Hoover Dam and the Evolution of Uplift Theory

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ABSTRACT

The earliest technical article by an American engineer on hydraulic uplift was the ASCE Presidential Address by James B. Francis in 1888. He suggested applying full hydrostatic pressure at the upstream heel of a dam, diminishing to zero pressure at the downstream toe. Thoughts on the potentially destabilizing role of uplift were mentioned by John R. Freeman in 1911 in his comments on the failure of Bayless Dam near Austin, PA. Between 1911–1918 Arnold C. Koenig, J. B. T. Coleman, and James B. Hays discussed how to account for uplift in masonry dams in the ASCE Proceedings and Transactions. Up until 1927–28 most engineers assumed that uplift pressure was tied to the permeability of the foundation and that of the dam structure. Most assumed that concrete was impervious and incapable of transmitting meaningful pore pressure. The textbooks cited by the engineers who drafted the plans for the St. Francis Dam cited examples that appeared in textbooks published between 1908–18. In 1918 Edward Wegmann asserted that it was impossible to accurately estimate the uplift that might develop beneath a dam and that engineers should rely on their own judgment. Others suggested that an uplift pressure diagram in the shape of a trapezoid be employed, assuming development of two-thirds the theoretical uplift, varying lineally to zero at the toe, unless uplift relief wells were employed. All the various theories were called into question when Hoover Dam was filled to capacity in 1934–41 and excessive uplift pressures developed beneath the dam's downstream face. This led to tripling the depth of the grout curtain, which took nine years to complete. In 1936 Karl Terzaghi introduced his theory of effective stress, which established a distinction between total stress and those ascribable to hydrostatic (pore water) pressure. In 1939 ASCE formed a Subcommittee on Uplift in Masonry Dams. In 1945 Terzaghi summarized the results of experimental work that suggested water was able to transmit pore pressures in concrete. The ASCE committee released their final report in 1952, which included uplift measurements of a modest number of high-head dams and commented on what they felt constituted reasonable bounds on how much uplift could develop if relief wells were installed immediately downstream of grout curtains.

EARLY APPRECIATION OF HYDRAULIC UPLIFT

The earliest murmurings about hydraulic uplift negating the dead weight of dams was first proposed by James B. Francis in his Presidential Address to ASCE in 1888 (Francis, 1888). Francis asserted: "*On the base of a wall founded on rock, having numerous seams, containing water under a head, the flow from which is not carried off by drains, an upward pressure may be transmitted through the mortar to the entire base*." He suggested applying full hydrostatic pressure at the upstream heel of a dam, diminishing to zero pressure at the downstream toe. The subject was next broached by John D. Van Buren (1895) after the failure of Bouzey Dam in France. In an article for ASCE Transactions he advocated that full hydrostatic head should be assumed at the upstream heel and assume a similar force at each horizontal joint in a masonry dam. Prof. L.J. Leconte (1895) submitted a discussion citing a number of dams in the San

Francisco area that exhibited leaky foundations that likely diminished the build-up of uplift pressure. In his design of the Wachusett Dam for the Boston Waterworks in 1900, Frederick P. Stearns assumed an uplift force beneath the masonry main dam of two-thirds of the full height reservoir pore pressure at the upstream heel, diminishing to zero at the downstream toe (Wegmann, 1927).

Despite some spectacular failures of masonry dams in Austin, Texas in 1892 and again in 1900 (Rogers, 2015), it wasn't until the failure of Bayless Dam near Austin, Pennsylvania in 1911 (Rose, 2013) that the subject of uplift appeared again, in a letter to the editor discussing the Bayless Dam failure by John R. Freeman in Engineering News (Freeman, 1911), and another in ASCE Transactions (Harrison, 1912).

Prior to 1928 most engineers assumed that uplift was intrinsically tied to the permeability of the foundation rock and that of horizontal partings in the dam material. During the two decades leading up to 1928 the four most cited textbooks provided rather vague suggestions. Turneaure and Russell (1908, p. 387) suggested that the uplift pressure could be altogether neglected. Morrison and Brodie (1910) suggested that an uplift pressure diagram in the shape of a trapezoid be employed, assuming development of two-thirds the theoretical uplift, varying lineally to zero at the toe, unless uplift relief wells were employed. William P. Creager (1917) was of the same general opinion as regards the amount and distribution of uplift. Wegmann (1918) asserted that it was impossible to accurately estimate the amount of uplift that might develop beneath a dam, so engineers should "*rely on their own judgment.*"

The first technical article focused solely on uplift was in 1915 by J.B.T. Coleman, which also appeared in the ASCE Transactions (Coleman, 1916). Its content triggered 11 discussions totaling more than 30 pages. In November 1920 ASCE's Board of Direction named "Results of Pressure Measurements of Uplift" as one of the subjects proposed for future technical activities of the society, which speaks to the need to collect uplift measurements from completed dams. The paucity of reliable data would continue for some time because instrumentation had to be organic to the dam's original construction.

The accepted references on the design of masonry gravity dams were largely outdated by the 1920s because they failed to reach any consensus on uplift theory, grout curtains, uplift relief wells, contraction joints, water stops, grouting of contraction joints after curing, and cement heat of hydration problems. These details became increasingly appreciated, beginning with Elephant Butte Dam in 1916. It was the Reclamation's largest mass concrete structure up to that time, and one of the first dams to employ a semi-continuous grout curtain extending into the rock foundation.

FAILURE OF THE ST. FRANCIS DAM

St. Francis Dam was a 63.4m (208 ft) concrete gravity arch dam with a constant radius of 152.4m (500 ft) built by the Los Angeles Bureau of Waterworks & Supply (today's Department of Water & Power). The structure was originally intended to have a maximum height of 54.9 m (180 ft), but was raised in two increments of 3m (10 ft) without increasing the dam's base width. It assumed a bearing pressure of 9.58 x 105 N/m^2 (10 $tons/ft^2$) on foliated mica schist and askosic sandstone, separated by an inactive fault in the right abutment. Ten test borings between 10 and 15 feet into the schist were converted into uplift relief wells beneath the dam's center section, but no uplift relief was installed beneath the steeply sloping abutments. The dam was constructed between August 1924 and April 1926.

The dam was comprised of 99,320m^3 (130,000 $yds^{3)}$ of mass concrete, which developed four prominent transverse shrinkage cracks that extended all the way through the main dam by late 1927. The exposed crack apertures on the dam's downstream face were initially between 1/8th and 3/16ths of an inch wide. What engineers didn't understand at the time was that the width was inconsequential; it was the transverse connectivity with the dam's upstream face that impacted internal stability (Rogers, 2017). In early 1928 the decision was made to caulk the transverse shrinkage cracks with oakum; a mixture of hemp or jute fiber that was often smeared with tar, creosote, or asphalt. The oakum served as a seal so lean grout could be inserted into the cracks to seal them (Left pane of Fig 1). All of the dam's visible shrinkage cracks were plugged along the downstream face, including the one on the dike section. Plugging of the dam's contraction joints on the downstream face had disastrous consequences by allowing full reservoir pressure against the interior faces of the main dam, especially if the grouting behind the caulked face was anything less than a 100% seal. This detail was neglected in post-failure investigations at the time.

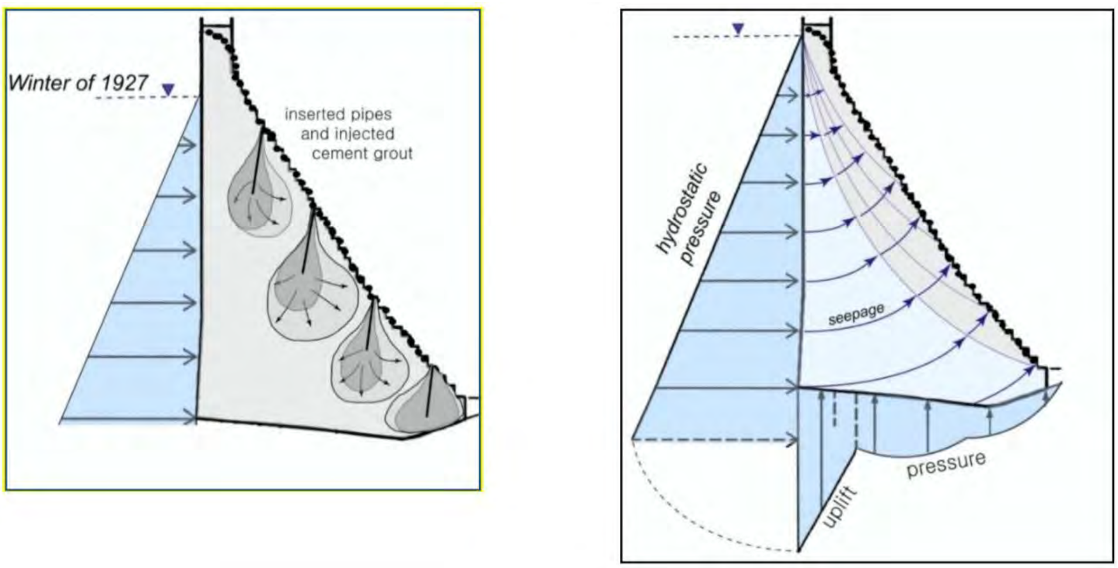

Figure 1. Left pane depicts the grouting of the transverse shrinkage cracks in the maximum section of the dam in early 1928. Right pane shows the likely impact of the 10 uplift relief wells beneath the main dam, prior to caulking of the transverse shrinkage cracks.

The St. Francis Dam collapsed catastrophically at midnight March 12/13, 1928, sending 38,160 acre-feet of water downstream in an initial wave 42.7m (140 ft) deep, which took 5-1/2 hours to flow 84km (52 mi) to the Pacific Ocean, and killing at least 430 people. Its dramatic failure set into a motion sudden interest and exchange of opinions about the potentially destabilizing effects of uplift pressures on hydraulic structures. Over the ensuing months a number of technical symposia were organized that addressed perceived needs for a better understanding of uplift, how to measure it, and, if it exists, how it might be estimated. These included articles or reports by Pagon (1927), Hinds (1928), Floris (1928), Pearce (1928a; 1928b), Terzaghi (1928) Torpen (1929), Parsons (1929), Jakobsen (1929), Houk (1930a), Westergaard (1930), Wiley (1931), Henny (1931, 1934), and Lane et al (1934, 1935).

It didn't take long for the fallout of the St Francis disaster to be felt. Floris (1928) maintained that it was impossible for full hydrostatic forces to develop at the dam-foundation interface unless the water could somehow get between the two materials, which seemed highly improbable unless there were a basal shearing as occurred at Austin Dam in 1900 and Bayless Dam in 1911. Jakobsen (1929) pointed out that the full hydrostatic head can never be reached because of the weight of the concrete being so much more than the water (this was before the concept of effective stress was introduced by Terzaghi in 1936).

In November 1929 A.J. Wiley, who had led the forensic team appointed by the State of California a year previous to investigate the St. Francis Dam failure asserted in an ASCE Proceedings article that "*There is no uplift in the body of a concrete dam except that caused by horizontal [pour] points, nor in foundation rock which is free from seams, nor in a perfect connection between the dam and the bedrock, except that which may be due to a different degree of porosity in the concrete and rock.*" Until 1945 these same individuals assumed that concrete was impervious and incapable of transmitting pore pressures.

In November 1931 a Board of Consulting Engineers appointed by the Los Angeles Board of Water & Power to evaluate the Mulholland Dam and Hollywood Reservoir. Two of the three board members concluded that *"The structure, has an insufficient base width with no allowance for uplift or earthquake stresses, and the foundation not properly terraced to provide against sliding of the dam on its foundation"* (Engineering News, 1931). It was a virtual copy of the ill-fated St. Francis Dam (but with a constant radius of 137m [450 ft]). Its reservoir volume was drawn down by two-thirds and an earth buttress of 252,000 m^3 (330,000 yds^3) placed against its downstream face and planted with trees to shield the dam from public view.

MEASUREMENTS OF UPLIFT

Ironically, the precedent-setting article on uplift appeared in the March 1928 issue of ASCE Proceedings, the very month the St. Francis Dam failed, and it was written by a new Los Angeles Bureau of Waterworks & Supply engineer named Julian Hinds. He joined the agency in January 1928, after several years of consulting on projects in Mexico, after working for Reclamation from 1910-26. His article was titled "Uplift Pressures Under Dams: Experiments by the United States Bureau of Reclamation." This appeared with 12 discussions in the 1929 ASCE Transactions, along with an article titled "Hydrostatic Uplift in Pervious Soils" by Harry De B. Parsons, which summarized experiments on porous coarse grained sandstone, which attained almost 100% uplift.

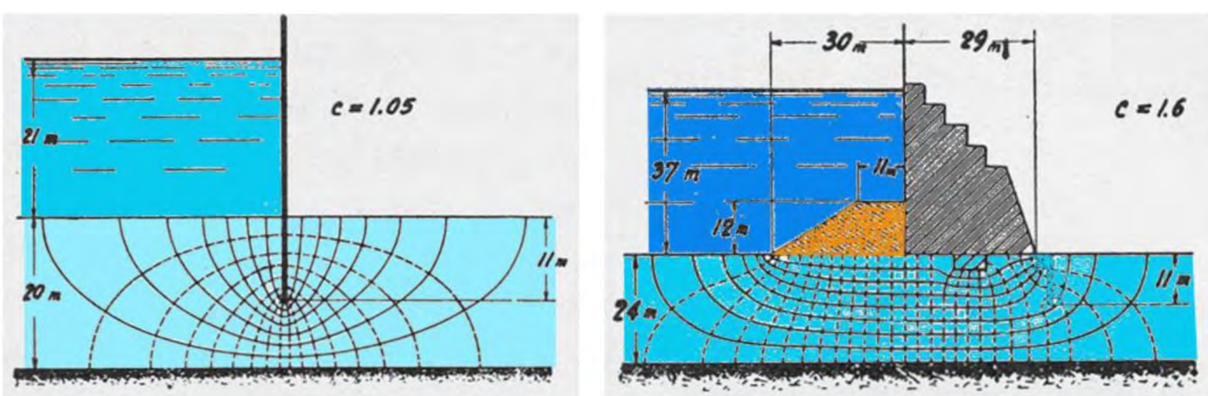

Figure 2. Seepage nets presented by Terzaghi (1929) for a bulkhead cutoff wall (left) and for a masonry concrete dam with an upstream toe berm. Flow nets allow a rational means of estimating uplift pressure in porous unconsolidated materials, like alluvium.

Hinds had recently moved to Los Angeles to work on the proposed Colorado River

Aqueduct. From 1925-27 he supervised the measurement of uplift developed beneath a concrete gravity dam (American Falls Dam on the Snake River in Idaho) and three concrete diversion dams: Colorado River Dam (Colorado), Percha Dam (New Mexico), and Willwood Dam (Wyoming). Hinds began the trend of plotting the maximum uplift pressures verses the uplift head pressure on the dam's base. This allowed engineers to compare the maximum theoretical value with the measured values, which were usually much less.

There was a very real need for engineers to develop a rational method of estimating likely uplift pressure that might develop under extreme flow conditions. This gave rise to electrical analog flow models (Terzaghi, 1929; Lane et al, 1934) and the construction of 2D steady state flow nets, where flow lines and equipotential lines, like the earth berm and sheetpile cutoffs in Fig. 2.

Ivan Houk (1930b) of Reclamation was the first worker to report uplift measurements of a "high dam," the 61 m (200 ft) high Gibson Dam, a concrete gravity arch structure on the Sun River about 113 km (70 mi) west of Great Falls, MT. The dam's maximum section was 27.4 m (90 ft) thick at the base, tapering to 4.6 m (15 ft) at the crest. It employed a constant radius of 123.4 m (405 ft) with a crest length of 292.6 m (960 ft), requiring 123,000 m^3 (161,000 yds^3) of mass concrete. Gibson Dam was constructed in 1926-29 on crystalline limestone dipping steeply upstream. A cutoff trench 1.5 to 9 m (5 to 30 ft) deep was used as the launching platform for a grout curtain 6 to 12 m (20 to 40 ft) deep and was installed on 1.5 m (5-ft) centers. It was the first Reclamation structure constructed with a regular line of uplift relief (drain) holes inserted at an angle of 30 degrees from vertical facing downstream, also on 1.5 m (5-ft) centers. These drains were connected to vertical tile drains embedded in the dam's concrete, also located on 1.5 m (5 ft) centers. Drains extending from the base of the dam into the bedrock were placed in three arrays spaced 6 m (20 ft) apart, to allow a profile across the dam at six locations between its abutments.

Uplift measurements commenced in May 1930, reporting pore pressures as equivalent water surface elevations measured in vertical pipes. The total head was taken as the height of the reservoir water surface above the base of the dam. The observed uplift pressures were found to be 25% of the total available head, with only one exception (where it reached 29.4%).

Hauk's (1930) report included a blueline plot illustrating the maximum uplift recorded beneath nine gravity dams on rock foundations, including two in Germany (this figure was reproduced many times thereafter). Hauk cautioned readers to appreciate that no data were available to indicate the areas on which the observed pressures act upon. Over the next several decades most who studied uplift recognized this same conundrum, and most everyone agreed that more data and case studies were needed.

UPLIFT PROBLEMS AT HOOVER DAM

During the exploration of Black Canyon prior to 1931, 22 borings were advanced in the channel beneath the proposed site for the dam, but only one deep boring was drilled to a depth of 166 m (545 ft) below low water level, to ascertain if the andesite breccia bedrock continued to great depth. A conventional grout curtain was drilled beneath the dam's upstream axis, comprised of a single line of holes 30.5 to 39 m (100 to 125 ft) deep, about 14 to 21% of the dam height. The depth of the curtain was selected on the basis of a survey of existing dams with grout curtains that Reclamation undertook prior to construction. Foundation grouting was completed in 1932-33, including grouting of some of the principal faults on both abutments. On the Nevada abutment four holes had to be abandoned, because of excessive grout take and leaks

(Simonds, 1953).

When the reservoir reached 335 m (1100 ft) elevation in 1937, the faults daylighted in the right abutment, and water began entering the fault zone. At this time the abutment drains in the Nevada side began discharging cool water. Warm water from the natural hot springs was collected along the right abutment drainage gallery near elevation 555, emanating from several “shattered zones.” The original grouting of this area was ineffective due to premature set of the cement grout, caused by the natural hot springs encountered in this area (described in Rogers, 2010)

A profile view of the original grout curtain is reproduced in Fig. 3-right. The layout included four rows of shallow B-holes, drilled 9 to 15 m (30 to 50 ft) deep and spaced 6 m (20 ft) apart, considered to be “dental work.” The C-holes were drilled on an incline from outside the upstream heel of the dam on 3 m (10-ft) spacings, to a maximum depth of 30.5 m (100 ft). The C-holes were grouted with pressures of up to 6.2 MPa (900 psi) prior to drilling of the A-holes, which were inclined upstream, from the lower drainage gallery (Fig. 3-right). The A-holes formed a curtain 46 m (150 ft) deep on five-foot centers, inclined 15 degrees upstream. A line of vertical uplift relief drain holes 30.5 m (100 ft) deep was drilled just downstream of the grout curtain (shown in Fig 3-left).

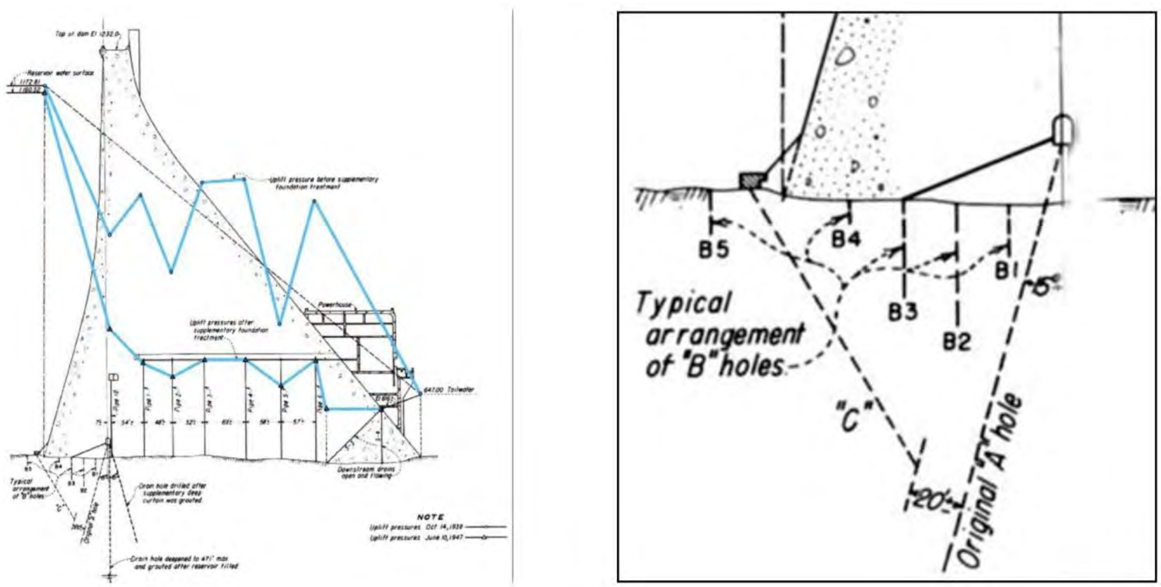

Figure 3. Left diagram shows blue profiles of uplift pressures measured between 1939-1947, before and after post-construction grouting (Simonds, 1953 and Rogers, 2010). Right figure is an enlargement of network of grout holes drilled beneath the dam’s upstream heel.

Post-construction corrective work

By June 1937 abnormally high uplift pressures began developing beneath the right center of the dam (Fig. 3-left). At the same time seepage inflow from the right abutment began overwhelming the lower galleries and pouring out of the canyon wall above the Nevada Powerhouse. In addition, alkaline water seeping into the lower penstock tunnel began accelerating corrosion of the steel penstock. Hot alkaline water also began seeping through the concrete liner of the inboard diversion tunnel on the Nevada side and spilling onto the 30-footdiameter steel penstock feeder, causing accelerated corrosion of the penstock. These seepage problems were mitigated by additional grouting around the diversion tunnel.

An extensive program of post-construction grouting was carried out during the 1940s to extend a grout curtain beneath the Nevada spillway and intake towers. This program succeeded in mitigating the seepage problems (Simonds, 1953). Excessive seepage also manifested itself along two fault strands through the right abutment when the reservoir reached elevation 1100 feet, 132 feet below crest.

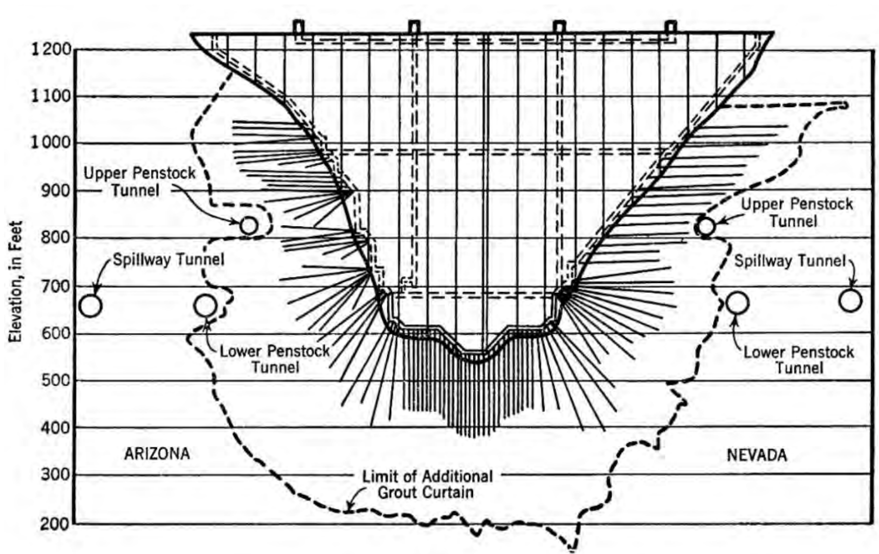

Figure 4. Profile of dam centerline showing the extent of original grout curtain and that of the deepened grout curtain, installed from 1938 to 1947, shown by dashed line (from Simonds, 1953).

Figure 5. Cramped working spaces typified the 9- year program of extending the grout curtain, from the dam's lowest centerline gallery between 1938 and 1947 (USBR).

The reservoir uplift reached its maximum levels in September 1938 (Fig. 3-left). This was unexpected, and a decision was quickly made to drill a series of BX-size (6 cm/2.38 in) core holes in the dam's foundation. These revealed that the grout curtain failed to penetrate six zones of intensely sheared rock, which were feeding seepage into a series of crisscrossing manganese gouge seams, which swelled up and became impervious (Simonds, 1953).

The dam's grout curtain was extended and deepened extensively between 1938 and 1947 (Fig. 4). The grout holes were extended to depths of 91 m (300 ft) beneath the dam's foundation, then pumped under pressure of full reservoir head. These were drilled from the dam's system of internal inspection galleries, shown in Fig. 5. A schematic view of the deepened grout curtain, as completed in 1947 is shown in Fig. 4. During the 12-year supplemental drilling program, 124,962 m (410,000 ft) of grout and drainage holes were drilled, and 11,951 m^3 (422,000 ft^3) of grout injected under pressure. This remedial program cost an additional $3.86 million. Uplift pressures were significantly reduced, as shown by the lower blue line in Fig. 3-left.

UPLIFT AND EFFECTIVE STRESS THEORY

When Karl Terzaghi began experimenting with consolidation of clay soils he discovered that he only discerned the correct relationship between stress and strain if he assumed that the measured compressibility and shear strength did not depend on the water pressure, it was wholly independent. His experiments showed it was the effective stress (total stress minus the hydrostatic pressure) that governed shear strength and compressibility of soils and deformable rock. It was essentially an empirical finding, substantiated by many experiments, without fully comprehending the mechanisms involved, which took years to sort out (Peck, 1997; Goodman, 1999).

In June 1936 Terzaghi published "Simple Tests to Determine Hydrostatic Uplift" in Engineering News Record. Not everyone agreed with his novel assertions, and many contrasting viewpoints appeared in the ASCE Transactions as discussions. The principal argument revolved around what was the "effective contact area," which surprised everyone by turning out to be almost 100%. Years earlier William Creager (1917) and others since that time had theorized that the effective contact area must be much less; something between 100% only for loose sand and 0% for glass.

Terzaghi developed his theory of effective stress from a series of three laboratory tests. It was from his so-called "buoyancy shearing test" that he derived the effective stress relationship for shearing resistance. He began by stating that specimens of soil are observed to shear along an rough inclined surfaces oriented at small epsilon, given as: 45 - 0.9 (phi)/2, where phi is the angle of internal friction (we don't know where the "0.9" constant came from). He went on to derive the relationships for the mobilized shear strength along an inclined surface.

In an undrained soil specimen, an increase in all-around pressure (total stress) does not increase the specimen's strength. But, in the drained specimens, he observed an increase in strength, correlative with an increase in cell pressure exerted in a triaxial test. Terzaghi concluded that the compressibility of a sample was due to void closure, caused by expulsion of incompressible pore water. He found that for porous materials, such as concrete or clay, the reduction factor was nearly 1.0, so full hydrostatic force was being realized, close to what was pretty much accepted for sand.

He reached this conclusion seven years after his discussion of Harry de B. Parsons' 1929 paper on measurement of hydrostatic uplift on pervious soils, when he opined that the effective contact area in clays could be no greater than about 60 to 90%. He used some elaborate computations to support these premises. Terzaghi assuaged that his theory of effective stress was the most fundamental principle of soil mechanics because it rationally explained how strength and compressibility of materials were related, which was critical to the stability of hydraulic fill embankment dams, which were common prior to 1945. Effective stress theory was unveiled by Terzaghi's students at the First International Conference on Soil Mechanics and Foundation Engineering at Cambridge, MA in June 1936, the same week his ENR article appeared.

Recognition of pore pressure conductance (1945-48)

In 1933 while teaching at the Technische Hochschule in Vienna, Terzaghi embarked on a series of triaxial tests on specimens of concrete described in Terzaghi and Rendulic (1934), which were funded by the Austrian Concrete Institute. These tests confirmed that concrete behaved as a "perfect porous media and obeyed his effective stress law." During the lull in teaching duties during World War II Terzaghi (1945) decided to undertake another appraisal of this question, using larger and more capable triaxial test frames. He employed jacketed specimens of concrete to explore the role of effective stress in the deformation and subsequent failure of concrete. His experiments were intended to probe the influence of pore water pressure on the "confined compressive strength" of concrete (as opposed to unconfined strength). He drew six conclusions from the tests, some which caused him to retract opinions he had professed earlier (Terzaghi, 1929a, 1929b). In 1928 he had assumed it improbable for uplift pressures to develop within/through low porosity materials like granite or mass concrete. The six conclusions are briefly summarized below:

1. Concrete and rock samples under hydrostatic confinement (confining stress) supplemented by an axial principal stress pressure were observed to fail by splitting, or pseudo-shear.
2. The excess axial pressure required to trigger failure depended on the total confining pressure and the internal pore water pressure. If the internal pore water pressure was near zero, the specimen's strength remained high. But, if the pore water pressure rose to a value equal to the confining pressure, the compressive strength remained constant (unlike soils).
3. It appeared that the solid constituents of concrete or rock are interconnected (cemented) over tiny portions of their total surface area, and the corresponding boundary porosity is 0.98%. This was a startling revelation because it meant one could expect hydrostatic uplift to develop in concrete almost as easily as in a cohesionless sand!
4. By comparing results of volume compression tests on jacketed specimens with unjacketed showed that confining pressures up to 1.44 to 1.92 x 10^6 N/m^2 (15 or 20 tons/ft^2) the boundary porosity of concrete or rock was negligible. Since the boundary porosity of stress-free specimens was shown to be almost 100%, meaning that the particle constituents were almost enveloped completely by water. This meant that the common practice since 1930 of assuming uplift only acting over 50 to 67% of the base of a concrete dam was erroneous.
5. The compressive strength of saturated concrete q_c was influenced by confining pressure p_c as well as internal pore water pressure u_w, when the effective stress ($p_c - u_w$) was less than 20 tsf. Compressive strength $q_c = q_u + 4.1 (p_c - u_w)$, where q_u is the unconfined compressive strength.
6. The impact of pore water pressure on the strength of rock at shallow depths (e.g. 30.5 m [100 ft] deep) can only be determined by the triaxial test procedure, but may become increasingly less valid with significant depth (e.g. 6000 m [20,000 ft]), because less pore water would be available.

From Terzaghi's view the empirical data was overwhelming, but it was difficult to explain scientifically. His theory of effective stress held to the premise that if pressure produces a change in mechanical properties, then it is an effective pressure, and that if it did not, he termed it a "neutral pressure." He reasoned that the unexpected pore pressure conductance was likely because of microfissuring of the concrete's mortar, which Terzaghi knew to be porous and thereby theoretically able to transmit pressures through incompressible molecules of water. A few years later, engineers with the Portland Cement Association (Powers, 1949) began exploring the nature of capillary pores in concrete, which were residue from water-filled voids in the cement as it set up. In the early stages of hydration water was able to seep through cement paste to fill pores.

Terzaghi's work was soon reinforced by the work of Serge Leliavsky (1947), who calculated the contact area beneath dams to be 0.91%, and Douglas McHenry (1948) who reported contact values of ~100% from triaxial tests on concrete by Reclamation in Denver.

At Terzaghi's request, in 1948 Ralph R. Proctor prepared an article titled "The Elimination of Hydrostatic Uplift Pressures in New Earthfill Dams" for the Second International Conference on Soil Mechanics in Rotterdam. His subject was Bouquet Canyon Dam, the replacement for the ill-fated St. Francis Dam described earlier. It was the first time that uplift pressures had been measured for more than 15 years beneath massive earthen embankments, which were the first to be constructed with embedded instrumentation (Proctor, 1948), as well as the first to employ Proctor's new method of compaction testing, which became an industry standard. The take-away was that no dam or foundation material could be assumed to develop near-zero uplift, which was quite a departure from where the status quo had been parked two decades previous.

ASCE SUBCOMMITTEE ON UPLIFT (1939-51)

In 1920 New York water supply engineer Colonel Thomas H. Wiggin suggested that ASCE form a committee on uplift and ice pressure. Almost 20 years later Wiggin chaired ASCE's Committee on Research, and at the 1939 annual meeting in Chattanooga he was successful in having a motion approved to establish a committee on uplift, in large measure because of excessive uplift being measured in the newly completed Boulder (Hoover) Dam.

This became the Subcommittee on Uplift in Masonry Dams of the Committee on Masonry Dams of the Power Division. Placing the uplift group within the Committee on Masonry Dams was a deliberate move to encourage cooperation of leading designers and dam owners to supply uplift data from dams then in service.

The committee's members consisted of Ross M. Riegel, Chairman, and members John B., Alexander, William P. Creager, Leroy F. Harza, Ivan E. Houk, A.V. Karpov, Gerard H. Matthes, Byram W. Steele, James H. Stratton, and Thomas H. Wiggin. This was a very learned group, as many of them had published articles on the subject. They initially convened at the 1940 annual meeting in New York City, but were precluded from meeting face-to-face from 1942-45 because of travel restrictions imposed by World War II. Their meetings reconvened in 1946 and concluded in 1951 with their final report "Uplift in Masonry Dams" published in the ASCE Proceedings as Separate No. 133 in June 1952, and in the society's annual Transactions of 1952 (Paper 2531 in Volume 117:1218 – 1252). The committee's report was broken down into 11 subject headings with three appendices, five recommendations, and five areas where more information was needed. These are briefly summarized below.

The report began with Topic #1 Failures of Dams, which only acknowledged the failure of three masonry gravity dams in the United States up to that time (Austin Dam in Texas in 1900; Dam No. 26 on the Ohio River in 1912; and the St. Francis Dam in 1928). There was no mention of the Bayless Dam failure in 1911 or the failure of Sweetwater Dam in 1916, among others. Topic #2 was Existing Dams, which noted that many dams were constructed without any consideration of uplift forces, cutoff walls, or tensile strength of their foundations. They also noted the exemplary stress distributions recently recorded at Shasta Dam.

Topic #3 was Uplift Pressures. This introduced the concept of the Area Factor, A, with a value of 100% being the maximum and zero being the minimum, beginning with the total height of the water column behind the dam, and the zero figure being the tailwater elevation just downstream of the dam's toe. The other variable is the Intensity Factor, I, which indicates how much of the theoretical uplift pressure is actually expected or recorded at the interface between the foundation and the dam structure. A 50% intensity factor indicates about half the theoretical pressure has actually developed. Sometime this can be because excess seepage is able to flow down-gradient through natural fissures in the foundation or embedded drains in the dam.

The committee produced a generic "Diagram of Uplift Intensity with Various Factors," shown in Fig. 6. This is similar to the plots Ivan Houk introduced in 1930, but facing in the opposite direction. These diagrams allowed data from multiple dams to be plotted over one another, so the viewer can appreciate the data scatter, which tend to follow lines of equal intensity factors. The sudden change in slope occurs along the "Line of Drains," where the uplift relief wells are located. This is normally beneath the lowest inspection gallery.

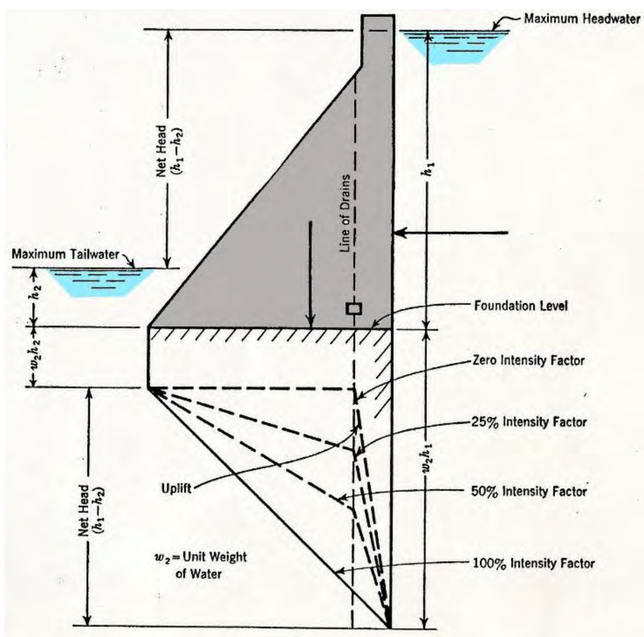

FIG. 1.—DIAGRAM OF UPLIFT INTENSITY WITH VARIOUS FACTORS

Figure 6. Diagram showing Uplift Intensity in percent of theoretical uplift pressure verses various other factors.

Topic #4 was Importance of Exploration and Geological Study. The uniqueness of foundation conditions was becoming more apparent with time because such conditions could cause the intensity and area factors to suddenly shift up or down between adjacent drains, without any apparent relation to the dam structure. This is what occurred at Hoover Dam because of some manganese-rich zones of fault gouge, and the converse at Shasta Dam, where resistant strata dipping upstream caused the dam's resultant thrust to be shifted significantly upstream, making it a very conservative design.

Topic #5 was Evidence as to Area Factors. In this discussion, the Subcommittee acknowledged the pioneering work of Terzaghi, Leliavsky and McHenry, described above. The committee cited measurements by the Tennessee Valley Authority (TVA) that showed area factors of 0.90 for marble and 0.75 for three different types of granite, which lent support to the experimental work. They concluded by stating "…most of the members of the Subcommittee believe that as a practical matter all new dams should generally be designed with an assumed factor of 1.0."

Topic #6 was Evidence As to Intensity Factor in Foundations. The committee members assuaged that the first method of reducing the intensity factor I should be employing a grout curtain to form a seepage cutoff, and employ trial drilling and grout-take programs beforehand. They also recommended the use of drainage holes drilled from the same lower heel gallery as the grout curtain, to intercept seepage that manages to penetrate the grout curtain. They then addressed clogging of internal drains by carbonate, alkali salts, and solids, including trash, etc. They then commented on the importance of internal galleries to monitor drainage over long periods of time.

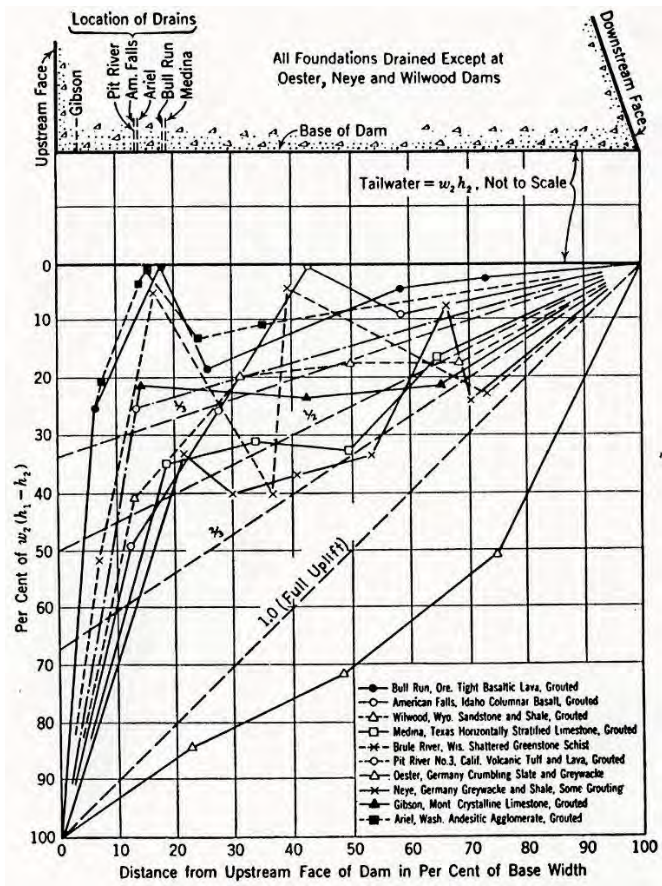

Figure 7. Observed uplift pressures under 10 gravity dams founded on rock foundations. Note the very high values recorded at Oester Dam in Germany, which were greater than 100% (full uplift). This dam is founded on crumbling slate and greywacke, whose particles may be clogging fissures, joints, and other discontinuities.

Topic #7 was Evidence As to Intensity Factor in Foundations. The committee cited the work of Hinds (1929) and Hauk (1930b), as well as other dams in Germany and the United States.

These data exhibited for more variation than any similar plots in their report, as shown in Fig. 7. Note the locations of drains in the dam's upstream heels. Most of these dams were exhibiting between 50% and 67% uplift, likely because of rock fissuring and joints.

Topic #8 was Assumptions for Design. This portion of the Subcommittee report described the basic assumptions relevant to uplift that the major dam-building agencies had come to employ because of measurements in their dams. These included the Bureau of Reclamation (USBR) in Fig. 8-left and the TVA in Fig. 8-right, with a written summary of the Army Corps of Engineers practices. These plots focused on the uplift intensity developed in the foundation rock beneath concrete gravity dams and assumed an effective grout curtain to curtail seepage, while employing uplift relief drains not more than 3 m (10 ft) apart immediately downstream of the curtain (dotted lines in Fig. 8-right). Reclamation's average of nine dams exhibited an uplift intensity of about 33% (1/3) effective uplift. TVA's four highest dams assumed their area factors to be 67% (2/3), except Fontana Dam (which was 1.0). Their dams were summarized in the "composite pressure line," while their "assumed design gradient" envelope was based on the assumption of existing (low) tailwater levels. These could be much higher during a flood event. That line does capture the high uplift intensities developed beneath the dam's upstream heel, which extends about 15% of the dam's basal width. It was noted that Owyhee and Hoover Dams were the world's highest dams when constructed, and were both founded on fractured volcanic rock with "innumerable joints and fissures" which were difficult to grout with certainty.

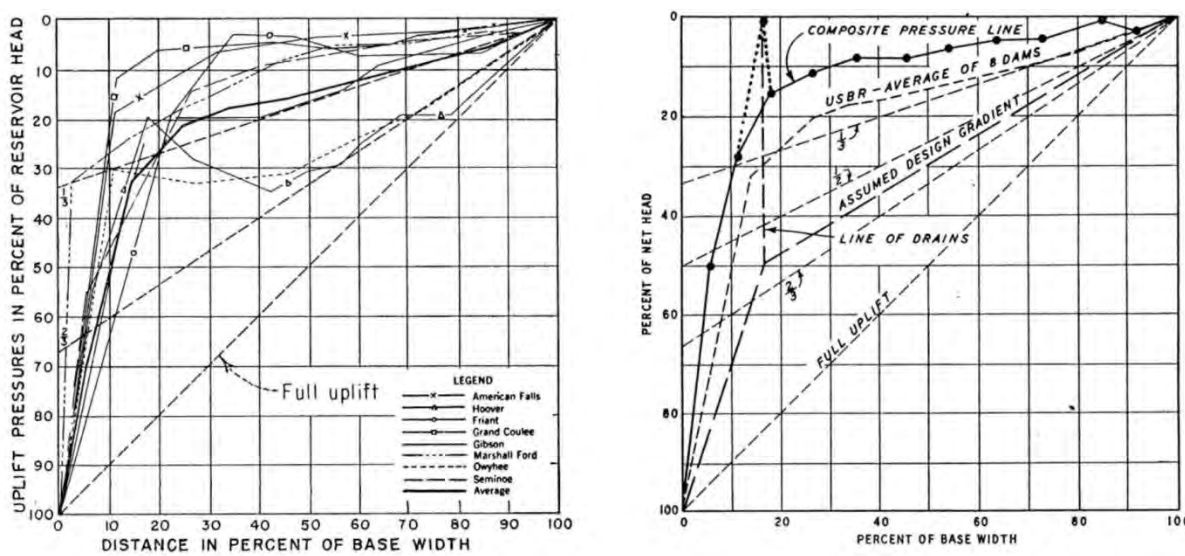

Figure 8 (left) presents uplift data for Reclamation's largest dams compiled by Keener (1949), with Hoover and Owyhee Dams yielding the highest values. Figure 8 (right) shows the composite foundation uplift intensity for TVA's four highest dams and Reclamation's average line (from Subcommittee, 1952).

Topic #9 was Design of Dam Above Foundation. The Subcommittee appreciated that the dimensions of any gravity masonry dam would be determined by the uplift assumptions for the foundation upon which it rests. Any reductions in uplift area or intensity were only justified if sufficient subdrainage measures were employed in the design and construction of the dam. There were many forms of drainage that could be employed, including open joints in the structure to safely convey seepage past the structure. The question of developing uplift pressures within the concrete described by Terzaghi in 1945 was not addressed by the Subcommittee.

Conclusions and Recommendations comprised Topic #10. The Subcommittee began by asserting that the report's real value was the data and measurements it contained, and that they were not making any attempt to codify design procedures. Most of the members agreed that in the design of new dams or hydraulic structures it was advisable to assume an area factor of 1.0, and that the systematic inspection and maintenance of drainage systems was of great importance. They also recognized the significance of internal inspection galleries near the base of the upstream heel, from which observations and measurements could be made, and that grout curtains or drains could be repaired or extended from these passages, if need be. With release of their report the Subcommittee felt its work was complete and that the group could be disbanded.

Topic #11 was Need for Information. The Subcommittee stressed that their work was of a preliminary nature, and that much more data needed to be collected to evaluate the actual intensity of pore pressure within concrete structures and what measures might be taken to limit their build-up. They also acknowledged the need for more data on the permeability of concrete, of heterogeneous rock masses, and the effective area in structures and foundations. And, they felt that the methods of measuring these factors at Holtwood Dam (Gisiger, 1938) should be more widely applied.

The report had a few detractors, including its founder Thomas E. Wiggin and Karl Terzaghi. They felt that the Subcommittee's assumptions about the long-term efficacy of grout curtains was unfounded and that the committee should be maintained to coordinate the collection and publication of more uplift data. The vagaries of grout curtains beneath embankment dams was a subject that would resurface years later (Casagrande, 1961; Chadwick et al., 1976).

CONCLUSIONS

The Subcommittee on Uplift included representatives from the Bureau of Reclamation, Army Corps of Engineers, and the TVA. Some fundamental research on mass concrete permeability was performed by Berkeley's Raymond E. Davis and Roy W. Carlson for the Corps of Engineers in the late 1940s, after an article on "The Significance of Pore Pressure in Hydraulic Structures appeared in late 1947 (Harza, 1947). Experiments were carried out at Berkeley on cylinders of concrete. In a summary of this work a few years later, Carlson (1957) opined that it would take about 1,000 years for a 122 m (400 ft) high concrete gravity dam to become fully saturated, if it contained fine sand in the aggregate. Very few concrete gravity dams were built after 1952, as arch, embankment, and concrete-faced rockfill dams dominated because of their economy.

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