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SOME UNUSUAL ASPECTS OF DAM SAFETY STUDIES IN WESTERN UNITED STATES (*)

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INTRODUCTION

The safety of dams in the western United States and particularly in California, has been a matter of public concern since 1928 following the failure of St. Francis Dam in Los Angeles County, California, which cost the lives of about 450 persons and millions of dollars in property damage. In 1929 the California State Legislature enacted legislation providing for state supervision of all non-Federal dams with regard to public safety, and a State unit, predecessor of the Division of safety of Dams was established to enforce the laws. On December 14, 1963, the dam impounding water in Baldwin Hills Reservoir in Los Angeles failed and brought devastation and loss of life to the communities in the path of the flood. As a result, more stringent regulations were enacted with regard to safety measures and classification of structures which are subject to the jurisdiction of the State of California. The spectacular failures of Malpasset and Vaiont dams in Europe further stimulated interest and public concern in the safety of such structures. In December, 1965, the Federal Power Commission issued the now wellknown Order 315, requiring safety investigations to be made of all licensed dams, by independent consultants, at intervals not greater than five years. Recently a Task Com-

(*) *Quelques aspects inhabituels de l'étude sur la sécurité des barrages dans la région Ouest des Etats-Unis.*

mittee of the United States Committee on Large Dams has completed the draft of a Model Law on Dam Safety which is being circulated to all 50 of the United States for review, comment, and possible adoption. In the United States, and particularly western United States, the administrators, designers, constructors and operators of dam projects are becoming truly safety conscious. Figure 1, plotted from

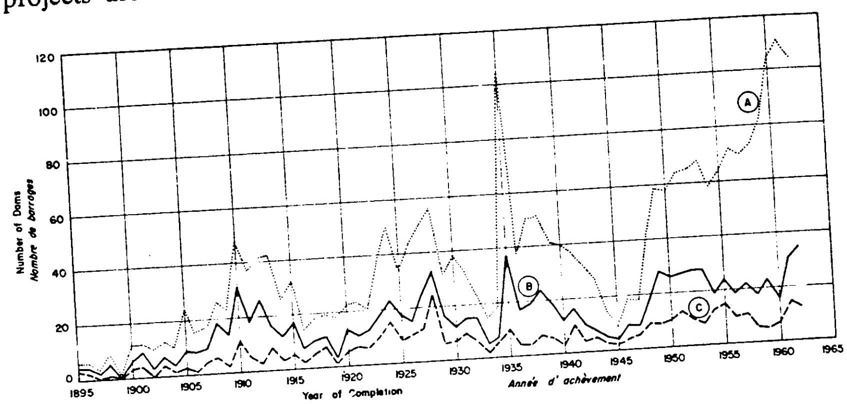


Fig. 1

Dams in the United States over 20 high above general foundation level.

(A) Total Constructed in 50 States and Puerto Rico.
(B) Total in Western States (California. Oregon, Washington, Idaho, Utah, Nevada, Arizona, Montana, Wyoming, Colorado, New Mexico).
(C) Total-California.

Barrages situés aux Etats-Unis ayant plus de 6 mètres de hauteur au-dessus du niveau moyen des fondations.

(A) Total construit dans les 50 Etats et Puerto Rico.
(B) Total dans les Etats de l'Ouest (Californie, Oregon, Washington, Idaho, Utah, Nevada, Arizona, Montana, Wyoming, Colorado, New Mexico).
(C) Total en Californie.

data published in Reference [1] shows the number of dams of all types over 20 feet (6.1 m) in height completed during the years 1895 through 1963, in (A) the United states and Puerto Rico, in (B) the western states of California, Oregon, Washington, Idaho, Utah, Nevada, Arizona, Montana, Wyoming, Colorado and New Mexico, and (C) in California alone. It may be noted that from 1895 through 1945, the dams constructed in the western states represented a substantial percentage of the total in the United States as a whole. Since 1945, he trend of construction of dams in the west has remained about as it was since the early 1900's, but in other areas of the United States the rate has increased markedly. Hence, consideration of safety of dams, which had been for many years centered largely in the west, has become a matter of importance in other areas as well.

A great many of the older dams in the west were built in areas originally remote from centers of population, usually to supply water for hydroelectric power, mining operations or as distant sources of water supply for cities and towns. Consequently, the hazards of failure were not great, beyond damage to, or complete loss of the facility itself. During the last several decades, however, and particularly since the mid-1940's after World War II, there has been a tremendous influx of population to the western states which, together with the greatly accelerated rate of population increase throughout the United States, has resulted in major centers of population and industry being located adjacent to, or in valleys below many of the older dams in the area. In such a situation, a failure of any dam could be catastrophic. Compared with modern practice, the design and construction techniques used on many of the aging western dams were primitive; important information regarding design criteria, material characteristics, foundation conditions, repairs and maintenance in past years, etc., is often available at present only in a fragmentary state. Some dams show visible evidence of considerable deterioration, while in other cases only detailed examinations will reveal subtle changes in condition which may pose a serious threat to the safety of the structure. Prompt and thorough investigation of every factor which could affect the integrity and safety of such structures, is mandatory. Each case is unique, and safety investigations in depth often require a high order of ingenuity to establish pertinent basic data, develop analytic approaches applicable to the situation, and arrive at a valid appraisal of the present condition of the structure with respect to stability, integrity, and safety.

SCOPE

During the last decade the author has been involved with studies in detail of the safety of many major dams, mostly located in the Pacific Coast states. This group has included compacted earth dams and embankments; hydraulic fill dams; rock fill structures of many types, including those with facings of soil, timber or concrete; concrete gravity dams; single arch structures; multiple arch dams; and a number of composite structures consisting of combinations of the aforementioned types. Some of the dams which were investigated were relatively new; others were more than 65 years old. The majority required only very minor modifications and repairs, or none at all, to meet acceptable safety criteria. The deficiencies of more serious nature disclosed in the remainder may be included in the following general categories :

a) Inadequate spillways and outlet works.

The usual practice when many of the dams surveyed were originally designed, was to size the spillway solely in accordance with the judgment of the engineer. He usually chose as the design flood some multiple, such as 1.5 to 2.0 times the maximum flood of record at that time. Many of the early records were of doubtful validity, or the adopted maximum flood was based on only a short period of authentic record. Modern hydrological methods demonstrate clearly that the probable maximum flood which should be considered can far exceed the spillway capacity of many of the older dams. The import of such a finding for any particular dam would depend upon the type and condition of the structure with regard to its ability to withstand overtopping ; the geologic conditions at the site ; the volume of impounded water which could be released ; in some cases, the rate of release of impounded water ; and the potential for damage and loss of life downstream should the structure fail.

b) Inadequate stability when subjected to major seismic forces.

Since the western states, and particularly the Pacific Coast states, are within a region subject to frequent and sometimes violent earthquakes, the stability of dams and appurtenant structures under the action of strong seismic forces, must be carefully and fully evaluated. It is only in recent years, however, that technical advances have given dam engineers a clearer insight regarding the magnitude and action of seismic forces, and provided them with analytical methods for approaching the problems of dynamics in dam design. Much still remains to be learned in this area, but even at the present stage of development, the analytical techniques certainly yield far better approximations of stresses and deformations in structures subjected to seismic loads than were formerly possible when methods, such as arbitrarily increasing static loads by some chosen percentage as an " earthquake allowance ", were in vogue. The aforementioned cases which have been investigated disclosed that in some instances not only were the original design criteria for seismic stability deficient, but changed conditions since construction, such as deterioration of materials, increased seepage and uplift, or changes in operating conditions, have further lowered resistance to seismic shock.

c) Foundation problems.

It is not unusual for dam sites in western United States to be adjacent to, or traversed by prominent faults in the bedrocks. Even though geologic evidence may indicate such faults to be ancient and dormant, prudence dictates that they be considered subject to reactivation during the life of the structure built thereon. The catastrophic failure of Baldwin Hills Dam was attributed to movements along known but supposedly inactive faults in the foundation bedrock beneath the structure. Several major dams in California have built-in provisions to accommodate significant movements along faults beneath the structures. A dam safety study must, therefore, give special consideration to such fault movements and evaluate the efficacy of means provided to counteract them ; or, where none are provided, to assess the probable effects on the structure should movement occur. Though not distinctly faulted, the bedrock may be highly jointed and fractured and adequate attention must be given to provisions to control leakage and underseepage. At some sites, soluble deposits of gypsum in foundations can create stability and seepage problems. Weak shales, claystones and siltstones are often present. The highly contorted formations of the Coast Range are notoriously troublesome with respect to foundation design and construction, and tunneling operations. Large areas of the western states are semiarid, and when some formations in these areas become saturated for the first time in a long geologic history, the loss of strength is significant, movements of structures founded upon them can occur, and landslides can develop around the periphery of the reservoir.

d) SEEPAGE AND DRAINAGE PROBLEMS.

Problems of this type encountered in safety investigations usually relate to either control of seepage through the dam itself, or to leakage through the foundation and abutments. In some instances both occur and are interrelated. A wide variety of situations can occur, as the case histories cited below will indicate.

e) DETERIORATION OF CONCRETE.

The techniques for manufacture and placement of sound, enduring concrete as they are known today, were only in the fomative state when a number of the dams under investigation were built. The quality of the concrete was sometimes erratic and inferior with respect to its resistance to wetting — drying and freezing — thawing cycles, but one of the most conspicuous causes of disintegration is alkali-aggregate reaction. Reactive aggregates are common in the western states, and a number of important structures which were built before the significance of alkali-aggregate reactivity was fully realized are at present exhibiting marked deterioration.

Dam safety investigations must of necessity be broad in scope and entail thorough, detailed examination of every aspect of design and performance of the dams and reservoirs from initial concept, design, and construction ; through current status ; to prediction of future behavior and performance. Frequently, the engineering work becomes more difficult and involved than that required for the original design of the structure. For example, in dams which have suffered serious deterioration because of alkali-aggregate reactivity, the expansion of the concrete has caused gross cracking of the structures, intimate microfracturing throughout the concrete, distortion, and substantial movements. Obviously, these structures are no longer " finite elastic bodies " as normally considered in stress analyses, so different concepts of behaviour must be considered and appropriate approaches to analyses devised.

EXAMPLES

The following cases have been selected as examples of the wide variety of situations which have been encountered in safety studies of dams in recent years.

CASE A. — *Concrete Arch Dam, height 190 feet (58 m), crest length 620 feet (189 m), built 1949, location California.*

This dam is located in a geologically complex area of folding and faulting, and is constructed on nearly vertical strata of fairly strong sandstone with interbeds of weak shale and soft sandstone. At the right abutment, the weak interbeds of shale are nearly normal to the line of thrust of the arch ; but in the left abutment, the weak shale and shattered sandstone strata are at an acute angle with the thrust of the dam. The channel section is crossed by a 150-foot (45.7 m) wide zone of crushed sandstone, shale, and fault gouge.

In 1964, when the safety studies were initiated, the records indicated that the left abutment had yielded about 1.0 inch (2.54 cm) in the direction of arch thrust and about ¼ inch (0.64 cm) in a downstream direction, and the right abutment had yielded about ¼ inch (0.64 cm) in the direction of thrust. Records also showed that shortly after the first filling of the reservoir, the crown of the arch had moved downstream about 1 ½ inches (3.82 cm) ; had remained in that position for seven years, then began to move progressively upstream until in 1964 the center point of the crown was 3/4 inch (1.91 cm) upstream and ¼ inch (0.64 cm) toward the right abutment from its initial position. It was later ascertained that the upstream movements of the crown and yielding of the abutments were caused by the expansion of the concrete due to alkali-aggregate reactivity. The upper part of the dam had expanded at a considerably greater rate than the lower part, resulting not only in warping the arch in cross section but doubtless inducing varying degrees of stress change and movement from top to bottom of each abutment. As mentioned, the geologic investigation disclosed the quality of the entire foundation to be poor and the left abutment to be less competent to resist the arch thrust than the right abutment, so the directions and relative movements of the abutment sections of the arch were consistent with the geologic findings. A comprehensive investigation was made, which included numerous core borings, physical tests, soniscope measurements to determine the rate of transmission of ultrasonic pulses through sections of the dam as an index of the approximate dynamic modulus of elasticity of the concrete, petrographic examinations of typical specimens, and detailed research regarding the cement and aggregate and the manufacture and placement of the concrete from all available records. Because of the concern as to the probable continued slippage and lateral movement at the abutments, four strain meters were installed in each abutment to monitor such movements.

It was found that all of the concrete in the dam had experienced a chemical reaction between the alkalies of the cement and certain siliceous constituents present mainly in the fine aggregate. The damaging effects of this reactivity were evidenced in the volumetric expansion of the concrete, the surficial pattern cracking, intense internal microfracturing, and the accompanying reduction in strength of the concrete and diminution of its elastic properties. By far the most serious damage occurred in portions of the upper 25 to 30 feet (7.62 to 9.14 m) of the arch. In this critically weakened zone, the compressive strength of the concrete had generally decreased to about one-third of its original value. In some zones, recovery of intact specimens was impossible.

Comprehensive stress analyses were performed, which took into account the effects of all discernible gross cracking, variations in strength and elastic modulus, and stresses induced by the volumetric expansion. It was found that under certain conditions of loading, the safety factor of the upper zone of the arch was unacceptably low and would continue to decrease because the alkali-aggregate reaction was incomplete and would further progress. In the interest of safety, the need for remedial action was obvious and several possible measures were considered. It was finally decided to reduce the stress on the dam, relieve the loading on the apparently overstressed abutments, and remove the dangerously deteriorated upper lifts of concrete by cutting away the entire central portion of the top of the arch and permanently lowering the crest about 25 feet (7.62 m). This work has been completed and monitoring of the remaining dam is continuing.

CASE B. — *Concrete Arch Dam, height 187 feet (57 m), crest length 253 feet (77 m), built 1938, location California.*

Within two years after completion of this dam in 1938, the development of an extensive pattern of cracking and movement was noted. It was immediately subjected to a rigorous program of investigation and testing which established that the distress was due to a reaction taking place between alkalis in the cement and certain reactive minerals in the aggregates. The predominant reactive minerals were later determined to be andesites and rhyolites which constituted 8.0 % by volume of the coarse aggregate and 5.4 % of the fine aggregates. The total alkali content of the cement was 1.44 %. Extensive studies were made of the concrete in 1940, 1942, 1945, and 1950. The latest comprehensive study of the structure, which included every aspect relating to stability, integrity, and safety of the dam, was made in 1966. The surficial pattern cracking had progressed to the point where large areas of the structure appeared to be composed of a mosaic of irregular, interlocking blocks of one to two square feet (0.09 to 0.18 m^2) of face area. This type of cracking apparently did not extend to great depth and resulted from the concrete in the interior of the mass having expanded slightly more than the near-surface material. A number of larger diagonal tension cracks up to ¼ inch (0.64 cm) in width had developed near the left abutment.

Excellent records were available of all movements of the dam from 1938, the year of completion, through 1965, when the last comprehensive analyses were made (and are continued up to date). The expansion or autogenous growth of the dam caused the arch to bow and the crown to move upstream, since the dam is rigidly constrained at the abutments. The maximum deflection of the crown near the top of the dam had increased 5.6 inches (14.2 cm) from the maximum downstream position in March, 1939, to a mean position in recent years; but, after 1958, there had been hardly any net increase attributable to autogeneous growth. The only measurable movement since 1958 had been a cyclic deflection reflecting the annual temperature changes as manifest throughout the period of record. The corresponding vertical movement at the crest was about 3.9 inches (9.9 cm) at the highest section of the dam, and varied along the crest in proportion to the height of the dam. The warping of the structure in cross section was measured with reference to a permanently installed plumbline. Numerous physical and soniscope tests, and petrographic examinations were made of the concrete to determine its characteristics and rate of regression of quality during the period of record. The compressive strength of uncracked concrete had decreased from about 4500 psi to about 3 000 psi. (316 kg/cm^2 to 210 kg/cm^2).

Several methods were used in performing the stress analyses of the structure. In one of these, the assumption was made that the expansion would eventually reach an ultimate value which would result in the dam being so completely cracked as to be composed of a mosaic-like structure of irregular, interlocking blocks, and be incapable of developing any tensile stresses. The dam was designed to impound water at a practically constant, unvarying reservoir level ; and, at this operating level, the maximum compressive stress in the hypothetical cracked dam would be about 455 psi (32 kg/cm^2) or about 15 % of the current average ultimate concrete strength.

To arrive at a more realistic determination of the actual state of stress in the dam, taking into account all known pertinent factors affecting the magnitudes and distribution of stresses, a finite element method of analysis was developed. All foreseeable static and seismic loading cases were investigated. The effects of the pattern cracking, all gross cracks which could be located by survey, stresses associated with both the horizontal and vertical components of expansion of the concrete, creep in the concrete, temperature changes, changes in concrete strength and modulus of elasticity and the elastic properties of the abutment rock, were all accounted for in the analyses. In order to investigate time-dependency effects, one series of step-by-step analyses was made starting with the dam in its original uncracked and undistorted condition in 1938. This included : (1) performing a complete stress analysis, (2) assuming cracks to form wherever the tensile stresses exceed about 150 psi (10.5 kg/cm^2), (3) assuming expansion of the concrete to occur so as to match the recorded movements of the dam, and (4) repeating the analyses step-by-step throughout the 27 years of record to arrive at the 1965 state of stress. Many other analyses made on a one-step basis wherein the total measured deflection was matched and cracks assumed to develop in high tension areas, were found to yield results compatible with the lengthy step-by-step trial series. For the most severe loading condition of full hydrostatic plus seismic forces, the maximum compressive stress by the finite element method was found to be 1744 psi (122 kg/cm^2), or 58 % of the average ultimate concrete strength.

Since the movements due to expansion of the concrete had apparently ceased, even though the alkali-aggregate reaction was determined to be incomplete, it was considered that the forces due to expansion, creep and hydrostatic pressure had reached a state of equilibrium ; and, as long as the water level in the reservoir was not substantially decreased for a considerable period of time, the autogeneous growth would probably remain dormant. From the results of the stress analyses and all the other related geologic, hydrologic, hydraulic, and structural studies that were performed, it was concluded that although the dam had apparently undergone considerable deterioration over the years, it was safe and, providing there would be no significant change in loading, it could remain in service under the close surveillance which it had received in the past.

CASE C. — *Concrete Gravity Dam, 328 feet (100 m) high, completed 1934, location California.*

The foundation rocks at this dam site are primarily granodiorite and diorite gneiss ; and, although the rocks are strong, the area is highly faulted and intensely jointed. There are at least eight faults of considerable size and numerous small shears passing through the foundation. These faults have crushed zones ranging from a fraction of an inch to more than five feet (1.53 m) in thickness. The dam is located in one of the most active seismic areas of the country, so these faults must be considered to be potentially active. One major fault through the foundation was singled out for special treatment in the design of the dam. A unique open joint with vertical sliding planes from top to bottom of the dam was provided to allow for earthquake and fault movements along the major fault. This joint crosses the axis of the dam at an angle of approximate 45° and is oriented in the direction of anticipated movement as shown by the strike, dip, and striae of the fault. The maximum total motion which could be accommodated along the direction of anticipated movement is 6 feet (1.83 m).

The dam was well designed, carefully constructed and thoroughly instrumented. During approximately the first 20 years of service only very minor and normal movements of the dam were recorded. Following this period, some small displacements and relative movements between monoliths of the dam were noticed, particularly at the articulated joint. The movements were complex, and in 1961 they appeared to undergo considerable acceleration. Joints between monoliths closed tightly and a conspicuous offset developed at the articulated open joint. In 1964 a detailed study was conducted to analyze the complex movements of the dam and arrive at their cause and significance.

Three possible causes of the movements were investigated in depth, (1) movement of the foundation, (2) movement of the abutments, and (3) autogenous growth. These investigations finally eliminated movements of the foundation or abutments as the causes of the movements in the dam, and found the source to be a very mild form of autogenous growth caused by a slow reaction of the cement alkalies with the silica of the aggregates. This type of reactivity differs markedly from that of the other two cases cited above in that it appears to progress very slowly, is only mildly active, and may be delayed or show no evidence of its action for many years after placement of the concrete. Tests demonstrated conclusively that the concrete in the dam was still in excellent condition and the superficial signs of possible distress had no significance whatever with respect to the safety of the structure.

Another problem requiring attention in the studies of this dam concerned spillway capacity. At the time of design, the spillway was to accommodate twice the then of flood of record. However, only three years after completion of the project, the flood of record at the time of design was not only exceeded but came close to the maximum spillway capacity. Had the dam been designed three years later, and if the same criteria were used, the spillway would have had twice its present capacity. It is interesting to note that, under those circumstances, the capacity would have been quite close to the maximum probable flood as estimated during the above-mentioned investigations. Since the dam would be overtopped should the maximum probable flood occur, the stability analyses were re-evaluated on the basis of this overtoping, and the dam was found still to be stable and no modifications were required in this respect.

CASE D. — *Hydraulic Fill Dam, 91 feet (28 m) high, crest length 830 feet (254 m), constructed 1918, location California.*

The method of construction of this dam was somewhat unique. The materials were excavated in the borrow areas by means of high-pressure water jets and transported and placed in the dam through a system of multilevel sluices. The resulting configuration and approximate zoning of the structure is shown in Figure 2. A layer, approximately 15 feet (4.65 m) thick, of native pervious sand and gravel overlying bedrock was left in place beneath the base of the dam, but a cutoff of impervious core material was extended through this layer to bedrock. The bedrock is a hard, fractured, metamorphic greenstone.

Seepage at the downstream toe and groin lines of the dam had been a continuing problem for many years. As part of a safety investigation conducted in 1964, a number of exploratory holes were drilled and piezometers installed to measure seepage pressures in the downstream section of the embankment and foundations. It was discovered

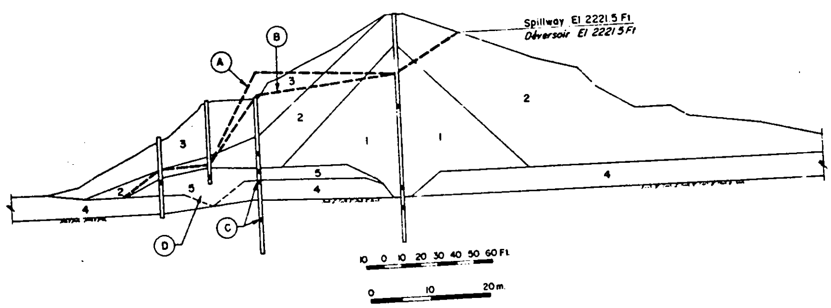


Fig. 2

Section of dam and foundation. Case D.

(A) Piezometric Line within Foundation.
(B) Piezometric Line within Embankment.
(C) Piezometer tip.
(D) Inferred cutoff.

Zone	Materials
1	Silts & Clays.
2	Gravelly Silts & Clays.
3	Silty & Clayey Gravels.
4	Gravels & Sands (Overburden).
5	Silts & Clays (Overburden).

Coupe de l'ensemble barrage et fondation. Cas D.

(A) Ligne piézométrique intérieure des fondations.
(B) Ligne piézométrique intérieure de berge.
(C) Tête piézométrique.
(D) Coupe estimée.

Zone	*Matériaux*
1	*Limons et argiles.*
2	*Limons et argiles graveleux.*
3	*Graviers limoneux et argileux.*
4	*Graviers et sables (surcharge).*
5	*Limons et argiles (surcharge).*

that unusually high seepage pressures had developed in the embankment and foundation as shown in Figure 2 by Piezometric Line B for the embankment and Piezometric Line A for the foundation. It was apparent that reservoir water was bypassing the central cutoff through fractures in the foundation rock. A partial cutoff through the gravel layer near the downstream toe resulted either from a pinching out of the gravel bed, or the presence of a remnant of a starter dike used in the initial placement of hydraulic fill. Stability analyses indicated the computed safety factor of the downstream slope to be precariously low under normal full-pool conditoins, and less than unity in the event of an earthquake with a maximum acceleration of only 0.1 gravity. The need for immediate remedial measures was evident ; and a number of alternative schemes were considered for relieving the inordinately high seepage pressures by drainage, grouting the bedrock under the core, or buttressing the downstream slope by construction of a substantial rock berm. The latter method proved to be the most feasible, and a rock fill of sufficient size to develop an ample safety factor was placed on the downstream slope to permit the dam to remain safety in service.

CASE E. — *Concrete Gravity Dam, height 50 feet (15.30 m) (concrete ogee section 32 feet (9.75 m) high, steel frame flashboard structure 18 feet (5.50 m) high), constructed 1915, location Montana.*

An investigation was made of this structure in 1966 as part of a routine safety check of a number of structures belonging to the same owner. The geologic conditions at the site are satisfactory and the dam in general is structurally sound. The spillway is capable of passing 1.4 times the maximum flood or record. The probable maximum flood, however, was calculated to be about 2.3 times the maximum flood of record, and would overtop the dam by about 2.4 feet (73 cm). Three core holes were drilled to sample the concrete and foundation, and to install piezometers to measure uplift pressures. Stability analyses were performed to determine stresses at the upstream and downstream faces, and overturning safety factors with respect to the base of the dam and a possible unbonded construction joint at about midheight of the concrete section. The calculated overturning safety factors for normal operating conditions were above 1.0 ; but, for the abnormal conditions of a probable maximum flood loading or that of normal full pool coincident with an earthquake having a maximum acceleration of 0.1 gravity, the calculated safety factors were slightly less than unity. Although the possibility of occurrence of the abnormal loading conditions was believed to be remote, it was considered prudent to take remedial measures to increase the stability against overturning and decrease the tension in the concrete at the upstream face. As shown on Figure 3, this was accomplished by installing post-tensioned steel

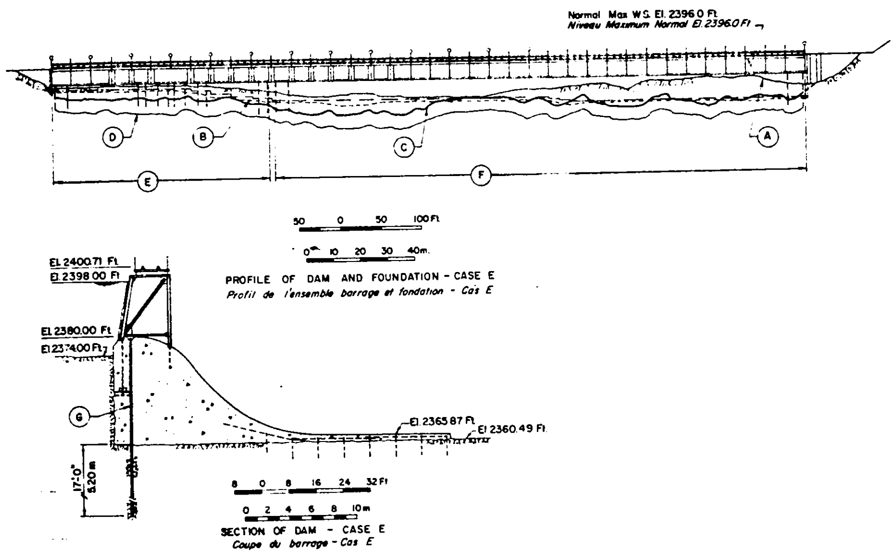


Fig. 3

(A) Upstream bedrock profile.
(B) Bedrock profile downstream side. Bottom at the toe.
(C) Bedrock profile under crest as determined by drilling.
(D) Bottom of Tendons.
(E) 11 panels, 2"-131 kip tendons per panel = 22 tendons.
(F) 27 panels, 2"-189 kip tendons per panel = tendons.
(G) Steel tendons.

(A) Profil amont du bedrock.
(B) Profil aval du bedrock extrémité avant de la semelle de fondation.
(C) Profil sous crête du bedrock (d'après forage).
(D) Pied des cables de tensionnement.
(E) 11 panneaux, 5,08 cm (131 000 livres, 60 tonnes) câbles par panneau = 22 câbles.
(F) 27 panneaux, 5,08 cm (189 000 livres, 85 tonnes) câbles par panneau = 54 câbles.
(G) Câbles en acier.

tendons spaced 5 feet to 12 feet (1.53 m to 3.65 m) on centers near the upstream face, which extended vertically through the dam and were anchored firmly in the foundation rock.

Case F. — *Earth and Rockfill Dam, height 85 feet (25.9), constructed 1939, location Montana.*

The investigations of this dam were made primarily to establish the feasibility of raising the crest elevation to increase the reservoir capacity. The evaluations resulting from the studies were influenced largely by the geologic conditions at the dam site and in the surrounding area.

The river where the dam is located is entrenched at varying depths below the general level of the surrounding country. The exposed rocks bordering the river valley are nearly horizontally bedded, stratified sedimentary formations, which include sandstones, siltstones, claystones, carbonaceous shales, and coal. The coal is sub-bituminous and several of the beds are of commercial thickness and quality. One coal bed in the area, penetrated by exploratory drill holes, varied from about 32 to 54 feet (9.75 m to 16.45 m) in thickness. Many thin beds are interspersed among the stratified siltstones and claystones, and in one 150-foot (45.7 m) interval above the river surface, four local coal beds are present ranging in thickness from one to four feet (30.5 cm to 122 cm).

One of the most striking features of the sedimentary formations results from effects of coal beds which burned many thousands of years ago. The heat from the burning coal beds locally changed the form of the adjacent beds, in some cases changing shales to red slaty or clinkery material ; fusing some of the claystones and siltstones to porcellaneous, slaty rocks ; and fusing some of the siltstones and sandstones into a cindery material closely resembling volcanic scoria. As the coal burned, voids were left which induced collapse of the overlying baked rocks, producing collapse breccia. Cooling of the baked rocks produced contraction joints and horizontal partings along incipient bedding in siltstone, which furthered breakdown of the rock. Because of the many fractures and joints, the baked rocks are extremely permeable, yet resistant to erosion because of their hardness. The coal fires burned at the outcrops ; and, where overburden was relatively thin, large surface areas were baked. Where the overburden was thick, the baking extended lesser distances underground but still often for several hundred feet. The vagaries of such baking are exhibited by abrupt lateral change from baked to unbaked rock.

At both abutments of the subject dam the geologic conditions are typical of those described above. In the right abutment there is less baked rock than in the left, but several prominent coal beds are present, one some 20 feet (6.09 m) thick. Much baked rock is present in the left abutment. In the spillway chute, which traverses the left abutment area, a remnant of the 20-feet (6.09 m) — thick coal bed is present. The valley floor is flat and has been filled with alluvial sand and gravel to depths of 25 (7.62 m) to more than 50 feet (15.30 m). The central section of the dam is founded on the alluvium.

The dam is an 85 - foot - (26 m) - high zoned, rolled earthfill structure with side slopes of 3 to 1 upstream and 2 ¼ to 1 downstream. The upstream half of the dam consists of an impervious core section with the outer slope protected by a 15-foot - (4.57 m) - thick blanket of baked rock and sandstone. Below the core, an impervious rolled-fill cutoff extends about 65 feet (19.80 m) through the valley alluvium to bedrock. The downstream half of the dam consists of a sand and gravel middle section and an outer pervious section of alternate layers of sand, gravel, baked rock, and sandstone. Foundation drains are provided under the outer pervious section. A chute-type, concrete-lined, spillway and a tunnel outlet works are provided at the left abutment area.

The dam has been plagued by seepage problems ever since the first filling in 1939. Corrective measures have included the construction of an additional 875-foot - (267 m) - long cutoff trench on the left abutment, considerable grouting, and extensive repairs and modifications to the spillway. For instance, in 1949 seepage caused a section of the spillway floor to raise about one foot (30.5 cm). On inspection it was found that a circuitous water passageway had developed under the slab which was large enough to admit a man. After this was repaired, no further problems developed until 1965 when large holes again formed in the spillway channel, which required major repairs.

After extensive investigations it was concluded that the dam could safely be raised the required 25 feet (7.63 m) ; but it would involve additional fill and drains on the dam, extensive blanketing of abutments with impervious fill, effectively sealing off the old spillway and construction of an entirely new spillway.

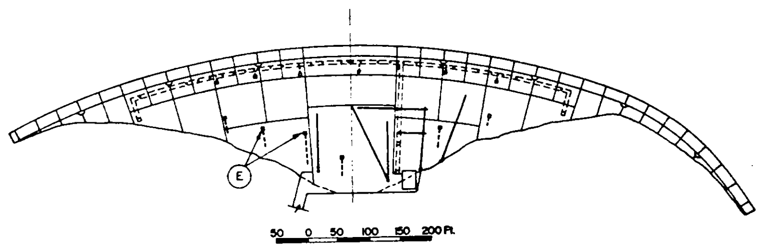
CASE G. — *Concrete Gavity Dam, height 358 feet (109 m), crest length 1337 feet (407 m), completed 1929, location California.*

The owners of this dam became concerned when it was observed that over the years the foundation drains that had been built into the structure were gradually discharging less and less water. If this decrease in flow were caused by a gradual silting and sealing off of the reservoir bottom and consequently less flow came through the joints and fractures in the dam foundation rock, the uplift on the base of the dam should be decreasing and the stability safety factor increasing. However, if the decreasing flow were caused by the drains becoming slowly plugged or blocked, then the uplift would be increasing and the stability decreasing. Prudence dictated that the situation be investigated and the safety of the dam be determined.

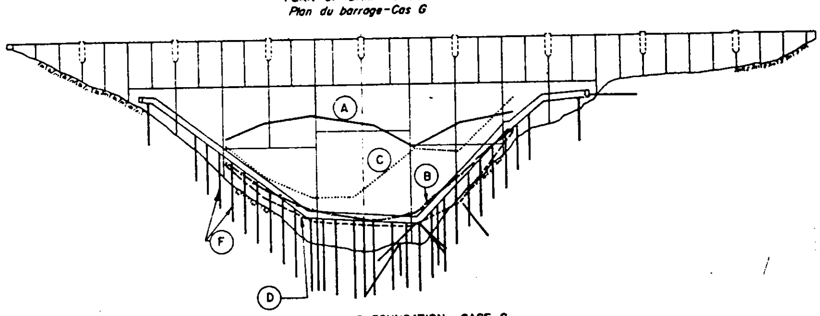
The original foundation drains consist of sections of porous concrete pipe placed at the contact surface between the foundation rock and concrete base. Each drain is connected to two vertical riser pipes which make a right-angle bend to discharge into a drainage gallery within the dam near the upstream face. The bedrocks at the site are hard, jointed, only moderately weathered greenstones ans slates of the Sierran metamorphic rock complex. The rocks are not permeable, but water does move in very complex routes along intersecting joint systems. A grout curtain extends some 30 feet (9.14 m) into bedrock upstream of the foundation drains. In order to measure the uplift at the base of the dam and pressures in the foundation rock at about

25 feet (7.63 m) below the base contact, 22 piezometers were installed in holes drilled through the concrete and into rock at strategic locations. The piezometer readings indicated that inordinately high uplift pressures existed over the entire base of the highest central part of the dam, and thorough stability analyses were mandatory.

Stability analyses using the original design uplift assumptions indicated 27.8 psi (1.95 kg/cm^2) compression at the heel of the dam and an overturning safety factor of 1.96 for normal pool loading conditions. When the measured uplift pressures were utilized in the same type of analysis, the stress at the heel was 68.8 psi (4.82 kg/cm^2) tension, and the overturning safety factor dropped to 1.25. Even more drastic changes in stresses and safety factors were indicated when seismic forces were also considered. The low safety factors made it



PLAN OF DAM - CASE G
Plan du barrage - Cas G



PROFILE OF DAM AND FOUNDATION - CASE G
Profil de l'ensemble barrage et fondation - Cas G

Fig. 4

— (A). Upstream uplift pressures before drainage.
—•— (B) Upstream uplift pressures after drainage.
······ (C) Downstream uplift pressures before drainage.
--- (D) Downstream uplift pressures after drainage.
▫ (E) Piezometer tips at base contact.
• | (F) Drain Holes.

— *(A) Sous pression amont, avant drainage.*
—•— *(B) Sous pression amont, après drainage.*
······ *(C) Sous pression aval, avant drainage.*
--- *(D) Sous pression aval, après drainage.*
▫ *(E) Têtes piézométriques au contact de base avec la fondation.*
• | *(F) Trous de drainage.*

apparent that prompt remedial measures were necessary. Schemes for reducing the uplift pressures, which involved both grouting and drainage, either separately or in combination, were considered at length. The grouting approach was finally abandoned in favor of drainage because a drainage scheme was considered to have greater probability of success in this case and did not entail the possibility of further plugging of existing drains by uncontrolled inflows of grout.

The required additional drainage of the foundation was accomplished by drilling 38 three-inch-(7.62 cm)-diameter drain wells from the interior gallery and downstream toe, as shown in Figure 4. After consideration of the geometry of the joint system in the foundation rock, it was estimated that the required depth of drain wells installed from the main gallery should be about 75 feet (22.9 m) into rock at the middle of the dam and reduce to 50 feet (15.3 m) toward the abutments. Slant holes of various lengths drilled from the toe, relieved local zones of high pressure caused by deep flows which bypassed the upstream well line. The new drain wells were very effective, as indicated on Figure 4, and the reduction in uplift pressure was as great as could possibly be achieved, since most of the new drains could discharge only into the existing drainage gallery in the dam.

Re-analyses were made of the stability of the dam based on the new uplift pattern, which showed the tension at the heel had been reduced to 2 psi (0.146 kg/cm²) and the overturning safety factor had been raised to 1.72. These results were considered satisfactory. The piezometers are read periodically to ascertain that no significant changes occur.

CASE H. — *Zoned Earth Embankment, maximum height 80 feet (24.4 m), crest length 5750 feet (1765 m), completed 1964, location California.*

This embankment or dike is part of a complex composed of a main dam and six dikes. At the time the project was designed in 1961, it was known that while most of the dike would be founded directly on competent bedrock, some 1500 feet (460 m) of the highest section would be founded on 8 to 12 feet (2,44 to 3.65 m) of impervious clayey sand and silty sand, below which pervious to semipervious, very weakly cemented or bonded sands and gravels extended to bedrock at a depth of some 90 feet (27.4 m). The foundation sands were erratically exposed in a river bank about 1000 feet (305 m) upstream of the dike, and the integrity of the impervious surface blanket was questionable. A positive cutoff to bedrock beneath the dike would obviously be very expensive, and might not be needed if the effective permeability of the deep sand formation were, in fact, low enough, and the overlying natural blanket efficient enough to limit the underseepage sufficiently to preclude the development of problems due to water loss or pore pressures in the foundation. It was decided to omit the positive cutoff in the original construction, compact the surface blanket within 1000 feet (305 m) upstream of the dike, extend a zone of impervious core material from the central core upstream beneath the upstream shell and berms in order to lengthen the seepage path, and observe carefully the performance of the dike when the reservoir was filled. Since the sandy foundation materials were known to be subject to considerable consolidation when saturated under load, 84 eighteen-inch-(46 cm)-diameter, gravel-packed saturation wells were drilled through the sand on about 100-foot-(30.48 m)-centers during the early foundation preparation work. The foundation area was then flooded by ponds created with 16-foot-(4.90 m)-high dikes, and the sand was saturated and partially consolidated before the dike was built.

After the project went into operation, the installed piezometers indicated that unfavorable pressure gradients did, in fact, develop in the dike foundation; numerous sink holes formed in the upstream blanket, boils appeared in areas downstream of the dike; and the seepage flows of about 1.5 cfs (0.042 cu.m./sec.) created a considerable nuisance to adjacent land owners. The principal concerns, however, were the effects of the foundation seepage pressures on the stability of the dike, and the possibility of boils developing to a stage that piping of fine materials from beneath the dike might occur. The safety investigations disclosed that the sink holes and boils resulted mostly from numerous animal burrows in the top blanket material, which persisted despite the compaction operations, and were of no great significance. However, when the gradients beneath the dike, as indicated by the piezometers, at the low reservoir levels which had been attained prior to and during the period of the investigations, were projected to full pool conditions, the forecast piezometric surface downstream of the dike would be well above ground level. This situation, together with the existing undesirable downstream seepage conditions, made it necessary to devise and install means for seepage control. Six schemes were considered, including relief wells in several configurations at the downstream toe, removal and recompaction of the upstream blanket, and the construction of a slurry cutoff trench along the upstream toe. The slurry trench method was finally chosen on the basis of cost and greater assurance of positive control. The slurry trench was located about 50 feet (15.30 m) upstream of the toe of the dike so as not to jeopardize the stability of the dike during construction. The slurry trench cutoff was connected to the impervious zone under the upstream shell by means of a compacted impervious blanket. The trench was about 8 feet (2.44 m) wide and reached a maximum depth of about 95 feet (29 m). During excavation, the trench walls were supported by a bentonite slurry. The excavated material, which was impregnated with the slurry, was mixed with gravel and the trench backfilled with the mixture. The completed cutoff proved to be very impervious. At full reservoir, a 60-foot (18.30 m) pressure drop occurred across the 8-foot-(2.44 m)-wide cutoff, and the piezometric surface was well below ground downstream of the dike. Analyses of the dike showed ample stability, and the downstream seepage problem was eliminated.

Many other cases could be cited which involve safety aspects of dams and reservoirs located astride major active faults; seismic shock resistance of lightly reinforced multiple arch dams; unique rehabilitation operations on unsafe outlet works; detailed safety analyses of unusual or outmoded types of dams which probably never before had been subjected to the detailed scrutiny necessary for assurance of safety; problems relating to settlement, seepage, slope instability and erosion through lack of adequate protection, and others, some of which are peculiar to the western environment and others more universal in nature.

COSTS

Forecasting the probable cost of conducting a comprehensive safety in investigation of a dam is inherently much less accurate than estimating the cost of engineering design work for a new structure of comparable size and type. It is usually impossible to predict the ramifications into which such studies might lead. In most cases, at least the preliminary investigations must be well under way before valid appraisals can be made of the necessary scope of the studies required to fully assess the condition of the structure and arrive, with confidence, at an evaluation of its safety. The ultimate costs of safety investigations, therefore, must be expected to vary widely; and experience has shown that they can range from about $ 5,000 to perhaps $ 75,000 or $ 85,000, with the average being about $ 30,000 to $ 40,000. Usually, about one-half of these costs are for engineering analyses ; and the other one-half for field work, test hole drilling, sampling, laboratory tests, installing instruments, or other data gathering activities.

LEGAL ASPECTS

The question is sometimes raised as to the responsibilities of the engineer charged with conducting a safety investigation of a dam. On the basis of such an investigation, if he reports the dam to be safe and it was later proved to be unsafe, and it partially or completely collapsed causing great damage and perhaps loss of life, does his liability extend beyond that normally assumed by a professional engineer in the design and construction of a new structure ?

The engineer in charge of a new structure presumably has full knowledge of all aspects of the design, characteristics and selection of materials, geologic and hydrologic conditions, construction operations, and all other matters which foreseeably could affect the safety of the structure. On the other hand, the engineer conducting a safety investigation of an existing structure, must glean whatever information he can from records (often incomplete and fragmentary) prepared by others; limited field explorations and tests; personal observations, and analyses which must of necessity be based on less complete technical data than are usually available for an original design. Despite the most careful investigations, it is possible for some subtle defects to remain undisclosed, which could reflect upon the general safety of the dam. Under these conditions, it would certainly seem unrealistic to expect the safety investigator to assume as much responsibility as that of an original design and construction engineer. Furthermore, the amount of compensation received by the safety investigator for his services does not justify his assuming any substantial liabilities as a result of performing the work.

The law requires that the professional engineer perform his services in accordance with the standards prevailing in the industry for the particular type of services being performed. Therefore, the safety investigator who complies with those standards should be free of any liability, even if it develops that there existed conditions which rendered the dam unsafe, so long as those conditions would not have been discovered had the safety investigator performed in accordance with those standards. As a practical matter, however, since the consequences of a dam failure are generally catastrophic, the safety investigator cannot, in the exercise of prudent business judgment, undertake a dam safety investigation without limiting his liability in such a manner that the risks are commensurate with the compensation he is to receive. The owner of the dam has a continuing liability to third parties for loss or damage which they may suffer as al result of a partial or total failure of the structure and, presumably, has established some program of insurance to protect against this liability. Since the safety investigation does not increase the liability of the dam owner, it would seem reasonable for the safety investigator to be afforded the protection of such insurance as the owner maintains for his own protection, subject perhaps to a proviso that the safety investigator will accept some liability for his failure to perform in accordance with professional standards, up to an amount commensurate with the compensation he is to be paid for his services. The risks inherent in this type of work are recognized in some jurisdictions, which have provided by statute that certain agencies will have no liability on account of failure of a dam or reservoir, or as a result of the operation of a dam or reservoir. For example, the State of California Water Code, Division 3, Part 1, Section 6028, referring to the Department of Water Resources, states :

"No action shall be brought against the state or the department or its agents or employees for the recovery of damages caused by the partial or total failure of any dam or reservoir or through the operation of any dam or reservoir upon the ground that such defendant is liable by virtue of any of the following:

a) The approval of the dam or reservoir.
b) The issuance or enforcement of orders relative to maintenance or operation of the dam or reservoir.
c) Control and regulation of the dam or reservoir.
d) Measures taken to protect against failure during an emergency."

Section 6029 states:

"Nothing in this part shall be construed to relieve an owner or operator of a dam or reservoir of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam or reservoir."

Section 6075 states:

"The department, under the police power of the state, shall supervise the construction, enlargement, alteration, repair, maintenance, operation, and removal of dams and reservoirs for the protection of life and property as provided in this part."

No similar legislation exists to limit the liability of a private engineer or consultant performing services in connection with dams or reservoirs.

In nearly all cases, the engineer (independent consultant or engineering firm) is retained by the owner of the dam to perform the safety investigations for the owner's information or in response to requests by regulatory agencies for such studies. Since the basic responsibility for the safety of the structure is clearly that of the owner, the limitations of liability to which the engineer may be exposed should be explicitly covered in the agreements or contracts between owner and engineer for performance of the work. It should be stipulated that the engineer, in performance of his services in connection with safety investigations, shall utilize his best efforts and judgment as a professional engineer; but the parties to the agreement should understand that, as a professional engineer and for the compensation provided in the agreement, the engineer cannot be expected to expose himself to liabilities unlimited in amount or disproportionate to the nature and scope of his services. Accordingly, it should be agreed that, although the engineer assumes the legal liability of an independent professional engineer, liability for damage to, or loss of property of the owner as a result of any act or omission of the engineer in connection with the services to be performed, shall not exceed a specified amount, such as the amount of compensation to be paid to the engineer, and the owner should release and hold the engineer harmless from all liability in excess of this amount. For a specified period of time the owner should include the engineer as a named insured on policies of insurance maintained by owner against liability to third parties. In no event should the engineer be liable to the owner for consequential damages, such as loss of use of a facility or loss of profit.

REFERENCE

[1] MERMEL, T. W. — Chairman, Committee on Register of Dams, United States Committee on Large Dams. Register of Dams in the United States.

SUMMARY

The safety of dams has been a subject of public concern in western United States for some years. California has led the nation in the adoption of legislation for the protection of the public against dam failures because of the catastrophic failure of St. Francis Dam in 1928 and the Baldwin Hills Dam in 1963 — both of which were located in California. In recent years there is a growing awareness of the problem throughout the United States as evidenced, in part, by the Federal Power Commission Order 315 for dam inspections issued in 1965, and the more recent activity of U.S.C.O.L.D. in developing a model law regarding dam safety for consideration by all states.

The shifting population distribution toward the west places greater emphasis in this area on the necessity for close inspection and safety investigation of dams, and particularly of the older dams, many of which were designed and constructed to standards which would be considered unacceptable at present. The principal deficiencies generally are inadequate spillways; insufficient stability if subjected to major seismic forces; foundation problems created by the extensive faulting in much of the western section of the country, the common occurrence of weak bedrocks, and substantial loss of strength of materials when saturated; seepage and drainage problems, often resulting from the intensive fracturing and distortion of the bedrocks; and quality of construction materials, particularly concrete aggregates. A number of typical examples of such conditions are discussed, the findings from the safety investigations are outlined, and remedial measures, if required, to enhance the safety of the structures are described. The costs of conducting safety investigations are briefly discussed, and the average cost estimated to be about $ 15,000 to $ 20,000 for engineering, and a similar amount for field exploration.

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Some legal aspects of safety investigations pertaining to the status of the engineer conducting the studies are also briefly mentioned.

RÉSUMÉ

La sécurité des barrages a fait l'objet de l'intérêt public dans l'ouest des Etats-Unis depuis quelques quarante années. La Californie a poussé la nation à adopter une législation sur la protection de la communauté publique contre les accidents des barrages à la suite des catastrophes du barrage St-Francis en 1928 et du barrage des collines Baldwin en 1963, ces deux barrages étant situés en Californie. Dans les dernières années, un intérêt grandissant pour ce problème s'est manifesté à travers les Etats-Unis, ainsi que le prouvent, d'une part l'arrêté 315 de la Commission Fédérale de l'Energie sur l'inspection des barrages paru en 1965 et d'autre part plus récemment l'active campagne menée par U.S.C.O.L.D. pour développer une loi modèle concernant la sécurité des barrages pour tous les états.

Par le fait de la migration vers l'ouest, il est apparu de plus en plus important de considérer, dans cette région, les nécessités d'une inspection sérieuse des barrages et d'étudier les problèmes de sécurité y afférant, et plus particulièrement celles concernant les barrages plus anciens, parmi lesquels beaucoup ont été conçus et construits selon des méthodes qui seraient considérées comme inacceptables de nos jours. Parmi les principaux défauts, il convient de noter en général : les évacuateurs de dimensions insuffisantes, une stabilité insuffisante en cas de forces séismiques importantes, les problèmes de fondation dûs à d'importantes failles dans la partie ouest du pays, le fait habituel de roches en place de mauvaise qualité, les pertes sérieuses de résistance des matériaux à saturation, les problèmes d'infiltration et de drainage, qui proviennent souvent de la fissuration intensive des roches ainsi que de leur déformation et de la qualité des matériaux de construction, plus particulièrement celui des agrégats du béton. Cet article présente un certain nombre d'exemples typiques de telles conditions, esquisse à grand traits les résultats de l'étude sur la sécurité et décrit les mesures de traitement afin de renforcer la sécurité des ouvrages. Cet article discute également rapidement les frais qui interviennent dans la conduite d'une enquête sur la sécurité. Le coût moyen pour le génie civil est estimé à environ $ 15 000 à $ 20 000, celui pour l'étude sur les lieux est du même ordre. Il est aussi fait mention de certains aspects d'ordre légal concernant les statuts de l'ingénieur conduisant les études de sécurité.