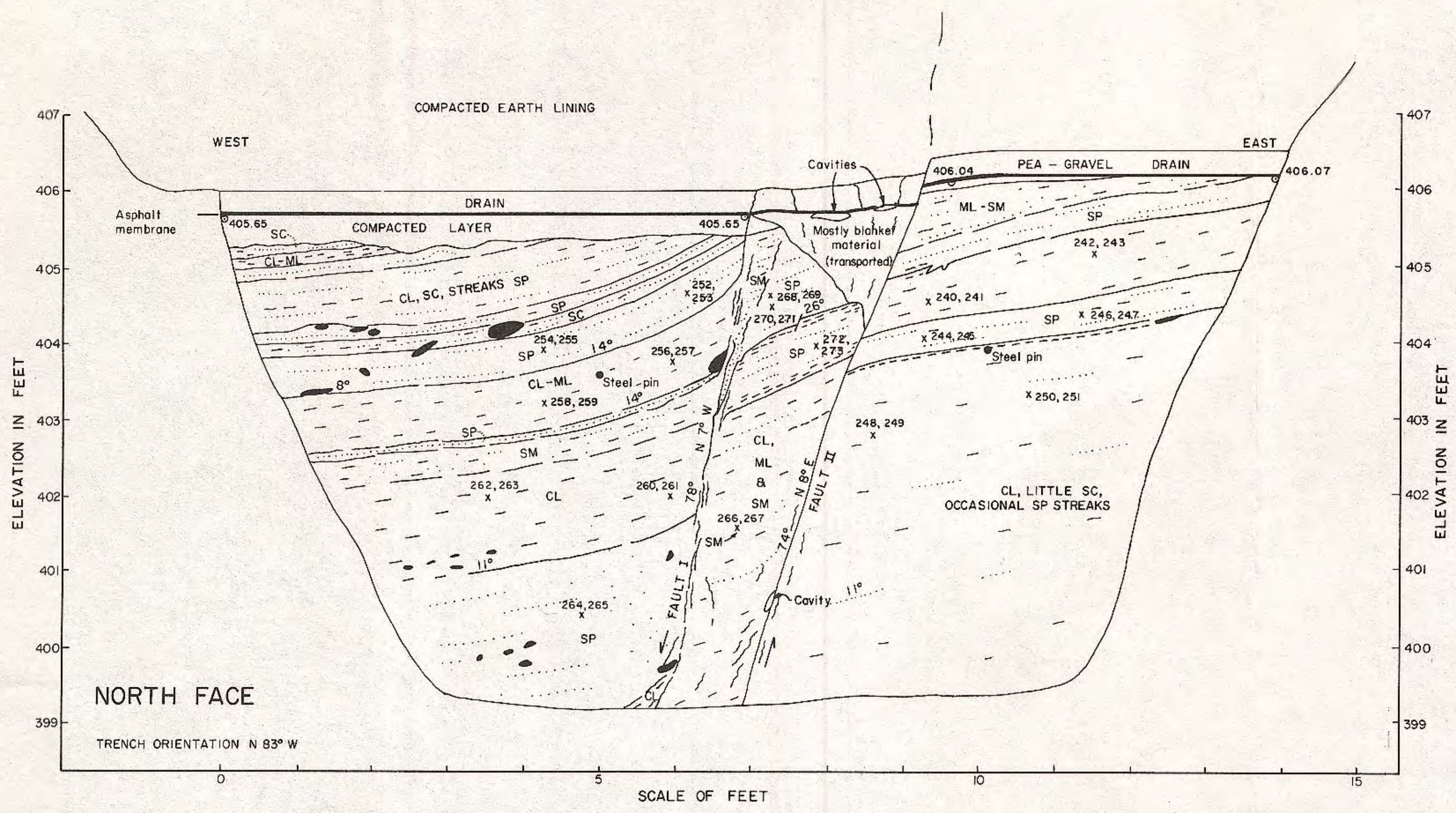
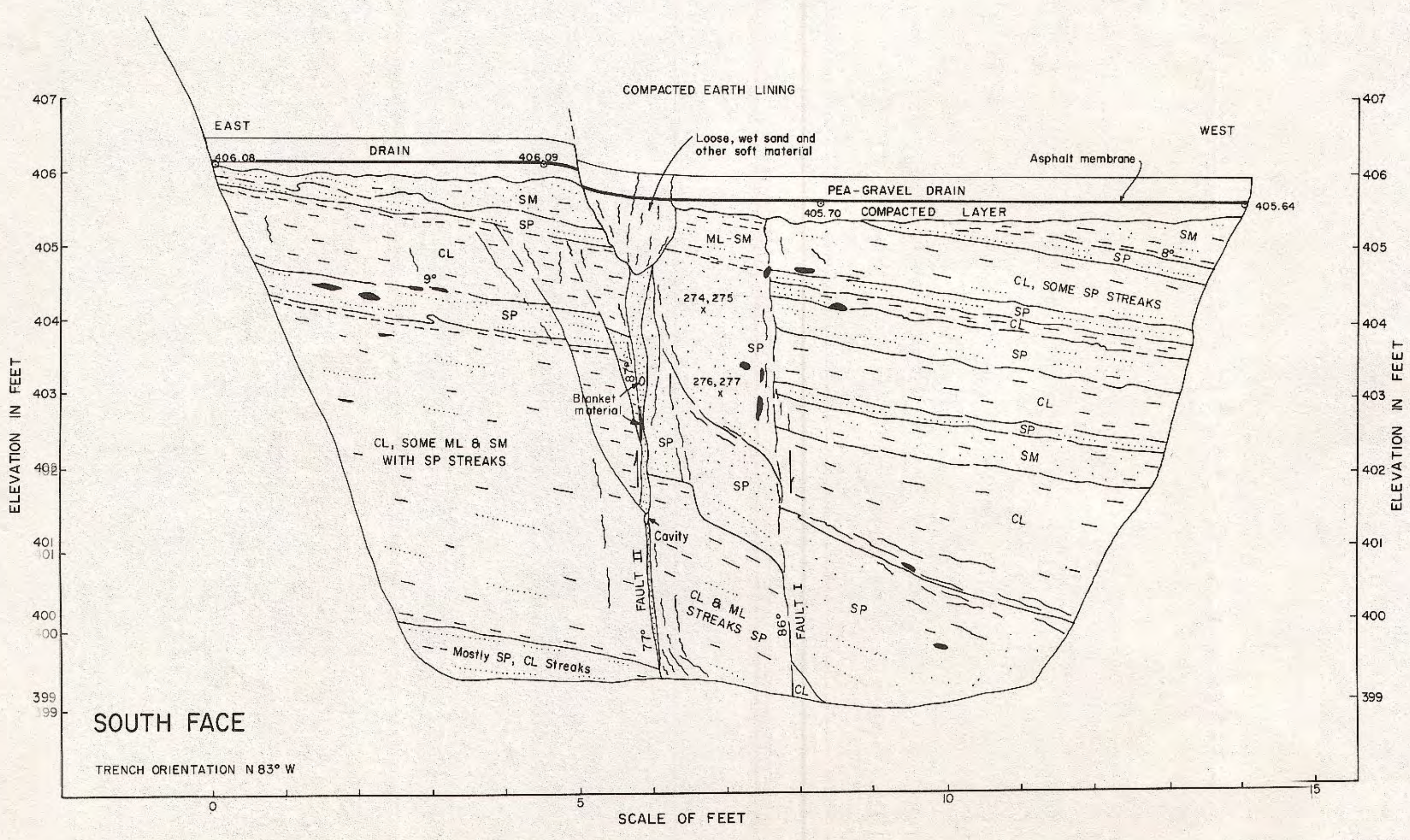
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 1





LEGEND

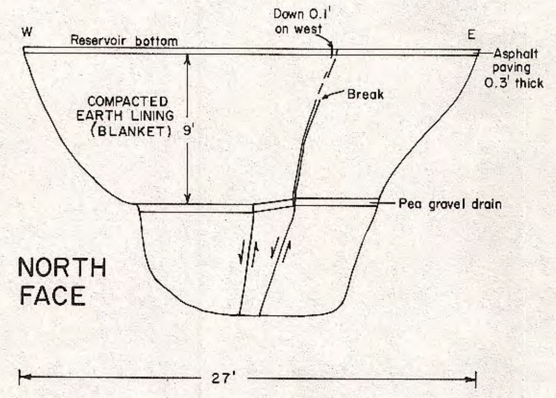
(dotted)	BEDDED SAND	GRAIN SIZE LARGER THAN NO. 200 SIEVE, SMALLER THAN NO. 4 SIEVE.
(stippled)	SAND WITH NO APPARENT BEDDING	
(dashed)	BEDDED SILT OR CLAY	GRAIN SIZE SMALLER THAN NO. 200 SIEVE.
(broken dashes)	SILT OR CLAY WITH NO APPARENT BEDDING	
(line)	CONTACT BETWEEN BEDS OR FAULTED MATERIALS, DASHED WHERE INDEFINITE.	
(line)	MINOR SHEAR	
264, 265 x	SAMPLING POINT. LAB NUMBERS INDICATED; FIRST NUMBER IS A 2-1/2" JACKED TUBE SAMPLE, SECOND NUMBER IS A JAR SAMPLE. VARIATIONS OF ABOVE ARE SHOWN ON TRENCH LOG. SAMPLES WERE TAKEN HORIZONTALLY, THAT IS PARALLEL TO BEDDING. FEW VERTICAL SAMPLES TAKEN ARE INDICATED BY	
(black)	CONCRETION, GENERALLY COMPOSED OF $CaCO_3$ and/or Fe_2O_3	

UNIFIED SOIL CLASSIFICATION SYMBOLS USED ON TRENCH LOGS

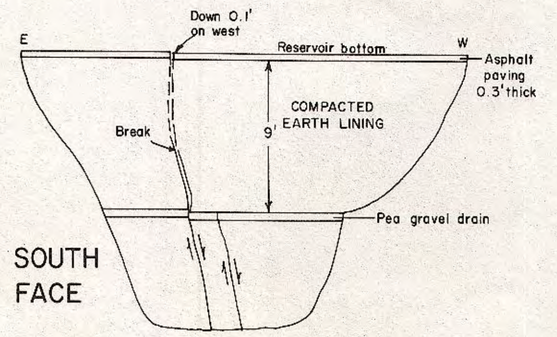
SP	POORLY GRADED SAND, PREDOMINANTLY ONE SIZE
SM	SILTY SAND, POORLY GRADED SAND-SILT MIXTURES, NON-PLASTIC FINES
SC	CLAYEY SAND, POORLY GRADED SAND-CLAY MIXTURES, PLASTIC FINES
ML	INORGANIC SILT AND VERY FINE SAND, SILTY OR CLAYEY FINE SAND WITH SLIGHT PLASTICITY
CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, SANDY CLAY, SILTY CLAY, LEAN CLAY

GENERAL NOTES

1. ALL MATERIAL DESIGNATED SP ON THESE TRENCH LOGS IS FINE TO MEDIUM SAND, GRAY TO BUFF, CLEAN, POORLY GRADED, LOOSE, MOIST, MICACEOUS, ARKOSIC, FREQUENTLY ASHY, GENERALLY LOW DENSITY, APPARENTLY HIGHLY PERMEABLE, EASILY ERODIBLE.
2. CL AND ML IS GENERALLY MORE FIRM AND COMPACT THAN SP. CL AND ML IS MOIST, THIN BEDDED TO MASSIVE, AND FREQUENTLY IRON-OXIDE-STAINED.
3. FAULTED MATERIAL IS USUALLY A MIXTURE OF LOOSE SAND, SILT, AND CLAY WITH NO APPARENT STRUCTURE.
4. MOST BEDS OF THE INGLEWOOD FORMATION CAN BE TRACED 20 OR 30 FEET, BUT PROBABLY CANNOT BE TRACED ACROSS THE RESERVOIR. THE INGLEWOOD FORMATION CONSISTS OF THIN TO THICK BEDDED, INTERBEDDED CLAY AND SILT AND SAND, WHICH ARE POORLY CONSOLIDATED.
5. TRENCHES WERE EXCAVATED WITH A GRADALL.
6. SOILS CLASSIFICATION BASED ON FIELD DETERMINATIONS.



NORTH FACE



SOUTH FACE

NOTES:

1. A ZONE OF LOOSE DISTURBED MATERIAL IS PRESENT ALONG FAULT ZONE JUST BELOW PEA GRAVEL DRAIN. THIS MATERIAL IS FOREIGN AND WAS DEPOSITED – PRESUMABLY IN A VOID – AFTER COMPLETION OF RESERVOIR.
2. SEDIMENTS THIS TRENCH ARE INGLEWOOD FORMATION.

STATE OF CALIFORNIA
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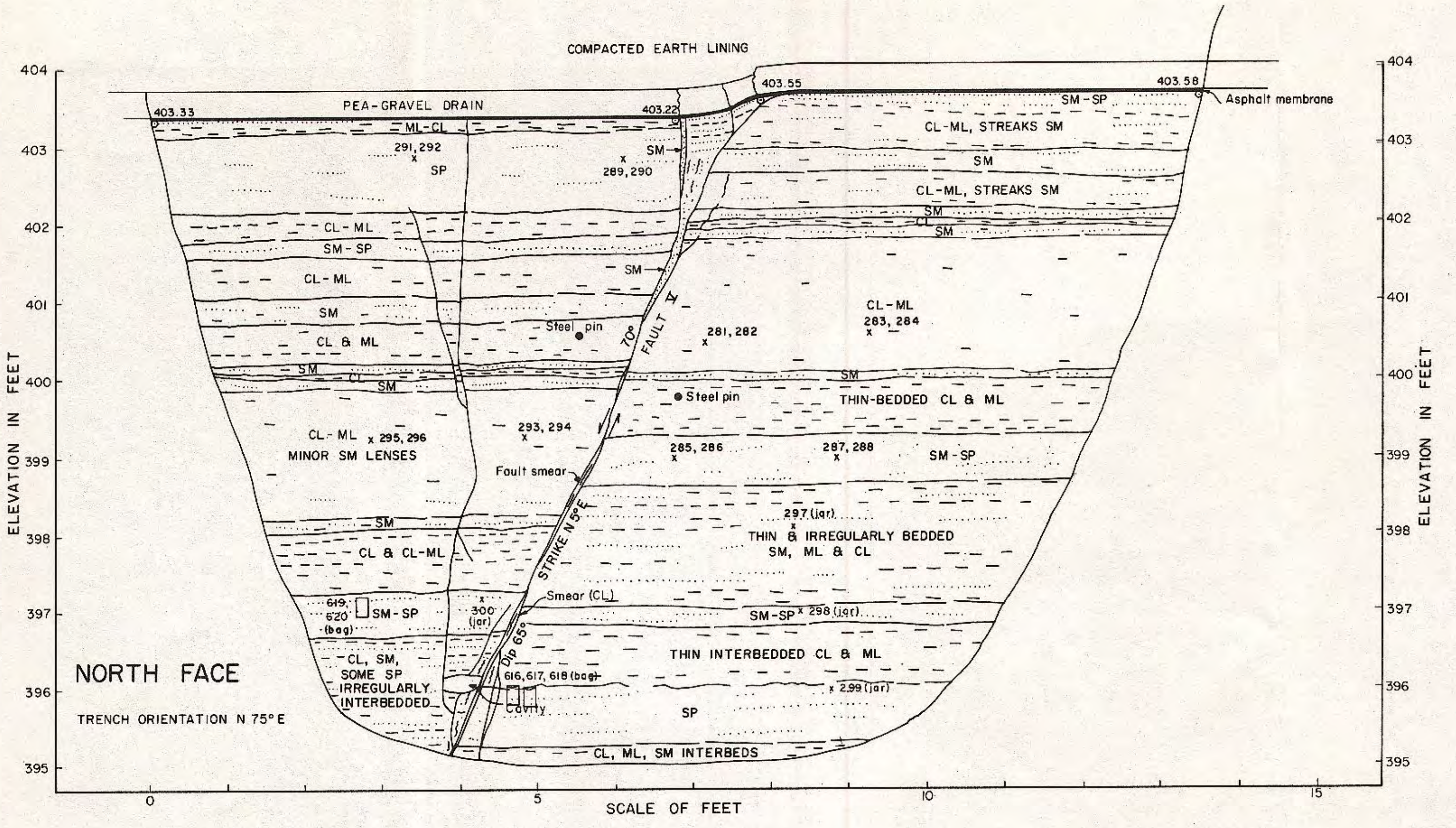
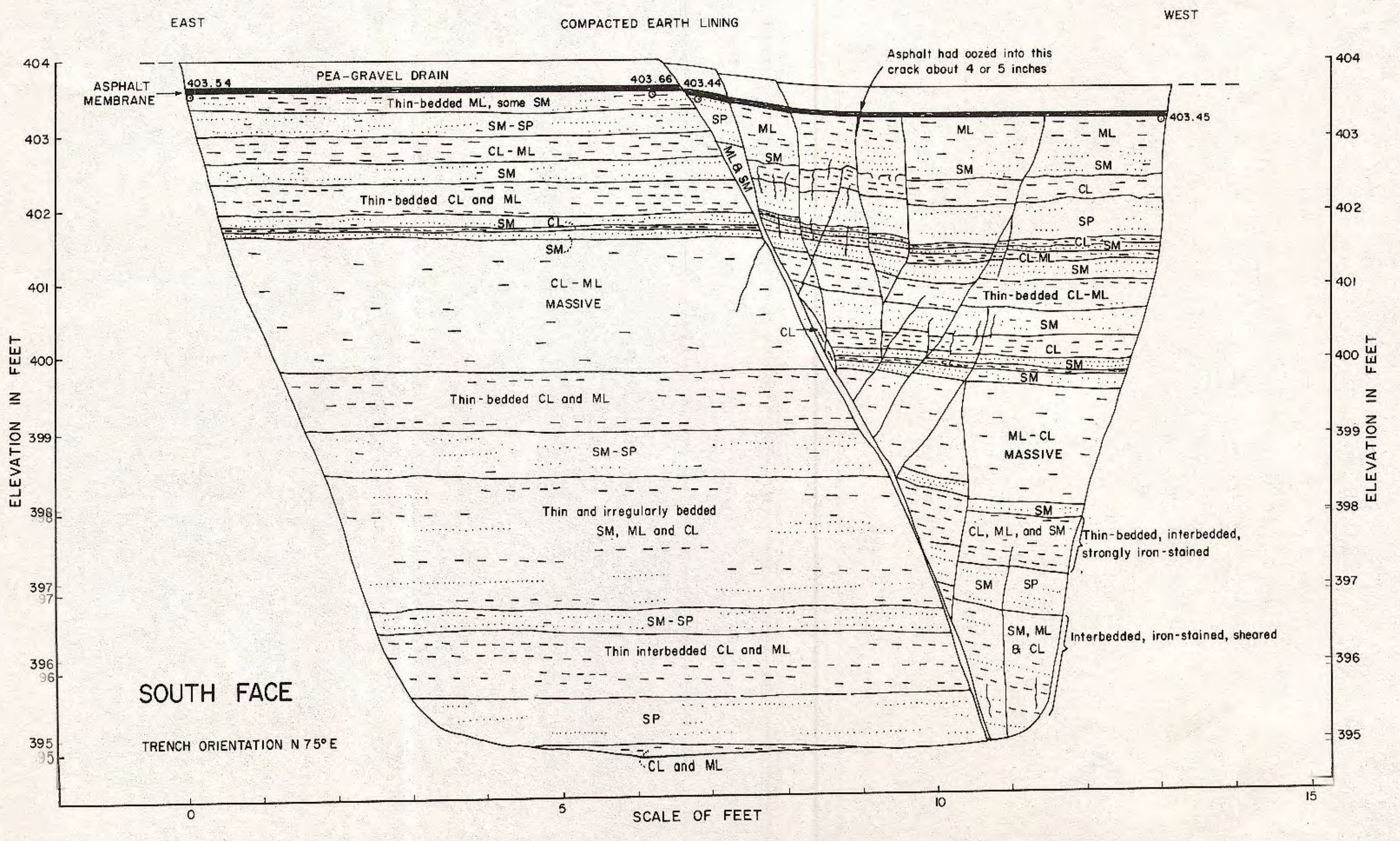
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 2





LEGEND

- BEDDED SAND — GRAIN SIZE LARGER THAN NO. 200 SIEVE, SMALLER THAN NO. 4 SIEVE.
- SAND WITH NO APPARENT BEDDING
- BEDDED SILT OR CLAY — GRAIN SIZE SMALLER THAN NO. 200 SIEVE.
- SILT OR CLAY WITH NO APPARENT BEDDING
- CONTACT BETWEEN BEDS OR FAULTED MATERIALS, DASHED WHERE INDEFINITE.
- MINOR SHEAR
- 264, 265 x — SAMPLING POINT. LAB NUMBERS INDICATED; FIRST NUMBER IS A 2-1/2" JACKED TUBE SAMPLE, SECOND NUMBER IS A JAR SAMPLE. VARIATIONS OF ABOVE ARE SHOWN ON TRENCH LOG. SAMPLES WERE TAKEN HORIZONTALLY, THAT IS PARALLEL TO BEDDING. FEW VERTICAL SAMPLES TAKEN ARE INDICATED BY
- CONCRETION, GENERALLY COMPOSED OF $CaCO_3$ and/or Fe_2O_3

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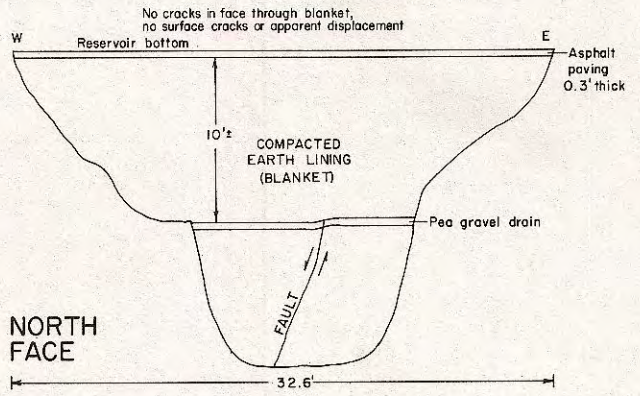
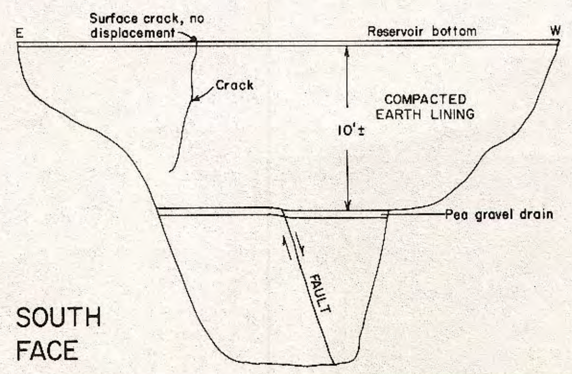


NORTH FACE



SOUTH FACE

NOTES:

1. CAVITY LOCATED NEAR BOTTOM OF TRENCH IN NORTH FACE IS ALONG FAULT ZONE AND OPENS INWARD TO A VOID ABOUT 11' LONG, 1' TO 2' WIDE AND ABOUT 3' HIGH. CAVITY DISCOVERED 1/11/64, THREE WEEKS AFTER TRENCH WAS FIRST EXCAVATED. A THIN, IRREGULAR FILM OF SECONDARY MATERIAL COVERED WALLS AND ROOF OF CAVITY.
2. AVERAGE DISPLACEMENT ALONG FAULT HERE IS 2.2'
3. DIP OF BEDS IS VERY FLAT WITH PERHAPS 2° - 3° DIP TO NORTHWEST.
4. IRON OXIDE STAINING IS VERY COMMON, PARTICULARLY IN SOME CLAYS AND ALONG CONTACTS OF BEDS.
5. SEDIMENTS THIS TRENCH ARE INGLEWOOD FORMATION.

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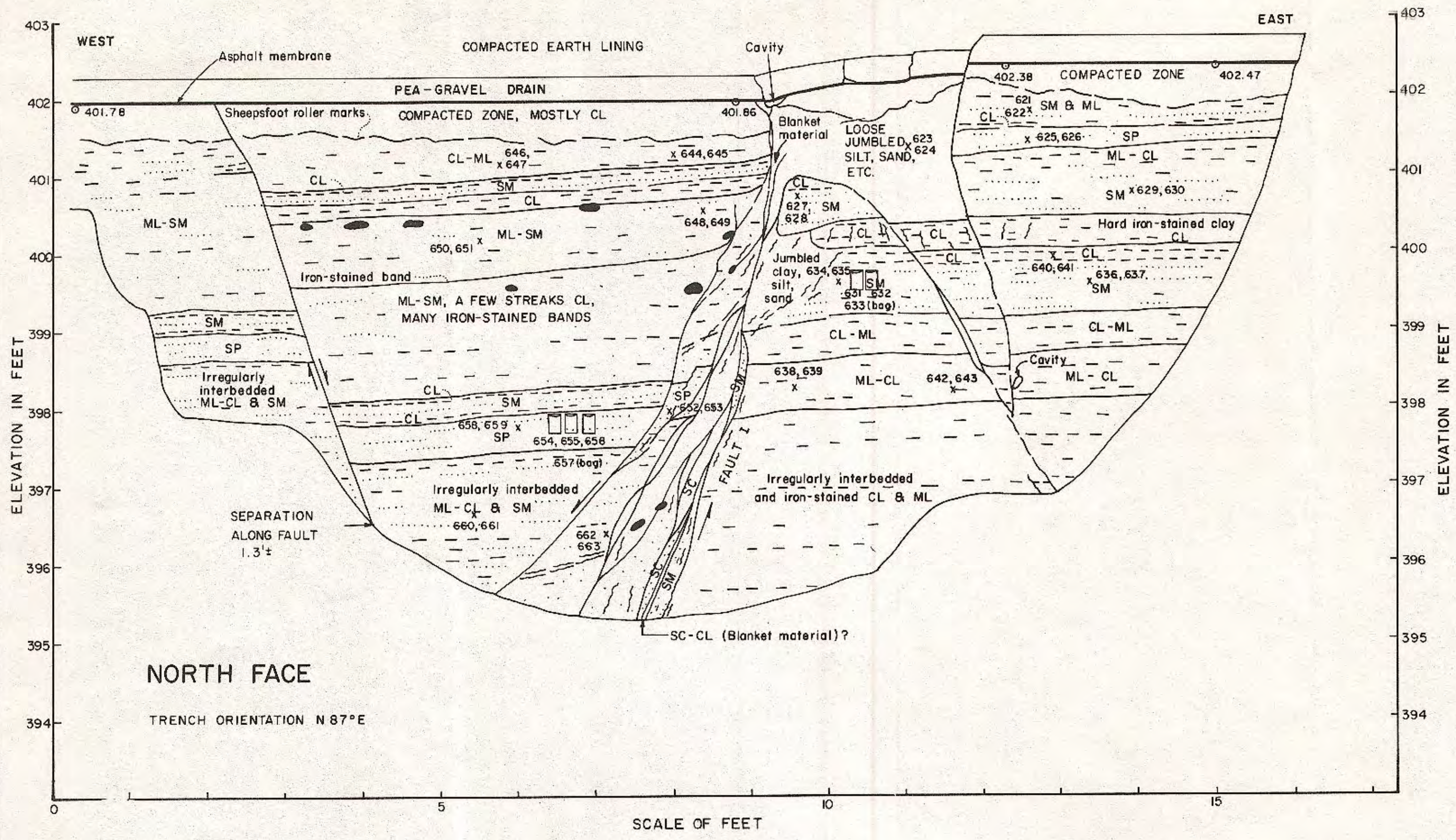
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 3



LEGEND

BEDDED SAND	}	GRAIN SIZE LARGER THAN NO. 200 SIEVE, SMALLER THAN NO. 4 SIEVE.
SAND WITH NO APPARENT BEDDING		
BEDDED SILT OR CLAY	}	GRAIN SIZE SMALLER THAN NO. 200 SIEVE.
SILT OR CLAY WITH NO APPARENT BEDDING		

CONTACT BETWEEN BEDS OR FAULTED MATERIALS, DASHED WHERE INDEFINITE.

MINOR SHEAR

264, 265 X — SAMPLING POINT. LAB NUMBERS INDICATED; FIRST NUMBER IS A 2-1/2" JACKED TUBE SAMPLE, SECOND NUMBER IS A JAR SAMPLE. VARIATIONS OF ABOVE ARE SHOWN ON TRENCH LOG. SAMPLES WERE TAKEN HORIZONTALLY, THAT IS PARALLEL TO BEDDING. FEW VERTICAL SAMPLES TAKEN ARE INDICATED BY

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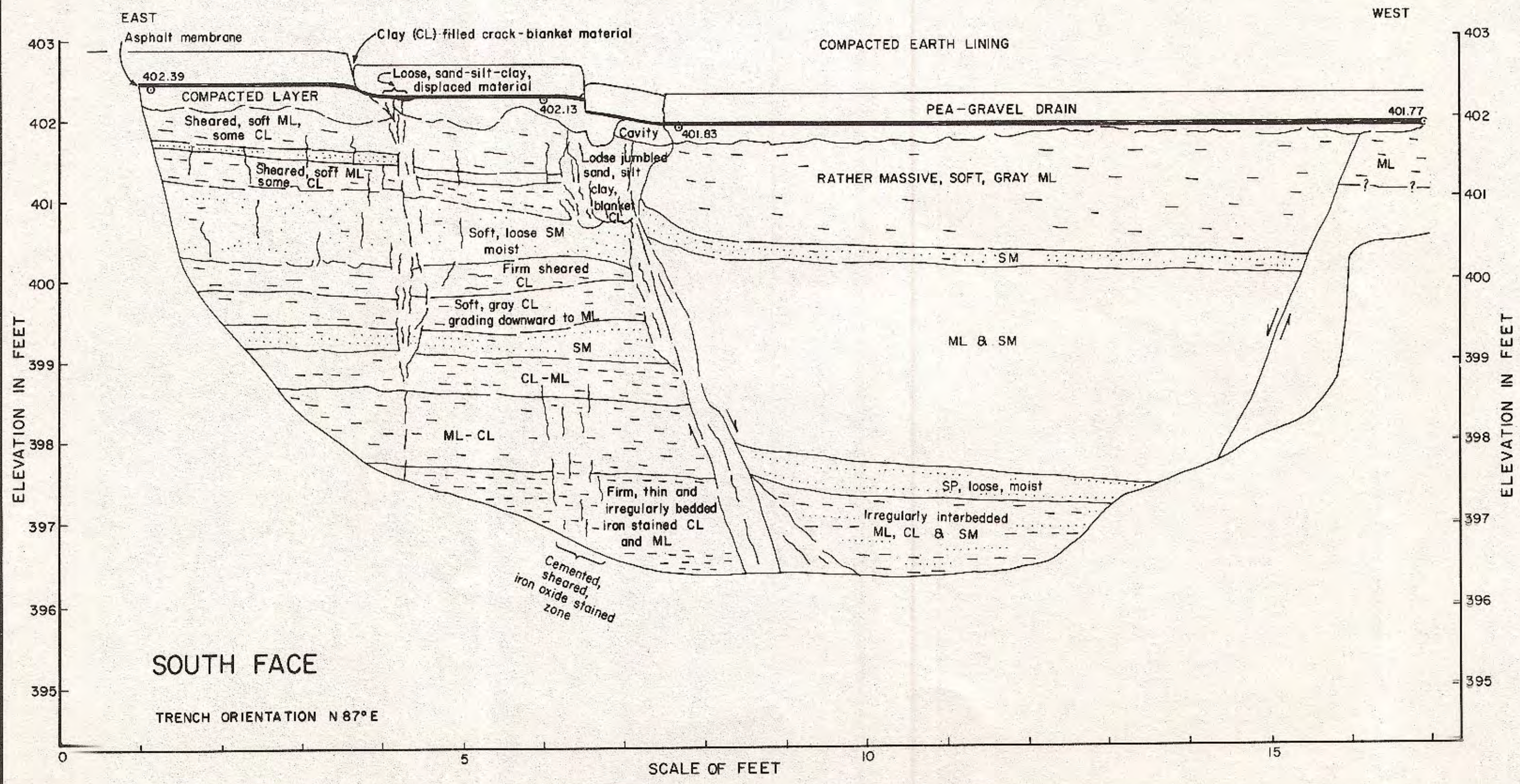
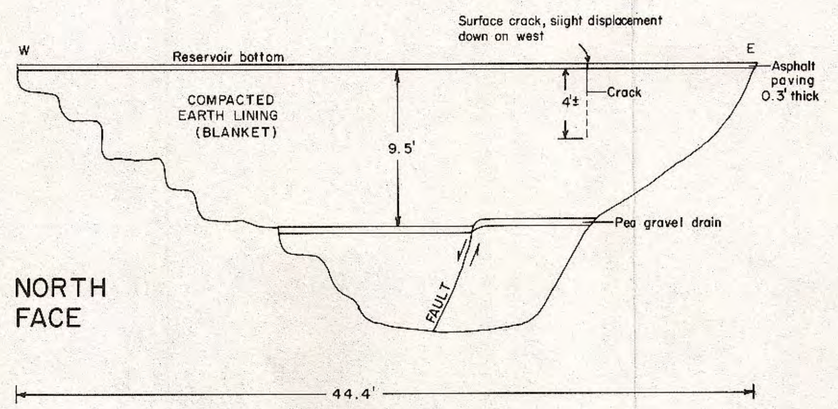
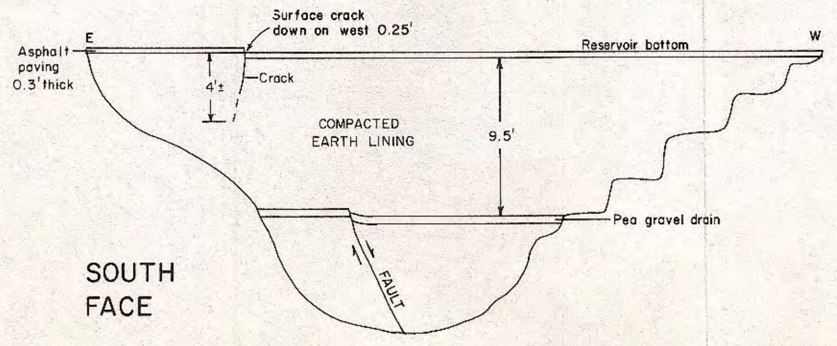
6. SOILS CLASSIFICATION BASED ON FIELD DETERMINATIONS.







NOTES:

1. LOG OF SOUTH FACE INCOMPLETE. RAIN WITH SUBSEQUENT COLLAPSE OF TRENCH INTERRUPTED LOGGING.
2. IN NORTH FACE A RATHER EXTENSIVE ZONE OF DISTURBED MATERIAL IS PRESENT ALONG FAULT ZONE JUST BELOW PEA GRAVEL DRAIN. THIS MATERIAL WAS DEPOSITED - PRESUMABLY IN A VOID - DURING A TIME INTERVAL FOLLOWING CONSTRUCTION OF RESERVOIR. THIS ZONE DIVIDES INTO TWO MUCH SMALLER ZONES IN SOUTH FACE.
3. THIN BAND OF CLAYEY SAND TO SANDY CLAY RESEMBLING BLANKET MATERIAL IS INDICATED NEAR BOTOM OF NORTH FACE ALONG FAULT ZONE.
4. DIPS IN SEDIMENTS ARE GENTLE, PERHAPS 3° - 4° TO THE NORTHWEST. SEDIMENTS THIS TRENCH ARE INGLEWOOD FORMATION.

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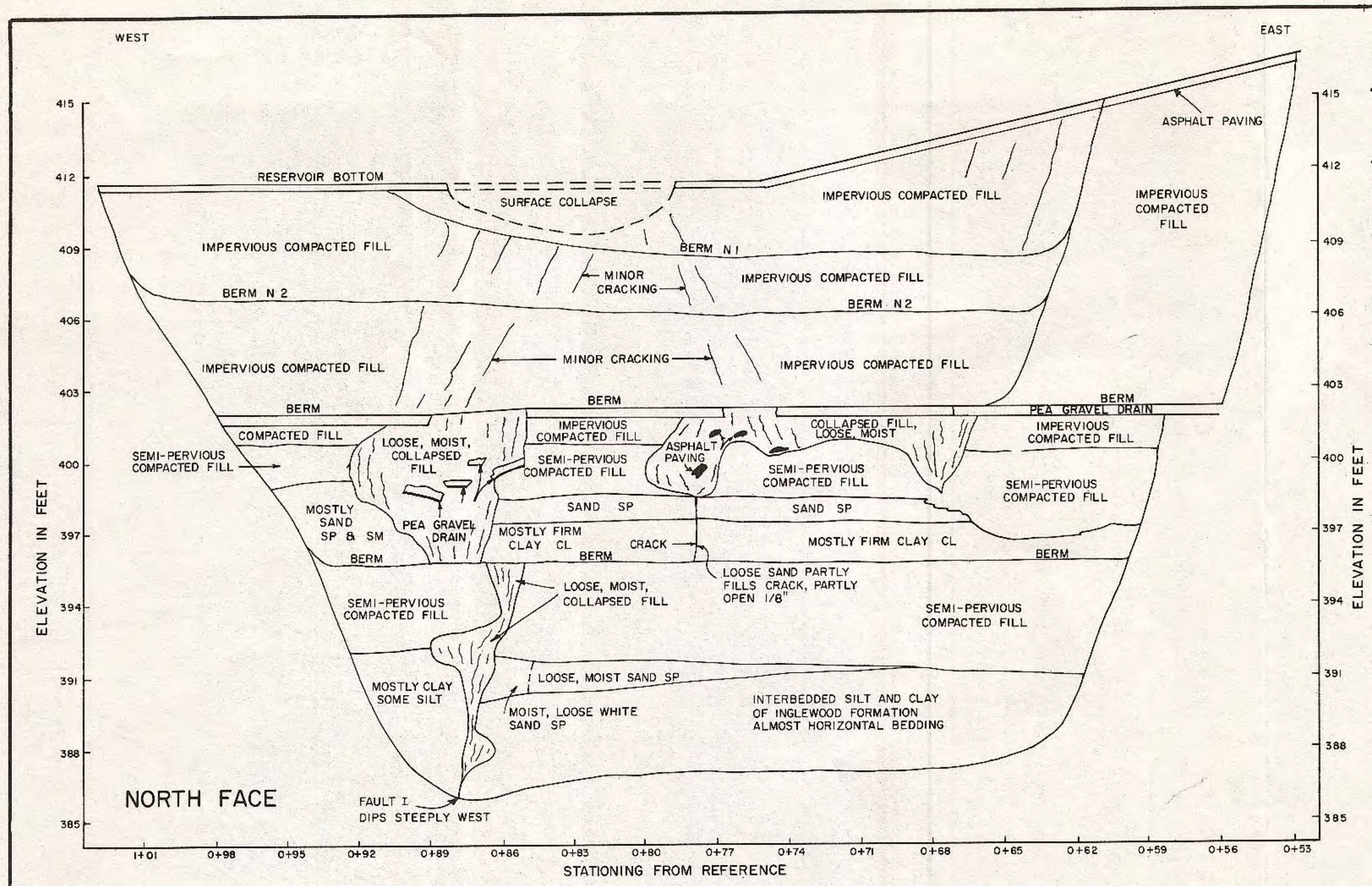
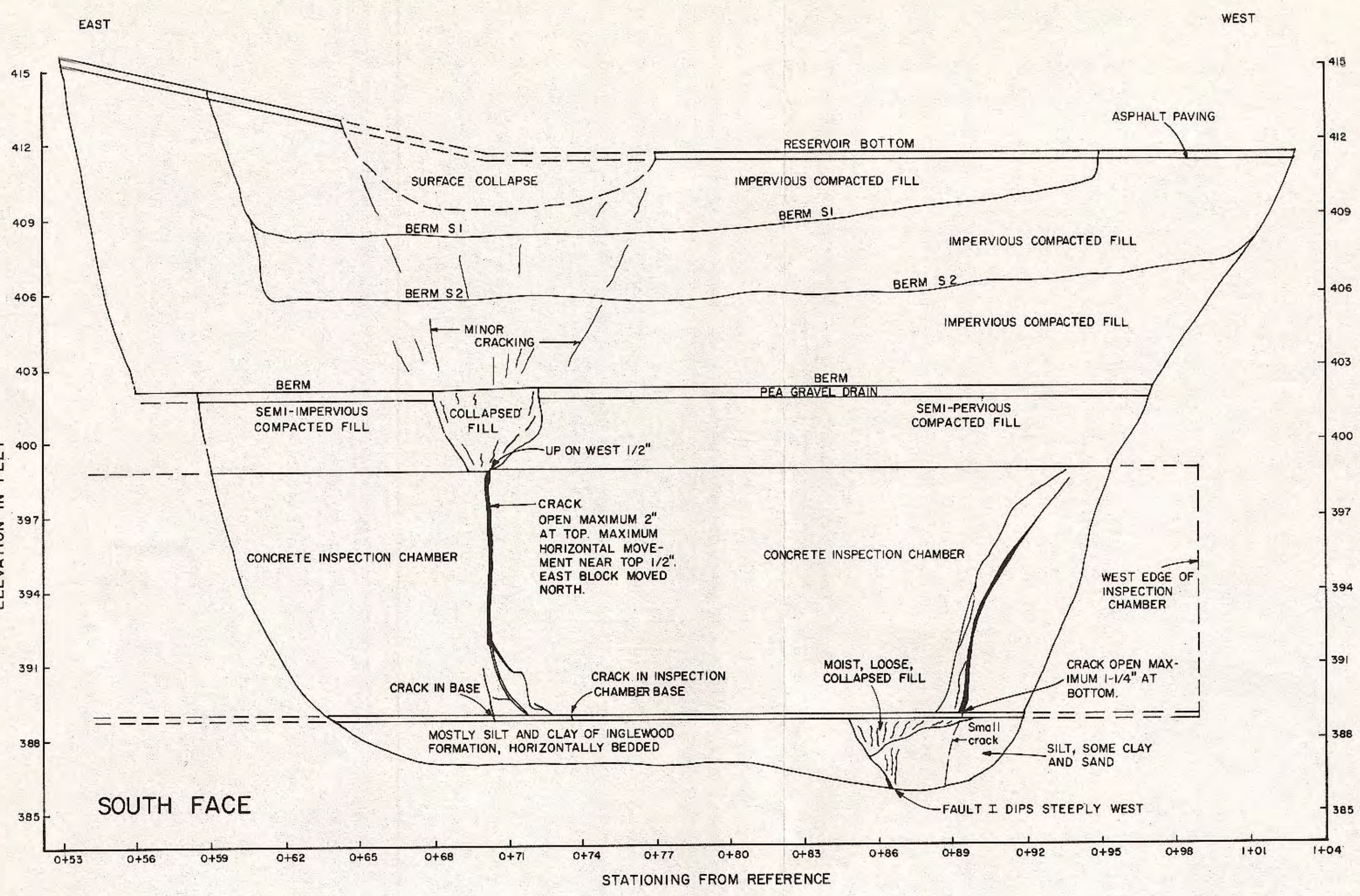
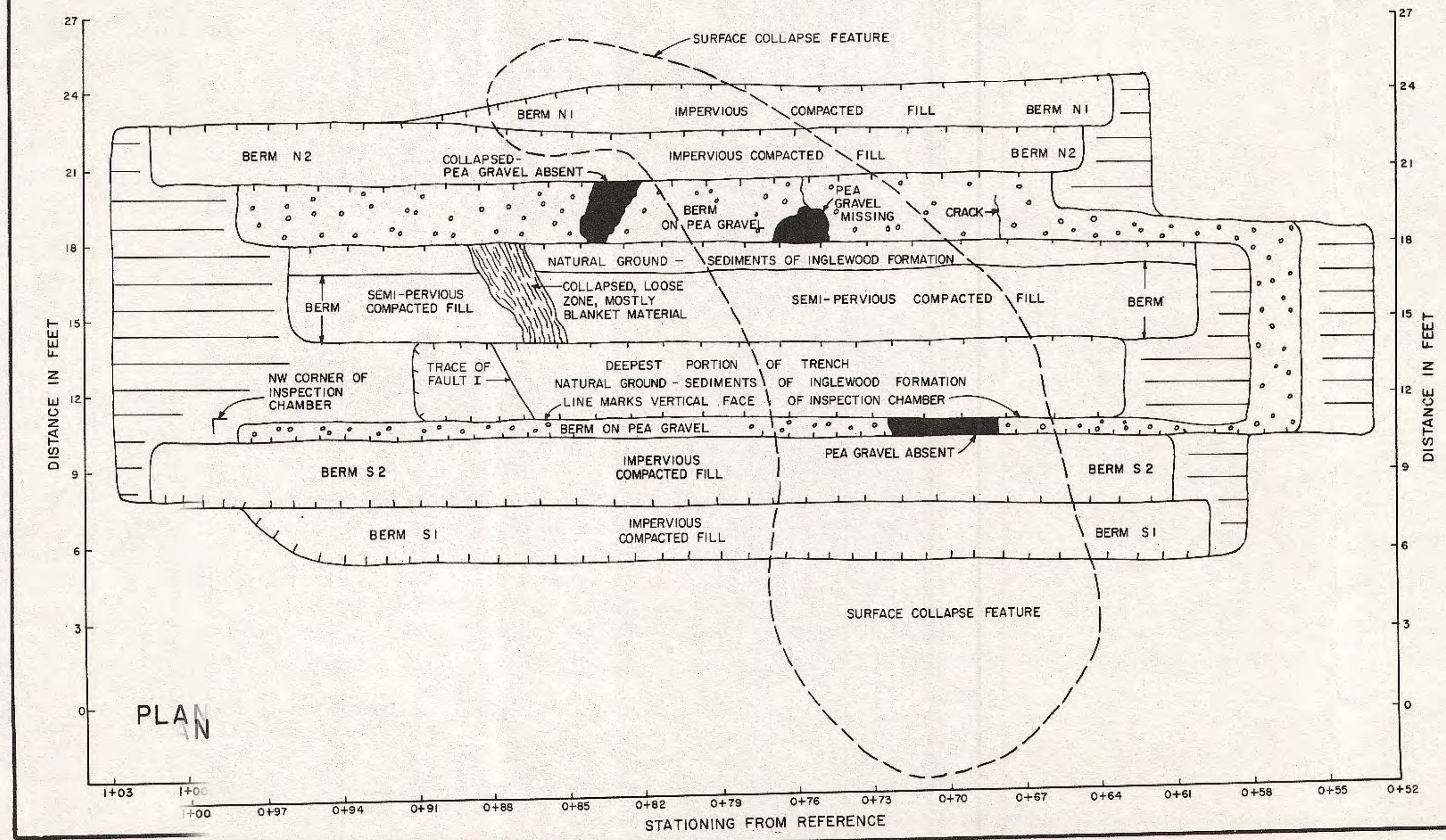
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 6







NOTES

1. THIS TRENCH WAS EXCAVATED WITH A GRADALL ON MARCH 11-14, 1964.
2. BERMS AS SHOWN ON DRAWING WERE A RESULT OF EXCAVATION AND SHORING PROCEDURE.

STATE OF CALIFORNIA
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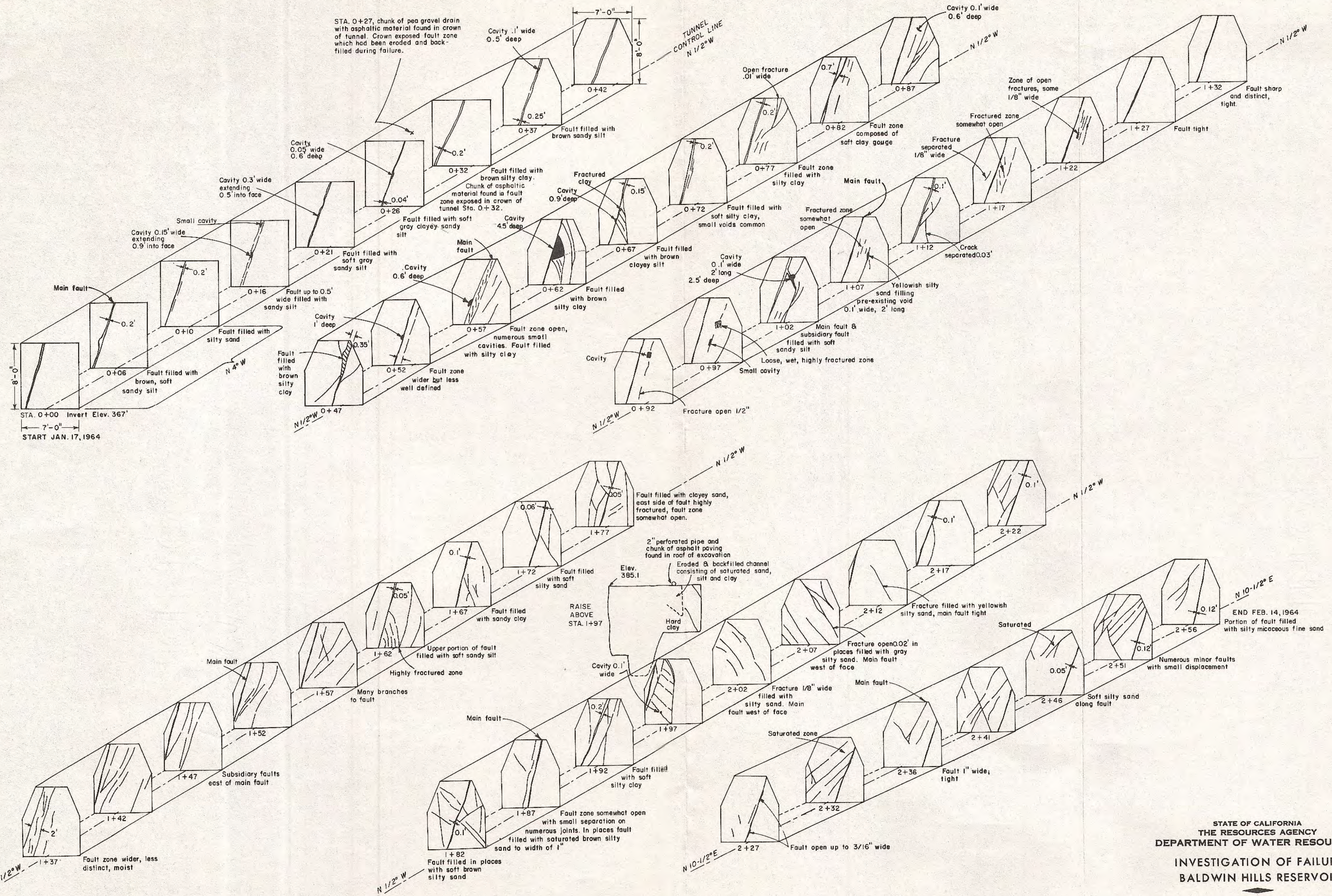
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

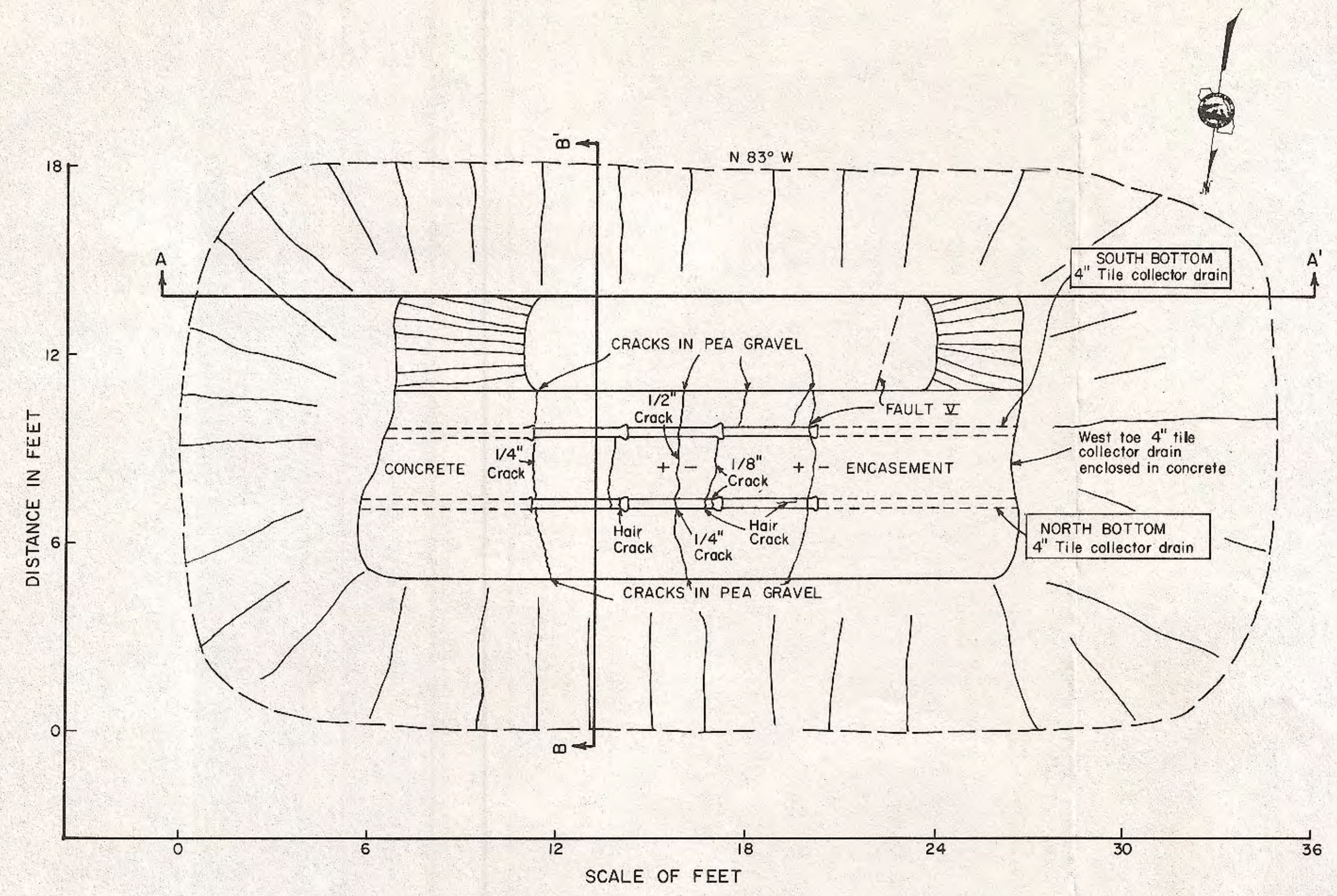
LOG OF EXCAVATION No. 7B



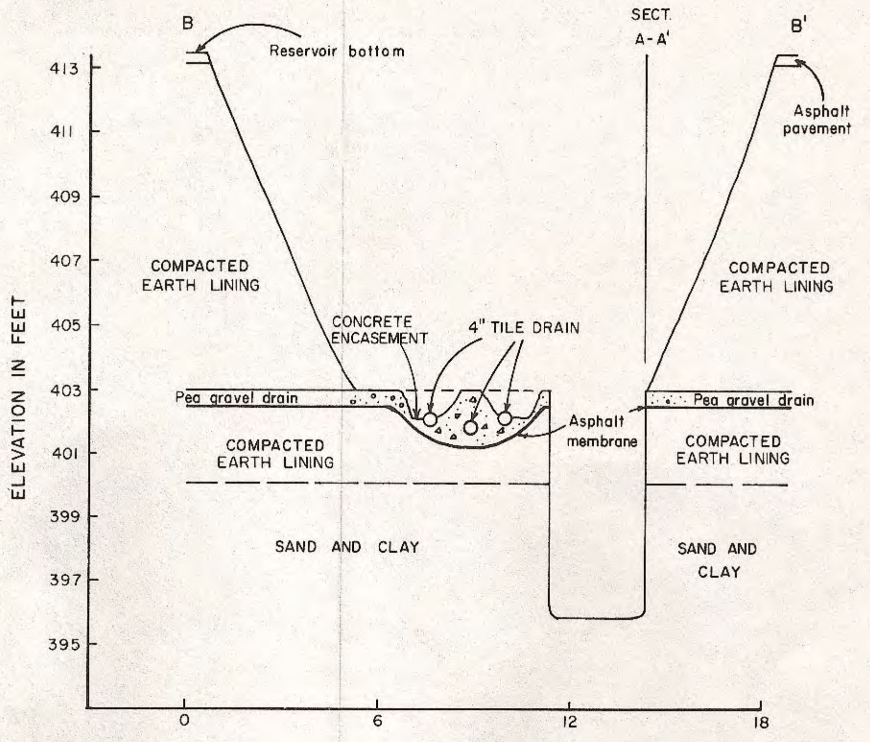
STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

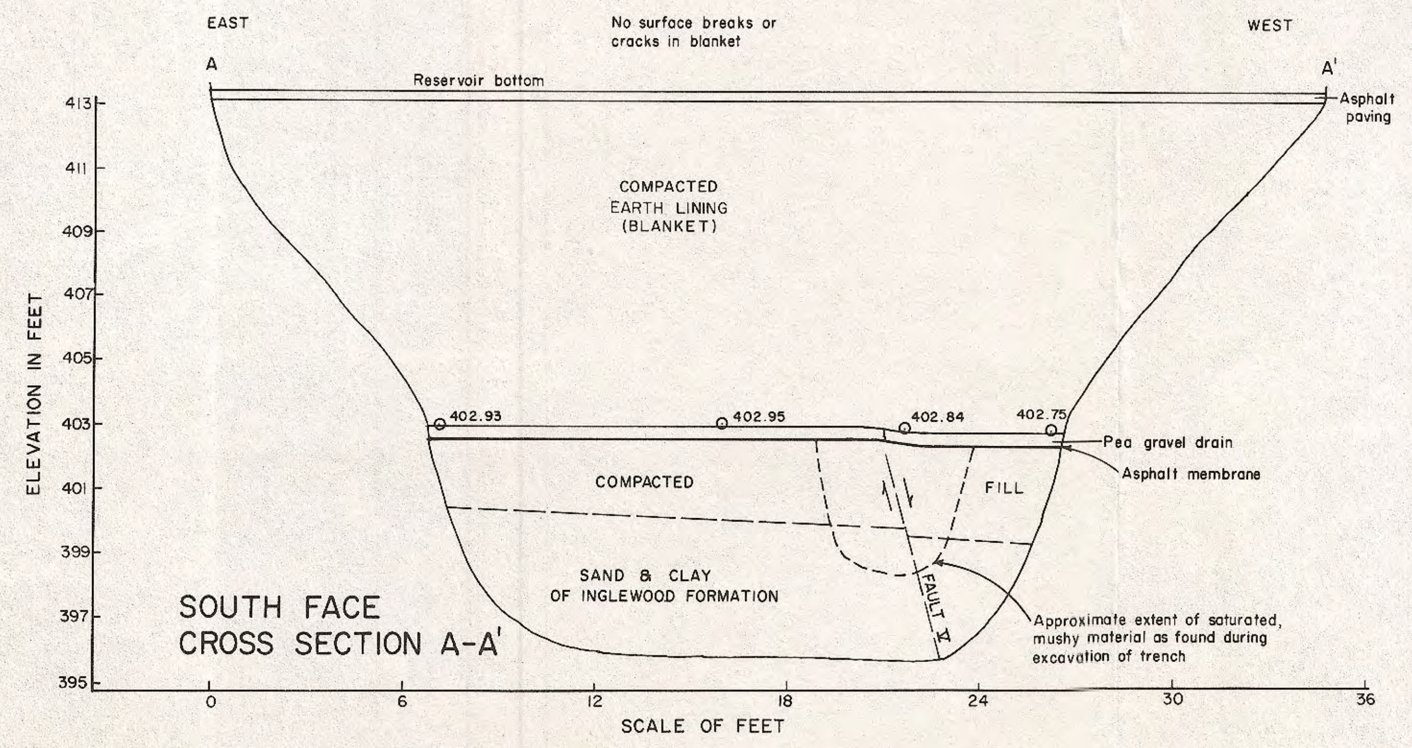
LOG OF EXCAVATION No. 9



PLAN



SECTION B-B'



SOUTH FACE
CROSS SECTION A-A'

NOTES

1. LIGHT-COLORED SECONDARY DEPOSITS ARE PRESENT ON CRACK FACES OF BOTH TILE DRAINS ON LARGE CENTRAL CRACK (LABELED 1/2" CRACK WHERE CRACK CROSSES CONCRETE ENCASEMENT). THIS SECONDARY DEPOSIT INDICATES THE CRACK HAS EXISTED FOR SOME TIME.
2. ON SECTION A-A' SATURATED, MUSHY MATERIAL IS LABELED. THIS OCCURRENCE INDICATES MOISTURE WAS GETTING THROUGH SEALING MEMBRANE INTO FOUNDATION.
3. TRENCH WAS EXCAVATED WITH A GRADALL.

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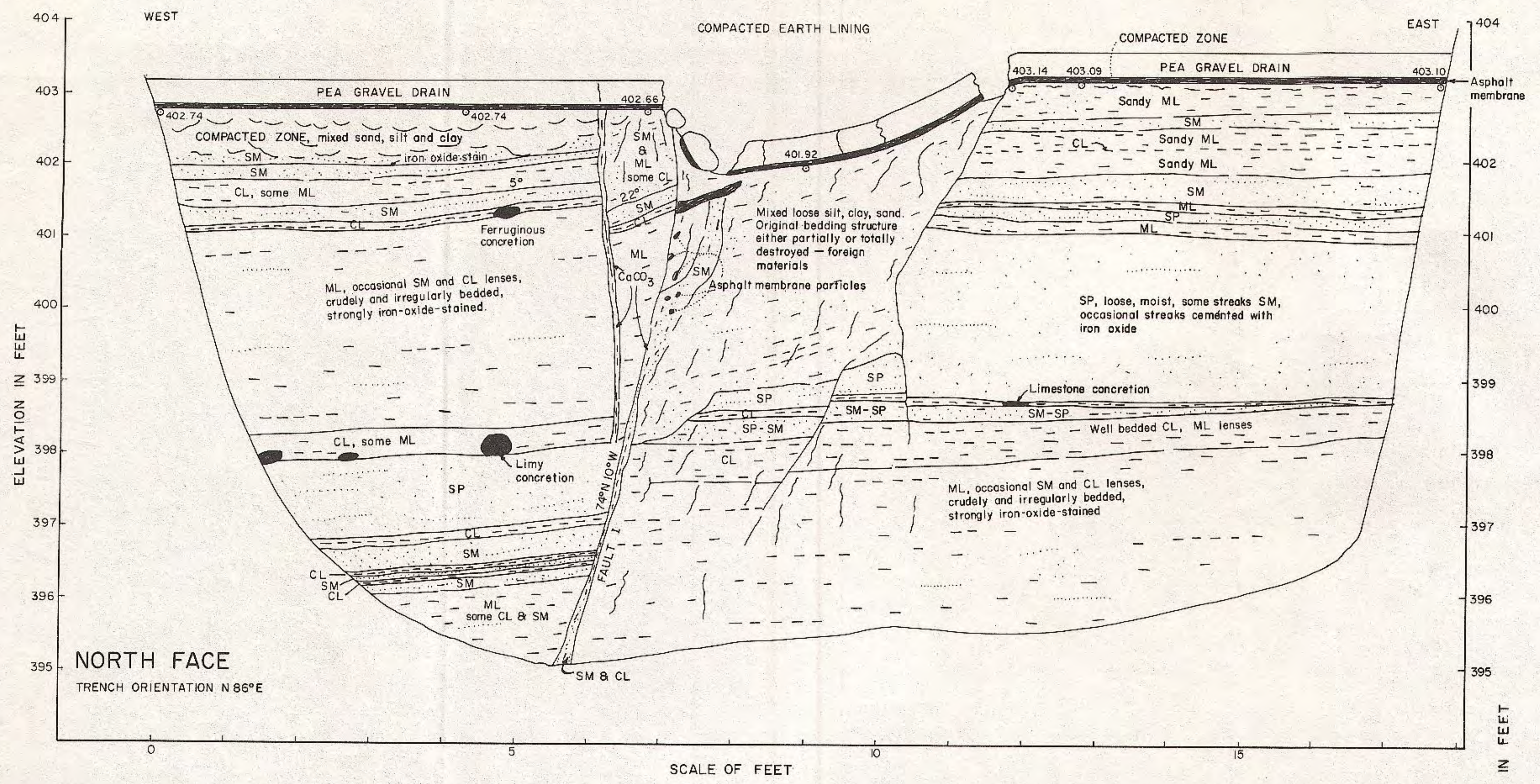
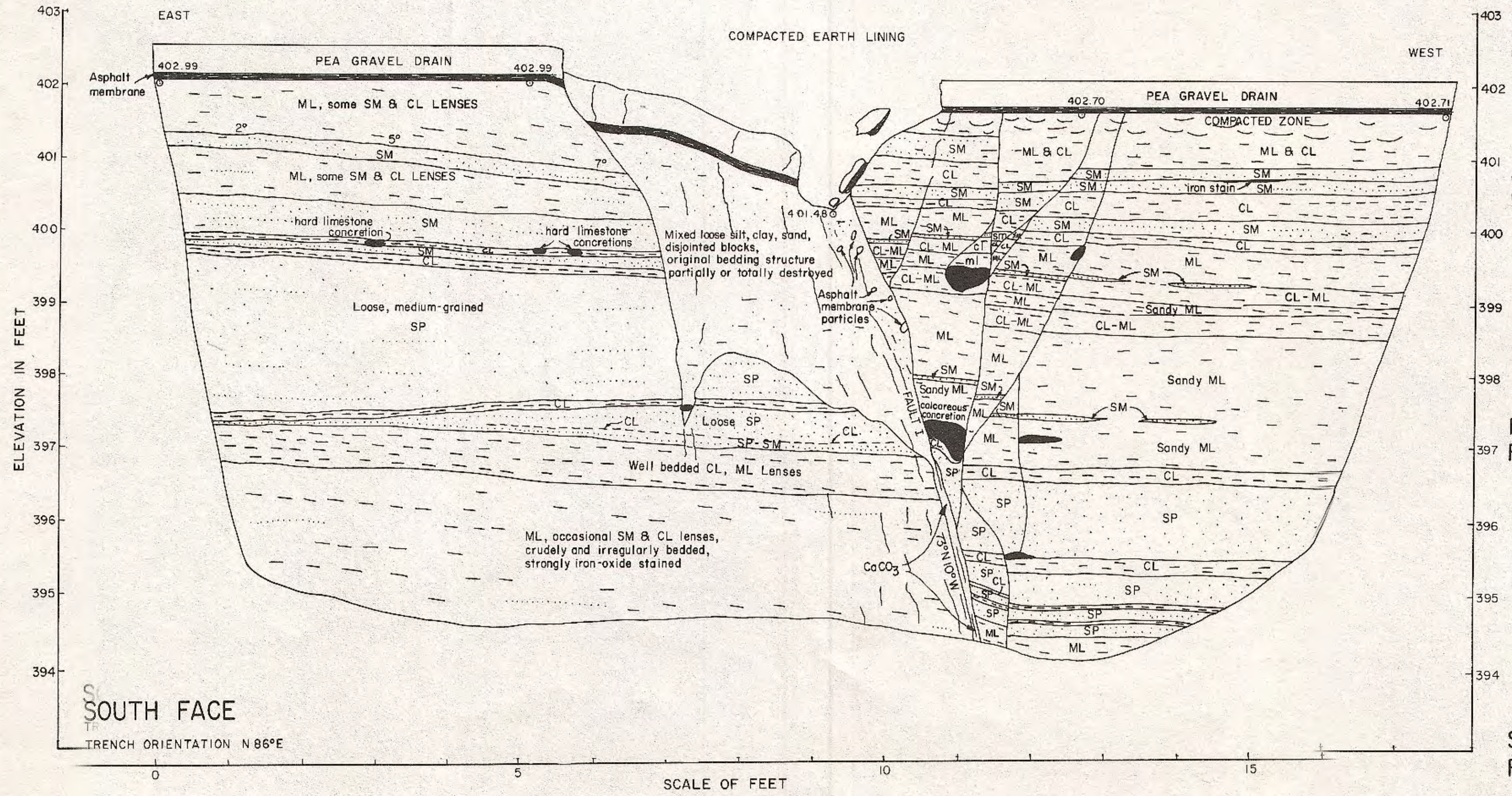
LOG OF EXCAVATION No. 11



NORTH FACE

TRENCH ORIENTATION N 86°E



SOUTH FACE

TRENCH ORIENTATION N 86°E

LEGEND

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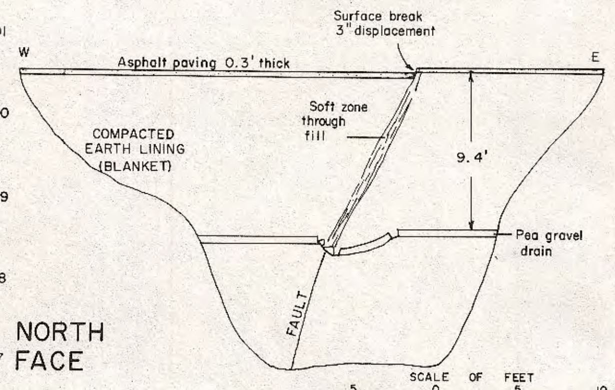
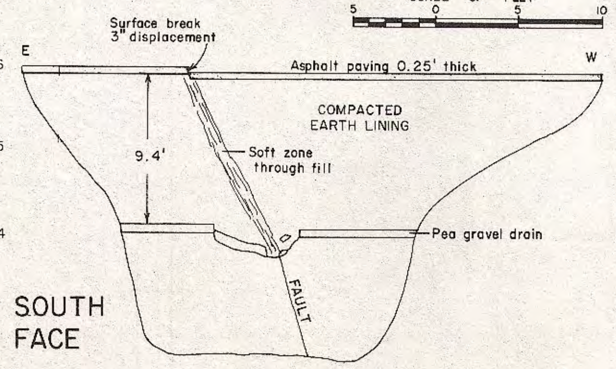


NORTH FACE



SOUTH FACE

NOTES:

1. BEDS ARE VERY NEARLY HORIZONTAL EXCEPT NEAR FAULT I. SEDIMENTS ARE INGLEWOOD FORMATION.

STATE OF CALIFORNIA
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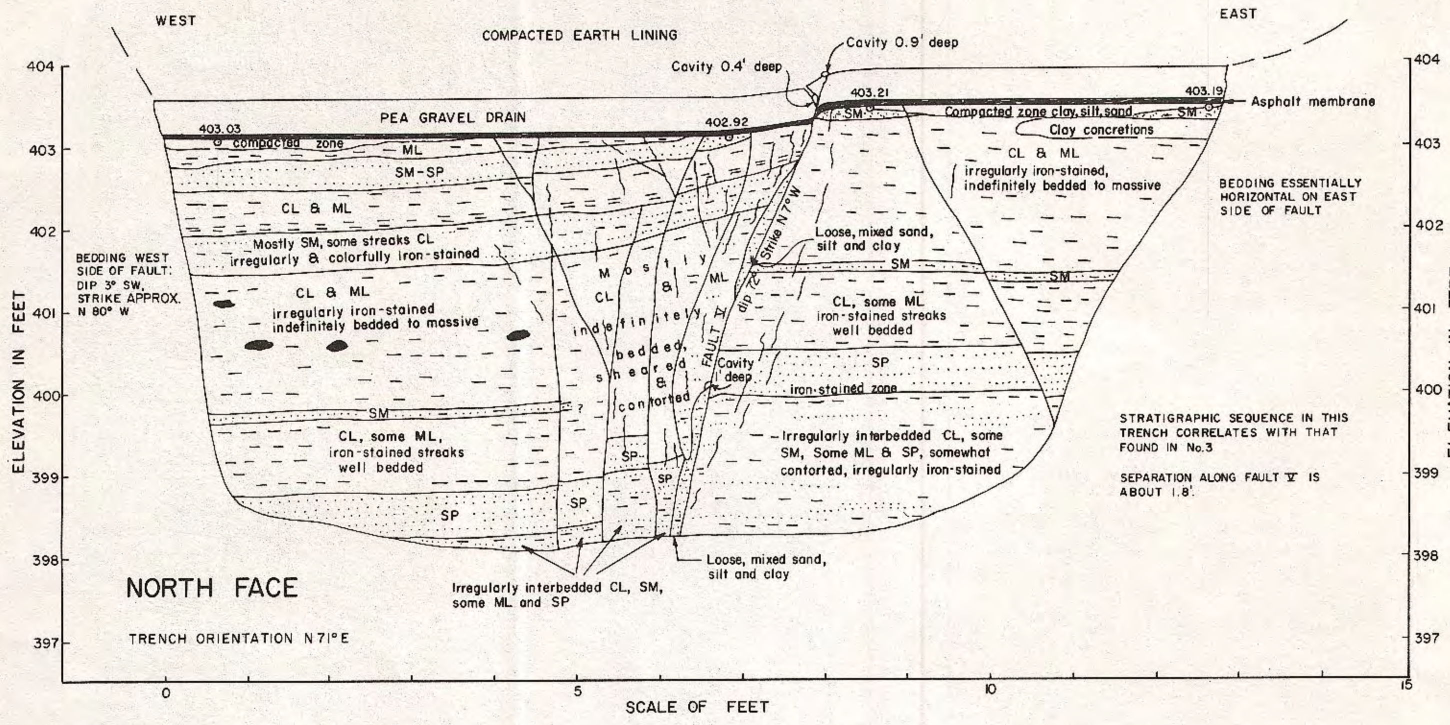
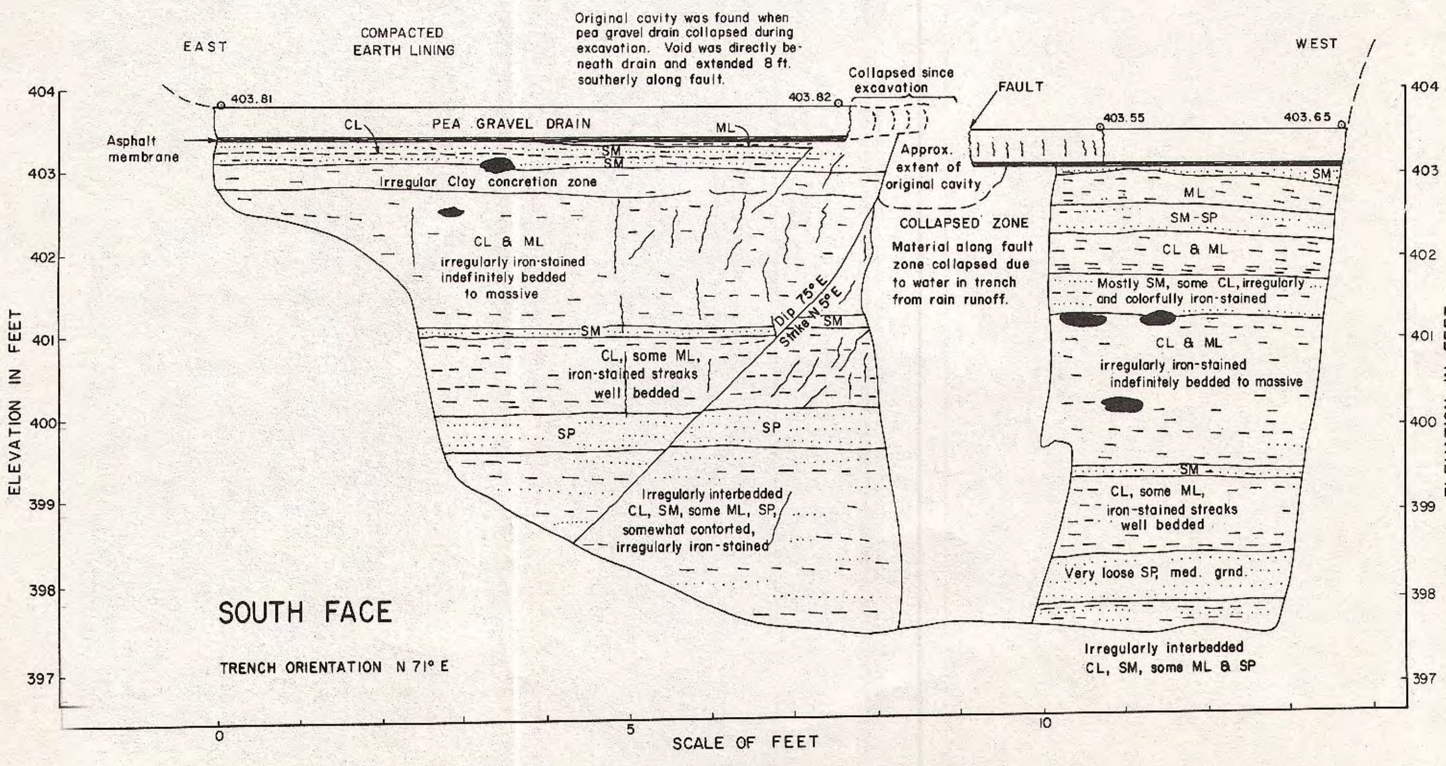
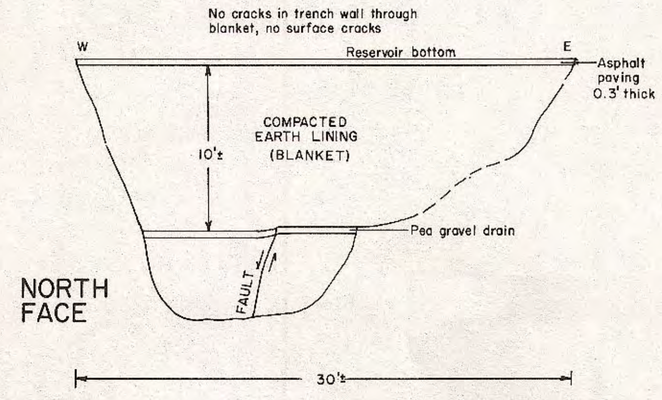
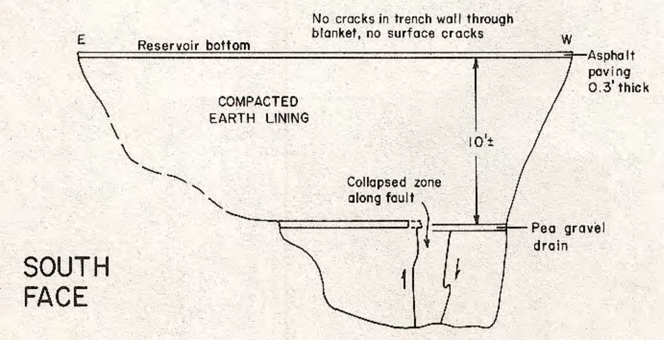
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 12A









LEGEND

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	SAND WITH NO APPARENT BEDDING	
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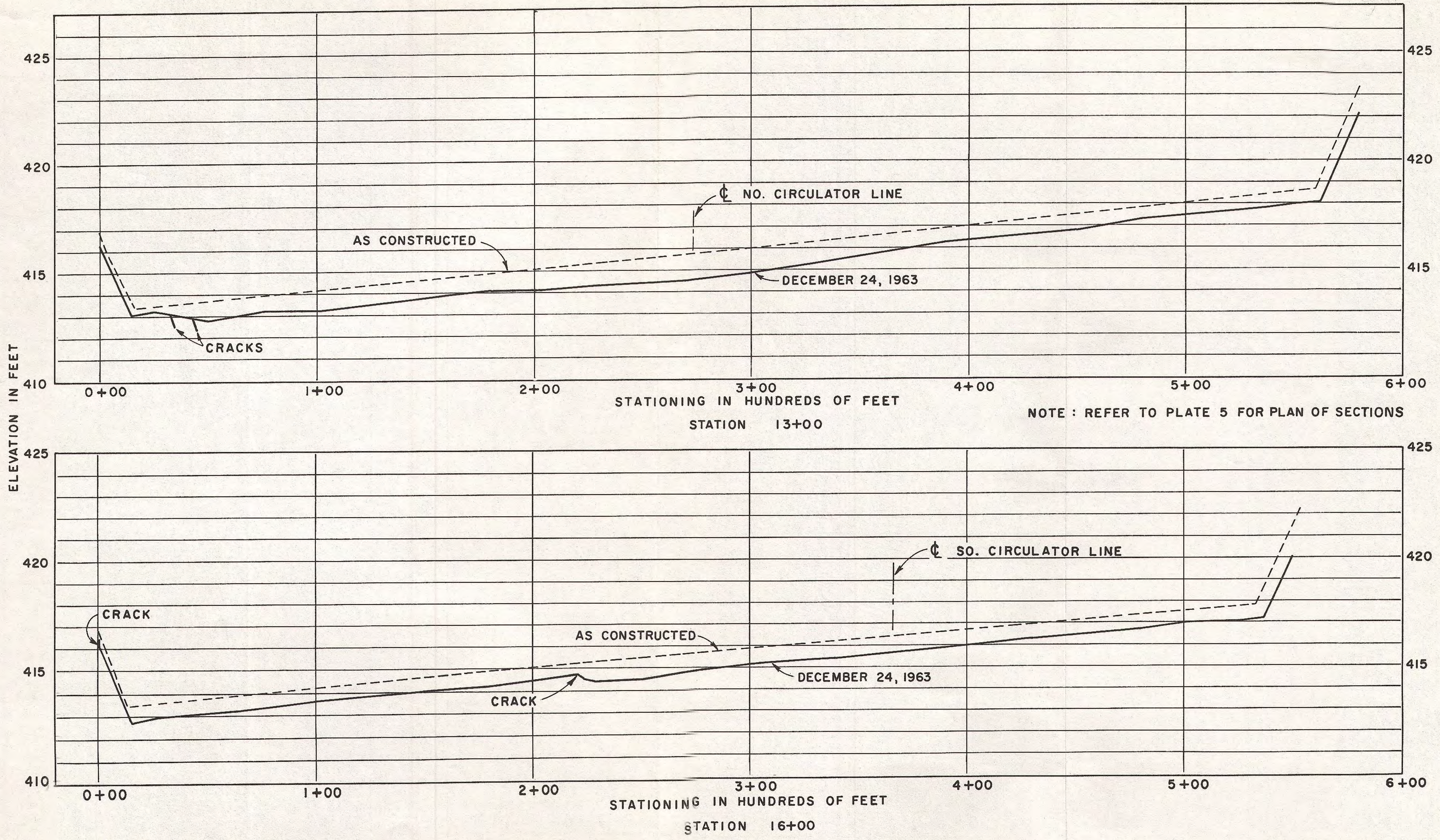
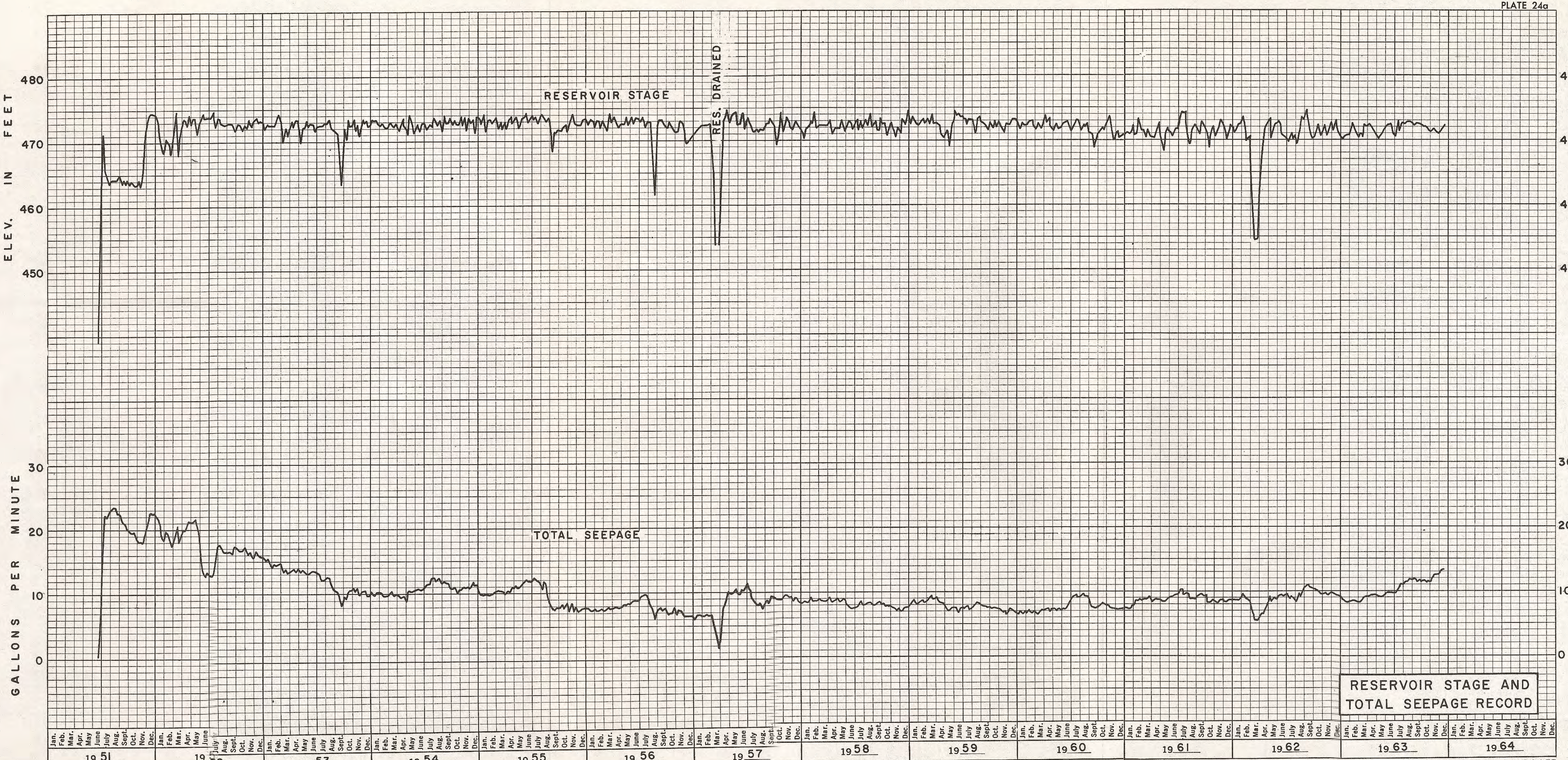
INVESTIGATION OF FAILURE
BALDWIN HILLS RESERVOIR

LOG OF EXCAVATION No. 13

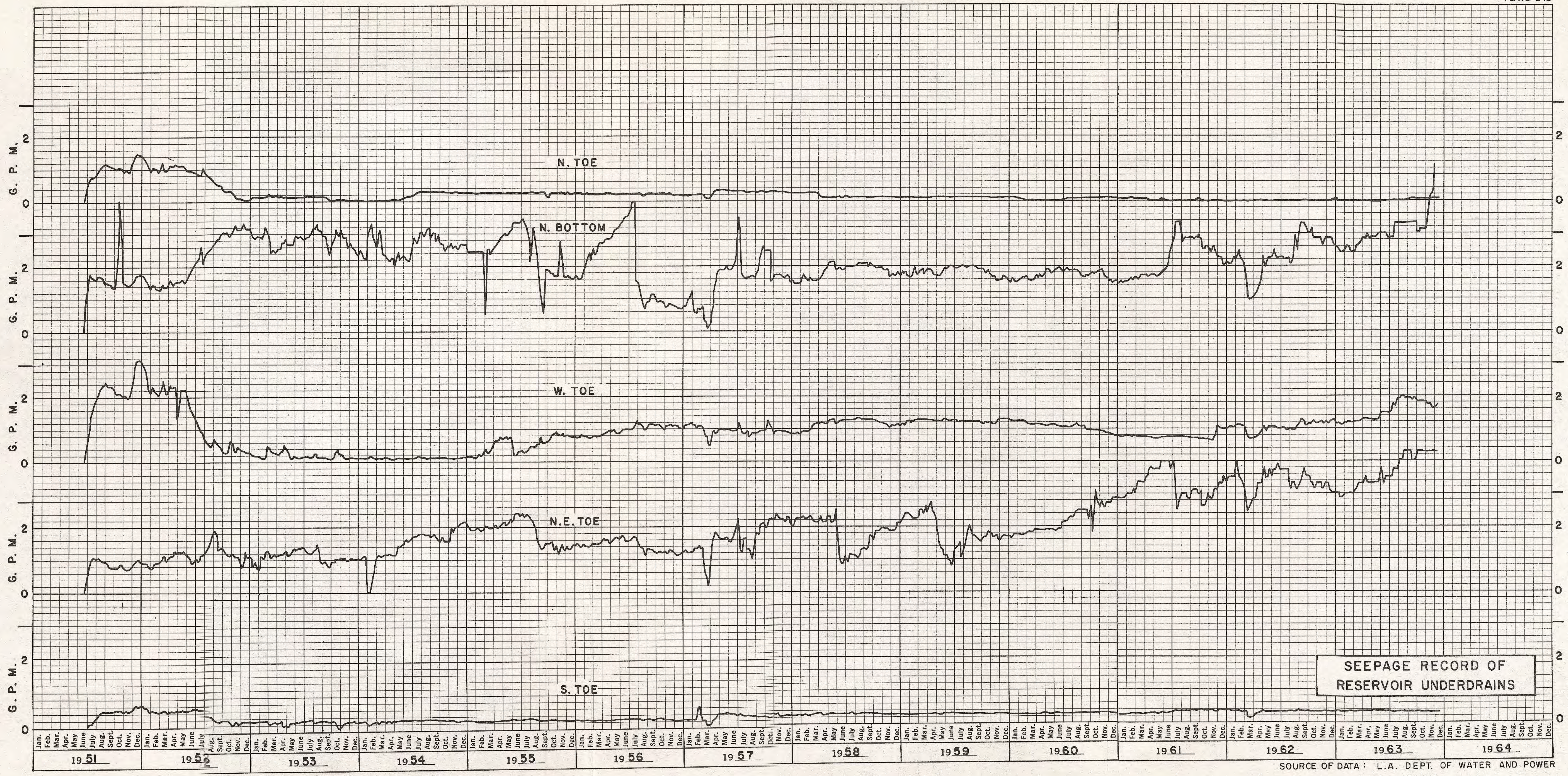


SETTLEMENT RECORD OF RESERVOIR FLOOR AT STATION 13+00 AND STATION 16+00



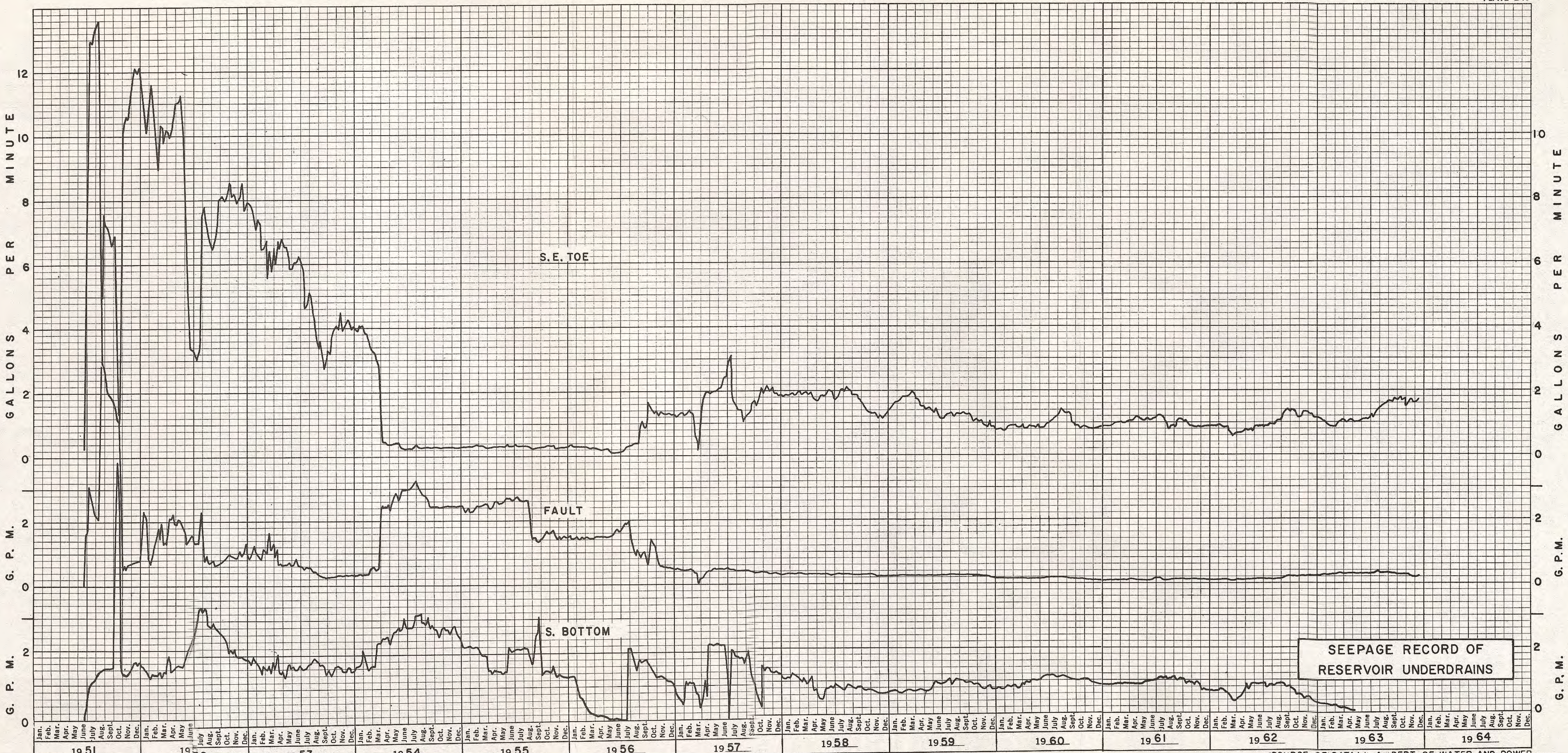
RESERVOIR STAGE AND TOTAL SEEPAGE RECORD

SOURCE OF DATA: L. A. DEPT. OF WATER & POWER

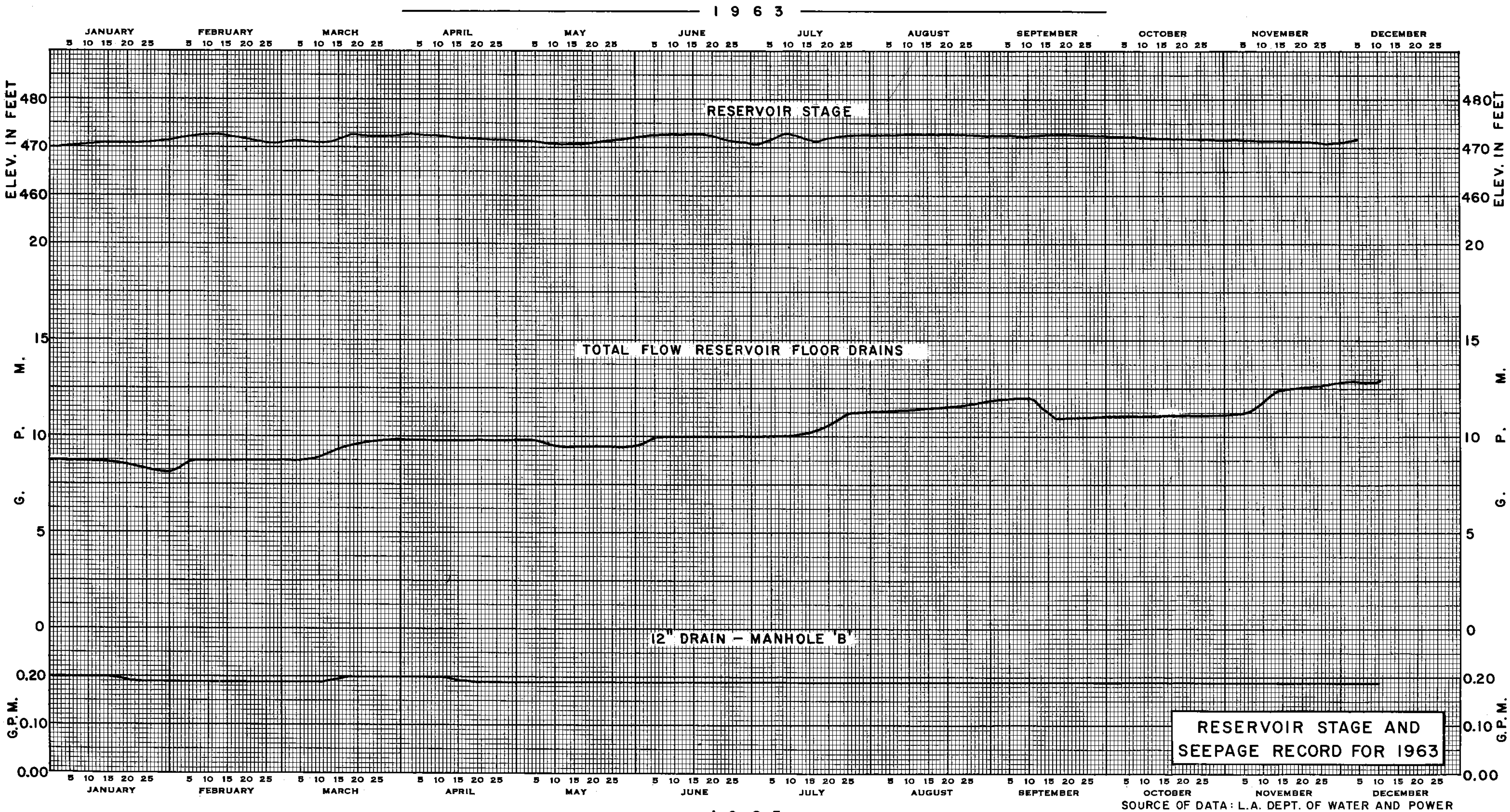


SEEPAGE RECORD OF RESERVOIR UNDERDRAINS

SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER

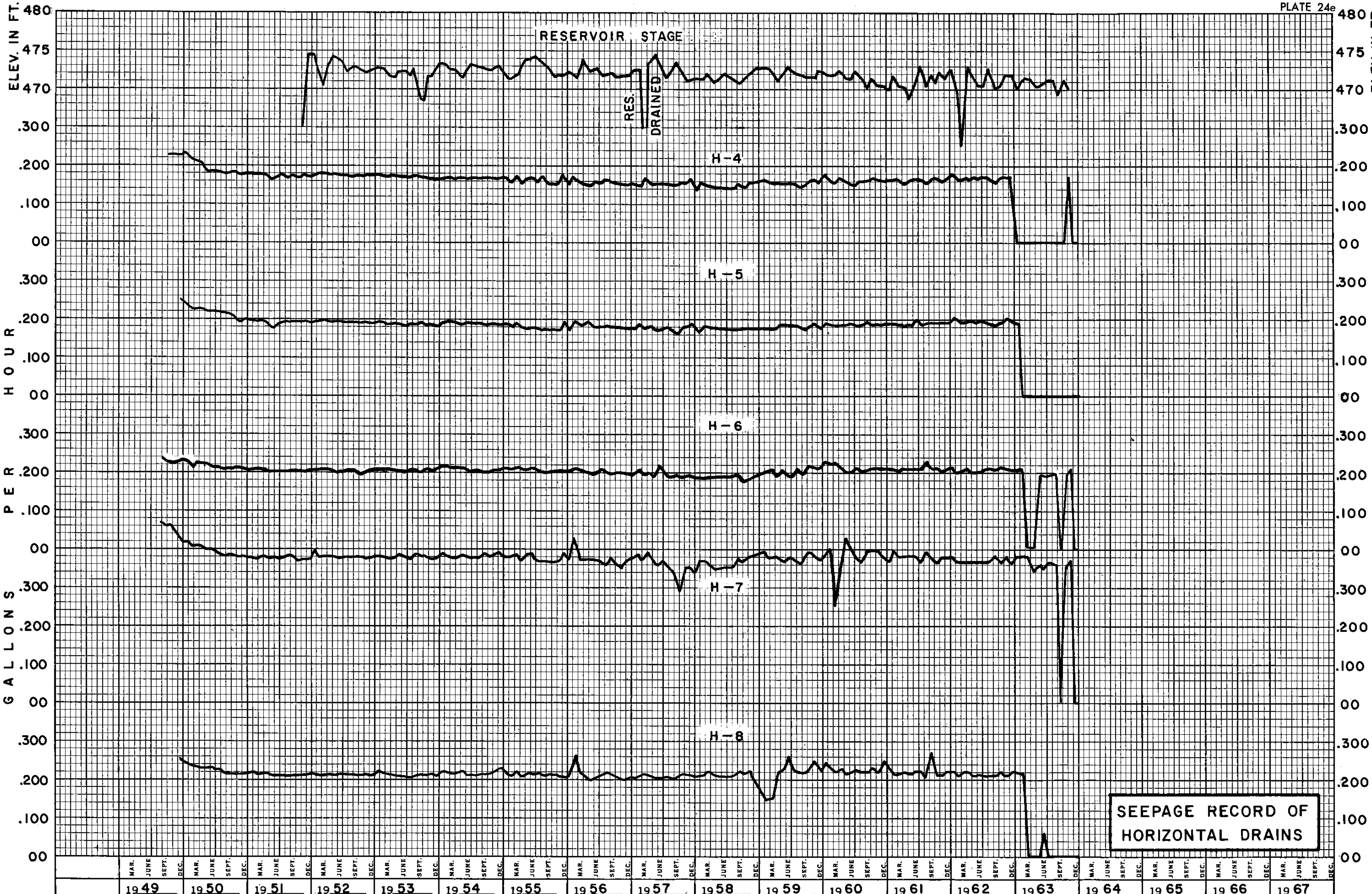


SOURCE OF DATA: L. A. DEPT. OF WATER AND POWER



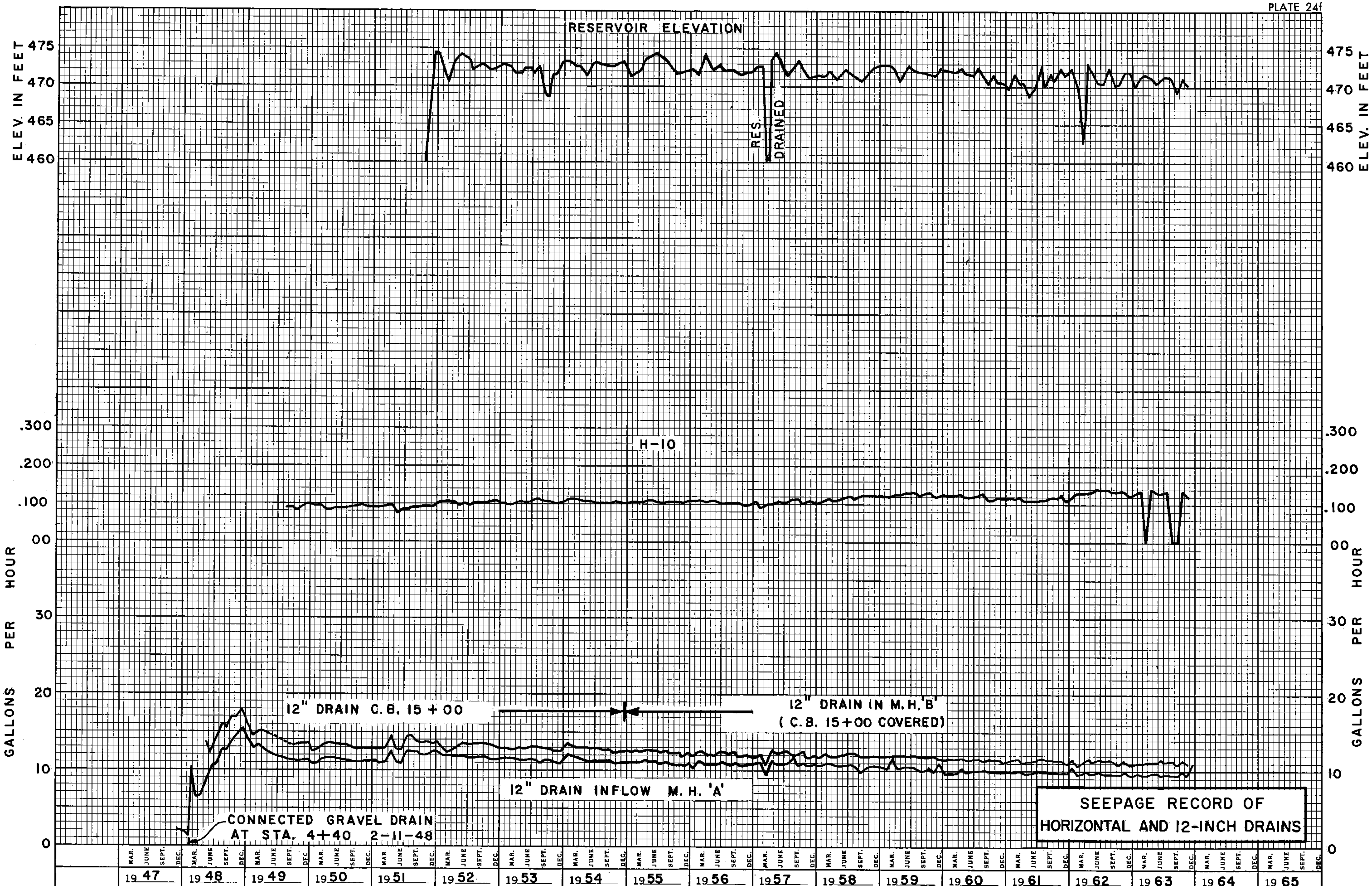
RESERVOIR STAGE AND SEEPAGE RECORD FOR 1963

SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER



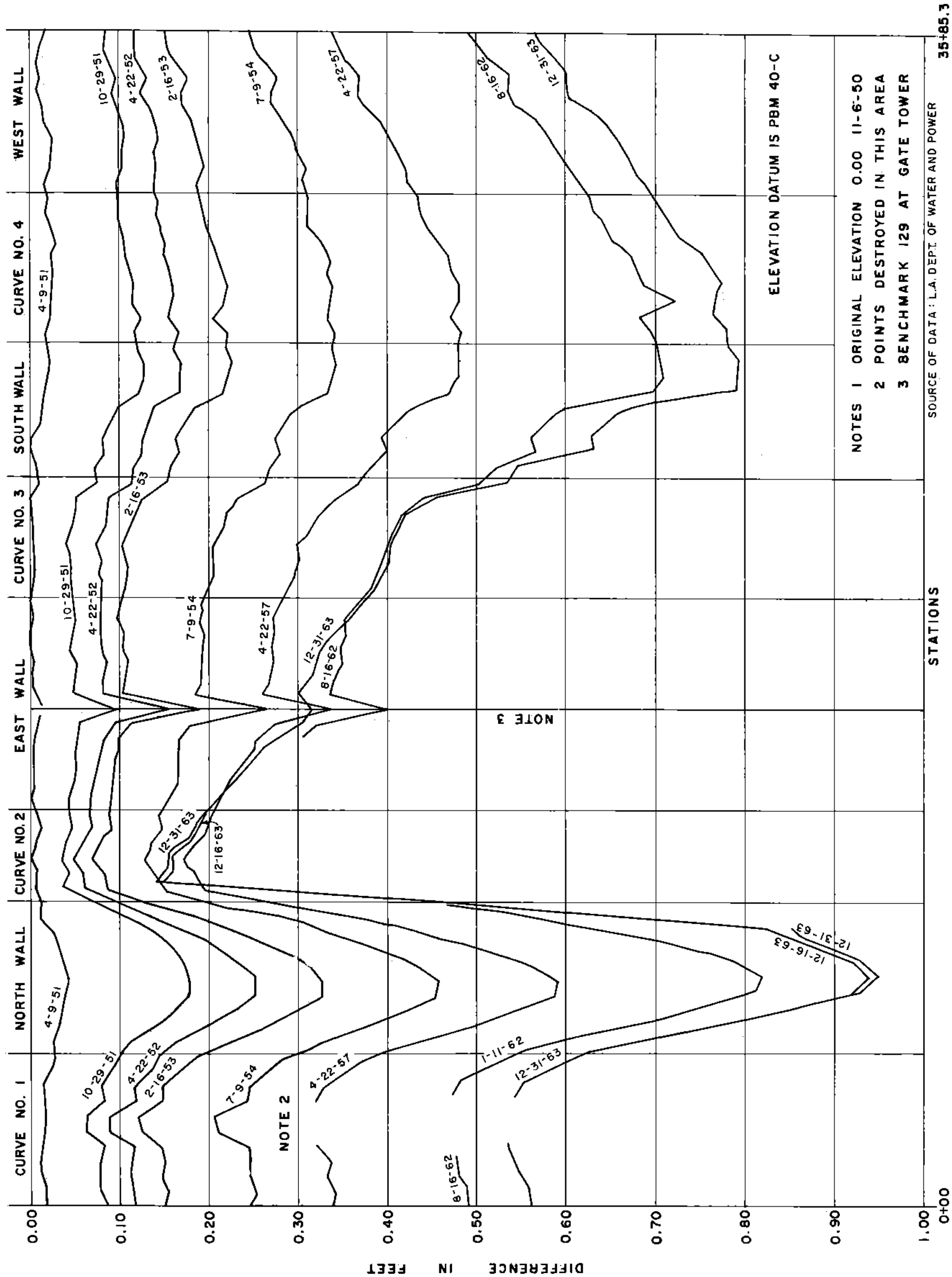
SEEPAGE RECORD OF HORIZONTAL DRAINS

SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER



SEEPAGE RECORD OF HORIZONTAL AND 12-INCH DRAINS

SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER

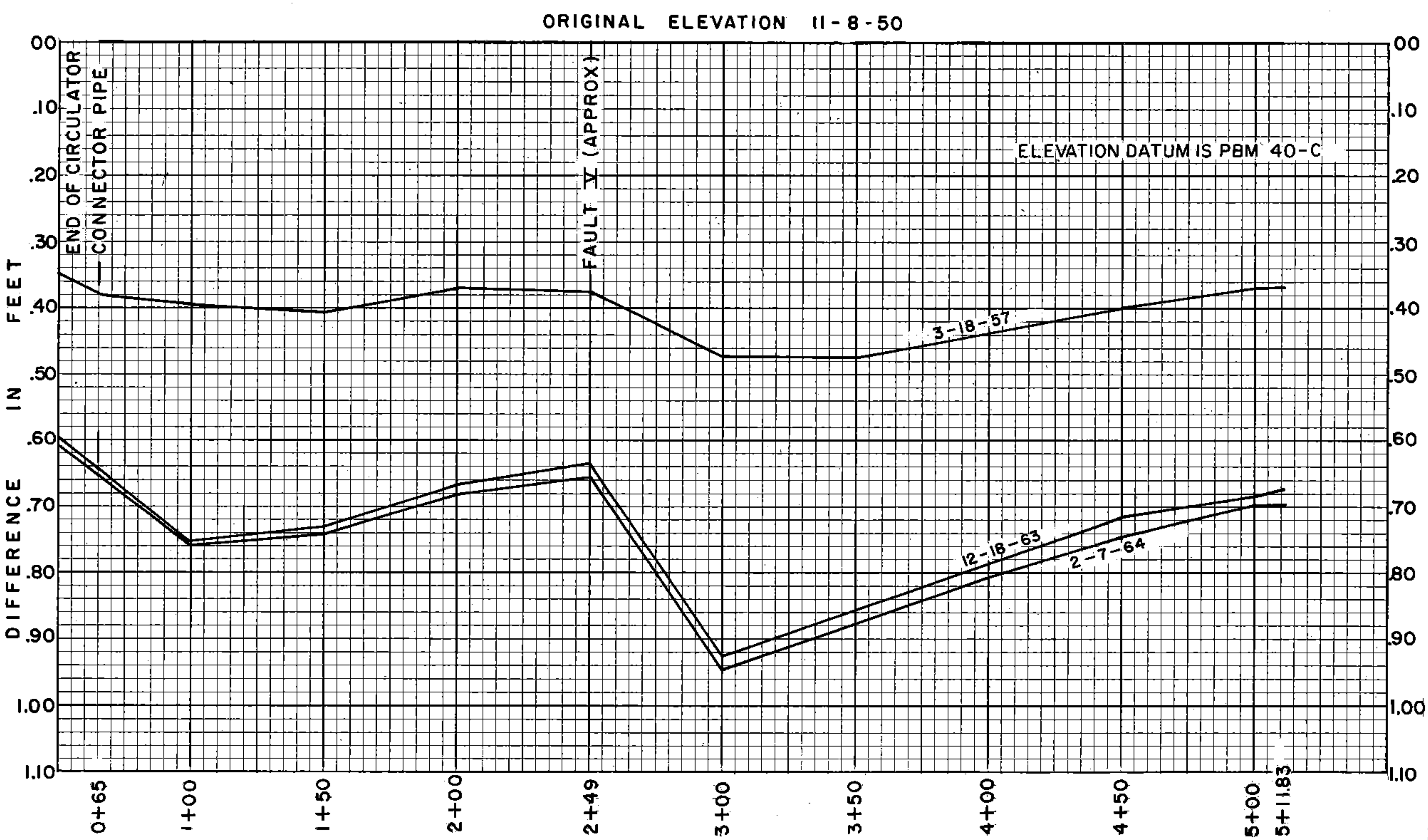


SETTLEMENT RECORD OF PERIMETER PARAPET WALL

SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER

SETTLEMENT RECORD OF NORTH CIRCULATOR LINE

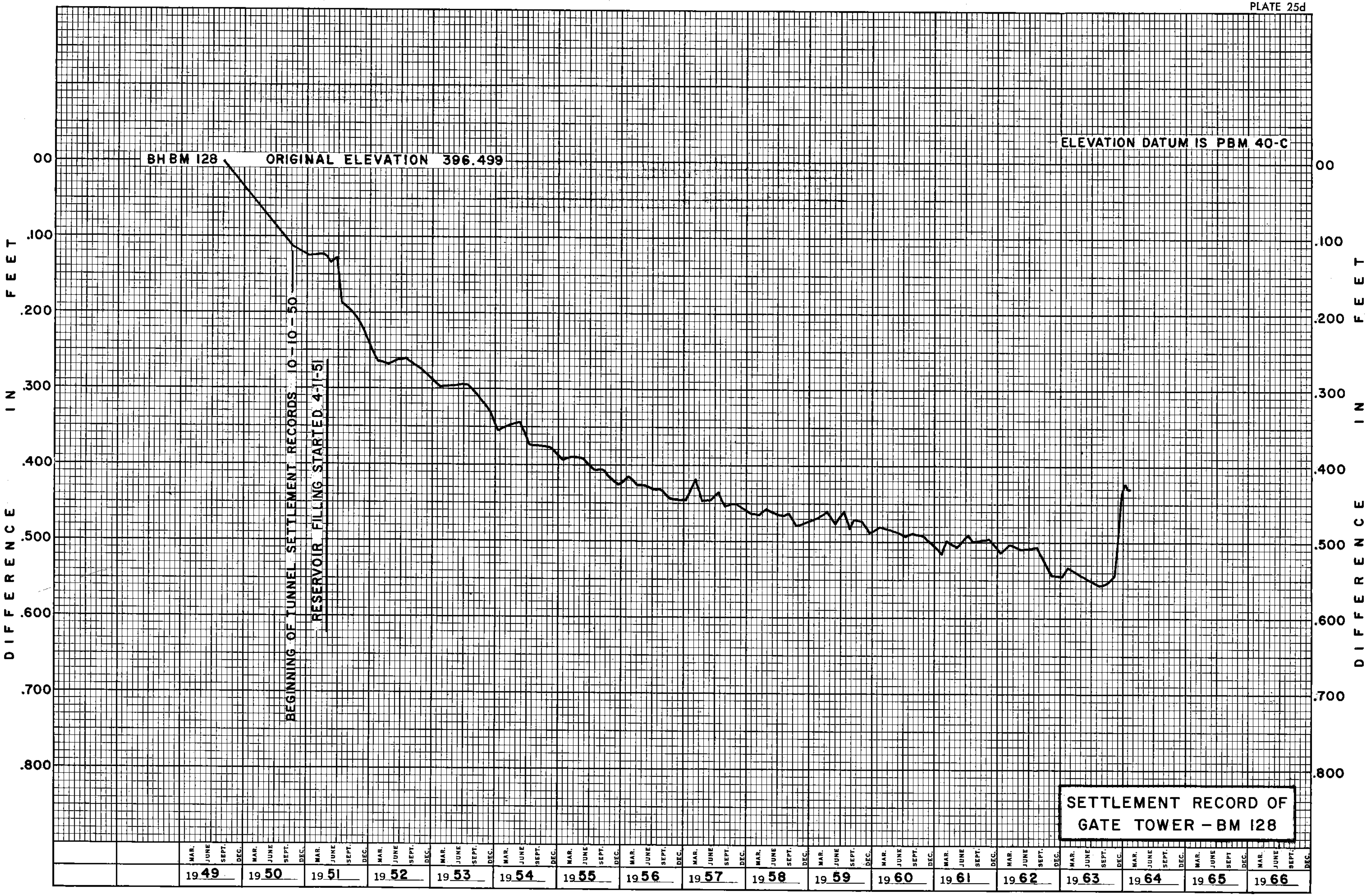
PLATE 25b



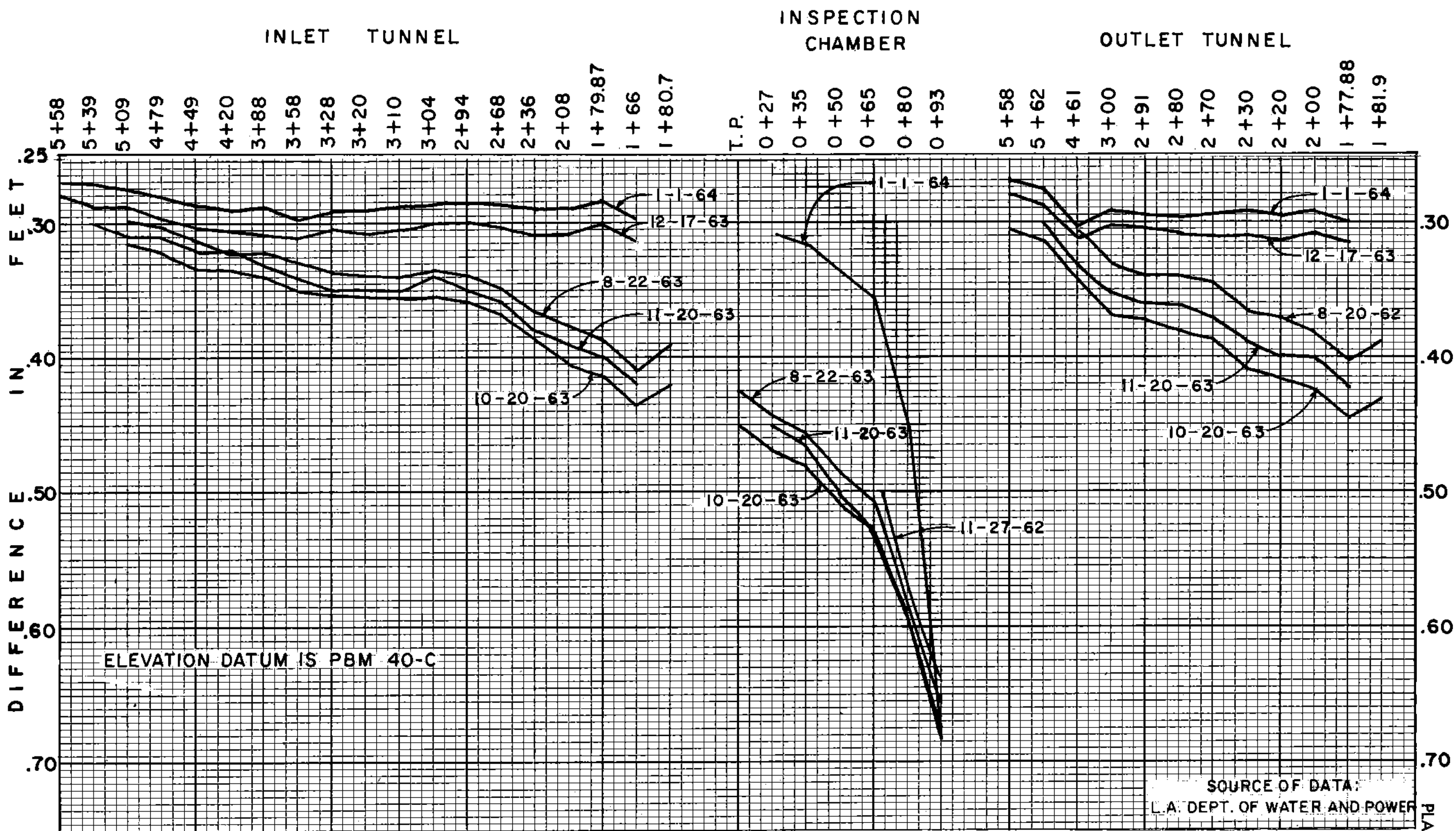
SOURCE OF DATA: L.A. DEPT. OF WATER AND POWER

SETTLEMENT RECORD OF SOUTH CIRCULATOR LINE

PLATE 25d

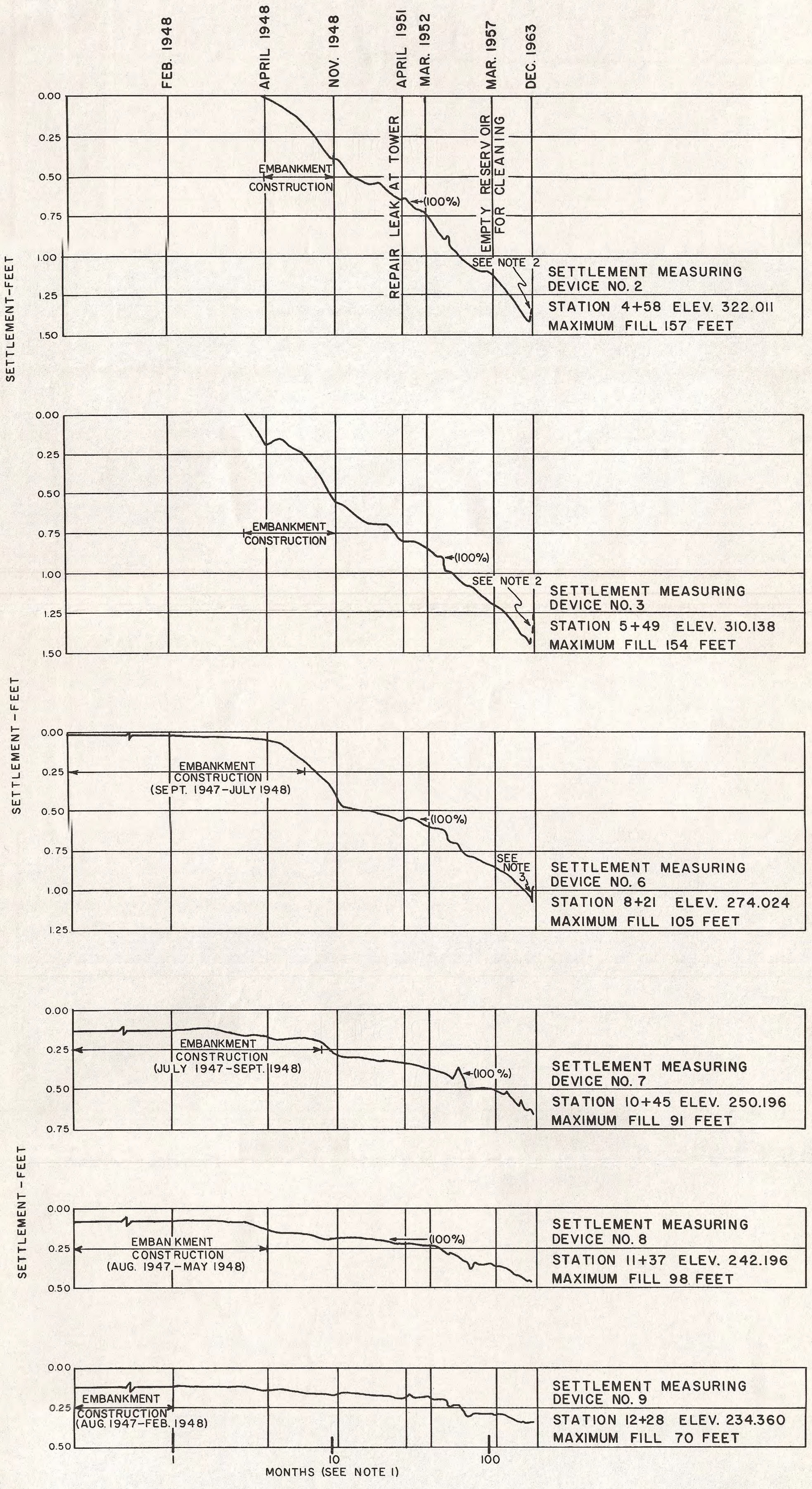


SOURCE OF DATA: L.A. DEPT. OF WATER & POWER



SETTLEMENT RECORD OF INLET TUNNEL, OUTLET TUNNEL, AND DRAINAGE INSPECTION CHAMBER

PLATE 25e



NOTES

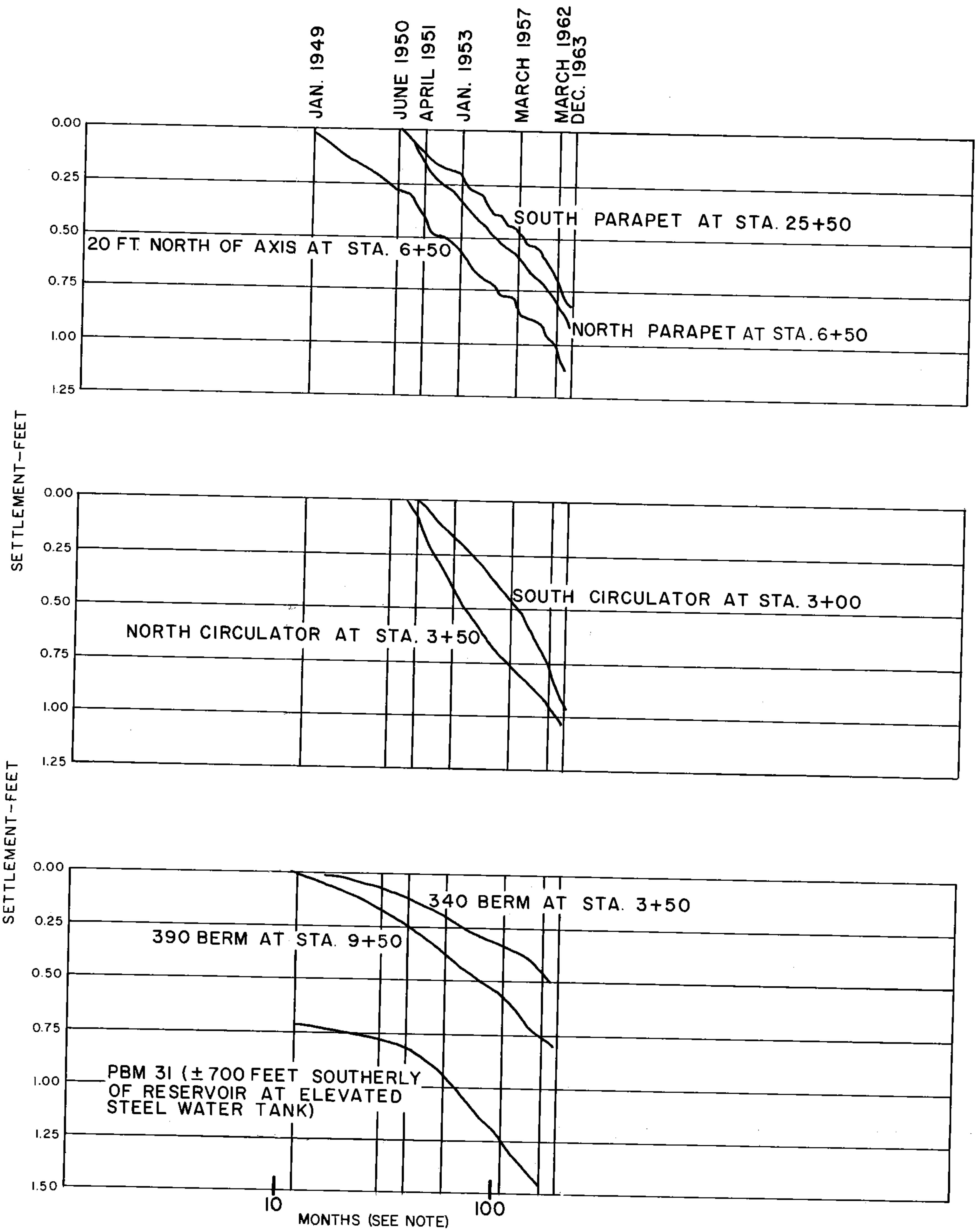
1. FOR CONVENIENCE OF PLOTTING TIME SCALE THE BASE MONTH HAS BEEN ARBITRARILY SELECTED AS JANUARY 1948.
2. POST-FAILURE READINGS MAY BE INACCURATE, MANOMETER LINES WERE SEMI-CLOGGED FROM INUNDATION AFTER FAILURE.
3. POST-FAILURE READING ASSUMED RELIABLE.
4. FOR LOCATION OF THESE POINTS SEE PLATE 13
5. ELEVATION DATUM IS PBM 40-C

LEGEND

←(100%) ESTIMATED TIME OF 100% FOUNDATION CONSOLIDATION UNDER EMBANKMENT LOAD.

RECORD OF FOUNDATION SETTLEMENT UNDER MAIN DAM

PLATE 26



NOTE
1. FOR CONVENIENCE OF PLOTTING TIME SCALE THE BASE MONTH HAS BEEN ARBITRARILY SELECTED AS JANUARY 1948.
2. FOR LOCATION OF THESE POINTS SEE PLATE 5
3. ELEVATION DATUM IS PBM 40-C

RECORD OF SURFACE SETTLEMENT ACROSS RESERVOIR