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LESSONS FROM NOTABLE EVENTS

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INTRODUCTION

Historical study of dams conceived in earlier times is essential. To continue advancing, the engineering profession must periodically review past problems and the lessons that they taught. Candid sharing of information on failures as well as successes is needed. In fact, some of the most valuable learning has come from projects where errors have been clear in retrospect. The following case histories are representative of the body of knowledge that has been accumulated in the interest of the future safety of dams.

THE BALDWIN HILLS RESERVOIR FAILURE

THOMAS M. LEPS AND ROBERT B. JANSEN

On December 14, 1963, at about 11:15 A.M., an unprecedented flow of water was heard in the spillway pipe at Baldwin Hills Dam in Los Angeles, California. The water came from drains under the reservoir lining.

At approximately 1:00 P.M., muddy leakage was discovered downstream from the east abutment of the dam, which formed the north side of the reservoir. At 2:20 P.M., lowering of the reservoir water level revealed a 3-ft-wide break in the reservoir's inner lining. A futile attempt was made to plug the hole with sandbags. Water broke violently through the downstream face of the dam. By 5:00 P.M., the reservoir had emptied, revealing a crack in the lining extending across the reservoir bottom in line with the breach in the dam (Fig. 2-1).

Geologic Setting

The Baldwin Hills, on the southwest edge of the Los Angeles Basin, are an expression of the Newport–Inglewood uplift, a series of structural domes and saddles extending about 40 miles (64 km) between Beverly Hills and Newport Beach. They are composed of sedimentary formations, principally of marine origin, overlying crystalline schist at depths of 10,000 to 12,000 ft (3050 to 3660 meters).

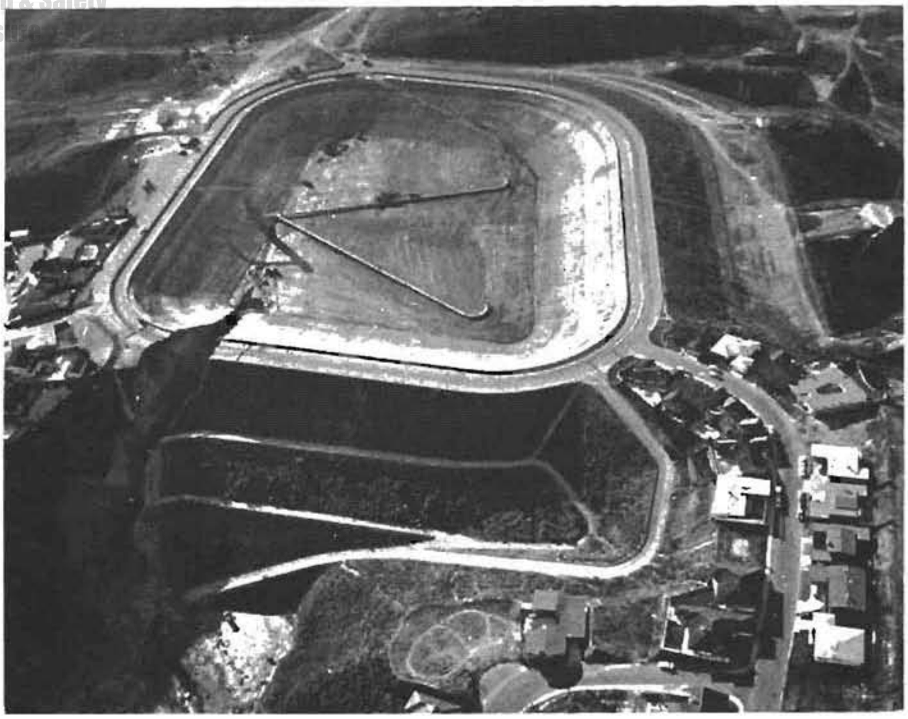

Fig. 2-1. Baldwin Hills Reservoir after failure.

The Inglewood Fault, the most northerly of four principal faults of the uplift, is about 500 ft (152 meters) west of the reservoir. The Newport–Inglewood uplift remains seismically active. However, at the Seismographic Laboratory of the California Institute of Technology, 15 miles (24 km) northeast of the reservoir, there was no report of any earthquake considered large enough to cause inertial damage at the project during the period 1950–63.

Several minor, steeply dipping faults were mapped in the Baldwin Hills during construction. Three of these, designated Faults I, II, and V, pass through the reservoir, angling away from the Inglewood Fault.

The reservoir foundation consisted of sediments that were susceptible to densification and erosion. During construction the formations were seen to be intensely jointed. Most of the joints were tight, but a few had gaps of as much as $\frac{1}{4}$ in. (6 mm).

Design and Construction

Construction began on January 13, 1947, and was completed on April 18, 1951. Located at the head of a northward-draining ravine, the reservoir was formed by the dam on the north side and compacted earth dikes on the other sides. Designed as a homogeneous earthfill, the dam was 232 ft (71 meters) high and 650 ft (198 meters) long.

The embankments were constructed of materials excavated from the reservoir bowl. To provide a uniform floor slope, part of the subgrade was compacted earth. The design incorporated underdrain systems and a reservoir lining (Figs. 2-2 and 2-3).

Below the earth lining lay a 4-in. (102-mm) lightly cemented pea-gravel drain blanket, with a system of 4-in. clay tile pipes placed beneath it to convey the leakage to a drainage inspection chamber. The pea-gravel blanket was capped with a $\frac{1}{4}$-in. porous sand gunite layer to prevent infiltration by soil particles. The lowest member of the lining was an asphaltic membrane about $\frac{1}{4}$ in. thick, sprayed on either the natural formation or compacted fill. Cotton fabric with an open weave served as reinforcement of the membrane where needed, mostly on the slopes and in the ditches for drain tile. The designers realized that the integrity of the reservoir would depend upon the impermeability of the asphaltic underseal.

Two separate underdrain systems were provided, one to drain the foundation under the earth embankments and the other to collect seepage passing through the earth lining and convey it through a central observation and measurement chamber to an outfall pipe.

In the bottom of the ravine under the main embankment was an open-jointed 12-in. (305-mm) clay tile foundation drain, its upper half covered with lightly cemented pea gravel to permit water entry. This drain passed through successive manholes where seepage flow was observed.

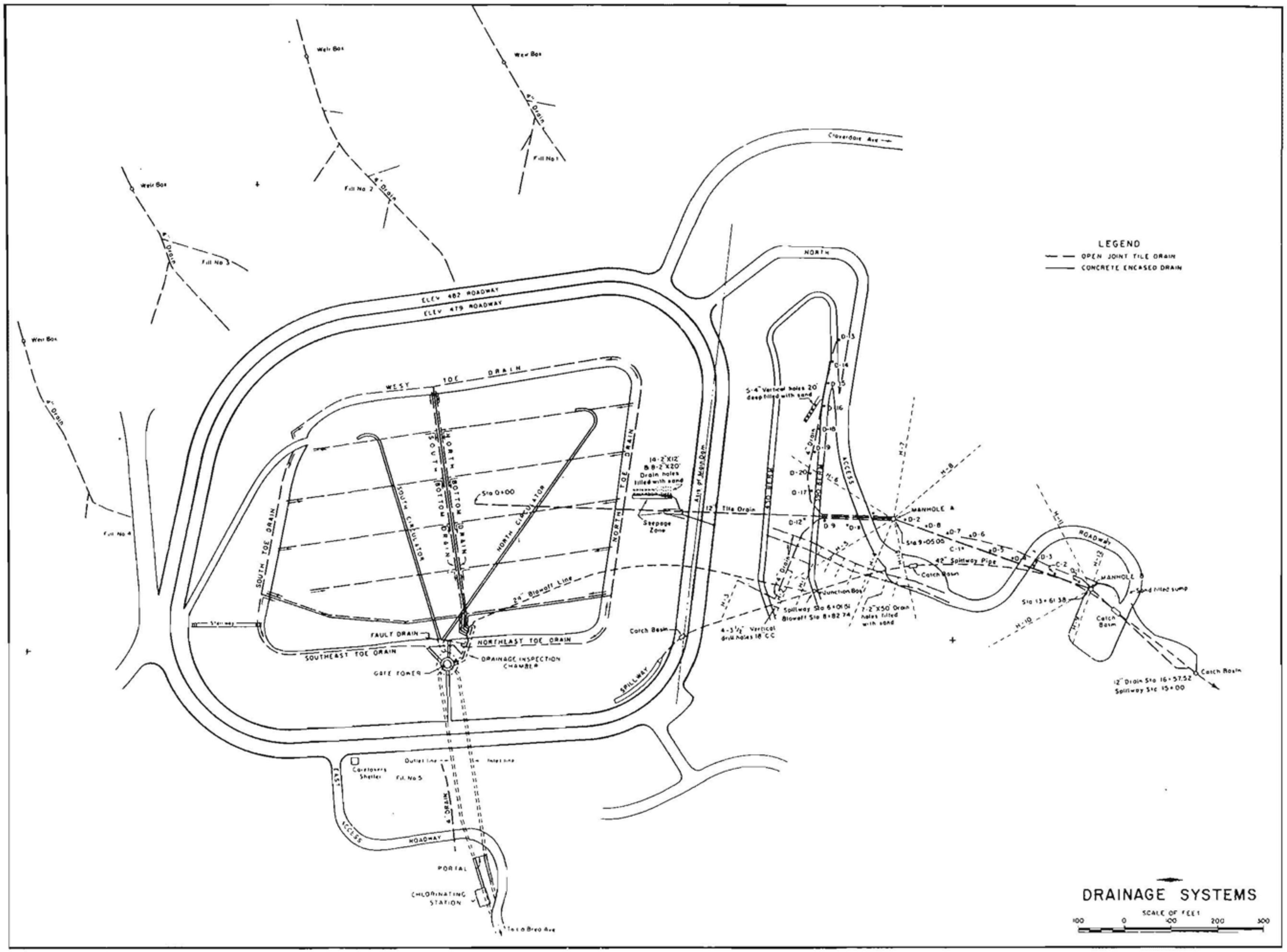

Fig. 2-2. Drainage systems, Baldwin Hills Reservoir.

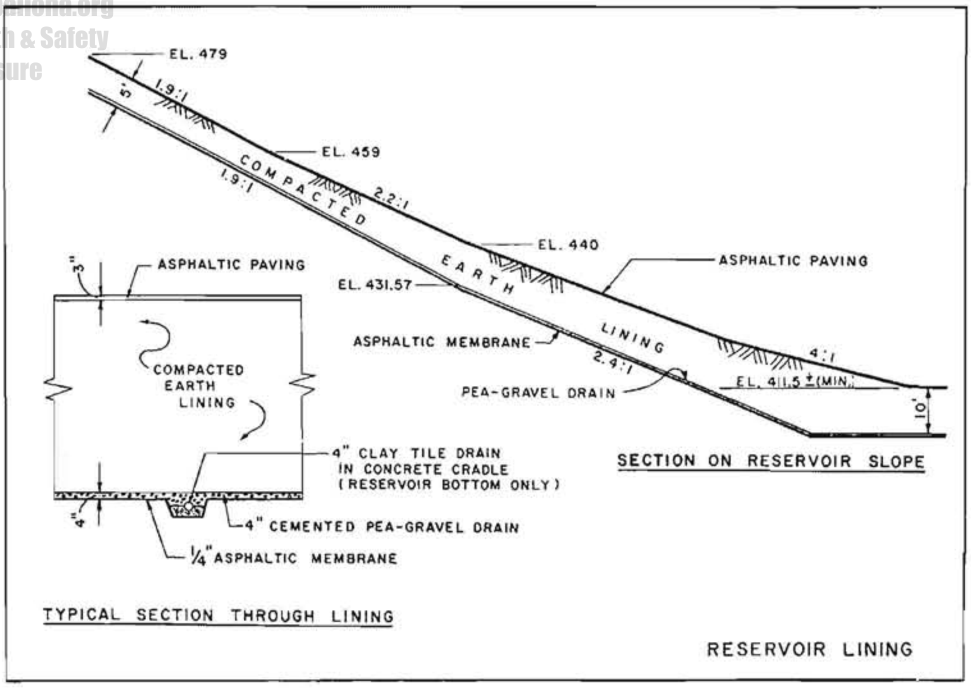

Fig. 2-3. Baldwin Hills Reservoir lining.

Horizontal and vertical holes were drilled into the dam foundation and were sand-filled. They were connected to 4-in. tile drainpipes leading to the 12-in. bottom pipe, which discharged into the city storm drain immediately downstream from the dam. Radiating from each of the manholes were other horizontal drill holes, backfilled with sand, to intercept seepage from the foundation.

Baldwin Hills Reservoir had a storage capacity of 897 acre-feet (1.1 million m^3) and was served by mortar-lined steel inlet and outlet conduits in tunnels through the east side. A 57-in. (145-cm) inlet line extended through the gate tower, supplying two circulator pipes leading to the west side of the reservoir. The tower had gates at various levels for discharge into the 66-in. (168-cm) outlet line.

Surveillance

The owner, the Los Angeles Department of Water and Power, conducted a comprehensive program to measure performance of the project. The reservoir was in service continuously from July 1951 until failure on December 14, 1963, except for a short time in 1957 when it was drained for cleaning and repairs.

A caretaker was on duty regularly at the reservoir from 7:45 A.M. to 4:15 P.M., seven days a week.

The Foundations and Structures Maintenance Section devoted one full day each month to inspection of the reservoir.

Leakage was regularly measured and sampled by hydrographers of the Water Operating Division, whose activities during a typical weekly inspection included entering Manholes A and B on the 12-in. tile drain and measuring flows from the horizontal drain holes. They also examined the spillway catch basins and the storm drains.

Flows of the reservoir underdrains were measured at weirs in the inspection chamber before discharge into a 24-in. (61-cm) outfall line, which connected to the spillway pipe.

Flows in the embankment foundation drains were measured monthly. Throughout the project operation no flows were observed in the drains for the embankments on the east, west, and south sides.

Periodic inspections were made of observation wells at the reservoir perimeter. Reportedly there was never any water in these wells.

History of Operation

During the first reservoir filling, begun on April 18, 1951, discharge from the underdrains increased substantially. Reservoir inflow was discontinued on May 2 with the storage level about 22 ft (6.7 meters) below the spillway. Repairs were made. Reservoir filling was resumed on June 18, 1951.

In the early years of operation following the initial remedial work in 1951, the underdrains required much maintenance. Appreciable volumes of asphalt, obviously from the asphalt membrane, flowed through the system from the west underside of the reservoir.

Calcium carbonate deposits developed in the drains, requiring frequent cleaning. Clogging, and possibly displacement of the drain tile, caused a reduction in the total seepage entering the inspection chamber. Seepage varied over the years from slightly more than 23 gpm (87 liters/min.) to a low of approximately 7 gpm (26 liters/min.), and measured about 8 gpm (30 liters/min.) in early 1963. Wide fluctuations of flow were observed in the individual underdrains, with much of the variation undoubtedly attributable to repeated acid treatment of the drains.

On October 29, 1951, a crack was discovered in the drainage inspection chamber near Fault I. Strain-gage points were set, and regular measurements of the opening were begun. The crack opened enough that the steel reinforcing bars in the wall were visible. Leakage through the opening was reportedly negligible prior to the reservoir failure.

The five drains entering the inspection chamber from the west were encased in concrete for a distance of about 20 ft (6 m). At that location they were below the $\frac{1}{4}$-in. (6-mm) asphalt membrane. On September 5, 1952, the concrete encasement was found to be cracked approximately 12 ft (3.7 meters) from the discharge end of the west toe drain. From the time that the crack opened, it was reasonable to assume that there was leakage into the foundation at that point.

In the period March 13–16, 1957, the reservoir was emptied, and the lining cleaned and checked. Inspectors found some cracking in the thin (about $\frac{1}{4}$-in.) cement coating on the asphaltic pavement, presumably due to creeping of the paving on the reservoir slopes. Although no substantial separation was found in the pavement, in some locations there was an overthrust of as much as 2 in. (5 cm). The reservoir bottom was found to be in generally sound condition.

In the weeks immediately preceding failure an apparent uplift developed in the inlet tunnel, the gate tower, and the part of the inspection chamber east of Fault I. This reversal of the settlement trend was similar to others that had occurred occasionally during the life of the reservoir, but it was larger than any previously recorded.

In the final year of operation, the flows from discharging horizontal drains under the main dam varied rapidly, declining to zero and then in some cases increasing to the earlier rate, followed by continued fluctuations.

Postfailure Conditions

After the failure, about 2 in. (5 cm) of fine silt and clay covered the reservoir floor.[1] There was a continuous crack approximately parallel to and near the toe of the east slope. Vertical displacement averaged about 2 in. (5 cm), but was as much as 7 in. (18 cm), with the west side of the crack lower than the east side. The crack extended up the south slope of the reservoir but with very little offset. Several sinkholes could be seen in the reservoir bottom along the continuous crack.

Inspection of preexisting cracks in the inlet tunnel showed no apparent change in the conditions of any of them.

On December 18, 1963, the drying of sediment on the floor disclosed a north–south-trending crack near the reservoir center. Subsurface investigation showed this crack to be at Fault V. There was very little displacement, but water apparently had been passing through the crack into the earth lining.

Postfailure investigators explored Faults I and V intensively. Thirteen test pits, two shafts, and 256 ft (78 meters) of tunnel were excavated. Also, drilling was done at the site to obtain samples of materials, for tests performed on both undisturbed and disturbed specimens of the natural and compacted soils.

Excavations at Faults I and V revealed downward displacement on the west side, which was evident in the reservoir paving but more pronounced at the pea-gravel drain, where offsets of as much as 7 in. (18 cm) were observed. These were generally extensions of rupture in the foundation.

At Fault I, cavities were discovered beneath the pea-gravel drain in some of the excavations.

At Fault V, in the north face of an exploratory trench, a hole found about 8 ft (2.4 meters) below the pea-gravel drain opened into a cavity about 11 ft (3.4 meters) long, 3 ft (1 meter) high, and as much as 2 ft (0.6 meter) wide. It extended north along the west side of Fault V. The pea-gravel drain at the fault was displaced, but it had not collapsed. A slight calcification on the surfaces of the large cavity suggested that it had developed over an appreciable time.

Excavation at Fault I disclosed cavities in the natural formation as deep as 47 ft (14 meters) below the pea-gravel drain. These cavities apparently had existed for an extended period.

In a drift excavated along Fault I, investigators encountered numerous cavities, some with maximum dimensions of several feet. The relatively smooth and straight fault planes suggested that separation of foundation blocks was the primary action in creating the gaps. Fine sand was found in some of the openings between the fault planes.

Some fragments of the asphaltic membrane under the cemented pea gravel contained small holes that could have allowed passage of water.

Settlement records indicated that a local trough of maximum settlement crossed the reservoir parallel to and just west of the trace of Fault V.

Surveys of the inlet and outlet tunnels, gate tower, and inspection chamber were conducted in the week after failure. With respect to measurements made on November 20, 1963, there was a relative uplift of 0.01 ft (3 mm) at the east portal, 0.11 ft (34 mm) at the tower, and 0.17 ft (52 mm) in the inspection chamber east of Fault I. There was no measured change in the chamber elevation west of Fault I. The top of the gate tower had moved to the east 0.12 ft (37 mm). Movement of the gate tower base had been generally downward over the life of the structure, reaching a maximum of 0.55 ft (168 mm) before a sudden uplift of 0.11 ft (34 mm) at about the time of failure.

The Inglewood Oil Field

The Inglewood Oil Field, discovered in September 1924, lies under the western half of the Baldwin Hills area. It covers about 1200 acres and in 1963 had more than 600 producing wells. The field adjoins the reservoir site on the south and west, the nearest reported production at the time of reservoir failure being from three wells within 700 ft (210 meters) of the south rim.

The geological structure of the field, the nature of the oil-bearing deposits, the relatively shallow depth, and the solution gas drive that prevailed in the early stages of production provided an environment favorable for subsidence.

Production at the Inglewood Oil Field commenced in 1924, and repressurization of the field was started in 1954.

Ground Movement

In 1917, the Department of Water and Power established benchmarks at the site of Centinela Reservoir, a proposed storage facility about 3000 ft (900 meters) southwest of the Baldwin Hills damsite. In 1943 levels again were run to some of these points, and the changes since 1917 determined. This and subsequent surveys indicated a general regimen of subsidence in the area (Figs. 2-4 and 2-5). In his extensive investigations, Leps[2] estimated the year of initial subsidence as about 1924.

The point of maximum cumulative subsidence in 1963 was about 3000 ft (900 meters) northwest of the intersection of La Cienega Blvd. and Stocker Street, over the most productive part of the Inglewood Oil Field. The total area of subsidence resembles an elliptical bowl with its center about 0.5 mile (0.8 km) west of the reservoir and its eastern periphery extending beyond La Brea Avenue. Subsidence at the reservoir site aggregated about 3 ft (0.9 meter) within the 1917–63 period. Its southwest corner had dropped more than the northeast corner. Between 1947 and 1962 the elevation difference between these reservoir corners increased by about 0.5 foot (15 cm).

Triangulation surveys in 1934, 1961, and 1963 showed stations in the Baldwin Hills to be moving laterally in the general direction of the subsidence depression. There was a progressive 0.4-ft (12-cm) elongation of the northeast–southwest diagonal of the reservoir between 1950 and 1963.

In an area between 0.5 mile (0.8 km) and 1 mile (1.6 km) southeast of the reservoir, several open cracks were detected in the ground as early as May 1957. They measured up to 2500 ft (760 meters) in length, and were similar to the 1963 reservoir cracks in terms of vertical offset, opening, orientation, and lack of longitudinal displacement. In general, both the reservoir cracks and those to the southeast paralleled old faults and joints. They developed in areas where tension could be expected as a result of warping of the ground surface.

Analysis of Failure

Several investigators have offered premises about the cause and mechanism of the reservoir failure. Although they share areas of substantial agreement, some of their differences have not been fully reconciled.

Hamilton and Meehan[3] concluded in 1971 that fluid injection caused shear displacements along Fault I, and that rupture propagating to the surface sheared the earth lining.

Casagrande et al.[4] were not convinced that any significant fault movement had taken place under the reservoir during its life. They believed that differential settlement occurred and could be explained by the greater compressibility of fractured and loosened material immediately west of each fault. They assumed that during pre-Holocene tectonic activity on Faults I and V the downthrown (west) block was dragged along the fault plane so that the materials near the surface on the downthrown side were loosened. Because the Inglewood formation is particularly friable, they regarded these effects as severe. They therefore concluded "that there was sharp differential compression of the foundation soils across Faults I and V, that this differential compression was initiated already during the first partial filling of the reservoir in 1951, and that it increased gradually during the life of the reservoir."

Leps[2] has stated that "the loosely articulated nature of the fault blocks has represented a foundation environment under the reservoir site which has been extremely sensitive, and responsive, to the localized but substantial changes in subsurface stresses caused by a subsurface, salt water injection program begun on a pilot basis by the oil field operators in 1954, and intensively pursued from 1957 . . . ."

The papers by Leps[2] and Hamilton and Meehan[3] demonstrated the apparent influence of oil field repressurization on the subsidence trend at the reservoir. The surveys of facilities on the East Sub-block showed clearly the slowing and short-term reversals of the downtrend, correlating with fluid injection. The cited papers concluded that differential vertical shearing at Faults I and II was accentuated by the repressuring. Recorded rebound east and south of the reservoir, and possibly some of the ground cracks in the environs, appear to support this argument.

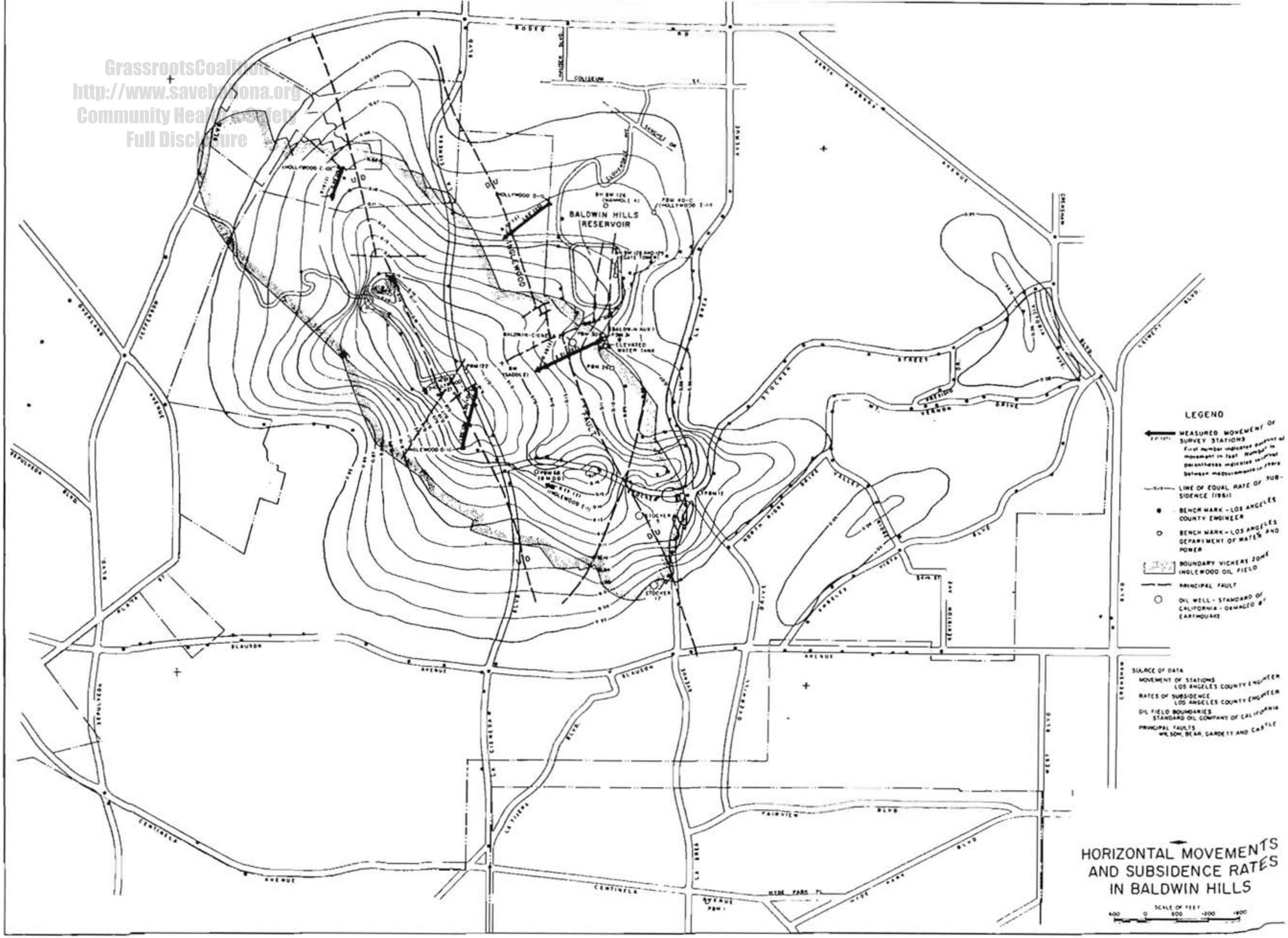

Fig. 2-4. Horizontal movements and subsidence rates in Baldwin Hills.

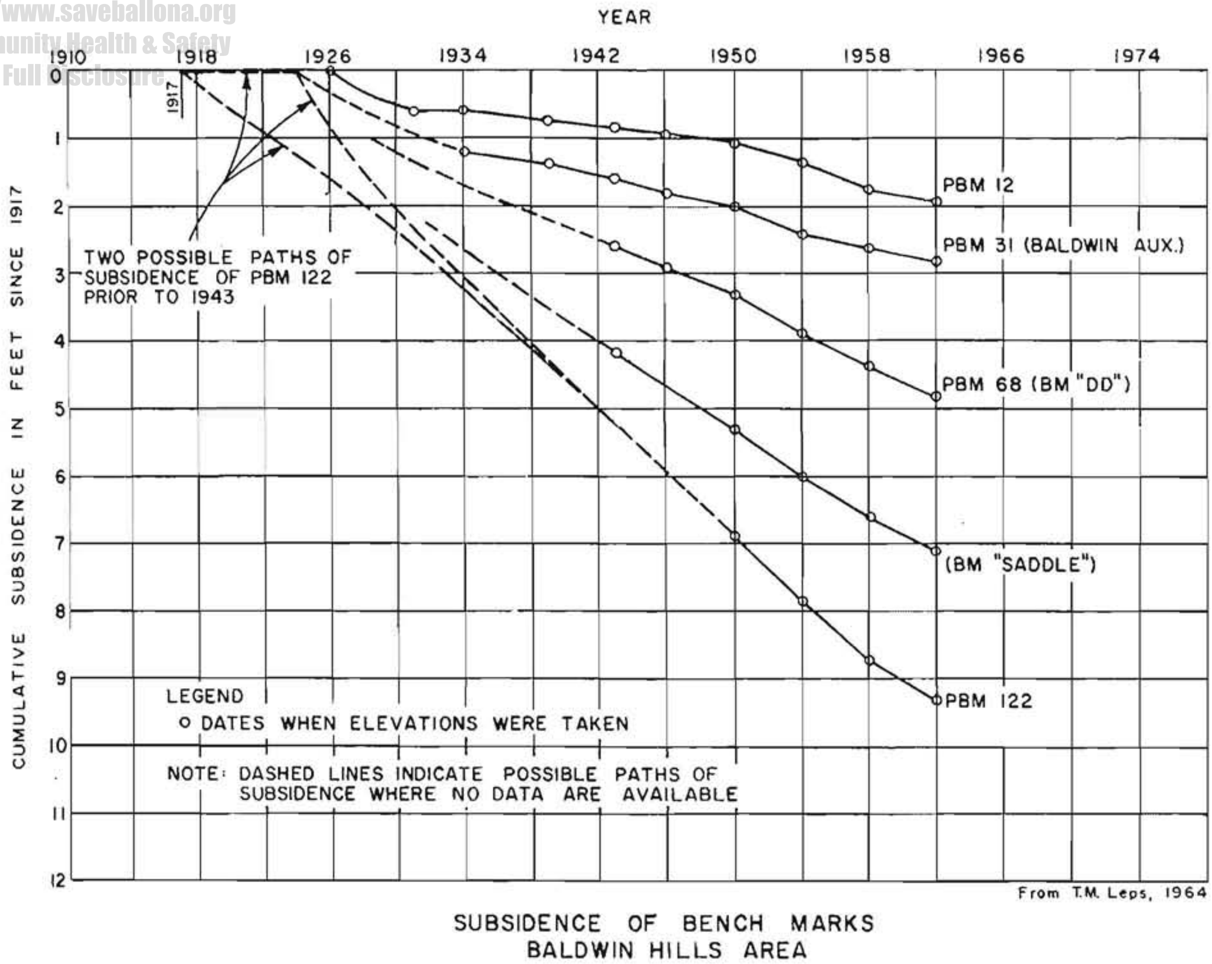

Fig. 2-5. Subsidence of benchmarks, Baldwin Hills area.

During drilling of oil wells south of the reservoir, circulation of drilling fluids reportedly was lost in the upper several hundred feet. Assuming this to be indicative of conditions under the reservoir, ready channels for seepage to great depth may have existed in the foundation, particularly at the faults.

There is evidence that gaps had developed between foundation blocks over a long period before the reservoir failure. They were seen and photographed at Faults I and II in an excavation near the gate tower site in 1948.

Further separation of the fault planes is believed to have occurred during the years of operation, which would tend to cause failure of the brittle underdrains, rupture of the asphalt membrane, and consequent wetting of any poorly consolidated materials in the fault openings.

In general, postfailure measurements showed the fault planes to be separated on the order of 0.25 in. (6 mm) to 0.5 in. (13 mm). In some places, the openings had been enlarged by erosion, which could be attributed to rainwater infiltration in years prior to construction, reservoir seepage in the period 1951–63, or outrush of water during the failure.

Separation of the fault blocks due to tension from warping of the subsidence bowl would not necessarily have resulted in differentially compressible materials of the nature, and to the degree, envisioned by the tectonic drag theory. However, overhanging fault-block corners that existed near the ground surface at the time of construction might have been broken by mobile equipment so that foundation material was forced into the accumulated fault gaps. Also, the wetting of the foundation could have made the corners of the overhanging foundation blocks susceptible to further breaking under the reservoir loading. All of this would have been conducive to localized settlement.

Heavy construction equipment also may have caused initial defects in the reservoir lining. Construction photographs show a scraper, a truck, and a motor grader operating directly on the cemented pea-gravel underdrain. A tracked crane was run over the asphaltic membrane, on plywood strips, during placement of the pea gravel. Much of this was membrane without fabric reinforcement.

Until the final episode of failure, the relatively small amounts of water that probably were entering the foundation could have been expected to pass through the permeable sedimentary materials and deeply into the fault gaps without developing sustained pressures of a magnitude that would move foundation blocks. On the other hand, the outrushing water at time of failure would have introduced much higher pressures. Although they might not have been able to move massive blocks, they might have reduced interplanar restraint enough to allow a small rebound, and they could have caused lateral shifting of the relatively thin foundation units between closely spaced faults such as that between Faults I and II. The Inglewood formation is intensely jointed, so that there might not be substantial resistance to such horizontal movement.

In summary, the reservoir and its immediate environs were subjected to many adverse forces, including horizontal and vertical displacement due to subsidence; local breaking of the weak foundation; some erosion at the faults; and rebound effects due to oil field repressurization, reservoir loading and unloading in 1951 and 1957, and the final inrush of water into the Fault I–II zone at time of failure.

With the benefit of the data that the failure provided, of course, the foundation problems would be predictable in such a setting. Under the same circumstances now, the settlement and seepage patterns that were observed at Baldwin Hills would signal the need for decisive corrective action. From a practical standpoint, present methods of interpreting performance data are not substantially better than those available in 1963. The instrumentation and the measurement procedures employed by the Department of Water and Power were adequate. In retrospect, they passed the test of showing what was happening.

Facing the same site conditions now, and armed with the knowledge of what went wrong at Baldwin Hills, designers could make several improvements. The obvious first step would be to avoid rigid drains so close to the water face and to the unstable and erodible foundation. A paved earth lining similar to that in the original design would be acceptable, but preferably of soils with higher plasticity. An impermeable, reinforced, multi-ply synthetic membrane, placed on a well-prepared subgrade, might provide an effective underseal. Any drain layer over this would have to be flexible and filter-protected. Overexcavation at the faults and backfilling with compacted clay would provide additional protection from cracking. A sandwiched lining of this type would be capable of appreciable deformation without rupture. However, it probably could not be guaranteed to remain completely watertight in the long term while its foundation continued to be stretched as a consequence of areal subsidence. Aging of the synthetic membrane might be a problem over a period of many years. The designers of any facility at the site also would have to be mindful of the seismic potential of the area.

Whether the engineers who conceived the original plan should have anticipated the nature of the foundation breaks is not easy to judge fairly in retrospect. The project was well reported in the technical journals and had few critics at the time. There was little precedent to offer guidance in the 1940s. The Baldwin Hills case, considering all its elements, is still unique in the history of dams.

Lessons Learned

1. Foundations in erodible rocks must be thoroughly explored to disclose any preexisting cavities or other defects.
2. The total prevention of leakage into a reservoir foundation over the lifetime of the facility may be unattainable under usual circumstances.
3. Associated faults that lie in close proximity and subparallel to an active fault should be regarded as susceptible to movement in a seismic event.
4. The possibility of differential fault movement unrelated to tectonic activity must be considered.
5. Potential effects of ground subsidence must be recognized in designing dams and reservoirs.
6. External causes and effects of subsidence must be closely monitored.
7. Foundation discontinuities should be given special treatment in construction.
8. Rigid buried elements such as cemented drains should not be incorporated into designs where differential settlement is a possibility.
9. Drains should be amply sized and provided with access, where possible, to facilitate maintenance.
10. Application of sprayed asphalt as a reservoir seal must be questioned as to its long-term effectiveness.
11. Earth linings preferably should have appreciable plasticity.
12. Erodible embankment and foundation elements must have adequate filter protection.
13. Structures placed across faults should be conservatively designed to accommodate predictable movements.
14. The use of heavy construction equipment must be carefully controlled to avoid damage of critical reservoir features on soft foundations.
15. Surveillance of a reservoir must be extended to its environs and to the consequences of adjacent developments and physical changes.