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Baldwin Hills Reservoir Failure

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Baldwin Hills is located in a suburb of the city of Los Angeles about midway between the city center and the L.A. International Airport.

The reservoir was designed by the staff of the Los Angeles Department of Water and Power. It was a small reservoir whose storage capacity was less than 900 acre-ft. and, on the average, the maximum depth of water was 65 ft. Site investigations were started in 1939 and continued with increasing intensity through 1947, which included the early part of the construction period. The reservoir location and a general site plan is shown in FIG. 1.

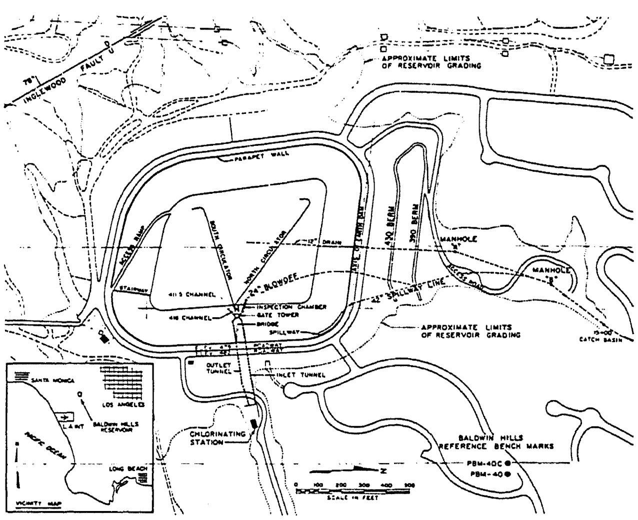


FIG. 1. Location and General Site Plan of Baldwin Hills Reservoir

The reservoir was fashioned out of a steep ravine by excavation at the abutments and filling in the eroded valleys. The main dam is at the north face of the reservoir and has a maximum height of about 200 feet. E-W cross sections through the axis of the main dam, and through the gate tower, are shown in FIGS. 2 and 3.

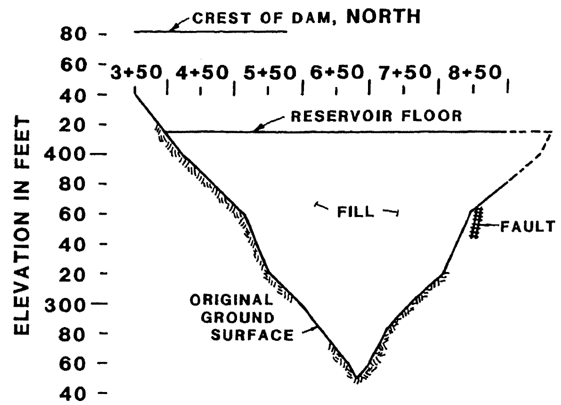


FIG. 2. E-W Cross-section Through Axis of Main Dam (tangent section only)

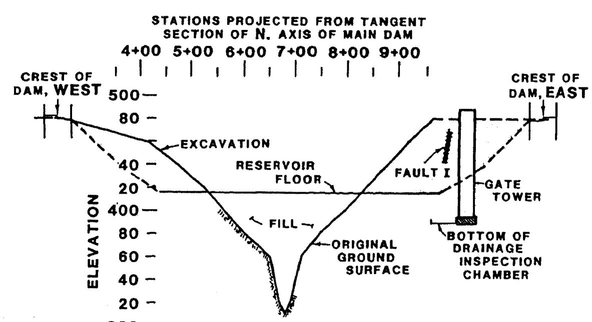


FIG. 3. E-W Cross-section Through the Gate Tower

SITE CONDITIONS

The relevant geologic stratum is the Inglewood formation, a marine deposit of early Pleistocene age that consists mainly of thinly bedded fine sand, silt, and clayey silt. Some of the strata are moderately well cemented but others are composed of loose, powdery sands and silts extremely susceptible to erosion.

Oil fields adjoin the reservoir on the south and west, and the existence of an associated subsidence bowl was known. The Inglewood fault, an active fault that is a branch of the San Andreas system, lies within 600 feet of the west rim of the reservoir. Tectonic and seismic activity associated with the Inglewood fault was well recognized. Auxiliary faults crossed the reservoir in a general N-S direction, as shown in FIG. 4. The gate tower was moved eastward from its original location to avoid positioning it directly over a fault. A photograph of fault I, exposed by excavation near the gate tower in April, 1948, is shown in FIG. 5. Note the fragile nature

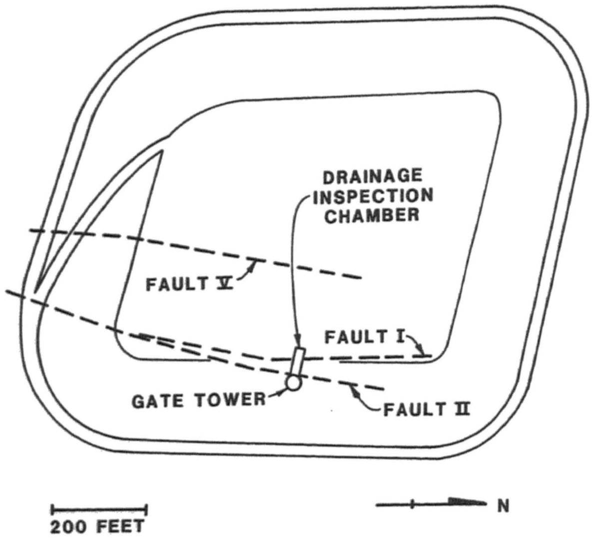


FIG. 4. Location of Auxiliary Faults

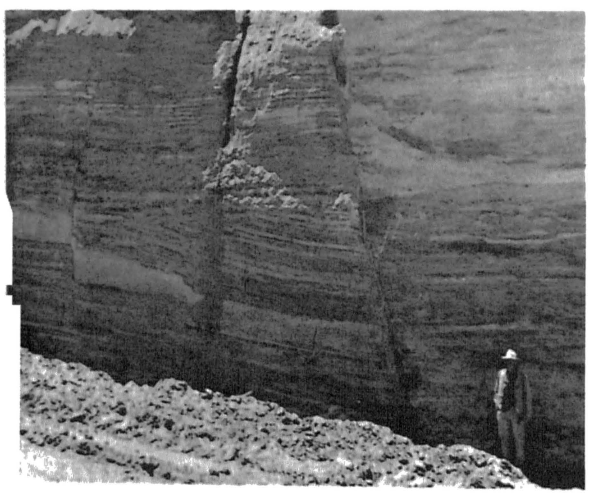


FIG. 5. Photograph of Auxiliary Fault I

of the soil along the planes of weakness. The consensus of opinion was that the auxiliary faults were not active, although the Department's geologist reported apparently fresh slickensides along the structural weakness planes (Wilson, 1949). Because the reservoir was in an active seismic zone, the embankments were designed to resist a horizontal acceleration of 0.2 g. To design, construct and operate a safe reservoir in an area subject to slow tectonic movements and seismic activity in nearby major faults, within a zone of regional subsidence due to pumping from adjacent oil fields, and at a site travers by auxiliary faults and underlain by low density, easil erodible soil was, indeed, an extremely challenging assignment.

DESIGN CONCEPT

The basic design concept adopted for the reservoi is illustrated in FIG. 6. The designers recognized th

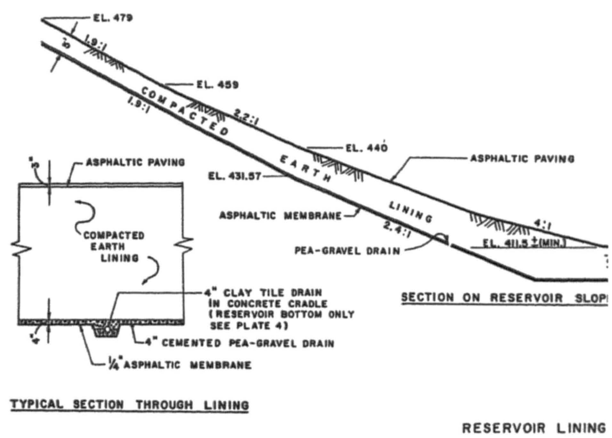


FIG. 6. Main Features of Reservoir Lining

the security of the reservoir was critically dependent preventing water from seeping into the foundation soil The main line of defense was a 10 ft. thick clay linir (tapering to 5 ft. at the top of the embankment slopes To maintain flexibility, the clay was compacted to 92 percent of the standard Proctor maximum density at a w content 5.5 percent wet of optimum. As a second line defense, a 1/4 in. asphalt membrane was sprayed on the subgrade soils in two coats. An open weave cotton fat was placed between the asphalt coatings at points of stress concentration. No effort was spared to constru a membrane as free of defects as possible and all oper tions were inspected with special thoroughness.

A 4 in. cemented pea gravel drain was constructe between the clay lining and the asphalt membrane to collect any seepage through the lining and convey it central observation and measuring station, called the drainage inspection chamber (FIG. 4).

A separate foundation drainage system was provid A 12-inch vitrified clay tile pipe with open joints, whose upper half was covered with cemented pea gravel was installed to drain the main dam. The drain passe through successive manholes where seepage could be ob- served. Markers were placed at regular intervals alo the drain to monitor settlements (FIG. 7). Each of t smaller embankments enclosing the reservoir had simil foundation drains (FIG. 8). Wherever local seepage z were encountered, vertical or horizontal drain holes drilled and filled with sand. These local drains wer connected to the foundation drain system with 4 in. c tile pipe (FIG. 8).

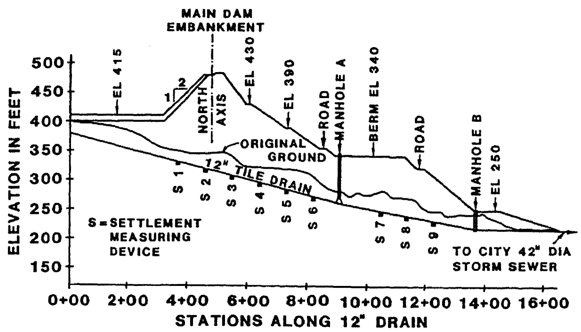


FIG. 7. Cross-section Through Main Dam and 12-Inch Foundation Drain

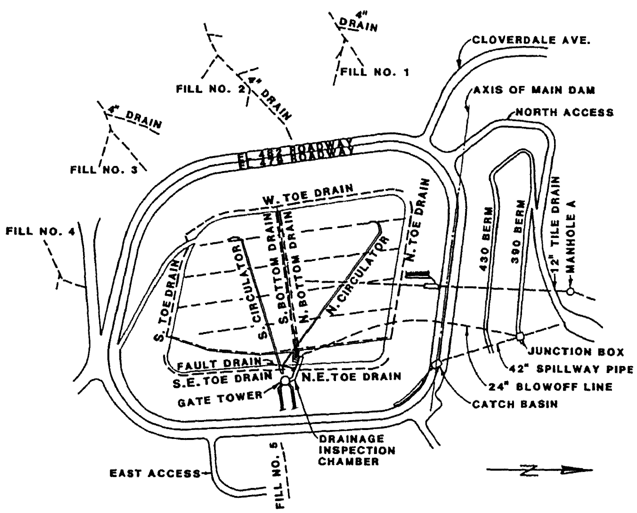


FIG. 8. Plan of Foundation and Reservoir Drainage Systems

Additional tile drains at the toes of the reservoir slopes, and a special fault drain were installed and fed directly into the drainage inspection chamber. Thus, leakage from separate segments of the reservoir underdrainage system, and from the toes of the reservoir slopes, could be measured independently.

The main design concept was to control leakage with the clay liner. Normal seepage through the lining would be collected and monitored. In the unlikely event that a crack would develop in the lining, it was expected that the reinforced, flexible asphaltic membrane would remain intact. Increased seepage through the lining would be observed in the inspection chamber and its location noted. Ample time would be available to drain the reservoir and effect repairs.

CONSTRUCTION

Construction was initiated by the Department's construction forces early in 1947 and was completed by contract in April 1951. Except for a small slide along a clay seam that occurred during excavation of the east abutment, which was readily stabilized, no unexpected conditions were encountered. Although the designers were confident of the dam's safety -- and a Consulting Board concurred in this judgment -- additional monitoring systems were installed to warn of any impending danger. These included surface monuments for settlement measurements at 50 ft. intervals along the crest, and bench marks to measure settlement of the gate tower and of the inlet and outlet tunnels. Later, as the need arose, strain gages to monitor separation of construction joints in the parapet wall and of cracks that developed in the drainage inspection chamber, as well as piezometers, inclinometers, and seismoscopes, were also installed.

DEVELOPMENT OF FAILURE

When put into service in 1951, the dam was considered a model of engineering excellence -- in design, construction methods, and monitoring systems. It was kept under close surveillance for 12 years. At 11:15 a.m. on Dec. 14, 1963 the caretaker heard a faint sound of running water emanating from the spillway discharge pipe. By 11:30 a.m. he had determined that the N.E. and S.E. toe drains, and the fault drain, were discharging muddy water in the inspection chamber like "fire hoses", and at 11:35 a.m. he sounded the alarm by telephone. At 12:20 p.m. measures to drain the reservoir were implemented; at 1:30 p.m., when sandbagging of a hole that developed in the north embankment slope proved futile, heroic evacuation measures were initiated. Five lives were lost, and the insurance carriers promptly paid off $12,000,000 dollars in damages. Law suits were then initiated to recover the losses.

INVESTIGATIONS OF THE FAILURE

A number of independent investigations were initiated immediately after the failure. However, the only readily available published report to be issued was prepared by a Board of Inquiry appointed by the State of California, and chaired by Robert Jansen (State of California, 1964). A Consulting Board chaired by J. Barry Cooke recorded their agreement with the findings in this report, which remains the primary source of information on the design, operation, surveillance, and post-failure investigations of the Baldwin Hills Reservoir. Additional valuable information and analyses were published in the Proceedings of the Purdue Conference by Leps (1972), and by Casagrande, Wilson and Schwantes (1972), following the settlement of lawsuits out of court in 1970. In briefest summary, the conclusions of the State Board of Inquiry were:

> I. "...that earth movement occurred at the Baldwin Hills Reservoir on December 14, 1963, following long-term development of stress and displacement in the foundation. The movement was apparently not seismic, but it did take place at faults which were planes of foundation weaknesses. Progressively increasing displacement finally resulted in rupture of the reservoir lining and consequent entry of water under pressure into the faulted foundation. Erosion of the foundation proceeded rapidly, causing uncontrolled leakage which led to total failure."
>
> II. "The earth movement which triggered the reservoir failure evidently was caused primarily by subsidence which had been observed in the vicinity for many years. Apparently the stage for destruction of the Baldwin Hills Reservoir was being set even before conception of the facility."

There appears to be general agreement with conclusion I. Conclusion II was supported by Hudson and Scott (1965) and Castle and Yerkes (1976), and strongly argued by Leps (1972). Hamilton and Meehan (1971) related the acceleration of crack openings in the drainage inspection chamber, which became evident early in 1958, to fluid injections initiated to increase the yield from the Inglewood oil field. Subsidence and slow tectonic movements certainly contributed to the observed displacements at the reservoir; however, in my opinion, it is not possible to establish, quantitatively, the relative contributions of regional movements, differential compression of the foundation strata due to reservoir loading, and slow seepage of water from the reservoir into the underlying erodible soils. On the other hand, it is possible to establish that events damaging to the reservoir's security occurred so soon after it was put into service that they could not possibly be attributed to slow regional movements. For example, during first filling of the reservoir in April of 1951, leakage was observed to increase dramatically in the foundation drains adjacent to the faults (FIG. 9). The reservoir was quickly

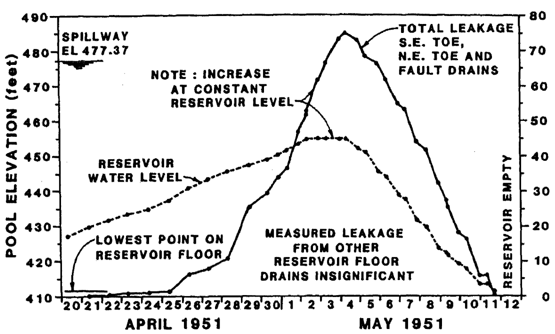


FIG. 9. Measured Leakage vs. Reservoir Level During First Filling of the Reservoir (after Casagrande, et al., 1972)

emptied. The asphaltic pavement protecting the clay lining was observed to have buckled along the toe of the inside slope along the east side of the reservoir. Also, a 3/4-in. differential settlement had occurred between the Elev. 418 channel inlet structure and the gate tower. Attempts to seal the leaks were made by grouting in the vicinity of the gate tower, and by replacing the roofing-paper gaskets in the joints between the inlet structure and the gate tower with rubber gaskets.

During June 1951, the reservoir was refilled. Again, the southeast toe and fault drains immediately responded with large flows. Subsequent attenuation of the flow rate was interpreted to mean that the leaks were "self-sealing" (FIG. 10). Instead, it appears that the asphalt

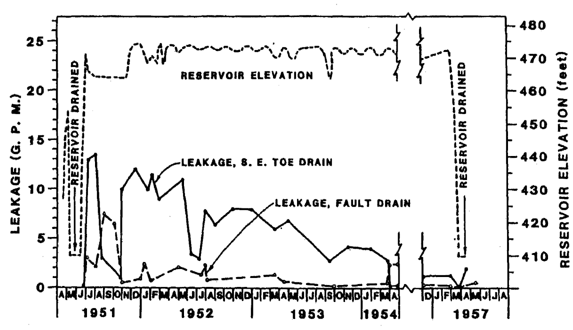


FIG. 10. Seepage Into Reservoir Underdrains

membrane was ruptured and seepage entered into the fou ation to begin a long-term process of progressive move ments -- aided and abetted by regional subsidence. Th view is supported by the settlement record of the gate tower, crack initiation and growth in the inspection chamber, and by the early pattern of subsidence that developed across the reservoir floor (FIG. 11). In ad

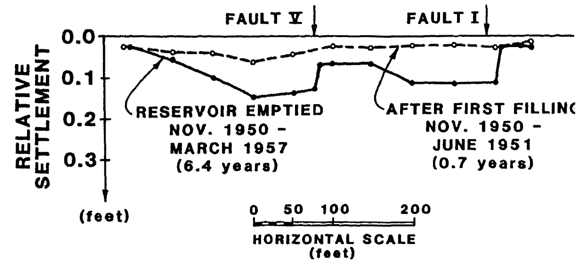


FIG. 11. Relative Settlement Across Reservoir Floor in E-W Direction (total movement minus regional subsidence, after Casagrande, et al., 1972)

tion, post failure investigations disclosed cavities i the locale of fault I nearly 50 feet below the reservo floor -- evidence of long-term seepage erosion -- whic is supported by calcification detected on the surfaces some of the cavities. It may be that some cavities pr dated construction of the reservoir. In any case, col lapse of cavities due to progressive seepage erosion a subsidence would occur suddenly; the effect on the already weakened clay liner would be to rupture it suffi ciently so that water could enter the foundation under pressure. I believe that this is the most likely mech anism that triggered the rapid demise of the reservoir December 14, 1963.

In the early years following the failure, the tri gering mechanisms were of paramount interest because t main concerns were with the assignment of liability. However, the thrust of this paper is to examine the le sons to be learned from the ensuing disaster. Thus fa only Casagrande, et al. (1972) have published views of the lessons learned. These are quoted as follows:

Lesson I

"The failure of the reservoir was caused by foundation strata highly sensitive to erosion and crossed by faults. The safety of the reservoir depended on preventing water from the reservoir entering these strata and the faults;"

"It is debatable whether a safe reservoir could have been designed for these conditions. Probably only a steel lining could have given reasonable assurance of safety. Other measures would have extended the life of the reservoir without ensuring the degree of safety that must be demanded of such a reservoir."

Lesson II

"The observational records show that the magnitude of these movements [subsidence, accompanied by tensile strains] was so small, including the differential settlements across Faults I and V, that it would not harm most of the earth dams and reservoir with which the authors are familiar."

"The authors seriously question whether any important dam and reservoir should be permitted to be constructed if it cannot withstand with perfect safety movements of such magnitude [i.e., those due to regional subsidence]. This may well be the principal lesson to be learned from this case record."

In the past two decades, and especially since Terzaghi's "observational method" was so eloquently articulated by Peck (1969), monitoring the performance of structures has become part and parcel of geotechnical engineering design. The approach is by no means new, as it was practiced some 800 years ago by the builders of the Tower of Pisa more boldly than most engineers would be willing to espouse today (Leonards, 1979). Among the essential features for success of the observational method, clearly delineated by Peck, is the necessity that the observations provide sufficient warning in time to prevent failure from occurring. This implies prior determination of a course of action whenever the measurements reach pre-determined critical values. The designers of Baldwin Hills were aware of all the hazards to safety posed by the site: they knew that the soils were highly erodible, and that it was crucial to prevent seepage from the reservoir from entering the foundation soils; they were aware of the ongoing ground movements, although some would argue they may not have appreciated fully the danger that was posed; and they not only knew of the faults crossing the reservoir but had the opportunity to observe the fragile nature of fault I, which was exposed during the early stages of construction (Fig. 5). They provided two lines of defense: (1) a 10 ft. thick clay lining specially constructed to possess considerable flexibility, and (2) an underlying cloth-reinforced asphalt membrane to protect the erodible foundation in the event the lining was ruptured. A comprehensive monitoring system was established to warn of impending danger, including settlements of the foundation drain under the main dam, the crest of the dam, the gate tower, and the surrounding area. Later on, piezometers, inclinometers and seismoscopes were added, and provisions were made to measure separation of construction joints on the parapet wall and of cracks that developed in the drainage inspection chamber. However, prime reliance was placed on a compartmentalized underdrain system to measure, independently, seepage from separate segments of the reservoir and its foundation. A rigorous surveillance regimen was established and faithfully executed, as follows:

Daily: - Seepage from underdrain networks

Monthly: - Surveys for settlement at the reservoir and of the surrounding area
- Inspection by a squad of maintenance personnel, and review of the results of all measurements, including those from the strain gages, tiltmeters, and seismoscopes. (The last regular inspection was on Nov. 26, 1963).

Annual: - Safety inspection by the State Dam Safety Office. (The last inspection was on April 3, 1963.)

All these efforts notwithstanding, the first indication of danger was at 11:15 a.m. on the day of failure, when the caretaker on routine rounds heard a faint sound of running water emanating from the spillway discharge pipe. Investigation revealed that the sound was the result of discharge from the 24 in. blowoff pipe where it joined the spillway discharge pipe. Upon verifying the high rate of muddy discharge from the underdrains directly in the inspection chamber, the caretaker sounded the alarm by telephone at 11:35 a.m. -- too late to stem the flow or lower the reservoir sufficiently to prevent breaching of the dam a mere four hours later.

REVIEW OF SURVEILLANCE MEASUREMENTS

It is pertinent to review the results of the surveillance program -- a 13-year record of measurements carefully taken, faithfully plotted, and regularly examined. In hindsight, and in the light of current knowledge, should the alarm have been raised and the reservoir drained before December 1963? Space limitations permit review only of a few key results. Settlements along the 12-inch foundation underdrain are shown in FIG. 12.

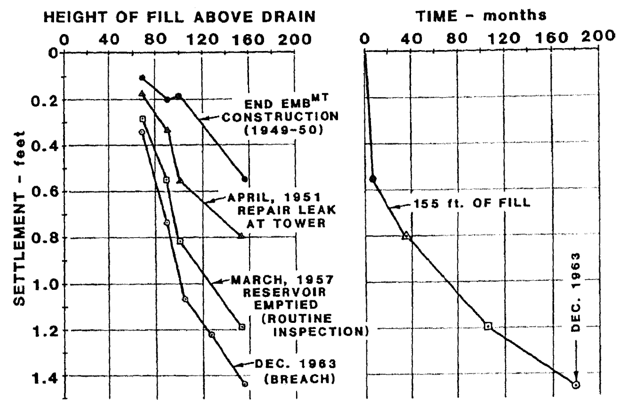


FIG. 12. Settlement Along 12-inch Foundation Underdrain

Settlement vs. fill height at any given time are reasonably regular. The settlement vs. time under 155 ft. of fill shows that the time dependent settlements are nearly double the immediate settlement. One might ask whether this should be expected if no water was seeping into the foundation, but this is easy to do in hindsight. At the time, the regular settlement pattern, and their monotonic attentuation with time, was apparently taken to be normal.

I was unable to obtain tabulations of the monthly crest settlement measurements. However, the crest settlement patterns published in the State of California Report (1964) are shown in FIG. 13. The sharp curvatures displayed are due to the distorted scales that were adopted; the patterns themselves reveal, positively, only the

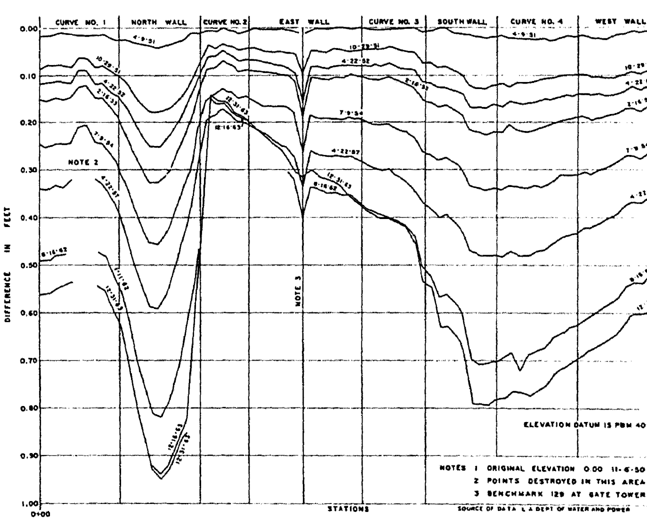


FIG. 13. Settlement Records of Perimeter Parapet Wall

locations of maximum subsidence, which correspond to the location of fault I. A plot of the maximum crest settlement vs. time is shown in FIG. 14. It appears that

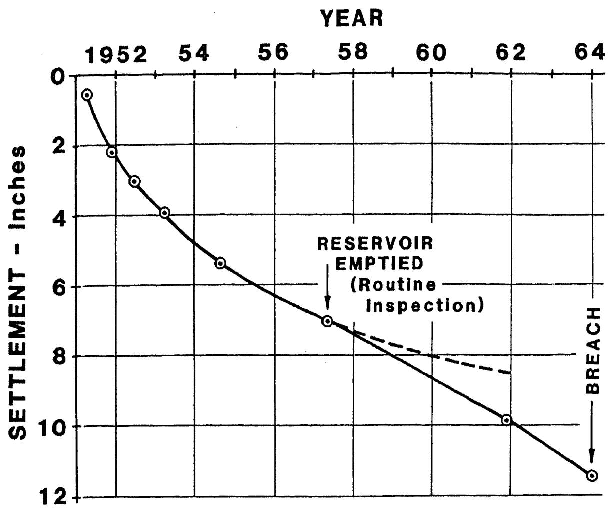


FIG. 14. Maximum Crest Settlement vs. Time

around 1957 (when fluid injection in the adjacent oil field was underway), the rate of settlement began to depart from the extrapolated path but, again, this is easy to recognize in hindsight. In the absence of an expected settlement-time relationship, and a stated criterion of unacceptable departures from the norm, the plot - in itself - is not decisive.

The simplified procedure proposed by Leonards and Narain (1963) was used to calculate the strain distribution along the crest and base of the dam using the measured settlement patterns (FIG. 15)**. The results,

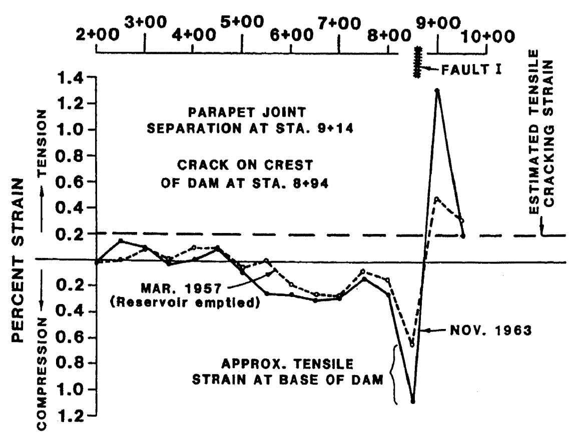


FIG. 15. Strain Distribution Along the Crest of the Main Dam

**The calculations were performed by Sunil Sharma, Graduate Assistant, Purdue University.

although approximate, are revealing. As it was not poss· ble to obtain samples to measure the tensile strain at cracking, it was estimated to be approximately 0.2% stra· (Leonards and Narain, 1963). The locations at which the calculated tensile strains exceeded the estimated crackii strain agreed with the locations at which cracking was ol served to occur at the crest of the dam. What is of cru· cial importance, however, is the observation that at the base of the dam the critical tensile cracking strain had already been exceeded in 1957. Had it been possible to make such calculations prior to 1963, they may have led to more frequent lowering of the reservoir and inspectioi of the lining. In that event, the catastrophe most like' would have been avoided.

The safety of the reservoir was crucially dependent on the ability of the asphaltic membrane to prevent seep· age from the reservoir from entering the highly erodible foundation soils. An extensive monitoring system was installed - which was very advanced for its time - in order to provide an early alert should the compacted cla; liner be damaged. The designers apparently expected the clay liner to crack first while the asphaltic membrane remained intact; in that event, increased flow in the underdrains would provide an alert that the clay liner was damaged in time to lower the reservoir and effect re pairs. The fatal flaw in the design was that the asphal tic membrane must have cracked either before or at the same time that the clay liner was damaged, thereby permi ting seepage to enter the foundation soils and begin the process that ultimately led to the demise of the reservo As described in the preceding paragraphs, the time-settlement records along the 12-inch foundation underdra and of the parapet wall, gave clues to impending disaste but these are clearly apparent only with the benefit of hindsight. Comparison of calculated strains in the embankment with expected cracking strains would have provided an early alert that cracking had developed in the lining, but the analysis needed for this purpose became available only nine months before the failure. Consider the crucial nature of the integrity of the asphaltic mem brane, instrumentation that could directly detect damage to the membrane was needed, but this would have been ver difficult to accomplish under the prevailing circumstanc

The lessons to be learned from these experiences ar summarized below.

LESSONS LEARNED

I. A MONITORING SYSTEM INTENDED TO WARN OF IMPENDING DANGER MUST BE OF SUCH A NATURE, AND BE SO LOCATED, THAT THE CRITICAL FACTORS CONTRIBUTING TO THE FAILURE ARE BEING SENSED. THIS IMPLIES FULL APPRECIATION OF THE PHYSICAL FACTORS INVOLVED -- AN APPRECIATION THAT IS NOT ALWAYS EASY TO COME BY.

II. A MONITORING SYSTEM THAT FAILS TO WARN OF IMPENDING DANGER IN TIME TO AVOID FAILURE CAN BE WORSE THAN NO SYSTEM AT ALL: IT TENDS TO INSTILL FALSE CONFIDENCE, AND MAY DELAY CAREFUL INSPECTION AND MAINTENANCE OPERATIONS.

III. TO BE SUCCESSFUL, A SUITABLE ANALYTICAL FRAMEWORK MUST BE AVAILABLE TO INTERPRET THE MEASUREMENTS. ACCEPTABLE LIMITS TO MEASURED VALUES MUST BE PREDETERMINED, AND A DECISIVE PLAN OF ACTION AGREED UPON, IN THE EVENT THAT LIMITING VALUES ARE EXCEEDED.

IV. IN SPITE OF OTHER SIMILAR INCIDENTS, e.g. VAJONT DAM AND BAKER POWERHOUSE SLIDES, I-95 AND KING'S LYNN TEST EMBANKMENTS (LEONARDS, 1982), THESE LESSONS ARE APPARENTLY NOT WIDELY APPRECIATED EVEN TODAY -- AS EVIDENCED BY THE UNEXPECTED CATASTROPHIC SLIDE (148 LIVES LOST) AT THE GUAVIO HYDROELECTRIC PROJECT NEAR BOGOTA, IN JULY 1983, AND OF THE SUDDEN LARGE SLIP OF THE UPSTREAM SLOPE OF THE CARSINGTON RESERVOIR, IN ENGLAND, IN JUNE OF 1984.

V. SUITABLE BASES TO INTERPRET IN ADVANCE WHEN THERE IS DANGER OF IMPENDING FAILURE ARE STILL LACKING IN THE CASE OF MANY TYPES OF GEOTECHNICAL STABILITY PROBLEMS. MORE RESEARCH ALONG THESE LINES IS BADLY NEEDED.

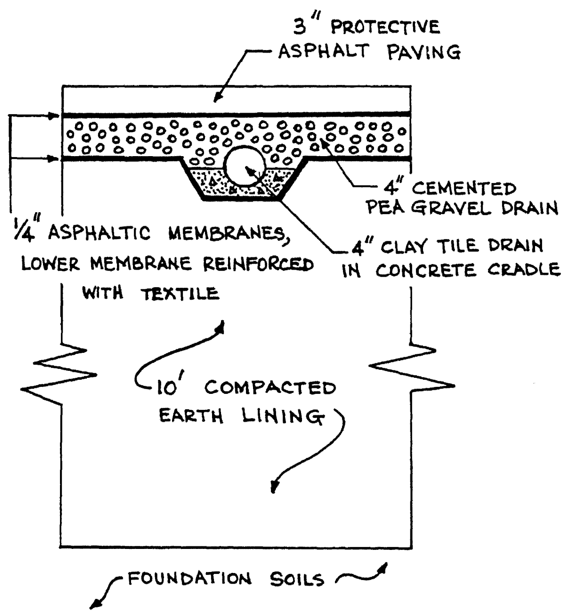
EPILOGUE

Could a safe and economical reservoir have been lt at the Baldwin Hills site? I believe the answer YES, provided the following prerequisites were isfied:

1. The monitoring system must provide a positive indication that seepage through the impervious asphalt membrane had commenced before significant erosion of the foundation soils could occur; and

2. The asphalt membranes must be accessible for ready repair.

Both these prerequisites could be satisfied by conucting the liner illustrated in FIG. 16. The main ference between this scheme and the one that was actlly used is the relocation of the asphalt membranes l pea gravel drains from below to above the compacted ay liner. As long as the asphalt membranes remain tact, no seepage could reach the foundation soils. ould sufficient movement occur to rupture the upper unreinforced asphalt membrane, the drains would have access to full reservoir head and large flows would immediately be detected in the drainage inspection chamber. Shut-off valves in the inspection chamber would close automatically to prevent water from flowing at high velocities in the discharge lines. It would be a simple matter to lower the reservoir and effect repairs. It is likely that the underlying reinforced asphalt membrane would remain intact, hence only the upper asphalt membrane (which is readily accessible) would need to be repaired. However, even if the reinforced asphalt membrane and the underlying clay blanket also cracked, the reservoir could be lowered long before significant damage to the foundation could occur because, in the absence of collapse in the foundation soils, the cracked clay blanket would prevent water under pressure from entering the foundation. Repairs would be less convenient but not overly expensive. There would never be any concern that slow insidious seepage could cause sudden collapses of sufficient magnitude to permit water under pressure to erode the foundation soils.



G. 16. Proposed Sandwich Construction for the Reservoir Lining

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