REASSESSMENT OF THE ST. FRANCIS DAM FAILURE

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INTRODUCTION

St. Francis Dam was built by the City of Los Angeles Bureau of Water Works and Supply in 1925-26 as a curved concrete gravity dam, approximately 200 feet high, in San Francisquito Canyon, about 5 miles northeast of what is now Magic Mountain, California. The stated purpose of the dam was to provide 38,000 acre-feet of emergency storage for Los Angeles-Owens River Aqueduct water should slippage along the San Andreas fault sever the aqueduct. The dam failed catastrophically upon its first full filling, near midnight on March 12/13, 1928, killing 450 in the San Francisquito and Santa Clara River Valleys. It was the greatest man-made disaster in California history.

No less than a dozen separate investigations of the failure followed, most notably, that of the State of California, which convened on March 19th, made one site visit, and issued their report 5 days later (Committee Report, 1928). The State's consulting board concluded that the dam's failure emanated from a fault contact between the Pelona Schist and Sespe formation on the right, or west abutment. This somewhat simplistic explanation was offered after observing that blocks from the dam's right abutment were found further downstream than those of the opposing, east side.

The failure of St. Francis Dam still serves to remind us that disaster awaits those who do not employ sound engineering geologic input, regardless of their respective engineering abilities. The public outcry following the disaster led to the humiliation and demise of William Mulholland, the dam's principal sponsor and the architect of Los Angeles' water supply system. It also led to the formation of the State's Division of Safety of Dams (DSOD), which retains regulatory authority over all but federally-owned dams in California.

Detailed geomorphic assessments suggest that St. Francis Dam was constructed with its eastern abutment against paleomegaslides, comprised of the Pelona Schist. The balance of this article explores what is currently being learned about the St. Francis disaster by reassessing its failure with modern forensic analytical techniques, many of which were unavailable to our professional contemporaries in 1928. Research-to-date suggests that St. Francis was not designed with a modern appreciation of uplift theory; the dam's base width was not as thick as previously assumed; and the designers were not aware that the left abutment was a paleomegalandslide or that the Sespe beds comprising the opposing abutment would slake upon submersion. A review of the available evidence suggests the dam failure sequence was brought about by the partial reactivation of the paleomegaslide, within the schist comprising the east abutment. As the slide mass was saturated to a depth of some 200 feet, "keyblocks", or discontinuity-bordered wedges beneath the sloping east abutment, appear to have been hydraulically "lifted", thereby breaking the dam and initiating a rapid chain-reaction failure, similar to the style of failure subsequently experienced at Malpasset Dam in France in 1959.

THE SITING OF ST. FRANCIS DAM

The St. Francis Dam was originally conceived by Bill Mulholland, who served as Chief Engineer of Los Angeles' Bureau of Water Works and Supply (later the Department of Water & Power) from 1886 until his untimely retirement in 1928. Mulholland was among the first in America to conceive and construct a long-distance municipal water system, the 300-mile long Los Angeles-Owens River Aqueduct, completed between 1906-1913.

Mulholland visualized a dam site along San Francisquito Creek, adjacent to one of the camps built during the aqueduct's construction. He had excavated exploratory adits into the Sespe beds and filled these with water to evaluate the adequacy of the Sespe beds to support a reservoir (Mulholland, 1928). Mulholland had also recognized the tendency of the Pelona Schist slip "along its slaty cleavage" in the outcrops along the southeast side of the canyon (City of Los Angeles, 1911). Mulholland's attraction to the dam site was linked to what he perceived as favorable topography: a natural narrowing of the canyon downstream of a broad, upstream platform, thereby creating a large water storage area (Mulholland, 1928). Unbeknownst to Mulholland, the reasons for this favorable topography lay in the fact that the site had already served as a natural reservoir, due to damming of San Francisquito Creek by ancient megalandslides in the Pelona Schist. The seemingly intact Pelona Schist had rotated downward, onto the opposing bank of Sespe conglomerate, thereby blocking the Canyon and creating a landslide dam (Figure 1). The waters of San Francisquito Creek had eventually overtopped the landslide dam and reexcavated a channel. The broad flat area, seen by Mulholland as an excellent reservoir site, had actually been created through sedimentation behind the paleolandslide dam. Evidence of the paleoslides and the impounded lake is clearly seen today, in the form of stepped terraces along the Pelona escarpment (Figures 1, 13 and 14).

DESIGN AND CONSTRUCTION

The Los Angeles Aqueduct was completed in 1913, a major engineering marvel in its time, bringing Mulholland a great deal of notoriety and respect. But, to the ranchers of the

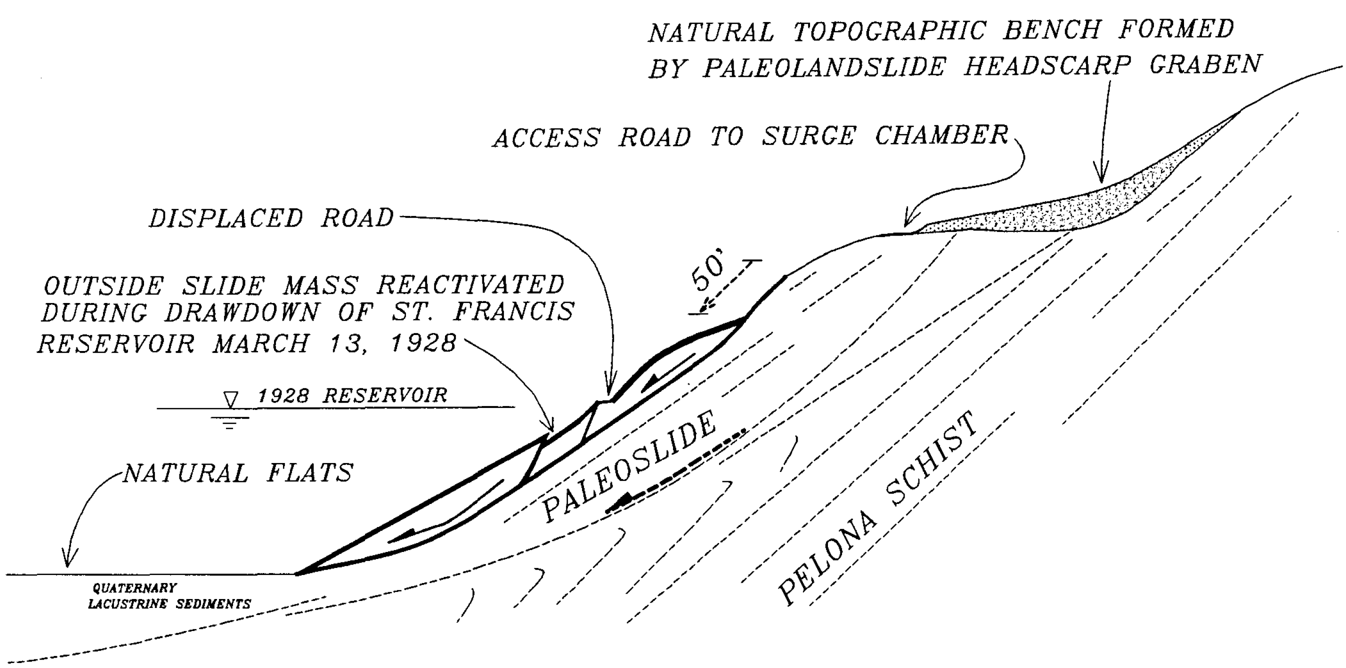


Figure 1. Present day view looking upstream at Pelona Schist escarpment along the southeastern side of San Francisquito Canyon. The remains of St. Francis Dam lie in the right foreground, while a drawdown-induced landslide, which slid approximately 50 feet into the reservoir, is at left center. The bench at the top of the slope appears to be the headscarp graben of an extensive paleomegaslide complex developed within the Pelona Schist, likely during the Pleistocene Epoch. The Los Angeles Aqueduct was built upon this surface in 1908-13. A schematic cross section through the 1928 slide and the surrounding slope is presented below.

Owens Valley, Mulholland was a fiend with no equal, as he was the point man for Los Angeles' tangled web of water rights acquisition that took away the water from their homesteads. The ranchers had been outmaneuvered by slick city lawyers who lobbied for favorable legislation in far-away Washington, D.C., long before the ranchers realized what the Angelinos were up to. The ranchers' sense of frustration often came to a head, and in mid-1920's, they struck back by dynamiting the aqueduct at vulnerable points along its remote traverse through the Owens Valley. The plight of these rural folks drew considerable sympathy at the time, even within newspapers, such as the Los Angeles Record, which never failed to take its shots at the City's Departments of Water and Power. These circumstances combined to create an air of secrecy and suspicion amongst Mulholland's Water Works people, which likely prevented much of St. Francis Dam's engineering records from ever being evaluated by outside entities (for example, no correct cross section of the dam had ever been released or published prior to this paper).

In 1916 the Los Angeles Department of Public Services bifurcated a new companion agency to Mulholland's Bureau of Water Works and Supply, called the Bureau of Power and Light. This new bureau constructed San Francisquito Powerhouse Number 1 along the aqueduct a year later (about 5.5 miles upstream of what would later become the St. Francis Dam site). In 1920, Power and Light constructed Powerhouse No. 2, six miles down the canyon, approximately 7000 feet downstream of the future damsite. Both of these facilities remain in service today.

Because of the political sensitivity of water rights acquisition, Mulholland appears to have proceeded with the planning of St. Francis Dam in a virtual veil of secrecy. Los Angeles Department of Water and Power (LADWP) records show that the City surveyed the entire dam and reservoir area in detail in 1922. The City filed a condemnation petition for the reservoir inundation area with the Federal government in April 1924. Their original plan called for a concrete gravity dam, virtually identical to the design then being utilized for the Hollywood Reservoir (which after the failure of the St. Francis Dam, was buttressed with an earthen berm against its downstream face, ENR, 1934). As originally conceived, the dam was to extend some 175 feet above the bed of San Francisquito Creek and provide storage for 30,000 acre feet of water. In July 1924, the lake's capacity was increased to 32,000 acre feet by adding a small wing dike, approximately 10 feet high, extending from the west abutment. Construction began in the spring of 1924, with the first concrete being placed the following August.

In July 1925, after eleven months of placing concrete, Mulholland's engineers apparently decided to increase the reservoir capacity once again (to 38,168 acre feet), by adding an additional 10 feet to the dam's height (but, without changing the base width). This increased storage was made possible by raising the dam to a height of 185 feet above the

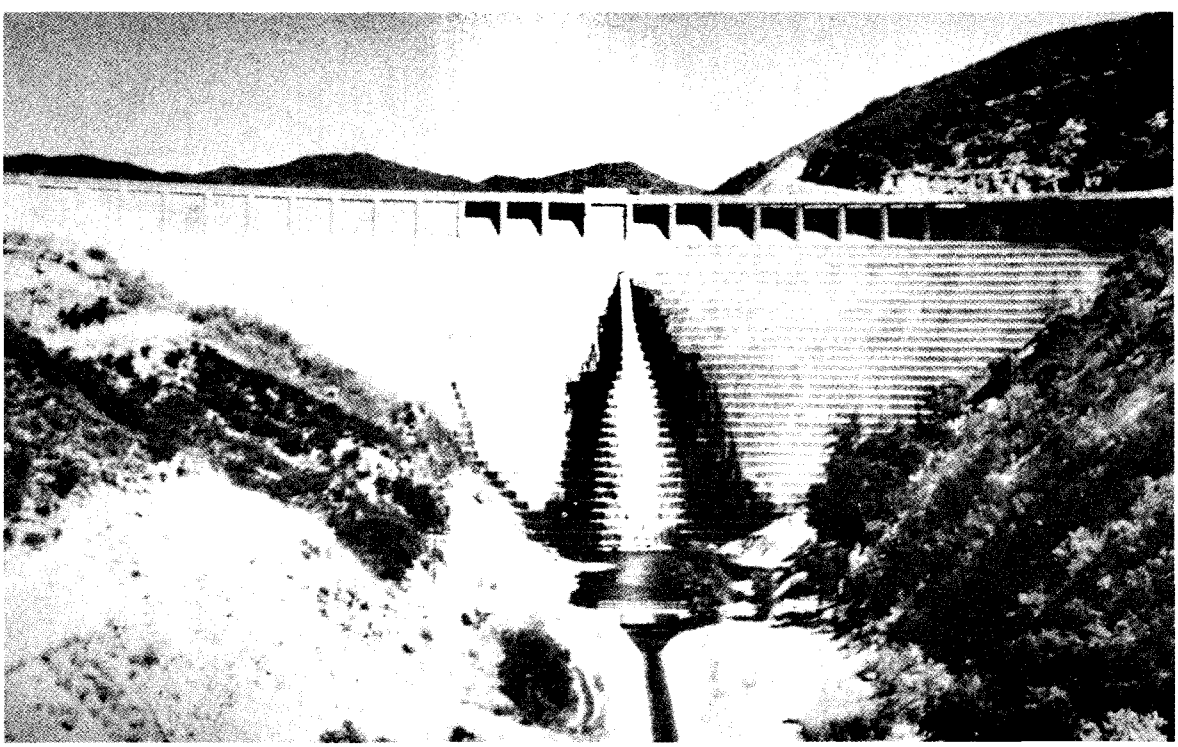
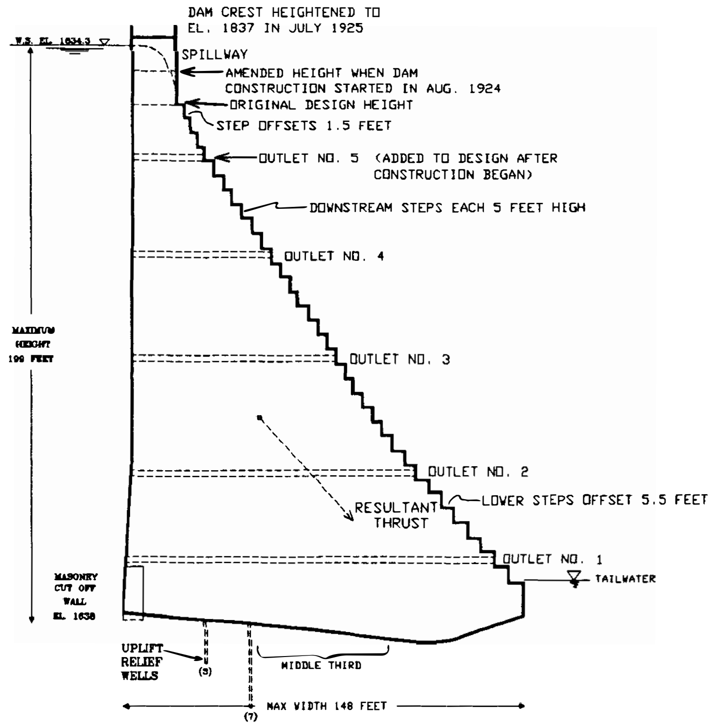
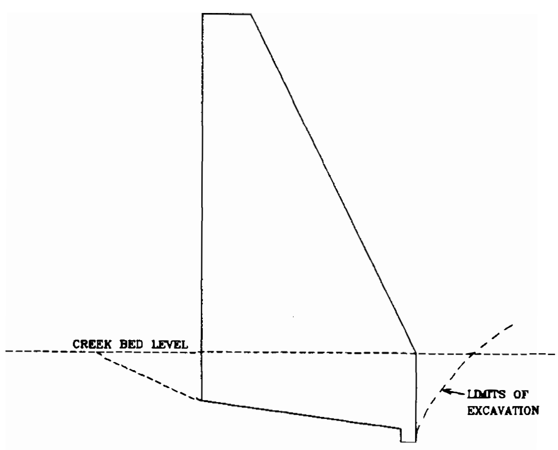


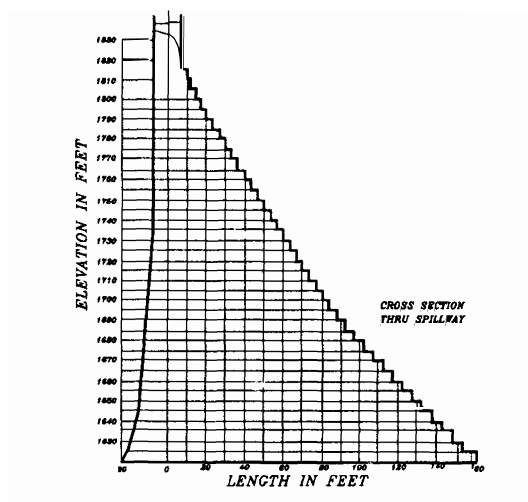
Figure 2. St. Francis Dam as it appeared just after its completion in May 1926. It was a curved concrete gravity section, standing approximately 200 feet high and was comprised of 130,000 cubic yards of concrete without benefit of any contraction joints, drainage galleries, cut-off walls, or grout curtain. The Pelona Schist made up the east abutment, on the right side of the photo, while the Sespe formation red beds comprised the upper two-thirds of the west abutment (shown at left).



ELEVATION VIEW OF ST. FRANCIS DAM
TAKEN THROUGH SURVIVING BLOCK 1



DESIGN SECTION THRU STA 1 + 25.00 FROM LA DWP FILES. LIKELY THE ORGINAL DESIGN, THE SECTIONS ARE CONSISTENT IN SHOWING THE DOWNSTREAM TOE CHOPPED OFF VERTICALLY, BELOW THE LEVEL OF THE STREAM BED.



CROSS SECTION PROVIDED BY THE LOS ANGELES BUREAU OF WATER WORKS AND SUPPLY TO GOVERNOR'S INQUIRY BOARD, SHOWING A FLARED DOWNSTREAM TOE. MULHOLLAND TOLD THE CORONER'S INQUEST THAT THE DAM WAS 170 FEET THICK AT THE BASE (WHICH, IF TRUE,APPEARS TO HAVE BEEN ADDED LATER)

Figure 3. Three cross sections of St. Francis Dam, through the outlet works contained in Block 1, the only section of the dam to survive the failure. The upper section represents a reconstruction by the author. That at the lower right is taken from LADWP files, and likely represents the original dam design. Note how the downstream toe was clipped, a feature Outland (1963) had discovered when examining construction photos. The section at lower right was the one provided to the various boards of inquiry into the failure. A few uplift relief wells had been provided for the central core of the dam, within the stream channel. No such protection was afforded the sloping abutments.

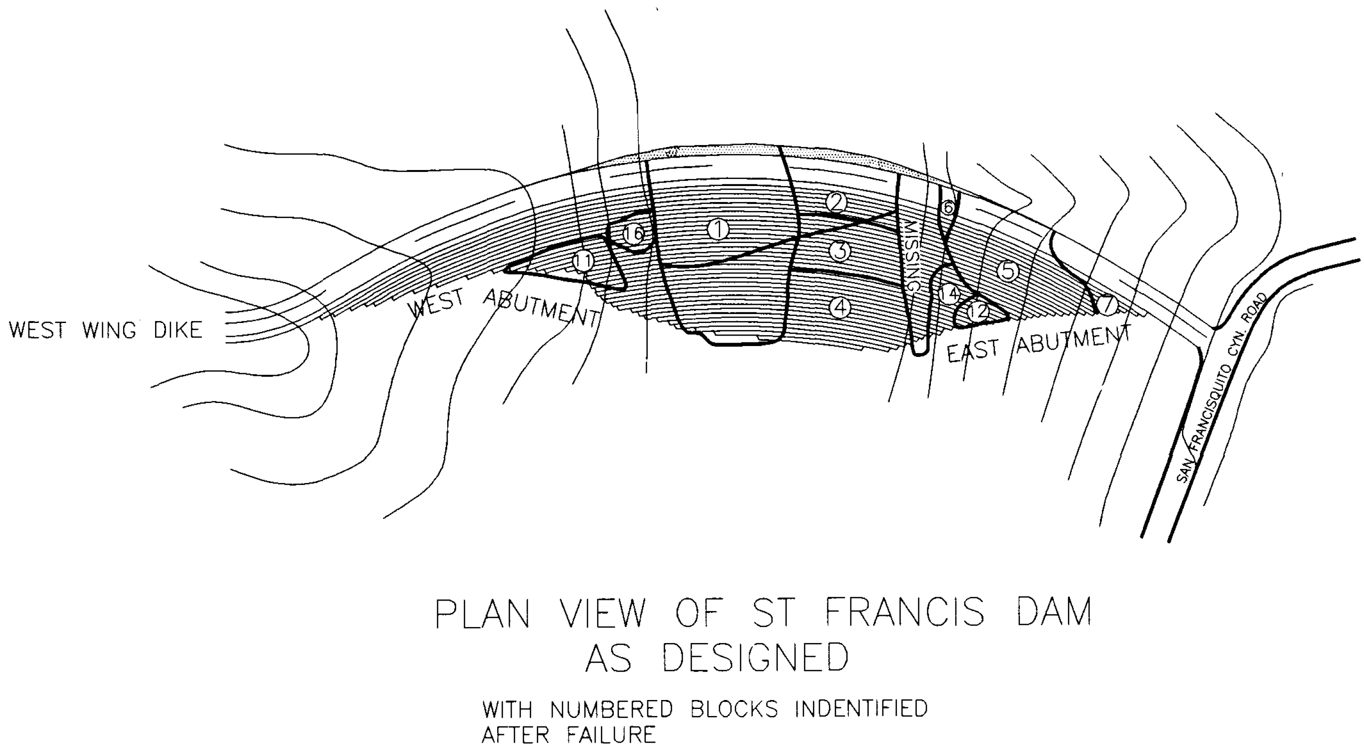


Figure 4. Plan view of the main section of St. Francis Dam as orginally designed, from the files of the Los Angeles Department of Water and Power. Diagonal lines and numbers delineate the blocks identified in the post-failure analysis by the State (shown in Figure 8). The portion of the dam between Blocks 2/3/4 and the East Abutment (Blocks 5/6/7) was never found, this portion of the dam becoming the so-called "missing section". The dam's horizontal steps were each five feet high; and varied in width, from 5.5 feet in the lower part of the structure, to as little as 1.5 feet in the uppermost part of of the dam. The unique spacing of the steps enables forensic assessment of where the blocks displaced downstream came from within the structure.

creekbed, and extending the west wing dike some 1300 feet along a natural ridgeline in the Sespe.

Owens Aqueduct water was first diverted into the reservoir on March 1, 1926, and the structure was completed the following May. The main dam section across San Francisquito Creek was designed as a free-standing gravity section, as shown in Figure 3. Very little seepage relief was afforded in the design, only a few uplift relief wells beneath the central core were employed (as shown in the upper half of Figure 3). No cutoff walls, grout curtains, or drainage inspection/collection galleries were employed in the structure's design, a fact that later investigative boards would repeatedly mention. In addition, no provision was made for emplacement of expansion joints within the main structure.

The main portion of the dam was arched upstream, on a 500 foot radius, as depicted in the plan view, shown as Figure 4. Additional load-carrying capacity due to arch action was neglected in the dam's design, a common and seemingly conservative decision. During the Los Angeles County Coroner's Inquest following the dam's failure, Mulholland would testify that the dam was designed with a "factor of safety of three or four" (Mulholland, 1928, p. 10).

INITIAL OPERATION AND FILLING

Filling of the St. Francis reservoir was nothing short of dramatic, the level rising at an average rate of 1.8 feet per day over the first three months (March 1 to June 1, 1926). The St. Francis reservoir pool reached elevation 1832, three feet below spillway crest, by May 10, 1927 (177 feet above the creekbed). This elevation was held until May 26, at which point the spring run-off ceased, and the reservoir lowered. Modest spillway capacity was afforded the structure through the employment of narrow slits, only 18 inches high, that were built into the main dam section (from elevation 1835 to 1836.5, as shown in Figure 3). In all, there were eleven 20-foot wide spillway bays (seen to good effect in Figure 2). An additional emergency overflow weir was built into the end of the wing dike section, approximately 1100 feet northwest of the main dam section.

During the initial filling of 1926-27, several cracks appeared, transverse to the dam's axis, in the downstream face of the main structure. These cracks appeared to Mulholland to be "transverse contraction cracks", presumably caused by the thermal stresses associated with the curing of the mass concrete (the main structure was comprised of 130,500 cubic yards of concrete, placed in just 16 months). Two sets of cracks appear to have formed, those on the steeply rising flanks of the structure, and two within the maximum section. The approximate locations of these fractures, deducted from verbal descriptions and photographs, are depicted schematically in Figure 5. The flank cracks were observed to be wider at their juncture with the abutments, narrowing upwards; while the two longer, main section cracks, appeared widest at the top, near the parapet wall. The transverse

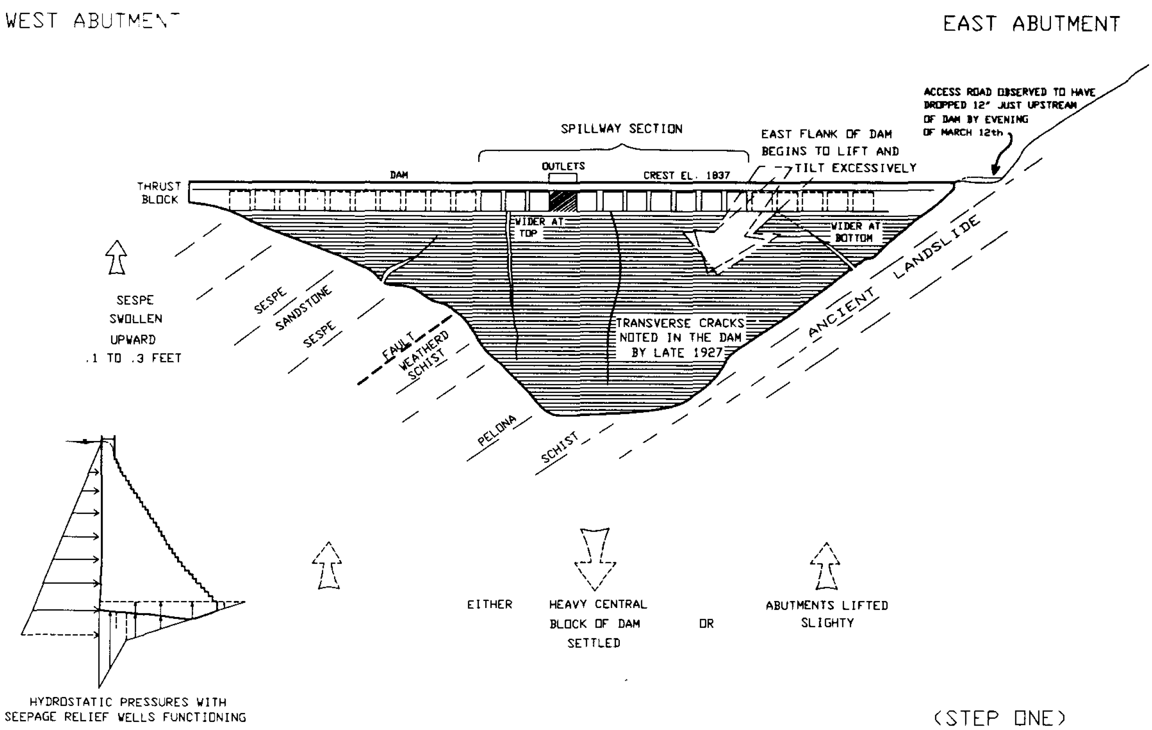


Figure 5. Schematic elevation view of transverse cracks which appeared on the downstream face of the dam prior to its failure. Th four cracks most often described are portrayed, their having been formed by late 1927. Both abutment cracks were angled obliquely, and widest at their junctures with the foundation rock, suggesting either abutment uplift or main section settlement. The two longest cracks, in the dam's central section, were later found to be those bordering Block 1.

cracks within the main dam block were wedged with oakum on the downstream face and backfilled with cement grout.

In February 1928 the lake level again rose to the dam crest and a number of leaks developed in the foundation, mostly on the west side, within the Sespe conglomerate. One of these leaks was in the vicinity of the fault separating the Sespe from the Pelona Schist. It would appear that the Sespe was little deformed in this location, having been conformably deposited upon an eroded surface of the Pelona, nearly coincident with the latter units' regional foliation. The paucity of structural distortion more than a few feet into either unit, suggests that inter-formational slippage along this boundary was responsible for the fault developing (due to the dramatic stiffness variance between the two units; see Clements, 1929).

When the spring run-off began arriving in January 1928, the reservoir was allowed to fill to maximum capacity, reaching elevation 1934.75 (three inches below spillway) by March 7, 1928, whereupon no additional aqueduct water was diverted into the lake. The previous years' leaks gushed forth with additional discharge and new leaks developed on both abutments. During the first week of March, a substantive leak developed along the wing dike, issuing artesian flow of about 0.60 cubic feet per second (cfs). Mulholland ordered a work crew to install a 8" concrete pipe underdrain from this point, eastward along the base of the dike, to discharge along the west abutment contact of the main dam section, giving an appearance that there was seepage emanating along this juncture (Figure 6).

INCIDENTS LEADING UP TO THE FAILURE

By Monday March 12th, the reservoir pool had been held at Elevation 1934.75, just three inches below spillway, for five days. Wind-driven waves were lapping up, over the spillways, complicating the task of discerning between last-minute leaks and spillage (Figure 6). With all downstream storage facilities similarly filled to capacity, excess water from the Los Angeles Aqueduct were released into San Francisquito Canyon for the first time in almost two years that morning; the 30 cfs discharge emanating from the aqueduct's overflow gates where it crossed Drinkwater Canyon (about 3000 feet downstream of Powerhouse 2). Some downcanyon residents, noticing water in the usually dry creekbed, wondered if something wasn't wrong at the dam.

That same morning, the damkeeper, Tony Harnischfeger, had telephoned Mulholland to say that a new, larger leak, had developed on the Sespe abutment, and that the discharge was "dirty", cause for concern to any dam engineer, as this would be an alarm to the possibility of piping. It was a Monday morning, and Mulholland, very much the personification of "the man in charge" (Mulholland made a practice of visiting all of his 19 dams once every 10 to 14 days according to his testimony before the Coroner), chose to personally inspect the dam immediately with his chief assistant engineer, Har-

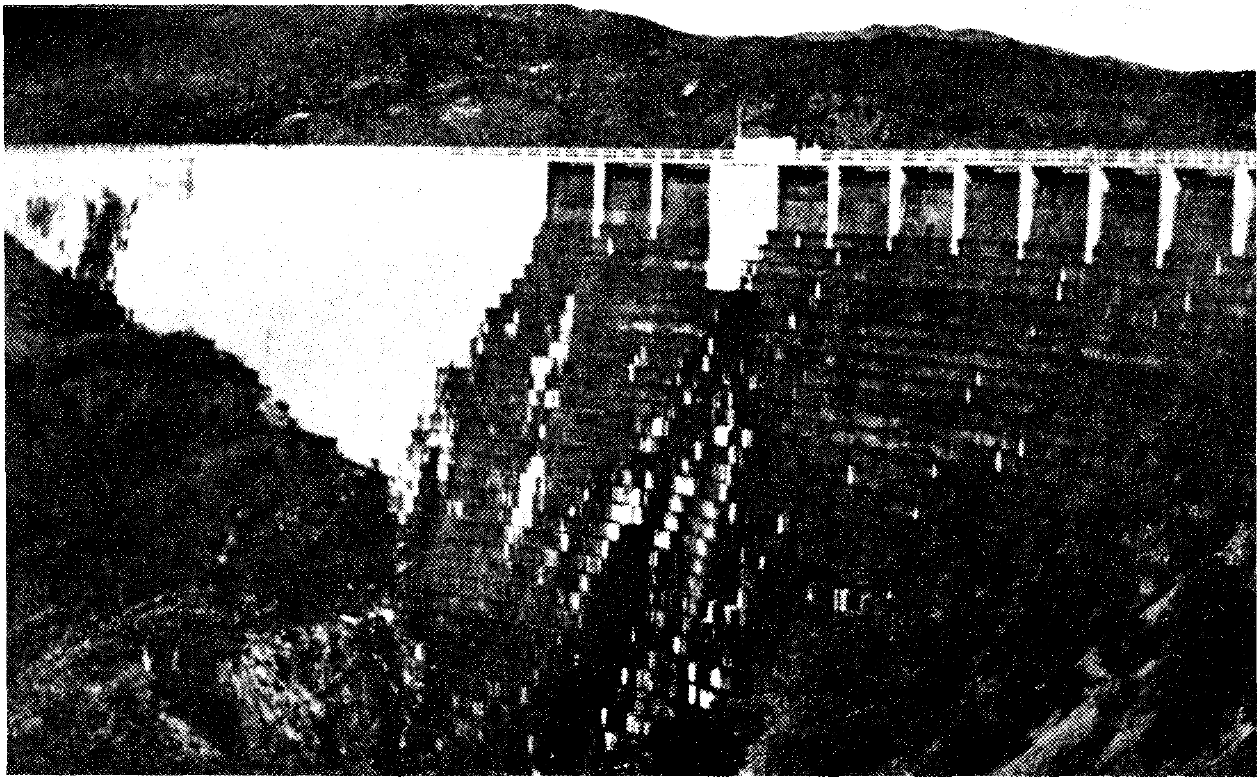


Figure 6. View of downstream face of St. Francis Dam, taken around noon on Monday March 12, 1928, the day of the failure. Note the windswept spillage coming over the spillways, thereby complicating the task of discerning last minute leakage from spillage. At this time approximately 2 cfs of water was running down the west abutment contact, shown at left center

vey Van Norman. Mulholland and Van Norman arrived by chauffeured car at the dam at 10:30 AM, whereupon they inspected the dam for the next two hours (Figure 7).

Wave-whipped water was issuing over the spillways, even though the lake had dropped to 1834.3 over the past two days (Figure 6). Their inspection revealed that about 2 cfs outflow was now emanating from the the western abutment, but the water was decidedly clear, which would preclude the possibility of piping (but not that of uplift). The volume of water issuing from the abutment was observed to be somewhat inconsistent, with a surging style of flow. Further investigation showed that the water had only become muddy where it washed over uncompacted sidecast fill, on the west abutment access road (which had a 18% uphill grade). Though not appreciating its significance at the time, the pair also noticed "a small stream of clear water" cascading down the exposed (downstream) juncture of the dam's east abutment, against the schist. Mulholland and Van Norman left the dam around 12:30 PM, assuring Harnischfeger of its apparent soundness. Within 12 hours, Harnischfeger and his six year old son would be the first victim's of the dam's collapse, their bodies never being found.

The apparent conditions at the dam site on Monday evening, March 12, 1928, are depicted schematically in Figure 5. No less than three separate parties drove by the dam within an hour of its apparent failure, just before midnight. All of these people were Bureau of Power employees stationed at Powerhouse No.1, five miles above the dam. The first of these was a family quoted by Charles Outland (1963, p.208) in his book, *Man-Made Disaster*, who had refused to be publicly identified. They told Outland that they had driven the San Francisquito access road, between Powerhouses 2 and 1, the evening of the failure. The unidentified source(s) related they had observed "the road had dropped at least 12 inches, just upstream of the dam's east abutment" (in the Pelona Schist). The location identified would seem to correspond to the upstream lateral scarp of the 1928 east abutment slide, shown in Figures 13, 14 and 15. Such an observation would be expected if the east abutment landslide was beginning to drop, and thereby, thrust against the back of the dam.

Two other employees stationed at Powerhouse 1, Elmer Steen and Katherine Span, had left their friends' home at Powerhouse 2 around 11:35 PM, climbing the canyon's southeast wall, and passing the dam's east abutment crest around 11:40 to 11:45 PM. In their testimony before the Coroner's inquest, they stated they had not seen "anything unusual", only that the dam was blacked out (which was normal) and appeared "spooky" in the moonlight as they passed along the rough San Francisquito Canyon Road (which was unpaved). Either of these individuals may also have been Outland's "mystery witness", as he had tracked them down for interviews in the 1950's. At the time of the interviews, Steen was on an LADWP pension.

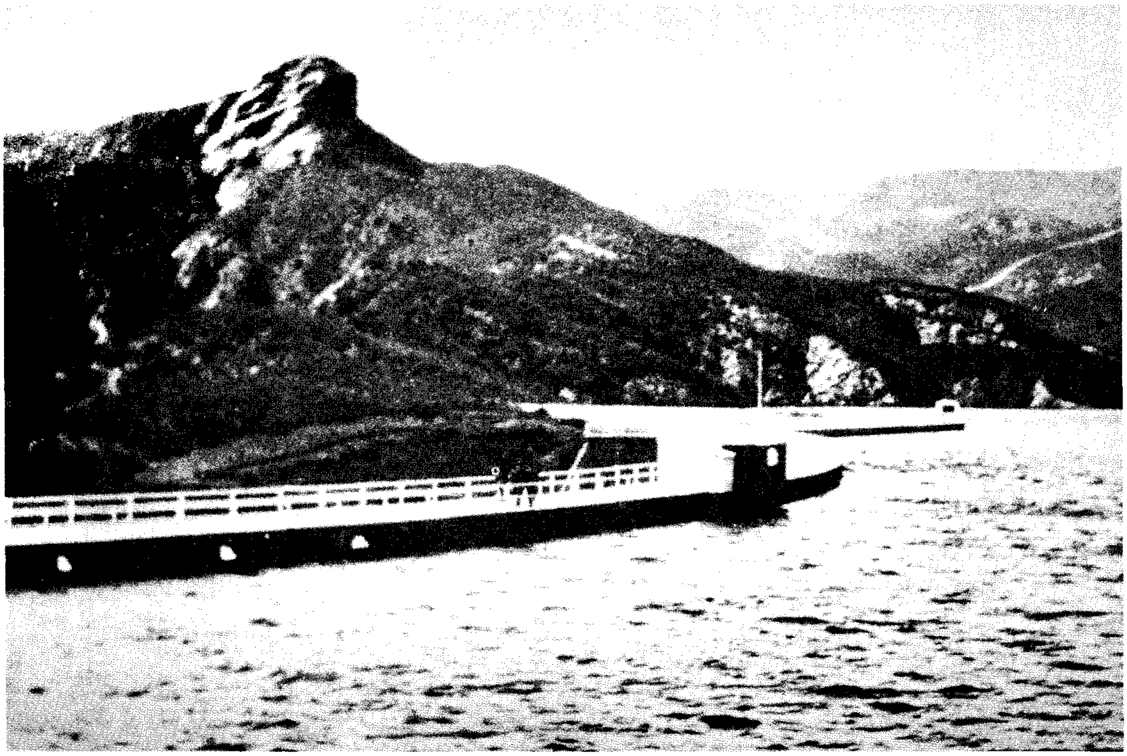


Figure 7. (upper) Bill Mulholland, Harvey Van Norman and damkeeper Tony Harnischfeger walking across the crest of the swollen dam, around noon on March 12, 1928, approximately 12 hours before the failure. The west dike, running across the top of a natural ridge in the Sespe formation red beds, lies at right center

The last person to see the dam and live to tell about it was Ace Hopewell, another Powerhouse 1 employee, who traversed the canyon road by the dam sometime between 11:50 and 11:55 PM. Driving alone on a motorcycle with sidecar, he should have been able to notice if anything significant (such as concentrated, orifice outflow) was occurring, but he did not. He did recall seeing car headlights up-canyon, presumably those of Steen and Span's vehicle, but noticed nothing else that was unusual. About a mile above the dam, Hopewell stopped suddenly, sensing an unusual sound or shaking. He pulled over, but kept the engine running on his motorcycle, smoking a cigarette while listening to the strange crashing sounds about a mile behind him. He then continued up the canyon, reaching Powerhouse 1 and learning of the disaster he had so narrowly missed.

What happened next has been subject to considerable speculation since the time of the failure. A Stevens Water Stage Gage, situated upon the deep, central core of the dam (Figure 17), was retrieved after the failure. An examination of the pencil trace on the recording graph paper indicated that, in the 40 or so minutes prior to the catastrophic failure, the lake level appeared to have lowered about 3.6 inches, in an accelerating manner. The dam was presumed to have failed at 11:57:30 PM, as that was the time the Southern California Edison (SCE) Lancaster power line, running along the east abutment, was suddenly cut. This power line extended the length of San Francisquito Canyon, on a series of poles placed in tandem, and was located about 90 feet above the

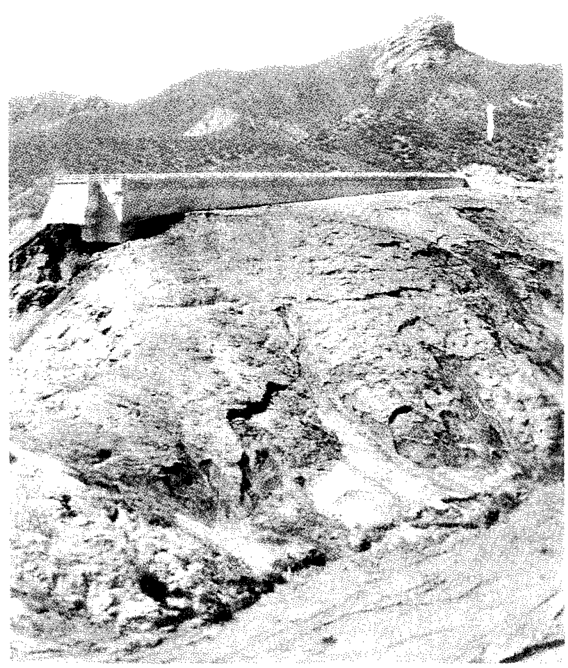


Figure 7. (lower) Approximately the same view, as seen the following day, after the dam failed, leaving the west dike in place. Water had been leaking from beneath this dike the previous day. Note the widespread occurance of shallow earthflows on the Sespe slope, likely due to rapid drawdown-induced pore pressure differential as the reservoir emptied in about an hour (photo from Ventura County Museum of History and Art).

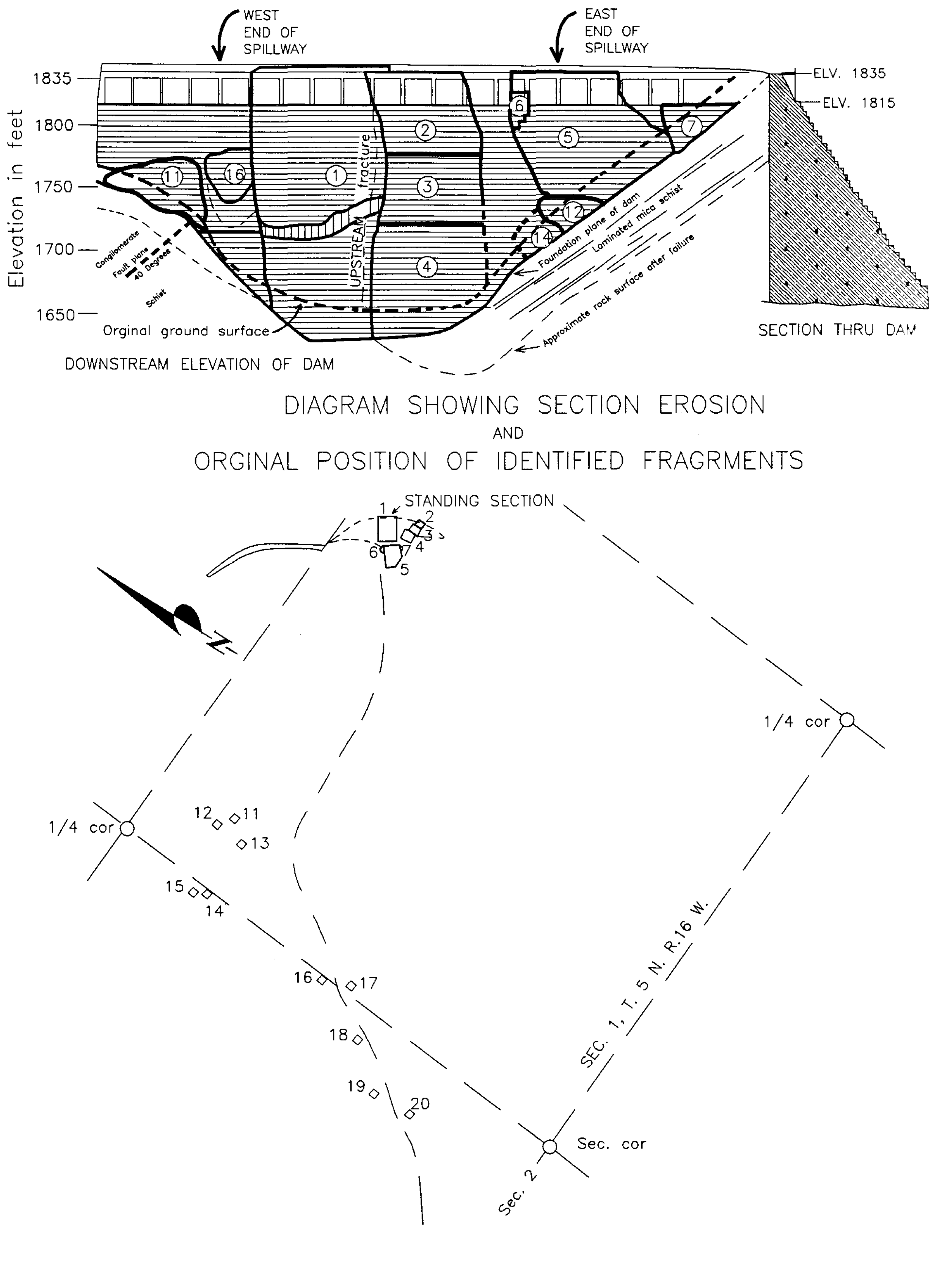


Figure 8. Distribution of fragments from the St. Francis Dam, as delineated in the State report of the disaster, issued 11 days after the failure. Some fragments from the west abutment were found as much as half a mile downstream, thereby influencing the Board's conclusion that the failure initiated on that side of the dam.

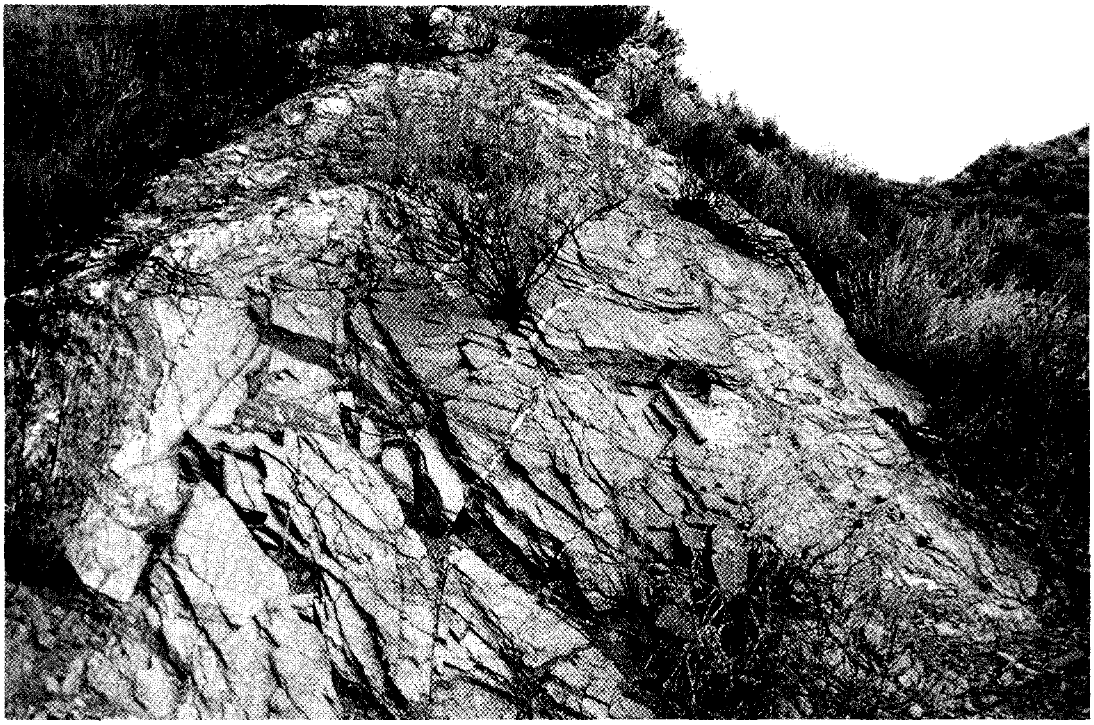


Figure 9. One of the better exposures of the Pelona Schist, seen today in the scar of the 1928 east abutment slide (shown in plan in Figure 15). Planes of foliation form the planar face, with numerous joints cutting the ground mass. A set of parallel shears cuts into the slope, while another, older set of joints is infilled with milky quartz. Discontinuity data from this and another, adjacent series of outcrops, was incorporated into the keyblock anaysis of the east abutment (photo by author).

east abutment crest of the dam (depicted in Figures 10, 13L, and 17).

The Stevens Gage consisted of a 12-inch diameter pipe affixed to the upstream face of the dam (Figure 17). Water fed into this so-called "stilling well" through a 1-inch diameter hole at its lower extremity, with the purported intent of filtering out the oscillatory effects of waves on the back of the dam. In the failure's aftermath, just about everyone attempted to correlate the apparent drop in lake level recorded by the gage with the theoretical outflow that could have lowered the lake by 3.6 inches. Predictably, this turned out to be a considerable quantity of water, beginning with around 1,000 cfs, and building up to over 15,000 cfs, just before the failure; an enormous amount of water in tiny San Francisquito Canyon not to be noticed by the 11:30 PM shift change at Powerhouse 2 or by any of the three aforementioned parties that drove along the creek, leading up to the dam just before the failure. The road connecting the dam to Powerhouse 2 paralleled the creek to a point about 2600 feet downstream of the dam, then climbed the east wall of the canyon, joining the east abutment crest at Elevation 1837. We can catch a glimpse of this road in Figure 26.

KINEMATIC ANALYSIS OF THE EAST ABUTMENT

In attempting to understand the potential failure mechanisms of either abutment, we first need to study the geometry and final positions of the dam's identifiable pieces (Figure 8). These positions can then be combined with LADWP survey data and geologic mapping to enable preliminary rock mechanics analysis to evaluate the abutments (Figure 9). As-built surveys indicate that at the base of the east abutment, a steep "step" was cut into the schist, leaving a slope inclined at approximately 55 degrees (from horizontal). By combining the excavated abutment profile survey (from LADWP files) with outcrop measurements, a computerized keyblock analysis (Goodman and Shi, 1985) was performed on the schist exposed in the east abutment (which was once part of the paleomegaslide). Figures 10 thru 12 present the results of such an analysis. Wedge-shaped keyblocks A, B and C were identified beneath the projection of the dam's original east abutment contact, in the vicinity of the steep cut shown on the construction surveys. The keyblock program serves to identify the geometry of these wedges, not their actual size. The representations made herein only approximate those that likely existed beneath the actual structure (since 40 vertical feet of schist, comprising the east abutment, was washed downstream during the 1928 failure, we are forced to presume that the fabric of the removed mass was similarly configured).

The original position of the dam's east side was then superposed over the identified keyblock wedges, as presented in Figure 11. Figure 12 graphically depicts an apparent imbalance of uplift forces and the dam's effective weight, absent any grouted seepage cutoff (in the style of analysis made

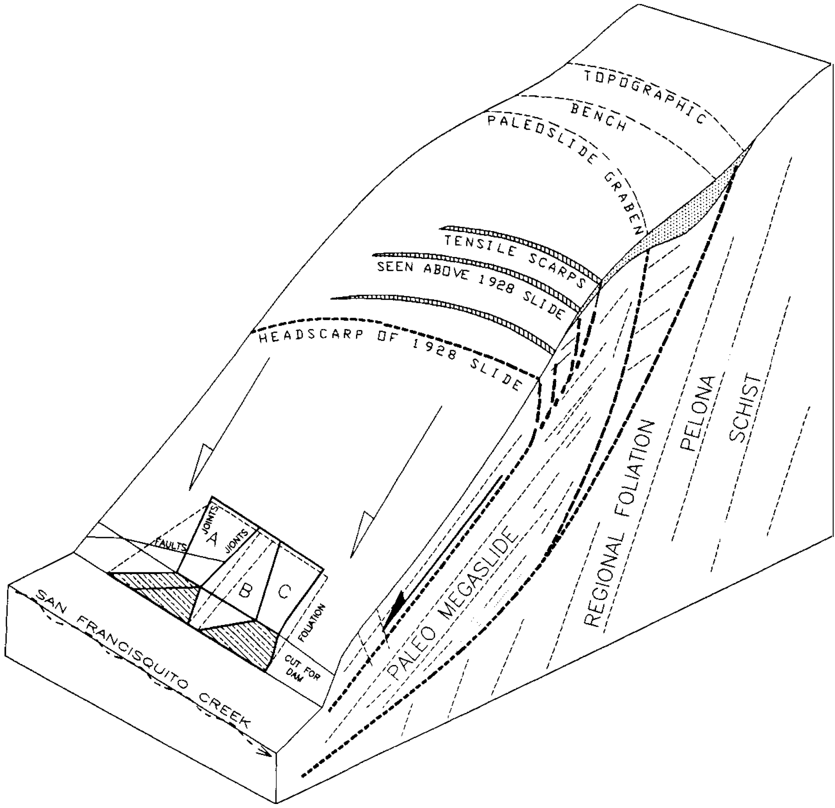


Figure 10. Schematic block diagram view of the St. Francis Dam east abutment, showing rock wedges A, B and C, identified in the keyblock analysis. The blocks are identifed by their geometry, the sizes portrayed here are only for purposes of illustration. Block B would present particularly unfavorable geometry with respect to uplift. The approximate positions of the paleomegaslide and the smaller, 1928 east abutment slide are shown.

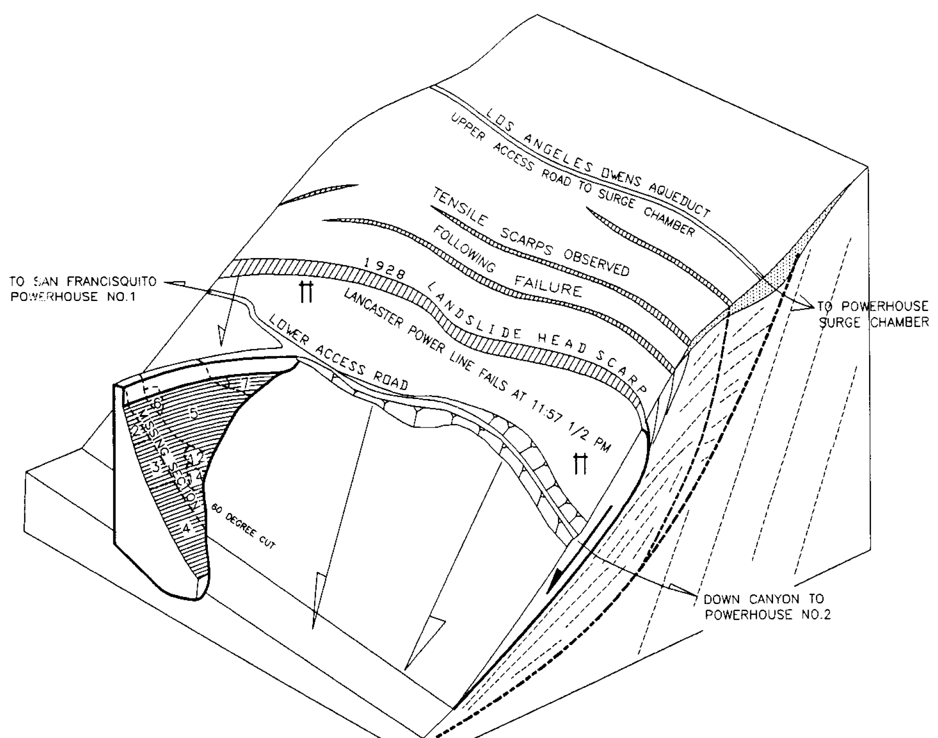


Figure 11. Schematic block diagram of the conditions on the east abutment of St. Francis Dam, just prior to its failure in March 1928. The "missing section" of the east abutment lies upon keyblocks A, B and C, shown in the previous figure, which coincide with the steepest excavation made for the dam. The approximate aereal limits of the east abutment slide are shown, as well as that of the Edison Company power lines that went down at 11:57:30 PM on the evening of the disaster. These lines were situated 90 feet above the dam crest.

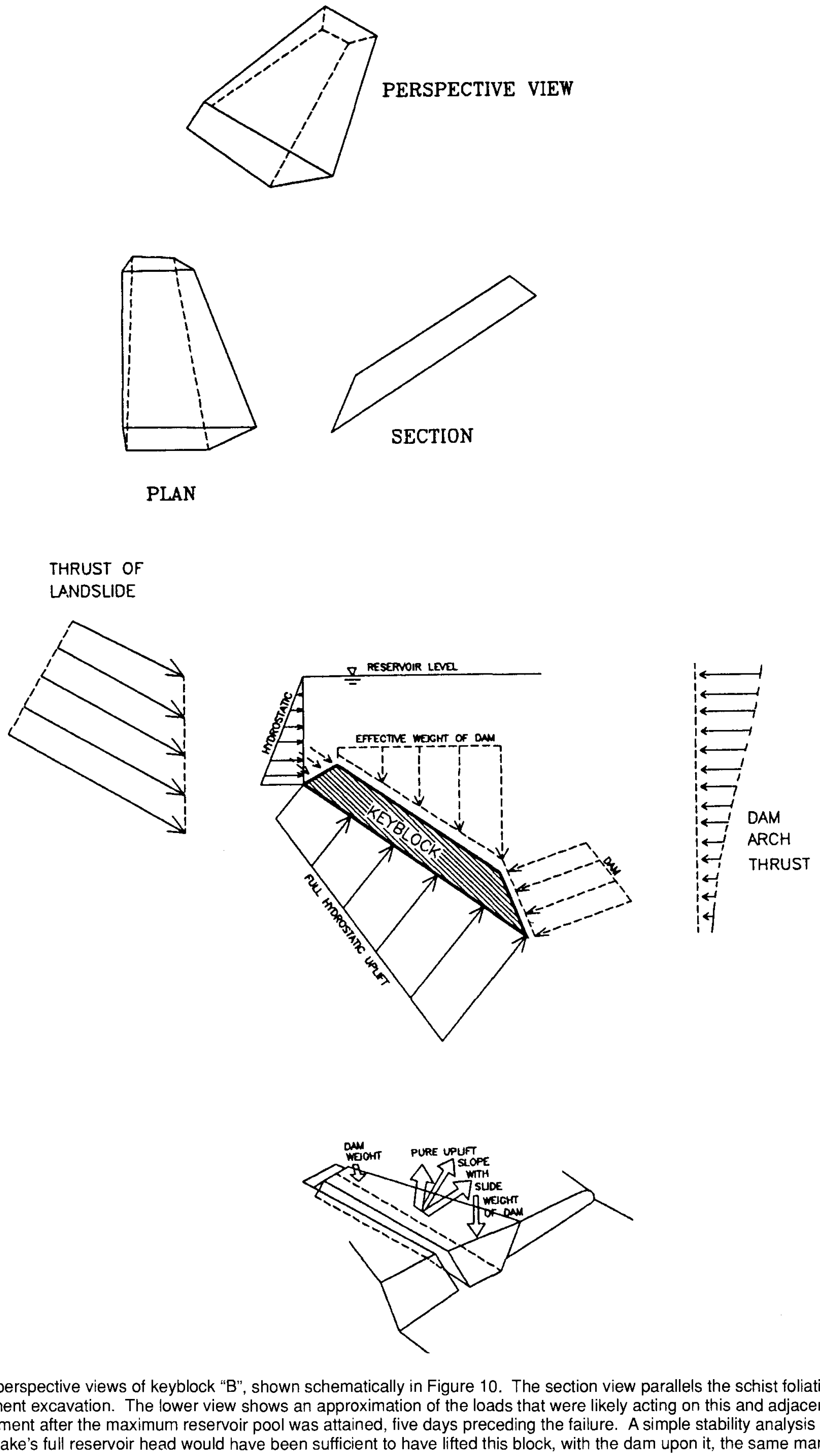


Figure 12. Plan, section and perspective views of keyblock "B", shown schematically in Figure 10. The section view parallels the schist foliation, the lower face that of the abutment excavation. The lower view shows an approximation of the loads that were likely acting on this and adjacent wedges, beneath the east abutment after the maximum reservoir pool was attained, five days preceding the failure. A simple stability analysis suggests that two-thirds of the lake's full reservoir head would have been sufficient to have lifted this block, with the dam upon it, the same manner of failure by which Malpasset Dam later failed in 1959.

by Pacher, 1964; and Londe, 1968). If two-thirds of the full reservoir hydraulic pressure were developed beneath any of the three wedge configurations ("B" being the worst of the three), it can be theoretically demonstrated that the dam abutment would have been lifted (as shown in the lower part of Figure 12), throwing parts of the sloping abutment into tension. Even with only 50% uplift, the normal effective weight of the dam, between Blocks 2/3/4 and 5/6, would appear prone to imminent failure (utilizing the Governor's Board of Inquiry block nomenclature, shown in Figure 8). Though slightly more complex (due to the sheer number of discontinuity suites present), the east abutment failure would appear similar to that experienced (due to a tetrahedrally-shaped wedge) at Malpasset Dam, a concrete arch structure in France, which collapsed catastrophically during initial filling in 1959 (Londe et al.,1968, 1969,1970; Pacher, 1964; Terzaghi, 1962).

There is no reason to openly discount Outland's (1963; p.208) "mystery witness" with regard to incipient motion of the east abutment, some 30 to 40 minutes preceding the failure. Such an observation would be expected, in light of the slide's enormity, seen the following morning (Figures 13 and 14). Realizing that landslide motion on the east abutment, regardless of how slight, preceded the dam's imminent demise, is a critical piece of information in unraveling the failure puzzle. We could reasonably expect that the incipient motion of such a large slide (877,500 tons/500,000 cubic yards of schist) would impose enormous loads on the dam

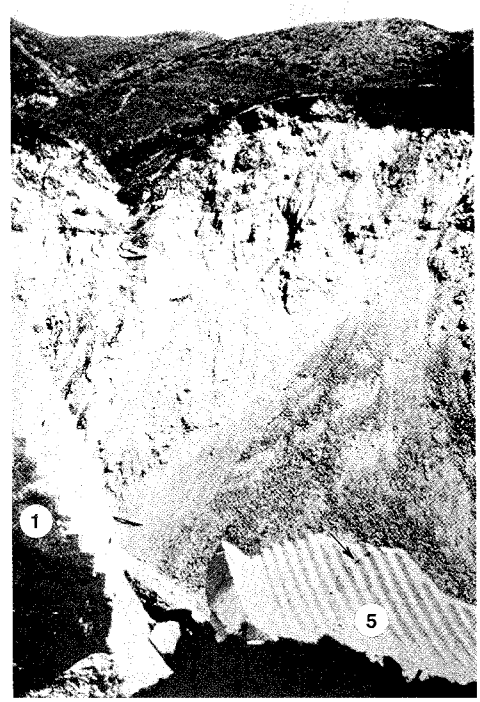


Figure 13 (upper) View looking southeast, at east abutment slide scar, from across canyon, atop the west abutment thrust block. The enormity of the slide can be vividly appreciated, the affected mass involving well over 500,000 cubic yards. The oakum patching of the diagonal crack in Block 5 can be clearly seen (arrow). Blocks 1 and 5 are indicated by their respective numbers

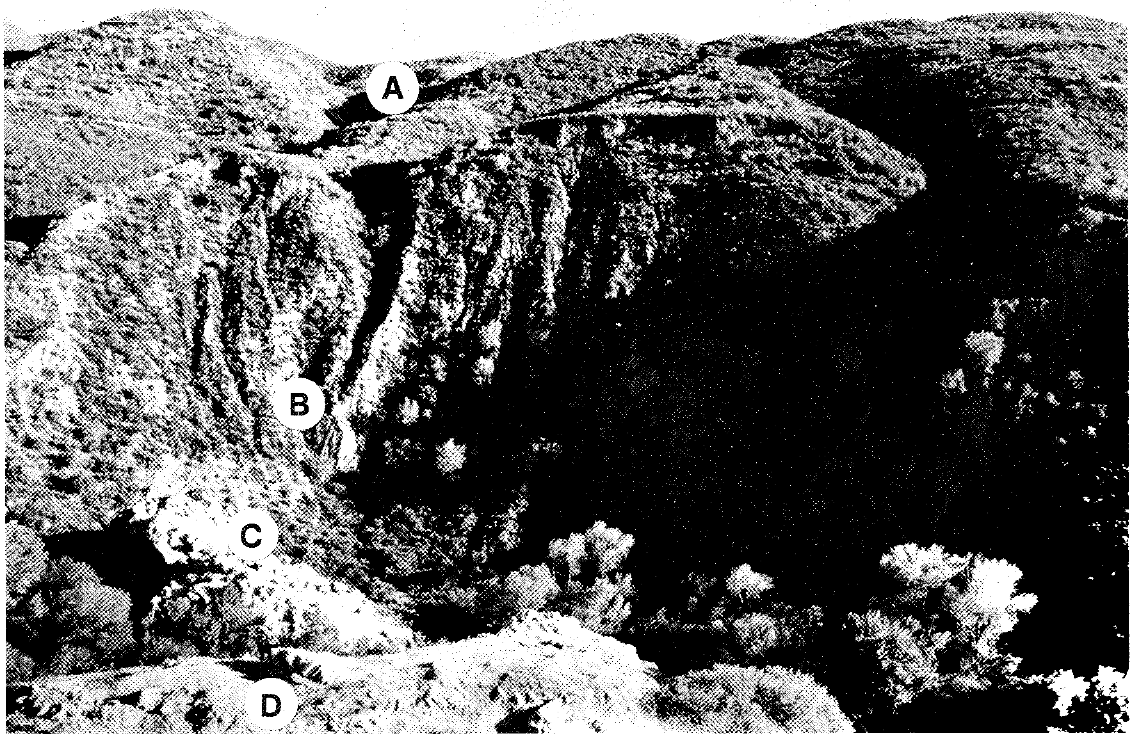


Figure 13. (lower) The same view as seen at the damsite today. "A" indicates the position of the old upper road along the LA Aqueduct (and within the paleoslide headscarp graben); "B" is the outcrop shown in Figure 9; "C" are the remains of dam Blocks 2 and 3; "D" is the remains of Block 1, demolished in 1929.

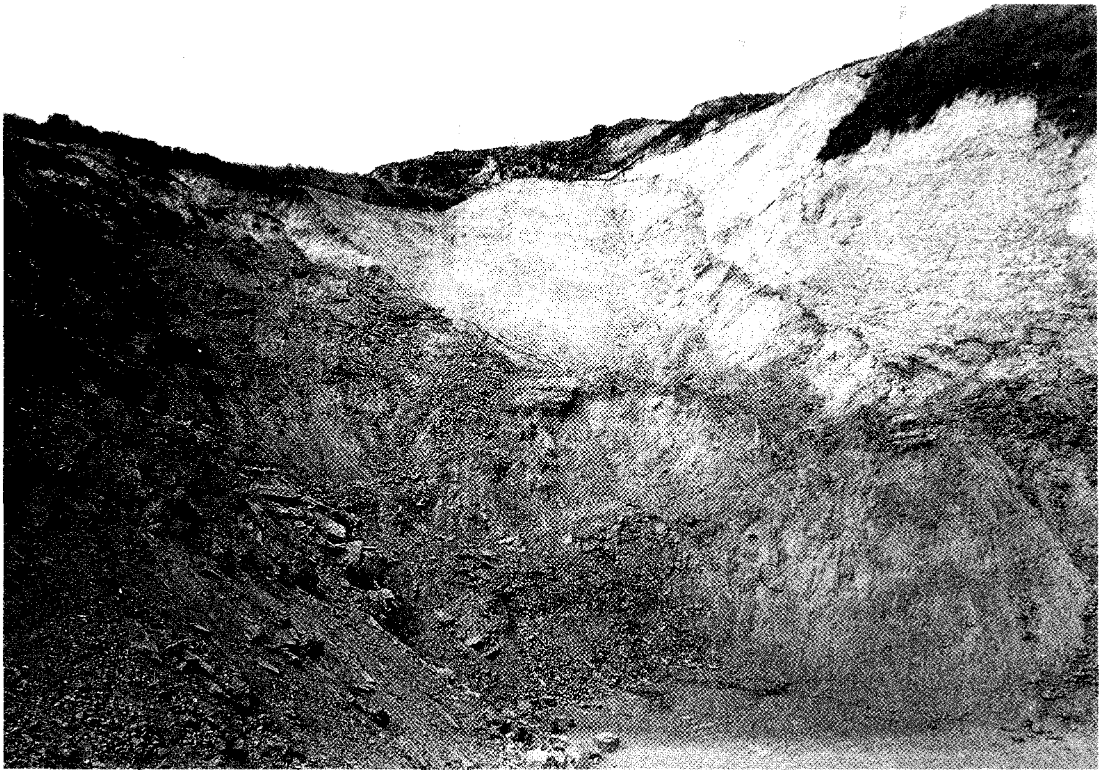


Figure 14. (upper) View looking uphill, into the evacuation scar of the 1928 east abutment slide, as seen from the floor of the canyon, just downstream of the dam's former position. This photo shows the guardrail of San Francisquito Canyon Road, parallel to the slide's west headscarp (thereby fixing its relative vertical position, shown in Figure 15). Photo reproduced by permission of Ventura County Museum of History and Art.

Figure 14. (lower) Post-failure view looking up-canyon from the higher, surge chamber road, above the east abutment slide headscarp (position "A" in lower portion Figure 13), a few days after the failure. Retrogressive tension scarps reached 200 feet uphill of the slide, encraoching on the high road as shown, dropping it by about 10 feet. The Edison Power line that went down at 11:57 PM is shown as "A" and the slide depicted in Figure 1 is delineated as "B".

(251,000 tons dry weight/130,000 cubic yards of concrete), thereby progressively distorting the distribution of stresses within the structure (and negating most of the design's conventional loading assumptions).

Figure 15 presents a present-day topographic plan map of the dam site, prepared by geology students at Cal Poly Pomona in 1980. The former position of St. Francis Dam was overlain on the present-day topography utilizing as-built surveys of the dam (some common points of fixity still exist, such as the base of the thrust block, atop the west abutment). From Figure 15, it can be appreciated that the upstream portion of the slide must have been dropping and thrusting against the lowest portion of the east abutment wall, in the vicinity of keyblocks A, B and C. As mentioned previously, this steeply sloping east abutment had not been provided with any manner of foundation cutoff, drainage, or uplift pressure relief. Given the fact that the eastern abutment was comprised of schist that had been displaced by paleolandsliding, it likely possessed sufficient porosity to develop extremely high pore pressures (a facet discussed in some detail by Willis, 1928).

From the pattern of transverse cracking, it would appear that uplift of the east abutment may have occurred with the first filling of the reservoir in 1926-27. If this were the case, excessively high toe pressures would have been generated at the base of the inclined section, between Blocks 2/3/4 and 5/6.

As the outermost piece of the paleomegaslide continued to creep downslope, it would have loaded the dam obliquely (Figure 15). Such lateral loading, much higher than that imposed by reservoir water, would have brought the upstream face of the dam into tension, induced by the downstream bending of the narrow abutment section (Blocks 5/6/7). The sudden creation of tensile stresses in the heel or upstream face of a brittle concrete dam is a requiem for disaster, as full reservoir hydraulic pressures would enter into such cracks and negate the dam's stabilizing dead load through bouyance (upper half, Figure 27). A simple calculation can demonstrate that, were full reservoir pressures allowed to leak into a transverse crack (or series of cracks within the "missing section", between Blocks 2/3/4 and 5/6/7), the dam's resultant thrust would be deflected far downstream, theoretically causing overturning failure.

Excessive overturning forces, brought about by tilting, could then be expected to initiate high bending stress at the downstream toe of the dam. As deflection continued, the downward concentration of force on the downstream toe usually initiates the failure process through one of two modes: bearing capacity failure of the dam foundation (Fishman, 1979), or induced tensile spalling, bringing about the "cleaving" of the sloping toe Preliminary calculations reveal that either of these initial failure modes would have been expectable results at St. Francis; the concrete tensile strength was fairly low (100 to 130 psi) and the displaced Pelona Schist would have been subjected to something around

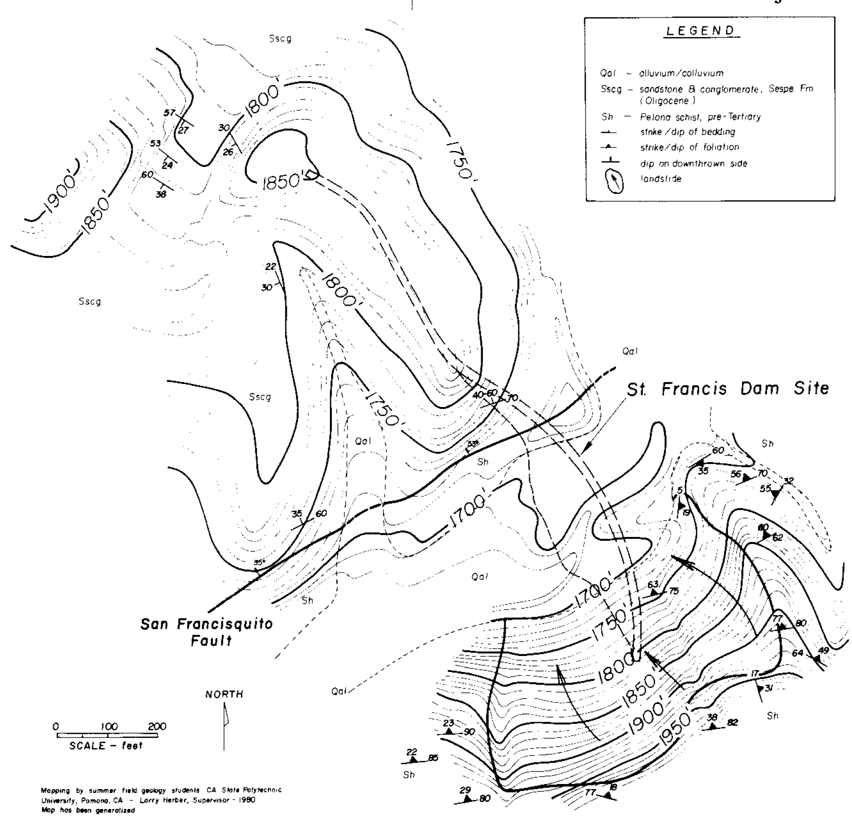


Figure 15. Surface geologic and topographic map of the St. Francis Dam site as it appears today, preparred under the direction of Professor Larry Herber at Cal Poly Pomona in 1980. The positions of the San Fran cisquito fault and the east abutment slide relative to the main section of the dam are easily discerned. Reversal of foliation attitudes in the Pelona schist are likely ascribable to buckling in the toe areas of the paleomegaslide.

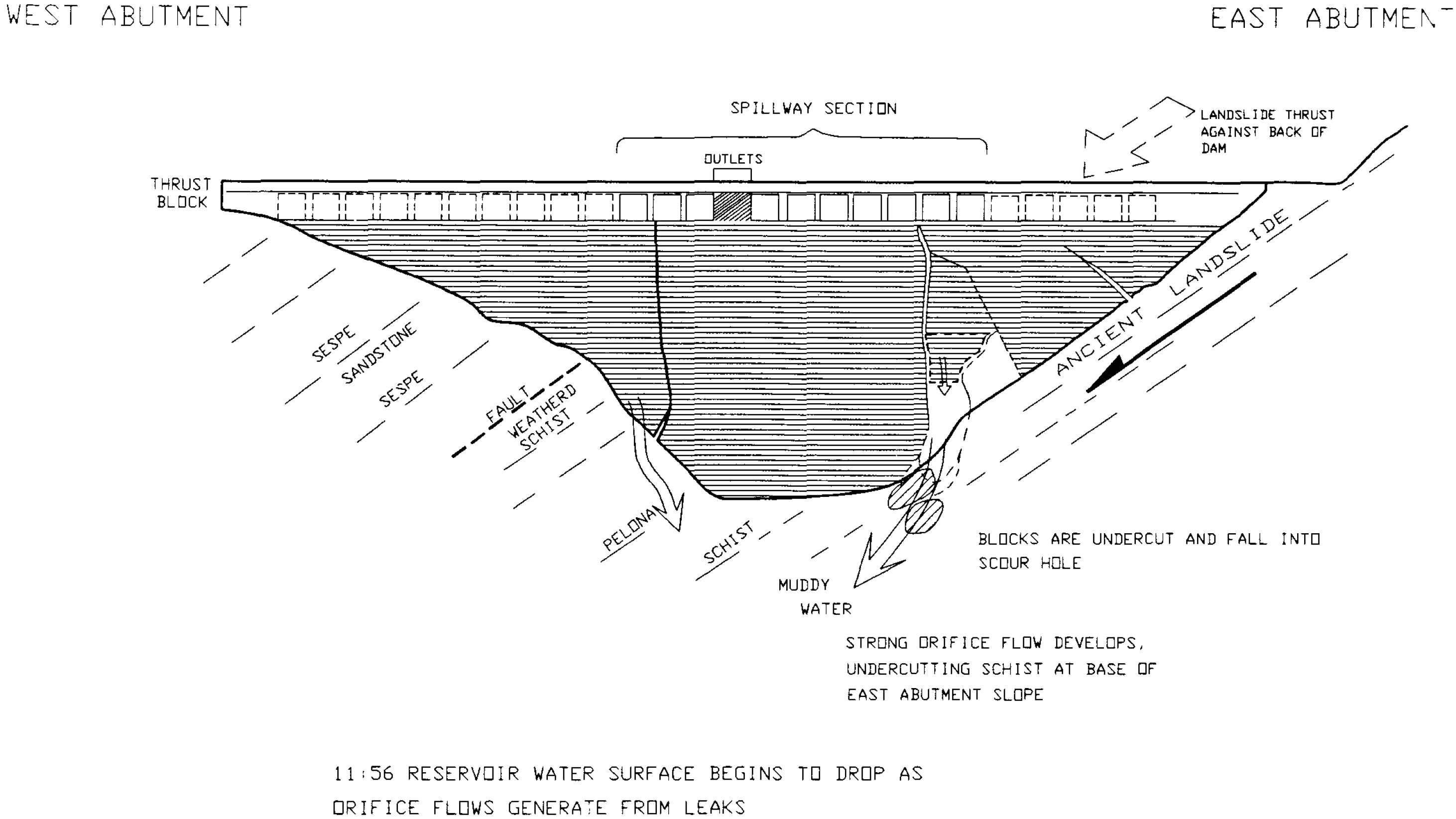


Figure 16. (upper) Schematic elevation view depicting the onset of under cutting beneath the "missing section" of the east abutment slide (see Figures 8 and 11), between Blocks 2/3/4 and 5/6/7. This sequence likely began around 11:50 to 11:55 PM, and would have been hiden from the view of people passing along the road leading to the east abutment crest.

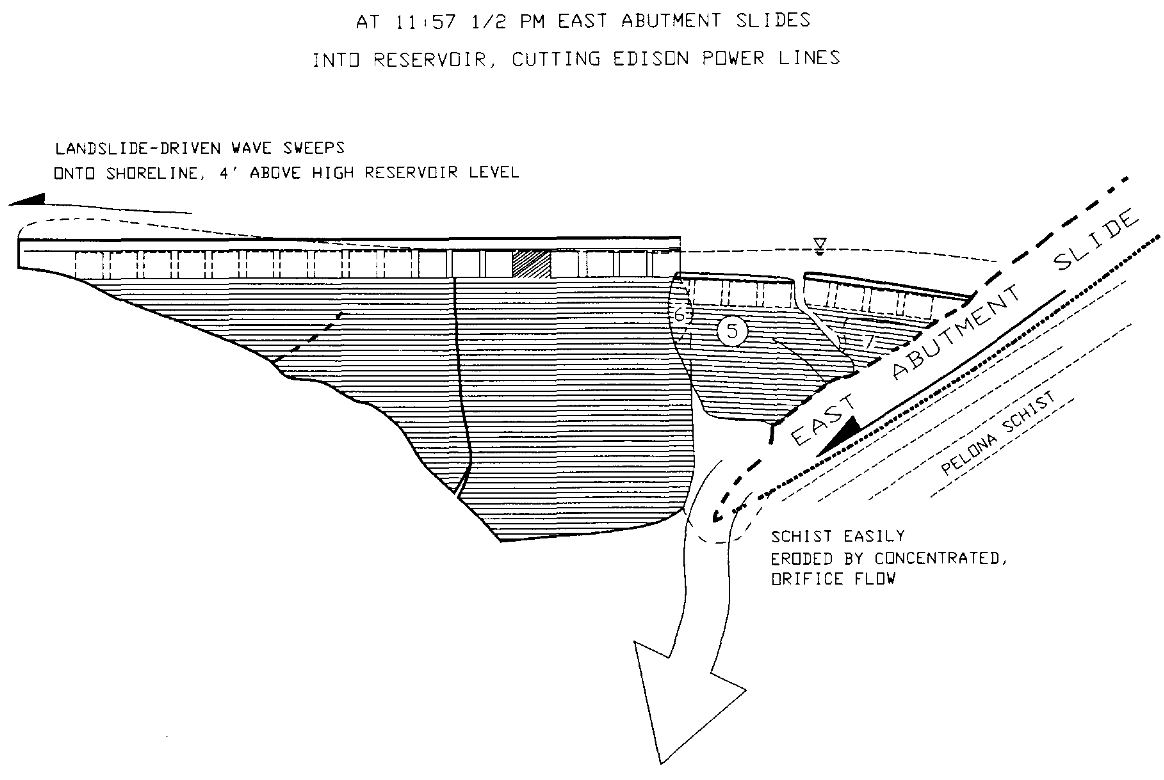


Figure 16. (lower) Schematic portrayal of the initial mobilization of the east abutment slide, around 11:57:30 PM. A sudden mass movement is suggested by several facts, including, but not limited to, the dropping of the Edison power lines some 90 feet above the dam crest at this instant (arguing against a toe-headward, progressive style of failure); the creation of wave floatsom 4 feet above the maximum reservoir level; and the apparent movement of Blocks 5/6/7 completely across the downstream face of the dam.

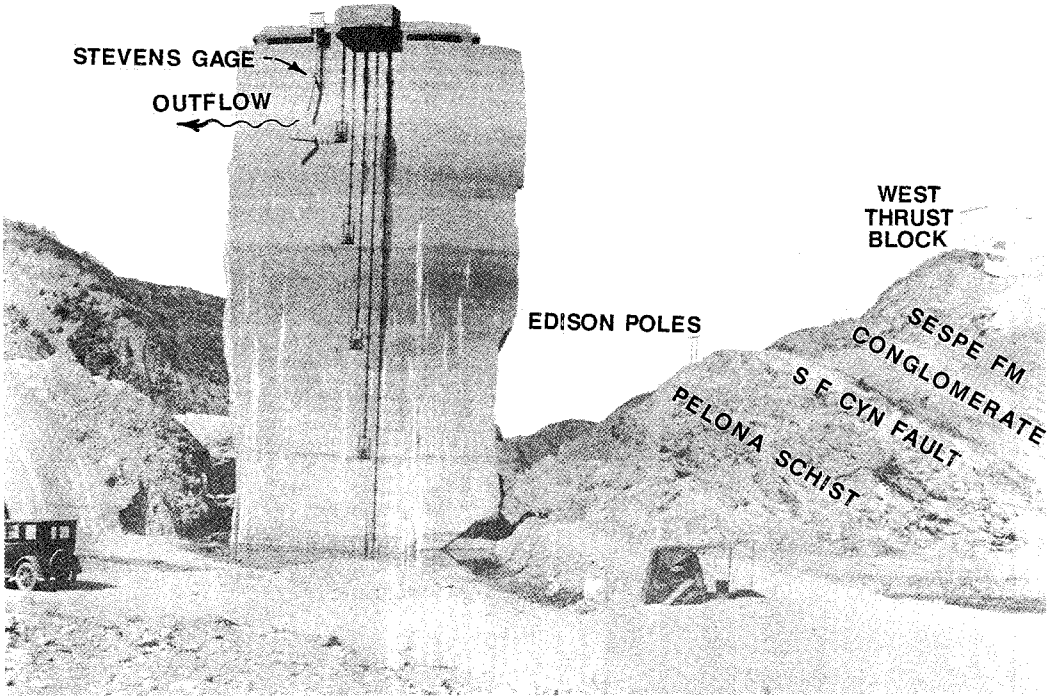


Figure 17. Post-failure view looking downsteam, at the upstream face of Block 1, the only remaining section of the dam. The thin black lines running vertically down the back of the dam are the gate controls for the dam's five outlet conduits (shown in Figure 3). The snapped portion of the 12-inch diameter housing for the Stevens Water Stage Gage is clearly seen to the left of the uppermost outlet gate, pointing towards the east abutment, between elevation 1800 and 1815 (arrow). The lake had been at elevation 1834.4 just prior to the failure. Photo used by permission of the Ventura County Museum of History and Art.

30,000 psf toe pressure, far above what we could expect it to accept without ample distortion, due to its accurate anisotropy and the fact that it must have dilated during translation by sliding.

If the dam's failure initiated in the lowermost east abutment, we could expect full hydrostatic pressures would then be exerted on the opposing faces of any transverse cracks, reaching the upstream face of the dam. Once this occurred, arching action towards the east abutment would have ceased. This loss of arching would then increase the cantilever loads on the remaining section, likely causing it to deflect excessively, lift and tilt. Such a scenario offers an alternative hypothesis to explain the 3.6 inch shift in maximum pool level faithfully recorded by the Stevens Gage: the dam could have been lifting, as opposed to the reservoir dropping just prior to the failure. Grunsky (1928) had been the only engineer to assert such a possibility in the days following the failure.

If reservoir water began to escape from either of the dam's lower abutments, its erosive effects would have been most acutely felt against the moving mountain of schist (as none of this material remained on the canyon walls following the failure). Orifice flow, emanating from an ever-enlarging breech would be expected, as depicted schematically in Figure 16 upper. The "missing section" of east abutment, between Blocks 2/3/4 and 5/6, can theoretically account for up to 400,000 cfs of escaping water with a near- full reservoir head (Figure 16 lower). But, if a hole that large were developing, it is almost certain that the unrestrained abutment (Blocks 5/6/7) would have been moving into such a void. It is reasonable to assume that some significant amount of concentrated, orifice flow must have preceded the catastrophic failure in order to create the "misty haze" that suddenly descended upon the Powerhouse 2 community near midnight, awakening Lillian Curtis and her husband (as discussed later).

Regardless of just how large a hole developed, a massive chain reaction failure must have ensued shortly after a substantive portion of the "missing section" (between Blocks 2/3/4 and 5/6/7) was removed (Figure 16 lower). The apparent outpouring of water thru the "missing section" (Figures 8 and 11) must have undercut the incipient slide mass; thereby setting about its apparent catastrophic failure (Figure 16). Blocks 12 and 14 were later found a half mile downstream (Figures 8 and 16 lower), about 20 feet above the channel level, and offset 50 yards to the outside of the first bend in the San Francisquito channel (Figure 8).

We are left to deduce that catastrophic sliding of the dam's east abutment occurred before the reservoir had experienced many appreciable drainage for six key reasons:

1. The morning after the failure, flotsam and debris was observed along the reservoir shoreline at a level 4 feet higher than the maximum level of the reservoir pool;

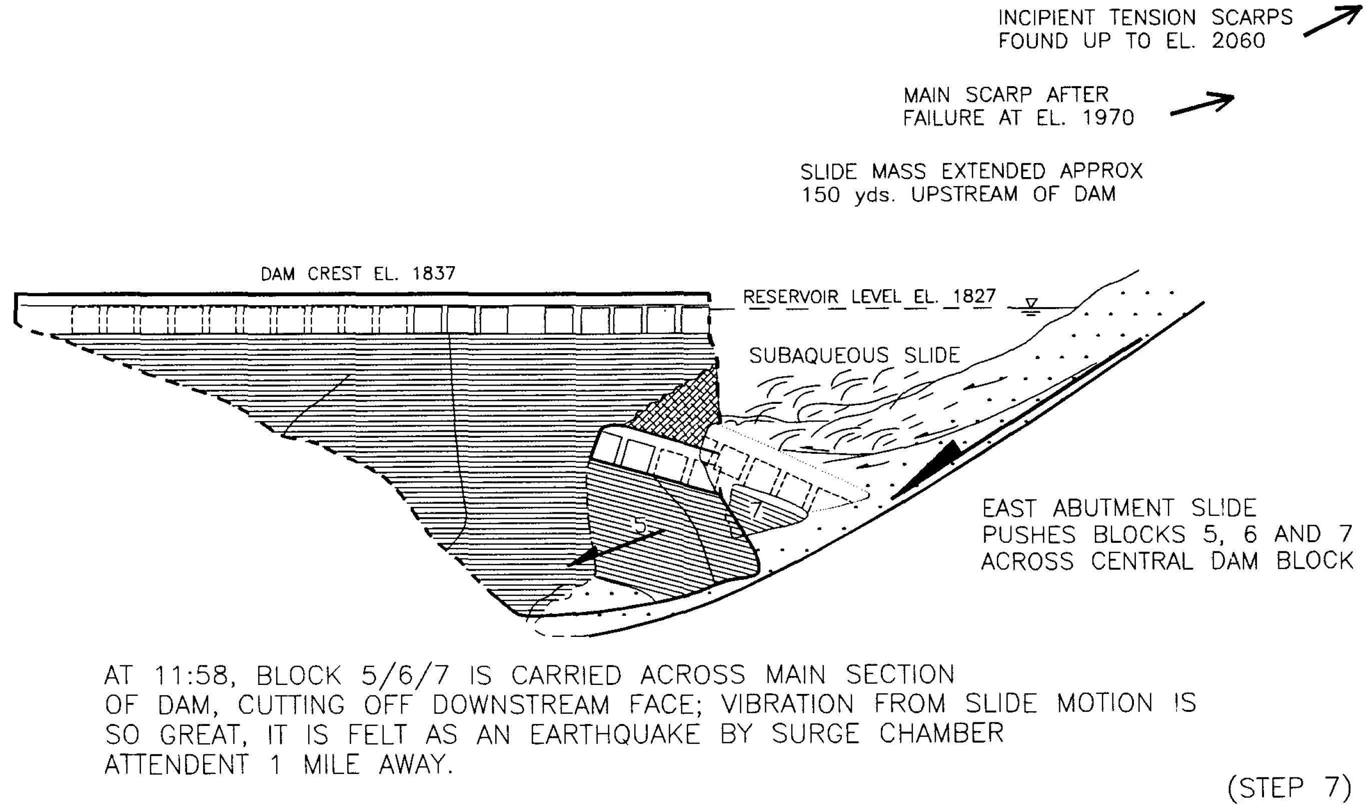


Figure 18. (upper) Schematic portrayal of the east abutment slide carrying Block 5/6/7 across the downstream face of the dam's central section (Blocks 1/2/3/4) early in the failure sequence. At this time, the lake level had likely only dropped 3 to 7 feet, to around elevation 1827. The outer 20 feet of the dam's face was abraided by the movement of Blocks 5/6/7.

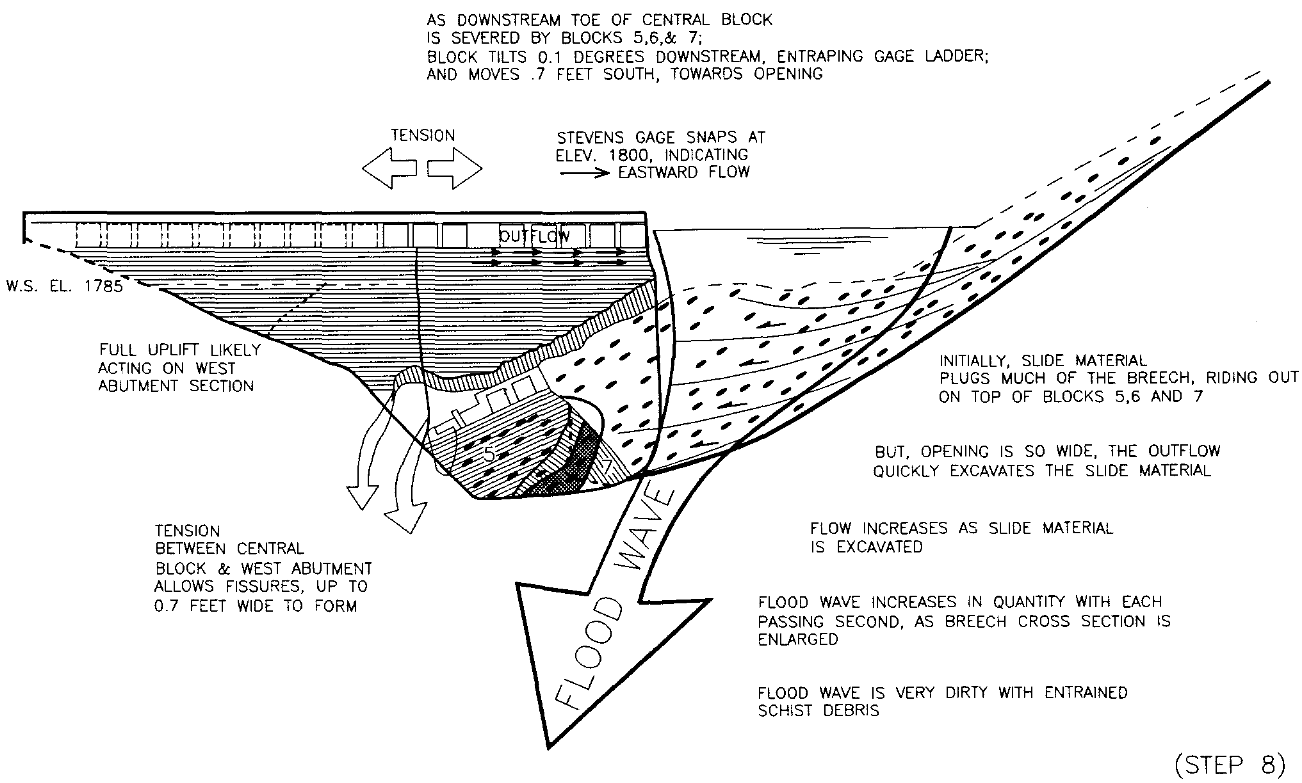


Figure 18. (lower) Schematic portrayal of the landslide dam, temporarily formed across the east abutment breech. Debris from this translational slide probably overthrust upon itself, building up a short-lived impedence to outflow. The formation of a temporary dam with increasing discharge accounts for the 30 to 90 seconds of precursory outflow related by survivors at Powerhouse 2 before the maximum discharge arrived, killing 120.

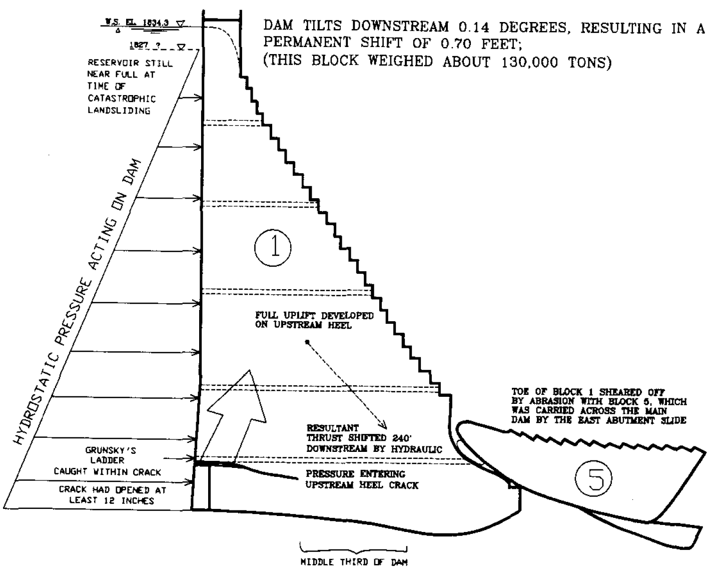


Figure 19. Loading conditions likely exerted upon Block 1, based on physical evidence observed immediately following the failure. The snapping of the Stevens Gage around elevation 1815 suggest that the reservoir was still full when maximum outflow conditions were developing through the east breech. When Block 5 sheared off the toe of Block 1, the dam's resultant thrust would have been shifted downstream of the structure, leaving the upstream heel in tension. The heel crack opened up at least 12 inches, allowing full hydrostatic pressures to develop under this section. The uplift wells placed beneath the dam's main section likely saved it from disaster. The sloping abutments were not afforded any such protection.

meaning that a sizable wave had been generated while the lake was still at a relatively high level;

2. The Stevens Gage stilling well pipe, affixed to the upstream face of Block 1 (on the west side of the dam's central core), was found to have been buckled between elevation 1800 and 1815, towards the east abutment (Figure 17); indicating that a massive outpouring was occurring to the east side of the central dam block when the lake was still high.

3. Blocks 5/6 and 7, comprising the majority of the east abutment, were carried across the entire main dam section, shearing off approximately 20 feet of the downstream toe of this main section (Figures 18 - 21). Enormous lateral forces had to have been acting on the dam when this occurred, in order to overcome the hydrostatic pressures of the reservoir (or its remnants), which would have been pushing normal to the downslope motion of these blocks. The fact that piles of schist were found atop Blocks 5 and 7, some 40 feet above the creekbed (Figure 20), also indicates that they were subsequently covered with schist detritus;

4. Lillian Curtis and Ray Rising, the only adult survivors of a population of 123 DWP employees/family members at Powerhouse 2, were both awakened by extra-normal sounds of high water discharge and an unusual "foggy haze" that had settled over their community. Curtis had sufficient warning to get out of her home with her 2-year old son, before the water reached its maximum level of 110 feet, whipping away the Powerhouse 2 community at 12:02:30 AM. She described the flood water as a sort of "liquid mud", indicating an extremely high suspended bedload in the initial flood wave. That this initial event was sweeping and occurred early on in the failure sequence is collaborated by E.H. Thomas, the surge chamber attendant atop the mesa above Powerhouse 2, who upon hearing and feeling what seemed to him like seismic tremors, ran down the canyon slope to see what had happened. He reached the high water line at 12:15 AM, only to find that the water had already receded 20 vertical feet.

5. There simply was not sufficient time for an appreciable lowering of the reservoir between the earliest time Ace Hopewell drove by (11:50 PM) and when the Edison power lines went down, on the east abutment slide at precisely 11:57:30 PM. The fact that these lines were located almost 100 feet above the crest level of the east abutment seems to have been overlooked in the post-failure assessments (Figures 11 and 14 lower).

6. The fixed times of known events suggests that the failure sequence was rapid and catastrophic, arguing against a conventional, piping style of failure. The dam looked fine shortly before the failure, between 11:50 and 11:55 PM (Hopewell); failed at 11:57:30 (downed Edison lines on east abutment); destroyed the Powerhouse 2 area at 12:02:30 (Curtis); and had already receded 20 feet by 12:15 AM (Thomas). The reservoir was emptied within an hour (when first reached by DWP employees Spainhower and Lindstrum at 1:09 AM). All the accounts are consistent with respect to time, but spaced very closely.

The upper and lower halves of Figure 18 present what likely occurred, given the verifiable parts of the puzzle (both physical evidence and eyewitness accounts). After having its toe rapidly undercut and eroded by strong orifice flow (which would produce a misty "fog"), the toe of the east abutment slide was severely undercut (Figure 16). Such undercutting should have been sufficient to trigger the east abutment landslide, which was already in a stage of incipient failure if the road had indeed dropped 12 inches by the evening of March 12th. The east abutment slide was a simple translational failure, along slippery planes of foliation in the mica schist. We can infer that the slide must have began its descent around 11:57:30, dropping the Edison Company Lancaster power lines. Depending on the speed at which the slide dropped, the moving mountain of schist should have infilled the tremendous void, at least temporarily (lower half. Figure 18). An alternative hypothesis could have the toe of the east abutment slide gradually being eaten away by the outpouring water, but such a scenario would then be unable to account for the very long trip taken by Block 5; which was shifted all the way over to the opposing abutment. We are left to conclude that the translation of Blocks 5/6/7 to the opposing side of the channel, below Block 1, argues for a rapid, dramatic slide motion, so large that it presented an inexorable force; temporarily greater than that posed by the force of the outpouring reservoir water. Mass balance assessments comparing the effective force of the slide, a saturated dam (Figure 27) and the reservoir water support such a premise.

In addition, if the east abutment blocks came down slowly, the outpouring water should have been capable of translating them some distance downstream (Figure 18). Years after the failure, journalist Outland (1963) had been the only person to correctly deduced this key part of the puzzle. The tremendous "grinding motion" of the massive concrete blocks (which sheared off a 20 foot thick section of the downstream face) was the likely source of the "seismic tremor" described by Ace Hopewell and surge chamber attendant Thomas and his mother, all atop the schist ridgeline. The block kinematics involved in such a massive translation can be better appreciated when viewing the dam in plan, as shown in Figures 4 and 15.

As Block 5 was carried upon the translating landslide, it sheared off the downstream face of Blocks 2/3/4 and Block 1, as depicted in Figures 19 and 20. By removing the downstream toe of the dam's central block, the resultant thrust would have been projected even further downstream, thereby leaving the upstream heel in tension (as the dam tried to overturn). Post-failure assessment by Grunsky (1928) showed that Block 1 had separated at its upstream heel, between the masonry cut-off wall and the dam proper (Figure 19). Such tilting would have progressively worsened, due to the development of full hydrostatic uplift pressures, within the crack. This must have occurred, as the gage ladder was found swallowed into the crack at the base of the remaining section, suggesting that the dam MUST have tilted at least 12 inches to allow complete access into this crack (Figures 17 and 22).

The approximately 500,000 yards of schist landslide debris would have been taken into the massive void created by the removal of the entire east abutment. Temporarily, the breech would have been partially blocked by the sheer volume of slide debris (as depicted schematically in the upper-half of Figure 23). The outpouring waters of the reservoir, some 200 feet deep, could be expected to rapidly excavate this temporary "landslide dam", issuing forth a progressively larger discharge, as the schist was quickly removed by the outpouring water. Short-lived, but dramatic increases in overflow discharge typify the breaching of erodable landslide dams (Lee and Duncan, 1975). Peak flows observed from landslide dams have been the largest recorded channel flows in modern time (Schuster, 1985).

The massive landslide failure sequence likely took only 30 to 90 seconds, just enough time for the few survivors at Powerhouse 2 to realize something was amiss and gain a brief chance to escape (Mrs. Curtis' husband had awakened her, pushed her out the uphill window, and told her to "run for

Figure 20. The remains of Blocks 5 and 6, as seen immediately after the failure. The two lowermost steps of Block 1 appear to have been left in tact at the west end. Note the accumlation of schist detritus upon Block 5, about 20 to 30 feet above the creek bed.

the hills" with their two year old son. He then went into an adjacent bedroom to retrieve their two older daughters, but did not escape; so there was precious little time to react). Mrs. Curtis and other survivors of the flood are consistent in describing the maximum wave front as consisting of "liquid mud", likely resulting from the 500,000 cubic yards of weathered schist that was rapidly being scoured and picked up in the east abutment breech. In the flood's aftermath, the Los Angeles County Coroner (1928) would testify as to the large amounts of silt found in the victim's bodies, attesting to their drowning in extremely turbid water. Based on a hydraulic assessment of the flood wave scour depth and channel cross section, the maximum flow quantity appears to have been approximately 1.7 million cfs, a much larger number than that supposed by the various boards of inquiry, as the mechanics of critical flow were little understood at the time. The height of the flood wave at Powerhouse No. 2, 7000 feet downstream of the dam, was later measured at 110 feet above the floor of the Canyon (the maximum water level of the dam being only 179 feet above Canyon floor). It would appear, therefore, that the reservoir emptied very quickly by excavating a "mega-breech" through the slide debris (Figure 23 upper). As the eastern side of the dam was torn away by the massive landslide, there would have been a marked relaxation of arch stresses to that side of the dam. The main dam section could be expected to relax slightly, towards the gap. This relaxation would have allowed the pre-existing transverse crack, bordering the right (west) side of Block 1 (depicted in Figure 5), to open itself up, thereby subjecting the fissure to full reservoir hydraulic pressure. The west abutment was now beginning to break up, for the same reasons experienced on the east side.

As the great quantity of water issued forth from the massive east abutment breech, the central core of the dam was being severely undercut by back-eddy flow (Figure 23 upper). We know from post-failure surveys that a hole 60 feet deep was cut beneath the dam's maximum section, undermining Block 2/3/4. As this occurred, Blocks 1 and 2/3/4 must have tilted towards the undercut/void, or towards the south. Post-failure surveys showed that Block 1 had moved 5.5 inches towards the east abutment and 6 inches downstream (a total of 0.70 feet on a bearing of south 3 degrees west). Such motion would release even all constraint on the west abutment (above the fault), thereby leaving it to absorb the remaining reservoir load as a free-standing gravity section.

As the central core of the dam moved towards the east abutment breech, the stability of the west abutment must have quickly deteriorated, likely initiating failure beneath Block 16 (Figure 25), in vicinity of the fault contact. We can deduce that the west abutment failed after the east side for two key reasons:

1. The Stevens Gage was pulled towards the east abutment at when the reservoir pool was at an elevation of between 1800 and 1815 feet (as seen in Figure 17); and
2. Post-failure photos clearly show that the west abutment construction road was not damaged above the 1785 elevation; despite being comprised of erodable sidecast fill from the Sespe formation (see photos in Figure 24). Undisturbed brush is clearly seen above this level in all the post-failure photos.

The west abutment was probably removed in rapid fashion in a chain-reaction sequence of progressive undercutting. Blocks 11 and 16 (shown in Figure 8), the two largest found more than a half mile downstream (Figure 25), were probably carried that distance by the maximum flow emanating from the dam, which could only have occurred after a substantial removal of the slide debris (in order to create a large opening, akin to the failure of a natural landslide dam). This brief passage of time allowed the reservoir to drain to a level between Elevation 1800 (the bottom of the weakened Stevens Gage casing) and Elevation 1785 (the top of the scour line on the west abutment).

The glaring separation of the west abutment pieces far downstream (Figures 8 and 25) could be ascribable to their smaller relative size (when compared to the blocks on the east side), and the fact that they were probably loosened at about the same time as the schist landslide debris was being excavated from the eastern breech. The tremendous amount of sediment entrained in the reservoir outflow would cause the discharge to have been of much greater relative density, than a comparable discharge of clean water. The relative submerged, or effective weight, of the concrete blocks would be directly proportional to the density of the fluid they displace. In this manner, such enormous blocks might weigh only a fifth of their dry weight, as shown diagrammatically in Figure 27. Back-analyses utilizing conventional, "clear water channel hydraulics", would not replicate the eyewitness descriptions of "moving mud" and the simple fact that 500,000 cubic yards of disintegrated Pelona Schist quickly vanished from the scene.

Towards the end of the failure sequence, Block 2/3/4 fell into the scour hole excavated beneath it by the massive east breech (Figure 21). This hole was some 60 feet deep, according to the LADWP post-failure surveys! As the east side of the dam's central core toppled over, it separated along near-horizontal cold/pour joints, markedly observed in the pieces remaining on site today. This final sequence is sketched in the lower half of Figure 23, and likely occurred when the reservoir had emptied to a level of 40 to 60 feet above the creek bed (based on the schist detritus left on Blocks 4 and 5, Figure 21).

POST-FAILURE NOTES

In the failure's aftermath, surveys by the City's Bureau of Water showed the enormity of the east abutment slide, as well as numerous shallow slides induced by rapid drawdown (Figures 1 and 7 lower). Precise post-failure surveys showed that the standing section (Block 1) had been permanently displaced 0.70 feet (Figure 26). These same surveys also found that the west wing dike (Figure 7 lower) had lifted 0.10 to 0.30 feet, presumably due to swell of the Sespe beds. This swelling may partly account for the many leaks observed in that area prior to the failure. However, nobody ever tried to tie the wing dike leaks to the collapse of the main dam.

A new road had to be constructed through San Francisquito Canyon as the east abutment slide had forever erased the old route. The new road was built shortly after the failure, cutting through what had been the west abutment (Figure 15). Over the past few years the City has constructed a new highway alignment which bypasses the dam site, its' cuts slashing through the former position of the overflow dike, approximately 1200 feet northwest of the dam (also shown on Figure 15). Were it not for engineering geology classes, the memory of California's greatest man-made disaster will simply lie as a footnote in the annals of the State's civil engineering history.

CONCLUSIONS

Many lessons can still be gleaned from the St. Francis failure. Foremost of these is the critical importance of soliciting sound engineering geologic input from more than one

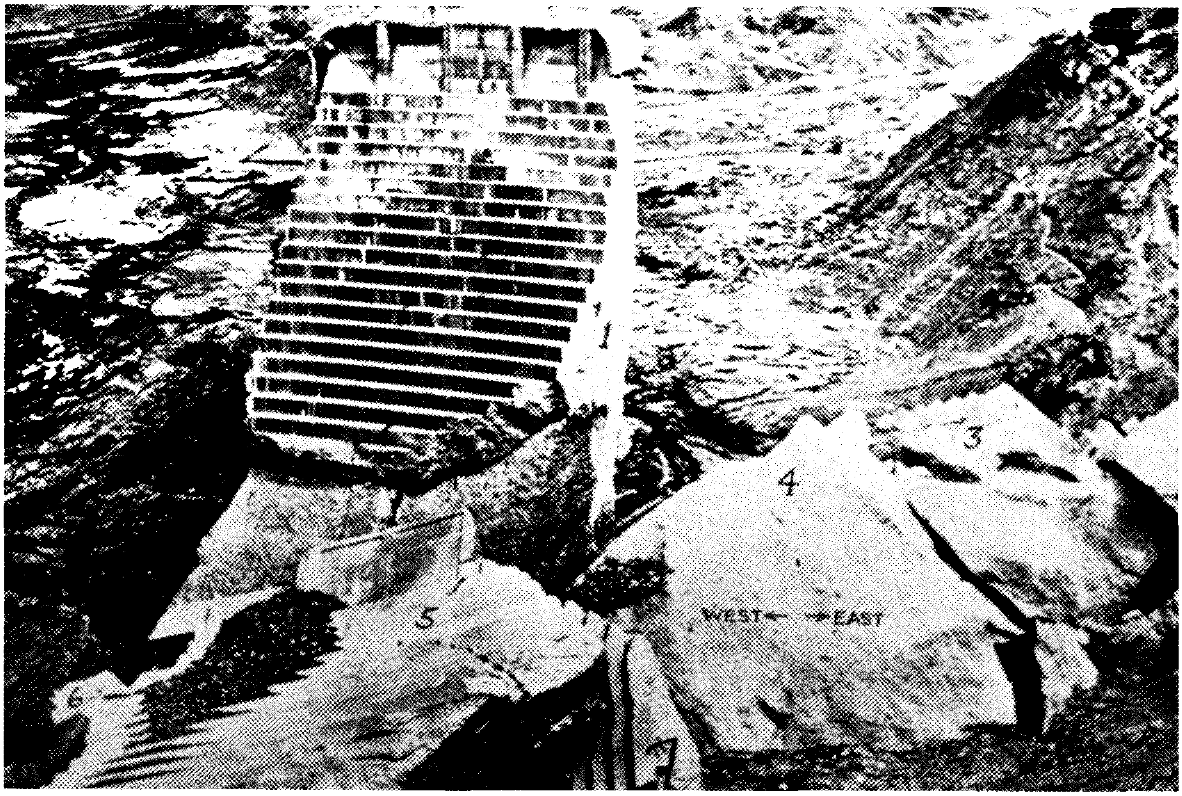


Figure 21. Photo looking upstream, showing the final resting positions of the east abutment and main section blocks. Note the schist detritus left upon Blocks 4 and 5. The oakum patch in Block 5 is also very apparent. Blocks 2/3/4 separated very late in the failure sequence, when much less water was available to disloge them (they toppled upstream).

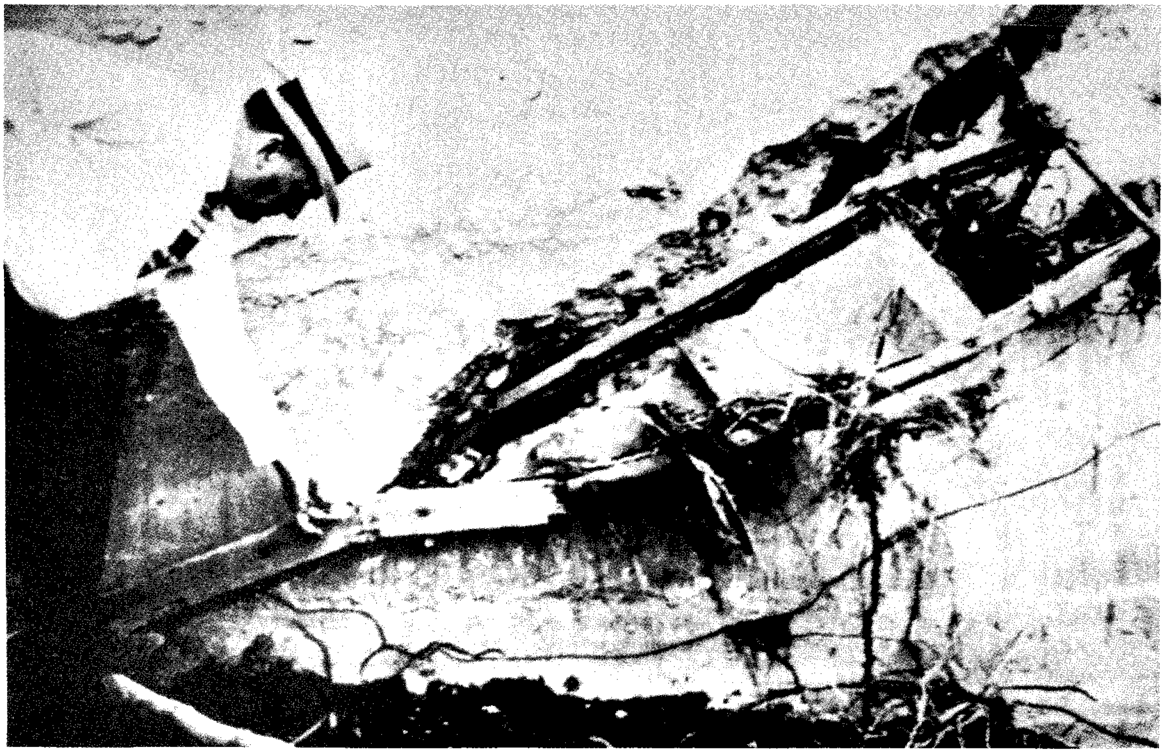


Figure 22. Grunsky's (1928) photo showing the upstream gage attendant's ladder wedged into a large crack at the west upstream heel of Block 1. Block 1 would have to have tilted at least 12 inches to swallow up the lad der, which was 18 inches wide. There is little doubt, therefore, that the upstream heel of the dam's central section was in tension and that, at some point, either before or during the failure sequence, full hydrostatic pressures must have acted upon the block's interior, significantly negat ing its effective dead load

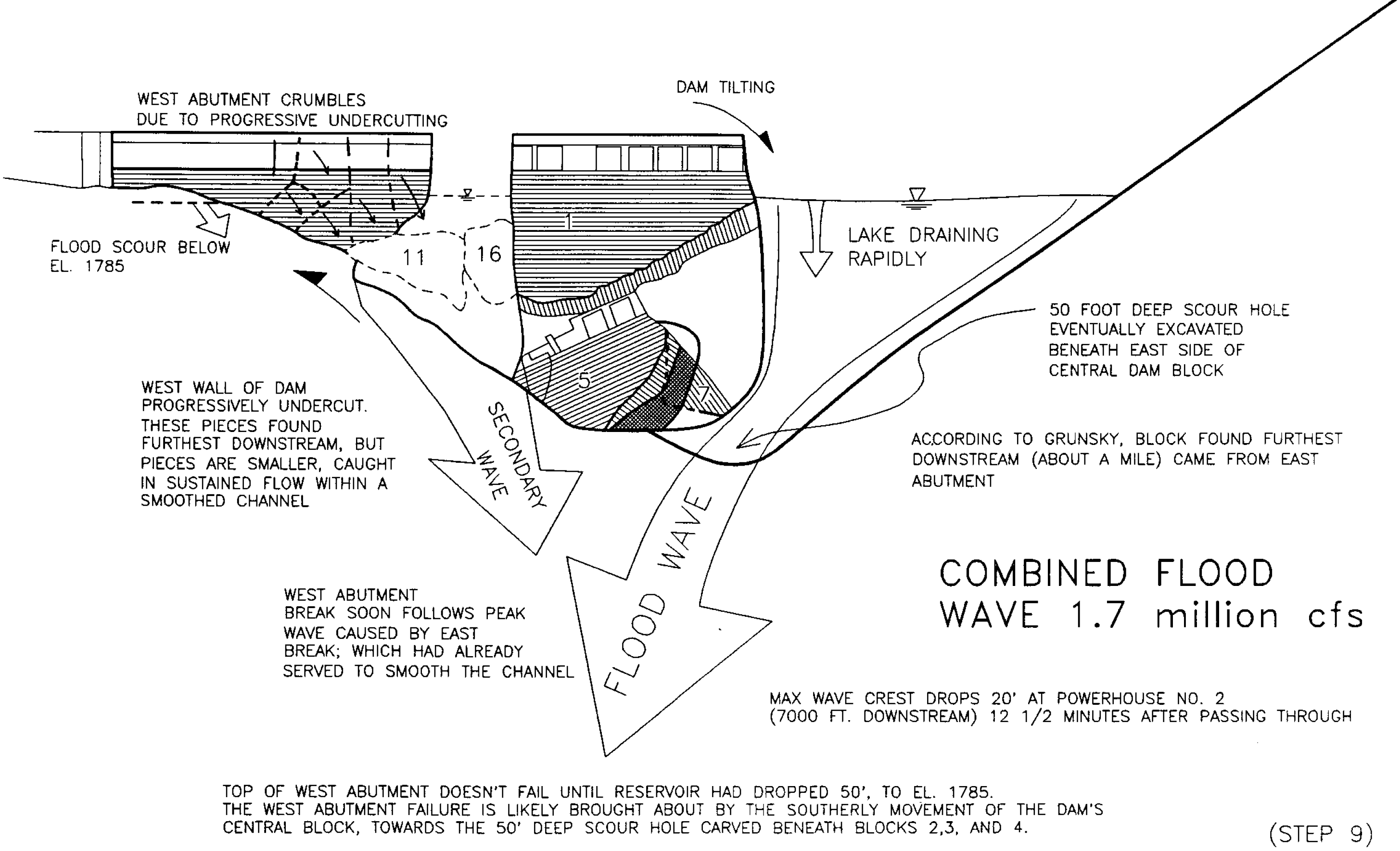


Figure 23. (upper) Schematic portrayal of east and west breeches combining to form the maximum discharge of 1.7 million cfs, sometime shortly after midnight on March 12/13, 1928. This wave crested at Powerhouse 2, 7000 feet downstream, sometime between 12:02:30 and 12:08; receeding 20 feet by 12:15 AM. The maximum outflow was likely caused by the rapid ero sion of the debris plug created by the east abutment slide at 11:57:30 PM.

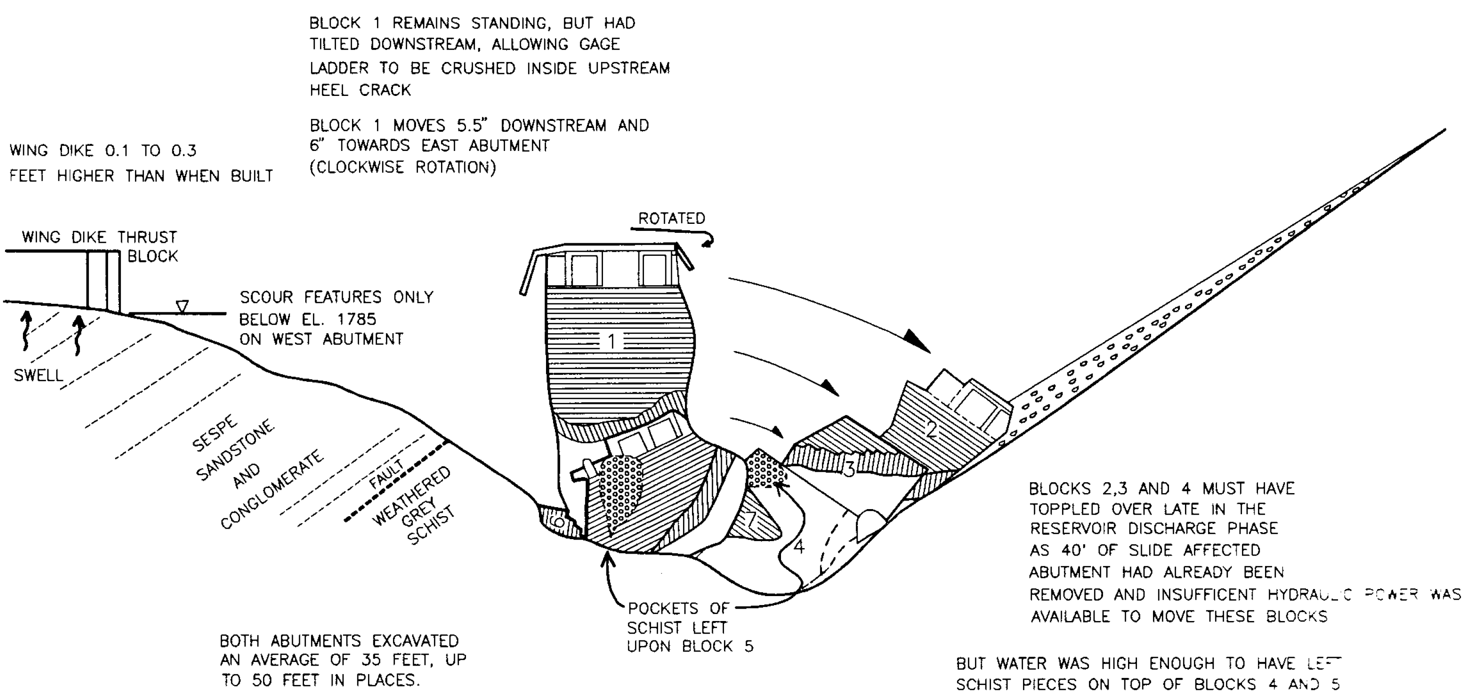


Figure 23. (lower) Schematic portrayal of the final stages of the St. Francis failure. The west abutment was scoured away very quicky once the initial piercement occurred, just west of Block 1 (see Figure 24). Block 2/3/4 was undercut by the outpouring through the east breech, eventually toppling over into a hole 50 feet deep, falling backward. The cutting of such a deep hole on the east side of the main structure also argues for more discharge emanating from this side of the structure. This is how the scene looked the following day (Figure 25).

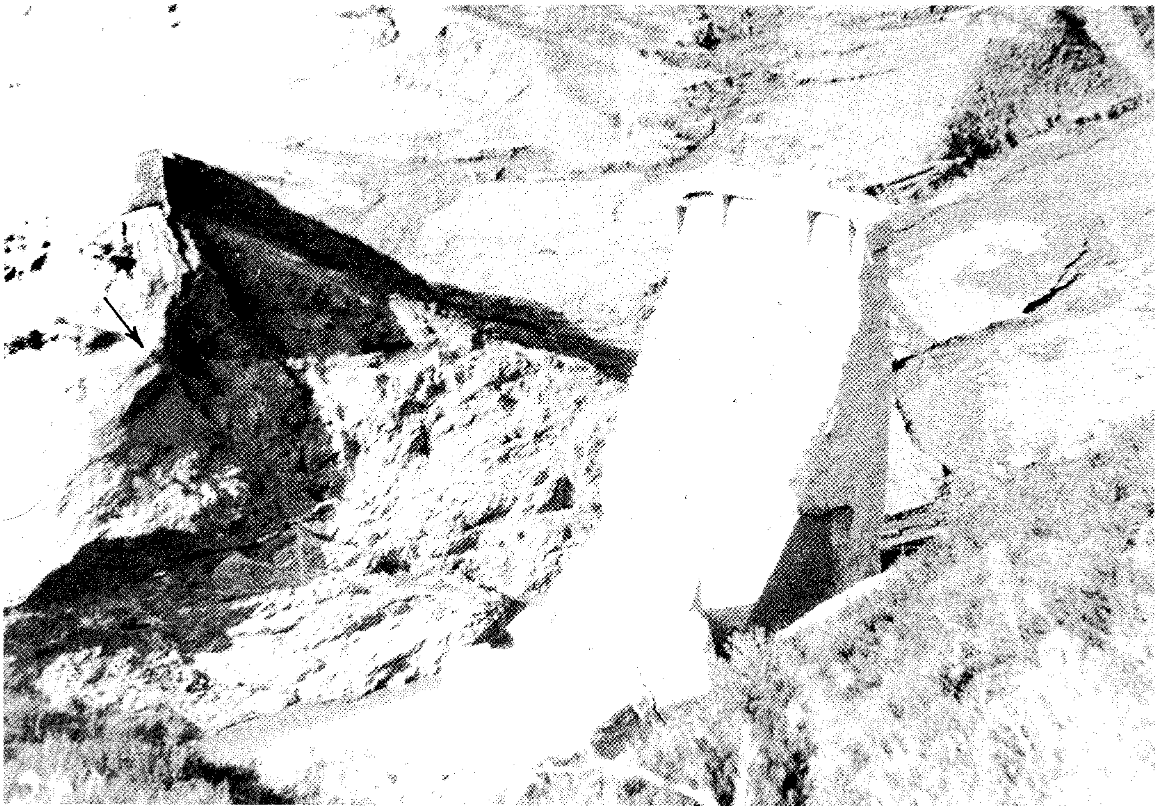


Figure 24. (upper) View looking down on the dam site from the upper surge chamber road (Figure 14 lower) the morning after the failure. Note how the original construction road, leading up to the wing dike, is not damaged above elevation 1785 (arrow). This suggsts that, at a minimum, the upper portion of the west abutment did not fail until the reservoir had drained at least 30 to 50 feet (photo courtesy of Ventura County Museum of History and Art).

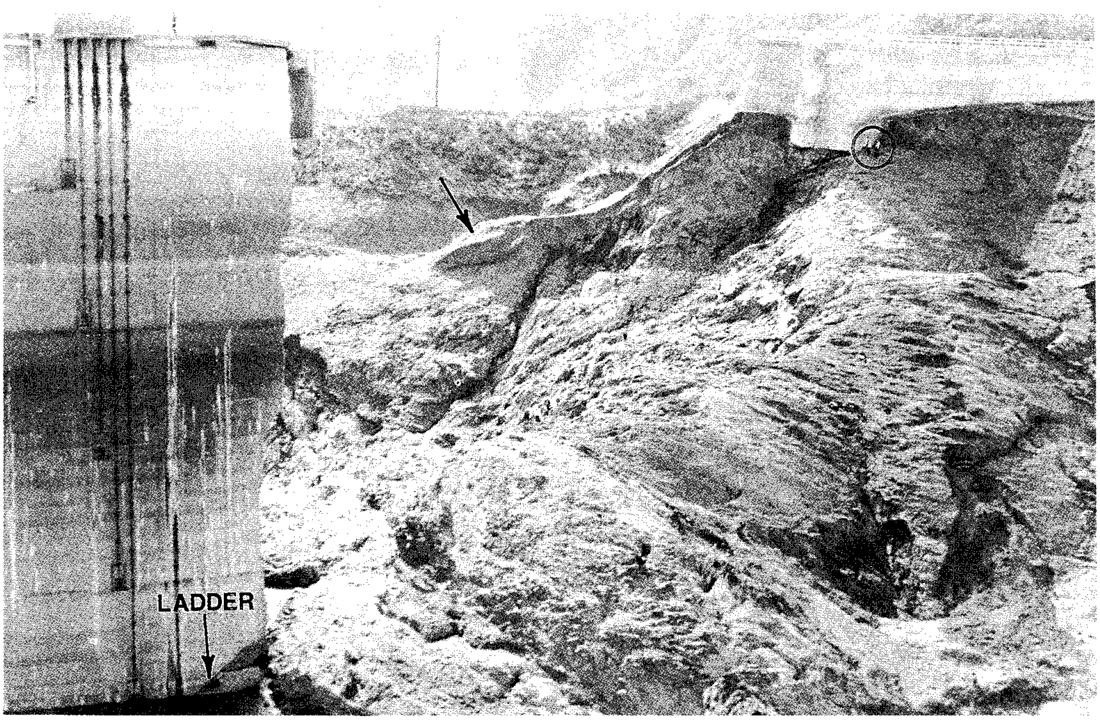


Figure 24. (lower) Detail of failed west abutment, looking downstream, with people circled for scale. Only a portion of the west dike access road side-cast fill was excavated by the outpouring water (arrow), with scrub brush left upon the cut slope, down to elevation 1785, 50 feet below the reservoir level. Grunsky's ladder, caught in the upstream heel crack as Block 1 tilted downstream, is annotated (photo courtesy of Ventura County Museum of History and Art).

expert source. No less than four prominent geologists viewed the site (Branner, Louderback, Ransome and Willis), but only Willis (1928) recognized the geomorphic indicators of ancient landsliding. Somehow, in the failure's aftermath, too much emphasis was placed on the fault, which was so vividly exposed on the west abutment (Figures 17 and 24).

It appears that William Mulholland simply didn't appreciate the concepts of effective stress and uplift, precepts that were then just beginning to gain universal acceptance (LaRue, 1928). Harza's (1949) award-winning paper on uplift theory had been written 15 years before it was accepted for publication, due to the reticence of some of the senior members of the dam engineering profession in the 1930s and '40s. The other weak link in Mulholland's design process had been the apparent omission of any outside consultants to review the dam's design, a curious decision considering that he had previously convened a consulting board to review the Los Angeles-Owens Aqueduct plans. During the Coroner's Inquest, Mulholland (1928) stated that he had brought Stanford Professor John Branner to the dam site to view it before construction had commenced. The Governor's Board of Inquiry concluded that the owners of all dams should submit their plans for review by an outside board of consultants; a recommendation almost identical to that made by Idaho's Teton Dam Inquiry Board a half century later (1976). The Coroner's Inquest reached a similar verdict, quoted by

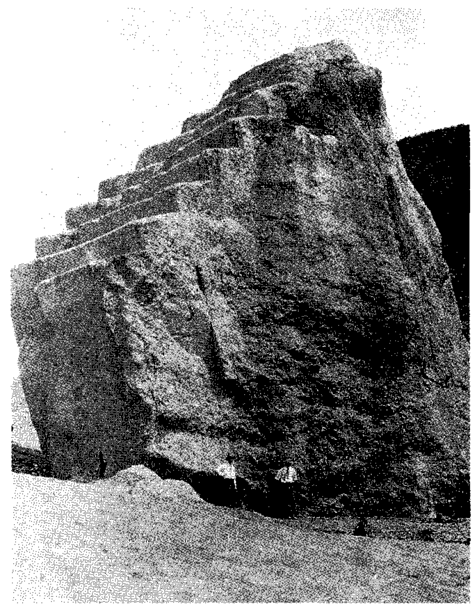


Figure 25. - Close up view of dam Block 16, which at 10,000 tons, was the largest piece of the dam moved any appreciable distance downstream (as shown in Figure 8). This block came from the lower part of the west abutment, adjacent to Block 1, which remained standing. The small holes visible on the upper cold joint (arrow) were for the rakers supporting the stepped forms for the dam's downstream face. Blocks of this size can be most easily transported in a dirty fluid or slurry, which greatly diminishes their effective weight, as suggested in Figure 27. Photo courtesy of Ventura County Museum of History and Art.

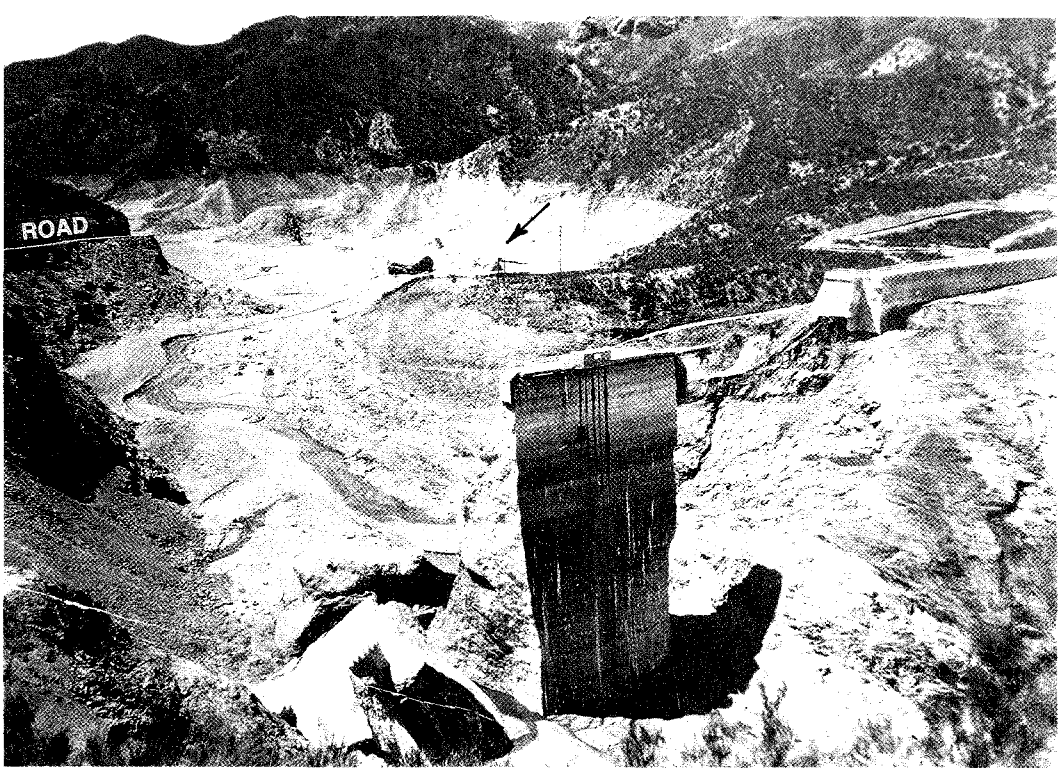


Figure 26. Overview of the flood's aftermath, immediately downstream of the dam. The flood wave scoured the canyon's slopes to a maximum depth of 120 feet above the stream level. The Edison power line crossed the channel, but was situated above the maximum scour line (arrow). San Francisquito Canyon road began climbing east wall of the canyon, at upper left. Photo courtesy of Ventura County Museaum of History and Art.

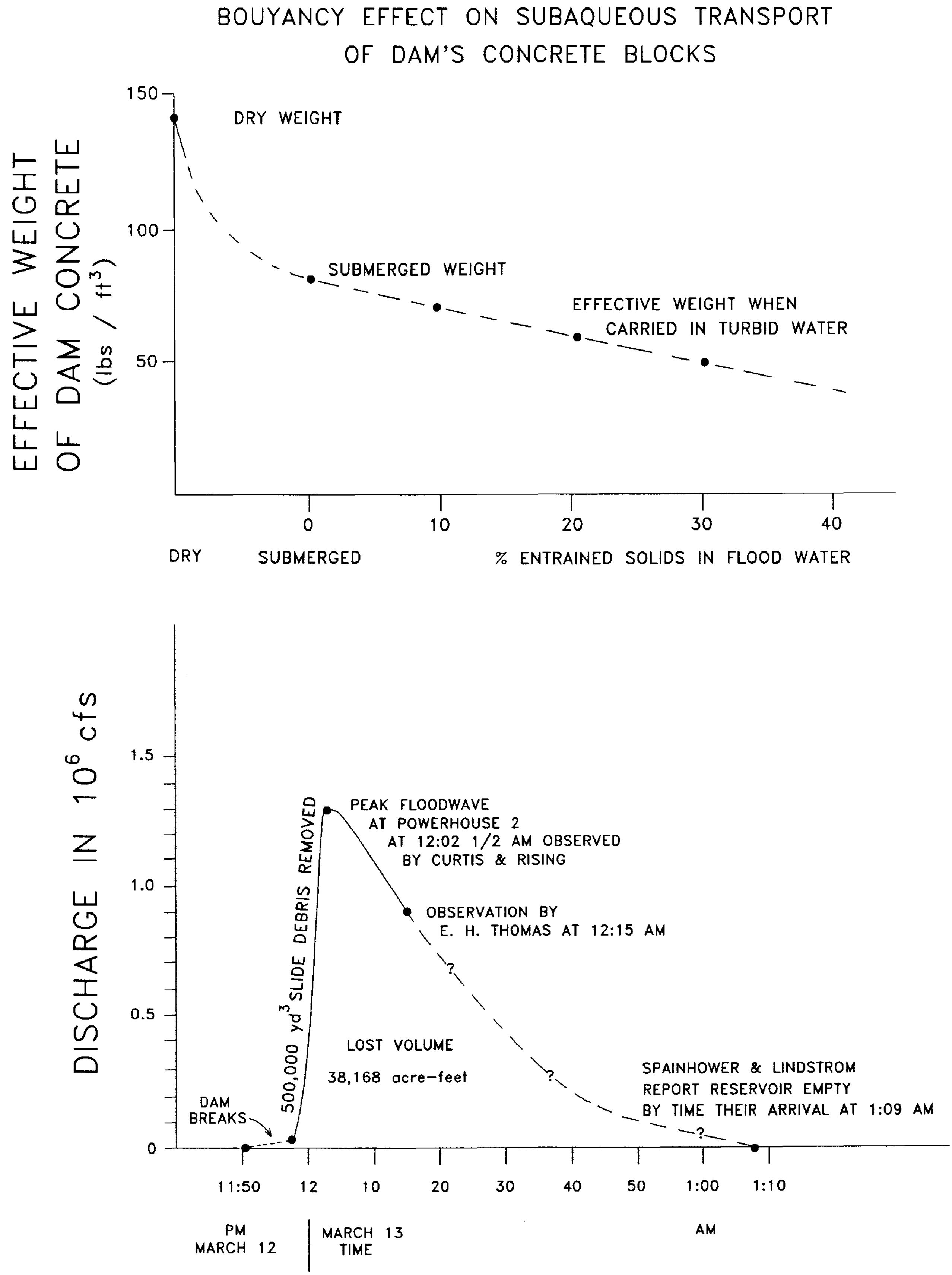


Figure 27. Preliminary evalation of flood wave turbidity effects on bouyancy of the dam's concrete blocks. Although heavy when dry, tests of the dam's concrete show it to have a void ratio of around 13%. Post- failure photos also suggest that the blocks were saturated, as they continued to weep moisture out of their cold joints for days following the failure. If saturated, and immersed in turbid flood waters, the effective weight of the blocks was reduced by as much as 67 percent

Raphael (1988), stating:

> *A sound policy of public safety and business and engin eering judgment demands that the construction and operation of a great dam should never be left to the sole judgment of one man, no matter how eminent, without...checking by independent experts."*

With those conclusions expressed, the State legislature enacted legislation to bring about a dam control law the following year. This new law mandated that the owner of proposed dams pay for a review of their projects by a board of eminent civil engineers and geologists retained by the State Engineer. This body subsequently became the Division of Safety of Dams (DSOD) within the State Department of Water Resources, and was one of the first agencies created strictly for dam safety review in the world.

Looking back, its particularly sad that the engineering profession's opinions were almost wholly predicated on little, if any, actual field investigation. Everyone seems to have accepted the City's cross sections and Mulholland's testimony with respect to the dam's maximum width, without having verified them through proper field measurements. With the exception of Grunsky and Willis, the published discussions focused on the fault, the propensity of the Sespe beds to slake under submersion, and the Sespe's low shear strength, which was about one fourth that of the dam's concrete. But, the more significant culprits appear to have been a proper appreciation of uplift theory and the incorporation of sound geomorphic and rock mechanics assessments. A similar failure mechanism befell Malpasset Dam 32 years later, and it was only after years of studying that failure that the importance of complex rock mechanics analyses were demonstrated. Had people kept working on St. Francis, the actual complexity of the failure might have been discovered.

It is of more than passing interest to note that most of what we can discern from the St. Francis failure didn't come from civil engineers or geologists, but through the efforts of a single journalist. The late Charles Outland (1910-1985), a long-time Ventura County historian and native of Santa Paula, spent the better part of his lifetime researching the failure. As a high school senior, Outland watched the flood decimate his hometown in the early morning hours of March 13, 1928. It was a sight he was never to forget. For 30 years he researched the St. Francis failure. This effort culminating with the publication of his first book, Man-Made Disaster, which appeared in 1963. It continues to be the only definitive work ever published on the construction and failure of the St. Francis Dam.

For engineering geologists, St. Francis will forever exist as a warning beacon to those who do not heed the importance of geologic input. A structure is only as strong as its foundation. No student of geology in southern California has completed their college curriculum without a visit to the imposing waste and heartfelt tragedy of the St. Francis site. Might we never forget it.

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